Benchmarking and Scoping Study of Hydraulic River Models

Stage Two - Final Report

University of Bradford

R&D Technical Report W88

Benchmarking and Scoping Study of Hydraulic River Models

Stage Two - Final Report

Richard A.Crowder, Y.Chen and R.A.Falconer

Research Contractor:
University of Bradford

Environment Agency
Rivers House
Waterside Drive
Aztec West
Bristol
BS12 4UD

R&D Technical Report W88

Publishing Organisation:
Environment Agency
Rio House
Waterside Drive
Aztec West
Almondsbury
Bristol BS12 4UD

Tel: 01454 624400 Fax: 01454 624409

Dissemination Status

Internal:	Released to Regions
External:	Released to Public Domain

Statement of use
This report summarises the findings from a series of benchmarking and scoping tests undertaken with a number of one-dimensional hydraulic river models. This report is not intended to be used so as to distinguish which particular one-dimensional hydraulic river models is the best. However, the report is intended to provide insight into the packages that may be more suitable for particular modelling requirements.

This document was produced under R&D Project 508 by:

Research Contractor
Department of Civil & Environmental Engineering
University of Bradford
West Yorkshire
BD7 1DP
England

Tel: 01274 – 383872 Fax: 01274 - 383888

Project Management
Flynn & Rothwell Consulting Engineers
Thomas Tredgold House
231 London Road, Bishops Stortford
Herts CM23 3LA

Tel: 01279 507346 Fax: 01279 758219

Environment Agency's Project Manager
The Environment Agency's Project Manager for R&D Project 508 was:
Mr David Pettifer – Environment Agency, Midlands Region

R&D Technical Report W88

CONTENTS

Section 1: General Introduction and Training

Section 2: Test Specification and Results

Section 3: Overview and Conclusion

APPENDICIES

EXECUTIVE SUMMARY

The purpose of this study was to test a range of hydraulic river modelling software ('models') in a variety of situations (both synthetic and real) to determine their applicability for use in UK river modelling. Testing the accuracy of the underlying algorithms in use within the software was not considered appropriate and has not been included in the study. It was originally anticipated that the models would be benchmarked against known results.

The models chosen for the tests were those models that were in use by the National Rivers Authority (NRA) at the commencement of the project (October 1995).

Stage One of the project was undertaken by Sir William Halcrow & Partners Ltd and HR Wallingford Ltd as a joint venture. This stage determined the models to be tested, using results from a survey of NRA users; it defined the tests to be undertaken; and it provided data sets in ISIS format together with benchmark results in certain cases.

Stage Two of the project was undertaken by the University of Bradford, and project managed by Flynn & Rothwell Ltd. This stage, the findings of which are the subject of this report, involved carrying out the defined tests on the models in order to provide an independent assessment of applicability and performance of each model. The tests were carried out, under strict supervision, by a research assistant at the University who had no previous knowledge of the models, so that assessments could be made of vendor training and help facilities.

When used in Stage Two, the test data provided by Stage One was found to have limitations due to errors in the downstream boundary of some data sets; a lack of benchmark results; scale effects; and the provision of data in ISIS format only. As a result it was felt that the models could not be definitively benchmarked and the brief was therefore changed to scoping where benchmarking was not possible. In addition, one of the Stage One tests was omitted, and two alternative tests substituted to widen the scope of the tests and vary the data sources employed.

The models tested ranged from simple steady state backwater programmes written for the NRA's (or predecessor bodies') internal use only, to complex hydrodynamic models which form part of much larger software suites. The models can broadly be divided into steady state models and full unsteady hydrodynamic models.

The version of the software employed in each case was that available immediately prior to the commencement of Stage Two of the project in October 1995. It was necessary to freeze the version of software used in testing in order to undertake a fair comparison and to prevent continual retesting. Many models have been updated significantly since then, particularly with regard to making them microsoft windows compliant. The vendors' own schedules of updates for each model are included within this report.

All models were found to have limitations to their use in particular situations, although when contacted most vendors stated that either approximations, simplifications, or in some cases additional programming, could be employed to model a particular structure or resolve a specific problem. The vendors advise was found to be critical to the successful modelling of some tests. The vendors ability to provide support in the form of additional programming was, however, deemed to be outside the scope of this report. The vendors' views on the draft report were invited, and their comments have been included in an appendix to this report.

One common problem with the tests was the size of channel employed. Many were small, being derived in several cases from laboratory experiments, and the effects of rounding in the results of some models was found to be significant. It should be noted that the testing was an academic exercise, quoting results to several decimal places in some instances. In reality the results required for engineering purposes, taking into account model accuracy, are to no greater degree of precision than 0.01m.

It should also be noted that none of the tests had any calibration data to allow refinement of the models to reflect observed data. As a result the differences in the test results contained in this report are greater than may be expected post calibration.

Other general observations noted were:

- Some of the tests required zero flow conditions to be tested. However, there are an infinite number of solutions that can be found for such conditions. In this study the steady state results from the software packages tested with zero flow were generally shown to be dependent upon the initial conditions specified.

- The models that were not designed to undertake supercritical flow analysis were nevertheless tested under such flow conditions to investigate their performance under such a scenario, and the results from such tests should take this design limitation into account. However, the ability of some of the software packages to recover subcritical flow upstream of supercritical flow accurately is an important feature of some of the models.

- The use of water level balance or energy level balance for split/looped flow situations has been shown to have a significant effect on the results that can be obtained.

- It was found that small changes in the input data can have a significant effect on the resulting water level or flow.

- This study has demonstrated the complexity of transporting data sets between different software packages. It has also enforced the need for survey data that are suitable and accurate to the modellers' requirements.

- The method employed to represent features was found to have a major effect on the results of the model.

- Modelling a weir by raising the cross-section profiles to represent the weir profile can be a suitable approximation in certain engineering situations. However, the spacing of the cross-sections is important and should be spaced very close where there is rapidly varying flow. The diversity of the results obtained by the models is an indicator that this method of representing weirs should be used with caution - where possible weir units should be used.

- Many of the features of the software packages have not been fully tested, or may not have been tested at all. Examples are bridges with sloping abutments, bridges being overtopped, flapped outfalls, and inverted siphons.

The report details in some depth the findings of the study to give engineers, planners and modellers a tool with which to decide which model is most appropriate for a particular application. A performance chart has been included in the report to simplify the selection process, but it is essential that this chart is read in conjunction with the tests summaries, and (if necessary) the detailed text to check a model's suitability for a specific application.

It is not the intention of this project to rank any of the models or to give comments on the best model for a particular situation. Rather, it is the intention to report, in an unbiased way, the performance of each model to each test undertaken, and this is reflected in the report.

LIST OF FIGURES

LIST OF TABLES

SECTION 1

General Introduction and Training

This section consists of:

- Introduction
- Training
- User Interface

1.1 INTRODUCTION

1.1.1 Background

This document has been produced as the stage two project record by the University of Bradford for the 'Benchmarking Study' originally begun with the National Rivers Authority under project 508. During the course of this project the National Rivers Authority has been incorporated into the Environment Agency and the title of the project has been amended to 'Benchmarking and Scoping Study of Hydraulic River Models' so as to reflect the overall substance of the study.

1.1.2 Software Packages

The versions of the software packages provided at the beginning of the testing period (September 1995) have been maintained throughout this study so as to allow a level platform for all packages to undertake the tests. The software packages selected from the stage one report are as follows:

<u>STEADY FLOW MODELS:</u>

- BAKWATER
- GLC-BACKWATER
- HEC-RAS
- HEC-2
- LD01
- CHANNEL

<u>UNSTEADY FLOW MODELS:</u>

- FLOODTIDE
- FLUCOMP
- HYDRO-1D
- ISIS
- MIKE 11

The packages listed in this report are in the order in which training was undertaken at the beginning of this study; this does not reflect the order in which the tests were undertaken for each package.

1.1.3 Tests

As a result of the Stage One study for the 'Benchmarking of Hydraulic Models', undertaken by Sir William Halcrow & Partners Ltd and H R Wallingford Ltd (published May 95), 11 tests have been defined. During the course of this study it was decided by the Steering Group to withdraw Test 8 (i.e. the Tidal River Thames) and to include Test 12 (i.e. the River Calder) and Test 13 (i.e. Analytical Flood Wave). The twelve Tests are as follows:

		STEADY	**UNSTEADY**
• Test 1 –	Structures	✓	
• Test 2 –	Simple steady state looped model	✓	
• Test 3 –	Subcritical flow in a triangular channel	✓	
• Test 4 –	Supercritical flow in a triangular channel	✓	
• Test 5 –	Transitional flow in a trapezoidal channel	✓	
• Test 6 –	Steady flow in compound and meander channel	✓	
• Test 7 –	Flood Channel Facility Blackwater physical model	✓	
• Test 8 –	Tidal Thames mathematical model (Dropped)		✓
• Test 9 –	River Blythe		✓
• Test 10 –	Circular culvert		✓
• Test 11 –	Bridges	✓	
• Test 12 –	River Calder (New)		✓
• Test 13 –	Flood wave (New)		✓

1.2 SOFTWARE TRAINING

1.2.1 Introduction

This section has been written so as to provide some general feedback on the training provided for each of the software packages being used within this study. Any initial indication of the capabilities or shortcomings of each software package should not be used as a guide to the overall performance of these packages. It should be noted that many of the features, strengths and weaknesses of the software packages only became apparent through their extended use.

1.2.2 SOFTWARE MODELS

FLOODTIDE

Installation of FLOODTIDE was a simple procedure, however, it should be noted that a revised copy of the program was provided by the vendor on the first day of training. The reason for the revision was due to errors that had been identified by the trainer since converting the software from a UNIX to PC platform. A senior employee for Babtie Group, Glasgow branch, provided the training for this software. The trainer's knowledge of the product was very comprehensive and was demonstrated by his ability to correct the FLOODTIDE source code when run time and output problems were encountered. The trainer advised that there might be further input/output commands that may arise and give rise to further errors. The trainer suggested that they (Babtie) should be contacted if such problems arose so that they could endeavour to make or suggest any changes.

There is no user interface provided with FLOODTIDE for the input of data. Any DOS editor that the user is familiar with can be used for the production and editing of the data files required by FLOODTIDE. The formatting and data structure required by the program are fundamental if a model set-up is to be run successfully. Any user of FLOODTIDE would require a detailed knowledge of the capabilities and required procedures for execution of the software in order to reduce model development time. However, the vendors state that they do not sell FLOODTIDE on a commercial basis for general application and that only full working calibrated and verified models dedicated to specific river systems are delivered to Clients.

Nevertheless it is very likely that such models will be subsequently amended by the client's staff, hence the reason for the inclusion of FLOODTIDE in the project.

Both an engineer's guide and an operator's manual provided good clear documentation. The engineer's guide details the method of calculation used within the program for various hydraulic structures and channels included in the program. The operator's manual details both the structure and data format required by the program to undertake any simulations.

It should be noted that it became apparent during the study that there are errors in the manual when describing the format required in the data file for the bridge and sluice gates.

Developers: *Babtie Group Ltd, UK.*
Vendors: *Babtie Group Ltd, Glasgow, UK.*
Platform: DOS
Version: 3
No. of Installation Disks: *2*
Dongle Required: *No*
Training Days: *3*
Steady/Unsteady State: *Both*

HEC-2

The installation of HEC-2 was done a couple of days prior to the training session and proved to be a simple procedure without causing any problems. An employee of Bullen Consultants of Bradford provided the training for the software, one of the UK vendors for HEC-2. The trainer provided a good explanation of HEC-2's background and was well prepared for the training session with overheads and handouts so as to provide a structured and well-organised session. Several tutorial examples were provided as a method of training, which by coincidence were of a similar nature to the benchmarking tests to be undertaken as part of this project.

HEC-2 uses a menu driven interface to run the program and create the data files and was relatively simple to operate. The training provided on the interface was adequate though the use of a standard text editor may be more convenient to a more experienced user. The program is capable of calculating both subcritical and supercritical flows though the user must be aware that the cross sections have to be input in reverse order and separate runs need to be undertaken for each condition.

Training

The manual provided is reasonably well structured. It details the input codes required by the editor in order to set up a river system and any associated structures. There is also an online help system that is useful for the inexperienced user. A detailed understanding of the file format is required by the user in order to create a data file with the correct structure and subsequently undertake a successful model calculation with HEC-2.

Developers: *US Army Corps of Engineers*
Vendors: *Bullen Consultants Ltd, Bradford, UK.*
Platform: *DOS*
Version: *4.6, May 1991*
No. of Installation Disks: *4 (+ 3 source code files)*
Dongle Required: *No*
Training Days: *1*
Steady/Unsteady State: *Steady State only.*

HEC-RAS

Installation was via the Windows setup routine. The package was fully Windows orientated, allowing use of the Windows clipboard utility. An employee of Bullen Consultants of Bradford provided the training for the software, one of the UK vendors for HEC-RAS. The quality of training was of a similar standard and format as that for HEC-2, although less formal.

The user interface was very appealing to use and the set up of a river system was very quick and simple. HEC-2 files could be imported into HEC-RAS via a filter built into the package. Unfortunately this version of HEC-RAS did not have all of the structural features required by the benchmarking tests. For example, to model a weir the bridge module has to be employed.

Documentation was well laid out, although the copy provided was clearly a photocopy, with the tables and graphs being of a poor quality.

Developers: *US Army Corps of Engineers*
Vendors: *Bullen Consultants Ltd, Bradford, UK.*
Platform: *WINDOWS 3.1 or later*
Version: *1.0c, October 1995*
No. of Installation Disks: *2*

Dongle Required: *Yes*
Training Days: *2*
Steady/Unsteady State: *Steady*

MIKE 11

An employee of the Danish Hydraulic Institute gave the training for MIKE 11. The trainer had prepared overheads but they were not used; instead handouts of the overheads were used. The knowledge of the instructor on MIKE 11's capabilities and procedures was extensive. Several tutorials and examples were provided as part of the training which were very useful. In addition to the examples provided several of the tests within this study were attempted over the three days of the training. With the limited knowledge of the tests to be undertaken within this study the instructor advised on possible ways that MIKE 11 could/should be used to set up and run each of the tests under this project.

The user interface of MIKE 11 is menu driven, which runs in a DOS window within the Windows operating system. It should be noted that the version of MIKE 11 under test is not fully windows integrated, however the trainer advised that such a version was under development. Both the user manual and reference manual provided with MIKE 11 are comprehensive in their content and are well laid out.

Developers: *Danish Hydraulic Institute (DHI), Denmark*
Vendors: *Danish Hydraulic Institute (DHI), Denmatk*
Platform: *Windows 3.1 or later*
Version: *3.11*
No. of Installation Disks: *8 (including various modules)*
Dongle Required: *Yes*
Training Days: *3*
Steady/Unsteady State: *Both*

HYDRO 1D

A senior employee of Mott MacDonald along with two other employees gave the training for HYDRO-1D. The training sessions provided an overview of the program as well as some examples.

Training

The user interface of HYDRO-1D is menu driven and should be run from DOS. A run time version of DBOS (required by Fortran programmes) is required by HYDRO-1D and is provided with the installation of the software. It is relatively simple to follow the installation, but the built in help system is not comprehensive.

Test 5 Part A was attempted during the training session for HYDRO-1D and took considerable time to set up due to the large amount of cross section files required by the program. It should be noted that a blank background map is required by the software to set up the model, but this was not provided with installation of HYDRO-1D and that a map from a previous test was used in order to continue with the training. A blank map was received by post for the second day's training.

During the training Test 1 Part A was attempted, however, with the default calculation settings it was not possible to obtain a stable result. On the advice of the instructors, some of the calculation settings were changed, with this producing more stable results. It should be noted that the calculation settings are not well documented in the manual; this leaves some ambiguity as to what values are suitable for the benchmarking tests.

The user manual that was provided in advance of the course was replaced on the first day of training. This was due to mistakes in the original copy. The manual is very basic, though colourful. It provides a basic reference for input of data and the running of a simulation with HYDRO-1D, however, it does not provide comprehensive information on any data settings.

Developers: *Mott MacDonald Ltd, UK*
Vendors: *Mott MacDonald Ltd, Cambridge, UK*
Platform: *DOS*
Version: *4.00*
No. of Installation Disks: *2*
Dongle Required: *No*
Training Days: *3*
Steady/Unsteady State: *Both*

FLUCOMP

An employee of HR Wallingford gave the training for FLUCOMP. His knowledge of the product was adequate for our needs although his lack of recent use of the product was apparent.

The time allocated to the training of FLUCOMP did not permit many examples to be attempted. The trainer briefly explained the background of FLUCOMP and then demonstrated some of its capabilities.

The FLUCOMP package is very limited in its use. A simple menu system with an editor is used by FLUCOMP to set up and undertake a simulation. Several different files are required to be set-up in order to undertake a model simulation. They are essentially the cross-section data file, the roughness data file and the flow data file. Additional files are required if there are bridges or if the simulation required is unsteady. It is crucial that all of the data required is provided in each of the files required and that it is coded and placed correctly so that a successful simulation is to be undertaken. Errors can not be detected until a run is completed and then it can be difficult to remedy unless the user is experienced in the use of the software.

It is not possible to simulate embanked rivers, looped systems and supercritical flow with FLUCOMP.

The manual is not easy to follow, but is comprehensive and detailed in its content. It explains the setting up of a river system and the structures that can be included in the program.

Developers: *HR Wallingford Ltd, UK*
Vendors: *HR Wallingford Ltd, UK*
Platform: *DOS*
No. of Installation Disks: *2*
DongleRequired: *YES*
Training Days: *1*
Steady/Unsteady State: *Both*

ISIS

An employee of Halcrow gave the training for ISIS, his knowledge and experience of the software package appeared to be very high. A basic overview of ISIS was first given (with sufficient detail) and then several on line examples were studied.

The version of ISIS provided for this study requires Windows 3.11 or later to run within, however, it is not fully windows integrated. The program uses a standard windows menu system and tool bar in order to setup and run any given river system. The method of setting up a system is fairly simple and was demonstrated well by the instructor.

The study data files provided by stage one of this project are in ONDA format. As ONDA is one of the predecessors of ISIS this facilitated the direct import of the data files into ISIS with the minimum of effort.

Several benchmarking tests were attempted over the three days with relative success. The Thames river system was successfully imported to ISIS, although varied results were obtained from runs of the model. The manual provided was adequate, well structured and compact.

Developers: *HR Wallingford Ltd, UK & Halcrow, UK*
Vendors: *HR Wallingford Ltd, UK & Halcrow, UK*
Platform: *Windows 3.11 or later*
No. of Installation Disks: *8 (including modules)*
Version: *1.0 (ISIS Flow 4.01, Workbench 1.01)*
Dongle Required: *Yes*
Training Days: *3*
Steady/Unsteady State: *Both*

CHANNEL

The developer of CHANNEL gave training for the software package. The training provided was very basic, however, it was felt that this was not restrictive since CHANNEL is limited to steady state, subcritical flows with no structures. The only means by which structures can be modelled is by the user specifying a head loss at the desired location.

Training

The input of data is via a menu driven system, which is DOS based although simple to use. The training in the use of the menu system was adequate and covered all aspects relevant to the study.

During the training period time was spent discussing the tests that CHANNEL could attempt as part of this study. The package is very limited in what it can achieve although the training in its capabilities was more than adequate. It should be noted that the user manual provided with CHANNEL is both comprehensive and laid out well.

Developers: *Micro Drainage, UK*
Vendors: *Micro Drainage, UK*
Platform: *DOS*
Version: *A.5*
No. of Installation Disks: *2*
Dongle Required: *No*
Training Days: *1*
Steady/Unsteady State: *Steady*

LD01, GLC-BACKWATER AND BAKWATER

A combined training session was undertaken for LD01, GLC, and BAKWATER. No formal presentation was made on the background of each program and no handouts or examples were provided. The limited time available for each package meant that only one of our study test cases could be attempted (TEST 1 : Part D - on BAKWATER)

It was made clear during the training that LD01 is limited to channels with a maximum of 50 cross-sections and steady flow conditions. It was also highlighted that LD01 can not cope with supercritical flows or sluice gates. LD01 is DOS based and requires the use of any DOS editor to set up the data files required to run a simulation. A detailed understanding of the structure required in the data files is essential by the user if errors and model development time is to be kept to a minimum.

The instruction and presentation for GLC-BACKWATER and BAKWATER was very similar to that of LD01. However, it should be noted that BAKWATER does provide a very basic user interface that aids model set up and allows the program to be run.

Training

The documentation provided with each of the packages is very rudimentary, only the GLC-BACKWATER manual was bound and no manual was provided for BAKWATER.

	LD01	**GLC**	**BAKWATER**
Developers :	*Environment Agency*		*ATPEC*
Vendors :	*Environment Agency*		*ATPEC*
Platform :	*DOS*	*DOS*	*DOS*
No. Of Installation Disks :	*1*	*1*	*1*
Dongal Required :	*No*	*No*	*No*
Training Days :	*1/3*	*1/3*	*1/3*
Steady/Unsteady State :	*Steady*	*Steady*	*Steady*

1.2.3 Summary

All software vendors provided their packages before the training, so that the packages could be installed and ready for training. Training was received for every package to be tested. In general, three days of intensive training were necessary for the unsteady models and one day of training for the steady models. The training provided for each of the packages has proved to be essential. The packages have generally fallen into one of three categories:

(i) Those that are completely DOS based with no user interface,
(ii) Those that have a user interface and run under either DOS or Windows, and
(iii) Those that are fully Windows integrated.

Except for BAKWATER, all of the packages included a user manual.

1.3 USER INTERFACE

1.3.1 Introduction

This section has been written so as to provide the reader with a background to the procedures required for setting-up, running and obtaining results for each of the models considered within this study.

1.3.2 Software Packages

FLOODTIDE

FLOODTIDE is a DOS based package that requires the user to set up the required data files through any DOS text editor available to the modeller. Essentially, two files are required to set up the model, they are the *.dat file, where the cross-sections, structures and river network are defined, and the *.bdr file, where the boundary data are defined. The *.bdr file are also used to specify the initial conditions (from a critical backwater calculation) and also to supply the results to a steady state calculation. The setting up and format required by both the *.dat and *.bdr files are described thoroughly in the user manual for this model.

A sequential procedure is required to undertake a simulation with FLOODTIDE. Once the FLOODTIDE executable file is run from the command line, the modeller enters the data file names and selects the required options presented on the screen and also enters the time-step required for the calculations. For any calculation where initial conditions are not available, then a critical backwater calculation should first be undertaken. The results from this run are appended to the *.bdr file, immediately after the input of the boundary conditions. These results should be inspected, and if necessary adjusted to acceptable values, as the values calculated are frequently inaccurate for complex networks (e.g. split flows). The modeller should then change the title of the critical backwater results to initial conditions, before a steady normal backwater calculation is undertaken. The results from the steady calculation are then added to the *.bdr file after the critical backwater results. If an unsteady run is required, then the results from the critical backwater calculations should be deleted from the *.bdr file and the results from the steady normal backwater calculations given the title of 'INITIAL

CONDITIONS'. This procedure can sometimes pose significant problems, especially for complex river networks where accurate initial conditions are difficult to obtain. However, the pseudo time-stepping method of calculation may sometimes overcome these problems.

Results from the steady backwater calculation are obtained from the *.bdr file. However, for an unsteady calculation the post-processor program, which is part of the FLOODTIDE package, has to be used. When the post-processor program is run, then results for the required cross-sections or time series can be obtained and are provided in an ASCII file for analysis by the modeller.

HEC-RAS

The HEC-RAS model has a fully integrated Windows graphical user interface. HEC-RAS uses 'Projects', which consist of a set of files associated with a particular river system and are categorised as follows: plan data, geometric data, steady flow data. There are five main steps which are taken by the modeller in order to undertake a study which include: starting the project, entering the geometric data, entering the flow data, performing the hydraulic calculations, and viewing and printing the results. Each step is fully described and illustrated in the HEC-RAS user manual.

The modeller can easily create the river network, which is built up of reaches, by drawing the river system schematically through the click and drag options available in the Graphical User Interface (GUI). After the rivers schematic is drawn, the modeller can then enter cross-sectional and hydraulic structure data (bridge only) through the respective pop-up editors. The cross-section editor easily facilitates the adding, copying, renaming and deleting of cross-sections; adjusting cross-section elevations, stations, Manning's n values and including ineffective flow areas, levees and blocked obstructions. Once the geometric data have been entered the modeller can then enter the flow data for the boundaries which can either be one of the following: water level, critical depth, normal depth, rating curve or discharge. However, it should be noted that an upstream water level is required when supercritical calculations are undertaken. Calculations are undertaken in a straightforward manner by clicking one of the compute buttons in the steady flow analysis pop-up window. The user has the opportunity to alter any of the default calculation settings through a sub-menu, accessible through the steady flow analysis window.

At all times during the model set-up and after computations the user has the ability to view the river network structures and cross-sections in 2-D, and the river network in 3-D, through the GUI capabilities of HEC-RAS. This is a very useful feature when building a model and when viewing the results. Results are also available in tabular form through the GUI, which allows the modeller to use predefined output tables, or user defined output tables, with any number of decimal places that the modeller may require.

HEC-2

The HEC-2 package is DOS based and includes its own DOS based GUI and editor to aid in the input of data. The user should be familiar with the codes that are used to define the data and the order in which the data is entered if model development time is to be optimised. Essentially there is a single data file that contains all of the cross-sectional and boundary data required to undertake a calculation. The user manual describes the sequence for the input of data and the codes required by the editor. However, the accomplished modeller who understands the data file structure may find that a standard DOS editor provides a simpler method of developing the data file required for a river model.

The GUI for HEC-2 is used to initiate a calculation using a model data file. Results from a calculation can obtained by running the post-processor GUI, which allows the modeller to define the required output values of the results file. Results can then be viewed with any text editor and printed out for inspection. HEC-2 is the predecessor to HEC-RAS. Hence, the usability of the package in comparison with other models that are currently available and on the market, make HEC-2 a package that is perhaps less suited to current day modelling demands.

MIKE 11

The MIKE 11 software package (version 3.11) is essentially a DOS based program and runs within a DOS window environment on Windows 3.11 or higher. The GUI with MIKE 11 is hierarchical in nature and allows an experienced user to navigate the package, so as to build up a river model, execute model runs, view results and to import and export data. To the inexperienced user MIKE 11 can appear to be a complex package to set up and execute calculations.

The setting up of a river model study through the GUI essentially requires the user to define: (i) the river set up with respect to cross-sections, networks and structures etc., (ii) the boundary conditions, and (iii) supplementary data. The abundance of features and add-on modules available with MIKE 11 make the package a very flexible model. However, this can also add to the complexity of setting up a river network. The error checking system within MIKE 11 aids the modeller in accurately setting up and executing the model.

The undertaking of a calculation for either steady or unsteady flow conditions is very similar and is undertaken through the calculation menu. The modeller has a number of calculation options available within the package, although essentially for a hydrodynamic simulation the user can select the start time, time step and simulation time.

Results can be obtained via the presentation menu, which allows the modeller to view numerous output values at specific times or over a range of times, which can be exported to a text file. MIKE 11 also has the capability to illustrate graphically output values with time series plots for specific cross-sections or reaches of a river network.

HYDRO-1D

The HYDRO-1D package uses a DOS based GUI, which is hierarchical, and allows the user to select the available data input and running options. To set up a model, the GUI requires a map on which a river network can be built. The map can either be blank or can represent the geographical features of the area being modelled. To build a model network, the user must first define nodes on the map (for cross-sections) and then the reaches, which connect the nodes. For a simple river network this is easy to use, however, for a complicated river network or a network that requires a large number of cross-sections this can be a somewhat cumbersome procedure. Cross-sections are input very conveniently in the GUI, which provides a useful graphical representation of the cross-sectional profile, once the data has been entered. However, each cross-section is stored in separate ASCII files, which can be a tedious procedure when building large or complicated river networks or amending cross-section values. The boundary data can easily be entered and visualised within the GUI.

Undertaking a simulation with HYDRO-1D requires a systematic approach. First, the modeller can select the start time step, end time step and the time step in hours for the simulation, and then also choose between either a steady or transient (unsteady) calculation. The modeller must initialise individual new or altered network, inflow, cross-section, boundary, backwater and results file for each calculation. The modeller can then enter the required calculation parameters or alter any default values before a calculation begins.

After each iteration of a calculation the 'Residual Limit' for convergence is displayed on the screen and also saved to a file for later inspection. This enables the modeller to asses the stability of the solution with the given calculation settings.

Results from a steady run are produced in an ASCII file, which can be viewed with any DOS editor. The steady results file can then be subsequently used to provide the initial conditions for any unsteady calculations. The GUI allows time series results to be view for either selected cross-sections or profiles of selected reaches of the river network. This is a very useful feature available within HYDRO-1D. The ASCII results file for both steady and unsteady calculations do not provide information on the velocities or the Froude number, with these parameters often being required by many modellers.

FLUCOMP

The FLUCOMP package has a DOS based GUI which has a built in editor for creating the various files required to undertake a calculation. Essentially five files are required:

*.SDF (Section data file for all cross-sectional information)
*.RDF (Roughness data file for defining roughness values)
*.BDF (Bridge data file for bridge data)
*.WDF (Flow data file for defining steady flow conditions and regulating structures)
*.FDF (Flow data file for defining unsteady flow conditions and regulating structures)
*.HDF (Hydrograph file for unsteady flows)

To produce any of the above files, either the built in editor or any DOS text editor can be used. However, the modeller must be aware of the required data structure in each of the files, as failure to produce any of the above files in the correct format can easily result in calculation failure. To undertake either a steady or unsteady calculation is a simple procedure, once the data files have been produced. First, the modeller should use the section data analysis algorithm to check and process the topographical data for the river and also the bridge data analysis to check and process the bridge data if required. The checking procedure can be a helpful tool when initially building a river model. To undertake a calculation the modeller can select either FLUCOMP 1 for a steady calculation, or FLUCOMP 3 for an unsteady calculation. During an unsteady calculation the simulation time is displayed on the screen to show that the calculations are under way.

Results for all variables from a steady run are given in a single text file, whereas the results from an unsteady run are given in separate files for each of the output variables available, such as water elevation, velocity and Froude number.

ISIS

ISIS has a GUI that is predominately Windows based. However, a DOS based editor may be used to define the single data file that contains all of the cross-sectional, network, boundary and initial condition values required to undertake a simulation. Once the modeller has created the data file, the GUI can then be used to build a Windows based visual representation of the river network. The user can also view cross-sectional and reach representations of the data that have been entered for a river network. The user manual for ISIS is well structured and describes the order in which data should be entered and the data requirements for the units used to model specific structures.

The undertaking of a calculation with ISIS is a simple procedure, initiated by the pop-up menus available within the package. The default calculation options can be altered for either a steady or unsteady calculation. A steady calculation provides results that are automatically delivered to the screen for viewing, as well as producing a text results file. An unsteady calculation provides various calculation values in a graphics windows environment throughout the simulation period. This is a valuable feature of ISIS and allows the modeller to monitor the progress of a calculation and the periods when tolerance values are exceeded. ISIS produces valuable diagnostics files that can assist the modeller when developing and running a river network.

Results from a steady calculation can be easily viewed as a text file, however, the results from an unsteady calculation must first be processed through the post processor within ISIS. This allows the user to select specific cross-sections or time periods within a simulation for analysis by the modeller. The GUI of ISIS also allows time series results to be viewed on the screen in relation to the geometric data provided for the river network. This feature is very useful to the modeller when analysing the results obtained from a simulation.

CHANNEL

The CHANNEL package has several modelling capabilities, however, only the 'Channel – Backwater Step Method' is suitable for modelling a single river channel without structures under steady flow conditions. The CHANNEL, package has a DOS based GUI, which is hierarchical, and allows the user to select the available modelling options and input the data. To begin a calculation the executable file should be run and then from the main menu the backwater option should be selected. The modeller can then either select an existing file or begin a new job. When beginning a new job the user is prompted to input: the starting water level (m), Manning's coefficient for the main channel, Manning's coefficient for the flood channel and the total flow (m^3/s). The modeller can choose to undertake either a calculation with only the main channel data, or both the main channel and flood plain data. Depending upon which option is chosen, the modeller must then input the relevant cross-sectional information, one section at a time and through the GUI. After the data for each cross-section has been entered, CHANNEL then automatically computes the water elevation at that cross-section and indicates the corresponding value on the screen. For a river channel the cross-sectional information has to be entered one section at a time and only when all of the cross-sectional data have been entered can the user obtain a printed copy of the calculated water elevations along the complete river channel

LD01

The LD01 program is a DOS based package that requires the modeller to produce a single data file in ASCII format ready for program execution. A calculation with LD01 is begun from the command line of DOS and hence the modeller is required to produce the data file in the required format by the use of any available text editor. The user manual provided with LD01 is very basic. However, it does assist in structuring the data correctly in the data file.

In general the setting up of a data file and the execution of a run is very simple procedure with LD01. However, if a calculation fails then the modeller is given little indication as to where the incorrectly defined data may be located in the data file.

GLC-BACKWATER

GLC-BACKWATER is a DOS based package which requires the modeller to produce a single data file in ASCII format ready for program execution. A calculation with GLC-BACKWATER is begun from the command line of DOS and hence the modeller is required to produce the data file in the required format by the use of any available text editor. Although the user manual provided with GLC-BACKWATER is very basic, it clearly explains the format and data structure required to develop a river model.

In general the setting up of a data file and execution of a calculation is a simple procedure with GLC-BACKWATER, once the data has been constructed correctly. The modeller simply executes the program and then enters the data file name to be used. If the structure of the data file is incorrect, or if the calculation fails, then error messages are given to assist the modeller in isolating the possible cause of the error prior to calculation failure.

Results from GLC-BACKWATER are provided in an ASCII data file which can be viewed and printed from any suitable text editor.

BAKWATER

The BAKWATER software package has a DOS based GUI which is simple and easy to use. The GUI allows the boundary data and cross-sectional data to be readily input and also graphically illustrates the cross-sectional data once it has been entered. Viewing a cross-section allows the user to define the main channel and flood-plain boundaries of a cross-section. The undertaking of a calculation is simply done through the GUI and also facilitates the production of a hard copy of the results. Although BAKWATER can take account of structures, it is a package that is simple and easy to set up for a simple river channel with steady flow conditions.

1.3.3 Summary

1. HEC-RAS is the only software package that has a GUI that allows the full input of data using the integrated capabilities of the Windows GUI.

2. ISIS has a Windows based GUI for viewing the network of a model, undertaking calculations and viewing the results. However, an integrated DOS based editor is used for the input of data for cross-sections, structures, boundaries and initial conditions. In addition data files may be created or edited externally of ISIS through any text editor.

3. MIKE 11 runs within Windows 3.11, or higher, but is essentially a DOS based package. It is a package that the user should be experienced with and be aware of its full capabilities if the model is to be used effectively.

4. HYDRO-1D, HEC-2, FLUCOMP, CHANNEL and BAKWATER all have DOS based GUI's to assist in the input of data.

5. Calculations with LD01 and GLC-BACKWATER require the modeller to produce a single data file with all of the model data and FLOODTIDE requires two data files, one with the network data and the other with boundary data. All these three packages are initiated at the DOS command level.

SECTION 2

Test Specification and Results

This section consists of:

		STEADY	UNSTEADY
Test 1 –	Structures	✓	
Test 2 –	Simple steady state looped model	✓	
Test 3 –	Subcritical flow in a triangular channel	✓	
Test 4 –	Supercritical flow in a triangular channel	✓	
Test 5 –	Transitional flow in a trapezoidal channel	✓	
Test 6 –	Steady flow in compound and meander channel	✓	
Test 7 –	Flood Channel Facility Blackwater physical model	✓	
Test 8 –	Tidal Thames mathematical model (Dropped)		✓
Test 9 –	River Blythe		✓
Test 10 –	Circular culvert		✓
Test 11 –	Bridges	✓	
Test 12 –	River Calder (New)		✓
Test 13 –	Flood wave (New)		✓

2.1 TEST 1 - STRUCTURES

2.1 Introduction

This test essentially assesses the ability of each software package to model the flows for four different structures. This test comprises of testing four types of hydraulic structures, which are as follows:

PART A -	Sluice Gate (11 modes)
PART B -	Flood Embankment
PART C -	Pump
PART D -	Crump Weir.

Each type of structure has been tested individually, however, only ISIS was able to complete the whole of Test 1. The other packages were able to undertake some parts of the test by approximation to the test configuration.

2.1A TEST 1 : PART A - Sluice Gate

2.1A.1 Aim of Test

Each of the software packages is to be tested so as to demonstrate its ability to calculate flow through and/or over a vertical sluice gate in all of the positive flow modes possible. It is expected that the equations used by each package should conform to 'standard' weir and gate flow equations (1). These have been reproduced in Table 1 of Appendix A.

2.1A.2 Introduction

The 'vertical' sluice gate in this test was tested in eleven modes as summarised in Appendix A, Table 1 and described in Section 4.4.1 of the Stage One Report. These eleven modes include zero flow, free and drowned flows over-gate, free and drowned weir under-gate flows, free and drowned through-gate under flows, free and drowned under-gate and free over-gate flows, and drowned under- and over-gate flows.

Figure 1.1 illustrates the plan location of the four cross sections used in Test 1 Part A. Each cross section had a constant Manning's roughness of 0.014 and was rectangular in profile with a width of 1.0m and height of 1.5m. The bed slope was 0.0005 between cross sections 1 and 2 and 0.001 between cross sections 3 and 4.

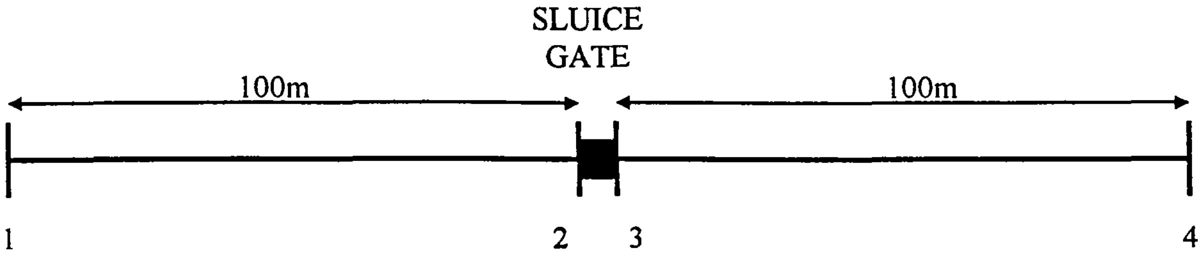


FIGURE 1.1 : *TEST 1 Part A - Plan location of cross-sections*

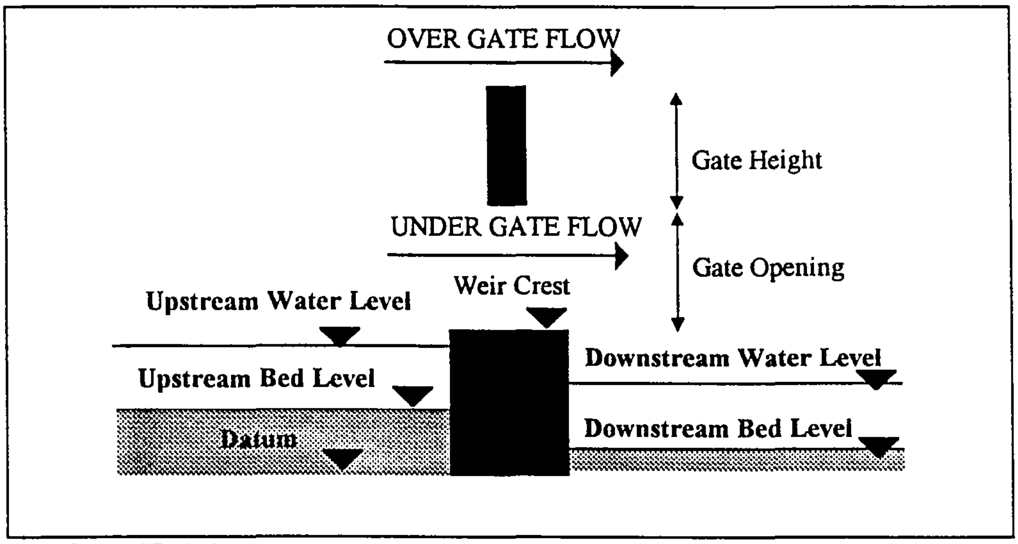


FIGURE 1.2 : *TEST 1 Part A - Schematic illustration of sluice configuration*

Figure 1.2 schematically illustrates the sluice configuration and the two flow paths possible i.e. over-gate and under-gate flows.

2.1A.3 Results

The stage one report (Table 4.3) provides equations for the calculation of flow over or under a vertical sluice(1). The equations have been reproduced in Table 1 of Appendix A and have been used as the method of assessing the accuracy of the results obtained from each of the software packages. The comparison of results between the different software packages for the non-zero flow modes is summarised in Appendix A, Table 2. The theoretical discharge in Appendix A,

Table 2 was obtained using the calculated water head by each package in the appropriate flow equation, listed in Appendix A, Table 1. This discharge shall then be compared with the original model discharge of 0.15cumec. Since ISIS, MIKE 11 and HYDRO-1D have been the only packages capable of the zero flow modes, these modes have been presented along with the comments for ISIS, MIKE 11 and HYDRO-1D.

The effect on the calculated theoretical discharge (from the equations in Table A.1), by small differences in the calculated water level upstream of the structure, has been investigated and tabulated in Appendix A, Table 3. It should be appreciated from this investigation that very small changes in water level can significantly affect the calculated discharge. For example a change of 5% head difference in the upstream water level alters the calculated discharge by 10% or more, clearly showing that careful consideration should be taken when assessing the results from each of the software packages.

A description of the undertaking of this test for each software package and the considerations that should be taken into account when assessing the results is given next.

FLOODTIDE

In the manual for FLOODTIDE(2) (Section 6.11) the data structure is described for the inclusion of an 'over flow' sluice in a river system. It is not possible to model over-gate flow with the sluice module, though it was suggested during training that the weir module be employed as an approximation. For the combined over flow and under flow scenario both a sluice and weir module would be required in parallel at the location of the structure, though this approximation should be treated with caution when considering the equations for flow under and over a sluice gate. However, this method of undertaking the test requires a split flow calculation.

Problems were encountered while setting up the sluice, mainly due to the unclear description of the required data and the incorrect listing for the data structure in the manual. The software vendors were contacted for assistance and were able to clarify the following:

- *A sluice gate can either be placed inline between two cross sections or alternatively between a channel and storage pond. If the gate is to be inline as this test requires, then a value of zero should be used for the 'ratio' in the data file.*
- *There should be five lines of data entry for the sluice and not four as the manual describes, both 'C_d' and 'cfact' which are shown to be on line four should be on line five.*

Although the problems with the data format and the reading-in of the data file by FLOODTIDE were overcome, major problems with the calculation were also encountered. The program for no apparent reason crashed while undertaking the critical backwater calculation to produce the initial conditions, hence the software vendors were contacted and they suggested the following measures to overcome the problems:

- *Run a critical calculation for 1 iteration, although this gives highly inaccurate values.*
- *Alter the discharge, water level and velocity values calculated by the critical calculation to values that are acceptable and then run an unsteady calculation with constant boundary conditions.*

The above suggestions were administered, though the program once again crashed; this time while undertaking the unsteady calculation. Babtie were advised of this continuing problem with the sluice module and could not offer any solution to the problem, which they also encountered on their PCs. The probable cause of the program failure might be due to the conversion of the program from UNIX to PC format, which was undertaken for the sole purpose of this study.

Due to the problems encountered with FLOODTIDE **no results** have been presented for this test. Although the closed gate, free and drowned over gate modes would be possible using the weir module, this has been omitted as this module is tested in Part D of Test 1 of this benchmarking study.

HEC-RAS

TEST 1 - Structures

The current version of HEC-RAS does not support the modelling of sluice gates directly. However, during the training period for HEC-RAS it was suggested that the bridge routine could be employed as an approximation.

To model the sluice extra cross sections were required at the location of the structure, this was done so as to raise the bed level to that of the assimilated weir crest in the sluice structure. The bridge routine was then set-up with the bridge opening set to that of the sluice opening and the bridge deck to that of the crest of the gate. For the modes that required the gate opening to be set to zero an opening of 0.01m was used, without which HEC-RAS would not undertake any calculation.

The results from the assimilated set-up are given in Table 2 of Appendix A, for the modes that were tested and should be treated with caution. It should be noted that HEC-RAS could not cope with the modes for zero flow hence no results have been presented. The following options which are available for HEC-RAS with the bridge computation were selected: (i) momentum equation, (ii) high flow method, (iii) submerged inlet and outlet coefficient = 0.8 - default value. For the drowned over gate and drowned under gate mode inlet and outlet coefficients of 1.0 and 0.5 were investigated; this altered the water level upstream of the structure from 2.023 to 2.016m and 2.044m respectively.

HEC-2

The same approach as with HEC-RAS was required to model the sluice gate, however the setting up was more difficult due to the archaic nature of data input, data structure, and the GUI used by HEC-2.

Several methods of bridge loss calculation can be adopted by HEC-2. For this test the special bridge method was adopted, which can cope with: (i) pressure flow, (ii) weir flow, and (iii) combinations of low or pressure flow. The results produced by the simulations can be found in Table 2, Appendix A, and should again be treated with caution.

MIKE 11

TEST 1 - Structures

It is possible for MIKE 11 to take into account gate structures only if the add on module 'Control Structures' is provided with the purchase of MIKE 11, as was the case for this study. The control structures are specified in menu A.5.A.2 of the add on modules and may be used whenever the flow through, or above, a structure is to be regulated by the operation of a moveable gate which forms part of a structure (3).

The treatment of a gate structure by MIKE 11 is both comprehensive and complex, though the manual clearly covers the input of data for: (i) gate definition (Section 13.3), (ii) control point definitions (Section 13.4), and (iii) control structure operation (Section 13.5). The gate structure defined in Menu A.5.A.2 can either be an overflow or underflow structure, but not both. To model both overflow and underflow for a sluice gate, it is required that two gate structures at the same location be defined, each with its own structure identification, structure definition and control structure operation values.

For Test 1 Part A both the overflow and underflow gates defining the sluice structure were configured with the default head loss factors of 0.5 and 1.0 for inflow and outflow respectively along with a free discharge coefficient (C_d) value of 1.0. The structure geometry was required to be defined in three stages: (i) definition of gate width, sill level and operating speed in menu A.5.A, (ii) definition of control locations for the operation of the gate in menu A.5.A.2.2 and (iii) definition of gate level according to the water level (at the control locations) in menu A.5.A.2.3.

The default head loss factors were used for all parts of Test 1. However, investigations into the effect of changing both head loss factors to 1.0, for the closed gate - free over flow mode, resulted in the water level upstream of the structure increasing from 1.242m to 1.274m, when using a C_d value of 1.0 for free flow and a value of 1.0 for the coefficient of contraction. Decreasing the C_d value to 0.92 (recommended in the MIKE 11 user manual to calibrate the discharge for critical flow conditions), resulted in an increase of the water level upstream of the structure to 1.289m. The cross sections which were intended to be either side of the gate had to be placed 1.0m away from the structure (the minimum possible) for all modes, except the free over gate and drowned under gate mode. To carry out a calculation without an error message for the free over gate and drowned under mode, the cross sections had to be placed

10.0m away from the structure when using an inlet coefficient of 0.5. However when using a value of 1.0 for the inlet coefficient, the distance of 1.0m was adequate.

MODE	Water Elevation at Structure		Discharge Q (m3/s)
	Upstream (m)	Downstream (m)	
Gate open – zero flow	0.300	0.300	0.000
Gate closed – zero flow	0.900	0.900	0.000

TABLE 1.1 : *Test 1 Part A – MIKE 11 results for zero flow modes*

It should be noted from the results in Table 1.1 that the initial conditions for the water level were set at 0.3m and 0.9m for the gate open and closed with zero flow respectively. If no initial conditions were specified then warning messages were given.

It is important to note that the input of data for the gate does not take into account the profile of the weir (length), however the crest of the weir was used for the sill level for both under-gate flow and over-gate flows.

HYDRO-1D

The modelling of a sluice gate with HYDRO-1D using the sluice gate option is not a straightforward operation and its appropriateness to the situation being modelled should be considered carefully.

An underflow sluice gate is modelled by defining the reach between two nodes as a gate, thus a cross section is defined directly upstream and downstream of the structure as required by the benchmarking data. However the sluice gate option of HYDRO-1D is only intended for conditions of submerged flow through a gate and not free flow. This is not covered in the manual and only became known when the accuracy of the results for this test were queried with the software vendors.

During the training for this package it was suggested that an overflow gate could be modelled by placing a weir in parallel with the sluice and then setting the crest level of the weir to that at the top of the gate.

The definition of the gate structure is very simple with respect to the fixed dimensions and coefficients. Although the length of the weir crest is not defined the weir width, crest and velocity coefficient are. It should be noted that HYDRO-1D internally sets the velocity coefficient to 0.9 and hence a value of 1 should be used in the weir menu unless the user requires a higher or lower value.

There is some scope for a clearer definition of the gate opening and operating rules. The manual does not cover the input of data for a sluice gate. It was only through the training for HYDRO-1D and a follow-up enquiry that the definitions for the operating parameters became known. It is crucial to have the correct operating data for the gate opening whether operating the gate with a fixed opening or a variable opening, with the latter being controlled by water level. For a steady flow calculation the backwater is calculated assuming the initial opening for the gate and hence for this calculation the operating rules are default parameters. However, for an unsteady flow calculation, the first time step again assumes the initial gate opening, but then shuts the gate if the calculated water level upstream of the structure is not greater than the minimum water level for gate opening. The zero flow modes were tested as a sluice and a weir the results of which are given in Table 1.2, with the water levels upstream and downstream of the structure being identical.

Due to the complex nature of the equations used to calculate flow over, or through, a sluice (Table 1 Appendix A), and the restriction imposed by HYDRO-1D to submerged flow, investigations have been made into combining a sluice and a weir in parallel. Also investigations have been made into the use of a weir in place of a sluice for the free weir flow-under gate, drowned weir flow-under gate, free gate flow, and drowned gate flow modes; the results of which are reported upon in Table 1.3.

Mode	Water Elevation at Structure		Discharge Q (m3/s)
	Upstream (m)	Downstream (m)	
Gate open - zero flow	0.300	0.300	0.000
Gate closed - zero flow	0.900	0.900	0.000

TABLE 1.2 : *Test 1 Part A - HYDRO-1D results for zero flow*

MODE	Water Elevation at Structure		Discharge Q (m3/s)	Comment
	Upstream (m)	Downstream (m)		
Free weir flow - under gate	0.383	0.348	Not Possible	weir & sluice
	0.383	0.348	Not Possible	sluice only
	0.680	0.348	0.105	sluice as a weir
Drowned weir flow - under gate	0.766	0.764	0.044	weir & sluice
	0.766	0.764	0.044	sluice only
	0.809	0.764	0.244	sluice as a weir
Free gate flow	0.489	0.348	Not Possible	weir & sluice
	0.490	0.348	Not Possible	sluice only
	0.680	0.348	0.069	sluice as a weir
Drowned gate flow	1.037	1.002	0.101	weir & sluice
	1.037	1.002	0.101	sluice only
	1.026	1.002	0.084	sluice as a weir

TABLE 1.3 : *Test 1 Part A - HYDRO-1D comparison of sluice and weir in parallel, sluice only and weir as a sluice for free and drowned weir and free and drowned gate flow modes.*

The results for the non zero flow modes using a sluice and a weir in parallel where appropriate are presented in Appendix A, Table 2, and should be treated with caution for the flow modes which require either over-gate flow or are not submerged under-gate flow.

FLUCOMP

The program FLUCOMP can model sluice structures, though the input of data was found to be rather problematic mainly due to the ASCII data structure required in the data files. Assistance was sought from the software vendors to set-up initially the data format for a sluice. Once this had been done the test was straightforward and required little effort to undertake the nine modes that were tested.

FLUCOMP requires a cross-section to be defined either side of the structure though they do not have to be directly adjacent. The user manual suggests that the cross-sections should be sufficiently close so that friction losses are small between sections yet far enough away from the sluice to be out of the influence of rapidly varying flow in the neighbourhood of the structure. The sluice gate structure is defined in the flow data file at a specific chainage within the river channel and with the following five parameters that describe the structure:

(i) invert level of the sill (m)
(ii) width of the gate (m)
(iii) gate opening (m)
(iv) height of the sluice gate (m)
(v) discharge coefficients (see appendix 2 of FLUCOMP user manual)

The modes with zero flow were not undertaken due to the inability of FLUCOMP to cope with a steady state calculation with zero flow. The results for the remaining flow modes are presented in Appendix A, Table 2.

ISIS

The setting up of the sluice gate unit in ISIS was straightforward. ISIS can cope with up to 50 identically sized sluice gates in one unit, with the gate opening being operated by either model time, water levels, logical rules defined within the data file entry, or an attached control unit. In order to undertake the zero flow modes the pseudo time stepping option was employed, whereas for the remaining modes the default steady state computation was possible. The results for the zero flow modes are given below in Table 1.4, whereas the remaining modes are presented in Appendix A, Table 2.

MODE	Water Elevation at Structure		Discharge Q (m3/s)
	Upstream (m)	Downstream (m)	
Gate open - zero flow	0.300	0.300	0.000
Gate closed - zero flow	0.900	0.900	0.000

TABLE 1.4 : *Test 1 Part A - ISIS results for zero flow modes*

It should be noted from the results in Table 1.4 that the initial conditions for the water level were set at 0.3m and 0.9m for the gate open and closed with zero flow respectively. If no initial conditions were specified for the calculations, then the water elevations upstream of the structure were calculated to be 0.151m and 0.828m for the gate open and closed (with zero flow) respectively. The effect of using different initial conditions was not investigated.

CHANNEL

CHANNEL does not have the ability to model sluice gates, however, the modes which simulate the sluice gate being closed with overflow and the modes which only require weir flow, could been modelled by treating the structure as a weir. This approach has not been undertaken as weir structures are tested in Part D of Test 1.

LD01

LD01 has the same limitations as CHANNEL and has been treated in a similar manner. Hence, no results have been presented for Test 1 Part A from LD01.

GLC-BACKWATER

GLC-BACKWATER has the same limitations as CHANNEL and has been treated in a similar manner. Hence, no results have been presented for Test 1 Part A from GLC-BACKWATER.

BAKWATER

BAKWATER has the same limitations as CHANNEL and has also been treated in the same manner. Hence, no results have been presented for Test 1 Part A from BAKWATER.

2.1A.4 Summary

1. Only ISIS and FLUCOMP can specify a single structure for both over-gate and under-gate flows, although this is a condition of limited practical application.

2. Both MIKE 11 and HYDRO-1D required approximations to the test data. MIKE 11 required two structures to be sited at the same location, with one for over-gate flow and the other for under-gate flow. HYDRO-1D required a sluice gate for under-gate flow and a weir for over-gate flow to be sited in parallel reaches. These approximations were found to be satisfactory for MIKE 11, but dependant upon flow conditions for HYDRO-1D.

3. HYDRO-1D is only designed for submerged flow conditions and is not recommended for other flow conditions.

4. MIKE 11, HYDRO-1D and ISIS all have the ability to model gates that can operate with different openings for unsteady flow conditions. However, their modes of operation and control are all different and were not tested as a requirement of this project.

5. Only MIKE 11, ISIS and HYDRO-1D were capable of directly simulating the zero steady flow modes, although this is thought to be of limited practical application (e.g. for looped systems).

6. HEC-RAS and HEC-2 are unable to model sluice gates. However, the results obtained using the bridge option, which allows both under-gate flow and over-gate flow, were not too dissimilar to those calculated by MIKE 11.

7. The sluice module in FLOODTIDE did not work and the developers were unable to resolve the problem.

8. The weir flow modes (modes 2, 3, 4 and 5) could have been undertaken with CHANNEL, LD01, GLC-BACKWATER and BAKWATER by approximation to the sluice structure, however, they have not been tested her as weir flow is tested in Test 1 Part D.

9. Overall ISIS and FLUCOMP gave discharge predictions which agreed most closely with the theoretical results.

2.1B TEST 1 : PART B - Flood Embankment

2.1B.1 Aim of Test

This test is designed to test the ability of each of the software packages to calculate flow from one trapezoidal channel (Channel A) to another parallel trapezoidal channel (Channel B) over a sloping embankment under free and drowned flow situations.

2.1B.2 Introduction

The prerequisite of being able to compute a split flow and handle multiple upstream and downstream boundaries limited the software packages suitable for this test. Only ISIS was able to complete the test as stipulated in the data file, however the following packages were able to undertake the test with some limitations: FLOODTIDE, HEC-RAS, MIKE 11 and HYDRO-1D.

Figure 1.3 illustrates the two channels (A and B), the embankment and the location of the cross sections used in Test 1 Part B. The cross-sections for both channels had a constant Manning's roughness of 0.025, a base width of 2.0m, and side slopes of 45°. The bed slope for both channels was also constant with a value of 0.0025 and the base elevations for channels A and B at the downstream boundaries are 0.0m and 0.25m respectively.

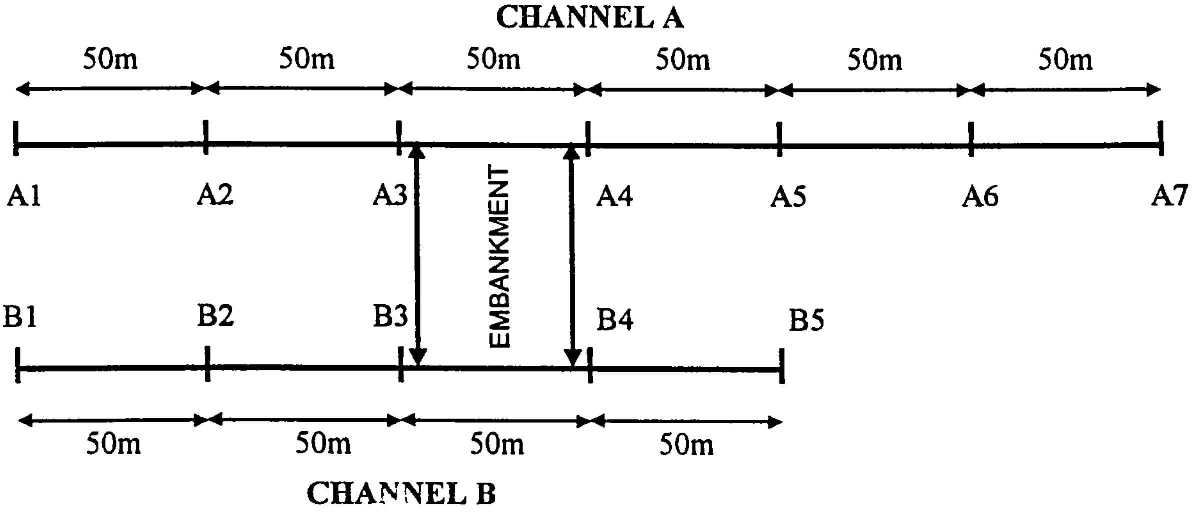


FIGURE 1.3 : *TEST 1 Part B - Plan location of cross-sections*

The embankment is 50m long and connects channels A and B between cross sections A3 and A4, and B3 and B4 respectively as illustrated in Figure 1.3. The embankment has a constant slope of 0.001 and has an elevation of 1.9 at the upstream sections and 1.85m at the downstream sections. The data set defines the embankment with a spill coefficient of 0.6 and a modular limit value of 0.9.

There were four modes of flow which were tested for Test 1 Part B, they are listed along with the boundary conditions in Table 1.5 and illustrated in Figure 1.4 below.

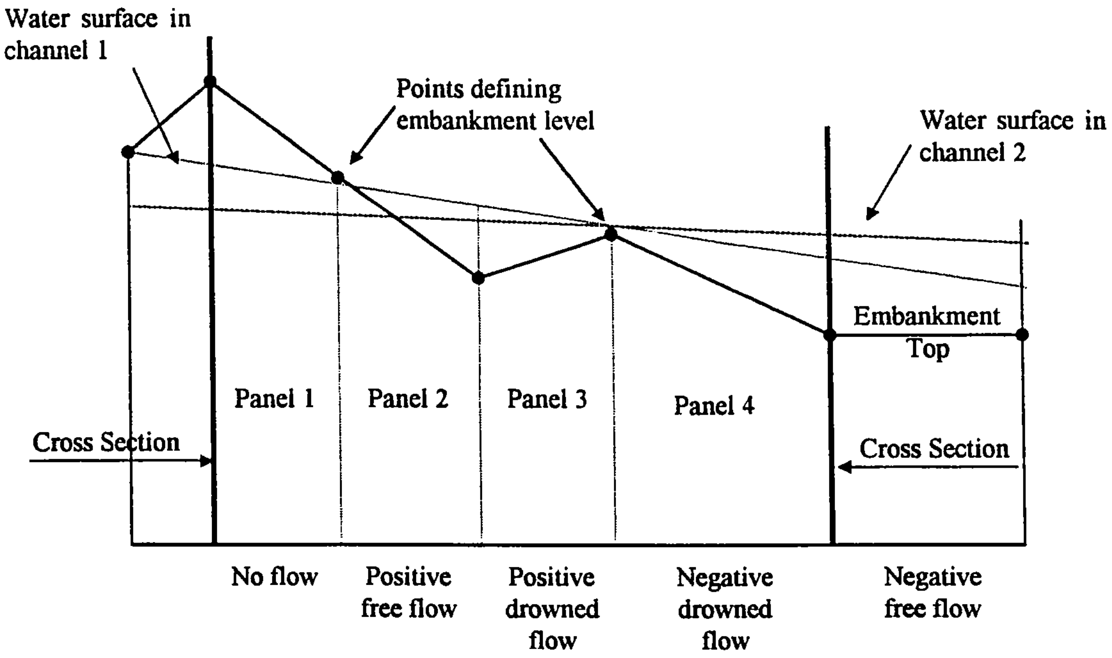


FIGURE 1.4 : *Test 1 Part B -Typical illustration of flow modes over an embankment*

2.1B.3 Results

The results from Test 1 Part B for each of the software packages are summarised in Table 1.6, and are quite different by each software package. However, it should be noted that no referenced results have been provided from the stage one report and hence comments for each of the software packages are limited to the ability of undertaking the test, rather than accuracy. A detailed breakdown of the flow, water elevation and velocity for the each package and for each mode tested is given in Appendix A, Tables 8 to 11.

Description	Channel A		Channel B	
	Downstream Water Depth (m)	Upstream Discharge (m^3/s)	Downstream Water Depth (m)	Upstream Discharge (m^3/s)
A→B : Free Flow - Positive Sense	1.547	10.00	1.850	5.00
A→B : Drowned Flow - Positive Sense	1.547	10.00	1.835	7.00
B→A : Free Flow - Negative Sense	1.550	5.00	1.900	7.00
B→A : Drowned Flow - Negative Sense	1.900	3.50	1.900	7.00

TABLE 1.5 : *Test 1 Part B - Modes of flow and boundary conditions.*

***Required link/weir to approximate test, **Result from unsteady run with fixed boundary conditions**

TABLE 1.6 : *Test 1 Part B - Comparison of calculated split flow for each package.*

FLOODTIDE

FLOODTIDE is able to cope with two channels, both with an upstream and downstream boundary conditions. The 'BANK' module in FLOODTIDE can be used to model flow over a floodbank connecting a main channel: (i) to a storage pond, (ii) between two storage ponds, or (iii) connecting two parallel channels - with the later being required in this test. Flow over flood banks in FLOODTIDE is calculated from the following weir equation(3) allowing for both free and drowned flow in both directions :

	A1→A3	A4→A7	Embankment A→B	B1→B3	B4→B5
Free Flow : Positive					
FLOODTIDE**	10.00	9.91	0.09	5.00	5.09
MIKE 11*	10.00	9.18	0.82	5.00	5.82
HYDRO-1D*	10.00	9.10	0.90	5.00	5.90
ISIS	10.00	9.53	0.47	5.00	5.47
Drowned Flow : Positive					
FLOODTIDE**	10.00	9.94	0.06	7.00	7.06
MIKE 11*	10.00	9.24	0.77	7.00	7.77
HYDRO-1D*	10.00	9.40	0.60	7.00	7.60
ISIS	10.00	9.58	0.42	7.00	7.42
Free Flow : Negative					
FLOODTIDE**	5.00	5.52	-0.52	7.00	6.48
MIKE 11*	5.00	5.91	-0.91	7.00	6.09
HYDRO-1D*	5.00	6.20	-1.20	7.00	5.80
ISIS	5.00	5.56	-0.56	7.00	6.44
Drowned Flow : Negative					
FLOODTIDE**	3.50	3.82	-0.32	7.00	6.68
MIKE 11*	3.50	4.41	-0.91	7.00	6.09
HYDRO-1D*	3.50	4.20	-0.70	7.00	6.30
ISIS	3.50	4.00	-0.50	7.00	6.50

$$Q = \left(\frac{2}{3}\right)^{1.5} g^{0.5} f C_b l h_u^{1.5}$$

where Q = flow (m3/s)
g = gravitational acceleration (m/s2)
f = drowned flow reduction factor
C_b = basic discharge coefficient
l = length of flood bank (m)

h_u = upstream flow level over bank crest (m)

The drowned flow factor, f, is a function of both upstream and downstream water levels with respect to the bank crest level (see FLOODTIDE user manual)

In the 'BANK' module of FLOODTIDE the embankment level is represented by defining the highest elevation between the two parallel channels at locations along its length, this divides the embankment into panels as previously illustrated in Figure 1.4. For this test the embankment level was defined by specifying an elevation of 1.9m at chainage 0m and 1.85m at chainage 50m. FLOODTIDE also allows a free discharge coefficient to be defined for each panel, hence a value of 0.6 was used as specified by the data set. Flow over each panel of the bank is then calculated by linearly interpolating the water levels between the upstream and downstream cross-sections. The total flow over the embankment is then calculated by adding the flow over each of the panels together.

It should be noted that other than ISIS, FLOODTIDE is the only software package which is able to model flow over an embankment as specified by the data set and that the use of junctions, connecting channels or additional weirs is not required.

When attempting to run FLOODTIDE in steady state mode as stipulated in the benchmarking data set several problems arose. In order to make a steady (and unsteady) flow calculation a critical backwater calculation was first required to provide initial conditions. The results obtained from the critical backwater calculation for all four modes were considered to be grossly inaccurate, as illustrated in Appendix A, Table 4 for the positive free flow mode. The inaccuracies in the critical backwater may be due to there being no solution with the specified boundary conditions.

In order to continue with the test the calculated backwater values were modified manually as in Appendix A, Table 5 and 6, so as to represent a more realistic initial condition (with the method being suggested during training). The values in Table 5, of Appendix A, are for a critical depth in each channel and have been calculated by hand assuming 1.0 cumecs passes from channel A to channel B over the embankment. It should be noted that the water level in channel A is significantly higher than that in channel B at the location of the embankment and that the boundary conditions have not been changed. The values used in Table A.6 are suggested values so that the water surface profile is smooth and similar in both channels at the location of the embankment. It should be noted that the areas, velocities and Froude values used are calculated according to the water levels.

Using the modified values, a backwater calculation could then be conducted to give a normal backwater result. The results from using the modified critical backwaters as in Appendix, Tables 5 and 6 produced exactly the same results that are illustrated in Appendix A, Table 7. It should be noted that the values obtained for the embankment are the same as those specified in the critical backwater solution.

The problem with this method is that the split flow needs to be defined by the user and is not adjusted by the model during a steady calculation. The resulting water levels have to be inspected to see if probable values are obtained and the split flow altered as required and a re-run made so as to produce a satisfactory result. A more accurate approach to this problem is to run an unsteady flow calculation with constant inflow at the upstream boundaries and a constant water elevation at the downstream boundaries. This method will then automatically calculate the split flow over the embankment so as to produce a stable and converged result. The test was therefore conducted using this approach with a time step of 1s, with the results being presented in Appendix A, Table 8.

To check the stability of the unsteady solution the results at several time steps were investigated. It was found that the results were exactly the same indicating that the solution had converged and that FLOODTIDE had produced a stable result.

HEC-RAS

Although HEC-RAS has the ability of defining a junction and a link-channel between the two main channels it does not have the capability of modelling an embankment or weir. The approximation of a weir by use of the bridge routine in not suited to the test since HEC-RAS is unable to calculate the split flow at the junction internally. Hence, this test has not been undertaken with HEC-RAS.

HEC-2

It is not mentioned in the manual how HEC-2 can cope with a model configuration which requires two channels, both with upstream and downstream boundaries, thus this test was not undertaken. It should be noted that HEC-2 could cope with split flow over a side weir which either loses or returns the flow to the system.

MIKE 11

It was not possible to set up this test with an embankment/side-weir between sections A3-A4 and B3-B4, as this facility is not available with MIKE 11. However a link channel with a weir placed centrally was used as a close approximation to the test conditions. The link channel had a width of 50m and bed level of 1.0m and was connected midway between sections A3-A4 and B3-B4. The length of the link channel was set at 2m, the minimum possible so as to allow the weir to be placed at mid channel, and had a Manning's roughness of 0.001 so as to be consistent with the similar testing approach used with HYDRO-1D. The weir option in MIKE 11 would not allow the sloping embankment profile to be exactly modelled since it defines a weir's width with respect to elevation, hence an approximation had to be made. Figure 1.5 below, illustrates the weir profile and the values used to define the weir. The loss coefficients used for the weir were 0.6 as required by the benchmarking data set and a value of 10 was used for calculating the Q/h tables for the weir.

ELEVATION (m)	WIDTH (m)
1.85	0.0
1.9	50.0
2.5	50.0

FIGURE 1.5 : *MIKE 11 Test 1 Part B - Illustration of weir configuration.*

MIKE 11 was easily set up for this approximated test, which made use of the split flow and diverging flow capabilities and the ability to use upstream and downstream boundaries for the two individual channels. However, it should be noted that MIKE 11 rounded the downstream boundary conditions for both the positive free and drowned flow conditions to two decimal places. Although the test was stipulated as a steady calculation an unsteady calculation was also made with a timestep of 6s to investigate any differences between the two solutions. The results from the two calculations are presented in Appendix A, Table 9, with some small differences given for the two solutions in the split flow for each flow modes with the exception of the negative free mode.

		A1→A3	A4→A7	Embankment A→B	B1→B3	B4→B5
Free Flow (Positive)	**Steady**	10.000	9.196*	0.809	5.000	5.809
	Unsteady	10.000	9.180	0.820	5.000	5.820
Drowned Flow (Positive)**	**Steady**	10.000	9.215*	0.790	7.000	7.790
	Unsteady	10.000	9.235	0.765	7.000	7.765
Free Flow (Negative)	**Steady**	5.000	5.912*	-0.912	7.000	6.088
	Unsteady	5.000	5.912	-0.912	7.000	6.088
Drowned Flow (Negative)	**Steady**	3.500	4.409*	-0.910	7.000	6.090
	Unsteady	3.500	4.412	-0.912	7.000	6.088

* Averaged value, ** Value of 20 used for weir Q/h tables

TABLE 1.7 : *Test 1 Part B – MIKE 11: Summary of calculated split flow for steady and unsteady runs.*

Table 1.7 summarises the split flow calculation for steady and unsteady calculations for each mode. It should be noted that the discharge results from all of the flow modes under steady flow conditions at sections A4-A7 showed small instabilities which have been average for Table 1.7. For the positive drowned flow calculation instabilities were found in the solution when using a value of 10 for the weir Q/h tables, however, increasing the value to 20 produced the stable result which is presented in Table 1.7 and in Table 9 of Appendix A.

As an investigative measure the Manning's *n* value for the embankment was altered to 0.025, the same as the main channel value, by doing this it was found that the results for both the steady and unsteady calculations were not changed. It was found by investigation that the results were not altered if the bed level of the link channel was reduced to 0.0m.

HYDRO-1D

It is not possible to set-up the test using a spillway or side weir as stipulated in the data file. However, the test was set-up using the junction capabilities of HYDRO-1D and a weir link between the two main channels. The setting up of the weir link was straight forward. However, to accommodate the link an extra nodal point was required between nodes 3 and 4 in each of the two main channels. Unlike MIKE 11 a link channel was not required since the weir could be directly attached to the additional nodes in each of the channels. As an approximation to the embankment the fixed weir option in HYDRO-1D was used with a width of 50m, a height of 1.875m (embankment average) and a weir coefficient of 0.6.

The results from a steady state calculation are presented in Appendix A, Table 10, and are also summarised in Table 1.6. Tests were also made with unsteady calculations for constant boundary conditions with a time step of 0.1hr. This produced exactly the same results as for the steady calculations.

FLUCOMP

The manual for FLUCOMP clearly stipulates in section 1.3 - 'Limitations of the Package that: the assumptions principally restrict the model to flows in a single channel river'. Hence this test was not possible using FLUCOMP.

ISIS

No problems were encountered with the setting-up of ISIS for this test. The forms editor was used to input the data and the SPILL module used to represent the embankment. The spill unit in ISIS calculates the flow over a jagged weir, which can either be for in line flows or lateral flows, such as those between two open channels or between an open channel and a reservoir.

The spill unit uses offset/elevation pairs to define the bank height between two channel cross sections. The unit then determines the flow over segments/panels of the weir, as illustrated in Figure 1.4, by using an integrated form of the weir equation for dry, free and drowned flow, in both the positive and negative directions. The equations of flow used by ISIS are given in Appendix A, Table 12.

Initially problems were encountered when attempting to run the test in steady mode, with the following warning displayed during calculation: 'no calculations are performed for overbank spills in the current version of the direct method'. ISIS would perform a calculation, but would not calculate any flow over the embankment. In order for ISIS to undertake this test the pseudo time stepping method for steady runs had to be adopted. This gave satisfactory results that were the same as the unsteady results, which were undertaken as an investigative measure with a time step of 1s. The results from the pseudo time stepping runs are given in Appendix A, Table 11.

CHANNEL

CHANNEL is not suited to this test as it can not cope with more than one channel, thus this test was not possible using CHANNEL.

LD01

The manual provided with LD01 makes no mention of a river system with more than one channel; it only describes the backwater calculation along a reach of a river. There is no reference to side weirs, split flow or multiple upstream and downstream boundaries, hence this test has not been undertaken with LD01.

GLC-BACKWATER

The documentation provided with GLC-BACKWATER clearly stipulates that 'the program can work out water profiles for channels in series and cannot be used directly for channels in parallel'. Hence it has not been possible to undertake this test with GLC-BACKWATER, since the test requires two channels in parallel.

BAKWATER

Although the manual provided does not list split flow calculations as a limitation of the package, it is clearly apparent that such calculations are not possible. Hence this test was not possible with BAKWATER.

2.1B.4 Summary

1. Only ISIS and FLOODTIDE have the ability of defining directly a sloping embankment that allows flow from one channel to another as required by the test specification. However, no benchmarking data were available to verify the accuracy of all of the models.

2. The critical backwater calculations from FLOODTIDE were clearly incorrect and required manual modifications in order to continue with a steady backwater and subsequent unsteady calculations. The same results were obtained for the steady backwater calculation by use of different modified critical backwater values.

3. An unsteady simulation was required by FLOODTIDE in order to calculate the split flow without having to specify manually the division of flow.

4. The test had to be approximated in HYDRO-1D by specifying a junction in each channel with a weir connecting the two. The sloping embankment crest was approximated by setting the crest level equal to the average embankment level. This approximation appeared to give sensible (but non-verifiable) results.

5. The test had to be approximated in MIKE 11 by defining a link channel between the two main channels and with a weir placed centrally in the link channel. The weir geometry was defined according to elevation and width so as to calculate Q/h tables for flow over the weir. This approximation again appeared to give sensible (but non-verifiable) results.

6. The test could not be approximated with the bridge option in HEC-RAS, as it is unable to calculate automatically the split flow.

7. HEC-2, FLUCOMP, CHANNEL, LD01, GLC-BACKWATER and BAKWATER are all unable to undertake this test as they are limited to single channel networks.

8. The lack of benchmarking data for this test makes it impossible to assess the accuracy of any of the software packages. However, with the exception of FLOODTIDE for the two

positive flow modes and HYDRO-1D for the negative free flow mode, the results from all of the appropriate models indicated similar general trends.

2.1C TEST 1 : Part C - Land Drainage Pump

2.1C.1 Aim of Test

This test is designed to test the ability of the software packages to incorporate a land drainage pump between two river cross-sections which is regulated by Q/h conditions.

2.1C.2 Introduction

Only MIKE 11, HYDRO-1D and ISIS have the capability of modelling flow with a pump as a regulating structure. However, only ISIS and HYDRO-1D were able to complete this test from the information available in the data set. MIKE 11 required alternative operating variables for the pump (see comments for each package) which could not be obtained from the data set provided. The following packages, according to their manuals, are not intended to cope with a pump in any manner and thus were not considered to be applicable for this test:

FLOODTIDE
HEC-RAS
HEC-2
FLUCOMP
CHANNEL
LD01
GLC-BACKWATER
BAKWATER

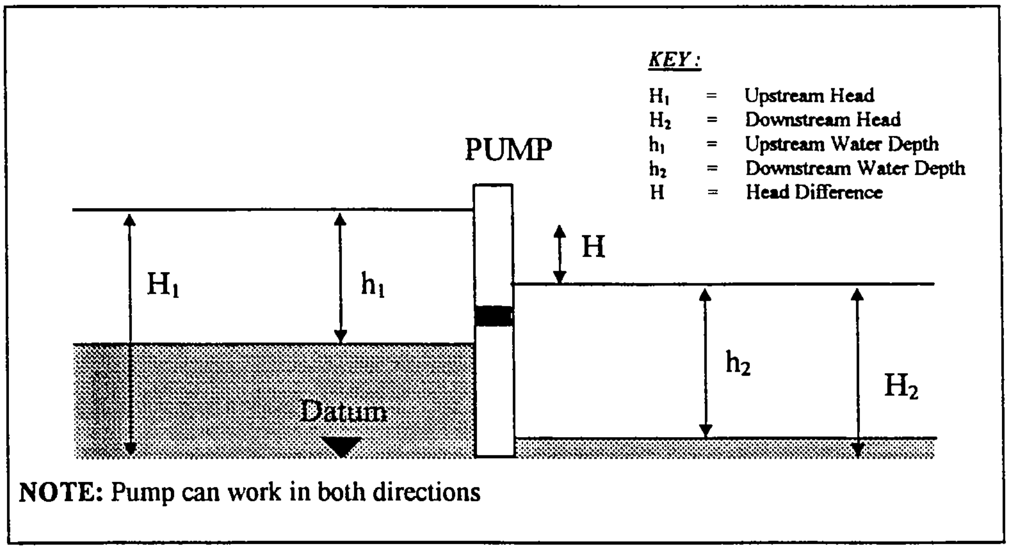


FIGURE 1.6 : *TEST 1 Part C - Schematic illustration of pump configuration*

Figure 1.6 schematically illustrates the pump configuration and the possible variations that can be considered for the head by each package. It should be noted that the test data provided for the pump was as follows:

980				Full Operating Speed (RPM)
1				No. of equivalent pumps in parallel
2				No. of speed, head, flow and efficiency data sets
740	25.3	1.20	0.71	Speed (RPM), Optimum Head (H), Optimum Flow (Q), Efficiency
980	44.6	1.48	0.71	Speed (RPM), Optimum Head (H), Optimum Flow (Q), Efficiency

2.1C.3 Results

It should be noted that there are no referenced results to allow a benchmarking of the software packages. The testing method and procedure adopted by the software packages that are capable of undertaking calculations with pumps is reported below.

MIKE 11

Although MIKE 11 can cope with regulating structures it could not cope with the data as provided for this test. The two types of regulating structures that can be modelled are defined in Menu A.5.6 and Menu A.5.7 in MIKE 11's GUI and are described in sections 2.2.6 and 2.2.7 of the manual. The regulating structures can define flow by either $Q = Q(t)$ i.e. discharge is a known function of time, or $Q = Q_a \times f(Z_b)$ i.e. discharge is regulated by flow at location *a*, and is a function of the water depth at *b*. The Danish Hydraulic Institute was contacted with respect to the possibility of undertaking this test for the data set that was available. Their response indicated that the test might be possible if attempted as described below.

> *To model a Q/H regulated pump is not a straightforward procedure and is certainly not user friendly. Firstly a gate control structure is required to resemble the Q/H pump in the system and should be regulated as a function of dH. Then the gate level should be adjusted for various dH values in order to achieve the desired Q. Then a pump should be included in the system, downstream of the gate and modelled by a Q/dH relationship - setting the discharge through the regulator equal to that through the gate structure. Then the gate level should be regulated by the dH value upstream and downstream of the pump.*

It was decided not to undertake this test with MIKE 11 as described above as it was considered to be beyond the scope of the current study and hence has been omitted from the testing. It should be noted that a methodology exists to carry out a simulation for a pump regulated in such a manner as described in this test.

HYDRO-1D

Two types of pump can be specified by HYDRO-1D. Pump type 1 can accommodate up to 5 different pumps, each with its own specified constant discharge with a cut-in and cut-out water level and pump type 2 requires the user to input a rating curve (up to 8 points) for flow and head. The manual for HYDRO-1D does not define what head value is required nor the order in which the data should be input. However, through technical support it became apparent that the head was the head difference H as defined in Figure 1.6, and that the rating curve data should be put in starting with the smallest head difference. It became apparent through undertaking this test that by inputting the rating curve data in the reverse order, significantly different results can be obtained by HYDRO-1D. This clearly demonstrated that the rating curve data must be input in the correct order. The rating curve values used in the test for HYDRO-1D were obtained from the rating curve values output by ISIS. Table 1.8 below gives the 8 values that were used for the rating curve in HYDRO-1D:

Q (m3/s)	H (m)
3.24	-55.0
2.71	-13.2
2.30	11.6
1.98	27.4
1.71	38.5
1.48	44.6
1.28	48.9
1.11	51.30

TABLE 1.8 : *TEST 1 Part C – HYDRO-1D: Rating curve values.*

In order to obtain a converged solution under steady flow conditions the maximum number of iterations in the calculation settings required a value of 100. Table 1.9 below gives the results obtained for this test from HYDRO-1D for a steady run.

NODE	Flow (m^3s^{-1})	Stage (m)	Velocity (ms^{-1})
A1	2.000	3.608	0.59
A2	2.000	3.588	
PUMP			
A3	2.000	30.00	0.08
A4	2.000	30.00	

NOTE: Velocity is for reach.

TABLE 1.9 : *TEST 1 Part C – HYDRO-1D: Results from pump test.*

ISIS

The setting up of this test was simple and took very little time to set-up using the data set provided. However, running a simulation was not as simple as envisaged at the outset. For a steady flow calculation the direct step method of calculation was not possible, hence the pseudo time stepping method of calculation was required. The manual describes the method of calculation and clearly highlights the importance of initial conditions and suggests time steps that could be adopted when conducting such calculations. For this test a time step of 60 seconds was first used until the flow convergence ratio was suitably reduced, then increased to 100 seconds for several more time steps and finally increased to 500 seconds until the ratio was at a stable value of around 10^{-5}. Table 1.10 below gives the results obtained for this test from ISIS.

It should be noted that ISIS internally calculates an operating curve (table) for the pump according to the operating values defined for the pump (see test data description) in the forms editor. The Q/H relationship defines the head H as the difference in head between the two faces of the pump and not the head/water level at either the upstream or downstream face of the pump. The operating table, which is stored internally by ISIS, can be printed out for inspection if required. The rating curve as produced by ISIS is illustrated in Appendix A, Graph 1.

NODE	Flow (m^3s^{-1})	Stage (m)	Froude	Velocity (ms^{-1})
A1	2.000	3.717	0.095	0.561
A2	2.000	3.700	0.092	0.548
PUMP				
A3	2.000	30.00	0.005	0.082
A4	2.000	30.00	0.005	0.080

TABLE 1.10 : *TEST 1 Part C – ISIS: Results from pump test.*

2.1C.4 Summary

1. Only ISIS and HYDRO-1D were capable of undertaking this test, which involved a bi-directional pump being operated according to the head difference across the pump. However, this test was limited in that only one operational pump mode was considered.

2. MIKE 11 has the ability to model pumps or related regulating structures (such as a syphon) if the required data are provided. However, for this test the data relating to the pump operation were not suitable to MIKE 11, hence this feature of MIKE 11 could not be tested.

3. FLOODTIDE, HEC-RAS, HEC-2, FLUCOMP, CHANNEL, LD01, GLC-BACKWATER and BAKWATER were not able to model a pump.

2.1D TEST 1 Part D - Crump Weir

2.1D.1 Aim of Test

This test is designed to test each software package with a crump weir in series under the following three flow conditions:

- Dry crest
- Free discharge
- Drowned discharge

2.1D.2 Introduction

The test configuration for Test 1 Part D required four cross sections each with a constant Manning's roughness of 0.014, rectangular profile, width of 1.0m and height of 1.5m. The location of the crump weir with respect to the cross-sections is illustrated below in Figure 1.7. The bed slope for the downstream and upstream reaches was 0.0005 and 0.001 respectively. The parameters defining the weir are presented in Table 1.11. A downstream boundary water elevation of 0.67m and an upstream inflow boundary condition of 0.15 cumecs was specified for the test.

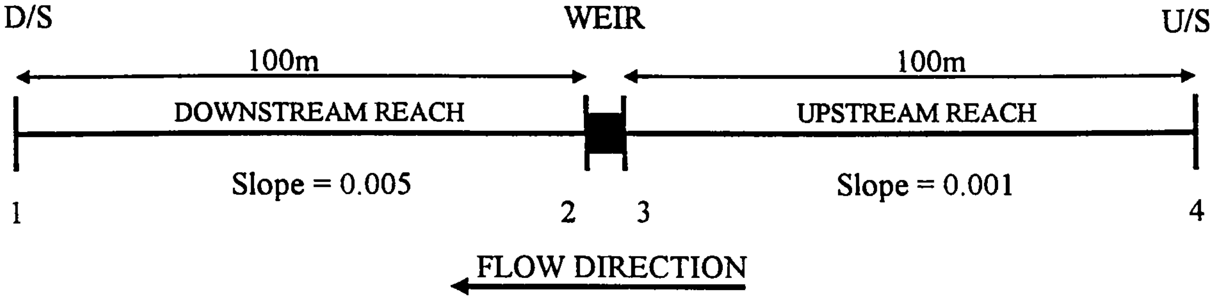


FIGURE 1.7 : *TEST 1 Part D - Plan location of cross-sections and weir*

It should be noted that only FLOODTIDE, FLUCOMP, ISIS, LD01 and GLC-BACKWATER were able to set up the test with a crump weir as stipulated in the data set. MIKE 11, HYDRO-1D and BAKWATER all approximated the test by use of alternative weir structures available by each of the packages. Hence the results obtained from these packages should be treated with caution especially when comparing the theoretical discharge values. Both HEC-RAS and HEC-2 are unable to model weirs, however the test was approximated by the use of the bridge capabilities of both packages.

PARAMETER	VALUE
Calibration Coefficient	1.0
Breadth of weir crest	0.90 m
Elevation of weir crest	0.50 m
Height of crest above upstream bed	0.45 m
Height of crest above downstream bed	0.45 m

TABLE 1.11 : *TEST 1 Part D - Weir parameters.*

2.1D.3 Results

Not all of the packages can cope with a weir, though some packages have been used to represent a weir by alternative means. Table 1.12 below gives the change in water level across the weir structure for each of the software packages tested in the free and drowned flow modes. In order to assess the accuracy of the results obtained from each of the software packages the following equation for discharge from White(4) have been used as a guide.

$$Q = f\ C_c\ C_d\ \sqrt{g}\ b\ H_1^{1.5}$$

where :
f = drowned flow coefficient
C_c = lateral contraction coefficient
C_d = discharge coefficient
g = acceleration due to gravity (m/s^2)
b = weir width (m)
H_1 = total upstream head above weir crest (m)

NOTE : *The discharge has been calculated for each of the software packages by using the calculated upstream head above the weir (H_1). The drowned flow coefficient has been set to one for the free flow condition and has been given by White(4) for drowned flow based on experimental work.*

The theoretical discharge for each software package in Table 1.12 was obtained using the calculated upstream total head above the weir crest in the above equation.

It can be seen from Table 1.12 for the free flow mode that the results for all of the packages are very similar to one another, with the exception of MIKE 11 and LD01, and for the drowned flow mode the results are very different.

SOFTWARE	Change in water elevation across structure (m)		Theoretical [1] discharge over weir (cumecs)	
	Free Discharge	Drowned Discharge	Free Discharge	Drowned Discharge
FLOODTIDE	0.362	0.019	0.153	0.144
HEC-RAS [2]	0.367	0.080	0.179	0.231
HEC-2 [2]	0.00	0.00	n/a	0.101
MIKE 11 [2]	0.421	0.063	0.209	0.208
HYDRO-1D [2]	0.332	0.060	0.140	0.205
FLUCOMP	0.363	0.037	0.175	0.168
ISIS	0.343	0.022	0.149	0.149
CHANNEL	1.370	1.040	2.382	2.228
LD01	0.392	0.030	0.210	0.201
GLC-BACKWATER	0.34	0.03	0.151	0.165
BAKWATER [2]	0.35	0.03	0.163	0.154

1. Actual discharge should be 0.15 cumecs 2. Assimilated test set-up used.

TABLE 1.12 : *TEST 1 Part D - Change in water level across weir structure and theoretical flow over weir for free and drowned flow conditions*

FLOODTIDE

The modelling of a weir with FLOODTIDE is a straightforward procedure which is clearly explained in the manual. There are seven types of weirs that can be modelled, including: (i) basic, (ii) rectangular profile crest, (iii) round nosed profile crest, (iv) crump weir, (v) flat-V profile crest, (vi) Ogee profile crest and (vii) natural profile crest. The crump weir was selected for this test as defined by the benchmarking data set.

The crump weir requires an upstream and downstream cross section, either side of the structure, thus allowing the test to be set up as stipulated by the benchmarking data set. The height of the weir crest above datum is required (0.5m) as well as the height of the crest above the upstream bed (0.45m). However instead of defining the height of the crest above the downstream bed, the slope of the downstream apron is required and was set at 2 (i.e. 1:2) - which is the smallest value possible. It should be noted that FLOODTIDE does not take into account user defined calibration coefficients for the structure. The results for the free and drowned discharge modes are given in Appendix A, Table 13, with the dry crest mode not being included since calculations were not possible with zero flow under steady state conditions.

It should be noted that FLOODTIDE uses the same equation as given previously for the theoretical discharge over the weir. The FLOODTIDE user manual clearly describes the method of calculation adopted by FLOODTIDE and refers the reader to Ackers(5) for further information.

HEC-RAS

With the current version of HEC-RAS it is not possible to model a weir, although a weir option does appear in the geometric data windows of the GUI. During the training for this package it was however suggested that the bridge routine be adopted as an approximation, as was the case for a sluice gate.

The bridge routine was very simple and easy to set up, with the cross sections either side of the structure being placed 0.5m away from the bridge. The bridge deck was set to that of the weir crest and the bridge opening set at 0.01m (the minimum possible width). The following options which were available by HEC-RAS for the bridge computation were selected: (i) momentum equation, (ii) high flow method, (iii) submerged inlet & outlet coefficient = 0.8, i.e. the default value. Expansion and contraction coefficients of zero were adopted (used for shock) for the calculations, the results of which can be found in Appendix A, Table 13 for the free and drowned flow modes. The dry crest mode is included since calculations were not possible with HEC-RAS for zero flow.

It is possible with HEC-RAS to extract the calculated discharge values for both underflow and overflow for the bridge. For the free flow mode the underflow and overflow was calculated as

0.05 and 0.10 cumecs respectively, and for the drowned flow mode as 0.02 and 0.13 cumecs. It should be noted that this method of undertaking the test permits flow under the bridge/weir for situations when the water level is below that of the weir crest/bridge deck.

The results from HEC-RAS should be treated with caution since both full orifice and weir flow equations are taken into consideration in the computation. The following orifice and weir equations used by HEC-RAS are as follows:

ORIFICE FLOW : $Q = C\,A\,\sqrt{2gH}$

WEIR FLOW: $Q = \sqrt{\dfrac{2gH}{K}}$

where : C = Coefficient of discharge for fully submerged pressure flow
A = Area of opening (m)
H = The difference between the energy gradient elevation upstream and the water surface elevation downstream (m)
K = Total loss coefficient

HEC-2

It is not possible to model a weir with HEC-2, however the same approach using a bridge routine as for HEC-RAS was adopted and the comments as for Test 1 Part A also apply here. The results presented for Test 1 Part D in Appendix A, Table 13, show a poor correlation to other packages that can model weirs and hence the use of this method for modelling a weir should be treated with caution. It should be pointed out that a zero water level difference was produced across the structure by HEC-2 for both the free and drowned flow cases. The water level upstream of the structure for free flow is also lower than the crest level. This clearly gives incorrect results.

MIKE 11

To model a crump weir with MIKE 11 the special weir option should be used, however, the user is required to specify Q-h relationships corresponding to free overflow conditions. Since the Q-h relations for the Crump Weir were not known, the broadcrested weir has been adopted as an approximation test. This will account for the higher water level calculated upstream of the weir in comparison with the Crump Weir case. To model a broadcrested weir it is necessary to specify inflow, outflow and free flow head loss factors for both positive and negative flow situations. The default values of 0.5 and 1.0 respectively for inflow and outflow head loss coefficients were used for the computation. The geometry of the weir was defined with a constant width of 0.9m for all elevations starting from 0.5m up to 1.5m and with a Q/h table calculated over the same range. The user manual (section 2.2.3) adequately explains the required input parameters for the broadcrested weir and the important features for the set-up of the river system. It is not possible to place the cross sections next to the structure with MIKE 11. As the nearest approximation they were placed 1.0m away from each face of the structure (minimum possible). The results for all three modes of flow are given in Appendix A, Table 13. However, it should be noted that the results for the zero flow mode are dependent upon the initial conditions supplied in the supplementary data file.

HYDRO-1D

HYDRO-1D is only able to model a fixed weir between two nodes (cross-sections) with the following user definable settings:

(i) upstream and downstream velocity head coefficients - both set to 1.0

(ii) coefficient C - set to 1.0, where $Q = 1.706\, C w h^{n}$,

(iii) width of weir w - set to 0.9m,

(iv) exponential coefficient n - set to 1.0 (multiplication factor)

(v) crest level of weir - set to 0.5m.

It should be noted that HYDRO-1D internally sets the 'n' value in the above discharge equation to 1.5 and that the value defined is a multiplication factor. Also the height of the crest level above the upstream and downstream channel cannot be defined along with the calibration coefficient.

The setting up of HYDRO-1D was simple and straightforward using the values as defined above. However, the manual does not give details on the set-up of a weir and gives no indication of suitable values for coefficients. The results for both the free and drowned flow modes are presented in Table 13, of Appendix A, and should be considered while taking into account the specifications for the weir definition. The dry flow mode was undertaken and resulted in a water level of 0.4m along the complete length of the river network.

FLUCOMP

The modelling of a weir is relatively straightforward and is adequately explained in the manual provided for FLUCOMP(6) (Section 8.2 and Table 13.6). However, it should be noted that FLUCOMP only includes formulae to compute flow over horizontal crested weirs. If the user wishes to model other types of weirs then they may either define the discharge properties of an equivalent horizontal crested weir or they must use the flow equations for the actual structure in the computer code.

FLUCOMP uses the following equation to calculate the discharge over a weir :

$$Q = \left(\frac{2}{3}\right)^{1.5} C_w \; B \; f \; \sqrt{g} \; h_u^{1.5}$$

where : C_w = modular flow discharge coefficient
B = breadth of weir crest (m)
f = drowned flow function
g = acceleration due to gravity (m/s^2)
h_u = water level upstream of weir (m)

To set up a weir it is required that the following values/coefficients be defined in the flow data file: (i) structure identification character - type 26, (ii) chainage of structure, (iii) the elevation of the weir crest - set to 0.5m, (iv) the breadth of the weir crest - set to 0.9m, (v) the weir modular discharge coefficient C_w - set to 1.00 (typical value suggested in manual), and (vi) the weir drowning ratio D_r - set to 0.75. The weir record does not take into account the height of the weir crest above the bed of the upstream or downstream channel.

The results file produced by FLUCOMP provides information for each cross-section and in addition at the end of the file a breakdown of the flow through the structure is given defining: (i) the upstream and downstream stages, (ii) the discharge over the weir, (iii) the discharge over the flood plain (if required), and (iv) the total discharge. The results for both the free and drowned flow modes are given in Appendix A, Table 13, with the dry flow mode not being included since calculations are not possible for zero flow. As an investigative measure the drowning ratio D_r was increased to 1.0. This did not alter the change in the water elevation across the structure for the free flow mode, but for the dry flow mode decreased the change to 0.037m.

ISIS

Several types of weirs can be modelled by ISIS including: (i) crump, (ii) flow-head control, (iii) gated, (iv) notional, (v) round nosed broad crested, (vi) sharp crested, (vii) syphon, and (viii) weir - for this test the crump weir was selected.

The equations used by ISIS to compute the flow over a crump weir are taken from White(4) and are the same as that expressed at the beginning of this section. It should be noted that ISIS internally stores the coefficients of discharge and the drowned flow reduction factors (based on the curve for the ratio of upstream and downstream total head with no truncation of the weir) from Figures 5 and 11 of White(4) respectively.

The forms editor enabled quick configuration and set-up of the test for each of the three modes considered. For the free and drowned flow modes the direct method for steady computations was available, however for the dry flow mode the pseudo time stepping method for steady runs had to be adopted. The results file produced by ISIS reports the mode that has been adopted for the calculation, along with the standard output values for each cross section. The results for the free and drowned flow modes can be found in Appendix A, Table 13. The dry flow mode produced a constant water elevation of 0.4m along the length of the river system, which remained the same as the initial conditions at each node. For a simulation with no initial conditions specified, the water depth was calculated at 0.4m at each node downstream of the structure and at 0.245m upstream of the structure. This clearly demonstrates that ISIS requires appropriate initial conditions if acceptable results are to be obtained from a zero flow calculation.

CHANNEL

The inability of CHANNEL to cope with structures has resulted in this test being undertaken as an approximation to the benchmarking data. To model this test the bed level at the location of the weir was raised to approximate the weir profile. The results for the free and drowned flow modes from this simulation can be found in Appendix A, Table 13, and should be treated with caution. No calculations have been made with the dry flow mode, as CHANNEL is unable to cope with zero flow.

LD01

LD01 has the ability to model weirs, however the archaic nature of the input data only defines the: (i) weir crest level - set to 0.5m, (ii) weir crest width - set to 0.9m, and (iii) weir thickness - set to 0.1m. The dry flow mode has not been possible as calculations with zero flow cannot be undertaken, however the results for the free and drowned flow modes have been presented in Appendix A, Table 13. It is possible with LD01 to print out cross-section information and the parameters defined for the weir. This is a useful method for checking the input of data by the user. It should be noted that LD01 internally calculates the weir discharge coefficients according to King(7) for free flow and by Van Beeston's curves for submerged flow.

GLC-BACKWATER

The modelling of a weir by GLC-BACKWATER is covered in Section 8.9 of the user manual. Both weirs within the watercourse and side spill weirs can be modelled. It is required to define the weir in two stages, firstly the weir parameters and secondly the weir configuration data. The weir parameters define the: (i) chainage of the downstream face of the structure, (ii) flag code for a weir, (iii) number of weirs, (iv) width of weir upstream, and (v) C_d multiplication factor. The weir configuration data define the: (i) downstream face of the structure, (ii) crest level, (iii) width of weir, (iv) height of crest above the upstream and downstream bed, (v) crest level difference, (vi) weir length, and (vii) type of weir - type 2 selected. In all, there are nine weir types that can be modelled and are listed as follows:

1. Flat V weir
2. Triangular weir (crump)
3. Thin plate
4. Rectangular (broad crested)
5. Round nosed broad based weir
6. Rectangular throated flume weir
7. Trapezoidal throated flume weir
8. Sharp drop, i.e. change in bed level
9. Side-spill weir

Each of the above weir types have a default C_d value as defined in Table 13 of the manual(8), which can be altered by the use of the C_d multiplication factor. The manual makes reference to the program's ability to adjust automatically the C_d value of a weir when the flow is non-modular, but makes no reference to the method of calculation for each type of weir.

The ability to define the weir as mentioned above enabled GLC-BACKWATER to be set-up as described by the benchmarking data set. The results from the calculations for both the free and drowned flow modes can be found in Appendix A, Table 13. The dry flow mode has not been undertaken, as the situation of zero flow cannot be modelled with GLC-BACKWATER.

BAKWATER

The inability of BAKWATER to cope with structures has resulted in this test being undertaken as an approximation to the benchmarking data as suggested during training. The manual states that BAKWATER does not calculate structure head losses, though discrete head losses can be added at any defined chainage. To model this test instead of using discrete head losses (unknown), the bed level at the location of the weir was raised to approximate the weir profile. The results for the free and drowned flow modes from this simulation can be found in Appendix A, Table 13. No calculations have been made with the dry flow mode, as BAKWATER is unable to cope with zero flow.

2.1D.4 Summary

1. FLOODTIDE, FLUCOMP, ISIS, LD01 and GLC-BACKWATER were directly able to model the crump weir. MIKE 11, HYDRO-1D and BAKWATER all approximated the test by use of alternative weir structures. However, this test was again limiting since only one weir type was considered.

2. Both HEC-RAS and HEC-2 required the test to be undertaken using the bridge capabilities within each package.

3. All of the packages produced water level differences across the weir and these differences were then converted to a discharge, which was compared to the steady state inflow. Several packages produced reasonable agreement, including: FLOODTIDE, ISIS, GLC-BACKWATER and BAKWATER. However, the other packages, notably CHANNEL, produced relatively larger discrepancies in the discharge but this is not surprising since the packages do not currently include a crump weir.

4. ISIS, MIKE 11 and HYDRO-1D were able to undertake this test for zero steady flow conditions with the results from each package being the same. Although the other models are not specifically set-up for this test configuration, the developers could easily modify their packages accordingly.

5. HEC-2 and HEC-RAS, which use the bridge routine to undertake this test, define the bridge structure in different ways. This may account for HEC-2 under-predicting the head loss across the structure, whereas HEC-RAS over predicts the head loss across the structure.

2.2 TEST 2 - Simple Steady State Looped Model

2.2.1 Aim of Test

Test 2 is designed to assess the ability of each software package to calculate a diverging and converging flow scenario, i.e. a looped system. Although stage one of this benchmarking study defines this test as a steady state test investigation, the unsteady flow models have also been tested using steady boundary conditions. This was done in order to establish possible numerical instabilities in the steady state calculation and to isolate any potential problems with an unsteady calculation.

2.2.2 Introduction

The configuration of the test as stipulated by the data set is illustrated in Figure 2.1. Essentially there are four reaches: Reach 1, Reach 2, Reach 3 and Reach 4. Each reach is defined with five cross sections with a constant Manning's *n* value of 0.012, 0.0125, 0.013 and 0.0135 for Reaches 1 through to 4 respectively. At the downstream end of Reach 1 the system diverges (splits) into Reaches 2 and 3, which then converge to Reach 4 at their downstream sections.

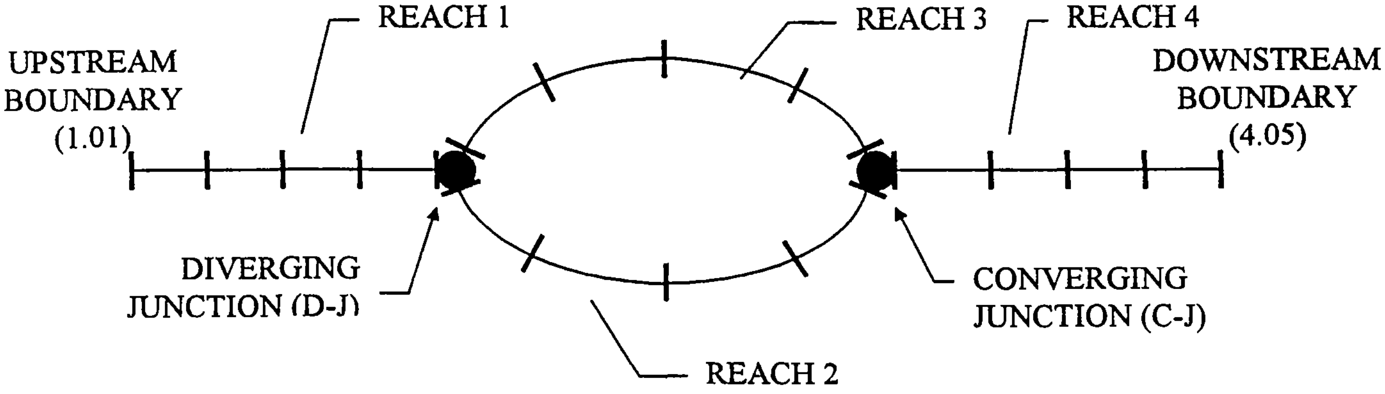


FIGURE 2.1 : *TEST 2 - Schematic Illustration of Test Configuration*

At each junction three cross sections are defined (one for each reach), though no information is provided by the data set for head losses or skew angles or method of computation (water level or energy). It should be noted that Reach 2 is three times as long as Reach 3 with the same vertical drop between the two junctions.

2.2.3 Results

In the phase one report, results of discharge and water depths for all four Channels of Test 2 are calculated according to Schulte and Chaudhry(9). Table 2.1 compares the results from each of the software packages to the referenced discharge through each of the four Channels and the water elevations at the boundaries and junctions. It can clearly be seen from Table 2.1 that the results differ significantly between the software packages. Both ISIS and HYDRO-1D calculated a split flow and water surface profile that is very similar to the referenced results. However, FLOODTIDE and HEC-RAS calculated an almost equal split flow, which is significantly different from the referenced results. It should be pointed out that multiple solutions may be obtained by FLOODTIDE for a steady run. The test method adopted for each package to calculate the split flow at a junction is listed in Table 2.2.

	SOFTWARE	Reach 1	Reach 2	Reach 3	Reach 4
Calculated Discharge (m^3/s)	**REFERENCED**	**250.00**	**75.181**	**174.819**	**250.00**
	FLOODTIDE[1]	250.00	125.00	125.00	250.00
	HEC-RAS	250.00	122.30	127.70	250.00
	MIKE 11	250.00	96.710	153.290	250.00
	HYDRO 1D	250.00	75.5	174.5	250.00
	ISIS	250.00	74.24	175.76	250.00
	SOFTWARE	Section 1.01	Diverging Junction	Converging Junction	Section 4.05
Calculated Water Elevation (m)	**REFERENCED**	**3.061**	**3.048**	**3.012**	**3.000**
	FLOODTIDE[1]	3.196	3.175	3.020	3.000
	HEC-RAS	3.149	3.137	3.061	3.000
	MIKE 11	3.084	3.070	3.010	3.000
	HYDRO 1D	3.070	3.057	3.015	3.000
	ISIS	3.064	3.052	3.011	3.000

NOTE : [1]using equal split flow in initial conditions.

TABLE 2.1 : *TEST 2 - Calculated flow distribution and water elevation for each package*

TEST 2 – Simple Steady State Looped Model

SOFTWARE	Method
FLOODTIDE	Water elevation balance
HEC-RAS	Energy level balance
MIKE 11	Water elevation balance
HYDRO 1D	Water elevation balance
ISIS	Water elevation balance

TABLE 2.2 : *TEST 2 – Method adopted by each software package to balance flows at a junction*

FLOODTIDE

The manual describes the method that should be adopted when a diverging or converging flow situation is to be modelled. The 'CONNECTION' module is used to define the location of the junction and the connecting cross-sections at that location; hence the benchmarking data set could be employed as directed. The ASCII format that is required to set-up the test provided a simple and straightforward method of undertaking the test.

The 'CONNECTION' module assumes that the difference in the velocity head at the connected labels is insignificant and therefore sets the water surface as equal. If the velocity head is to be considered then the 'JUNCTION' module should be used(10). The 'JUNCTION' module requires the definition of head losses hence it was not suitable for this test.

The main problem connected with FLOODTIDE and split flows under steady conditions is the manual iterative procedure that has to be undertaken to find the correct solution. Once a backwater calculation has been completed the computed results of the water elevations have to be inspected and compared at the split for each branch to see if they are equal. If the water levels are not equal, then the split flow used in the initial conditions was incorrect and needs to be adjusted so as to take account of the discrepancy. A re-run then has to be undertaken. This has to be done repeatedly until the user is satisfied that the results obtained are an accurate solution. However, it was found through investigation that the water levels were balanced at the junctions when various different split flows were specified. For example when an even split of 125cumecs was specified the result was almost identical to that when the known referenced split flows were used. This would suggest that this approach is not a suitable method for determining split flows with FLOODTIDE.

An alternative approach to calculate the split flow in a looped system, suggested during training, is to undertake an unsteady calculation using constant boundary conditions. This method was adopted, but produced unsatisfactory results since solutions at different time intervals produced conflicting results. Investigations were undertaken by altering the time step used in the calculation, which again did not lead to a satisfactory solution. The software vendors were approached with this problem and were unable to offer a remedy and could not explain this occurrence.

HEC-RAS

The GUI used to set up a model configuration provided a very simple and convenient method of setting up the model network. However, it should be noted that the GUI would not allow the required network to be defined. HEC-RAS would allow the two looped reaches to be defined (reaches 1 and 2) and one of the two remaining reaches (reaches 1 and 4). However, when the last reach was defined the program failed with the error "subscript out of range". This was overcome by building the network with an extra junction, located mid-length along Reach 2.

The model uses junctions to account simply for the splitting or converging of flows to/from as many branches as desired. Two or more cross sections are required (from either the beginning or end of a reach) to connect to a junction and the distance between cross-sections has to be specified in the junction data menu. In this test a value of 0.0m was used for the distance between cross sections, with values larger than this not being investigated. There is an option to use either the momentum or energy balance computation for the solution. In this test the energy option was selected as an appropriate method. It should be noted that the momentum option allows the user to specify a tributary angle and, if required, friction and weighting terms can be added to the computation.

HEC-RAS does not calculate the spilt flow internally; the user is required to suggest an initial discharge through each reach and then compare the calculated energy values as described very clearly in the manual (page 4-11) and then redistribute the flow accordingly in a manual iterative procedure. This method was rather tedious for this simple test and for larger river systems could pose problems and inconsistencies for combined flow situations. Table 2.3 gives the results at the end of each channel reach as given by HEC-RAS for this test. It should be noted that the addition of the extra junction in Reach 2, as previously mentioned, produced no energy, water level or velocity changes from the cross-section immediately upstream of the junction to the cross-section immediately downstream of the junction.

Cross Section	Q (m^3/s)	Water Elevation (m)	Velocity (m/s)	Energy Elevation (m)
1.01	250.0	3.149	1.902	3.334
1.05	250.0	3.138	1.870	3.316
2.01	122.3	3.215	1.409	3.316
2.05	122.3	3.130	1.147	**3.197**
3.01	127.7	3.060	2.244	3.316
3.05	127.7	3.042	1.742	**3.197**
4.01	250.0	3.013	1.898	**3.197**
4.05	250.0	3.000	1.873	3.179

TABLE 2.3 : *TEST 2 - HEC-RAS : Summary of results*

HEC-2

The split flow option in HEC2 can only cope with situations where flow is lost from the main channel, over levees or weirs, over topping of the watershed, and divisions/splits created by diversion structures. It is not possible for HEC2 to model the test situation that requires the flow to split into two channels and then converging flow as required by the benchmarking data set. Hence, no Test 2 results have been obtained for HEC2.

MIKE 11

It was straightforward to set-up and run this test for MIKE 11. This model can easily cope with the required split and converging flow configuration. It should however be noted that MIKE 11 can only input Manning's number to three decimal places, whereas Reaches 2 and 4 require Manning's number to four decimal places. This may account for some discrepancy in the results when compared to other software packages.

Although the test is stipulated in the stage one report as being a steady state test, both steady and unsteady runs were undertaken for a constant water elevation and discharge at the appropriate boundaries. The results from the two runs were very similar and are shown in Table 2.4. It should be noted that the main difference in these two simulations is the difference of 0.577 cumecs in the split flow. It was found that oscillations occurred in the solution for the unsteady mode when using a time step of 6s although for a time step of 0.6s stable results were produced.

	Steady Run			Unsteady Run		
Cross Section	Q (m^3/s)	Water Elevation (m)	Velocity (m/s)	Q (m^3/s)	Water Elevation (m)	Velocity (m/s)
1.01	250.0	3.084	1.956	250.0	3.085	1.955
1.05	250.0	3.070	1.925	250.0	3.071	1.925
2.01	96.710	3.070	1.191	97.287	3.071	1.192
2.05	96.702	**3.010**	0.951	97.287	**3.010**	0.952
3.01	153.290	3.070	2.677	152.713	3.071	2.671
3.05	153.290	**3.010**	2.115	152.713	**3.010**	2.111
4.01	250.0	**3.010**	1.899	250.0	**3.010**	1.899
4.05	250.0	3.000	1.872	250.0	3.000	1.872

TABLE 2.4 : *TEST 2 - MIKE 11: Summary of results*

The results in Table 2.4 are from calculations that have been made setting the node compatibility option (menu G.5.5) to water levels. However it should be noted that MIKE 11 lists the energy compatibility as an alternative compatibility option, although MIKE 11 would not allow this to be selected.

HYDRO-1D

The test was simple to set-up and run with HYDRO-1D. The method adopted to represent split flows in HYDRO-1D allowed the looped system defined in the data set to be accurately represented. The results from HYDRO-1D for this test are given in Table 2.5. It should be noted that the results give the water elevation and not the energy elevation at nodal points. The results given in Table 2.5 are from a steady state run, though it should be noted that several iterations were required (automatically) in order to obtain a converged solution. By undertaking an unsteady run with constant boundary the same solution as that from the steady run calculation was obtained.

Cross Section	Q (m^3/s)	Water Elevation (m)	Velocity (m/s)
1.01	250	3.070	1.969
1.05	250	3.057	1.938
2.01	75.5	3.057	0.932
2.05	75.5	3.015	0.739
3.01	174.5	3.057	3.066
3.05	174.5	3.015	2.404
4.01	250	3.015	1.895
4.05	250	3.000	1.871

TABLE 2.5 : *TEST 2 – HYDRO-1D: Summary of results*

FLUCOMP

The manual for FLUCOMP clearly stipulates in Section 1.3 - Limitations of the package, that: 'the assumptions principally restrict the model to flows in a single channel river'. Since a split flow calculation was required it has meant that Test 2 has not been possible with FLUCOMP and has thus not been undertaken.

ISIS

Although the data file supplied for the test was already in ISIS format the data was manually input by hand using the GUI and the 'Forms Editor'. The setting-up of Test 2 with ISIS was quick and simple. The test itself was performed without any problems and as expected utilised all of the data as provided by the benchmarking data, set to the required number of decimal places.

In ISIS the split flow at junctions is calculated by equating water levels at the nodes of the junction and also conserving mass by applying Kirchhoff's Law to the flows(11). For flows where the velocity is significant then it is suggested that total heads should be investigated. Although the total head loss at junctions is not included explicitly within ISIS the Bernoulli loss unit can be used.

	Steady Run			Unsteady Run		
Cross Section	**Q (m^3/s)**	**Water Elevation (m)**	**Velocity (m/s)**	**Q (m^3/s)**	**Water Elevation (m)**	**Velocity (m/s)**
1.01	250.0	3.064	1.974	250.0	3.065	1.973
1.05	250.0	3.052	1.940	250.0	3.052	1.940
2.01	74.241	3.052	0.919	74.761	3.052	0.925
2.05	74.241	**3.011**	0.727	74.761	**3.011**	0.732
3.01	175.759	3.052	3.095	175.239	3.052	3.084
3.05	175.759	**3.011**	2.424	175.239	**3.011**	2.417
4.01	250.0	**3.011**	1.899	250.0	**3.011**	1.899
4.05	250.0	3.000	1.871	250.0	3.000	1.871

TABLE 2.6 : *TEST 2 – ISIS: Summary of results.*

Although the test is stipulated in the stage one report as a steady state test both steady and unsteady simulations were undertaken with a constant water elevation and discharge at the appropriate boundaries. The results from the two runs were similar, with the only difference being the small change in the velocities, which can be seen in Table 2.6. It should be noted that the unsteady run results in Table 2.6 were for a 10 second time step; however, results from a smaller time step of 1s produced unreliable results, showing an ingress of flow into the system.

CHANNEL

CHANNEL is not suited to this test, as it cannot cope with more than one channel. Thus Test 2 has not been undertaken using CHANNEL.

LD01

The manual provided with LD01 makes no mention of a river system which requires a split flow or has two channels; it only describes the backwater calculation along a single reach of a river system, hence Test 2 has not been undertaken using LD01.

GLC-BACKWATER

The documentation provided with GLC-BACKWATER clearly stipulates that 'The program can work out water profiles for channels in series and cannot be used directly for channels in parallel'. Hence Test 2 has not been possible with GLC-BACKWATER since the test requires two channels in parallel for the spilt flow section of the test.

BAKWATER

It is not possible to undertake Test 2 with BAKWATER, as diverging and converging flows as required by the data set can not be defined by the package. Hence, Test 2 was not undertaken using BAKWATER.

2.2.4 Summary

1. Only FLOODTIDE, HEC-RAS, MIKE 11, HYDRO-1D and ISIS were able to undertake this test directly. The other software packages were unable to undertake calculations for split or looped channels, unless separate calculations were undertaken for each channel. This latter approach does not comply with the original test requirements.

2. The test method adopted for each of the software packages in order to calculate the split flow at the diverging and converging junctions falls into two categories. They are based upon either a water elevation balance, as used by FLOODTIDE, MIKE 11, HYDRO-1D and ISIS, or an energy balance approach as used by HEC-RAS. The energy balance approach was considered by the authors to be the more appropriate criteria for split flow conditions since the velocity head is included.

3. The results fall into three distinct categories: those that calculate a split flow similar to the referenced solution (e.g. ISIS and HYDRO-1D), those that split the flow almost equally between the two channels (e.g. FLOODTIDE and HEC-RAS) and those that fall somewhere in between the previous two solutions (e.g. MIKE 11).

4. The referenced solution is based on a numerical model simulation and is therefore of limited value for model validation.

5. FLOODTIDE produced the same water elevations at the upstream and downstream junctions respectively, irrespective of the flows specified *in the* split channels for a steady run. By specifying steady state boundary conditions for unsteady simulations of FLOODTIDE, the model failed to produce convergent results.

6. MIKE 11 has the option to undertake the computation using the energy balance approach, however, this option failed when applied to this test.

7. Energy losses at the junctions were not taken into account for this test.

8. The modellers attention is drawn to the fact that there is a significant difference in flow between the model solutions even though water levels are very similar and that care must be exercised when designing works based upon model results.

2.3 TEST 3 - Subcritical Flow in a Triangular Channel

2.3.1 Aim of Test

This test is designed to assess the ability of each of the software packages to calculate a normal subcritical flow depth in a triangular channel with a side slope of 1:2. The results are to be checked with the analytical solution.

2.3.2 Introduction

Test 3 is a simple triangular channel with eleven cross-sections placed 300m apart each with a side slope of 1:2 and a constant Manning's *n* value of 0.035. There is a constant bed slope of 0.001 and a uniform flow of 20m^3/s. The test is designed to produce a constant water depth of 3m along the length of the channel with subcritical flow under steady flow conditions. To check the results from each of the software packages the Manning's equation for a triangular channel provides the following formula for the normal depth:

$$h = 5^{\frac{1}{8}}\left(\frac{Q.n}{2\sqrt{S}}\right)^{\frac{3}{8}}$$

where Q = flow (m^3/s)
n = Manning's n
S = bed slope

From the above equation, the normal depth of flow may be obtained to be 3.012m. The critical depth of flow in the channel may be calculated to be 2.412m.

2.3.3 Results

As a whole each package was prepared easily and executed the test satisfactorily with the exception of HYDRO-1D. Technical assistance was sought from Mott MacDonald in order to complete the test.

To assess the accuracy of the results from each of the software packages the root mean square (RMS) error in water depth was obtained for the complete length of the channel has been calculated. In addition the maximum percentage error in depth (either positive or negative) along the length of the channel has also been calculated. The RMS error is defined as follows:

$$RMS = \frac{1}{N}\sqrt{\sum_{j=1}^{N}\left(S_j - S_j^*\right)^2}$$

where:
N = number of results
S_j = calculated water depth
S_j^* = analytical water depth

The results for all of the software packages are presented in Graphs 1 to 11 of Appendix B and are summarised in Table 3.1 with the statistical comparisons. The overall results were very close to the expected values, with the majority producing a 0.001 RMS error. However, it can be clearly seen from the result in Table 3.1 and Graph 4 of Appendix B that when the resistance radius method for calculating conveyance is adopted with MIKE 11 as opposed to the hydraulic radius method then the water level profile calculated is considerably lower than the analytical solution. It can also be seen from Table 3.1 that GLC-BACKWATER calculates a result that is quite different to the other software packages.

	ERROR	
SOFTWARE	Max. (±%)	RMS (m)
FLOODTIDE	-0.162	0.005
HEC-RAS	-0.015	0.001
HEC-2	-0.015	0.001
MIKE 11 – Resistance Radius	-1.769	0.068
MIKE 11 – Hydraulic Radius	-0.058	0.001
HYDRO 1D	-0.053	0.001
FLUCOMP	-0.053	0.001
ISIS	-0.053	0.001
CHANNEL	-0.038	0.000
LD01	-0.038	0.000
GLC-BACKWATER	-0.015	0.008
BAKWATER	-0.015	0.001

NOTE: Both errors were with regard to the analytical water depths in the channel

TABLE 3.1 : *TEST 3 - Statistical comparisons*

It should be noted that FLOODTIDE required the number of layers for the conveyance tables to be set at 50 in the parameters file in order to obtain the above result. Using the default value of 15 layers produced a water level much lower than the analytical solution. It should also be noted that the manual does not mention the possibility of increasing the number of layers used in a calculation, this was only found out through technical support. In addition with FLOODTIDE the three types of conveyance calculation were tested all of which produced the same result (see comments for Test 7).

1.3.3 Summary

1 This test was considered to be fairly basic and routine by the authors.

2 All of the software packages with the exception of GLC-BACKWATER were capable of accurately calculating a normal subcritical backwater profile for the triangular channel, with RMS errors being within 0.001m for the majority of packages.

3 The errors calculated by GLC-BACKWATER are likely to be due to the number of layers used internally by the model to calculate the conveyance parameters at each cross-section.

4 The resistance radius method for computing conveyance in MIKE 11 produced a result that was significantly lower than the analytical solution whereas the hydraulic radius method produced a result that was very close to this solution.

5 The number of layers used to calculate the conveyance tables in FLOODTIDE was found to be crucial for producing results which compared accurately with the analytical solution. The default value of 15 layers was not sufficient for this test and a value of 50 was found to be more appropriate.

2.4 TEST 4 - Supercritical Flow in a Triangular Channel

2.4.1 Aim of Test

This test is designed to assess the ability of each of the software packages to calculate a normal supercritical flow depth in a triangular channel with a side slope of 1:2. The results are to be checked with the analytical solution.

2.4.2 Introduction

Test 4 is a simple triangular channel with eleven cross-sections placed 15m apart each with a side slope of 1:2 and a constant Manning's *n* value of 0.035. There is a constant bed slope of 0.02 and a uniform flow of $20\text{m}^3/\text{s}$. The test is designed to produce a constant water depth of 1.7m along the length of the channel with supercritical flow under steady flow conditions. To check the results from each of the software packages the Manning's equation for a triangular channel provides the following formula for the normal depth:

$$h = 5^{\frac{1}{8}}\left(\frac{Q.n}{2\sqrt{S}}\right)^{\frac{3}{8}}$$

where Q = flow (m^3/s)
n = Manning's n
S = bed slope

From the above equation the normal depth of flow may be obtained to be 1.718 and the critical depth of flow in the channel may be calculated to be 1.828m (from theory).

2.4.3 Results

No problems were encountered in setting up and executing Test 4, each package gave a solution for the test. To assess the accuracy of the results from each of the software packages the root mean square (RMS) error in water depth was obtained for the complete length of the channel. In addition the maximum error in water depth (either positive or negative) along the length of the channel has been calculated. The RMS error is defined as follows:

TEST 4 – Supercritical Flow in a Triangular Channel

$$RMS = \frac{1}{N}\sqrt{\sum_{j=1}^{N}\left(S_j - S_j^*\right)^2}$$

where: N = number of results
S_j = calculated water depth
S_j^* = analytical water depth

The results for all of the software packages are presented in Graph 1 to 11 of Appendix C and Table 4.1 summarises the statistical comparisons. The packages that are able to calculate supercritical flow produced very accurate results with the exception of MIKE 11 when using the resistance radius method.

HEC-RAS, HEC-2, MIKE 11, HYDRO-1D and ISIS are able to calculate the supercritical levels accurately. The other software packages, as can clearly be seen in Table 4.1 and Graphs 1 to 11 of Appendix C, could not accurately calculate the water elevation, however, they were able to undertake the calculation. It can be seen from the result in Table 4.1 and Graph 4 of Appendix C that the water level profile calculated is considerably lower than the analytical solution for MIKE 11 when the resistance radius method for calculating conveyance is adopted as opposed to the hydraulic radius method.

The results from both HEC-RAS and HEC-2 required the upstream boundary to be defined with both the inflow boundary and water level boundary. The water level was set at a value that resulted in the same water level at the downstream boundary as specified in the data set.

During the course of this study the vendors of HYDRO-1D made it clear that HYDRO-1D can undertake calculations with supercritical flows. The vendors also suggested that the following calculation settings be used when undertaking calculations with supercritical flows so that water surface profile results were as smooth as possible:

		Suggested	Default
Minimum iteration	=	1	1
Maximum iteration	=	200	20
Weighting factor	=	0.55	0.55
Maximum number of iterations before applying optimal head correction	=	201	15
Maximum allowable head correction	=	0.001	0.1
Maximum allowable flow correction	=	0.10	1.0

TEST 4 – Supercritical Flow in a Triangular Channel

When using the default calculation settings with HYDRO-1D the resultant water surface profile varied above and below the analytical solution and along the length of the channel.

It should be noted that the results obtained from HYDRO-1D for Test 4 used the above calculation settings for HYDRO-1D, as suggested by the vendors, and that all the other software packages under test used the default calculation settings.

From Graphs 1 through to 11 of Appendix C it can be clearly seen that the results fall into two distinct categories, namely:

(i) those that give an accurate representation of the supercritical water surface profile: HEC-RAS, HEC-2, MIKE 11, HYDRO-1D, ISIS and CHANNEL;

(ii) those that return a constant water slope above the true water profile (usually at critical depth): FLOODTIDE, FLUCOMP, LD01, GLC and BAKWATER;

	ERROR	
SOFTWARE	**Max. (±%)**	**RMS (m)**
FLOODTIDE	1.456	0.038
HEC-RAS	0.315	0.003
HEC-2	0.923	0.010
MIKE 11 – Resistance Radius	-1.161	0.039
MIKE 11 – Hydraulic Radius	0.069	0.004
HYDRO-1D	-0.153	0.005
FLUCOMP	2.087	0.054
ISIS	-0.101	0.008
CHANNEL	0.174	0.000
LD01	2.097	0.061
GLC-BACKWATER	1.145	0.035
BAKWATER	1.300	0.034

NOTE: Both errors were with regard to the analytical water depths in the channel

TABLE 4.1 : *TEST 4 - Statistical comparisons*

2.4.4 Summary

1 HEC-RAS, HEC-2, MIKE 11, HYDRO-1D, ISIS and CHANNEL were capable of accurately calculating the supercritical depth along the triangular channel.

2 The result obtained with HYDRO-1D required the user to specify the calculation settings that were suggested by the vendor, without which the test would produce a water surface profile that varied both above and below the analytical solution and along the length of the channel.

3 The software packages that were not capable of calculating a supercritical depth were only able to compute the water level to the critical level, before continuing to produce critical (and hence incorrect) water levels. Hence, these models should not be used for supercritical flow calculations.

2.5 TEST 5 - Steady State Analytical Solutions

2.5.1 Aim of Test

The test is designed to assess the ability of each of the software packages to model subcritical, supercritical and transitional flows under steady flow conditions. The results are to be compared with the proposed analytical solution by MacDonald(12).

2.5.2 Introduction

Test 5 has five separate parts designed to test each software package under subcritical, supercritical and transitional flows. An analytical solution proposed by MacDonald(12) is used to assess the accuracy of each software package. Each part of Test 5 uses a single channel with 101 rectangular cross-sections placed 1m apart each with a constant Manning's *n* value of 0.035. The bed level along the channel and the downstream water elevation boundary are both varied for each part of the test to produce the required flow conditions. However, a constant inflow of 20 cumecs was specified for each part of the test at the upstream boundary.

2.5.3 Results

A large number of cross sections were required for this test (5555 in total) together with the associated network files. Packages like HYDRO-1D, CHANNEL and MIKE 11 proved to be the most tedious due to the required data structure. Table 5.1 summarises the method used to input the cross section data for each software package.

To assess the accuracy of the results from each of the software packages the root mean square (RMS) error of the results obtained for the complete length of the channel has been calculated. The RMS is defined as follows:

$$RMS = \frac{1}{N}\sqrt{\sum_{j=1}^{N}\left(S_j - S_j^*\right)^2}$$

where: N = number of results
S_j = calculated water depth
S_j^* = analytical water depth

SOFTWARE	Method of input for cross-sectional data.
FLOODTIDE	A computer program was written so as to read the bed level from the data file supplied by the NRA and output the cross sections in the required format.
HEC-RAS*	Data could be imported from HEC-2 data files.
HEC-2*	A computer program was written so as to read the bed level from the data file supplied by the NRA and output the cross sections in the required format.
MIKE 11*	A computer program was written so as to read the bed level from the data file supplied by the NRA and output the cross sections in the required format so as to be imported into MIKE 11.
HYDRO 1D	A computer program was written so as to read the bed level from the data file supplied by the NRA and output the cross sections in separate files in the required format.
FLUCOMP	A computer program was written so as to read the bed level from the data file supplied by the NRA and output the cross sections in the required format.
ISIS*	Individual cross section would have had to be typed in if we had not been provided with the Xchanger program.
CHANNEL	Individual cross-sections were typed. A computer program could not be written since i) CHANNEL calculates the results during the input of data and then stores the results in the data file and ii) the file data structure was not know or reported upon in the manual.
LD01	A computer program was written so as to read the bed level from the data set and to output the cross-sections in the required format.
GLC-BACKWATER	A computer program was written so as to read the bed level from the data set and to output the cross-sections in the required format.
BAKWATER	A computer program was written so as to read the bed level from the data set and to output the cross-sections in the required format.

* Denotes package able to calculate supercritical conditions

TABLE 5.1 : *TEST 5 -Input method for cross sections*

In order to make the statistical analysis a large amount of data need to be transferred to Microsoft Excel™. Not all of the packages output the results in a format that can be easily edited by either a standard text editor or within Microsoft Excel™. This meant significant time had to be spent in typing in the results for the calculated water level at each cross section. Table 5.2 summarises the actions required to transfer the results to Microsoft Excel™.

RESULTS TYPED IN BY HAND	RESULTS IMPORTED AS TEXT FILE
MIKE 11*	FLOODTIDE
CHANNEL	HEC-RAS
LD01	HEC-2
GLC-BACKWATER	HYDRO 1D
	FLUCOMP
	ISIS
	BAKWATER

* Possible to export data in convenient format at a specified time

TABLE 5.2 : *TEST 5 - Most common method of input of results to MS Excel*

The RMS error results for each Part of Test 5 are presented in Table 5.3 and the water surface profile results are illustrated in Appendix D, Graphs 1 to 55. The graphs also illustrate the percentage error in the calculated water depth when compared to the analytical solution. A brief description of the undertaking of each part of the test and the results is given next.

		Root Mean Square ERROR (m)				
	SOFTWARE	Part A	Part B	Part C	Part D	Part E
Models Capable of Supercritical Flow	HEC-RAS	0.001	0.002	0.004	0.003	0.004
	HEC-2	0.004	0.010	0.014	0.063	0.089
	MIKE 11 - Resistance Radius	0.062	0.039	0.087	0.160	0.100
	MIKE 11 - Hydraulic Radius	0.029	0.034	0.054	0.149	0.095
	HYDRO 1D	0.002	0.095	0.070	0.085	0.185
	ISIS	0.006	0.038	0.060	0.104	0.099
Models Capable of Subcritical Flow Only	FLOODTIDE	0.098	0.142	0.092	0.197	0.118
	FLUCOMP	0.001	0.197	0.092	0.141	0.164
	CHANNEL	0.015	N/A	N/A	N/A	N/A
	LD01	0.026	0.203	0.097	0.141	0.163
	GLC-BACKWATER	0.004	0.143	0.060	0.139	0.130
	BAKWATER	0.003	0.144	0.057	0.080	0.128

TABLE 5.3 : *TEST 5 - Statistical comparisons*

During the course of this study the vendors of HYDRO-1D made it clear that HYDRO-1D can undertake calculations with supercritical flows. The vendors did suggested that the following calculation settings be used when undertaking calculations with supercritical flows so that water surface profile results were as smooth as possible.

		Suggested	Default
Minimum iteration	=	1	1
Maximum iteration	=	200	20
Weighting factor	=	0.55	0.55
Maximum number of iterations before applying optimal head correction	=	201	15
Maximum allowable head correction	=	0.001	0.1
Maximum allowable flow correction	=	0.10	1.0

It should be noted that for Test 5 the vendor suggested calculation settings for HYDRO-1D (as above) were used and that all the other software packages under test used the default calculation settings.

TEST 5: Part A - Subcritical Flow

Test 5 Part A is designed to produce subcritical flow. Graphs 1 through to 11 of Appendix D, illustrate the results for the water surface profile and percentage error along the length of the channel for the individual software packages. Table 5.3 summarises the statistical values calculated when comparing the calculated solution to the analytical value.

With the exception of CHANNEL(0.015m), LD01(0.026m), MIKE 11(0.029m) and FLOODTIDE(0.098m) each package calculated the water surface profile to within 0.01m RMS for the complete length of the channel. A proportion of the error for LD01 can be accounted by the alternate cross sections that were used for the test as LD01 only allows a maximum of 50 cross sections. Similarly it should also be noted that BAKWATER is limited to 80 cross-sections hence the use of alternate cross sections as suggested by the software vendors. However, for MIKE 11 it should be noted that the hydraulic radius method for calculating the conveyance produced results which were much closer to the analytical solution than those using the resistance radius method. The hydraulic radius method for calculating the conveyance produced results with a RMS error of 0.029m compared to a RMS error of 0.062m for the resistance radius method.

TEST 5: Part B - Supercritical Flow

Test 5 Part B is designed to produce supercritical flow. Graphs 12 through to 22 of Appendix D, illustrate the results for the water surface profile and percentage error along the length of the channel for the individual software packages. Table 5.3 summarises the statistical values calculated when comparing the calculated solution to the analytical value.

Each of the packages tested was able to return a value for the water surface profile along the whole length of the channel. Only the HEC-RAS, HEC-2, MIKE 11 and ISIS were able to accurately calculate the water surface profile. With the exception of CHANNEL all the other packages calculated a water level above the analytical solution. CHANNEL calculated a water surface profile that was completely inaccurate along the length of the channel. The software vendors were contacted about the results obtained with CHANNEL and indicated that a bug had been found in the program. The problem was reported to be due to 'the program calling the wrong algorithm (subcritical instead of supercritical) and as a result it could not find a solution'.

Graph 13 clearly illustrates how well HEC-RAS performed in this test. The error line is almost flat along its whole length and has the smallest RMS error of 0.002m. The trend of the error line for MIKE 11 (Graph 15) and ISIS (Graph18) are very similar and illustrate the closeness of their performance. For MIKE 11 it should be noted that the hydraulic radius method for calculating the conveyance produced results with a RMS error of 0.034m compared to a RMS error of 0.039m for the resistance radius method.

TEST 5: Part C - Supercritical to Subcritical Flow

Test 5 Part C is designed to produce transcritical flow (subcritical to supercritical). Graphs 23 through to 33 of Appendix D, illustrate the results for the water surface profile and percentage error along the length of the channel for the individual software packages. Table 5.3 summarises the statistical values calculated when comparing the calculated solution to the analytical value.

FLUCOMP, GLC-BACKWATER, BAKWATER and LD01 all performed much the same with trend for high inaccuracy at the downstream end of the channel, the region of supercritical flow and then a gradual recovery at the upstream end where flow is subcritical. Both FLOODTIDE and HYDRO-1D produced similar trends by calculating a water level higher than the analytical solution at the downstream end of the channel but then a lower water level at the upstream end of the channel. As with Part B, CHANNEL calculated a water surface profile that was completely inaccurate along the length of the channel and can be explained in a similar manner as for Part B of the test.

From the packages that are capable of calculating supercritical flow HEC-RAS compared the best with the analytical solution by producing a RMS error of 0.004m. From the results of HEC-2 it was not possible to accurately assess the water elevation in the region of transitional flow. Two runs were required, one for the subcritical part and the other for the supercritical, the results were combined and produce an overlapping region of ambiguity. Graph 25, which illustrates the percentage error and water profile along the length of the channel, omits the water elevation in the transitional region. Once again MIKE 11 has not performed as might be expected, the results obtained are on average 5% lower than the expected analytical values. However, it should be noted that the hydraulic radius method for calculating the conveyance produced results with a RMS value of 0.054m compared to a RMS value of 0.087m for the resistance radius method.

TEST 5: Part D - Subcritical to Supercritical to Subcritical Flow

Test 5 Part D is designed to produce subcritical to supercritical to subcritical flow. Graphs 34 through to 44 of Appendix D, illustrate the results for the water surface profile and percentage error along the length of the channel for the individual software packages. Table 5.3 summarises the statistical values calculated when comparing the calculated solution to the analytical value.

With the exception of CHANNEL each of the packages tested was able to resolve a water surface profile for the complete length of the channel satisfactorily. Only HEC-RAS was able to accurately calculate the water profile across the hydraulic jump with an overall RMS error of 0.004m. HEC-2 produced a reasonable representation but as with Test 5 Part C resolved a region of ambiguity due to the overlapping of results.

With the exception of MIKE 11, ISIS and CHANNEL each package accurately calculated the water elevation both upstream and downstream of the hydraulic jump. Although ISIS locates the position of the hydraulic jump it produces a RMS error of 0.104m and the results upstream of the jump are significantly lower than the analytical solution. For MIKE 11 the hydraulic radius method for calculating the conveyance produces results with a RMS error of 0.149m compared to a RMS error of 0.160m for the resistance radius method.

CHANNEL accurately calculated the water surface profile in the downstream section of the channel, the region of subcritical flow, however, once the flow became supercritical the results became completely inaccurate which can again be explained by the reasons as discussed for Part B of the test. However, in the supercritical region FLOODTIDE, HYDRO-1D, FLUCOMP, LD01, GLC-BACKWATER and BAKWATER were all able to continue with the calculation, although inaccurately, and were able to successfully recover the subcritical water level at the upstream end of the channel.

TEST 5: Part E - Supercritical to Subcritical to Supercritical Flow

Test 5 Part E is designed to produce supercritical to subcritical to supercritical flow. Graphs 45 through to 55 of Appendix D, illustrate the results for the water surface profile and percentage error in water depth along the length of the channel for the individual software packages. Table 5.3 summarises the statistical values calculated when comparing the calculated solution to the analytical value.

With the exception of HEC-RAS and HEC-2 none of the packages accurately calculated or represented the water surface profile. HEC-RAS was extremely accurate and produced a RMS error of 0.004m, while HEC-2's inaccuracies are mainly due to incorrectly calculating the position of the hydraulic jump. MIKE 11 calculated a water elevation continuously below the analytical solution except for the region of the hydraulic jump. The hydraulic radius method for calculating the conveyance in MIKE 11 produced results with a RMS error of 0.095m compared to a RMS error of 0.100m for the resistance radius method.

2.5.4 Summary

1. This test assesses the ability of the models to undertake subcritical, supercritical and transitional flows and compares the results to the analytical solutions proposed by MacDonald et al (1995). In the opinion of the authors this is a good test of the scope of the models.

2. All of the software packages were able to model subcritical flows, however, only HEC-RAS, HEC-2, MIKE 11 and ISIS were capable of accurately modelling supercritical flows.

3. HEC-RAS gave the closest agreement with the analytical solution for all parts of the test.

4. For Part A (subcritical) of the test all of the software packages compared well with the analytical solution along the complete length of the channel, with the exceptions of FLOODTIDE, MIKE 11 and LD01.

5. For each part of the test that involved supercritical flows, the RMS error for MIKE 11 and ISIS was similar and these models were found to be significantly superior to the others – except for HEC-RAS.

6. In order to undertake combined flow calculations (i.e. subcritical and supercritical) with HEC-2 and HEC-RAS both upstream and downstream water elevation boundaries needs to be defined.

7. Results from MIKE 11 using the hydraulic radius method for calculating the conveyance showed closer agreement with the analytical solution when compared with the results obtained using the resistance radius method.

8. With the exception of CHANNEL the software packages that were tested but unable to calculate supercritical flows, were all capable of computing each part of the test. However, each package raised the water level to the critical water level in locations where supercritical flows prevailed and then were able to recover subcritical flows upstream of a supercritical section. Hence, these models should again not be used for such supercritical flow conditions.

9. To undertake supercritical flow calculations with HYDRO-1D it is important that the user specifies the calculation settings that were suggested by the vendor. This will allow HYDRO-1D to accurately calculate subcritical flows when supercritical flows are also present within a channel.

10. For this test scenario CHANNEL produced totally inaccurate results when the flow became supercritical and was unable to recover subcritical flows upstream of a supercritical section. The problem cause of the failure was due to a bug in the test version of the program that was identified by the program vendors.

2.6 TEST 6 - SERC Flood Channel Facility

2.6.1 Aim of Test

This test examines the ability of each software package to model firstly two straight channels with a flood plain (Part A) and secondly a meandering channel with a flood plain (Part B).

2.6.2 Introduction: Part A – Straight channels with flood plain

The Stage One Report describes the test as set up for the SERC Flood Channel Facility (Section 4.9). However, it does not mention whether the expected flow conditions should be either subcritical or supercritical. It should be noted for both Parts A and B that no data have been provided for the validation of results obtained from the software packages. For Part A the theoretical, critical and normal water elevations have been calculated by the authors according to Manning's using an iterative procedure, and is represented in Appendix E, Graphs 2 to 13 for Part A1 and Graphs 15 to 25 for Part A2.

Two configurations for Test 6 Part A were tested. Firstly Part A1 considered a narrow channel with wide flood plains (floodplain to channel ratio of 5.55), and secondly Part A2 considered a wide channel and narrow flood plains (floodplain to channel ratio of 2.11), both are illustrated in Figure 6.1. Figure 6.2 illustrates the plan locations of the cross-sections used for both Parts A1 and A2 and Table 6.1 presents additional information on the test set-up.

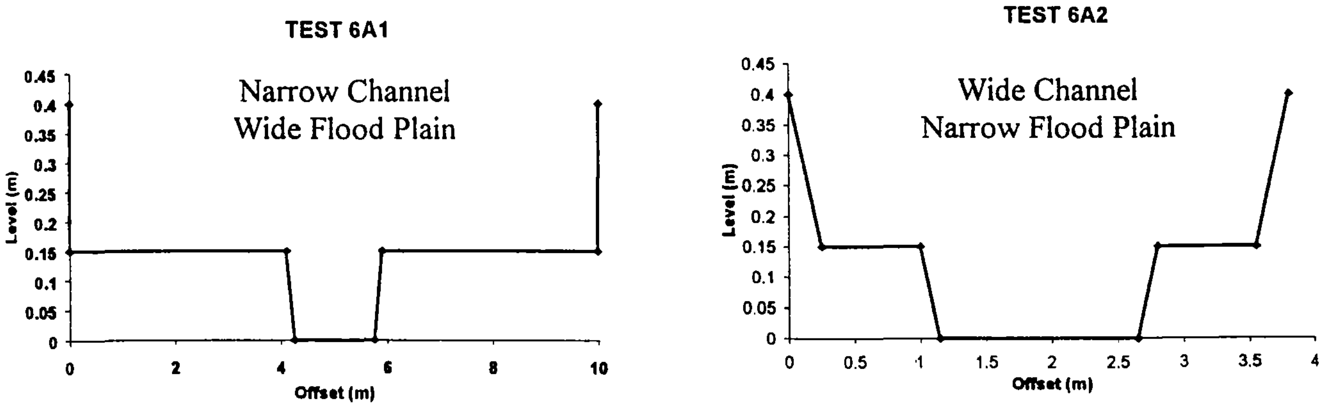


FIGURE 6.1 : *TEST 6 Part A - Cross-section profiles*

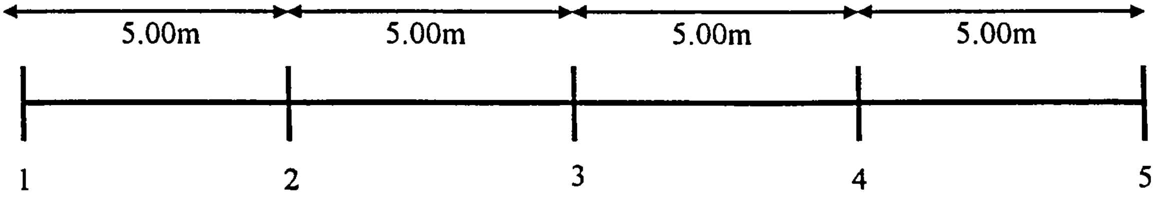


FIGURE 6.2 : *TEST 6 Part A - Plan location of cross-sections for Parts A1 and A2*

	PART A1	PART A2
UPSTREAM BOUNDARY CONDITION	0.5002 m^3/s	0.3323 m^3/s
DOWNSTREAM BOUNDARY CONDITION	0.1718 m	0.1730 m
DOWNSTREAM CROSS-SECTION FLOOD PLAIN LEVEL	0.0150 m	0.0150 m
CONSTANT BED SLOPE	0.001027	0.001027

TABLE 6.1 : *TEST 6 Part A - Additional test information for Parts A1 and A2*

For both Parts A1 and A2 the water elevation at the downstream boundary was specified in the benchmarking data set at a level lower than the hand calculated critical water elevations of 0.188m and 0.176m for Parts A1 and A2 respectively. Since the constant bed slope is mild the backwater curve is of type M3 (i.e. water depth increases in the flow direction).

2.6.3 Results: Part A – Straight channels with flood plain

The results for Parts A1 and A2 of Test 6 are presented in Table 6.2 and 6.3 respectively. These results enable a comparison to be made between all of the software packages tested under this benchmarking study. For some of the packages more than one result has been presented, to show the difference in results that can be obtained depending upon the method of calculation adopted using the software (see notes at the bottom of Table 6.2)

It should be noted that the benchmarking data set provides the cross-sectional data (at times) to six decimal places and the boundary conditions to four decimal places. However, the majority of software packages cannot represent either the cross-sections or the boundary values to such accuracy and hence round-off discrepancies may contribute to differences in the calculated results for each of the software packages.

Most of the software packages calculate a supercritical water elevation at the downstream boundary hence the results from packages that cannot calculate supercritical flow should be treated with caution. It should be noted that only HYDRO-1D, ISIS, MIKE 11, HEC-2 and HEC-RAS have the ability to calculate supercritical flow conditions. However, it can be seen from Table 6.2 and Graph 6 of Appendix E that HYDRO-1D can not cope with supercritical flow conditions properly, therefore resulting in sudden water level rises. The authors of the stage one report were queried to ascertain the validity of the supercritical downstream boundary conditions specified in the data set. It was reported that the boundary conditions had initially been intended to be subcritical and had somehow been incorrectly specified. However, the test was undertaken with the boundary conditions as specified in the data set provided for the study.

		Calculated Stage (m) at Section				
	SOFTWARE	1	2	3	4	5
	THEORETICAL	0.184	0.180	0.177	0.174	0.172
Subcritical solution	FLOODTIDE[1,2]	0.217	0.210	0.202	0.194	0.172
	FLOODTIDE[1,3]	0.214	0.209	0.204	0.198	0.172
	HEC-RAS[1]	0.226	0.221	0.216	0.211	0.202
	HEC-2[1]	0.22	0.22	0.21	0.21	0.20
	HYDRO 1D[4]	0.242	0.238	0.239	0.236	0.170
	FLUCOMP[1]	0.223	0.217	0213	0.208	0.172
	CHANNEL[1]	0.258	0.257	0.256	0.255	0.172
	LD01[1]	0.221	0.216	0.212	0.207	0.203
	GLC-BACKWATER[1]	0.23	0.23	0.22	0.20	0.19
	BAKWATER[1]	0.22	0.22	0.21	0.21	0.20
Supercritical solution	HEC-RAS	0.190	0.186	0.181	0.176	0.172
	HEC-2	0.19	0.18	0.18	0.18	0.17
	MIKE 11[5]	0.204	0.196	0.189	0.181	0.170
	MIKE 11[6]	0.215	0.206	0.191	0.174	0.170
	HYDRO 1D[4]	0.190	0.233	0.234	0.231	0.231
	ISIS	0.192	0.187	0.182	0.177	0.172

[1]water level automatically raised to critical water level as a minimum.[2]convayance type 0, [3]conveyance types 1and2, [4]typical result, [5]Resistance radius, [6]Hydraulic radius

TABLE 6.2 : *TEST 6 Part A1 - Theoretical result compared to calculated result for each software package*

		Calculated Stage (m) at Section				
	SOFTWARE	**1**	**2**	**3**	**4**	**5**
	THEORETICAL	0.174	0.171	0.168	0.167	0.173
Subcritical solution	**FLOODTIDE[1,2]**	0.205	0.197	0.189	0.181	0.173
	FLOODTIDE[1,3]	0.201	0.196	0.190	0.184	0.173
	HEC-RAS[1]	0.216	0.210	0.205	0.199	0.178
	HEC-2[1]	0.20	0.20	0.19	0.18	0.18
	MIKE 11[5]	0.199	0.191	0.185	0.177	0.170
	MIKE 11[6]	0.210	0.202	0.192	0.183	0.170
	HYDRO 1D[4]	0.289	0.288	0.285	0.175	0.170
	FLUCOMP[1]	0.208	0.203	0.199	0.193	0.173
	CHANNEL[1]	0.210	0.195	0.194	0.184	0.173
	LD01[1]	0.205	0.199	0.193	0.186	0.180
	GLC-BACKWATER[1]	0.22	0.20	0.21	0.19	0.18
	BAKWATER[1]	0.21	0.20	0.20	0.19	0.18
Super-critical solution	**HEC-RAS**	0.194	0.186	0.186	0.180	0.172
	HEC-2	0.19	0.19	0.18	0.17	0.18
	HYDRO 1D[4]	0.200	0.193	0.186	0.191	0.183
	ISIS	0.193	0.188	0.183	0.178	0.173

[1]water level automatically raised to critical water level as a minimum.[2]convayance type 0, [3]conveyance types 1and2, [4]typical result, [5]Resistance radius, [6]Hydraulic radius

TABLE 6.3 : *TEST 6 Part A2 - Theoretical result compared to calculated result for each software package*

SOFTWARE	Defines the difference between the main channel and flood plain	Can specify a varying Manning's 'n' value across a section	Can account for bends in channel by use of relative path length or equivalent
FLOODTIDE	✓	✓	✓
HEC-RAS	✓	✓	✓
HEC-2	✓	✓	✓
MIKE 11	✓[1]	✓	×
HYDRO 1D	✓	✓	×
FLUCOMP	✓	✓	✓
ISIS	✓	✓	✓
CHANNEL	✓	✓	×
LD01	✓	✓	×
GLC-BACKWATER	×	×	×
BAKWATER	✓	✓	×

[1]requires hydraulic radius method for conveyance calculation and a differing relative path length across section

TABLE 6.4 : *TEST 6 - Summary of qualities of software packages considered useful for flood plain calculations*

Table 6.4 summarises the capabilities of each of the software packages which are considered useful for flood plain calculations. The ability to set up this test on each of the packages and the results obtained for Parts A1 and A2 are discussed individually for each software package and should be read in conjunction with Tables 6.2 and 6.3

FLOODTIDE

To model a channel with a flood plain is a simple task using FLOODTIDE. The complete cross-section, including the flood plain and the main channel, is set-up with markers being used to indicate the limits of the main channel. The Manning's *n* value, although constant across the whole cross-section, can be defined at each co-ordinate of the cross-section allowing a varying Manning's roughness.

There are three 'conveyance types' which can be defined for the river channel, they are(13):

Type 0 - Einstein's Method: uses individual Manning's *n* values for each segment to calculate a weighted average Manning's *n* value for the whole cross-section.

Type 1 - The Sum of Segments Method: calculates the conveyance of the channel as the sum of the conveyances of the segments.

Type 2 - The Divided Channel Method: applies Einstein's Method to the main channel and flood plain areas separately to obtain K_{mc} and K_{fp} respectively.

Calculations using all three of the above types were undertaken and the results are presented in Tables 6.2 and 6.3 for Parts A1 and A2 and illustrated in Appendix E, Graphs 1 and 2, and Graphs 13 and 14 receptively. The results from FLOODTIDE are given to three decimal places, although the input of data was to four decimal places at the boundaries and to six places at the cross-sections.

Although not essential to these calculations, a 'FLOODTIDE' relative path length can be defined. For Test 6A this value was not defined as a straight channel, with a constant path length of 1.0 does not require a value to be defined.

It should be noted that FLOODTIDE (from the results printout) calculates a Froude number of 1.589 and 1.044 at the downstream boundary for Parts A1 and A2 respectively, thus indicating supercritical flow conditions. FLOODTIDE automatically increases the water level at the next upstream cross-section to the critical water level as a minimum in order to carry on with the calculations. Hence, the rapid increase in the water elevation at cross-section 4. The Froude number then remains around 1.0 along the complete length of the channel for both Parts A1 and A2 indicting a constant supercritical flow regime.

HEC-RAS

The GUI of HEC-RAS was used to set up this test with some ease, allowing the cross-sectional data and boundary data to be input to the required accuracy. The cross-section is split into three sections, namely the left flood plain, the main channel, and the right flood plain, with the distance to the next cross-section and the Manning's n value being defined for each section.

The subcritical option was initially undertaken for both Parts A1 and A2, the results of which are presented in Tables 6.2 and 6.3. However, the downstream water elevation was automatically increased by HEC-RAS to the critical water elevation, which was calculated at 0.20181m indicating supercritical flow conditions existed for Part A1. The supercritical option was also undertaken setting the upstream water elevation boundary at 0.19016m so as to give a resultant downstream water elevation of 0.1718m. The supercritical result is illustrated in Appendix E, Graphs 1 and 3 and also in Table 6.2.

The same problem as with PART A1 was found for Part A2 and again a supercritical calculation was undertaken. However, for Part A2 it was not possible to set the upstream boundary level so as to produce a similar downstream boundary level as that for the data set. Table 6.3 shows the result of setting the upstream boundary level at 0.1940m, giving a downstream water elevation of 0.17211m. However, it should be noted that if an upstream level of 0.1941m is used then the downstream boundary level is calculated as 0.17444m and that an upstream level of 0.1939m gives a downstream level of 0.17212m. It can be seen from Table 6.3 and Graphs 13 and 15 in Appendix E, for Part A2, that the results show a varying water surface profile and hence these results should be treated with caution.

HEC-2

The setting up of this test was relatively straightforward once the format for the input data defining the flood plains had been clarified. HEC-2 divides the cross-section into three main sections, namely: the left flood plain, main channel and right flood plain. For each sub-section of the cross-section the Manning's *n* value should be defined as well as the length to the next cross-section. Hence, if required a greater or shorter distance can be defined for the flood plains, allowing a meandering channel to be represented.

It was not possible to input the cross-sectional data to five or six decimal places as required by the benchmarking data, instead data was rounded-off to three decimal places. However, the boundary conditions were input to four decimal places as required, though it should be noted that the output of results only gave values to two decimal places.

The subcritical option was initially undertaken for both Parts A1 and A2 with the results being presented in Tables 6.2 and 6.3. The results show an increased downstream boundary water elevation of 0.20m and 0.18m, instead of 0.17m (when rounded) for Parts A1 and A2. HEC-2 has automatically increased the water level to a critical level as a minimum, hence the program considers the initial boundary value as supercritical flow. The supercritical option was then undertaken for both Parts A1 and A2 using the same upstream boundary level as used with HEC-RAS. The results are illustrated in Appendix E, Graphs 1 and 4, and Graphs 13 and 16 respectively as well as in Tables 6.2 and 6.3. The results from HEC-2's supercritical run for Part A1 were very similar to the supercritical water profile as produced by HEC-RAS, although the result for Part A2 was significantly different.

MIKE 11

The usual procedure for setting up a river system with MIKE 11 was adopted for Test 6 Part A. The only problem that MIKE 11 encountered was with the number of decimal places to which the input data could be specified. The cross-sectional data and the downstream water level boundary were rounded-off to two decimal places, whereas the upstream discharge boundary was rounded-off to three. MIKE 11 does not define the boundary between the main channel and the flood channel, although markers are used to define the riverbanks and the riverbed in the cross-sectional data menu.

With sections with significant variations in shape (such as over bank flow paths) then the resistance radius method of calculating conveyance is more suited. While the hydraulic radius is more appropriate for deep and narrow uniform sections. For hydraulic radius calculations where over bank flow paths occur then adjustment of the relative resistance should be made to invoke the parallel channel analysis facility as described in the reference manual for MIKE11(14). Although the Manning's n value specified for each cross section was constant the hydraulic radius method was adopted for continuity from other tests, however, investigations using the resistance radius were made.

The results for Parts A1 and A2 are presented in Tables 6.2 and 6.3 and illustrated in Appendix E, Graphs 1 and 5, and Graphs 13 and 17 receptively. MIKE11 is able to calculate supercritical flow conditions, although the only method of knowing this was to inspect an unsteady calculation output file which includes the Froude number. It was not possible to obtain additional output values from a steady calculation, although for Part A1 (from an unsteady run) the Froude number along the complete length of the channel was calculated at a value greater than one, indicating supercritical flow. However for Part A2 the Froude number was less than one, indicating subcritical flow.

It can be seen from Table 6.2 and 6.3 that the effect of using either the resistance radius or hydraulic radius method for the calculation of conveyance noticeably alters the result. For Part A1 the resistance radius method reduces the calculated water elevation at the upstream boundary by 0.011m a 5% reduction over the hydraulic radius method. For Part A2 a similar result occurs.

HYDRO-1D

HYDRO-1D's GUI provided a very quick and simple method for setting up the test configuration. However, problems arose when attempting to input the cross-sectional data. HYDRO-1D can only input data to two decimal places, whereas Test 6 requires five decimal places for the cross-sectional data and four decimal places for the boundary conditions. This resulted in varying bed slope along the channel length and boundary values that differ slightly from the benchmarking data.

It is possible to define what is termed as the convective region and the normal region for a cross-section. The meaning and the use of these terms are not explained in the manual, although the training for this package dealt with this, i.e. normal region defines the main channel and the convective region defines the complete cross-section, including the flood plains. Although not specifically required by Test 6, HYDRO-1D has the ability to define a Manning's value for both the left and right flood plains, the channel in and out of bank flow, and a low normal channel flow.

Although not vital for Part A of Test 6, the use of a relative path length or its equivalent is not used by HYDRO-1D to take account of the effects of bends in a channel. Hence, the relative path length of 1.0 stipulated by the data set could not be employed for Test 6 Part A.

PART A1: The subcritical option was first attempted with HYDRO-1D for which a typical result for Part A1 is given in Table 6.2 at the beginning of this section and illustrated in Appendix E, Graphs 1 and 6. However, it became apparent while undertaking the steady state test that different results could be obtained by using different calculation settings. Appendix E, Table 1 shows with comments the results that were obtained from the calculations made for the various calculation settings investigated. It can be clearly seen from these results that HYDRO-1D produces varying results for the steady state calculations.

The output of results from HYDRO-1D do not provide the values of the Froude number and hence it may not be known if the calculated water surface levels should be above or below the critical water level. Since the theoretical analysis indicated a supercritical flow and many other packages calculated supercritical flow for Test 6 Part A1, it seemed prudent to undertake a supercritical calculation. To undertake the supercritical option the upstream boundary node has to be specified as a fixed water level boundary (set at 0.19m) and the most downstream reach defined with code 21. A typical result for a steady supercritical calculation for Test 6 Part A1 is given in Table 6.2. However, it can be seen from Table 1 in Appendix E that the results from calculations with various calculation settings were varied as for the subcritical calculation. An unsteady calculation with constant boundary conditions was made for both subcritical and supercritical calculations. The subcritical calculation was unable to calculate a converged solution, although the supercritical calculation produced a converged result, which is presented in Appendix E, Table 1.

PART A2: The same problems as with Part A1 were encountered with Part A2 hence the same approach was undertaken to this test. A typical result for a steady subcritical and supercritical calculation is presented in Table 6.3 and a detailed listing is given in Appendix E, Table 1. The typical result is illustrated in Appendix E, Graphs 13 and 18.

The results obtained for Test 6A should be treated with caution and only appraised while considering the limitations of the input data to two decimal places. Results are generally poor and are non convergent, with a varying water surface profile along the length of the channel as can be seen in Appendix E, Graphs 13 and 18.

FLUCOMP

The setting up of FLUCOMP for a river with flood plains is relatively straightforward. For each cross-section the co-ordinates of the section are required to be input in the data file on separate lines for the left flood plain, main channel and right flood plain using L, C and R respectively at the beginning of each line. The Manning *n* value is defined in the roughness data file for both the flood plains and the main channel, allowing some variation in the roughness over the cross-section. The field width in the data file allows the cross-sectional data to be specified to four decimal places and to eight decimal places for the flood plain and channel lengths. The definition of the flood plain requires the user to define either the flood plain area or the flood plain length, or both if desired. For Test 6 Part A, the later option was adopted.

The appropriate method of defining the flood plain is discussed in Sections 6.6 and 6.7 of the Engineers Guide for FLUCOMP and is summarised in Table 6.4(15).

Define Flood Plain Plan Area	Define Flood Plain Length	Comments
Yes	Yes	Suitable in all cases. Plan areas used to estimate storage, flood plain lengths used to estimate surface slopes on the flood plain.
Yes	No	Suitable for all cases where flood limit is reasonably regular. Flood plain lengths estimated from the plan area and surface width of the flood plain.
No	Yes	Suitable for steady flow should only be used with caution for unsteady flow. Plan area is estimated from the flood plain length and the surface width of the flood plain

TABLE 6.4 : TEST 6 – *FLUCOMP: Use of plan areas and flood plain lengths*

The flood plain length is defined as the average length of the streamlines on the left and right flood plains, between each pair of cross-sections. Since the channel is straight for Test 6 Part A the flood plain length is always the same as the distance between cross-sections along the main channel (5m).

The results from Parts A1 and A2 are presented in Tables 6.2 and 6.3 and illustrated in Appendix E, Graphs 1 and 7, and Graphs 13 and 19 receptively. It can clearly be seen from these graphs that the water surface profiles were smooth once the water elevation had been increased to the critical water level at section four. The output file for both Parts A1 and A2 indicated that the downstream boundary water level was supercritical and hence automatically raised the water level to the critical water level as a minimum in order to continue with a backwater calculation.

ISIS

The forms editor in ISIS allowed a straight forward set-up of this test, although the cross-sectional and boundary data could only be specified to three decimal places, where six and four decimal places respectively were provided. Markers are used to define panels separating the flood plains from the main channel, these are conveniently set in the cross-sectional data input form. The relative path length although not essential for a straight channel - is set by default in ISIS to a value of 1.0 and was not changed.

ISIS is able to calculate supercritical flow conditions and for such situations returns a Froude number of 1.0 or greater in the output file. The Froude number for both Parts A1 and A2 was calculated to be greater than 1.0 along the whole length of the channel indicating supercritical flow conditions. The results for Parts A1 and A2 are presented in Tables 6.2 and 6.3 and illustrated in Appendix E, Graphs 1 and 8, and Graphs 13 and 20 receptively.

CHANNEL

The setting up of a flood plain calculation with CHANNEL is simple and straightforward. When initially setting up the river channel the user has the option to carry out a calculation for either the main channel only or the flood plain and main channel. The cross-sectional data is then input in two parts, firstly for the main channel and secondly for the flood plain, this is described clearly in the manual. The Manning's *n* value is defined for the main channel and the flood plain separately, allowing some variation over the cross-section.

The cross-sectional data can only be specified to two decimal places, whereas Test 6 requires up to six decimal places at times. The first x co-ordinate defining the flood plain cannot have a value of zero, as zero is used as an indicator for the end of input data. Hence a value of 0.01m was used for the first x co-ordinate. CHANNEL does not permit two x co-ordinates to have the same value. Hence, the second x co-ordinate for the flood plain required a value of 0.02m and the last two required values of 9.99m and 10.00m for Test 6 Part A1.

The results for Test 6 Part A1 and A2 are given in Tables 6.2 and 6.3 and illustrated in Appendix E, Graphs 1 and 9, and Graphs 13 and 21 receptively. For Part A1 it can clearly be seen from these graphs that the water surface profile is smooth once the water elevation has been increased to the critical water level at section four. The rapid increase in water level from cross-section five to four can be attributed to the supercritical water level that CHANNEL calculates at section five. In order for CHANNEL to continue with the calculation the water elevation at the next upstream cross-section is automatically increased (CHANNEL cannot undertake supercritical flow calculations). The results for Test 6 PartA2 show a backwater profile which does not have a sudden increase in the water level at the cross-section immediately upstream of the downstream water elevation boundary. It should be noted that the output file, which also provides information on the split flow between the main channel and flood plain, did not indicate supercritical levels at any of the cross-sections in the channel.

LD01

The manual for LD01 describes the different formats required by the data file for a cross-section that can either be with or without a flood plain. Standard x, y co-ordinates are used to define the cross-section, along with a Manning's *n* value defined for each pair (except the last), with markers being used to define the limits of the main channel. The format of the data are separated into fields of eight, thus allowing cross-sectional data and boundary conditions to be specified to the required number of decimal places. It is vital to have the correct positioning of data and to adhere to blank values where required (see section 3.3 of LD01 manual). Although not critical to this test, there is no means of representing the relative path length, hence for LD01 the value of 1.0 defined in the data file is a defunct parameter.

The results for Parts A1 and A2 are presented in Tables 6.2 and 6.3 and illustrated in Appendix E, Graphs 1 and 10, and Graphs 13 and 22 receptively. It should be noted that LD01 cannot calculate supercritical flow conditions and increases the water level to the critical water level as a minimum. It can be seen from the results for both Parts A1 and A2 that the calculated water profiles are very similar to those calculated using HEC-RAS and HEC-2, which automatically raise the boundary value for supercritical flow conditions.

GLC-BACKWATER

The setting up of GLC-BACKWATER for Test 6 Part A was straightforward. However, no provision is made within GLC-BACKWATER to define/indicate the difference between the main channel and the flood plain. Cross-sectional data and boundary values are input in fields of 10 digits, thus allowing the required number of decimal places for the data. The Manning's *n* value can only be defined for a complete cross-section and as required for this test remained constant for the whole length of the river system (Manning's *n* can vary along the length of a channel - see table 17 of manual).

The results for Parts A1 and A2 are presented in Tables 6.2 and 6.3 and illustrated in Appendix E, Graphs 1 and 11, and Graphs 13 and 23 receptively. It should be noted that GLC-BACKWATER cannot undertake supercritical calculations and gives a warning in the output file of locations where the water level has been raised to critical level in order to carry on with the calculation. For both Parts A1 and A2 the water level at the downstream boundary was calculated as being supercritical and was raised accordingly. When considering round off effects (results given to two decimal places) it can be seen from Table 6.2 and Graph 1 of Appendix E for Part A1, that the calculated water surface profile was similar to that of other packages which also raised the water level at the downstream boundary. However, Table 6.3 and Graphs 13 and 23 of Appendix E for Part A2 show that the water surface profile was slightly higher along the length of the channel when compared to the other packages.

BAKWATER

To set-up and run Test 6A one BAKWATER was quick and simple. The cross-sectional data was input through the DOS based input menu, with rounding off to two decimal places on the GUI and three decimal places in the input data file. The GUI allows the cross-sections to be viewed on screen and provides a means of defining the limits of the main channel and flood plains. This feature is a simple, quick and amenable tool within BAKWATER. The boundary data was stored to three decimal places in the ASCII file (when viewed outside of the GUI) and results printed and stored to two decimal places.

The results for both Parts A1 and A2 are presented in Tables 6.2 and 6.3 and illustrated in Appendix E, Graphs 1 and 12, and Graphs 13 and 24 respectively. It should, however, be noted that the result for Part A1 is for a critical water surface profile along the complete length of the channel as indicated on the results printout. It should also be noted that the varying water surface profile is a result of round off to two decimal places. BAKWATER cannot calculate supercritical flow conditions and will automatically raise the water level in order to continue with a calculation. For Part A2 BAKWATER has increased the downstream boundary level to a critical water level, indicating supercritical flow conditions were initially calculated. BAKWATER then continues to calculate a subcritical water surface profile along the remainder of the channel.

Once a backwater calculation has been undertaken with BAKWATER the results can be viewed in long section with the GUI. The long section profiles can be saved in a format that can be readily imported into a spreadsheet such as Microsoft Excel™ for processing. This feature has been illustrated in Figure 1 of Appendix E.

2.6.4 Summary: Part A – Straight channels with flood plain

1. This test assesses the ability of the software packages to predict flows throughout a narrow main channel with wide flood plains and also along a wide main channel with narrow flood plains. However, no laboratory data were made available for model comparisons and hence analytical results were used for basic comparative purposes.

2. For this test (and subsequent part B) it should be borne in mind that the flume is of limited length, i.e. 49m, and that only small variations in the predicted water elevations were to be expected. Also in the physical model it was anticipated that the flow conditions could be transitional turbulent, whereas in all of the numerical models the flow was inherently assumed to be fully developed turbulent flow. Hence, comparisons with this laboratory set-up were, in the opinion of the authors, of limited value for comparisons.

3. For both parts of the test the theoretical and calculated results from the software packages indicated that the boundary conditions were supercritical. This limited the suitability of the test to those packages that could calculate supercritical flow conditions, that is: HEC-RAS, HEC-2, MIKE 11, ISIS and HYDRO-1D.

4. The results of the test for HYDRO-1D using the supercritical flow mode produced results which were generally non-convergent, with a varying water surface profile along the length of the channel. Although the test is supercritical this model was also run for the inappropriate subcritical case, which also gave non-convergent results. HYDRO-1D did not provide information as to the value of the Froude number, making it difficult to ascertain which flow condition was appropriate for this test.

5. Results from HEC-RAS, HEC-2, MIKE 11 and ISIS were all very similar, although slightly higher than the theoretical solution.

6. Results using the resistance radius option, rather than the hydraulic radius option, in MIKE 11 produced results that were closer to the theoretical solution, indicating that the resistance radius option was more appropriate for overbank flows.

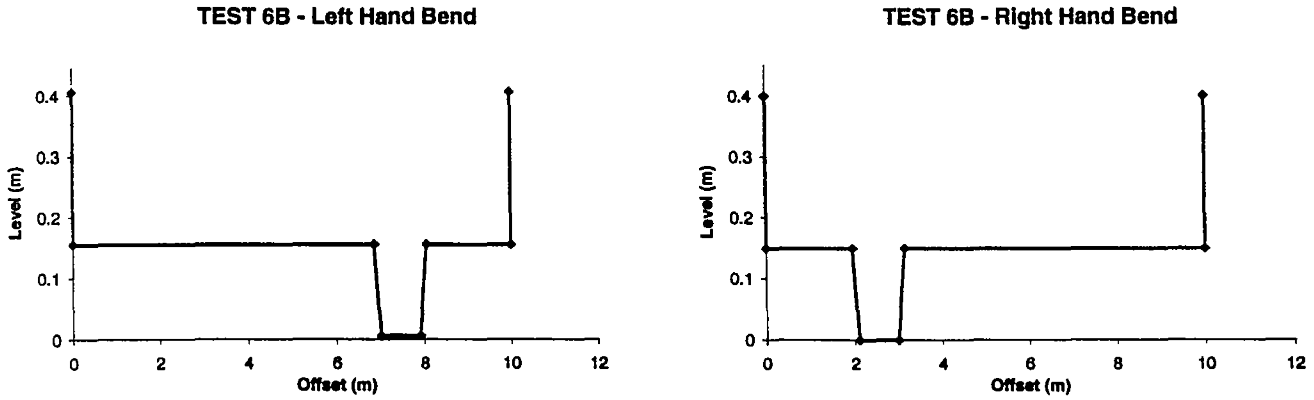
7. As before all of the packages that were unable to calculate supercritical flows led to the water level being raised automatically to the critical level in order to produce a result. The results from these packages were all very similar and incorrect.

2.6.5 Introduction: Part B – Meandering channel with flood plain

The benchmarking data set provided for this test gives nine cross-sections at the apex of the bends for a 60° meandering channel intending to represent the SERC Flood Channel Facility at H.R.Wallingford. The stage one report (Appendix E) for this study gives a detailed description of the flood channel facility at H.R.Wallingford and Figure 6.3 illustrates the cross-sections used to represent the meandering channel as provided by the benchmarking data set.

It should be noted that cross-sectional data has not been specified between the apex of the bends, which is not the same as the description in Appendix E of the Stage One Report. For some of the software packages it may be more appropriate to use additional cross-sections so as more accurately represent the meandering channel and the over bank flow paths. However, for this study the information supplied in the data set has been solely used so as to allow comparison of results.

The main channel has a bed slope of 0.0007 assuming an alternating distance of 8.245m and 9.245m between the first and last cross-sections. Figure 6.4 illustrates the meandering channel where the cross-sections are located at the apex of the bends. The boundary conditions in the data set specified a constant discharge of 0.5002m^3/s at the upstream boundary and a constant water level of 0.2030m at the downstream boundary (flood plain level of 0.15m). A constant Manning's roughness value of 0.01 was specified for both the main channel and flood plain along the complete length of the channel as specified in the data set.


TEST 6B - Left Hand Bend

TEST 6B - Right Hand Bend

FIGURE 6.3 : TEST 6 Part B - Cross-sections at apex of bends

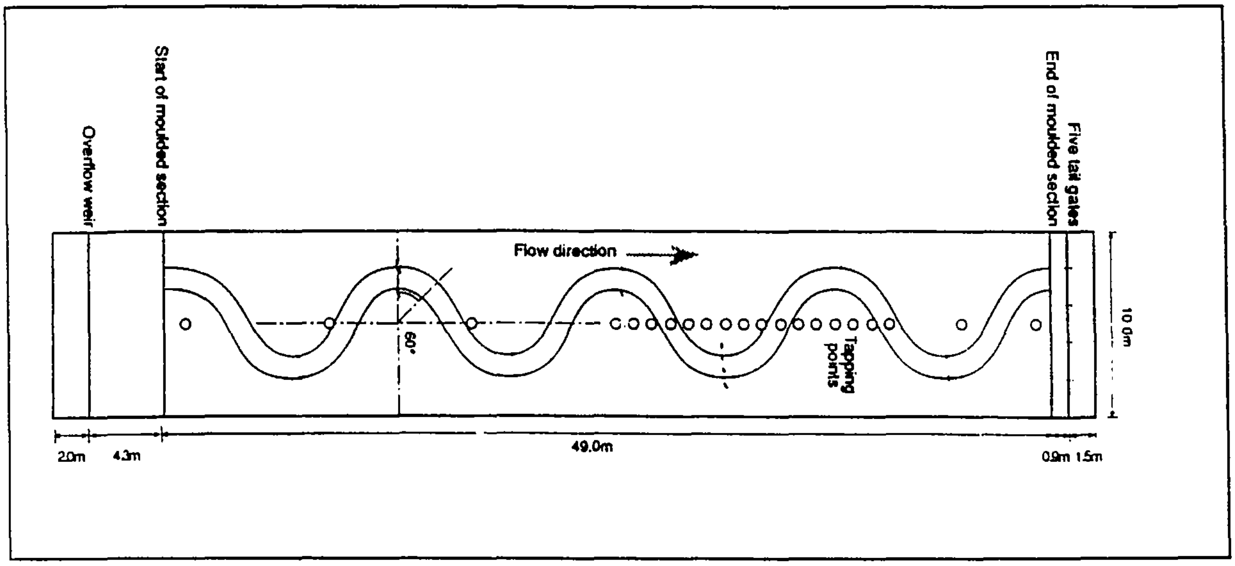


FIGURE 6.4 : *TEST 6 Part B - Meandering channel*

2.6.6 Results: Part B – 60° Meandering channel with flood plain

The results for Test 6 Part B for each of the software packages tested is discussed next and presented in Table 6.5 and illustrated in Appendix E, Graphs 25 to 36. It can clearly be seen that the results are generally similar although the results for HEC-RAS, HEC-2 and ISIS using supercritical conditions stand out as giving the three lowest water surface profiles. However, it should be noted that the Stage One Report does not provide relevant data to allow validation of the results.

	SOFTWARE	Calculated Stage (m) at Section								
	TEST 6 Part B	1	2	3	4	5	6	7	8	9
subcritical solution	FLOODTIDE[1]	0.265	0.257	0.251	0.244	0.236	0.229	0.221	0.212	0.203
	FLOODTIDE[2]	0.250	0.244	0.239	0.233	0.227	0.221	0.215	0.209	0.203
	HEC-RAS[3]	0.266	0.259	0.253	0.246	0.240	0.233	0.226	0.218	0.211
	HEC-2[3]	0.26	0.26	0.26	0.25	0.25	0.24	0.24	0.23	0.21
	MIKE 11[1]	0.273	0.268	0.260	0.252	0.245	0.237	0.228	0.216	0.200
	HYDRO 1D[1]	0.272	0.268	0.258	0.253	0.250	0.240	0.236	0.220	0.200
	FLUCOMP	0.236	0.231	0.227	0.222	0.218	0.214	0.211	0.207	0.203
	CHANNEL[1]	0.273	0.266	0.257	0.251	0.245	0.236	0.228	0.218	0.203
	GLC-BACKWATER[1]	0.28	0.27	0.26	0.26	0.25	0.24	0.24	0.21	0.20
	BAKWATER[1]	0.28	0.27	0.26	0.26	0.25	0.24	0.23	0.22	0.20
	LD01[1&3]	0.270	0.263	0.257	0.250	0.243	0.236	0.228	0220	0.211
Super-critical solution	HEC-RAS	0.218	0.217	0.215	0.213	0.212	0.213	0.208	0.206	0.203
	HEC-2	0.22	0.22	0.21	0.22	0.23	0.22	0.22	0.22	0.21
	ISIS	0.233	0.227	0.222	0.217	0.213	0.208	0.204	0.203	0.203

[1] without use of relative path length, [2]with use of relative path length, [3] water level automatically raised to critical water level as a minimum.

TABLE 6.5 : *TEST 6 Part B - Results from each software package*

FLOODTIDE

The test was simple to set up with FLOODIDE. The cross-sections were input to the three decimal places required and the boundaries to four decimal places. The boundary between the main channel and flood plain was defined with the use of markers as for Part A and the meandering channel was represented by the use of 'Relative Path Length' (RPL). The RPL is a factor used by FLOODTIDE to account for the increased flow path on the outside (reduced on inside) of bends. This is necessary to calculate the true area (and hence storage) of the flood plains based on the section width and the distance between sections measured along the centreline of the main channel(16).

$$RPL = \frac{A}{\left(L \times \left(dx_1 + dx_2\right) / 2\right)}$$

where: dx_1 = distance along the main channel from the current cross section to the next cross-section (m).
dx_2 = distance along the main channel from the current cross section to the previous cross section (m).
L = width of left/right flood plain (m).
A = area of left/right flood plain measured mid-distance between the previous and next cross-sections (m^2).

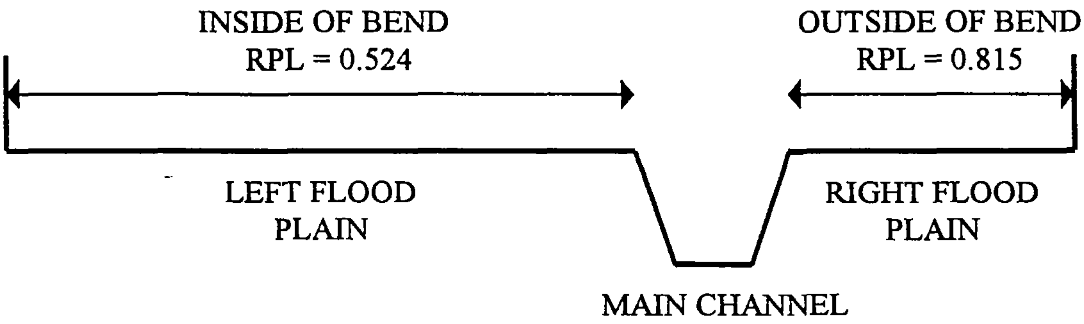


FIGURE 6.5 : *TEST 6 Part B – FLOODTIDE: Calculated Relative Path Lengths (RPL)*

This is in effect the ratio of the actual surface area to that assumed by the model that uses *dx* values along the channel centreline, and is not the same as the area defined in the benchmarking data set. For this test (Part B) the relative path length was calculated by hand using the dimensions as laid out in Figures 5 and 6 of Appendix E of the Stage One Report. The Relative Path Length values calculated are illustrated in Figure 6.5.

It should be noted that the benchmarking data set defines the relative path length from one cross-section to the next, whereas FLOODTIDE defines the relative path length from mid-distance between the previous and next cross-sections. This approach requires the relative path length to be defined differently for the flood plains on the left hand and right hand sides of the flood plains. It does not require the same relative path length for both flood plains, which alternate between 0.781 and 0.728 for cross-section in the benchmarking data set.

The undertaking of the test was straightforward and provided no problems once the data file had been set up. Initially a critical backwater calculation was undertaken to provide the initial conditions required for a backwater calculation. The backwater results for Test 6 Part B when using conveyance type one and RPL values of 0.524 and 0.815 along the length of the channel are presented in Table 6.5 and illustrated in Appendix E, Graphs 25 and 26. It can been seen from Graph 25 that the water surface profile is similar to, although higher than, that produced by ISIS. As an investigative measure a calculation was made without the use of the relative path length so as to allow comparisons with the packages which cannot represent the meandering channel, but have been tested using a straight channel. The results for this test are also presented in Table 6.5 and show a similar water surface profile as packages which have been tested only for a straight channel.

As a further investigative measure tests were undertaken with an averaged 'FLOODTIDE' relative path length of 0.695, which was used for both left and right flood plains. This resulted in a smooth backwater profile with a calculated upstream boundary level of 0.249m. Also undertaken was a calculation using an alternating relative path length of 0.781 and 0.728 for both flood plains as stipulated in the benchmarking data set. This also resulted in a smooth backwater profile with a calculated upstream boundary level of 0.253m.

It should be noted that the Froude number was calculated to be less than one for the complete length of the channel for all situations tested indicating that the results presented in Table 6.5 are for subcritical flow conditions.

HEC-RAS

The nine cross-sections representing the meandering channel were input to the four decimal places required by the data set. However, HEC-RAS does not consider relative path length and only considers reach length for the left over bank, right over bank and main channel. The HEC-RAS user manual describes the reach length as follows:

> *DOWNSTREAM REACH LENGTH:* The downstream reach length describes the distance between the current cross-section and the next cross-section downstream. Channel reach lengths are typically measured along the thalweg and overbank reach lengths along the anticipated path of the centre of mass of the overbank flow.

The values used in this test for channel length alternated between 9.245m and 8.245m as per data file and over-bank reach lengths alternated between 7.220m and 6.001m as calculated from:

Over-bank Reach Length = Relative Path Length x Main Channel Length.

Undertaking the subcritical calculation option for Part B resulted in the water level at the downstream boundary being increased automatically to the critical water level and thus indicating that supercritical flow conditions had initially been specified. As an alternative solution a supercritical calculation was undertaken setting the upstream water level at exactly 0.21847m so as to give a downstream boundary value of 0.203m as stipulated in the data set. The results for both the subcritical and supercritical calculations are presented in Table 6.5, which used expansion and contraction coefficients of zero (used to evaluate shock). The results for the supercritical calculation are illustrated in Appendix E, Graphs25 and 27.

HEC2

Test 6 Part B was set up using the same method as required for Part A and used a similar approach as for HEC-RAS Test 6 Part B with regards to reach lengths for the left overbank, right overbank and main channel. Again the downstream water level boundary was automatically increased to the critical water level when undertaking a subcritical calculation. This indicates that a supercritical water level had been initially specified. As with HEC-RAS a supercritical calculation was undertaken setting the upstream boundary level at 0.2184m, the same as for HEC-RAS. The results for the supercritical calculation are presented in Table 6.5 and illustrated in Appendix E, Graphs 25 and 28. The varying water surface profile along the length of the channel can be attributed to the results being rounded off to two decimal places in the output file.

MIKE 11

It is not possible with MIKE 11 to take into account directly the effects of a meandering channel by means of relative path length or equivalent. The only way in which MIKE 11 is able to take into account the effects of bends is to define additional storage areas(17) of the flood plain and/or with the use of additional channels to represent the flow path over the flood plains. This method although appropriate to some situations, has not been employed as it would deviate considerably from the benchmarking data set and would require considerable calibration for accurate results to be obtained. It should also be noted that additional storage would have no effects for steady flow calculations.

As a crude approximation to the test MIKE 11 was set up using the cross-sectional data as defined in the data file and thus treating the channel as straight. It is not possible to define difference between the flood plains and main channel as discussed with Test 6 Part A. The distance used between cross-sections alternated between 9.245m and 8.245m as specified in the benchmarking data set for the main channel. The comments made for Test 6 Part A with respect to setting up also apply here for Part B and should be taken into account when analysing the results. In addition it should be noted that the chainage for the cross-sections could only be input to three decimal places when converted to kilometres, i.e. the format required by MIKE 11.

The results obtained from Test 6 Part B are presented in Table 6.5 and illustrated in Appendix E, Graphs 25 and 29 and should be treated with caution. The backwater profile is similar to those obtained by other packages which cannot represent meandering channels by the use of a relative path length or similar and had assumed a straight channel. The Froude number at the cross-sections as calculated from an unsteady run (with constant boundary conditions) was less than 0.7 for all but the two cross-sections at the downstream end of the river system, with values of 0.94 and 0.82. Indicating that some of these results are very close to critical condition.

HYDRO-1D

HYDRO-1D is unable to represent meandering channels by using relative path lengths or equivalent as set out by the benchmarking data. However, a possible method of accounting for the effects of flow through a bend with HYDRO-1D may be to add additional channels so as to provide a flow path for flow over the flood plains and/or with the use of storage areas of the flood plains. This method has not been used as it would deviate considerably from the benchmarking data set and would require considerable calibration for accurate results to be obtained.

As an a approximation to the test HYDRO-1D was run using the cross-sectional information as provided in the data set, however, the channel was defined as being straight. The comments made for Test 6 Part A with respect to setting-up also apply for Test 6 Part B and should be taken into account when analysing the results. In order to obtain the results as presented in Table 6.5 (subcritical calculation) the minimum number of iterations in the calculation settings had to be set at 30 to allow convergence, values less than this produced a smooth backwater although lower water levels than these quoted. An unsteady calculation with constant boundary conditions was also undertaken as an investigative measure and produced the same results as for the steady calculations. Since the backwater profile from a subcritical calculation was smooth and the result converged, a supercritical calculation was not undertaken. The results for the subcritical calculation is illustrated in Appendix E, Graphs 25 and 30. The varying water surface profile along the length of the channel can be attributed to the results being rounded off to two decimal places in the output file.

FLUCOMP

As discussed for Test 6 Part A FLUCOMP can cope with flood plains and can also take into account the effects of meandering channels but only by means of plan areas and/or the use of the flood plain length. The relative path length as defined in the benchmarking data cannot be used directly by FLUCOMP to define the meandering of a channel.

Using the dimensions in Figures 5 and 6 of Appendix E in the Stage One Report, it was found from hand calculations that the total plan area for both the left and right flood plains between cross-sections was 45.282m^2. Since the cross-sections are defined only at the apex of the bends the flood plain areas are equal between cross-sections, hence both the left and right flood plains have a plan area of 22.641m^2. The perpendicular distance of 6m was used for the flood plain length. Record type 6 in the data file defines the distance between cross-sections and the data for the flood plain, it can also be used to define the ground level for the flood plains. The description in the manual for the ground level of the flood plain is not clear, although through technical support for FLUCOMP this was quickly clarified.

As an investigative measure two calculations were made by: (i) defining the flood plain area with the flood plain length, and (ii) defining just the flood plain length. The same result was obtained for both approaches, the result for which is presented in Table 6.5 and illustrated in Appendix E, Graphs 25 and 31. It should be noted that warning messages were given in the output file, indicating supercritical flow conditions at the downstream water elevation boundary. FLUCOMP automatically increased the water elevation at cross-section four to the critical water level in order to continue with the backwater calculation.

ISIS

The benchmarking data supplied for this study was suited for direct input to ISIS without alterations or approximations and caused no problems as was anticipated. The tutorial in the manual describes the modelling of flood plain flow(18) with reference to a simple situation. It is mentioned in the discussion for this tutorial that the cross-sections on the flood plain must be perpendicular to the flow directions, implying that the cross-sections may be curved, but one cross-section cannot intersect another. Since the cross-sections given in the data file were at the apex of the bends this point was irrelevant, but should be considered carefully if more cross-sections were to be used to represent the meandering channel.

The definition of relative path length by ISIS is as follows: it is a measure of the sinuosity associated with each of the vertical panels, relative to the sinuosity of the main channel. It is calculated from the midpoint of the distance from the previous section to the midpoint of the distance to the subsequent section.

$$RPL_{left} = \frac{dx_{left}}{dx_{centre}} \qquad RPL_{right} = \frac{dx_{right}}{dx_{centre}}$$

Since the data set provided for this test was in ISIS format no problems were encountered in setting up the test as described in the introduction to Test 6 Part B earlier. The results for Part B using an unsteady direct method calculation are presented in Table 6.5 and illustrated in Appendix E, Graphs 25 and 32. The output file produced by ISIS gives that the calculated Froude number is greater than one at all cross-sections except at the downstream boundary (value of 0.89). This indicates that a supercritical flow regime is being calculated for the majority of the channel.

As an investigative measure an unsteady calculation was made with constant boundary conditions, the results of which showed only a 2mm increase in the calculated water level at the upstream boundary.

CHANNEL

The same approach as used for Part A was adopted with Part B and produced the same problems as discussed previously. It should be noted that the manual for CHANNEL gives no reference to calculations with meandering channels or bends and hence the relative path length as defined in the data set was irrelevant. In addition, problems were encountered with the cross-sections when attempting to input the desired chainage. CHANNEL does not allow the input of chainage to any decimal places, thus as an approximation the chainage was rounded off cumulatively.

The results for the backwater calculation are presented in Table 6.5 and illustrated in Appendix E, Graphs25 and 33. No warning messages were given to indicate critical flow conditions although the results should be treated with caution considering the limitations of CHANNEL and the inaccuracies produced from the input of data.

LD01

A meandering channel cannot be accurately represented in LD01 as the effects of path lengths for flow over flood plains are not taken into account. The test was undertaken by inputting the cross-sections as given in the data file and with distances between cross-sections alternating between 8.245m and 9.245m along the length of the channel. The flood plain was represented in the same way as for Part A, the comments for which should be taken into account when considering the results for Part B which are presented in Table 6.5 and illustrated in Appendix E, Graphs 25 and 34. The downstream boundary condition set at 0.2030m was automatically raised by LD01 to a critical water level of 0.211m, thus indicating that a supercritical flow condition had initially been encountered.

GLC-BACKWATER

A meandering channel or channel with flood plains cannot be represented by GLC-BACKWATER. The same approach as used for Part A was adopted for Part B and hence the results which are presented in Table 6.5 and illustrated in Appendix E, Graphs 25 and 35 should be treated with caution. In addition the comments made for Test 6A with respect to the setting up of the test also apply for Test 6B and should be read accordingly. It should be noted that GLC-BACKWATER calculated Froude values of less than one along the complete length of the channel indicating subcritical flow conditions.

BAKWATER

A meandering channel cannot be represented by BAKWATER, although flood plains can be taken into account and hence the results are presented in Table 6.5 and illustrated in Appendix E, Graphs 25 and 36 should be treated with caution. In addition the comments made for Part A with respect to the setting up of the test also apply for Part B and should be read accordingly. It should be noted that BAKWATER calculated Froude values of less than one along the complete length of the channel indicating subcritical flow conditions.

2.6.7 Summary: Part B – 60° Meandering channel with flood plain

1. This test assessed the ability of the software packages to model a meandering channel by incorporating the use of relative path lengths to take account of the meanders and floodplains. The suitability of the data set was solely limited to ISIS, although FLOODTIDE, HEC-RAS, HEC-2 and FLUCOMP were all able to simulate this test using alternative approaches. As for Part A, this test was felt to be of limited value by the authors.

2. This test was also undertaken with HYDRO-1D, MIKE 11, CHANNEL, GLC-BACKWATER, LD01 and BAKWATER by treating the channel as straight, and then approximating the main channel length by the centreline distances between adjacent cross-sections. The results from these packages were very similar to one another, except for LD01.

3. As before relevant data were not made available to allow validation of the results and hence only a comparison between the software packages was possible.

4. Three of the packages, including HEC-RAS, HEC-2 and ISIS which take account of the meandering channel using relative path lengths or equivalent, have produced supercritical results for all, or a majority, of the channel. However, both FLOODTIDE and FLUCOMP, which also accounted for the meandering channel, have produced results that were subcritical. All of the other packages tested produced subcritical flow predictions.

5. FLOODTIDE calculated a water surface profile that was noticeably higher than that calculated by HEC-RAS, HEC-2, FLUCOMP and ISIS, with these packages producing a similar water surface profile.

6. Except for LD01, which raised the water level to the critical level, all of the other software packages that do not take direct account of the effects of the meandering have produced subcritical predictions with higher upstream water levels

7. The test data only supplied cross-sectional data at the apex of the bends, although more cross-sections would have been appropriate for some of the software packages.

2.7 TEST 7 - FCF Blackwater Model

2.7.1 Aim of Test

This test is designed to assess the ability of each of the software packages to model the FCF Blackwater model at H.R.Wallingford, UK. Results are to be compared with experimental measured stage values but are not expected to show precise agreement.

2.7.2 Introduction

The benchmarking data file provided for this test gives eight cross-sections representing the FCF Blackwater Model as described in Appendix F of the Stage One Report for this study. The model represents a proposed alleviation scheme involving the re-routing/reconstruction of an existing river to a meandering compound channel with berms, approximately 0.5m below the general level of the natural flood plain. Appendix F, Figure 1 shows a plan of the FCF Blackwater Model and location of the cross-sections and Figures 2 to 9 in Appendix F illustrate the eight cross-sections used in Test 7. Each cross-section had a constant Manning's value of 0.017 for the main channel and 0.019 for the flood plains.

In all three different boundary conditions are to be tested to show how each of the software packages cope with flows which are either in bank or out of bank. They are as follows in Table 7.1:

	Inflow at Upstream Boundary (m3/s)	Water Level at downstream boundary (m)
Part A	0.06126	0.174
Part B	0.12470	0.251
Part C	0.17327	0.279

TABLE 7.1 : *Boundary conditions for each part of Test 7*

The water level as specified by the experimental data is illustrated at each cross section in Figures 1 through to 9 of Appendix F. It can be seen from these figures that Part A tests the ability of each software packages to cope with flow which just encroaches the floodplain, whereas Parts B and C test for higher water levels.

2.7.3 Results

PART A:

The results for Part A from each of the software packages are given in Table 7.2 along with the experimental result. The results are also illustrated in Appendix F, Graphs 1 to 11 and should be read in conjunction with the comments made for each package.

	TEST 7 Part A	Calculated Stage (m) at Section							
	SOFTWARE	1	2	3	4	5	6	7	8
	EXPERIMENTAL	0.213	0.207	0.204	0.195	0.190	0.184	0.178	0.174
Use RPL* or equivalent	HEC-RAS	0.206	0.201	0.198	0.192	0.187	0.184	0.178	0.174
	HEC-2	0.21	0.20	0.20	0.19	0.19	0.18	0.18	0.17
	FLUCOMP	0.202	0.197	0.195	0.189	0.185	0.182	0.177	0.174
	ISIS	0.205	0.199	0.196	0.189	0.184	0.182	0.177	0.174
RPL* not used	FLOODTIDE[1]	0.218	0.211	0.207	0.197	0.191	0.187	0.180	0.174
	MIKE 11[2]	0.204	0.198	0.196	0.189	0.183	0.179	0.174	0.17
	HYDRO 1D	0.204	0.198	0.196	0.188	0.183	0.179	0.174	0.17
	CHANNEL	0.207	0.202	0.199	0.192	0.187	0.183	0.178	0.174
	LD01	NO RESULT							
	GLC-BACKWATER	0.21	0.21	0.20	0.20	0.19	0.18	0.18	0.17
	BAKWATER	0.21	0.21	0.20	0.20	0.19	0.19	0.18	0.17

* Relative Path Length, [1]convayance type 1, [2]Hydraulic radius, NOTE: shaded cells indicate level is below flood plain.

TABLE 7.2 : *TEST 7Part A - Results from each software package*

It can be seen from Table 7.2 and Graphs 1 to 11 in Appendix F that the results for all of the packages are very similar and close to the experimental results. When compared to the experimental results FLUCOMP produced the maximum difference in the upstream boundary water elevation about 5% lower than experimental results. The results from FLOODTIDE, HEC-RAS and HEC-2 were within 3% of the experimental results and 4% for ISIS, all of which take account for the meandering of the channel by relative path length of equivalent. However, it should be noted that MIKE 11 and HYDRO-1D which do not take into account the relative path length were also within 5% of the experimental result. In table 7.2 it can be seen that for a number of packages the calculated water elevation is below the flood plain level at the upstream.

PART B:

The results for Part B from each of the software packages are given in Table 7.3 along with the experimental result. The results are also illustrated in Appendix F, Graphs 12 to 23 and should be read in conjunction with the comments made for each package.

	TEST 7 Part B	Calculated Stage (m) at Section							
	SOFTWARE	1	2	3	4	5	6	7	8
	EXPERIMENTAL	0.292	0.285	0.283	0.274	0.269	0.265	0.257	0.251
Use RPL* or equivalent	HEC-RAS	0.273	0.269	0.267	0.262	0.257	0.257	0.251	0.251
	HEC-2	0.28	0.28	0.27	0.27	0.26	0.26	0.26	0.25
	FLUCOMP	0.270	0.267	0.265	0.261	0.256	0.256	0.251	0.251
	ISIS	0.264	0.260	0.259	0.257	0.254	0.254	0.251	0.251
RPL* not used	FLOODTIDE[1]	0.289	0.284	0.281	0.274	0.269	0.263	0.258	0.251
	MIKE 11[2]	0.280	0.274	0.272	0.266	0.260	0.257	0.251	0.25
	HYDRO 1D	0.270	0.267	0.265	0.260	0.256	0.256	0.251	0.25
	CHANNEL	0.275	0.271	0.269	0.264	0.260	0.258	0.254	0.251
	LD01	0.274	0.270	0.268	0.263	0.259	0.257	0.253	0.251
	GLC-BACKWATER	0.28	0.28	0.28	0.27	0.26	0.26	0.25	0.25
	BAKWATER	0.28	0.28	0.28	0.27	0.26	0.26	0.25	0.25

*Relative Path Length, [1]convayance type 1, [2]Hydraulic radius

TABLE 7.3 : *TEST 7Part B - Results from each software package*

It can be seen from Table 7.3 and Graphs 12 to 23 in Appendix F that the results for all of the software packages are lower than the experimental results. ISIS which accounts for the meandering channel by use of relative path lengths calculated the lowest water elevation at the upstream boundary by almost 10%. Generally the software packages which do not take into account of the meanders by use of relative path length or equivalent show a better correlation to the experimental results. This questions the relevance of relative path length to this test situation and the use of 1-D models to predict 3-D flow regimes that this test case exhibits.

PART C:

The results for Part C from each of the software packages are given in Table 7.4 along with the experimental result. The results are also illustrated in Appendix F, Graphs 24 to 35 and should be read in conjunction with the comments made for each package.

	TEST 7 Part C	Calculated Stage (m) at Section							
	SOFTWARE	1	2	3	4	5	6	7	8
	EXPERIMENTAL	0.322	0.315	0.313	0.305	0.302	0.295	0.288	0.279
Use RPL* or equivalent	HEC-RAS	0.302	0.296	0.294	0.290	0.284	0.285	0.278	0.279
	HEC-2	0.31	0.29	0.30	0.29	0.29	0.28	0.28	0.28
	FLUCOMP	0.299	0.294	0.293	0.289	0.284	0.285	0.278	0.279
	ISIS	0.291	0.287	0.286	0.285	0.281	0.281	0.279	0.279
RPL* not used	FLOODTIDE[1]	0.316	0.312	0.309	0.301	0.296	0.291	0.286	0.279
	MIKE 11[2]	0.307	0.301	0.299	0.294	0.289	0.286	0.281	0.28
	HYDRO 1D	0.301	0.296	0.294	0.291	0.286	0.286	0.278	0.28
	CHANNEL	0.303	0.299	0.297	0.292	0.289	0.286	0.281	0.279
	LD01	0.303	0.299	0.297	0.292	0.287	0.285	0.280	0.280
	GLC-BACKWATER	0.31	0.30	0.30	0.29	0.29	0.29	0.28	0.28
	BAKWATER	0.31	0.30	0.30	0.29	0.29	0.29	0.28	0.28

* Relative Path Length, [1]convayance type 1, [2]Hydraulic radius

TABLE 7.4 : *TEST 7Part C - Results from each software package*

As with Part B of Test 7, Part C shows a similar outcome for the test results. However, this time with the exception of FLUCOMP and ISIS all of the packages consistently calculated the water surface around 6% lower than the experimental result. Both ISIS and FLUCOMP calculated a water surface profile around 10% and 7% respectively lower than the experiment result at the upstream boundary. This would again question the relevance of relative path length to this test situation and also the use of 1-D models to predict what is considered to be a 3-D flow regime.

FLOODTIDE

As with Test 6 the boundary values and the cross-sections were input to the required number of decimal places, with the boundaries between the main channel and flood plain being defined by the use of markers. The Manning's *n* value could be defined for the changing roughness across the section, although the meandering channel which is represented by defining the Relative Path Length could not be taken into account, as the flood plain area could not be obtained from the information available with the data set.

There are three 'conveyance types' which can be defined for the river channel, they are(13):

Type 0 - Einstein's Method: uses individual Manning's *n* values for each segment to calculate a weighted average Manning's *n* value for the whole cross-section.

Type 1 - The Sum of Segments Method: calculates the conveyance of the channel as the sum of the conveyances of the segments.

Type 2 - The Divided Channel Method: applies Einstein's Method to the main channel and flood plain areas separately to obtain K_{mc} and K_{fp} respectively.

The test was undertaken assuming a relative path length value of 1 and proved to be simple and straightforward to execute. As always with FLOODTIDE a critical backwater calculation was required so as to provide the initial conditions for a subsequent 'normal' backwater calculation. Calculations using all three of the above conveyance types were undertaken, however, only the results from Type 0 which show the closest correlation to the experimental results are presented in Tables 7.2, 7.3 and 7.4 for Parts A, B and C respectively. However, the results from all three conveyance types are illustrated in Graphs 2, 13 and 25 of Appendix F. The results from conveyance Types 1 and 2 produced a water surface profile which was noticeably lower than that when conveyance type 0 was used. The water level at the upstream boundary was generally 0.01m lower when conveyance types 1 and two were used.

Although FLOODTIDE can take account of a meandering channel the results from this test should be treated with some caution as the 'FLOODTIDE' Relative Path Length required by the package could not be obtained from the information provided for the test.

HEC-RAS

As with Test 6 the boundary values and the cross-sections were input to the required number of decimal places. HEC-RAS allows a Manning's *n* value to be defined for each pair of co-ordinates representing the cross-section; hence the changing roughness across the section could be accurately represented. As discussed for Test 6 HEC-RAS does not consider relative path length and only considers reach length for the left over-bank, right over-bank and main channel. The values used in Test 7 for the main channel length were taken directly from the data set and the over-bank reach lengths were calculated from:

Over-bank Reach Length = Relative Path Length x Main Channel Length.

The values used at each cross-section as calculated from the above equation are presented in Appendix F, Table 1.

The undertaking of a subcritical calculation produced a smooth backwater profile for all parts of the test and are illustrated in Appendix F, Graphs 3, 14 and 26. The calculated water levels at each cross-section are presented in Tables 7.2, 7.3 and 7.4 for Parts A, B and C respectively. For Part A the results show a close agreement with the values calculated using ISIS, FLUCOMP and HEC-2, all of which use relative path lengths or equivalent. It should be noted that a Froude number of less than 0.37 was calculated for the complete length of the channel and the expansion and contraction values (shock) were set at zero for each cross-section. For Parts B and C the calculated water levels are higher than those calculated by ISIS, and FLUCOMP, and are closer to the results from the other software packages which do not use relative path length.

HEC-2

The setting up of this test was relatively straightforward and required the same approach as used for Test 6. HEC-2 divides the cross-section into three main sections namely: the left flood plain, main channel and right flood plain. For each division of the cross-section the Manning's *n* value should be defined as well as the length to the next cross-section. Hence the varying roughness for the flood plain and main channel could be accurately defined and the meandering of the channel could be represented in a similar manner to that for HEC-RAS.

The boundary and cross-sectional data were input to the required number of decimal places and the over-bank reach lengths were the same as those used for HEC-RAS (see Appendix F, Table 1)

For all parts of the test the undertaking of a subcritical calculation produced a smooth backwater profile as illustrated in Appendix F, Graphs 4, 15 and 27. The calculated water level at each cross-section is presented in Tables 7.2, 7.3 and 7.4m, and was calculated only to two decimal places resulting in the jagged appearance of the water surface profile. The results for all parts of Test 7 show a close comparison to the values calculated by HEC-RAS. For part A it should be noted that the Froude number throughout the channel was calculated at less than 0.34. As with HEC-RAS the water level results for Parts B and C are noticeably higher than those calculated by ISIS and FLUCOMP, and are closer to the results from the other software packages which do not use relative path length.

MIKE 11

It is not possible with MIKE 11 to take into account the effects of a meandering channel by means of relative path lengths or equivalent. The only way in which MIKE 11 is able to include the effects of bends is to define additional storage areas(17) and/or with the use of additional channels to represent the flow path over the flood plains. This method has not been used as it would deviate considerably from the benchmarking data set and would require considerable calibration for accurate results to be obtained.

MIKE 11 approximated the test by employing the cross-sectional data as defined in the data file but treated the channel as straight. MIKE 11 can take the varying Manning's roughness across a section into account by the use of the relative resistance value defined in the raw cross-section data (Menu A.6.5.R). Once a global Manning's *n* value has been set-up in the supplementary data (0.017 for this test) then any variation in the roughness can be included by entering a relative resistance value different from 1 for that part of the section. This is clearly described and illustrated in MIKE 11's user manual in Section 2.9. To define a Manning's value of 0.019 on the flood plain a relative resistance number of 1.118 is required. However, MIKE 11 rounds-off this value to one decimal place, which results in a Manning's *n* value of 0.0187.

It should be noted that the chainage for the cross-sections could only be input to three decimal places when converted to kilometres (the format required by MIKE 11), when five decimal places was given by the data set. In addition, the water level boundary value and the cross-sectional data were rounded-off to two decimal places when three decimal places were required, and the discharge boundary was rounded-off to three decimal places when five decimal places were required.

I was observed that the resistance radius method of calculating conveyance is more suitable for cross-sections with significant variations in shape (such as over bank flow paths), whereas the hydraulic radius is more appropriate for deep and narrow uniform sections. For hydraulic radius calculations where over bank flow paths occur then adjustment of the relative resistance should be made to invoke the parallel channel analysis facility as described in the reference manual for MIKE 11 (p7). Since the Manning's n values for the main channel and flood plain are different for Test 7 the hydraulic radius method was adopted for the test. However, investigations using the resistance radius were also made.

The results obtained from Test 7 for MIKE 11 using the hydraulic radius method for conveyance are presented in Tables 7.2, 7.3 and 7.4 for Parts A, B and C respectively. The results using both the hydraulic radius and resistance radius methods are illustrated in Appendix F, Graphs 5, 16 and 28. It can be seen from all these three graphs that the resistance radius method for conveyance calculation consistently calculates even lower water surface profiles along the length of the channel. For parts A, B and C it can be seen from Tables 7.2, 7.3 and 7.4 that the backwater profiles are very similar to those calculated by HYDRO-1D, which uses the same approach to the test. The Froude number at cross-sections as calculated from an unsteady run (with constant boundary conditions) was less than 0.37 for all cross-sections throughout the river system and for all parts of the test.

HYDRO-1D

As discussed for Test 6 HYDRO-1D does not use relative path lengths or an equivalent to account for the meandering of a channel. However, a possible method of accounting for the effects of flow through a bend with HYDRO-1D may be to add additional channels so as to provide a flow path for flow over the flood plains and/or with the use of storage areas. This method, although appropriate in some situations, has not been used for Test 7 as it was considered inappropriate and would require considerable calibration for accurate results to be obtained.

As an approximation to the test HYDRO-1D employed the cross-sectional data as specified in the benchmarking data set and considered only the main channel length.

HYDRO-1D's GUI provided a very quick and simple method of setting up the test configuration, however problems did arise when attempting to input the cross-sectional data. HYDRO-1D can only input data to two decimal places whereas Test 7 requires three decimal places for the cross-sectional data and up to five decimal places for the boundary conditions. This resulted in varying bed slopes along the length of the channel and boundary values that differ slightly from the benchmarking data.

As discussed for Test 6 HYDRO-1D has the ability to define a Manning's *n* value for the left flood plain, right flood plain, normal channel in and out of bank flow, and normal channel low bank flow. This has enabled an accurate representation of the varying Manning's value for a cross-section as stipulated in the benchmarking data set.

The results obtained from a converged subcritical calculation are presented in Tables 7.2, 7.3 and 7.4 and are illustrated in Appendix F, Graphs 6, 17 and 29 for Parts A, B and C respectively. Each of the backwater profiles are very similar to the solution using MIKE 11 which uses the same approach for this test and are not dissimilar to the results from the software packages which can represent a meandering channel by the use of relative path lengths or equivalent. HYDRO-1D does not output the Froude number and hence its value cannot be reported. However, it is useful to point out that although the calculation results are converged it does not mean the results are correct.

FLUCOMP

As discussed for Test 6 FLUCOMP can cope with flood plains and can also take into account the effects of meandering channels. The relative path length as defined in the benchmarking data set is not a value used to define the meandering of a channel, instead the plan area of the flood plain and/or plain lengths are required. Unfortunately, the flood plain area could not be obtained from the information available hence flood plain lengths as calculated for HEC-RAS provided the most appropriate method of undertaking the test (see Appendix F, Table 1).

FLUCOMP defines the Manning's roughness values at a cross-section in either full format or simple format. The full format method allows the roughness to vary between the points used to define the cross-section in the section data file, whereas the simple format defines a roughness value for the left and right flood plains and the main channel as previously described and adopted in Test 6. Since the section roughness was constant over the left and right flood plains and the main channel, the simple format was considered to be more appropriate for Test 7.

The results obtained from Test 7 for FLUCOMP are presented in Tables 7.2, 7.3 and 7.4 and are illustrated in Appendix F, Graphs 7, 18 and 30 for Parts A, B and C respectively. For all parts of the test the backwater profiles are much lower than the experimental results although they are very similar to the solutions for HEC-RAS and HEC-2 which also use the same approach (and values) to undertake the test. FLUCOMP calculated the critical flow number(19) at cross-sections throughout the channel at a value of less than 0.14, indicating subcritical flow conditions.

As an investigative measure three calculations were made for Part A of the test defining the local maximum flood level at 8.0m, 1.0m and 0.5m while using 30 points in the data base table (see Table 11.4 of FLUCOMP user manual). Results using a flood plain level of 1.0m and

0.5m gave the same water level of 0.202m at the upstream boundary cross-section, whereas a value of 8.0m produced a reduced water elevation of 0.196m. This suggests that the local maximum flood plain level should be selected whilst considering the likely maximum water level along the length of the channel.

ISIS

The benchmarking data supplied for this study was in ISIS format and hence did not require any alterations or approximations. As discussed for Test 6 the manual for ISIS clearly describes in section 4.3.4 the set up and technique employed to model flood plain flow and the meandering of a channel. The reader is directed towards the comments made for Test 6, which also apply for Test 7 with the following addition.

The forms editor provides a convenient and simplistic method of allowing input and editing of the roughness data for each cross-section. ISIS takes into account the varying roughness across a section by defining a Manning's *n* value for each set of co-ordinates used to define a cross-section.

The results for an unsteady direct method calculation for Test 7 are presented in Table 7.2, 7.3 and 7.3 and are illustrated in Appendix F, Graphs 8, 19 and 31 for Parts A, B and C respectively. The output file produced by ISIS shows that the calculated Froude number is no greater than 0.35 at any cross-section in the river system for any part of the test. As an investigative measure unsteady calculations were made with constant boundary conditions the results of which were identical to the steady state calculations.

CHANNEL

The same approach as used for Test 6 was adopted for Test 7 and produced the same problems as discussed previously. It should be noted that the manual for CHANNEL gives no reference to calculations with meandering channels and makes no use of the relative path length as defined in the data set.

TEST 7 – FCF Blackwater Model

In addition to the problems discussed for Test 6, several further problems were encountered for this test: (i) CHANNEL does not allow the input of chainage to any decimal places, hence the chainage was rounded off cumulatively, (ii) cross-sectional off-set values (x co-ordinates) must increase from one value to the next, i.e. the repeated values in sections 5 and 7 were removed, and (iii) the input of the boundary values in CHANNEL can only be made to three decimal places, hence the discharge boundary was rounded off to 0.061cumecs.

The results for the backwater calculation are presented in Tables 7.3 and 7.4 and are illustrated in Appendix F, Graphs 9, 20 and 32 for parts A, B and C respectively. Each of the backwater profiles are very similar to the solution using MIKE 11 which uses the same approach for this test. The results are not dissimilar to those from the software packages that can represent a meandering channel by the use of relative path lengths or equivalent. No warning messages were given to indicate critical flow conditions, although the results should be treated with caution considering the limitations of CHANNEL and the input of data.

LD01

A meandering channel cannot be accurately represented in LD01 as the effects of path lengths for flow over the flood plain are not taken into account. The test was undertaken by inputting the cross-sections as given in the data file and with distances between cross-sections taken as the distance along the main channel. The flood plain was represented in the same manner as for Test 6, the comments for which should be taken into account.

The undertaking of Test 7 with LD01 did not produce a satisfactory outcome for Part A. Results could not be obtained when using the discharge boundary value as stipulated in the data set. Through investigation it was found that results could not be obtained when using a discharge boundary value of 0.1 cumecs or below, however when using a value of 0.11 or greater then a solution was produced. However, for Parts B and C of the test with and increased boundary conditions a result was obtainable from LD01. The results of which are presented in Table 7.3 and 7.4 and Graphs 21 and 33 for Parts and B respectively. It can be clearly seen that the water surface profile obtained by LD01 for Parts B and C of the test falls within the bounds of an envelope encompassing the results from other software packages. It should be noted that the water elevation along the length of the channel was not altered by LD01 to a critical water level, thus indicating the result is subcritical throughout.

GLC-BACKWATER

GLC-BACKWATER cannot take account of relative path length or equivalent to represent the meandering of a channel. In addition GLC-BACKWATER can only define the Manning's n value for a complete cross-section, hence a Manning's n value of 0.017 was adopted to represent the whole cross-section. Investigations with other averaged values were not investigated since the results obtained were similar to those obtained by the other software packages. The same approach as used for Test 6 was again used here for Test 7, the comments for which also apply and should be read accordingly. It should be noted that GLC-BACKWATER can take into account varying Manning's *n* values across a section as required by the benchmarking data set.

The results for Test 7 are presented in Tables 7.2, 7.3 and 7.4 and illustrated in Appendix F, Graphs 11, 22 and 34 for Parts A, B and C respectively. It should be noted that GLC-BACKWATER gives results to two decimal places, hence the jagged appearance of the water surface profile in Graphs 10, 21 and 33 of Appendix F. However, it can be clearly seen that the water surface profile obtained by GLC-BACKWATER for each part of the test falls within the bounds of an envelope encompassing the results from other software packages. It should be noted that the water elevation along the length of the channel was not altered by GLC-BACKWATER to a critical water level, thus indicating the result is subcritical throughout.

BAKWATER

The use of relative path length or equivalent to represent the meandering of a channel cannot be taken in account by BAKWATER. However, BAKWATER does have the ability to consider the flood plains. The same approach as used for Test 6 was again used here for Test 7, the comments also apply and should be read accordingly. It should be noted that BAKWATER is able to define a global Manning's roughness value for the flood plains and the main channel of a river.

TEST 7 – FCF Blackwater Model

The results obtained for Test 7 are presented in Table 7.2, 7.3 and 7.4 and illustrated in Appendix F, Graphs 1 and 11 for Parts A, B and C respectively and should be treated with caution considering the approach used to undertake the test. It can be clearly seen that the water surface profile obtained by BAKWATER for Parts A, B and C of the test falls within the bounds of an envelope encompassing the results from other software packages. The results are very similar to those from software packages that do not use relative path length as a means of representing a meandering channel. It should be noted that the water elevation along the length of the channel was not altered by BAKWATER to a critical water level, thus indicating the result is subcritical throughout.

2.7.4 Summary

1 Only HEC-RAS, HEC-2, FLUCOMP and ISIS were able to take account of the bends in the model by using the relative path length or equivalent. The data available did not allow FLOODTIDE to be run properly to include the effects of bends.

2 Results from HEC-RAS, HEC-2, FLUCOMP and ISIS were all noticeably lower than the experimental results and also lower than the results from the other software packages that did not take account of the effects of the bends.

3 The scale of the laboratory model set-up and the accuracy required by the data set to define the cross-sectional and boundary data, make it difficult to assess for this test the accuracy and suitability of each package for modelling meandering flows. Hence, as for Test 6, this test is again of limited value.

4 The results from the software packages that did not take account of the meanders in the physical model and treated the channels as being straight generally showed closer agreement with the experimental results. This was thought to be due to several factors, including Reynolds number and scale effects.

5 Once the effects of round off are taken into consideration all of the software packages, except LD01, demonstrated that they could calculate a smooth water surface profile for

each of the three flow conditions tested. LD01 did not provide results for Part A of this test.

6 In MIKE 11 the hydraulic radius method, as opposed to the resistance radius method produced results that compared better with the experimental results. This was in contrast to the previous Flood Channel Facility test, where the resistance radius method gave better results.

7 In FLOODTIDE the results using conveyance Type–0 (i.e. Einstein's method), as opposed to the results obtained using Types 1 and 2, produced results that compared better with the experimental results.

2.8 TEST 8 – Tidal River Thames

2.8.1 Aim of Test

This test was to examine the ability of each software package to model unsteady flows for the tidal section of the river Thames between Molesey and Southend. Calibration and verification of the models were to be made with the measured data available from 7 level gauges.

2.6.2 Introduction

The data set provided for this test was in ISIS format and was generally not well suited to the other software packages being tested within this study. Problems were encountered with the running of the model with ISIS and also the setting up of the test for the other software packages. Hence, the project Steering Group decided that Test 8 should be dropped and the test data available for the river Calder was adopted as an alternative.

Test 12 is the replacement for Test 8 and examines the ability of each software package to model unsteady flows with a complex river network. The test data was available in MIKE 11 format, and not ISIS format, with calibration data also being available to verify the models.

2.9 TEST 9 - River Blythe

2.9.1 Aim of Test

The aim of this test is to investigate the ability of each of the software packages to model a section of the river Blythe under unsteady flow conditions by use of measured and interpolated cross sections. It is also designed to test the stability of the software packages when a relatively small channel flow overflows onto a wide flood plain

2.9.2 Introduction

A section of the river Blythe is to be modelled by the use of measured and interpolated cross-sections. There are three parts of the test designed to assess the importance of selecting suitable locations for the interpolated cross-sections and an additional part to test the stability of the software packages when a relatively small channel flow overflows onto a wide flood plain. Altogether there are four parts to the test are namely:

Test 9A - River Blythe as supplied by Environment Agency Midlands Region with 39 measured cross sections and 26 interpolated cross sections.

Test 9B - River Blythe with all interpolated cross sections from Test 9A removed.

Test 9C - River Blythe as for Test 9B but with the addition of 30 new interpolated cross-sections, generally at critical points of the network, e.g. steep gradients, and most of them being at different locations from those for Test 9A.

Test 9D - River Blythe as for Test 9C but with increased inflow hydrographs designed to produce over bank flow.

The total length of the river reach is 13,614 m, with 5 relatively steep regions located approximately at chainage 4,500 m, 5,800 m, 9,300 m, 10,700 m and 12.700 m. For each part of Test 9 there is one water level and three inflow boundaries defined along the river network. At 13614m there is a fixed downstream water elevation of 72.6m, which is considered by the authors to be inappropriate for unsteady flow conditions since local effects on the results will occur close to the downstream boundary. At 0m there is an upstream inflow hydrograph, and at 6559m and 4244m there are tributary inflow hydrographs. The same three inflow hydrographs were used for Parts A, B and C as illustrated in Appendix G, Graph 1. However, for Part D the hydrographs are increased, as illustrated in Appendix G, Graph 2 so as to produce flow on the flood plains. Each cross-section throughout the river network has a Manning's roughness of 0.06 for the flood plains and 0.03 for the channel.

A plan location of the river Blythe used for the test is given in Figure 4.7 of the Stage One Report. Information pertaining to bridges within the length of the Blythe being modelled has not been supplied with the data set and has thus not been accounted for, hence calibration data is not appropriate for comparison and has therefore not been provided.

2.9.3 Results

This test is an unsteady flow test, hence none of the steady state models were considered for testing. The ability of each of the unsteady models to undertake each part of Test 9 is summarised below in Table 9.1 and a summary of the calculated maximum water elevations at selected locations is given in Table 9.2.

	TEST 9			
SOFTWARE	Part A	Part B	Part C	Part D
FLOODTIDE	×	×	×	×
MIKE 11	✓[1]	✓	✓[1]	✓[1]
HYDRO 1D	✓[1]	✓	✓[1]	✓[1]
FLUCOMP	✓[1]	✓	✓[1]	✓[1]
ISIS	✓[1]	×	✓	✓

[1] Only possible with external interpolations

TABLE 9.1 - *Summary of software packages able to undertake Test 9*

Table 9.3 summarises the comparison graphs in Appendix G, by each software package for all parts of Test 9. Model comparisons have been made at times: 0hr, 15hr, 17.5hr, 25hr, 35hr and 55hr, and also at locations: 12,104m, 9,579m, 5,060m, 2,514m and 0m.

		Maximum Water Elevation at Chainage :				
	SOFTWARE	0 m	2,514 m	4,560 m	7,061 m	10,079 m
Part A	MIKE 11[2]	84.908	82.908	81.376	79.474	76.947
	HYDRO 1D[2]	84.947	83.033	81.418	79.476	77.183
	FLUCOMP[2]	84.927	82.994	81.393	79.494	78.068
	ISIS[1]	84.920	83.008	81.418	79.459	77.192
Part B	MIKE 11[2]	84.963	82.886	81.370	79.458	76.951
	HYDRO 1D[2]	85.033	82.924	81.414	79.474	77.613
	FLUCOMP[2]	85.028	82.917	81.399	79.309	77.473
	ISIS[1&2]	NO RESULT				
Part C	MIKE 11[2]	84.954	82.902	81.369	79.465	76.982
	HYDRO 1D[2]	85.061	82.960	81.425	79.478	79.947
	FLUCOMP[2]	85.052	82.950	81.405	79.313	76.774
	ISIS[1]	85.030	82.928	81.430	79.461	76.931
	ISIS[2]	85.013	82.955	81.428	79.462	76.946
Part D	MIKE 11[2]	85.298	83.061	81.893	79.927	77.239
	HYDRO 1D[2]	85.061	82.960	81.425	79.478	79.947
	FLUCOMP[2]	85.283	83.144	81.529	79.419	76.887
	ISIS[1]	85.338	83.074	81.835	79.818	77.192
	ISIS[2]	85.333	83.119	81.829	79.826	77.177

[1] with internal interpolations. [2] with external interpolations.

TABLE 9.2 : *TEST 9 - Summary of calculated maximum water elevations at selected location*

GRAPH	RESULT
1 – 2	Inflow hydrographs
3 – 6	Comparisons of maximum water levels along the channel for Parts A, B and C by MIKE 11, HYDRO-1D, FLUCOMP and ISIS respectively.
7 - 10	Comparisons of maximum water levels along the channel among packages for Parts A, B, C and D respectively.
11 – 17	Comparisons of water levels among packages at selected times and locations for Part A.
18 – 24	Comparisons of water levels among packages at selected times and locations for Part B.
25 – 31	Comparisons of water levels among packages at selected times and locations for Part C.
32 – 39	Comparisons of water levels for ISIS internal and external interpolations at selected times and locations for Part C.
40 – 46	Comparisons of water levels among packages at selected times and locations for Part D.
47 - 70	Comparison of model results for Parts A, B and C at selected times and locations.

TABLE 9.3 : *TEST 9 - Summary of graph results to be found in Appendix G.*

INTERPOLATIONS

In ISIS several interpolated cross-sections can be defined between two measured cross sections, without providing cross-sectional geometry to the interpolated sections. Instead of directly calculating conveyance values for the section from geometric data the cross-sectional parameters are interpolated from corresponding values of adjacent measured cross-sections. Such an approach is referred as internal interpolation. Alternatively, externally generated interpolated cross-sectional geometry can be provided at desired locations.

The data set provided for Test 9 is in ISIS format and is generally not suited to the other software packages which can cope with unsteady flow conditions. HYDRO-1D, FLUCOMP and FLOODTIDE do not have the ability to cope with internal interpolations. MIKE 11 can only handle internally interpolated sections at equal distance and is not suited to the data set since specific locations of varying chainage for the interpolated sections are required by the test.

To overcome the problem of interpolated cross sections for Test 9 the University of Bradford have developed a programme which will calculate geometric cross sections at a specified location between two measured cross sections. Therefore the effects of providing additional cross-sections by way of interpolation can be studied. The locations of the measured and interpolated cross sections are illustrated for Parts A, B and C in Figure 9.1 as defined by the Stage One report.

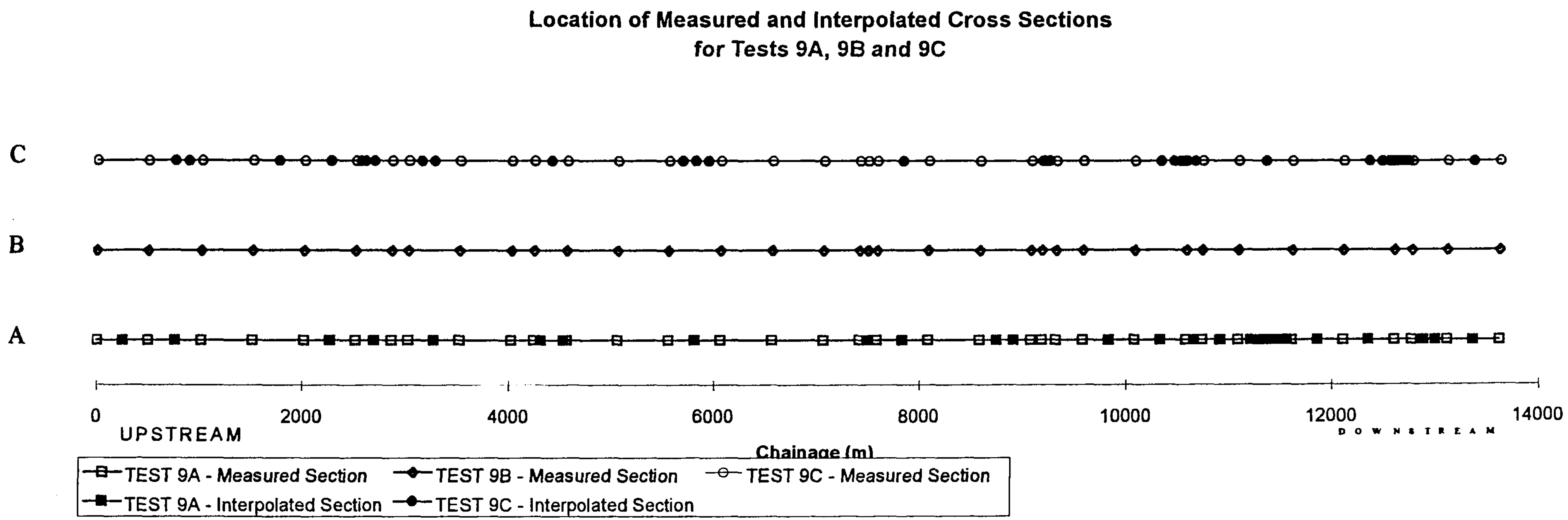


FIGURE 9.1 : *Test 9 - Location of measured and interpolated cross-sections*

NOTE :

TEST 9A: 39 measured cross sections + 26 interpolated cross sections

TEST 9B: 39 measured cross sections

TEST 9C: 39 measured cross sections + 30 interpolated cross sections

TEST 9D: same as 9C

FLOODTIDE

Since FLOODTIDE is unable to calculate internal interpolated sections the external interpolated sections as calculated by the University of Bradford were deployed for the test. The setting up of the river network was straightforward. However, great care had to be taken when producing the ASCII data file so as to avoid typing errors for the cross sectional and boundary data. As discussed for Test 7, FLOODTIDE can easily cope with varying Manning's roughness coefficients across a section and also for different conveyance regions, including the main channel and floodplain.

This test was relatively straightforward to set up and run for the steady (backwater) calculation, whereas, considerable problems were encountered when attempting to undertake the unsteady simulation. The undertaking of an unsteady calculation with FLOODTIDE should be a simple procedure. The user simply selects the data and boundary files, the method of calculation either Dynamic or Kinematic, the start and finish times for the simulation, and finally the time step to be used. For all parts of Test 9 a simulation would always begin. However, the simulation was short-lived due to the program crashing with a floating-point overflow. In an attempt to overcome this problem investigations were made with: (i) the time step - as low as 0.01 second, (ii) the number of layers used to calculate the conveyance tables, and (iii) Theta (θ) - the weighting parameter for the 4-point Preissmann scheme, but all with no success. The software vendors were contacted about this problem and they suggested that the pseudo time-stepping method be adopted once the backwater calculation had been undertaken. This was suggested so as to provide better initial conditions from the full hydrodynamic equations which the pseudo time-stepping method employs. Unfortunately this did not remedy the problems as previously encountered so the test files were forwarded to the vendors for checking. However, the vendors did not identify any problems with the test files and were unable to suggest any alternative methods of obtaining a solution, hence for Test 9 no results have been presented.

MIKE 11

Although MIKE 11 can cope with internal interpolations the external interpolated sections as calculated by the University of Bradford were deployed for the test since the location of the interpolations as provided in the data set were not suitable for the internal interpolations of MIKE 11.

Once the geometric data for the interpolated cross sections had been obtained from the external interpolation program, they were then subsequently converted into MIKE 11 format for the cross sectional database. This enabled the test to be simply and quickly set up leaving only the boundary files, river system and supplementary data to be defined within MIKE 11. The simulations for all parts of Test 9 were very simple to undertake. However, selection of the time step was important for a successful simulation. Generally time steps greater than two minutes resulted in MIKE 11 crashing for all four parts of the test. Therefore, a time step of 15 seconds was used for all of the results presented within this report which was also consistent with the other packages under test. The default calculation settings as specified in the supplementary data file were used for all parts of Test 9. Investigations into the effect of altering the default values were not undertaken since it was considered to be beyond the scope of this study.

The MIKE 11 long section results for Part A are illustrated in Appendix G, Graphs 11 through to 17 and show smooth water surface profiles along the complete length of the river for each of the time steps illustrated. With the exception of the upstream boundary at t = 0.0hr, it can clearly be seen from Graph 17 that the calculated water elevations at the selected locations throughout the simulation are consistently in the region of 10-20cm lower than the results from HYDRO-1D, FLUCOMP and ISIS, for Part A. However, it should be noted that the results from MIKE 11 for Parts A, B and C were more consistent than any other software package, as discussed later on.

For Part B, the test without interpolations, it can be seen from Appendix G, Graphs 18 through to 24 that the calculated water surface profiles are much smoother than the profiles calculated by FLUCOMP and HYDRO-1D. From Appendix G, Graphs 53 through to 58 it can be seen that the removal of the interpolated cross sections has little effect on the calculated water levels at the selected time steps. Again for Part C the calculated water surface is much smoother than that calculated by FLUCOMP and HYDRO-1D and at times is very similar to the results calculated by ISIS. It can be seen in Graphs 53 through to 58 that the effect of adding new interpolated sections at different locations does not significantly affect the results calculated by MIKE 11.

The results for Part D with the increased inflow hydrographs consistently show a smooth water profile as illustrated in Appendix G, Graphs 40 through to 45. With the exception of the profiles at 0.0hr and 55.0hr, the water surface profiles for the majority of the river reach are very similar to those calculated by ISIS. The difference with the profiles at 55.0hr can be attributed to the phase difference in the peak water levels.

Generally the results from MIKE 11 are consistent and show that the addition or removal of the externally interpolated cross sections has little effect on the results. Increasing the inflow hydrographs so that flow is produced on the floodplains does not produce instabilities in the solution at the point when the water level changes from within the channel to the floodplain.

HYDRO-1D

Since HYDRO-1D is unable to calculate internally interpolated sections the external interpolated sections as calculated by the University Bradford were deployed for the test. The setting up of the river network was not as simple as with some of the other packages, and care had to be taken when producing the individual cross sectional data files so as to avoid typing errors. The network file although straightforward to set up gave rise to some problems with the results file if sections were to added, moved or deleted. It was found that if a section was added after the network had been set up then the result from that section would be printed at the end of the results file and not at the point of insertion in the network. Since the test essentially required the addition and removal of many cross sections for the different parts of the test a network was created for each part so as to avoid errors in analysing the data. The boundary files were simple to set up and define within the network, with the inflow location for the upstream boundary and tributaries being defined at nodes (cross sections). As discussed for Test 7, HYDRO-1D splits a cross section into two regions, the convective region and the normal region. Hence, different Manning's *n* values for the main channel and floodplains were accounted for in the test.

Parameter	Value
Minimum number of iterations per time step	5
Maximum number of iterations per time step	50
Preissmans Factor (default = 0.55)	0.55
Range of linearisation (default = 0.22)	0.22
Total Residual Limit for Convergence (default = 0.1)	0.1
Number of iteration before applying optimal head correction	15
Maximum allowable head correction	0.1
Maximum allowable flow correction	1.0

TABLE 9.4 - *Summary of calculation settings in HYDRO-1D for Test 9*

To undertake an unsteady calculation with HYDRO-1D a steady calculation has first to be undertaken so as to provide initial conditions for an unsteady/transient run. There are several calculation settings that can be altered in the run menu of HYDRO-1D, with the values used for this test being given in Table 9.4.

The selection of the time-step in HYDRO-1D is an important consideration when undertaking an unsteady run. The user not only needs to consider the model set up but also the volume of results that will be generated by HYDRO-1D. Results are produced for each time-step for the whole period of the simulation for which the user has no control. The smallest time-step that HYDRO-1D allows is 0.01hr. However, for Test 9 a time-step of 0.1hr was the smallest value possible for a successful simulation. In order to analyse the results a simple conversion program was written so as to extract the required data from the 4MB result file(s). The results file does not give information about the convergence of the solution at each time step, hence the user should monitor the values for the residual limit and number of iterations throughout the simulation on screen or either check the screen dump file in the HYDRO-1D source code directory.

The long section results from Part A are illustrated in Appendix G, Graphs 11 through to 16 for the time steps selected. At 0hr the water surface profiles are reasonably smooth, although there are some locations where the water levels are very close to the bed level or noticeably higher than the results from MIKE 11 and ISIS. The results at 15.0hr, 25.0hr and 35hr are very similar to all of the other packages at those times. However, at 17.5hr and 55hr the results are noticeably different - especially at 17.5hr. In Appendix G, Graph 17 the time series plots show several different trends. At 12104m the water level initially follows the trend of the results from MIKE 11 and then increases to follow the trend from the FLUCOMP and ISIS results. At 9579m and 5060m the water level is very similar to that of FLUCOMP and ISIS throughout the simulation, although there are periods of slight deviation. However, the results at 2514m show significant instabilities in the solution during the early and late periods of the simulation, even though the water level is below the channel crest (floodplain level 83.78m). At the upstream boundary (chainage 0m) the water level rises and falls in close approximation with the results from all of the other packages.

For Part B the removal of the interpolated cross sections noticeably changed the results for HYDRO-1D as illustrated in Appendix G, Graphs 59 through to 64. The long section results in Appendix G, Graphs 18 through to 23, show a similar trend to those results from FLUCOMP, although, the results are noticeably different to those from MIKE 11. Graph 24 in Appendix G, clearly shows that the results for Part B are equivocal throughout the simulation.

The introduction of new interpolated cross sections for Part C of the test generally resulted in the results becoming not too dissimilar from those obtained using the other packages. With the exception of around 12604m and at 15.0hr, it can clearly be seen from Appendix G, Graphs 25 through to 30, that the water surface profiles are generally similar to that obtained using FLOODTIDE and MIKE 11. Graph 31 shows that there are significant instabilities in the solution at 12,104m, which is immediately upstream of the sudden jump in water level at 12,700 m in Graphs 26 to 29. However, at 9,579m, 5,060m, 2,514m and 0m the water levels follow a smooth and similar trend to the results from the other packages under test.

The long section results for Part D with the increased inflow hydrographs are presented in Appendix G, Graphs 40 through to 45, and consistently show a similar result to that from FLUCOMP but which is noticeably lower than those results for MIKE 11 and ISIS. As with Part C there is a sudden jump in the water level at 12604m at each of the time steps selected, with the exception of 0.0hr. The time series plot at 12104m, in Graph 46, of Appendix G, clearly illustrates the instabilities for the solution in this region of the river network. While undertaking the simulation it was noticed that the residual limit was frequently not reached and that the maximum number of iterations (50) were being executed. It was found that by increasing the maximum number of iterations to 150 the instabilities could be smoothed out. However, significantly different results were obtained after around 17hr, as can be seen from Graph 46.

In order to obtain satisfactory results from HYDRO-1D, it was felt that a much smaller time step needed to be used to overcome the instability problem. However, this was not possible with the existing version of the software.

FLUCOMP

Since FLUCOMP is unable to calculate internally interpolated sections the external interpolated sections as calculated by the University of Bradford were deployed for the test. Several files are required to make an unsteady flow simulation in FLUCOMP. The input of data is not a straightforward procedure. The user must fully understand the required elements of each of the following files:

*.SDF - section data file
*.RDF - roughness data file
*.FDF - flow data file
*.HDY - hydrograph file

In order to minimise errors in the section data file a program was written to produce the required section data file incorporating all of the measured and externally interpolated cross sections. The roughness data file was simple to set up as the Manning's roughness coefficient for the main channel and the flood plains was constant along the complete length of the river. Although the manual was comprehensive, it was difficult to follow when attempting to set up the flow data hydrograph files. However, once the requirements had been clarified with the vendors the undertaking of the test was a simple procedure.

To undertake the test the user is firstly required to use FLUCOMP to produce a backwater solution and then to undertake an unsteady simulation. The user specifies the start and finish times for the simulation, the time step to be used and the frequency that results are to be recorded in the flow data file before the simulation is started. The user must take caution with the file names given for each of the data files required, as files with the same name but different extensions will cause problems with the undertaking of a calculation and production of results.

All the simulations undertaken for Test 9 were with a time step of 15 s so as to be consistent with the other software packages under test. The long section results for Part A are illustrated in Appendix G, Graphs 11 through to 16 and consistently show a similar water surface profile for each of the time steps selected. However, there are several locations where the variation in the water surface profiles differs noticeably from the profiles produced by MIKE 11 and HYDRO-1D. Consistently at around 13,200m the water level is undulating, which could be attributed to the fixed downstream water elevation throughout the simulation. Also at 10,654m and 2,264m there are noticeable reductions in the calculated water levels at each of the time steps selected. From Graph 17 it can be seen that at the selected cross sections the water level varies smoothly throughout the simulation period with a similar trend being observed to the results obtained from ISIS.

For Part B the removal of the interpolated cross sections generally eliminated the small variations in the water level that were previously produced in Part A. However, as a result the water level at around 10,000m was noticeably higher for each of the time steps selected for illustration in Appendix G, Graphs 47 through to 52. It can be seen in Graphs 18 through to 23 that the increase in the water levels gives a similar result to that predicted from HYDRO-1D. Generally the results from FLUCOMP are similar to those from HYDRO-1D. However, the instabilities that were produced for HYDRO-1D - as illustrated in Graph 24 - are not produced by FLUCOMP, although the variation in the water level at 12,104m is not as might be expected.

The addition of 30 new interpolated cross sections for Part C significantly improved the results obtained by FLUCOMP. It can be seen from Appendix G, Graphs 24 through to 30, that the water surface profiles at the selected time steps are smooth along the complete length of the channel. The results are similar to those from MIKE 11 and ISIS, with the exception of the result at 15.0hr. From Graph 31 it can be seen that the variation in the water level at 12,104m is significantly different from the result obtained for Part B and of a similar trend to the results from MIKE 11 and ISIS.

It can be seen from Appendix G, Graphs 41 through to 45 that for Part D smooth water surface profiles are calculated along the complete length of the river, unlike Parts A and B. It can be seen from Graph 46 that increasing the inflow hydrograph does not produce instabilities in the results. However, there is a significant difference in the water level at selected cross sections throughout the simulation when compared to the results from MIKE 11 and ISIS.

ISIS

The original data set provided for Test 9 was in ISIS format, hence the setting up of the test with internally interpolated cross sections was simple. However, the test did produce some unexpected results. Undertaking Parts A and B of Test 9 resulted in calculation failure, hence the software vendors were contacted for assistance. The vendor reported that neither Parts A nor B were expected to work and that only Parts C and D should be able to undertake the test. However, the test was successfully undertaken for Part A by use of the externally interpolated cross sections as calculated by the University of Bradford and as used for FLOODTIDE, MIKE 11, HYDRO-1D and FLUCOMP. As an investigative measure the externally interpolated cross sections for Part C were also tested for ISIS so as to provide a comparison with the internally calculated interpolations.

All of the simulations for Test 9 were undertaken using a time step of 15 s so as to be consistent with the other software packages under test. However, it should be noted that time steps of one minute or greater generally produced results with poor convergence or resulted in the model failing to complete a simulation.

For Part A, using external interpolations the results are illustrated in Appendix G, Graphs 11 through 17. From the long section plots (Graphs 11 through to 16) it can be seen that the calculated water surface profiles are smooth along the complete length of the river. However, due to the fixed downstream water level boundary there is on occasions a rapid increase in water elevation at the cross section immediately upstream of the water level boundary. From Graph 17 it can be seen that results using the externally interpolated cross sections produce a smooth variation in the water elevation with respect to time and have similar values as those obtained using the other software packages.

TEST 9 – River Blythe

The results using internally interpolated cross sections for Part C are illustrated in Appendix G, Graphs 25 through to 31. The long section results show smooth water surface profiles along the complete length of the river and are generally of a similar profile to those calculated using MIKE 11 and FLUCOMP. From Graph 31 it can be see that the variation in the water level with respect to time is consistent with the results that are calculated by MIKE 11 and FLUCOMP. However, there is some instability in the water level at 2,514m during the early and later stages of the simulation.

Graphs 32 through to 39 illustrate the difference in the results when using internal and external interpolations for Test C. Overall the results are extremely similar, with the maximum difference in the long section plots being less than 0.25% at 3,250m, as illustrated in Graph 38. It can be seen from Graph 39 that the instabilities that were previously produced by the internal interpolations at 2,514m are removed with the use of the external interpolations, although now the peak water level is slightly lower. Results for the other cross sections show almost identical profiles throughout the simulation. It can be seen from Appendix G, Graphs 65 through to 70 that results obtained using the internal interpolation for Part C are noticeably different to the results obtained using the externally interpolated cross-sections for Part A. The differences are especially noticeable at 10,000m in each of the long section plots and throughout the long section at 55hr.

The comparison produced between the internal and external interpolations for Part C, validate the program written by the University of Bradford for calculating the external interpolated cross sections. This ensures that testing can be undertaken for all the packages.

It can be seen from Appendix G, Graphs 40 through to 46 for Part D, that smooth water surface profiles are calculated along the complete length of the river - a result which is similar to that calculated for MIKE 11. From Graph 46, it can seen that the effect of increasing the inflow hydrographs does not produce instabilities in the results. However, at 12,104m there is a noticeably higher and delayed maximum water level.

2.9.4 Summary

1. Five packages were tested, namely: FLOODTIDE, MIKE 11, HYDRO-1D, FLUCOMP and ISIS, with additional cross-sections being placed at various regions by external interpolation where necessary. In the opinion of the authors this test was a good test case to study the scope of the models, but it was unfortunate that no data were again available to validate the predictions.

2. External interpolation was found to be comparable with internal interpolation, as illustrated by comparison of Test 9 Part C for ISIS.

3. FLOODTIDE was unable to produce any results for this test and the developers were unable to remedy the problem.

4. MIKE 11 produced consistent results for Parts A, B and C with little variation in the results for different sets of interpolated cross-sections. All other packages showed different predictions for various sets of specified cross-sections.

5. ISIS and HYDRO-1D exhibited instabilities at various stages during the model simulation. HYDRO-1D also produced steep water surface gradients along some regions of the river network.

6. Selection of a suitable time step for HYDRO-1D was limited to 0.1hr for this test. This was felt to be restrictive by the authors.

7. ISIS was only able to undertake Part A of the test using external interpolation, whereas internal interpolation resulted in failure of the model. ISIS also failed to run for Part B, where no interpolation was required..

8. Only ISIS and MIKE 11 were capable of undertaking unsteady calculations with internal interpolations. However, ISIS defines the specific location of a required interpolated cross-section, whereas MIKE 11 defines multiple interpolated cross-sections at equal incremental distances between measured cross-sections.

9. The fixed downstream water level boundary for each part of the test gave different results for each of the software packages. The distance over which the downstream boundary condition affected the results was found to be as much as 2km for HYDRO-1D and ISIS and 1km for FLUCOMP. However, it did not appear to affect significantly the results for MIKE 11.

10. For Parts A and B of this test both HYDRO-1D and FLUCOMP performed unsatisfactory, producing a varying water surface profile along the length of the channel. HYDRO-1D produced identical maximum water levels at all selected locations for Parts C and D.

11. In general, FLUCOMP produced the lowest water surface profile for the majority of river cross-sections.

12. The water level results produced by MIKE 11 and ISIS for Parts A, C and D were reasonably close for most of the time and for the majority of the river reach.

13. This test confirmed that it is always preferable to use measured cross-sections for all locations for any model study. It is useful to incorporate a cross-sectional interpolation facility into any model, although any user should always be aware of the corresponding limitations.

14. Modellers should be aware that the different software packages produced significantly different water surface profiles. Model results must therefore be calibrated against real data if they are to be regarded with any confidence. The use of uncalibrated models is not recommended and results of such models should be treated with extreme caution.

2.10 TEST 10 - Circular Culvert

2.10.1 Aim of Test

The test is designed to show that each of the software packages is able to model unsteady flows in a circular culverts of both mild and steep slopes, giving rise to subcritical and supercritical flows respectively, and flowing from non-pressurised to pressurised states.

2.10.2 Introduction

Test 10 is designed to test the ability of each of the software packages to model circular culverts under unsteady flow conditions. This limits the undertaking of the test to FLOODTIDE, MIKE 11, HYDRO-1D and ISIS. It should be noted that FLUCOMP, which can cope with unsteady flow conditions, does not have the ability to model culverts unless the bridge routine is used as an approximation. For this test this was considered inappropriate. The test also investigates the ability of the software packages to cope with culverts that have both steep and mild slopes, giving rise to subcritical and supercritical flows respectively. The unsteady flow conditions are designed to simulate a transition from open channel flow to pressurised flow.

The benchmarking data set is in ISIS format, which is generally not suited to the other software packages, hence where necessary the benchmarking data set has been adjusted so as to allow the test to be undertaken. Figure 10.1 schematically illustrates the test set up for both Parts A and B, as given in the benchmarking data set. For Part A the mild bed slope was 0.0025, and for Part B the steep bed slope was 0.01.

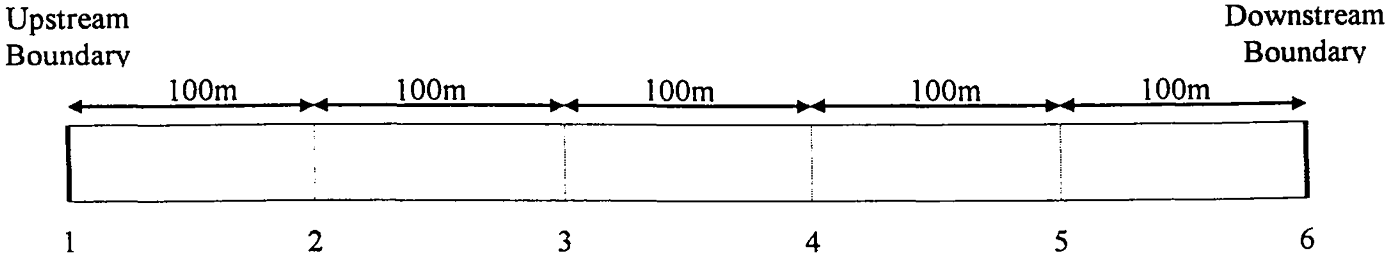


FIGURE 10.1 : *TEST 10 Parts A and B -Illustration of test set up*

The test employs six culvert units, five of length 100m and one of zero length (five culvert system). These culvert units are connected in series without using any inter-connecting cross-sections. The boundary conditions are also connected directly to the culvert units at the ends of the system, hence no cross-sections are defined in the original data set. The upstream boundary had constant inflows of 0.75 and 1.5 cumecs for Parts A and B respectively, whereas the downstream boundary had an initial water level of 0.5m which rose linearly to 4.2m for Part A and 9.7m for Part B during the simulation time of 82 minutes. Each culvert was defined with a diameter of 1.5m, a Colebrook-White Ks value of 0.003m, an invert level as appropriate, and the distance to the next section as appropriate.

Under fully submerged conditions, the theoretical velocities in the culvert may be obtained as 0.4244 m/s and 0.8488 m/s for Parts A and B respectively, based upon the above mentioned flow and geometry conditions. However, there are no known water levels to allow the results to be compared with.

2.10.3 Results

The model results of water levels at the inlet, velocities and Froude values at both the inlet and outlet for all the software packages tested are presented and compared in Appendix H, Graphs 1 to 3 for Part A and 9 to 11 for Part B. Comparisons of using open and closed sections for MIKE 11 are shown in Appendix H, Graphs 4 and 5 for Part A and Graphs 12 and 13 for Part B. Furthermore, Comparisons of modelling single and five culvert systems for ISIS are illustrated in Appendix H, Graphs 6 to 8 for Part A and 14 to 16 for Part B. Only MIKE 11 is able to produce velocity values at mid-culvert.

FLOODTIDE

The version of FLOODTIDE provided for the benchmarking study is designed to model hydraulically short culverts and is described in detail in the user manual. The set up of the test required a single culvert of length 500m to be defined since open channel cross-sections would have been required to connect the culverts in series. To connect the boundary conditions to the system two cross-sections of similar profile to the lower half of the culverts were defined at both the upstream and downstream end of the system. This was necessary since boundary conditions cannot be directly connected to a culvert unit in FLOODTIDE.

Since FLOODTIDE is limited to subcritical flow conditions, Part B of the test was not attempted. However, Part A was undertaken but produced a result with zero flow and a 24.9m water level at the upstream section of the culvert throughout the simulation period. Investigations were made with various time steps to see if feasible results could be obtained. However, this was not successful. The software vendors were approached with the problems encountered with FLOODTIDE, unfortunately they confirmed that the culvert unit gave inaccurate results and they could not offer any solution. Since the culvert unit in FLOODTIDE was not performing satisfactorily no results have been presented in this report.

MIKE 11

MIKE 11 is able to cope with culvert flow under many different flow conditions, these are illustrated and discussed in detail in the MIKE 11 user manual and are listed below:

1. Zero flow
2. Inflow critical
3. Inflow partially full and outflow critical
4. Inflow submerged and outflow critical
5. Orifice flow
6. Full culvert flow with free flow
7. Inflow and outflow partially full
8. Inflow submerged and outflow partially full
9. Inflow partially full and outflow submerged
10. Fully submerged

Appendix A of the user manual gives a detailed scientific background of the above flow conditions and also describes how the Q-h relationship tables for the culvert are calculated. The specifications required to define a culvert are given in menu A.5.C of the GUI and are listed below in Table 10.1, along with the values used in Parts A and B of the test.

TEST 10 – Circular Culvert

Specification	Value
Culvert Length (m)	500
Upstream invert level for Part A/ B (m)	1.25/5.0
Downstream invert level (m)	0
Number of parallel culverts	1
Friction coefficient (Manning's n)	0.015
Valve regulation	0
Closed/Open section flag (closed = 0, open = 1)	0
Entrance loss coefficient (typically 0.5)	0.5
Exit loss coefficient (typically 1.0)	1.0
Bend loss coefficient (straight culvert 0.0)	0.0
Critical flow correction coefficient (typically 1.0)	1.0

TABLE 10.1 : *TEST 10 - MIKE 11: Culvert specifications*

The test could not be set up as required in the benchmarking data set as MIKE 11 requires boundaries to be attached to cross-sections and for culverts to be defined within a channel. To approximate the test a channel of length 500m was defined with cross-sections defined with a similar profile to the lower half of the culvert with extended vertical side walls (open cross-section), and then also with a profile similar to the culvert but with a narrow vertical slot (closed cross-section). Figure 10.2 illustrates the two types of cross-sections deployed.

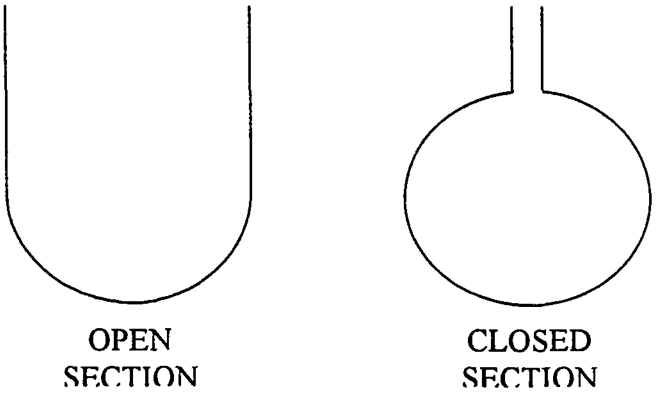


FIGURE 10.2 : *TEST 10 - MIKE 11: Profile of cross-sections used.*

To define the location of a culvert(s) within a channel the chainage of the middle of a culvert needs to be defined in menu A.5.C. Although MIKE 11 can cope with several culverts within a reach, attempts to define adjacent culverts produced warning messages when attempting to calculate the Q-h tables. Hence, a single culvert of length 500m was defined centrally between the upstream and downstream cross-sections. It is important to note that the culvert requires higher invert levels than the adjacent upstream and downstream cross-sections, hence both the upstream and downstream culvert inverts were raised by 0.001m. Loss coefficient values as suggested by the user manual were employed and a Manning's *n* value of 0.015, equivalent to a Colebrook Ks value of 0.003m, was used for both the culvert and cross-sections.

It should be noted for a closed cross-section that the 'hydraulic radius using total area' formulation is the only option available for the radius formulation in menu A.6.5.R, whereas for an open section both the 'resistance radius' and 'hydraulic radius using effective area' are also available. For continuity, only the hydraulic radius option was used.

The undertaking of unsteady calculations using MIKE 11 was a straightforward procedure. A steady state calculation was required before a unsteady/hydrodynamic simulation could be made. For both Parts A and B a time step of 0.1min was selected for the unsteady calculations, with results being saved every minute. The GUI of MIKE 11 allowed simple output and viewing of the results, which assisted in checking and analysing of the results.

The results from Part A using a closed section are illustrated in Appendix H, Graphs 1 to 3. It can be seen from Graph 1 that the water profile at the upstream section remains constant for the first 28 minutes of the simulation and then begins to rise in line with the downstream water elevation. There are no sudden jumps in the water level when the downstream boundary causes the inflow to become fully submerged and also when the culvert becomes completely full. Compared to the results from ISIS and HYDRO-1D the upstream boundary water level is noticeably different at the beginning of the simulation. It can be seen from Graph 2 that the velocity profiles for both the upstream and downstream boundaries have two distinct regions. The velocity at the downstream boundary gradually reduces as the water level rises and once the inlet becomes fully submerged the velocity remains almost uniform. However it can be seen that the upstream velocity is constant until the point when the upstream water level begins to rise, resulting in the velocity gradually reducing over the remainder of the simulation. From Graph 2 it can be seen that when the culvert becomes full the velocities at the boundaries and mid-culvert are almost identical to the theoretical full culvert flow velocity.

The values calculated for the Froude number throughout the simulation are illustrated in Graph 3 and clearly show that the flows are subcritical throughout, though again these values are at the boundary and not within the culvert. The variation in the Froude number follows similar profiles to the velocity profiles.

The results from undertaking Part A of the test with an open section are compared to those for a closed section in Appendix H, Graphs 4 and 5. It can be seen from Graph 4 that the water levels are very similar throughout the simulation. However, it can be seen from Graph 5 that the velocity values are considerably different. Unlike the results from the closed section the velocities at the upstream and downstream boundaries are not equal once the culvert becomes completely full, this can be attributed to the boundary cross-sections not being within the culvert.

The results from Part B using a closed section are illustrated in Graphs 9, 10 and 11. It can been seen in Graph 9 that the upstream water level follows a similar profile to that for Part A. However, due to the steep slope of the culvert, the upstream water level now begins to rise in line with the downstream boundary after 46 minutes of the simulation time. Graphs 10 and 11 for the velocity and the Froude number respectively follow a similar trend as for Part A, however, the full culvert velocity is slightly lower than the theoretical value.

The results from undertaking Part B of the test with an open section are compared to those from a closed section in Appendix H, Graphs 12 and 13. It can be seen from Graph 12 that the water level from the open section test is initially higher than that for the closed section. However, once the water level begins to rise at the upstream section the water level is slightly lower than that for the closed section. It can be seen from Graph 13 that the velocity values are considerably different when using the open section. Unlike the results from the closed section the velocities at the upstream and downstream boundaries are considerably lower than the theoretical full culvert velocity which can be attributed to the boundary cross-sections not being within the culvert.

HYDRO-1D

HYDRO-1D has the ability to model culverts between two given cross-sections, hence as for MIKE 11 the test had to be modified from the benchmarking data set. The cross-sections are also required so that boundary conditions can be connected to the system. The cross-sections used in the test were of a similar profile to the lower half of the culvert with extended vertical sided walls, since closed sections can not be defined within the GUI of HYDRO-1D (closed cross-sections can be defined externally as described for the bridge test). The setting up of the test configuration required a single culvert of length 500m, since open cross-sections would have been required to join five 100m culvert sections. To define the single culvert a circular conduit was defined between the upstream and downstream boundary cross-sections with the specifications as given in Table 10.2. It should be noted that no entrance or exit coefficients could be defined for the culvert.

Although HYDRO-1D has the ability to model supercritical flow conditions in open channels, it is not designed to model such flow conditions through a culvert. However, both Parts A and B of the test have been undertaken.

Specification	Value
Upstream invert level (m)	1.25/5.0
Downstream invert level (m)	0
Radius of Conduit (m)	0.75
Slot width (m)	0.0001
Manning's n	0.015
Number of conduits in parallel	1

TABLE 10.2 : *TEST 10 – HYDRO-1D:- Culvert specifications*

While seeking assistance from the vendors for the running of an unsteady test it was suggested for the culvert test that 45 levels be defined for the 'hydpar' file in the run menu. This was recommended so as to reduce the effects of a known bug in the model relating to the hydraulic characteristics above the pipe/culvert crown.

Since the test was an unsteady simulation a suitable time step had to be chosen for the simulation. HYDRO-1D is not a program that allows simple execution of unsteady runs as the user must be aware of the way in which HYDRO-1D undertakes the calculation. Selection of the time step is restricted to whole or fractions of an hour, with the smallest time step possible being 0.01 hr (36 seconds) and that was the value selected for this test. Initially a steady calculation is undertaken at time zero hour using the corresponding boundary conditions, the result of which is assigned time step one in the results file. For the unsteady calculation, results from the first time step are read in and then over written with time step two and any subsequent time steps. Hence, if the results from the first time step are needed then the steady results file has to be saved with a different name. The duration of the simulation is governed by the total number of time steps that the user selects in the run menu and is limited to 99999. Hence the longest simulation with a time step of 0.01 hour is 41.66 days.

The results from Part A are illustrated in Appendix H. It can be seen from Graph 1 for Part A, that the water profile at the upstream boundary is lower than that calculated by MIKE 11, though similar to that calculated for ISIS. It can also be seen that the water level initially reduces from the calculated initial water level before gradually rising over the first 28 minutes of the simulation. Once the downstream water level is high enough to influence the upstream water level, the upstream level then begins to rise in parallel with the downstream water elevation. There are no sudden jumps in the water level when the downstream boundary causes the inflow to become fully submerged and also when the culvert becomes completely full.

The calculations undertaken with Part B for supercritical flow conditions produced converged results, the results of which are illustrated in Appendix H, Graph 9. It can been seen from Graph 9 that the upstream water level is slightly lower than that calculated by MIKE 11 and ISIS for the first 46 minutes of the simulation. However, once the downstream water level is high enough, then both the upstream and downstream water levels rise in parallel with the upstream values similar to those calculated using MIKE 11. There are no sudden jumps in the water level when the downstream boundary causes the inflow to become fully submerged and also when the culvert becomes completely full.

Unfortunately the version of HYDRO-1D provided for this study does not provide output of the velocity value or Froude number, hence, these values are unknown for both Parts A and B.

ISIS

Since the test data was designed for ISIS the setting up and undertaking of the test was relatively straightforward. However, problems did arise in selecting a suitable time step for the simulations. In addition to undertaking the test as stipulated in the benchmarking data set, the test was also undertaken using a single culvert of length 500m so as to allow some comparison with the other packages tested.

The undertaking of the test for Parts A and B with a single culvert required the use of a 20 second time step or greater as the use of a 10 second time step resulted in the calculations failing due to diverging results. It should be noted that when using a time step of 10 seconds, the calculation tolerance was exceeded on occasions during the simulation period. However, increasing the time step reduced such an occurrence. For the original test configuration of six culvert units the time step could be reduced to 10 s, with values smaller than this resulting in calculation failure for both Parts A and B.

The GUI of ISIS allowed a useful inspection of the longitudinal water surface profile results at each time step and also enabled selective output of calculated values. However, it should be noted that for this test the tabulated output was not in a structure that is suitable for data analysis if imported to a spreadsheet.

The conduit unit that is used to model the culvert is based on the St.Venant equations, and uses an infinitesimally thin slot for the case of pressurised flow. The water level calculated by the program is the piezometric level at each unit label, and hence the results at the boundaries are also given as piezometric levels.

*For clarity the word **head** is used to describe the **piezometric level** when discussing the results from ISIS and water level for the results from HYDRO-1D and MIKE 11.*

The results from a single culvert test set up for Part A are illustrated in Appendix H, Graphs 1, 2 and 3. It can be seen from Graph 1 for Part A that the head at the upstream boundary is lower than the water level calculated by MIKE 11 but almost identical to the water level calculated by HYDRO-1D. It can also be seen that the head initially reduces from the initial backwater result before gradually rising over the next 32 minutes of simulation. Once the downstream head is high enough, then both the downstream and upstream heads rise at a similar rate. However, unlike the results from MIKE 11 and HYDRO-1D there is a slight flecture before the culvert becomes full. When the upstream opening of the culvert becomes fully submerged there is a sudden instability in the calculated upstream head. However, this does not occur when the original test configuration of six culvert units is used as illustrated in Graph 6. It can be seen from Graph 2 that the velocity profiles for the upstream boundary have three distinct regions. After the velocity at the upstream boundary has reached a peak, the velocity reduces steeply until the downstream outlet becomes fully submerged. As the downstream water level rises further, the velocity continues to reduce though now gently until the culvert becomes completely full. Under full flow conditions both the upstream and downstream velocity are constant and equal to the theoretical full flow velocity of 0.4244m/s.

The calculated Froude number values throughout the simulation are illustrated in Graph 3 and clearly show that the flows are subcritical, with the exception of the values calculated during the first few minutes of the simulation. The Froude values follow similar profiles to the velocity profiles. The Froude number becomes zero when the culvert is full, since the top width (B) in the following equation for the calculation of the Froude number becomes zero:

$$Fr^2 = \frac{Q^2 B}{gA^3}$$

The results for a single culvert from Part B are illustrated in Graphs 9 to 11. It can been seen in Graphs 1 and 9 that the upstream head for Part B follows a similar profile to that for Part A. However, due to the steep slope of the culvert, the upstream head now gradually rises over the first 46 minutes of simulation. Once the upstream head begins to rise in line with the downstream water level, which coincides with the culvert approaching full flow conditions, then instabilities are apparent in the solution. It can be seen from Graphs 6 and 14 that significant instabilities in the upstream head are reduced for Part B, though not eliminated but extended, when the original system of six culvert units is adopted for the test. Once the culvert becomes full and the head stabilises, then the upstream head continues to rise smoothly in parallel with the downstream head. The velocity and Froude number profiles for Part B are shown in Graphs 15 and 16 respectively, and follow similar profiles to those for Part A.

The effect of splitting the culvert into five lengths of 100m, instead of one length of 500m, is presented in Graphs 6 to 8 for Part A and Graphs 14 to 16 for Part B. When the culvert is split into sections it can be seen from Graph 6, for Part A, that the head at the upstream boundary remains almost constant for the early part of the simulation and then rises without fluctuation in parallel with the downstream boundary head. Graph 7 also illustrates this by having a near constant upstream boundary velocity for the early stages of the simulation. There is then a rapid decrease in the velocity to the point when the downstream culvert opening becomes full, followed by a gradual further decrease in velocity until the culvert becomes completely full. It should also be noted that the initial sudden increase in velocity at the upstream boundary is no longer produced when the culvert is split into sections. For Part B Graphs 14, 15 and 16 show that similar results are achieved for the steep slope when the culvert is split into several sections. It should, however, be noted that for the early part of the simulation the velocity at the upstream boundary is not as uniform as is the case for the mild slope situation. In addition to this, the velocities from Parts A and B are not stable during the simulation as shown in Graphs 6 and 15.

2.10.4 Summary

1. FLUCOMP cannot model culvert systems and FLOODTIDE failed to produce realistic results.

2. Tests were undertaken with MIKE 11, ISIS and HYDRO-1D for mild and steep longitudinal slope conditions.

3. Both single unit and multi unit culvert systems were tested for ISIS, with the multi unit culvert producing slightly better results. Only a single unit culvert (of 500m) could be used for MIKE 11 and HYDRO-1D and this was judged by the authors to be a restriction in these models.

4. Unlike MIKE 11 and HYDRO-1D, no explicit cross-sections were required to connect to the culvert units in ISIS.

5. Little difference was found in the predicted water levels for MIKE 11 when open and closed cross-sections were used. However, velocity results indicated that closed cross-sections should always be used.

6. No velocity and Froude number output were available for HYDRO-1D.

7. Calculated upstream head results for ISIS exhibited instabilities when the culvert just became full, although the results were improved by the use of a multi-unit culvert.

8. ISIS calculated supercritical flow conditions during the early stages of Part A i.e. for a mild slope.

9. The invert level of the culvert in MIKE 11 had to be above the minimum level of the cross-section immediately downstream of the culvert. The practicality of this situation in the field was thought to be restrictive, as the invert level of a culvert may be below that of the measured cross-section immediately adjacent to the structure.

2.11 TEST 11 - Bridges

2.11.1 Aim of Test

Test 11 assesses the ability of each of the software packages to model and adequately take account of the effects of flow through bridges by the US Department of Commerce (USBPR) bridge modelling approach, and the HR Wallingford Arch Bridge Method. The test does not model a real case hence results are to be compared with hand calculations.

2.11.2 Introduction

Test 11 assesses the ability of each of the software packages to model and adequately take account of the effects of flow through bridges. The data set provides information for the testing of bridges by two different approaches - (i) the USBPR bridge modelling approach, and (ii) the HR Wallingford Arch Bridge Method, both of which are described in detail below. It should be noted that reference data are not available for this test, hence a true benchmarking of the software packages has not been possible. However, bridge afflux results are compared to theoretical calculations made by hand.

USBPR Bridge Modelling Approach

Figure 11.1 illustrates the bridge dimensions as used for the USBPR method of computing bridge afflux. Table 11.1 presents the data provided by the benchmarking data set which defines the bridge dimensions and other relevant data for the USBPR calculation.

For the USBPR bridge the theoretical total afflux across the bridge is the sum of the inlet and outlet losses produced by each bridge opening and is calculated as follows(20):

Inlet losses: $$\Delta E = k\frac{U_2^2}{2g} \times \text{number of openings}$$

Outlet losses: $$\Delta E = \varepsilon\frac{(U_2 - U_3)^2}{2g} \times \text{number of openings}$$

where: ΔE = Energy loss (m)
k = inlet loss coefficient (taken as 0.10)
ε = outlet loss coefficient (taken as 0.82)
U_2 = velocity in middle of bridge (m/s)
U_3 = velocity downstream of bridge (m/s)

Over-topping of the bridge has not been tested, hence the level of the bridge deck has not been specified in the data set. The bridge piers are defined as vertical and the effects of sloping abutments have also not been tested.

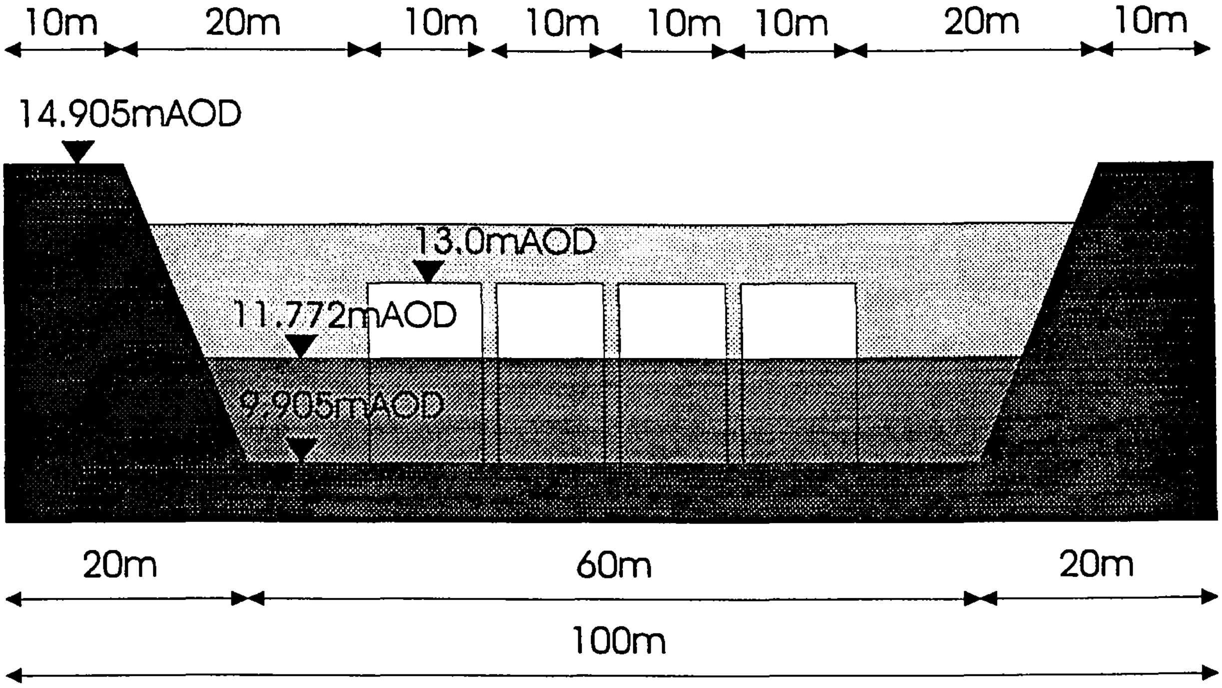


FIGURE 11.1 : *TEST 11 Part A - USBPR Bridge Dimensions*

Decription	Value
Friction Type	Manning's
Scaling factor for calibration of afflux	1
Skew angle (deg)	0
Distance between u/s and d/s bridge faces (m)	5
Abutments type	3
Total width of piers normal to flow direction (m)	3
Width of single pier (m)	1
Pier shape	Cylinder
Abutment alignment with flow direction	Aligned
Number of data points defining channel	30
Data points defining channel (not presented)	-
Number of bridge arches	1
Offset at beginning of arch (m)	30
Offset at end of arch (m)	70
Spring height of arch (mOAD)	13
Soffit of arch (mOAD)	13
Number of culverts in arch	0

TABLE 11.1 : *TEST 11 Part A - USPBR bridge data*

HR Wallingford Arch Bridge Method

The test is designed to asses the ability of each of the software packages to model bridge afflux according to the Hydraulics Research publication 'Afflux at Arch Bridges', December 1988. Since this test is specifically designed for ISIS none of the other software packages have been able to model this bridge using only the test data set provided. However, it should be noted that several packages could undertake this part of the test by calculating to the bridge geometry externally knowing that the arch shape is parabolic(21). Results from each of the software packages have been compared with hand calculations for the bridge afflux according to the HR Wallingford method(22).

Figure 11.2 illustrates the arched bridge dimensions, as used for Test 11 Part B, and Table 11.2 presents the data provided by the benchmarking data set to define the bridge dimensions and other relevant data for the Arched Bridge calculation.

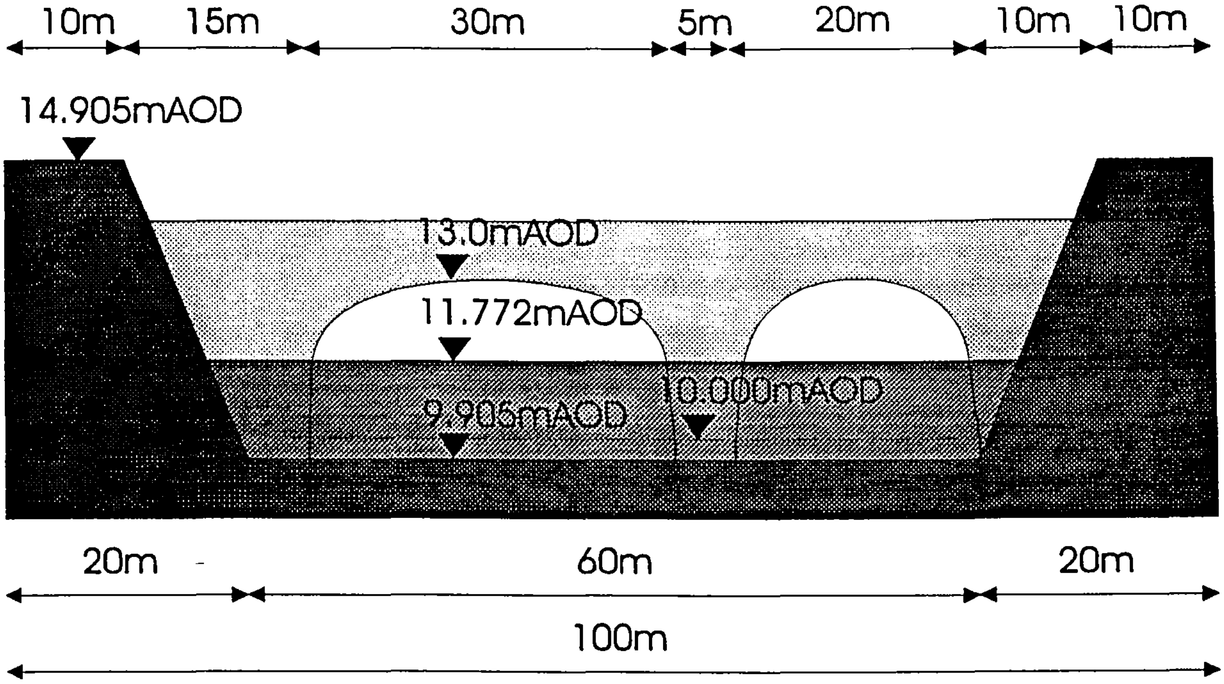


FIGURE 11.2 : *TEST 11 Part B - Arched bridge dimensions*

Decription	Value
Friction Type	Mannings
Scaling factor for calibration of afflux	1
Skew angle (deg)	0
Distance between u/s and d/s bridge faces (m)	5
Number of data points defining channel	35
Data points defining channel (not presented)	-
Number of bridge arches	2
Offset at beginning of 1st arch	25
Offset at end of 1st arch	55
Spring height of 1st arch	10
Soffit of 1st arch	13
Offset at beginning of 2nd arch	60
Offset at end of 2nd arch	80
Spring height of 2nd arch	10
Soffit of 2nd arch	13
Number of culverts in bridge	0

TABLE 11.2 : *TEST 11 Part B - Arch bridge data*

Figure 11.3 illustrates the plan location of the eight cross sections used to define the channel in Test 11. The only changes required for the two bridge modelling approaches was the actual bridge data. Each cross section had a constant Manning's 'n' value of 0.025 and had a cross sectional profile as illustrated in Figures 11.1 and 11.2. The bed slope was 0.0002 between cross sections 1 and 4, and 6 and 9, and 0.004 between cross sections 4 and 5. There was no bed slope between sections 5 and 6. Both Parts A and B used the same constant boundary conditions of 100 cumecs inflow at the upstream boundary and a 11.712m water level at the downstream boundary.

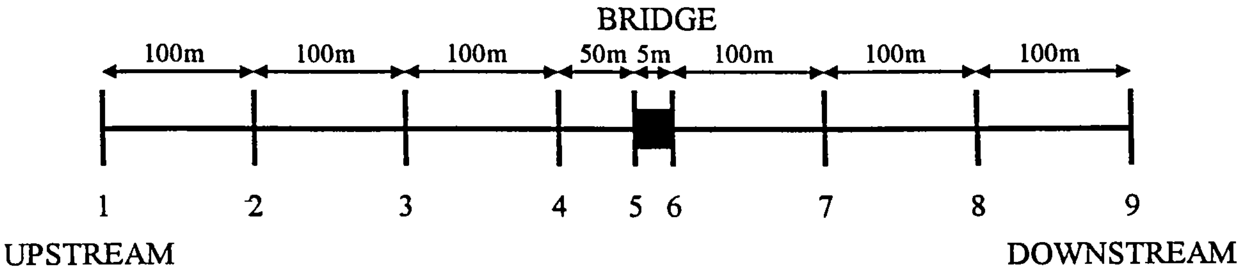


FIGURE 11.3 : *TEST 11 Parts A & B - Plan location of cross sections*

According to the bridge configurations for Part A and B the following relationships between the flow area under the bridge (A) and the water elevation (Z) may be obtained

For Part A: $A = 37 * (Z - 9.905)$ $\quad when{:}9.905m \le Z \le 13.0m$

For Part B:

$$A = \begin{cases} 50 \times (Z - 9.905) & when{:}9.905m \le Z \le 10.0m \\ 104.75 - \dfrac{100}{3\sqrt{3}}\sqrt{(13 - Z)^3} & when{:}10.0m \le Z \le 13.0m \end{cases}$$

The flow area under the bridge is compared for Parts A and B in Figure 11.4, from which it can be seen that a larger flow areas exists under the bridge for Part B than for Part A for the same water elevation for this test.

Error! Not a valid link.

FIGURE 11.4 : *TEST 11 - Comparison of flow area under bridge for Parts A and B.*

2.11.3 Results

The results obtained for Test 11 from each of the software packages tested for both Parts A and B are summarised below in Tables 11.3 and 11.4 respectively. It should be noted that neither CHANNEL nor BAKWATER have the ability to model structures and have thus been omitted from the tables.

	Water Level (m)			
SOFTWARE	Theoretical[1]	Calculated		
	Upstream face of bridge	Upstream face of bridge	Downstream face of bridge	Change ΔH (m) @ Bridge
FLOODTIDE	11.869	11.953	11.772	0.181
HEC-RAS[2]	11.871	11.811	11.775	0.036
HEC-2	11.867	11.780	11.770	0.010
MIKE 11	11.872	11.982	11.776	0.206
HYDRO 1D	11.869	11.981	11.772	0.209
FLUCOMP	11.871	11.893	11.775	0.118
ISIS	11.869	11.868	11.772	0.096
LD01	11.871	11.799	11.775	0.024
GLC-BACKWATER	11.876	11.870	11.780	0.090

[1]using calculated downstream water level as calculated by individual packages [2]Energy method of computation.

TABLE 11.3 : *TEST 11 Part A - Summary of USBPR bridge results*

	Water Level (m)			
SOFTWARE	Theoretical[1]	Calculated		
	Upstream face of bridge	Upstream face of bridge	Downstream face of bridge	Change ΔH (m) @ Bridge
FLOODTIDE	11.827	11.790	11.772	0.018
HEC-RAS[2]	11.830	11.799	11.775	0.024
HEC-2	11.825	11.790	11.770	0.020
MIKE 11	11.831	11.940	11.776	0.164
HYDRO 1D	11.827	11.918	11.772	0.146
FLUCOMP	Not Possible			
ISIS	11.827	12.335	11.772	0.563
LD01	11.830	11.790	11.775	0.015
GLC-BACKWATER	11.835	11.800	11.780	0.020

[1]using calculated downstream water level as calculated by individual packages [2]Energy method of computation.

TABLE 11.4 : *TEST 11 Part B - Summary of Arch Bridge Results*

The theoretical water levels upstream of the bridge in Tables 11.3 and 11.4 have been calculated using the water levels calculated by each of the software packages at the downstream face of the bridge.

Different water level changes across the bridge were produced by different software packages. They varied from as little as 0.03m to 0.04m for both Parts A and B to as much as 0.21m and 0.56m for Parts A and B respectively. It is worth noting from Tables 11.3 and 11.4 that water levels downstream of the bridge (cross-section six) by various software packages were between 11.77m and 11.78m and will not be affected by the use of different bridge configurations. One would therefore expect a greater water level change across the bridge for Part A than for Part B, due to a smaller flow area under the bridge and more bridge openings for Part A than for Part B. It can be seen from these two tables that FLOODTIDE, HEC-RAS, MIKE 11, HYDRO-1D, GLC-BACKWATER and LD01 produced greater water level changes for Part A than for Part B. However, ISIS and HEC-2 produced unexpected smaller water level changes across the bridge for Part A than for Part B.

For Part A, the USBPR modelling approach, it can be seen from Table 11.3 that the upstream water level results from both ISIS and GLC-BACKWATER are almost identical to the theoretical solution. However, the remaining software packages all calculate the water elevation to within 0.10m of the theoretical solution. For Part B, the arch bridge method, all of the software packages except MIKE 11 and ISIS calculate the upstream water level to within 0.05m of the theoretical solution. MIKE 11 calculates the upstream water level 0.109m higher than the theoretical solution and ISIS calculates a water elevation that is 0.508m higher than the theoretical solution.

A description of the undertaking of the test with each of the software packages is given next highlighting the relevance of the results presented in Tables 11.3 and 11.4.

FLOODTIDE

FLOODTIDE has the ability to model bridges using the USBPR and the HR Wallingford Arch Bridge methods, although the data format is somewhat different to that provided in the benchmarking data set. To model a bridge the user is required to define a cross section both upstream and downstream of the bridge module. However, the cross section in the middle of the structure as provided by the data set is not required. The data required for these two methods are very similar and are described below in Table 11.5 along with the values used for both Parts A and B of the test. Figure 11.5 illustrates the five pier types available in FLOODTIDE in the USBPR module. The pier types are also defined and illustrated in Figure 6.9.1 of the user manual.

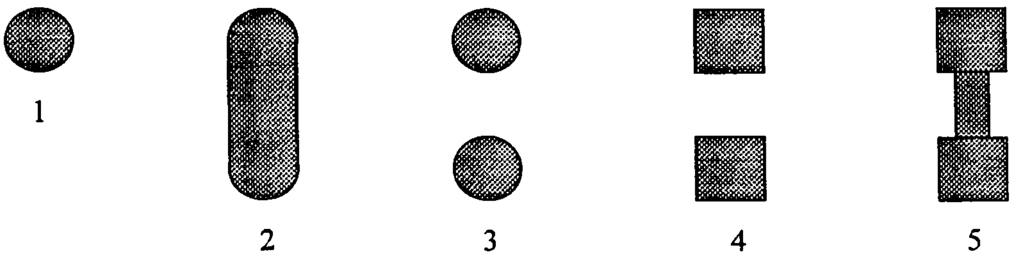


FIGURE 11.5 : *TEST 11 Part A - FLOODTIDE : Pier Types*

For Part A the area normal to the flow direction was simple to calculate since each of the three piers was 1m wide for the complete pier columns. However, for Part B the total obstruction area produced by the bridge required calculation at various levels in order to represent accurately the consequence of the parabolic shape of the bridge arches.

The results obtained from FLOODTIDE for Parts A and B of Test 11 are presented in Tables 11.3 and 11.4 respectively.

Variable	Decription	Value	
		USBPR	ARCH
ptype	Pier type descriptor	1	1
Hbed	Bed level at upstream cross section (m)	9.905	9.905
Width	Constricted width of channel at centre line of bridge	40	55
Cfact	Correction factor	1.0	1.0
soffit	Soffit level	13	n/a
hdeck	Deck level	15	15
ldeck	Length of bridge deck	5	5
cdeck	Discharge coefficient for overtopping - *not essential for this test since the flow level is not high enough for weir flow.*	1.0	1.0
nlevel	Number of elevations defining pier area/obstruction	as appropriate	
level	Elevation for projected pier area in the flow direction	as appropriate	
area	Total area normal to the flow direction below 'level'	as appropriate	

TABLE 11.5 : *TEST 11 Parts A and B - FLOODTIDE : Bridge data*

HEC-RAS

The modelling of bridges by HEC-RAS is both comprehensive and relatively simple. A cross section is required at both the upstream and downstream faces of the bridge. Bridge data were input through the bridge and culvert input menu where the bridge dimensions and features can be defined through the following sub-menus:

Deck/Roadway - the cross sectional profile of the upstream and downstream faces of the bridge are defined separately by specifying the high and low cords across the bridge width. This allows both the invert level of the bridge to be defined as well as the roadway. It also allows irregular bridge profiles to be defined.

Piers - as many piers as required can be defined within the bridge structure. The centre line of both the upstream and downstream faces is defined, thus allowing skew angles to be taken into account. For each pier the width at as many levels as required is specified so as to define accurately any variation in the pier width over height.

Sloping Abutments - although not required for this test the shape of abutments can also be taken into account. This can be done by defining the abutment geometry through the x-y co-ordinates.

Bridge modelling approach - there are several options that are available to the user for the computation which include (i) Energy, (ii) Momentum, and (iii) Yarnell. For this test both the energy and momentum methods were tested.

Culvert - although not required for this test any culvert data are also entered through the culvert option in the bridge data menu.

Multiple openings - although not required for this test HEC-RAS has the ability to model multiple bridge and/or culvert openings at any individual river crossing. This is ideally suited to complex crossings where openings are at different levels and are only taken into account for certain flow conditions.

A useful feature of HEC-RAS is the graphical image of the bridge that is provided in the bridge and culvert window menu. This is useful for checking that the bridge has been defined correctly. Another tool in HEC-RAS, which is also useful, is the ability to check data in an x-y-z perspective plot.

The set-up of both Parts A and B was such that the data entered approximated to the geometry and characteristics of the two types of bridges. For Part A this was not a problem. However, for Part B the parabolic profile was calculated according to the information available. The results for Part A are presented in Table 11.3. They are from a calculation using the energy computation method. However, it should be noted that undertaking the computation by the momentum approach produces a slightly lower water level of 11.87m at cross section 1 and a much reduced water level difference of 0.011m across the bridge structure. The results for Part B using both the energy and momentum approaches were the same and are presented in Table 11.4. It should be noted that the number of points used to define the parabolic shape of the bridge openings is limited to the array size that HEC-RAS can cope with. It was found that 66 points defining the bridge were too many, however, 42 points were acceptable.

HEC-2

The modelling of bridges using HEC-2 is not a simple task. The user must be accustomed to the data flagging required to implement the bridge options available in order to undertake an accurate simulation. Essentially there are two bridge options which can be employed for the modelling of bridges: the simple method and the special bridge method. Since the data set defines a bridge with piers the special bridge option had to be employed for Part A of this study. However, for Part B the special bridge method is not suitable for arched bridges and hence the simple method was adopted.

The user manual tentatively describes the input of data required for both bridge methods, and considerable cross-referencing throughout the manual is required in order to obtain a complete understanding of the data requirements and structure.

For both Parts A and B contraction and expansion coefficients of 0.3 and 0.4 respectively were used as suggested by the HEC-2 user manual (Chapter 4 - Example of Input). These values were called upon as appropriate, using the 'X3' record as described in the examples given in the Hec-2 user manual.

Part A - USBPR Bridge

The special bridge routine computes losses through structures for either low flow, pressure flow, weir flow or a combination of these, with the first case being the flow situation for this test.

To set up the special bridge method a cross section has to be defined at both the upstream and downstream faces of the structure, as provided with the data set. Once the downstream river cross section has been defined in the data file the 'SB' record follows, defining some of the bridge dimensions and calculation variables. These are defined in Table 11.6 along with the values used for Test 11 Part A.

After the 'SB' record the remaining bridge data are defined after the cross section upstream of the bridge with an 'X2' record. The 'X2' record is used to employ the special bridge method in the computation and is also used to define the low cord of the bridge and the roadway deck for weir flow.

Three calculations were undertaken with various 'K' values used to define the pier shape as recommended in Chapter 3 of the user manual (Special Bridge Coefficients). The 'K' values used were : 0.95 for a twin-cylinder piers with connecting diaphragm, 1.05 for a 90° triangular nose and tail, 1.25 for a square nose and tail, and 2.5 for a Ten pile trestle bent pier, with the results being presented in Table 11.7 below. The results in Table 11.3 are for the twin cylinders with a connecting diaphragm as this represents the data set provide for the test.

Variable	Decription	Value
IA	Record identification character.	SB
XK	Pier shape coefficient 'K', for use in Yamell's energy equation for Class A flow.	Various
XKOR	Total loss coefficient, 'K', between cross sections on either side of bridge, for use in orifice flow equation - *not essential for this test since the flow level is not high enough for weir flow.*	1.6
COFQ	Coefficient of discharge 'C' for use in weir flow equation - *not essential for this test since the flow level is not high enough for weir flow.*	2.6
RDLEN	1) Indicator (0) for no flow over roadway or table of roadway elevations from BT record for determining 'L' in weir equation or 2) Average length of roadway 'L'.	0
BWC	Bottom width of bridge opening including obstruction.	40
BWP	1) Indicator (0) for no piers or 2) total width of obstruction.	3
BAREA	Net area of bridge opening below the low chord.	114.52
SS	1) Indicator (0) for vertical bridge abutments, or 2) side slope of abutments.	0
ELCHU	Elevation of the channel invert at the upstream side of the bridge.	9.905
ELCHD	Elevation of the channel invert at the downstream side of the bridge.	9.905

TABLE 11.6 : *TEST 11 Part A - HEC-2: SB record values*

It can be clearly seen from Table 11.7 that for this test the pier coefficient K has no marked effect on the small water level change (i.e. 0.01m) produced at the bridge when values of 0.95 through to 1.25 are used. However, a much-increased value of 2.5 for K produces a bigger water level difference of 0.03m at the bridge structure.

Value of K	Water Level at Cross Section (m) 1	5	6	Water level change ΔH (m) @ Bridge
0.95	11.85	11.78	11.77	0.01
1.05	11.85	11.78	11.77	0.01
1.25	11.85	11.78	11.77	0.01
2.50	11.87	11.80	11.77	0.03

TABLE 11.7 : *TEST 11 Part A - HEC-2*
Effect of pier shape coefficient K on change in water level at bridge.

Part B - Arch Bridge

The simple bridge method is suited to model arched bridges and is described in the user manual with an example. Once again a cross section is required to be defined downstream of the bridge structure, with this being the only similarity with the special bridge method. The upstream face of the bridge is defined as a cross section with a 'X1' record and the bridge openings are described through a 'BT' record. The 'BT' records define the roadway elevation and low cord elevation across the bridge width. It should be noted that the 'BT' records should coincide with the records for the previous downstream cross section, this is because the program computes the conveyance of the cross section incrementally. The downstream face of the bridge is also defined as a cross section with an 'X1' record and for this test followed an 'X2' record. The 'X2' record is employed to copy the downstream profile of the bridge openings, although another 'BT' record could be used to describe any changes in the bridge openings. It should be noted that the bridge opening was defined as parabolic and was calculated according to the information available. Once the bridge data had been completed the upstream section and any further sections were defined by the usual means required by HEC-2.

It should be noted that no other information is required for the definition of the bridge other than the shape of the bridge opening through the 'BT' record and any coefficients accounting for constriction or expansion as described previously. The results from the calculation for Test 11 Part B can be found in Table 11.4.

MIKE 11

MIKE 11 does not directly have an option to model bridges. However, the culvert options available in MIKE 11 were suggested by the vendors as an appropriate method for modelling the test conditions. There are two culvert options available in MIKE 11 : (i) culverts with user specified Q-h relationships, and (ii) culverts with Q-h relationships calculated automatically - the later option being more suitable for the available data set. As with any river model the river system was set up with the appropriate cross-sections and boundary conditions. The structure was defined at the specified river chainage and the variables defined in Table 11.8 were used for the bridge openings. For Part A the rectangular option was selected and for Part B the Irregular option. This provided a simple and efficient way of defining the bridge for both Parts A and B. The results are presented in Tables 11.3 and 11.4 for Parts A and B respectively.

Variable		Value
Invert Upstream (m)		9.905
Invert Downstream (m)		9.905
Length (m)		5.0
Manning's n		0.025
Number of culverts		4/2
Rectangular / Irregular	Depth	As Appropriate
	Width	As Appropriate
Valve regulation (0-closed, 1 - open)		0
Coefficients	Inflow	1.0
	Outflow	1.0
	Bends	1.0
	Critical Flow	1.0

TABLE 11.8 : *TEST 11 Part A and B - MIKE 11: Bridge structure/calculation variables.*

HYDRO-1D

Setting up HYDRO-1D to model a bridge is a straightforward task once the river network has been created through the GUI. Cross sections are required to be defined both upstream and downstream of the structure and then a connecting 'bridge' reach is required for each bridge opening. Each bridge opening is given a reference 'X' and the associated cross-section is given the file name Xbrd.xsn. This file can be created through the cross section editor in HYDRO-1D. However, it should be noted that the version of HYDRO-1D supplied for this study has a known bug, which is the value of 0.00m being stored as the maximum cross-section level. The solution is to edit the file outside of HYDRO-1D and change the maximum cross-section level to an appropriate value. If the cross-sections which defines the bridge opening are closed then a text editor must be used to define the Xbrd.xsn file as closed cross sections can not be defined using the GUI of HYDRO-1D. HYDRO-1D does not take account of overtopping, abutment skew angles or pier types. only the structure/calculation variables are definable as listed below in Table 11.9:

Variable	Value
Upstream velocity head coefficient	1.0
Velocity/Head coefficient	1.0
Downstream velocity head coefficient	1.0
Energy loss coefficient	1.0
Exit loss coefficient	1.0
Bridge No.	As Appropriate

TABLE 11.9 : *TEST 11 Part A - HYDRO-1D :*
Bridge structure/calculation variables.

For Part A the USBPR bridge was modelled using four connecting channels between the upstream and downstream cross-sections, using the structure/calculation values as defined above in Table 11.9. The results from this calculation are presented in Table 11.3 and show a water level difference of 0.209m across the structure. As an investigative measure the test was also undertaken using just one opening but with a combined net opening of 37m. This resulted in a change in water level of 0.118m across the structure as would be expected.

For Part B, the Arch bridge was modelled using two connecting channels between the upstream and downstream cross-sections, again using the structure/calculation values as defined above in Table 11.9. The parabolic shape of each opening was the same as that used for HEC-RAS. By using two channels to define the bridge openings the water level difference across the structure was calculated to be 0.146m. However, as an investigative measure a single cross section file was created with two arches. This resulted in a change in water level of 0.097m across the structure.

FLUCOMP

The modelling of bridges using FLUCOMP is achieved by creating a bridge data file (BDF) which contains all of the information for each bridge to be modelled. The structure of the BDF file is well documented in the user manual. However, a clear understanding of the flow data file (WDF) and a methodical approach is essential since amendments are required to 'WDF' file in order to initiate the bridge afflux calculations.

FLUCOMP uses the USBPR design method to calculate the afflux at each bridge. However, the amount of information that is to be input for each bridge is significantly more detailed than that supplied by the data set. Table 11.10 illustrates some of the integral variables required to define the bridge and the values adopted for the Test 11 Part A.

Since the set-up of a bridge is extensive and is comprehensively covered in the user manual an in-depth account of the test set-up is not given herewith. Essentially to model a bridge cross sections have to be defined both upstream and downstream of the bridge structure, although these sections do not have to be directly in contact with the face of the bridge. The chainage of the bridge is specified together with the distance from the upstream cross-section to the bridge and the bridge pier length. In the BDF file the upstream face of the bridge is defined including any flood plains, and the bridge opening is defined using the spring and soffit levels. FLUCOMP does not allow arched bridges to be defined, hence Part B of Test 11 was not possible. Table 11.10 illustrates the major values which need to be defined in the data file and the values used in Test 11 Part A. The results from Part A are illustrated in Table 11.3 and shows that the change in water level across the bridge structure is 0.118m.

	Variable	Value
Control Parameters	Number of bridges	1
	Number of velocity heads	9
	Number of downstream levels	21
	Friction law indicator	1
Integer bridge Data	Chainage of cross-section upstream of bridge.	As Appropriate
	Chainage of bridge (m)	As Appropriate
	Number of arches	4
	Abutment type	1
	Pier type	1
	Number of culverts	0
	Drowning coefficient for road flow - *not essential for this test since the flow level is not high enough for weir flow.*	0.67
Decimal bridge data	Distance of bridge from upstream cross-section (m)	0
	Skew angle for bridge (°)	0
	Skew angle of embankment (°)	0
	Skew angle of piers (°)	0
	Bankfull discharge (m^3/s)	200
	Length of bridge piers (m)	5
	Discharge coefficient for road flow - *not essential for this test since the flow level is not high enough for weir flow.*	1
	Discharge coefficient for culvert flow - *not essential for this test since the flow level is not high enough for weir flow.*	1
Bridge or Road section data	Chainage of cross section (m)	As Appropriate
	Horizontal offset (m)	As Appropriate
	Ground level (m)	As Appropriate
Arch data	Pointer to offset of left arch	As Appropriate
	Pointer to offset to right arch	As Appropriate
	Spring level (m)	9.905
	Soffit level (m)	13.00

TABLE 11.10 : *TEST 11 Part A - FLUCOMP: Bridge structure/calculation variables.*

ISIS

The data set provided for both Parts A and B were in ISIS format. The setting up of Part A was straight forward and the results are presented in Table 11.3. However, problems arose with Part B as described below.

For both Parts A and B cross-sections were defined both upstream and downstream of the bridge structure. The structure details were then input through the GUI as defined in Table 11.1 for the USBPR bridge and Table 11.2 for the arched bridge. For Part B of the test ISIS crashed each time an attempt was made to save the data. The software vendors were contacted with this problem and it was suggested that the data file be created through a text editor outside of ISIS. This method was adopted and enabled Test 11 Part B to be undertaken successfully.

CHANNEL

CHANNEL is not suited to this test as it cannot cope with structures, hence Test 11 has not been conducted for this package.

LD01

The manual for LD01 suggests that the culvert option in the program be adopted for modelling bridges. This allows bridges of rectangular profile to be modelled with up to 5 different openings each with frictional losses taken into account at the inlet and outlet. This method of modelling bridges allows arched bridges to be modelled as required by Part B of test 11, although the definition of each arch is limited.

The setting up of the 'culvert' is straightforward and is illustrated clearly in the manual, however, there is no description of the five flow types as mentioned in the manual although they are referenced. There is no suggestion of suitable discharge coefficients. Each opening is defined by up to a maximum set of ten co-ordinates, with each set defining the offset, invert level and soffit level. For Part A this was straight forward, whereas for Part B the parabolic profile of the two arches was calculated by hand and defined rather coarsely by the ten co-ordinates permitted for each arch.

The results presented for Part A in Table 11.3 are from a calculation using discharge coefficients of 0.9 for both the inlet and outlet. This gives rise to a water level change across the structure of +0.024m. However, it should be noted that when using coefficients of 1.0 for both the inlet and outlet coefficients the change in water level across the structure changes to

– 0.002 m; clearly a result which should be treated with caution. The results for Part B are presented in Table 11.4 and are from a calculation using inlet and outlet coefficients of 0.9. The water level change across the structure of 0.015m is reduced to 0.001m when the inlet and outlet coefficients are increased to a value of 1.0 for Part B.

GLC-BACKWATER

GLC-BACKWATER models both bridges and culverts in the same manner. However, if over topping of the bridge is required then open parapets at deck level also need to be specified. The bridge/culvert can be either rectangular, circular or irregular in shape and is defined in the data file after the cross-sectional data has been defined. The user manual clearly describes the data structure required to define the bridge and the openings, Table 11.11 illustrates these and the values used in Part A. Up to four openings can be defined, each with a different depth, width and invert level.

To define an irregular opening GLC-BACKWATER requires a maximum of nine co-ordinates for each opening. The co-ordinate pairs have to be defined in an anticlockwise direction and input commencing from the highest point of the opening. For Test 11 Part B the co-ordinates were the same as those used for the other packages, which also required the opening to be defined by co-ordinates. The results from Part B are presented in Table 11.4.

Variable	Value
Chainage of downstream face of structure (m)	As Appropriate
Depth of opening (m)	3.095
Coefficient for outlet losses	1.0
Coefficient for inlet losses	1.0
Manning's number for structure	0.025
Length of structure (m)	5.0
Number of openings	4
Width of opening (m)	2 @ 9.5, 2 @ 9.0
Downstream invert level (m)	9.905

TABLE 11.11 : *TEST 11 Part A - GLC-BACKWATER: Bridge structure/calculation variables.*

BAKWATER

BAKWATER is not suited to this test as it cannot cope with structures, hence Test 11 has not been conducted for this package.

2.11.4 Summary

1. Two different configurations of USBPR and Arched bridges were tested under steady flow condition.

2. CHANNEL and BAKWATER were unable to model bridge structures and FLUCOMP was unable to undertake Part B of this test.

3. Different water level changes across the bridge were produced using the various software packages. They varied from 0.03m to 0.21m for Part A and from 0.04m to 0.56m for Part B.

4. The GUI editor for the arched bridge with ISIS would not allow the bridge configuration data to be saved. However a simulation was undertaken when the data file was created in a text editor external to the ISIS GUI.

5. FLOODTIDE, HEC-RAS, MIKE 11, HYDRO-1D and LD01 produced greater water level changes for Part A than for Part B, whereas GLC-BACKWATER produced slightly different water levels in Part A and Part B. However, ISIS and HEC-2 produced unexpectedly smaller water level changes across the bridge for Part A than for Part B, this result is clearly questionable.

2.12 TEST 12 - River Calder

2.12.1 Aim of Test

The test is an addition to the study and is designed to assess the ability of the software packages to model a complex river network under unsteady flow conditions. The authors of the stage one report did not produce the data set provided for the test. WS Atkins provided the data set in MIKE 11 format

2.12.2 Introduction

The test was originally prepared to demonstrate that a development in the floodplain at Lowlands along the River Calder, West Yorkshire, would not adversely affect the conveyance of the river. However, for Test 12 the model configuration is based upon the river system before this development.

The river Calder itself forms the main channel in the modelled river network and runs from Sowerby Bridge through to Brighouse and beyond, passing under a main road and ending with a Q (h) boundary condition, as illustrated in Figure 12.1. The upstream boundary condition is an inflow Q(t) of a 1967 storm event. The Calder and Hebble Navigation canal runs alongside the river from upstream of the area known as Wimpenny, passes Ganny Lock and rejoins the river downstream of Brighouse. There are a number of locks along the canal and Whitrose Lake lies to the north of the canal between Lillands Loop and Tagcut.

During an initial survey a number of areas were identified which could potentially be affected by flooding. These are the Lowfields area and the low lying areas near Lillands Loop and to the south of the canal in the vicinity of the Whitrose Lake. In the data set provided these are represented by a network of wide channels and weirs. There are also several weirs along the river itself including Elland Weir, Brookfoot Weir, Rastrick Weir and two others downstream of Brighouse. The actual weir profiles used in the model are presented in Appendix I, Graph 1. It should be noted that Rastick Bridge was modelled using a culvert.

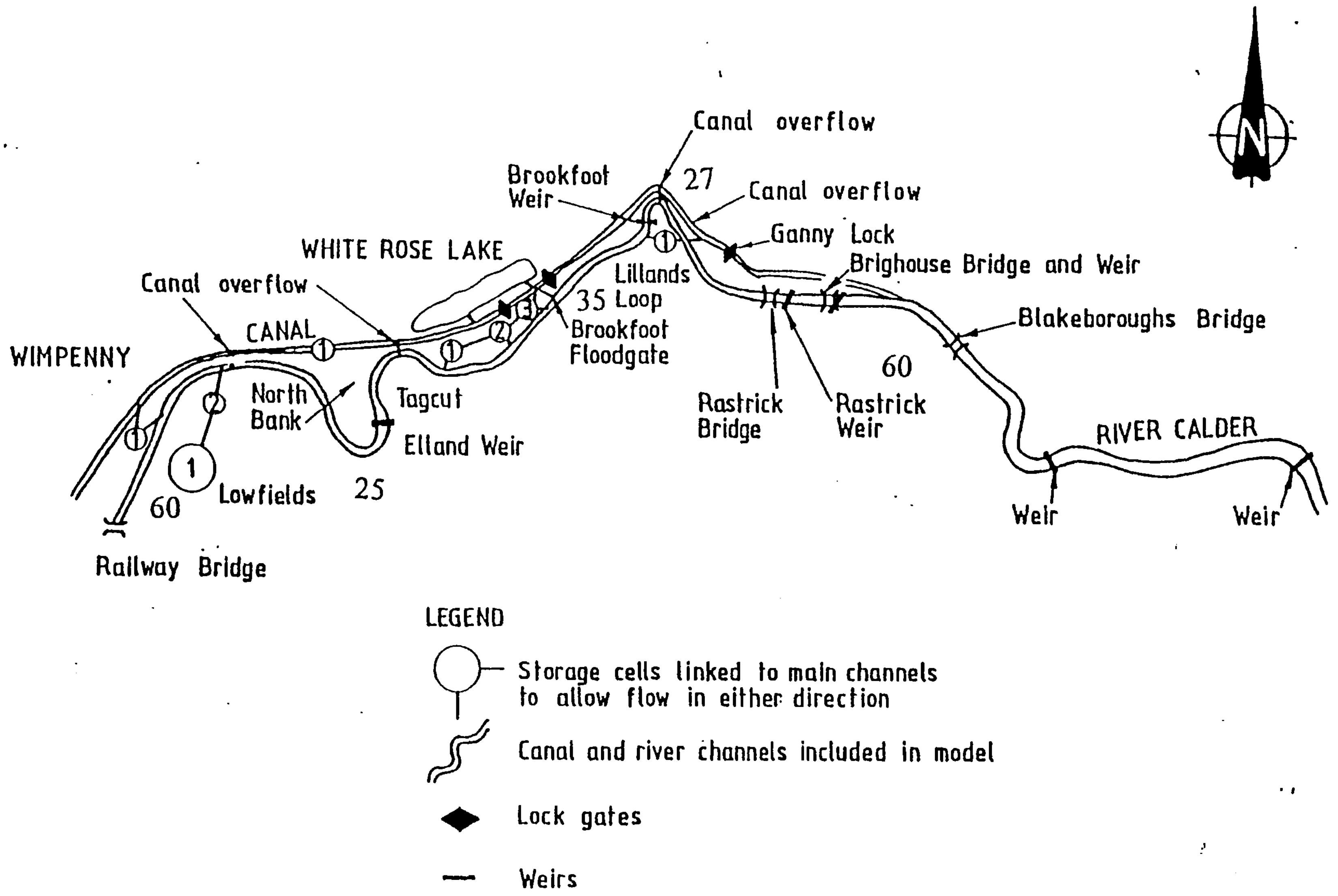


FIGURE 12.1 : *TEST 12- Illustration of river Calder network*

Calibration information for the model is available for peak water levels only at a number of locations measured by observing debris levels after the 1967 storm event.

Initial conditions for the river network were provided in the form of 'HOT START' file for MIKE 11. A base flow of 20 cumecs in river Calder and a nominal flow of 0.1 cumec in the canal were specified as the initial conditions, with an initial water level of 58.0m being specified in the Whiterose lake. Although better (or worse) calibration results may be obtained by altering the friction resistance, either globally or locally, the Manning's *n* values as originally provided for the test were used consistently for all the packages tested. The Manning's *n* being 0.033 for majority of cross-sections in the river Calder.

2.12.3 Results

The ability of each of the unsteady models to undertake Test 12 with the river network provided is summarised below in Table 12.1. A summary of the calculated maximum water elevations at selected locations is given in Table 12.2

SOFTWARE	TEST 12
FLOODTIDE	✗
MIKE 11	✓
HYDRO 1D	✓
FLUCOMP	✗
ISIS	✗

TABLE 12.1 - *Summary of software packages able to undertake Test 12*

Although FLUCOMP is capable of undertaking unsteady flow calculations it was not possible to undertake Test 12 as the software package can not cope with the split flow requirements of the test.

The data set provided for this test was in MIKE 11 format and required approximations to be made in order to attempt the test with FLOODTIDE, HYDRO-1D and ISIS. However, due to calculation instabilities with the approximated model set up both FLOODTIDE and ISIS were unable to complete the test. A description of the undertaking of Test 12 with FLOODTIDE, HYDRO-1D, MIKE 11 and ISIS is given next.

TEST 12 – River Calder

Location along River Calder		Maximum Water		
Chainage (km)	Description	1976 Storm	MIKE 11	HYDRO-1D
10.312	First cross-section downstream of inflow boundary	62.680	62.664	62.920
10.744	Junction with WIMPENNY	62.660	62.645	62.545
13.030	Junction with BROOKFOOT	59.040	59.019	60.872
13.449	Junction with BSIDE1	58.560	58.545	60.809
14.425	Junction with CANAL	57.290	57.256	60.530
15.292	Cross-section between weirs at 15.145 km and 15.349 km	55.100	55.040	58.079
16.044	Downstream of junction with CANALEX	54.190	54.150	54.079

TABLE 12.2 : *TEST 12 - Summary of calculated maximum water elevations at selected locations*

FLOODTIDE

The test set-up for FLOODTIDE essentially required a number of approximations to the original data set so that the test could be attempted. FLOODTIDE uses a connection unit to model any node at which three or more flows join. It requires a cross-section to be defined for each of the connecting river branches. Hence, to accommodate the connecting flows the cross-section specified in the original data set at each junction was repeated. To insert a weir into the river network at the required locations, cross-sections were repeated from the previous upstream and next downstream sections and inserted at either side of a natural weir unit. This unit accurately approximates the required weir profiles as illustrated in Appendix I, Graph 1 by defining the weir level with respect to offset. However, for the weirs specified in CANAL 3 at 0.090 m and the Calder at 15.090 m the weir geometry had to be adjusted since a weir of a closed profile can not be defined in FLOODTIDE. A culvert unit was inappropriate for the weir profile.

The river network was gradually built up in different stages. Firstly the river Calder was set up with the appropriate boundary conditions and the appropriate approximations as described above. Once the first network had been set up it was found that critical, backwater and unsteady simulations with a time step of 60 s could be made successfully. Next the canal was added including the Wimpenny connection so as to provide the looped system. Unfortunately FLOODTIDE produced inaccurate critical calculation results and hence backwater and unsteady calculations were not possible. In order to proceed with an unsteady calculation the critical backwater results were modified, an approach used with previous tests, according to the initial conditions specified in the MIKE 11 'HOT START' file. However, it was found by using such initial conditions that an unsteady simulation failed in the same manner as for Test 9. It was felt at this point to suspend further development of the river network as the network would become more complicated and the same problem would continue to exist. Hence, no results have been presented for Test 12 from FLOODTIDE.

HYDRO-1D

The test set-up for HYDRO-1D essentially required two approximations to the original data set to enable the testing. Firstly HYDRO-1D requires a cross-section at both the upstream and downstream faces of a junction. To accommodate this the cross-section specified in the original data set at each junction was repeated as appropriate. Secondly HYDRO-1D does not specify the weir insertion and the weir geometry in the same manner as for MIKE 11. In order to represent the weir geometry as illustrated in Appendix I, Graph 1, approximated cross-sections were added at both the upstream and downstream faces of the structure at the required locations. The same profile as the weir crest was then defined with the exception of the central portion. For this portion of the weir the depths was increased to that of the bed levels at either the previous upstream or next downstream cross-sections as appropriate. Between these approximated cross-sections a normal weir was then specified with the appropriate crest levels. For the weirs specified in CANAL 3 at 0.090 m and the Calder at 15.090 m the cross-section geometry had to be adjusted since closed cross-sections can not be defined in HYDRO-1D.

The river network was gradually built up in different stages. Firstly the river Calder was set up with the appropriate boundary conditions and the appropriate approximations as described above. Once the main river network had been set up a successful simulation could be made with a time step of 0.1 hr. However, the residual limit for convergence was not satisfied during the early stages of the storm event. It should be noted that the time step of 0.1 hr was the smallest time step possible for this simulation period due to the internal limitations of array size imposed within HYDRO-1D. Once the main river network was set up successfully the canal was added with the Wimpenny connection so as to include a complete looped system. All other network branches were omitted at this stage. Again a simulation was made, with the residual limit for convergence not being satisfied for the early stages of the storm event. The remaining network branches were finally added one by one, with further test simulations being carried out after each addition to check that HYDRO-1D could cope with the increased complexity of the network.

Once the complete river network was set up, a simulation was successful. It should again be noted that the residual limit for convergence was not satisfied during the early stages of the simulation. However, the residual limit became almost constant at every time step and gradually reduced over the early stages of the storm event until the required convergence value was met.

It should be noted that for each build-up stage of the network HYDRO-1D was able to produce a converged solution for the backwater profile. This is then subsequently used as the initial conditions for the unsteady simulation.

The maximum water level results obtained for Test 12 are presented in Table 12.2. It can be seen from this table that the results compare well with the calibration data at chainage's 10.312km, 10.744km and 16.044km, however, at 13.030km, 13.449km, 14.425km and 15.292km the results are noticeably lower than calibration data. It should be noted that improved results might be obtained if calibration of the model is undertaken, however, this has not been undertaken as it is beyond the scope of this study.

MIKE 11

The data set provided for this test was already in MIKE 11 format which made the setting up of the river network a straight forward task. The supplier of the data set recommended that a 'HOT START' file be used to begin a model simulation. Attempts to start the model without the 'HOT START' file proved unsuccessful, clearly demonstrating the complex river network and the need for appropriate initial conditions.

It should be noted that the method adopted by MIKE 11 to define a weir is unique compared to the other software packages as it defines a weir's width with respect to elevation. This allows MIKE 11 to comprehensively define the flow area over a weir for various water depths by means of Q (h) tables calculated internally. It was advised that the number of levels which is used to calculate the Q (h) tables be checked especially in the canal as the default range may be larger than required. The cross-section profiles of the weirs defined in the test are illustrated in Appendix I, Graph 1.

Simulations with a time step of 0.1hr were used and provided results that appeared to be stable. By observing the graphical output in MIKE 11 it could clearly be seen that the water levels in the main river channel varied smoothly over the simulation period. It should be noted that the graphical illustrations available in MIKE 11 are extremely useful when analysing the results produced from a complex river network.

It can be seen from Table 12.1 that the results obtained show very good comparisons with the 1967 storm event. However, as with any computer model these values have only been obtained through model calibration by the data supplier.

ISIS

The test set-up for ISIS essentially required two approximations to the original data set to enable the testing. Firstly ISIS, like HYDRO-1D, requires a cross-section at both the upstream and downstream face of a junction. To accommodate this the cross-section specified in the original data set at each junction was repeated as appropriate. Secondly ISIS does not specify the weir insertion and weir geometry in the same manner as for MIKE 11. In order to represent the weir geometry as illustrated in Appendix I, Graph 1, approximated cross-sections were added at both the upstream and downstream faces of the structure at the required locations. To specify the weir profile two different approaches were attempted as describe below:

APPROACH 1: To be consistent with HYDRO-1D cross-sections at the weir were defined with the same profile as the weir crest with the exception of the central portion. For this portion of the weir the depth was increased to that of the bed levels at either the previous upstream or next downstream cross-sections as appropriate. Between these approximated cross-sections a normal weir was then specified with the appropriate crest levels. For the weirs specified in CANAL 3 at 0.090m and the Calder at 15.090m the cross-section geometry was adjusted so as to avoid the use of a closed cross-section.

The river network was gradually built up in different stages. Firstly the river Calder was set up with the appropriate boundary conditions and the appropriate approximations as described above, with the same initial conditions being used as specified in the MIKE 11 'HOT START' file. Once the first main river network had been set up it was found that a successful simulation could be made for a time step of 60 s. The Lillands Loop was then added to the river network. This produced significant instabilities in the solution and resulted in calculation tolerances being exceeded at various times throughout the simulation. However, a complete calculation could be undertaken. Next the canal was added including the Wimpenny connection. This time the simulation was not possible due to instabilities in the calculation. Investigations with pseudo time stepping so as to produce more suitable initial conditions, a reduced time step and also with changes to the calculation settings were without any success. The complete river network was also built to see if the instabilities previously encountered could be eliminated. However, as anticipated a calculation was not possible.

APPROACH 2: ISIS can define weirs with irregular shapes by the use of the SPILL unit as previously used for Test 1 Part B. Instead of defining the weir as an embankment as for Test 1 Part B the unit can be used to define a weir between two cross-sections in a river channel. To insert the weir into the network cross-sections were repeated from the previous upstream and next downstream sections and inserted at either side of the SPILL unit at the required locations. Then the SPILL unit was defined between the cross-sections with the required profile as illustrated in Appendix I, Graph 1. For the weirs specified in CANAL 3 at 0.090m and the Calder at 15.090m the weir geometry was adjusted in a similar manner as for HYDRO-1D and the previous approach as the SPILL unit would not accept closed weir profiles. The value for the weir modulus at each structure was set to 0.6, the same as that for Test 1 Part B.

As with the previous approach the river network was gradually built up in stages. Firstly the river Calder was set up with the appropriate boundary conditions and approximations as described above. It was then tested with both the initial conditions as specified in the MIKE 11 'HOT START' file and those from a steady run by ISIS. With such initial set up it was found that a successful unsteady simulation could be made with both of the initial conditions for a time step of 60s. Next the canal was added including the Wimpenny connection so as to provide the looped system. At this stage, ISIS also produced a successful simulation if the MIKE 11 'HOT START' values were used as the initial conditions. However, it should be noted that a simulation using the initial conditions from a steady run by ISIS resulted in the convergence criteria being exceeded throughout the simulation period. Further additions of the remaining channels into the network unfortunately proved to be unsuccessful. Calculation failure occurred due to instabilities in the solution for both of the initial conditions from the MIKE 11 'HOT START' and a steady run by ISIS. Several investigations were made with - (i) the pseudo time stepping method so as to produce more suitable initial conditions, (ii) changes to the calculation settings and (iii) a reduced time step, all of which proved to be unsuccessful.

No results have been presented for Test 12 from ISIS due to the problems encountered by both approaches for the test. Further modifications to the network were not undertaken due to the availability of the data provided.

2.12.4 Summary

1. The complex river Calder network was tested, with the original data set being provided by W. S. Atkins in MIKE 11 format.

2. The river network was built up in stages for HYDRO-1D and ISIS; The main river Calder was first simulated, followed by the canal and then the remaining channel branches. Except for MIKE 11, approximations had to be made for the weir geometry, as well as the inserted cross-sections at both upstream and downstream faces of the weir.

3. Among the five unsteady flow models, FLUCOMP was not able to undertake this test; and FLOODTIDE and ISIS also failed to simulate the complete network..

4. The results produced by HYDRO-1D were not as close to the calibration values as those by MIKE 11 and was assumed to be due to the approximations required in HYDRO-1D. Further calibration may have improved this situation.

5. MIKE 11 could not undertake an unsteady simulation when using initial conditions from a steady state run. A 'HOT-START' file, which was provided with the test data, was required to undertake the test successfully and this was deemed to be restrictive. The acquisition of such a 'HOT-START' file may require extensive modelling expertise.

6. For a complex river networks, such as this test case, it would be expected that modellers would obtain advice or modifications of the software from the developers in order to undertake the test successfully and accurately.

7. Site knowledge and further survey data may be useful for such a complex river network, particularly when approximations have to be made to represent the storage and weir structures in a network.

8. In principle, this should have been the most comprehensive and valuable test case for this study. However, the river system was extremely complex and the available data was limited. These limitations have undoubtedly restricted the scope of this test.

2.13 TEST 13 - Flood Wave

2.13.1 Aim of Test

The aim of this test is to establish the response of various unsteady models to a sinusoidal flood wave in a flat-bottomed idealised estuary as given in Estuary and Coastline Hydrodynamics(23). The results are to be compared to the analytical solution for a flood wave in a channel of finite length originally obtained by A.T.Ippen.

2.13.2 Introduction

The analytical solution for a flood wave in a channel of finite length and was based upon two crucial assumptions:

1. bed friction may be linearised.
2. the non-linear advective acceleration term may be neglected.

These assumptions are acceptable when the ratio of wave amplitude to the mean water depth is relatively small. The analytical solution calculates the water elevation in channel of finite length with a closed end by superimposing the incident wave with the reflecting wave as illustrated in Figure 13.1. However, due to friction resistance, the amplitude of the incident wave at the closed end will be smaller than that at the channel entrance. Similarly the amplitude of the reflecting wave at the entrance will be even smaller than that at the closed end. Hence, the analytical solutions at distance x and time t for a damped co-oscillating wave can be expressed for the wave amplitude $\eta_{x,t}$ as:

$$\eta_{x,t} = a_o\left[e^{-\mu x}\sin\left(\sigma t - k x\right) + e^{\mu(x-2l)}\sin\left(\sigma(t-t_1) + k(x-l)\right)\right]$$

and for the flow velocity $U_{x,t}$ as :

$$U_{x,t} = \frac{a_o\sigma}{H\sqrt{\mu^2+k^2}}\left\{\begin{matrix} e^{-\mu x}\sin\left(\sigma t - k x + \alpha\right) - e^{-\mu l}\sin\left(\sigma t - k l + \alpha\right) \\ -e^{\mu(x-2l)}\sin\left[\sigma(t-t_1) + k(x-l) + \alpha\right] + e^{-\mu l}\sin\left[\sigma(t-t_1) + \alpha\right] \end{matrix}\right\}$$

where: H = mean water level (m) a_o = incident wave amplitude (m)

TEST 13 – Flood Wave

l = channel length (m)
$t_1 = l/\sqrt{gH}$
μ = channel characteristic
$k^2 = \mu^2 + (\sigma^2/gH)$

t = time (s)
$\alpha = \tan^{-1}(\mu/k)$
$\sigma = 2\pi / T$
T = wave period

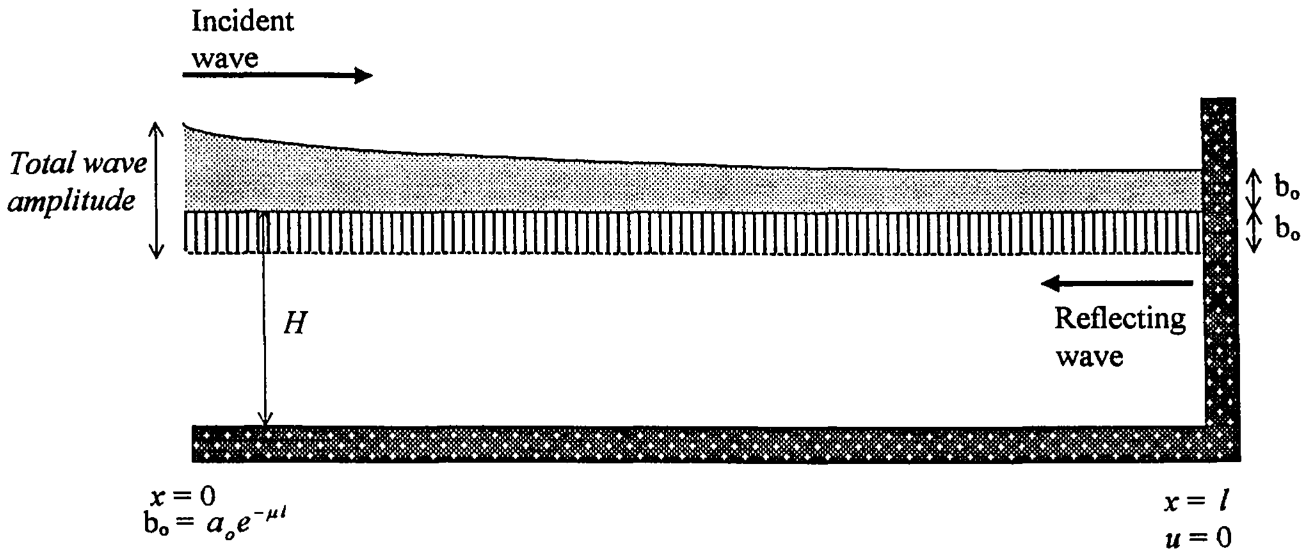


FIGURE 13.1: *TEST 13 - Illustration of channel of definitive length with a closed end.*

To test each of the models for a flood wave a wave period of 2 hours, with an initial amplitude a_o = 1m and a mean water level H = 20m, was selected. A flat rectangular channel of total length 100,000m, width 1,000m, cross-sectional depth 25m and constant Manning's roughness of 0.025 was specified with 201 equally spaced cross-sections (a Manning's roughness of 0.001 were also investigated). The closed boundary for each model was defined as a discharge boundary of zero inflow throughout the simulation. The downstream water level was specified as a sinusoidal wave of height 2m and period 2 hours. To be consistent with the analytical solution the initial water level was set to 20m throughout the channel. A time step of 60s was employed for the test with each of the software packages with the exception of HYDRO-1D, for which a time step of 72s was used. The wave speed in the channel may be calculated to be approximately 14m/s ($=\sqrt{gH}$). It will take about 2 hours for the initial incident wave at the entrance to reach the closed end of the channel and about another 2 hours for the reflecting wave to reach the open boundary. Since the open boundary condition at the channel entrance will be influenced by the reflecting wave after 4 hours and becomes difficult to define from then on, the simulation time was set to 4 hours. Comparisons of the model results with the above analytical solutions are then made for the duration of 2 to 4 hours.

2.13.3 Results

All the unsteady flow software packages were able to undertake Test 13, including FLOODTIDE, MIKE 11, HYDRO-1D, FLUCOMP and ISIS. Results for the water elevations above the mean water level (i.e. water level - mean water level) and the velocities at 25km, 50km 75km and 100km from the entrance of the channel are presented in Appendix J, Graphs 1 through to 10 for each individual package, together with the analytical solutions. Graphs 1 through to 10 also show the results at time phases of t/T = 1.25, 1.5, 1.75 and 2.0 for each individual package, together with the analytical solutions at these phases. Comparisons between all of the software packages were also made, as illustrated in Appendix J, Graphs 11 through to 14. It should be noted that the results for the whole channel length for each of the software packages are at 2.5hr (t/T=1.25), 3.0hr (t/T=1.5), 3.5hr (t/T=1.75) and 4.0hr (t/T=2.0).

It should also be noted that model results are only comparable with the analytical solutions for the duration of 2 to 4 hour at x = 100km, 2.5 to 4 hours at x = 75km, 3 to 4 hour at x = 50km, and 3.5 to 4 hour at x = 25km. This is when the reflecting wave has arrived. Similarly, the comparisons should be made for the channel reaches of 75km to 100km, 50km to 100km, 25km to 100km and the whole channel when t/T = 1.25, 1.5, 1.75 and 2.0 respectively.

Reasonable agreements exist for both water elevation and velocity comparisons for the appropriate comparable duration and channel reaches for all of the packages tested, with little difference in wave phases, as can be seen in Graphs 11 to 14. It can also be observed from the results that some degree of fluctuations in water level and velocity was produced at some stage by all the packages.

The incident wave has also been applied at the entrance for ten cycles assuming the effects of reflecting wave on the open boundary may be ignored. The results exhibited much stronger oscillations in water levels and velocities for all the packages, clearly indicating the existence of the reflecting wave and its impact. Therefore no results for such a long run have been presented. HYDRO-1D failed to produce any wave pattern for such a long run.

Although not presented here the results from the investigative tests with a Manning's roughness of 0.001 showed that the water surface profiles and velocity profiles were very similar to those when using a Manning's roughness of 0.025. However, in order to get the results to compare well with the analytical solution the bed friction term in the analytical solution had to be reduced considerably. It should also be noted that both the water level and velocity profiles were generally higher when using a Manning's roughness of 0.001 as opposed to 0.025.

FLOODTIDE

Setting up this test with FLOODTIDE was a straightforward procedure. A program was written by the University of Bradford so as to produce the data file in the required format with the appropriate cross sectional geometry.

Problems were observed when attempting to produce an accurate backwater solution for the initial boundary condition. The backwater solution produced by FLOODTIDE would not allow the unsteady calculation begin, hence, the backwater profile was altered with more suitable values, i.e. a constant water level of 20m, a constant zero velocity and a constant cross sectional area of 20,000m^2 throughout the channel. It should also be noted that due to the large number of cross sections defined in the test the number of layers used for the data tables had to be limited to 30 if an unsteady simulation was to begin.

The results from Test 13 are illustrated in Appendix J, Graphs 1 and 2. It can be seen that the model reproduced the flood wave. The effects of the reflecting wave can be clearly seen when it met the incident wave, producing a non-smooth water surface at x=25km at about 3.5 hour and at x=75km at about 2.5 hour in Graph 1. Velocity variations can also be seen when t/T=1.25 (at about 75km) and t/T=1.75 (at about 25km) in Graph 2. It can also be seen from Graph 2 that the water level near the open boundary was affected by the reflecting wave at t/T=2.0.

MIKE 11

Setting up this test with MIKE 11 was a straightforward procedure. A program was written by the University of Bradford so as to produce a cross sectional text file in the required format for import to MIKE 11.

TEST 13 – Flood Wave

The results from Test 13 are illustrated in Appendix J, Graphs 3 and 4. It should be noted that velocity values in the graphs are one minute out of phase to the water level results, as velocity values are calculated at each half time step whereas water levels are calculated at whole time steps.

It can be seen from Graph 3 that the model reproduced the flood wave. The effects of the reflecting wave can be clearly seen when it met the incident wave, producing a non-smooth water surface at x=25km at about 3.5 hour and at x=75km at about 2.5 hour in Graph 3. Velocity variations are stronger than those of FLOODTIDE, as can be seen when t/T=1.25 (at about 75km) and t/T=1.75 (at about 25km) in Graph 4. It can also be seen from Graph 4 that the water level near the open boundary was affected by the reflecting wave at t/T=2.0.

HYDRO-1D

Setting up this test with HYDRO-1D was straightforward by a program written by the University of Bradford so as to produce the 201 cross sectional data files required by the package. In addition a program was written so a to produce the network file in the required format without which the test would have taken considerable time to set up.

The undertaking of the test with HYDRO-1D required a time step of 0.02hr as selecting a time step of 0.01hr (smallest possible) would not allow a calculation to commence. The software vendors were contacted with this problem and explained that is was due to the array size set within the programs code. The vendors were able to offer an updated version of the program that would overcome this problem. However, to maintain the testing consistency the version as originally provided was used for the test.

The results from Test 13 with HYDRO-1D are illustrated in Appendix J, Graphs 5 and 6. However, it should be noted that in order to obtain a converged solution throughout the simulation the value for the flow correction in the run menu had to be set at 150 (suggested value is 10% of flow). The results obtained from HYDRO-1D are very similar to those from the other software packages. It should also be noted that the investigative tests with a Manning's roughness of 0.001 were not possible with HYDRO-1D as the package crashed when attempting to undertake a simulation. However, it should be noted that the test could be undertaken when a constant Manning's roughness value of 0.005 was defined along the complete length of the channel.

FLUCOMP

A program was written by the University of Bradford so as to produce the section data file in the required format for FLUCOMP. The additional files required to undertake the backwater calculation and unsteady calculation were set up without any problems. The undertaking of the backwater calculation produced the expected backwater profile without any problems. However, attempts to undertaking the unsteady simulation initially posed significant problems. It was found through investigation that by reducing the number of cross section to 200 (limit of package) instead of 201 a calculation could be made. Therefore the distance between the last two cross-sections was increased from 500 m to 1,000 m in order to maintain the total channel length of 100km.

The results for Test 13 from FLUCOMP are illustrated in Appendix J, Graphs 7 and 8. It can clearly be seen from these two Graphs that both water levels and velocities are slightly smoother than those produced by MIKE 11 and FLOODTIDE. A small hump in water level occurred near the entrance at t/T=1.75 and soon smoothed out at t/T=2.0, as can be seen in Graph 8.

ISIS

The setting up of this test with ISIS was a straightforward procedure. A program was written by the University of Bradford so as to produce the data file in the required format. The undertaking of the test posed not problems, the results of which are illustrated in Appendix J, Graphs 9 and 10. It can be seen from these two Graphs that the results appear to be much smoother than those produced by other packages. However, weaker effects of the reflecting wave can still be seen when it meets the incident wave at x=25km at about 3.5 hour and at x=75km at about 2.5 hour in Graph 9. Velocity variations can be seen when t/T=1.25 (at about 75km) and t/T=1.75 (at about 25km) in Graph 4.

2.13.4 Summary

1. A flood wave propagating in an idealised rectangular channel with a flat bottom has been studied, with the analytical solutions for the water elevation and velocity being obtained and compared with the model results.

2. The effects of the reflecting wave were apparent and could not be ignored. Different packages produced similar degrees of oscillations in the water level and velocity profiles. It appeared that the results by ISIS were smoother than other packages, but that all model results failed to agree closely with the analytical water level results.

3. The wave imposed at the open boundary was unable to propagate into the channel when values of less than 150 were used for the flow correction factor in the calculation settings of HYDRO-1D.

4. FLOODTIDE, HYDRO-1D, MIKE 11, FLUCOMP and ISIS all produced reasonable results of the wave phase.

5. When using a low Manning's roughness value of 0.001 as opposed to 0.025, the results were very similar in profile to the analytical solution. However, the bed friction term in the analytical solution had to be reduced considerably in order to achieve this level of agreement.

6. Through investigative tests it became apparent that the velocity and water elevation profiles were generally lower when a Manning's roughness of 0.025 was used in comparison with a value of 0.001.

7. It was disappointing that none of the models were able to reproduce the analytical results for this test to a high degree of accuracy. However, the models included the non-linear advection and bed friction terms and these would ideally need to be removed for direct comparisons.

SECTION 3

Overview and Conclusion

This section consists of:

- Discussion and Conclusions
- Vendors Comments
- Software Updates
- Performance Chart
- References
- Bibliography

3.1 DISCUSSION AND CONCLUSIONS

The stage one report for this study, produced by Sir William Halcrow and Partners Ltd and HR Wallingford Ltd, describes the eleven tests that were originally programmed for this stage of the study. However, during the testing period it was decided by the Project Steering *Group* that Test 8 (i.e. the Tidal Thames Model) should be dropped and that Test 12 (i.e. the River Calder), provided by WS Atkins, should be used as a replacement. In addition Test 13 (i.e. the analytical Flood Wave), suggested by the University of Bradford, was commissioned by the Environment Agency during the period of the program.

The original title of this study was changed from 'Benchmarking of Hydraulic Models' to Benchmarking and Scoping of Hydraulic River Models'. This was primarily due to the lack of calibration and field data available for most of the tests suggested in the stage one report and hence for most tests it was only possible to check if tests could be undertaken by the models.

There were several common problems and limitations with the data sets provided for this study, which surfaced during the testing period. Except for Tests 12 and 13, all of the data sets provided for the testing were in a format that was more suited to the requirements of ISIS. This bias undoubtedly turned out to be a disadvantage for many of the software packages and approximates frequently had to be made to the data sets in order to attempt the tests. Furthermore, boundary and cross-sectional data was often provided to three, four or even five decimal places, whereas many of the software packages were only set-up to input data to two decimal places as this is typically the order of accuracy of measured data normally required by practicing engineers.

Various structures in the flow path have been tested in Tests 1 and 11, including sluice gates, weir, embankment, pump and bridges, under steady flow conditions. The testing of the sluice gate was comprehensive, whereas only a specific type of weir, i.e. the crump weir, and only a specific operation mode of the pump was tested.

Test 1 Part A provided a comprehensive range of tests cases for a sluice gate. The ability of ISIS and FLUCOMP to model both under-gate flow and over-gate flow of a sluice with a single structure was an advantage over MIKE 11, which required two structures at the same location, one for over-gate flow and one for under-gate flow. The limitations of HYDRO-1D to submerged flow conditions severely restricted the suitability of this package to model flow through a sluice. The bridge modelling approach adopted by HEC-RAS and HEC-2 to model the sluice allowed both over-gate and under-gate flow conditions to be modeled. The similarity between these results and the MIKE 11 results suggested that the bridge option in both HEC-RAS and HEC-2 may be a suitable approximation to a sluice gate.

FLOODTIDE and ISIS were able to undertake Test 1 Part B according to the data set, however the test required approximations to be made with HYDRO-1D and MIKE 11. Nevertheless, the use of a link channel and weirs to control flow from one channel to another over an embankment was shown to give a satisfactory approximation to the test. The inability of FLOODTIDE to calculate automatically the split flow over the embankment from a steady simulation necessitated an unsteady calculation with constant boundary conditions to be undertaken. Although this was not a problem for this particular test, this requirement was though to be unsatisfactory for a complex river system, which require accurate initial conditions.

The ability of ISIS and HYDRO-1D to undertake Test 1 Part C with a land drainage pump has demonstrated the bias of the data set. It would be a relatively straightforward task for any developer to include such a pump requirement in any 1-D model. Although MIKE 11 is capable of modelling pumps it was not tested, since the data set did not match the input requirements.

Only FLOODTIDE, FLUCOMP, ISIS, LD01 and GLC-BACKWATER have the ability to model a crump weir. However, MIKE 11 and HYDRO-1D both have alternative weir structures that can be used to approximate a crump weir. HEC-RAS and HEC-2, which do not have weir structures, required the use of their bridge routines to model weirs, which approximate a weir structure by allowing overflow of the structure. This approach although not ideal has been shown to be a crude method of representing flow over weirs with these two packages. An alternative approach for these two packages is to raise the bed level at the weir site to the crest of the weir. To approximate a weir with both CHANNEL and BAKWATER the bed level at the location of the weir has to raised so as to approximate the weir profile. This approach has proven to be effective for BAKWATER but not for CHANNEL. The lack of benchmarking data for Test 1 Part D necessitated the comparison of results with theoretical solutions for flow over a crump weir. Although FLOODTIDE, ISIS, GLC-BACKWATER and BAKWATER have all produced reasonable agreement with the theoretical solution, calibration of the structure that would usually be undertaken in a river modelling project was not investigated with any of the software packages requiring approximation to the structure.

For the simple steady state looped model test (i.e. Test 2) it has been demonstrated that different solutions are possible from models for such a common flow situation. The results obtained from the software packages have used either a water elevation balance or an energy balance approach to calculate split flows at a junction, the later option being considered to be a more appropriate criteria for split flows. Only ISIS and HYDRO-1D, which use the water level balance approach, have been able to produce a solution that compared well with the referenced split flow data. HEC-RAS, which used the energy balance method, has produced results that have split flow almost evenly between the two channels. However, MIKE 11, which used the water level balance method, produced a solution that was between the referenced split flow solution and an even split flow solution. An alternative and perhaps more suitable method of assessing the performance of the software packages was to calculate water elevations at split flow locations. Using this approach HYDRO-1D, MIKE 11 and ISIS showed very similar water levels at the junctions, especially when the results were compared to only two decimal places. For the test configuration specified the effect of a 0.01m change in the water level can alter the split flow by as much as 10 cumecs, which clearly highlights the sensitivity of the system. The variable results produced by FLOODTIDE and the semi-internal iteration procedure required by HEC-RAS have indicated that for this test only HYDRO-1D, MIKE 11 and ISIS are suited to solving the split flow scenario.

Tests 3 and 4 enabled the software packages to be checked for predicting normal depths in a simple triangular channel, for subcritical and supercritical flows respectively. For Test 3 the results from GLC-BACKWATER compared poorly with the analytical solution, possibly due to the number of levels used internally to calculate conveyance values. The relevance of this was also evident with FLOODTIDE, which allows the user to define the number of levels used to calculate the conveyance. Using 15 layers with FLOODTIDE produced results that did not compare well with the analytical solution, whereas using 50 layers produced results that agreed almost perfectly. For MIKE 11 it became evident that the method adopted for calculating the conveyance had a significant effect on the results obtained. For both Test 3 and Test 4 the hydraulic radius method for calculating the conveyance produced results that were much closer to the analytical solution, as compared with the resistance radius method which did not produce such good agreement. It was observed that the resistance radius is more appropriate for cross-sections with complex shape or over bank flows, whereas the hydraulic radius is more suited for regular cross-sections. Only HEC-RAS, HEC-2, MIKE 11, HYDRO-1D and ISIS were clearly capable of accurately calculating the normal water depth in a channel for supercritical flow conditions.

With the exception of CHANNEL all of the software packages have demonstrated in Test 5 that they are capable of undertaking calculations which involve subcritical, supercritical and transitional flows in a straight rectangular channel. However, only HEC-RAS, HEC-2, MIKE 11, and ISIS have demonstrated that they can successfully calculate both supercritical and transitional flows that compare accurately with the analytical solutions. Both HEC-RAS and HEC-2 required an upstream boundary water level to be given when supercritical flow conditions were to be modeled, and both an upstream and downstream water level boundaries for mixed flow regimes. HYDRO-1D required specific calculation settings so as to cope with supercritical regions of flow as the version under test does not suppress the convective term in the momentum equation when the Froude number exceeds a specified upper value. The ability of both MIKE 11 and ISIS to accurately undertake unsteady calculations with supercritical or transitional flows with a single downstream water level boundary was seen to be a major advantage over the other software packages under test.

The two parts to Test 6 were designed to assess the ability of the software packages to model floodplain flows for straight and meandering channels, i.e. Parts A and B respectively. However, for Part A, the test produced mixed results due to the supercritical boundary conditions specified in the test configuration. All of the software packages except GLC-BACKWATER were able to take account of the varying Manning's roughness across the cross-section. However, only FLOODTIDE, HEC-RAS, HEC-2, FLUCOMP and ISIS were also able to take account of the meandering effects. The results from HEC-RAS, HEC-2, MIKE 11 and ISIS for the straight channels (both wide and narrow floodplains for Part A) were all similar, suggesting that any one of these packages was suitable for straight channels with floodplains. Part B of the test for a meandering channel produced two distinct sets of results. However, the remaining software packages, with the exception of LD01 and FLUCOMP, with the latter being able to take account of the meanders, produced similar subcritical solutions. The difference between the two sets of solutions was around 0.06m. However, the small scale laboratory model, the lack of benchmarking data, and the closeness of the results from the packages which do not take account of the meanders preclude an overall assessment of the most suitable packages for modelling flows over meandering channels.

The data set provided for Test 7 included the water level boundary and cross-sections to three decimal places and the inflow boundary to five decimal places for a physical model of the River Blackwater, sited in the Flood Channel Facility at HR Wallingford. The practicality of this test to real modelling projects and the measurement of field data was considered to be limited. Most modellers only have field data and require results to two to three decimal places. Only HEC-RAS, HEC-2, FLUCOMP and ISIS were able to take account directly of the meandering channel, as for the previous test, and the information required to set up FLOODTIDE was not available. However, treating the channel as a straight channel FLOODTIDE and the remaining software packages were able to undertake the test and produced similar results. In fact the models which treated the channel as a straight channel produced results that were in closer agreement with the experimental results, and slightly higher than the results from the models which took account of the meanders by use of relative path lengths or equivalent. As for Test 6 the difference between the two sets of solutions was around 0.01m for all three flow conditions tested. However, the scale of the laboratory model and the closeness of the results from the packages which do and do not take account of the meanders again preclude an overall assessment of the most suitable packages for modelling the laboratory model of the Blackwater River and the flows over the meandering channels.

In Test 9, which has four parts, for unsteady flows in the River Blyth, the relevance, effect and suitability of interpolated cross-sections and their locations to modelling flows in a river network has been clearly demonstrated. The two unsteady models that can internally calculate interpolated cross-sections (and which do not require geometric data) are ISIS and MIKE 11. However, the provision of interpolated cross-sections at specific locations has resulted in the data set only being suited to ISIS format – this being a major restriction of the data set. However, this has been successfully overcome by providing externally calculated interpolated cross-sections for MIKE 11, FLUCOMP and HYDRO-1D, all of which can model unsteady flows. FLOODTIDE, which is also an unsteady package, demonstrated sever problems in modelling the flows for Test 9 and did not produce any results. The inability of ISIS to undertake Test 9 Part A and B with and without the internally specified internal interpolated cross-sections respectively, demonstrated that ISIS requires engineering judgement and experience by the user when modelling natural rivers and selecting locations for the internal interpolated cross-sections. However, the successful use of the external interpolated cross-sections for Part A with ISIS demonstrated that the interpolations with geometric cross-sectional data had advantages over internal interpolations, although the user should be aware of limitations in using interpolated cross-sections, as opposed to measured cross-sections. For both Parts A and B of the test HYDRO-1D and FLUCOMP exhibited poor results, due to the selection of the interpolated cross-section locations. However, satisfactory results were obtained for Part C using these packages when suitable locations for interpolated cross-sections were used. Both HYDRO-1D and ISIS produced instabilities in the solutions at various times during the simulations. HYDRO-1D produced steep water surface gradients at regions for the different sets of cross-sections specified. Overall MIKE 11 proved to be the most stable model for this test and the most consistent performer for Test 9. Results from MIKE 11 were very similar for each part of Test 9 and were least affected by the number and location of the interpolated cross-sections. MIKE 11 also proved to be the model that was least influenced by the fixed downstream water level boundary, as specified for each part of the test. The same maximum water level results were calculated using HYDRO-1D for Parts C and D of Test 9, even though different inflow hydrographs were used. Also, the instabilities observed in the results placed some scepticism over the accuracy of HYDRO-1D for this test.

For the culvert test (Test 10), the inaccurate results obtained using FLOODTIDE, for both parts A and B, preclude this model from being a suitable tool for calculating flow through a culvert. However, the reasonably satisfactory results obtained for both parts of the test using ISIS, MIKE 11 and HYDRO-1D suggest that any one of these models could be used to model culvert flows. The approximations required to the test configuration in order to undertake the test using MIKE 11 and HYDRO-1D, which were due to the provided data set being set-up for ISIS, have been shown to be satisfactory. However, several key points have become apparent. In order to achieve accurate velocity results at mid-culvert from MIKE 11, closed cross-sections that represent the culvert opening need to be defined. However, the water level predictions were not noticeably affected by the use of either open or closed cross-sections. The requirement of the invert level of the culvert to be higher than the cross-section immediately downstream of the structure was a feature in MIKE 11 that was thought to be a limitation for some engineering situations. The inability of HYDRO-1D to provide velocity results within the culvert only allowed an assessment of the water levels to be made. For both ISIS and MIKE 11 the calculated velocities were very similar to the theoretical full culvert flow velocities. Finally, the ability of ISIS to connect culvert units directly to each other in series is useful for long culverts and for culverts with varying slopes. Using a multi-unit culvert also helps ISIS to reduce the instabilities when the flow in the culvert just becomes full.

For the bridge test (i.e Test 11), CHANNEL is the only software package that does not have the ability to model a bridge. However, neither FLUCOMP nor BAKWATER have the capability to model bridges that are arched in profile. For Part A of Test 11, i.e. the USBPR Bridge, all of the software packages were able to calculate the bridge afflux very close to the theoretical solution. This suggested that any of the packages tested for Part A of Test 11 were suitable for modelling bridge structures similar to the data set. However, the test did not assess the ability of the software packages to model flow that required over-topping of the bridge, nor did it assess the ability of the packages to model bridges with sloping abutments or piers that were at an angle to the normal flow direction. Some of the software packages can model such features and may be more suitable for specific modelling requirements. For Part B of Test 11 the irregular shaped opening used by some packages for bridges were successfully adopted to approximate the arched bridge. With the exception of ISIS, all of the software packages were able to calculate the bridge afflux close to the theoretical value. However, ISIS, which was most suited to the data set, produced a result that was noticeably higher than the theoretical solution. The problems encountered with setting up the arch bridge test and the accuracy of the results obtained from the package have placed some scepticism on the ability of ISIS to predict flows through an arched bridge.

The River Calder data set (i.e. Test 12) provided an opportunity to test the software packages against a data set that was not specifically suited to the requirements of ISIS. The data set, which was provided in MIKE 11 format, required approximations to be made to undertake the test using FLOODTIDE, HYDRO-1D and ISIS. However, the approximations made using both FLOODTIDE and ISIS, which were considered most appropriate, have resulted in calculation failure. It is conceivable that both of these packages require highly accurate initial conditions in order to overcome the instabilities that have been encountered with each model for this test. For both FLOODTIDE and ISIS neither the use of a steady backwater calculation or the use of the 'HOT-START' values produced from MIKE 11 were able to provide suitable initial conditions to overcome the instabilities and calculation failure encountered during the unsteady simulations. Conversely, HYDRO-1D was able to complete the model setup and run. However, the inability of MIKE 11 to undertake the test without a suitable 'HOT-START' file has illustrated the importance of suitable initial conditions and the problems that inexperienced modellers can encounter. The test has highlighted the need to survey data to be generated in such a format that is directly suited to the requirements of the specific package, hence used to model a specific study.

The unsteady Flood Wave test (i.e. Test 13) has clearly demonstrated that none of the models is currently capable of dealing with a reflecting boundary condition. Although the theoretical solution assumed that bed friction was linearised and that the non-linear advective acceleration terms were neglected, the results have shown that each package produced similar degrees of oscillations in the water elevations and velocities once the reflecting wave had met the incident wave. All of the models produced reasonable phase results for comparable periods of the channel reaches. However, ISIS produced results that are smoother than those obtained using the other models.

Overall the Benchmarking and Scoping Study has enabled all of the models to be assessed according to functionality and applicability to the test conditions imposed. Furthermore, there has been some opportunity to test the accuracy of the models. The data sets have generally been biased towards ISIS format. Many of the software packages also have capabilities that have not been investigated or commented upon due to limitations of this study. All of the models have been shown to be suitable for modelling simple subcritical flows. However, only MIKE 11, ISIS, HEC-RAS and HEC-2 have the ability to model supercritical flows accurately. The ability of FLOODTIDE, MIKE 11, HYDRO-1D, FLUCOMP and ISIS to undertake unsteady flows increased their potential application. However, FLOODTIDE, which was unable to complete several of the tests satisfactorily and FLUCOMP, which was only capable of modelling flows for a single downstream boundary, have exhibited restrictions in their ability to model unsteady flows. MIKE 11 and ISIS were deemed to be the only models with few limitations for unsteady and supercritical flows.

It should also be appreciated that several of the tests highlighted deficiencies in the models which could readily have been addressed by some of the model developers had they been given the opportunity to modify their models accordingly. In addition it is felt that most packages could readily be amended to accept or display data with more decimal points. However, in practical engineering terms three decimal places for boundary conditions and two decimal places for cross-sections is generally considered acceptable.

3.2 PERFORMANCE CHART

3.2.1 Introduction

Any general assessment to suggest the best overall model(s) would be very subjective and limited if only the ability of the models to undertake the tests as required by the data set were to be considered. Most managers and users of 1-D river models are aware that many of the developers are able to modify their packages to suit the specific conditions for any model study.

This study has tested only some of the features available in the models, with many of the features of some of these models not being included in the testing and hence not being covered within this report. The data set provided for this study has been predominantly suited for input to ISIS and hence many of the tests have required approximations to the test setup in order to undertake some of the testing.

3.2.2 Understanding the Chart

The manager and user who is expected to use the performance chart *should not use* the chart to assess which package is either the most accurate or the one which has been able to undertake the most tests. The attached chart *should only be used* as a guide to the models that are more suited to model a specific problem and those that are likely to produce acceptable results. Below is the key for reading the chart:

KEY FOR PERFORMANCE CHART:

✓ = Test undertaken as required.
✓' = Test undertaken as required, although approximations to the test setup required due to data set.
✗ = Test not possible with model.
? = Test not possible due to limitations of data set provided.
! = Refer to comments in report.

TEST	SOFTWARE PACKAGE	FLOODTIDE	HEC-RAS	HEC-2	HYDRO-1D	MIKE 11	FLUCOMP	ISIS	BAKWATER	GLC- BACKWATER	CHANNEL	LD01
Test 1 - Structures	Sluice Gate	×!	✓’	✓’	✓!	✓’	✓	✓	×	×	×	×
	Flood Embankment	✓!	×	×	✓’	✓’	×	✓	×	×	×	×
	Land Drainage Pump	×	×	×	✓	?	×	✓	×	×	×	×
	Crump Weir	✓	✓’!	✓’!	✓’!	✓’	✓	✓	✓!	✓!	✓!	✓!
Test 2 – Simple Steady Looped Model		✓!	✓	×	✓	✓	×	✓	×	×	×	×
Test 3 – Subcritical Flow in Triangular Channel		✓!	✓	✓	✓	✓	✓	✓	✓	✓!	✓	✓
Test 4 – Supercritical Flow in Triangular Channel		×	✓	✓	✓!	✓	×	✓	×	✓!	×	×
Test 5 – Steady State Analytical Solutions	Part A: Subcritical Flow	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
	Part B: Supercritical Flow	×	✓	✓	✓!	✓	×	✓	×	×	×	×
	Part C: Supercritical to Subcritical	×	✓	✓	✓!	✓	×	✓	×	×	×	×
	Part D: Subcritical to Supercritical to Subcritical	×	✓	✓	✓!	✓	×	✓	×	×	×	×
	Part E: Supercritical to Subcritical to Supercritical	×	✓	✓	✓!	✓	×	✓	×	×	×	×

TABLE 14.1: Benchmarking and Scopeing Study - Performance Chart (Part 1 of 2)

TEST / SOFTWARE PACKAGE		FLOODTIDE	HEC-RAS	HEC-2	HYDRO-1D	MIKE 11	FLUCOMP	ISIS	BAKWATER	GL-C BACKWATER	CHANNEL	LD01
Test 6 – SERC Flood Channel Facility	Part A1: Straight Channel with wide flood plain	✓	✓	✓	✓	✓!	✓	✓	✓	✓!	✓	✓
	Part A2: Straight Channel with narrow flood plain	✓	✓	✓	✓	✓!	✓	✓	✓	✓!	✓	✓
	Part B: Meandering Channel with flood plain	✓	✓	✓	✓’!	✓’!	✓	✓	✓’!	✓’!	✓’!	✓’!
Test 7 – FCF Blackwater Model		✓	✓	✓	✓’	✓’	✓	✓	✓’	✓’	✓’	✓’
Test 8 – Tidal Thames Model		Test Not Undertaken										
Test 9 – River Blythe (unsteady)		×!	×	×	✓!	✓	✓!	✓!	×	×	×	×
Test 10 – Circular Culvert (unsteady)	Part A: Mild Slope	×!	×	×	✓’	✓’	×	✓	×	×	×	×
	Part B: Steep Slope	×!	×	×	✓’	✓’	×	✓	×	×	×	×
Test 11 – Bridges	Part A: USBPR Bridge	✓	✓	✓	✓	✓’	✓	✓	×	✓	×	✓’
	Part B: HR Wallingford Arch Bridge	✓’	✓’	✓’	✓	✓’	×	✓!	×	✓’	×	✓’
Test 12 – River Calder (unsteady)		×!	×	×	✓’	✓	×	×!	×	×	×	×
Test 13 – Flood Wave (unsteady)		✓	×	×	✓	✓	✓	✓	×	×	×	×

TABLE 14.1: Benchmarking and Scopeing Study - Performance Chart (Part 2 of 2)

3.3 VENDORS COMMENTS

Each of the software vendors/developers has been given the opportunity to respond to the contents of this report with respect to their own software packages. A strict limit of no more than two pages of text was stipulated in the invitation to supply a comment. This section contains the responses that were received from the vendors/developers verbatim.

3.4 SOFTWARE UPDATES

Each of the software vendors/developers has been given the opportunity to provide information on any updates or their latest version of their software which has been tested within this study.. A strict limit of no more than two pages including any graphical illustrations was stipulated in the invitation to supply this information. This section contains the responses that were received from the vendors/developers verbatim.

3.5 REFERENCES

(1) BOS, M.G, Discharge Measurement Structures, ILRI publication 20, 1989, 3rd revised edition.

(2) User Manual, FLOODTIDE *Hydrodynamic model*, Version 3, Volume II – Operators Guide, Babtie Group Limited, Glasgow, 1995, pp1-42

(3) User Manual, MIKE 11 *A Microcomputer Based modeling System for River Channels*, Version 3.11, Danish Hydraulic Institute, 1st Edition, 1995

(4) WHITE W.R, The performance of two-dimentional and flat-V triangular profile weirs, Proc.I.C.E, Suppl.(ii), 1971, paper 7350S, pp21-48

(5) ACKERS.P, The hydraulic design of straight compound channels, Report SR 281, Volume 1 and 2, HR Wallingford, 1991

(6) User Manual, The FLUCOMP River Model Package *An engineers guide*, Report EX 999, HR Wallingford, 1982

(7) KING H.W, Handbook of Hydraulics for the solution of hydrostatic and fluid-flow problems, 5th Edition, London, McGraw-Hill Book co., 1963

(8) Operation Manual, GLC-BACKWATER, Revision 4, National Rivers Authority, Thames Region, UK, pp1-24

(9) CHAUDRHY M.H, SCHULTE A.M, Canadian Journal of Civil Engineering, Volume 13, 1986

(10) FLOODTIDE. op cit

(11) User Manual, ISIS Flow, Version 1.1, Halcrow/HR Wallingford, UK, 1995

(12) Mc DONALD I, BAINS M.J, NICHOLS N.K and SAMUELS P.G, Steady Open Channel Test Problems with analytical solutions – submitted to ASCE Journal of Hydraulic Engineers

(13) FLOODTIDE. op cit

(14) MIKE 11. op cit

(15) FLUCOMP. op cit

(16) FLOODTIDE. op cit

(17) MIKE 11. op cit.

(18) ISIS. op cit

References

(19) HENDERSON F.M, Open Channel Flow, MacMillian, London, 1981.

(20) CHOW V.T, Open Channel Hydraulics, McGraw-Hill, 1959

(21) ISIS. op cit

(22) Hydraulics Research Publication, Afflux at Arch Bridges, Report SR182, HR Wallingford, 1988

(23) IPPEN, A.T. (Ed.), Estuary and Coastline Hydrodynamics, Engineering Society Monographs, New York, MacGraw-Hill Book Co., 1966, Chapter 10

3.6 BIBLIOGRAPHY

Backwater Program Suite, August 1980, Southwest Water Authority

User Manual, FLOODTIDE *Hydrodynamic model*, Version 3, Volume I – Engineers Guide, Babtie Group Limited, Glasgow, 1995

User Manual, FLOODTIDE *Hydrodynamic model*, Version 3, Volume II – Operators Guide, Babtie Group Limited, Glasgow, 1995

User Manual, MIKE 11 *A Microcomputer Based modeling System for River Channels*, Version 3.11, Danish Hydraulic Institute, 1st Edition, 1995

Reference Manual, MIKE 11 *A Microcomputer Based modeling System for River Channels*, Version 3.11, Danish Hydraulic Institute, 1st Edition, 1995

User Manual, The FLUCOMP River Model Package *An engineers guide*, Report EX 999, HR Wallingford, 1982

Introduction to MicroFLUCOMP, HR Wallingford

User Manual, Micro Drainage, Version A.5, Micro Drainage, UK, Jan 95

Quick Reference Guide, MicroFLUCOMP, August 1989, HR Wallingford

Operation Manual, GLC-BACKWATER, Revision 4, June 1991, National Rivers Authority, Thames Region, UK.

User Manual, HEC-RAS, River Analysis System, Version 1.0, July 1995, US Army Corps of Engineers, Hydrological Engineering Centre.

Hydraulic Reference Manual, HEC-RAS, River Analysis System, Version 1.0, July 1995, US Army Corps of Engineers, Hydrological Engineering Centre

User Manual, HEC-2 Water Surface Profiles, September 1990, CPD-2A, US Army Corps of Engineers, Hydrological Engineering Centre

User Manual, HYDRO-1D, Hydraulic Model for River Systems, Version 4.0, November 1995, Mott MacDonald, UK

User Manual, ISIS Flow, Version 1.1, Halcrow/HR Wallingford, UK, 1995

LD01 PC Version, Data & Performance, August 1994, National Rivers Authority, South West Region. Ref:DI/04/10/0002

COED Corps of Engineers Editor, CPD-56, February 1987, US Army Corps of Engineers, Hydrological Engineering Centre

Appendix A

MODE	DESCRIPTION	CONDITIONS	EQUATION
0	Gate open, water levels below weir crest.	$h_1 \le 0.005(L-r)$	$Q=0$
1	Gate closed, water levels below gate top.	$h_1 \le 0.0001$; $h_1 - h_g \le 0.005(L-r)$	$Q=0$
2	Gate closed, free flow over gate.	$h_o \le 0.001$; $\left(h_1 - h_g\right) > 0$; $\dfrac{\left(h_2 - h_g\right)}{\left(h_1 - h_g\right)} \le m$	$Q = C_{vs} C_e \dfrac{2}{3}\sqrt{2g}\, b\left(h_1 - h_g - h_o\right)^{1.5}$ where : $C_e = 0.602 + 0.075\dfrac{\left(h_1 - h_g - h_o\right)}{\left(p_1 + h_g + h_o\right)}$
3	Gate closed, drowned flow over gate.	$h_o \le 0.001$; $\left(h_1 - h_g\right) > 0$; $\dfrac{\left(h_2 - h_g\right)}{\left(h_1 - h_g\right)} > m$	$Q = C_{rf} C_e \dfrac{2}{3}\sqrt{(2g)}\, b\left(h_1 - h_g - h_0\right)^{1.5}$ where : $C_e = 0.602 + 0.075\dfrac{\left(h_1 - h_g - h_o\right)}{\left(p_1 + h_g + h_o\right)}$ $C_{rf} = \left[1 - \dfrac{\left(h_2 - h_g - h_o\right)^{1.5}}{\left(h_1 - h_g - h_o\right)^{1.05}}\right]^{0.385}$
4	Free weir flow under gate.	$h_o \ge 0.001$; $\dfrac{h_2}{h_1} \le m$; $0.005(L-r) < h_1 < 1.5h_o$; $h_2 < h_0$	$Q = C_d C_{vw}\left(2/3\right)^{1.5}\sqrt{g}\, b\left(h_1 - h_2\right)^{1.5}$ where: $C_d = \left[1 - \dfrac{\delta(L-r)}{(hb)}\right]\left[1 - \dfrac{\delta(L-r)}{2h_1}\right]^{1.5}$ $r = 0.1$ *and* $\delta = 0.01$

TABLE 1 : *TEST 1 Part A - Equations For Flow Over or Through a Vertical Sluice After BOS (1989) (Part 1 of 2)*

5	Drowned weir flow under gate.	$h_o \ge 0.001\ ;\ \frac{h_2}{h_1} > m\ ;\ 0.005(L-r) < h_1 < h_o$	$Q = C_d C_{vw} (\frac{2}{3})^{1.5} \sqrt{g}\, b h_1 \left[\frac{(h_1 - h_2)}{(1-m)}\right]^{0.5}$ where : $C_d = \left[1 - \frac{\delta (L-r)}{(hb)}\right]\left[1 - \frac{\delta (L-r)}{2h_1}\right]^{1.5}$ $r = 0.1$ $\delta = 0.01$
6	Free gate flow	$h_o \ge 0.001\ ;\ h_1 \ge 1.5 h_o$ $\frac{h_2}{h_o} < \frac{\alpha}{2}\left(\sqrt{1 + 16\left(\frac{h_1}{(\alpha h_o - 1)}\right)} - 1\right)$	$Q = 0.60 C_{vg} \sqrt{2g}\, b h_o^{1.5} \left[\frac{h_1}{h_o} - \alpha\right]^{0.5}$ where: α = Contraction Coefficient
7	Drowned gate flow.	$h_o \ge 0.001\ ;\ h_1 \ge 1.5 h_o$ $\frac{h_2}{h_o} \ge \frac{\alpha}{2}\left(\sqrt{1 + 16\left(\frac{h_1}{(\alpha h_o - 1)}\right)} - 1\right)$	$Q = C_e C_{vg} b h_o \sqrt{2g} (h_1 - h_2)^{0.5}$ where: $C_e = 0.61\left(1 + 0.15\left(\frac{(b + 2h_o)}{2b + 2h_o}\right)\right)$ α = Contraction Coefficient
8	Free over gate and free under gate flow.	As Mode 6 and ; $h_1 \ge h_g$	Combination of Mode 6 and mode 2 equations.
9	Free over gate and drowned under gate flow.	As Mode 7 and ; $(h_1 - h_g) > 0\ ;\ (h_2 - h_g) \le 0$	Combination of Mode 7 and Mode 2 equations.
10	Drowned over gate and drowned under gate flow.	As Mode 7 and ; $(h_1 - h_g) > 0\ ;\ (h_2 - h_g) > 0$	Combination of Mode 7 and Mode 3 equations.

TABLE 1 : *TEST 1 Part A - Equations for Flow Over or Through a Vertical Sluice After BOS (1989) (Part 2 of 2)*

Where :
L = length of weir (m)
r = radius of curvature for weir crest (m)
h_1 = upstream water depth (m)
h_2 = downstream water depth (m)
h_g = height of gate (m)

h_o = gate opening (m)
m = modular limit (from Figure 4.2 in BOS (1989))
b = breadth (m)
C_d, C_e = discharge coefficients
C_{rf} = drowned flow reduction factor

Q = flow (m^3/s)
α = contraction coefficient
C_{vg}, C_{vs}, C_{vw} = velocity coefficients
g = gravitational acceleration (m/s^2)

MODE 2 - Free Over Gate Flow : Model Discharge 0.15m^3/s

Software	Water Elevation at Structure		Calculated Head Difference (m)	Theoretical Discharge Q (m^3/s)
	U/S (m)	D/S (m)		
FLOODTIDE	N/A			
HEC-RAS	1.244	0.803	0.441	0.212
HEC2	1.250	0.800	0.450	0.220
MIKE 11	1.242	0.803	0.439	0.209
HYDRO 1D	1.175	0.803	0.372	0.126
FLUCOMP	1.210	0.803	0.407	0.168
ISIS	1.200	0.803	0.397	0.155

MODE 3 - Drowned Over Gate Flow : Model Discharge 0.15m^3/s

Software	Water Elevation at Structure		Calculated Head Difference (m)	Theoretical Discharge Q (m^3/s)
	U/S (m)	D/S (m)		
FLOODTIDE	N/A			
HEC-RAS	1.318	1.301	0.017	0.114
HEC2	1.320	1.300	0.020	0.123
MIKE 11	1.317	1.301	0.016	0.112
HYDRO 1D	1.341	1.301	0.040	0.172
FLUCOMP	1.367	1.301	0.066	0.226
ISIS	1.331	1.301	0.030	0.149

MODE 4 - Free Weir Flow : Model Discharge 0.15m^3/s

Software	Water Elevation at Structure		Calculated Head Difference (m)	Theoretical Discharge Q (m^3/s)
	U/S (m)	D/S (m)		
FLOODTIDE	N/A			
HEC-RAS	0.701	0.345	0.356	0.126
HEC2	0.710	0.290	0.411	0.135
MIKE 11	0.749	0.349	0.400	0.177
HYDRO 1D	0.383	0.348	0.035	Not Possible
FLUCOMP	0.713	0.347	0.366	0.138
ISIS	0.713	0.345	0.368	0.138

MODE 5 - Drowned Weir Flow : Model Discharge 0.15m^3/s

Software	Water Elevation at Structure		Calculated Head Difference (m)	Theoretical Discharge Q (m^3/s)
	U/S (m)	D/S (m)		
FLOODTIDE	N/A			
HEC-RAS	0.772	0.764	0.008	0.089
HEC2	0.770	0.760	0.010	0.100
MIKE 11	0.782	0.764	0.018	0.140
HYDRO 1D	0.766	0.764	0.002	0.044
FLUCOMP	0.774	0.764	0.010	0.101
ISIS	0.780	0.764	0.016	0.131

Table 2 : *Comparisons of Results for Test 1 Part A (Part 1 of 3)*

MODE 6 – Free Gate : Model Discharge 0.15m^3/s				
Software	Water Elevation at Structure		Calculated Head Difference (m)	Theoretical Discharge Q (m^3/s)
	U/S (m)	D/S (m)		
FLOODTIDE	N/A			
HEC-RAS	0.809	0.345	0.464	0.112
HEC2	0.780	0.290	0.490	0.104
MIKE 11	0.804	0.349	0.455	0.110
HYDRO 1D	0.489	0.348	0.141	Not Possible
FLUCOMP	0.979	0.347	0.632	0.150
ISIS	0.881	0.345	0.536	0.130

MODE 7 – Drowned Gate Flow : Model Discharge 0.15m^3/s				
Software	Water Elevation at Structure		Calculated Head Difference (m)	Theoretical Discharge Q (m^3/s)
	U/S (m)	D/S (m)		
FLOODTIDE	N/A			
HEC-RAS	1.016	1.002	0.014	0.064
HEC2	1.040	1.000	0.040	0.108
MIKE 11	1.050	1.002	0.048	0.118
HYDRO 1D	1.037	1.002	0.035	0.101
FLUCOMP	1.061	1.002	0.059	0.131
ISIS	1.067	1.002	0.065	0.138

MODE 8 – Free Over Gate, Free Under Gate : Model Discharge 0.15m^3/s				
Software	Water Elevation at Structure		Calculated Head Difference (m)	Theoretical Discharge Q (m^3/s)
	U/S (m)	D/S (m)		
FLOODTIDE	N/A			
HEC-RAS	1.099	0.345	0.754	0.108
HEC2	1.110	0.290	0.820	0.116
MIKE 11	1.124	0.349	0.775	0.126
HYDRO 1D	0.910	0.348	0.562	Not Possible
FLUCOMP	1.158	0.347	0.811	0.154
ISIS	1.142	0.345	0.797	0.140

MODE 9 – Free Over Gate, Drowned Under Gate : Model Discharge 0.15m^3/s				
Software	Water Elevation at Structure		Calculated Head Difference (m)	Theoretical Discharge Q (m^3/s)
	U/S (m)	D/S (m)		
FLOODTIDE	N/A			
HEC-RAS	1.156	0.903	0.253	0.124
HEC2	1.180	0.900	0.280	0.149
MIKE 11*	1.139	0.902	0.237	0.109
HYDRO 1D	1.122	0.903	0.219	0.095
FLUCOMP	1.208	0.902	0.306	0.179
ISIS	1.175	0.902	0.273	0.143

Table 2 : *Comparisons of Results for Test 1 Part A (Part 2 of 3)*

MODE 10 - Drowned Over Gate, Drowned Under Gate : Model Discharge 0.15m^3/s				
Software	Water Elevation at Structure		Calculated Difference (m)	Theoretical Discharge Q (m^3/s)
	U/S (m)	D/S (m)		
FLOODTIDE	N/A			
HEC-RAS	2.023	2.000	0.023	0.118
HEC2	2.000	2.000	0.000	0.000
MIKE 11	2.001	2.000	0.001	0.026
HYDRO 1D	2.020	2.000	0.020	0.109
FLUCOMP	2.056	2.000	0.056	0.194
ISIS	2.030	2.000	0.030	0.136

*Upstream and Downstream cross section 10.0m away from structure and not 1.0m as for all others with MIKE11

Table 2 : *Comparisons of Results for Test 1 Part A (Part 3 of 3)*

Water Elevation Upstream (m)	Water Elevation Downstream (m)	Head Difference (m)	Variation in ΔH (%)	Discharge Q (m3/s)	Variation in Q %
MODE 2 - GATE CLOSED FREE OVER GATE FLOW					
1.180	0.803	0.377	-4.07	0.132	- 12.405
1.190	0.803	0.387	-1.53	0.143	- 4.725
1.196	0.803	0.393	0.00	0.150	0.000
1.210	0.803	0.407	3.56	0.168	11.359
1.220	0.803	0.417	6.11	0.180	19.756
MODE 3 - GATE CLOSED DROWNED OVER GATE FLOW					
1.310	1.301	0.009	-70.00	0.088	-41.322
1.320	1.301	0.019	-36.67	0.121	-19.085
1.331	1.301	0.030	0.00	0.149	0.000
1.340	1.301	0.039	30.00	0.170	13.836
1.350	1.301	0.049	63.33	0.191	28.199
MODE 4 - FREE WEIR UNDER GATE FLOW					
0.710	0.345	0.365	-4.20	0.134	-10.429
0.720	0.345	0.375	-1.57	0.144	-3.956
0.726	0.345	0.381	0.00	0.150	0.000
0.730	0.345	0.385	1.05	0.154	2.667
0.740	0.345	0.395	3.67	0.164	9.434
MODE 5 - DROWNED WEIR UNDER GATE FLOW					
0.770	0.760	0.010	-52.15	0.100	-33.692
0.780	0.760	0.020	-4.31	0.147	-2.511
0.781	0.760	0.021	0.00	0.150	0.000
0.790	0.760	0.030	43.54	0.186	23.950
0.800	0.760	0.040	91.39	0.223	48.380
MODE 6 - FREE GATE FLOW					
0.900	0.345	0.555	-11.90	0.134	-10.557
0.950	0.345	0.605	-3.97	0.145	-3.391
0.975	0.345	0.630	0.00	0.150	0.000
1.000	0.345	0.655	3.97	0.155	3.280
1.050	0.345	0.705	11.90	0.164	9.545

TABLE 3 : *TEST 1 Part A - Sensitivity of Head Difference on the Calculated Discharge (Part 1 of 2)*

Water Elevation Upstream (m)	Water Elevation Downstream (m)	Head Difference (m)	Variation in ΔH (%)	Discharge Q (m3/s)	Error in Q %
MODE 7 - DROWNED GATE FLOW					
1.060	1.002	0.058	-24.68	0.130	-13.210
1.070	1.002	0.068	-11.69	0.141	-6.026
1.079	1.002	0.077	0.00	0.150	0.000
1.090	1.002	0.088	14.29	0.160	6.904
1.100	1.002	0.098	27.27	0.169	12.815
MODE 8 - FREE OVER GATE AND FREE UNDER GATE FLOW					
1.130	0.345	0.785	-2.85	0.130	-12.833
1.140	0.345	0.795	-1.61	0.138	-7.437
1.153	0.345	0.808	0.00	0.150	0.000
1.160	0.345	0.815	0.87	0.156	4.193
1.170	0.345	0.825	2.10	0.165	10.404
MODE 9 - FREE OVER GATE AND DROWNED UNDER GATE FLOW					
1.160	0.901	0.259	-7.50	0.129	-14.049
1.170	0.901	0.269	-3.93	0.138	-7.490
1.181	0.901	0.280	0.00	0.150	0.000
1.190	0.901	0.289	3.21	0.159	6.335
1.200	0.901	0.299	6.79	0.170	13.586
MODE 10 - DROWNED OVER GATE AND DROWNED UNDER GATE FLOW					
2.020	2.000	0.020	-44.44	0.109	-27.201
2.030	2.000	0.030	-16.67	0.136	-9.612
2.036	2.000	0.036	0.00	0.150	0.000
2.040	2.000	0.040	11.11	0.160	6.147
2.050	2.000	0.050	38.89	0.182	20.860

TABLE 3 : *TEST 1 Part A - Sensitivity of Head Difference on the Calculated Discharge (Part 2 of 2)*

Calculated Critical Backwater (Free Flow Positive)						
NODE	**FLOW m^3/s**	**STAGE m**	**VELOCITY m/s**	**FROUDE No.**	**AREA m^2**	**TOP WIDTH m**
A1	10.000	1.901	2.758	0.959	3.625	4.301
A2	9.997	1.775	2.758	0.959	3.624	4.301
A3	9.993	1.650	2.758	0.959	3.623	4.300
A4	-54768.379	-42939.172	*******	********	0.210	2.200
A5	-54689.715	-42877.621	*******	********	0.210	2.200
A6	-54611.160	-42816.176	*******	********	0.210	2.200
A7	-54546.098	1.547	-9940.321	-3057.833	5.487	5.094
A→B	-657.948	6.821	0.000	0.000	0.000	0.000
B1	5.000	1.518	2.351	0.968	2.126	3.536
B2	4.998	1.393	2.351	0.968	2.126	3.536
B3	4.996	1.268	2.351	0.968	2.126	3.536
B4	54833.977	12.375	806.382	76.476	68.000	6.000
B5	548333.949	1.850	9519.342	2887.705	5.760	5.200

Table 4 : *FLOODTIDE - TEST 1 Part B - Calculated Critical Backwater*

Modified Critical Backwater "A" (Free Flow Positive)						
NODE	**FLOW m^3/s**	**STAGE m**	**VELOCITY m/s**	**FROUDE No.**	**AREA m^2**	**TOP WIDTH m**
A1	10.000	1.874	2.848	1.000	3.511	4.248
A2	10.000	1.749	2.848	1.000	3.511	4.248
A3	10.000	1.624	2.848	1.000	3.511	4.248
A4	9.000	1.434	2.778	1.000	3.239	4.118
A5	9.000	1.309	2.778	1.000	3.239	4.118
A6	9.000	1.184	2.778	1.000	3.239	4.118
A7	9.000	1.547	1.640	0.255	5.487	5.094
A→B	1.000	1.529	0.000	0.000	0.000	0.000
B1	5.000	1.504	2.408	1.000	2.077	3.508
B2	5.000	1.379	2.408	1.000	2.077	3.508
B3	5.000	1.254	2.408	1.000	2.077	3.508
B4	6.000	1.214	2.519	1.000	2.077	3.678
B5	6.000	1.850	1.042	0.100	5.760	5.200

Table 5 : *FLOODTIDE - TEST 1 Part B - Modified Critical Backwater "A"*

Modified Critical Backwater “B” (Free Flow Positive)						
NODE	FLOW m^3/s	STAGE m	VELOCITY m/s	FROUDE No.	AREA m^2	TOP WIDTH m
A1	10.000	2.200	1.999	0.3999	5.003	4.900
A2	10.000	2.100	1.951	0.375	5.126	4.950
A3	10.000	1.980	1.942	0.370	5.150	4.960
A4	9.000	1.860	1.739	0.296	5.175	4.970
A5	9.000	1.740	1.731	0.293	5.200	4.980
A6	9.000	1.620	1.722	0.289	5.225	4.990
A7	9.000	1.547	1.640	0.255	5.487	5.094
A→B	1.000	1.950	0.001	1.000	0.800	40.000
B1	5.000	1.950	1.302	0.198	3.840	4.400
B2	5.000	1.925	1.166	0.149	4.290	4.600
B3	5.000	1.900	1.050	0.113	4.760	4.800
B4	6.000	1.875	1.143	0.127	5.250	5.000
B5	6.000	1.850	1.042	0.100	5.760	5.200

Table 6 : *FLOODTIDE - TEST 1 Part B - Modified Critical Backwater "B"*

Backwater Results (Free Flow Positive)						
NODE	FLOW m^3/s	STAGE m	VELOCITY m/s	FROUDE No.	AREA m^2	TOP WIDTH m
A1	10.000	2.195	2.007	*	4.983	4.900
A2	10.000	2.071	2.006	*	4.985	4.950
A3	10.000	1.948	2.004	*	4.991	4.960
A4	9.900	1.834	1.783	*	5.047	4.970
A5	9.000	1.733	1.743	*	5.165	4.980
A6	9.000	1.637	1.695	*	5.311	4.990
A7	9.000	1.547	1.640	*	5.487	5.094
A→B	1.000	*	*	*	*	*
B1	5.000	2.035	1.183	*	4.225	4.400
B2	5.000	1.981	1.099	*	4.549	4.600
B3	5.000	1.396	1.014	*	4.932	4.800
B4	6.000	1.892	1.124	*	5.337	5.000
B5	6.000	1.850	1.042	*	5.760	5.200

* same value returned as given in modified critical backwater.

Table 7 : *FLOODTIDE - TEST 1 Part B - Backwater Results*

FLOODTIDE	Free Flow : Positive			Drowned Flow : Positive		
NODE	FLOW m^3/s	STAGE m	VELOCITY m/s	FLOW m^3/s	STAGE m	VELOCITY m/s
A1	10.00	2.19	2.02	10.00	2.19	2.02
A2	10.00	2.06	2.02	10.00	2.07	2.02
A3	10.00	1.94	2.02	10.00	1.94	2.01
A4	9.91	1.83	1.96	9.94	1.83	1.97
A5	9.91	1.73	1.93	9.94	1.73	1.93
A6	9.91	1.63	1.88	9.94	1.63	1.88
A7	9.91	1.55	1.82	9.94	1.55	1.81
A→B	0.09	1.88	0.00	0.06	1.89	0.00
B1	5.00	1.95	1.30	7.00	2.04	1.65
B2	5.00	1.92	1.18	7.00	1.97	1.56
B3	5.00	1.89	1.06	7.00	1.91	1.45
B4	5.09	1.86	0.98	7.06	1.87	1.36
B5	5.09	1.85	0.88	7.06	1.84	1.24

Table 8 : *FLOODTIDE TEST 1B - Free and Drowned Flow (Positive Sense)*

(Part 1 of 2)

FLOODTIDE	Free Flow : Negative			Drowned Flow : Negative		
NODE	FLOW m^3/s	STAGE m	VELOCITY m/s	FLOW m^3/s	STAGE m	VELOCITY m/s
A1	5.00	1.84	1.48	3.50	1.96	0.90
A2	5.00	1.78	1.38	3.50	1.94	0.80
A3	5.00	1.73	1.26	3.50	1.93	0.71
A4	5.52	1.64	1.34	3.82	1.91	0.70
A5	5.52	1.60	1.22	3.82	1.91	0.63
A6	5.52	1.57	1.11	3.82	1.90	0.57
A7	5.52	1.55	1.00	3.82	1.90	0.52
A→B	-0.52	1.92	0.00	-0.32	1.93	0.00
B1	7.00	2.04	1.65	7.00	2.05	1.63
B2	7.00	1.97	1.55	7.00	1.99	1.53
B3	7.00	1.92	1.44	7.00	1.94	1.42
B4	6.48	1.92	1.18	6.68	1.92	1.22
B5	6.48	1.90	1.08	6.68	1.90	1.11

Table 8 : *FLOODTIDE TEST 1B - Free and Drowned Flow (Negative Sense)*

(Part 2 of 2)

	Free Flow : Positive					
MIKE 11	Steady Calculation			Unsteady Calculation		
NODE	FLOW m³/s	STAGE m	VELOCITY m/s	FLOW m³/s	STAGE m	VELOCITY m/s
A1	10.00	2.287	1.840	10.000	2.286	1.83.9
A2	10.00	2.151	1.866	10.000	2.152	1.865
A3	10.00	2.013	1.881	10.000	2.015	1.878
A4	9.201	1.884	1.744	9.180	1.883	1.743
A5	9.201	1.769	1.720	9.180	1.768	1.718
A6	9.201	1.656	1.708	9.180	1.656	1.706
A7	9.201	1.550	1.668	9.180	1.550	1.666
A→B	0.809	*	*	0.820	*	*
B1	5.000	2.115	1.085	5.000	2.115	1.085
B2	5.000	2.076	1.003	5.000	2.076	1.002
B3	5.000	2.014	2.178	5.000	2.014	2.177
B4	5.809	1.884	1.102	5.820	1.884	1.104
B5	5.809	1.850	1.006	5.820	1.850	1.008

Table 9 : *MIKE 11 TEST 1B - Free Flow (Positive Sense)*

(Part 1 of 4)

	Drowned Flow : Positive					
MIKE 11	Steady Calculation			Unsteady Calculation		
NODE	FLOW m³/s	STAGE m	VELOCITY m/s	FLOW m³/s	STAGE m	VELOCITY m/s
A1	10.000	2.287	1.839	10.000	2.288	1.836
A2	10.000	2.152	1.865	10.000	2.154	1.861
A3	10.000	2.014	1.879	10.000	2.017	1.873
A4	9.210	1.885	1.746	9.235	1.886	1.748
A5	9.220	1.770	1.723	9.235	1.771	1.724
A6	9.220	1.657	1.710	9.235	1.657	1.714
A7	9.210	1.550	1.671	9.235	1.550	1.676
A→B	0.790	*	*	0.765	*	
B1	7.000	2.254	1.328	7.000	2.253	1.329
B2	7.000	2.196	1.254	7.000	2.196	1.254
B3	7.000	2.129	2.623	7.000	2.128	2.625
B4	7.790	1.899	1.467	7.765	1.898	1.453
B5	7.790	1.830	1.374	7.765	1.830	1.370

Table 9 : *MIKE 11 TEST 1B - Drowned Flow (Positive Sense)*

(Part 2 of 4)

	Free Flow : Negative					
MIKE 11	Steady Calculation			Unsteady Calculation		
NODE	FLOW m^3/s	STAGE m	VELOCITY m/s	FLOW m^3/s	STAGE m	VELOCITY m/s
A1	5.000	1.921	1.341	5.000	1.921	1.341
A2	5.000	1.841	1.283	5.000	1.841	1.283
A3	5.000	1.775	1.197	5.000	1.774	1.198
A4	5.912	1.709	1.333	5.912	1.709	1.333
A5	5.911	1.644	1.245	5.912	1.644	1.245
A6	5.911	1.591	1.167	5.912	1.591	1.167
A7	5.912	1.550	1.073	5.912	1.550	1.073
A→B	-0.912	*	*	-0.912	*	*
B1	7.000	2.261	1.320	7.000	2.260	1.320
B2	7.000	2.204	1.245	7.000	2.204	1.245
B3	7.000	2.138	2.594	7.000	2.138	2.594
B4	6.088	1.933	1.102	6.088	1.933	1.102
B5	6.088	1.900	1.008	6.088	1.900	1.008

Table 9 : *MIKE 11 TEST 1B - Free Flow (Negative Sense)*

(Part 3 of 4)

	Drowned Flow : Negative					
MIKE 11	Steady Calculation			Unsteady Calculation		
NODE	FLOW m^3/s	STAGE m	VELOCITY m/s	FLOW m^3/s	STAGE m	VELOCITY m/s
A1	3.500	1.996	0.865	3.500	1.996	0.865
A2	3.500	1.971	0.779	3.500	1.971	0.779
A3	3.500	1.953	0.696	3.500	1.954	0.696
A4	4.410	1.937	0.795	4.412	1.937	0.796
A5	4.408	1.921	0.717	4.412	1.921	0.718
A6	4.408	1.909	0.655	4.412	1.909	0.656
A7	4.410	1.900	0.594	4.412	1.900	0.594
A→B	-0.910	*	*	-0.912	*	*
B1	7.000	2.261	1.320	7.000	2.260	1.320
B2	7.000	2.204	1.245	7.000	2.204	1.245
B3	7.000	2.138	2.594	7.000	2.138	2.594
B4	6.090	1.933	1.102	6.088	1.933	1.102
B5	6.090	1.900	1.009	6.088	1.900	1.008

Table 9 : *MIKE 11 TEST 1B - Drowned Flow (Negative Sense)*

(Part 4 of 4)

HYDRO-1D	Free Flow : Positive			Drowned Flow : Positive		
NODE	FLOW m^3/s	STAGE m	VELOCITY m/s	FLOW m^3/s	STAGE m	VELOCITY m/s
A1	10.0	2.284	1.845	10.0	2.288	1.838
A2	10.0	2.153	1.855	10.0	2.159	1.845
A3	10.0	2.009	1.889	10.0	2.021	1.867
A4	9.1	1.869	1.743	9.4	1.883	1.777
A5	9.1	1.756	1.723	9.4	1.767	1.762
A6	9.1	1.651	1.691	9.4	1.658	1.736
A7	9.1	1.550	1.654	9.4	1.550	1.708
A→B	0.9	*	*	0.6	*	*
B1	5.0	2.012	1.215	7.0	2.123	1.512
B2	5.0	1.959	1.124	7.0	2.037	1.453
B3	5.0	1.920	1.030	7.0	1.966	1.378
B4	5.9	1.885	1.113	7.6	1.906	1.410
B5	5.9	1.850	1.024	7.6	1.840	1.331

NOTE : Velocity calculated by hand

Table 10 : *HYDRO-1D TEST 1B - Free and Drowned Flow (Positive Sense)*

(Part 1 of 2)

HYDRO-1D	Free Flow : Negative			Drowned Flow : Negative		
NODE	FLOW m^3/s	STAGE m	VELOCITY m/s	FLOW m^3/s	STAGE m	VELOCITY m/s
A1	5.0	1.915	1.356	3.5	1.989	1.246
A2	5.0	1.841	1.279	3.5	1.965	1.117
A3	5.0	1.779	1.192	3.5	1.948	1.001
A4	6.2	1.711	1.391	4.2	1.932	1.119
A5	6.2	1.645	1.309	4.2	1.918	1.013
A6	6.2	1.593	1.218	4.2	1.908	0.919
A7	6.2	1.550	1.127	4.2	1.900	0.837
A→B	-1.2	*	*	-0.7	*	*
B1	7.0	2.129	1.502	7.0	2.133	1.496
B2	7.0	2.045	1.441	7.0	2.051	1.433
B3	7.0	1.976	1.364	7.0	1.983	1.355
B4	5.8	1.930	1.049	6.3	1.935	1.044
B5	5.8	1.900	0.963	6.3	1.900	0.963

NOTE : Velocity calculated by hand

Table 10 : *HYDRO -1D TEST 1B - Free and Drowned Flow Part (Negative Sense)*

(Part 2 of 2)

ISIS	Free Flow : Positive			Drowned Flow : Positive		
NODE	FLOW m^3/s	STAGE m	VELOCITY m/s	FLOW m^3/s	STAGE m	VELOCITY m/s
A1	10.000	2.270	1.860	10.00	2.273	1.856
A2	10.000	2.131	1.884	10.00	2.134	1.878
A3	10.000	1.981	1.927	10.00	1.987	1.916
A4	9.533	1.893	1.777	9.577	1.895	1.781
A5	9.533	1.774	1.767	9.577	1.776	1.772
A6	9.533	1.659	1.752	9.577	1.660	1.758
A7	9.533	1.547	1.732	9.577	1.547	1.739
A→B	0.467	*	*	0.424	*	*
B1	5.00	2.016	1.203	7.000	2.125	1.496
B2	5.00	1.968	1.104	7.000	2.047	1.427
B3	5.00	1.932	1.010	7.000	1.981	1.349
B4	5.467	1.879	1.032	7.424	1.893	1.384
B5	5.467	1.850	1.850	7.424	1.835	1.305

Table 11 : *ISIS TEST 1B - Free and Drowned Flow (Positive Sense)*

(Part 1 of 2)

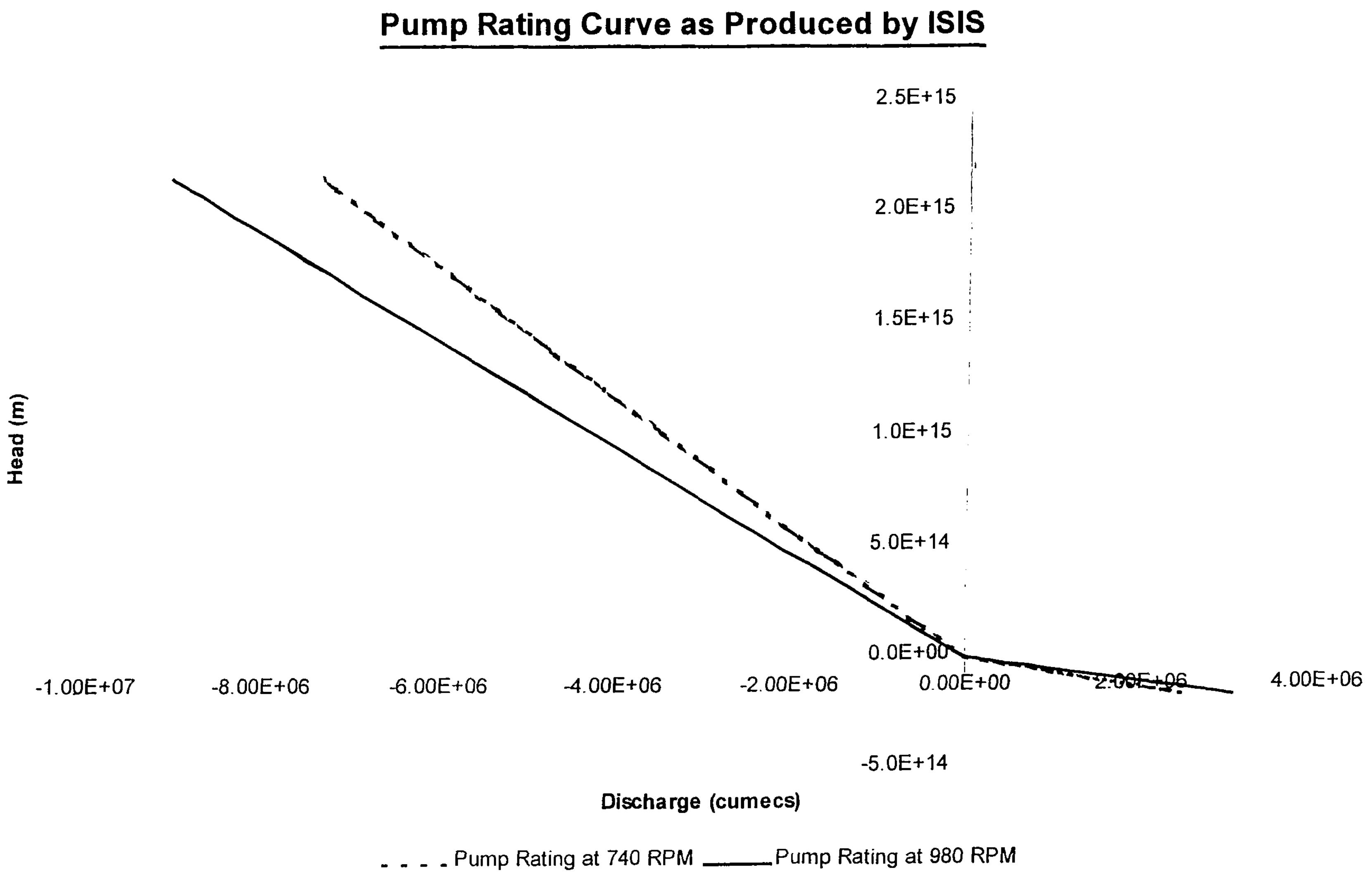
ISIS	Free Flow : Negative			Drowned Flow : Negative		
NODE	FLOW m^3/s	STAGE m	VELOCITY m/s	FLOW m^3/s	STAGE m	VELOCITY m/s
A1	5.000	1.913	1.354	3.500	1.995	0.863
A2	5.000	1.837	1.283	3.500	1.973	0.769
A3	5.000	1.777	1.188	3.500	1.957	0.690
A4	5.562	1.680	1.280	3.996	1.929	0.722
A5	5.562	1.626	1.187	3.996	1.916	0.652
A6	5.562	1.584	1.095	3.996	1.907	0.589
A7	5.562	1.550	1.008	3.996	1.900	0.537
A→B	-0.562	*	*	-0.496	*	*
B1	7.000	2.109	1.520	7.000	2.112	1.516
B2	7.000	2.026	1.457	7.000	2.030	1.451
B3	7.000	1.955	1.382	7.000	1.960	1.376
B4	6.438	1.935	1.156	6.503	1.936	1.167
B5	6.438	1.900	1.066	6.503	1.900	1.077

Table 11 : *ISIS TEST 1B - Free and Drowned Flow (Negative Sense)*

(Part 2 of 2)

Free Flow (Positive sense)	**Drowned Flow (Positive sense)**
Condition : $\frac{y_{21}+y_{22}}{y_{11}+y_{12}} < m$	Condition : $\frac{y_{21}+y_{22}}{y_{11}+y_{12}} < m$
Equation : $q_s = \frac{2C_d b\left(y_{12}^2\sqrt{y_{12}} - y_{11}^2\sqrt{y_{11}}\right)}{5\left(y_{12}-y_{11}\right)}$	Equation : $q_s = Ab\left\{\left(\frac{2}{3}\right)y_k D - \left(\frac{4}{15}\right)y_i\left(dy_{22}+dy_{12}\right)\right\}$
Special Case : When water surface nearly parallel to bank $y_{12} \approx y_{11}$ $q_s = C_b b.y_{11}\sqrt{y_{11}}$	Special Cases : 1. If $y_{12} - y_{11} = y_{22} - y_{21}$, then $q_s = -\frac{1}{2}Aby_k^2\left(y_{11}+y_{12}\right)\sqrt{\left(y_{11}-y_{12}\right)}$ If $y_{11} - y_{21} << y_{12} - y_{11} - y_{22} + y_{21}$, then $q_s = -2Aby_k^{\frac{5}{2}}\left\{\frac{y_{11}}{3} + \frac{\left(y_{12}-y_{11}\right)}{5}\right\}$ 2. If $y_{12} - y_{21} << y_{12} - y_{11} - y_{22} + y_{21}$, then $q_s = -\frac{1}{2}Ay_k^2\left(2y_{11} + y_i + \frac{y_k y_{11}}{y_m}\right)\sqrt{\left(y_{11}-y_{21}\right)}$
Where : y_{11} = upstream water depth in channel 1 y_{12} = downstream water depth in channel 1 y_{21} = upstream water depth in channel 2 y_{22} = downstream water depth in channel 2 m= modular limit b = width of spill section	Where : $y_k = y_{12} - y_{11} - y_{22} + y_{21}$ $y_m = y_{12} - y_{21}$
Free Flow (Negative Sense)	**Drowned Flow (Negative Sense)**
The same formulas apply as for positive sense but with y_{21} interchanged with y_{11}, and y_{12} interchanged with y_{22}.	The same formulae apply as for positive flow but with y_{21} and y_{11} interchanged, and y_{12} and y_{22} interchanged as above.

TABLE 12 : *ISIS Test 1 Part B - Equations of flow used by Spill Unit*

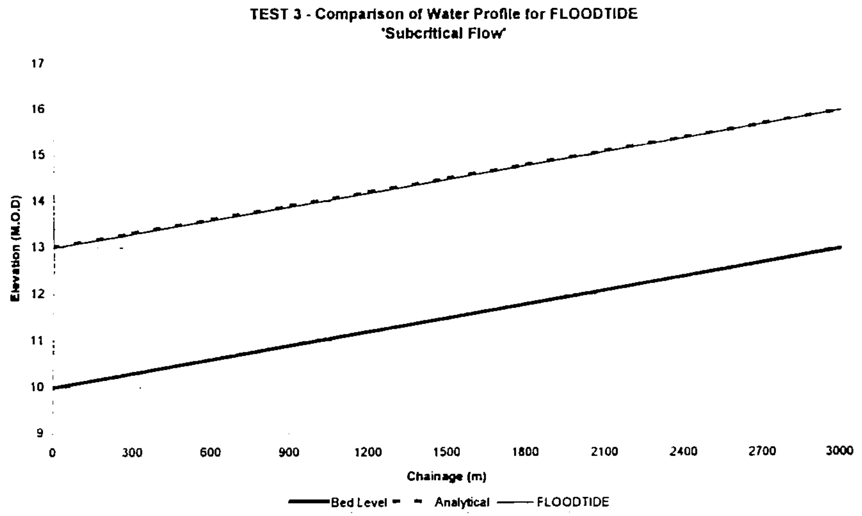


GRAPH 1 : *TEST 1C - ISIS Pump Rating Curve*

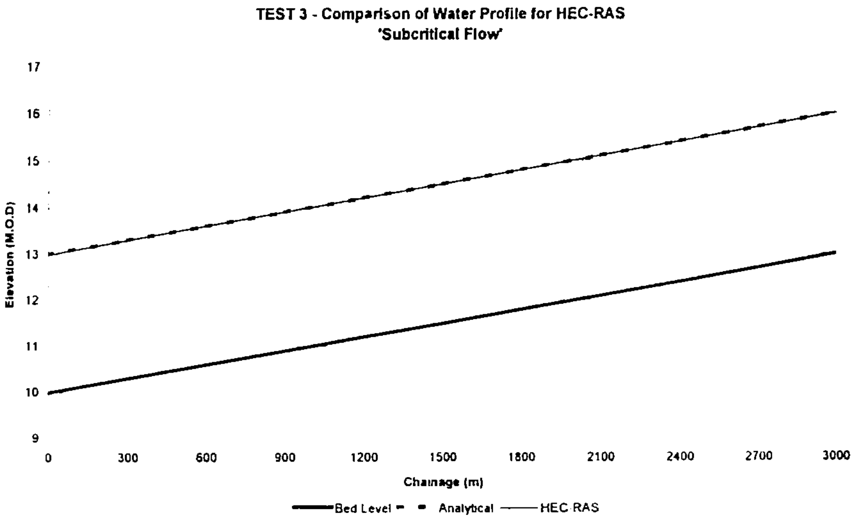
	Water Level at Node :			
	A1	A2	A3	A4
SOFTWARE	**Zero Flow Mode**			
MIKE 11	0.400	0.400	0.400	0.400
HYDRO-1D	0.400	0.400	0.400	0.400
ISIS	0.400	0.400	0.400	0.400
SOFTWARE	**Free Flow Mode**			
FLOODTIDE	0.696	0.692	0.330	0.300
HEC-RAS	0.719	0.713	0.346	0.300
HEC-2	0.400	0.350	0.350	0.300
MIKE 11	0.770	0.766	0.345	0.300
HYDRO 1D	0.686	0.680	0.348	0.300
FLUCOMP	0.716	0.710	0.347	0.300
ISIS	0.694	0.688	0.345	0.300
CHANNEL	1.716	1.716	0.346	0.300
LD01	0.743	0.738	0.346	0.300
GLC-BACKWATER	0.700	0.690	0.350	0.300
BAKWATER	0.700	0.700	0.350	0.300
SOFTWARE	**Drowned Flow Mode**			
FLOODTIDE	0.696	0.692	0.673	0.670
HEC-RAS	0.760	0.755	0.675	0.670
HEC-2	0.680	0.680	0.680	0.670
MIKE 11	0.742	0.737	0.675	0.670
HYDRO 1D	0.741	0.736	0.676	0.670
FLUCOMP	0.718	0.712	0.675	0.670
ISIS	0.702	0.697	0.675	0.670
CHANNEL	1.716	1.716	0.676	0.670
LD01	0.745	0.711	0.675	0.670
GLC-BACKWATER	0.710	0.710	0.680	0.670
BAKWATER	0.700	0.700	0.670	0.670

Table 13 : *TEST 1D - Results for Flow over a Weir.*

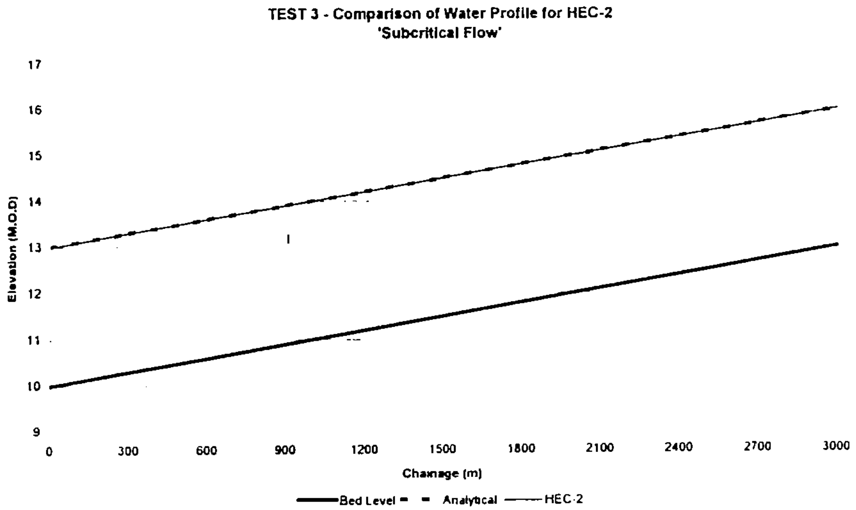
Appendix B



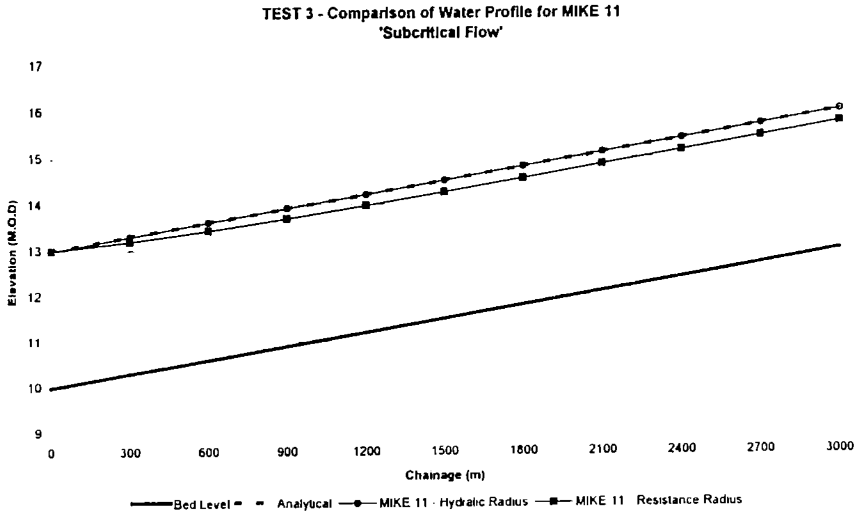
GRAPH 1 : *Test 3 – FLOODTIDE*



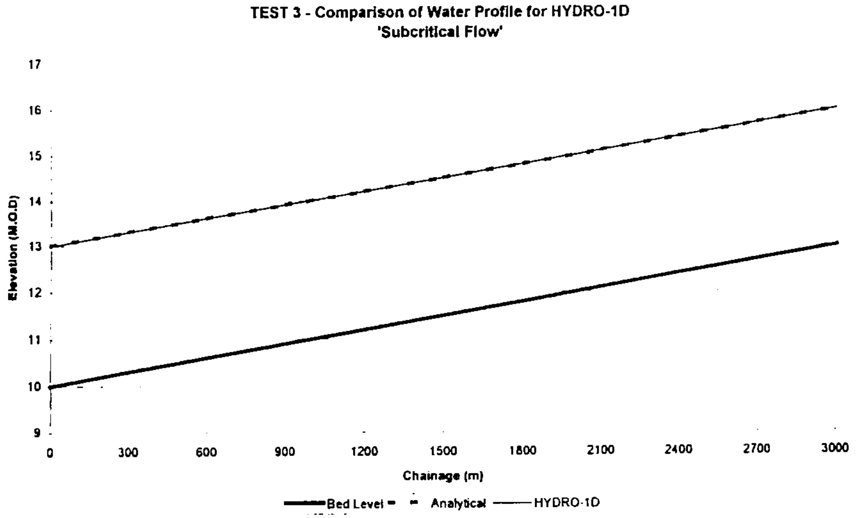
GRAPH 2 : *Test 3 -- HEC-RAS*



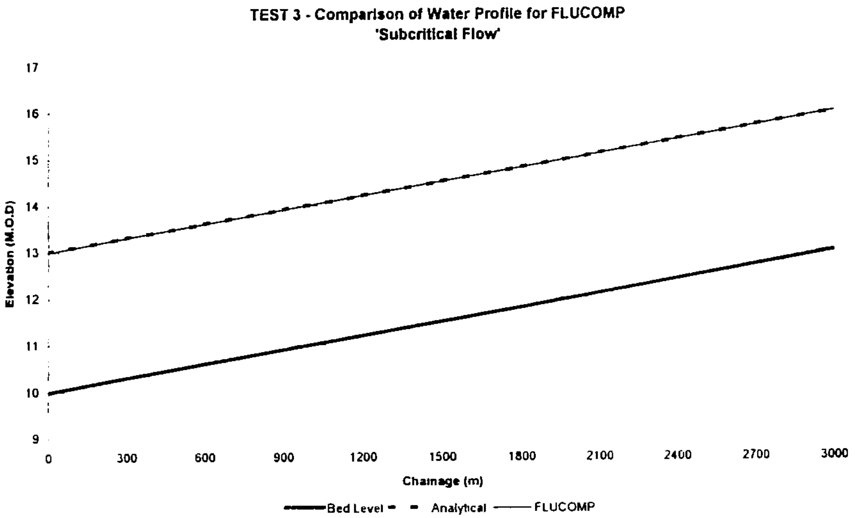
GRAPH 3 : *Test 3 – HEC-2*



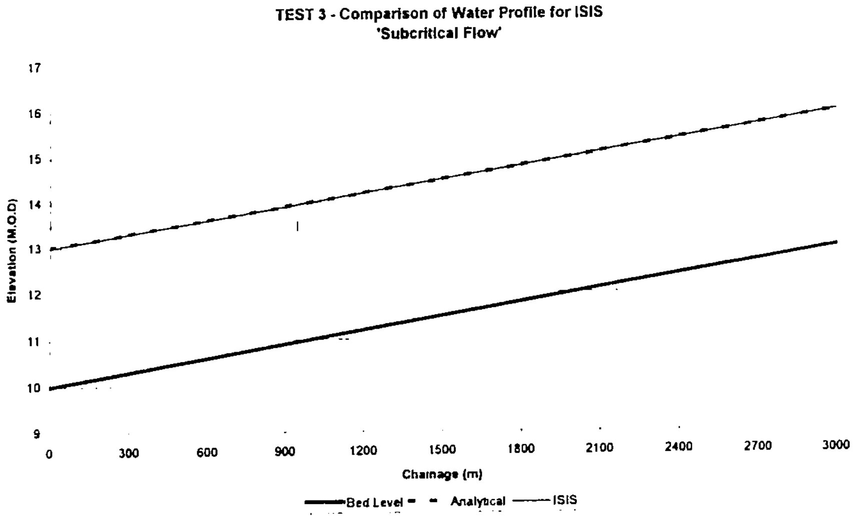
GRAPH 4 : *Test 3 – MIKE 11*



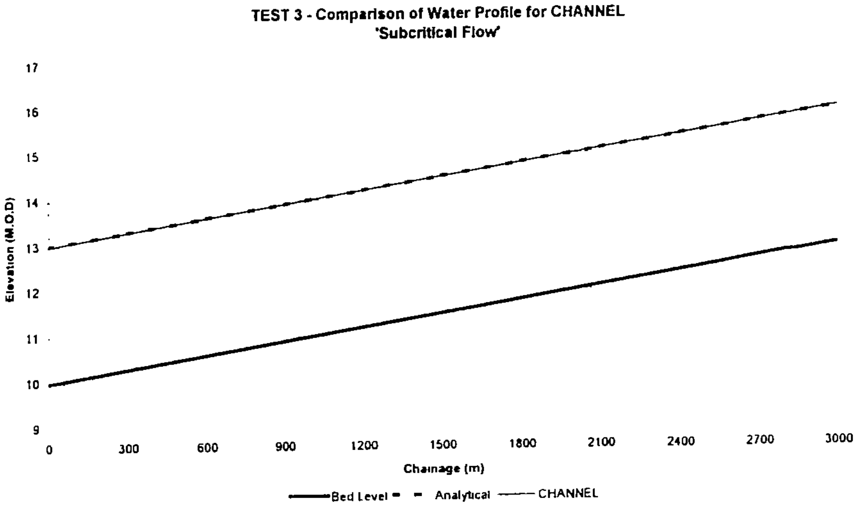
GRAPH 5 : *Test 3 – HYDRO-1D*



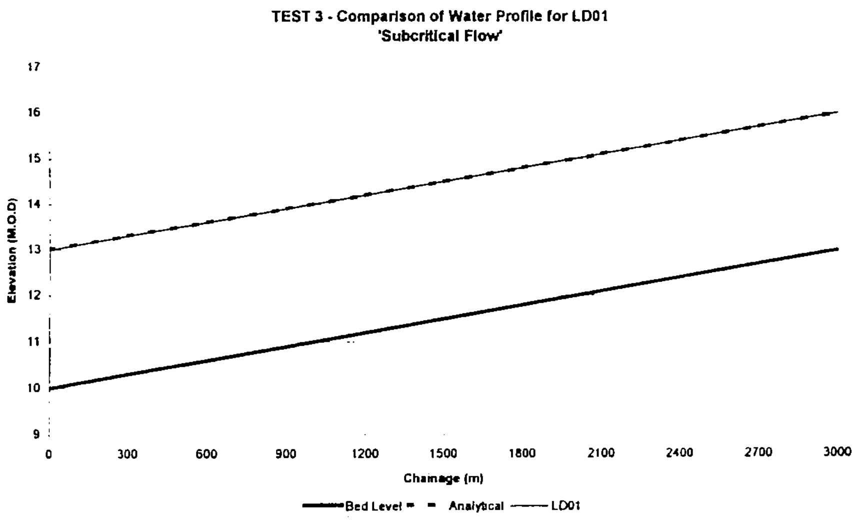
GRAPH 6 : *Test 3 - FLUCOMP*



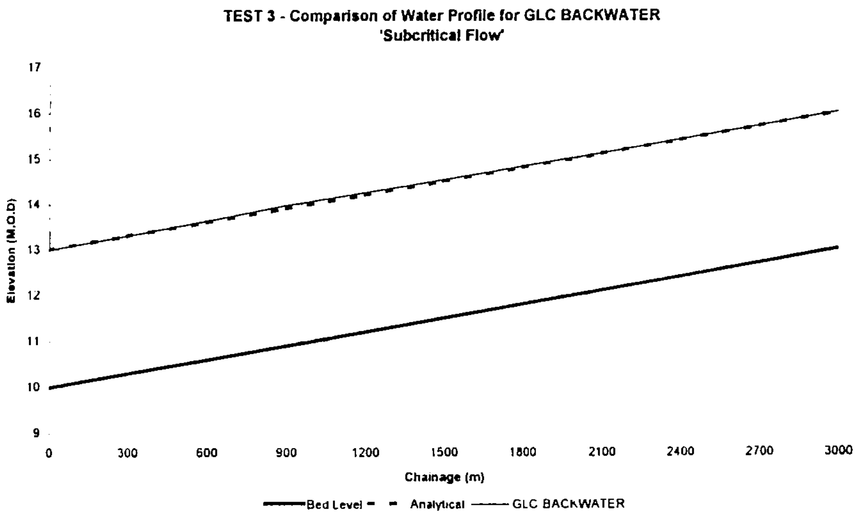
GRAPH 7 : *Test 3 – ISIS*



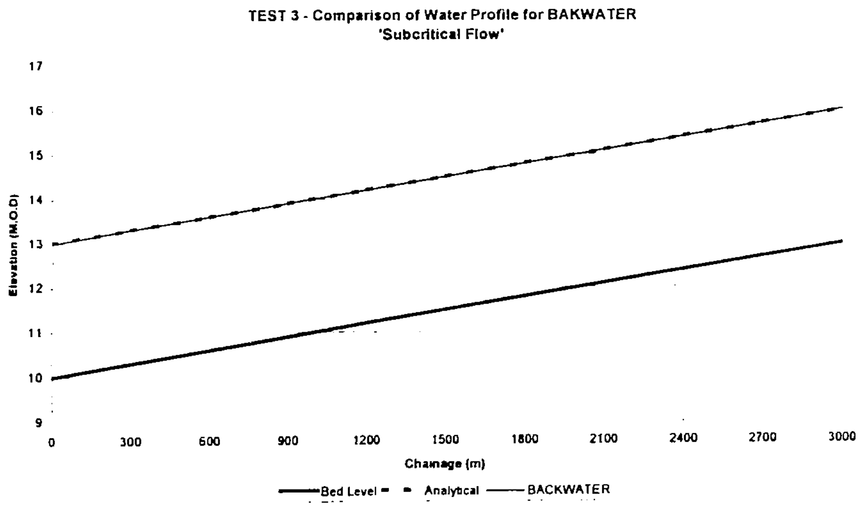
GRAPH 8 : *Test 3 – CHANNEL*



GRAPH 9 : *Test 3 – LD01*

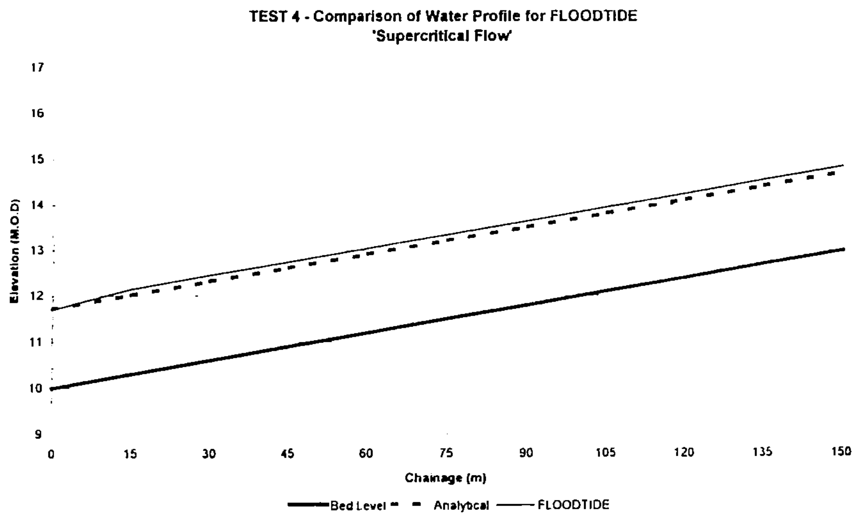


GRAPH 10 : *Test 3 – GLC BACKWATER*

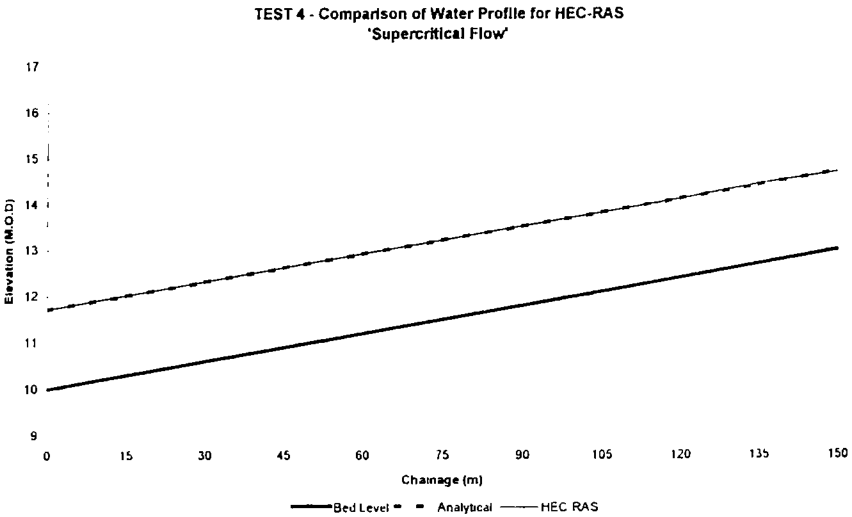


GRAPH 11 : *Test 3 - BAKWATER*

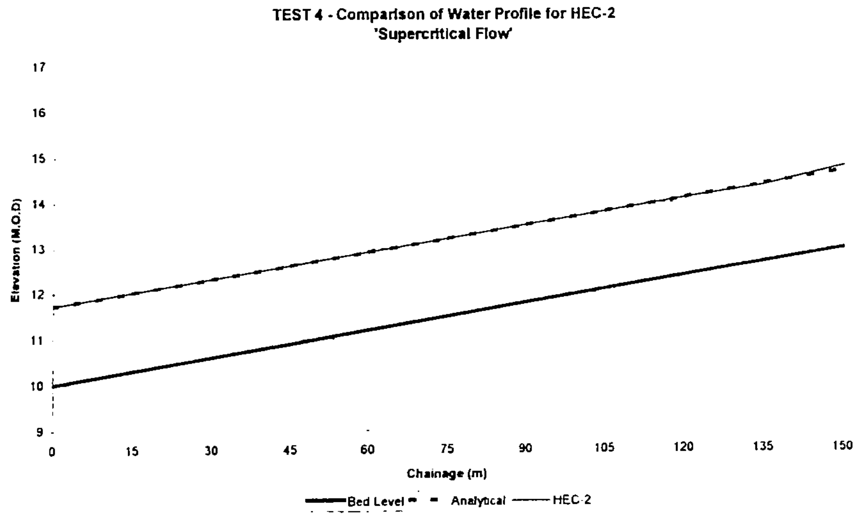
Appendix C



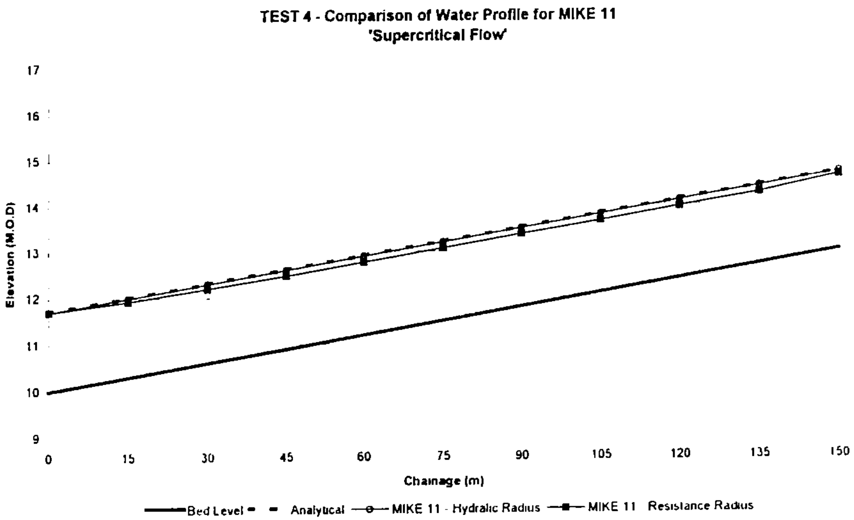
GRAPH 1 : *Test 4 – FLOODTIDE*



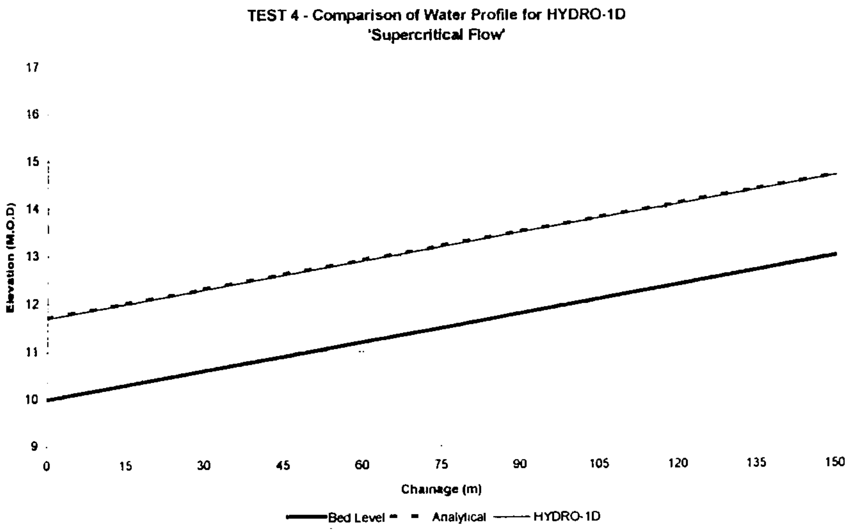
GRAPH 2 : *Test 4 - HEC-RAS*



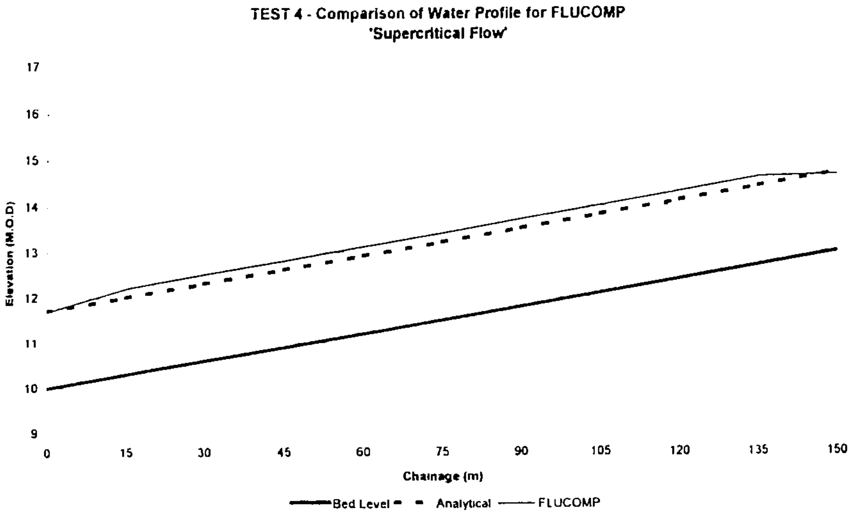
GRAPH 3 : *Test 4 - HEC-2*



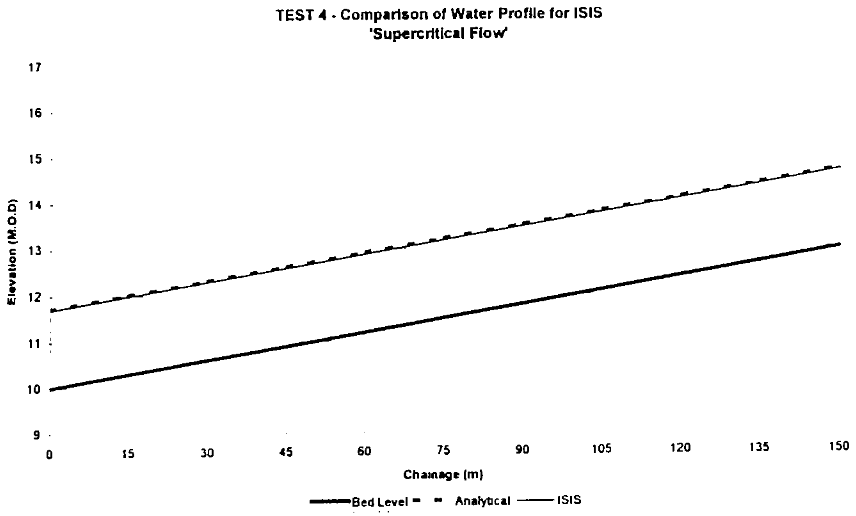
GRAPH 4 : *Test 4 – MIKE 11*



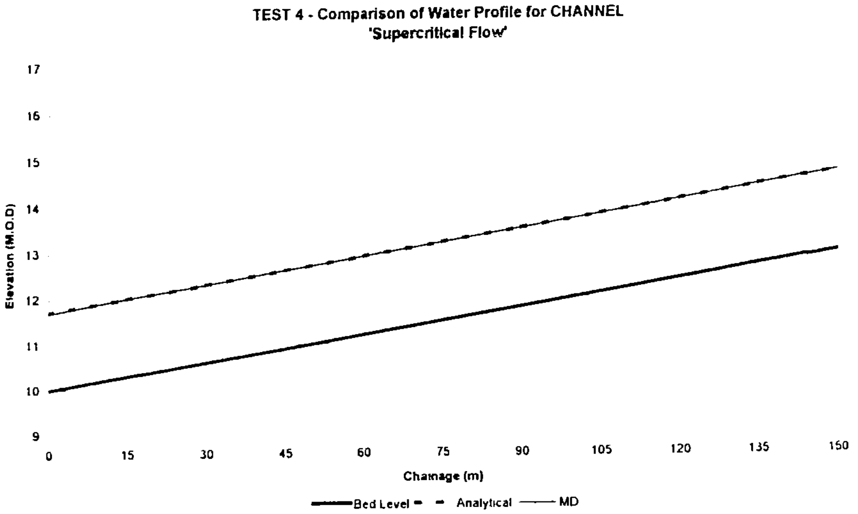
GRAPH 5 : *Test 4 -- HYDRO-1D*



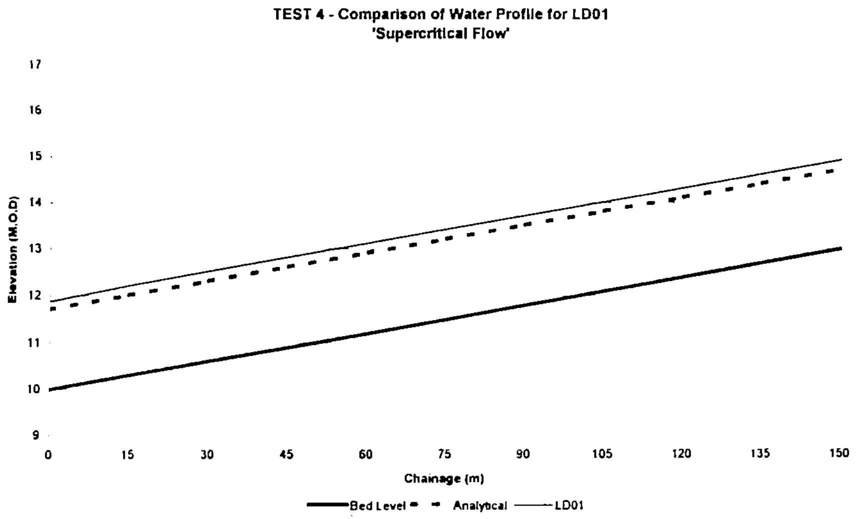
GRAPH 6 : *Test 4 - FLUCOMP*



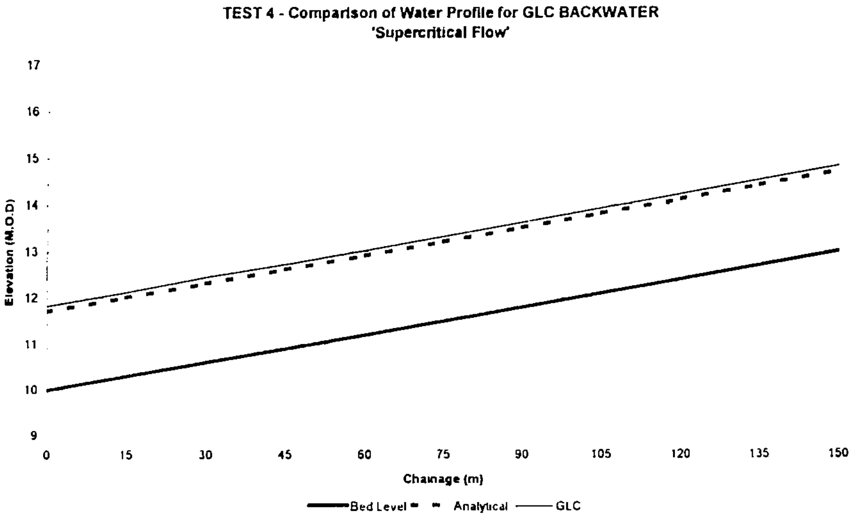
GRAPH 7 : *Test 4 - ISIS*



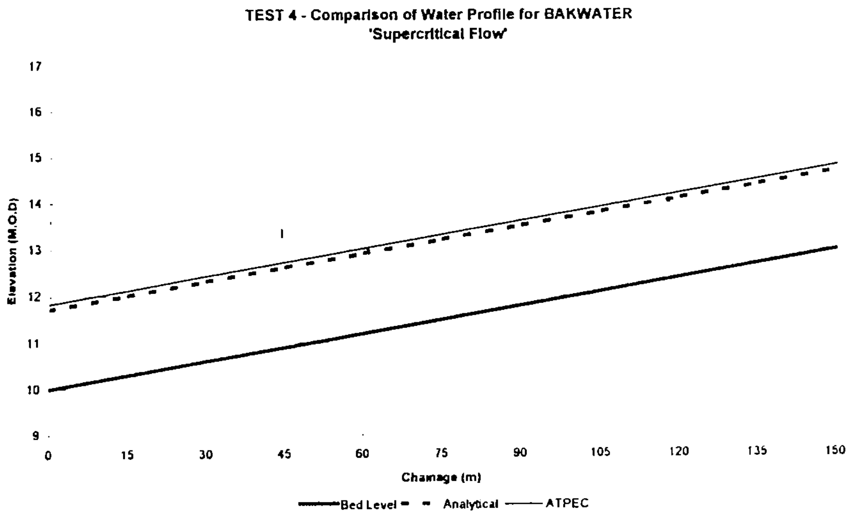
GRAPH 8 : *Test 4 - CHANNEL*



GRAPH 9 : *Test 4 – LD01*

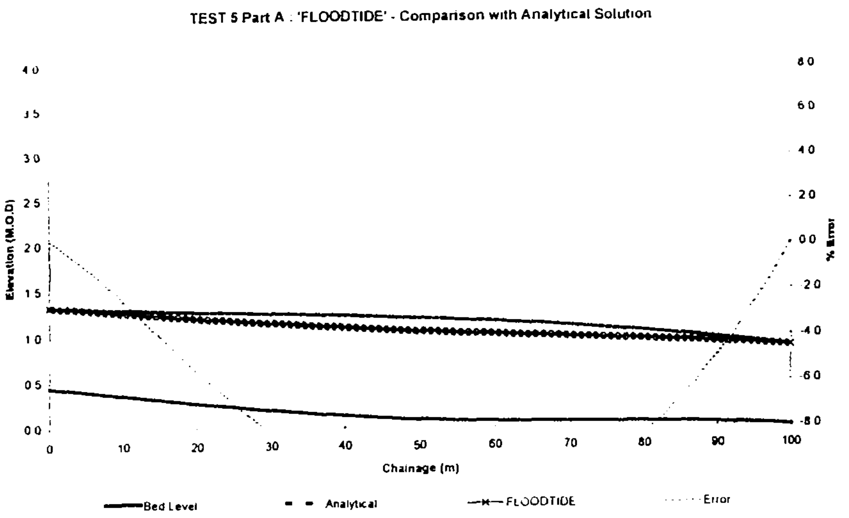


GRAPH 10 : *Test 4 - GLC-BACKWATER*

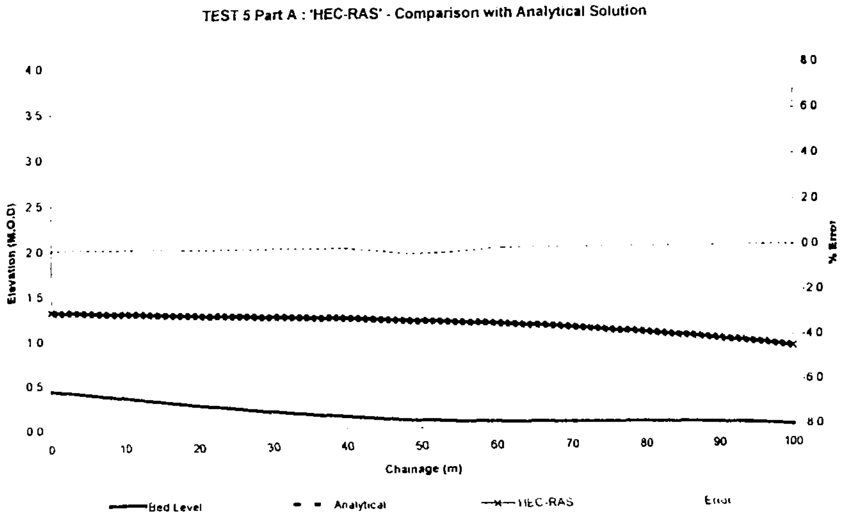


GRAPH 11 : *Test 4 – BAKWATER*

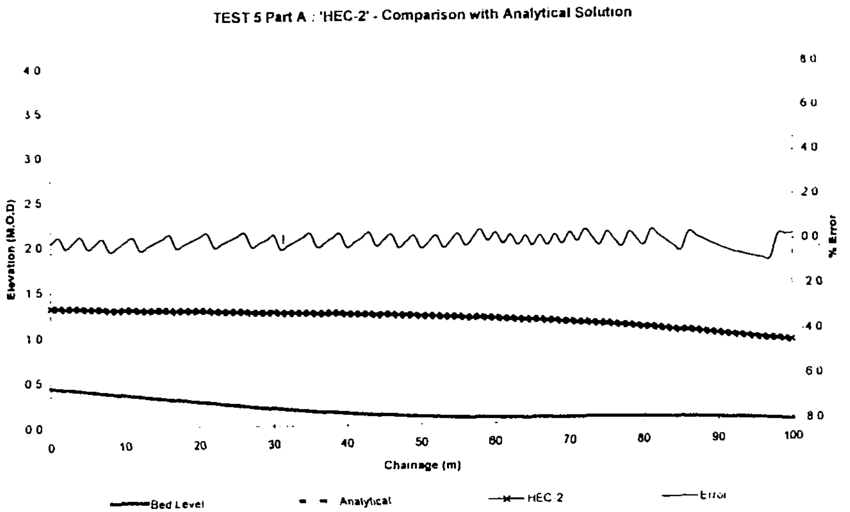
Appendix D



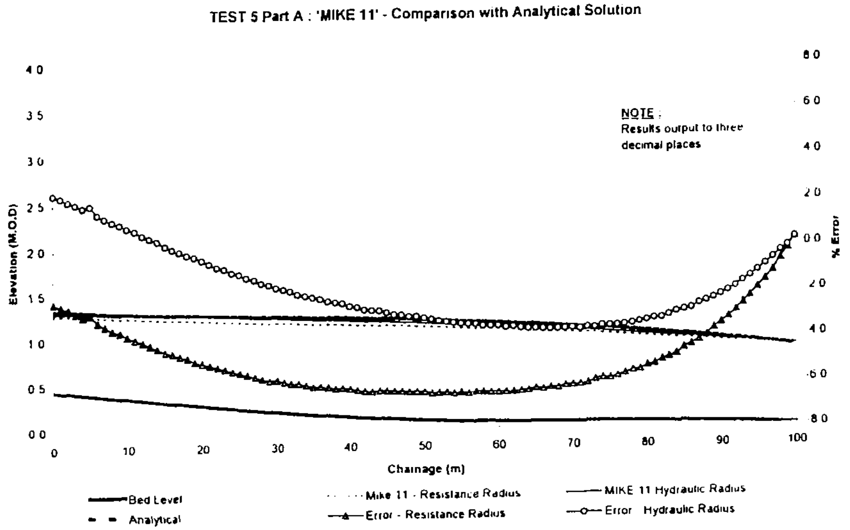
GRAPH 1 : *Test 5 Part A - FLOODTIDE*



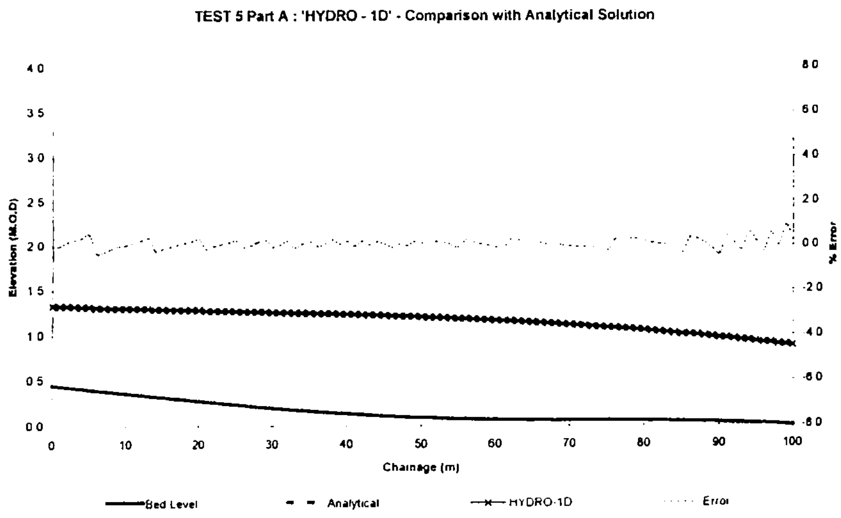
GRAPH 2 : *Test 5 Part A - HEC-RAS*



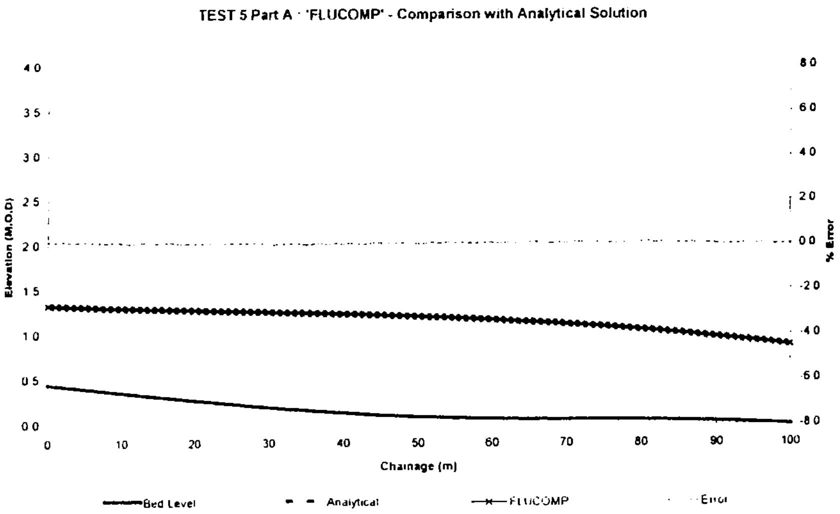
GRAPH 3 : *Test 5 Part A – HEC-2*



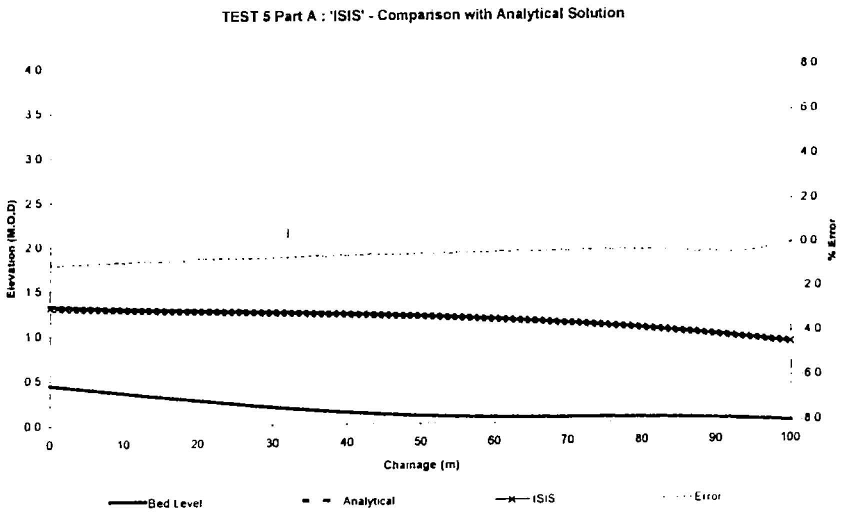
GRAPH 4 : *Test 5 Part A -- MIKE 11*



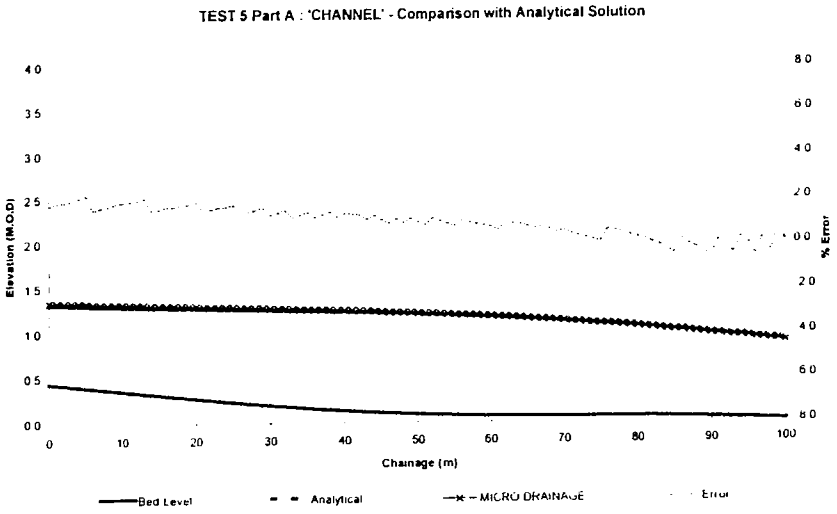
GRAPH 5 : *Test 5 Part A – HYDRO-1D*



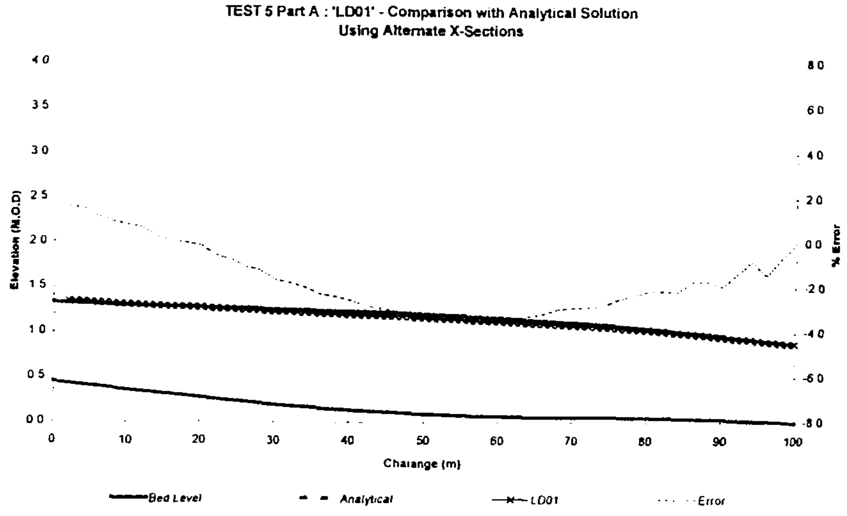
GRAPH 6 : *Test 5 Part A - FLUCOMP*



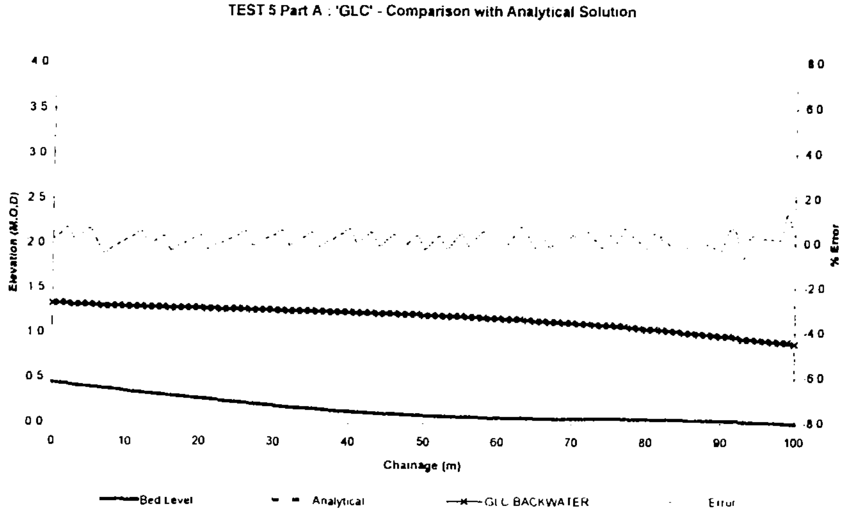
GRAPH 7 : *Test 5 Part A -ISIS*



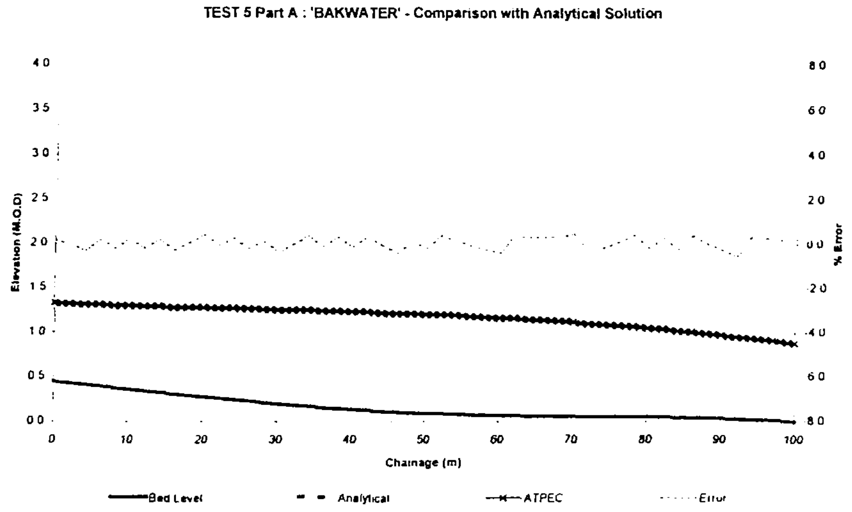
GRAPH 8 : *Test 5 Part A – CHANNEL*



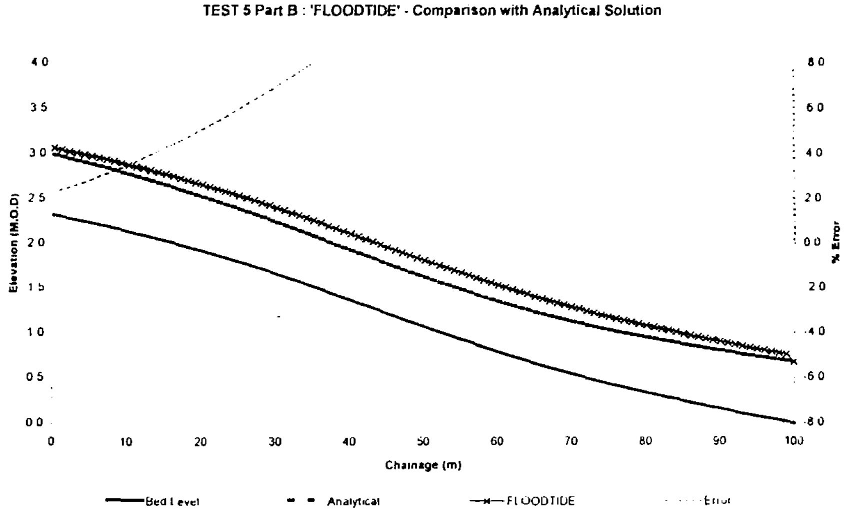
GRAPH 9 : *Test 5 Part A – LD01*



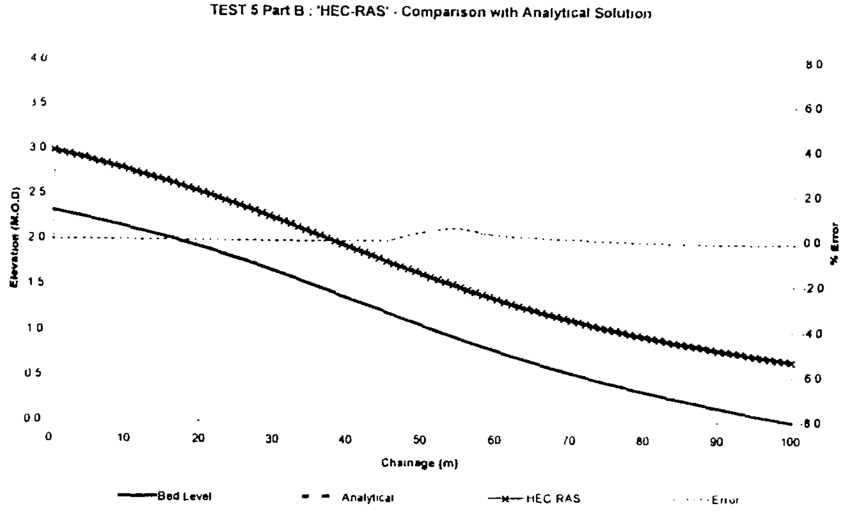
GRAPH 10 : *Test 5 Part A – GLC BACKWATER*



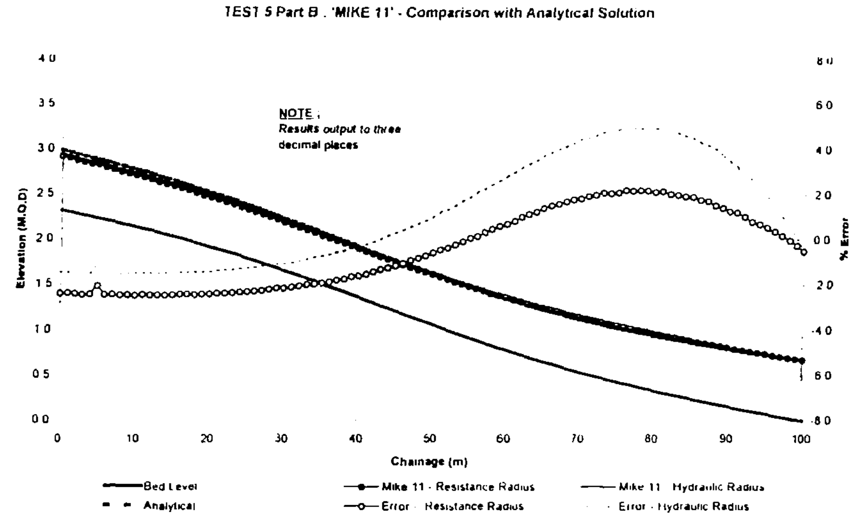
GRAPH 11 : *Test 5 Part A - BAKWATER*



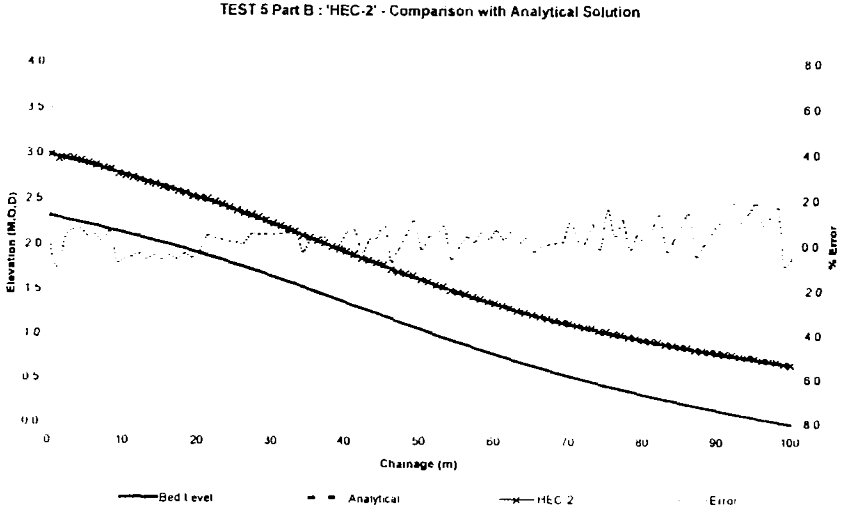
GRAPH 12 : *Test 5 Part B - FLOODTIDE*



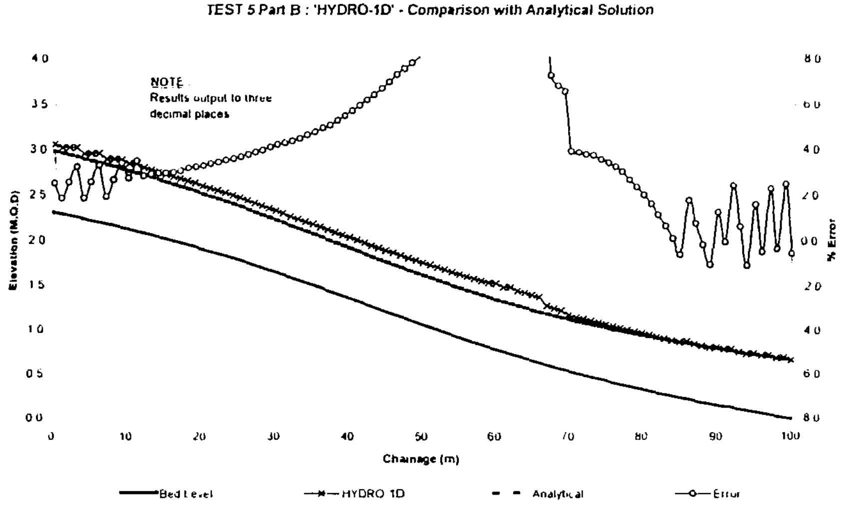
GRAPH 13 : *Test 5 Part B – HEC-RAS*



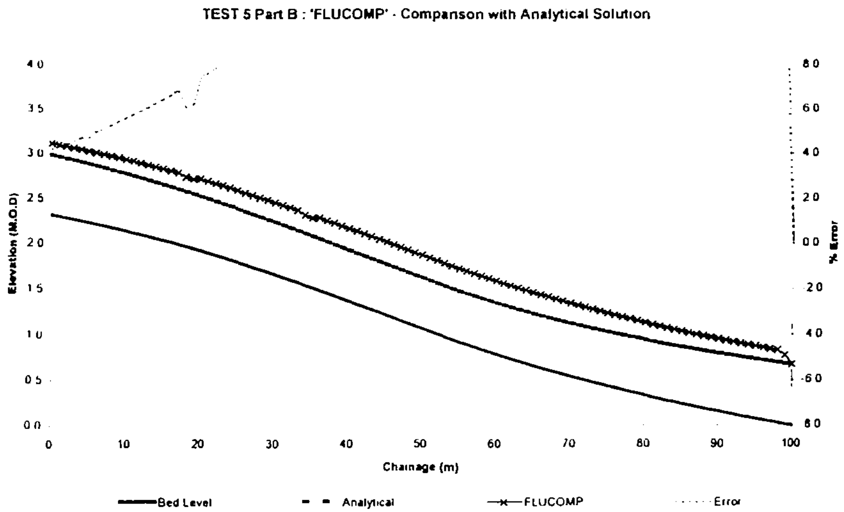
GRAPH 15 : *Test 5 Part B – MIKE 11*



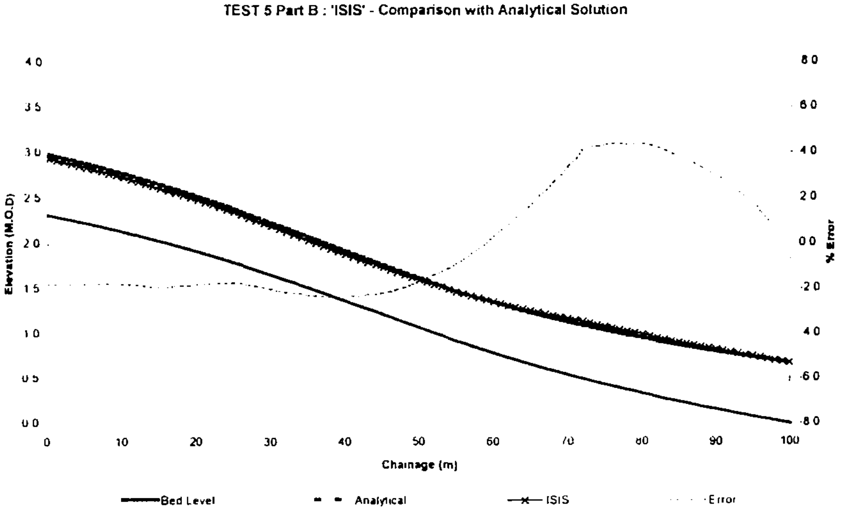
GRAPH 14 : *Test 5 Part B – HEC-2*



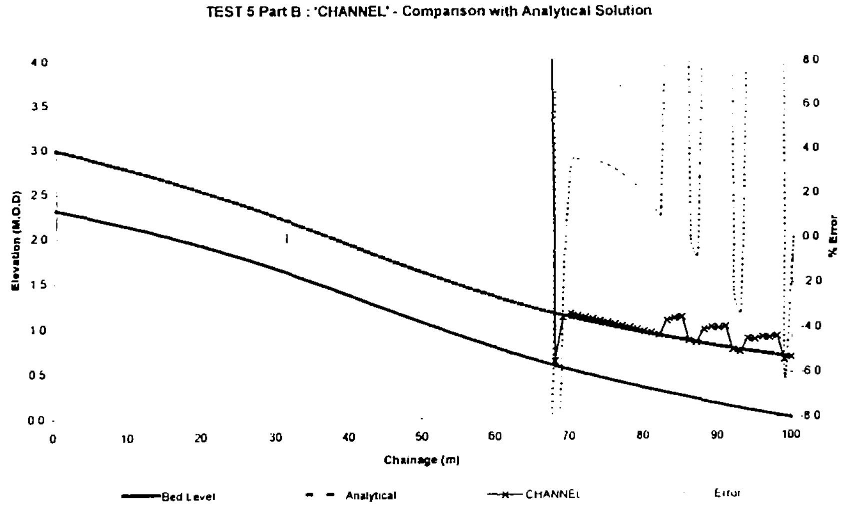
GRAPH 16 : *Test 5 Part B – HYDRO-1D*



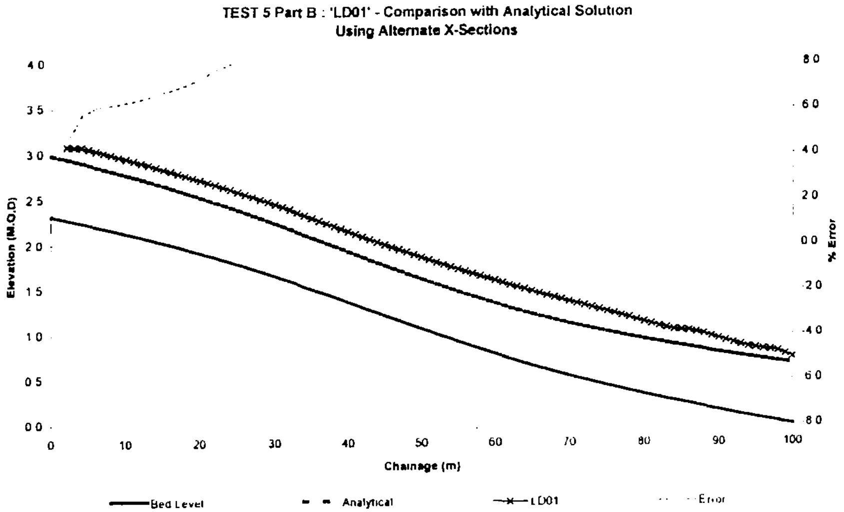
GRAPH 17 : *Test 5 Part B - FLUCOMP*



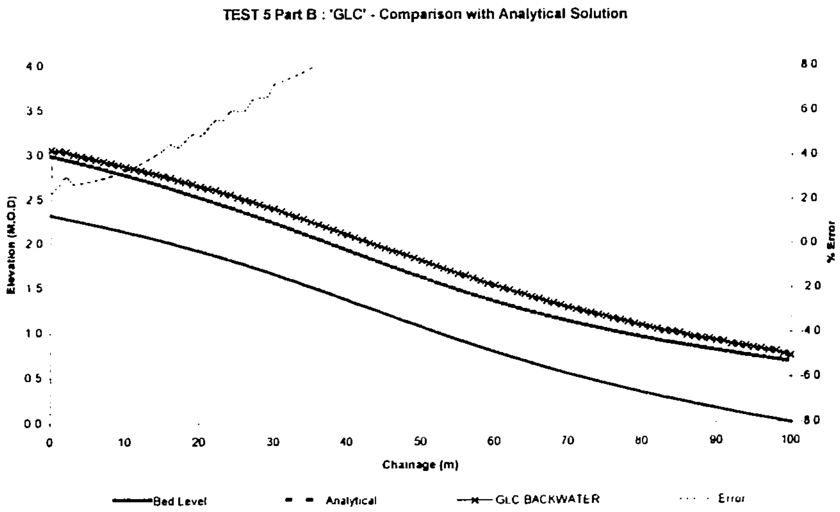
GRAPH 18 : *Test 5 Part B - ISIS*



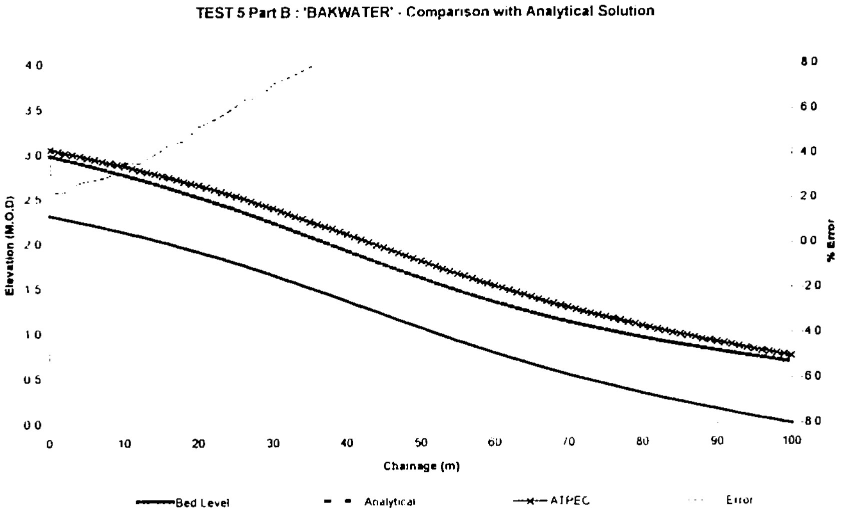
GRAPH 19 : *Test 5 Part B - CHANNEL*



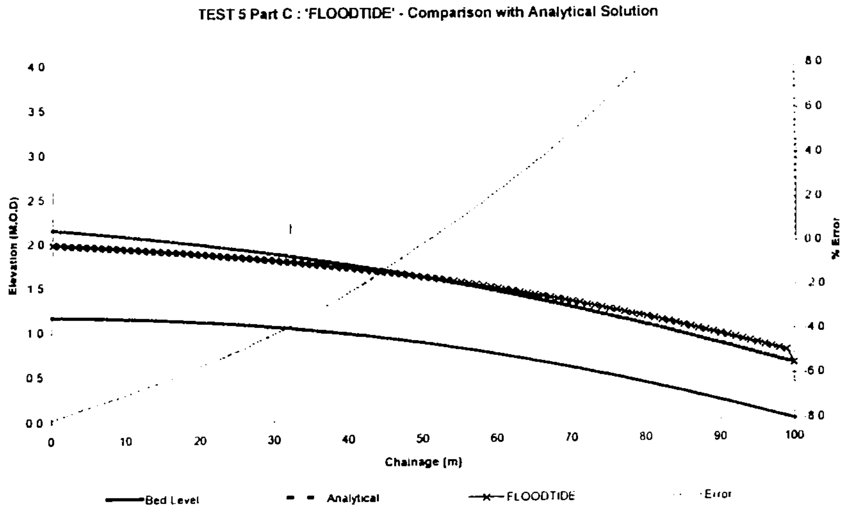
GRAPH 20 : *Test 5 Part B - LD01*



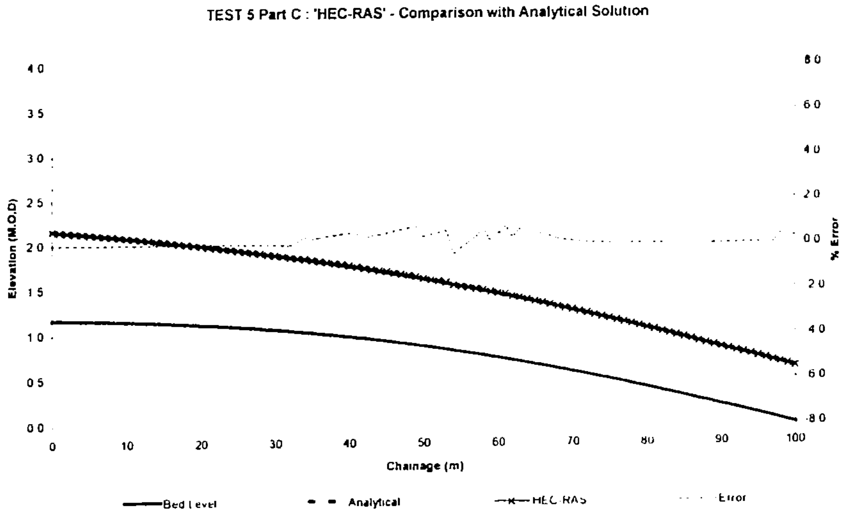
GRAPH 21 : *Test 5 Part B – GLC BACKWATER*



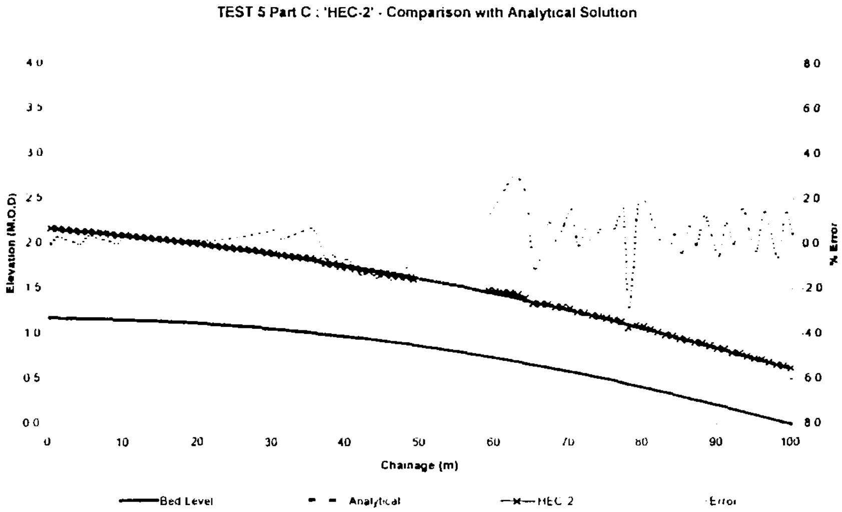
GRAPH 22 : *Test 5 Part B – BAKWATER*



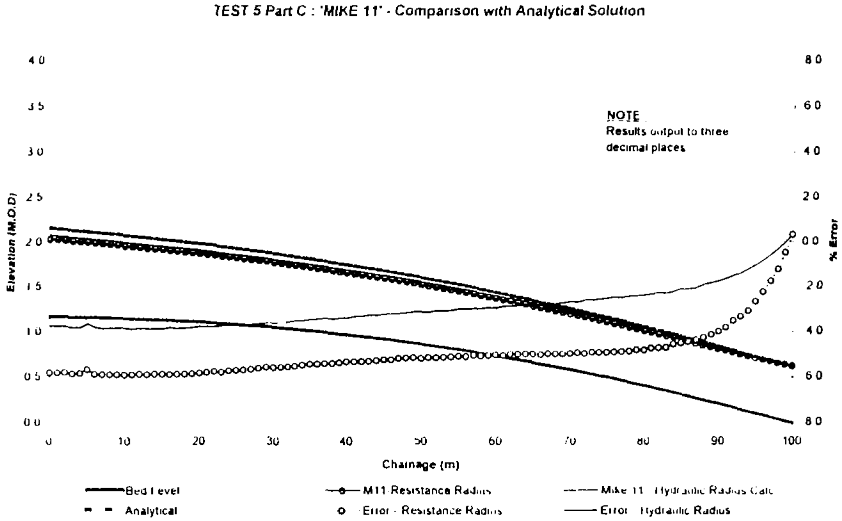
GRAPH 23 : *Test 5 Part C – FLOODTIDE*



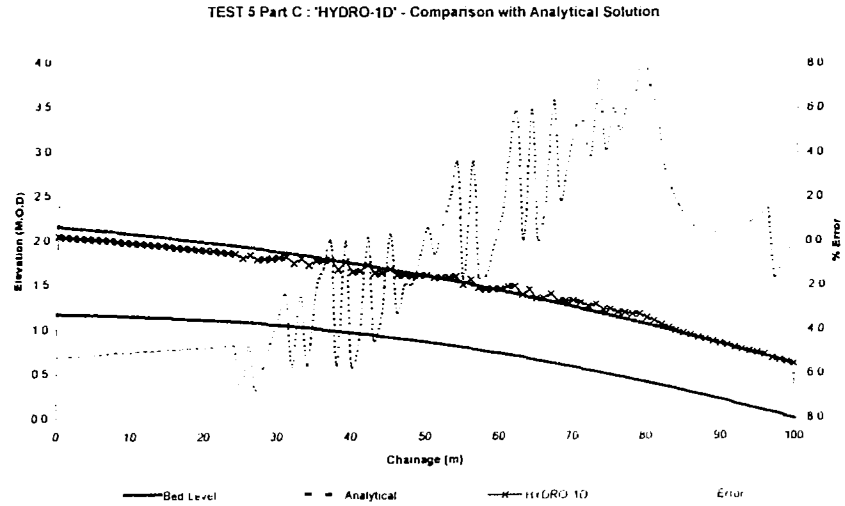
GRAPH 24 : *Test 5 Part C – HEC-RAS*



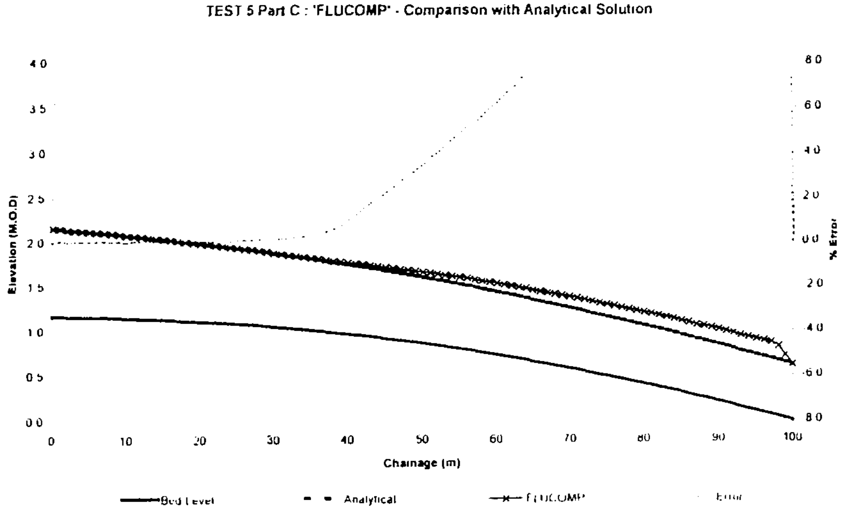
GRAPH 25 : *Test 5 Part C – HEC-2*



GRAPH 26 : *Test 5 Part C - MIKE 11*

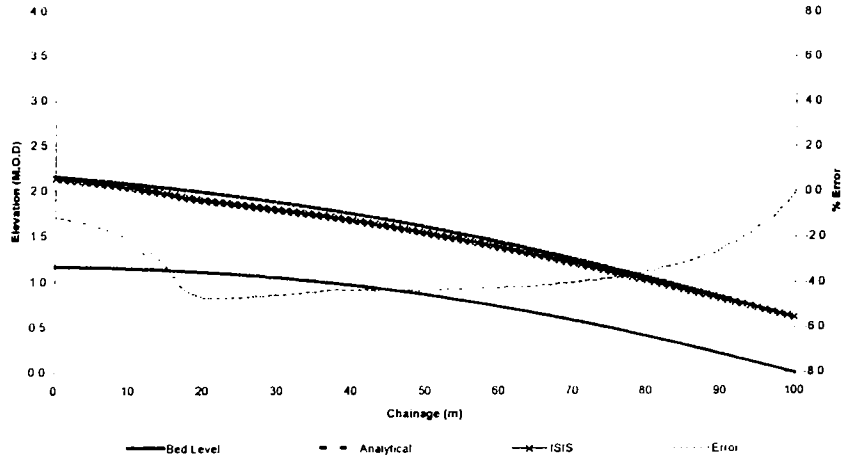


GRAPH 27 : *Test 5 Part C – HYDRO-1D*



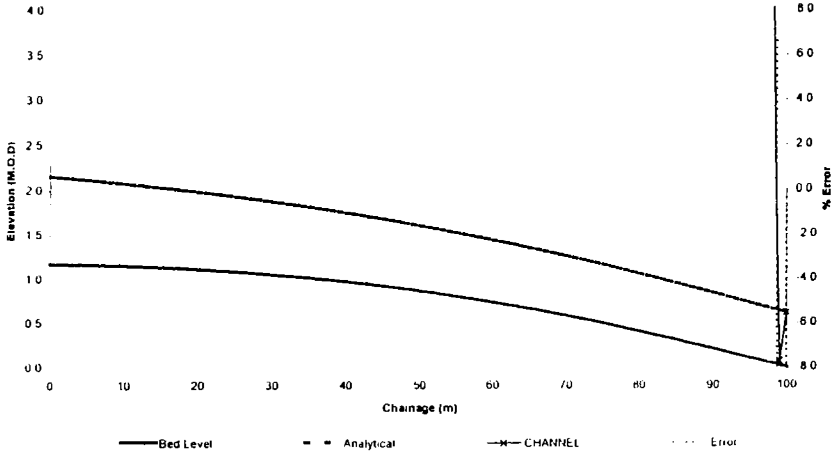
GRAPH 28 : *Test 5 Part C - FLUCOMP*

TEST 5 Part C : 'ISIS' - Comparison with Analytical Solution



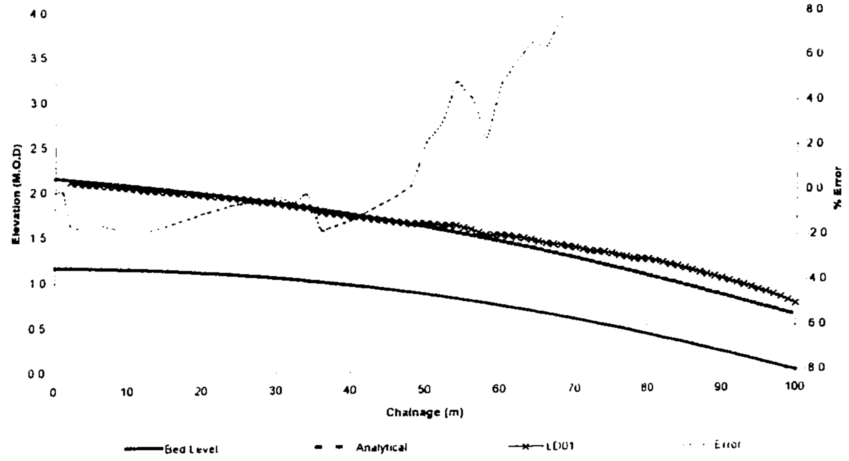
GRAPH 29 : *Test 5 Part C - ISIS*

TEST 5 Part C : 'CHANNEL' - Comparison with Analytical Solution



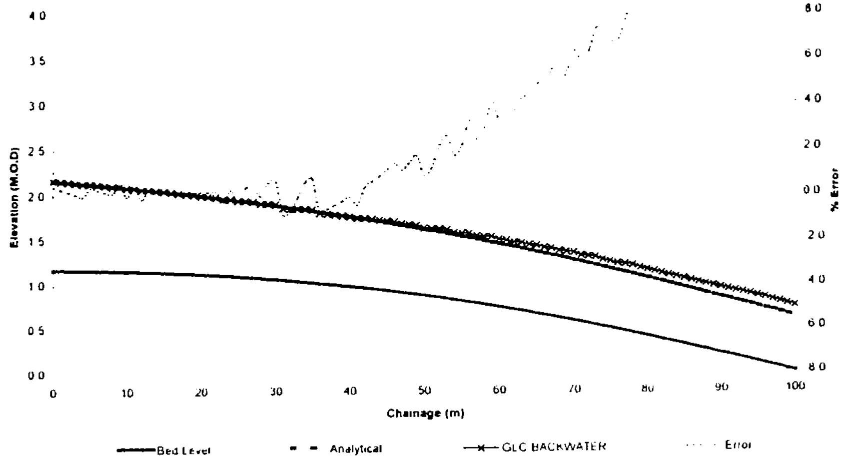
GRAPH 30 : *Test 5 Part C - CHANNEL*

TEST 5 Part C : 'LD01' - Comparison with Analytical Solution
Using Alternate X-Sections



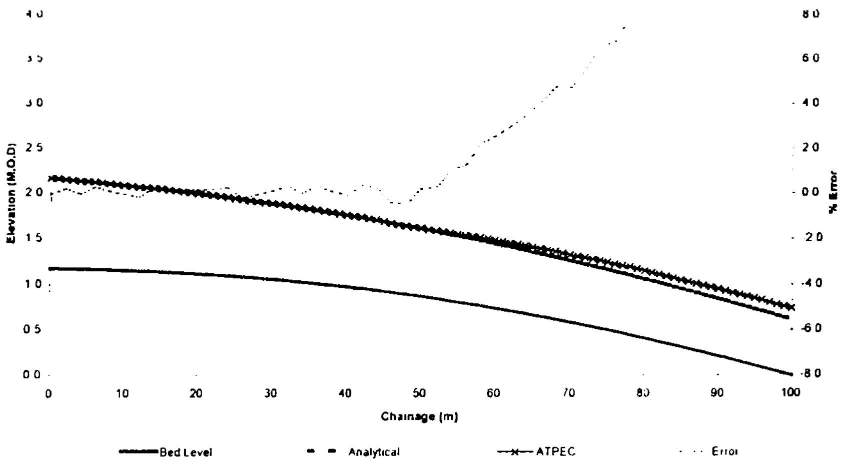
GRAPH 31 : *Test 5 Part C - LD01*

TEST 5 Part C : 'GLC' - Comparison with Analytical Solution



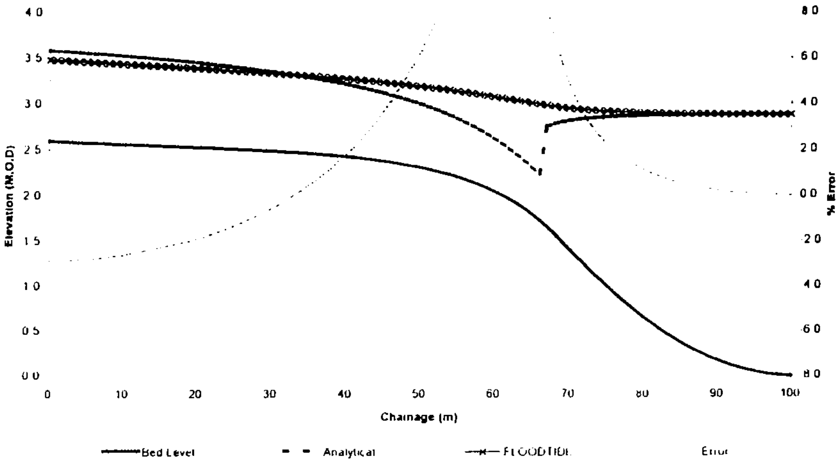
GRAPH 32 : *Test 5 Part C - GLC BACKWATER*

TEST 5 Part C . 'BAKWATER' - Comparison with Analytical Solution



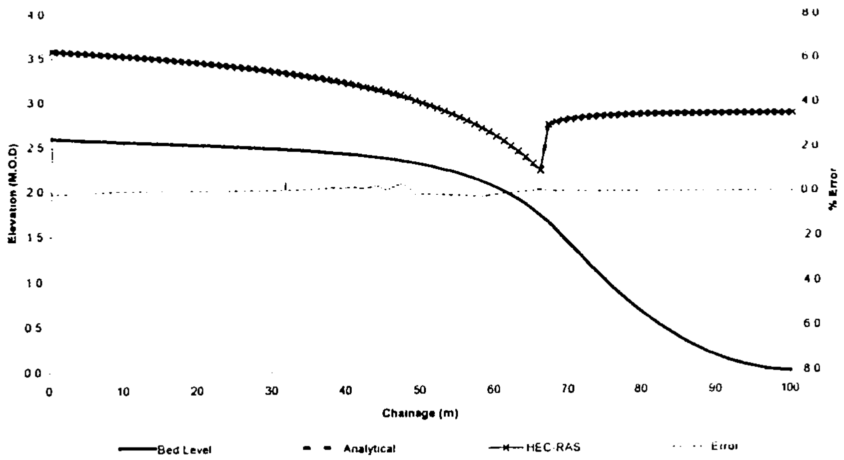
GRAPH 33 : *Test 5 Part C - BAKWATER*

TEST 5 Part D . 'FLOODTIDE' - Comparison with Analytical Solution



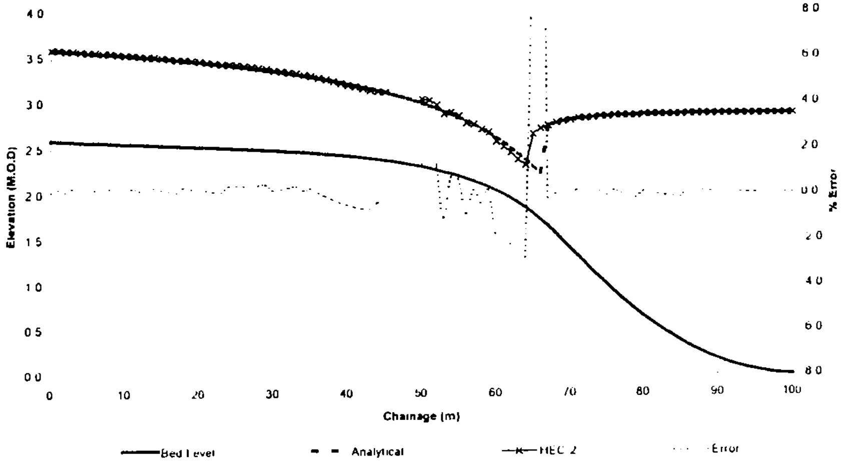
GRAPH 34 : *Test 5 Part D - FLOODTIDE*

TEST 5 Part D : 'HEC-RAS' - Comparison with Analytical Solution



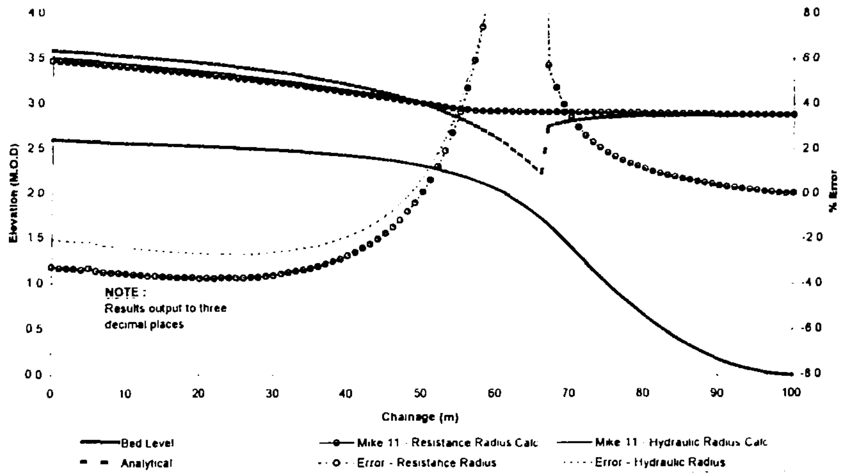
GRAPH 35 : *Test 5 Part D - HEC-RAS*

TEST 5 Part D : 'HEC-2' - Comparison with Analytical Solution



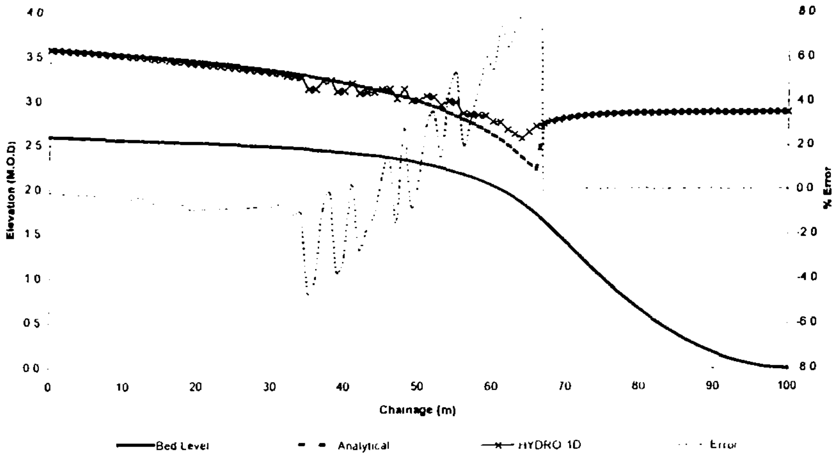
GRAPH 36 : *Test 5 Part D - HEC-2*

TEST 5 Part D : 'MIKE 11' - Comparison with Analytical Solution



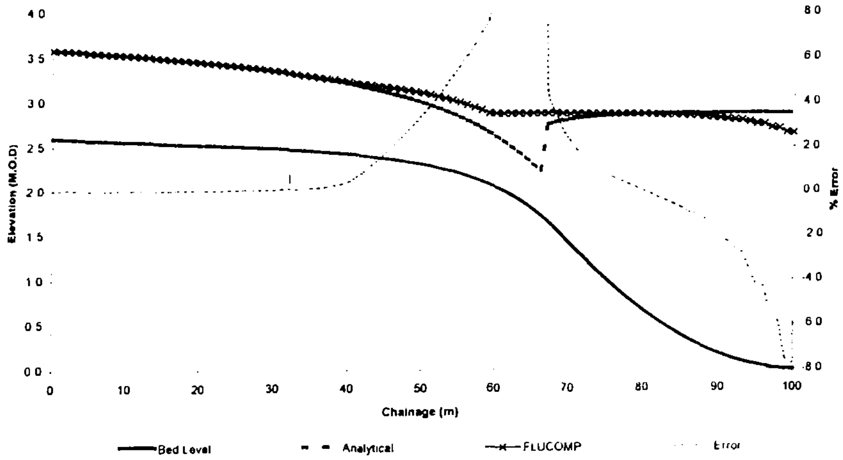
GRAPH 37 : *Test 5 Part D – MIKE 11*

TEST 5 Part D : 'HYDRO-1D' - Comparison with Analytical Solution



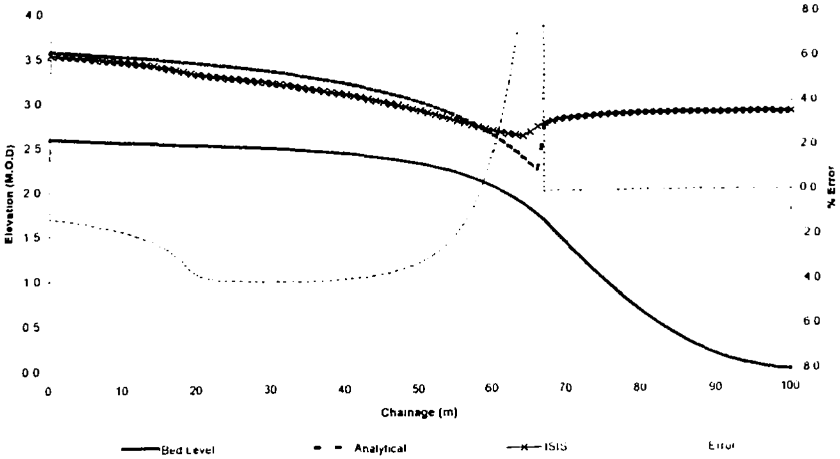
GRAPH 38 : *Test 5 Part D – HYDRO-1D*

TEST 5 Part D : 'FLUCOMP' - Comparison with Analytical Solution

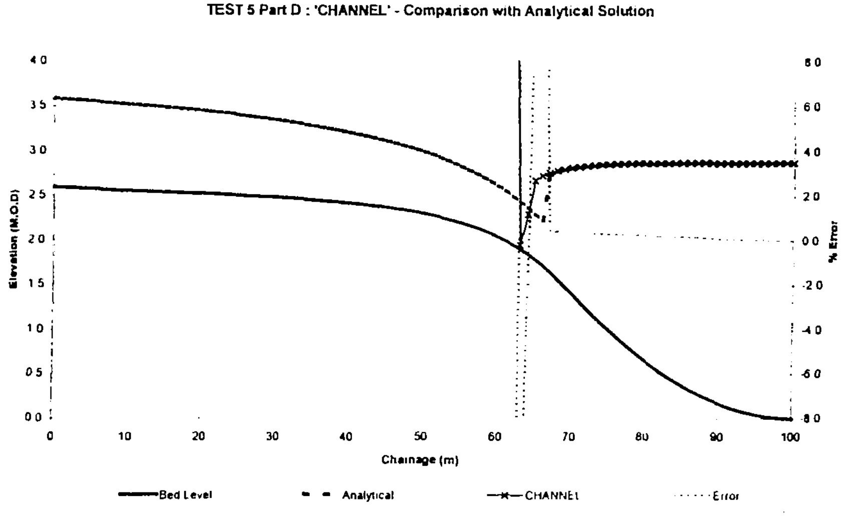


GRAPH 39 : *Test 5 Part D - FLUCOMP*

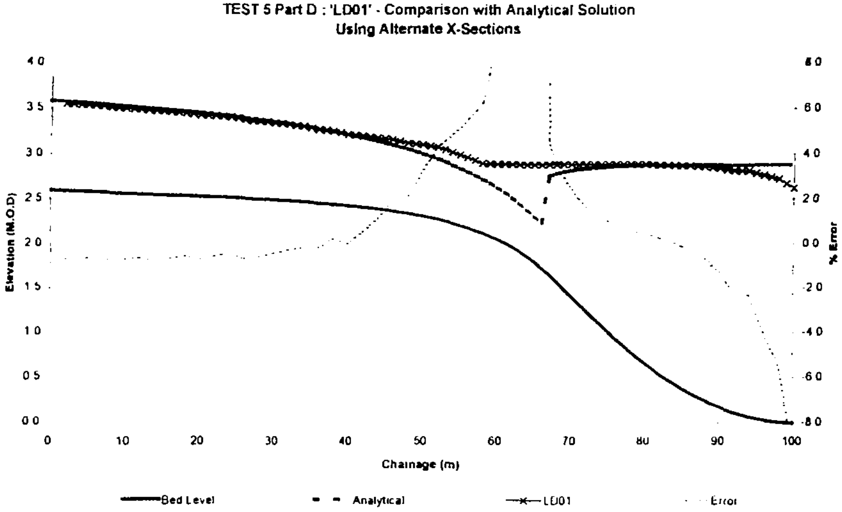
TEST 5 Part D : 'ISIS' - Comparison with Analytical Solution



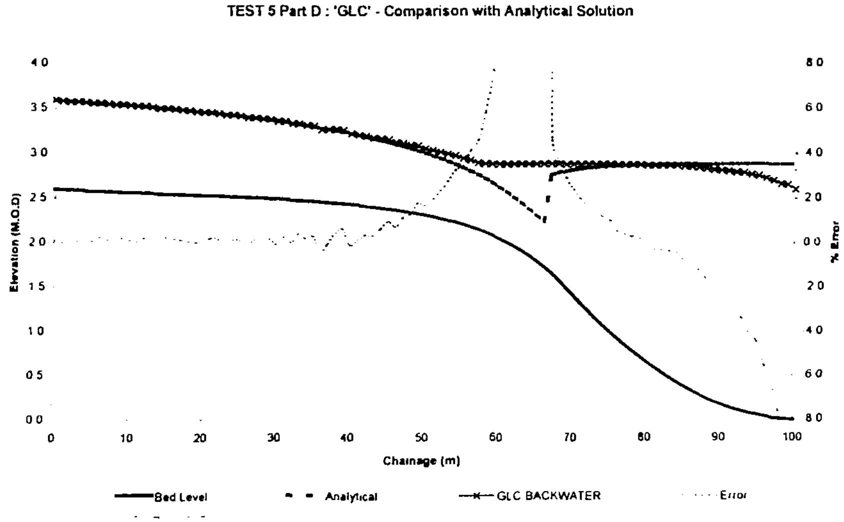
GRAPH 40 : *Test 5 Part D – ISIS*



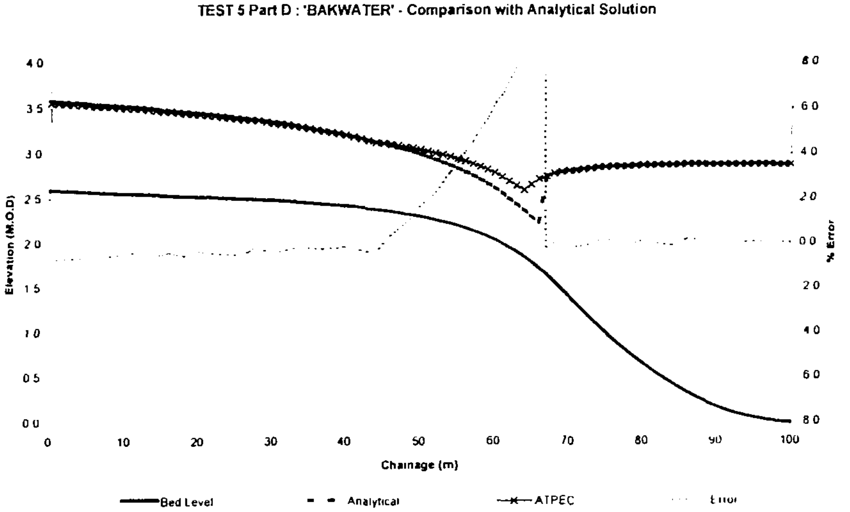
GRAPH 41 : *Test 5 Part D – CHANNEL*



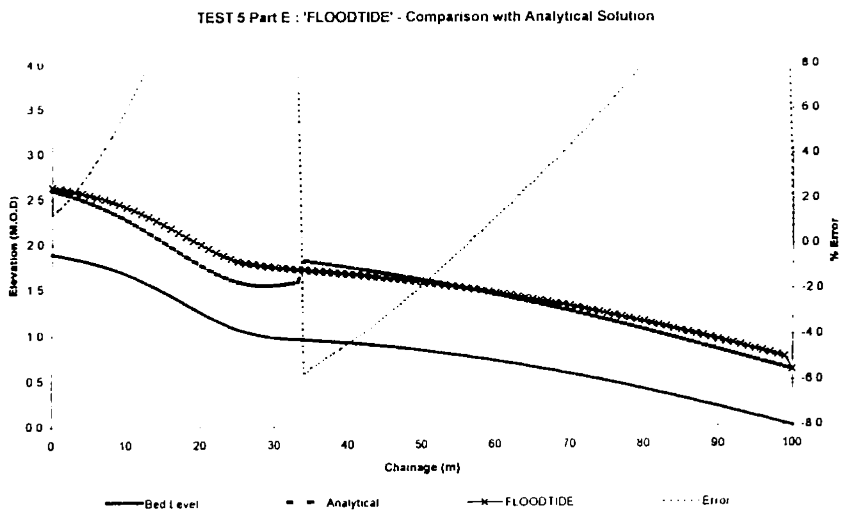
GRAPH 42 : *Test 5 Part D – LD01*



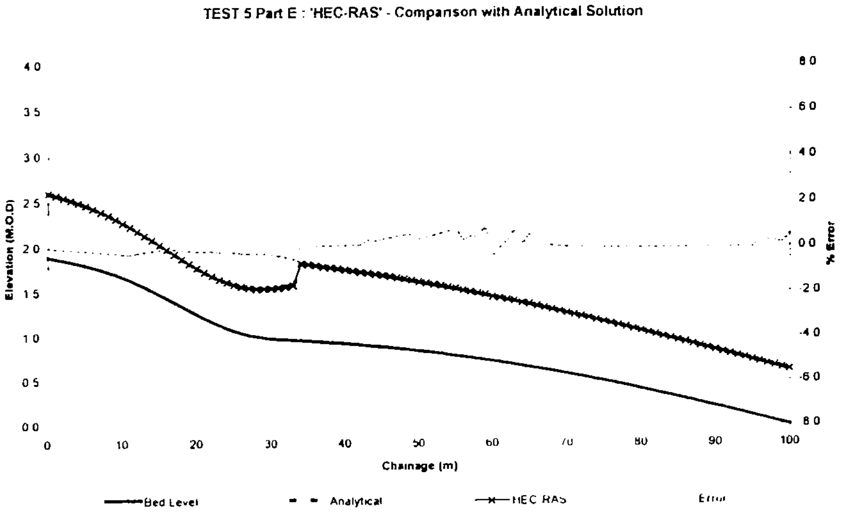
GRAPH 43 : *Test 5 Part D – GLC BACKWATER*



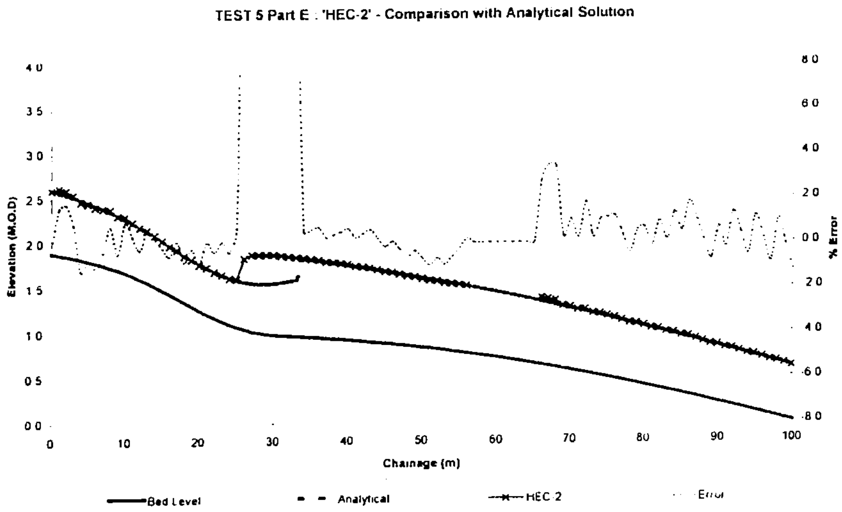
GRAPH 44 : *Test 5 Part D - BAKWATER*



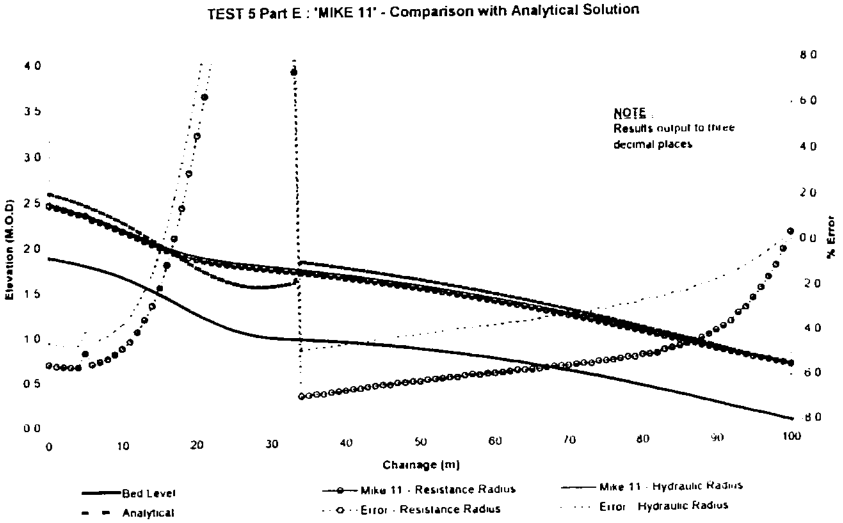
GRAPH 45 : *Test 5 Part E - FLOODTIDE*



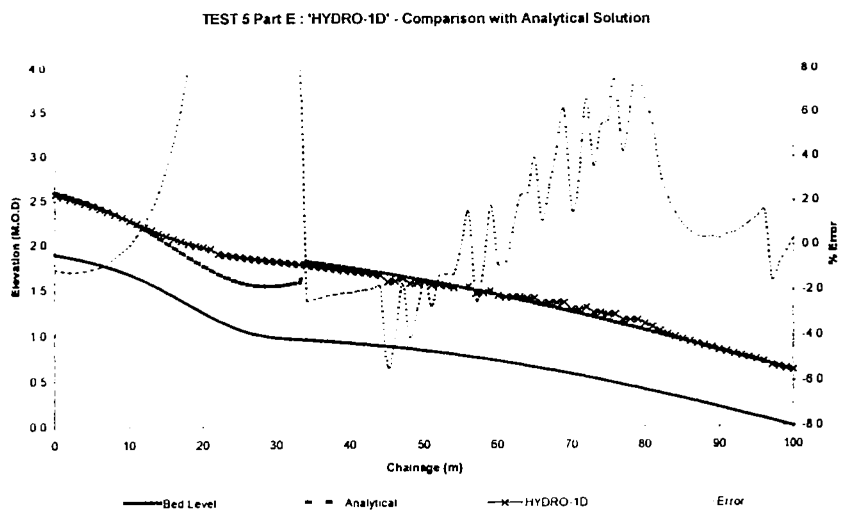
GRAPH 46 : *Test 5 Part E - HEC-RAS*



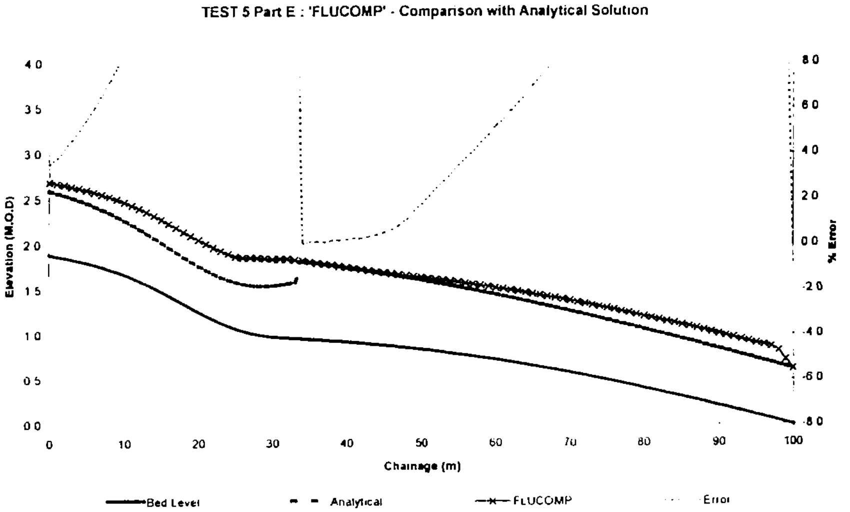
GRAPH 47 : *Test 5 Part E – HEC-2*



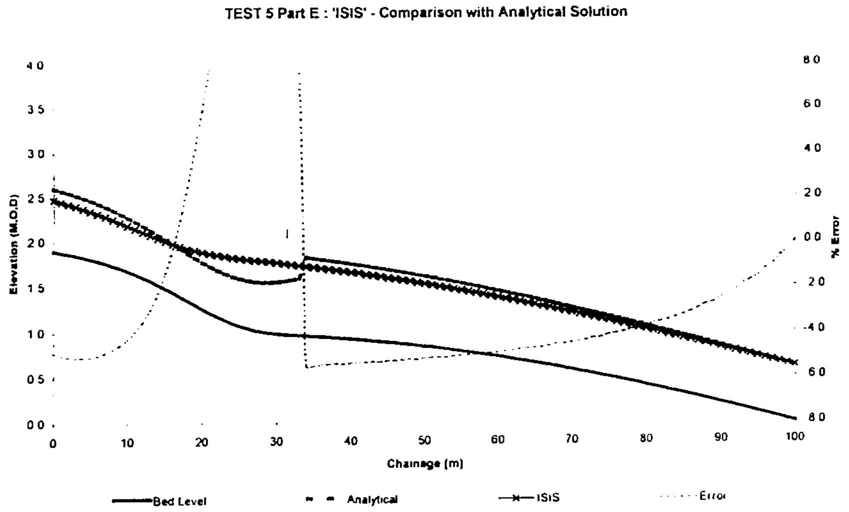
GRAPH 48 : *Test 5 Part E – MIKE 11*



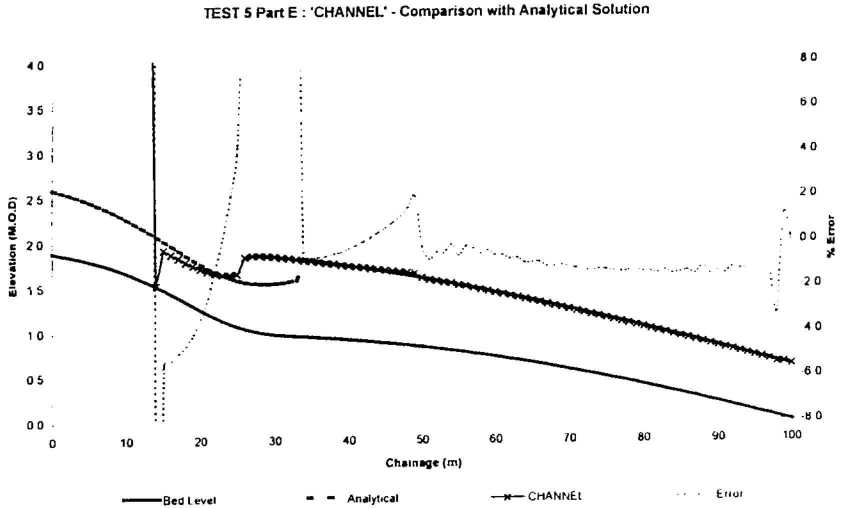
GRAPH 49 : *Test 5 Part E – HYDRO-1D*



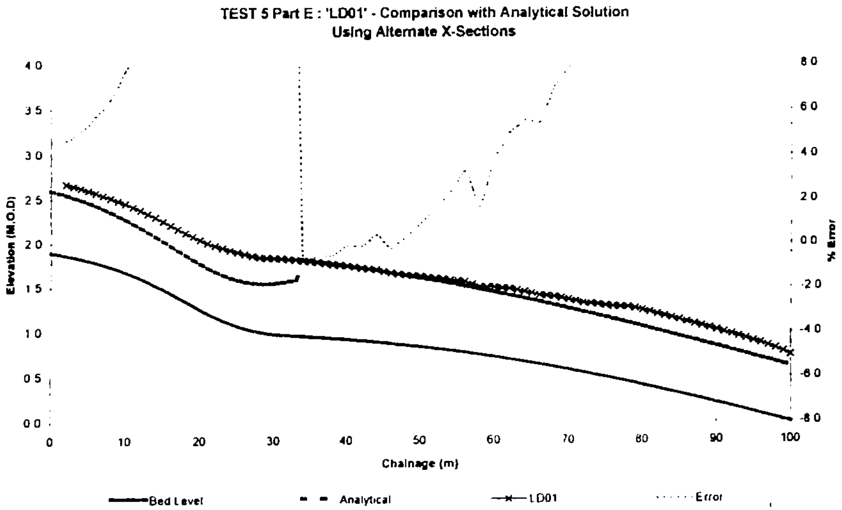
GRAPH 50 : *Test 5 Part E - FLUCOMP*



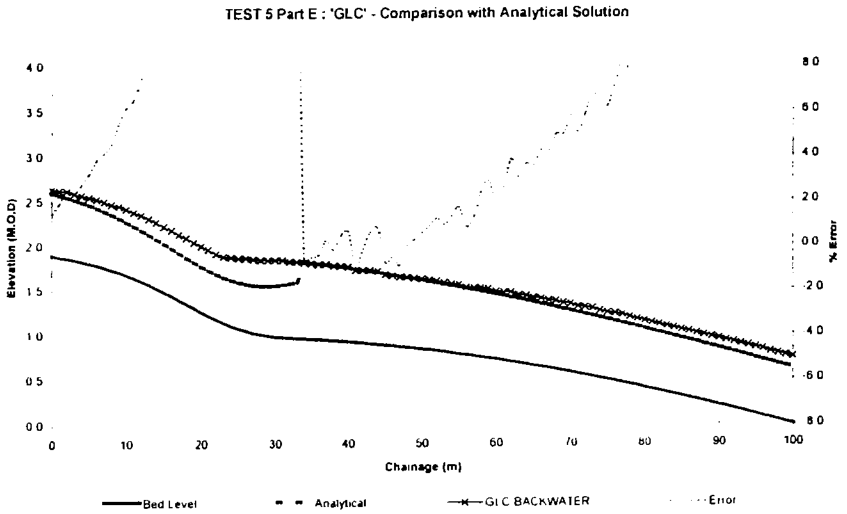
GRAPH 51 : *Test 5 Part E - ISIS*



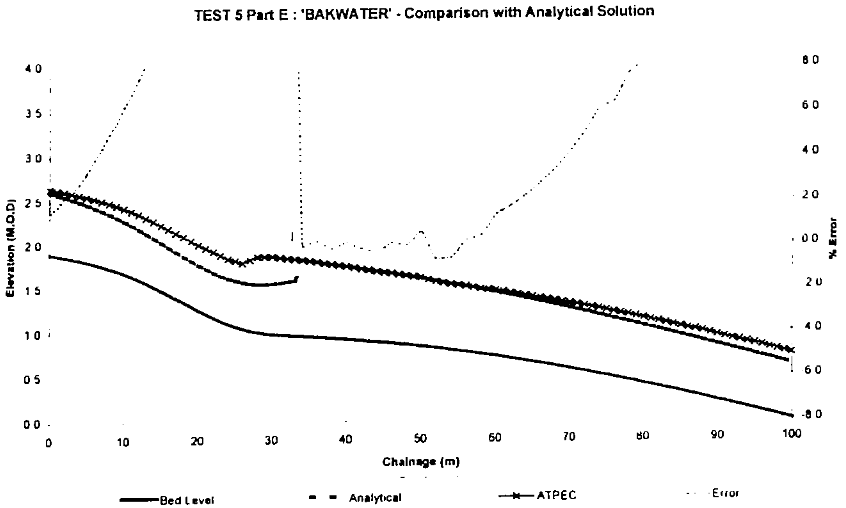
GRAPH 52 : *Test 5 Part E - CHANNEL*



GRAPH 53 : *Test 5 Part E – LD01*

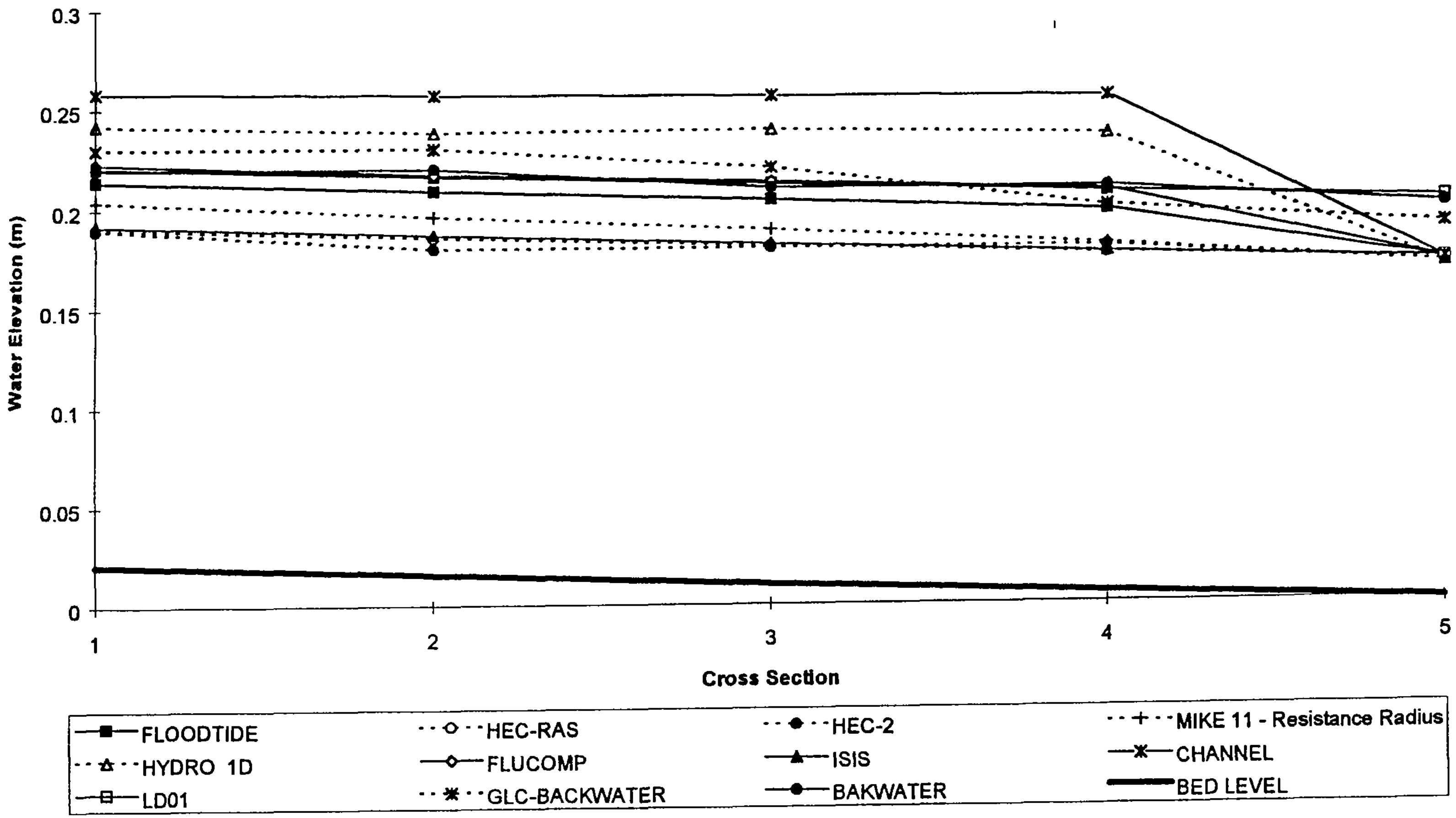


GRAPH 54 : *Test 5 Part E – GLC BACKWATER*

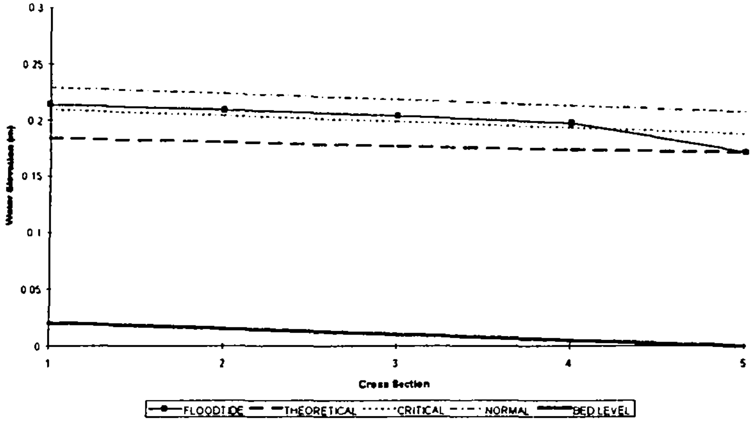


GRAPH 55 : *Test 5 Part E - BAKWATER*

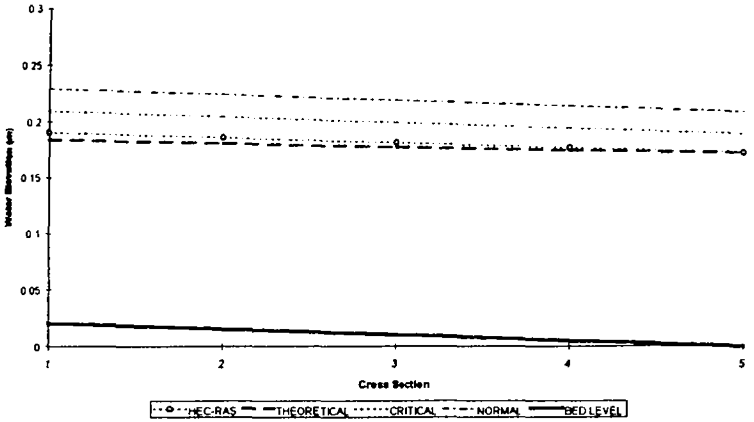
Appendix E



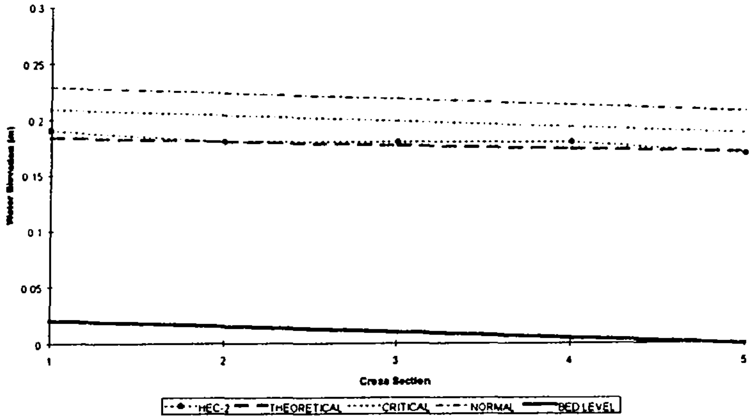
GRAPH 1 : *TEST 6A1 - Comparison of Calculated Water Surface Profiles*



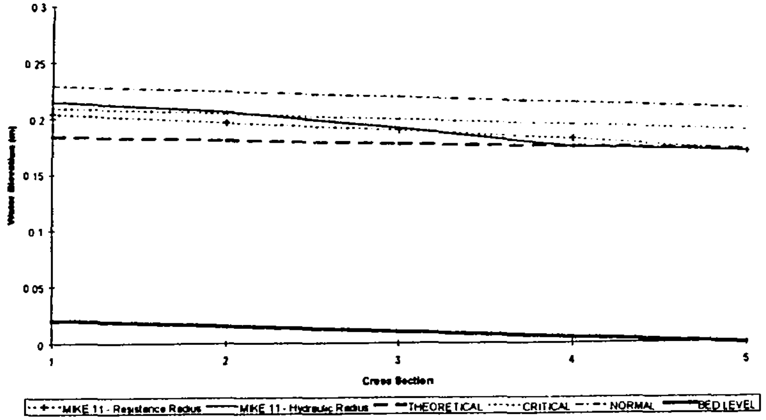
GRAPH 2 : *TEST 6 Part A1 - FLOODTIDE Water Surface Profile*



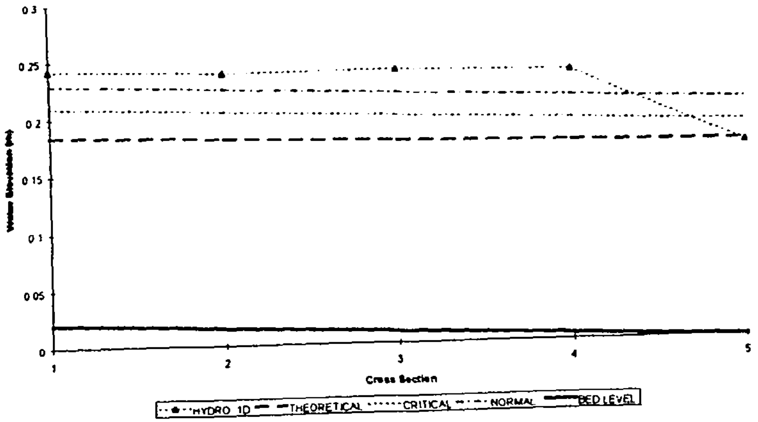
GRAPH 3 : *TEST 6 Part A1 - HEC-RAS Water Surface Profile*



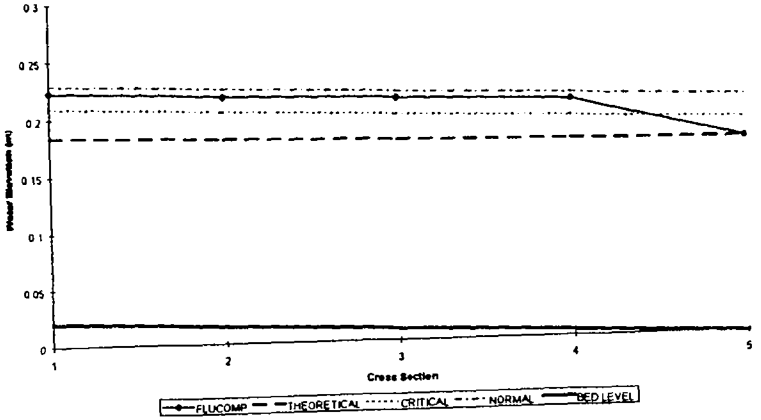
GRAPH 4 : *TEST 6 Part A1 – HEC-2 Water Surface Profile*



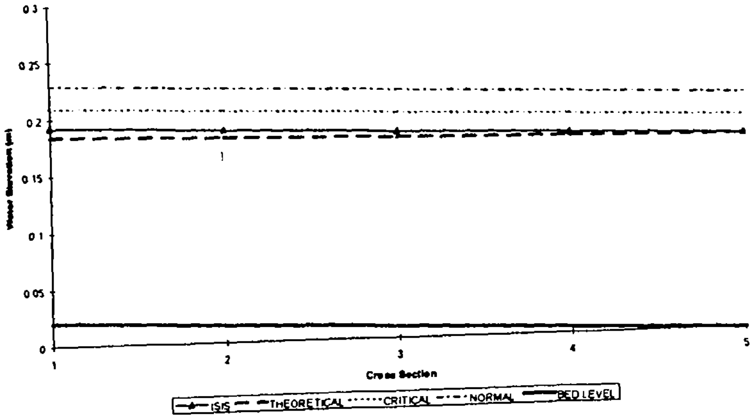
GRAPH 5 : *TEST 6 Part A1 - MIKE 11 Water Surface Profile*



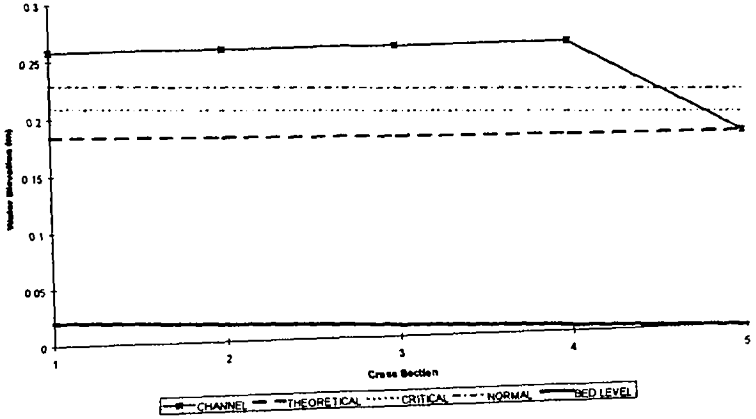
GRAPH 6 : *TEST 6 Part A1 - HYDRO 1D Water Surface Profile*



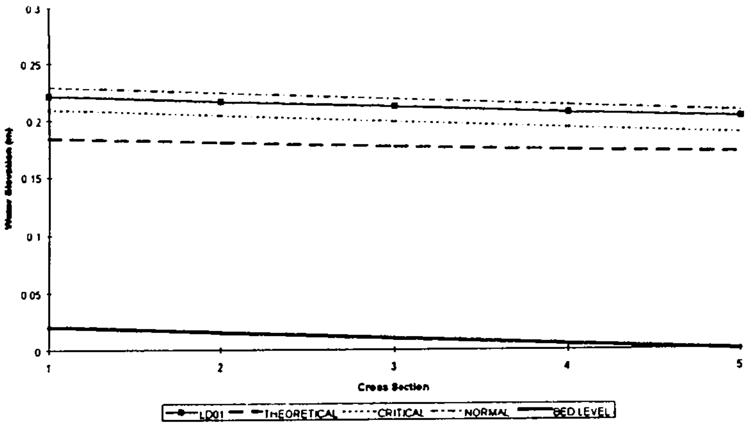
GRAPH 7 : *TEST 6 Part A1 - FLUCOMP Water Surface Profile*



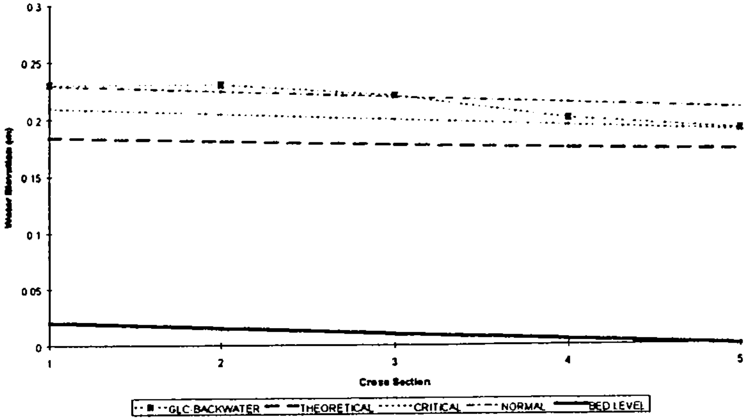
GRAPH 8 : *TEST 6 Part A1 - ISIS Water Surface Profile*



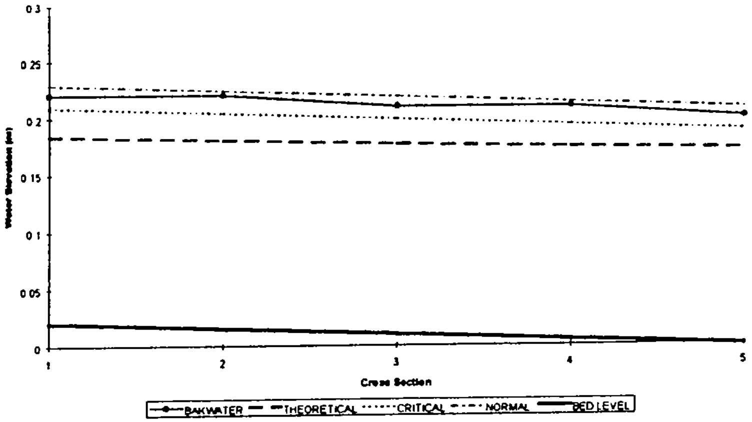
GRAPH 9 : *TEST 6 Part A1 - CHANNEL Water Surface Profile*



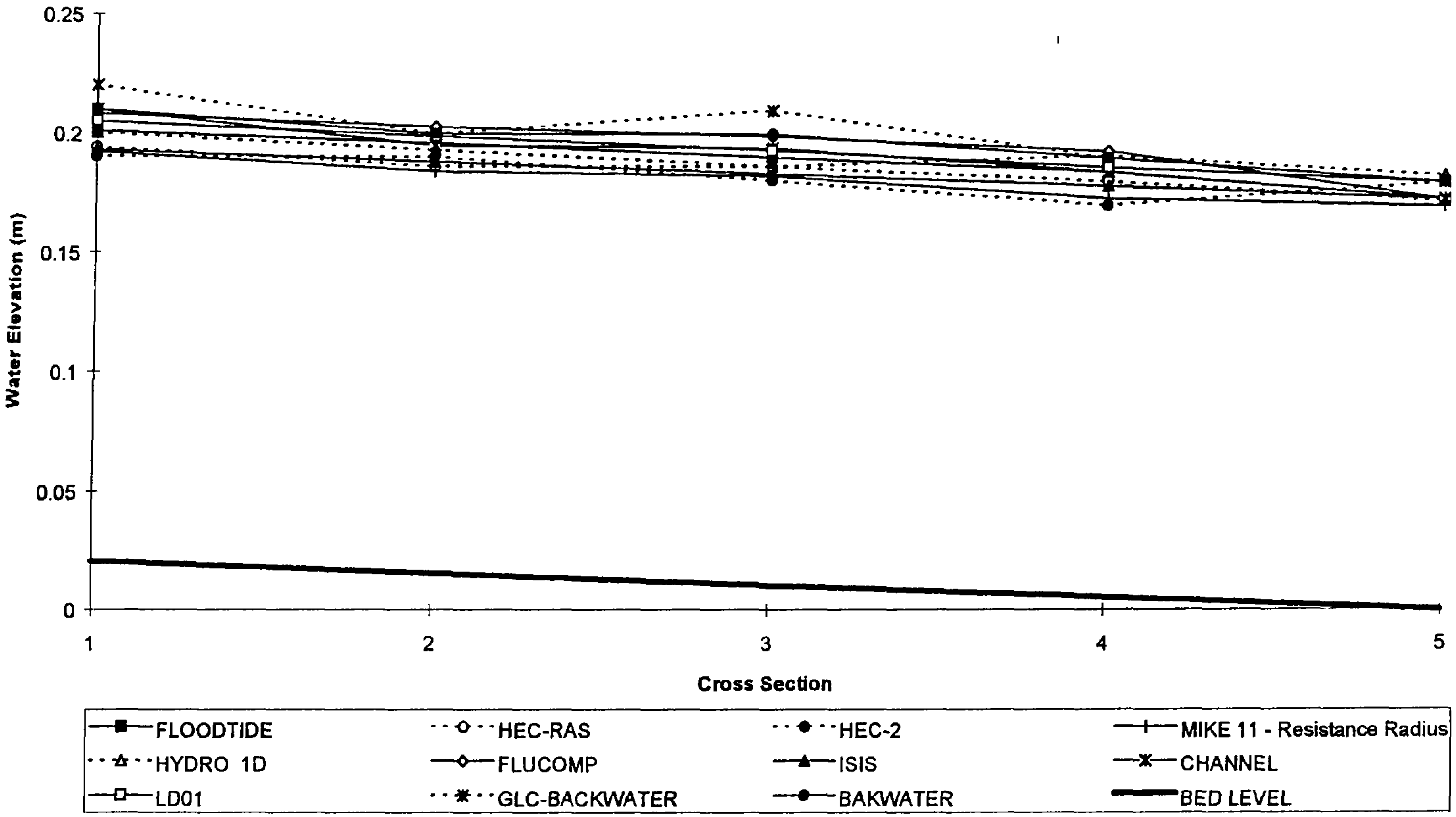
GRAPH 10 : *TEST 6 Part A1 - LD01 Water Surface Profile*



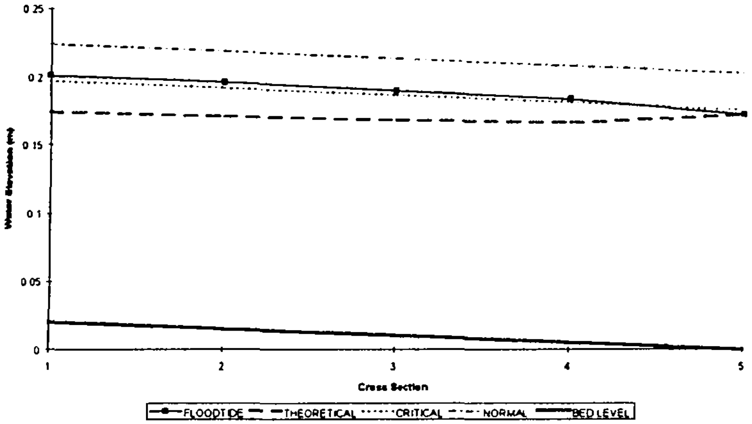
GRAPH 11 : *TEST 6 Part A1 - GLC-BACKWATER Water Surface Profile*



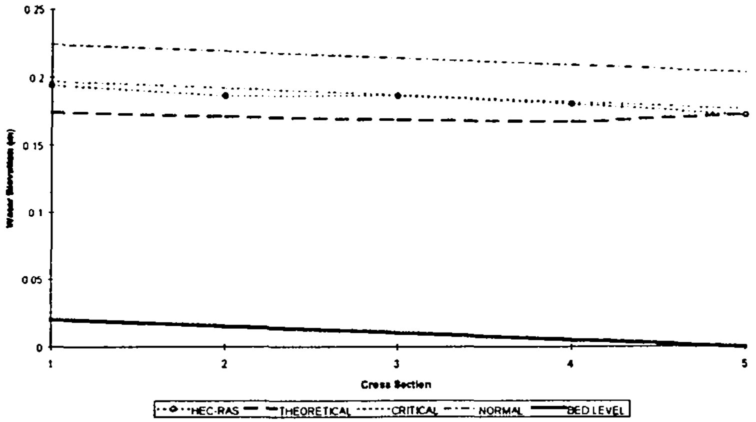
GRAPH 12 : *TEST 6 Part A1 - BAKWATER Water Surface Profile*



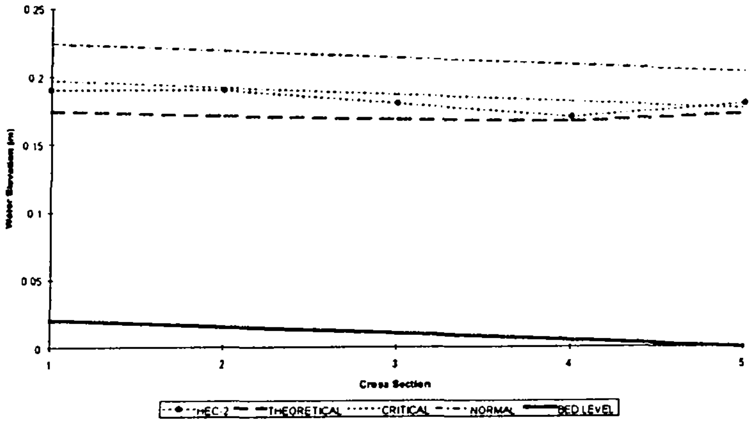
GRAPH 13 : *TEST 6A2 - Comparison of Calculated Water Surface Profiles*



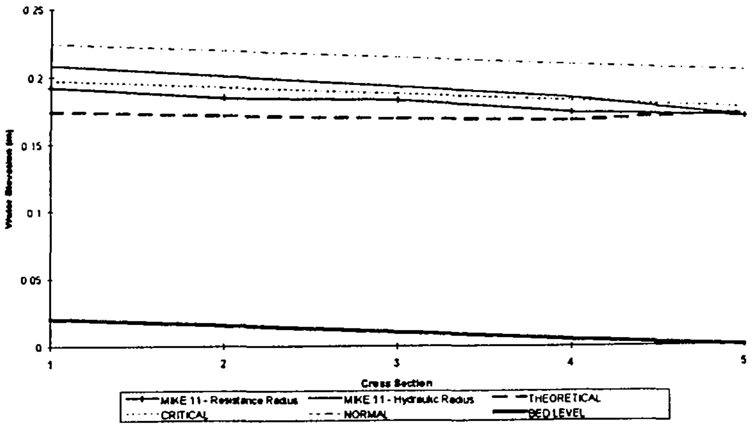
GRAPH 14 : *TEST 6 Part A2 - FLOODTIDE Water Surface Profile*



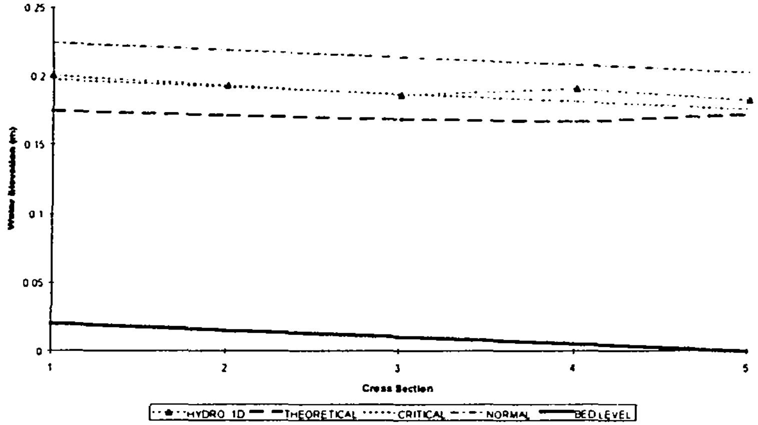
GRAPH 15 : *TEST 6 Part A2 - HEC-RAS Water Surface Profile*



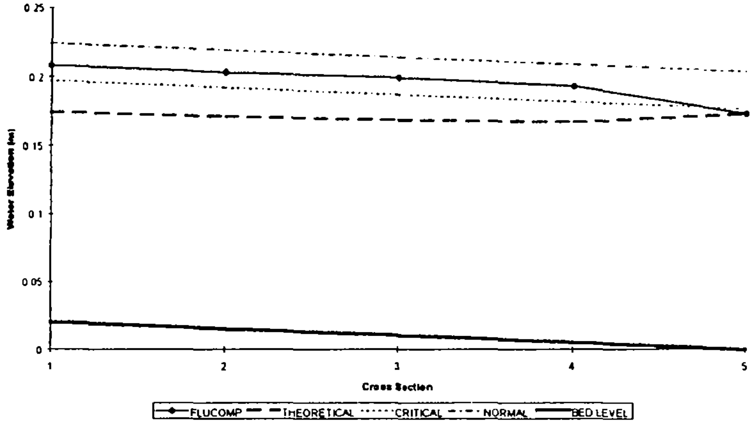
GRAPH 16 : *TEST 6 Part A2 - HEC-2 Water Surface Profile*



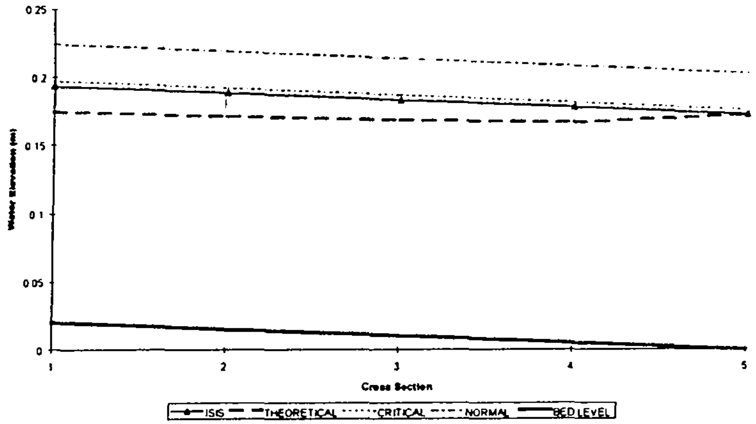
GRAPH 17 : *TEST 6 Part A2 - MIKE 11 Water Surface Profile*



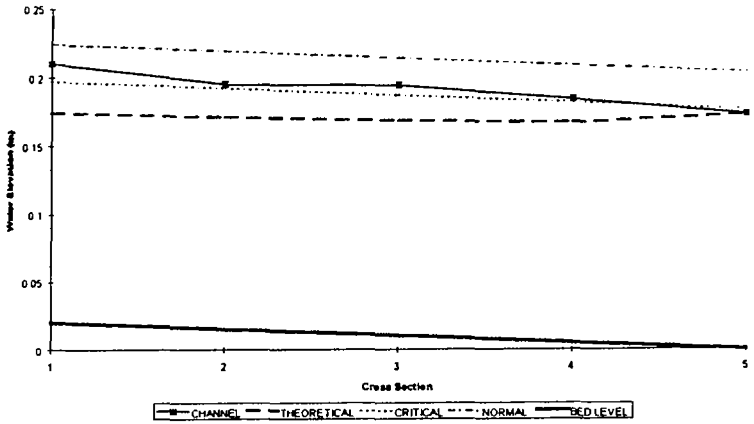
GRAPH 18 : *TEST 6 Part A2 - HYDRO 1D Water Surface Profile*



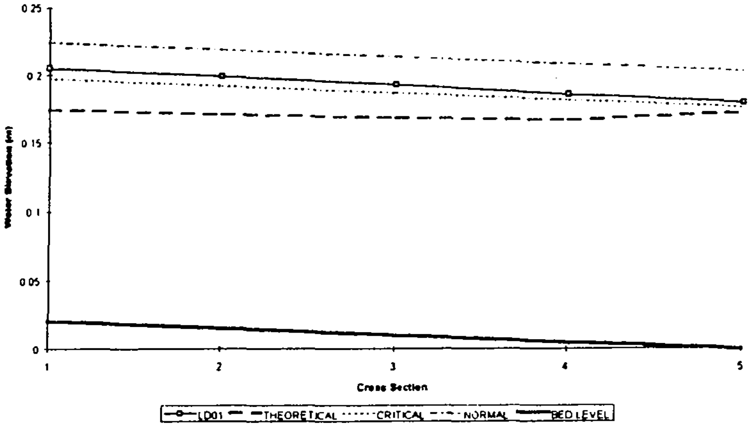
GRAPH 19 : *TEST 6 Part A2 - FLUCOMP Water Surface Profile*



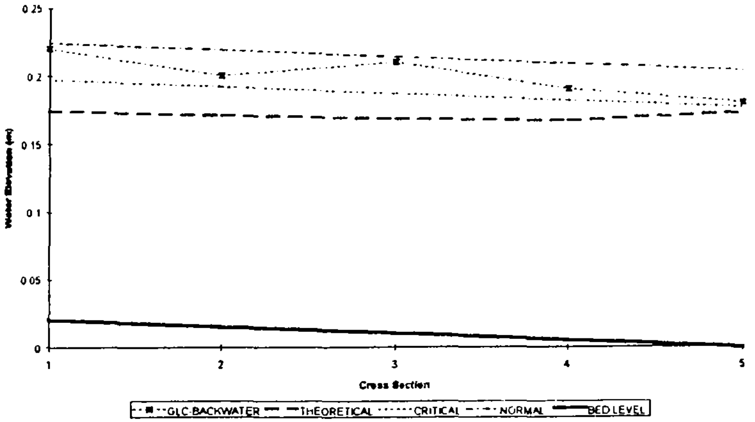
GRAPH 20 : *TEST 6 Part A2 - ISIS Water Surface Profile*



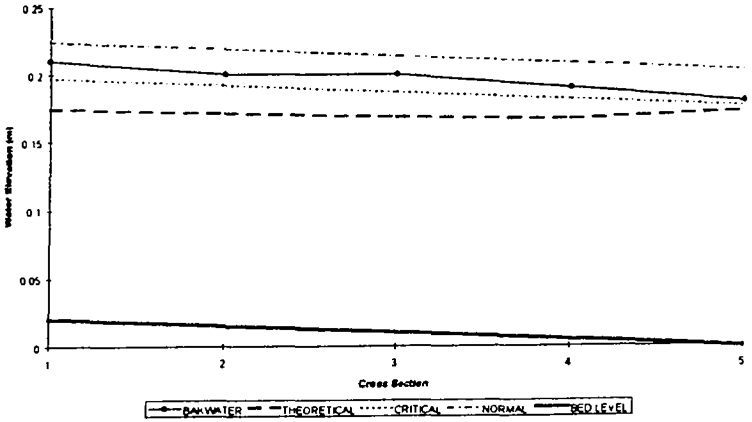
GRAPH 21 : *TEST 6 Part A2-CHANNEL Water Surface Profile*



GRAPH 22 : *TEST 6 Part A2 - LD01 Water Surface Profile*



GRAPH 23 : *TEST 6 Part A2 - GLC-BACKWATER Water Surface Profile*



GRAPH 24 : *TEST 6 Part A2 - BAKWATER Water Surface Profile*

Chainage (m)	Water Elevation (m)	Comment
CASE 1 - Steady Subcritical Calculation		
0	0.242	Time Step = 0.01, Min/Max iterations = 5/20
5	0.238	Prissman Factor = 0.5, Residual Limit = 0.1
10	0.239	Max No. iterations before optimal head correction = 15
15	0.236	Max allowable head correction = 0.1
20	0.170	Max allowable flow correction = 0.01
CASE 2 - Steady Subcritical Calculation		
0	0.380	Same as case 1 with :
5	0.380	Max allowable head correction = 0.01
10	0.370	
15	0.370	
20	0.170	
CASE 3 - Steady Subcritical Calculation		
0	0.189	Same as case 2 with :
5	0.238	Residual limit = 0.01
10	0.239	
15	0.236	
20	0.170	
CASE 1 - Steady Supercritical Calculation		
0	0.190	Time Step = 0.01, Min/Max iterations = 5/20
5	0.233	Prissman Factor = 0.5, Residual Limit = 0.1
10	0.234	Max No. iterations before optimal head correction = 15
15	0.231	Max allowable head correction = 0.1
20	0.231	Max allowable flow correction = 0.01
CASE 2 - Steady Supercritical Calculation		
0	0.190	Same as case 1 with :
5	0.380	Max allowable head correction = 0.01
10	0.370	
15	0.370	
20	0.360	
CASE 3 - Steady Supercritical Calculation		
0	0.190	Same as case 2 with :
5	0.380	Residual limit = 0.01
10	0.370	
15	0.370	
20	0.360	
Unsteady Supercritical Calculation		
0	0.190	Same as case 3 supercritical calculation
5	0.199	
10	0.190	
15	0.210	
20	0.181	

TABLE : 1 : *HYDRO 1D TEST 6 Part A1 Results from Investigations with Various Calculation Settings*

NRA BENCHMARKING TEST 6 PART A1

CS Ref	Chainage	Water Level	Total Head	Bed Level	Left Highest	Right Highest	Left Bank	Right Bank
1	0	0.2	0.23	0	0.4	0.4	0.15	0.15
1	5	0.21	0.24	0.01	0.41	0.41	0.16	0.16
1	10	0.21	0.24	0.01	0.41	0.41	0.16	0.16
1	15	0.22	0.25	0.02	0.42	0.42	0.17	0.17
1	20	0.22	0.26	0.02	0.42	0.42	0.17	0.17

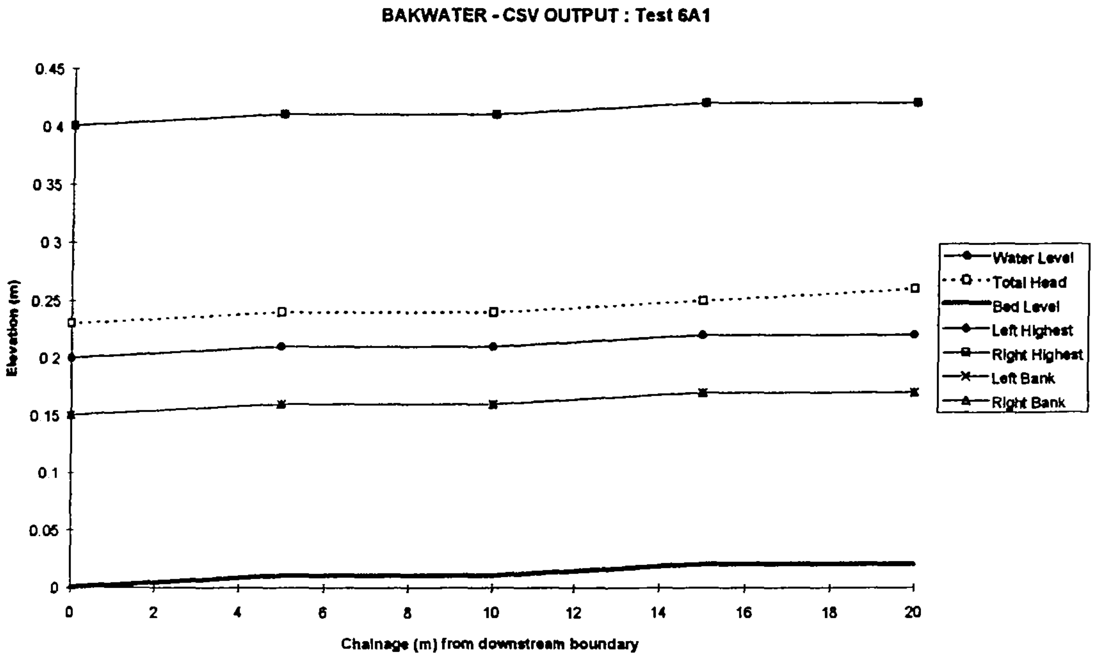
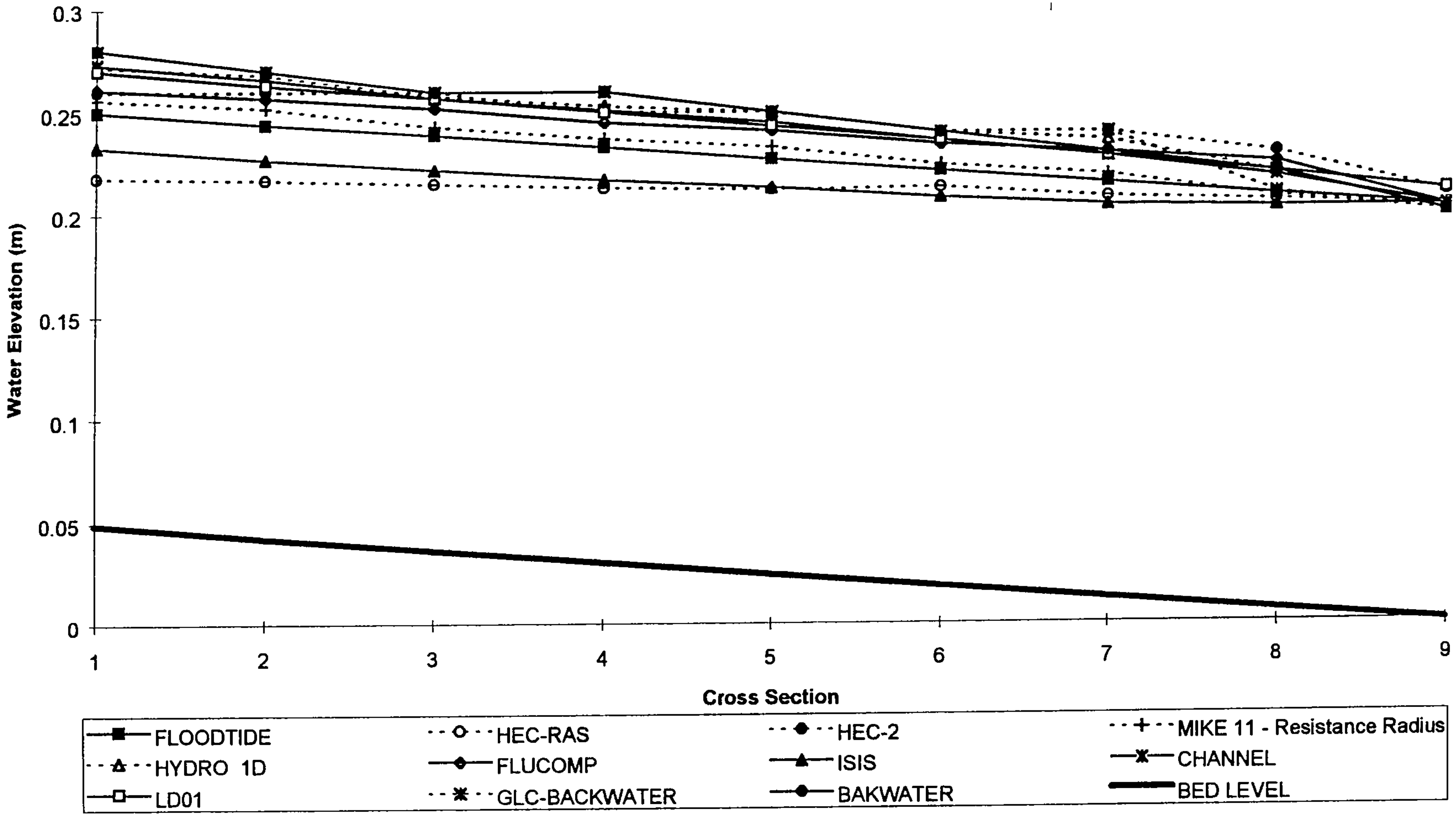
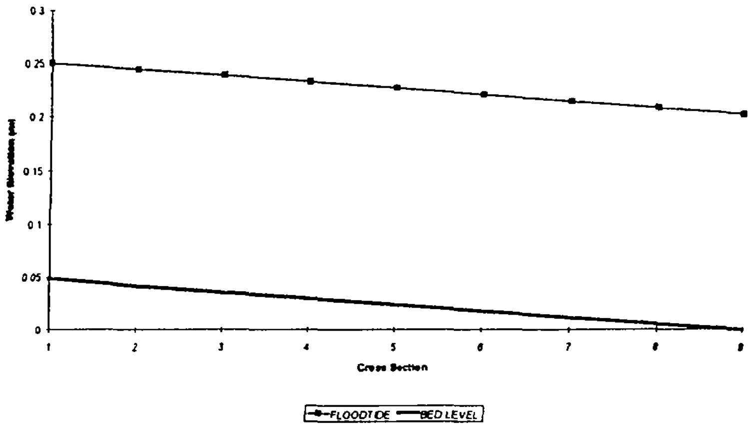


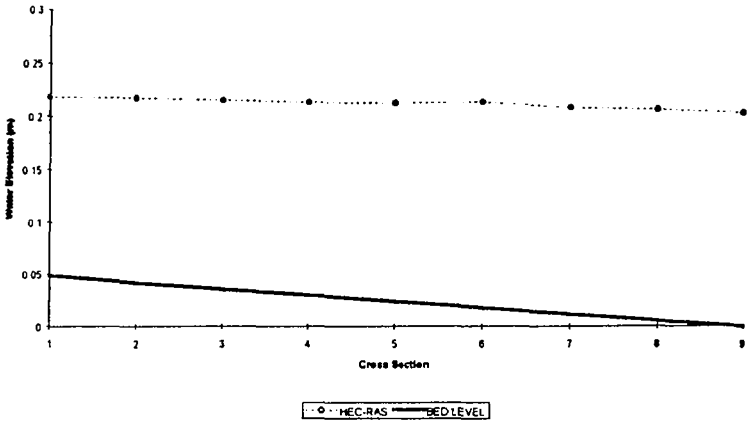
FIGURE 1 : *TEST 6A1 - Microsoft Excel Tabular Import and Graphical Processing of an BAKWATER - CSV Output File*



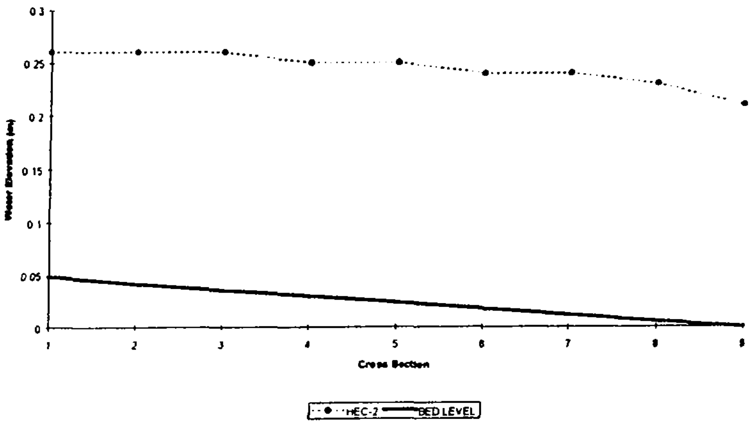
GRAPH 25 : *TEST 6 Part B - Comparison of Calculated Water Surface Profiles*



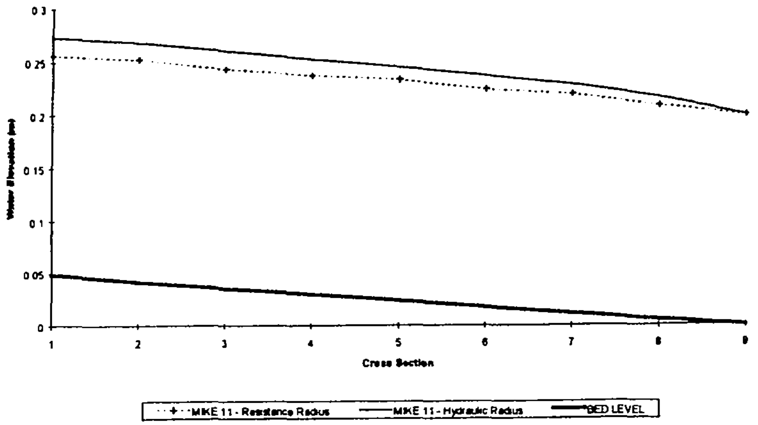
GRAPH 26 : *TEST 6 Part B - FLOODTIDE Water Surface Profile*



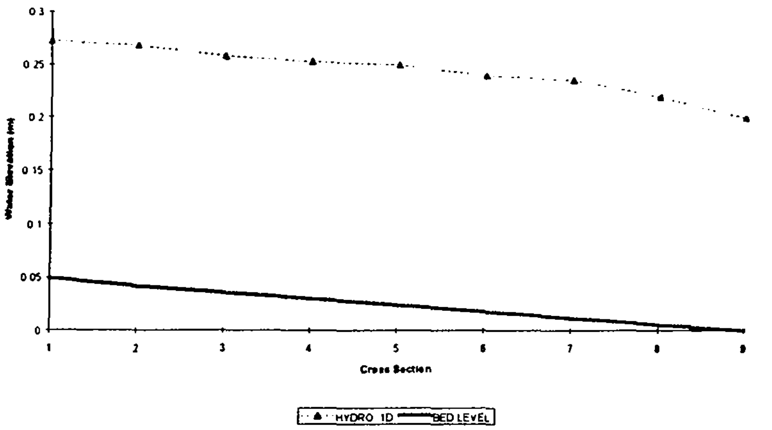
GRAPH 27 : *TEST 6 Part B - HEC-RAS Water Surface Profile*



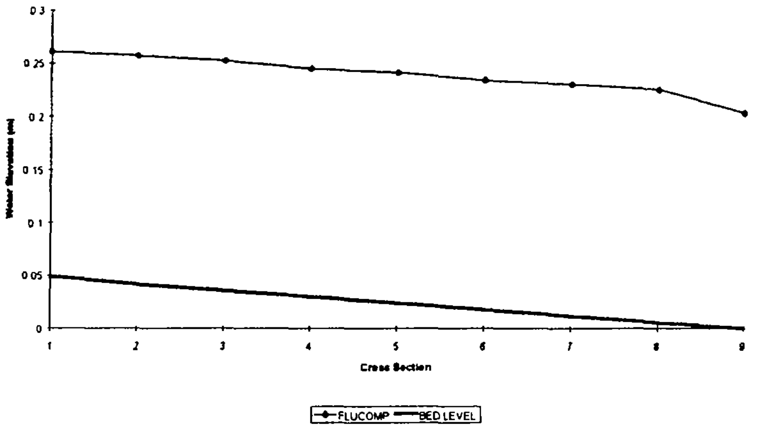
GRAPH 28 : *TEST 6 Part B - HEC2 Water Surface Profile*



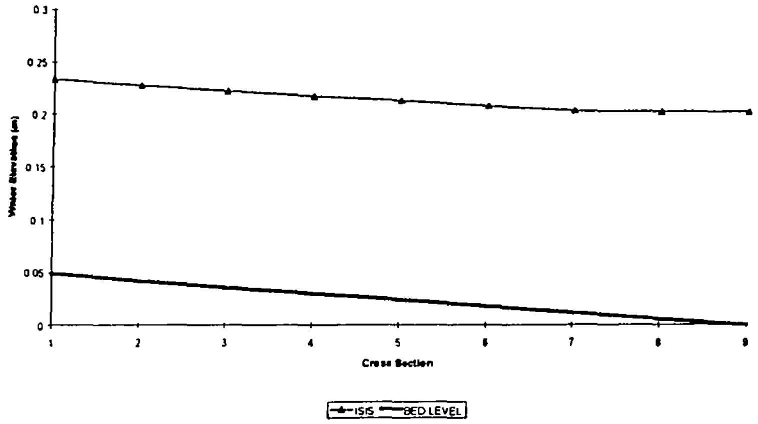
GRAPH 29 : *TEST 6 Part B - MIKE 11 Water Surface Profile*



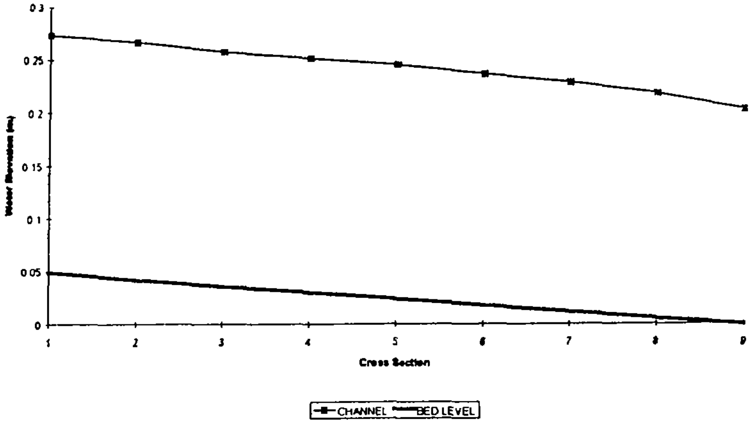
GRAPH 30 : *TEST 6 Part B - HYDRO 1D Water Surface Profile*



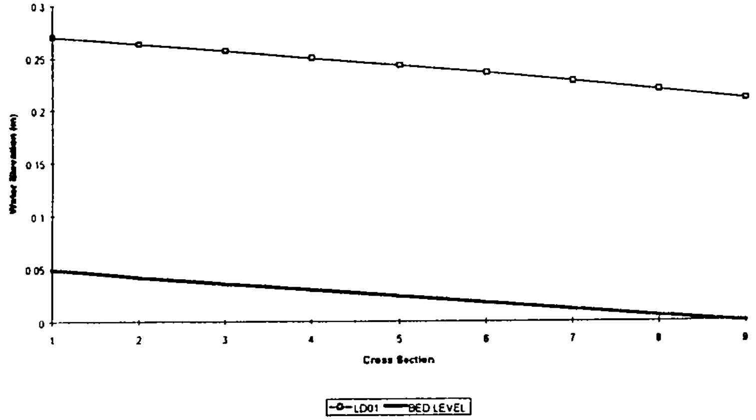
GRAPH 31 : *TEST 6 Part B - FLUCOMP Water Surface Profile*



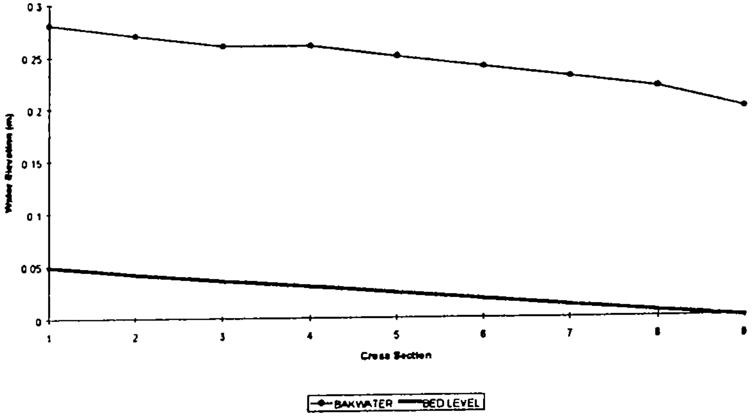
GRAPH 32 : *TEST 6 Part B - ISIS Water Surface Profile*



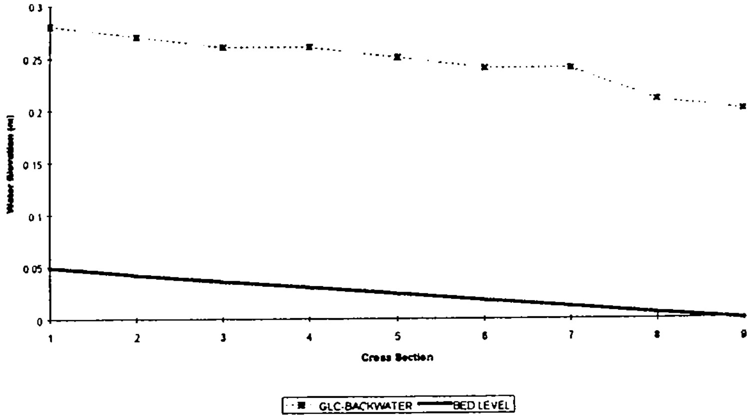
GRAPH 33 : *TEST 6 Part B - CHANNEL Water Surface Profile*



GRAPH 34 : *TEST 6 Part B - LD01 Water Surface Profile*



GRAPH 36 : *TEST 6 Part B - BAKWATER Water Surface Profile*



GRAPH 35 : *TEST 6 Part B - GLC-BACKWATER Water Surface Profile*

Appendix F

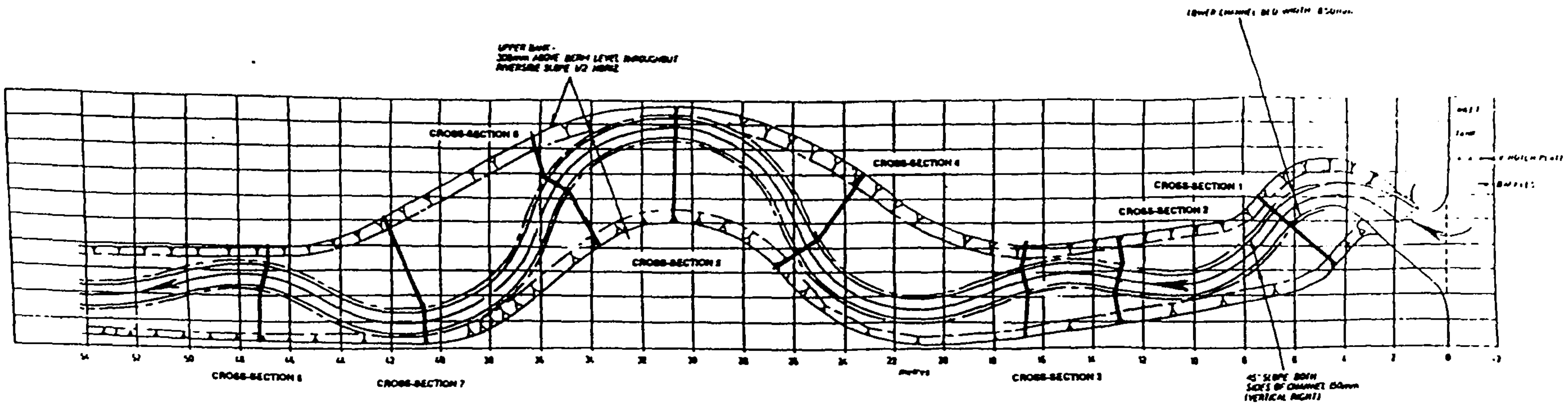


FIGURE 1 : *TEST 7 - FCF Blackwater Model*

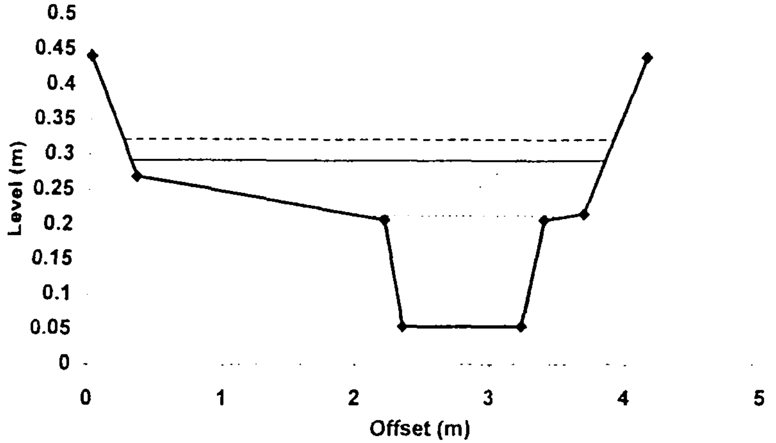


FIGURE 2 : *TEST 7 - Cross Section 1*

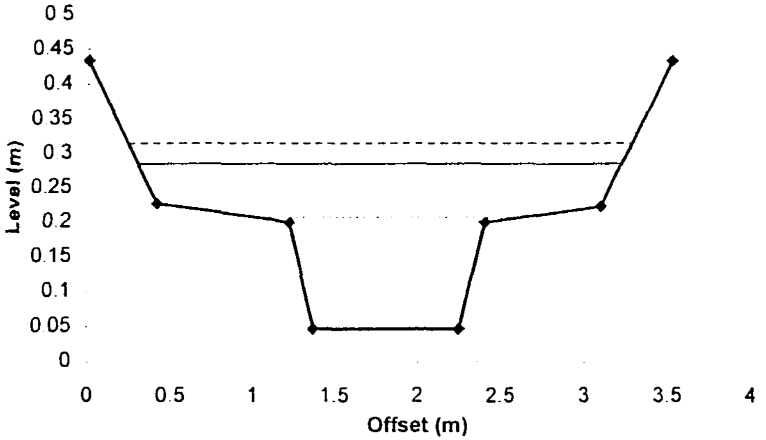


FIGURE 3 : *TEST 7 - Cross Section 2*

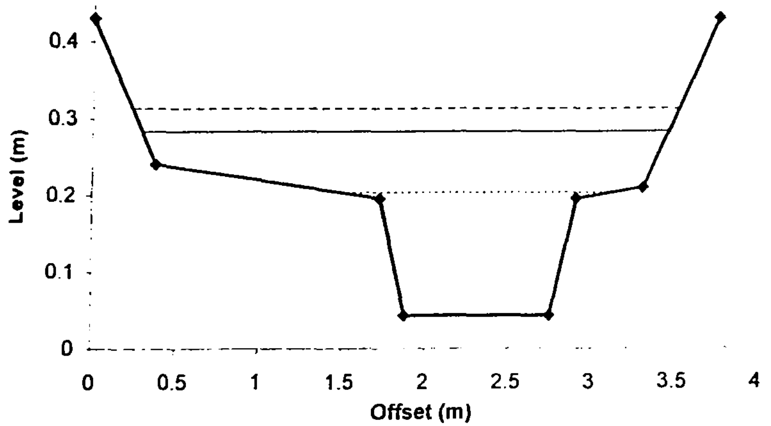


FIGURE 4 : *TEST 7 - Cross Section 3*

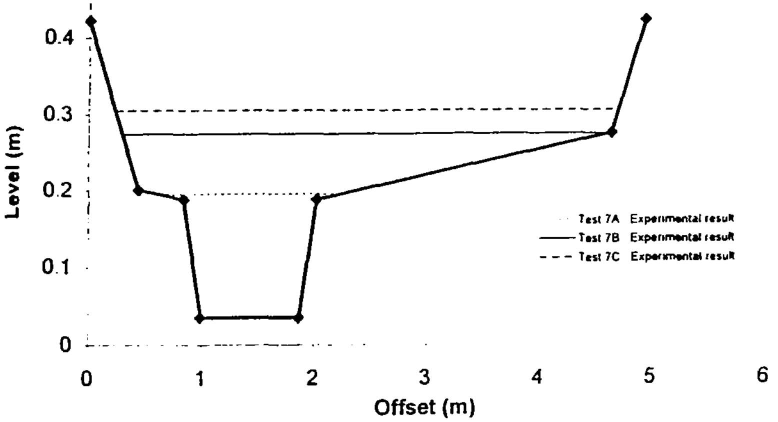


FIGURE 5 : *TEST 7 - Cross Section 4*

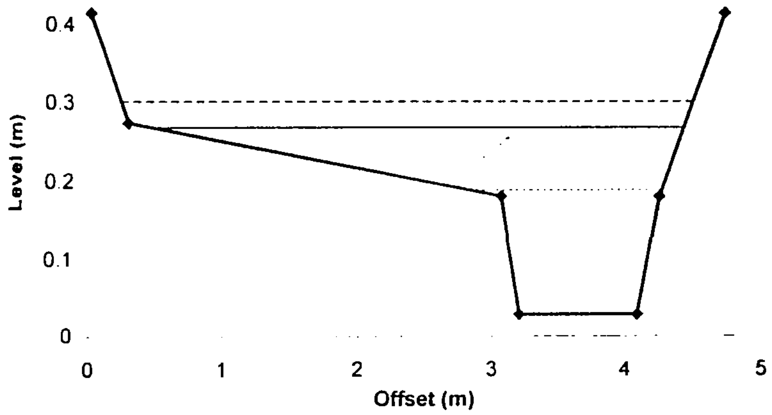


FIGURE 6 : *TEST 7 - Cross Section 5*

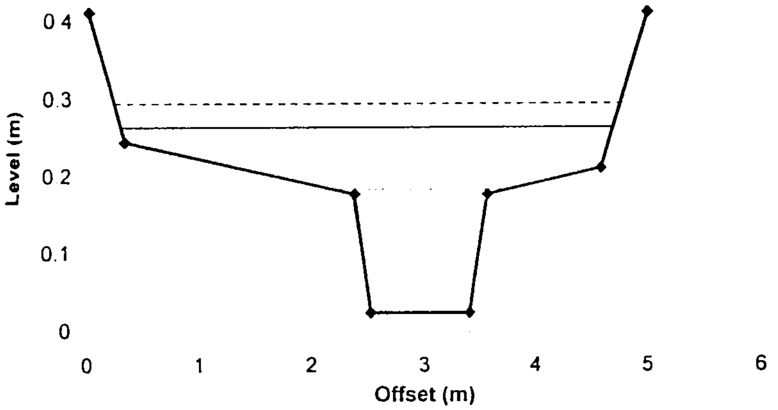


FIGURE 7 : *TEST 7 - Cross Section 6*

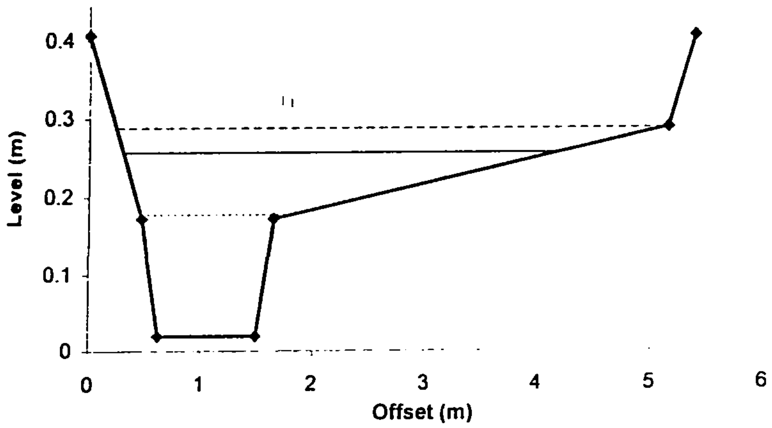


FIGURE 8 : *TEST 7 - Cross Section 7*

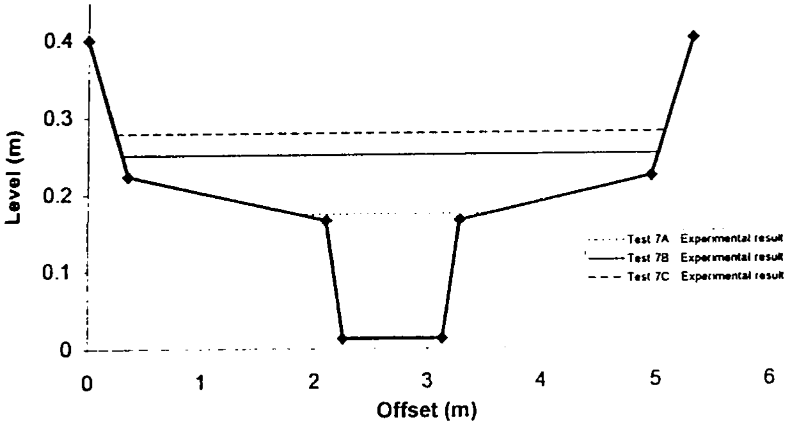
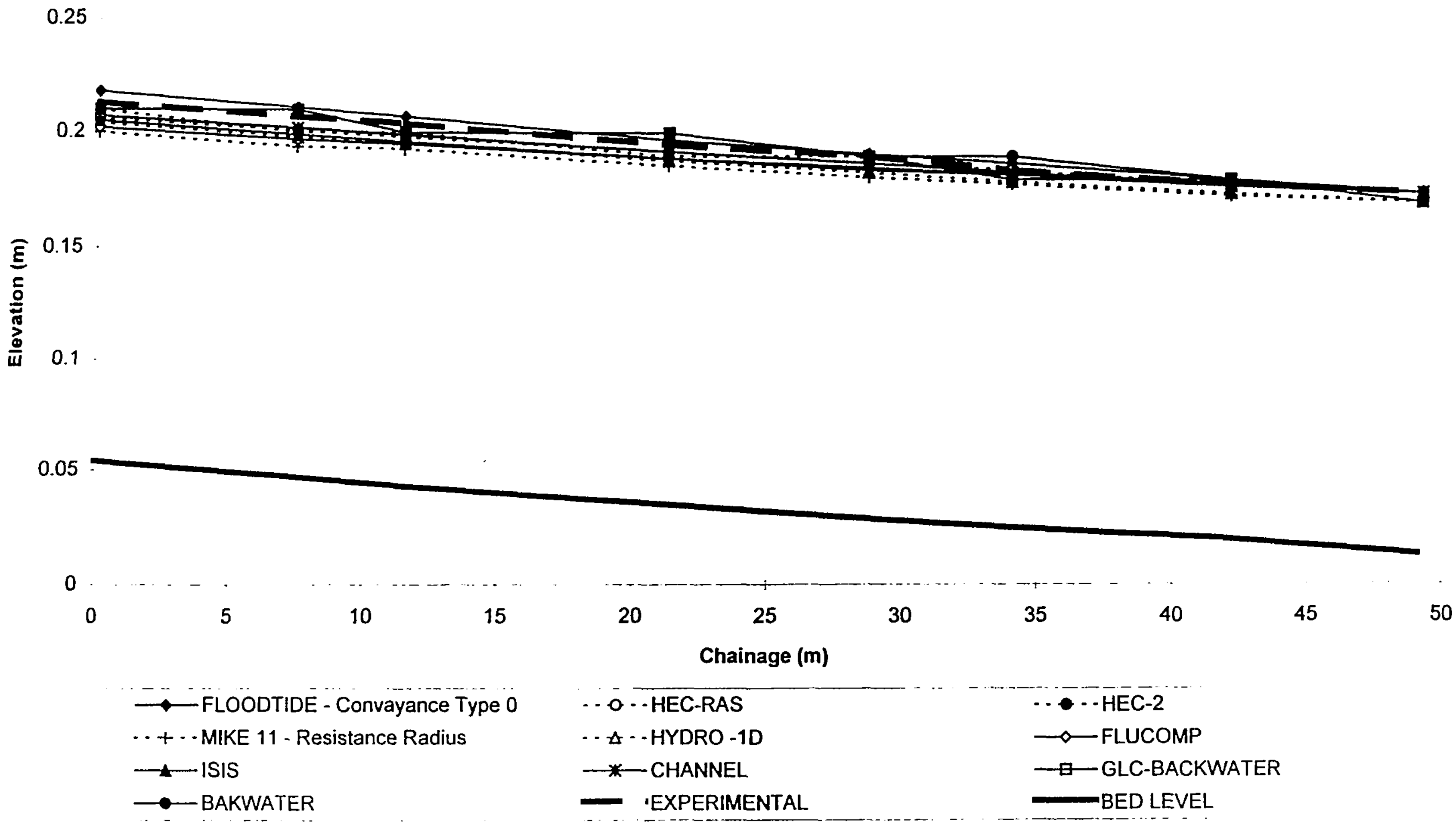
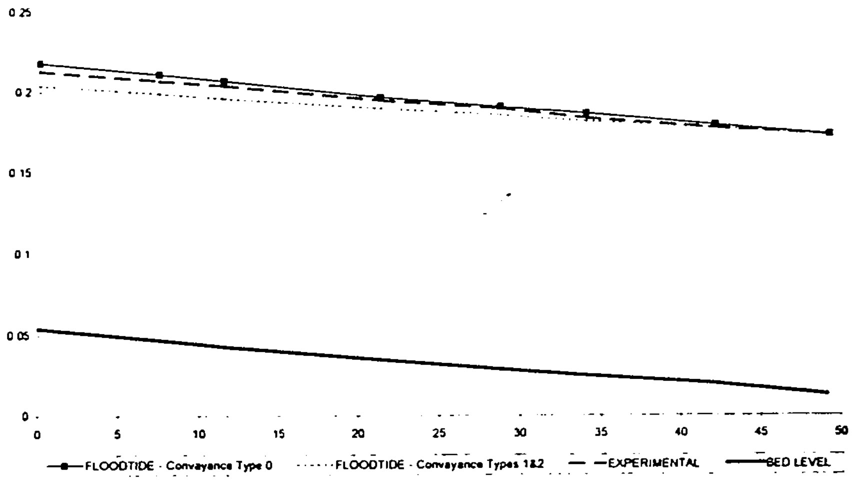


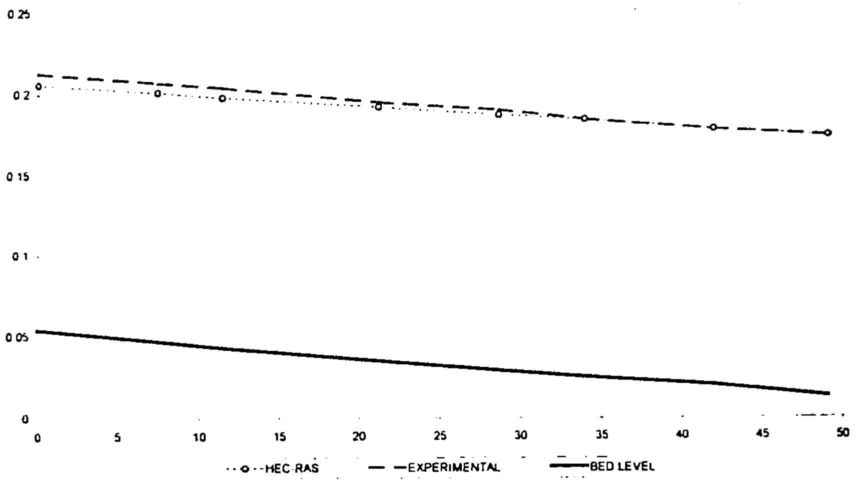
FIGURE 9 : *TEST 7 - Cross Section 8*



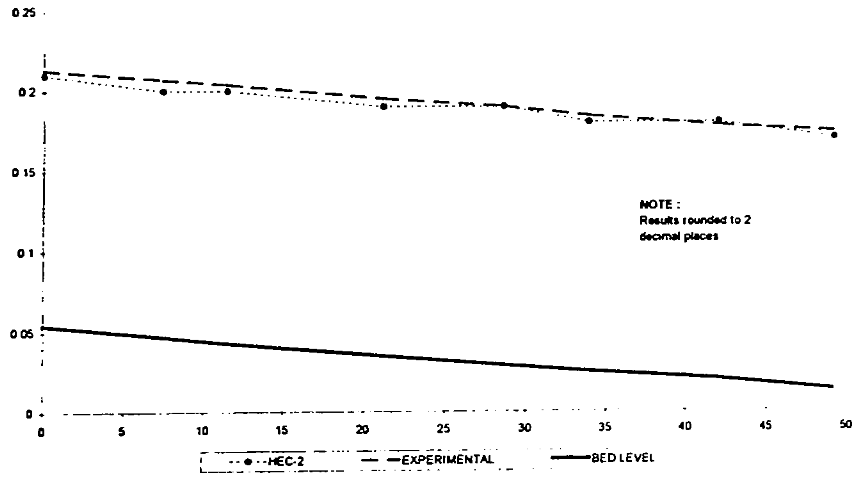
GRAPH 1 : *TEST 7A - Comparison of Water Surface Profiles*



GRAPH 2 : *TEST 7A - FLOODTIDE Water Surface Profile*

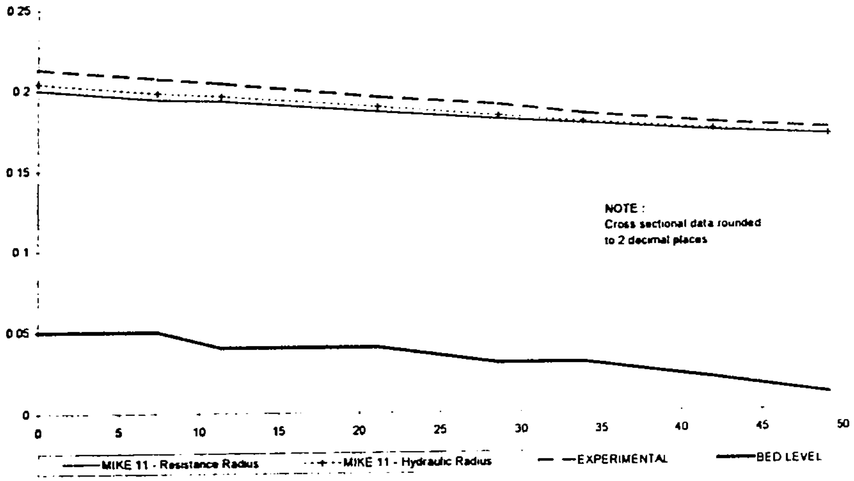


GRAPH 3 : *TEST 7A - HEC-RAS Water Surface Profile*



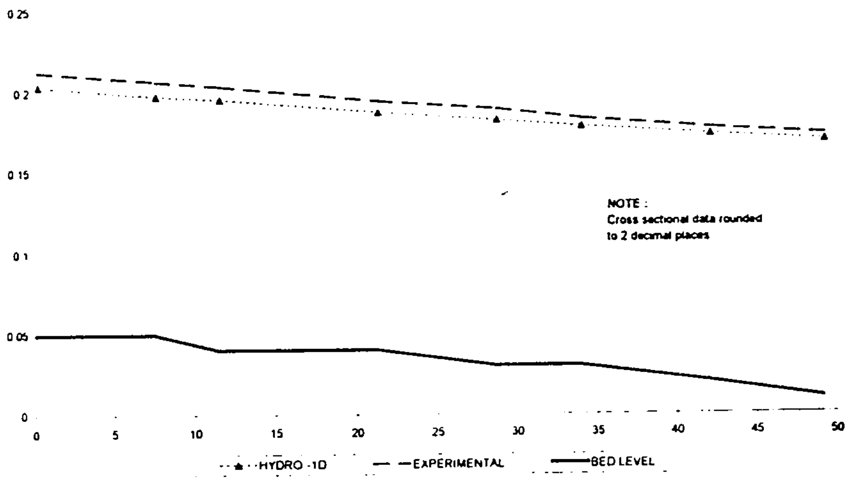
NOTE :
Results rounded to 2
decimal places

GRAPH 4 : *TEST 7A - HEC2 Water Surface Profile*

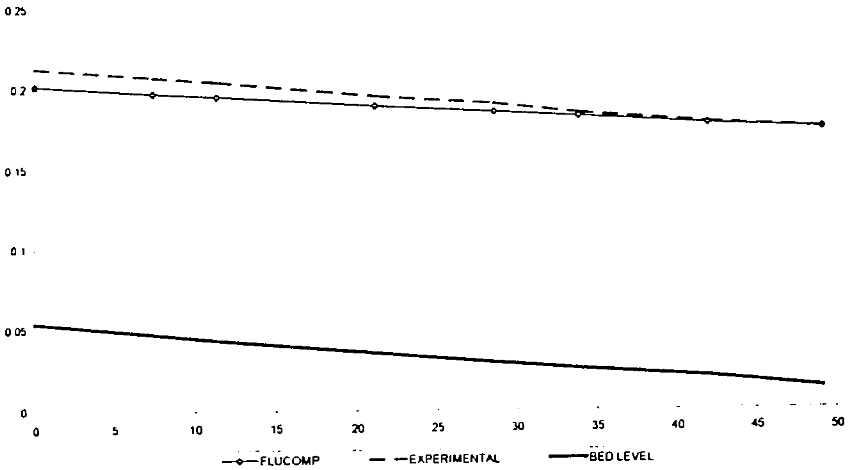


NOTE :
Cross sectional data rounded
to 2 decimal places

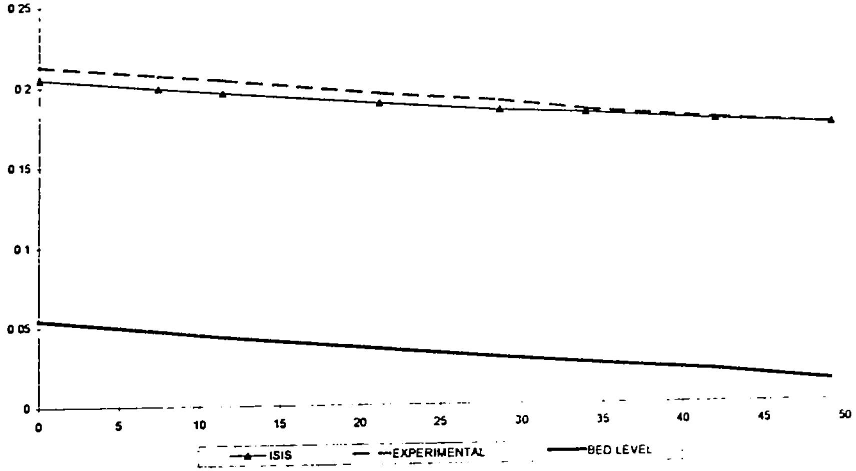
GRAPH 5 : *TEST 7A - MIKE 11 Water Surface Profile*



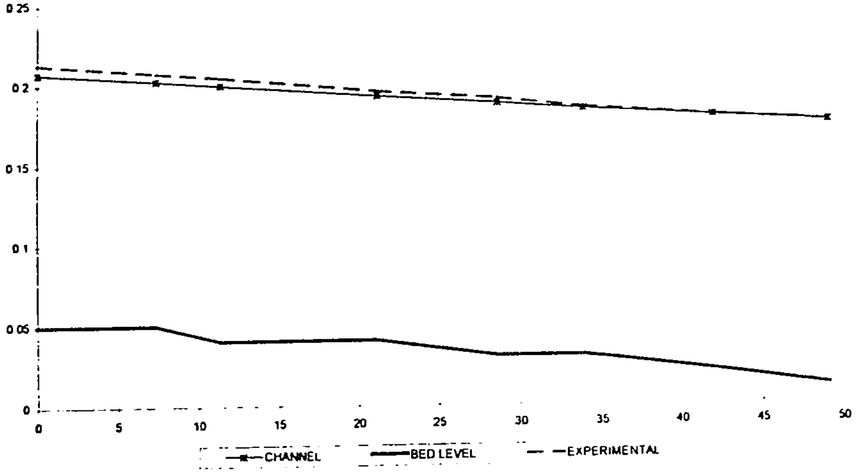
GRAPH 6 : *TEST 7A - HYDRO 1D Water Surface Profile*



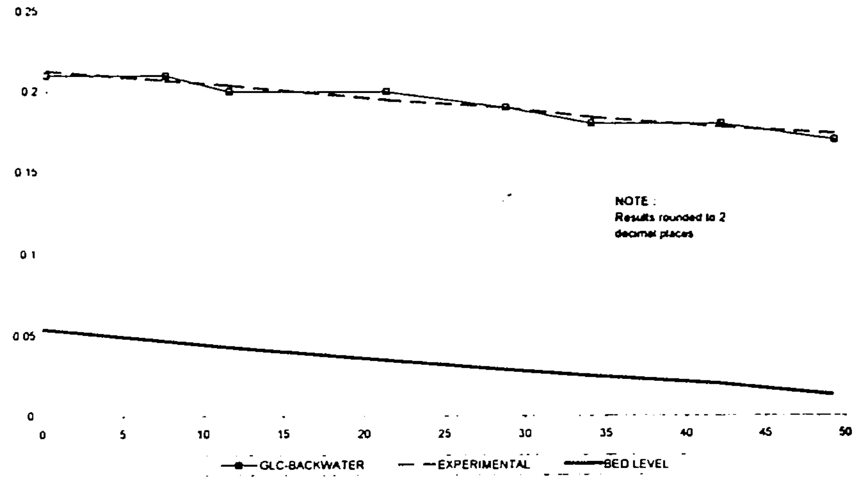
GRAPH 7 : *TEST 7A - FLUCOMP Water Surface Profile*



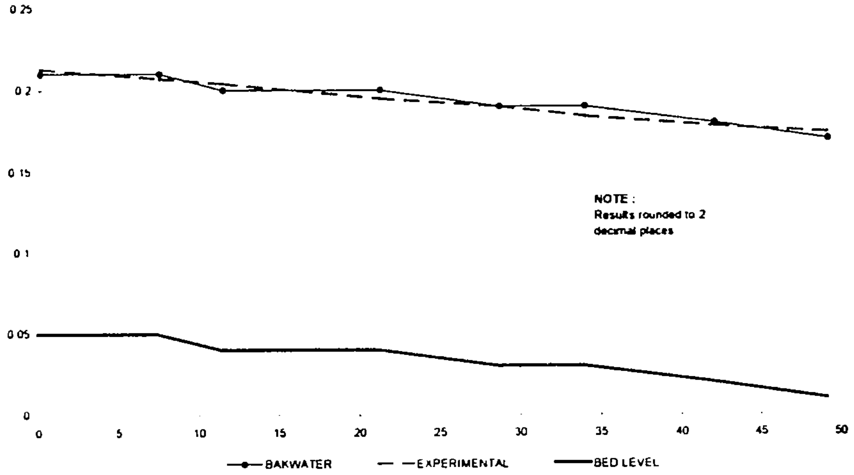
GRAPH 8 : *TEST 7A - ISIS Water Surface Profile*



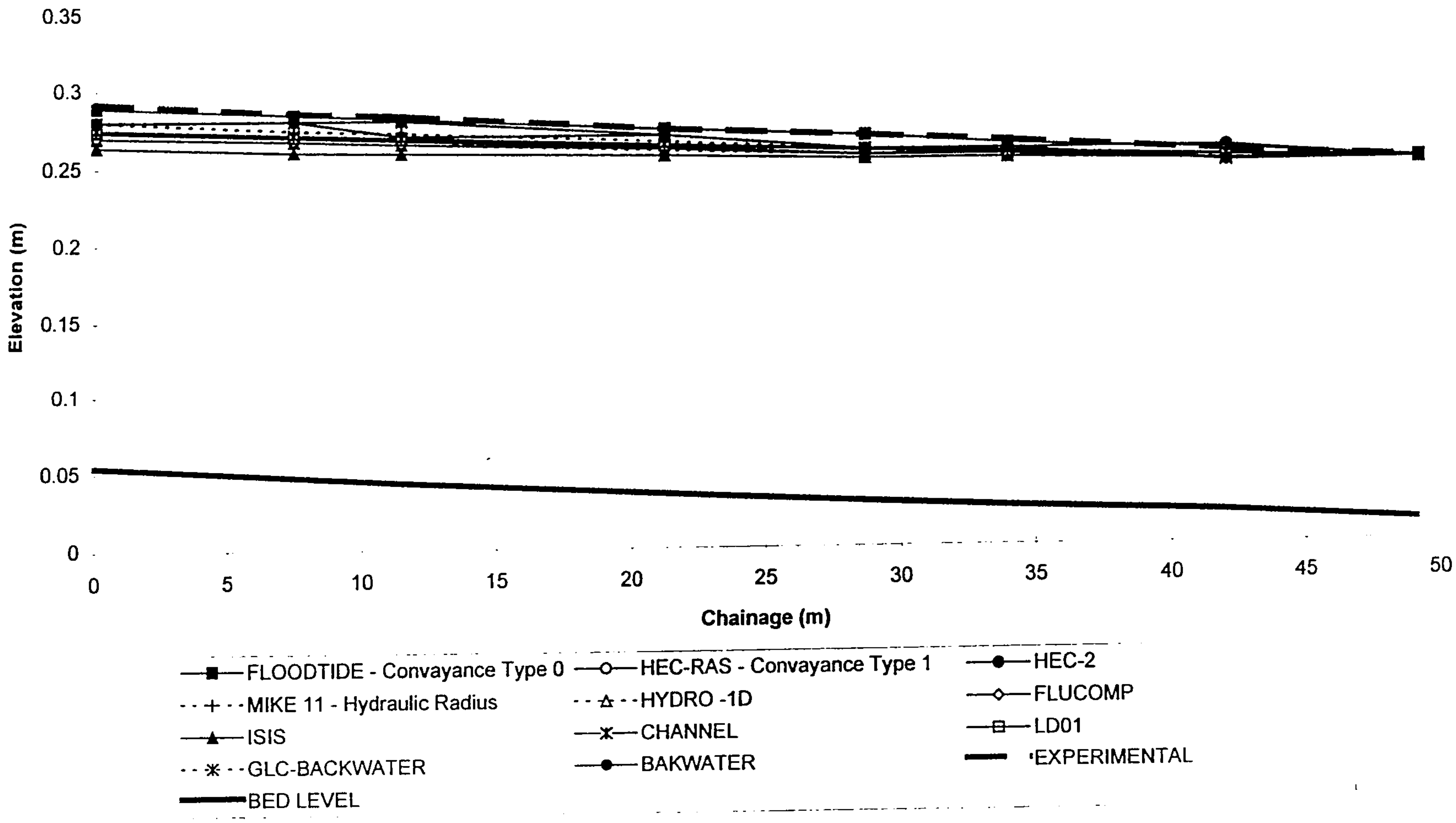
GRAPH 9 : *TEST 7A - CHANNEL Water Surface Profile*



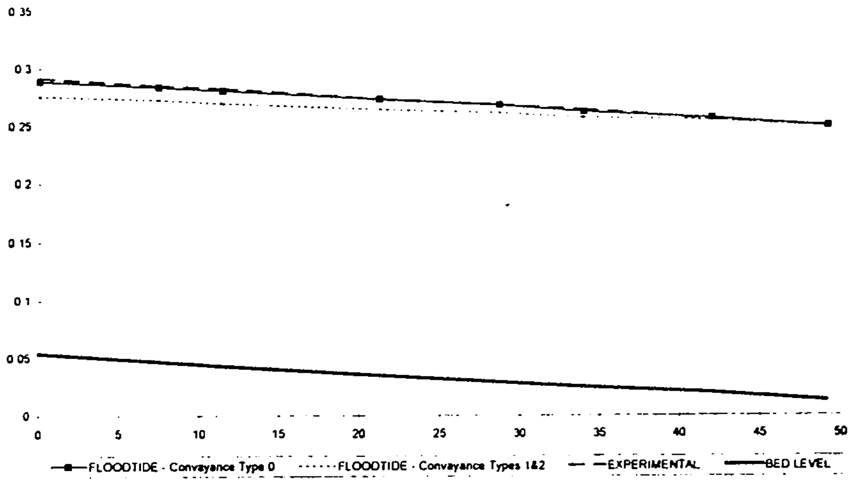
GRAPH 10 : *TEST 7A – GLC-BACKWATER Water Surface Profile*



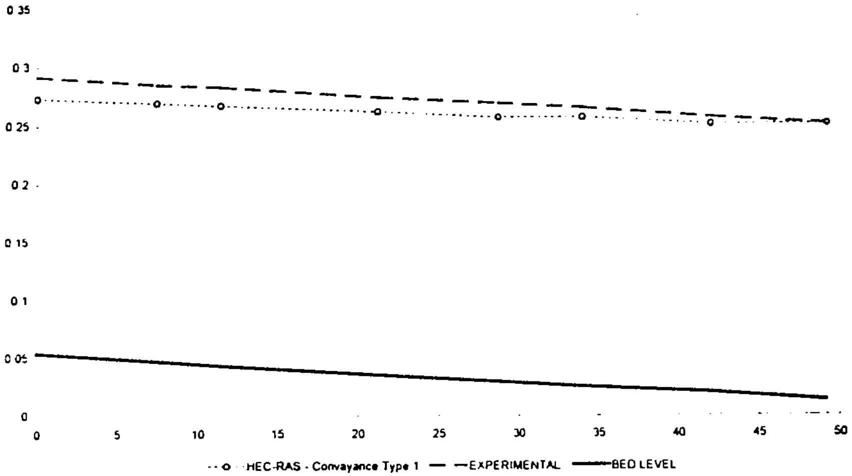
GRAPH 11 : *TEST 7A - BAKWATER Water Surface Profile*



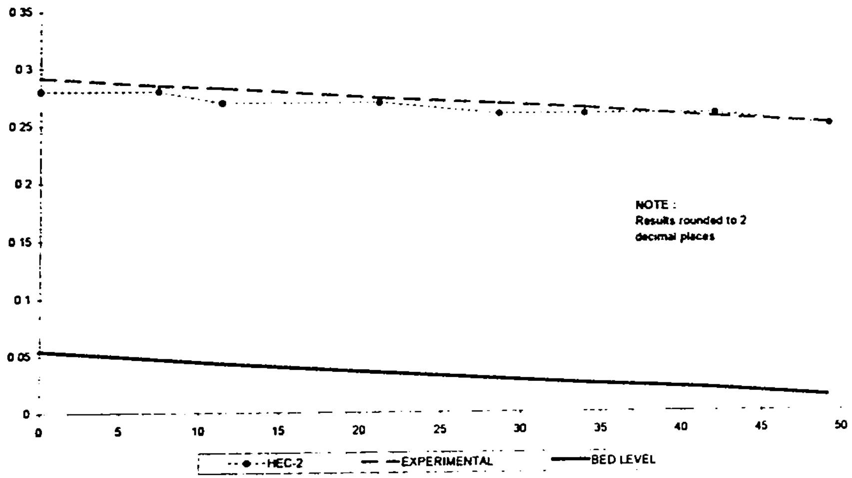
GRAPH 12 : *TEST 7B - Comparison of Calculated Water Surface Profiles*



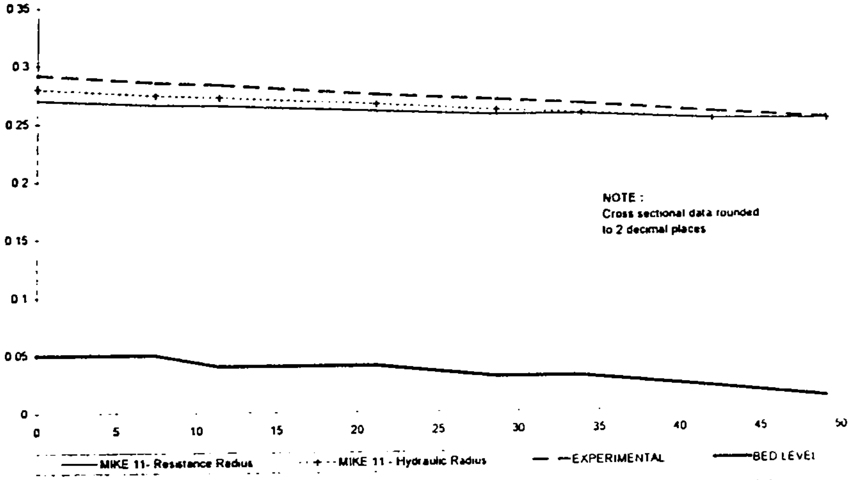
GRAPH 13 : *TEST 7B - FLOODTIDE Water Surface Profile*



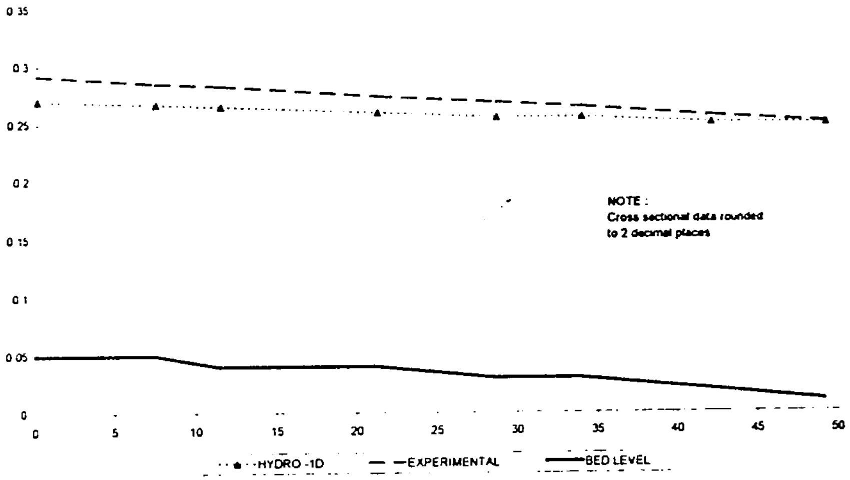
GRAPH 14 : *TEST 7B - HEC-RAS Water Surface Profile*



GRAPH 15 : *TEST 7B - HEC-2 Water Surface Profile*

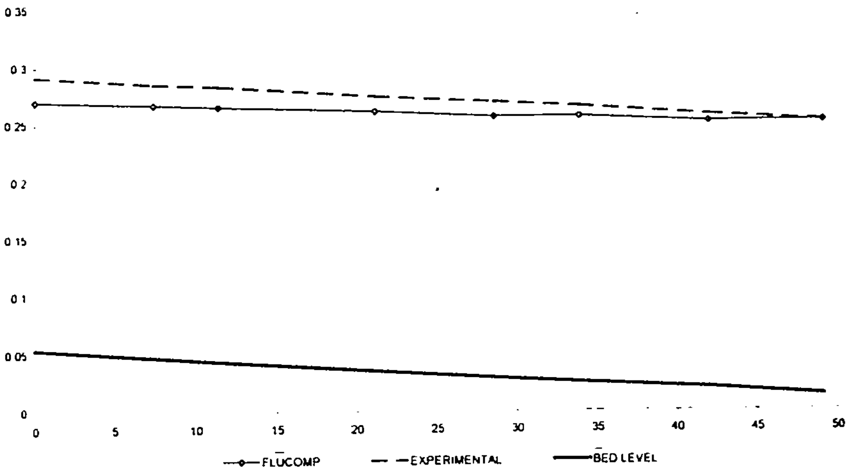


GRAPH 16 : *TEST 7B - MIKE 11 Water Surface Profile*

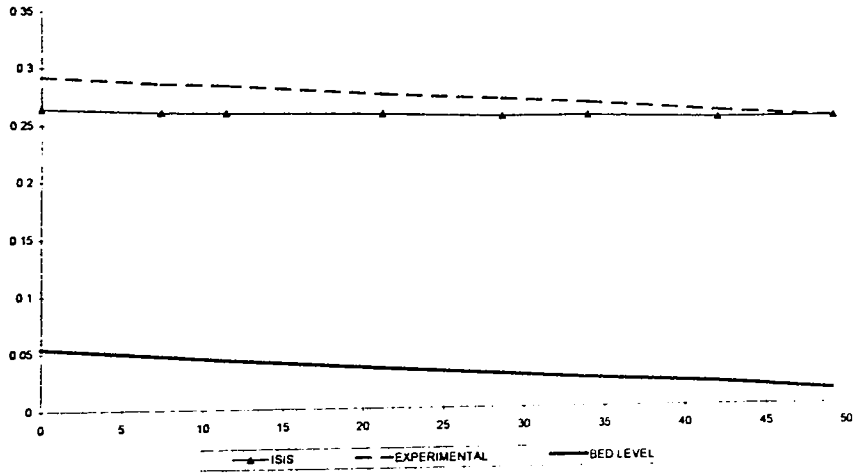


NOTE :
Cross sectional data rounded
to 2 decimal places

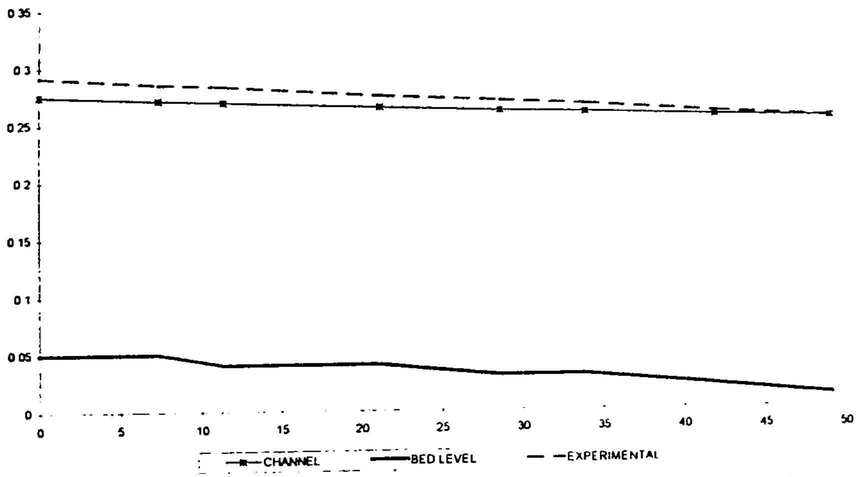
GRAPH 17 : *TEST 7B - HYDRO-1D Water Surface Profile*



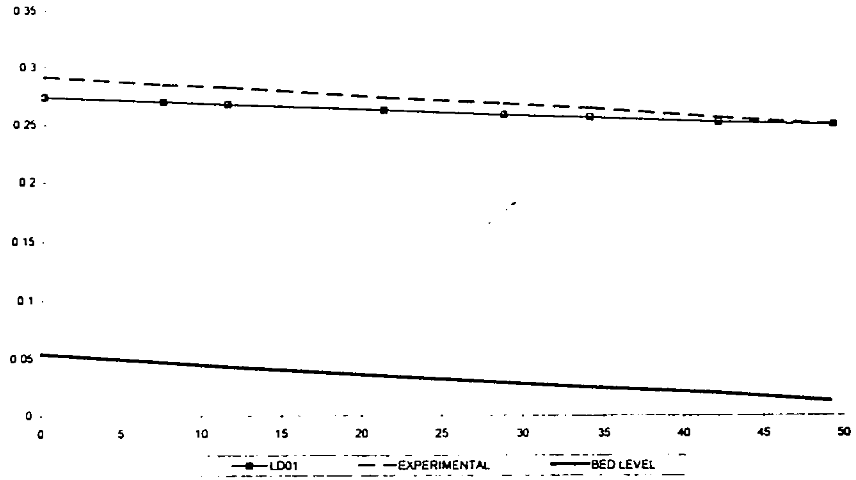
GRAPH 18 : *TEST 7B - FLUCOMP Water Surface Profile*



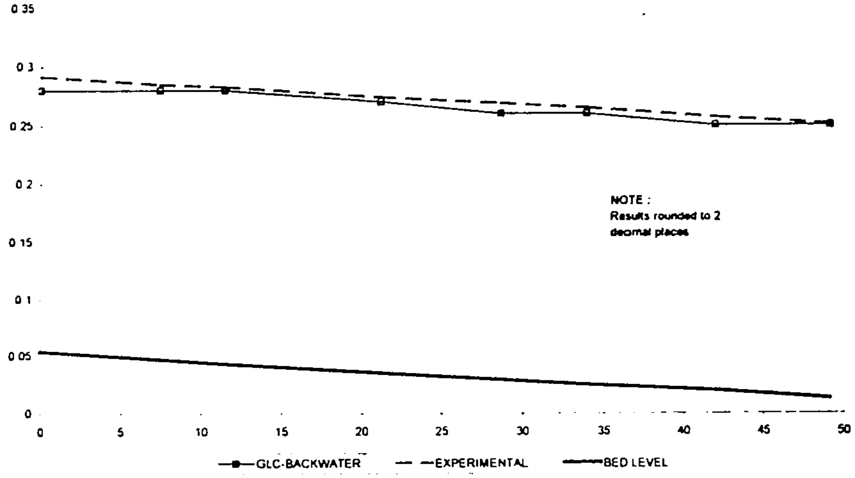
GRAPH 19 : *TEST 7B - ISIS Water Surface Profile*



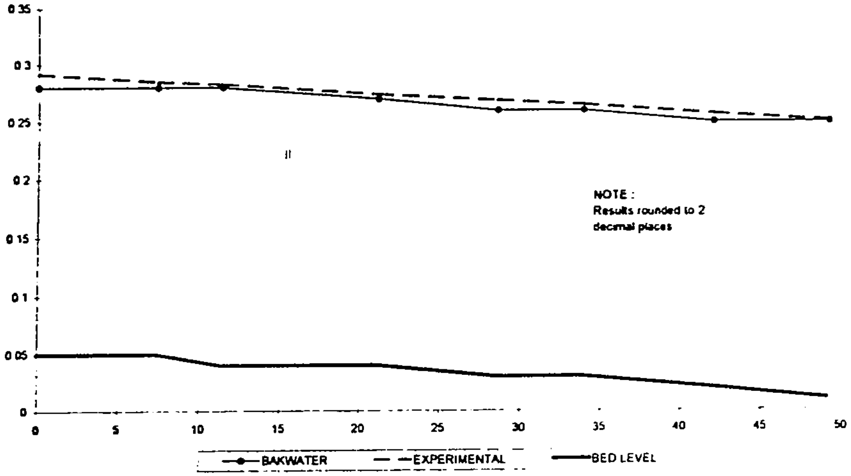
GRAPH 20 : *TEST 7B - CHANNEL Water Surface Profile*



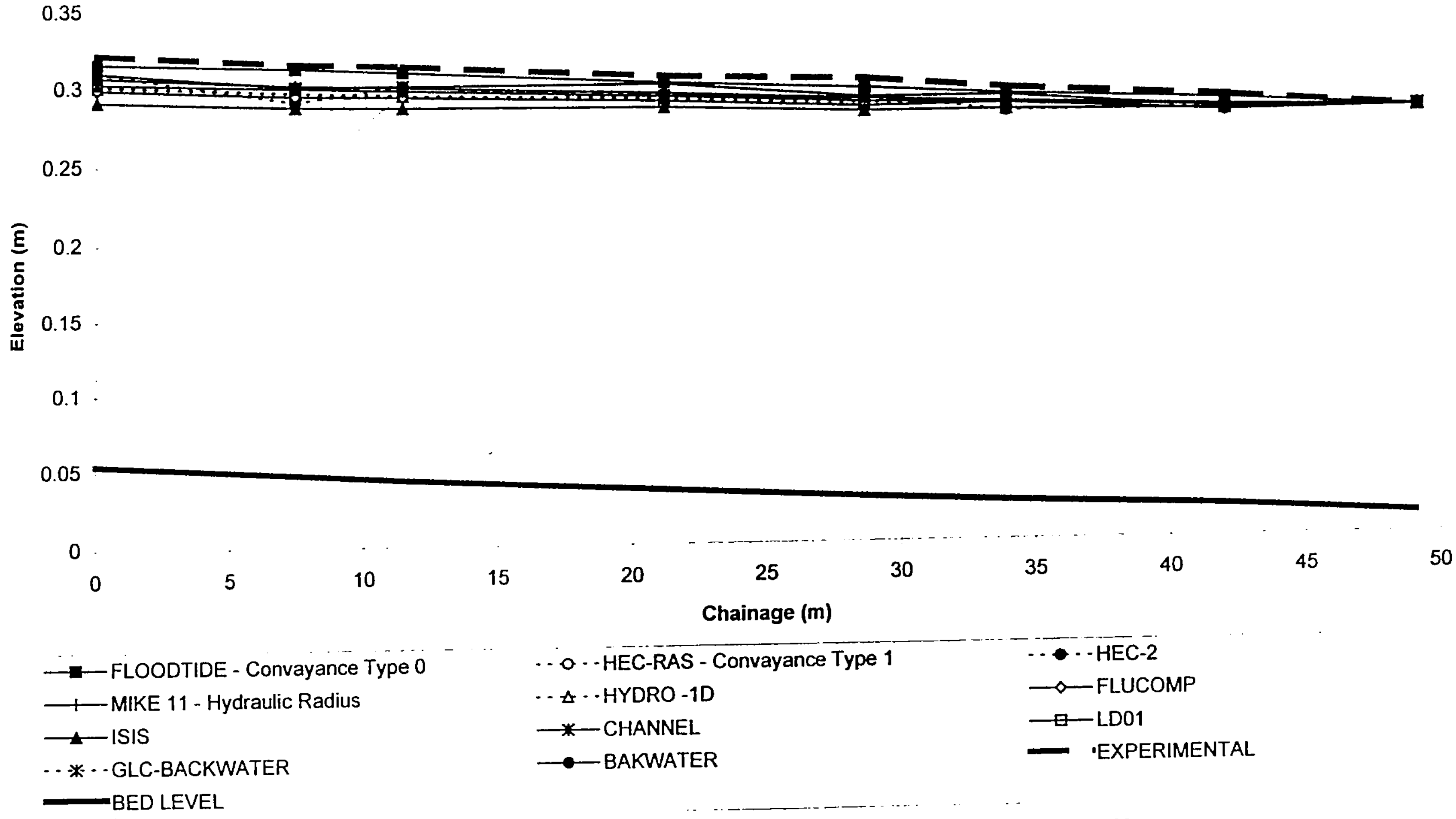
GRAPH 21 : *TEST 7B - LD01 Water Surface Profile*



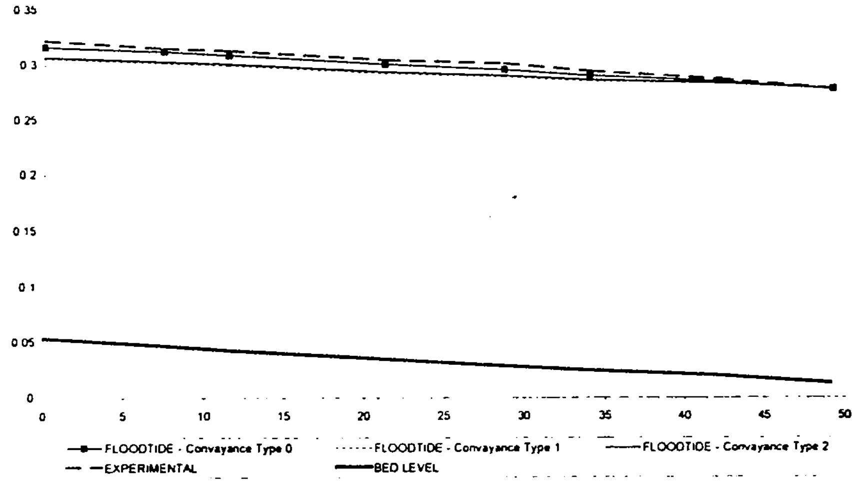
GRAPH 22 : *TEST 7B - GLC-BACKWATER Water Surface Profile*



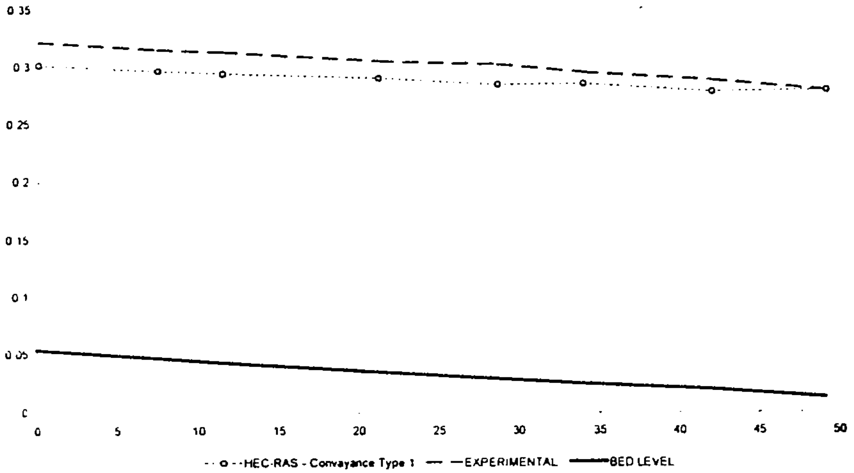
GRAPH 23 : *TEST 7B - BAKWATER Water Surface Profile*



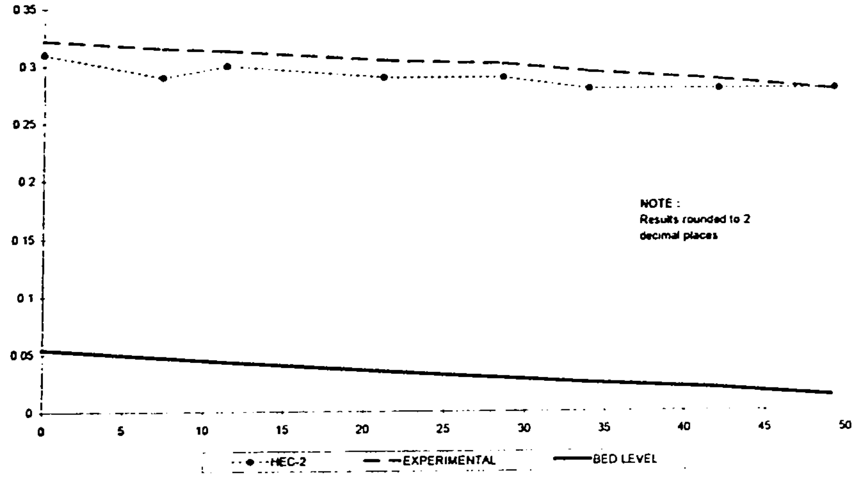
GRAPH 24 : *TEST 7C - Comparison of Calculated Water Surface Profile*



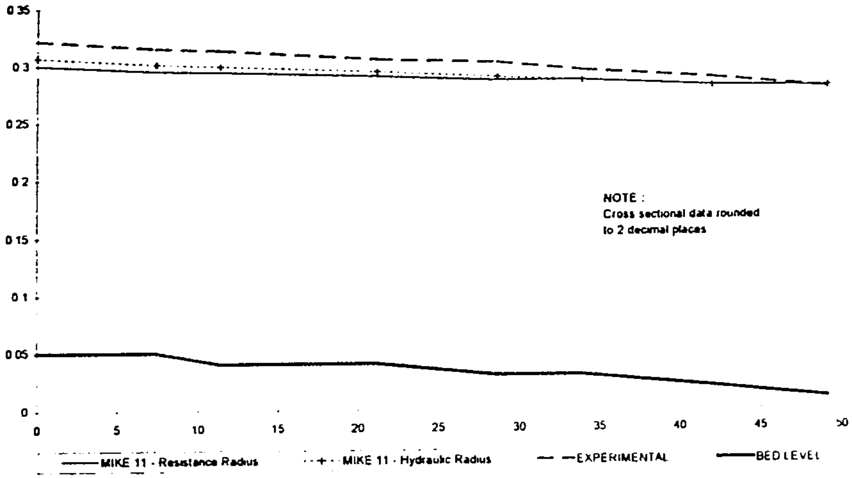
GRAPH 25 : *TEST 7C - FLOODTIDE Water Surface Profile*



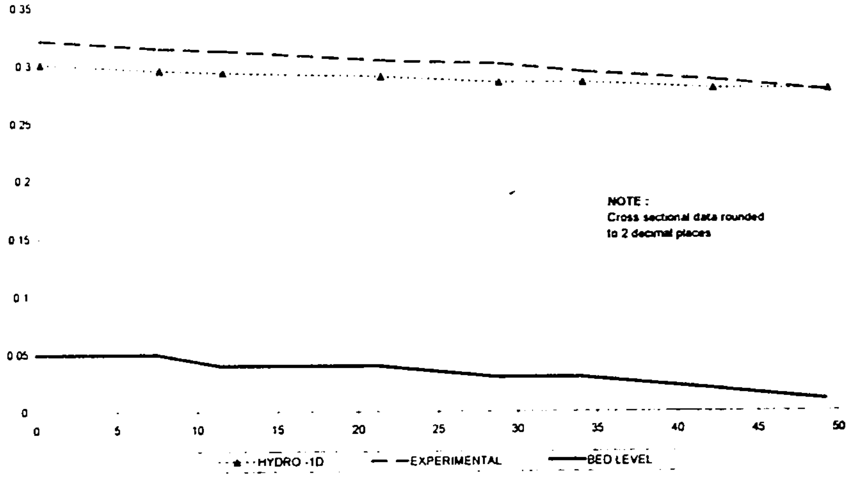
GRAPH 26 : *TEST 7C -HEC-RAS Water Surface Profile*



GRAPH 27 : *TEST 7C - HEC-2 Water Surface Profile*

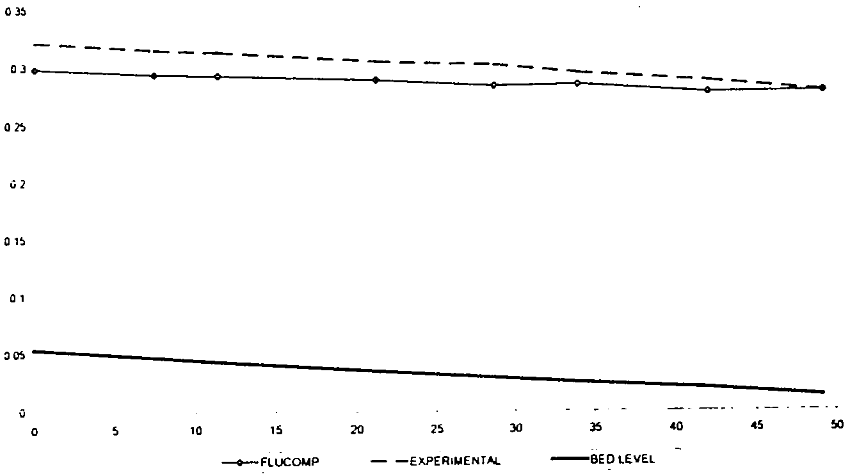


GRAPH 28 : *TEST 7C - MIKE 11 Water Surface Profile*

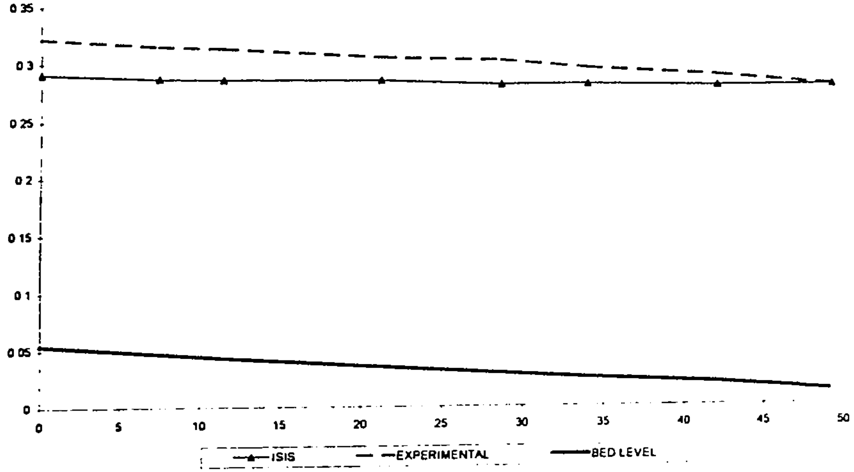


NOTE :
Cross sectional data rounded
to 2 decimal places

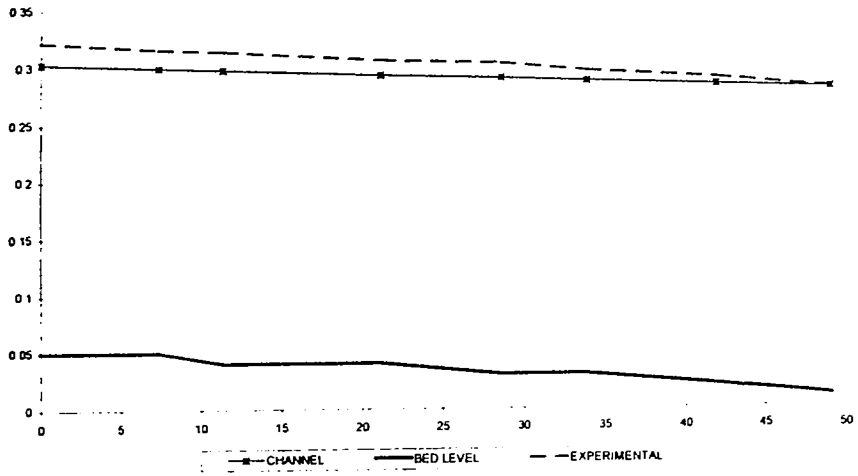
GRAPH 29 : *TEST 7C -HYDRO-1D Water Surface Profile*



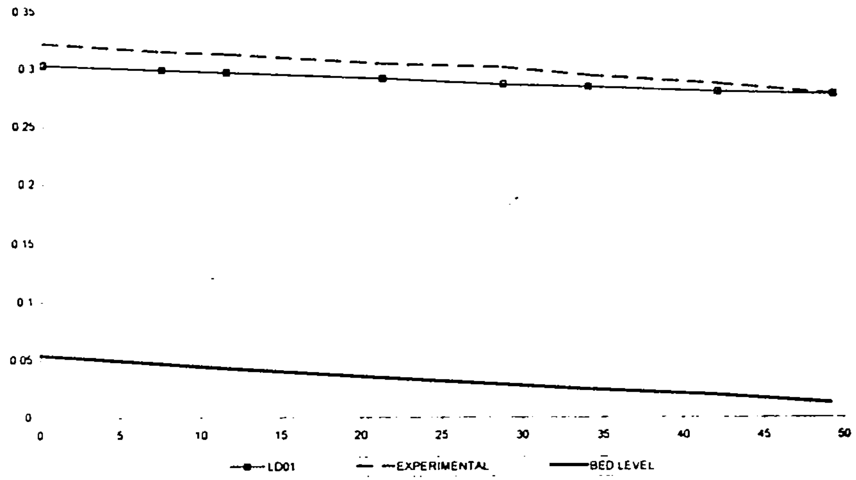
GRAPH 30 : *TEST 7C -FLUCOMP Water Surface Profile*



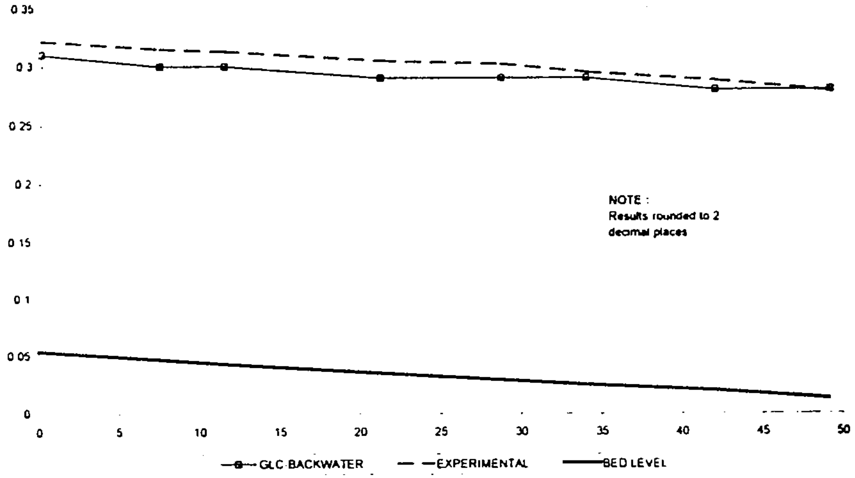
GRAPH 31 : *TEST 7C - ISIS Water Surface Profile*



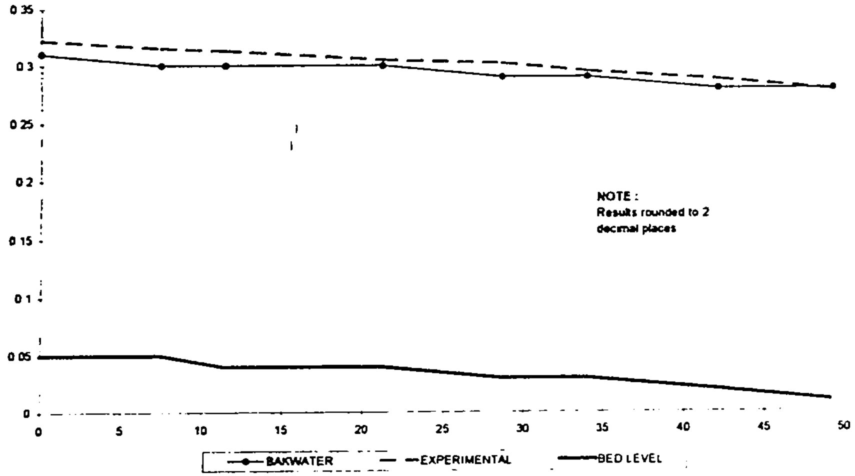
GRAPH 32 : *TEST 7C -CHANNEL Water Surface Profile*



GRAPH 33 : *TEST 7C -LD01 Water Surface Profile*



GRAPH 34 : *TEST 7C -GLC-BACKWATER Water Surface Profile*

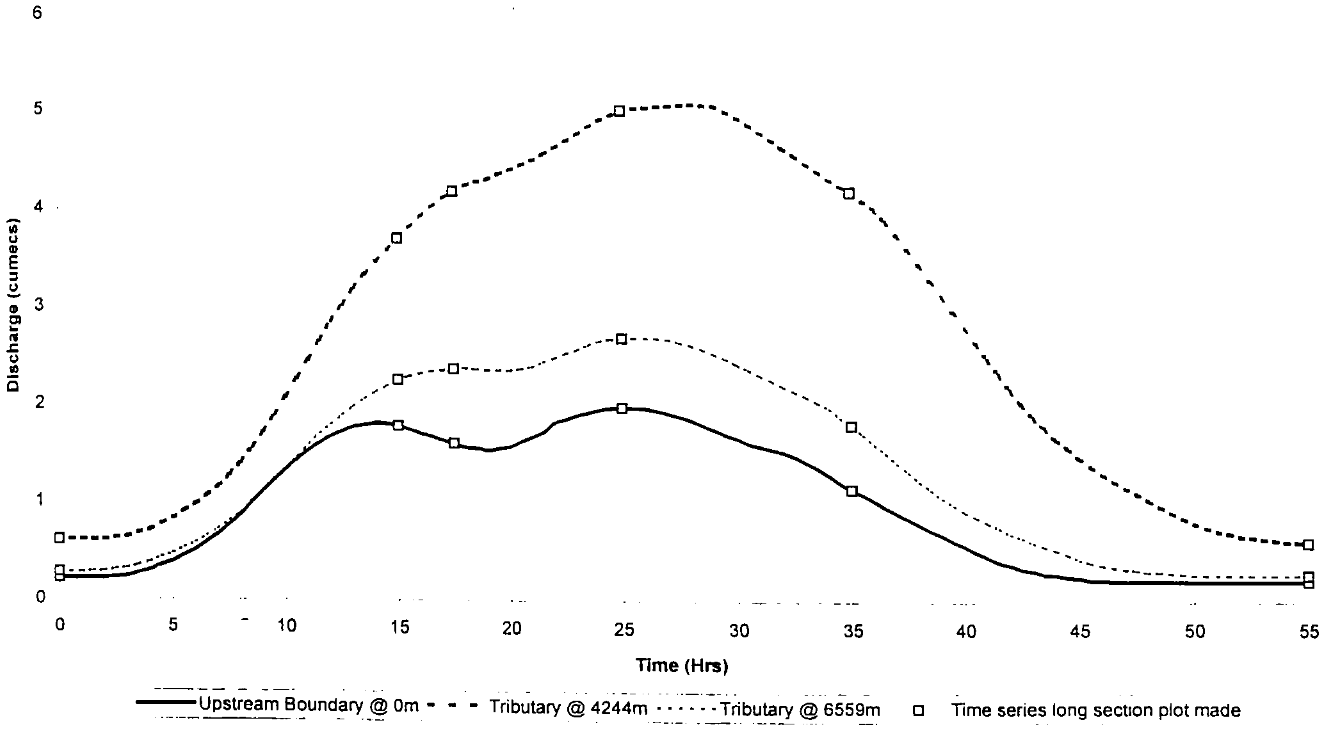


GRAPH 35 : *TEST 7C - BAKWATER Water Surface Profile*

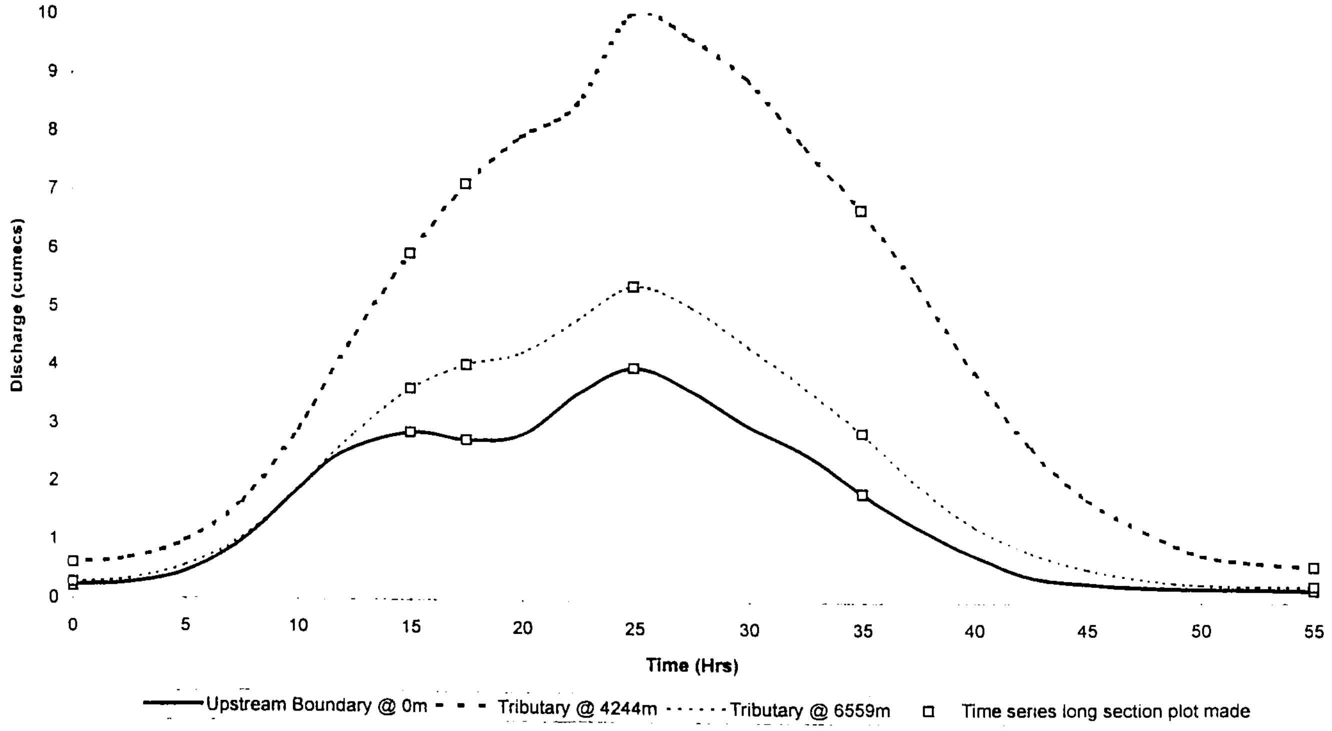
	Left Overbank Reach Length (m)	Main Channel Length (m)	Right Overbank Reach Length (m)
SECTION 8	NOT REQUIRED		
SECTION 7	7.14	7.14	6.07
SECTION 6	7.83	8.07	7.42
SECTION 5	4.07	5.29	5.29
SECTION 4	7.36	7.43	6.89
SECTION 3	10.24	9.75	8.78
SECTION 2	4.12	4.0	3.76
SECTION 1	8.79	7.39	6.65

TABLE 1 : TEST 7 - *HEC-RAS Overbank and Channel Lengths*

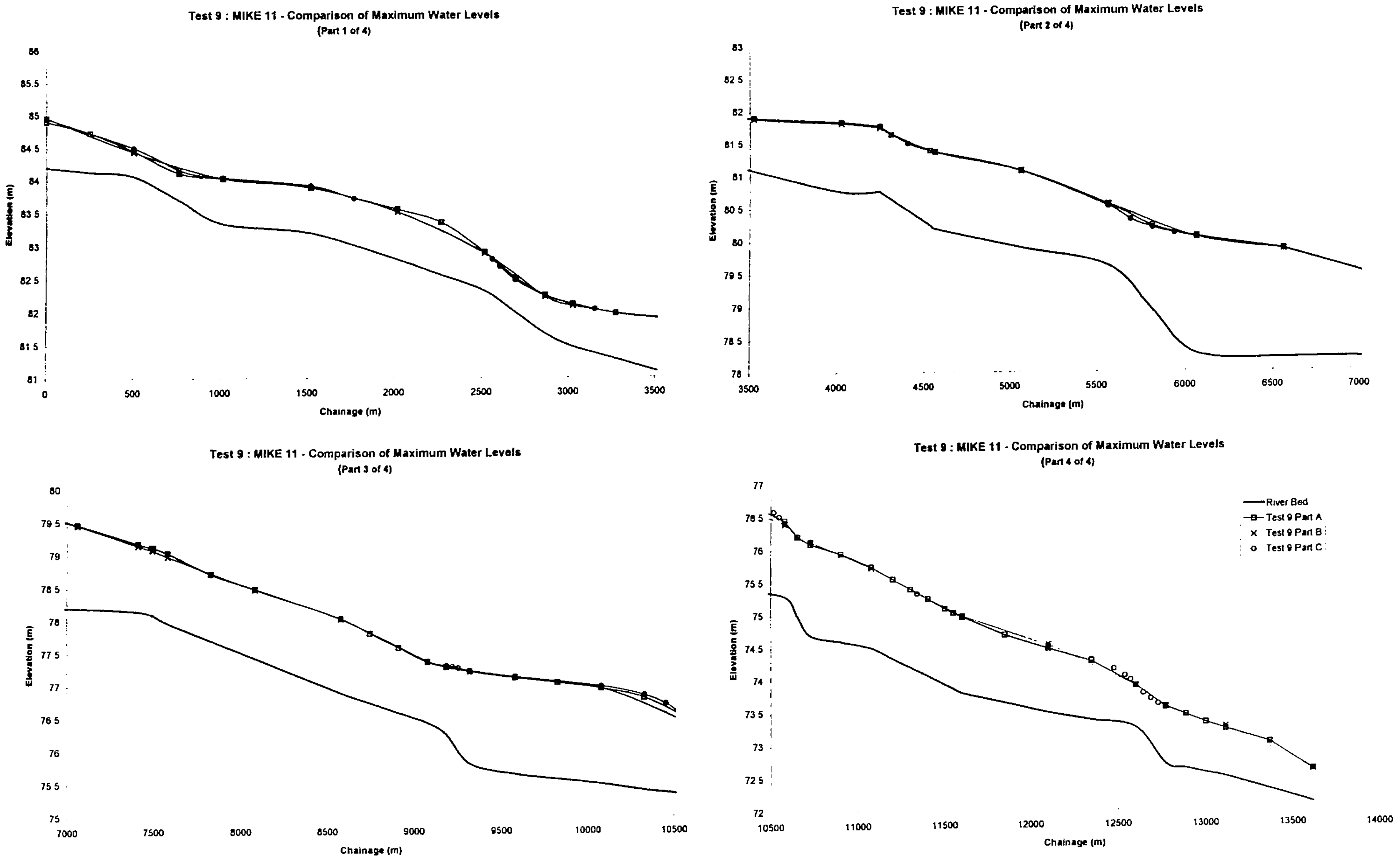
Appendix G



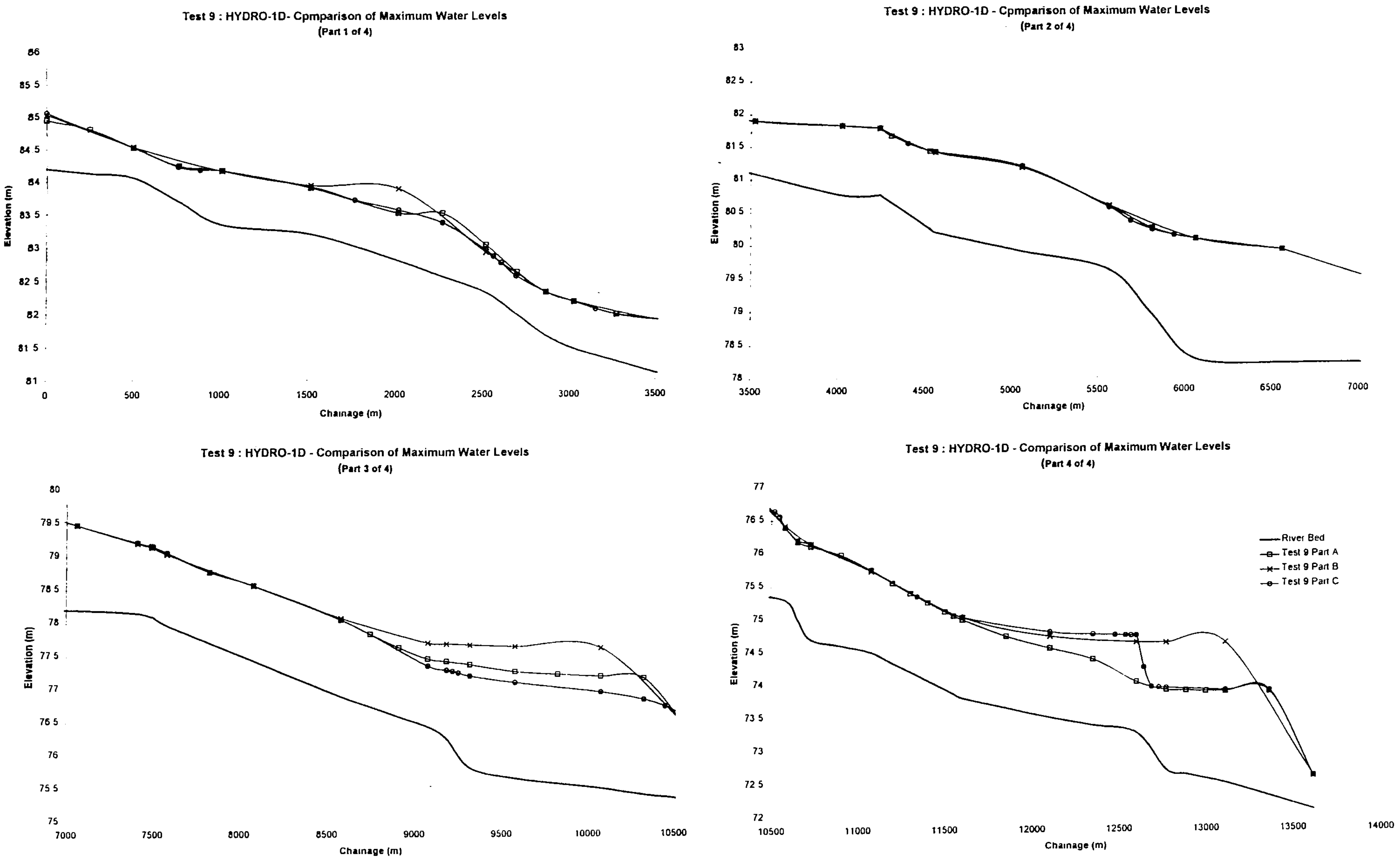
GRAPH 1 - *Inflow Hydrographs for Test 9 Parts A, B and C*



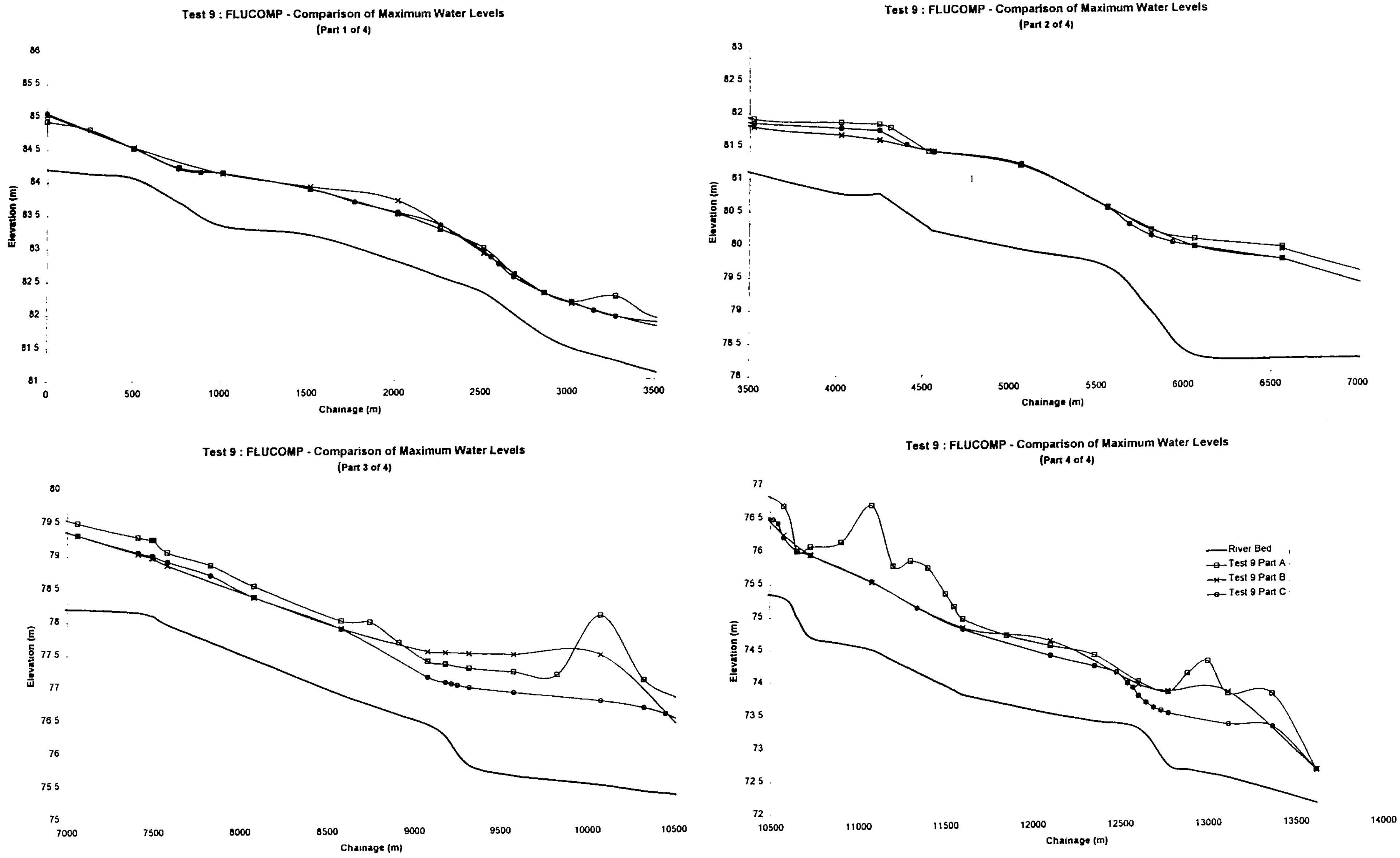
GRAPH 2 - *Inflow Hydrographs for Test 9 Part D*



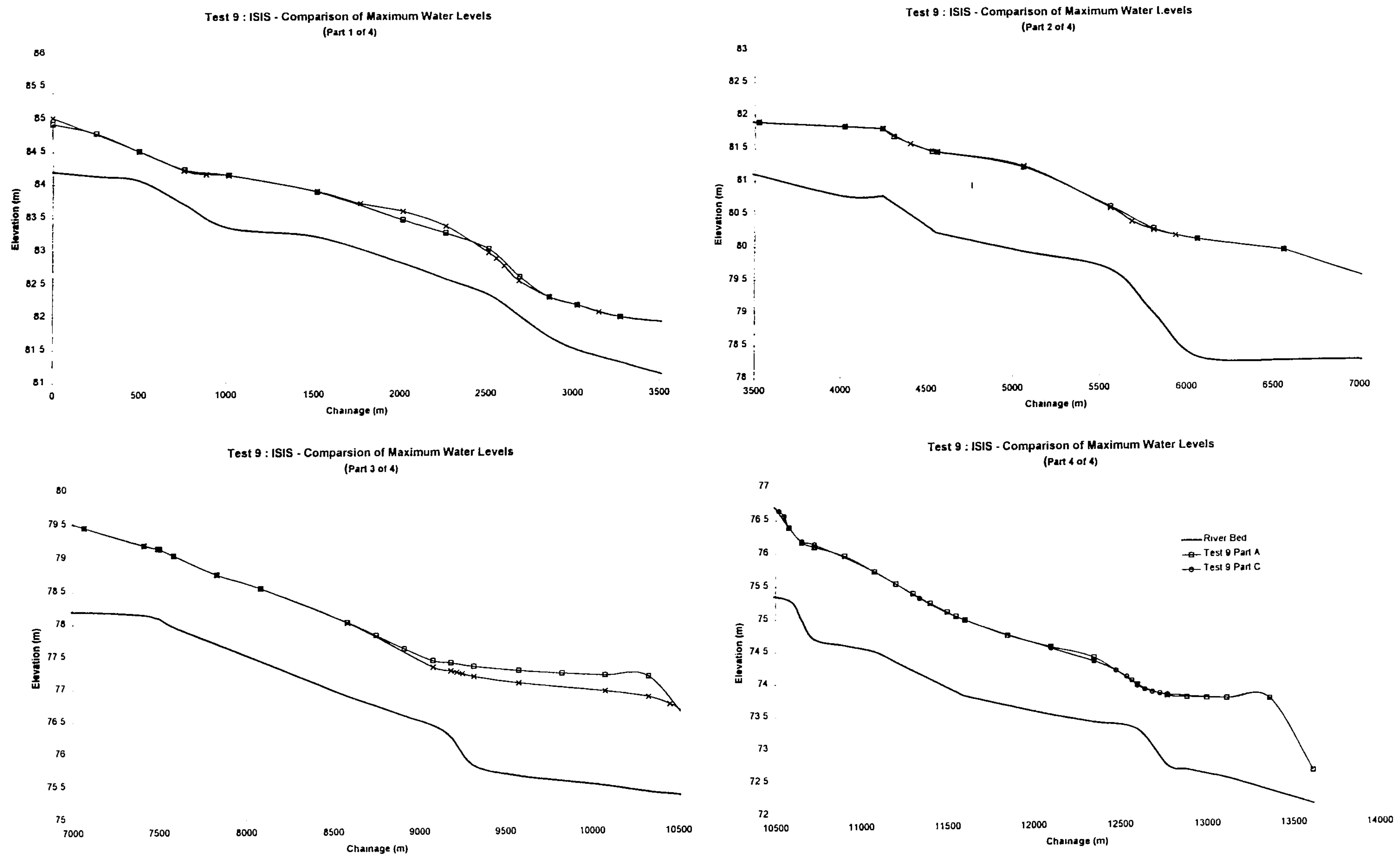
GRAPH 3 : *TEST 9 : MIKE 11 - Comparison of maximum water elevations*



GRAPH 4 : *TEST 9 : HYDRO-1D - Comparison of maximum water elevations*



GRAPH 5 : *TEST 9 : FLUCOMP - Comparison of maximum water elevations*



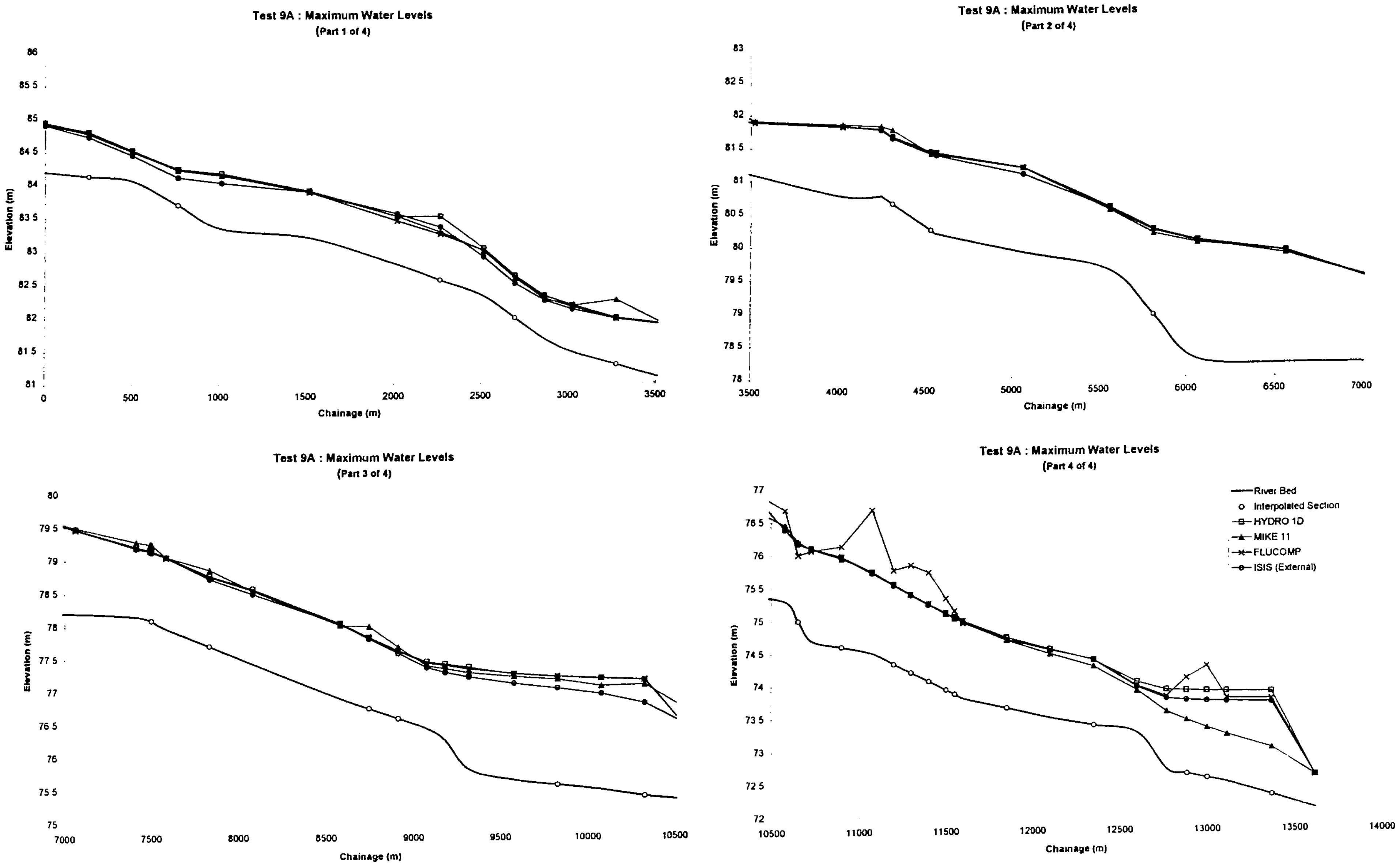
Test 9 : ISIS - Comparison of Maximum Water Levels (Part 1 of 4)

Test 9 : ISIS - Comparison of Maximum Water Levels (Part 2 of 4)

Test 9 : ISIS - Comparsion of Maximum Water Levels (Part 3 of 4)

Test 9 : ISIS - Comparison of Maximum Water Levels (Part 4 of 4)

GRAPH 6 : *TEST 9 : ISIS - Comparison of maximum water elevations*



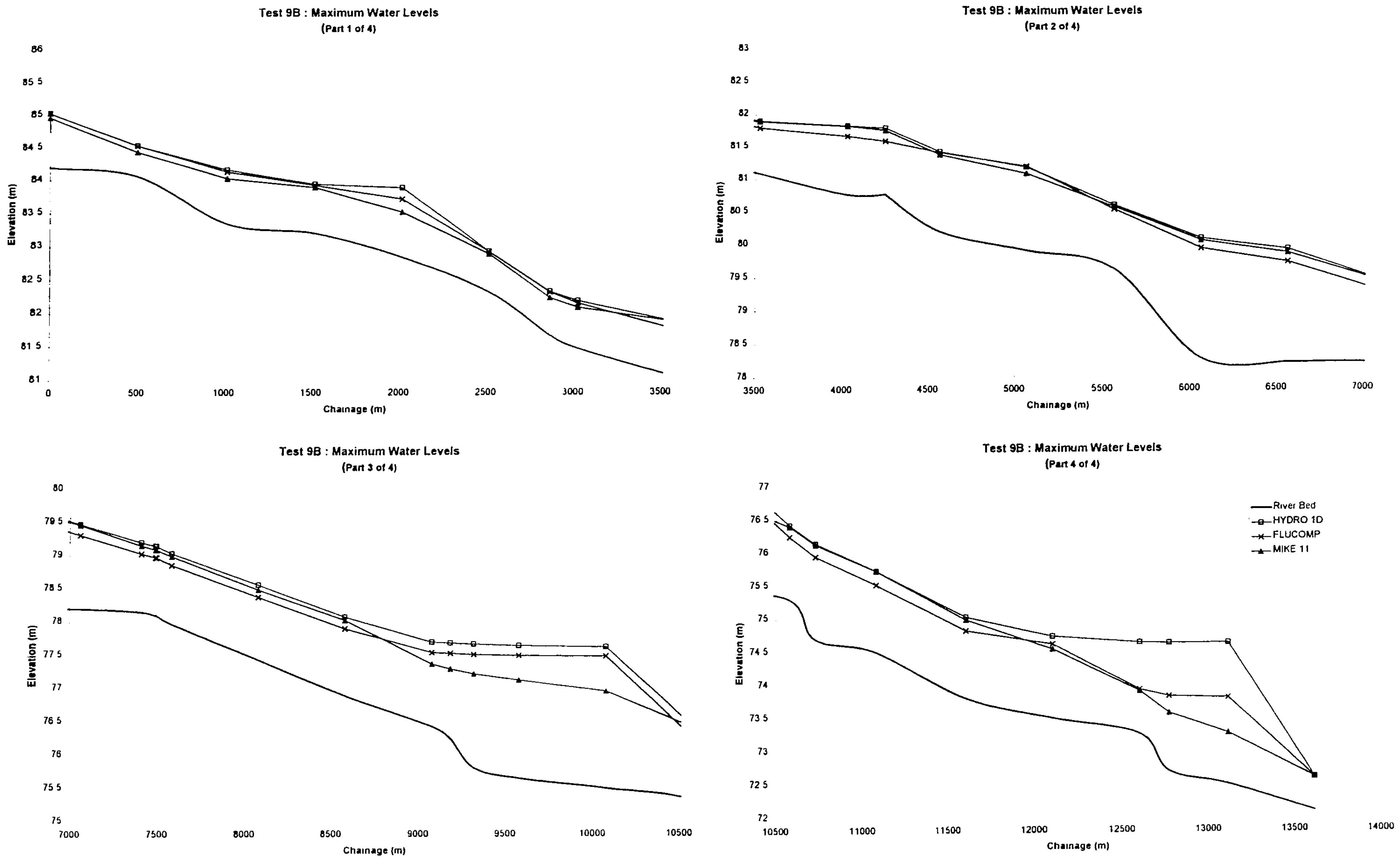
Test 9A : Maximum Water Levels (Part 1 of 4)

Test 9A : Maximum Water Levels (Part 2 of 4)

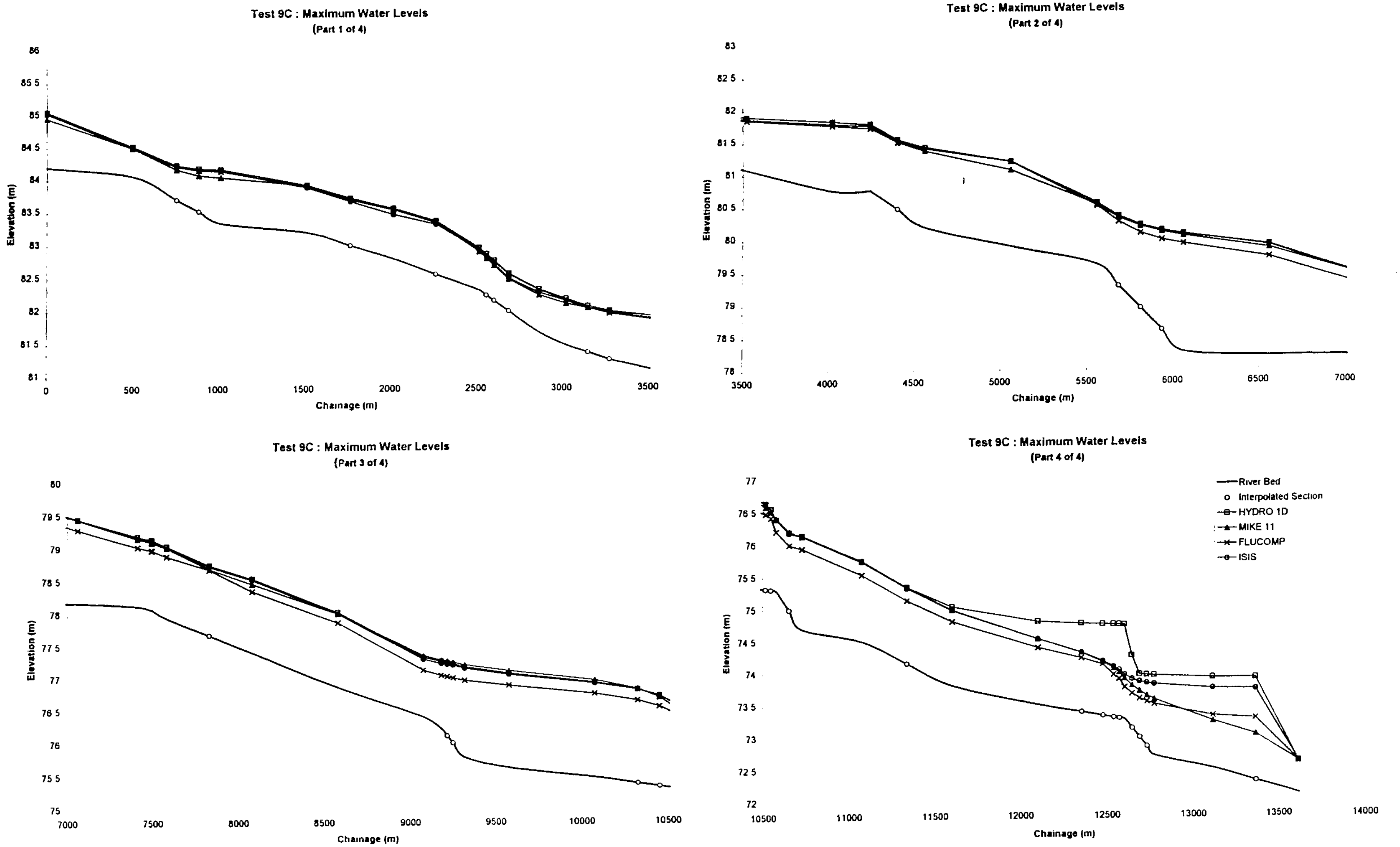
Test 9A : Maximum Water Levels (Part 3 of 4)

Test 9A : Maximum Water Levels (Part 4 of 4)

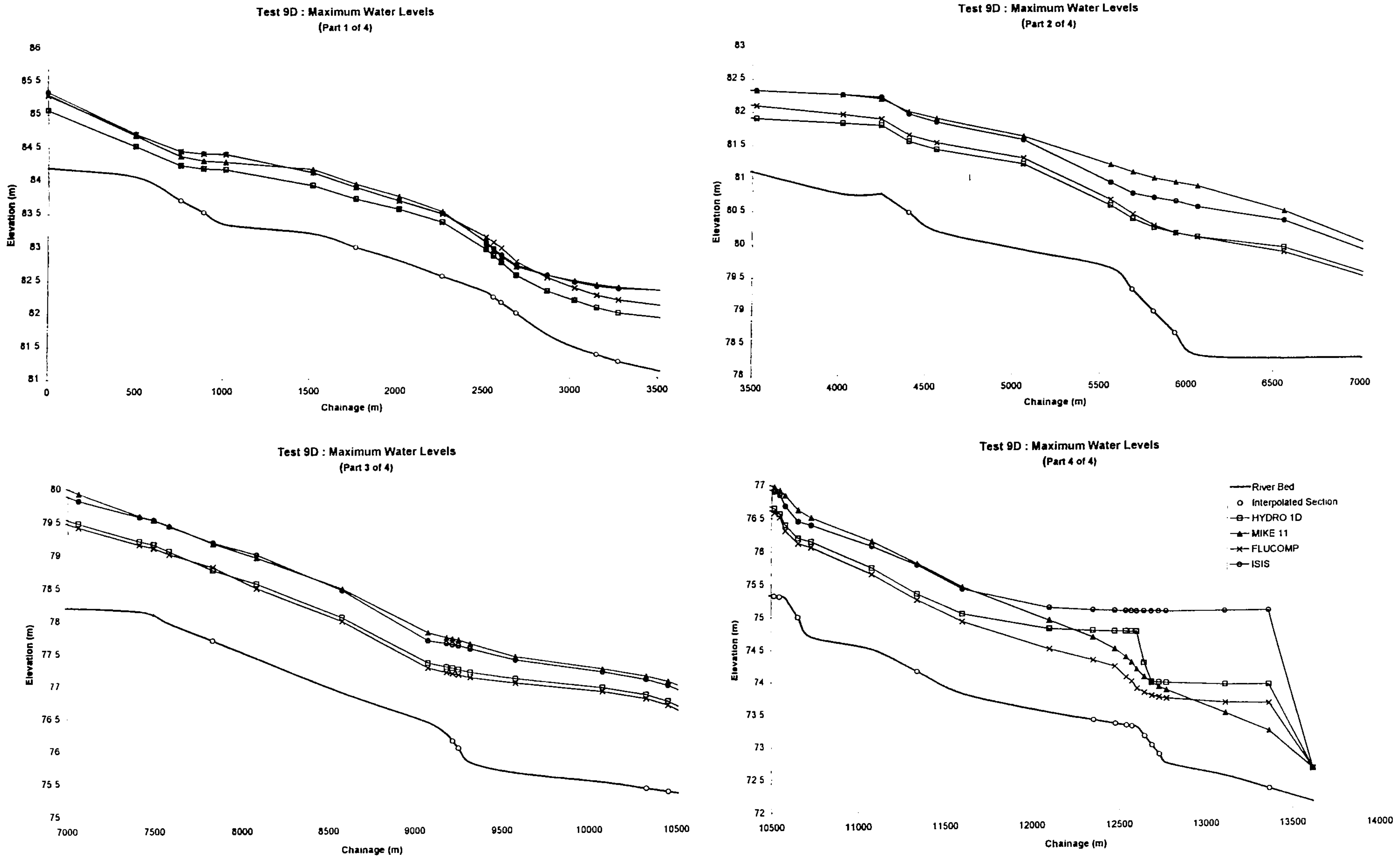
GRAPH 7 : *TEST 9 Part A - Comparison of maximum water elevations*



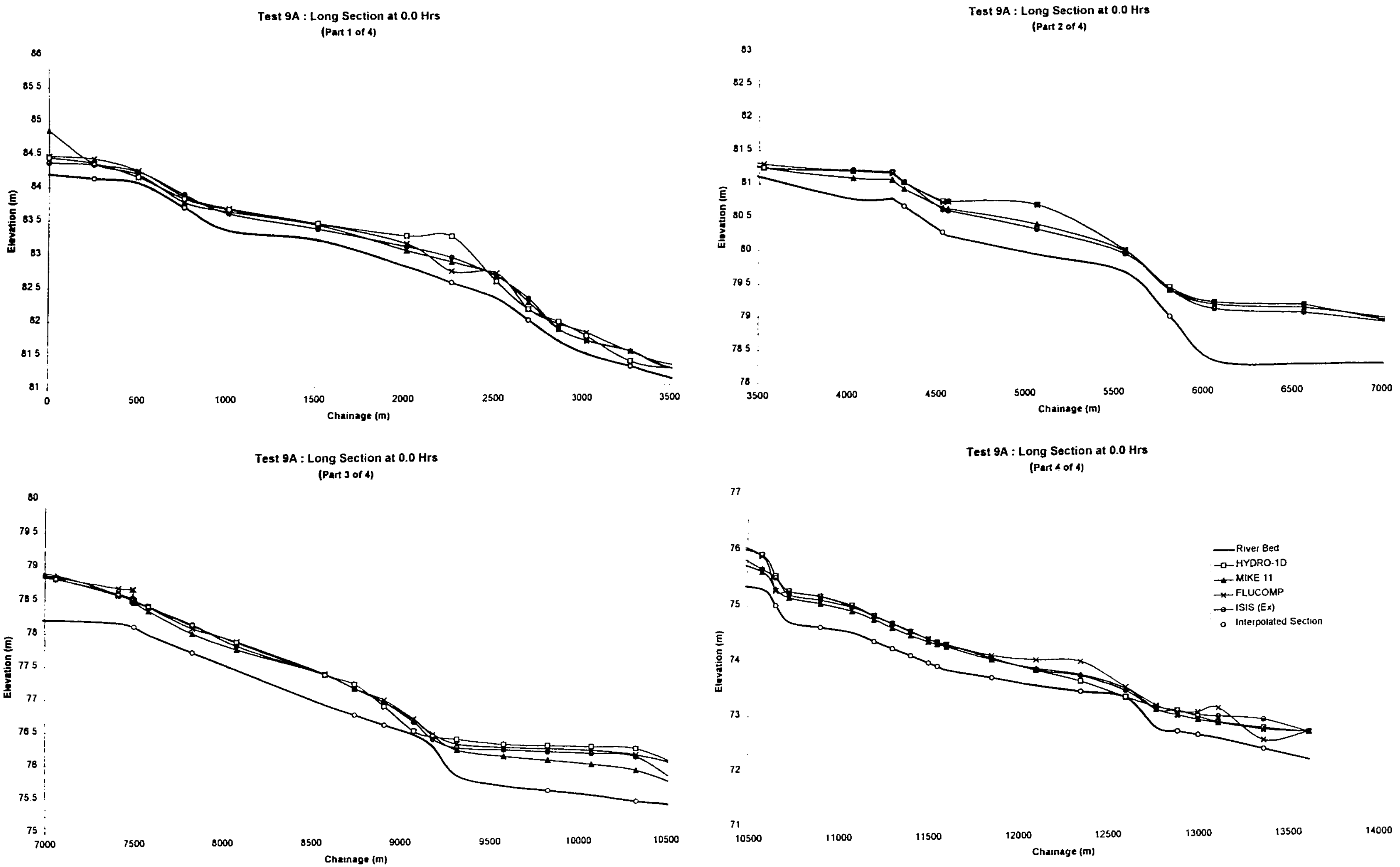
GRAPH 8 : *TEST 9 Part B - Comparison of maximum water elevations*



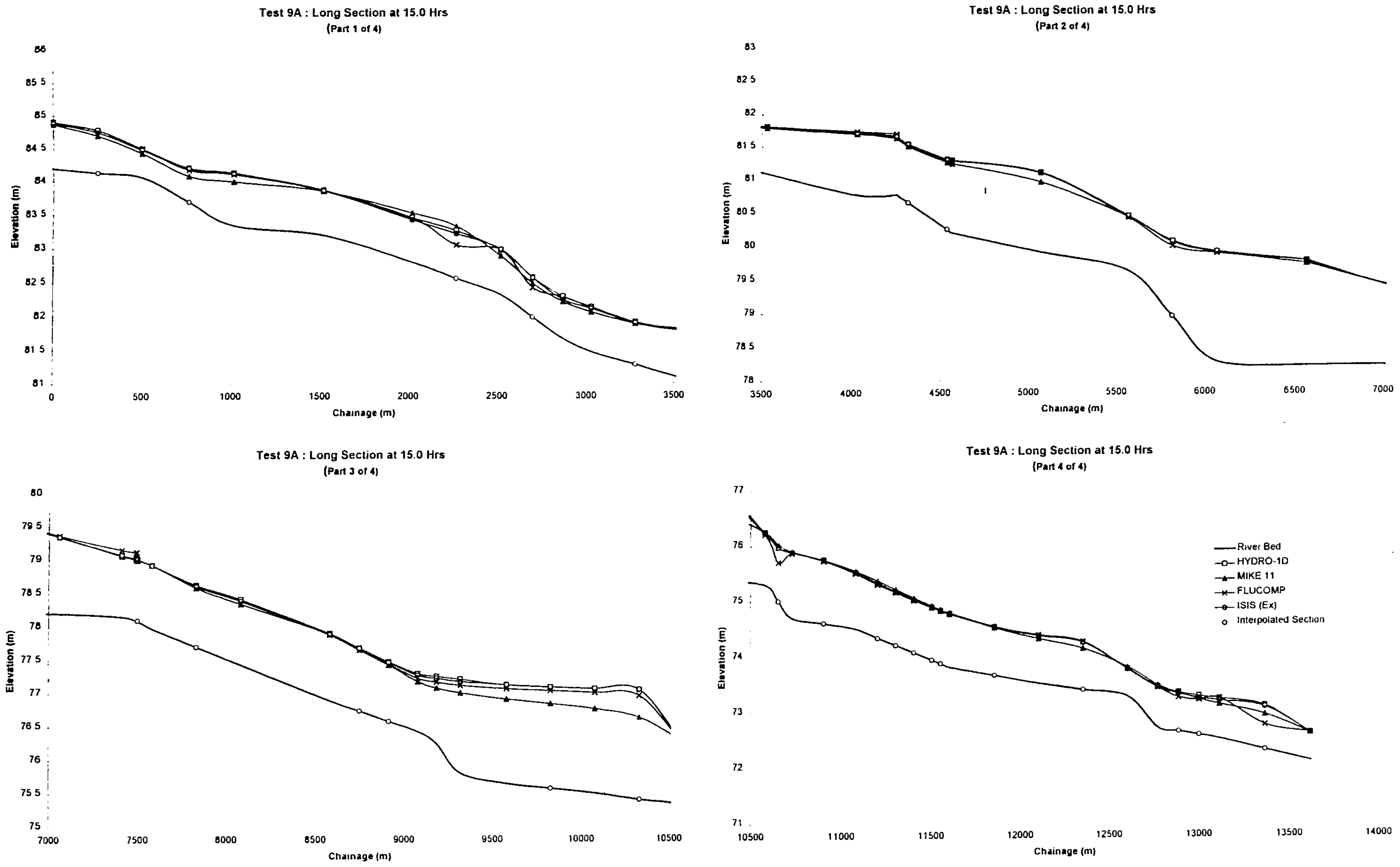
GRAPH 9 : *TEST 9 Part C - Comparison of maximum water elevations*



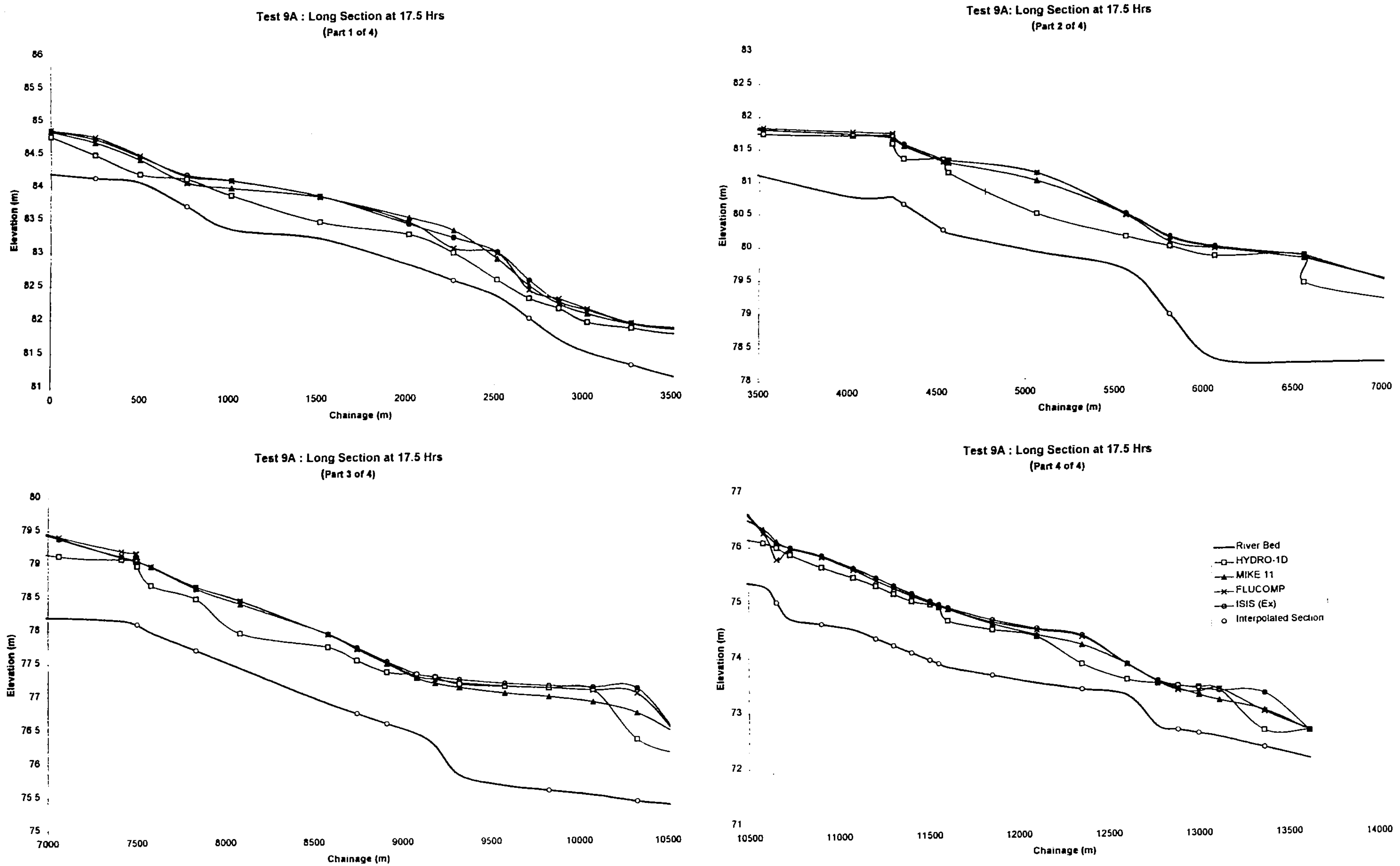
GRAPH 10 : *TEST 9 Part D - Comparison of maximum water elevations*



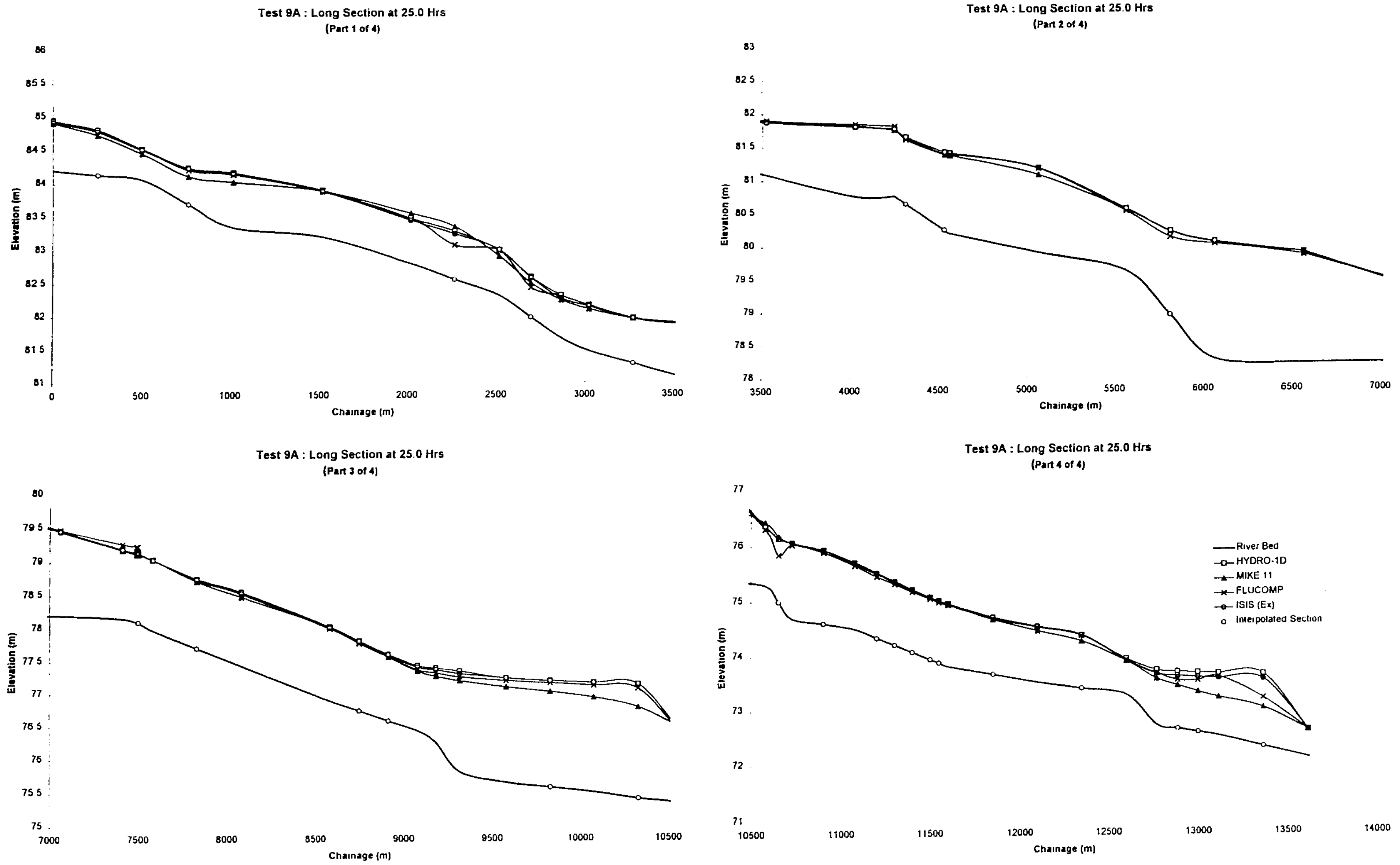
GRAPH 11 : *TEST 9A - Comparison of water elevations at 0.0 Hrs*



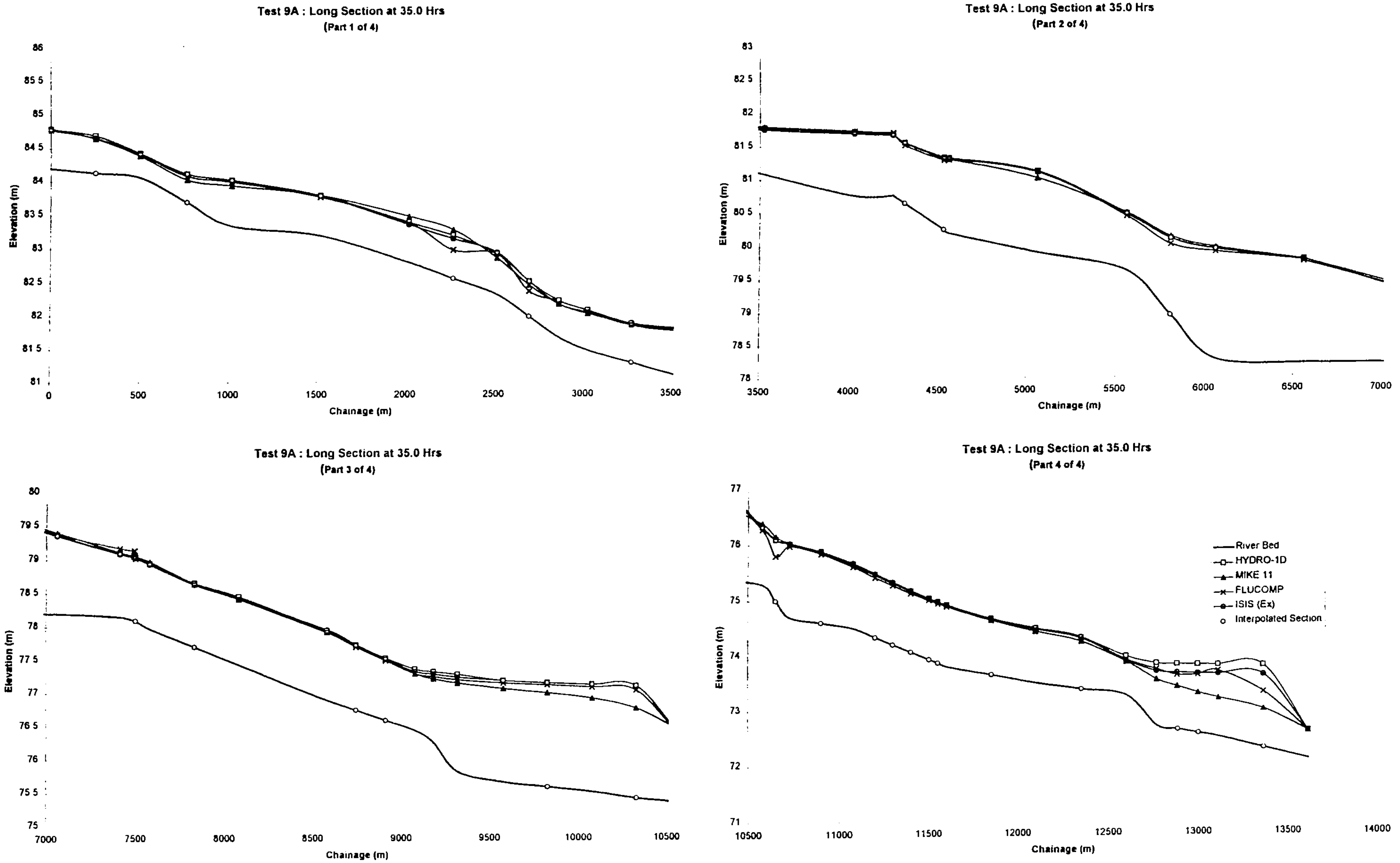
GRAPH 12 : *TEST 9A - Comparison of water elevations at 15.0 Hrs*



GRAPH 13 : *TEST 9A - Comparison of water elevations at 17.5 Hrs*



GRAPH 14 : *TEST 9A - Comparison of water elevations at 25.0 Hrs*

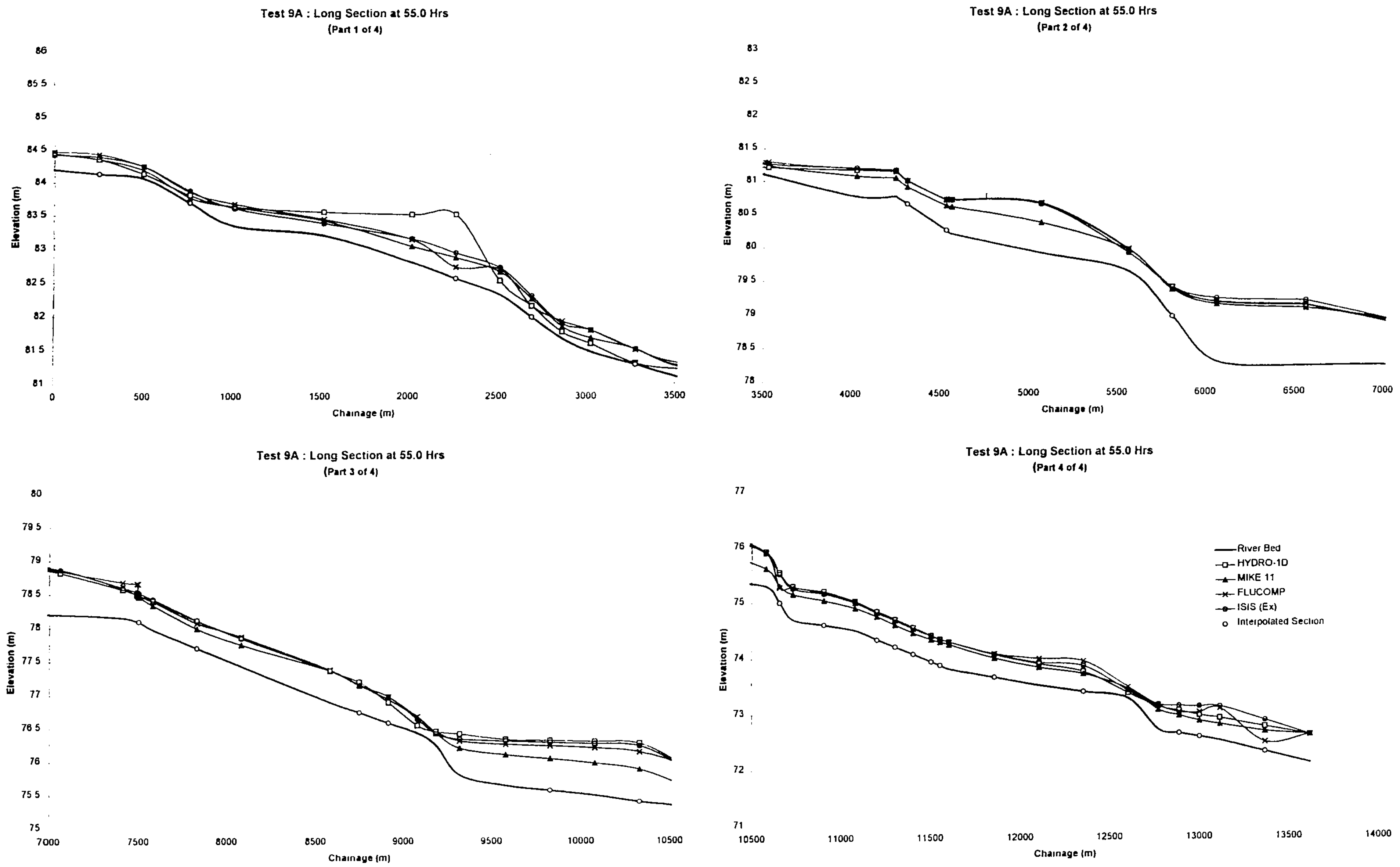

Test 9A : Long Section at 35.0 Hrs (Part 1 of 4)

Test 9A : Long Section at 35.0 Hrs (Part 2 of 4)

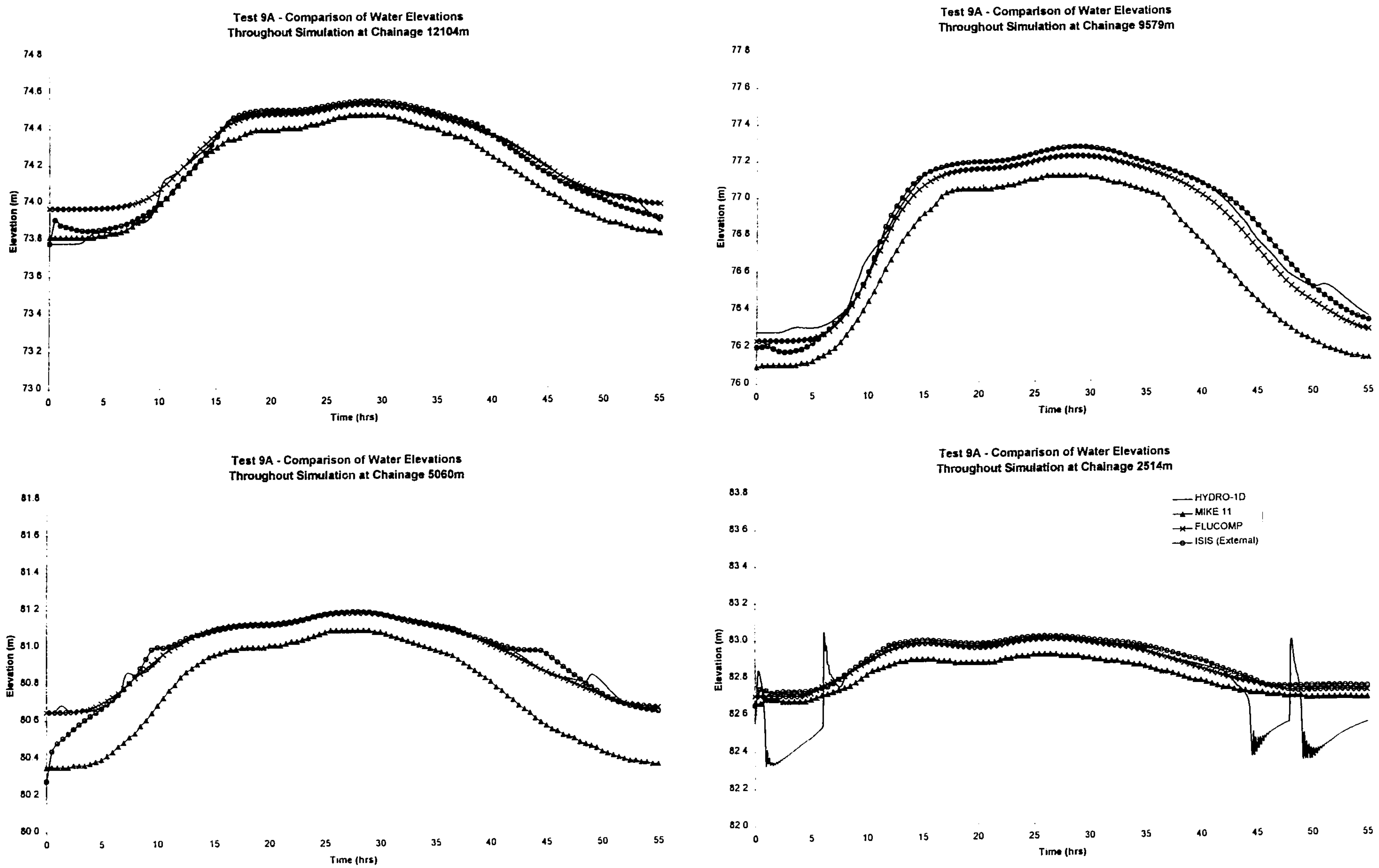
Test 9A : Long Section at 35.0 Hrs (Part 3 of 4)

Test 9A : Long Section at 35.0 Hrs (Part 4 of 4)

GRAPH 15 : *TEST 9A - Comparison of water elevations at 35.0 Hrs*



GRAPH 16 : *TEST 9A - Comparison of water elevations at 55.0 Hrs*

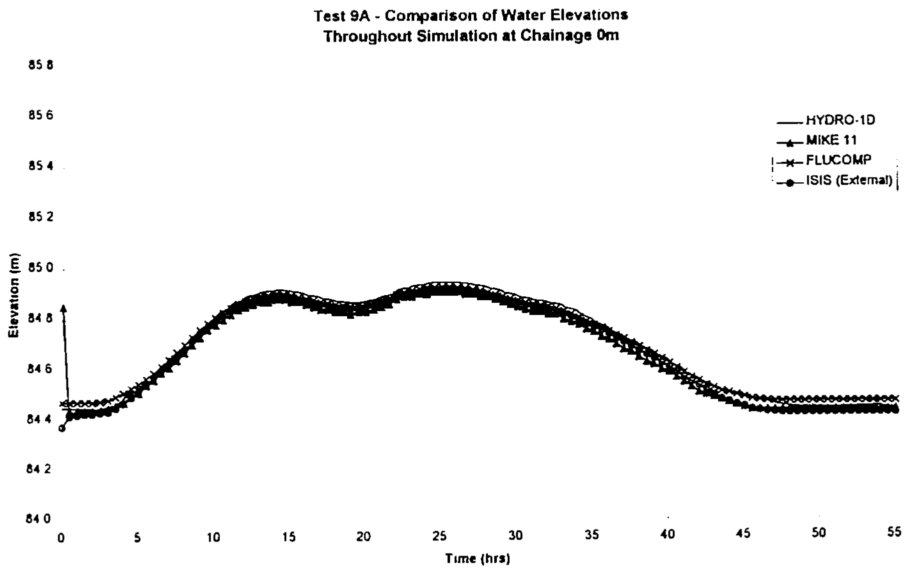

Test 9A - Comparison of Water Elevations Throughout Simulation at Chainage 12104m

Test 9A - Comparison of Water Elevations Throughout Simulation at Chainage 9579m

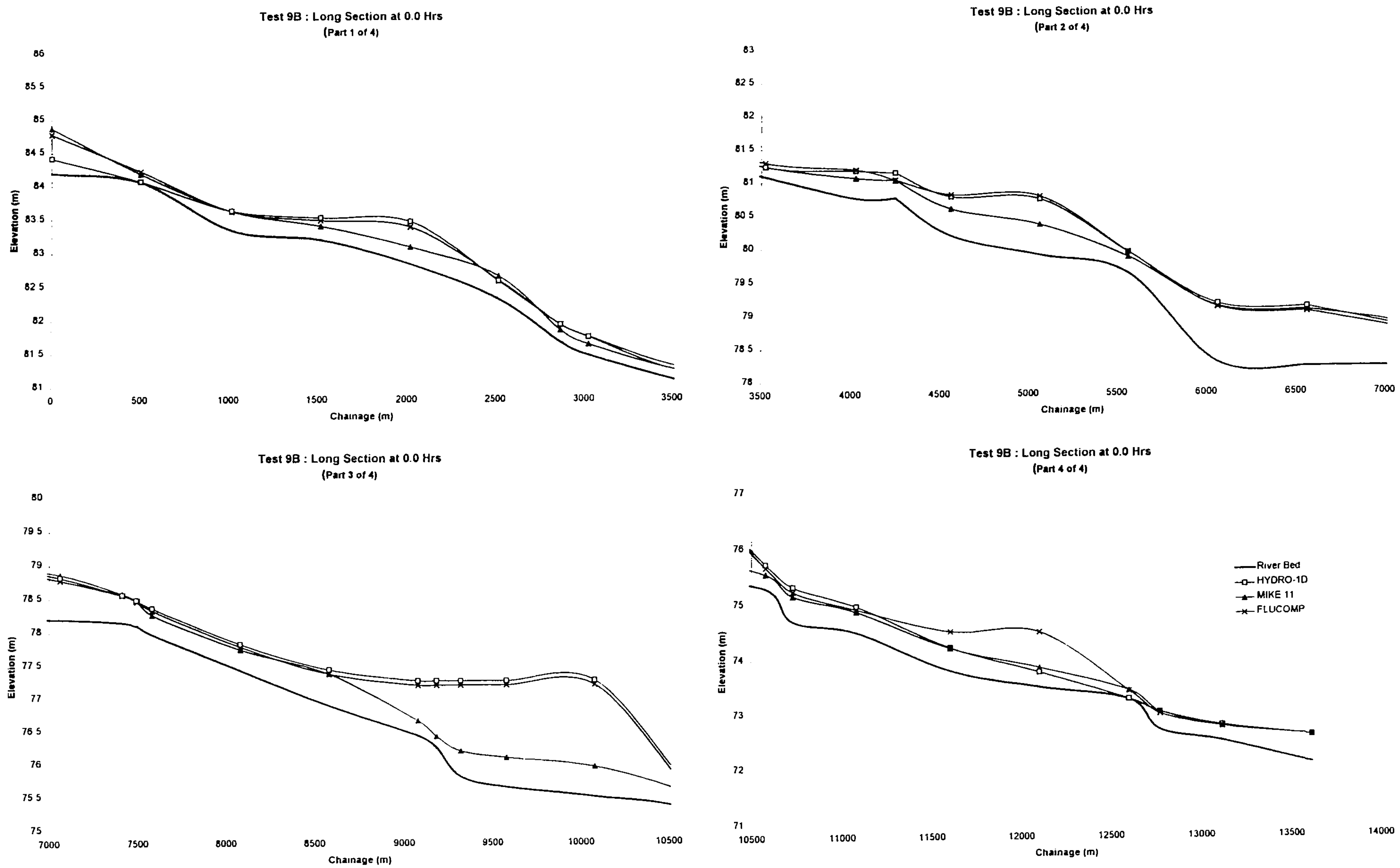
Test 9A - Comparison of Water Elevations Throughout Simulation at Chainage 5060m

Test 9A - Comparison of Water Elevations Throughout Simulation at Chainage 2514m

GRAPH 17 : *TEST 9A - Comparison of water elevations throughout simulation.*



GRAPH 17 : *TEST 9A - Comparison of water elevations throughout simulation.*



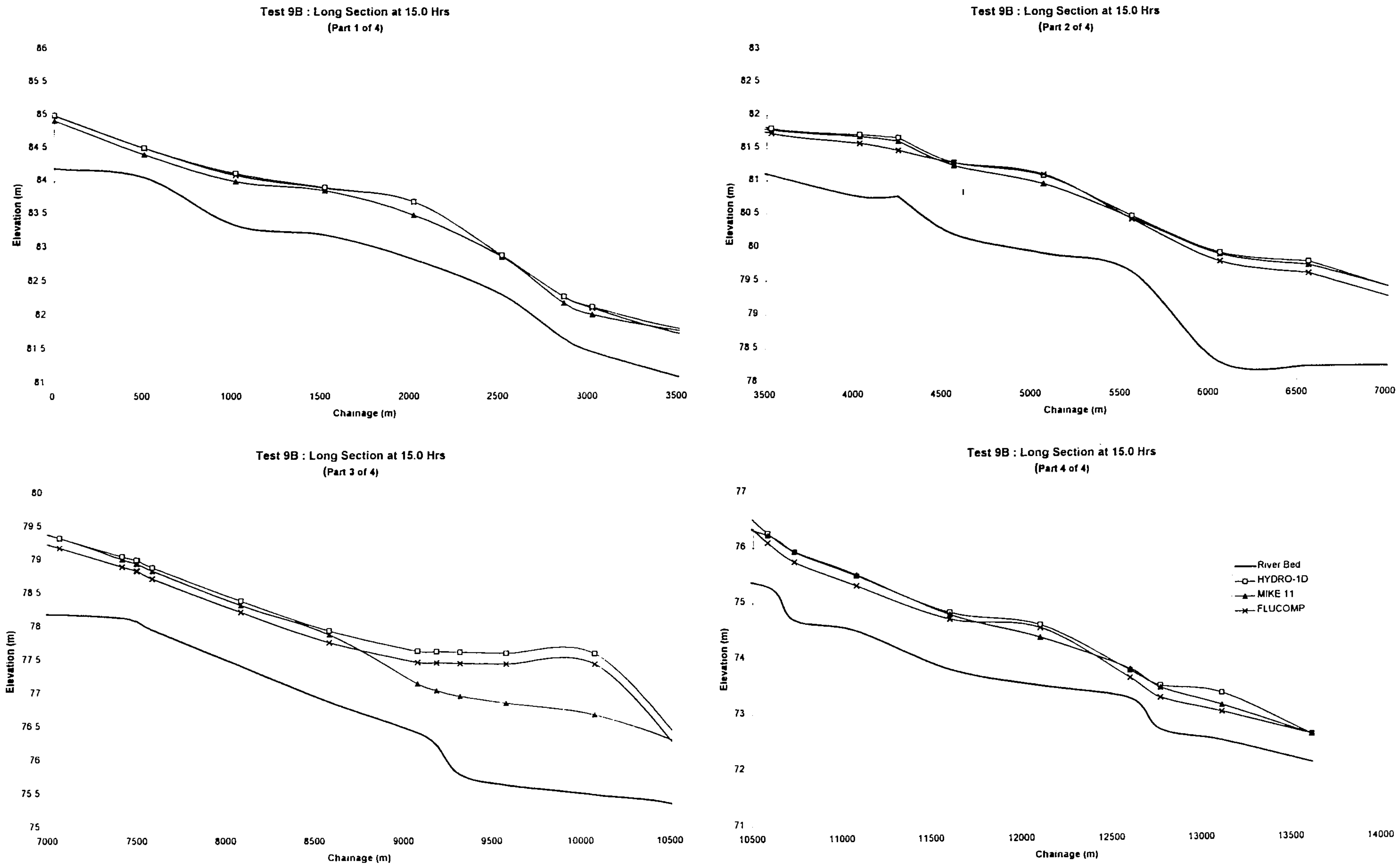
Test 9B : Long Section at 0.0 Hrs (Part 1 of 4)

Test 9B : Long Section at 0.0 Hrs (Part 2 of 4)

Test 9B : Long Section at 0.0 Hrs (Part 3 of 4)

Test 9B : Long Section at 0.0 Hrs (Part 4 of 4)

GRAPH 18 : *TEST 9B - Comparison of water elevations at 0.0 Hrs*



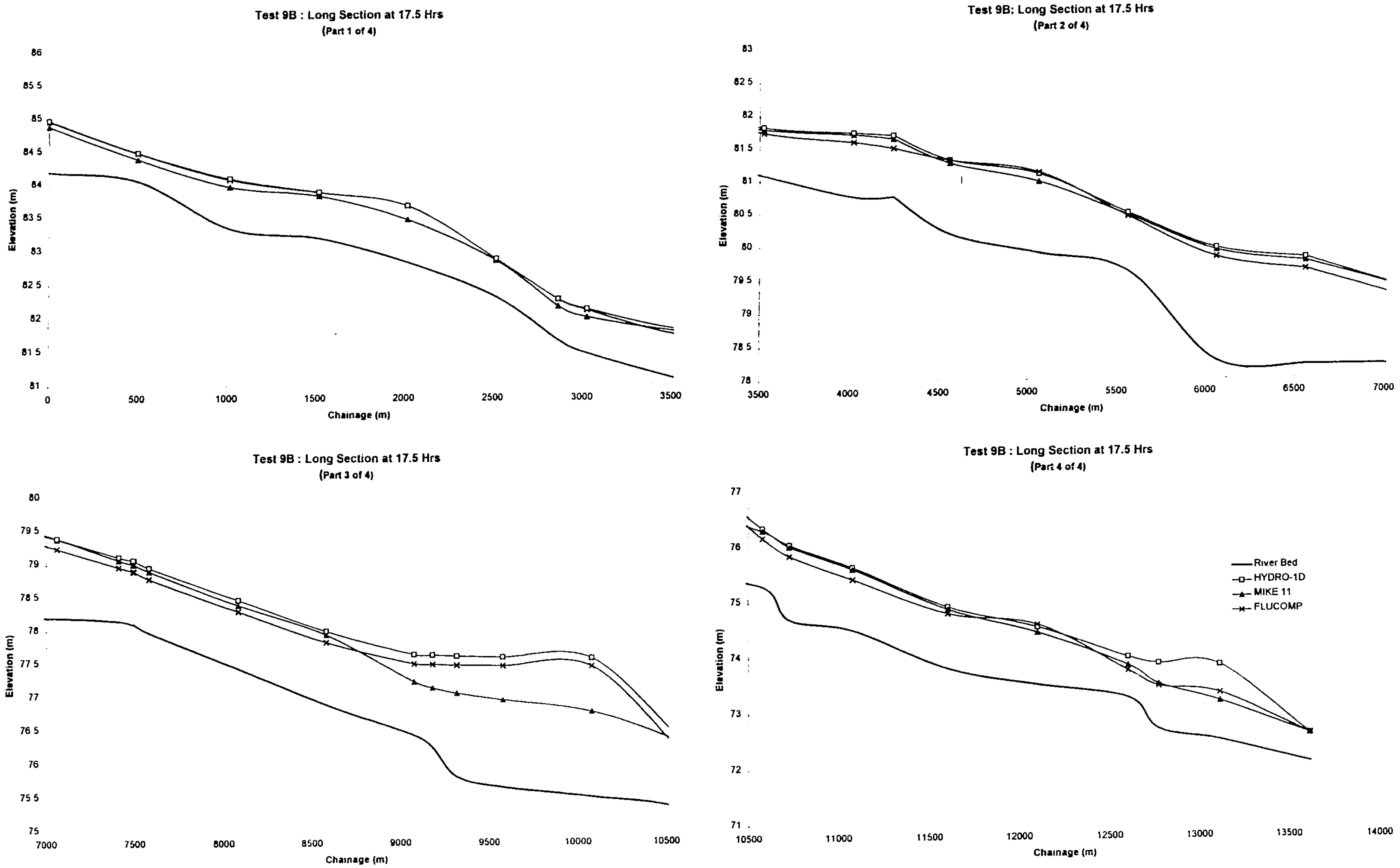
Test 9B : Long Section at 15.0 Hrs
(Part 1 of 4)

Test 9B : Long Section at 15.0 Hrs
(Part 2 of 4)

Test 9B : Long Section at 15.0 Hrs
(Part 3 of 4)

Test 9B : Long Section at 15.0 Hrs
(Part 4 of 4)

GRAPH 19 : *TEST 9B - Comparison of water elevations at 15.0 Hrs*

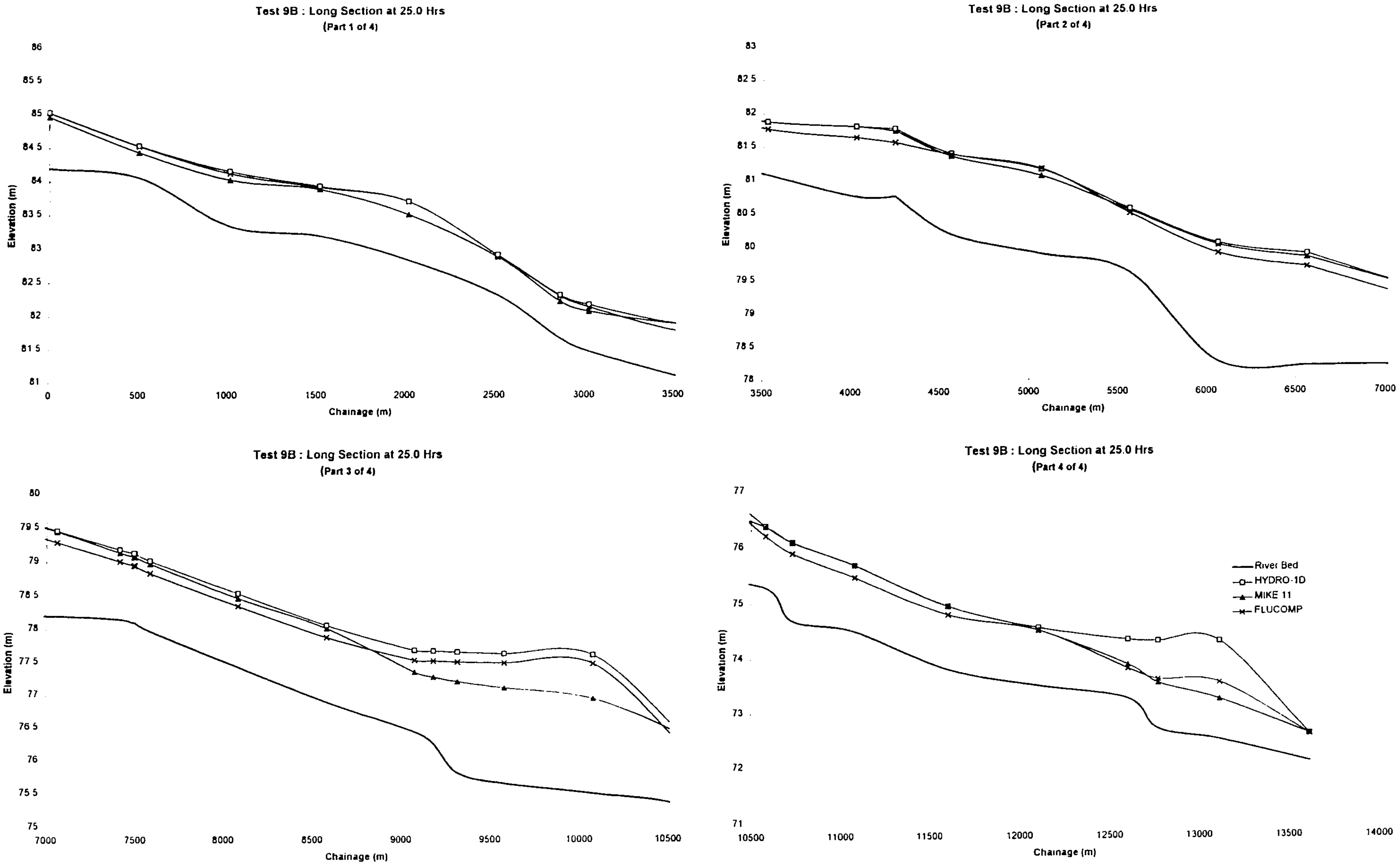

Test 9B : Long Section at 17.5 Hrs (Part 1 of 4)

Test 9B: Long Section at 17.5 Hrs (Part 2 of 4)

Test 9B : Long Section at 17.5 Hrs (Part 3 of 4)

Test 9B : Long Section at 17.5 Hrs (Part 4 of 4)

GRAPH 20 : *TEST 9B - Comparison of water elevations at 17.5 Hrs*

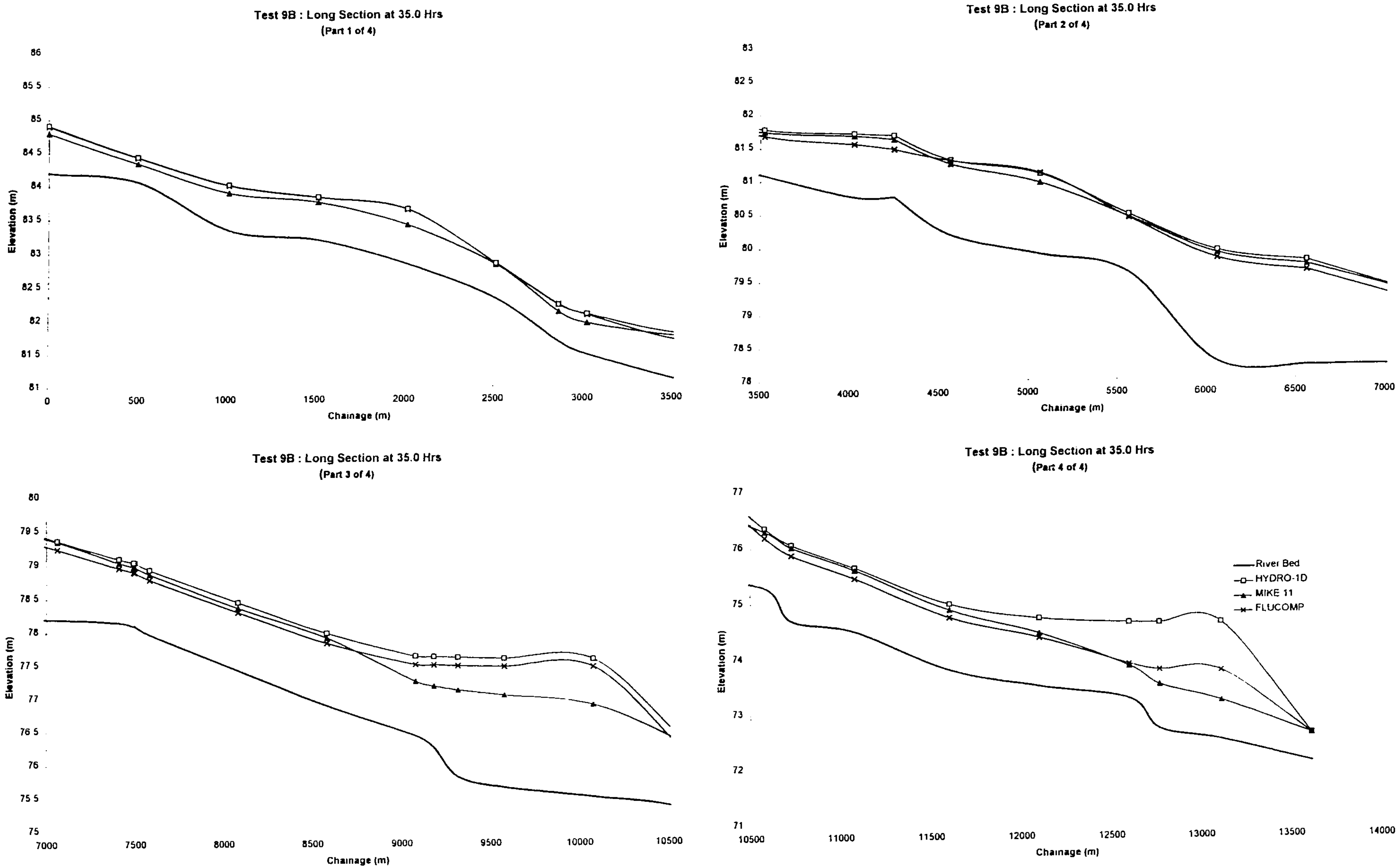

Test 9B : Long Section at 25.0 Hrs (Part 1 of 4)

Test 9B : Long Section at 25.0 Hrs (Part 2 of 4)

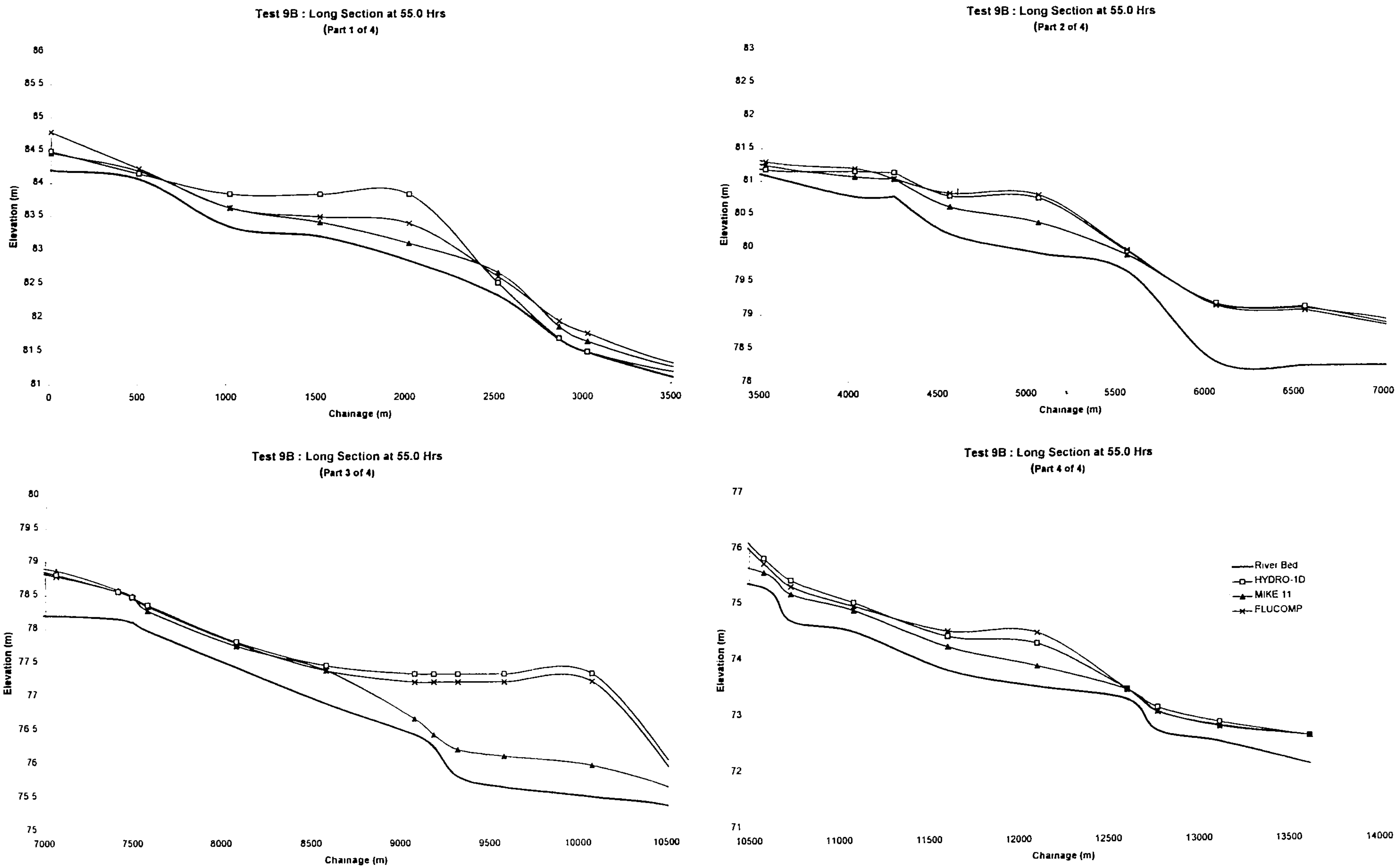
Test 9B : Long Section at 25.0 Hrs (Part 3 of 4)

Test 9B : Long Section at 25.0 Hrs (Part 4 of 4)

GRAPH 21 : *TEST 9B - Comparison of water elevations at 25.0 Hrs*



GRAPH 22 : *TEST 9B - Comparison of water elevations at 35.0 Hrs*

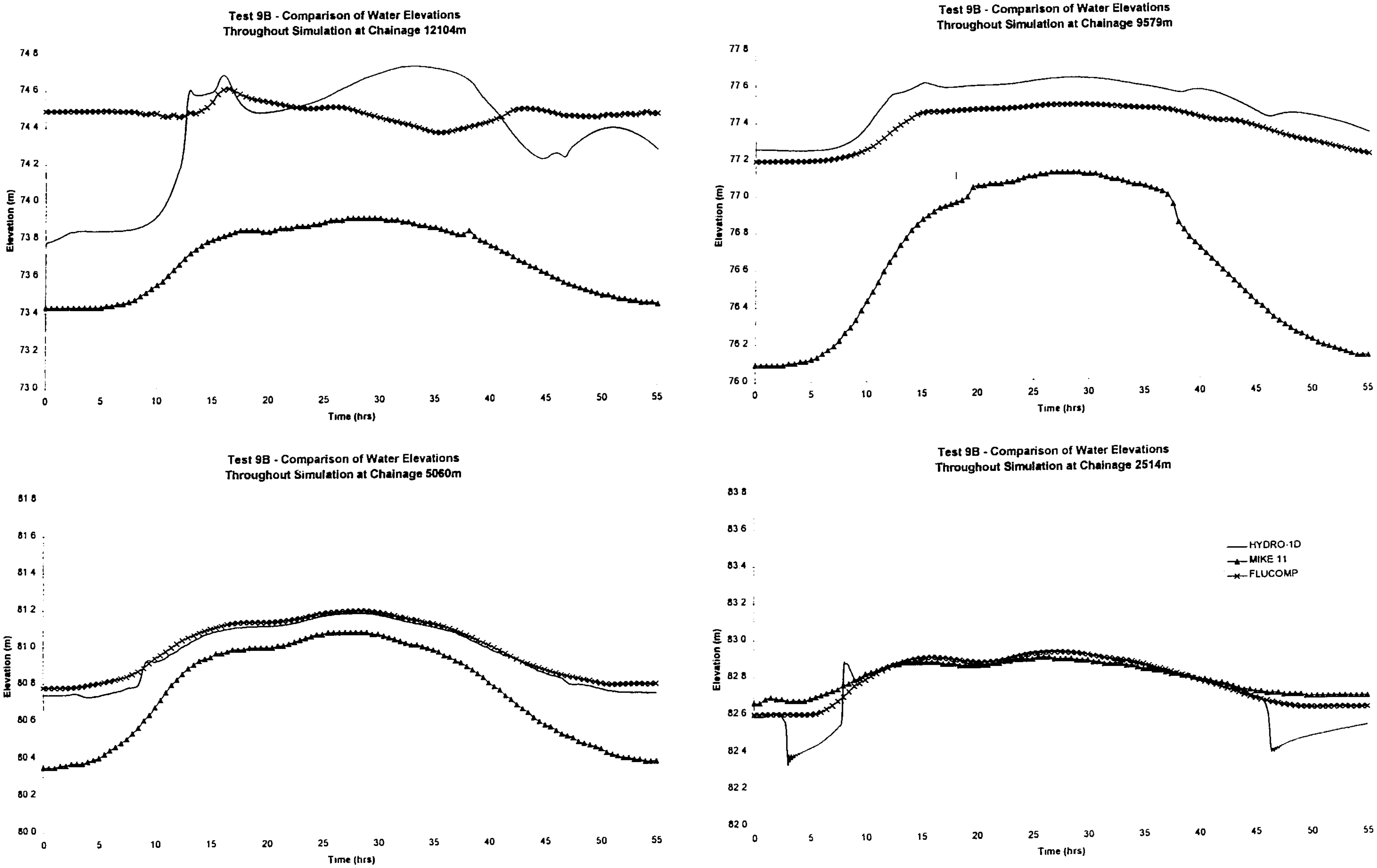

Test 9B : Long Section at 55.0 Hrs (Part 1 of 4)

Test 9B : Long Section at 55.0 Hrs (Part 2 of 4)

Test 9B : Long Section at 55.0 Hrs (Part 3 of 4)

Test 9B : Long Section at 55.0 Hrs (Part 4 of 4)

GRAPH 23 : *TEST 9B - Comparison of water elevations at 55.0 Hrs*



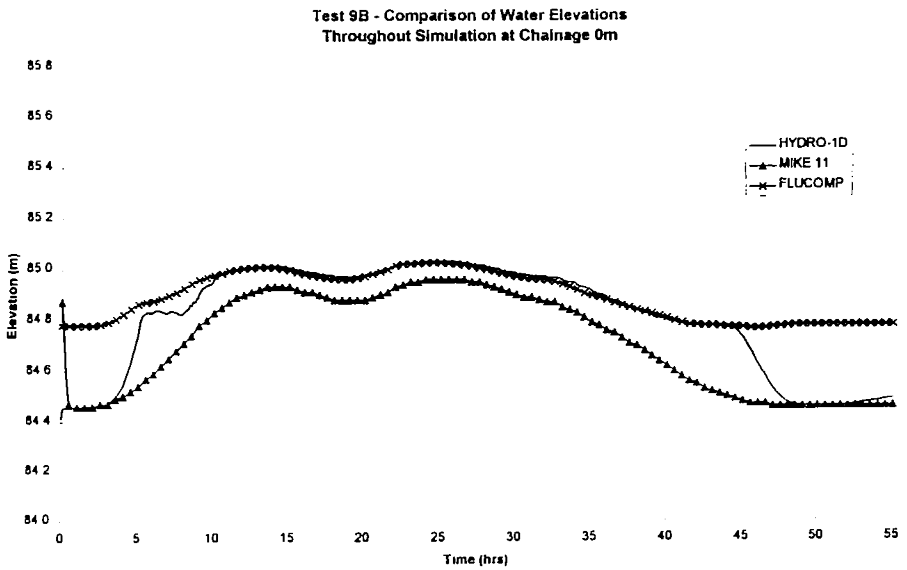
Test 9B - Comparison of Water Elevations Throughout Simulation at Chainage 12104m

Test 9B - Comparison of Water Elevations Throughout Simulation at Chainage 9579m

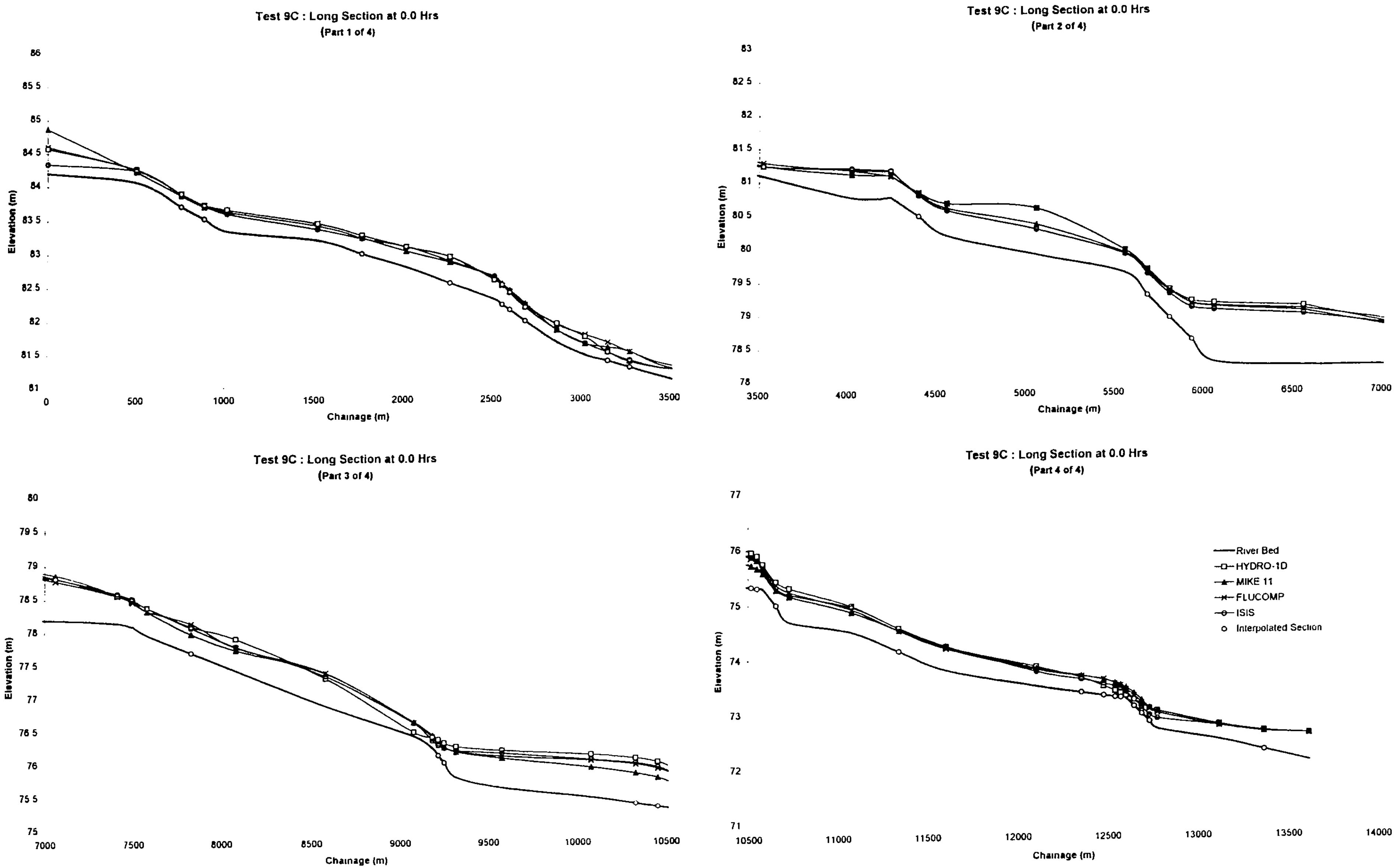
Test 9B - Comparison of Water Elevations Throughout Simulation at Chainage 5060m

Test 9B - Comparison of Water Elevations Throughout Simulation at Chainage 2514m

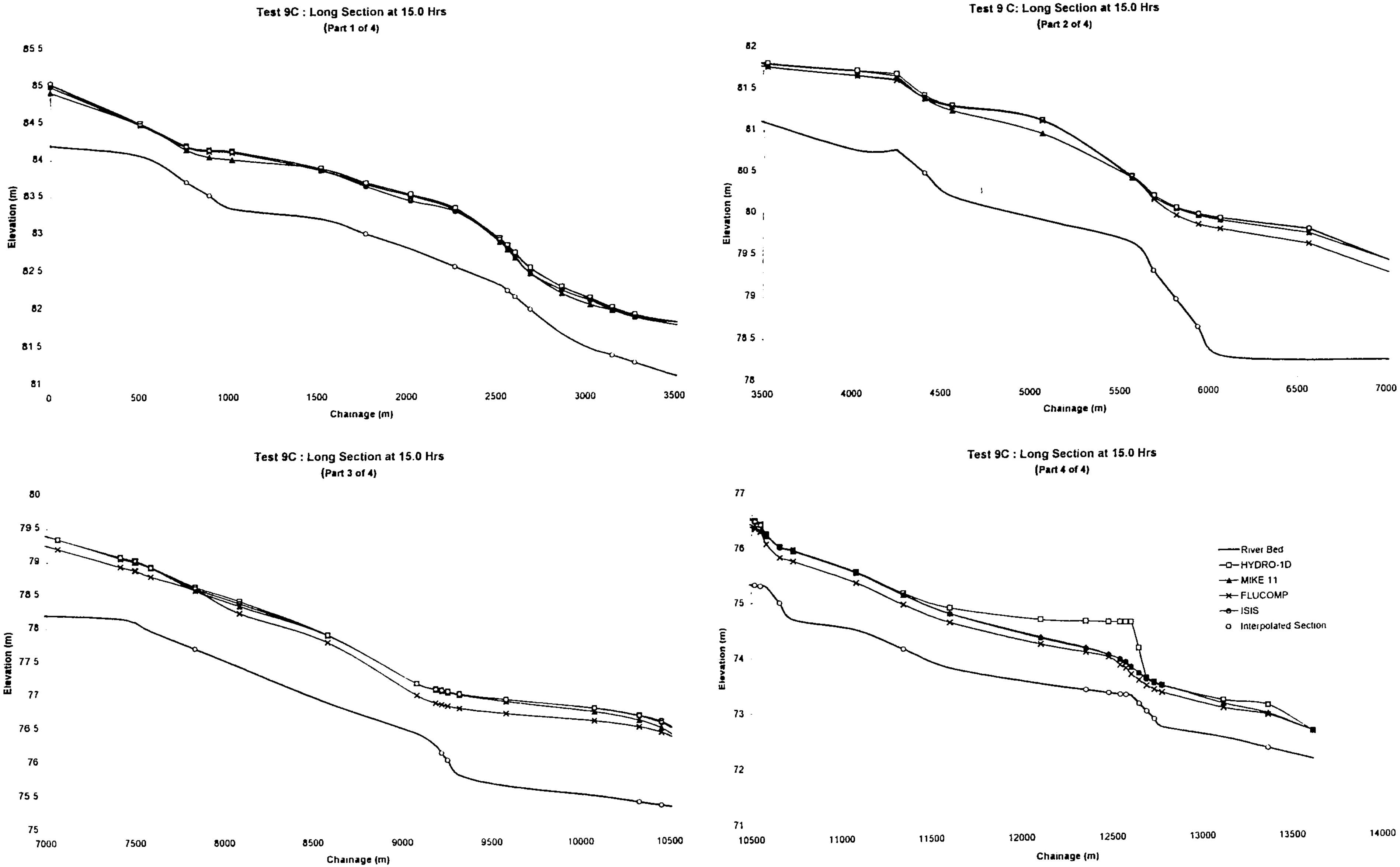
GRAPH 24 : *TEST 9B - Comparison of water elevations throughout simulation*



GRAPH 24 : *TEST 9B - Comparison of water elevations throughout simulation*



GRAPH 25 : *TEST 9C - Comparison of water elevations at 0.0 Hrs*

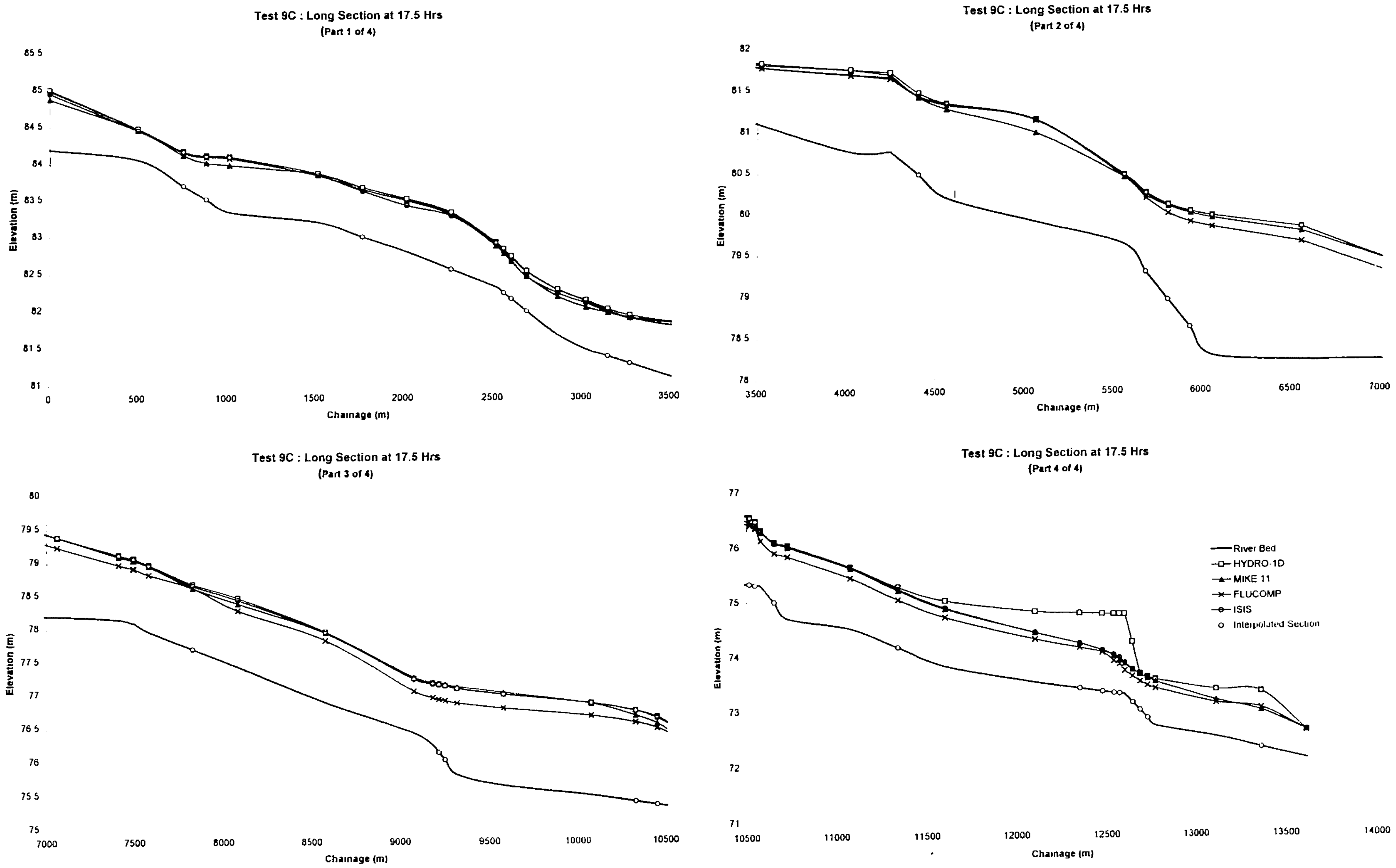

Test 9C : Long Section at 15.0 Hrs (Part 1 of 4)

Test 9 C: Long Section at 15.0 Hrs (Part 2 of 4)

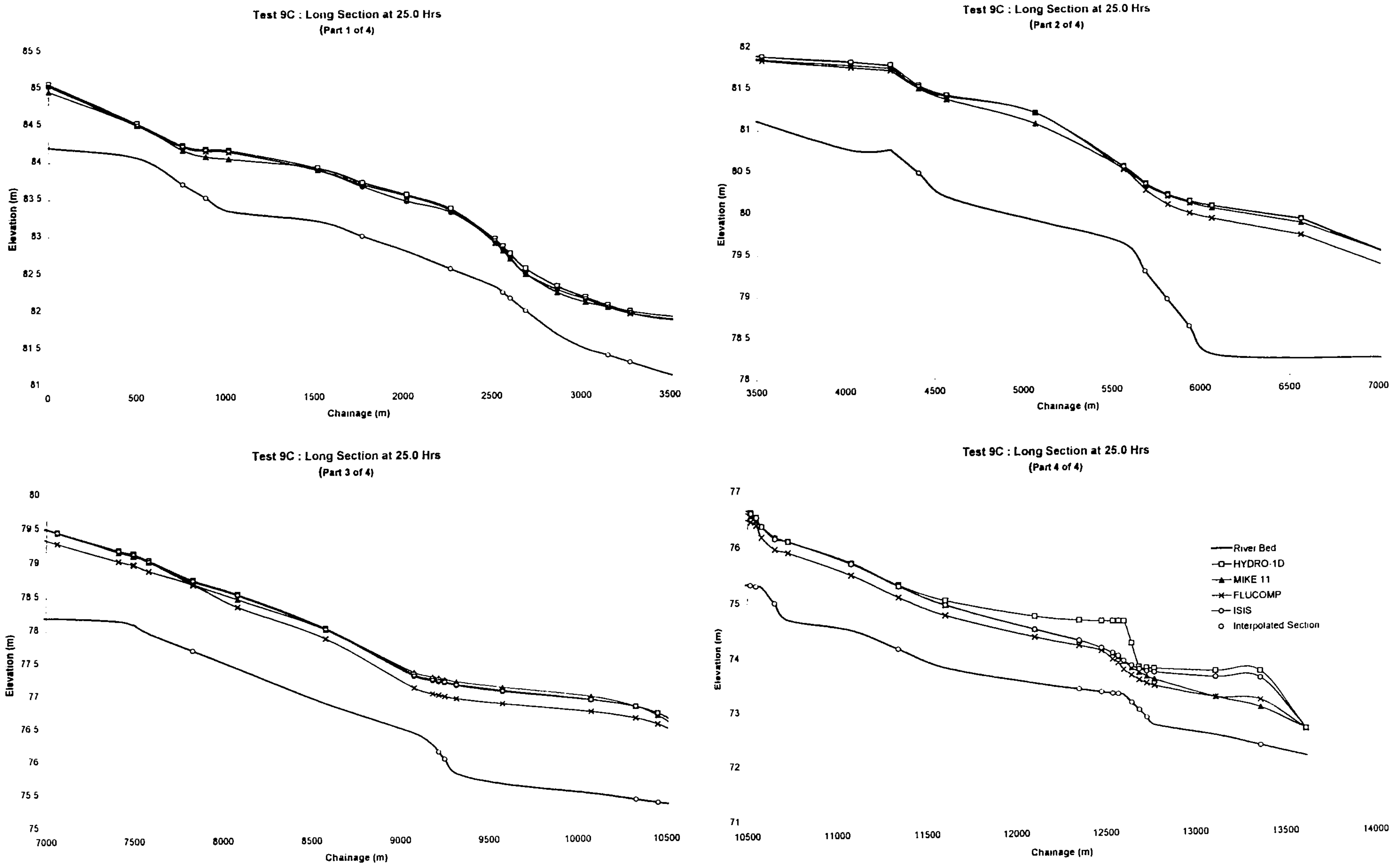
Test 9C : Long Section at 15.0 Hrs (Part 3 of 4)

Test 9C : Long Section at 15.0 Hrs (Part 4 of 4)

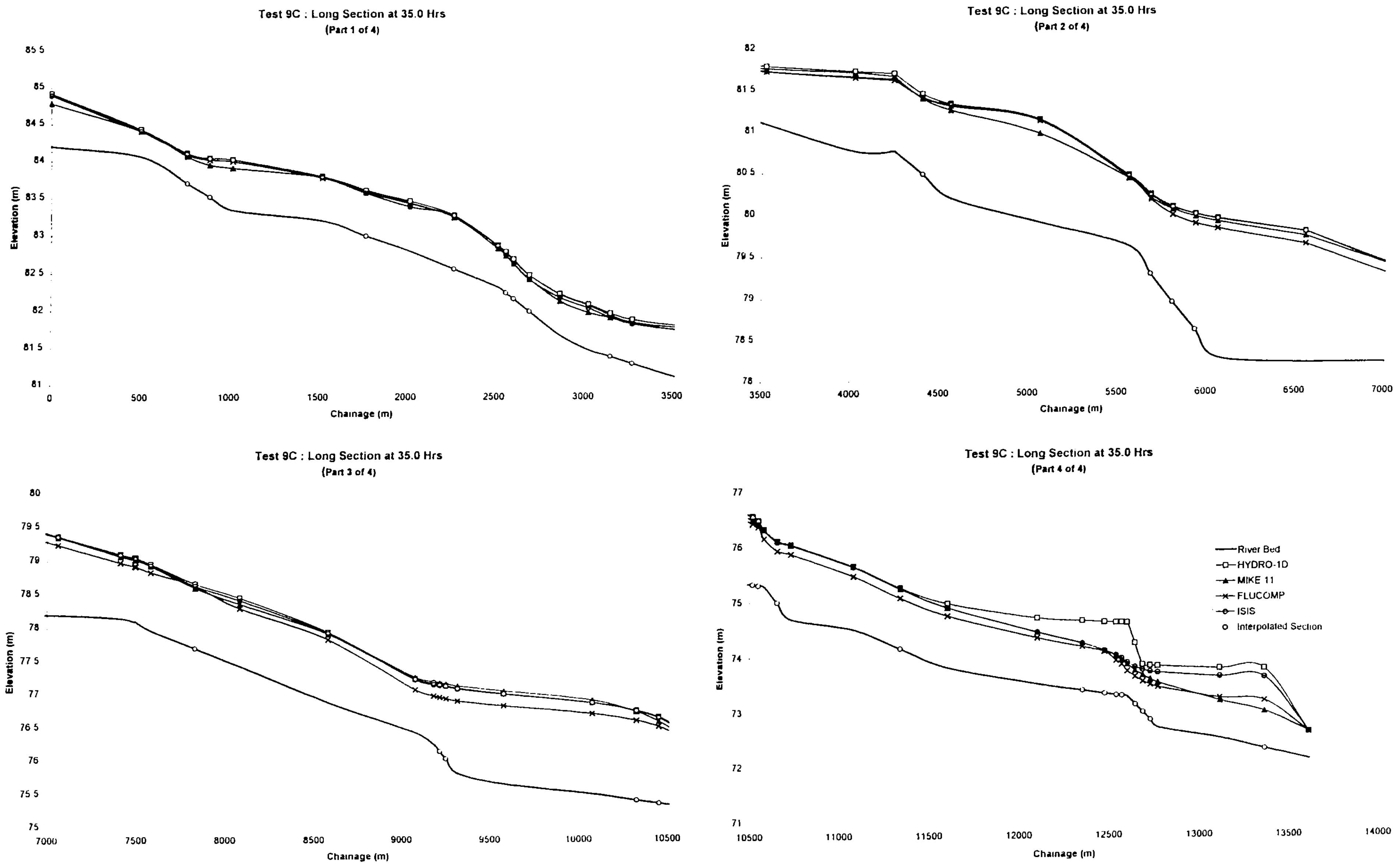
GRAPH 26 : *TEST 9C - Comparison of water elevations at 15.0 Hrs*



GRAPH 27 : *TEST 9C - Comparison of water elevations at 17.5 Hrs*

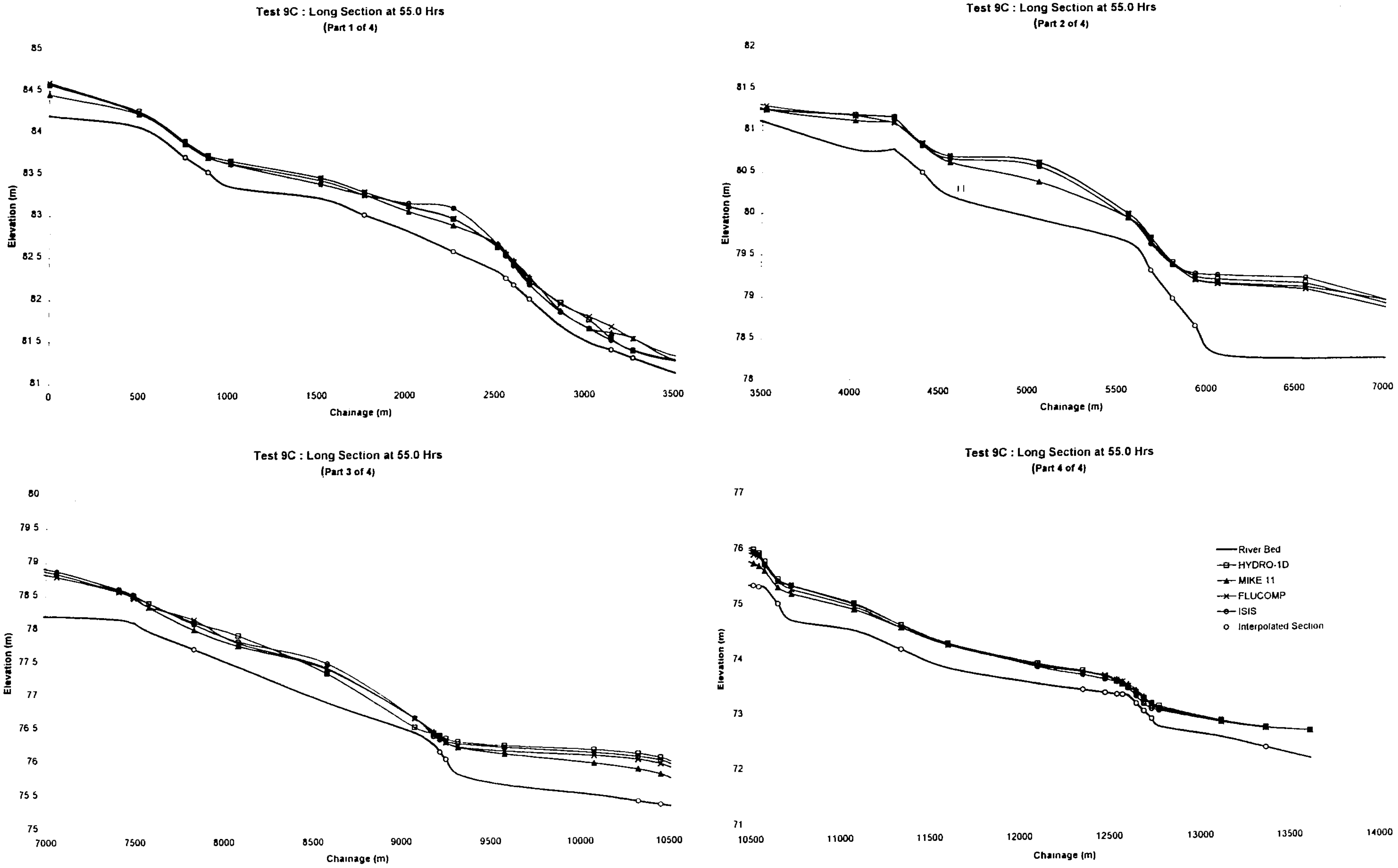


GRAPH 28 : *TEST 9C - Comparison of water elevations at 25.0 Hrs*

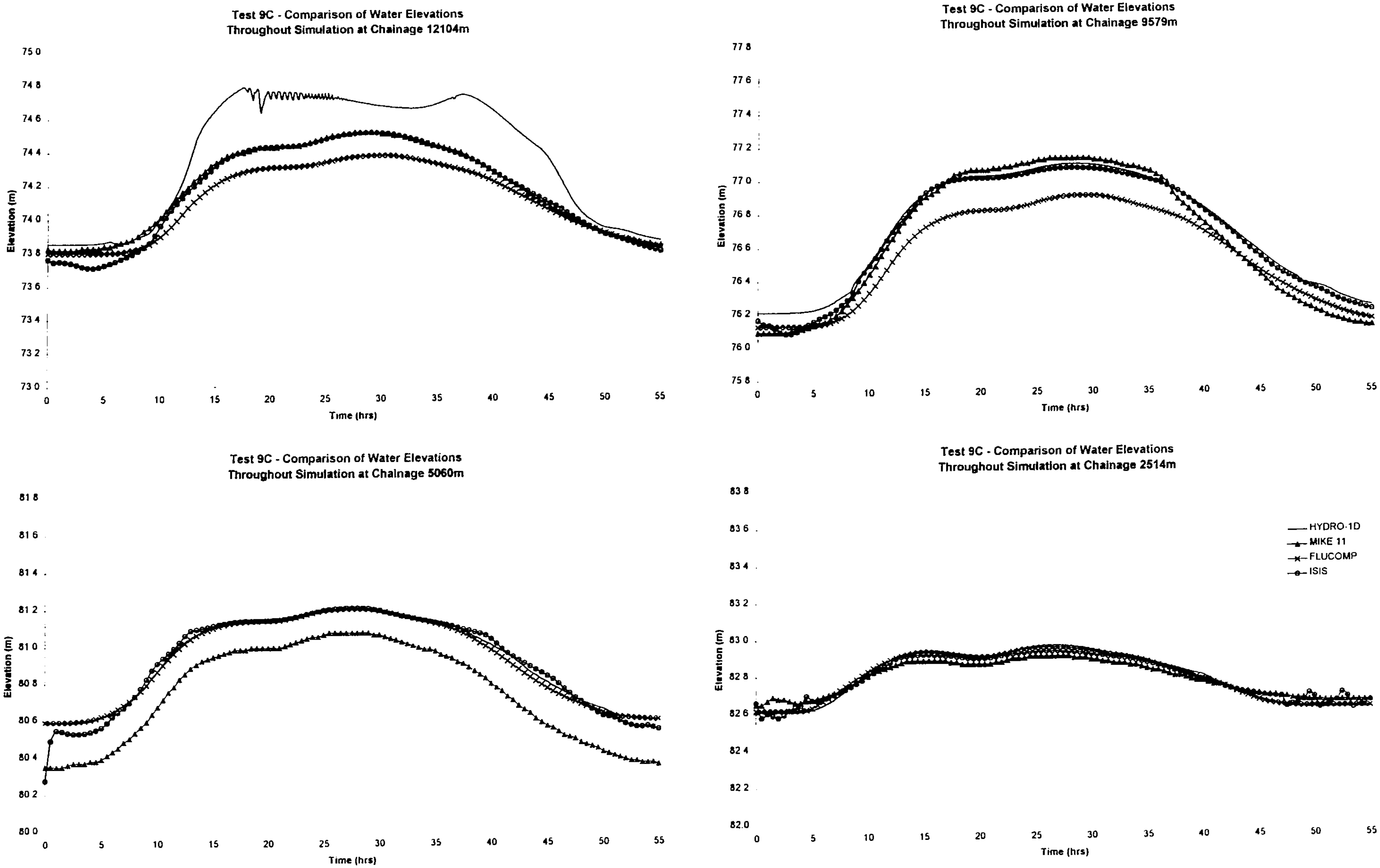


GRAPH 29 : *TEST 9C - Comparison of water elevations at 35.0 Hrs*

11



GRAPH 30 : *TEST 9C - Comparison of water elevations at 55.0 Hrs*

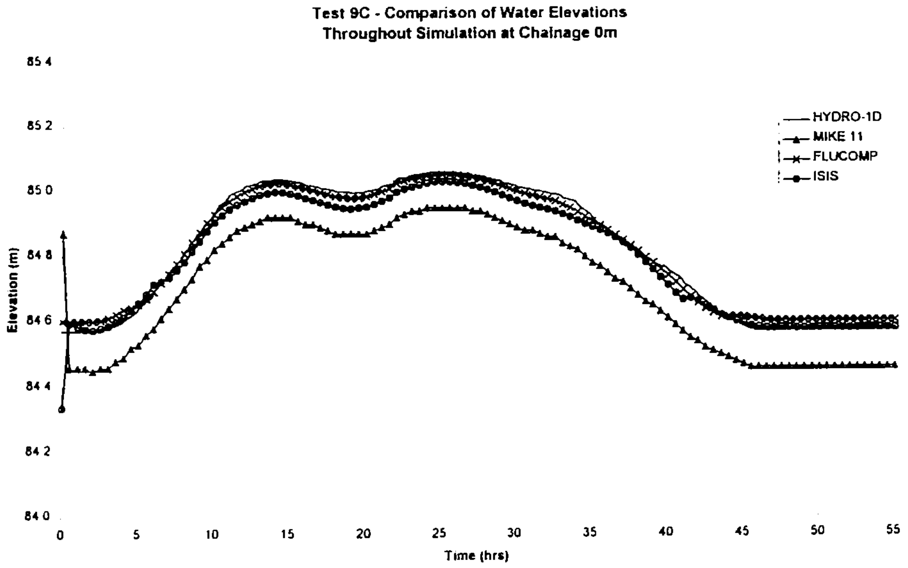

Test 9C - Comparison of Water Elevations Throughout Simulation at Chainage 12104m

Test 9C - Comparison of Water Elevations Throughout Simulation at Chainage 9579m

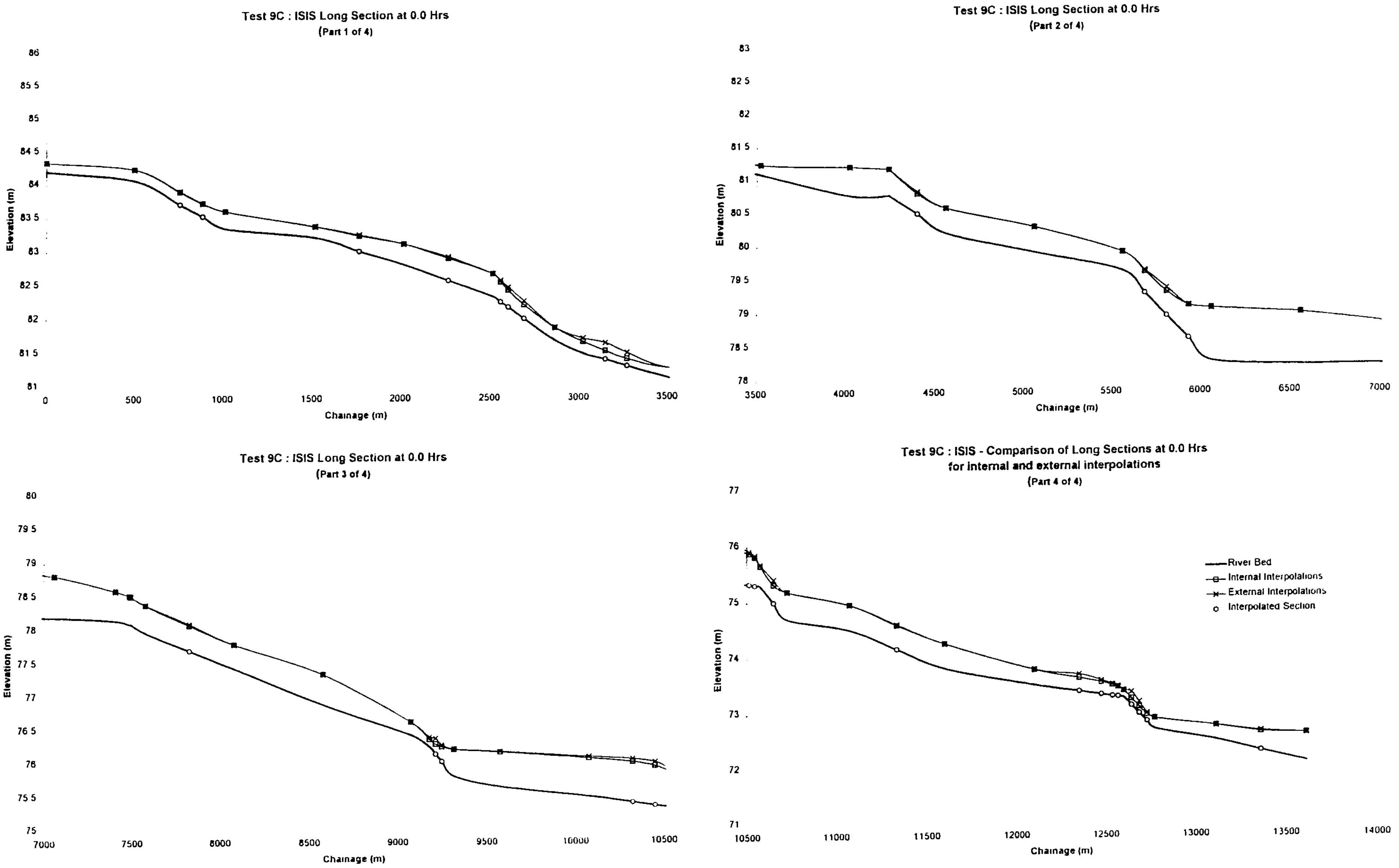
Test 9C - Comparison of Water Elevations Throughout Simulation at Chainage 5060m

Test 9C - Comparison of Water Elevations Throughout Simulation at Chainage 2514m

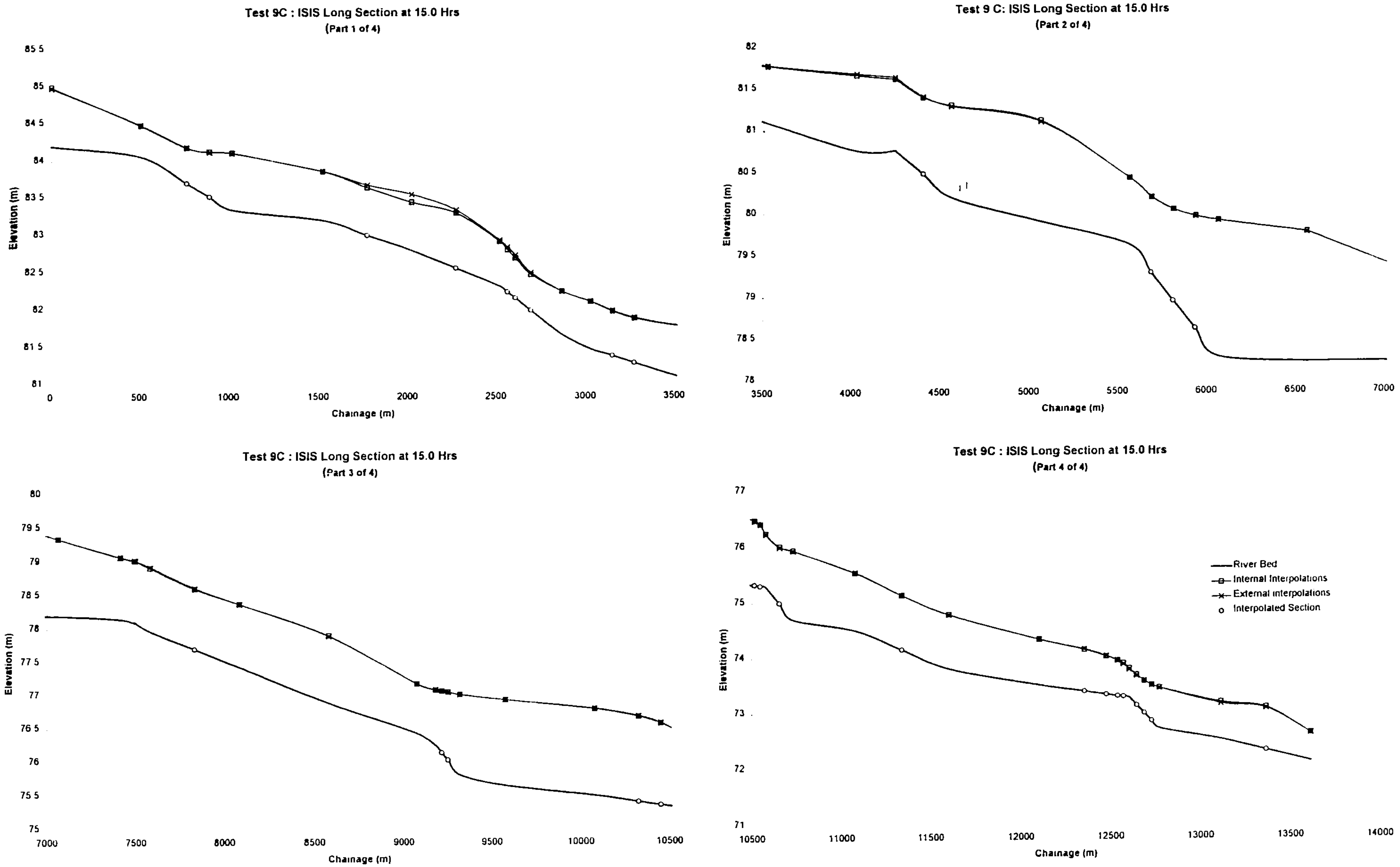
GRAPH 31 : *TEST 9C - Comparison of water elevations throughout simulation*



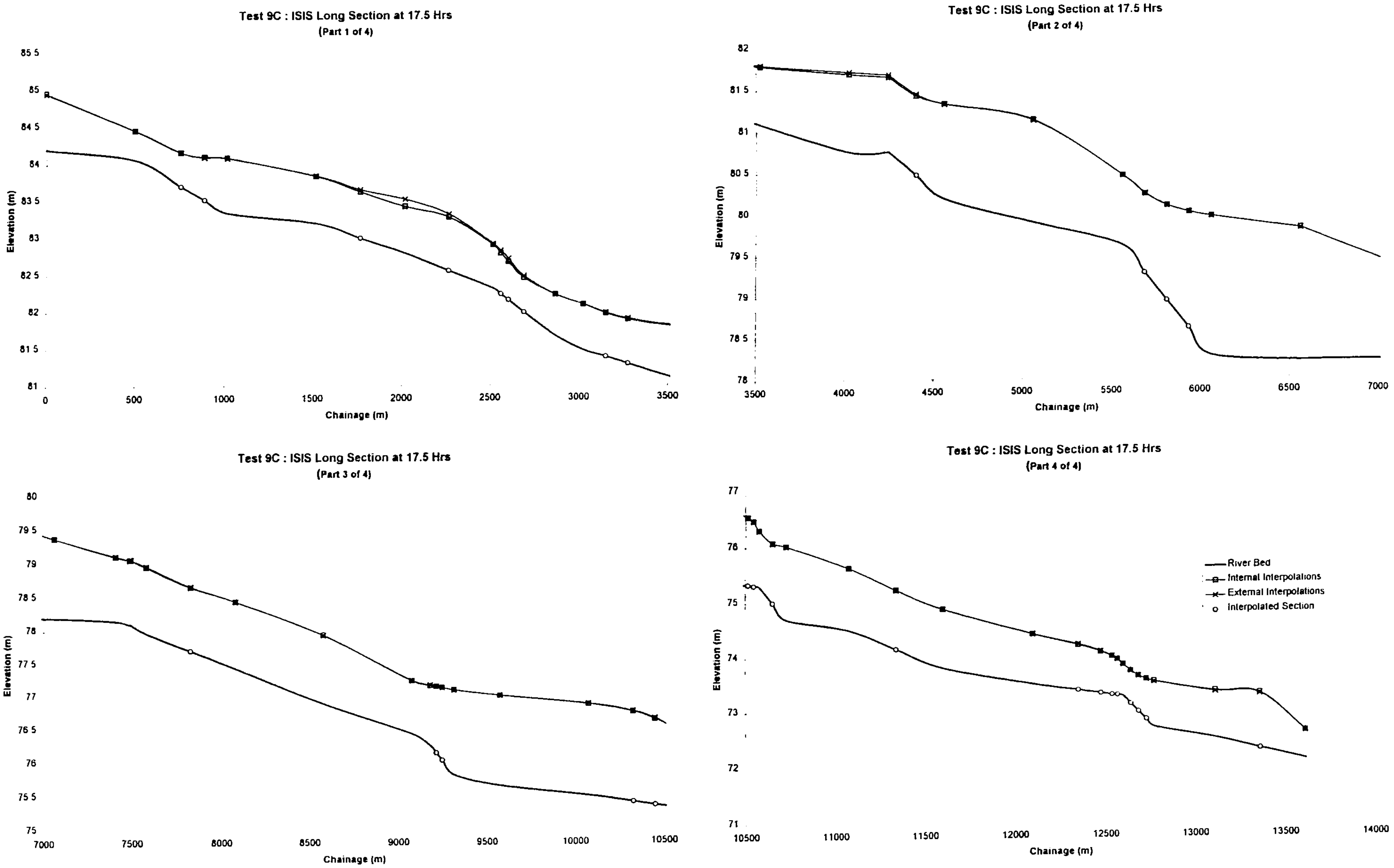
GRAPH 31 : *TEST 9C - Comparison of water elevations throughout simulation*



GRAPH 32 : *TEST 9C - Comparison of water elevations for internal and external interpolations with ISIS at 0.0 Hrs*



GRAPH 33 : *TEST 9C - Comparison of water elevations for internal and external interpolations with ISIS at 15.0 Hrs*



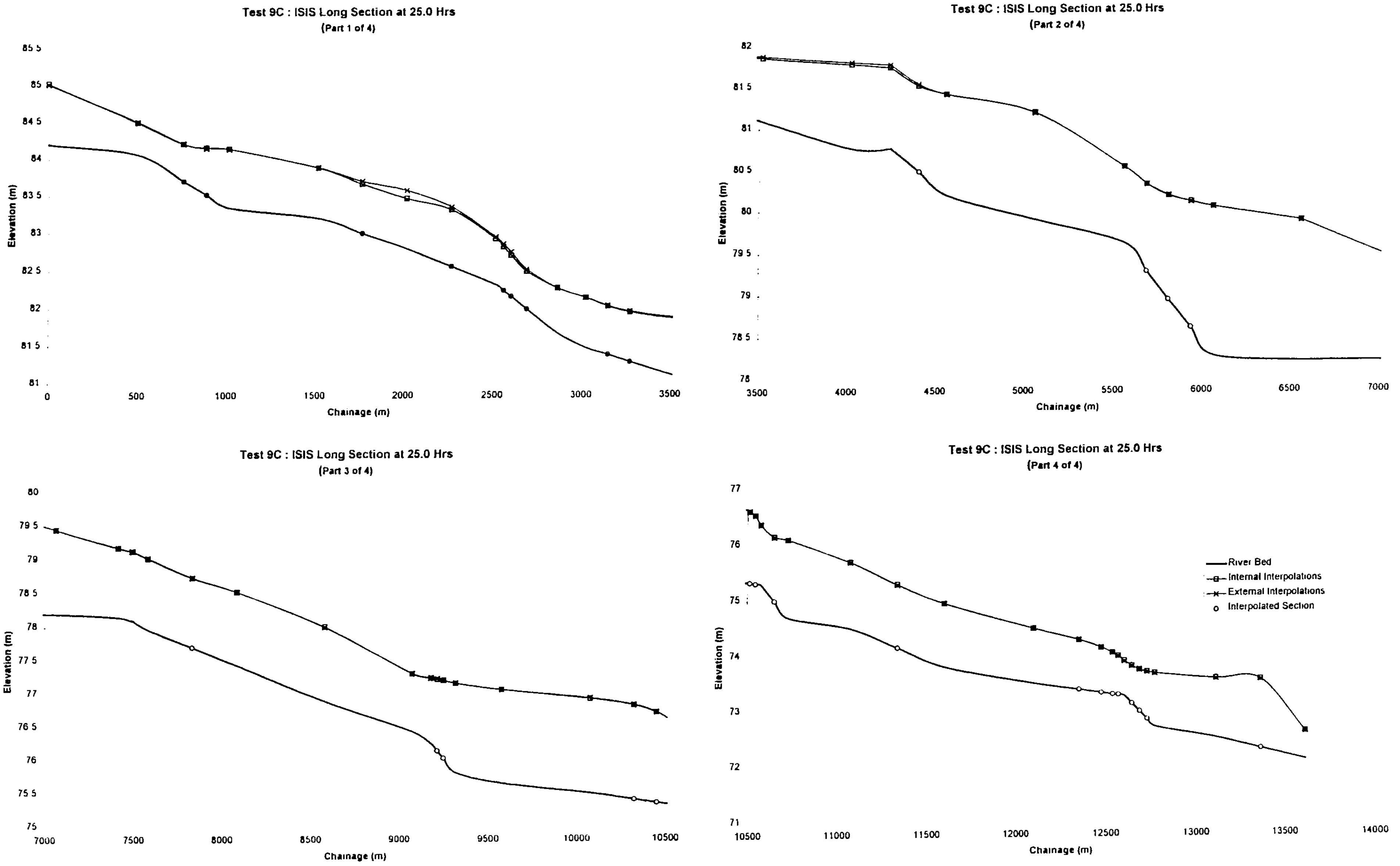
Test 9C : ISIS Long Section at 17.5 Hrs
(Part 1 of 4)

Test 9C : ISIS Long Section at 17.5 Hrs
(Part 2 of 4)

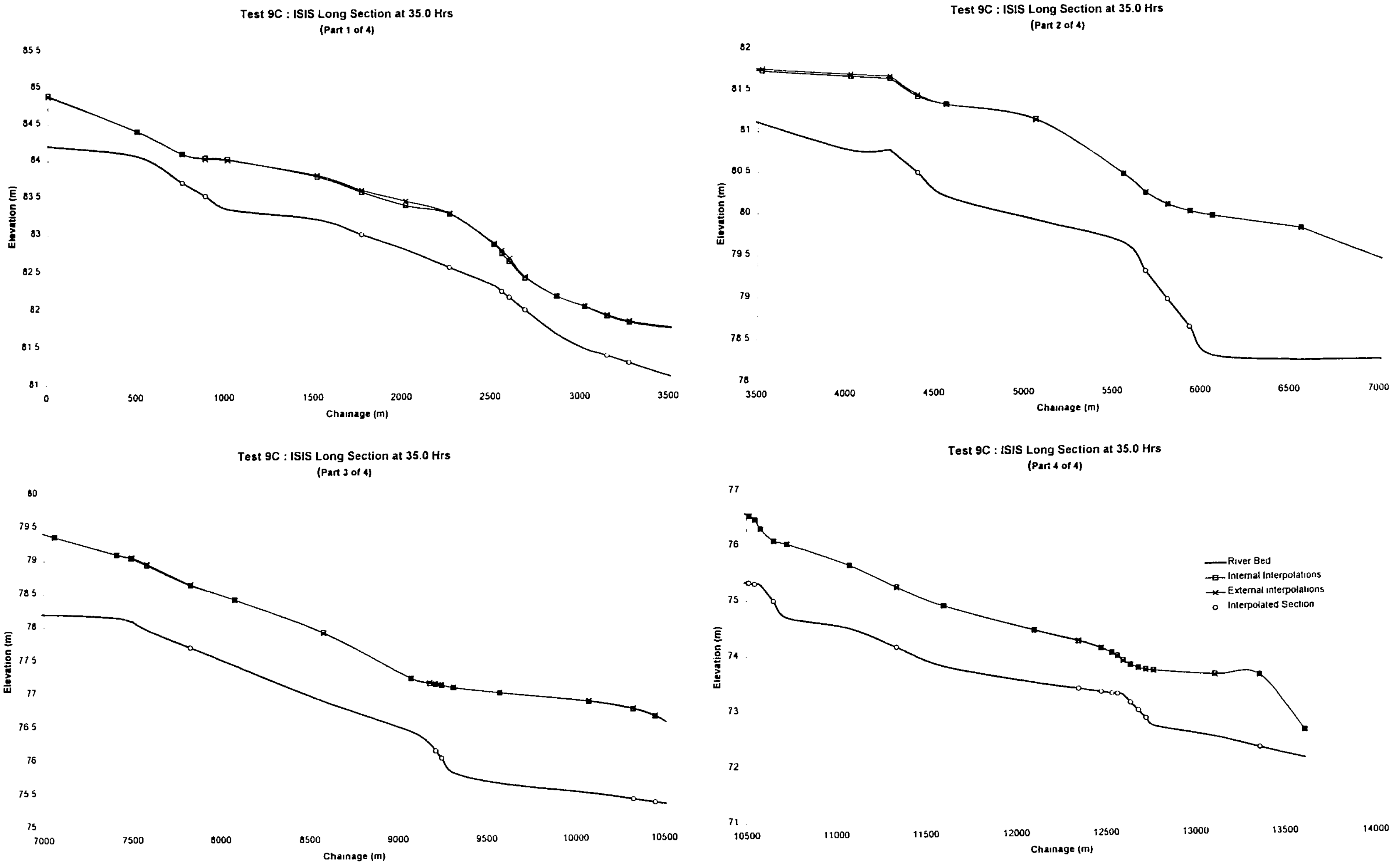
Test 9C : ISIS Long Section at 17.5 Hrs
(Part 3 of 4)

Test 9C : ISIS Long Section at 17.5 Hrs
(Part 4 of 4)

GRAPH 34 : *TEST 9C - Comparison of water elevations for internal and external interpolations with ISIS at 17.5 Hrs*



GRAPH 35 : *TEST 9C - Comparison of water elevations for internal and external interpolations with ISIS at 25.0 Hrs*

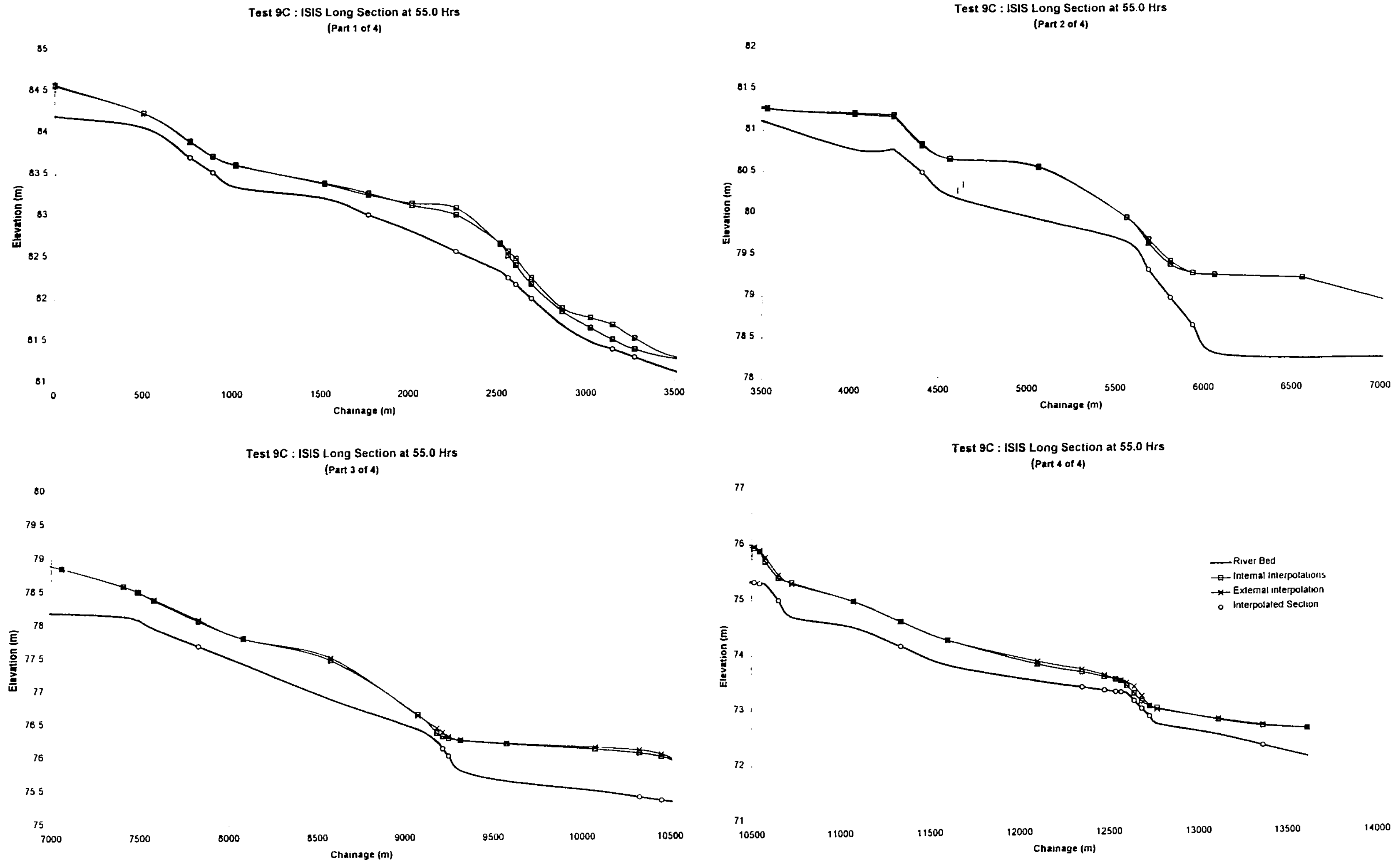

Test 9C : ISIS Long Section at 35.0 Hrs (Part 1 of 4)

Test 9C : ISIS Long Section at 35.0 Hrs (Part 2 of 4)

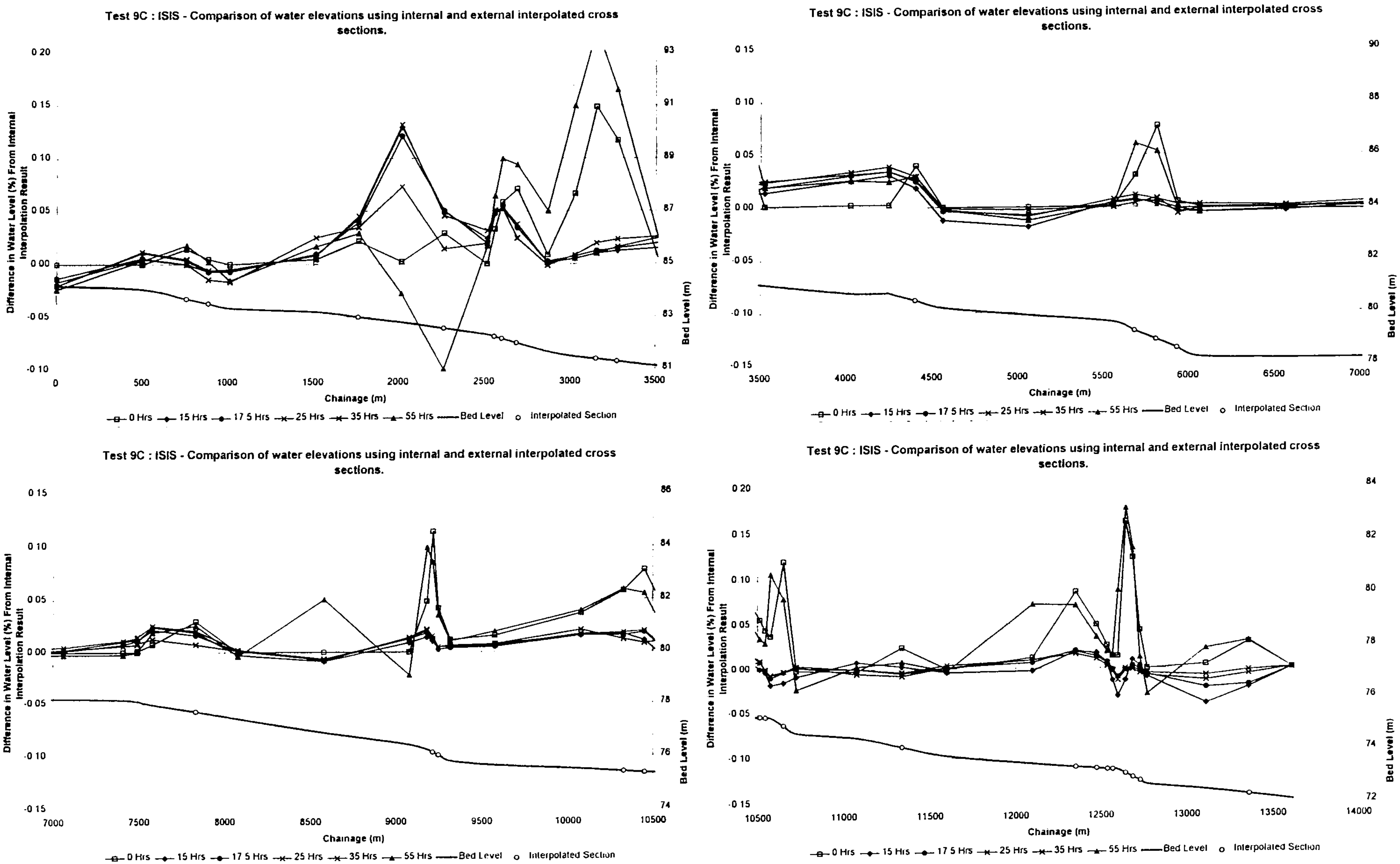
Test 9C : ISIS Long Section at 35.0 Hrs (Part 3 of 4)

Test 9C : ISIS Long Section at 35.0 Hrs (Part 4 of 4)

GRAPH 36 : *TEST 9C - Comparison of water elevations for internal and external interpolations with ISIS at 35.0 Hrs*



GRAPH 37 : *TEST 9C - Comparison of water elevations for internal and external interpolations with ISIS at 55.0 Hrs*


Test 9C : ISIS - Comparison of water elevations using internal and external interpolated cross sections.

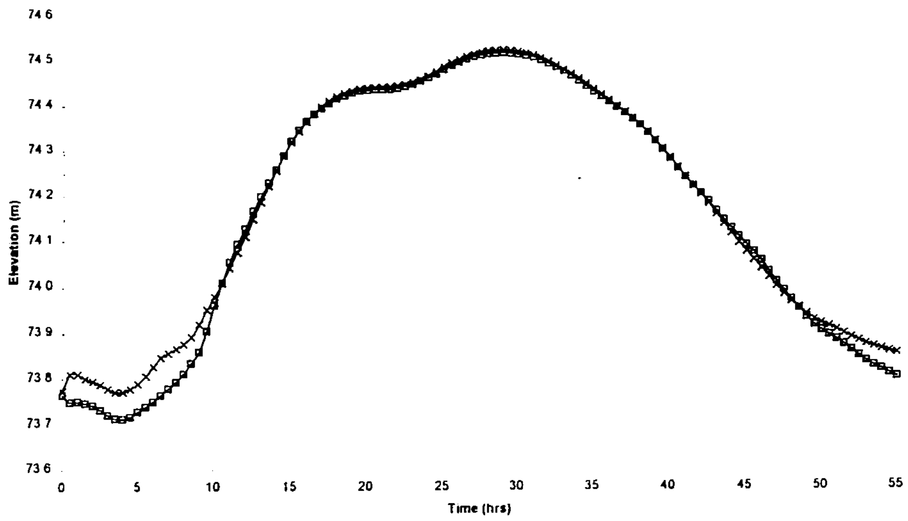
Test 9C : ISIS - Comparison of water elevations using internal and external interpolated cross sections.

Test 9C : ISIS - Comparison of water elevations using internal and external interpolated cross sections.

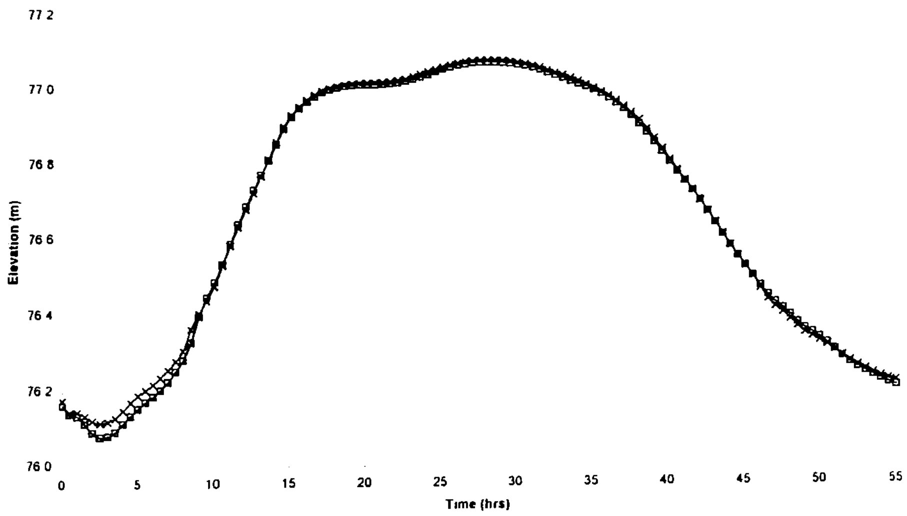
Test 9C : ISIS - Comparison of water elevations using internal and external interpolated cross sections.

GRAPH 38 : *TEST 9B - Difference in water elevations for internal and external interpolations with ISIS.*

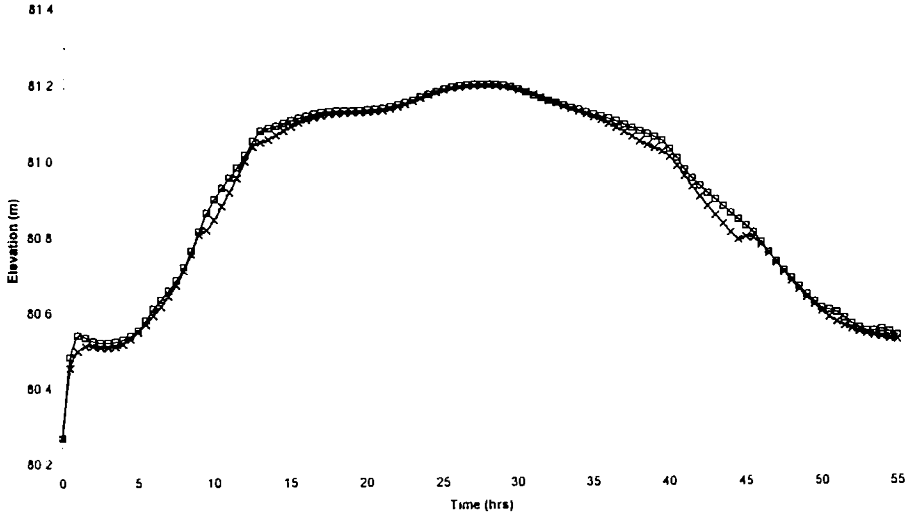
Test 9C : ISIS - Comparison of water elevations for internal and external interpolations throughout simulation at chainage 12104m



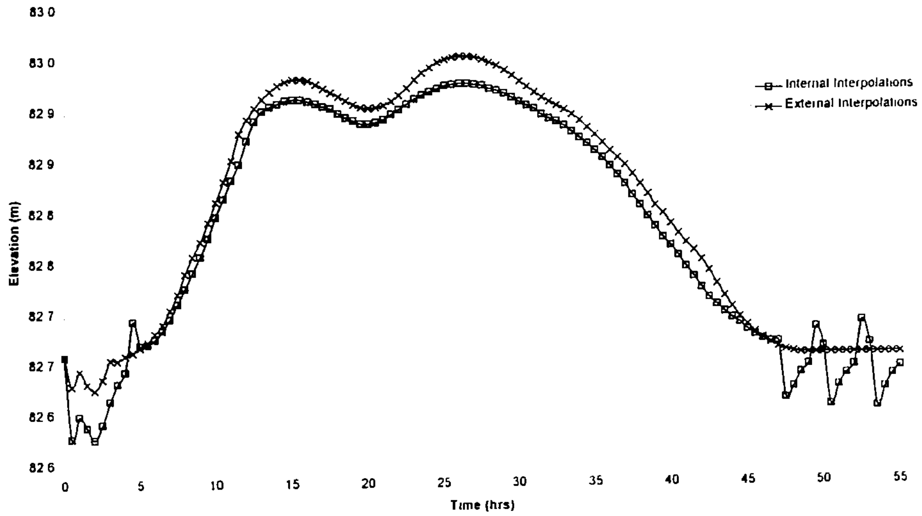
Test 9A : ISIS - Comparison of water elevations for internal and external interpolations throughout simulation at chainage 9579m



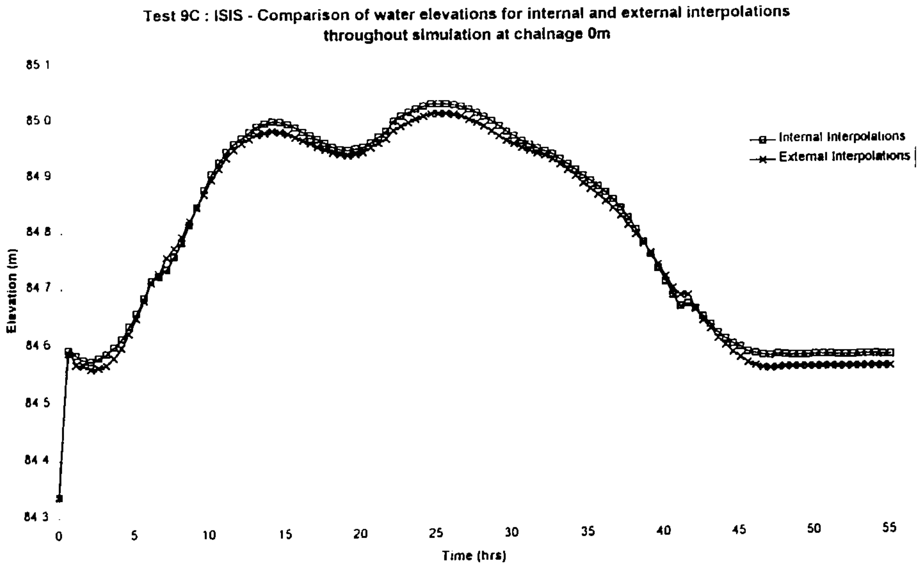
Test 9A : ISIS - Comparison of water elevations for internal and external interpolations throughout simulation at chainage 5060m



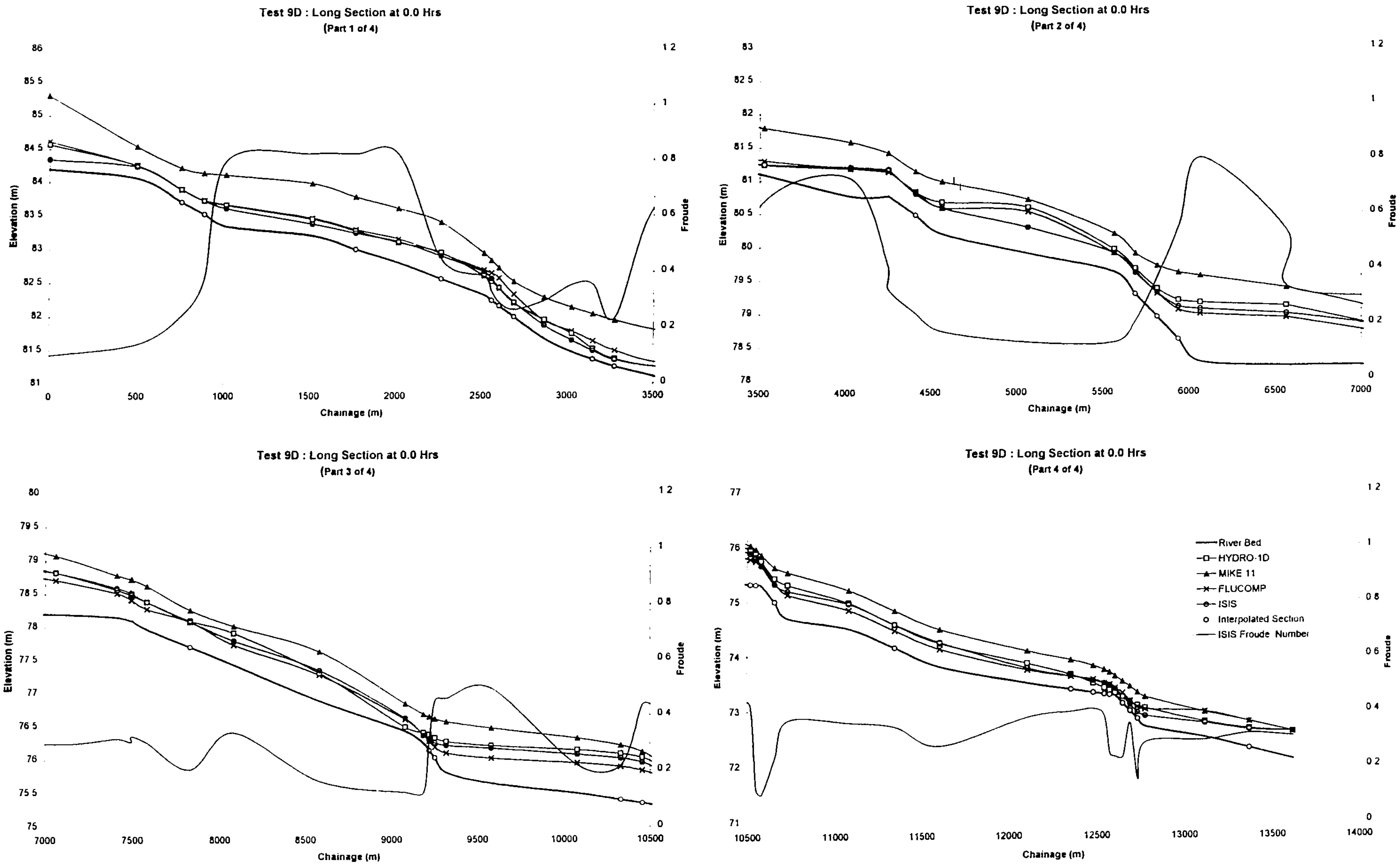
Test 9C : ISIS - Comparison of water elevations for internal and external interpolations throughout simulation at chainage 2514m



GRAPH 39 : *TEST 9C - Comparison of water elevations for internal and external interpolations with ISIS at cross sections*



GRAPH 39 : *TEST 9C - Comparison of water elevations for internal and external interpolations with ISIS at cross sections*

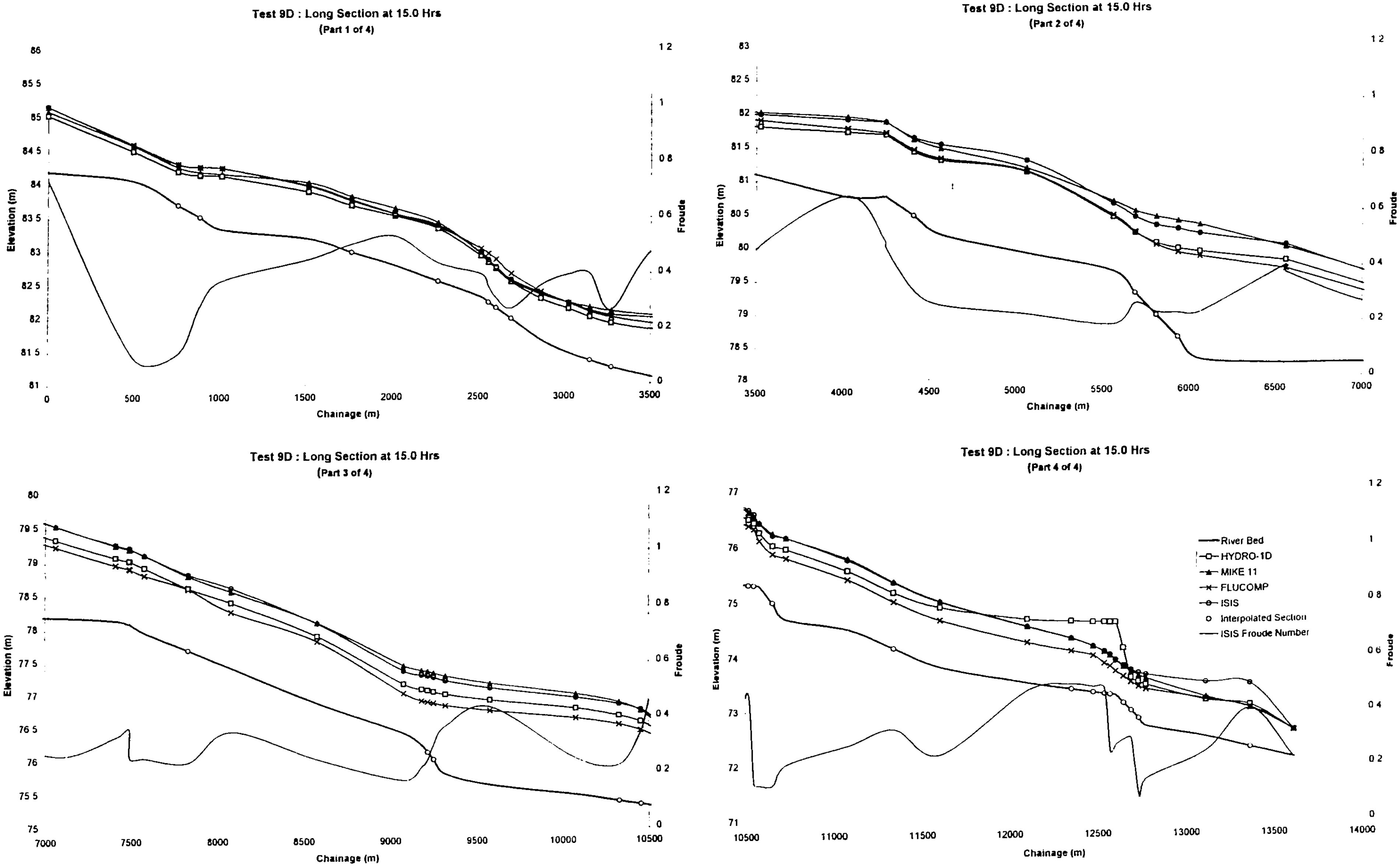

Test 9D : Long Section at 0.0 Hrs (Part 1 of 4)

Test 9D : Long Section at 0.0 Hrs (Part 2 of 4)

Test 9D : Long Section at 0.0 Hrs (Part 3 of 4)

Test 9D : Long Section at 0.0 Hrs (Part 4 of 4)

GRAPH 40 : *TEST 9D - Comparison of water elevations at 0.0 Hrs*



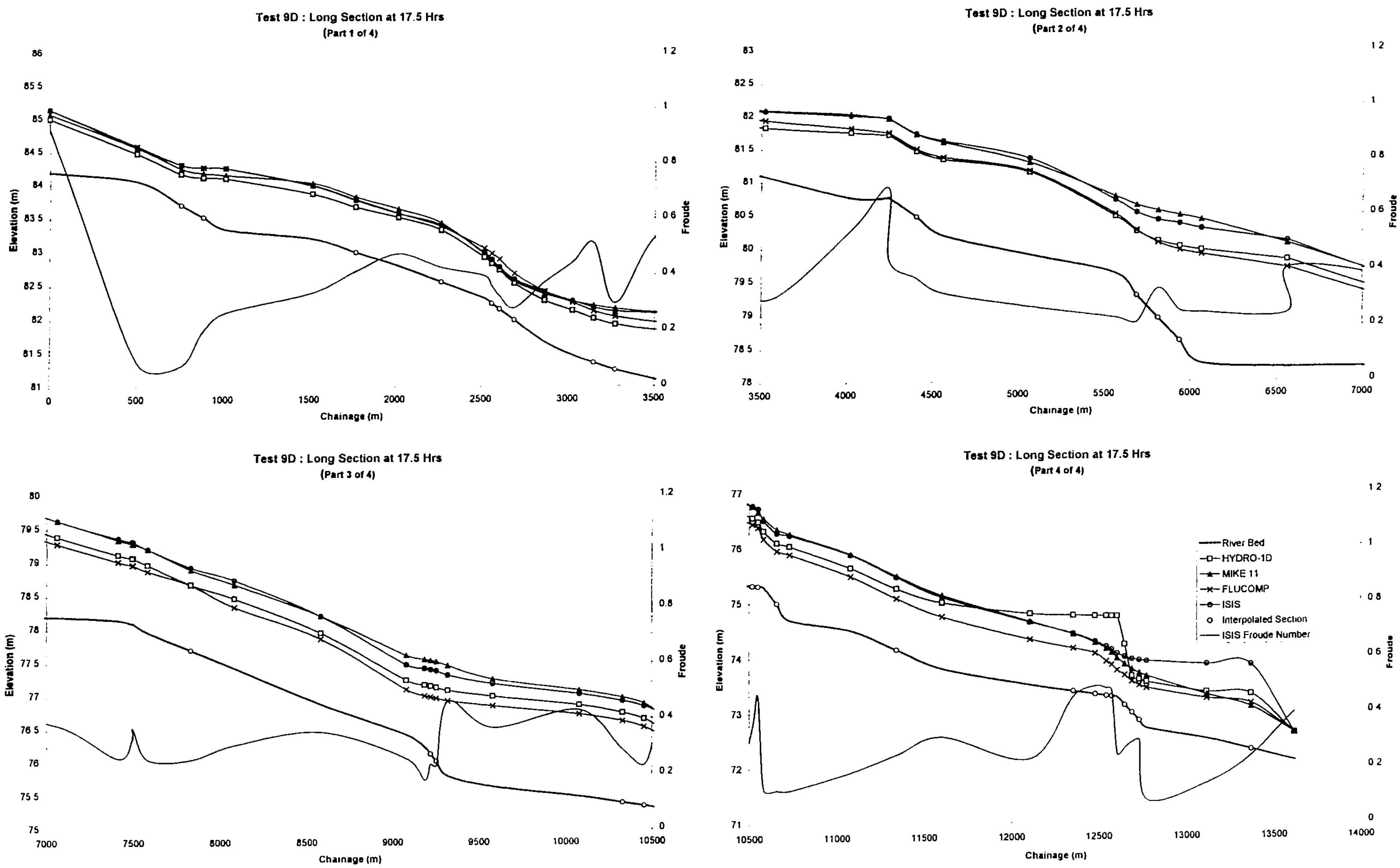
Test 9D : Long Section at 15.0 Hrs (Part 1 of 4)

Test 9D : Long Section at 15.0 Hrs (Part 2 of 4)

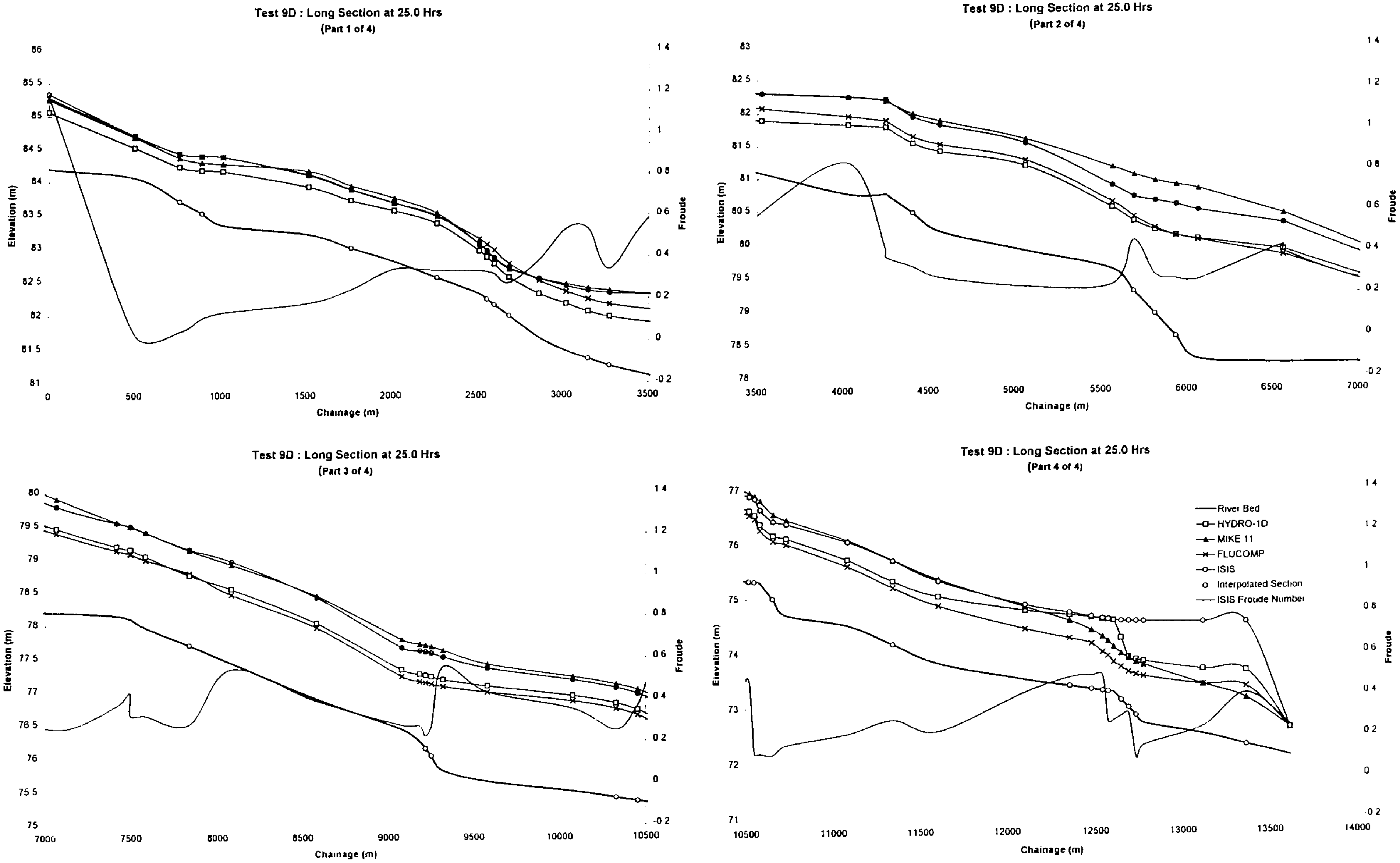
Test 9D : Long Section at 15.0 Hrs (Part 3 of 4)

Test 9D : Long Section at 15.0 Hrs (Part 4 of 4)

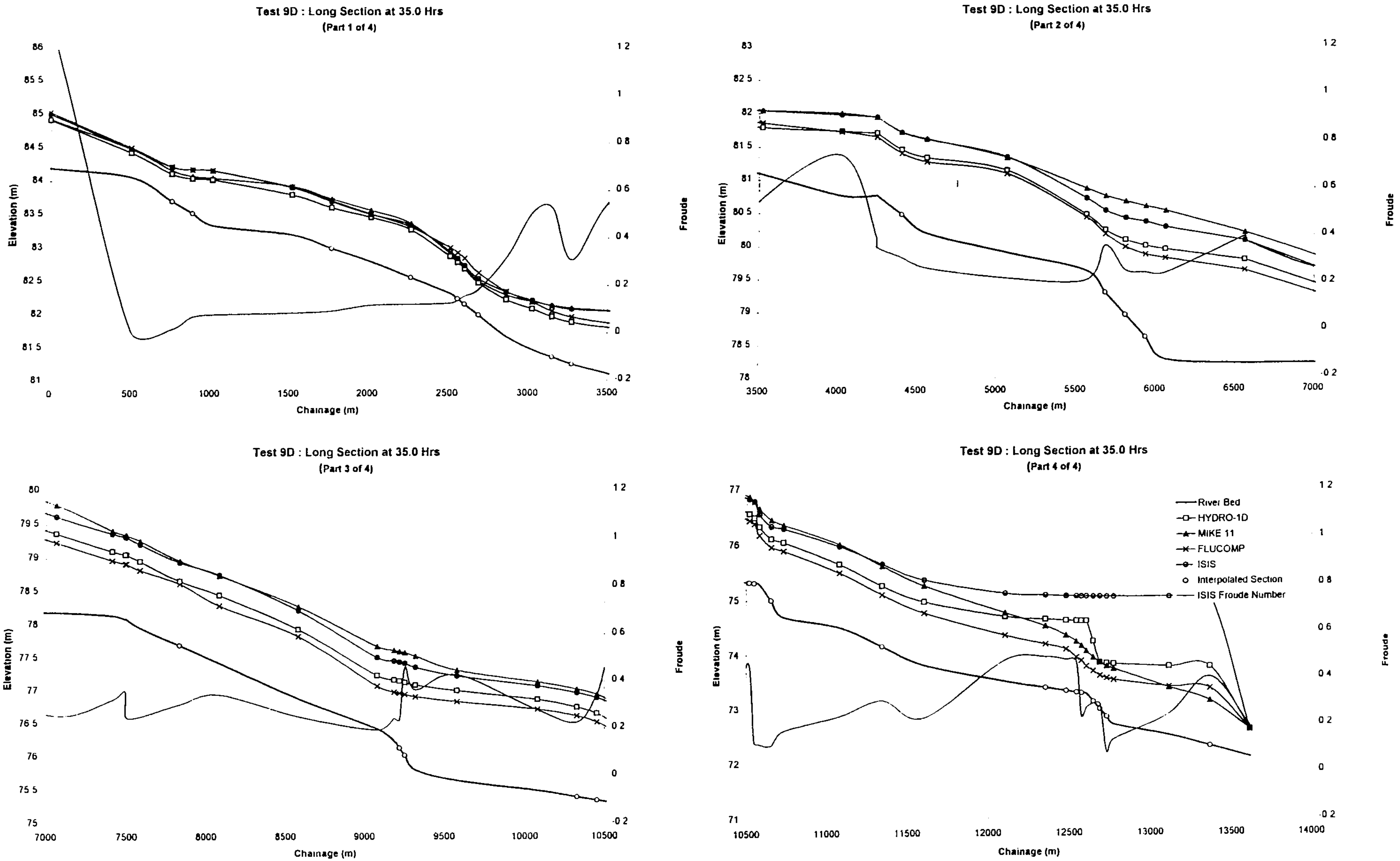
GRAPH 41 : *TEST 9D - Comparison of water elevations at 15.0 Hrs*



GRAPH 42 : *TEST 9D - Comparison of water elevations at 17.5 Hrs*



GRAPH 43 : *TEST 9D - Comparison of water elevations at 25.0 Hrs*

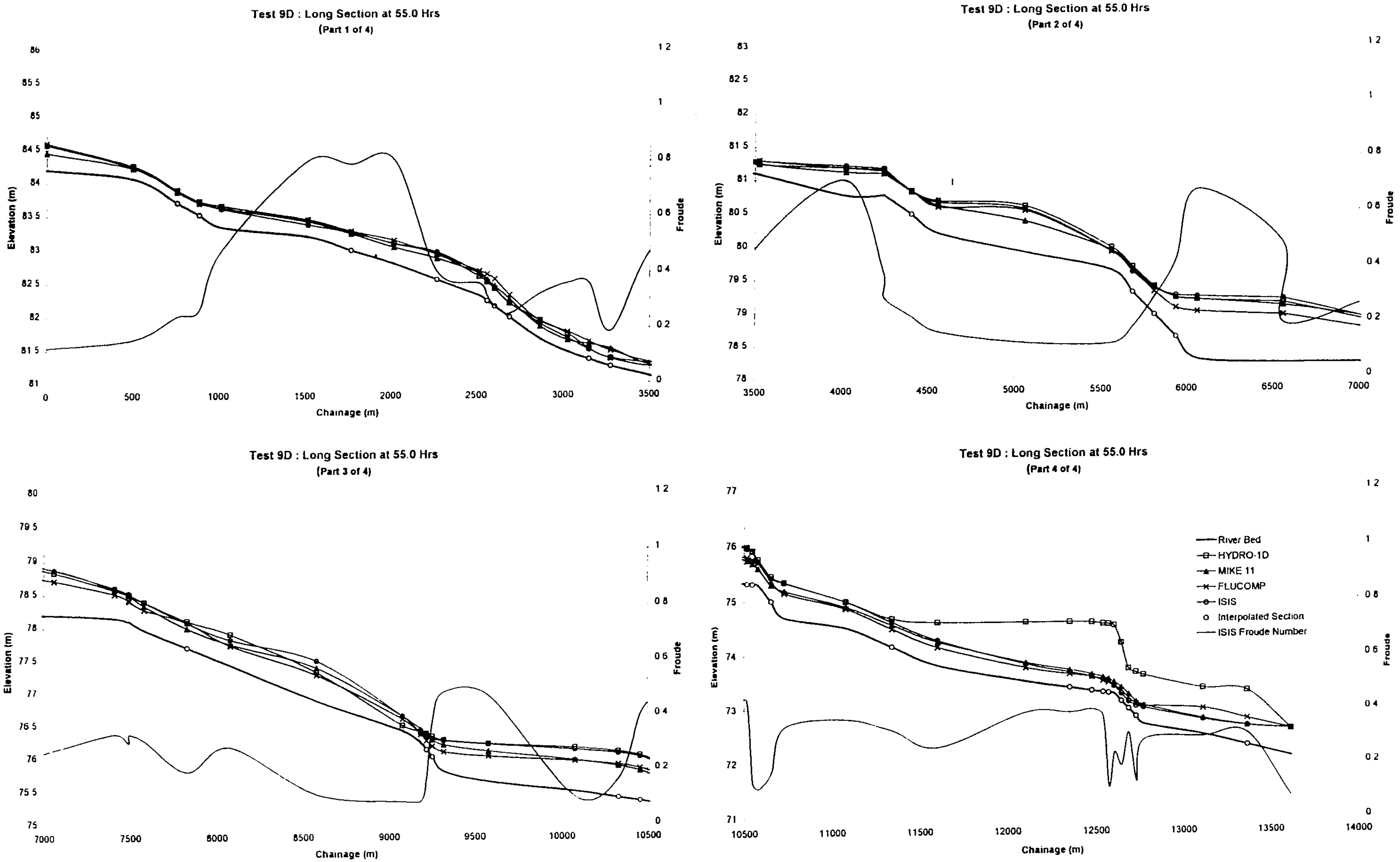

Test 9D : Long Section at 35.0 Hrs (Part 1 of 4)

Test 9D : Long Section at 35.0 Hrs (Part 2 of 4)

Test 9D : Long Section at 35.0 Hrs (Part 3 of 4)

Test 9D : Long Section at 35.0 Hrs (Part 4 of 4)

GRAPH 44 : *TEST 9D - Comparison of water elevations at 35.0 Hrs*



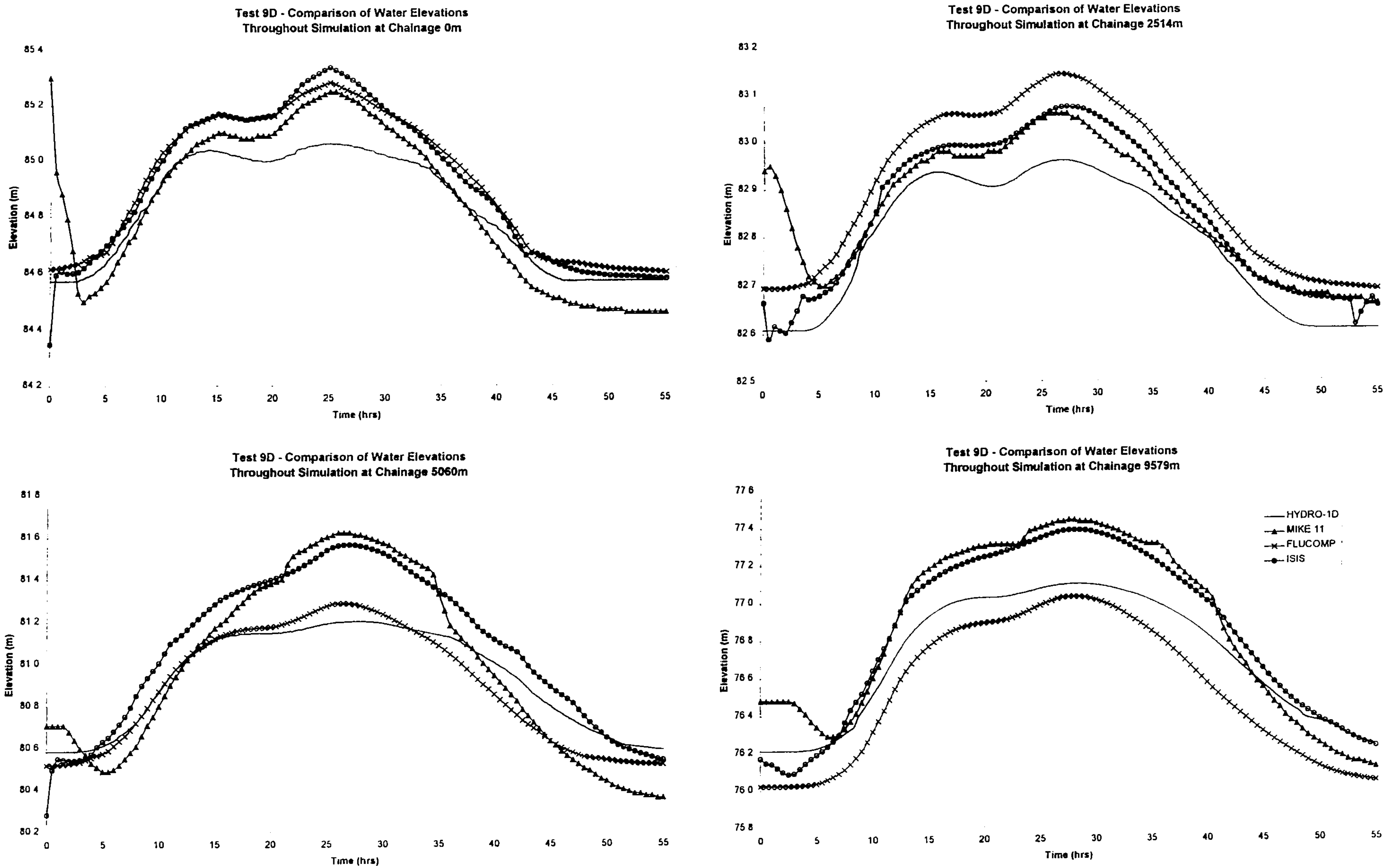
Test 9D : Long Section at 55.0 Hrs
(Part 1 of 4)

Test 9D : Long Section at 55.0 Hrs
(Part 2 of 4)

Test 9D : Long Section at 55.0 Hrs
(Part 3 of 4)

Test 9D : Long Section at 55.0 Hrs
(Part 4 of 4)

GRAPH 45 : *TEST 9D - Comparison of water elevations at 55.0 Hrs*

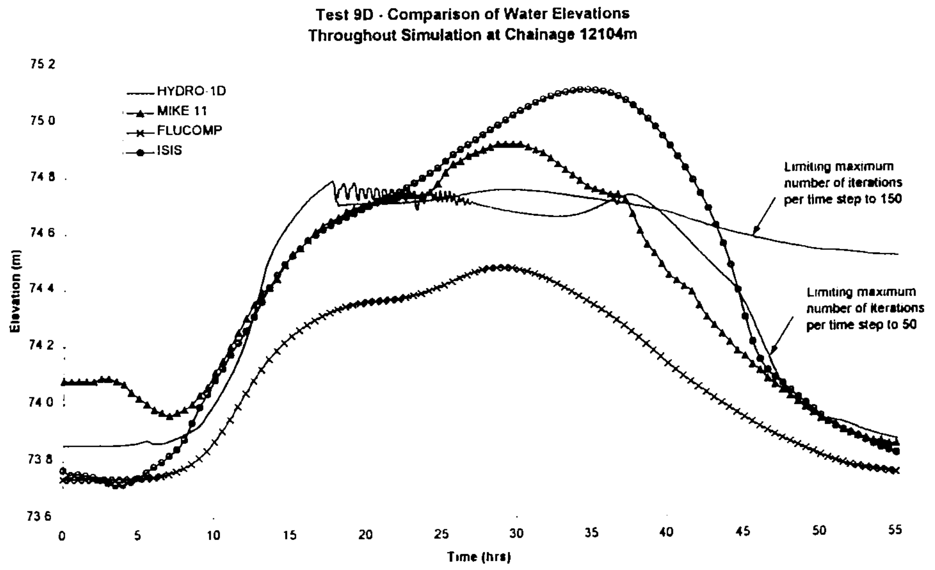

Test 9D - Comparison of Water Elevations Throughout Simulation at Chainage 0m

Test 9D - Comparison of Water Elevations Throughout Simulation at Chainage 2514m

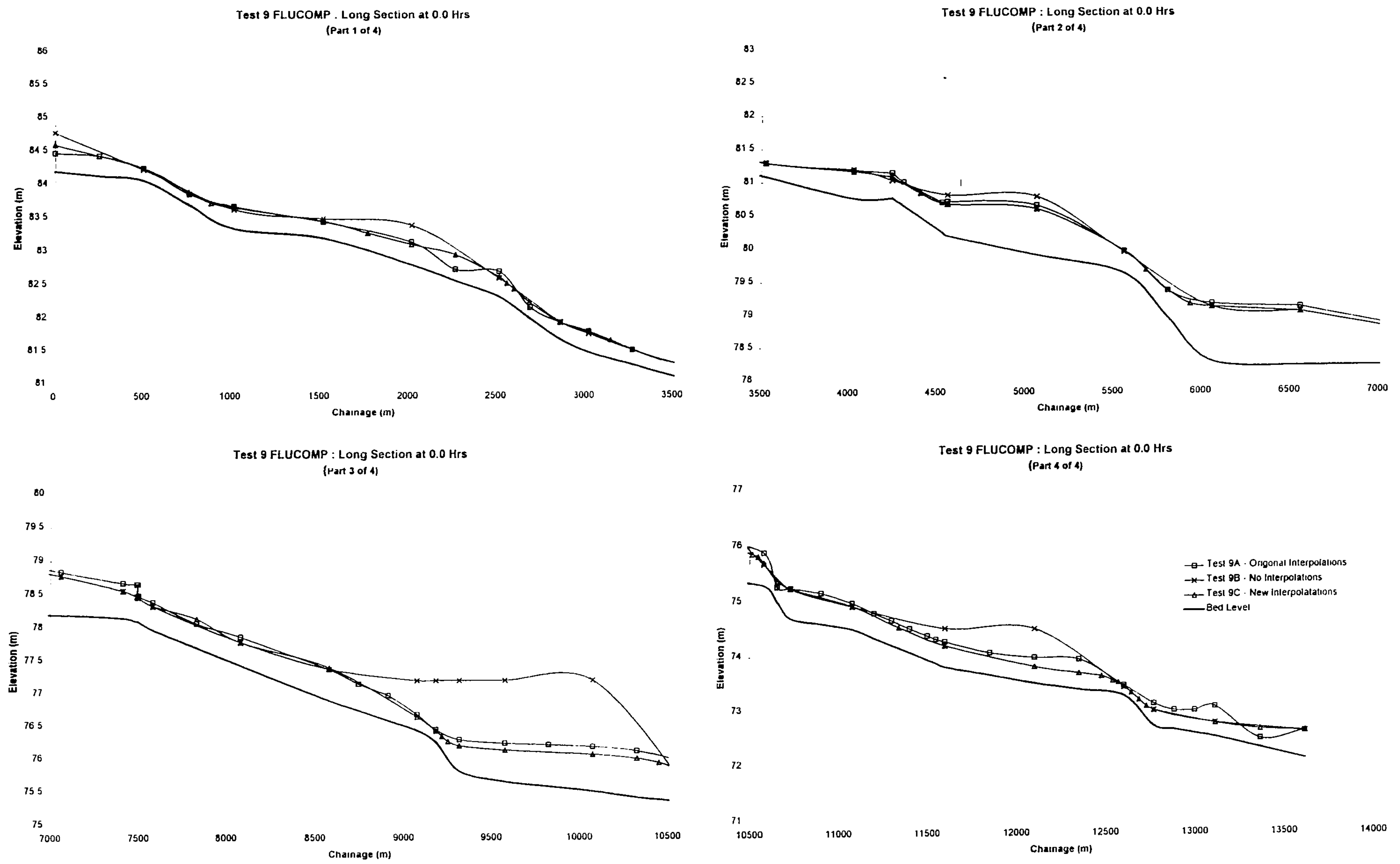
Test 9D - Comparison of Water Elevations Throughout Simulation at Chainage 5060m

Test 9D - Comparison of Water Elevations Throughout Simulation at Chainage 9579m

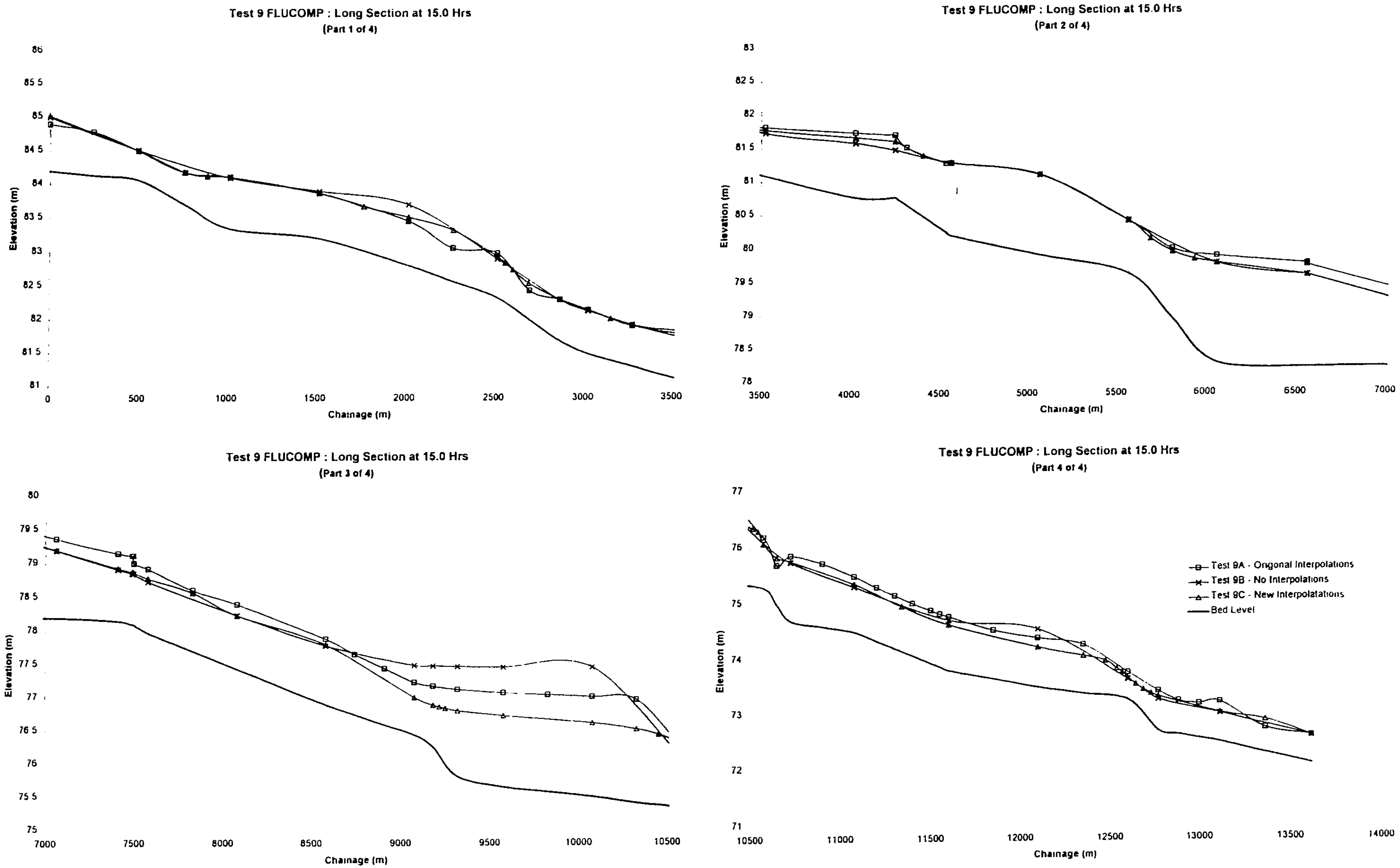
GRAPH 46 : *TEST 9D - Comparison of water elevations throughout simulation*



GRAPH 46 : *TEST 9D - Comparison of water elevations throughout simulation*



GRAPH 47 : *TEST 9 FLUCOMP - Comparison of water elevations at 0.0 Hrs*

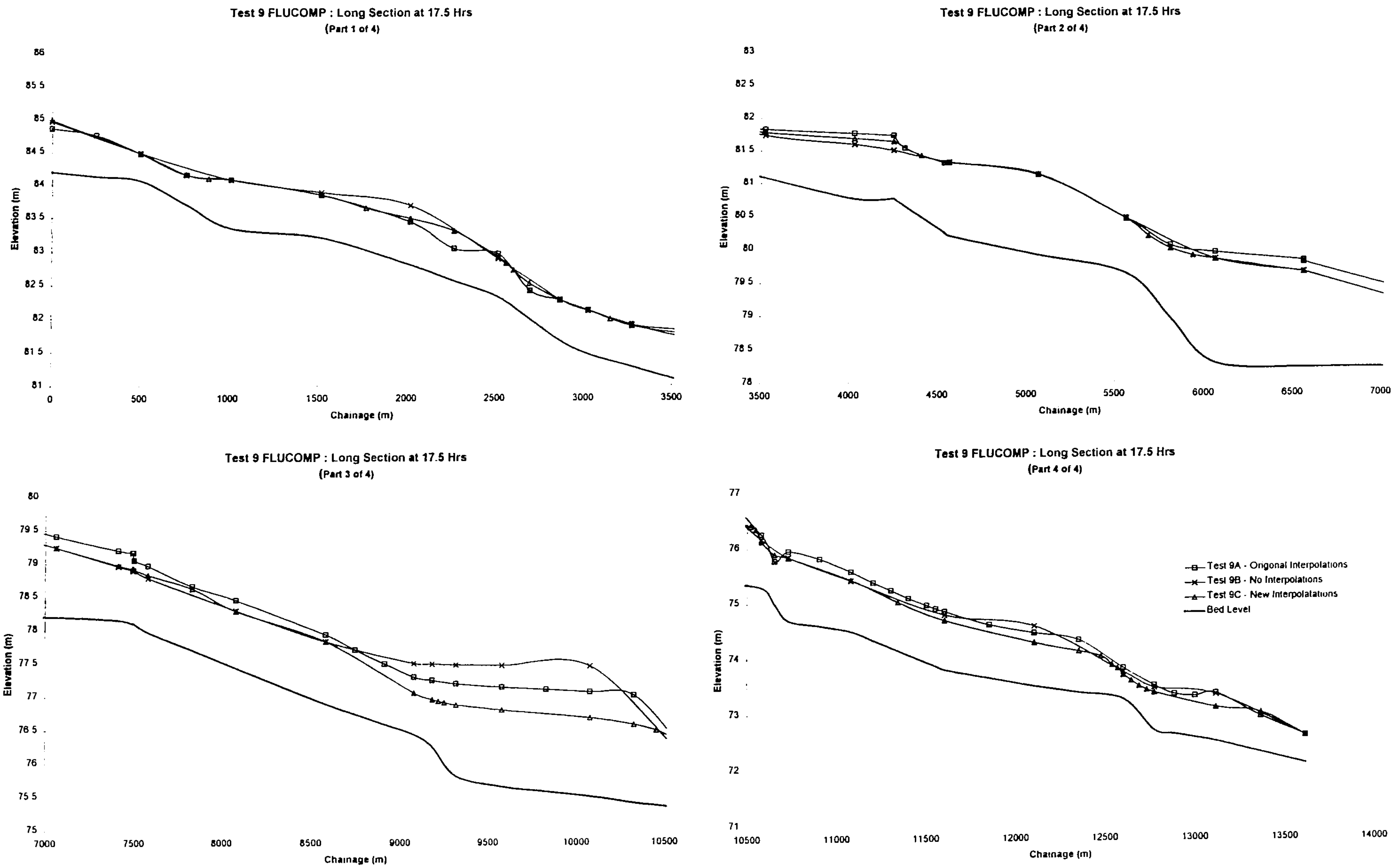

Test 9 FLUCOMP : Long Section at 15.0 Hrs (Part 1 of 4)

Test 9 FLUCOMP : Long Section at 15.0 Hrs (Part 2 of 4)

Test 9 FLUCOMP : Long Section at 15.0 Hrs (Part 3 of 4)

Test 9 FLUCOMP : Long Section at 15.0 Hrs (Part 4 of 4)

GRAPH 48 : *TEST 9 FLUCOMP - Comparison of water elevations at 15.0 Hrs*



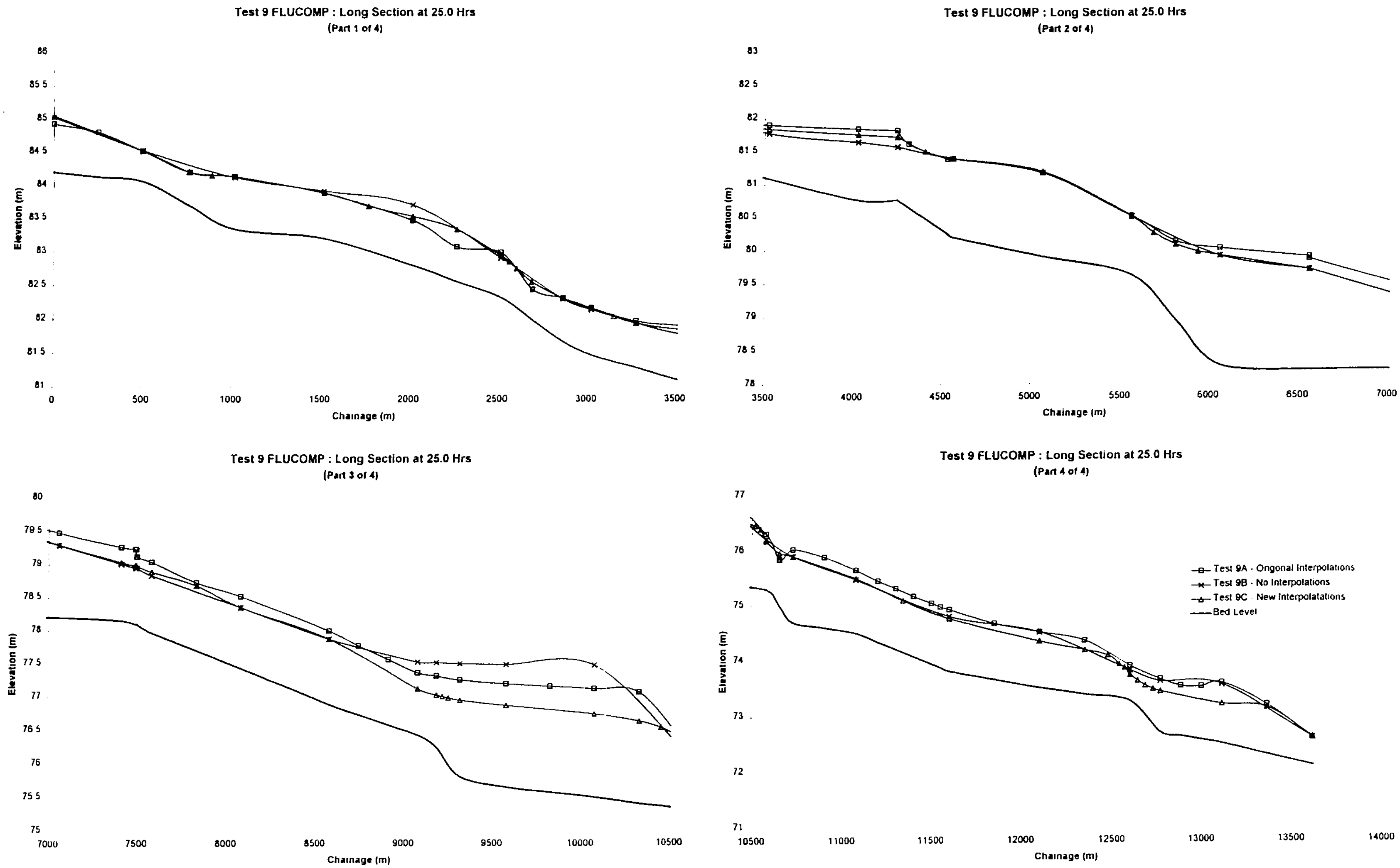
Test 9 FLUCOMP : Long Section at 17.5 Hrs (Part 1 of 4)

Test 9 FLUCOMP : Long Section at 17.5 Hrs (Part 2 of 4)

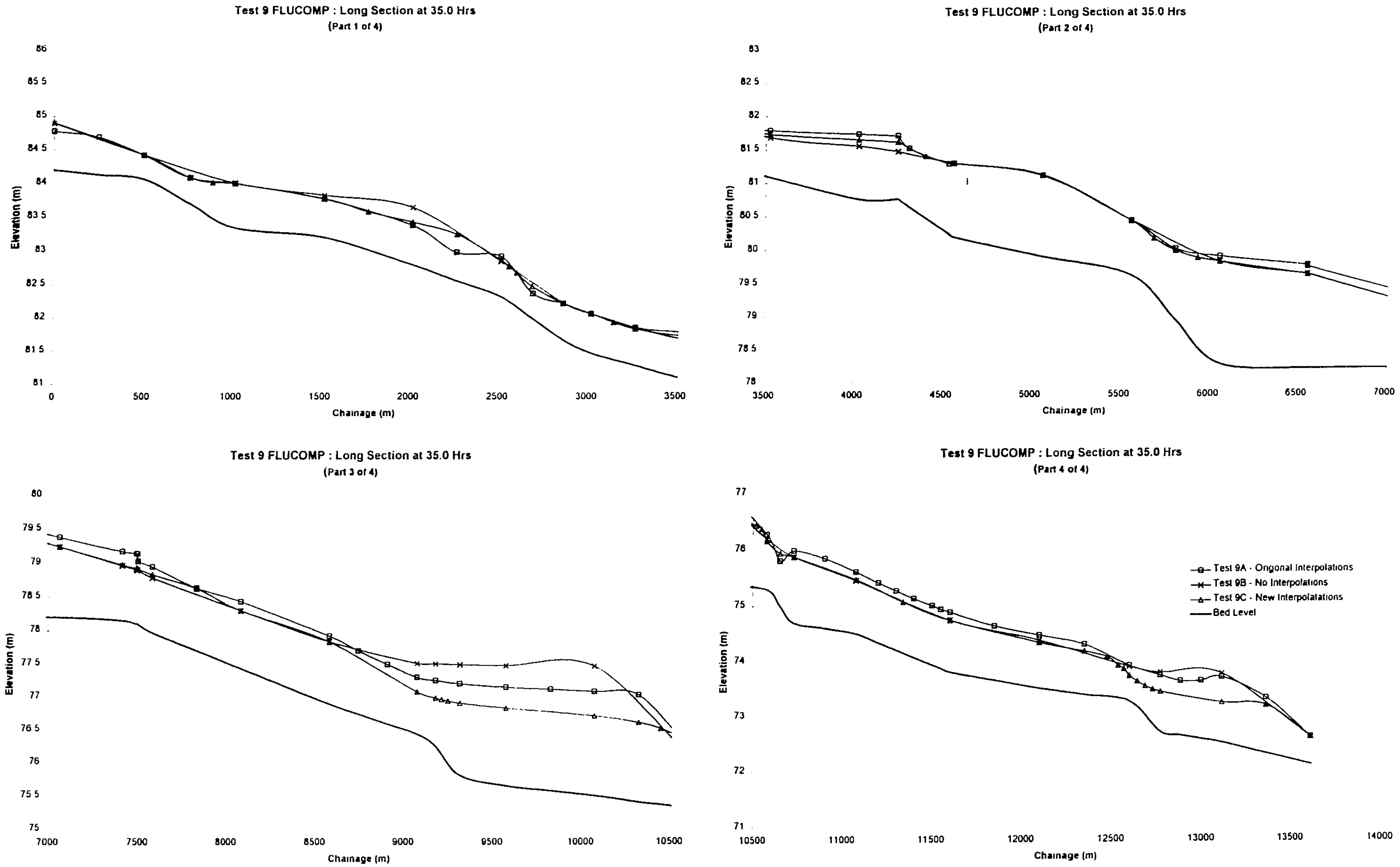
Test 9 FLUCOMP : Long Section at 17.5 Hrs (Part 3 of 4)

Test 9 FLUCOMP : Long Section at 17.5 Hrs (Part 4 of 4)

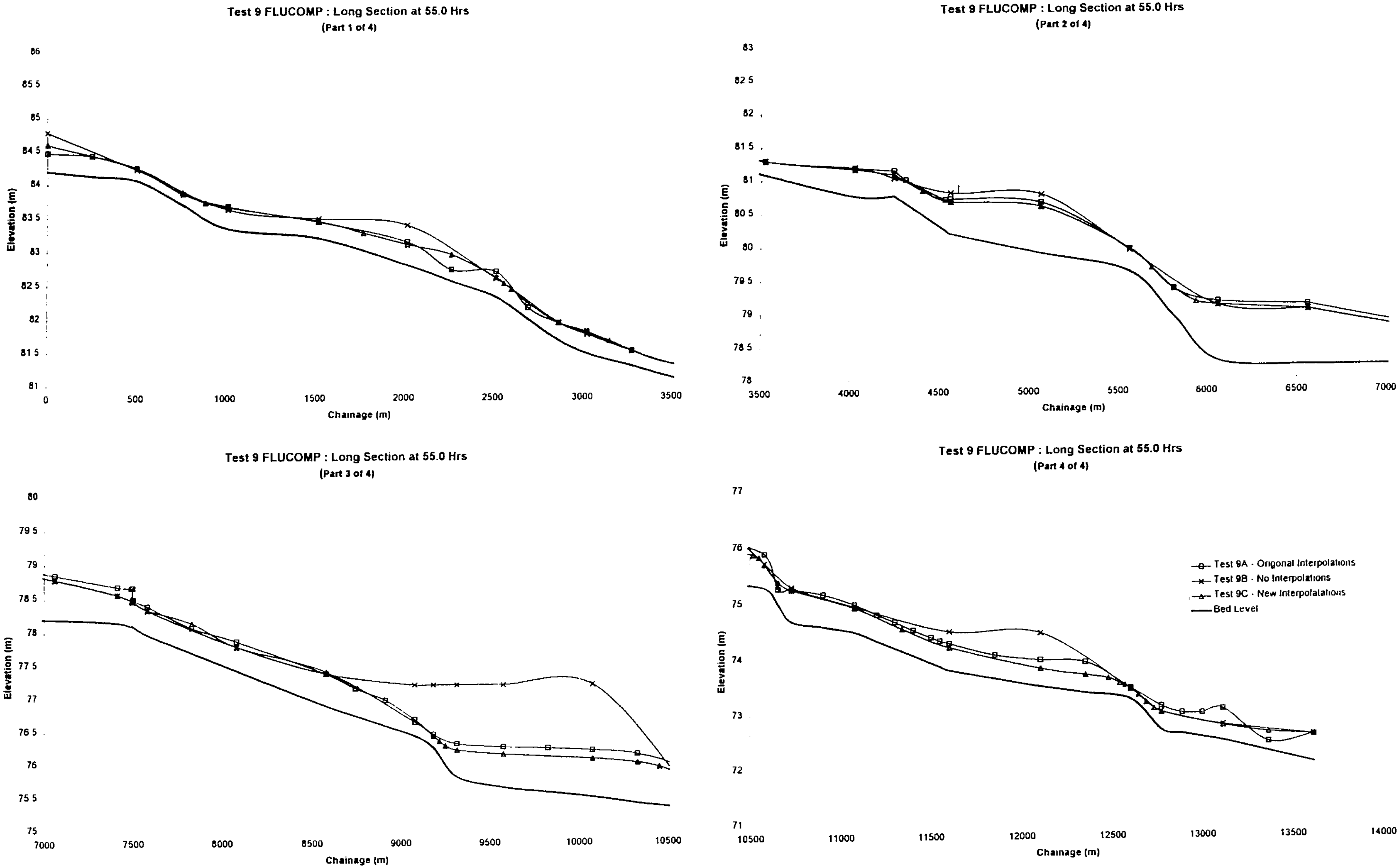
GRAPH 49 : *TEST 9 FLUCOMP - Comparison of water elevations at 17.5 Hrs*



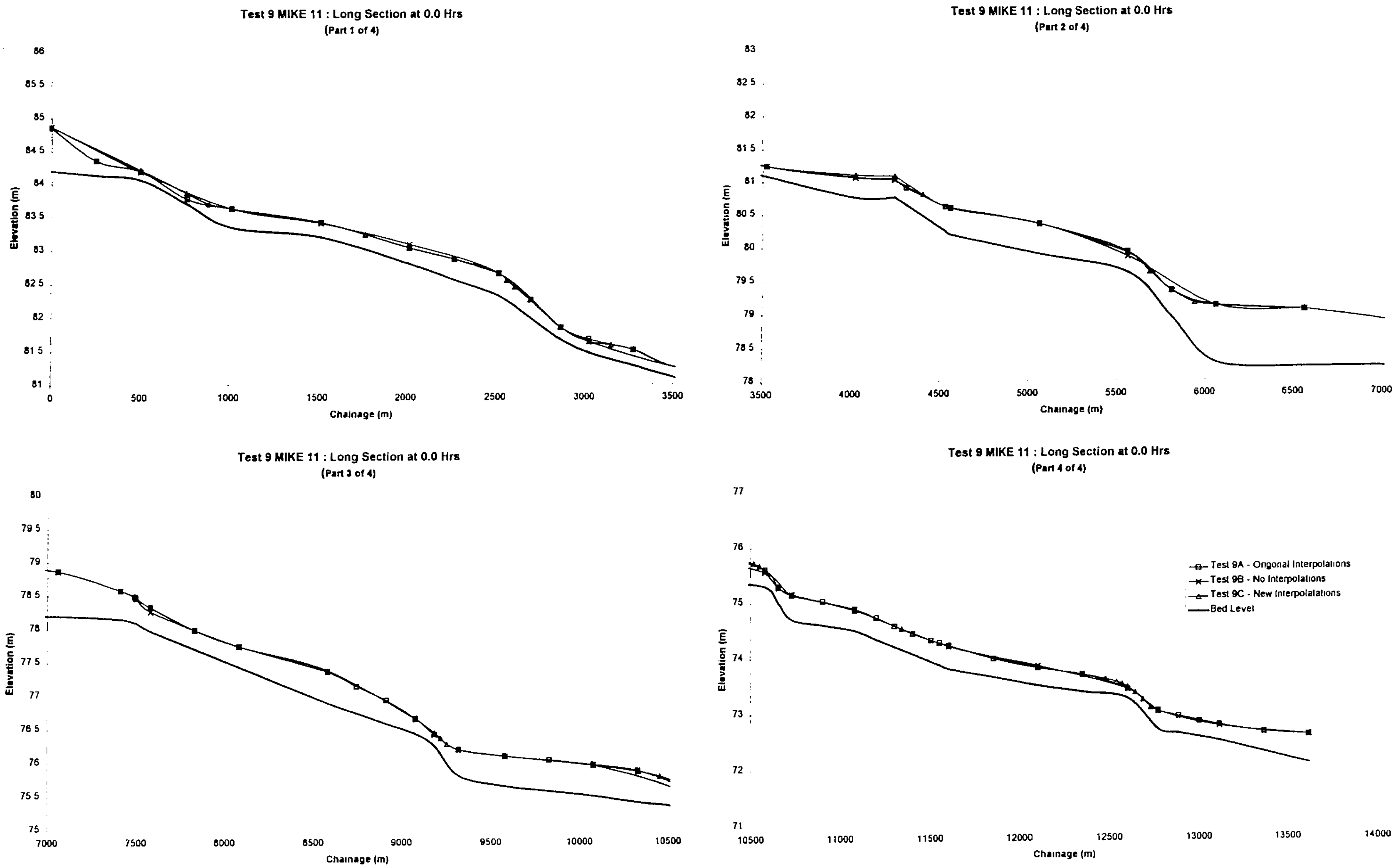
GRAPH 50 : *TEST 9 FLUCOMP - Comparison of water elevations at 25.0 Hrs*



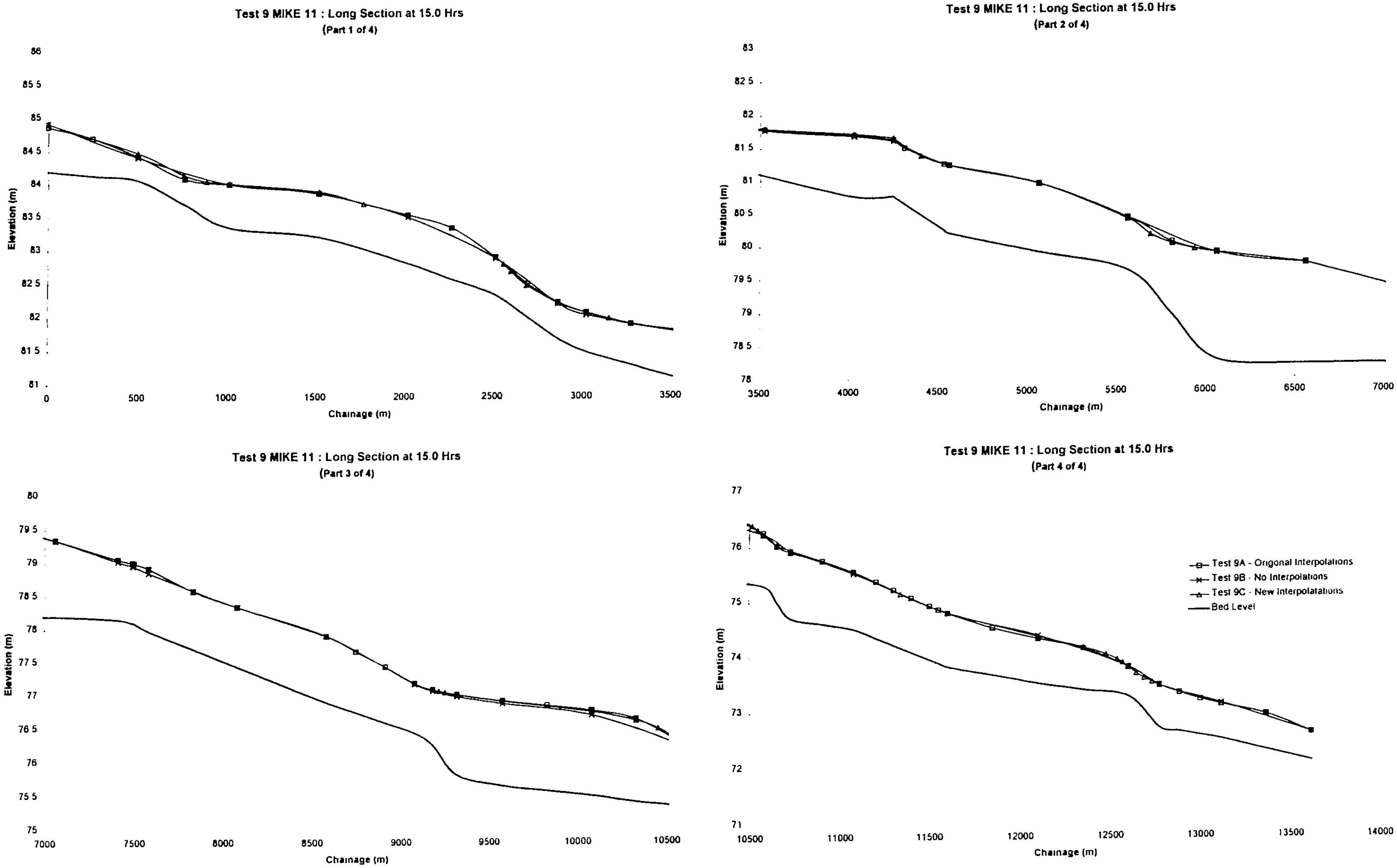
GRAPH 51 : *TEST 9 FLUCOMP - Comparison of water elevations at 35.0 Hrs*



GRAPH 52 : *TEST 9 FLUCOMP - Comparison of water elevations at 55.0 Hrs*



GRAPH 53 : *TEST 9 MIKE 11 - Comparison of water elevations at 0.0 Hrs*

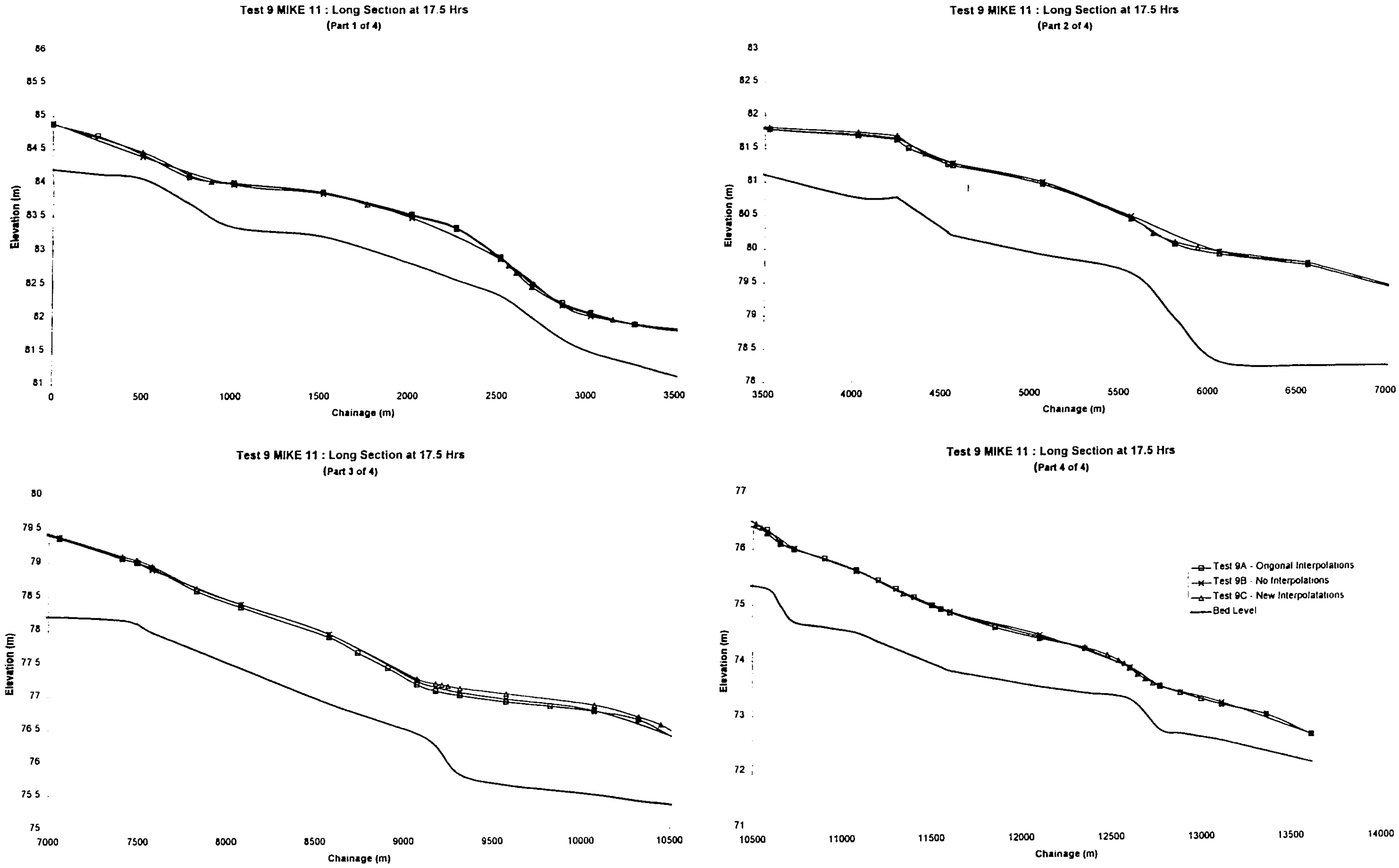

Test 9 MIKE 11 : Long Section at 15.0 Hrs
(Part 1 of 4)

Test 9 MIKE 11 : Long Section at 15.0 Hrs
(Part 2 of 4)

Test 9 MIKE 11 : Long Section at 15.0 Hrs
(Part 3 of 4)

Test 9 MIKE 11 : Long Section at 15.0 Hrs
(Part 4 of 4)

GRAPH 54 : *TEST 9 MIKE 11 - Comparison of water elevations at 15.0 Hrs*

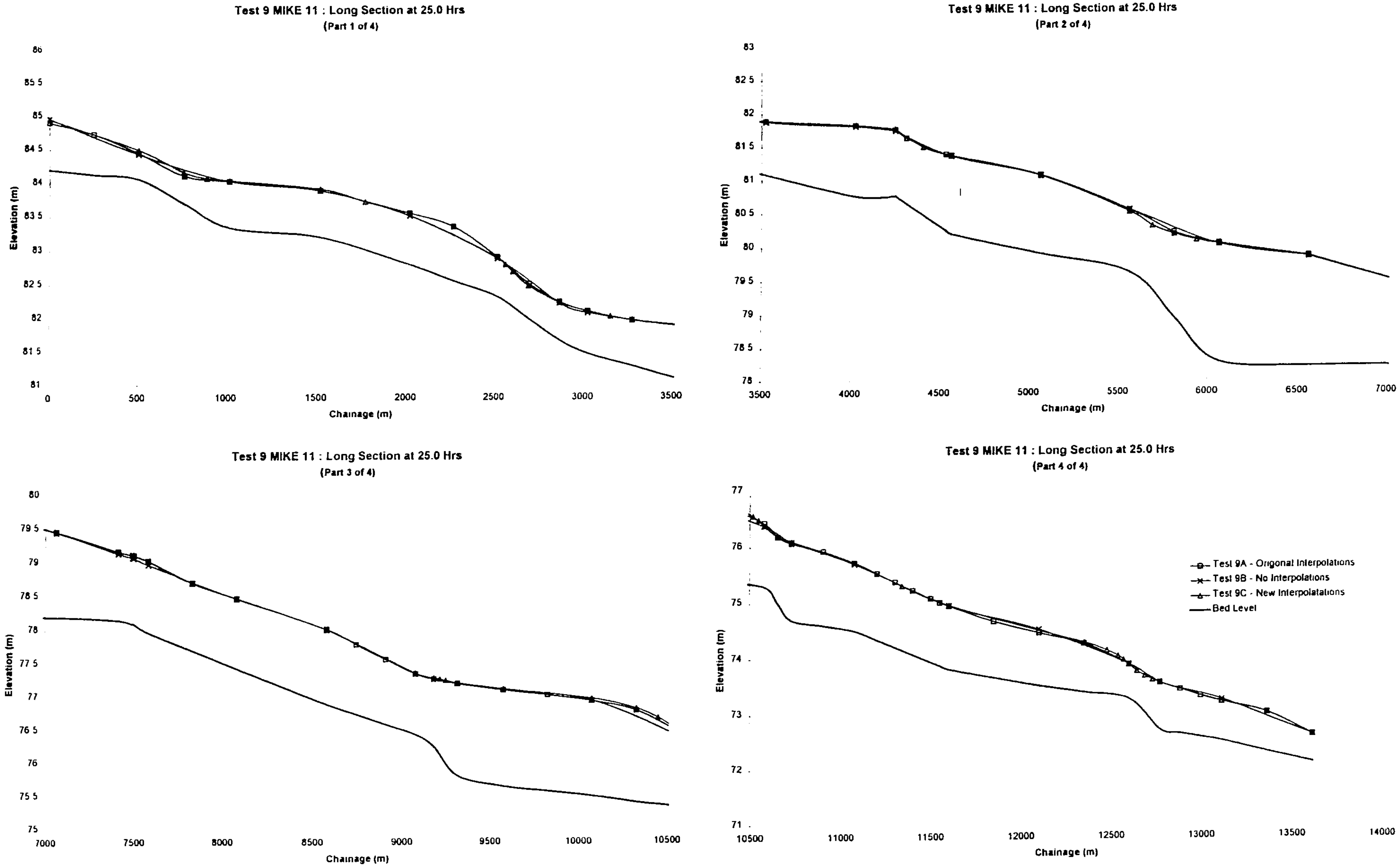

Test 9 MIKE 11 : Long Section at 17.5 Hrs (Part 1 of 4)

Test 9 MIKE 11 : Long Section at 17.5 Hrs (Part 2 of 4)

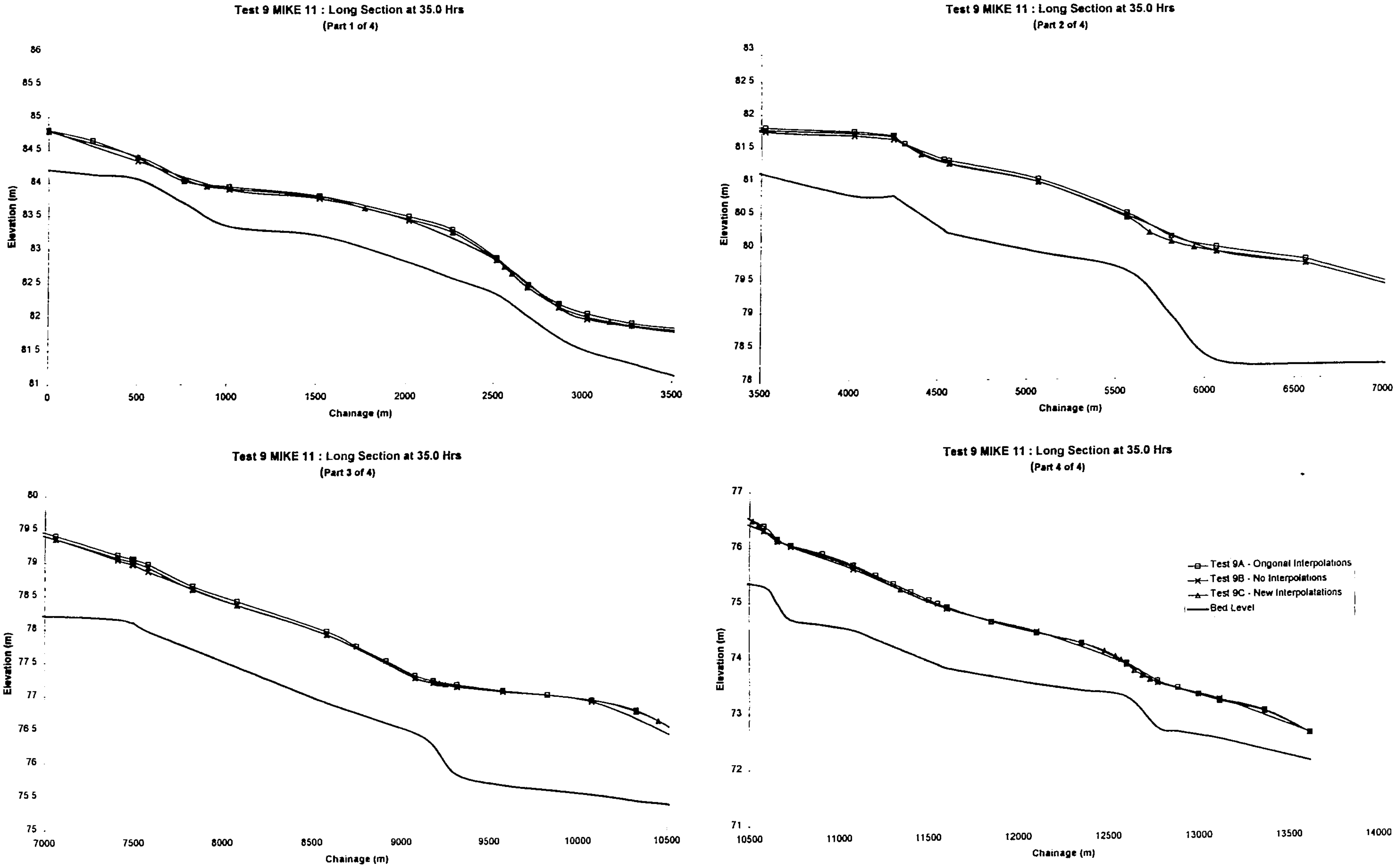
Test 9 MIKE 11 : Long Section at 17.5 Hrs (Part 3 of 4)

Test 9 MIKE 11 : Long Section at 17.5 Hrs (Part 4 of 4)

GRAPH 55 : *TEST 9 MIKE 11 - Comparison of water elevations at 17.5 Hrs*



GRAPH 56 : *TEST 9 MIKE 11 - Comparison of water elevations at 25.0 Hrs*



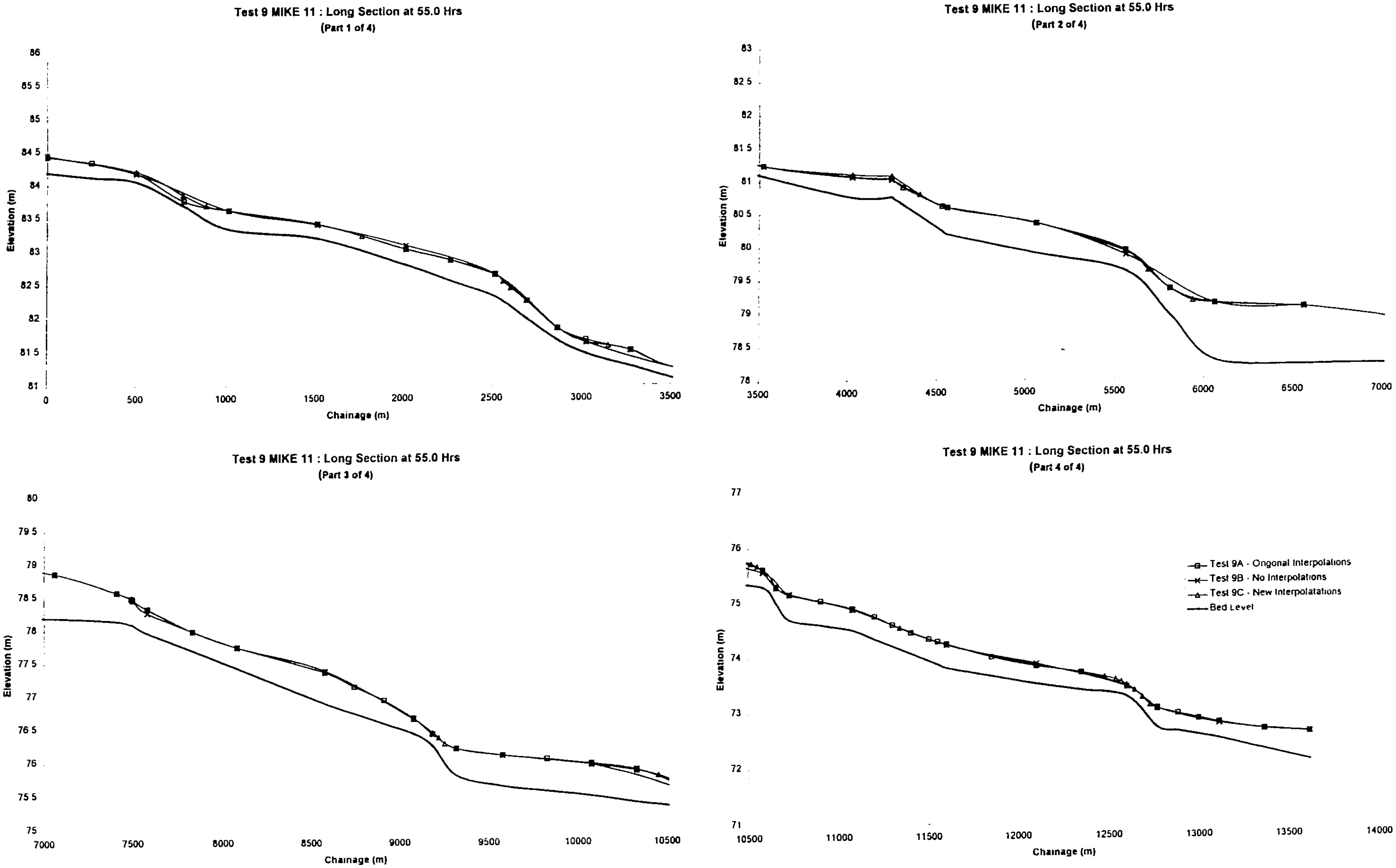
Test 9 MIKE 11 : Long Section at 35.0 Hrs (Part 1 of 4)

Test 9 MIKE 11 : Long Section at 35.0 Hrs (Part 2 of 4)

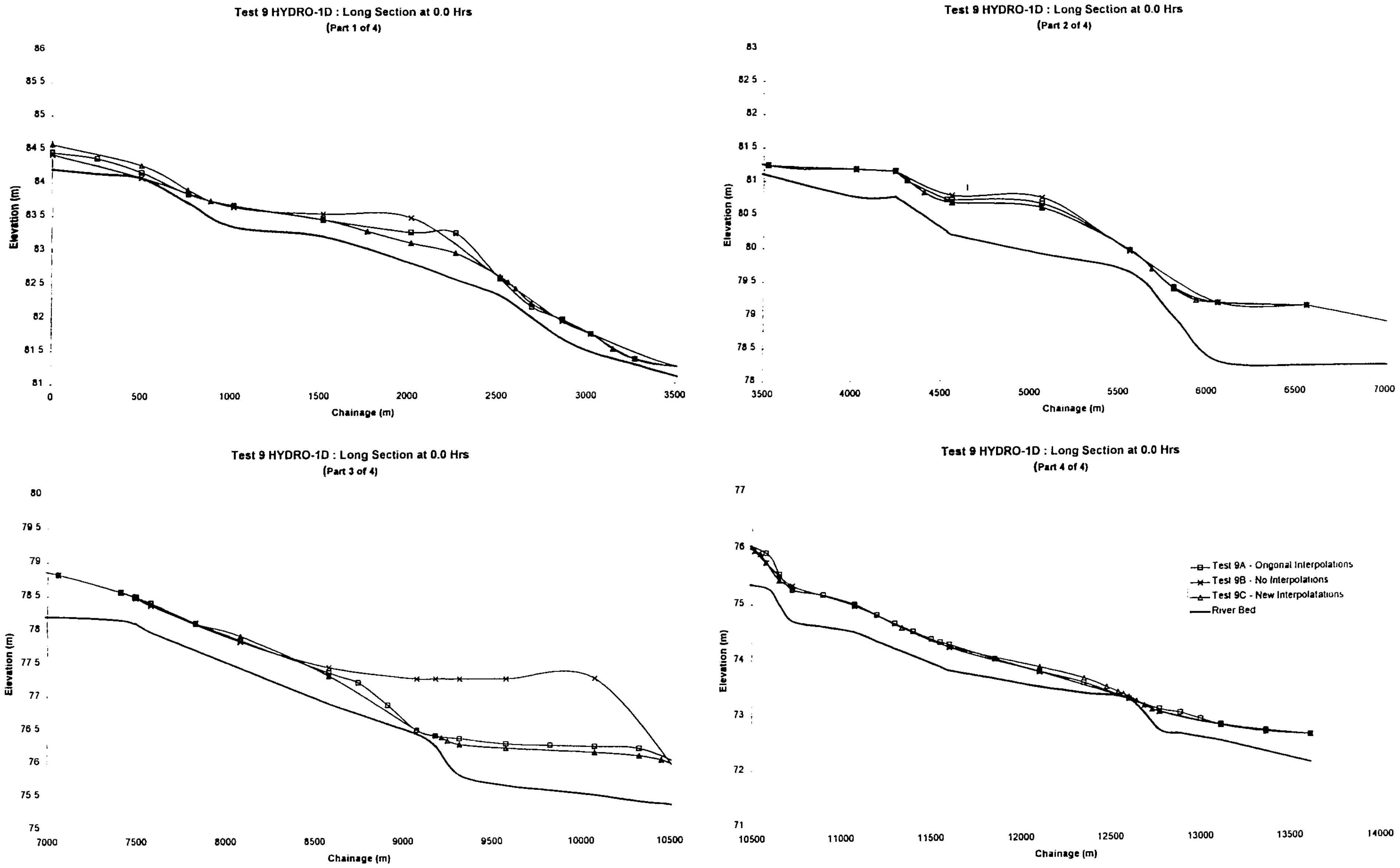
Test 9 MIKE 11 : Long Section at 35.0 Hrs (Part 3 of 4)

Test 9 MIKE 11 : Long Section at 35.0 Hrs (Part 4 of 4)

GRAPH 57 : *TEST 9 MIKE 11 - Comparison of water elevations at 35.0 Hrs*



GRAPH 58 : *TEST 9 MIKE 11 - Comparison of water elevations at 55.0 Hrs*



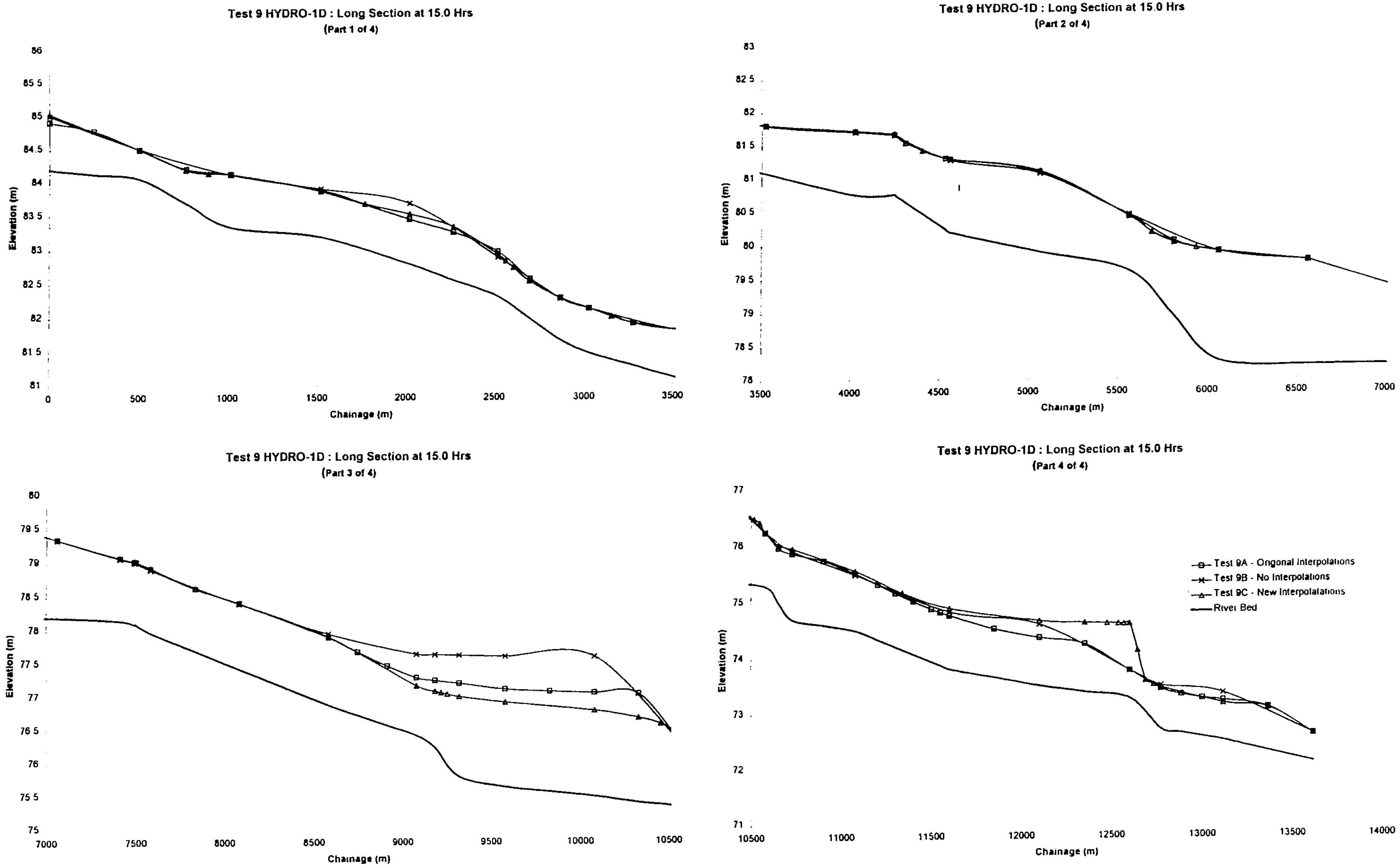
Test 9 HYDRO-1D : Long Section at 0.0 Hrs (Part 1 of 4)

Test 9 HYDRO-1D : Long Section at 0.0 Hrs (Part 2 of 4)

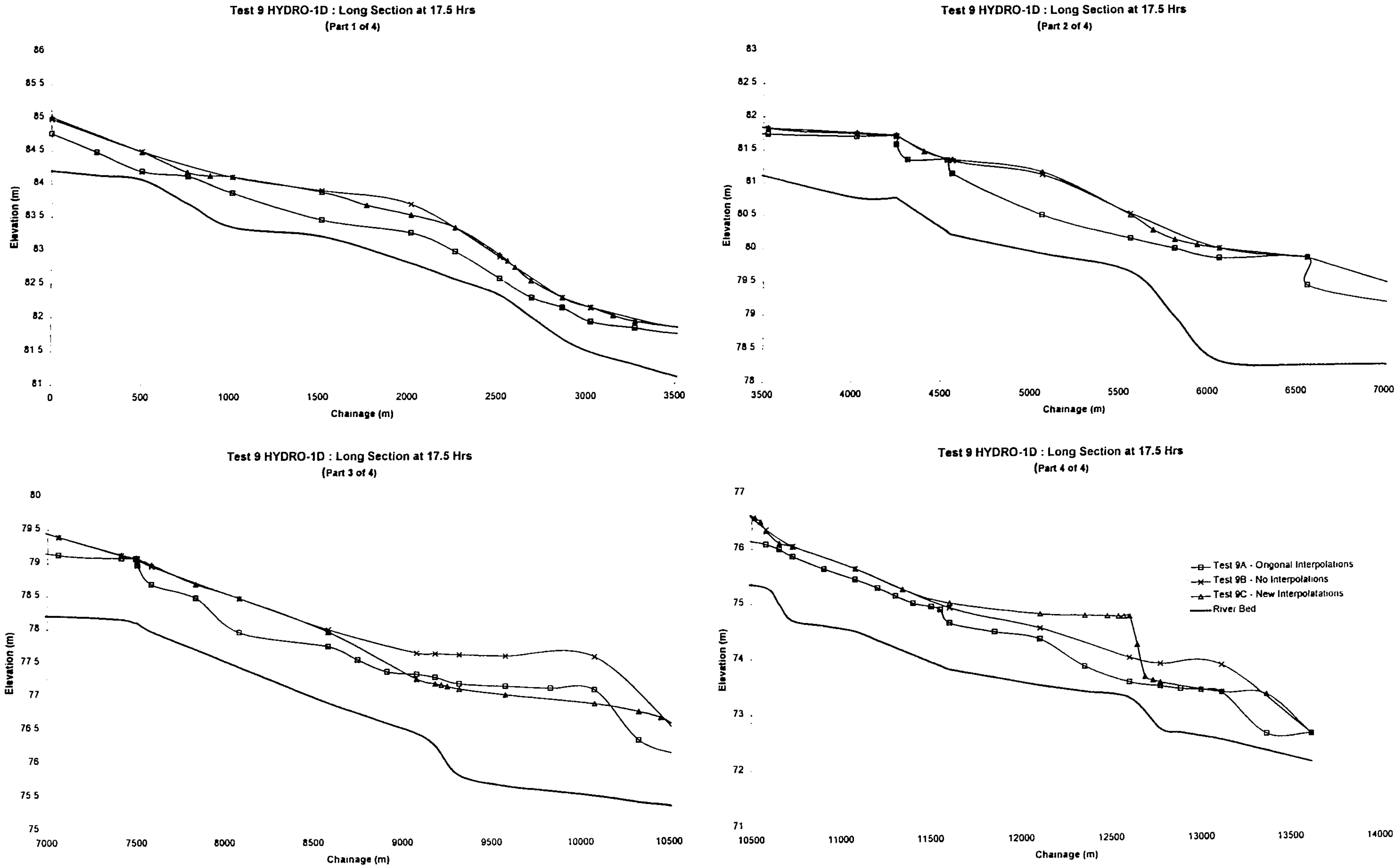
Test 9 HYDRO-1D : Long Section at 0.0 Hrs (Part 3 of 4)

Test 9 HYDRO-1D : Long Section at 0.0 Hrs (Part 4 of 4)

GRAPH 59 : *TEST 9 HYDRO-1D - Comparison of water elevations at 0.0 Hrs*



GRAPH 60 : *TEST 9 HYDRO-1D - Comparison of water elevations at 15.0 Hrs*



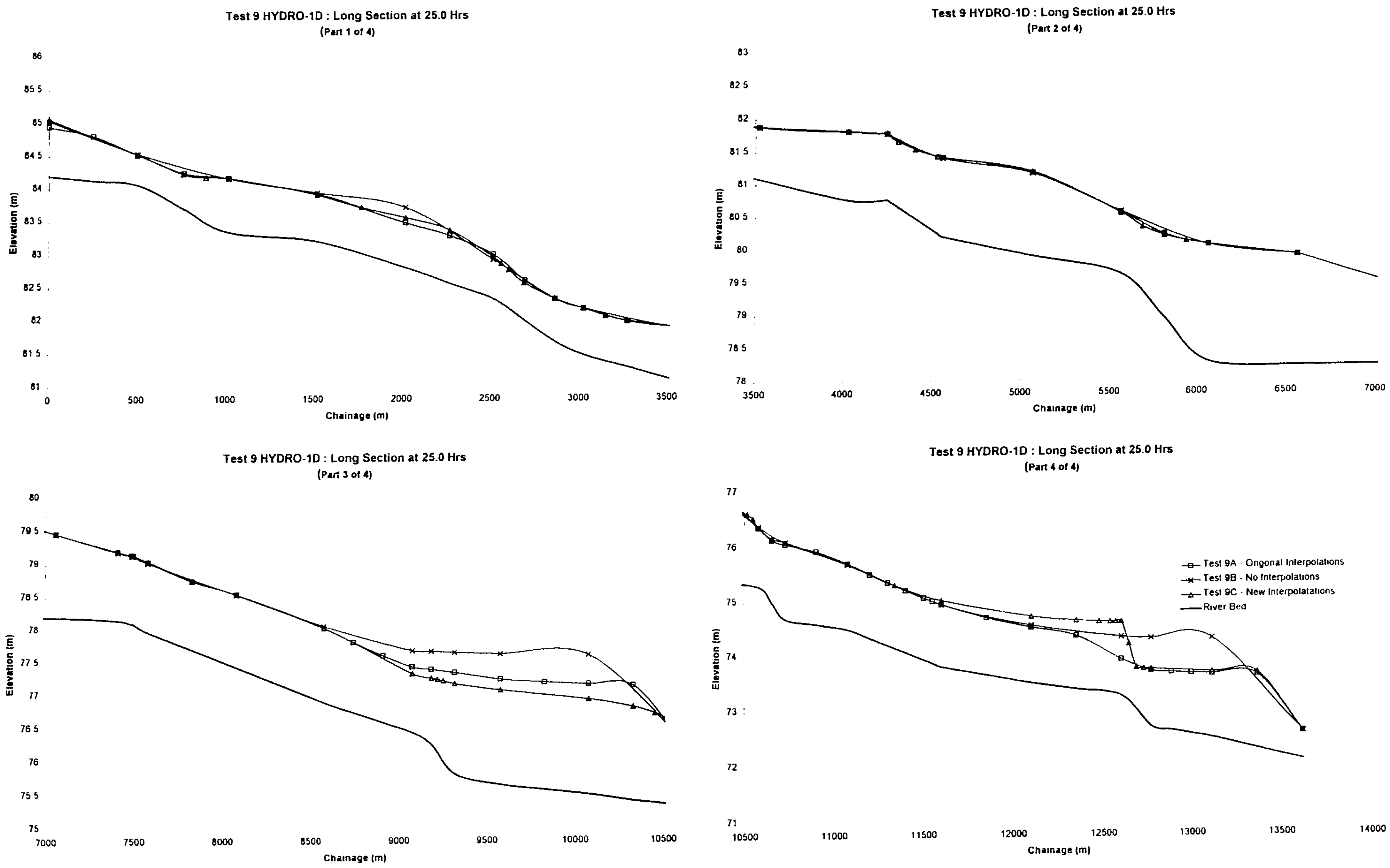
Test 9 HYDRO-1D : Long Section at 17.5 Hrs (Part 1 of 4)

Test 9 HYDRO-1D : Long Section at 17.5 Hrs (Part 2 of 4)

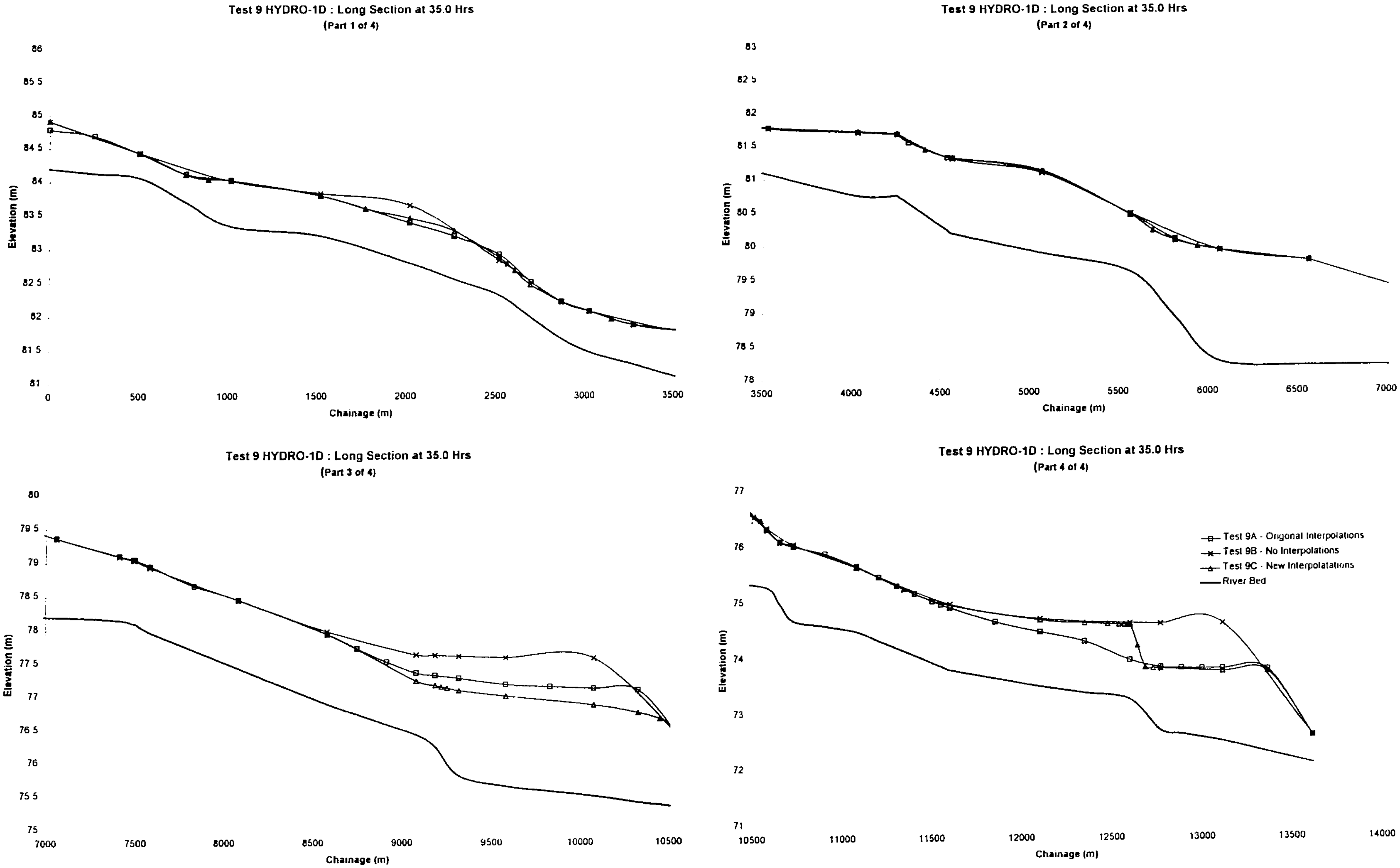
Test 9 HYDRO-1D : Long Section at 17.5 Hrs (Part 3 of 4)

Test 9 HYDRO-1D : Long Section at 17.5 Hrs (Part 4 of 4)

GRAPH 61 : *TEST 9 HYDRO-1D - Comparison of water elevations at 17.5 Hrs*



GRAPH 62 : *TEST 9 HYDRO-1D - Comparison of water elevations at 25.0 Hrs*


Test 9 HYDRO-1D : Long Section at 35.0 Hrs (Part 1 of 4)

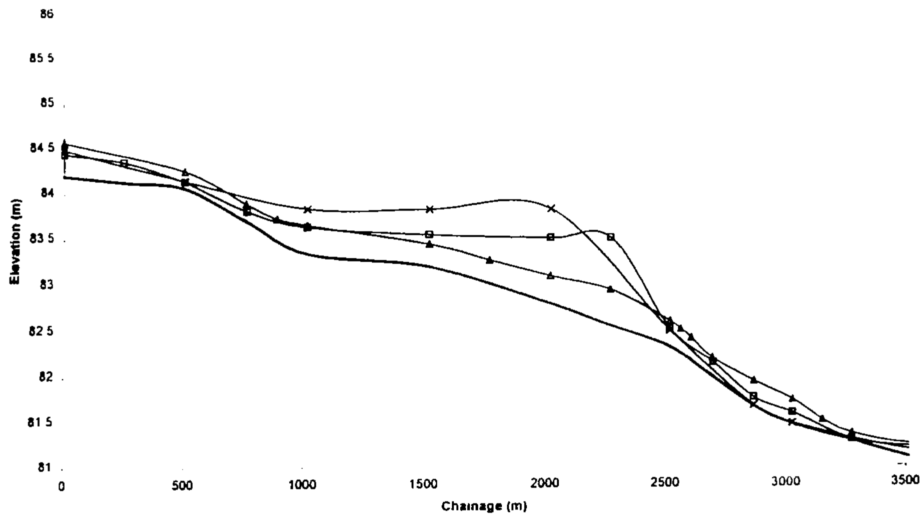
Test 9 HYDRO-1D : Long Section at 35.0 Hrs (Part 2 of 4)

Test 9 HYDRO-1D : Long Section at 35.0 Hrs (Part 3 of 4)

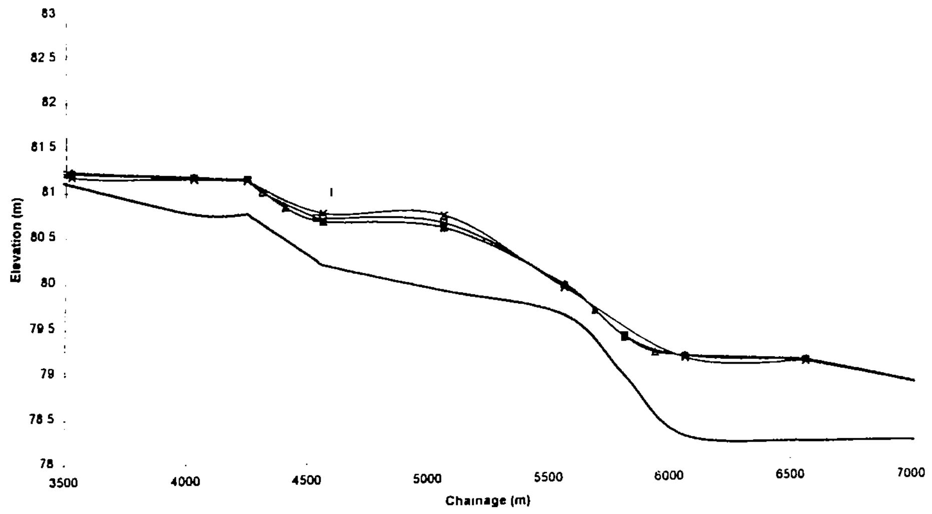
Test 9 HYDRO-1D : Long Section at 35.0 Hrs (Part 4 of 4)

GRAPH 63 : *TEST 9 HYDRO-1D - Comparison of water elevations at 35.0 Hrs*

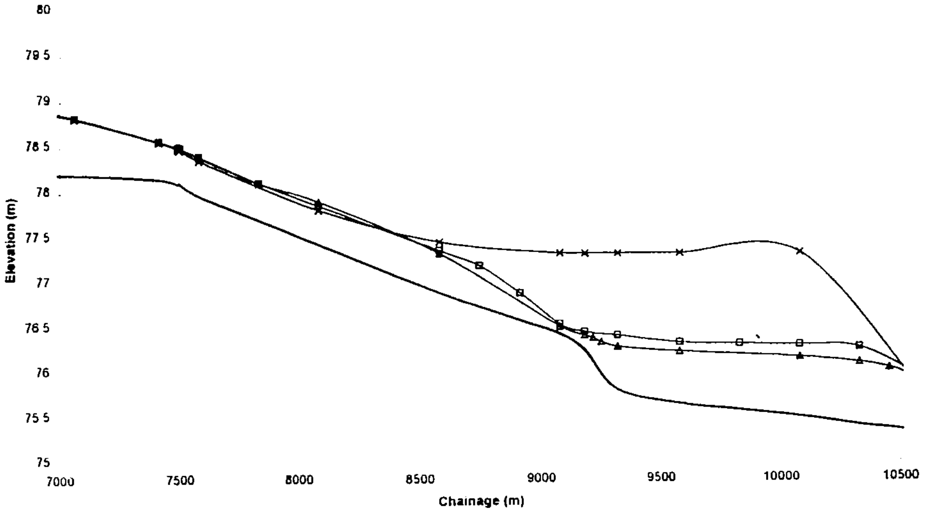
Test 9 HYDRO-1D : Long Section at 55.0 Hrs
(Part 1 of 4)



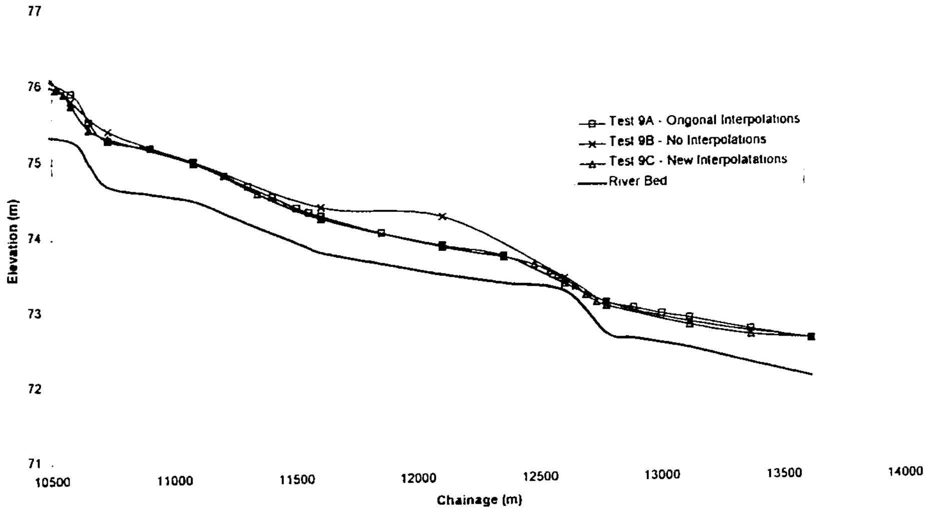
Test 9 HYDRO-1D : Long Section at 55.0 Hrs
(Part 2 of 4)



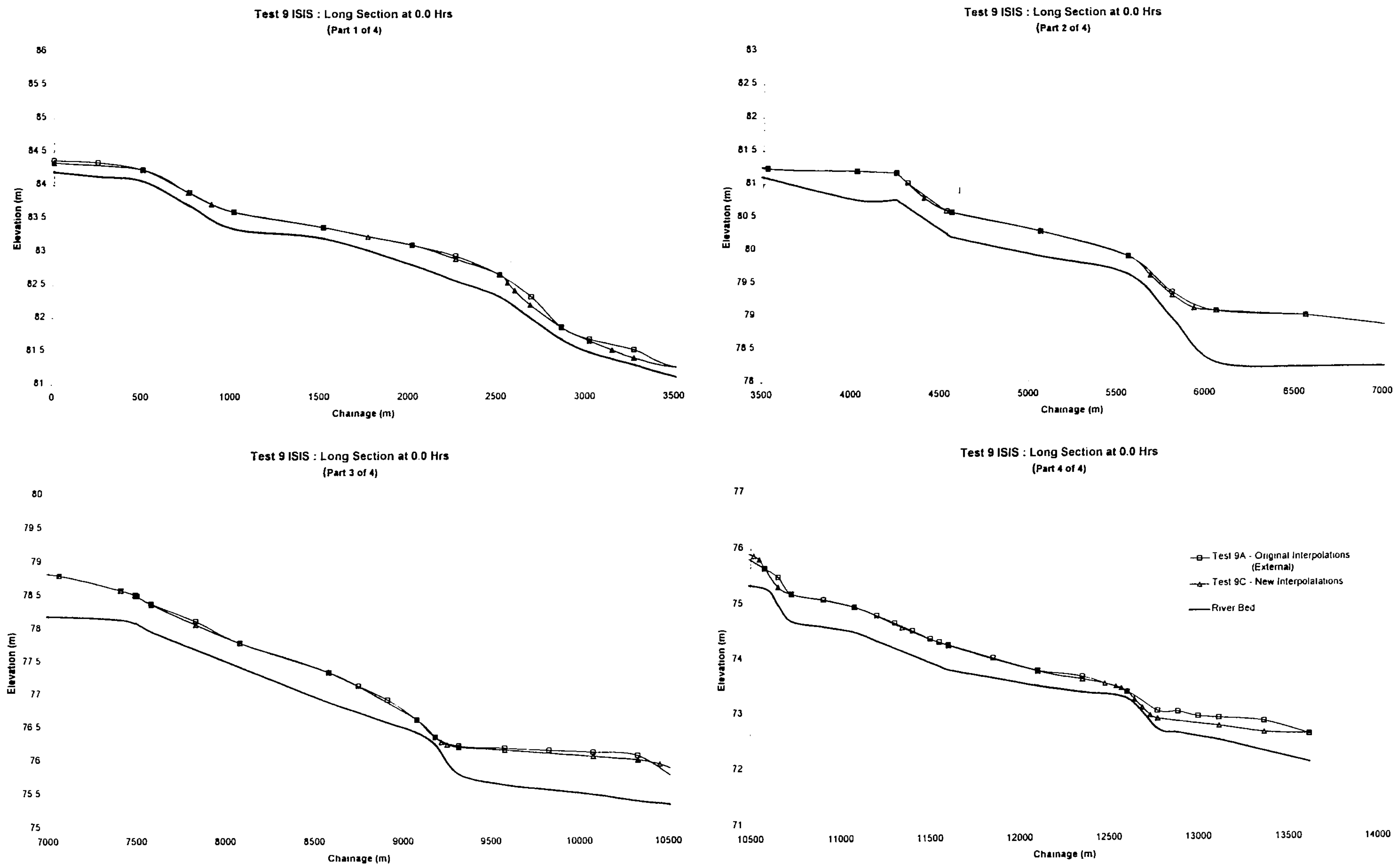
Test 9 HYDRO-1D : Long Section at 55.0 Hrs
(Part 3 of 4)



Test 9 HYDRO-1D : Long Section at 55.0 Hrs
(Part 4 of 4)



GRAPH 64 : *TEST 9 HYDRO-1D - Comparison of water elevations at 55.0 Hrs*



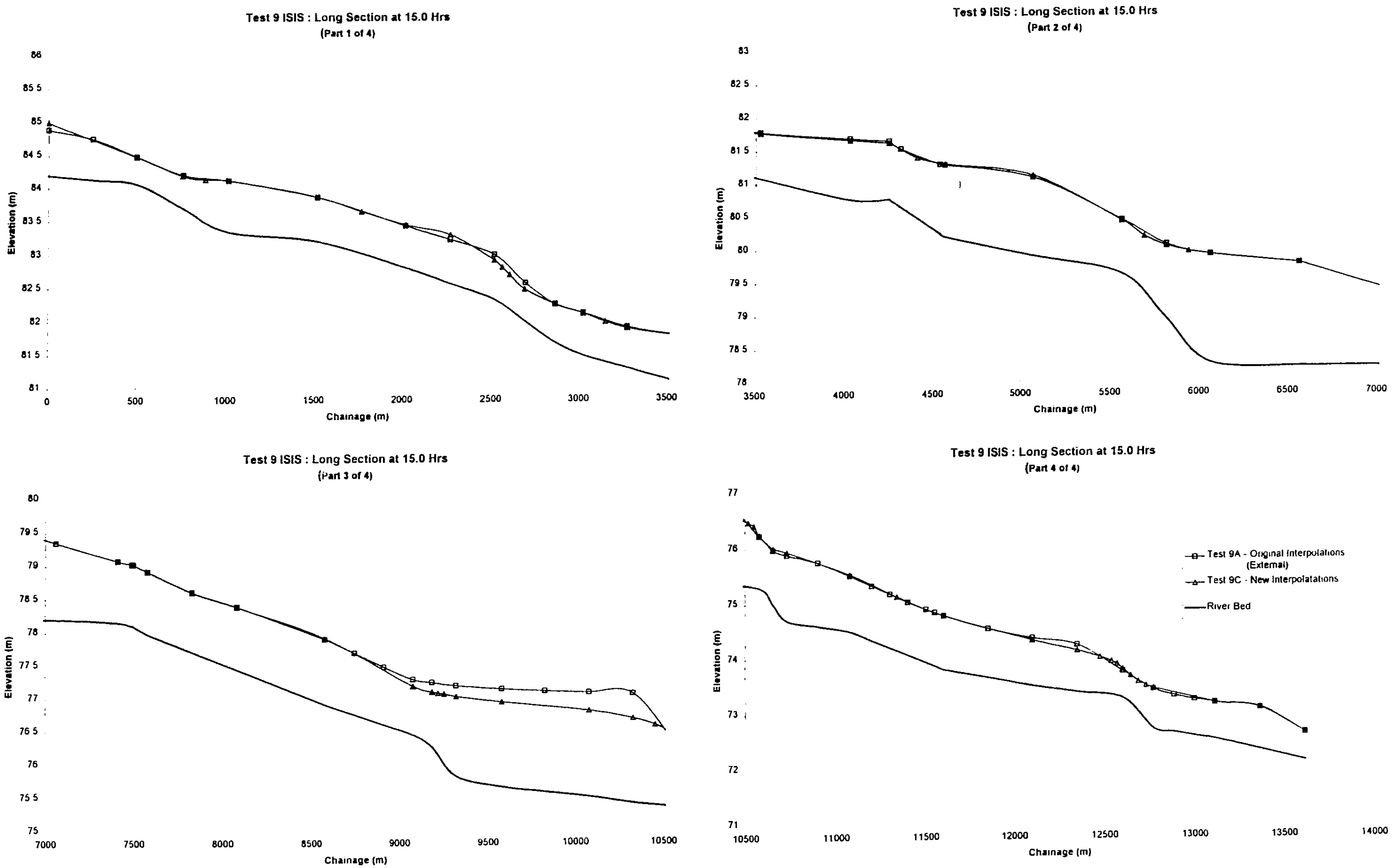
Test 9 ISIS : Long Section at 0.0 Hrs
(Part 1 of 4)

Test 9 ISIS : Long Section at 0.0 Hrs
(Part 2 of 4)

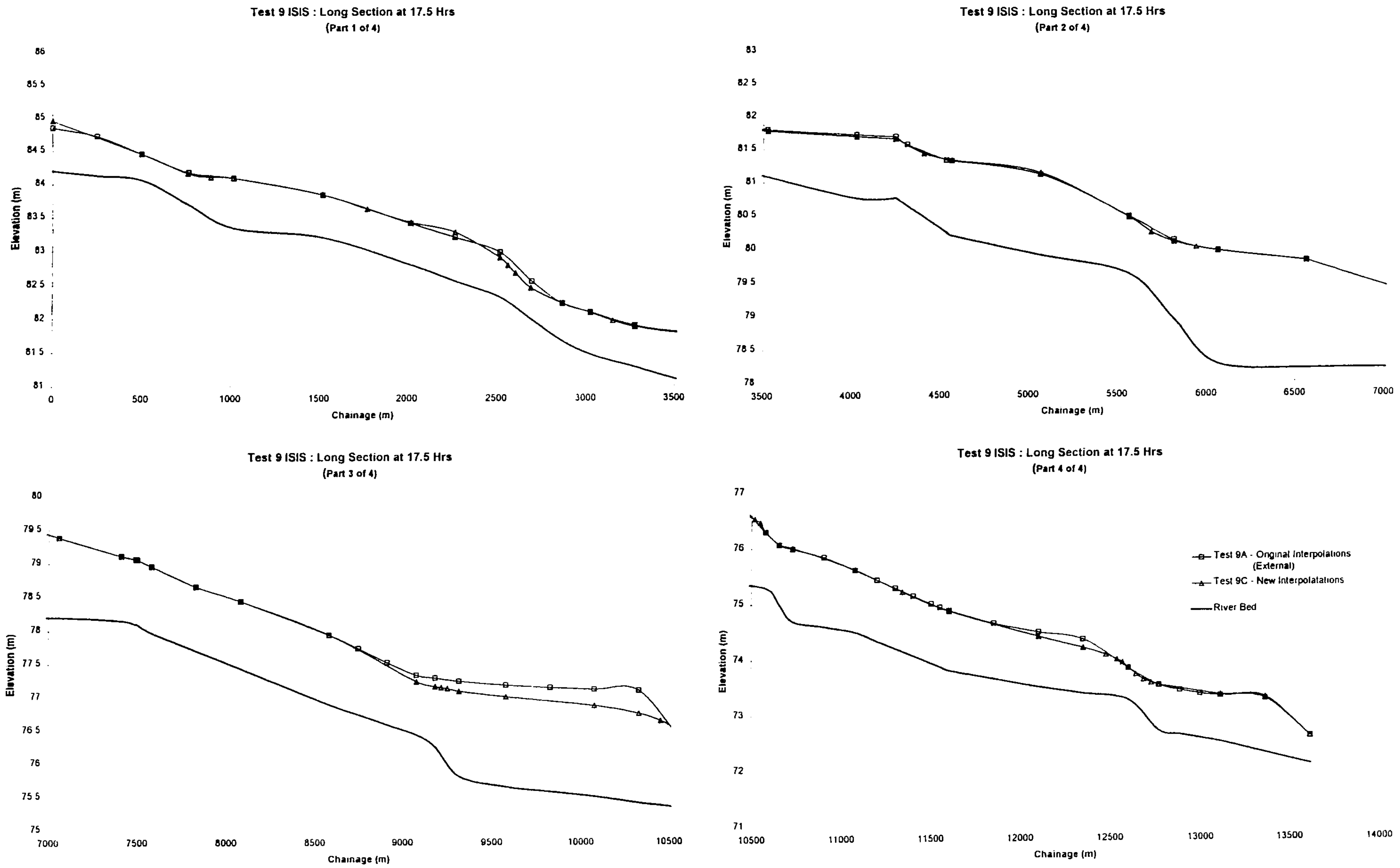
Test 9 ISIS : Long Section at 0.0 Hrs
(Part 3 of 4)

Test 9 ISIS : Long Section at 0.0 Hrs
(Part 4 of 4)

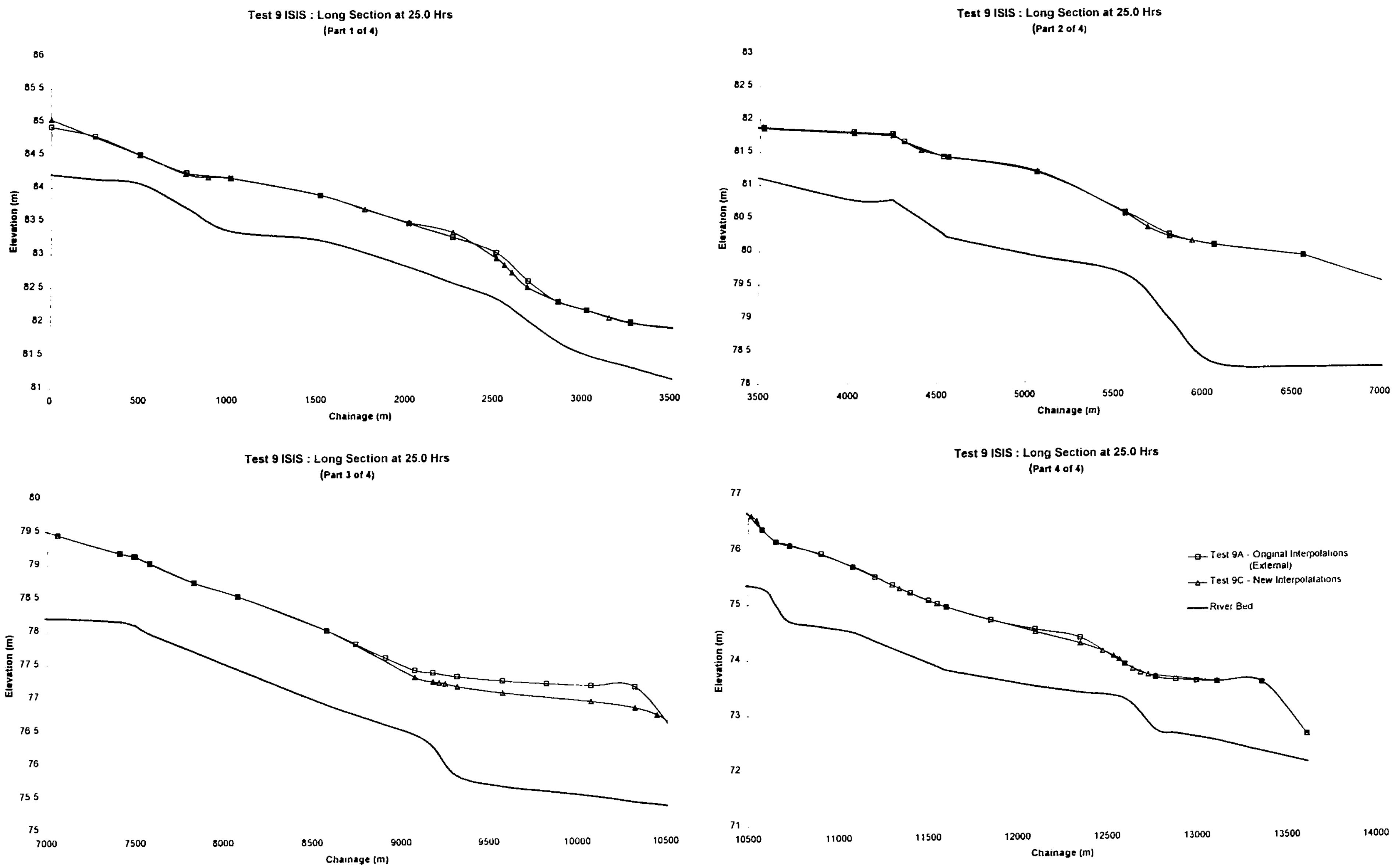
GRAPH 65 : *TEST 9 ISIS - Comparison of water elevations at 0.0 Hrs*



GRAPH 66 : *TEST 9 ISIS - Comparison of water elevations at 15.0 Hrs*



GRAPH 67 : *TEST 9 ISIS - Comparison of water elevations at 17.5 Hrs*



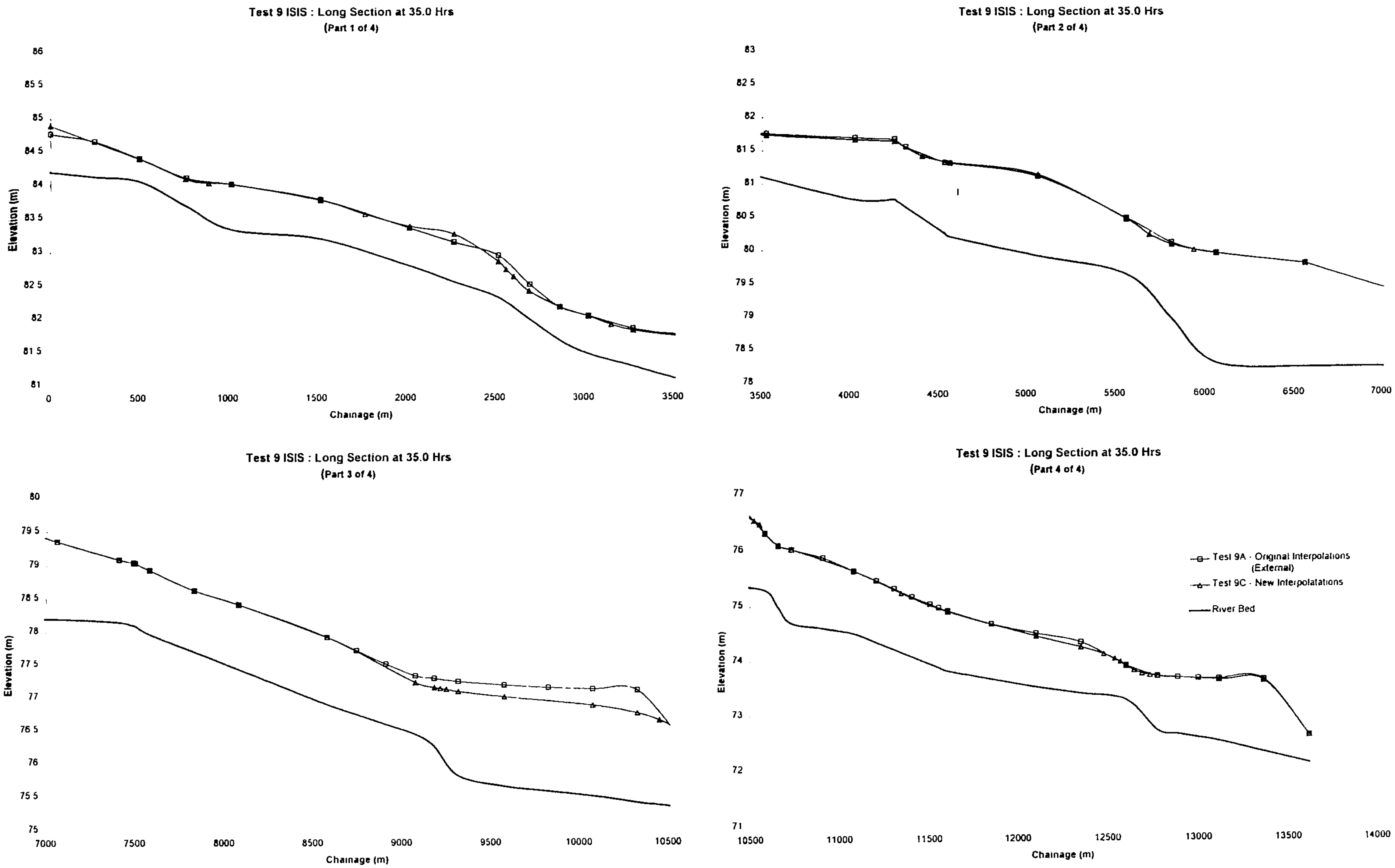
Test 9 ISIS : Long Section at 25.0 Hrs (Part 1 of 4)

Test 9 ISIS : Long Section at 25.0 Hrs (Part 2 of 4)

Test 9 ISIS : Long Section at 25.0 Hrs (Part 3 of 4)

Test 9 ISIS : Long Section at 25.0 Hrs (Part 4 of 4)

GRAPH 68 : *TEST 9 ISIS - Comparison of water elevations at 25.0 Hrs*



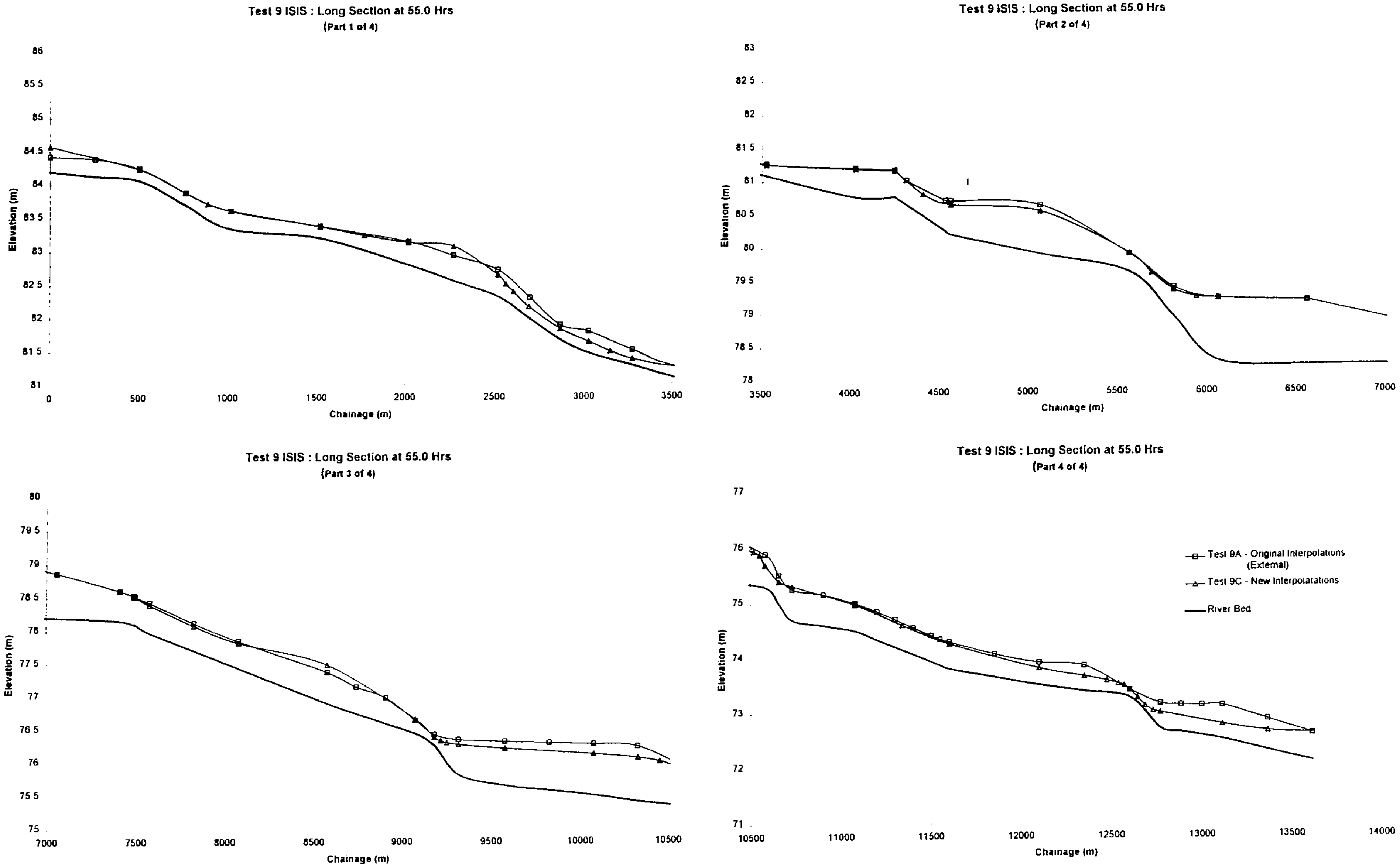
Test 9 ISIS : Long Section at 35.0 Hrs
(Part 1 of 4)

Test 9 ISIS : Long Section at 35.0 Hrs
(Part 2 of 4)

Test 9 ISIS : Long Section at 35.0 Hrs
(Part 3 of 4)

Test 9 ISIS : Long Section at 35.0 Hrs
(Part 4 of 4)

GRAPH 69 : *TEST 9 ISIS - Comparison of water elevations at 35.0 Hrs*



Test 9 ISIS : Long Section at 55.0 Hrs
(Part 1 of 4)

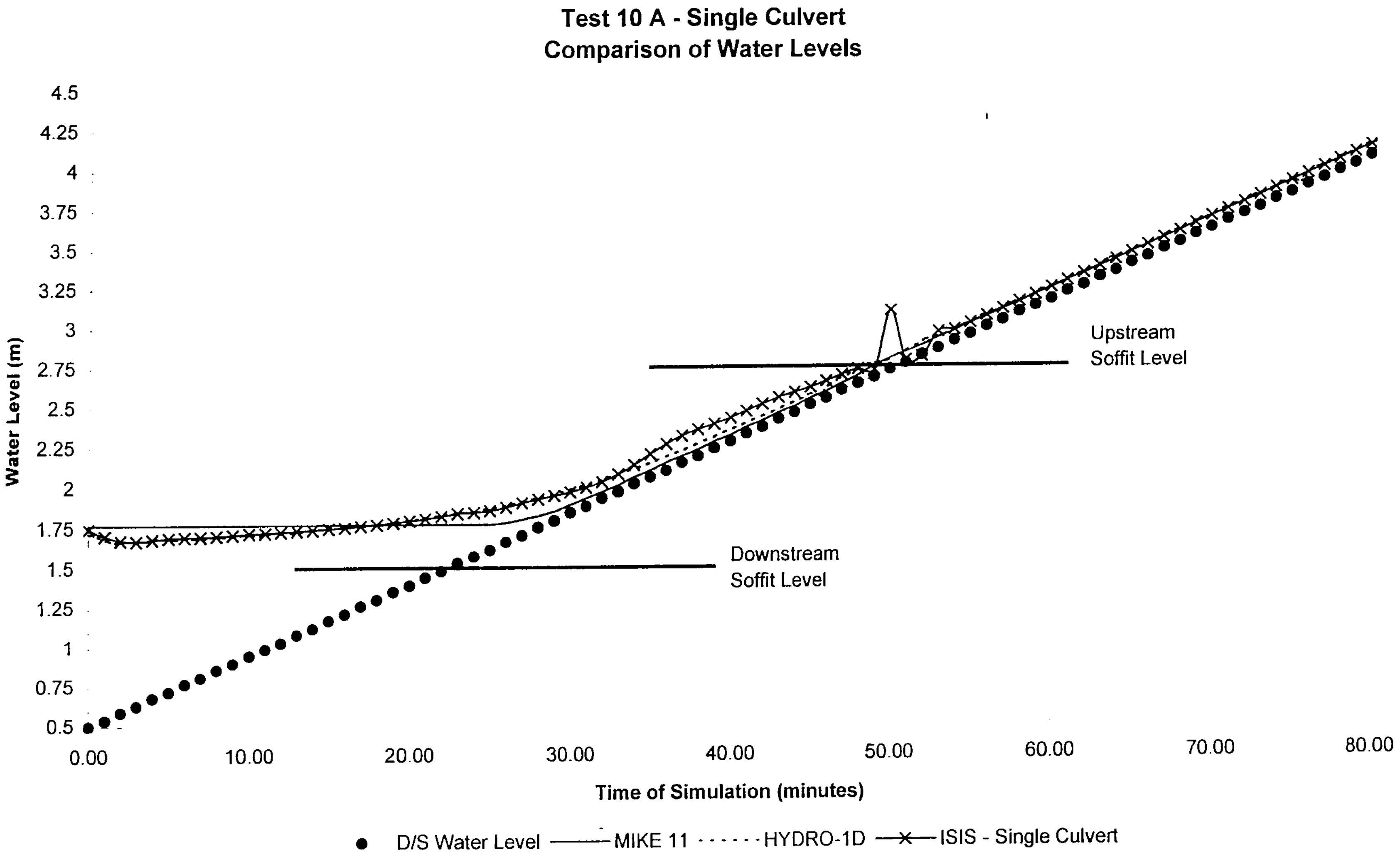
Test 9 ISIS : Long Section at 55.0 Hrs
(Part 2 of 4)

Test 9 ISIS : Long Section at 55.0 Hrs
(Part 3 of 4)

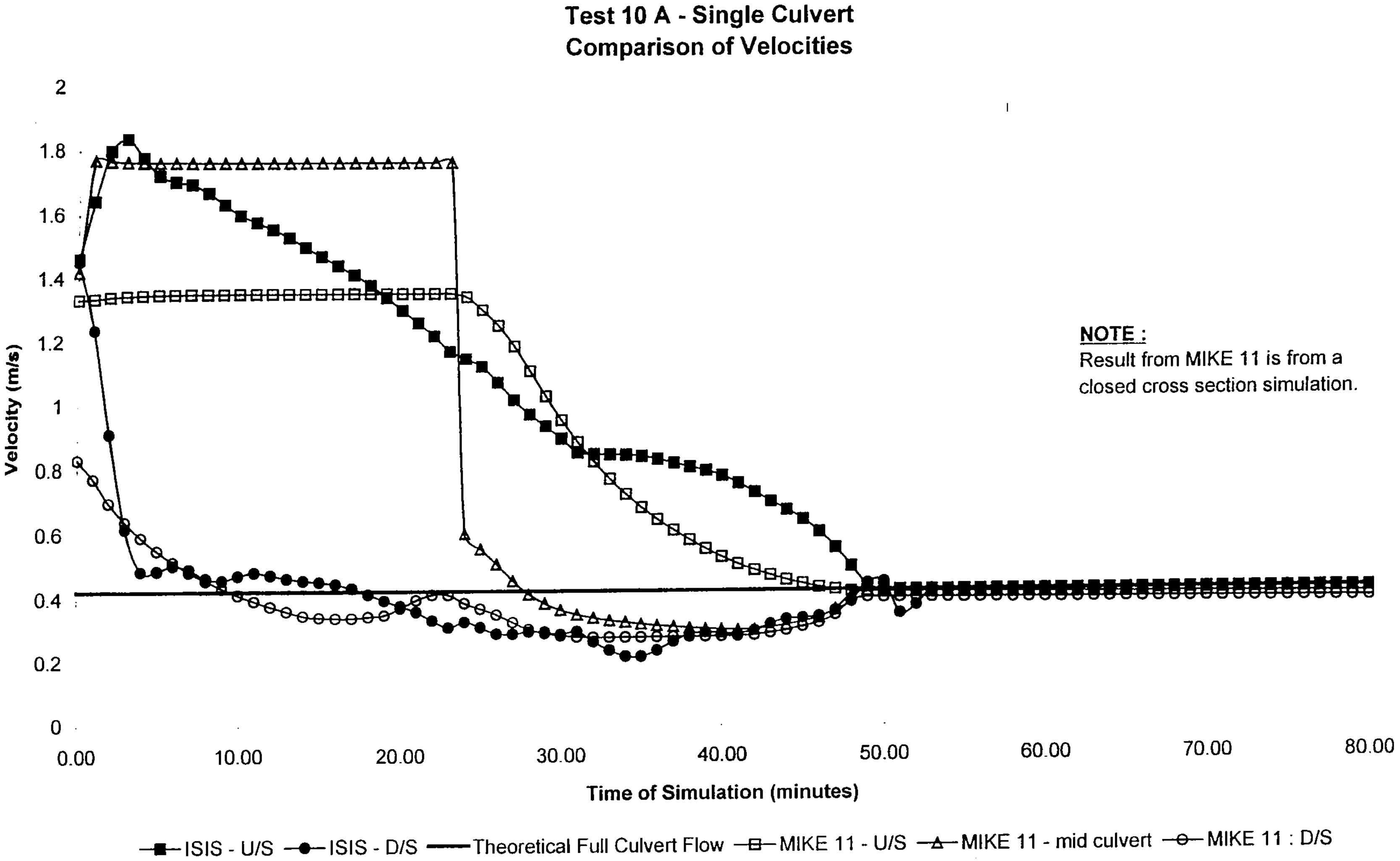
Test 9 ISIS : Long Section at 55.0 Hrs
(Part 4 of 4)

GRAPH 70 : *TEST 9 ISIS - Comparison of water elevations at 55.0 Hrs*

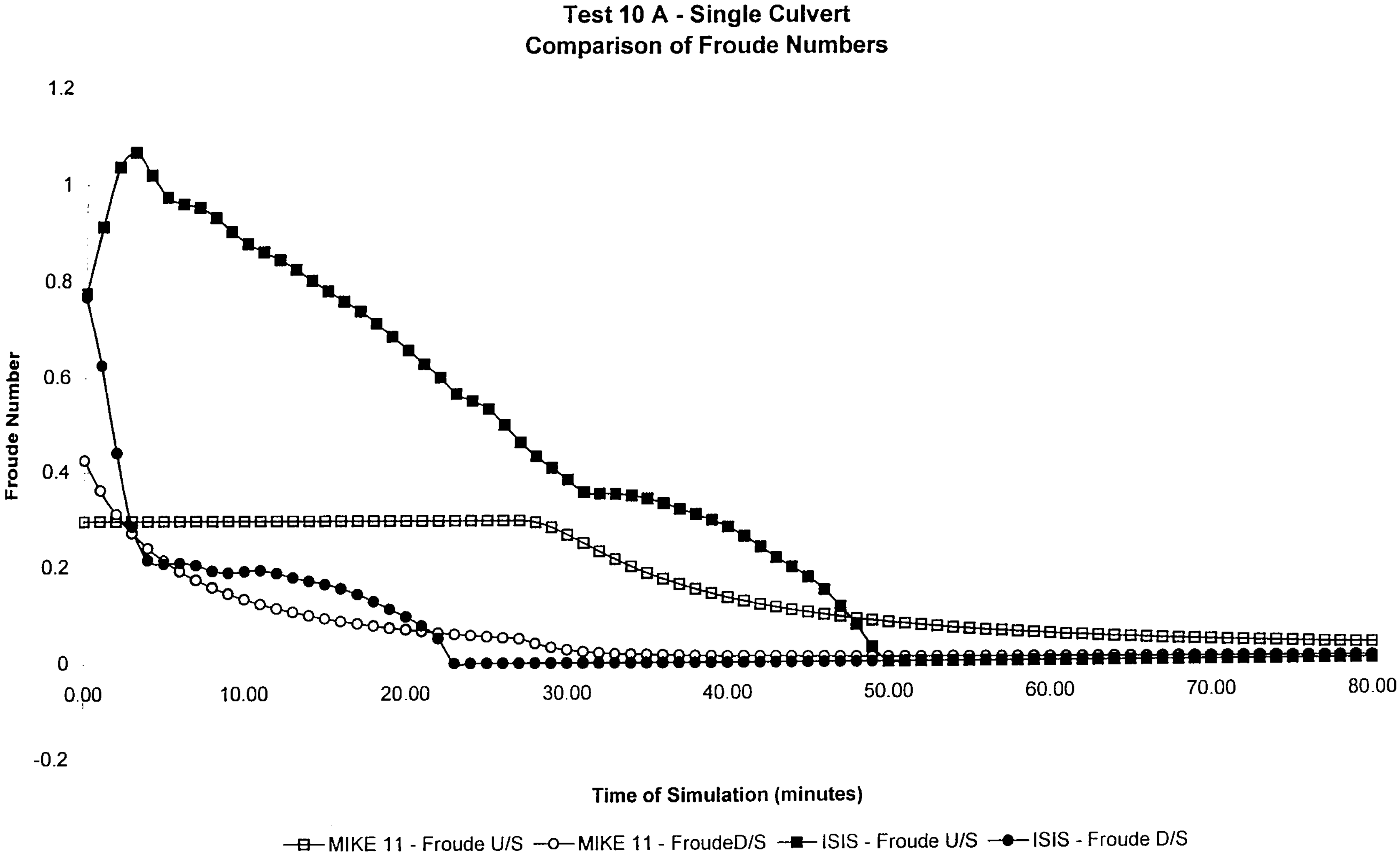
Appendix H



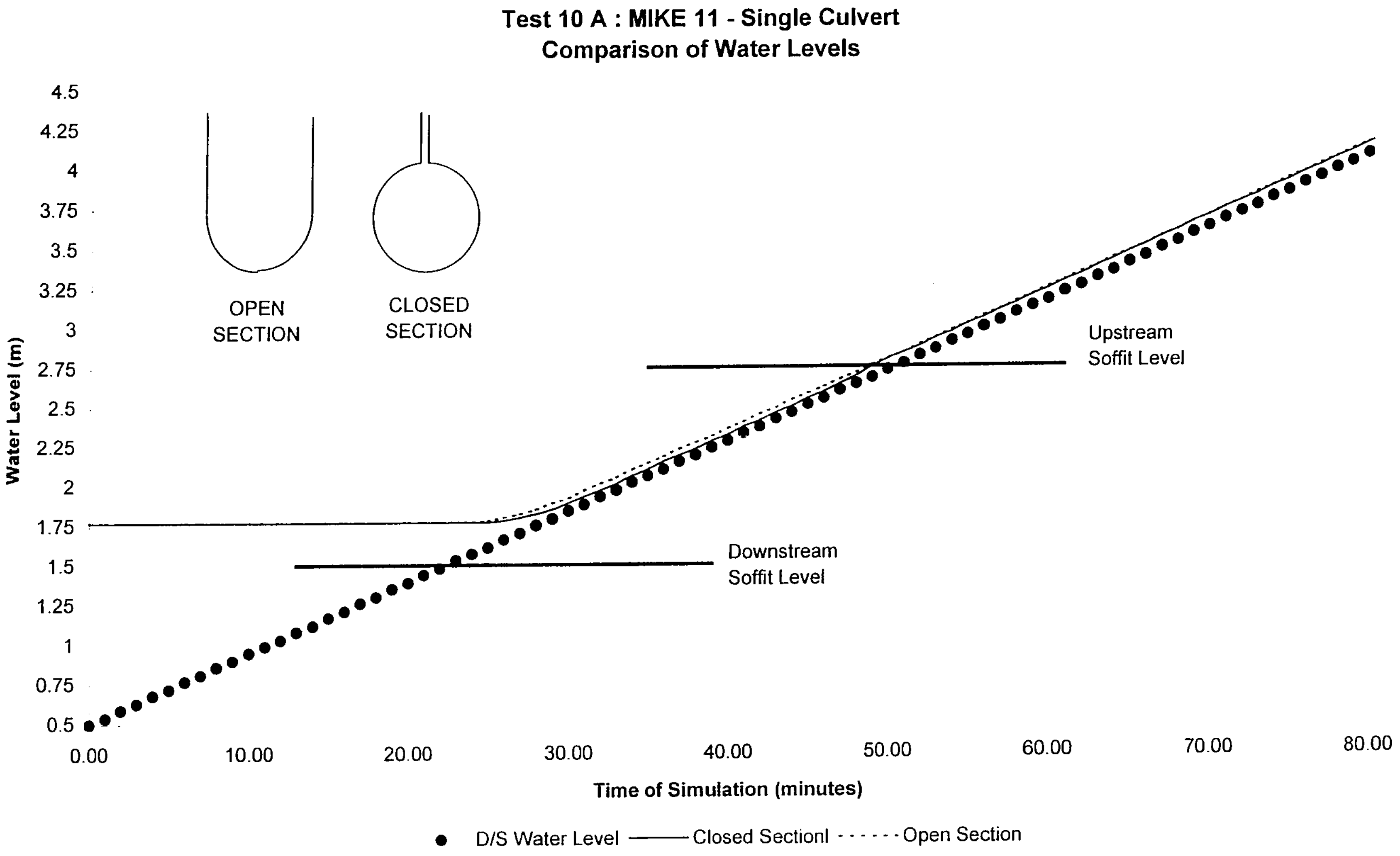
GRAPH 1 : *TEST 10A - Comparison of calculated upstream boundary water elevations throughout simulation.*



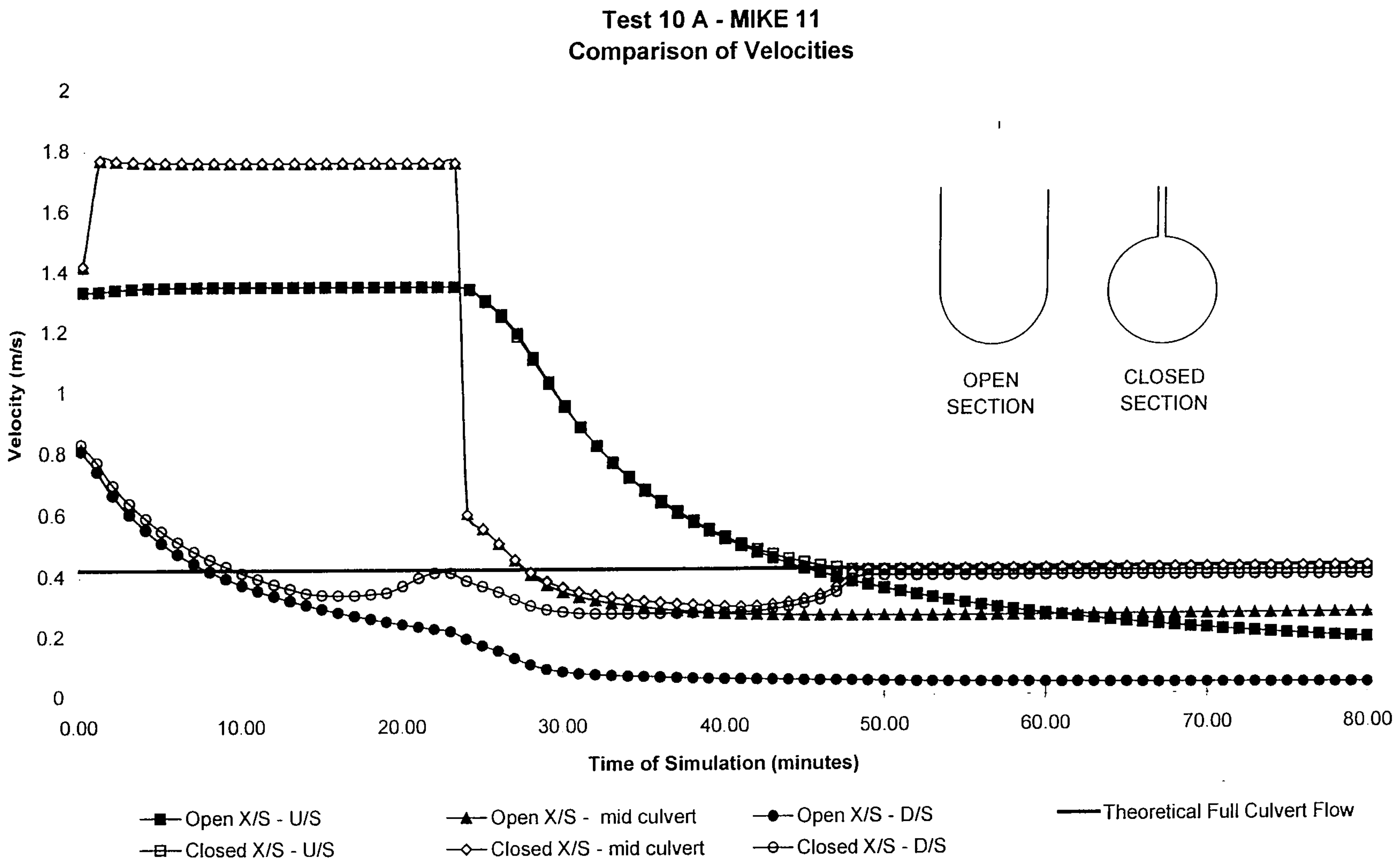
GRAPH 2 : *TEST 10A - Comparison of calculated upstream and downstream boundary velocities throughout simulation.*



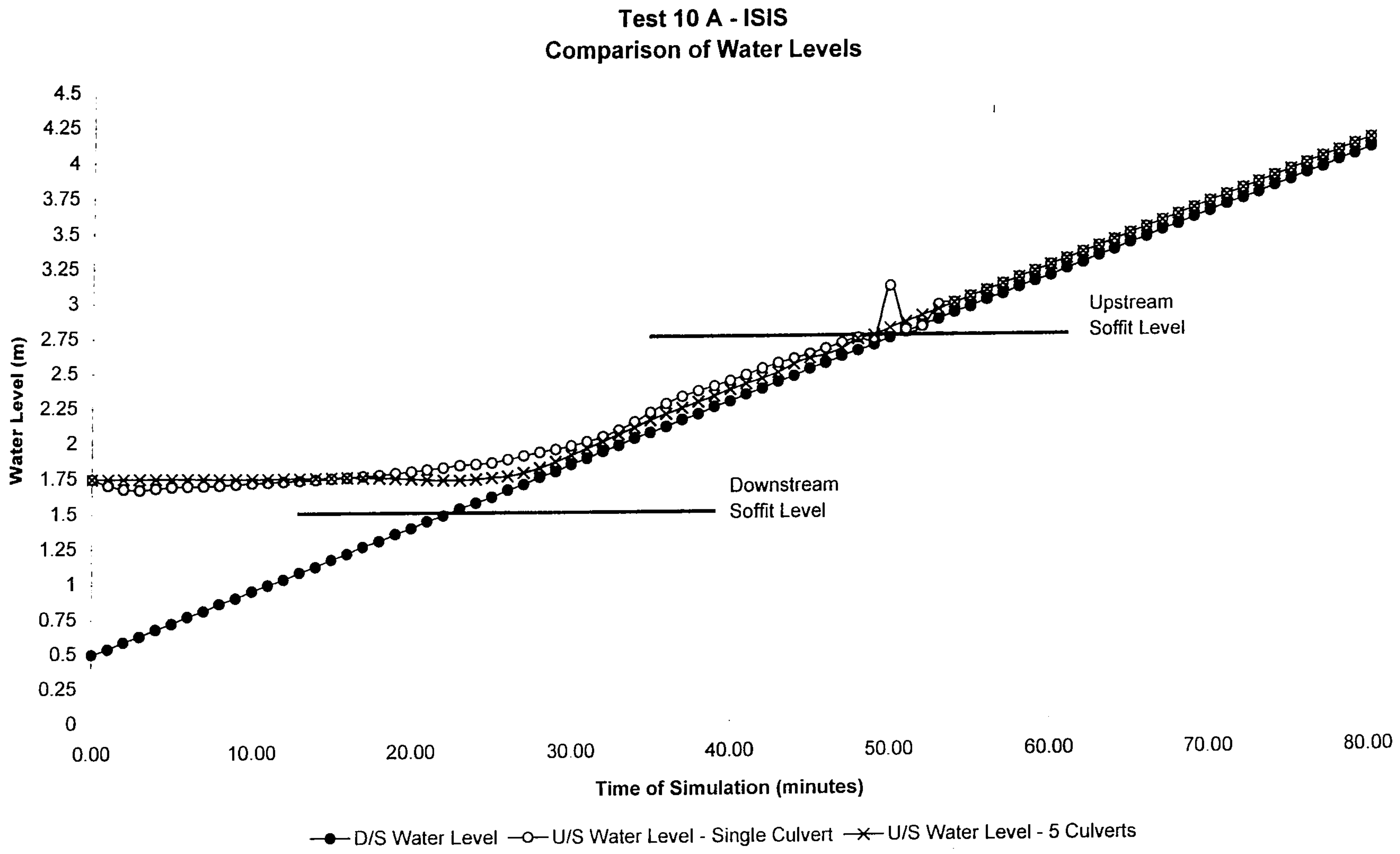
GRAPH 3 : *TEST 10A - Comparison of calculated upstream and downstream boundary Froude numbers throughout simulation.*



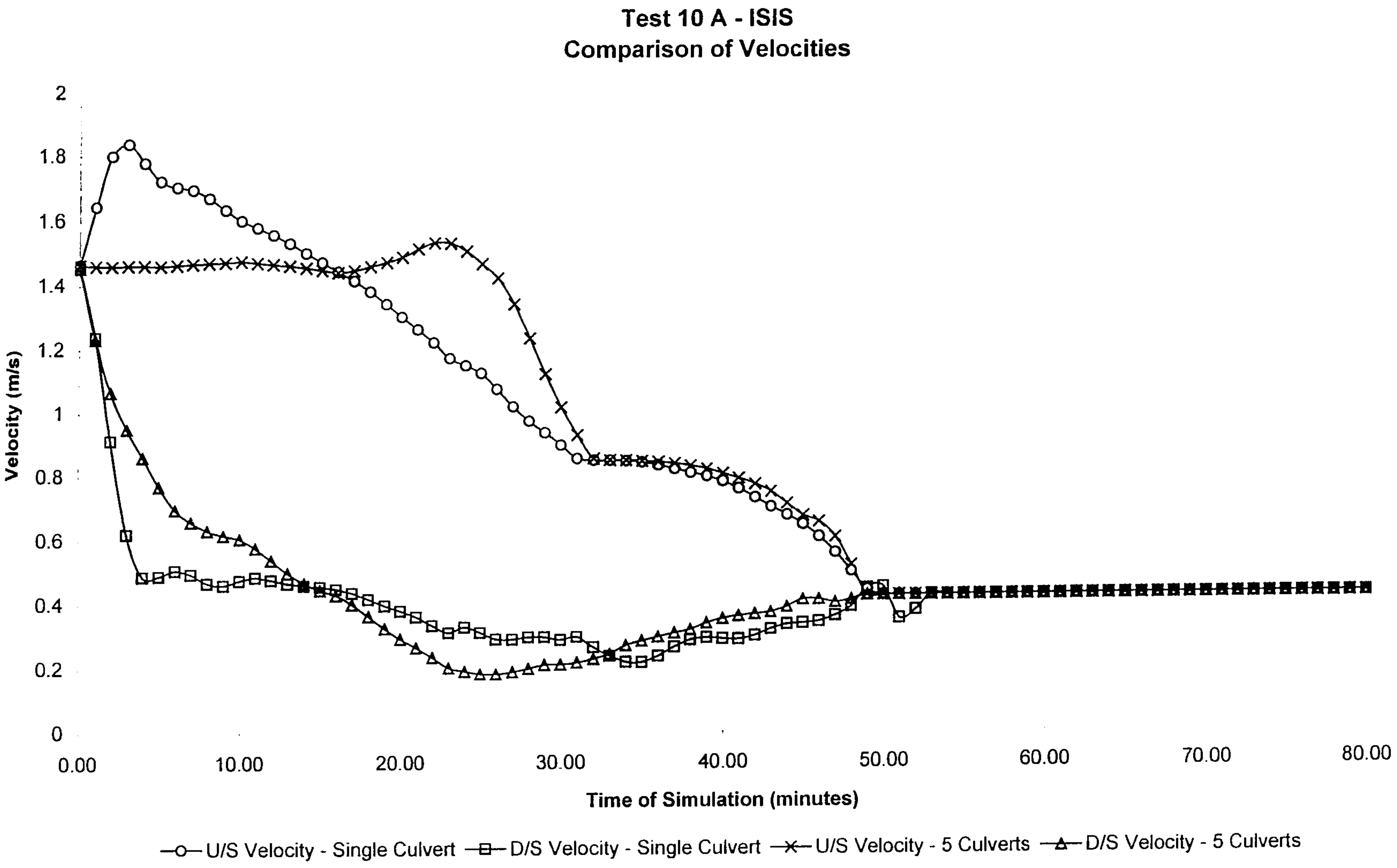
GRAPH 4 : *TEST 10A MIKE 11 - Comparison of boundary water elevations using open and closed sections.*



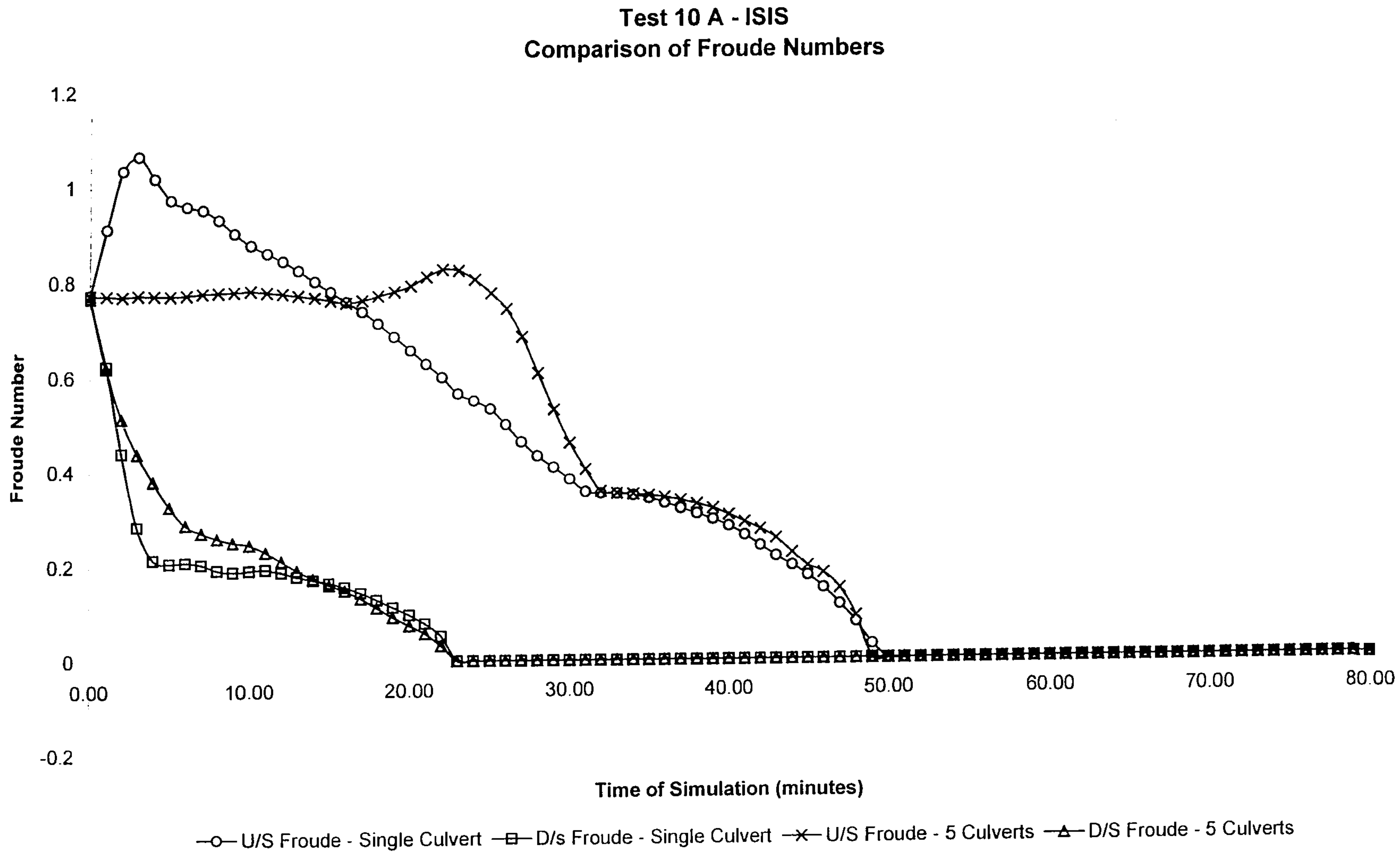
GRAPH 5 : *TEST 10A MIKE 11 - Comparison of velocities using open and closed sections.*



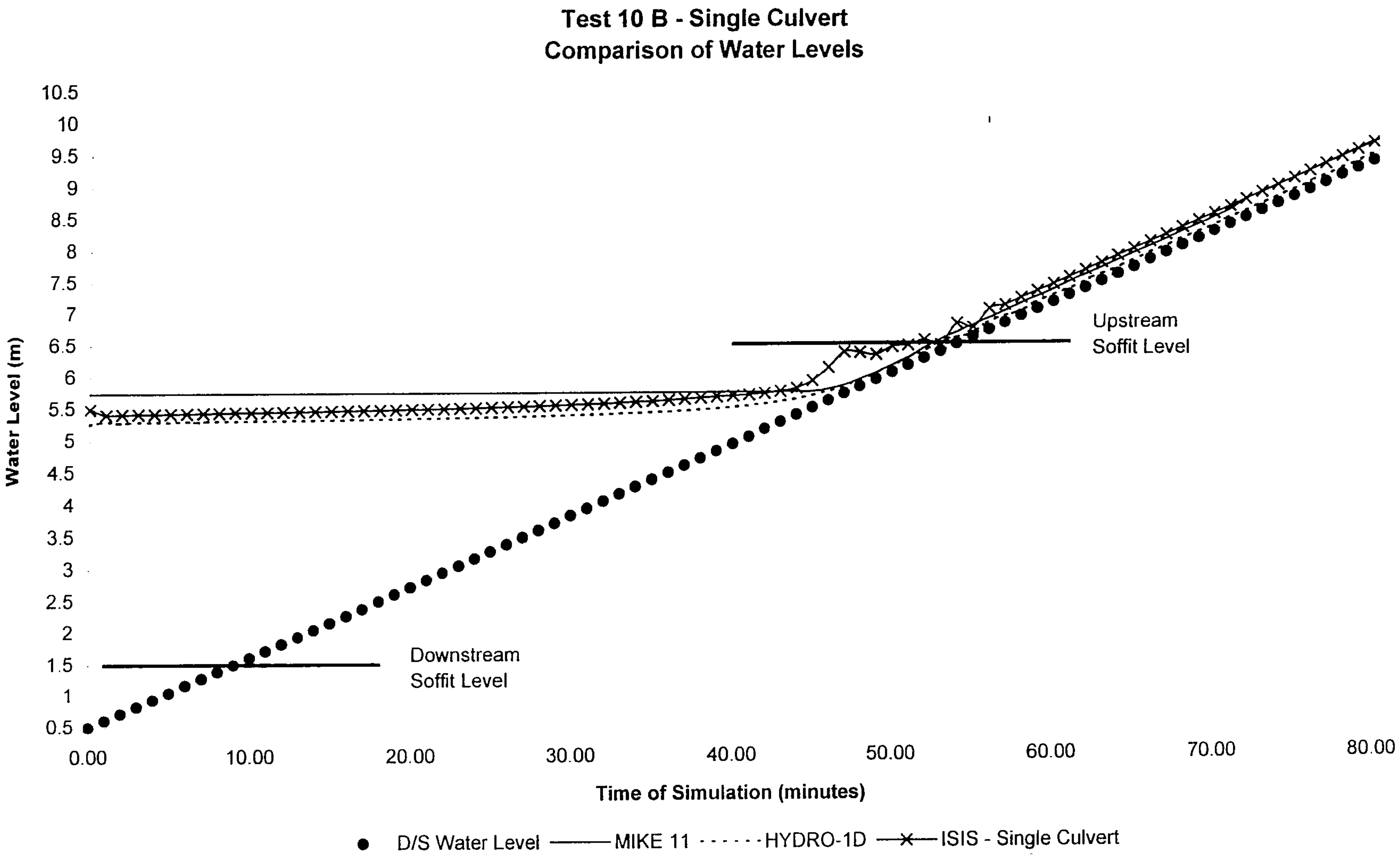
GRAPH 6 : *TEST 10A ISIS - Comparison of boundary water elevations for a single and five culvert system.*



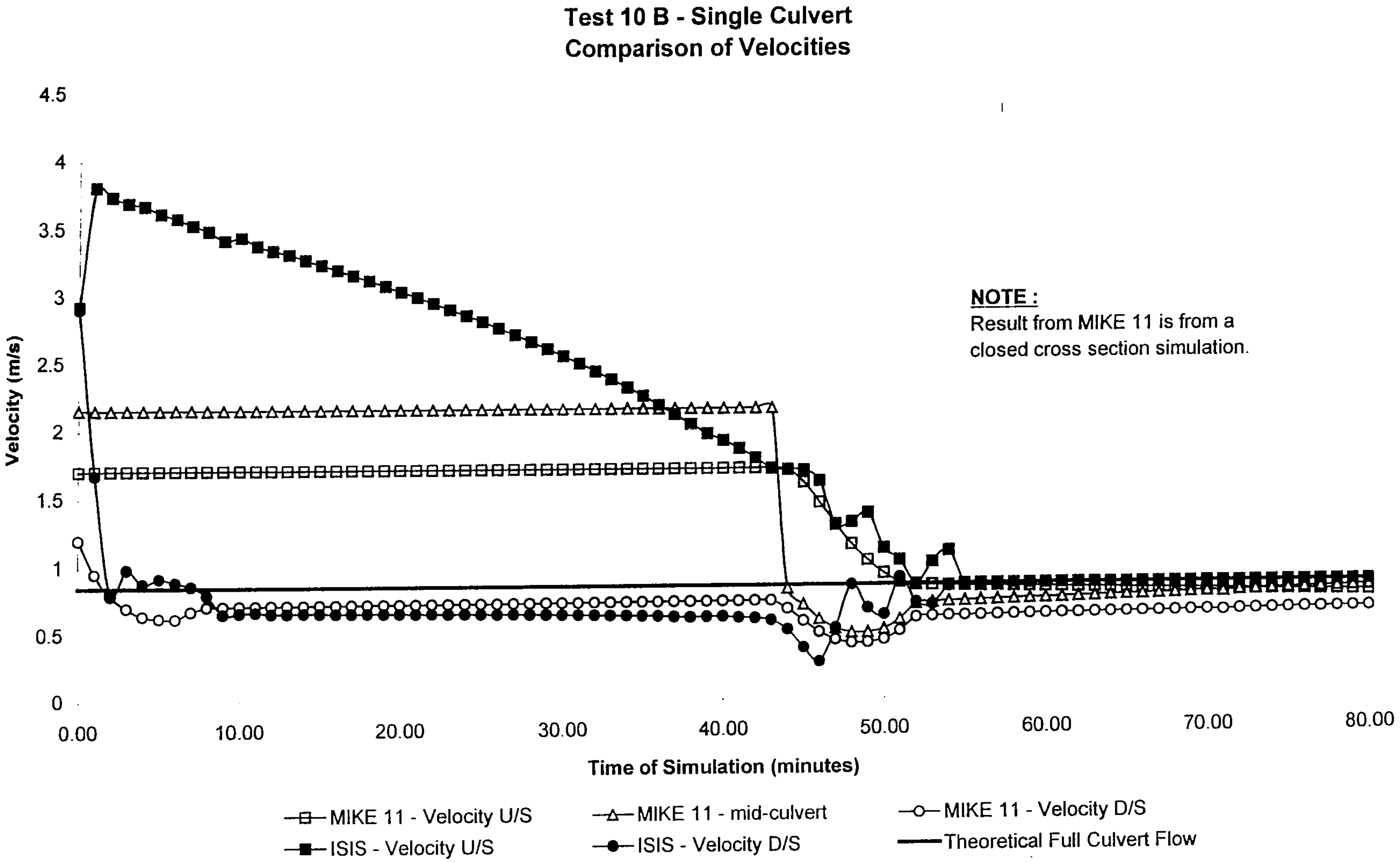
GRAPH 7 : *TEST 10A ISIS - Comparison of boundary water velocities for a single and five culvert system.*



GRAPH 8 : *TEST 10A ISIS - Comparison of boundary Froude numbers for a single and five culvert system.*



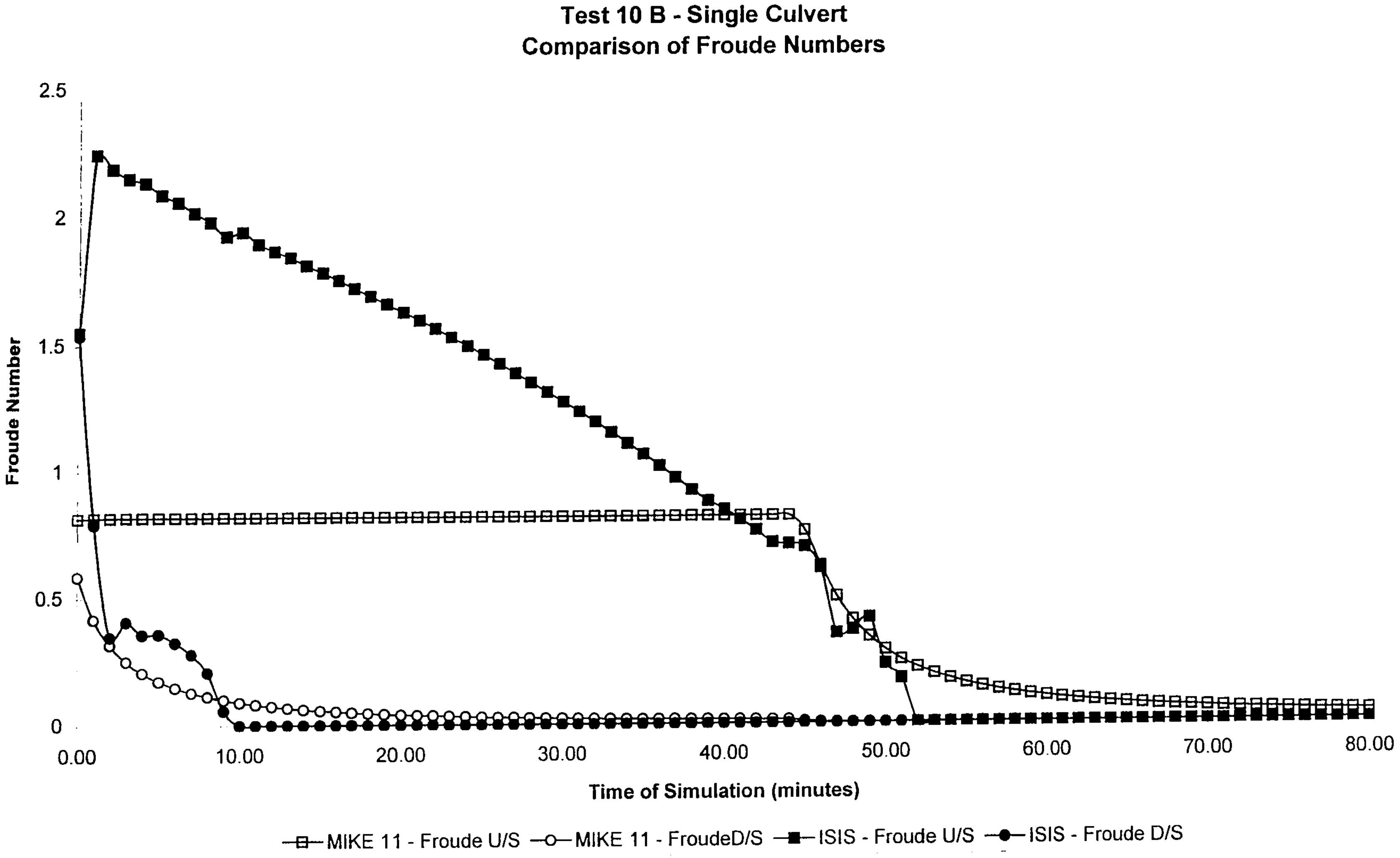
GRAPH 9 : *TEST 10B - Comparison of calculated upstream boundary water elevations throughout simulation.*



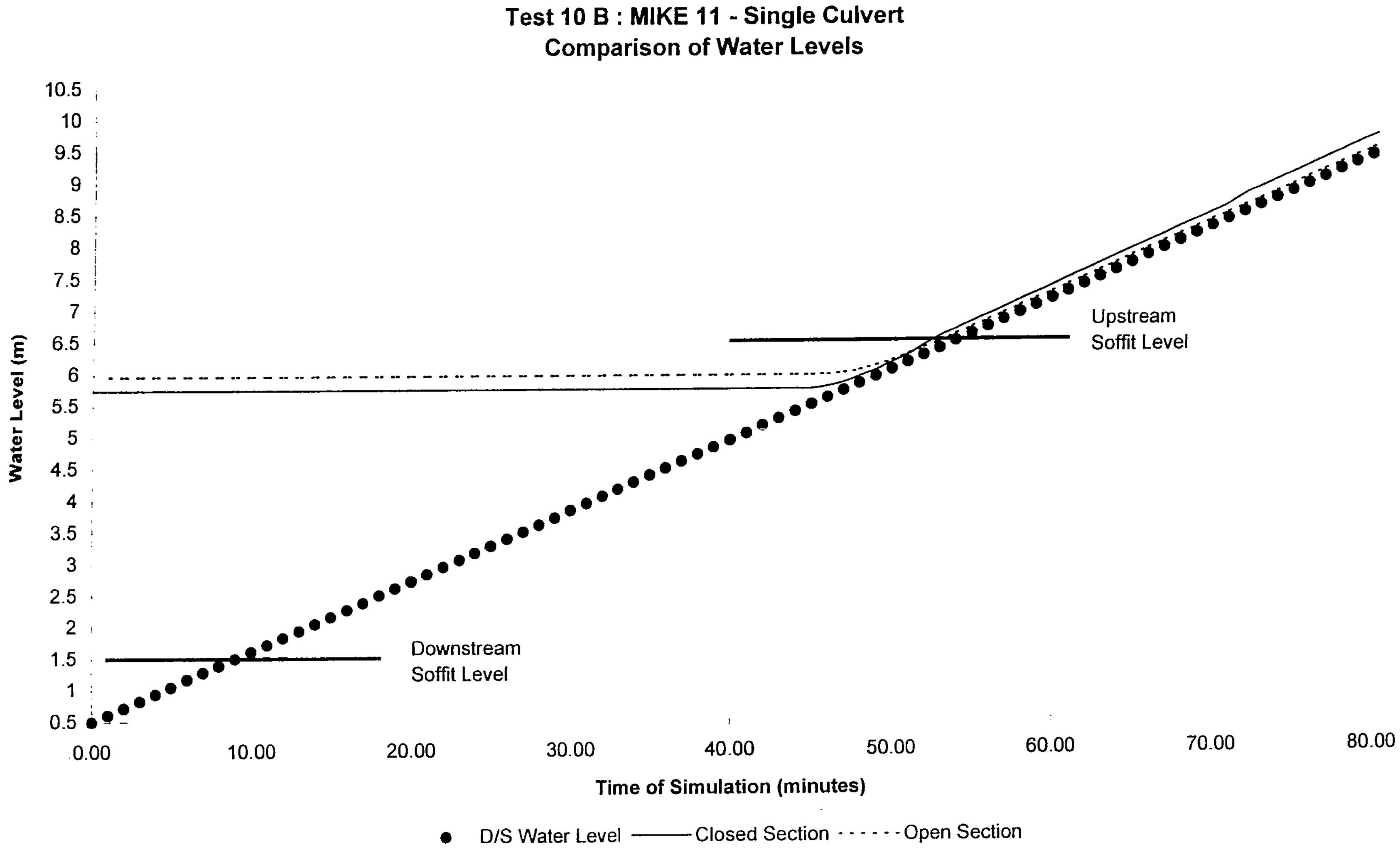
Test 10 B - Single Culvert
Comparison of Velocities

NOTE :
Result from MIKE 11 is from a
closed cross section simulation.

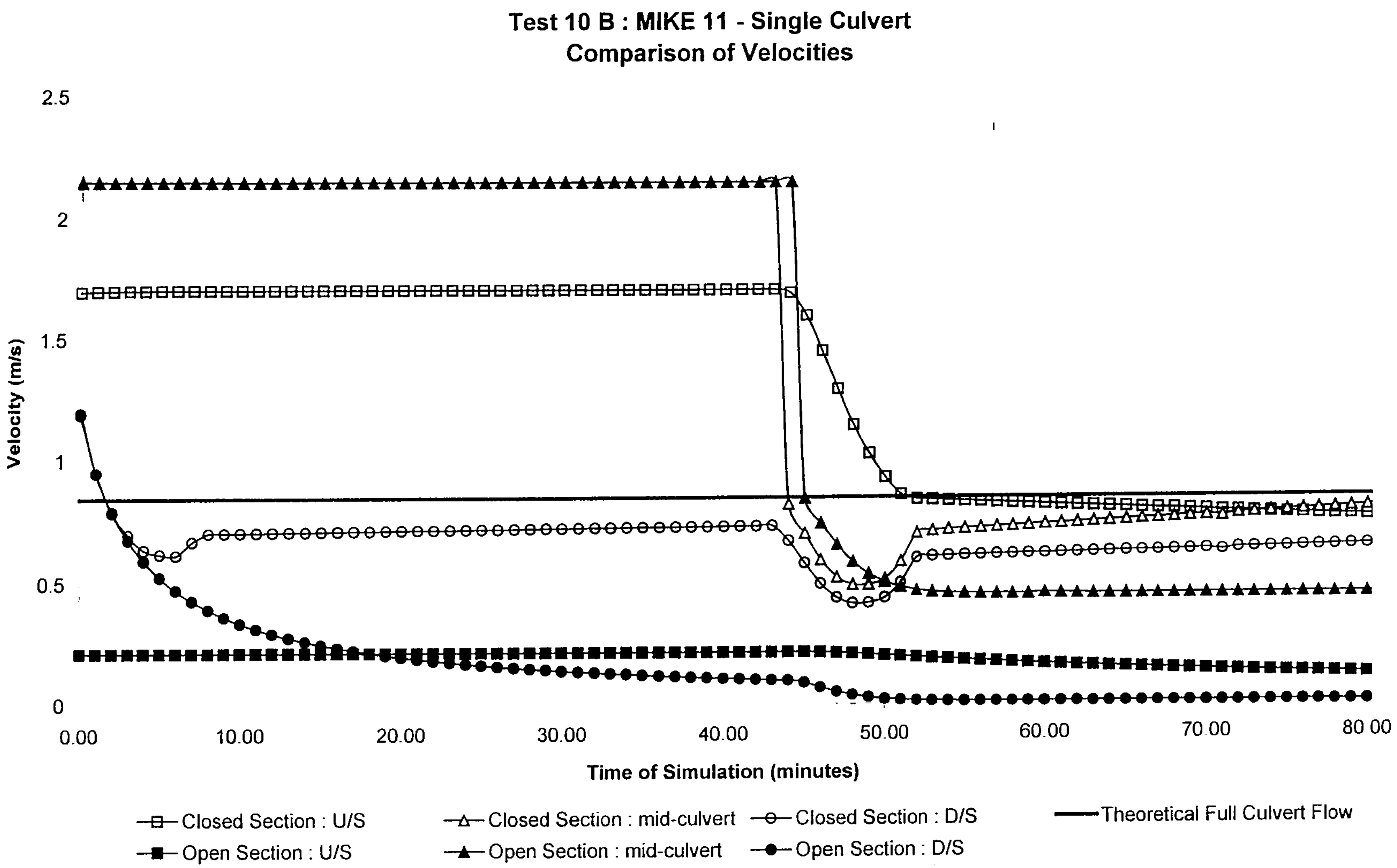
GRAPH 10 : *TEST 10B - Comparison of calculated upstream and downstream boundary velocities throughout simulation.*



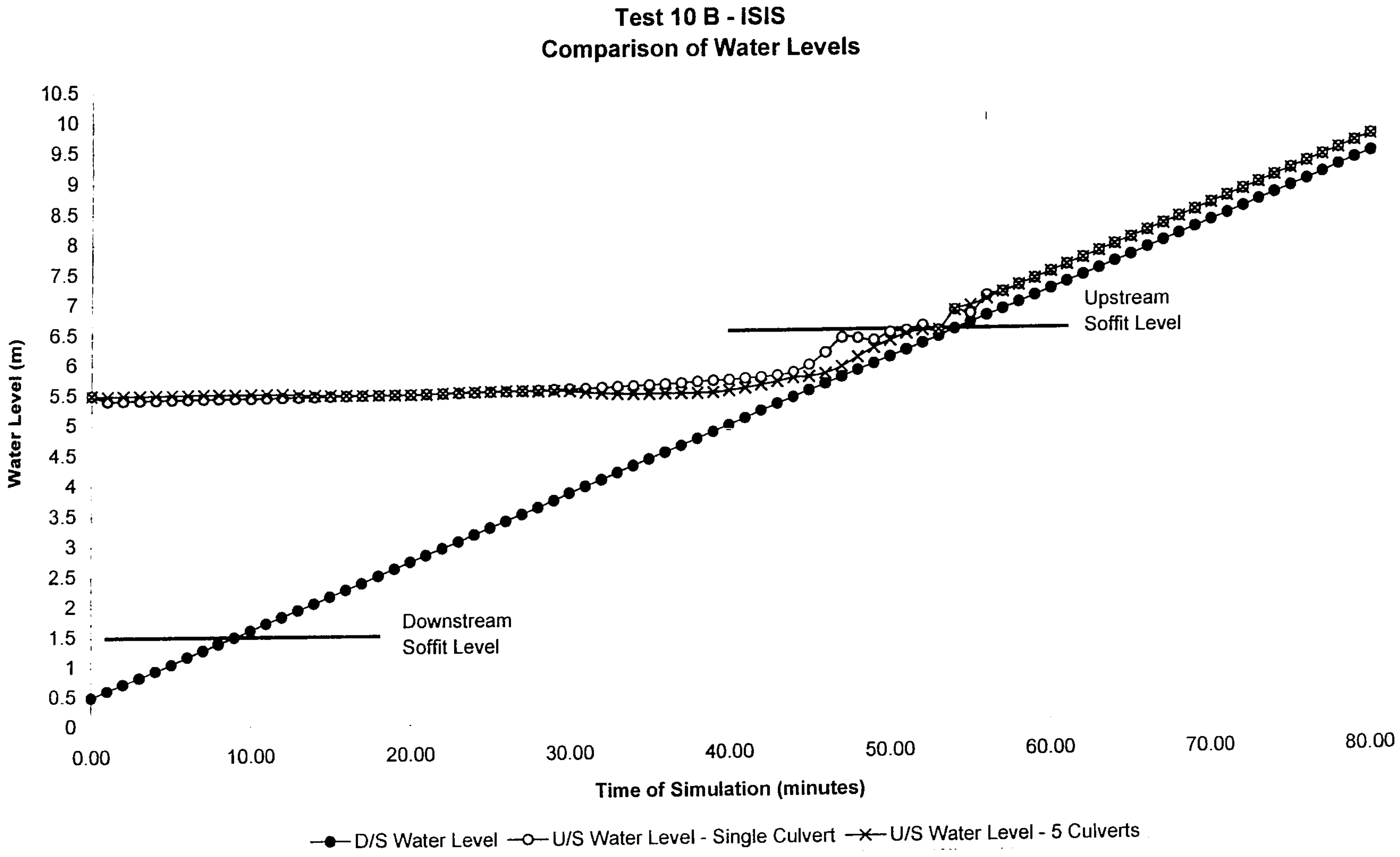
GRAPH 11 : *TEST 10B - Comparison of calculated upstream and downstream boundary Froude numbers throughout simulation.*



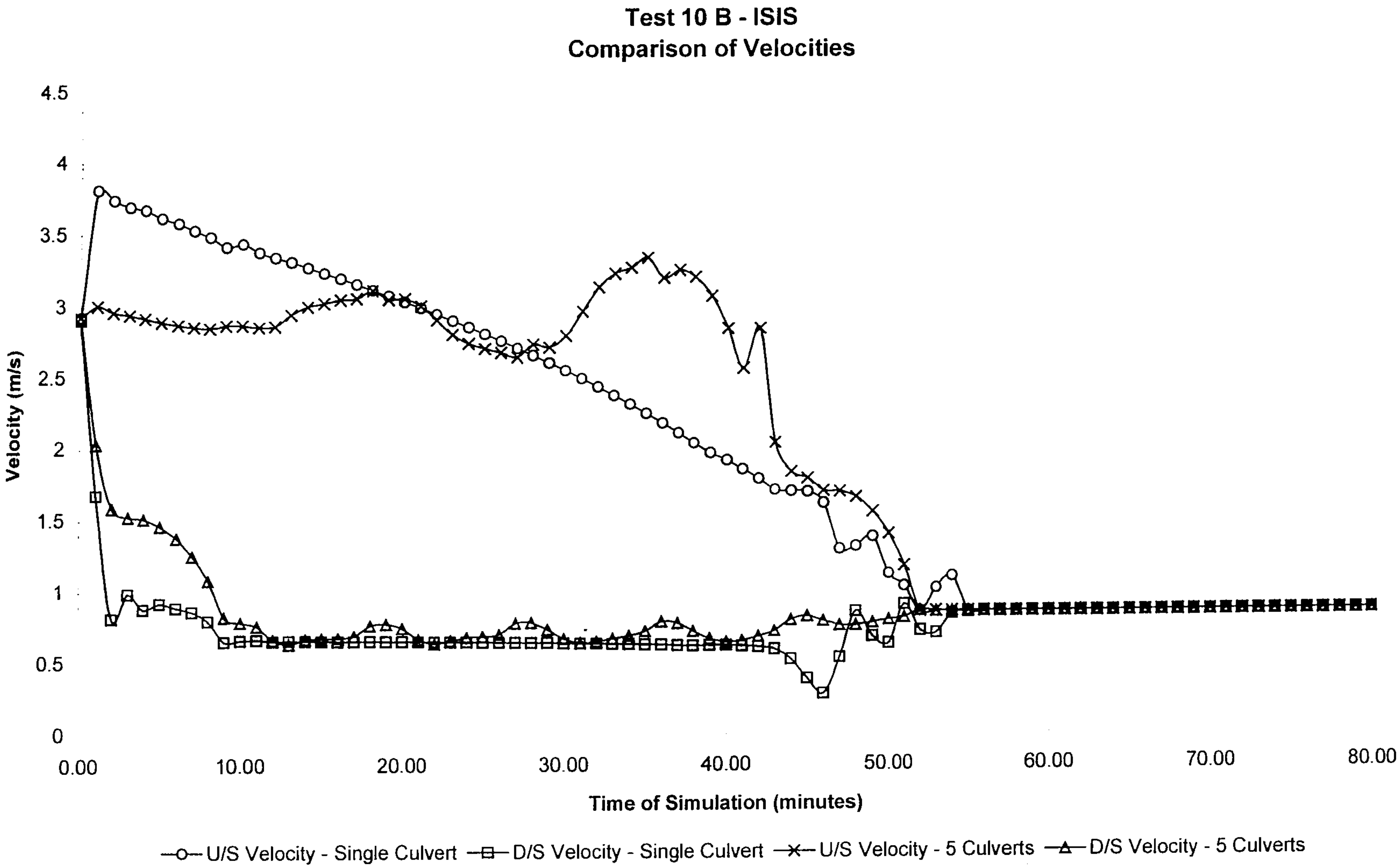
GRAPH 12 : *TEST 10B MIKE 11 - Comparison of boundary water levels using open and closed sections.*



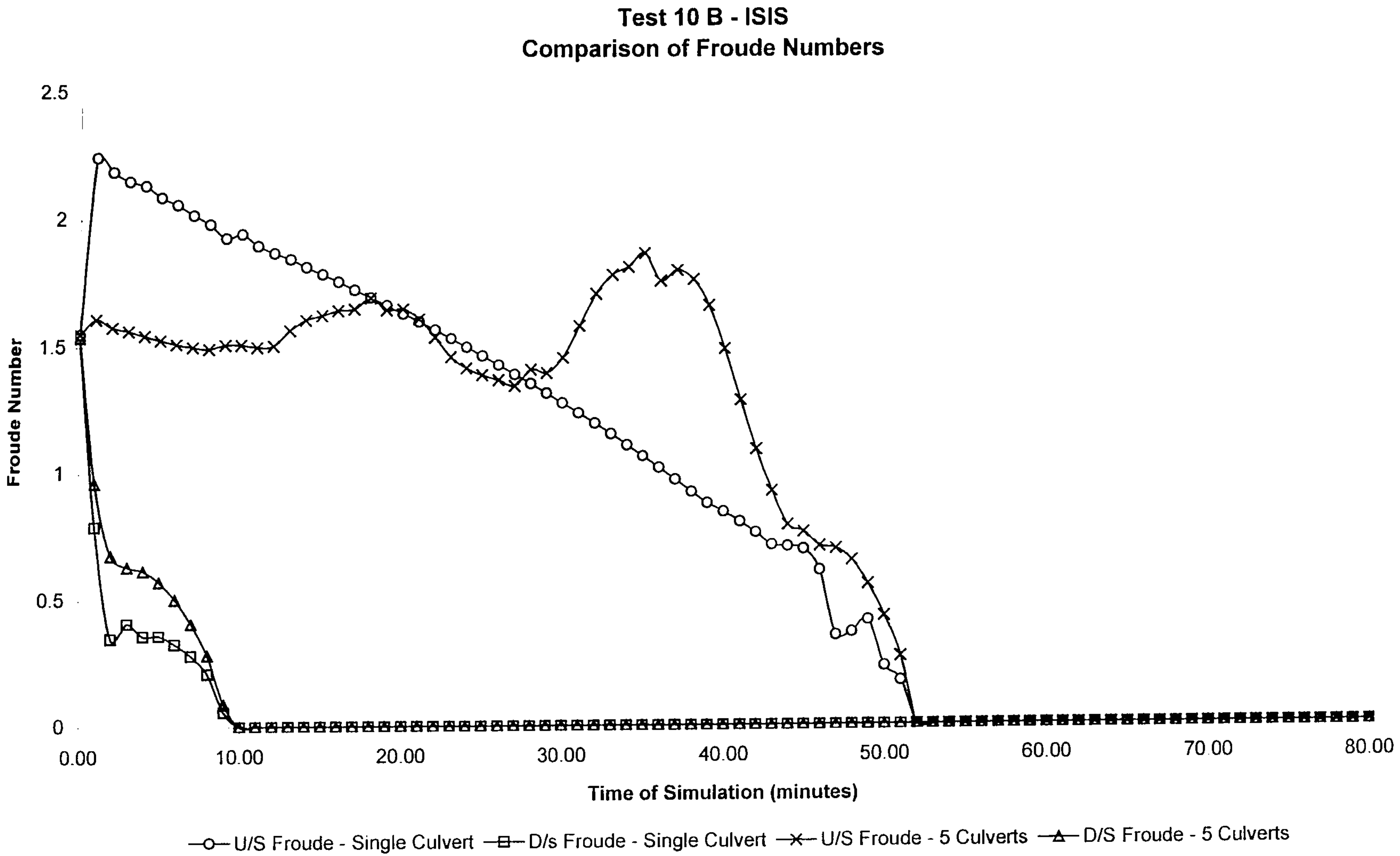
GRAPH 13 : *TEST 10B MIKE 11 - Comparison of velocities using open and closed sections*



GRAPH 14 : *TEST 10B ISIS - Comparison of boundary water levels for a single and five culvert system.*

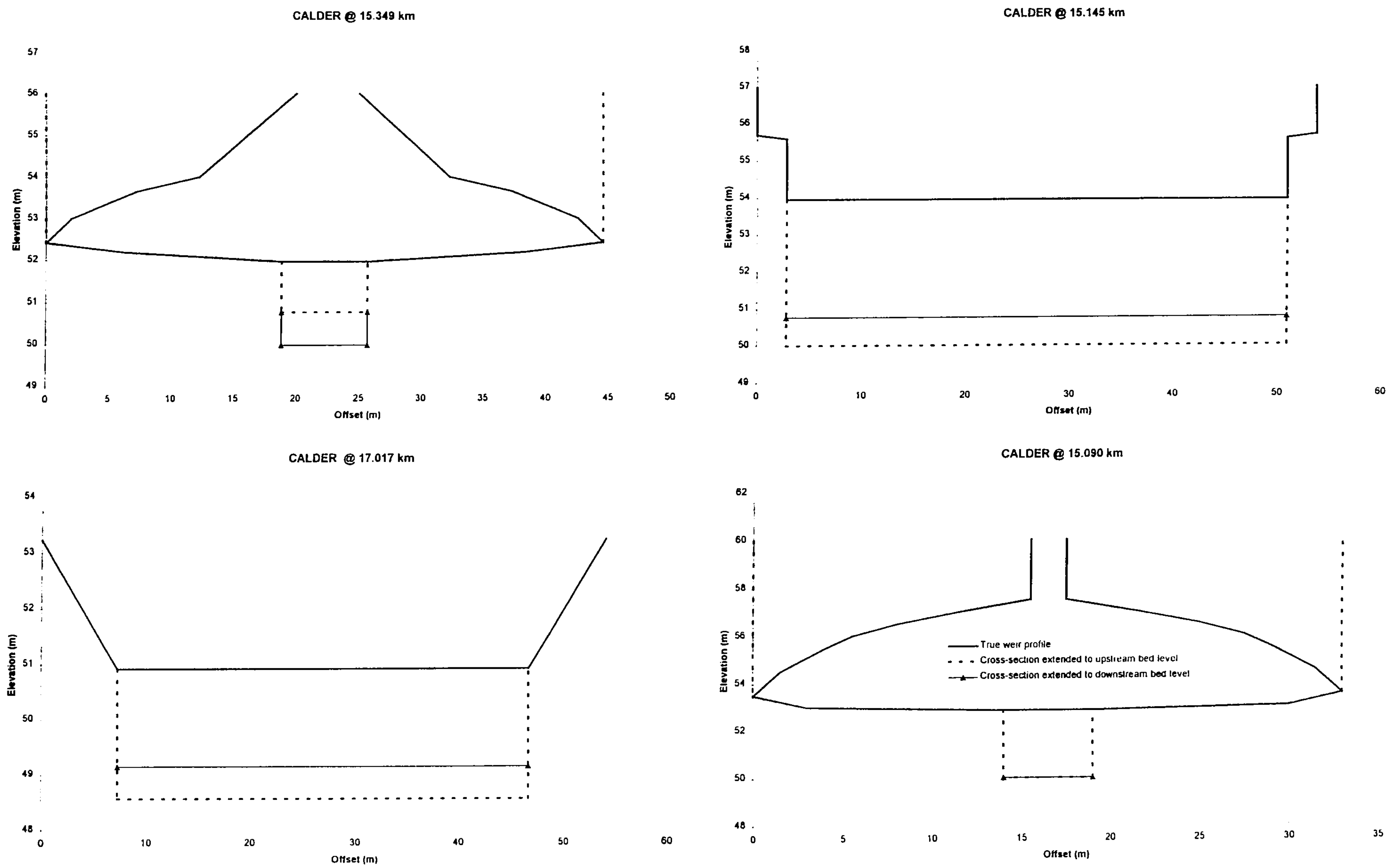


GRAPH 15 : *TEST 10B ISIS - Comparison of boundary water velocities for a single and five culvert system.*

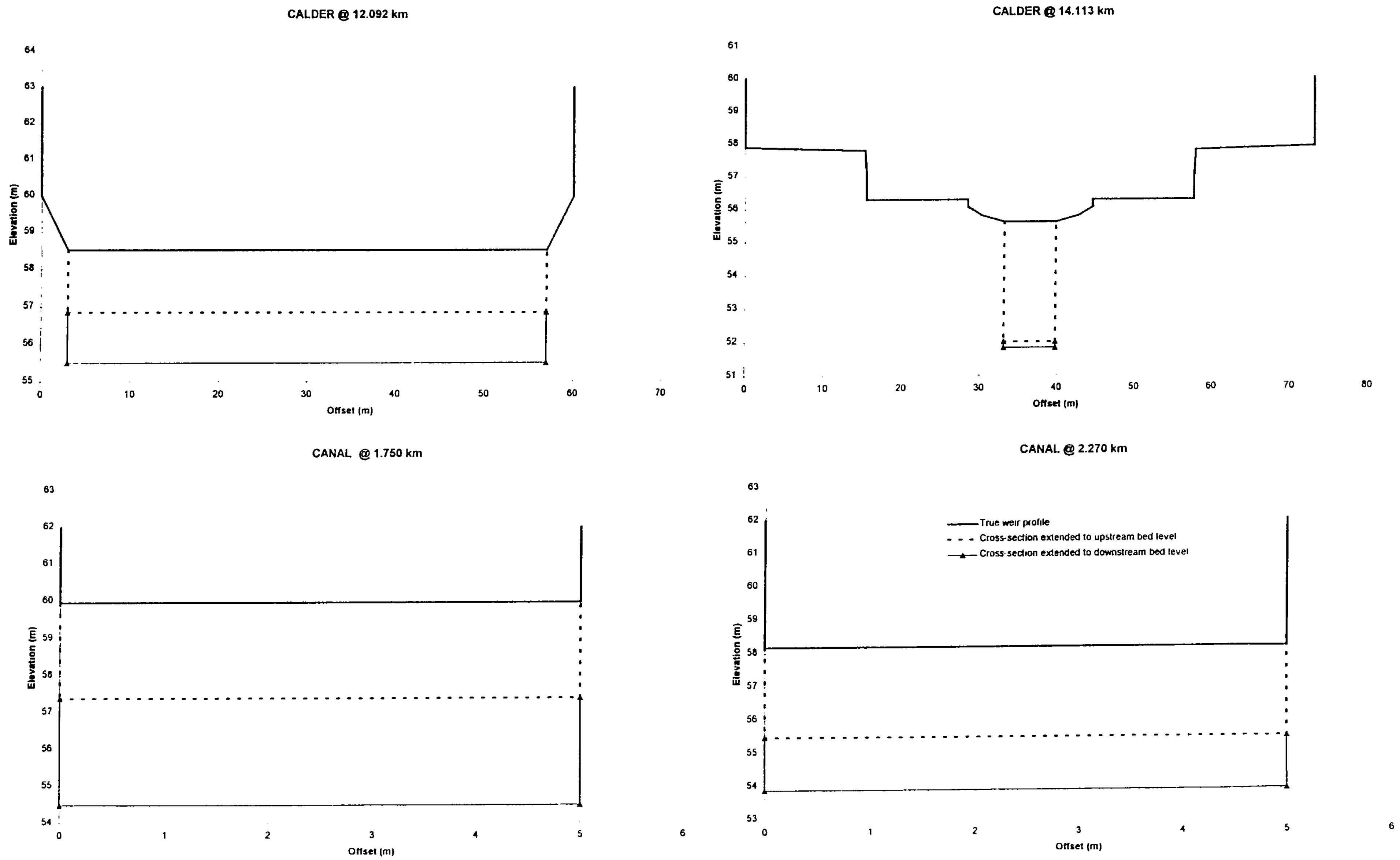


GRAPH 16 : *TEST 10B ISIS - Comparison of boundary Froude numbers for a single and five culvert system.*

Appendix I



GRAPH 1 : *Test 12 - Weir profiles (Part 1 of 7)*

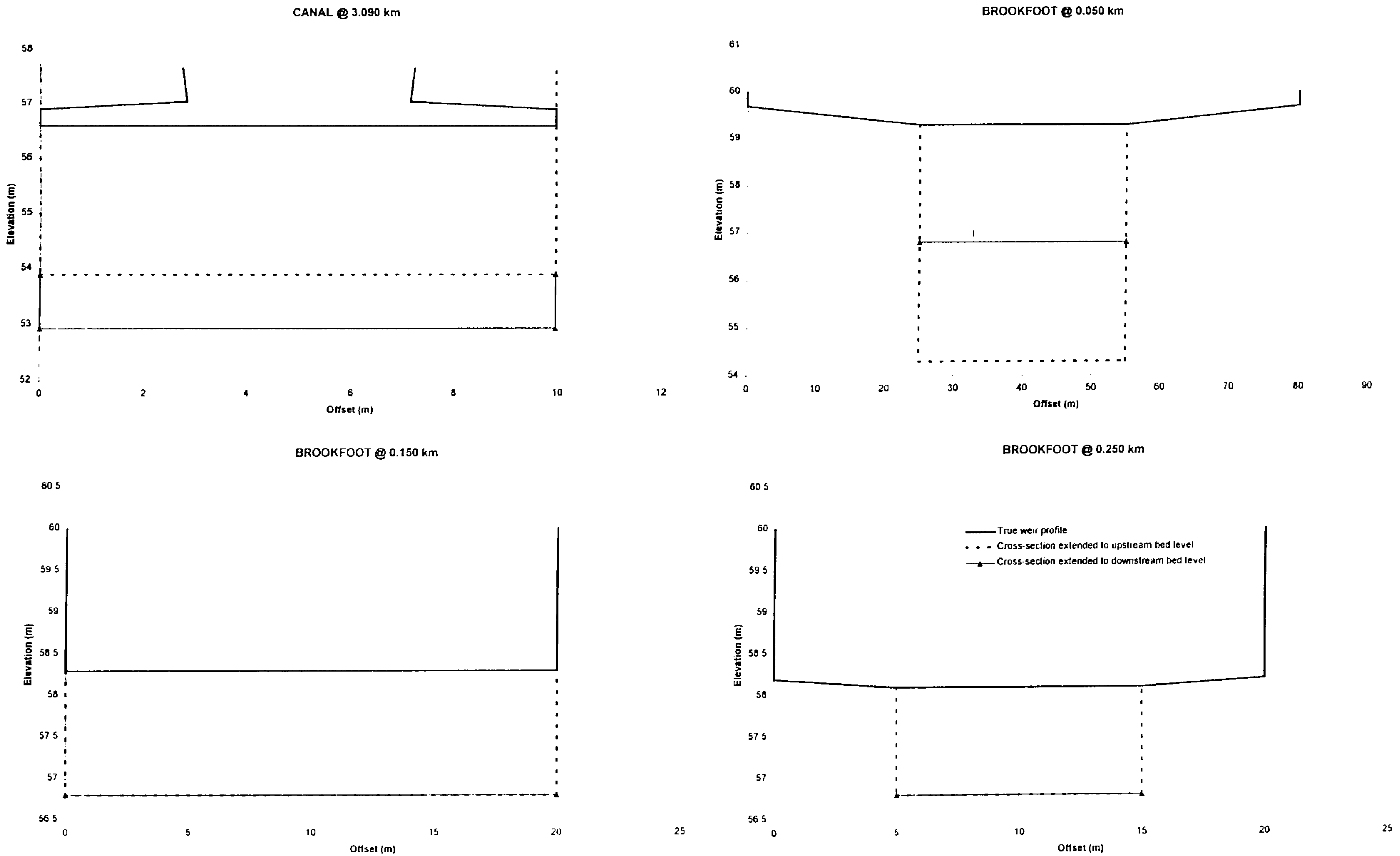

CALDER @ 12.092 km

CALDER @ 14.113 km

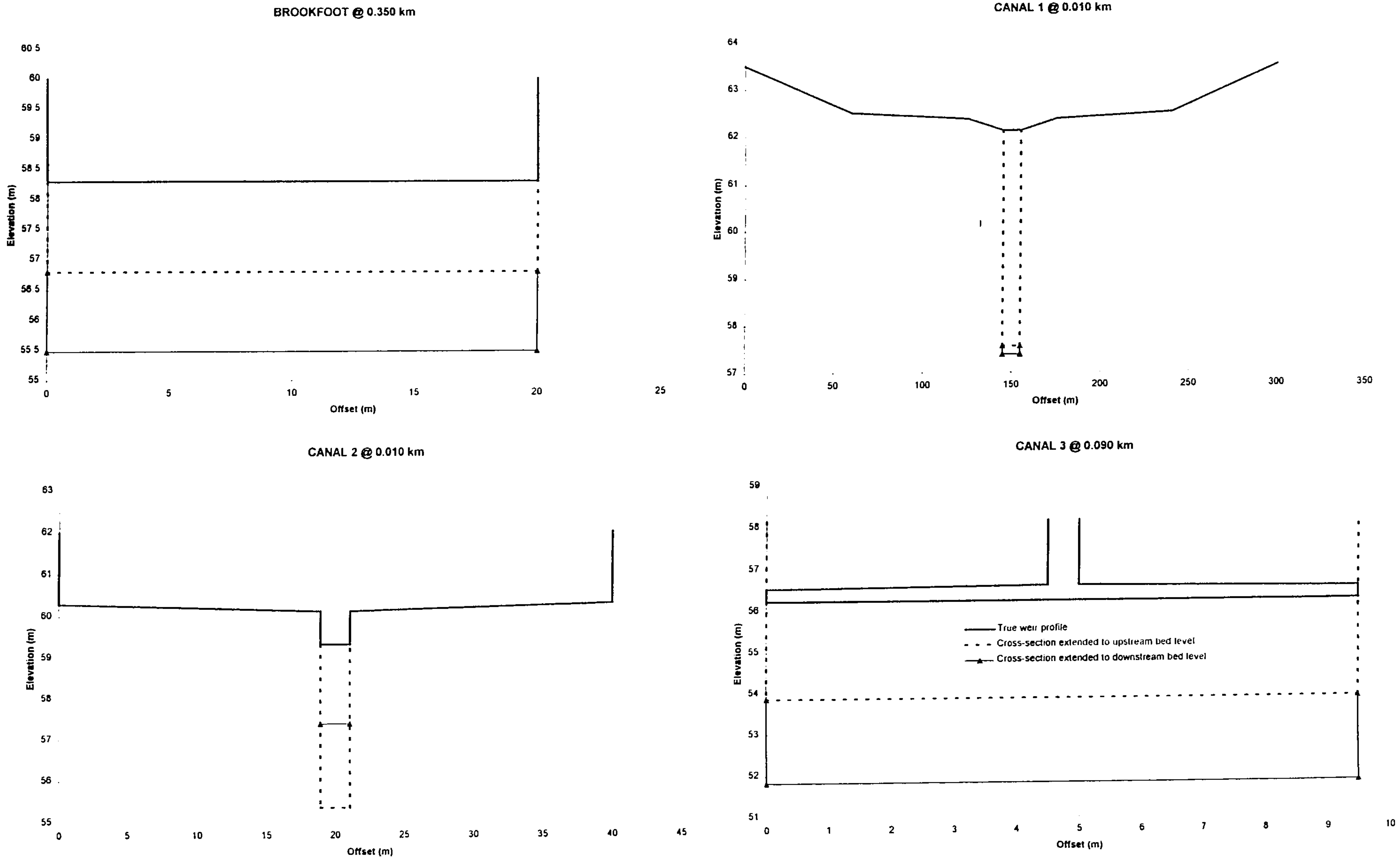
CANAL @ 1.750 km

CANAL @ 2.270 km

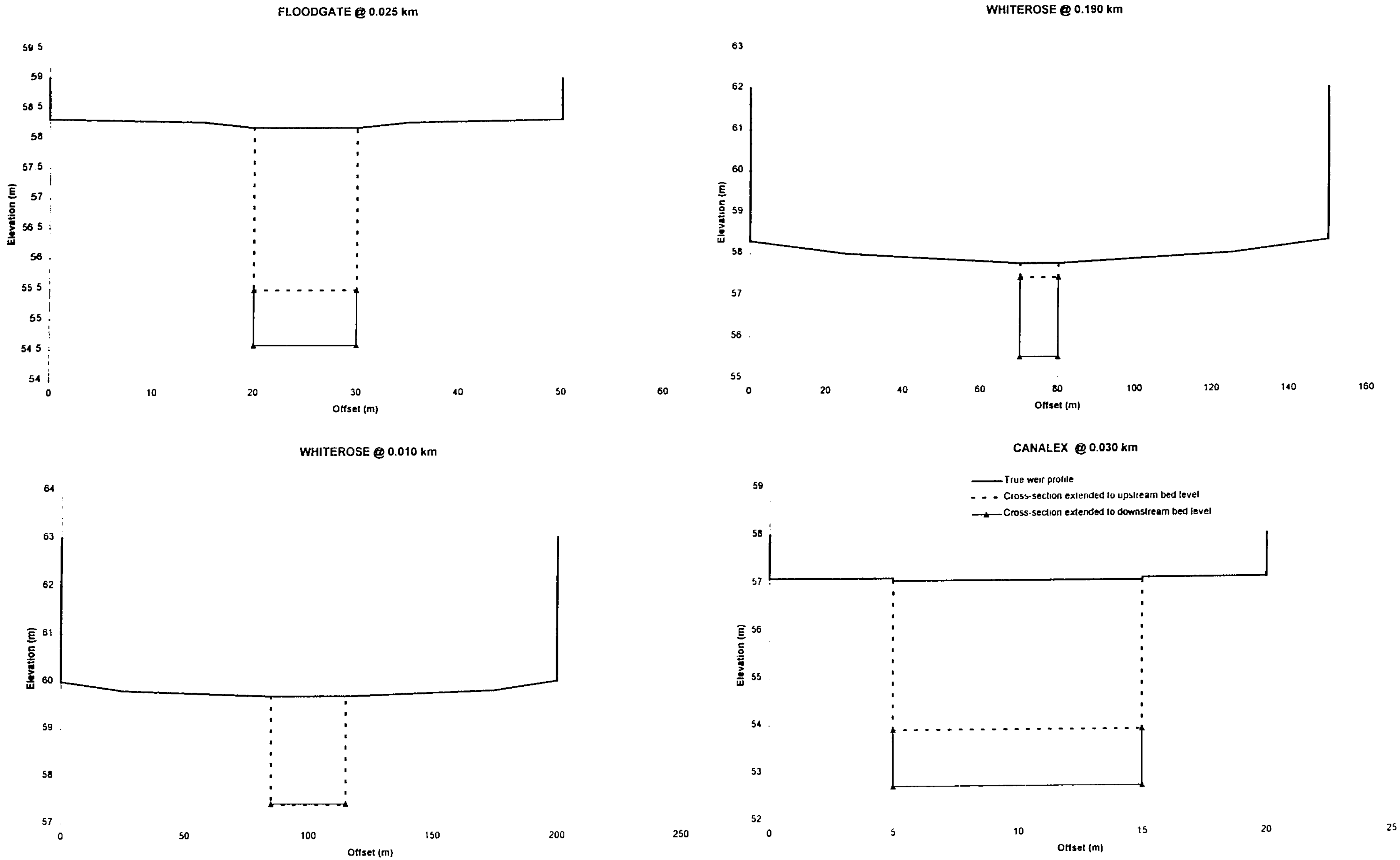
GRAPH 1 : *Test 12 - Weir profiles (Part 2 of 7)*



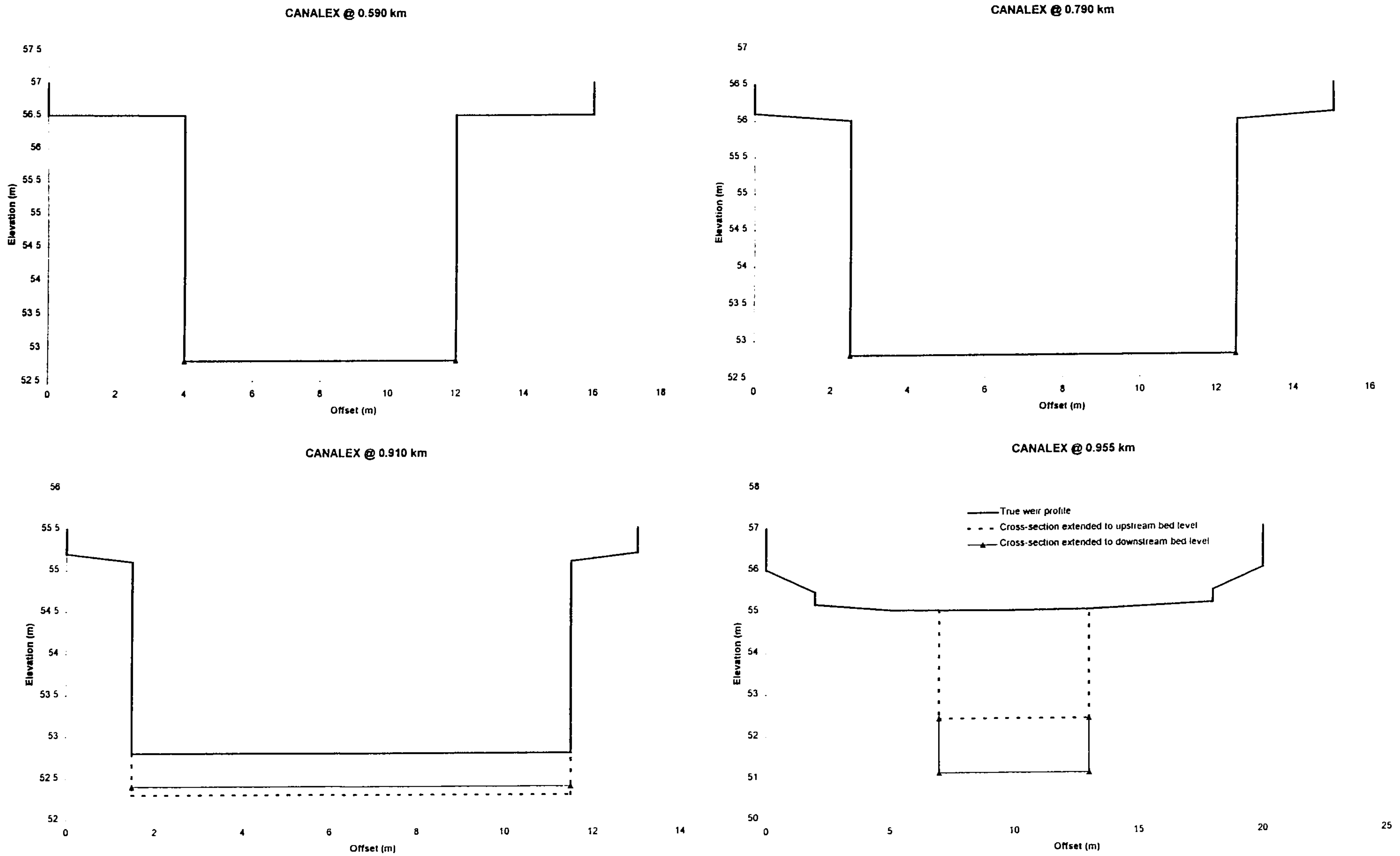
GRAPH 1 : *Test 12 - Weir profiles (Part 3 of 7)*



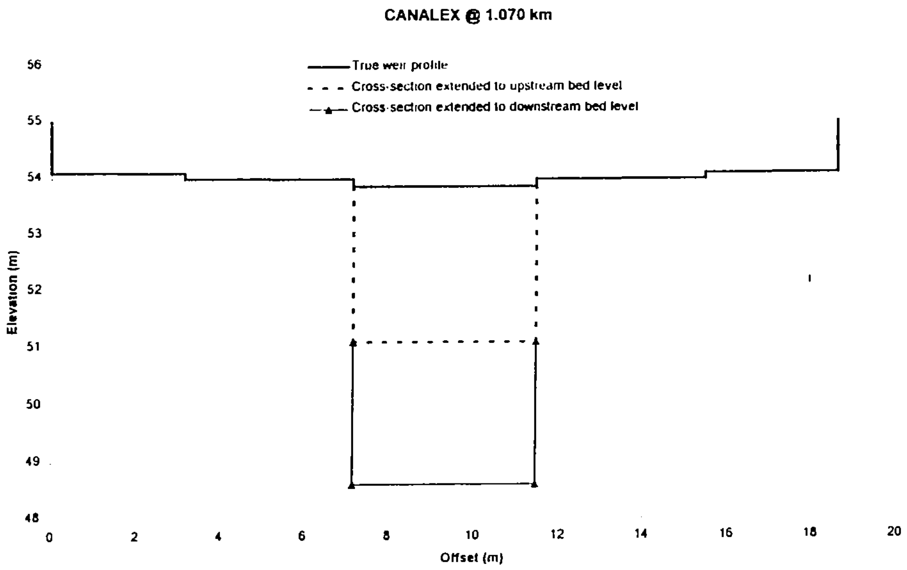
GRAPH 1 : *Test 12 - Weir profiles (Part 4 of 7)*



GRAPH 1 : *Test 12 - Weir profiles (Part 5 of 7)*

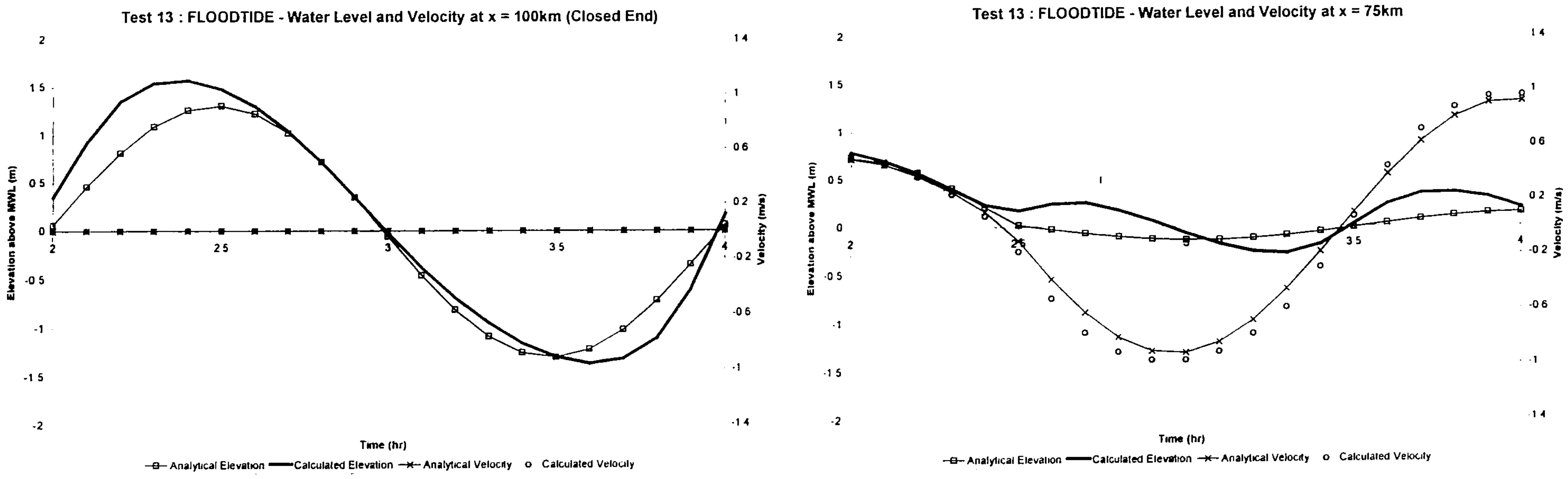


GRAPH 1 : *Test 12 - Weir profiles (Part 6 of 7)*

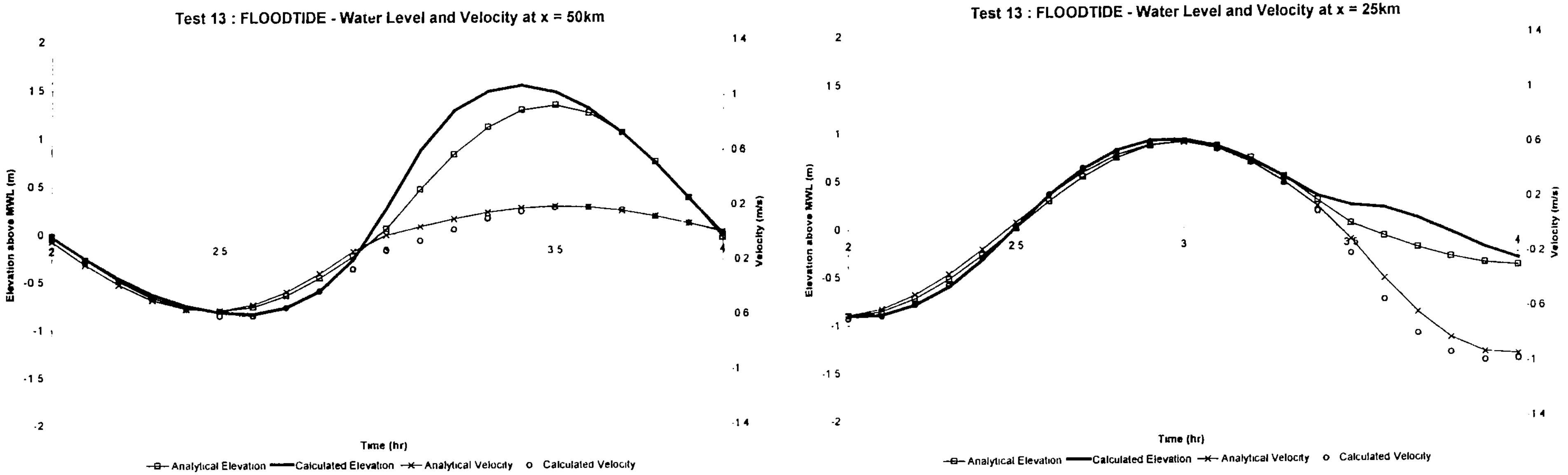


GRAPH 1 : *Test 12 - Weir profiles (Part 7 of 7)*

Appendix J

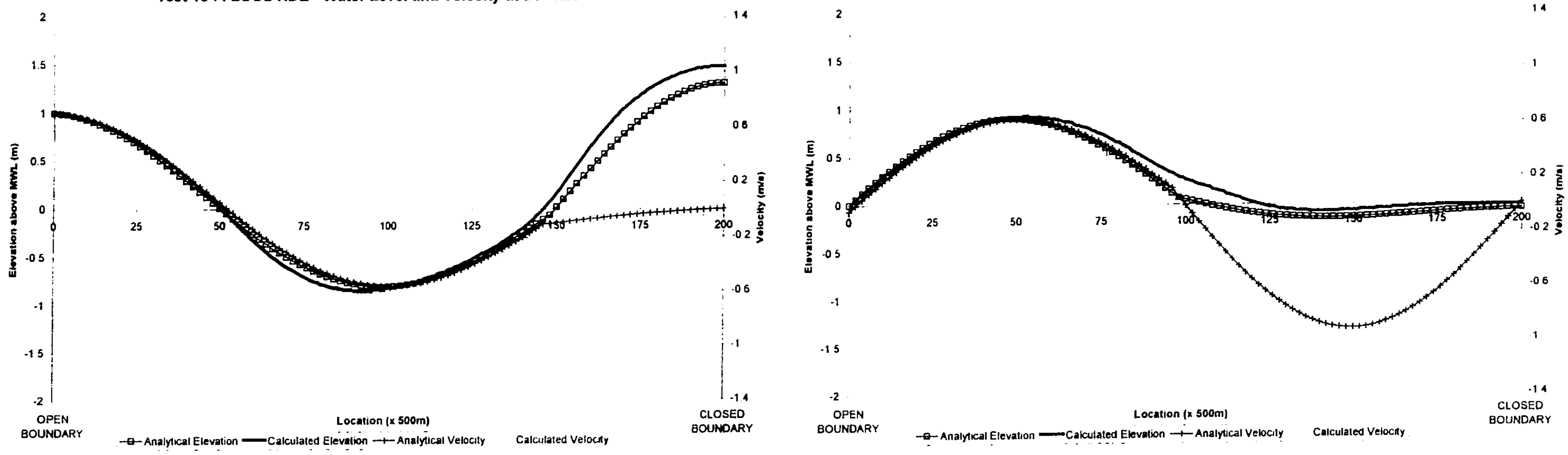

Test 13 : FLOODTIDE - Water Level and Velocity at x = 100km (Closed End)

Test 13 : FLOODTIDE - Water Level and Velocity at x = 75km


Test 13 : FLOODTIDE - Water Level and Velocity at x = 50km

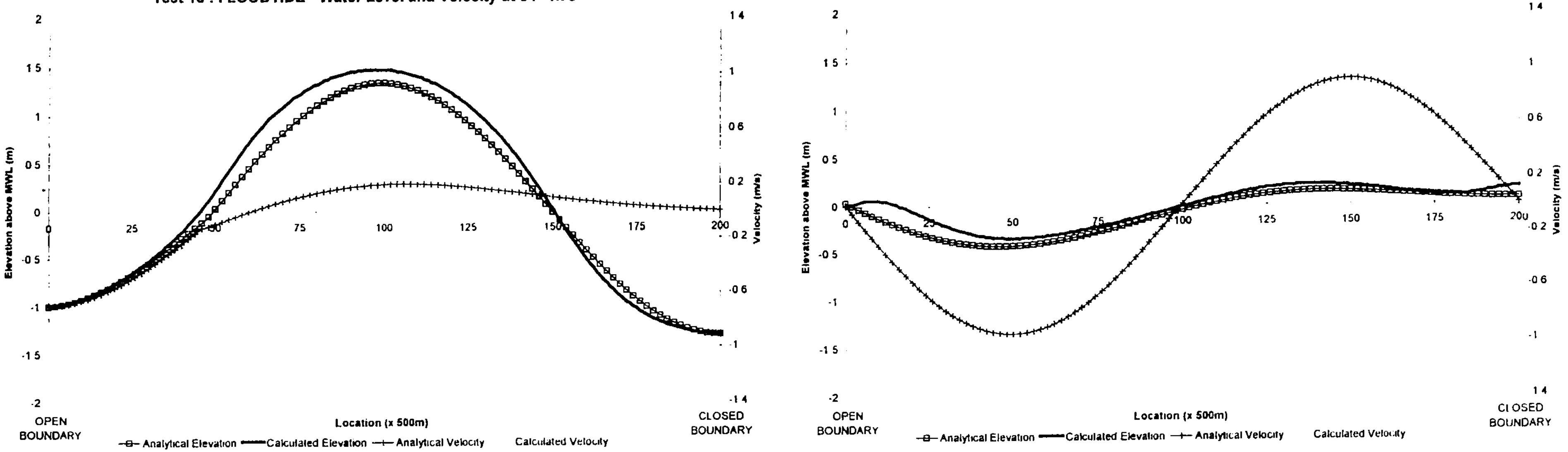
Test 13 : FLOODTIDE - Water Level and Velocity at x = 25km

GRAPH 1 : *Test 13 : FLOODTIDE - Calculated velocities and water elevations at fixed locations.*



Test 13 : FLOODTIDE - Water Level and Velocity at t/T=1.25

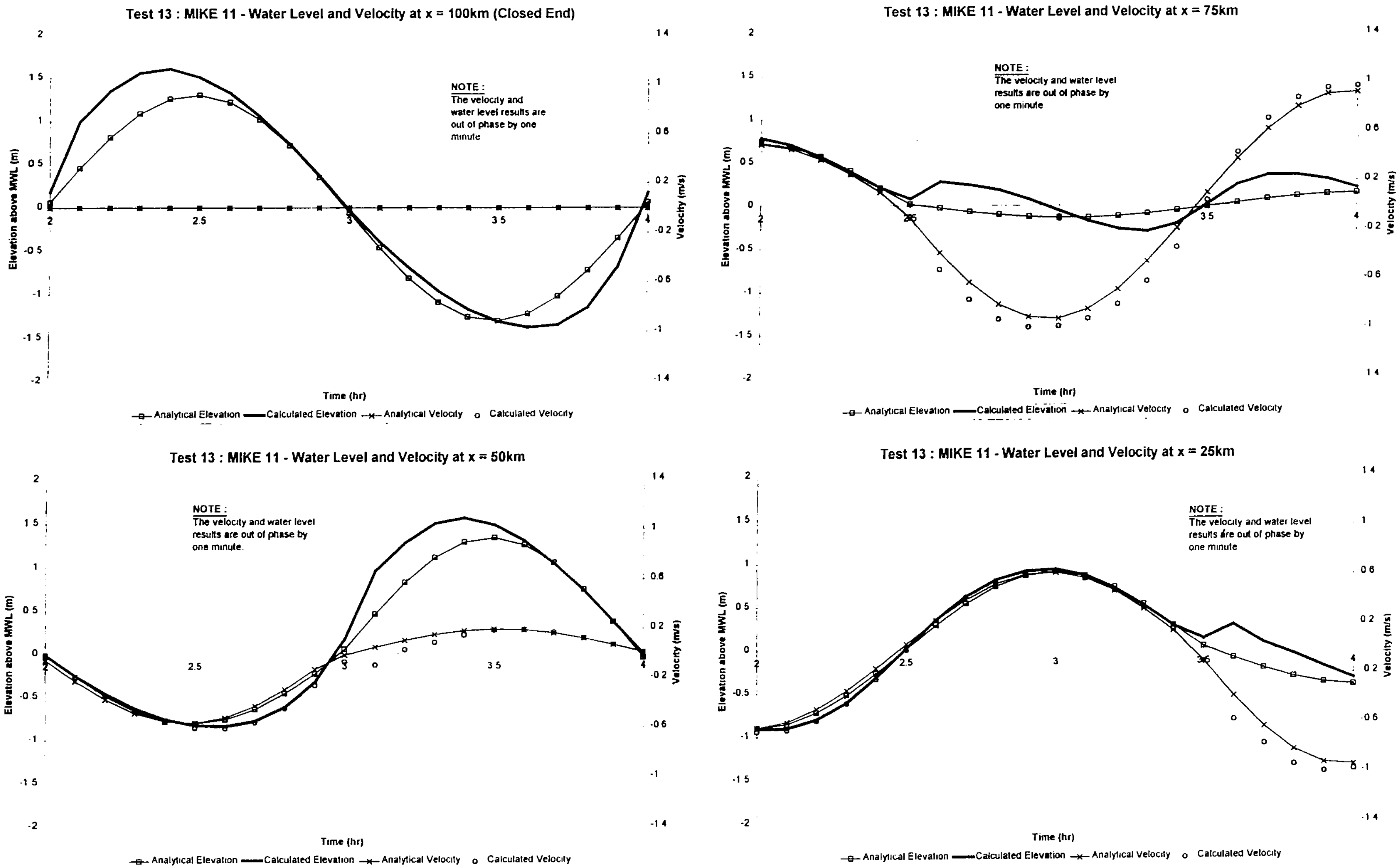
Test 13 : FLOODTIDE - Water Level and Velocity at t/T=1.5



Test 13 : FLOODTIDE - Water Level and Velocity at t/T=1.75

Test 13 : FLOODTIDE - Water Level and Velocity at t/T=2.0

GRAPH 2 : *Test 13 : FLOODTIDE - Calculated velocities and water elevations at fixed times.*


Test 13 : MIKE 11 - Water Level and Velocity at x = 100km (Closed End)

NOTE :
The velocity and water level results are out of phase by one minute

Test 13 : MIKE 11 - Water Level and Velocity at x = 75km

NOTE :
The velocity and water level results are out of phase by one minute

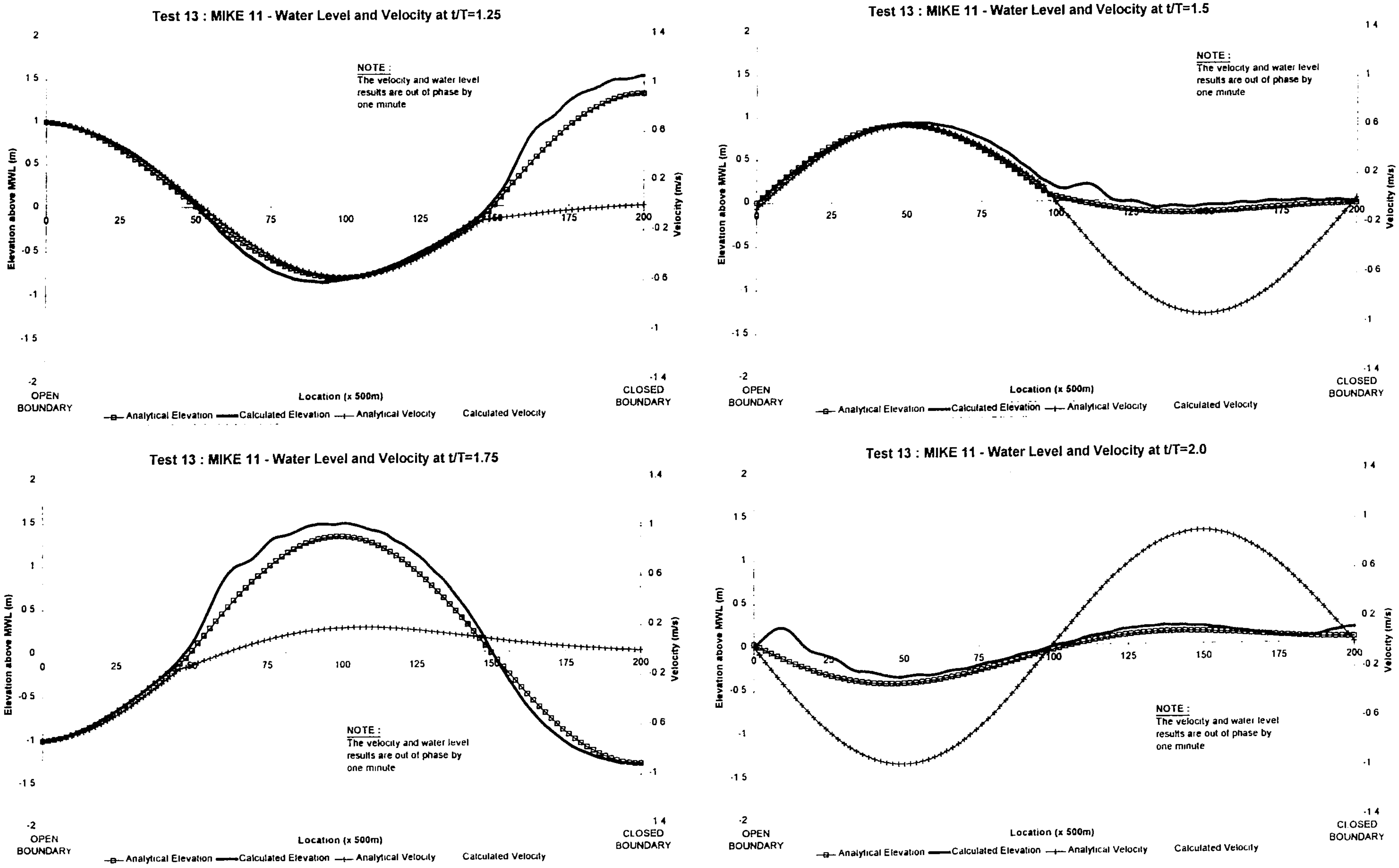
Test 13 : MIKE 11 - Water Level and Velocity at x = 50km

NOTE :
The velocity and water level results are out of phase by one minute.

Test 13 : MIKE 11 - Water Level and Velocity at x = 25km

NOTE :
The velocity and water level results are out of phase by one minute

GRAPH 3 : *Test 13 : MIKE 11 - Calculated velocities and water elevations at fixed locations.*



Test 13 : MIKE 11 - Water Level and Velocity at t/T=1.25

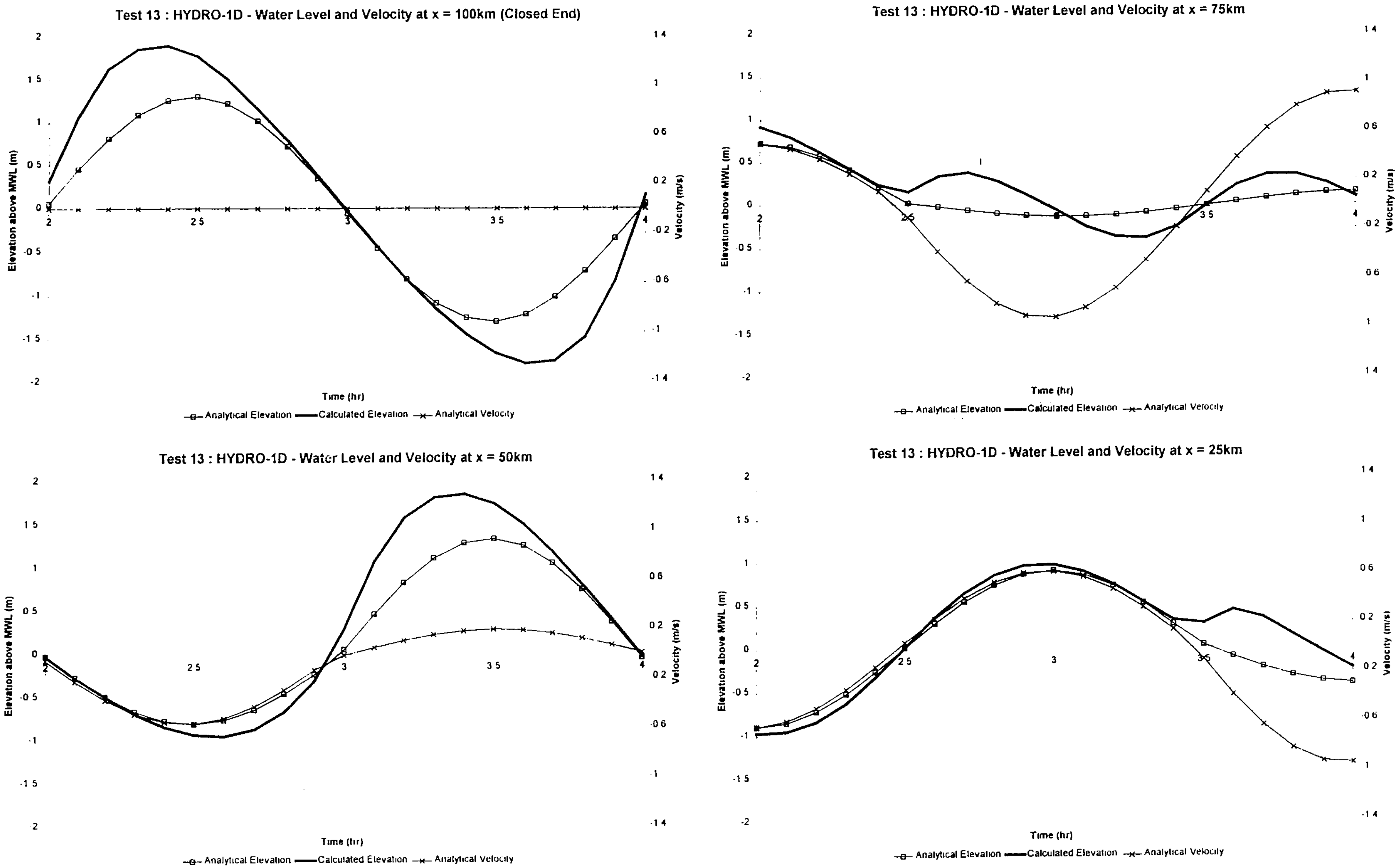
Test 13 : MIKE 11 - Water Level and Velocity at t/T=1.5

Test 13 : MIKE 11 - Water Level and Velocity at t/T=1.75

Test 13 : MIKE 11 - Water Level and Velocity at t/T=2.0

NOTE : The velocity and water level results are out of phase by one minute

GRAPH 4 : *Test 13 : MIKE 11 - Calculated velocities and water elevations at fixed times.*

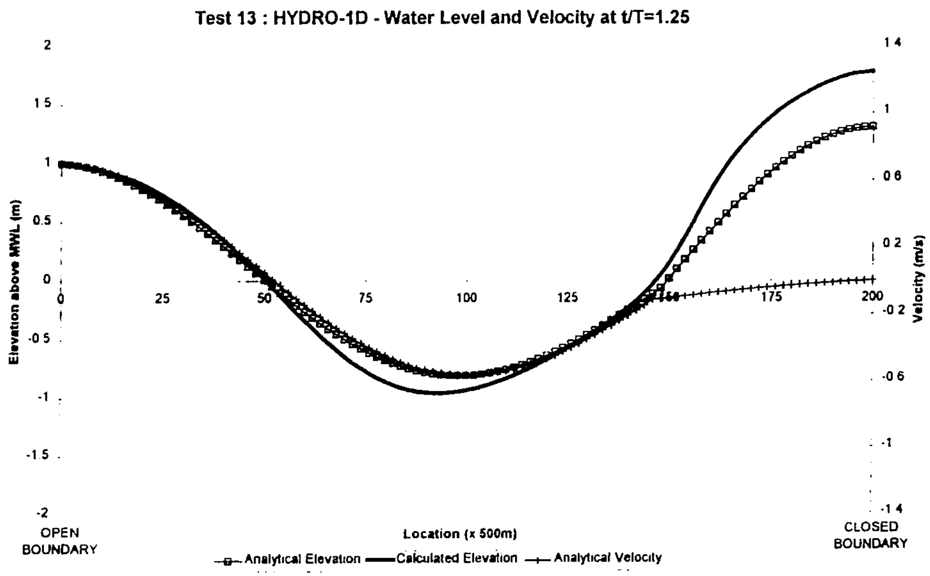

Test 13 : HYDRO-1D - Water Level and Velocity at x = 100km (Closed End)

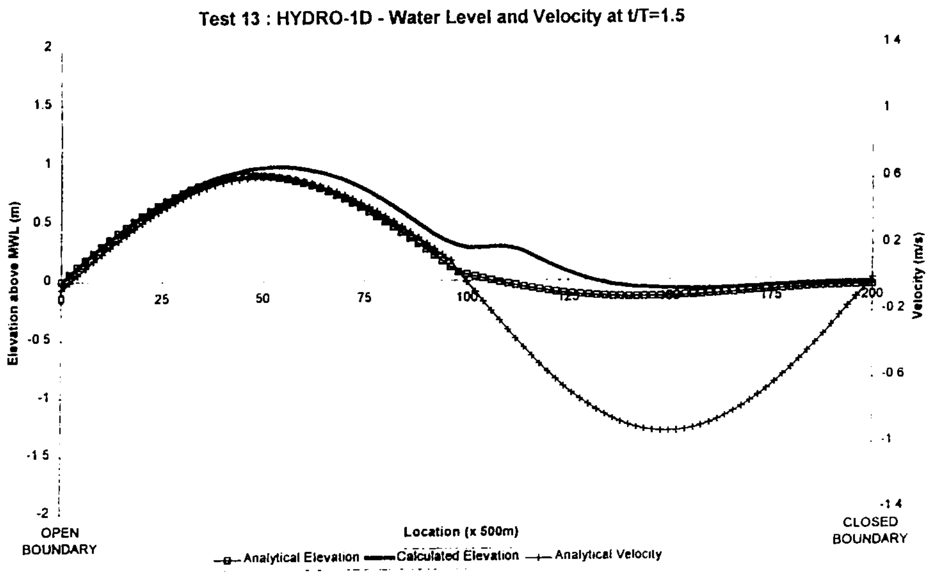
Test 13 : HYDRO-1D - Water Level and Velocity at x = 75km

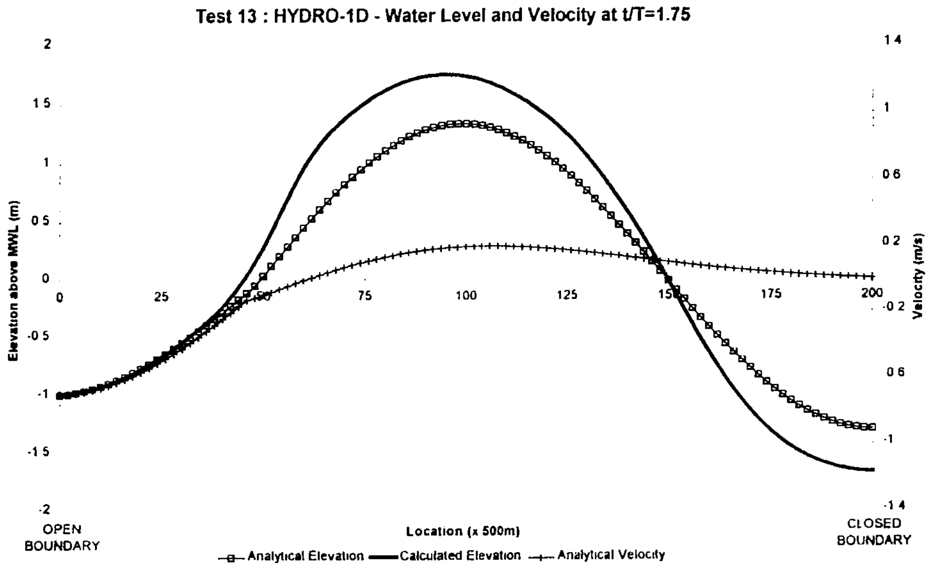
Test 13 : HYDRO-1D - Water Level and Velocity at x = 50km

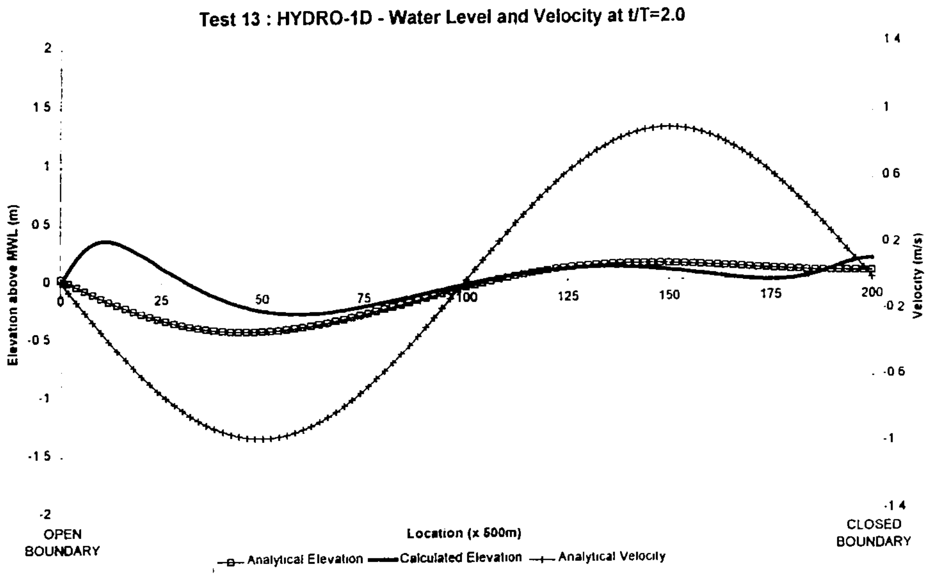
Test 13 : HYDRO-1D - Water Level and Velocity at x = 25km

GRAPH 5 : *Test 13 : HYDRO-1D - Calculated velocities and water elevations at fixed locations.*

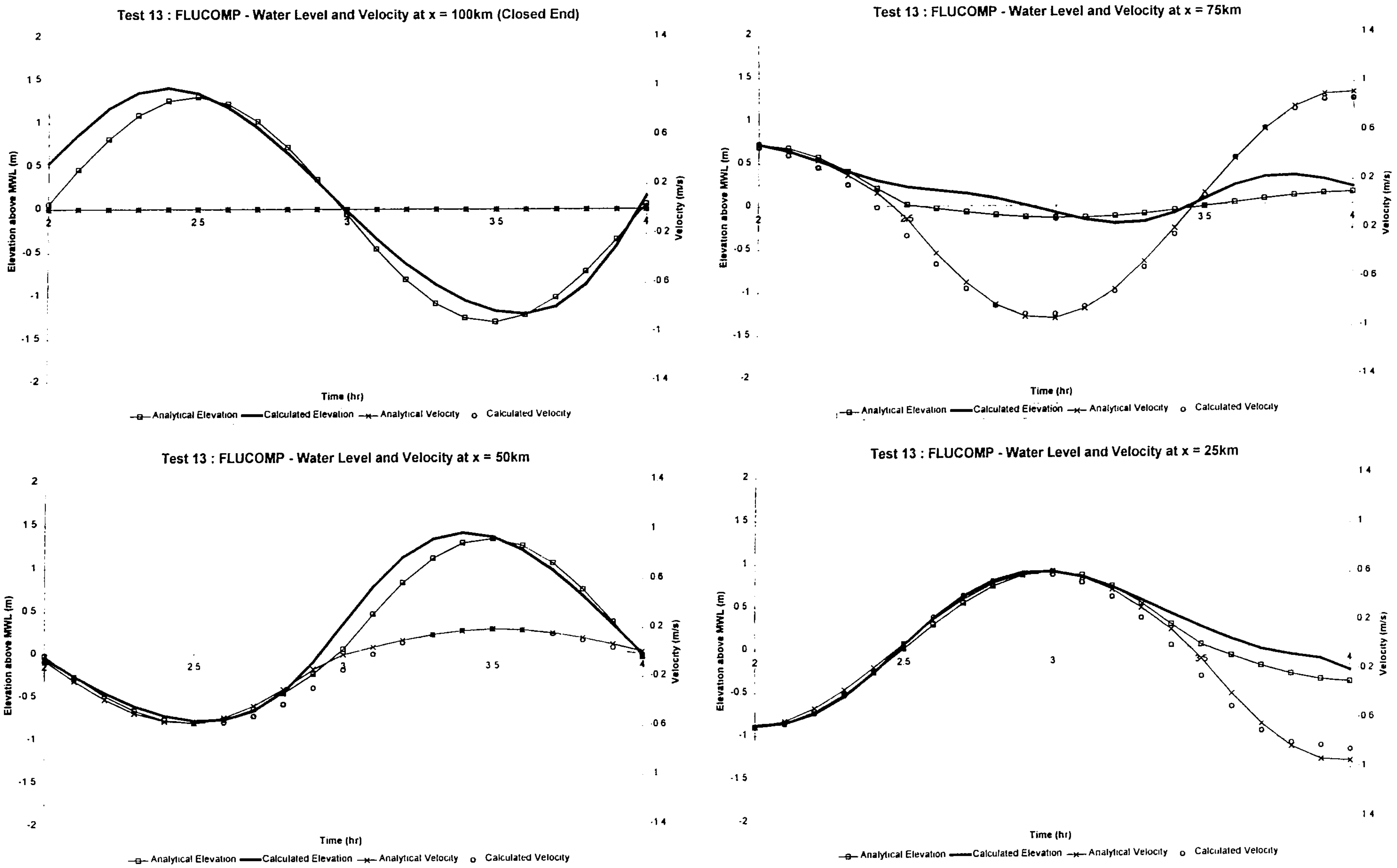

Test 13 : HYDRO-1D - Water Level and Velocity at t/T=1.25


Test 13 : HYDRO-1D - Water Level and Velocity at t/T=1.5

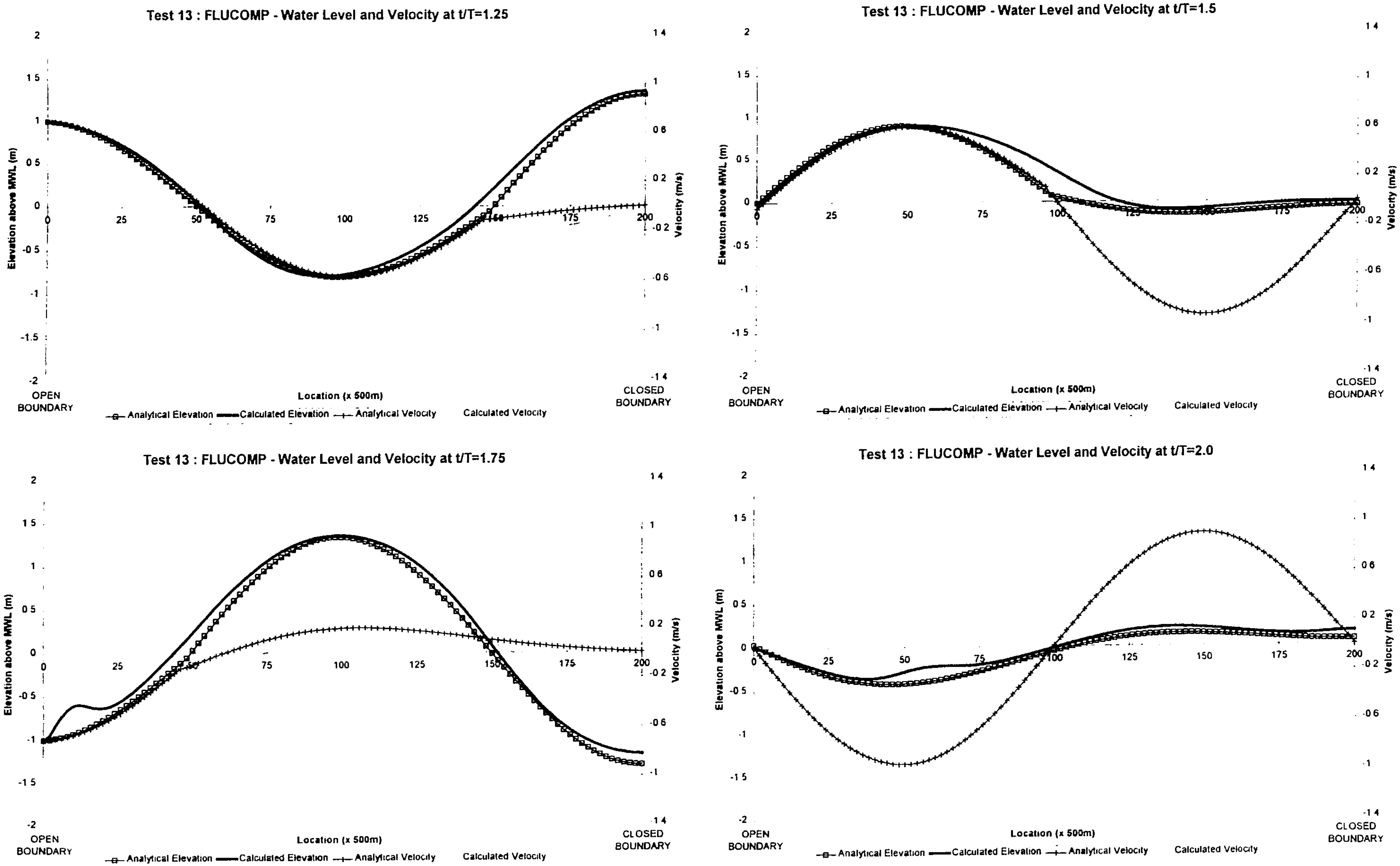

Test 13 : HYDRO-1D - Water Level and Velocity at t/T=1.75


Test 13 : HYDRO-1D - Water Level and Velocity at t/T=2.0

GRAPH 6 : *Test 13 : HYDRO-1D - Calculated velocities and water elevations at fixed times.*



GRAPH 7 : *Test 13 : FLUCOMP - Calculated velocities and water elevations at fixed locations.*

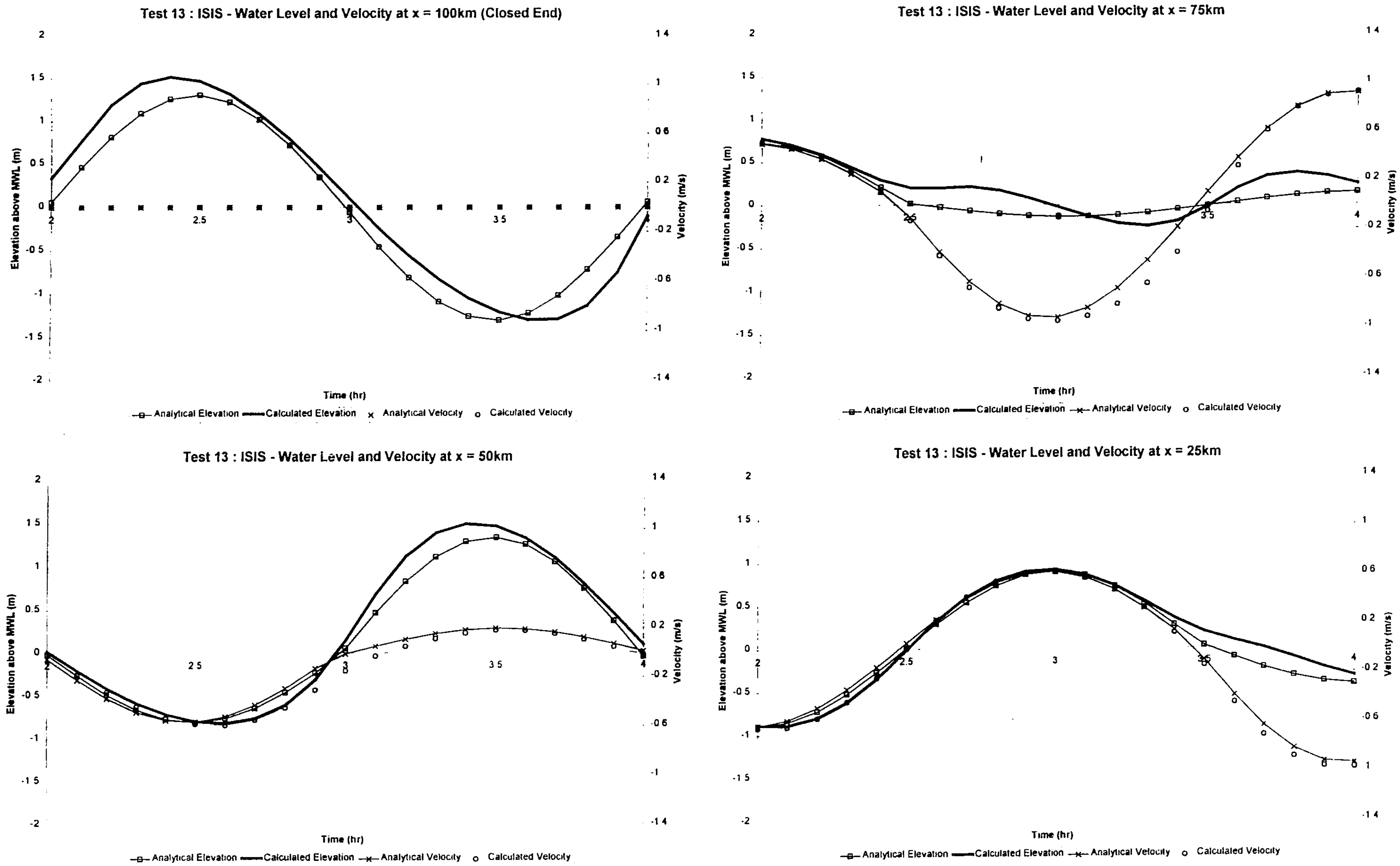

Test 13 : FLUCOMP - Water Level and Velocity at t/T=1.25

Test 13 : FLUCOMP - Water Level and Velocity at t/T=1.5

Test 13 : FLUCOMP - Water Level and Velocity at t/T=1.75

Test 13 : FLUCOMP - Water Level and Velocity at t/T=2.0

GRAPH 8 : *Test 13 : FLUCOMP - Calculated velocities and water elevations at fixed times.*

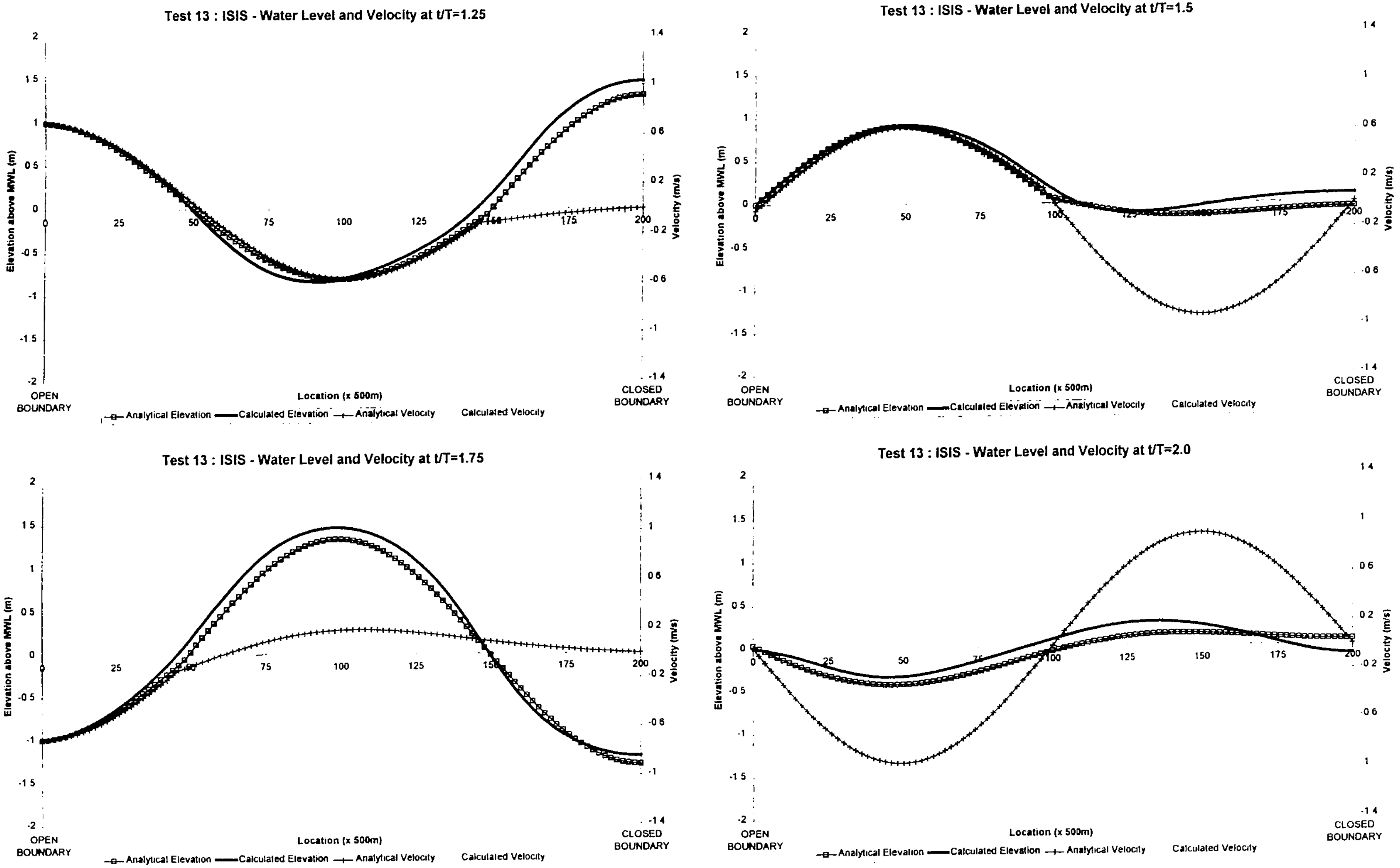

Test 13 : ISIS - Water Level and Velocity at x = 100km (Closed End)

Test 13 : ISIS - Water Level and Velocity at x = 75km

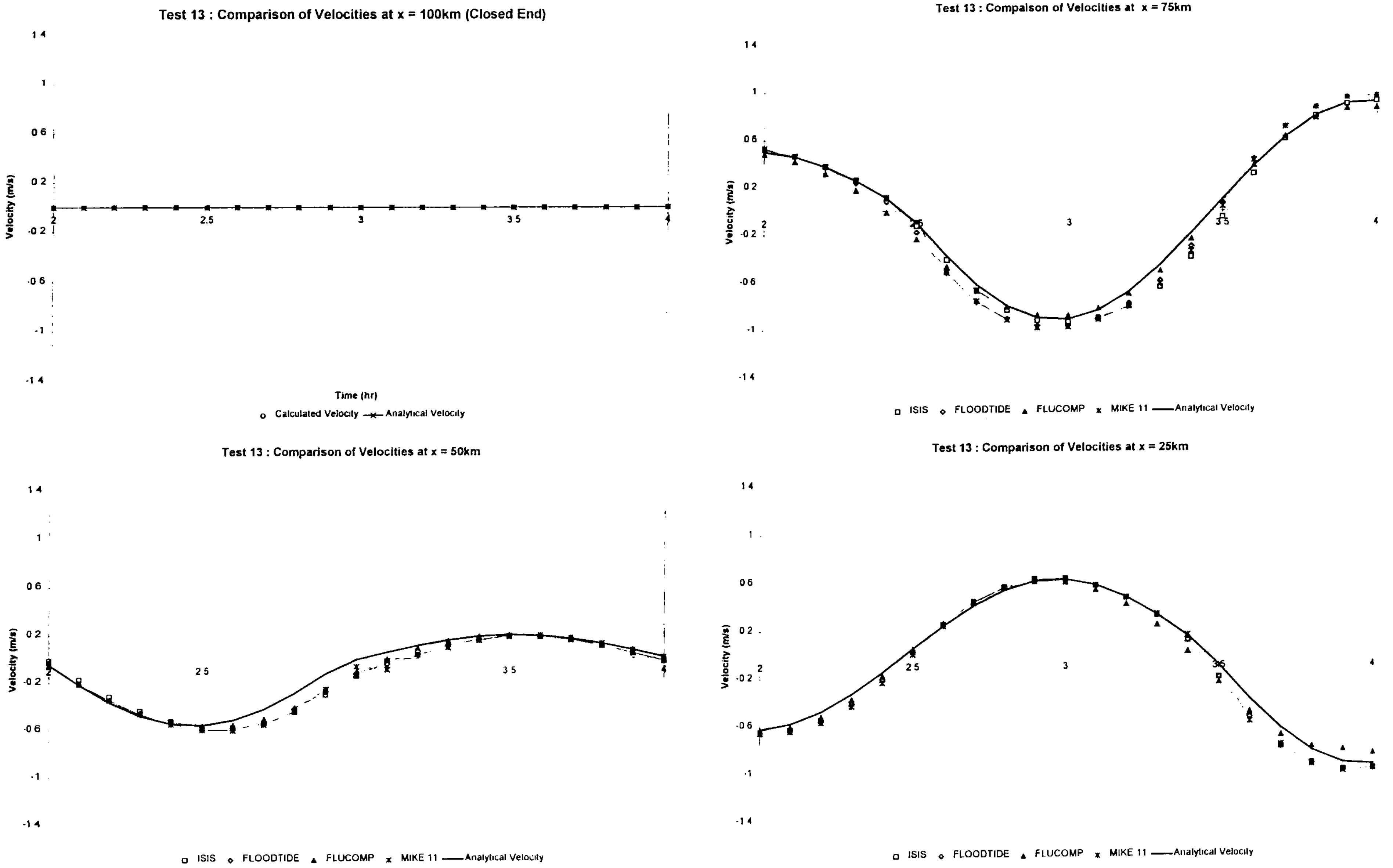
Test 13 : ISIS - Water Level and Velocity at x = 50km

Test 13 : ISIS - Water Level and Velocity at x = 25km

GRAPH 9 : *Test 13 : ISIS - Calculated velocities and water elevations at fixed locations.*



GRAPH 10 : *Test 13 : ISIS - Calculated velocities and water elevations at fixed times.*

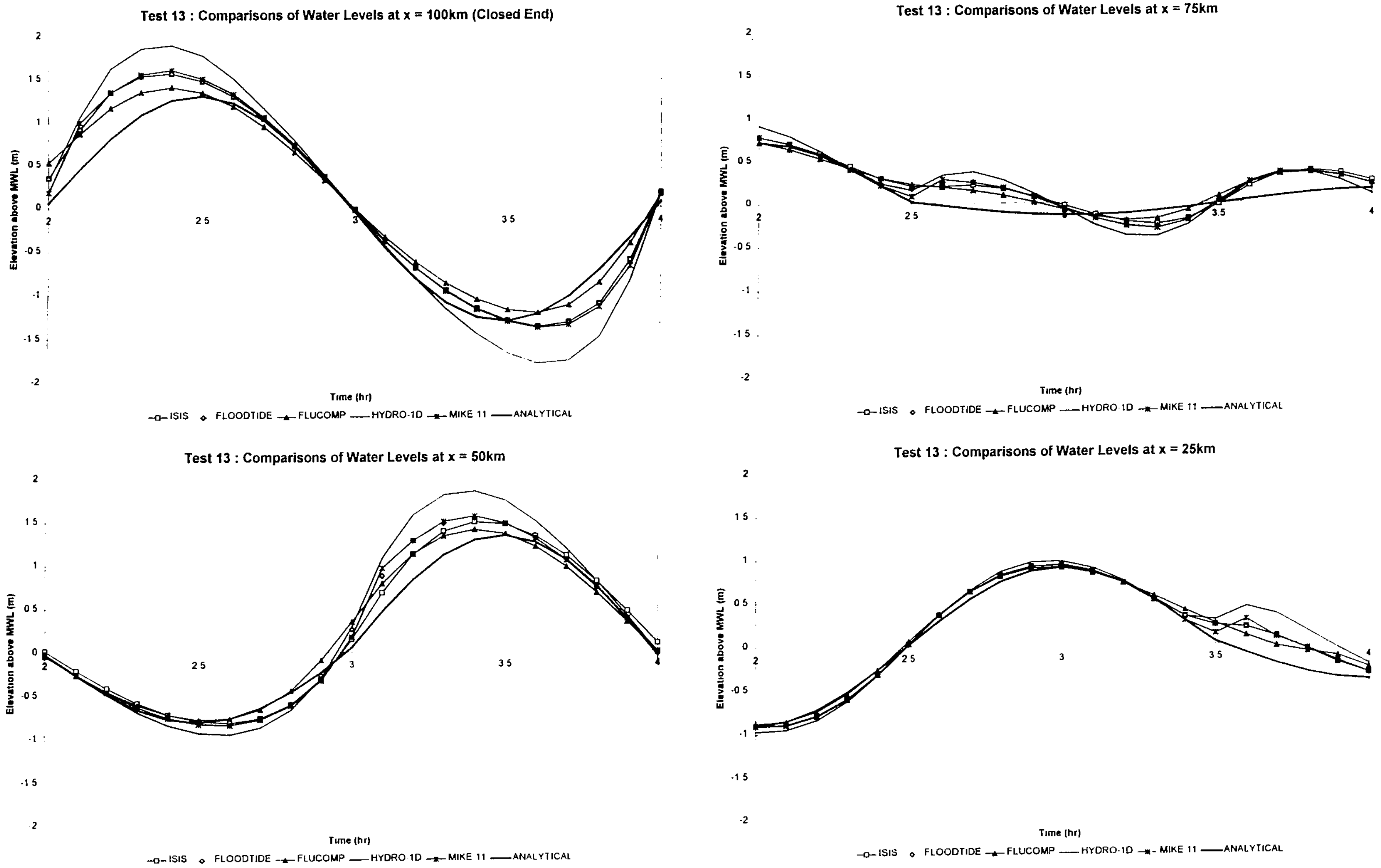

Test 13 : Comparison of Velocities at x = 100km (Closed End)

Test 13 : Compalson of Velocities at x = 75km

Test 13 : Comparison of Velocities at x = 50km

Test 13 : Comparison of Velocities at x = 25km

GRAPH 11 : *Test 13 : Comparison of calculated velocities at fixed locations.*

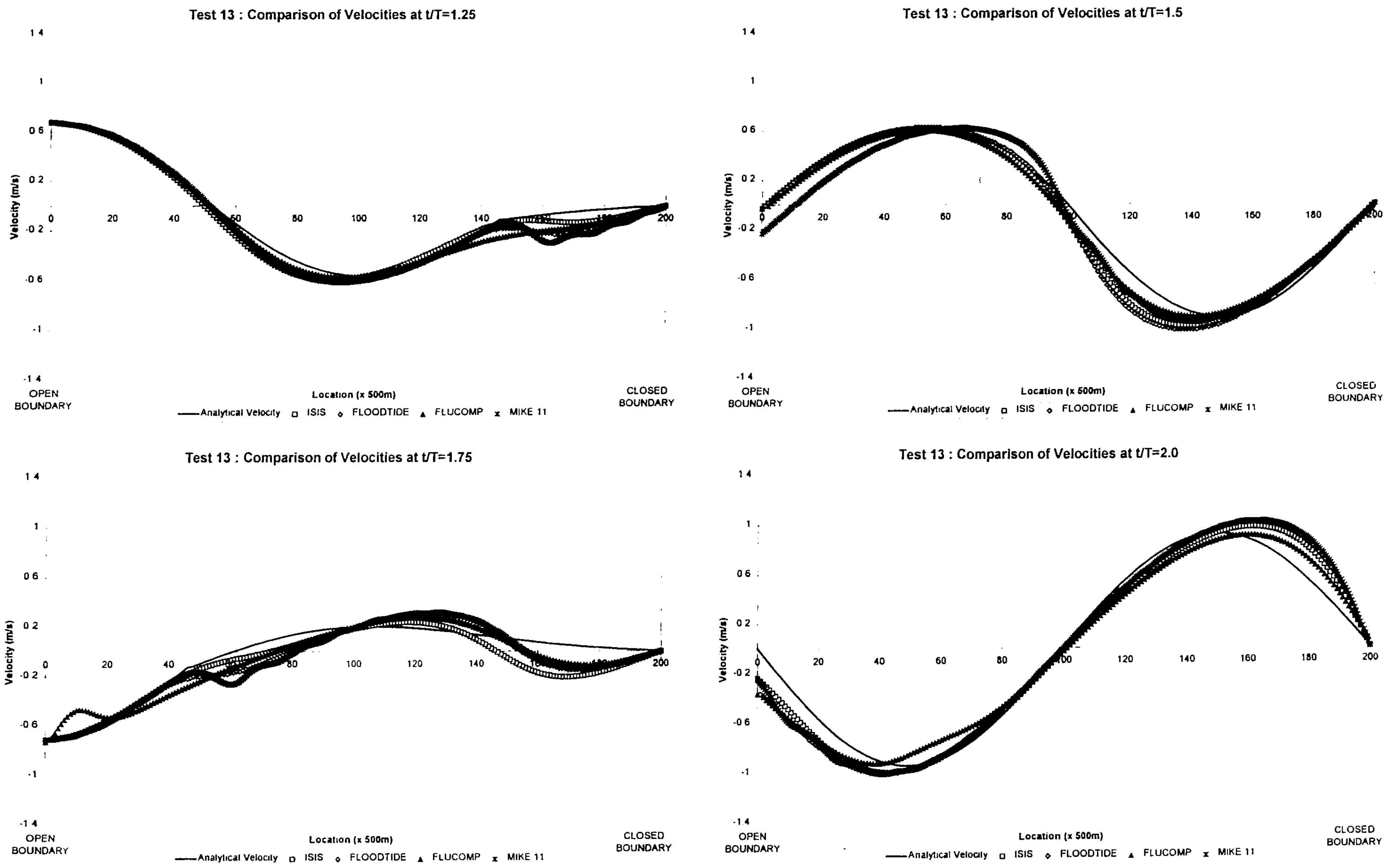

Test 13 : Comparisons of Water Levels at x = 100km (Closed End)

Test 13 : Comparisons of Water Levels at x = 75km

Test 13 : Comparisons of Water Levels at x = 50km

Test 13 : Comparisons of Water Levels at x = 25km

GRAPH 12 : *Test 13 : Comparison of calculated water elevations at fixed locations.*

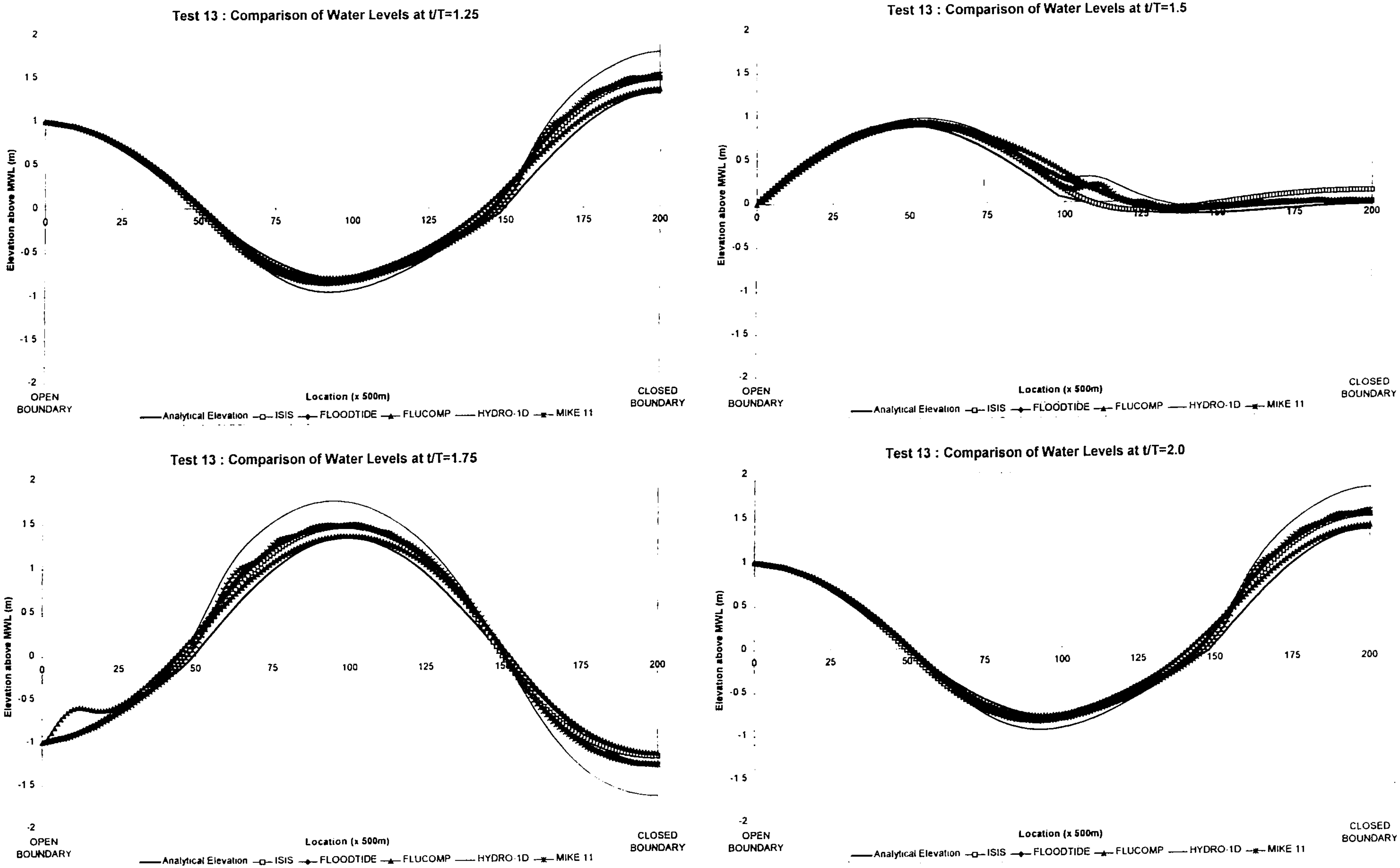

Test 13 : Comparison of Velocities at t/T=1.25

Test 13 : Comparison of Velocities at t/T=1.5

Test 13 : Comparison of Velocities at t/T=1.75

Test 13 : Comparison of Velocities at t/T=2.0

GRAPH 13 : *Test 13 : Comparison of calculated velocities at fixed times.*



Test 13 : Comparison of Water Levels at t/T=1.25

Test 13 : Comparison of Water Levels at t/T=1.5

Test 13 : Comparison of Water Levels at t/T=1.75

Test 13 : Comparison of Water Levels at t/T=2.0

GRAPH 14 : *Test 13 : Comparison of calculated water elevations at fixed times.*