University of Southern Queensland
Faculty of Engineering and Surveying

Advantages and Practicality of using a 2D Hydrodynamic Model in Comparison to a 1D Hydrodynamic Model in a Flood Prone Area

A dissertation submitted by

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ABSTRACT

This project investigates the advantages and practicality of using a 2D hydrodynamic model in comparison with a 1D hydrodynamic model for a flood plain area within Southern Queensland. The project aims to make progress towards determining when a 2D hydrodynamic is necessary and when a 1D hydrodynamic model would suffice.

Three hydrodynamic models were created for an effective comparison, a simple 1D model, a refined 1D model and a 2D model. The chosen location for the project was a 43 hectare flat site subject to flooding from multiple sources including the downstream Mooloolah River. Flooding was only considered in this project from the local upstream catchment. Flows for the models were generated using the RAFTS runoff-routing software package. The 1D hydrodynamic modelling was undertaken using the MIKE11 river and creek system hydrodynamic modelling package. The TUFLOW 2-dimensional hydrodynamic floodplain software was utilised for 2D modelling. Model comparisons were analysed using the 1, 2, 5, 10, 20, 50 and 100 year Average Reoccurrence Interval (ARI) events for the local upstream catchment of the chosen site.

It was found that for conditions where the flows were contained within a well defined channel, 1D hydrodynamic modelling is an effective method of representing flood characteristics. However, when flows become more complex 2D hydrodynamic modelling provides a more complete indication of flooding extents and other characteristics. A significant finding of the project was the tendency for instabilities in the 1D model when there are multiple branches in close proximity to each other.

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ENG4111 & ENG4112 *Research Project*

Limitations of Use

CERTIFICATION

I certify that the ideas, designs and experimental work, results, analyses and conclusions set out in this dissertation are entirely my own effort, except where otherwise indicated and acknowledged.

I further certify that the work is original and has not been previously submitted for assessment in any other course or institution, except where specifically stated.

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27/10/2010

Date

ACKNOWLEDGMENTS

I would like to thank Mark Gibson of MRG Water Consulting for providing the majority of software and all the data required for the project. Many thanks also to Darren Rogers of Storm Water Consulting for the continuing support, sound advice and allowing me the flexibility to work on this project as needed. Bill Syme of BMT WBM generously provided a license for TUFLOW to complete the required 2D hydrodynamic modelling.

Personal thanks must be given to my wife Naomi. Naomi deserves as much credit for this project as myself due to her unwavering support and patience as I have undertaken my academic pursuits.

Table of Contents

LIST OF FIGURES

LIST OF TABLES

1 INTRODUCTION

This dissertation contains all information relevant to the investigations undertaken by the author in relation to the comparison of a 1D and 2D hydrodynamic model of a flood prone area in South East Queensland. This project aims to give a clear understanding of the advantages and disadvantages of each model in relation to the specific site investigated.

1.1 BACKGROUND

South East Queensland is subject to much growth in the area of urban development (Department of Infrastructure and Planning 2010), certain locations of proposed and completed developments are located adjacent to or within flood plains and flood prone areas. Local Government generally requires an analysis be undertaken to determine the post development flood levels within an urban development and for the developer to demonstrate no adverse impacts are caused to neighbouring properties and the environment (City Policy & Strategy Division 2008). Determination of flood levels within urban developments often takes the form of computational hydraulic models of either the one-dimensional (1D) or two-dimensional (2D) type. 1D hydraulic models, which determine fluid flow perpendicular to defined cross sections, are often used, particularly if there is a defined flow path/channel and it is not foreseen that the water level will not exceed the level of the levee (Yang et al. 2007).

Increasingly 2D hydraulic models are utilised to determine flood characteristics of urban development. 2D hydraulic models are generally hydrodynamic, that is they calculated fluid flow over time. 2D hydrodynamic models calculate flow in all directions within the x-y plane (WBM BMT 2007). The advantage of a 2D hydrodynamic model is that location, extent and level of flows outside the defined channel (break out) can be determined, flood plains with complex flow characteristics can be modelled and flow path interaction can be simulated (Syme 2006).

1D hydraulic models can also take the form of a hydrodynamic (flow over time) model. A well defined channel, thorough investigation of the channel network and determination of possible break out points based on topography can lead to a 1D hydrodynamic being, potentially, as effective as a 2D hydrodynamic model.

As a result, it is not always entirely clear from an initial review of a site whether a 1D or 2D hydrodynamic model is more appropriate for a flood prone area. Therefore, this project aims to compare the model set up and model outputs from a 1D and 2D hydrodynamic model of the same area and determines which model would be more appropriate to use. The site investigated in this study is located at 166 Parklands Boulevard, Meridan Plains, Queensland.

1.2 CHARACTERISTICS OF THE CHOSEN SITE

The 43 hectare (ha) site at 166 Parklands Boulevard is a flat lot varying in height from RL 11 m AHD at the highest point to below RL 2 m AHD at the lowest point. Figure 1.1 provides an aerial view of the undeveloped site. Generally the slope on the site is less than 1%. The site has three major flow paths running from the south to north of the allotment. The western most flow path conveys water from an upstream lake that acts as a detention structure. That water then flows to the thickly vegetated neighbouring property. The neighbouring property provides storage and slows flows within the flood plain. The neighbouring property, therefore, has a significant impact on flooding characteristics within the flood plain. The central most flow path conveys runoff from almost the entire eastern portion of the upstream catchment. These flows go through another detention structure before entering the site. The central flow path runs along the western boundary of the site. From a site inspection carried out by the author it was evident that the central and western most flow paths will interact during major flood events. On the eastern part of the site there is another flow path which conveys a small portion of the eastern part of the upstream catchment. Figure 1.2 displays the topographic conditions of the site.

Upon inspection of Figure 1.2 it can be seen that there is potential for considerable interactions between the flow paths during flood events. This is confirmed further in this report. The significant potential for these paths to interact provides a justification for hydrodynamic modelling to be undertaken and for 2D hydrodynamic to be considered.

The site is subject to flooding from three sources. The sources of flooding are:

- Discharges from the upstream Catchment
- Backwater from Currimundi Creek
- Backwater from the Mooloolah River

Tailwater (downstream boundary) conditions can be included in either 1D or 2D hydrodynamic models in the form of a constant water level, constant discharge, a discharge-height relationship or hydrographs (water level or discharge over time) (MIKE by DHI 2009, WBM BMT 2007). For this project the height-discharge relationship was chosen to be used as the downstream boundary condition for the model. Though the site is located close to the Mooloolah River and Currimundi Creek and may be subject to backwater (flood water from the river's flood plain) from them the height discharge relationship boundary was calculated using the slope and roughness values from within the model. This was done for simplicity as the calculation of water levels within the Mooloolah River or Currimundi Creek is outside the scope of this project. Also, the consideration of backwater would have been the dominant element resulting in the majority of the flooding in the model. Therefore an auto generated height discharge relationship was used for both the 1D and 2D models and hydrologic calculations were only undertaken for the catchment upstream of the site. . Figure 1.3 and Figure 1.4 provide the location of the site relevant to Currimundi Creek and the Mooloolah River respectively.

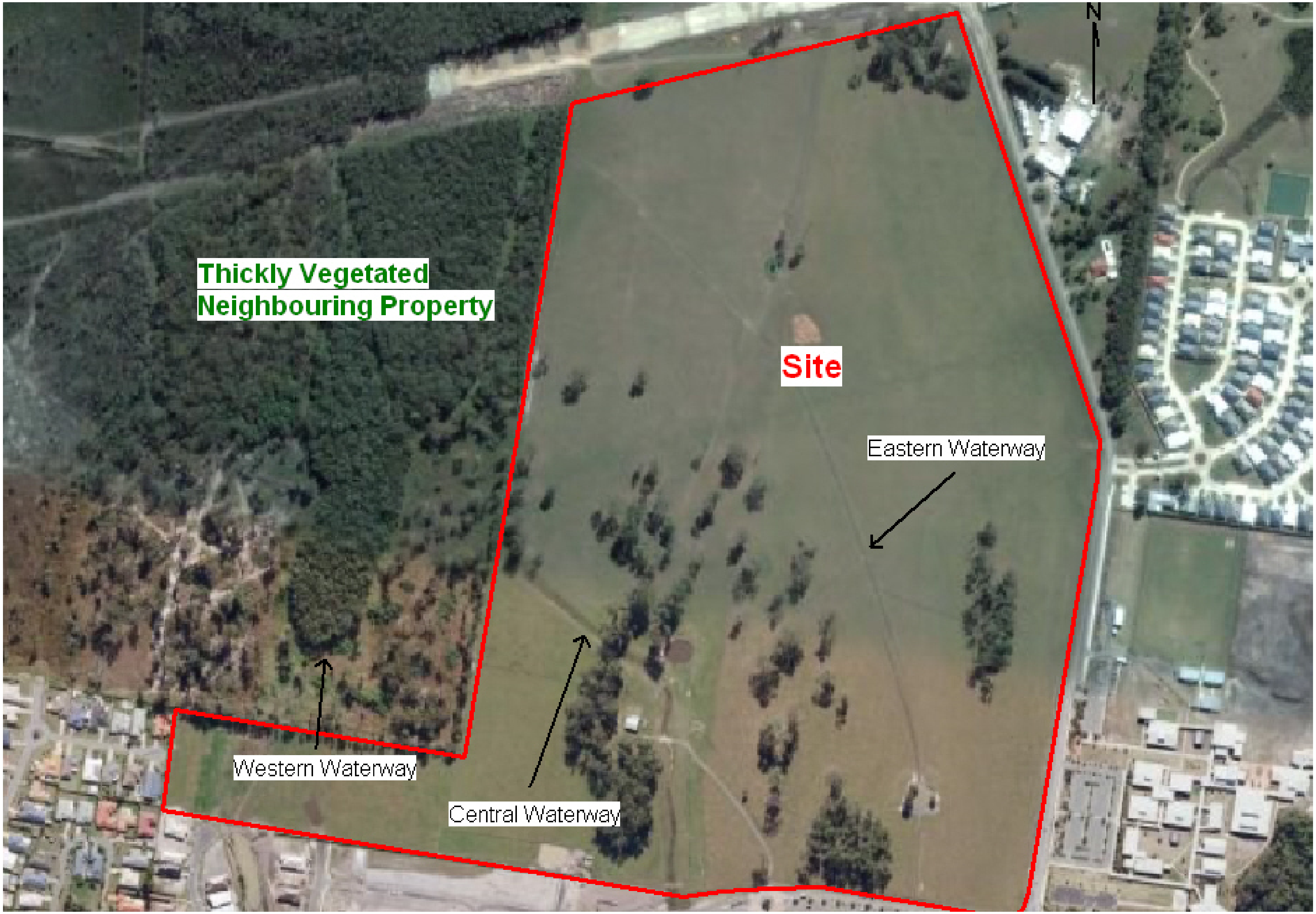

Figure 1.1 – Site Aerial Photograph

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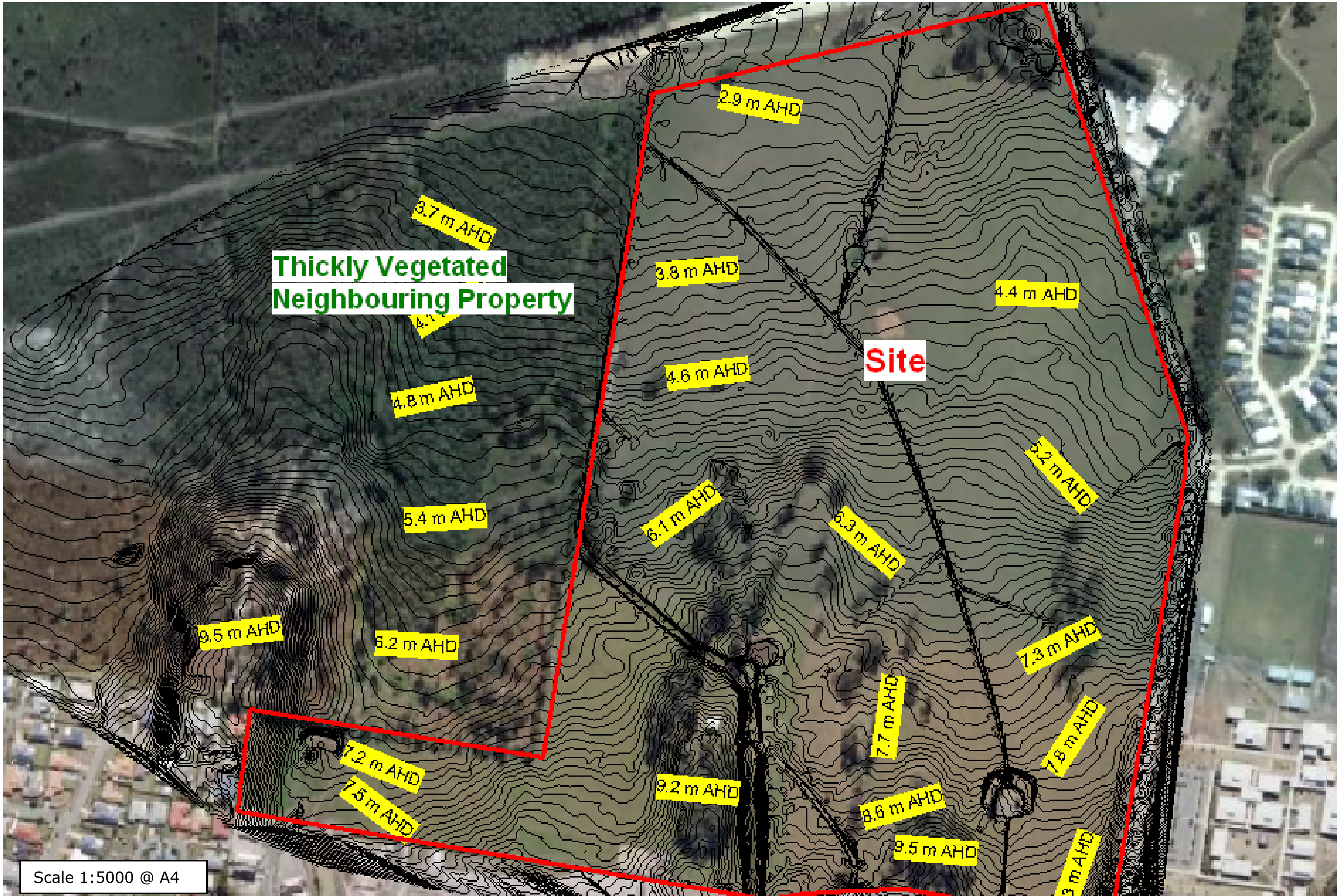

Figure 1.2 – Site Topographic Conditions

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Figure 1.3 – Site Locality (Currimundi Creek)

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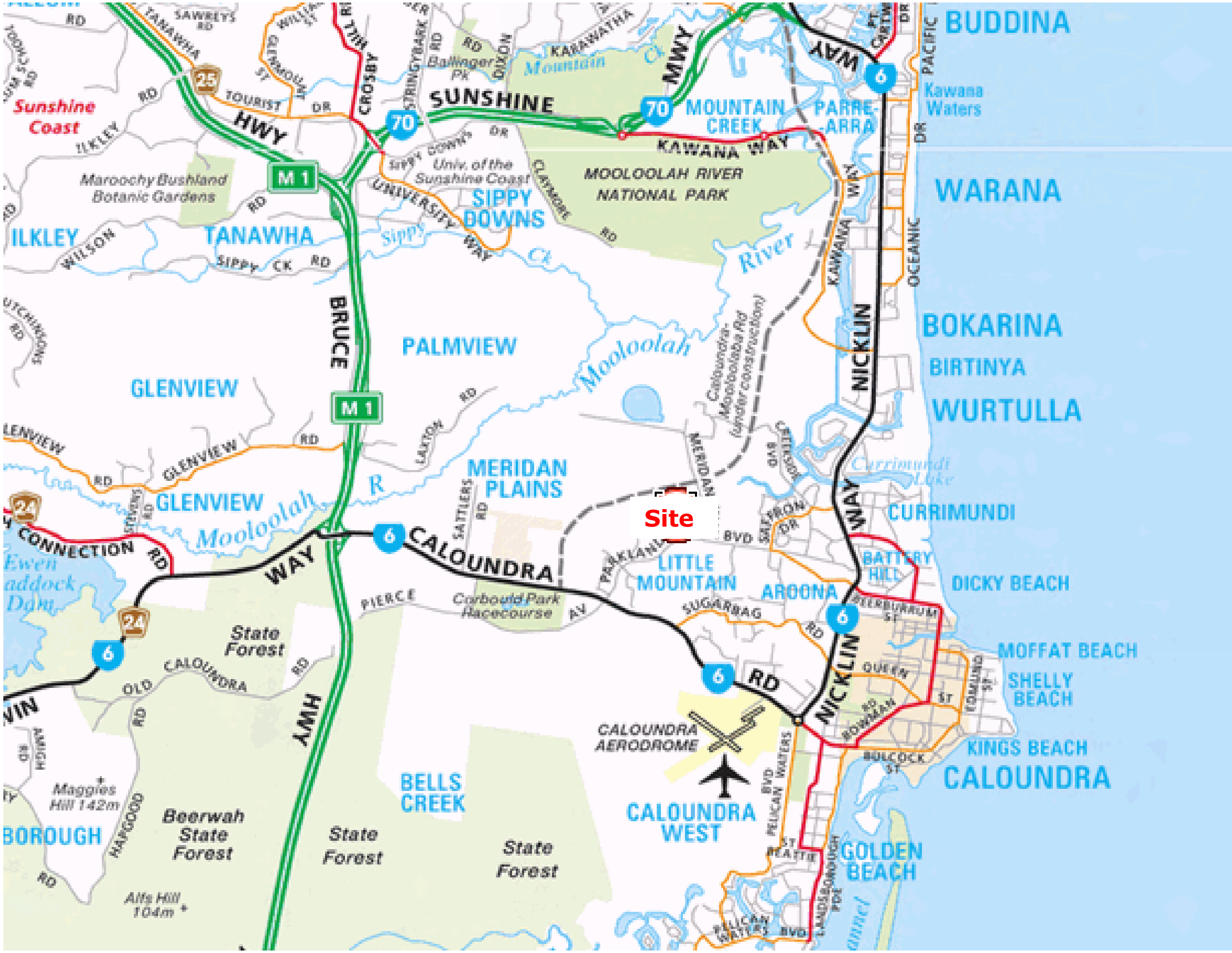

Figure 1.4 – Site Locality (Mooloolah River)

1.3 PROJECT TASK STATEMENT

This research project investigates the advantages and practicality of utilising 2D hydrodynamic modelling for a proposed urban development in a flood prone area compared to a 1D hydrodynamic model.

1.4 PROJECT OBJECTIVES

The project objectives are to demonstrate the effectiveness of representing flood characteristics in an urbanised area using a 2D hydrodynamic model in comparison with a 1D hydrodynamic model in terms of both flood inundation extent and flood level. Secondly, to demonstrate the practicality of setting up and utilising a 2D hydrodynamic model in comparison with a 1D hydrodynamic model comprising of multiple 1D branches.

Two 1D hydrodynamic models will be set up. The first will consist of a simple model representing the major flow paths and the second will be set up based on the 2D hydrodynamic model output.

1.5 ANTICIPATED PROJECT OUTCOMES

Prior to undertaking this study it was envisaged that the outcomes of this project would demonstrate that a 2D hydrodynamic model is more powerful in predicting the inundation extents of flood events in flood prone areas, in addition to calculating flood characteristics with a greater spatial variance than a 1D model. Furthermore, it was anticipated that the results of this project would demonstrate that a 1D hydrodynamic model with networks representing the possible flood paths will provide results somewhat similar to a 2D hydrodynamic model. However due to an increased potential for human error, the results would be potentially less reliable. Furthermore, it was anticipated that the 1D hydrodynamic with multiple branches would be more cumbersome to construct than the pure 2D hydrodynamic.

1.6 PROJECT RESOURCES

Resources that were utilised in this project consisted of project specific software including:

- Licenses and software locks;
- A computer capable of running the required software and reading the data; and
- Project data supplied by MRG Water Consulting.

The project specific software utilised in this project consists of MapInfo, XP-RAFTS, MIKE11, MIKEVIEW, TUFLOW and SMS. MRG Water Consulting has provided the required software and the associated licenses and software locks for this project and arrangements were been made in such a way that these software packages were available to the author. The exception to this was the TUFLOW License provided by BMT WBM. Table 1.1 below contains a list of the software and the relevant use of this software in regards to the project.

Table 1.1 – List of Software Used in Project

Software	Use
MapInfo	Mapping software used to display and topographic information. Also used in the generation of the 3D DTM model for 2D hydrodynamic modelling.
XP-RAFTS	Generates discharge hydrographs from location specific rainfall and catchment information. The discharge hydrographs are used for boundary conditions in hydrodynamic modelling.
MIKE11	1D hydrodynamic modelling software.
MIKEVIEW	Result display software for MIKE11 output.
TUFLOW	2D hydrodynamic modelling software.
SMS	Graphical results display software for 2D hydrodynamic modelling software.

1.7 PROJECT TASKS

The tasks required to carry out this project were very specific. This was an advantage in planning and preparing the required work, as the tasks could be itemised therefore providing a framework for specific goal setting. The list below contains the tasks required to carry out this project. Naturally all of the tasks listed each have a sub set of tasks which will be described in further detail in subsequent chapters of this dissertation.

1. *Hydrologic Calculations:* Preliminary hydrologic calculations were done to determine the peak discharge on the site for storms that would statistically occur from 1 in a year to 1 in every 100 years.
2. *Hydrologic Calculations:* This step was undertaken to generate catchment specific discharge-time relationships (hydrographs) to enter into the hydrodynamic models as inflows.
3. *Topographic Investigation:* The topography of the site was inspected from visits to the site by the author and inspection of ground level information. Ground level information used in this study was in the form of ground level contours from Council and detailed professional survey provided by MRG Water Consulting.
4. *Simple 1D Hydrodynamic Model:* A simple 1D model was set up to gain an initial understanding of the flood characteristics on the site. The model was originally set up by the author for commercial purposes but has been adjusted to specifically apply to this study. Initially the eastern flow path was ignored as the flows were considered relatively small compared to the other major flow paths. Therefore all flows were entered into the western and central flow paths.
5. *Digital Terrain Model (DTM) Preparation:* This was a significant step in the study. A DTM is required for the 2D hydrodynamic modelling. Great detail must be taken to ensure the DTM has data that is as close as possible to the actual ground level so as to most effectively represent the topographic conditions in the 2D model.

6. *2D Hydrodynamic Model:* Modelling of the subject site with a 2D hydrodynamic model to assess the difference between the simple model and the 2D output.
7. *Further 1D Hydrodynamic Modelling:* A second 1D model was constructed base on the observed output from the 2D model to ascertain if the differences in the first 1D model and the 2D were a result of the simplistic approach or differences due to the type of model (i.e. 1D or 2D)
8. *Documentation:* Includes the preparation of the Dissertation.

1.8 RISK ASSESSMENT

The risks associated with the activity are minimal from a physical point of view.

The risks of damage are somewhat more prevalent as the computer software, hardware and data files could be damaged. The strategies for avoiding these circumstances are as follows;

- All data to be used in the project was copied from MRG Water Consulting's archives to an external computer hard drive that is stored at the home of the author. Therefore if any data is lost or corrupt there is no loss to MRG Water Consulting and the data could be re obtained.
- Software for the project was provided by MRG Water Consulting and installed on the author's personal computer. Therefore if there were any problems with software they would be unique only to one system.
- As much as practically possible the computer hardware of the author was used.

1.9 DISSERTATION STRUCTURE

The dissertation consists of five main sections namely; the literature review, methodology- hydrology, methodology hydrodynamic, results discussion and conclusions.

Literature Review

An overview of current literature in the field of 1D and 2D hydrodynamic modelling is included within this section of the dissertation. It presents information regarding the calculation methods and solutions schemes of the different hydrodynamic models utilised in this dissertation.

Methodology – Hydrology

This section contains information regarding the methodology utilised in determining the inflow boundaries for the hydrodynamic modelling. Rational Method calculations were undertaken to determine the peak flows entering the site for the chosen rainfall events. A RAFTS hydrologic model was then set up and the parameters within the RAFTS model were then adjusted to match the Rational Method peak discharges.

Methodology – Hydrodynamic Modelling

The Hydrodynamic Modelling section contains information regarding the methodology utilised in creating the three hydrodynamic models. Discharge hydrographs calculated from the RAFTS model were included as upstream boundary conditions in all hydrodynamic modelling. Both the simple and refined 1D hydrodynamic models were created using MIKE 11. The 2D hydrodynamic model was created utilising TUFLOW. Significant investigation was carried out on the creation of a 3D surface to represent the terrain in the 2D model.

Results Discussion

Presents result comparisons predominantly of the 100 year ARI event for the three hydrodynamic models. The comparison locations for all hydrodynamic models were taken as the same as the cross section locations for both 1D models.

Conclusions

This section presents the overall summary of the dissertation with future recommendations.

2 LITERATURE REVIEW

2.1 INTRODUCTION

The literature reviewed for this project comprised largely of literature containing technical information about the 1D and 2D hydrodynamic modelling software considered. Other literature containing relevant information has also been discussed below.

2.2 1D HYDRODYNAMIC MODELLING

1D hydraulic analysis assumes fluid flow perpendicular to cross sections of a flow path. Parameters such as bed resistance, bed slope, tailwater (downstream boundary) and inflow, geographic characteristics and structures within the flow path all contribute to the flow characteristics in a 1D hydraulic model (Hydrologic Engineering Center 2010, MIKE by DHI 2009). In the case of the site for this project all of these parameters will need to be considered.

MIKE11 is the model to be utilised for the projects 1D modelling. MIKE11 is a 1-dimensional time variant (hydrodynamic) model developed by the DHI group based in Denmark. It is world renowned hydraulic modelling software. This 1D hydrodynamic software is capable of calculating flood characteristics for waterways from small urban waterways to large river and creek systems. MIKE11 has the ability to model flood storage within a flood plain along with conveyance through the main channel.

The MIKE11 model used to simulate the 1D flows for the site will be run in the hydrodynamic mode using the Saint Venant equations to calculate flows within the channel and flood plain. The model is hydrodynamic as it considers the motion of the water in the vertical direction (change in water levels) and the changes in velocity and discharge with time (Tsanis et al. 2006, MIKE by DHI 2009).

The Saint Venant equations used in the MIKE11 hydrodynamic simulation undertaken were derived assuming the water to be incompressible and homogenous, a small bed slope, large wave lengths compared to water depth and sub-critical flow. The Saint Venant equation variations for the MIKE11 model used are as follows;

For the Conservation of Mass:

$$\frac{\partial(A)}{\partial t} + \frac{\partial(Q)}{\partial x} = q$$

For the Conservation of Momentum:

$$\frac{\partial Q}{\partial t} + \frac{\partial\left(\alpha \frac{Q^2}{A}\right)}{\partial x} + gA\frac{\partial h}{\partial x} + \frac{gQ|Q|}{C^2 AR} = 0$$

Where:

A=total cross sectional area (m^2)

t=time (sec)

Q=discharge (m^3/s)

q=lateral inflow (m^2/s)

h=stage (height) above datum (m)

x=distance parallel to the flow (m)

α=velocity distribution coefficient

g=gravitational acceleration (m/s^2)

R=hydraulic radius (m)

C=Chezy coefficient, which is related to Manning's 'n' roughness by $C = \frac{R^{\frac{1}{6}}}{n}$

Of particular note is the assumption of sub-critical flow. The MIKE11 solution scheme is not designed for supercritical flow but does make allowance for supercritical flow in order to achieve smooth simulation of the generally subcritical flow (MIKE by DHI 2009, p. 183). The software recognises when flow conditions change from sub-critical to supercritical by use of the Froude number and adopts a reduced momentum equation:

$$\frac{\partial Q}{\partial t} + gA\frac{\partial h}{\partial x} + \frac{gQ|Q|}{C^2AR} = 0$$

The solution scheme for the 1D hydrodynamic model to be used for the site consists of h-points (location where the water level is determined) at the cross sections and Q-points (locations where the discharge is determined) midway between each h-point. The Saint Venant equations are then implicitly solved for each time step using the finite difference method (MIKE by DHI 2009). Below gives a schematic representation of the MIKE11 solution scheme taken from the MIKE11 reference manual where the points labelled 'h' are cross section locations.

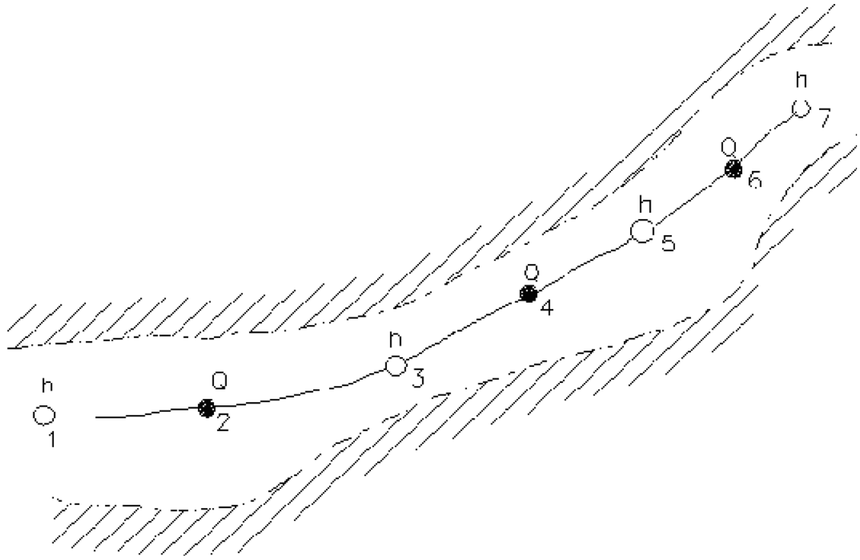

Figure 2.1 – Schematic Representation of MIKE11 Solution Scheme

Delis et al. (2000) mentioned that finite difference methods for solving the Saint Venant equations contains a weakness in calculating discontinuous flow and significant changes in discharges. This point is not relevant to the analysis which is to be undertaken for this project as the inflow hydrographs to be calculated for the upstream boundaries will not have abrupt changes in magnitude.

2.3 2D HYDRODYNAMIC MODELLING

The 2D hydrodynamic model used to assess the flow conditions for the site is TUFLOW. TUFLOW was developed by Bill Syme of BMT WBM in conjunction with the University of Queensland (WBM BMT 2007). TUFLOW is widely used in Australia but is also used in the United Kingdom and is recognised throughout the world. TUFLOW calculates surface in the x-y plane based on a 3D digital terrain model (DTM) surface roughness and structures within the DTM or model space. Depth averaged shallow water equations which calculate fluid flow for long waves with respect to depth are used for the calculations of fluid characteristics within the TUFLOW model. The equations are solved using a finite difference method (Syme 1991). WBM BMT (2007, pp. 1.2-1.3) describes the formulae used in the depth averaged shallow water equations as follows:

For the Conservation of Mass

$$\frac{\partial \zeta}{\partial t} + \frac{\partial (Hu)}{\partial x} + \frac{\partial (Hv)}{\partial y} = 0$$

For the Conservation of Momentum:

$$\frac{\partial u}{\partial t} + u\frac{\partial u}{\partial x} + v\frac{\partial u}{\partial y} - c_f\, v + g\frac{\partial \zeta}{\partial x} + g\, u\frac{\sqrt{u^2+v^2}}{C^2 H} - \mu\left(\frac{\partial^2 u}{\partial x^2} + \frac{\partial^2 u}{\partial y^2}\right) + \frac{1}{\rho}\frac{\partial p}{\partial x} = F_x$$

$$\frac{\partial v}{\partial t}+u\frac{\partial v}{\partial x}+v\frac{\partial v}{\partial y}+c_f u+g\frac{\partial \zeta}{\partial y}+g\,v\frac{\sqrt{u^2+v^2}}{C^2 H}-\mu\left(\frac{\partial^2 v}{\partial x^2}+\frac{\partial^2 v}{\partial y^2}\right)+\frac{1}{\rho}\frac{\partial p}{\partial y}=F_y$$

Where:

ζ =Water Surface elevation (m)

u =velocity in the x direction (m/s)

v =velocity in the y direction (m/s)

H =depth of water (m)

t =time (sec)

x =distance in the x direction (m)

y =distance in the y direction (m)

c_f =coriolis force coefficient

C =Chezy coefficient

p =atmospheric pressure (Pa)

μ =horizontal diffusion of momentum coefficient

ρ =density of water (kg/m^3)

F_x =sum of external forces in the x direction (N)

F_y =sum of external forces in the y direction (N)

The additional parameters accounted for in the TUFLOW model of note in comparison with the MIKE11 Saint Venant equations are the Coriolis force and the horizontal diffusion of momentum. This can be attributed to the 2D nature of the TUFLOW model.

The TUFLOW model determines flood depths at the centre mid-point on cell sides and corners of the grids defined in the model. Flow velocities are determined at the mid-points of the cell sides. The cell side mid points and the point at the cell centre determine the direction of flow. The information which is input into the model is associated with each cell including the surface elevation (WBM BMT 2007). Figure 2.2 is a schematic representation of the TUFLOW grid taken from the TUFLOW manual.

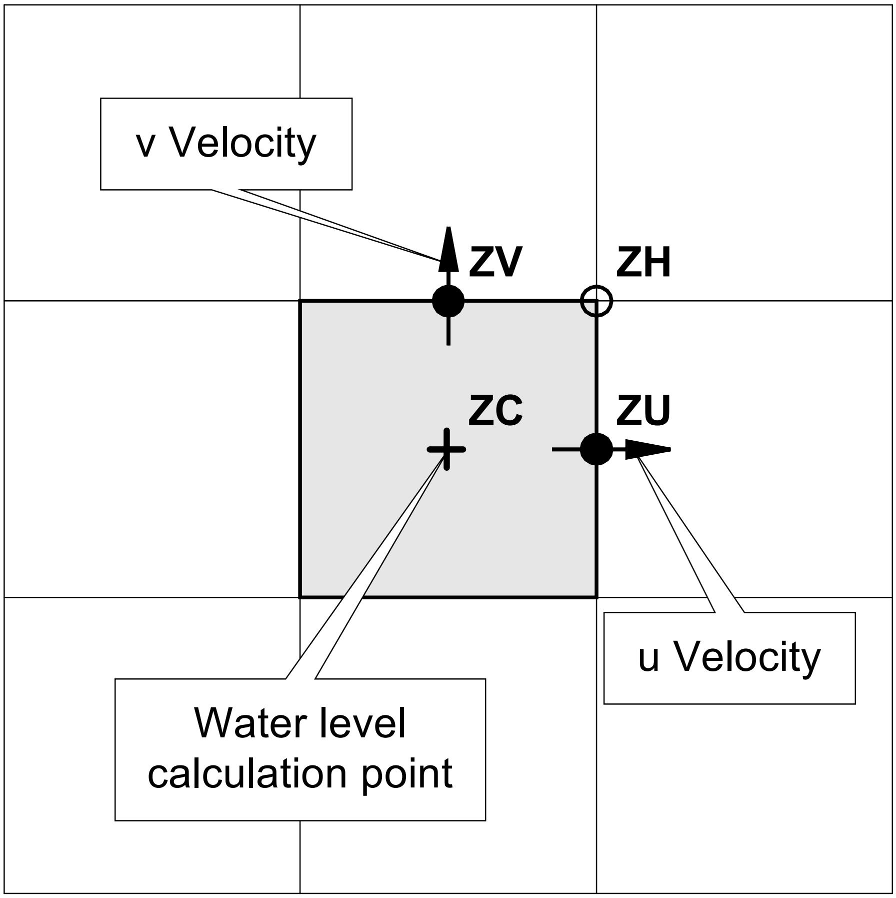

Figure 2.2 – Schematic Representation of TUFLOW Grid

2.4 SUMMARY

Both the 2D and 1D hydrodynamic model considered for this study appear to have detailed parameters and methods for determining the flow characteristics for flood plains and the structures contained therein. The power and accuracy for determining the flow characteristics within a flood plain by either a 2D or 1D model will be best assessed by testing rather than a study of the methods used for calculation.

3 METHODOLOGY – HYDROLOGY

3.1 THESIS METHODOLOGY

The methodology for the calculation of flood levels using 2D and 1D hydrodynamic modelling software for statistical rainfall events can be summarised in the following steps. The six steps below refer specifically to the calculations of the flood levels caused by flooding from the upstream catchment. Backwater from Currimundi Creek and the Mooloolah River was omitted for ease in comparison.

1) Calculate preliminary estimates of peak discharges generated by the site and upstream catchment for a range of average recurrence intervals (ARI).
2) Generate hydrographs for the site and upstream catchment that include catchment characteristics such as roughness, slope and flood storage.
3) Obtain topographic and structure information of the site and arrange the data to be utilised in the hydrodynamic models
4) Construct hydrodynamic model from the topographic and structure information using the hydrographs generated as upstream boundaries for the models.
5) Analyse the model outputs and determine the key differences.
6) If applicable determine strategies to ensure model outputs are similar in terms of extent and magnitude.

A comprehensive description of these methodologies is included in the following sections.

3.2 HYDROLOGY - GENERAL

To simulate flooding and runoff conveyance using either 1D of 2D hydrodynamic modelling requires boundary conditions both upstream and downstream. Downstream boundary conditions were automatically generated by both hydrodynamic modelling packages based on slope and Manning's 'n' roughness. Therefore no hydrologic calculations were undertaken for Mooloolah River or Currimundi Creek.

Upstream boundary conditions require the input of either rainfall or discharge hydrographs for both the 1D and 2D models (MIKE by DHI 2009, WBM BMT 2007). Local and state government regulations within Queensland require that floor levels within new urban developments have flood immunity for rainfall events with a statistically determined average recurrence interval (ARI) of, generally, 100 or 50 years plus a pre defined safety factor (Department of Natural Resources & Water et al. 2007, City Policy & Strategy Division 2008, Sunshine Coast Regional Council 2004). Therefore the upstream boundary conditions within the 1D and 2D models were calculated for a statistically recurring storm with an ARI of 100 years. Other ARI events were also considered (1, 2, 5, 10, 20 and 50 year ARI) for both hydrologic and hydrodynamic purposes but the primary event for comparison was the 100 year ARI event.

Calculation of discharge hydrographs using the RAFTS runoff routing software will be used in this project. Peak discharges and critical storm durations were fitted to 'Rational Method' peak discharge estimates according to the guidelines found in the Queensland Urban Drainage Manual (QUDM) (Department of Natural Resources & Water et al. 2007)

3.3 PRELIMINARY HYDROLOGIC CALCULATIONS

The initial calculation of peak discharges for the site and upstream catchment were undertaken using the 'Rational Method' for comparison of the runoff routing model as mentioned above and outlined in QUDM. Requirements for utilising the Rational Method to calculate peak discharges in QUDM include an urbanised catchment, catchment area less than 500 ha and a lack of significant storage within the catchment. The catchment contributing to flows within the site meets all three of the criteria mentioned above except there are two areas of significant flood storage within the catchment. However, the significant storage areas can be ignored at the initial calculations stages due to the limitations of the Rational Method and be included after the runoff routing model is manipulated to match the Rational Method peak discharges.

The catchment boundaries for the 159 ha site and upstream catchment were determined using topographic maps obtained from the Sunshine Coast Regional Council (SCRC) and MRG Water Consulting. Figure 3.1 displays the catchment boundaries for the site and upstream catchment. The peak stormwater discharges for the upstream catchment were calculated using the Rational Method at Point 1 shown on Figure 3.1. In reality Point 1 actually consists of the entirety of the northern catchment boundary. This is due to the multiple flow paths within the catchment. The Rational Method, however requires a discharge point and assumes the longest travel path for runoff in the calculation of peak discharges (Department of Natural Resources & Water et al. 2007).

The Rational Method calculations have been completed in accordance with the parameters recommended in the QUDM for the 1, 2, 5, 10, 20, 50 and 100 year ARI storm events for existing and developed site conditions. Table 3.1 contains a summary of the parameters used in the Rational Method Calculations. Appendix B contains full details of the rational method calculations.

Rainfall intensities for the Rational Method calculations are also displayed in Appendix B. Rainfall intensities were determined using the methods found in Australian Rainfall and Runoff (AR&R) (Canterford 1987, Pilgrim 1998) for Caloundra.

Table 3.1 – Rational Method Parameters

Parameter	Value	Comments/Description
Area	159 ha	Area of the site and upstream catchment combined.
*Time of Concentration	71 min (ex), 61 min (dev)	A combination of standard inlet time and pipe/channel travel time.
*Runoff Coefficient (C_{10})	0.73 (ex) 0.76 (dev)	Determined for individual land use throughout the catchment.

The upstream catchment includes regional detention basins to mitigate the effect of any future development. Therefore it was assumed that any undeveloped land in the upstream catchment would remain undeveloped from a peak discharge perspective. This is due to the presence of the regional basins. The currently developed areas in the upstream catchment were assumed as such in the calculations. Analysis of the regional detention basins will be undertaken in the hydrologic modelling stage.

Table 3.2 compares existing and developed peak discharges for the site and upstream catchment at Point 1.

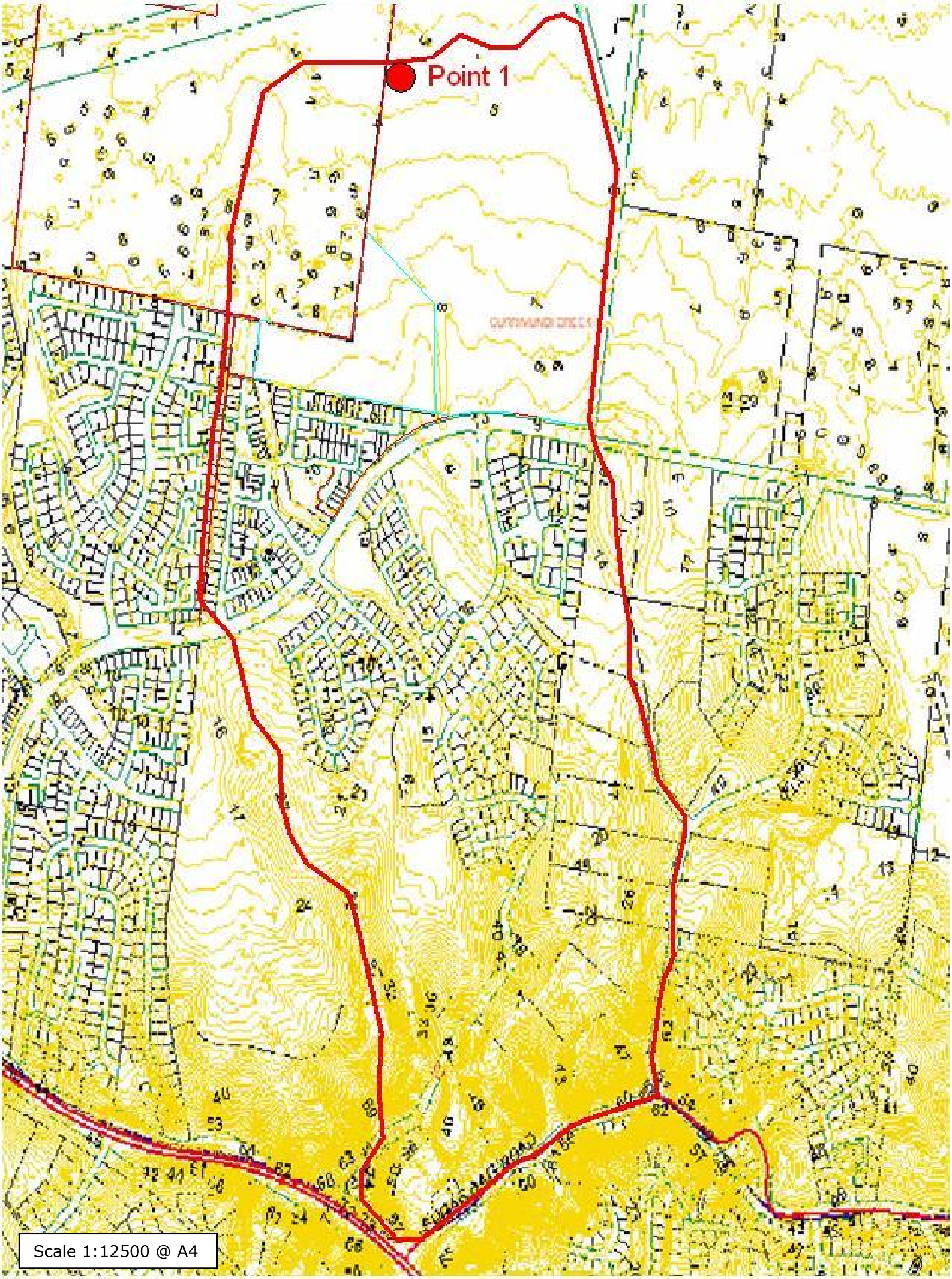

Figure 3.1 – Site and Upstream Catchment

Table 3.2 – Rational Method Peak Discharges at Point 1

ARI (years)	Peak Discharge (m^3/s)	
	Existing Site	Developed Site
1	9.19	10.53
2	12.49	14.23
5	17.57	20.04
10	20.85	23.77
20	25.03	28.48
50	32.09	36.58
100	37.08	42.19

3.4 HYDROLOGIC MODELLING

Full discharge hydrographs for the site and upstream catchment were calculated using a runoff routing model. The RAFTS runoff routing model, which was used, calculates discharge hydrographs utilising rainfall intensity data, temporal patterns, lose rates, degree of urbanisation, catchment area and slope (XP Software 2009).

RAFTS is a non-linear runoff routing model that calculates flood hydrographs from rainfall hyetographs. The hyetographs are determined by applying zone appropriate temporal patterns to rainfall depth appropriate for the catchment location (XP Software 2009, Pilgrim 1998). It can be used for the analysis and management of both urban and rural runoff and the design of flood storages and river analysis. RAFTS can also assist with the design of smaller urban drainage systems, on-site detention systems and large detention basins. In the case of this study RAFTS is used to determine urban runoff which is routed through detention structures.

The hydrologic model was only built for developed site conditions. This is due to the hydrologic model being utilised for commercial purposes. However, as the inflow boundary conditions for all hydrodynamic models were only considered at the upstream location, not across the site there would be no difference in using existing or developed flows. This is because the upstream catchment was assumed to be the same for both existing and developed site conditions. As a result the hydrographs from the developed model were considered appropriate for this study.

Hydrographs for the 1, 2, 5, 10, 20, 50 and 100 year ARI events for the site and upstream catchments were determined by setting up a RAFTS hydrologic model for developed conditions. The RAFTS model was manipulated until peak discharges were similar to those calculated by the Rational Method. Sub-catchments used in the RAFTS modelling and the modelling layout are shown in Figure 3.2.

Each sub-catchment of the RAFTS model was divided into its pervious and impervious areas. The B_x parameter used in the RAFTS model was 0.929. This was determined by setting the initial pervious losses for the 100 year ARI event to 0 and adjusting the B_x parameter until the peak discharges matched those of the Rational Method. The remainder of the losses for the 100 year ARI event were the same as those described below. This stage of the hydrologic modelling did not include the detention structures.

The B_x parameter is the global storage parameter within the RAFTS. Increasing the B_x number assumes more storage within the catchment and decreasing it assumes less. As all catchments have storage within it of some form or another and this number can be adjusted to suit the catchment. Therefore the B_x was adjusted to match the Rational Method flows for the 100 year ARI.

Continuing losses of 2.5 and 0 mm/hr were used for the pervious and impervious areas respectively. An initial loss of 1 mm was used for the impervious areas. The initial pervious loss was adjusted for the remainder of the ARI events until peak discharges matched those Rational Method peak discharges for the developed site. Table 3.3 presents the RAFTS model parameters used in the calibration.

Table 3.3 – RAFTS Model Parameters

ARI (Years)	Lag Parameter (B_x)	Pervious Area Initial Loss (mm)
1	0.929	6
2	0.929	9
5	0.929	8
10	0.929	8
20	0.929	8
50	0.929	1
100	0.929	0

The RAFTS model was run for the 1, 2, 5, 10, 20, 50 and 100 year ARI design events for the 25, 45, 60, 90 and 120 minute durations and compared to the Rational Method peak discharges. Table 3.4 compares the peak discharges from the Rational Method to those generated by the RAFTS model.

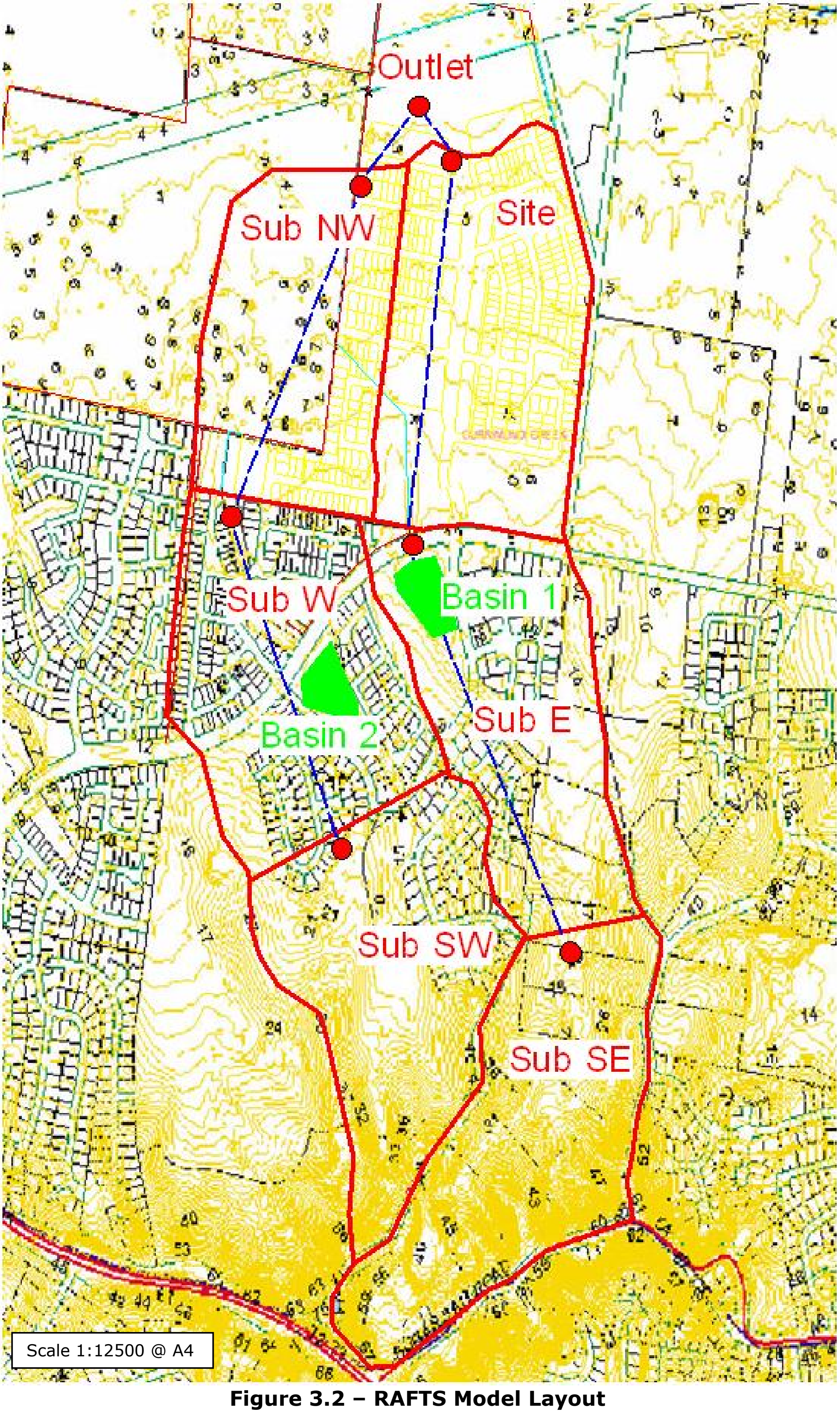

Figure 3.2 – RAFTS Model Layout

Table 3.4 – RAFTS - Rational Method Peak Discharges Comparison

ARI (years)	Peak Discharge (m3/s)		Difference (m3/s)
	Rational Method	RAFTS	
1	10.53	10.45	-0.07 (-0.7%)
2	14.23	14.09	-0.14 (-1.0%)
5	20.04	20.17	0.13 (0.6%)
10	23.77	23.74	-0.04 (-0.2%)
20	28.48	28.37	-0.10 (-0.4%)
50	36.58	36.63	0.05 (0.1%)
100	42.19	42.20	0.01 (0.01%)

Table 3.4 displays that the maximum difference between peak discharges is -0.14 m^3/s or 1.0%. Therefore the model was considered to adequately represent the peak design discharges for the catchment.

The two detention basins within the upstream catchment were subsequently added to the hydrologic model. Figure 3.2 presents the locations of the detention structures. The basins provide storage for stormwater runoff in the catchment and are intended to attenuate peak runoff for the upstream catchment. Table 3.5, Table 3.6, Table 3.7 and Table 3.8 show the basin characteristics which were entered into the runoff routing model. Storage and outlet information for these structures was determined from information provided by MRG Water Consulting and is subject to commercial confidence. Therefore, the information below is all that will be provided on these structures.

Table 3.5 – Basin 1 Height/Storage Relationship

Height (m AHD)	Storage (m^3)
8.0	0
9.0	6623
10.0	15289

Table 3.6 – Basin 1 Outlets

No. of Conduits	Details
2	• 1840 X 780 mm box culverts @ I.L 8.22
2	• 1830 X 980 mm openings @ I.L 8.22
1	• 1840 X 740 mm box culvert @ I.L 8.22

Table 3.7 – Basin 2 Height/Storage Relationship

Height (m AHD)	Storage (m^3)
8.5	0
10.0	12582

Table 3.8 – Basin 2 Outlets

No. of Conduits	Details
3	• 2100 X 760 mm box culverts @ I.L 8.50

The basins were then added to the RAFTS model and the model was re-run. It was found that the 90 minute storm was the critical duration for the 5, 10, 20, 50 and 100 year ARI events and the 120 minute storm was critical for the 1 and 2 year ARI events. Table 3.9 compares the peak discharges and without the detention basins included in modelling.

Table 3.9 – RAFTS Peak Discharges Comparison

ARI (years)	Peak Discharge (m^3/s)		Difference (m^3/s)
	RAFTS (no Basins)	RAFTS (with Basins)	
1	10.45	9.93	-0.52 (-5.0%)
2	14.09	13.44	-0.65 (-4.6%)
5	20.17	19.00	-1.17 (-5.8%)
10	23.74	22.33	-1.40 (-5.9%)
20	28.37	26.60	-1.77 (-6.3%)
50	36.63	34.53	-2.10 (-5.7%)
100	42.20	39.27	-2.93 (-6.9%)

4 METHODOLOGY – HYDRODYNAMIC MODELLING

4.1 SIMPLE 1D HYDRODYNAMIC MODEL

A simple 1D hydrodynamic model was firstly constructed and run. This prevented branch locations being determined from 2D hydrodynamic model output and yields a more realistic comparison. This model was set up and run by the author initially to be used for commercial purposes. Its intended use was to give an indication of existing flood levels across the site and minimise modelling time whilst giving an indication of flood characteristics. The model was set up for the 1, 2, 5, 10, 20, 50 and 100 year ARI events using the boundary conditions calculated above and topographic information provided by MRG Water Consulting.

The initial MIKE 11 model consists of two reaches, the central and western reach. The western waterway extends from the southern site until it joins the central branch 259 m downstream. The central waterway extends from the southern property boundary to 799 m downstream.

The Manning's 'n' roughness varied throughout the model to represent the terrain within the model. Roughness values were determined by first examining the aerial photography and utilising knowledge of the site gained from a site visit. The values were then assigned based on the tables and guidelines in the HECRAS reference manual (Hydrologic Engineering Center 2010). The HECRAS manual was used due to its clarity and comprehensive tables and description of the conditions for each value. Figure 4.1 illustrates the variation in roughness for the flood plain in relation to the cross sections. The roughness values are shown for each area by the number highlighted yellow. For locations not marked in Figure 4.1 the roughness was assumed to be 0.06.

The discharge hydrographs from the calibrated RAFTS model were used as boundary conditions in the MIKE11 model. Discharges for the eastern waterway were not considered in the simple model as it was assumed that the discharges flowing through this channel would be of little consequence in comparison to the central and western waterways. Therefore, the discharges for eastern waterway were entered at the central waterway. Table 4.1 shows the location and description of the boundaries.

Table 4.1 – Initial 1D Model Boundary Conditions

Boundary Description	MIKE11 Boundary Type and Chainage
RAFTS Sub-catchment W (Total Flows)	Open – 2000
RAFTS Sub-catchment E (Total Flows)	Open – 1000
Q-H relationship	Auto Generated – 1699

Table 4.2 shows the peak discharge for each of the inflow boundaries for the range of ARI events.

Table 4.2 – Initial 1D Model Inflow Boundary Peak Discharges

ARI (years)	Peak Discharge (m^3/s)	
	Central Branch	Western Branch
1	4.36	3.51
2	5.86	4.76
5	8.81	6.96
10	10.44	8.25
20	12.52	9.89
50	15.31	12.85
100	17.45	14.77

The model was run with a 0.5 sec time step. This was consistent for all hydrodynamic modelling in this study. The 0.5 sec time step was determined by the size of the grid for the 2D model. It is recommended by BMT WBM (2007) that an alternative 'rule of thumb' method to using a courant number for determining the time step is assuming a time step which is half the grid size. That would mean a 5 m grid has a 2.5 sec time step. As the grid selected for the TUFLOW model was 1 m the time step chosen was 0.5 sec. For consistency this was kept constant for all hydrodynamic models. The MIKE11 models were not found to be unstable due to this time step.

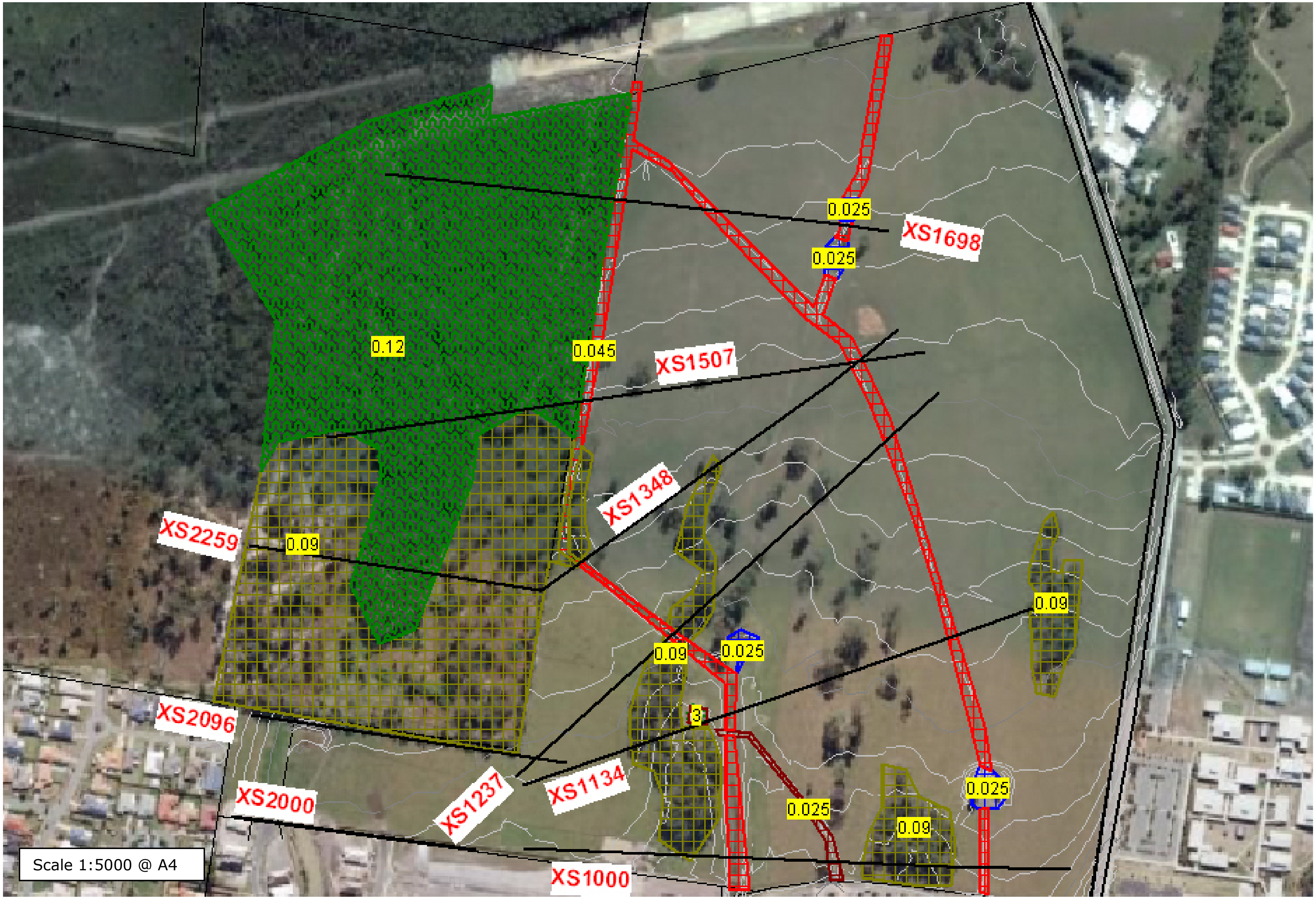

Figure 4.1 – Initial 1D Model Roughness

4.2 2D HYDRODYNAMIC MODEL SET UP

4.2.1 General Model Set Up

The 2D hydrodynamic model was subsequently created from a digital terrain model (DTM) constructed from topographic information provided by MRG Water Consulting. There were significant iterations in the construction of the DTM in order to obtain a surface the best represented the topography. The creation of the DTM will be discussed further in Section 4.2.2 of this report.

Along with the DTM the 2D hydrodynamic model was constructed from the boundary information discussed in Section 3.4 and roughness information mentioned in Section 4.1. Numerical output from the model was taken at key locations.

4.2.2 DEM Creation

MRG Water Consulting provided survey information for the site addressed 166 Parklands Boulevard. However, the only topographic information that was made available to the author for the neighbouring, thickly vegetated area was Council contours at a 0.5 m elevation interval. Figure 4.2 illustrates the extent of the survey information used for the creation of the DTM with Figure 4.3 displaying the survey points at a closer view.

By inspection of Figure 4.2 it can be seen that there was a significant amount of survey information for 166 Parklands Boulevard. The black stars and red circles constitute survey points and the grey lines show the contour lines. As mentioned before the contour lines were taken from SCRC raster images and therefore have less detail. Some points were manually inserted at key locations to assist in the creation of the DTM. Lines have been introduced as shown in Figure 3.5 marking the top levees and inverts of the waterways and dams (breaklines). These lines are marked in magenta. The breaklines were in fact created by the author based on site inspection and a study of the survey data. Due to the limitations of MapInfo's Vertical Mapper the breaklines were represented as closely situated points, as shown on Figure 4.3.

An add-on product of MapInfo named Vertical Mapper was used to generate the DTM. As an investigation into the 3D surface creation is outside the scope of this study the capabilities and functioning of Vertical Mapper will not be discussed here in full.

Three main iterations were undertaken in generating the 3D surface and are described as follows:

Iteration 1

The contour lines shown on Figure 4.2 and Figure 4.3 were translated into points with a 1 m spacing. The translated lines were added to the survey points and the complete data set was translated using the triangulation method within vertical Mapper. This method was found to create a 'streaky' surface on the neighbouring property and was therefore not entirely desirable.

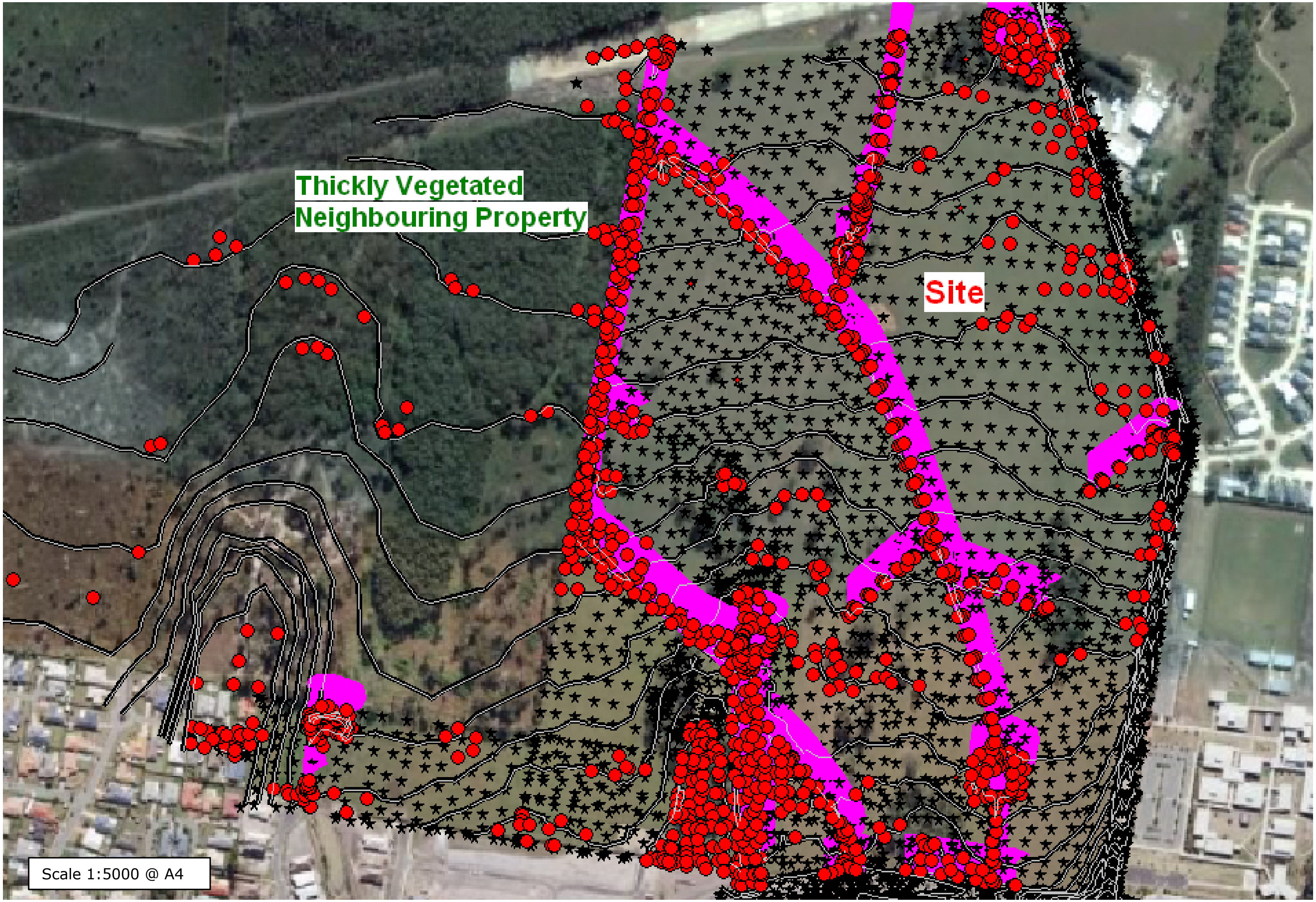

Figure 4.2 – Site Topographic Information

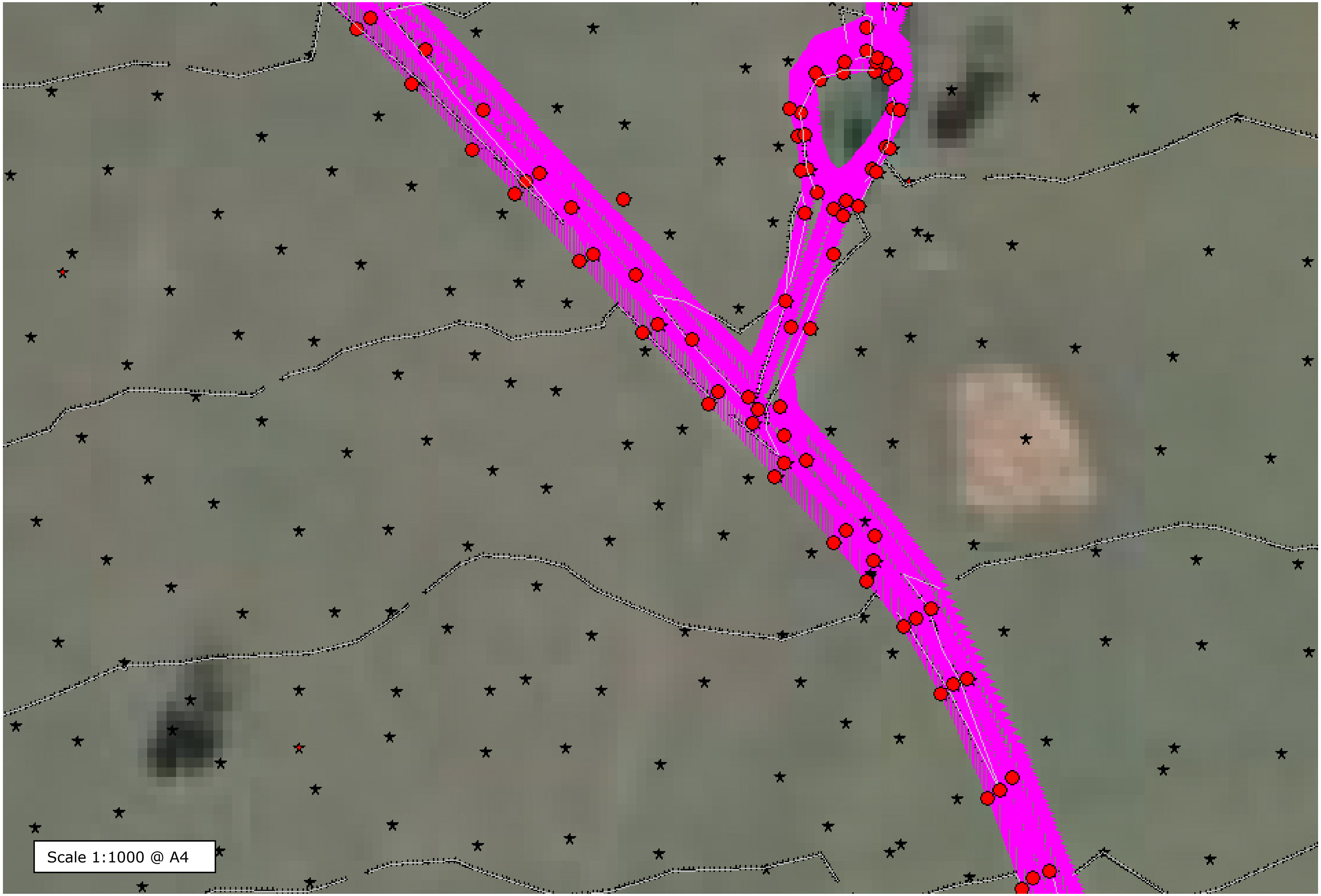

Figure 4.3 – Survey Points and Breaklines

Iteration 2

As in iteration 1 the contour lines shown on Figure 4.2 and Figure 4.3 were translated into points with a 1 m spacing. The complete data set was then translated using the natural neighbour method within vertical Mapper. The natural neighbour method assumes, via mathematical relationships, which of the nearest points to take the height information for the interpolated grid points. This method created a grid with a too many local high points creating the appearance of 'mounds' throughout the surface.

Iteration 3

Iteration 3 was the same as Iteration 1 only that the contour lines were translated to points at the vertices of the lines rather than every metre. This yielded the most favourable result, as the surface looked most realistic.

Figure 4.4 , Figure 4.5 and Figure 4.6 show the 3D surface created by Iterations 1, 2 and 3 respectively. All 3D surfaces were created with a 1 m grid.

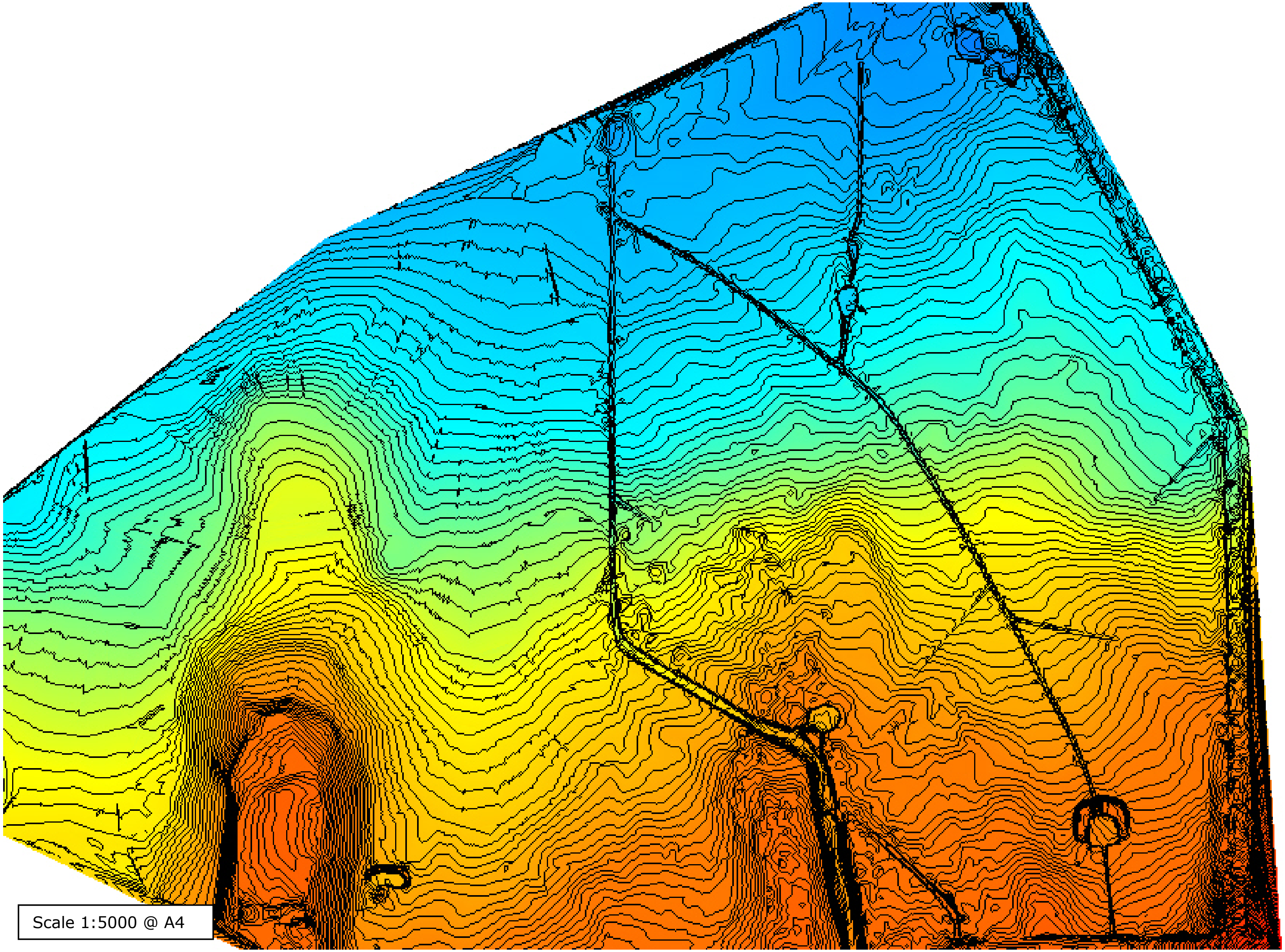

Figure 4.4 – Iteration 1 DTM

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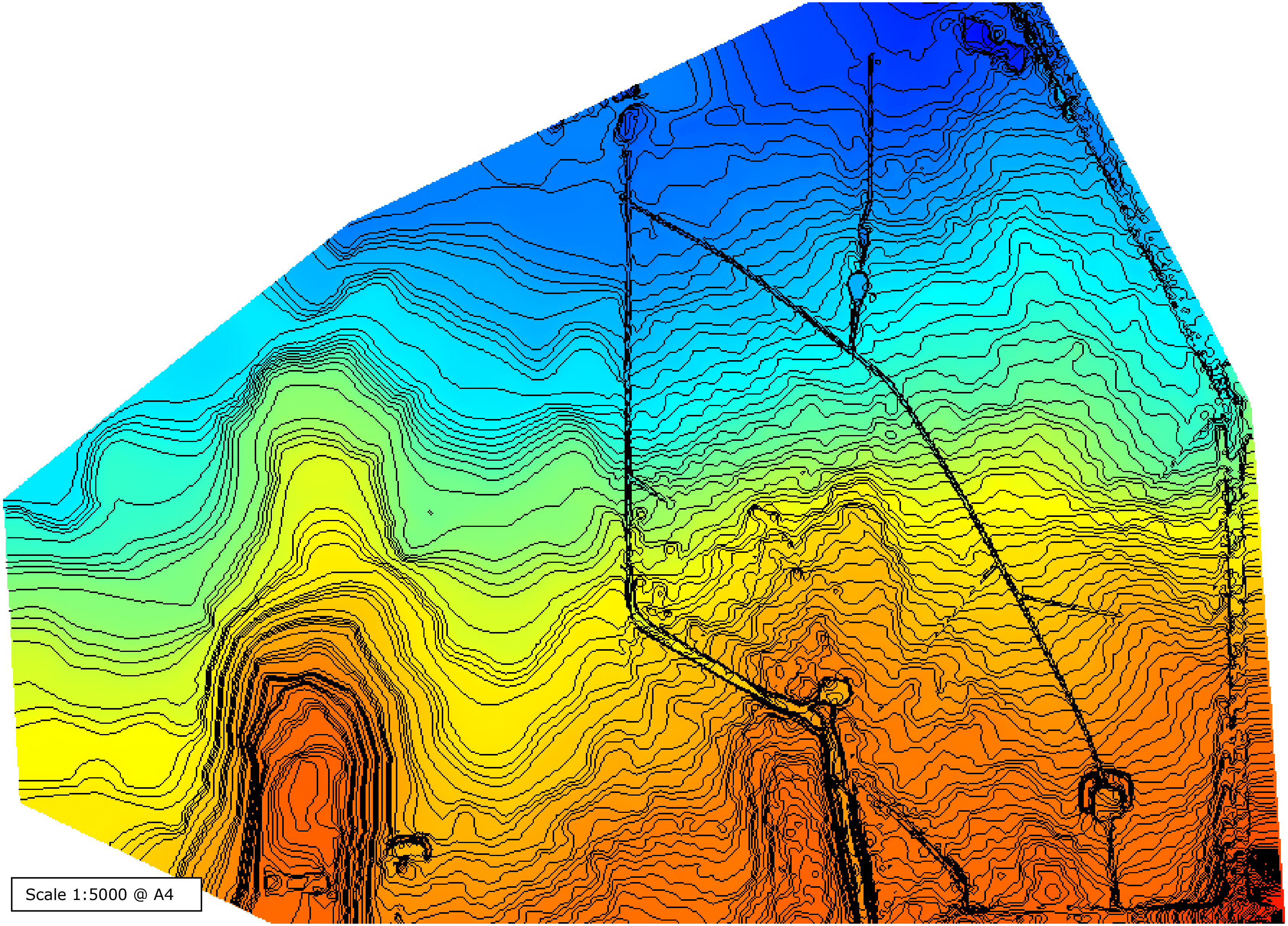


Figure 4.5 – Iteration 2 DTM

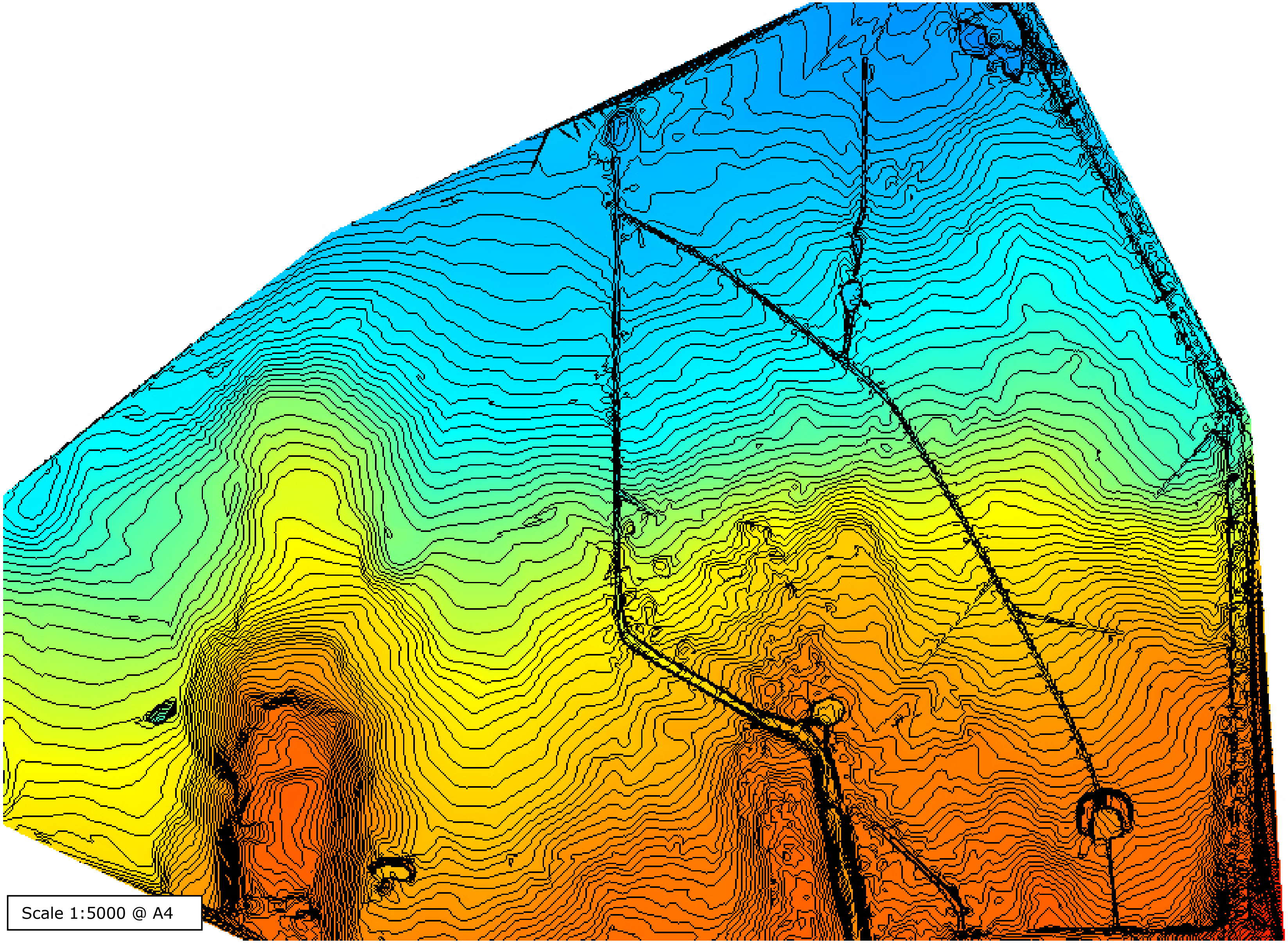


Figure 4.6 – Iteration 3 (Adopted) DTM

4.2.3 Model Grid

The grid size selected for the 2D model was 1m. The 1m grid was chosen in order to gain a reasonable definition of the waterways within the site. A grid size that is coarse over a waterway will not have enough definition of the waterway to accurately represent the shape and profile. This concept is illustrated in Figure 4.7 which was taken from the TUFLOW User Manual (WBM BMT 2007).

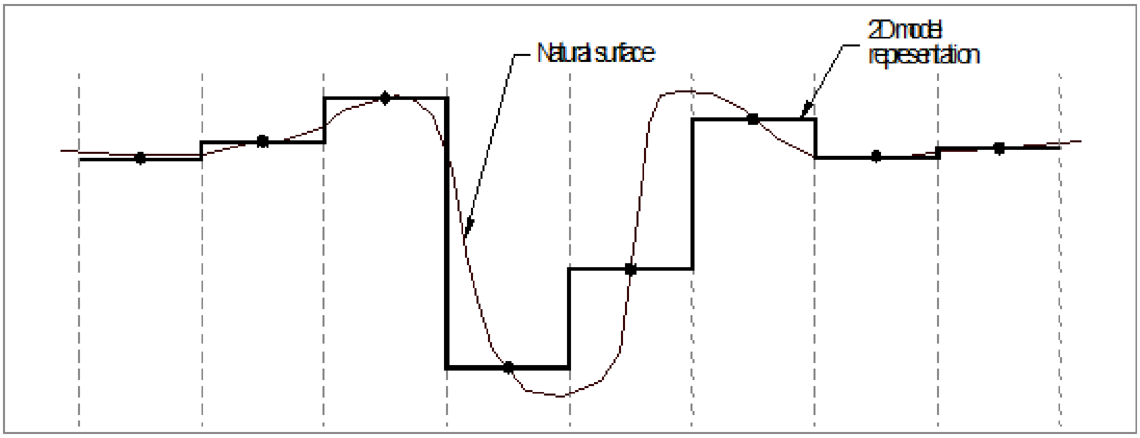


Figure 4.7 – Representation of a Coarse Grid over a Channel

The TUFLOW manual recommends that a 1D channel be defined within the 2D grid where a grid fine enough to represent the channels is not feasible. Hence, the 1m grid was selected to prevent the use of an embedded 1D channel within the 2D grid. This process was outside the scope of this study.

4.2.4 Model Boundaries and Roughness

The inflow hydrographs used in the initial 1D model were used also for this model. However, the hydrograph from the eastern portion of the upstream catchment was split up to account for water flowing along the eastern most waterway. Table 4.3 contains the peak flows for the inflow boundaries into the 2D model.

Table 4.3 – 2D Model Inflow Boundary Peak Discharges

ARI (years)	Peak Discharge (m^3/s)		
	Central Branch	Western Branch	Eastern Branch
1	4.09	3.51	0.27
2	5.50	4.76	0.36
5	8.27	6.96	0.54
10	9.80	8.25	0.64
20	11.75	9.89	0.77
50	14.37	12.85	0.94
100	16.38	14.77	1.07

Figure 4.8 shows the model extents and boundary locations. Mannings 'n' roughness was taken from the same polygons shown on Figure 4.1 site for the initial 1D model, signifying roughness values for the site roughness. As in was the case for the initial 1D hydrodynamic model the area not shaded in Figure 4.1 were considered to have a Manning's 'n' roughness value of 0.06.

4.2.5 Output Locations

Output for the 2D hydrodynamic model was taken every 1 min for the entire grid. In order to make comparison to the 1D hydrodynamic models simpler, comparison lines were also identified for the 2D model and results were output at the same interval for discharge, velocity and water level. Figure 4.9 displays the output locations.

Figure 4.8 – 2D Model Extents

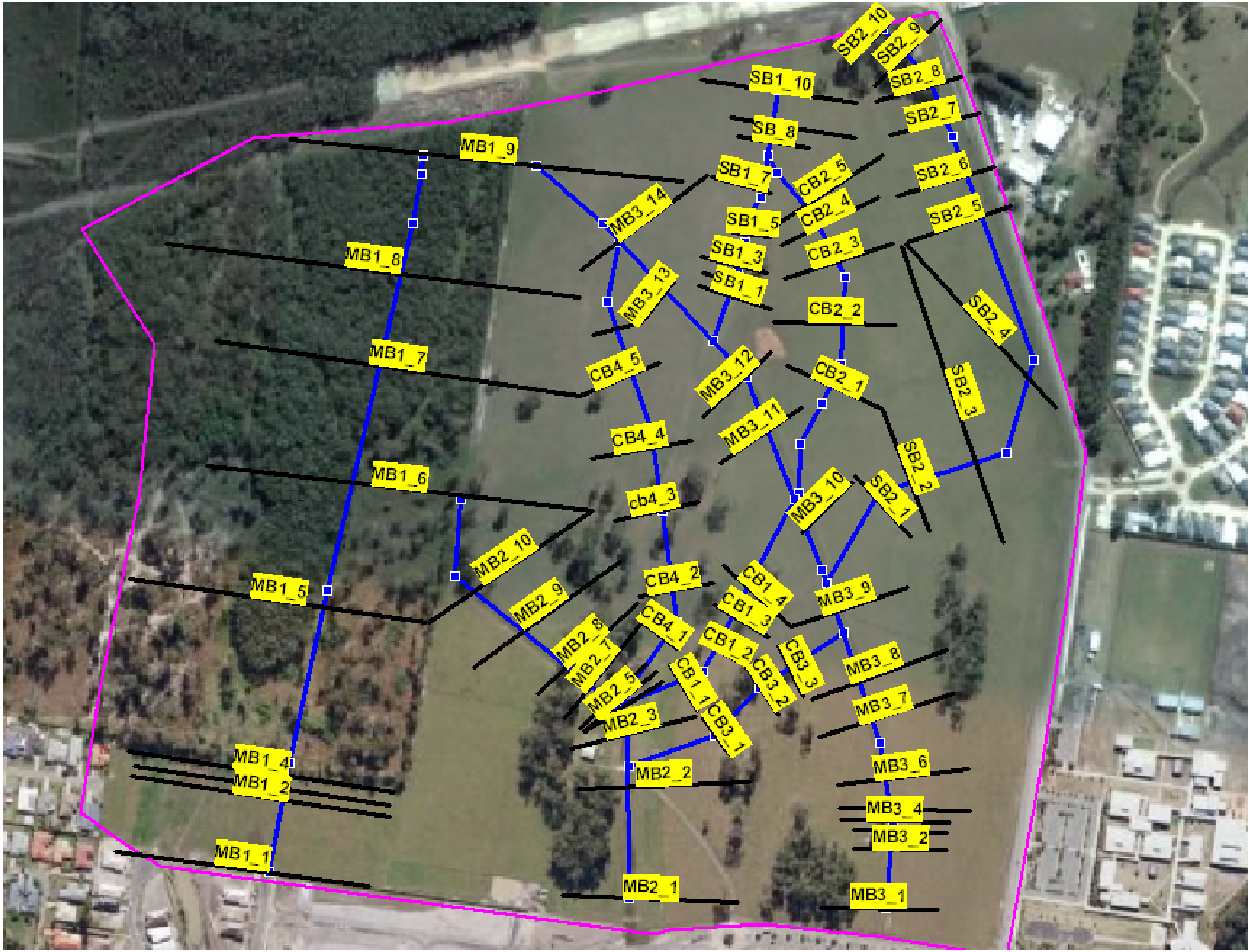
Figure 4.9 – 2D Model Comparison

4.3 REFINED 1D HYDRODYNAMIC MODEL

The second 1D hydrodynamic model was set up based on an inspection of the 2D model output. The second 1D model consisted of 3 major branches, 3 connection branches and 2 sub-branches that account for break outs from the main branches that do not connect to another branch. The branches are therefore given the names of MB1 to MB3 (major branches), CB1 to CB4 (connection branches) and SB1 and SB2 (sub-branches).

In order for the MIKE11 model to smoothly calculate the transitions into the branches small slots were put in the adjoining sections so they joined at the same at the same elevation from a model perspective. The slots were made small enough so as to not hinder the hydraulics of the model. Appendix C contains all cross sections used in modelling.

Inflows for the second 1D hydrodynamic model were the same magnitude and location as that of the 2D hydrodynamic model (that is there was a cross section corresponding to each of the 2D model inflow locations). The Mannings 'n' roughness and downstream boundary conditions were determined in the same manner as the two previous hydrodynamic models. Figure 4.10 displays the model set out for the refined 1D model, including cross section and branch locations.


SB2_10
SB2_9
SB2_8
SB1_10
SB2_7
SB_8
MB1_9
SB2_6
SB1_7
CB2_5
MB3_14
CB2_4
SB1_5
SB2_5
SB1_3
CB2_3
MB1_8
SB1_1
MB3_13
CB2_2
SB2_4
MB1_7
CB4_5
MB3_12
CB2_1
SB2_3
MB3_11
CB4_4
SB2_2
MB1_6
cb4_3
SB2_1
MB3_10
MB2_10
CB4_2
CB1_4
MB1_5
MB2_9
CB1_3
MB3_9
CB4_1
MB2_8
CB1_2
CB3_3
MB2_7
CB1_1
CB3_2
MB3_8
MB2_5
MB3_7
MB2_3
CB3_1
MB1_4
MB3_6
MB1_2
MB2_2
MB3_4
MB3_2
MB1_1
MB2_1
MB3_1

5 RESULTS DISCUSSION

5.1 MODEL SET UP TIME

Hydrodynamic model creation time was observed throughout the study to present a comparison of set up duration for a 1D simple model, 1D refined model and a 2D model. These times exclude any hydrologic calculations and modelling. Set up times for the models are reported below. The time to learn the processes of 2D modelling such as DEM creation were omitted therefore, the duration was calculated based on the assumption of the modeller having prior knowledge and skills.

- 1st 1D model set up time – 10hrs
- 2D model set up time – 6hrs
- 2nd 1D model set up time – 9hrs

The actual time spent on the 2D model was well over 30 hrs as the processes and workings of TUFLOW modelling and DEM creation needed to be learnt. However, excluding the time taken to learn TUFLOW, it can be seen from the list above that a pure 2D model was less time consuming to set up for the given site. This would however, vary with a differing complexity in roughness and the addition of hydraulic structures. The addition of hydraulic structures is considered to add equally to the set up time for both 1D and 2D model. In contrast, a greater spatial change in material roughness would tend to be increasingly more time consuming for a 2D hydrodynamic model.

Furthermore, the 2nd 1D model was considerably reduced in set up time by:

- The creation of the DEM;
- The use of utilities that accompany the TUFLOW software; and
- The use of the 2D roughness MapInfo file.

It is believed that the 2nd 1D model would have been at least 300% longer to set up without these three key tools.

However, model run times for the 1D and 2D models were contrasting. With 2D and 1D model run times up 8 hrs 15 min, respectively. This would be an advantage when undertaking multiple modelling iterations but does not affect the required set up time.

In conclusion a 2D model is equally difficult and time consuming to set up as a 1D model consisting of a few branches. However, as a 1D hydrodynamic model increases in the number of branches the set up time exceeds that of a pure 2D hydrodynamic model.

5.2 COMPARISON OF INITIAL 1D AND 2D HYDRODYNAMIC MODELS

The initial 1D model was run for the 1 year and 100 year ARI events. Figure 5.1 shows the 100 year ARI inundation across the site for the initial 1D model and Figure 5.2 displays the maximum depths and flood extent for the 100 year ARI flood event for the 2D model.

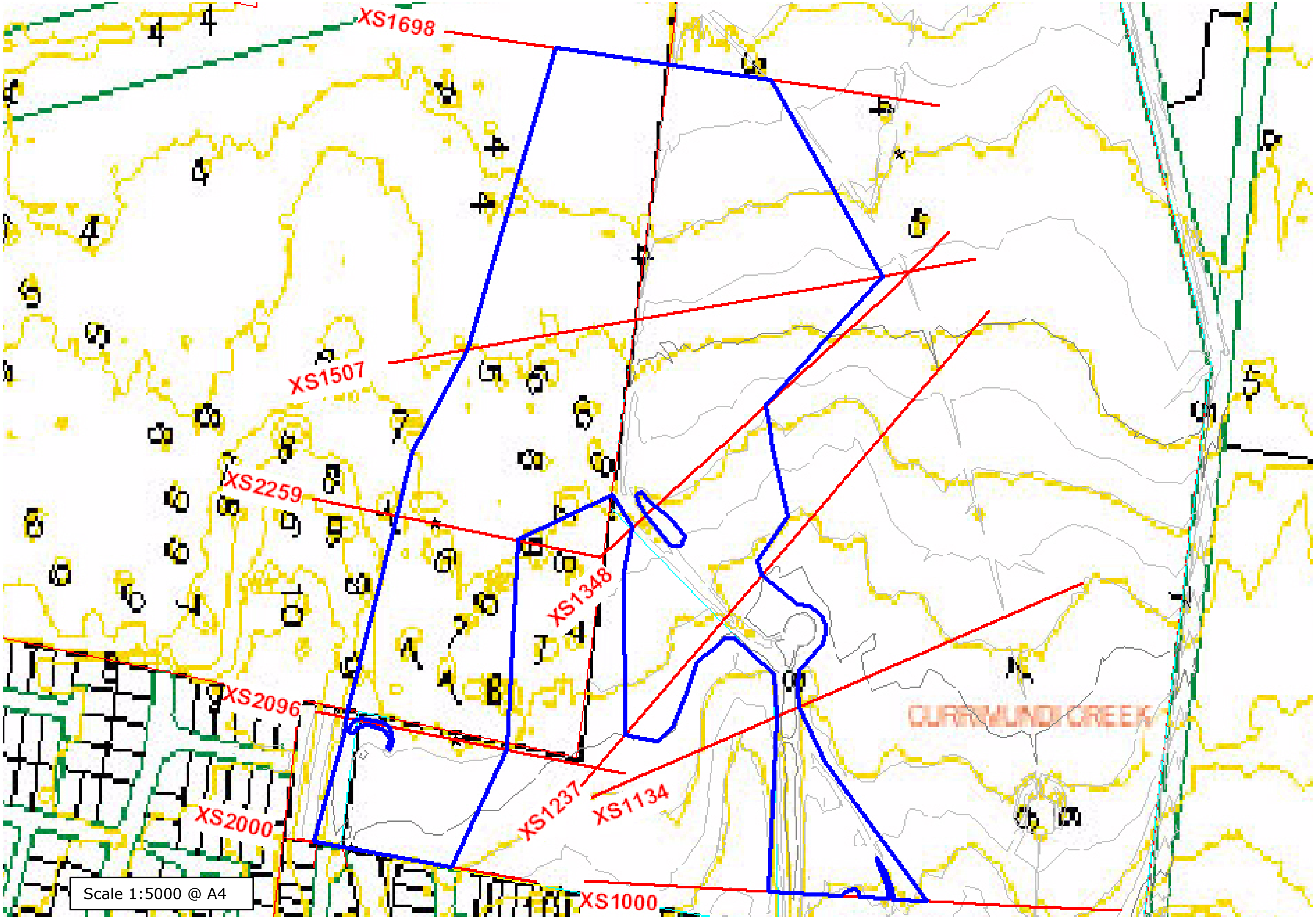


Figure 5.1 – Initial 1D Model 100 Year ARI Inundation

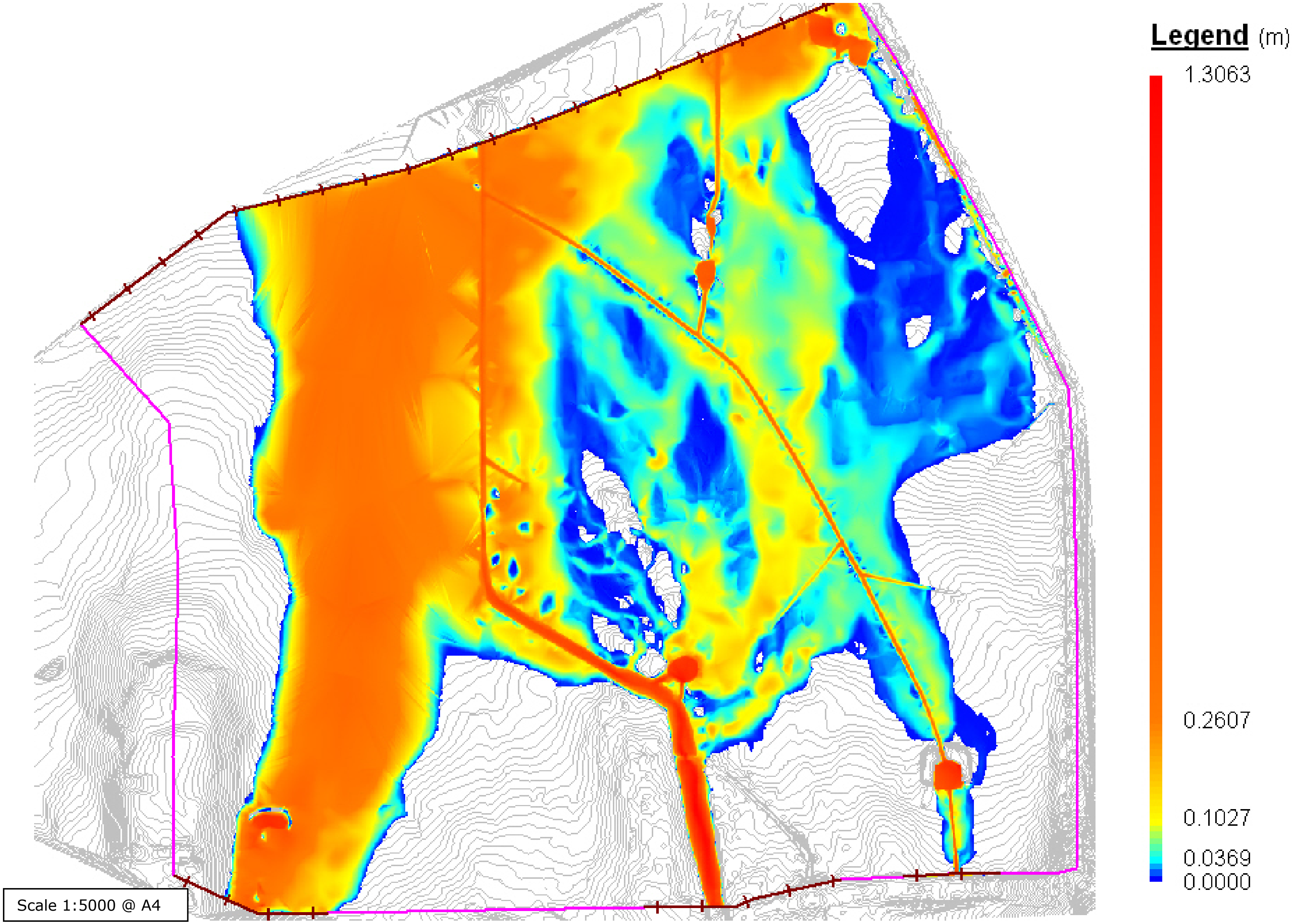


Figure 5.2 – 2D 100 Year ARI Peak Flood Depth

It can be seen from these figures that the 2D model presents a far more comprehensive indication of the inundation extent. It is evident from Figure 5.1 that there are some model set up aspects to the simple 1D model that have affected the results. Firstly, whilst cross sections 1000 to 1348 are perpendicular to the central waterway they are not for the eastern. This has affected how the model calculates the flows through the eastern portion of the site. The other aspect that has affected the results is the crossover of cross sections 1348 and 1507. While these aspects contribute to the difference in model output between the 1D and 2D models these differences are model set up difference and not attributed to the limitations of 1D modelling.

5.2.1 Comparison of 100 Year ARI Water Levels

Water levels at the simple 1D model cross section locations were compared for the 1D and 2D models. Table 5.1 gives a comparison of the water levels between the two models. The 2D model output in this table is from the TUFLOW print output lines which records model output along a line or lines for a set time interval as the model runs.

Table 5.1 – Simple 1D & 2D 100 Year Comparison (Water Levels)

Chainage	1D Peak Water Level m AHD	2D Peak Water Level m AHD	Difference m
1000	8.575	8.919	0.344
1134	8.078	8.071	-0.007
1237	7.377	7.304	-0.073
1348	6.087	6.195	0.108
1507	4.814	5.052	0.238
1698	3.472	3.554	0.082
2000	7.875	7.896	0.021
2096	7.198	7.179	-0.019
2259	5.491	6.155	0.664

It can be seen from Table 5.1 that of the nine cross sections listed six of them show an increase in water levels from the 1D model to the 2D model. Of the three cross sections that show a decrease in water levels two of those are almost comparable with chainage 1134 and 2096 having a difference of 7 mm and 19 mm, respectively. Nonetheless the maximum decrease in water levels was 73 mm which is still comparatively small. The maximum increase in water levels is 664 mm. This is a significant increase in water levels. However, Figure 5.3 below shows that the initial water level for this cross section is markedly different between the 2D and refined 1D model. This is attributed to the difference in determining and representing the topography for the site. The 1D cross sections were determined by inspection and interpolation of topographic data where the 2D topographic conditions were taken from the DEM. Figure 5.3 to Figure 5.6 below contain water level over time comparisons at key locations for the simple 1D and 2D model.

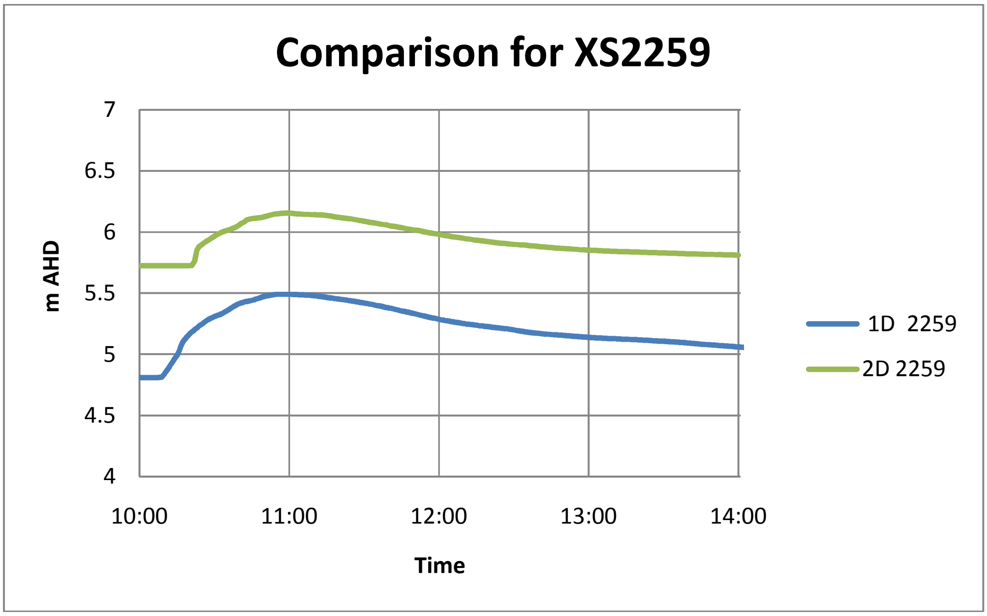


Figure 5.3 – Q100 WL Comparison for XS2259

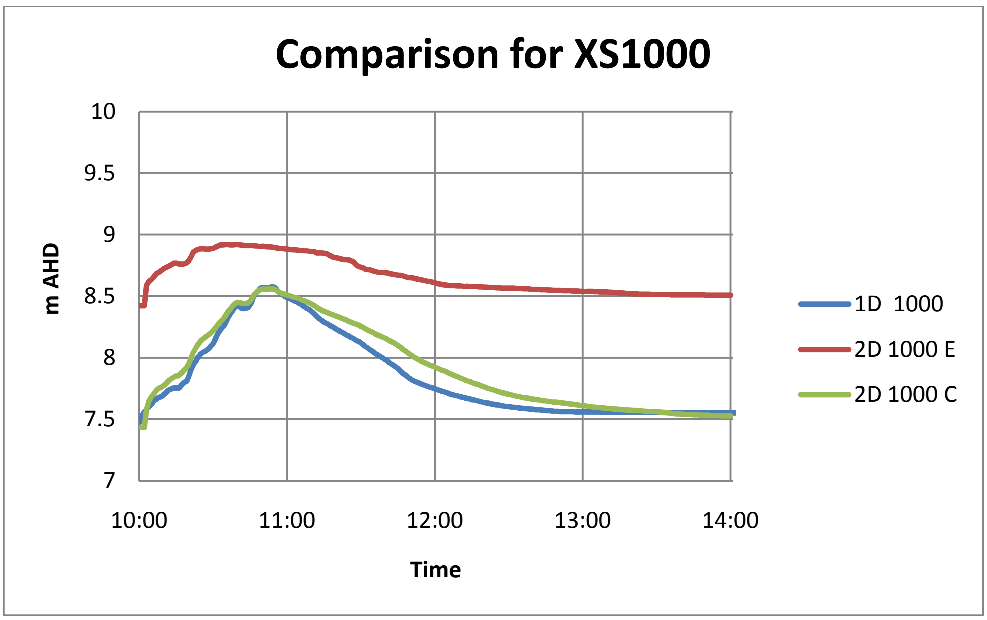


Figure 5.4 – Q100 WL Comparison for XS1000

Figure 5.4 illustrates that for the central channel the water levels are comparable in both magnitude and shape, but that the 1D model does not represent water levels in the eastern channel. This can be seen by the clear difference in the line labelled '2D 1000 E' (representing the water level at the eastern waterway in the 2D model) and the other two water level-time lines.

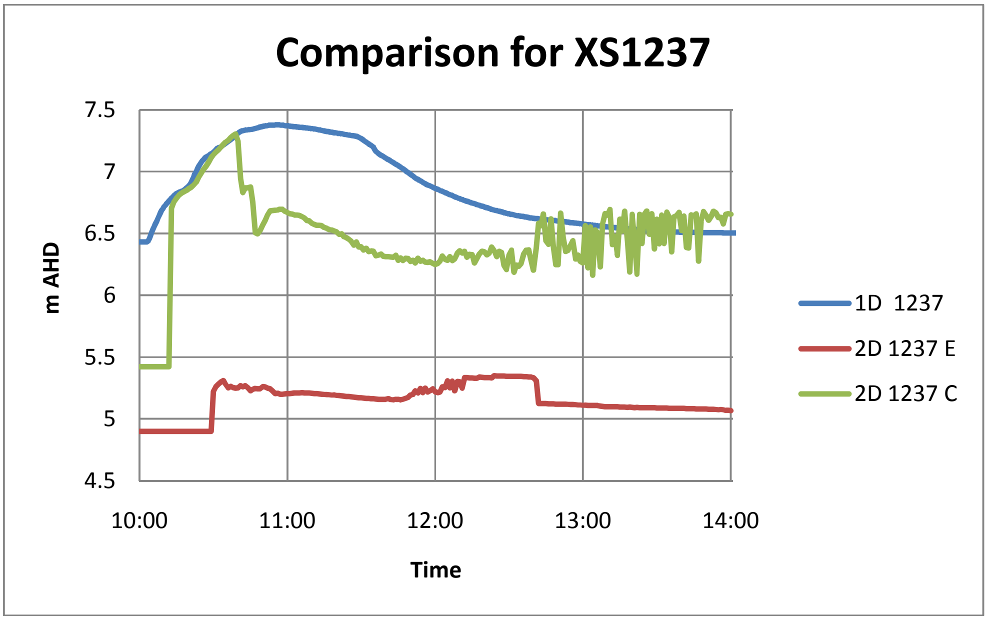


Figure 5.5 – Q100 WL Comparison for XS1237

It can be deduced from Figure 5.5 that the water level in the central waterway drops quite dramatically and suddenly at approximately 40 minutes of model run time. Furthermore the comparison identifies that the initial water level is quite different. However, upon inspection of the graphical output from the 2D model it was determined that this is simply demonstrating a limitation in the numerical print output function of TUFLOW. The output comparison lines that were entered into the TUFLOW model have the capability of recording water levels for two major flow paths within that line. The locations at which to record the water level are determined by the width of the flow path. As the 2D model starts it assumes the invert at a location which is not in the central channel, then as the flow comes through the central channel (one of the dominant flow paths) the model records the water levels at that location. However as the water starts to break out from the farm dam upstream of cross section 1237, a wider yet shallower flow path is evident, the water levels are subsequently taken from that location. From the 2.5 hour model run point the central water level appears to oscillate, this is a result of water levels being taken at differing location as the model progresses. To more accurately compare water levels at this location the graphical output for each time step requires interrogation. This would however be a very time consuming task. Inspection of the maximum water level graphical output revealed that the maximum water level at this location was 7.579 m AHD which is 202 mm greater than the 1D output.

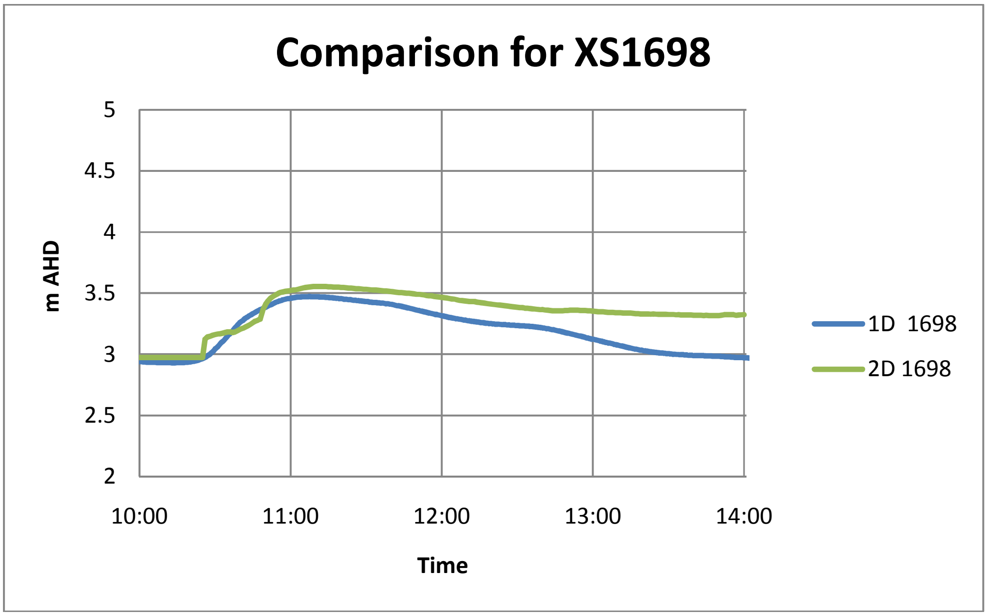


Figure 5.6 – Q100 WL Comparison for XS1698

Figure 5.6 demonstrates that the peak water level for both models at the downstream end is similar. Figure 5.6 however, also illustrates that the 2D model has a slower rate of change in water level as the level drops. This would be due to the 2D model being able to store more water due to the spatial variance in ground levels then the 1D model.

5.2.2 Comparison of 100 Year ARI Discharges

Table 5.2 compares the discharges at the simple 1D model cross section locations. It should be noted that MIKE11 actually calculates the discharges between the cross sections but the calculated discharges are applied at the cross sections to calculate water levels. The first upstream cross sections were not compared as the discharges at this location are equal to the inflow boundary conditions. Table 5.2 shows that discharges in the 2D model were generally lower in the cross section locations. This demonstrates the ability of the 2D model to vary the flooding extent within the grid as required. That is to say the discharges were lower across the specified lines as the inundation extent in the 2D model was greater.

Table 5.2 – Simple 1D & 2D 100 Year Comparison (Discharges)

Chainage	1D Peak Discharge m^3/s	2D Peak Discharge m^3/s	Difference m^3/s	Difference %
1134	17.209	17.278	0.069	0.4%
1237	17.056	15.599	-1.457	-8.5%
1348	16.887	14.639	-2.248	-13.3%
1507	30.429	26.511	-3.918	-12.9%
1698	29.692	27.357	-2.335	-7.9%
2096	14.641	14.625	-0.016	-0.1%
2259	14.236	14.44	0.204	1.4%

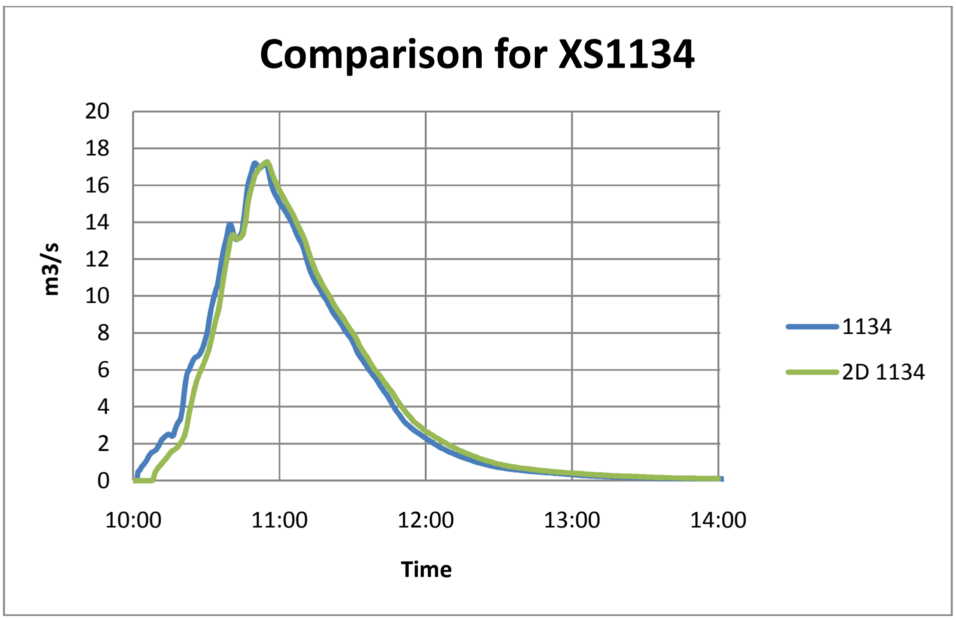


Figure 5.7 – Q100 Q Comparison for XS1134

Figure 5.7 illustrates that the discharges in the upstream section of the simple 1D and 2D models are very similar in magnitude and shape. It can seen from Figure 5.8 and Figure 5.9 that as the models progress further downstream there is more attenuation within the 2D model. That is to say that the 2D model is calculating more flood storage then the initial 1D model. This is shown by the peak of the 2D graph being lower and slightly later. The 2D model has the ability to calculated flood storage by the spatial variance in ground levels which will allow the water to be trapped or slowed by dips or low points in the terrain.

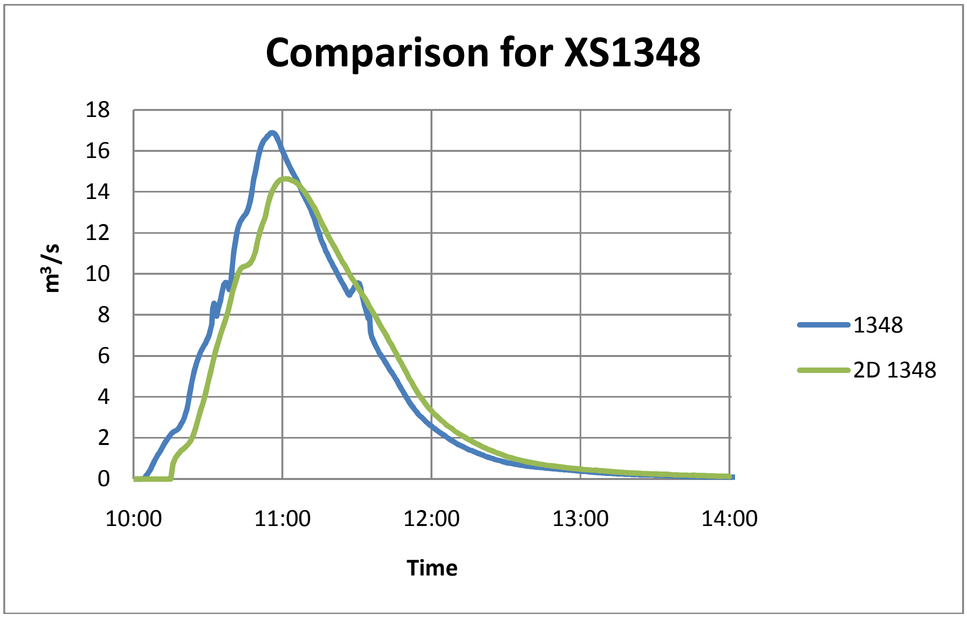


Figure 5.8 – Q100 Q Comparison for XS1348

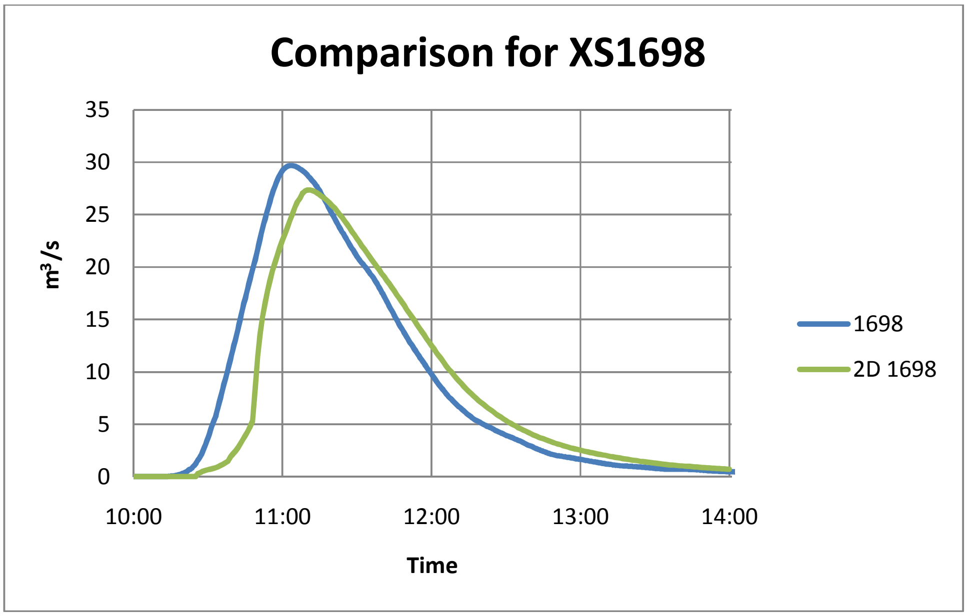
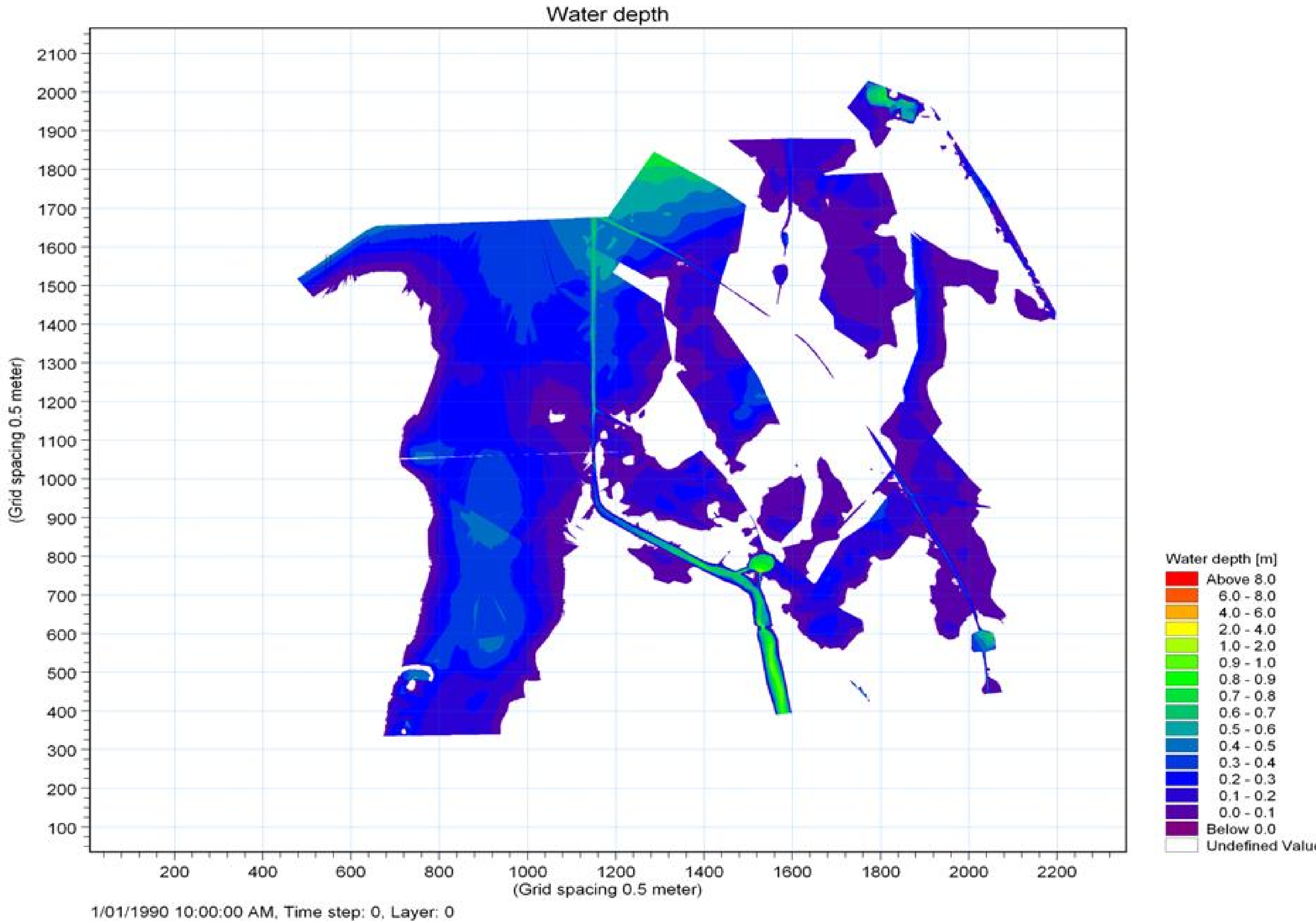


Figure 5.9 – Q100 Q Comparison for XS1698

5.3 COMPARISON OF 2D AND SECOND 1D HYDRODYNAMIC MODELS

Figure 5.10 presents the 1D model output from the refined 1D model. Though the cross sections were positioned so as to not overlap, the model was still not able to accurately represent or interpolate the model output.



Water depth
2100
2000
1900
1800
1700
1600
1500
1400
1300
1200
1100
1000
900
800
700
600
500
400
300
200
100
(Grid spacing 0.5 meter)
200
400
600
800
1000
1200
1400
1600
1800
2000
2200
(Grid spacing 0.5 meter)
1/01/1990 10:00:00 AM, Time step: 0, Layer: 0
Water depth [m]
Above 8.0
6.0 - 8.0
4.0 - 6.0
2.0 - 4.0
1.0 - 2.0
0.9 - 1.0
0.8 - 0.9
0.7 - 0.8
0.6 - 0.7
0.5 - 0.6
0.4 - 0.5
0.3 - 0.4
0.2 - 0.3
0.1 - 0.2
0.0 - 0.1
Below 0.0
Undefined Value

5.3.1 Comparison of 100 Year ARI Water Levels

Table 5.3 below compares the water levels for the refined 1D model and the 2D model. The comparison locations for the 2D Model were the same as the 1D model cross sections. Columns 1 and 2 of Table 5.3 contain the names of these locations for each model.

Table 5.3 – Refined 1D & 2D 100 Year Comparison (Water Levels)

1D Section Name	2D Location Name	1D Peak Water Levels m AHD	2D Peak Water Levels m AHD	Difference m
MB1 1000	2D MB1_1	7.798	7.868	0.070
MB1 1067	2D MB1_2	7.244	7.426	0.182
MB1 1077	2D MB1_3	7.175	7.350	0.175
MB1 1089	2D MB1_4	7.114	7.179	0.065
MB1 1245	2D MB1_5	6.151	6.156	0.005
MB1 1361	2D MB1_6	5.410	5.488	0.078
MB1 1469	2D MB1_7	4.452	4.541	0.089
MB1 1555	2D MB1_8	3.892	3.930	0.038
MB1 1664	2D MB1_9	3.454	3.287	-0.167
MB2 1000	2D MB2_1	8.334	8.559	0.225
MB2 1101	2D MB2_2	8.068	8.371	0.303
MB2 1149	2D MB2_3	7.810	8.028	0.218
MB2 1173	2D MB2_4	7.810	7.820	0.010
MB2 1178	2D MB2_5	7.704	7.818	0.114
MB2 1186	2D MB2_6	7.700	7.802	0.102
MB2 1207	2D MB2_7	7.543	7.656	0.113
MB2 1288	2D MB2_9	6.744	6.843	0.099
MB2 1345	2D MB2_10	6.106	6.213	0.107
MB3 1000	2D MB3_1	8.892	8.920	0.028
MB3 1055	2D MB3_2	8.077	8.392	0.315
MB3 1070	2D MB3_3	8.073	8.377	0.304
MB3 1079	2D MB3_4	8.073	8.378	0.305
MB3 1091	2D MB3_5	7.996	8.273	0.277
MB3 1119	2D MB3_6	7.662	7.669	0.007
MB3 1211	2D MB3_8	6.930	6.957	0.027
MB3 1278	2D MB3_9	6.497	6.491	-0.006
MB3 1359	2D MB3_10	5.735	5.800	0.065
MB3 1455	2D MB3_11	4.630	5.088	0.458

MB3 1508	2D MB3_12	4.332	4.629	0.297
MB3 1617	2D MB3_13	3.512	3.886	0.374
MB3 1679	2D MB3_14	3.456	3.429	-0.027
SB1 1043	2D SB1_1	3.688	4.078	0.390
SB1 1056	2D SB1_2	3.536	3.965	0.429
SB1 1072	2D SB1_3	3.516	3.918	0.402
SB1 1091	2D SB1_4	3.363	3.711	0.348
SB1 1102	2D SB1_5	3.295	3.603	0.308
SB1 1114	2D SB1_6	3.261	3.513	0.252
SB1 1145	2D SB1_7	3.076	3.160	0.084
SB1 1189	2D SB1_8	2.808	2.803	-0.005
SB1 1205	2D SB1_9	2.715	2.725	0.010
SB1 1235	2D SB1_10	2.592	2.628	0.036
SB2 1088	2D SB2_1	5.671	5.600	-0.071
SB2 1124	2D SB2_2	5.151	5.310	0.159
SB2 1186	2D SB2_3	4.423	4.776	0.353
SB2 1285	2D SB2_4	4.227	4.205	-0.022
SB2 1442	2D SB2_5	3.433	3.716	0.283
SB2 1482	2D SB2_6	3.266	3.463	0.197
SB2 1570	2D SB2_8	2.667	2.618	-0.049
SB2 1639	2D SB2_10	2.502	2.617	0.115
CB1 1062	2D CB1_1	7.522	7.600	0.078
CB1 1109	2D CB1_2	7.096	7.154	0.058
CB1 1172	2D CB1_4	6.418	6.517	0.099
CB2 1103	2D CB2_1	4.690	4.693	0.003
CB2 1162	2D CB2_2	4.363	4.341	-0.022
CB2 1220	2D CB2_3	3.979	3.962	-0.017
CB2 1280	2D CB2_5	3.250	3.350	0.100
CB3 1082	2D CB3_1	7.769	7.624	-0.145
CB3 1141	2D CB3_2	7.358	7.251	-0.107
CB3 1172	2D CB3_3	7.090	6.962	-0.128
CB4 1075	2D CB4_1	7.404	7.411	0.007
CB4 1124	2D CB4_2	6.949	6.982	0.033
CB4 1197	2D CB4_3	6.121	6.243	0.122
CB4 1326	2D CB4_5	4.482	4.523	0.041
CB4 1368	2D CB4_6	4.074	4.038	-0.036

Water levels across the 2D model are generally higher than the refined 1D model. As can be seen from Table 5.3 there are some exceptions to this. The 2D model has lower water levels than the 1D in branches CB2 and CB3 where the 1D model has considerably more flow as shown in Section 5.3.2 below. Branch SB2 in the refined 1D model is also shown to have considerably more flow than the 2D model in Section 5.3.2. However, Table 5.3 reveals that the water levels are greater in the 2D model. This is due to the water breaking out of the eastern channel (MB3) in the 2D model and slowly flowing in a south east direction. Though the discharges are lower there is still a reasonable amount of water flowing that way but at a much lower velocity. The maximum area-averaged velocity for SB2 1124 for the refined 1D model is 0.62 m/s while the maximum area-averaged velocity in the 2D model was 0.13 m/s. However at the location of SB2 1124 in the 2D model the local velocity is comparable to the 1D model with a maximum localised velocity of 0.55 m/s. This is, however, over a very small portion of that section. This very small portion of that section is the area in which the water levels exceed the water levels of the 1D model.

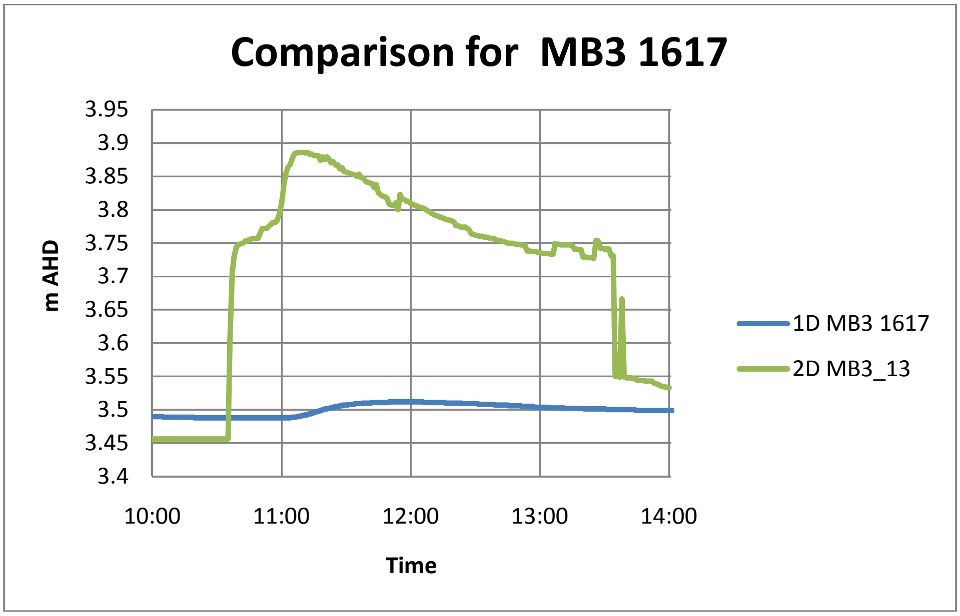


Figure 5.11 – Q100 WL Comparison for MB3 1617

Figure 5.11 demonstrates a significant difference in the refined 1D and 2D models. The water levels are so much greater at this location due to the refined 1D model not representing the breakouts and minor flow paths similar to the 2D model. There is significantly less flow through the downstream portion of MB3, hence the major difference in flood level above. Section 5.3.2 discusses the difference between the refined 1D and 2D models around MB3 in more detail.

The apparent difference in initial water levels is due to the initial conditions within the refined 1D model.

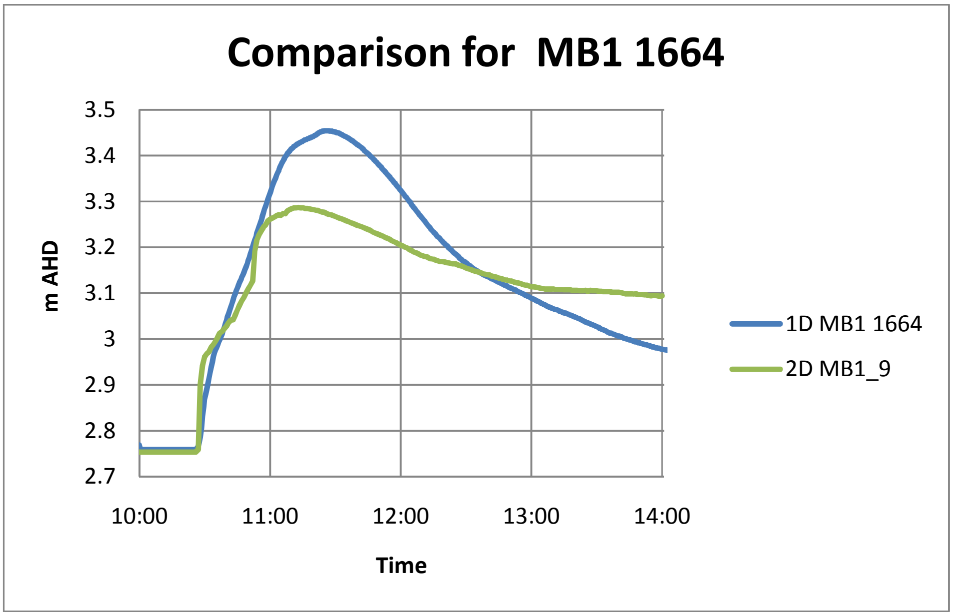


Figure 5.12 – Q100 WL Comparison for MB3 1664

Figure 5.12 represents the water levels over time of the downstream cross section within the prominent flow path. The shape of the two graphs on Figure 5.12 are noticeably different. The 2D water levels-time graph has a much lower peak but the rate of decline after the peak is much lower. This is due to the 2D model having the entirety of the northern boundary to spread the flows through while the 1D models are only confined to represent flows within the bounds of the cross section.

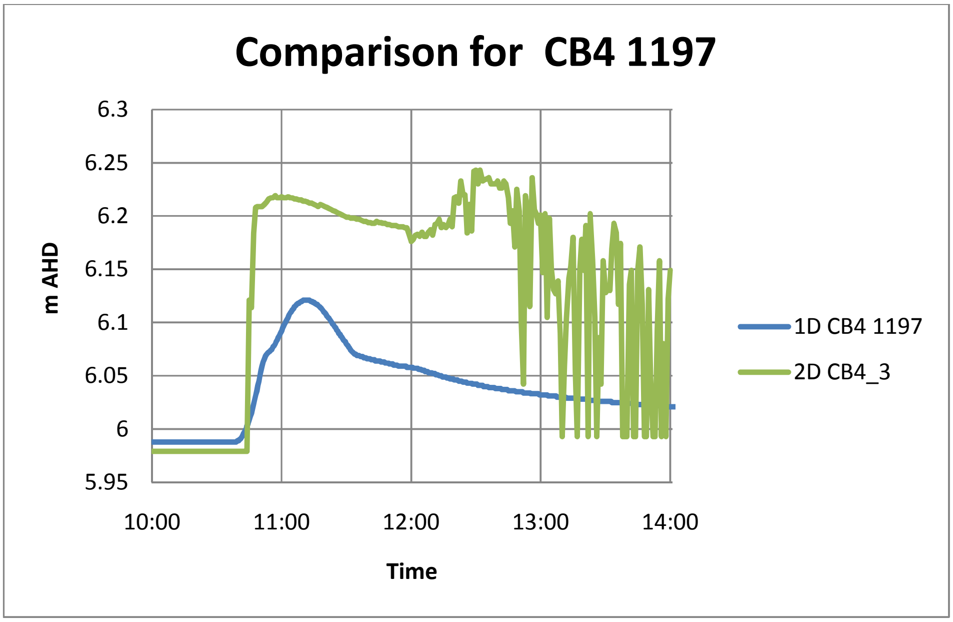


Figure 5.13 – Q100 WL Comparison for CB4 1197

The comparison of water levels at CB1197 in Figure 5.13 demonstrates a marked difference in the two models. As discussed in Section 5.2 the TUFLOW print output lines assume the flow path. Again the representation of this water surface level-time graph shows the model assuming a flow path at different locations. Inspection of the 2D model graphical time series revealed that this has resulted from water ponding on the site in a 'dip' or 'pool' of water. As the flows drain from the secondary flow path the CB4 represents the water held within the model due to the spatial variance in ground levels representing a low spot. The water was conveyed to that location when there was sufficient energy however as it slowly drains there is not sufficient energy for the water to continue. This phenomenon is more obvious in the farm dams located in MB1, MB2, MB3, SB1 and SB2 but is also evident throughout the site. The 2D model is only able to account for such features if the survey is sufficiently detailed and the grid spacing is fine enough.

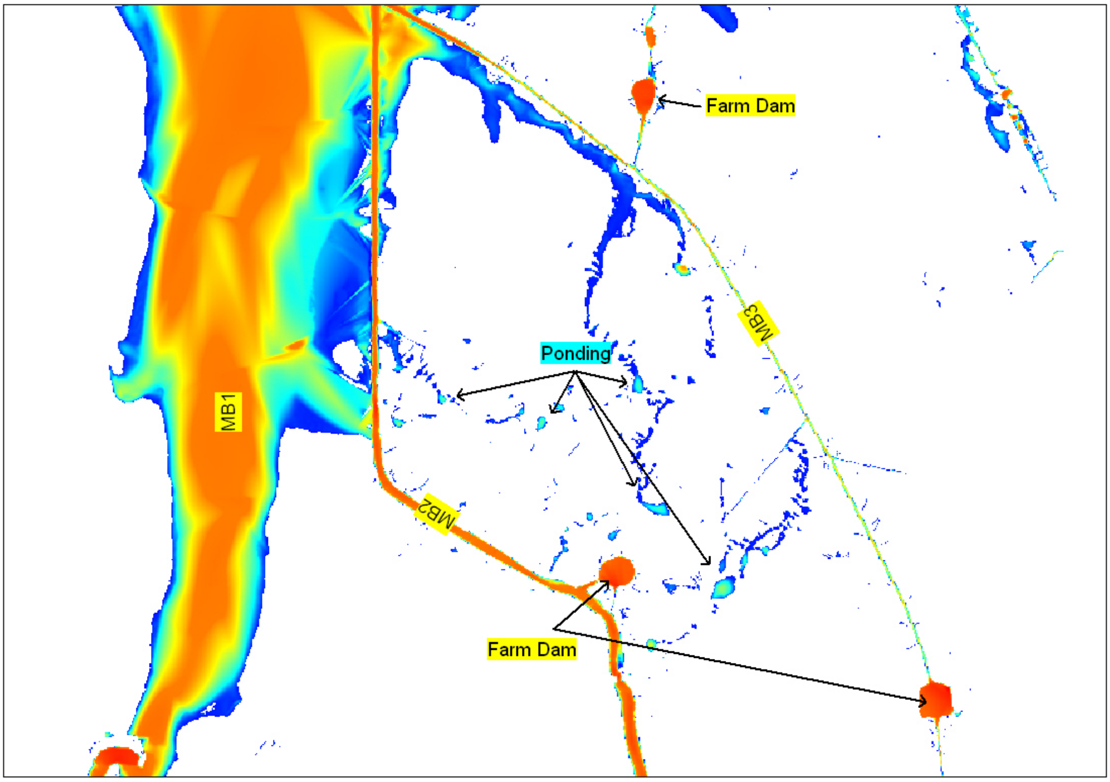


Figure 5.14 – Ponding in 2D Model Output @ 2.95 hrs (NTS)

5.3.2 Comparison of 100 Year ARI Discharges

A comparison of the peak discharges for the refined 1D and 2D models is contained in Table 5.4. As was the case with the comparisons to the refined 1D model the upstream cross sections close to the inflow boundary conditions have similar peak values. This can also be seen by inspection of Figure 5.16. Table 5.4 does, however, show significant differences in peak discharges. In particular along MB3 where there are many connections. As a result there are noteworthy differences in the connected branches.

Table 5.4 – Refined 1D & 2D 100 Year Comparison (Discharges)

1D Section Name	2D Location Name	1D Peak Discharges m^3/s	2D Peak Discharges m^3/s	Difference m3/s	Difference%
MB1 1067	2D MB1_2	14.691	14.634	-0.057	-0.4%
MB1 1077	2D MB1_3	14.648	14.617	-0.031	-0.2%
MB1 1089	2D MB1_4	14.631	14.625	-0.006	0.0%
MB1 1245	2D MB1_5	14.483	14.436	-0.047	-0.3%
MB1 1361	2D MB1_6	14.051	25.546	11.495	81.8%
MB1 1469	2D MB1_7	23.668	25.347	1.679	7.1%
MB1 1555	2D MB1_8	23.472	25.212	1.74	7.4%
MB1 1664	2D MB1_9	20.611	26.793	6.182	30.0%
MB2 1101	2D MB2_2	16.205	16.013	-0.192	-1.2%
MB2 1149	2D MB2_3	12.415	15.409	2.994	24.1%
MB2 1173	2D MB2_4	15.104	13.894	-1.21	-8.0%
MB2 1178	2D MB2_5	38.537	13.177	-25.36	-65.8%
MB2 1186	2D MB2_6	17.1	12.411	-4.689	-27.4%
MB2 1207	2D MB2_7	11.579	11.628	0.049	0.4%
MB2 1288	2D MB2_9	10.77	11.541	0.771	7.2%
MB2 1345	2D MB2_10	10.638	11.403	0.765	7.2%
MB3 1055	2D MB3_2	1.522	1.491	-0.031	-2.0%
MB3 1070	2D MB3_3	1.519	1.473	-0.046	-3.0%
MB3 1079	2D MB3_4	1.516	1.463	-0.053	-3.5%
MB3 1091	2D MB3_5	1.517	1.46	-0.057	-3.8%
MB3 1119	2D MB3_6	1.514	1.449	-0.065	-4.3%
MB3 1211	2D MB3_8	1.477	1.419	-0.058	-3.9%
MB3 1278	2D MB3_9	4.867	1.677	-3.19	-65.5%
MB3 1359	2D MB3_10	3.716	1.49	-2.226	-59.9%
MB3 1455	2D MB3_11	0.187	2.977	2.79	1492.0%
MB3 1508	2D MB3_12	0.171	2.521	2.35	1374.3%
MB3 1617	2D MB3_13	0.1	1.045	0.945	945.0%
MB3 1679	2D MB3_14	0.1	2.31	2.21	2210.0%
SB1 1043	2D SB1_1	0.167	1.52	1.353	810.2%
SB1 1056	2D SB1_2	0.166	1.091	0.925	557.2%
SB1 1072	2D SB1_3	0.164	1.178	1.014	618.3%
SB1 1091	2D SB1_4	0.164	0.851	0.687	418.9%
SB1 1102	2D SB1_5	0.164	0.898	0.734	447.6%
SB1 1114	2D SB1_6	0.162	0.735	0.573	353.7%
SB1 1145	2D SB1_7	0.17	0.769	0.599	352.4%

SB1 1189	2D SB1_8	2.323	1.857	-0.466	-20.1%
SB1 1205	2D SB1_9	2.313	2.45	0.137	5.9%
SB1 1235	2D SB1_10	2.235	2.891	0.656	29.4%
SB2 1088	2D SB2_1	3.577	0.376	-3.201	-89.5%
SB2 1124	2D SB2_2	3.523	0.25	-3.273	-92.9%
SB2 1186	2D SB2_3	3.493	0.213	-3.28	-93.9%
SB2 1285	2D SB2_4	3.169	0.211	-2.958	-93.3%
SB2 1442	2D SB2_5	2.558	0.195	-2.363	-92.4%
SB2 1482	2D SB2_6	2.555	0.177	-2.378	-93.1%
SB2 1570	2D SB2_8	2.558	0.142	-2.416	-94.4%
SB2 1639	2D SB2_10	2.853	0.301	-2.552	-89.4%
CB1 1062	2D CB1_1	0.944	2.202	1.258	133.3%
CB1 1109	2D CB1_2	0.552	1.507	0.955	173.0%
CB1 1172	2D CB1_4	0.549	2.579	2.03	369.8%
CB2 1103	2D CB2_1	2.928	1.3	-1.628	-55.6%
CB2 1162	2D CB2_2	2.695	1.414	-1.281	-47.5%
CB2 1220	2D CB2_3	2.505	1.143	-1.362	-54.4%
CB2 1280	2D CB2_5	2.39	1.436	-0.954	-39.9%
CB3 1082	2D CB3_1	3.774	0.586	-3.188	-84.5%
CB3 1141	2D CB3_2	3.745	0.706	-3.039	-81.1%
CB3 1172	2D CB3_3	3.729	0.547	-3.182	-85.3%
CB4 1075	2D CB4_1	3.268	1.404	-1.864	-57.0%
CB4 1124	2D CB4_2	1.063	1.309	0.246	23.1%
CB4 1197	2D CB4_3	1.046	0.833	-0.213	-20.4%
CB4 1326	2D CB4_5	1.018	0.465	-0.553	-54.3%
CB4 1368	2D CB4_6	0.988	0.396	-0.592	-59.9%

Figure 5.15 demonstrates the significant differences in peak discharges along MB3. This shows that the refined 1D model conveys more water through CB3 and subsequently conveys the additional discharges through MB3 1278 then splits a disproportionate amount of flow to SB2 whilst the remainder of the additional flow is conveyed through MB3 1359 and then through CB2. Not only is the remainder of the additional MB3 flows taken through CB2 but almost all the MB3 flow from upstream of CB2 is taken down the CB2 branch. This results in very little flow in the downstream portion of MB3.

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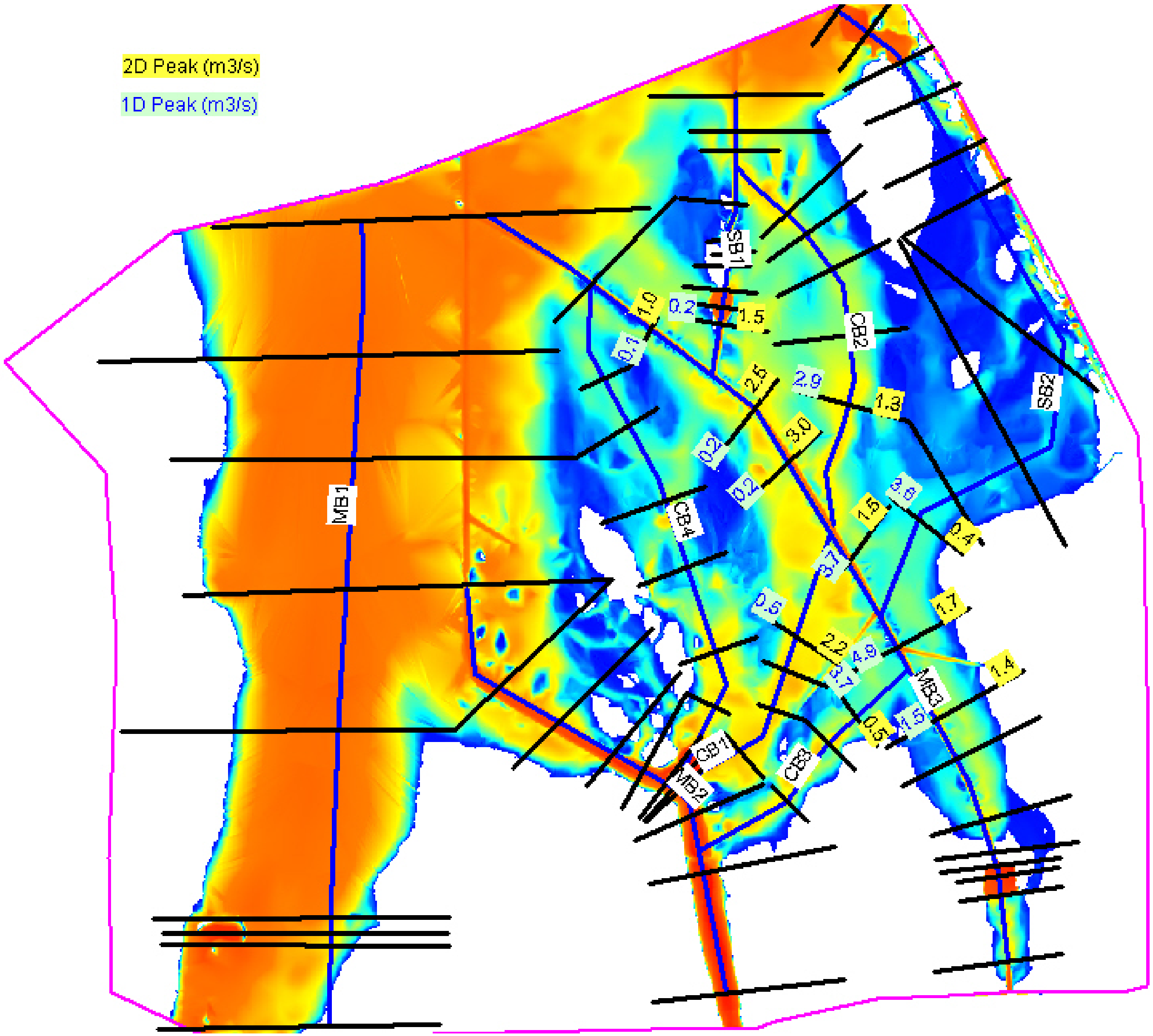

Figure 5.15 – Refined 1D-2D 100 Year ARI Discharges around Branch MB3

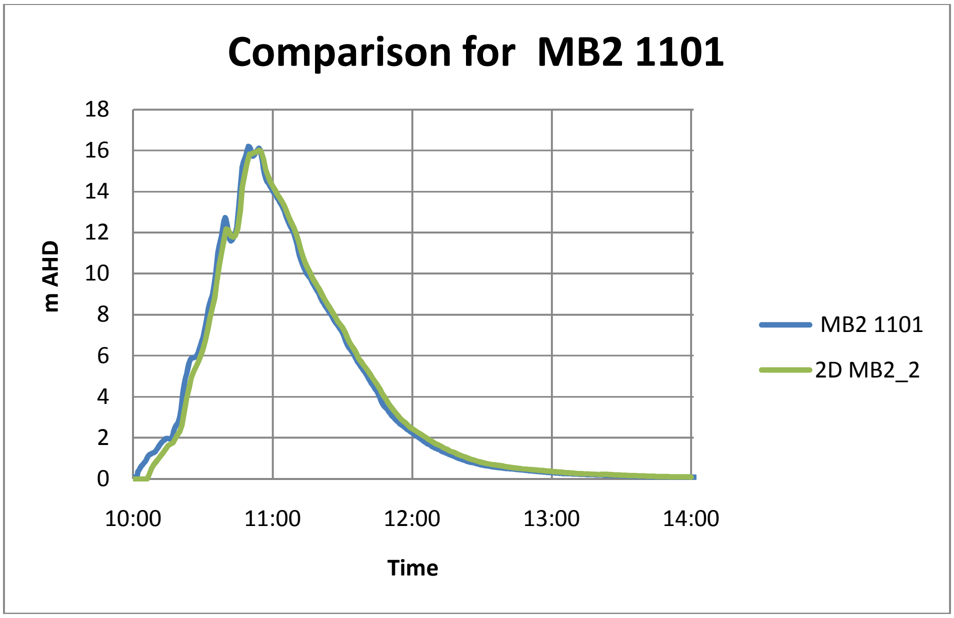

Figure 5.16 – Q100 Q Comparison for MB2 1101

As mentioned previously the peak discharges at MB2 1101 are comparable for the refined 1D and 2D models. Figure 5.16, also demonstrates that the shape of the discharge hydrograph for both models is very much comparable.

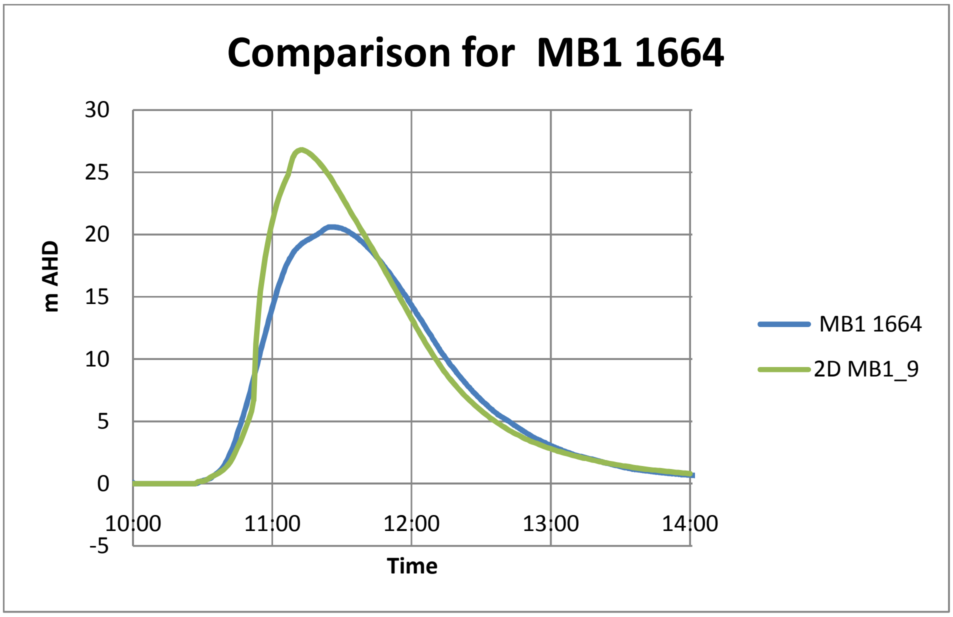

Figure 5.17 – Q100 Q Comparison for MB1 1664

Figure 5.17 above compares discharges for the refined 1D and 2D models for the most prominent major branch. It can be seen that there is a significant difference in both the peak and the timing of the peak. This is attributed to more flow being conveyed through SB2 and attenuation as the flows split and are routed through the many different branches.

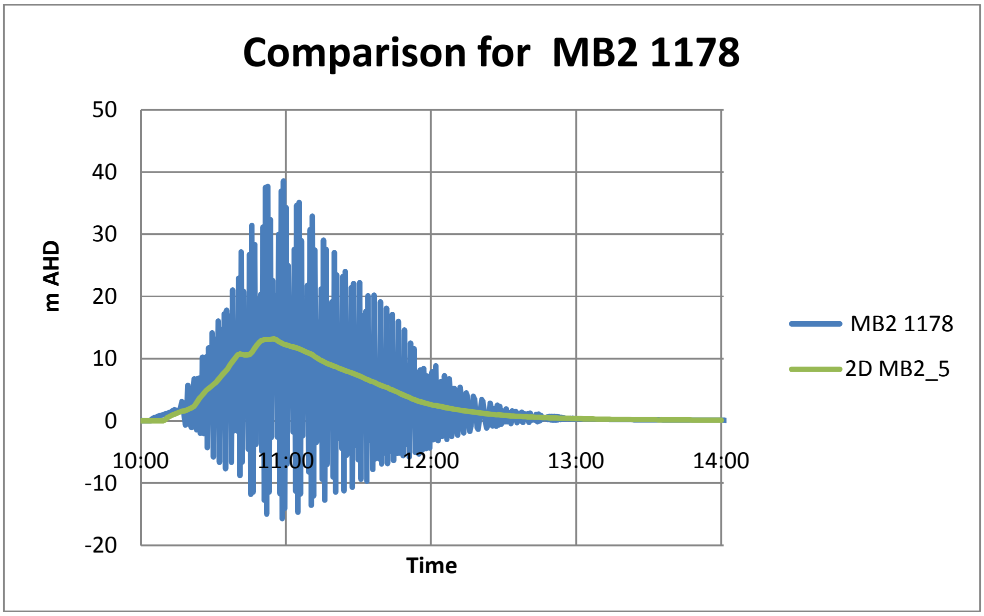

Figure 5.18 – Q100 Q Comparison for MB2 1178

The refined 1D model did contain inherent instabilities due to the close connection of various branches. This was somewhat unavoidable due to a limit of 77 h points within the model. The distance between cross sections is also required to be similar in order to yield reasonable results. Figure 5.18 shows the major model instability that was detected. The graph presents a significant oscillation in the hydrograph. This is due to the position of two branches off MB2 at this specific location. If further 1D modelling was undertaken the connection location could be repositioned by a small amount either side of 1178. It is envisaged that the implementation of this strategy would reduce the instability. However, in order to do this two more h points will be used. This will result in a reduction of two more sections in the model as h points are automatically created by MIKE11 when a connection is inserted, even if there is no cross section at this location.

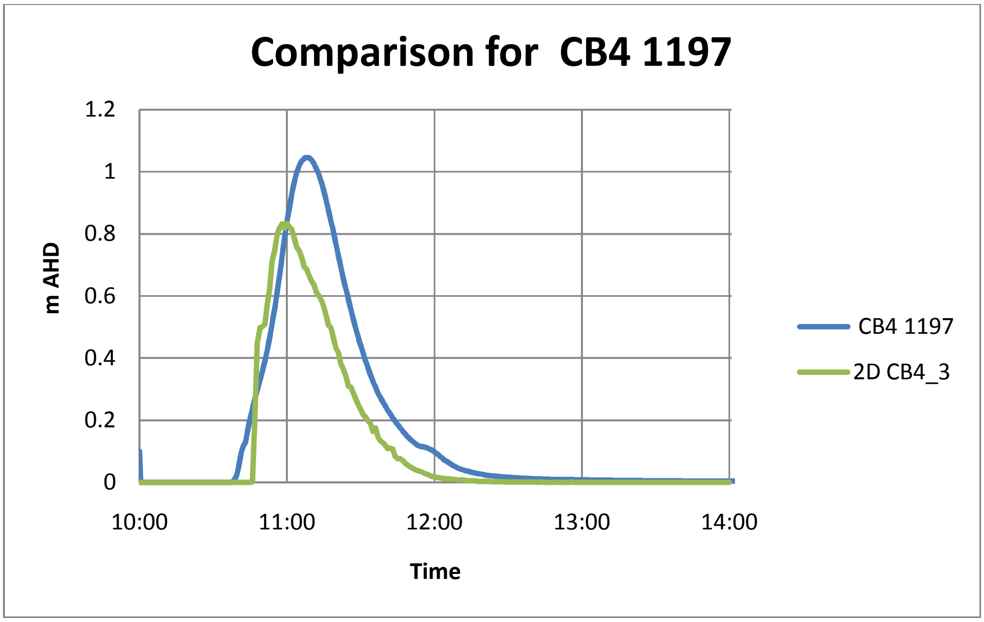

Figure 5.19 – Q100 Q Comparison for CB4 1197

The comparison of CB4 1197 in Figure 5.19 demonstrated that through the connection branch CB4 the discharges are approximately 20% greater in the 1D model. It can be seen, however, that the shape of the hydrograph is similar.

5.4 COMPARISON FOR SMALLER ARI EVENTS

As mentioned previously the models were run for the 1, 2, 5, 10, 20 and 50 year ARI events as well as the 100 year ARI event. The peak discharges for each of the events is progressively less. Therefore, as the ARI event gets smaller the flood behaviour approaches that of the 1 year ARI event. As a result of this the 1 year ARI event will be analysed in this section to demonstrate the differences between the 1D and 2D modelling for the smaller event on the subject site.

It was found that the minor flow paths of CB1, CB2, CB3 and SB2 were not utilised in the smaller 1 year ARI event for the 2D model. Though there was water within the SB2 portion of the site this was only backwater from the eastern waterway and the downstream portion of the site. It was not found that water flowed entirely down the SB2 branch in the 2D model. Figure 5.20 below shows the maximum depths for the 2D model for the 1 year ARI event. It can be seen that the flow is mostly contained within the defined channels. This is an indication that such a flood event does not require 2D modelling but that a 1D model would suffice. Notwithstanding this philosophy there is still significant difference in both the water level and discharges calculated between the 2D model and the 1D models. Table 5.5 and Table 5.6 below contain peak water levels and peak discharges comparisons for the simple 1D and the 2D model. Table 5.7 and Table 5.8 contain peak water levels and peak discharges comparisons the refined 1D model to the 2D model.

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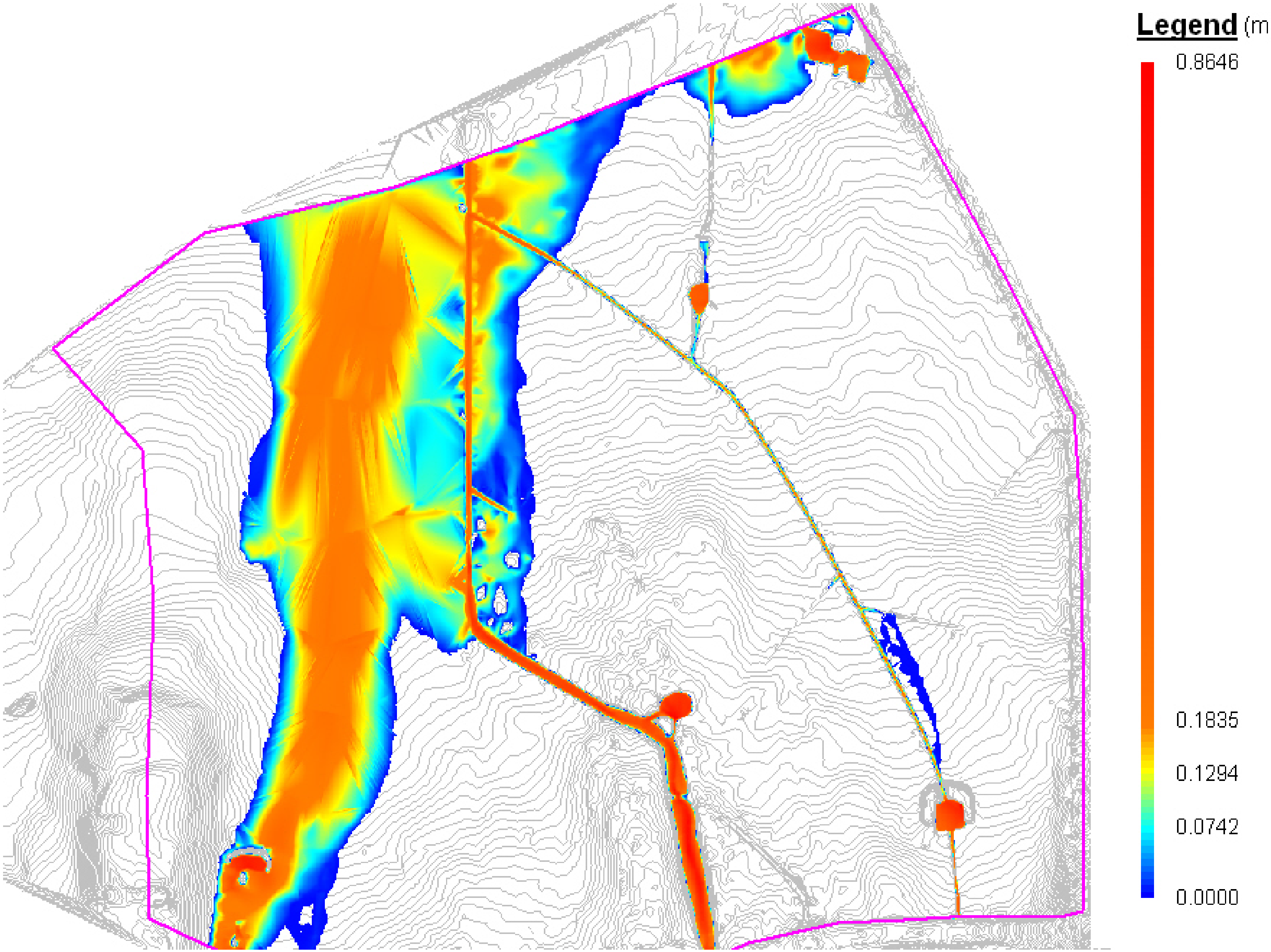

Figure 5.20 – 2D 1 Year ARI Maximum Flood Depths

Table 5.5 – Simple 1D & 2D 1 Year Comparison (Water Levels)

Chainage	1D Peak Water Level m AHD	2D Peak Water Level m AHD	Difference m
1000	7.883	8.695	0.812
1134	7.609	7.754	0.145
1237	6.998	7.056	0.058
1348	5.953	6.164	0.211
1507	4.706	5.015	0.309
1698	3.27	3.414	0.144
2000	7.654	7.784	0.13
2096	7.041	6.996	-0.045
2259	5.272	5.951	0.679

Table 5.6 – Simple 1D & 2D 1 Year Comparison (Discharges)

Chainage	1D Peak Discharge m^3/s	2D Peak Discharge m^3/s	Difference m^3/s	Difference %
1134	4.326	4.244	-0.082	-1.9%
1237	4.275	4.213	-0.062	-1.5%
1348	4.273	4.205	-0.068	-1.6%
1507	7.616	7.339	-0.277	-3.6%
1698	7.478	7.415	-0.063	-0.8%
2096	3.502	3.502	0	0.0%
2259	3.471	3.492	0.021	0.6%

Table 5.4 and Table 5.5 identicate that there is generally lower discharges in the 2D model however the water levels are generally higher. This is due to the water in the simple 1D model not having a comparitable attenuation. Therefore the water moves at a greater velocity and does not generate water levels as high. The 812 mm increase in the 2D model at chainage 1000 is due to the representation of the eastern channel in the 2D model as was the case in the 100 year ARI event.

Table 5.7 – Refined 1D & 2D 1 Year Comparison (Water Levels)

1D Section Name	2D Location Name	1D Peak Water Levels m AHD	2D Peak Water Levels m AHD	Difference m
MB1 1000	2D MB1_1	7.647	7.750	0.103
MB1 1067	2D MB1_2	7.149	7.296	0.147
MB1 1077	2D MB1_3	7.017	7.235	0.218
MB1 1089	2D MB1_4	6.961	6.996	0.035
MB1 1245	2D MB1_5	5.964	5.955	-0.009
MB1 1361	2D MB1_6	5.272	5.452	0.180
MB1 1469	2D MB1_7	4.346	4.398	0.052
MB1 1555	2D MB1_8	3.729	3.778	0.049
MB1 1664	2D MB1_9	3.164	3.169	0.005
MB2 1000	2D MB2_1	7.973	8.061	0.088
MB2 1101	2D MB2_2	7.781	7.952	0.171
MB2 1149	2D MB2_3	7.525	7.710	0.185
MB2 1173	2D MB2_4	7.447	7.523	0.076
MB2 1178	2D MB2_5	7.391	7.517	0.126
MB2 1186	2D MB2_6	7.383	7.507	0.124
MB2 1207	2D MB2_7	7.268	7.343	0.075
MB2 1288	2D MB2_9	6.577	6.661	0.084
MB2 1345	2D MB2_10	6.010	6.167	0.157
MB3 1000	2D MB3_1	8.619	8.696	0.077
MB3 1055	2D MB3_2	7.946	8.095	0.149
MB3 1070	2D MB3_3	7.946	8.096	0.150
MB3 1079	2D MB3_4	7.946	8.096	0.150
MB3 1091	2D MB3_5	7.935	8.015	0.080
MB3 1119	2D MB3_6	7.418	7.593	0.175
MB3 1211	2D MB3_8	6.055	6.847	0.792
MB3 1278	2D MB3_9	5.702	6.299	0.597
MB3 1359	2D MB3_10	5.504	5.708	0.204
MB3 1455	2D MB3_11	4.462	4.806	0.344
MB3 1508	2D MB3_12	4.145	4.454	0.309
MB3 1617	2D MB3_13	3.496	3.687	0.191
MB3 1679	2D MB3_14	3.144	3.280	0.136

Table 5.8 – Refined 1D & 2D 1 Year Comparison (Discharges)

1D Section Name	2D Location Name	1D Peak Discharges m^3/s	2D Peak Discharges m^3/s	Difference m^3/s	Difference%
MB1 1067	2D MB1_2	3.504	3.505	0.001	0.0%
MB1 1077	2D MB1_3	3.501	3.503	0.002	0.1%
MB1 1089	2D MB1_4	3.501	3.502	0.001	0.0%
MB1 1245	2D MB1_5	3.488	3.492	0.004	0.1%
MB1 1361	2D MB1_6	3.444	7.328	3.884	112.8%
MB1 1469	2D MB1_7	7.003	7.281	0.278	4.0%
MB1 1555	2D MB1_8	6.969	7.253	0.284	4.1%
MB1 1664	2D MB1_9	6.651	7.381	0.73	11.0%
MB2 1101	2D MB2_2	4.057	4.008	-0.049	-1.2%
MB2 1149	2D MB2_3	3.854	4.007	0.153	4.0%
MB2 1173	2D MB2_4	4.556	4.008	-0.548	-12.0%
MB2 1178	2D MB2_5	13.262	4.011	-9.251	-69.8%
MB2 1186	2D MB2_6	6.26	4.006	-2.254	-36.0%
MB2 1207	2D MB2_7	3.991	4.003	0.012	0.3%
MB2 1288	2D MB2_9	3.769	4	0.231	6.1%
MB2 1345	2D MB2_10	3.752	4.002	0.25	6.7%
MB3 1055	2D MB3_2	0.266	0.26	-0.006	-2.3%
MB3 1070	2D MB3_3	0.262	0.254	-0.008	-3.1%
MB3 1079	2D MB3_4	0.261	0.254	-0.007	-2.7%
MB3 1091	2D MB3_5	0.261	0.253	-0.008	-3.1%
MB3 1119	2D MB3_6	0.26	0.253	-0.007	-2.7%
MB3 1211	2D MB3_8	0.246	0.253	0.007	2.8%
MB3 1278	2D MB3_9	0.34	0.253	-0.087	-25.6%
MB3 1359	2D MB3_10	0.11	0.252	0.142	129.1%
MB3 1455	2D MB3_11	0.1	0.252	0.152	152.0%
MB3 1508	2D MB3_12	0.1	0.252	0.152	152.0%
MB3 1617	2D MB3_13	0.1	0.201	0.101	101.0%
MB3 1679	2D MB3_14	0.1	0.201	0.101	101.0%

The key differences between the refined 1D model and the 2D models for the 1 year ARI event are:

1. The same difference as the 100 year at the downstream end of MB3 where the refined 1D model does not let water flow beyond MB3 1359 (0.1 m^3/s was the initial condition throughout the refined 1D model);
2. The 2D model combines the discharges of MB1 and MB2 before the refined 1D model, hence the difference in discharges at MB1 1361

3. Again similarly to the 100 year ARI event there is evidence of the instability at MB2 1178.
4. The water levels are generally lower in the refined 1D model.

It is apparent that for the smaller rainfall events the 2D model is yielding higher water levels. One of the great advantages of a 2D model is the ability to have a spatial variance in flood characteristics. Since the flows are contained mainly within the channels this is not a significant advantage. The higher water level in the 2D model is most likely due to the grid not having enough definition through the channel as it is limited only to where a grid point is located for height information. In comparison, the 1D model can have any definition required.

It can be concluded that for smaller flows on the subject site a 1D model of the flow paths would be most appropriate. However, TUFLOW does have the capacity to embed a 1D channel through the 2D grid (WBM BMT 2007).

6 CONCLUSIONS

Three hydrodynamic models were conducted on the site located at 166 Parklands Boulevard, Meridian Plains, Queensland. These models consisted of a simple 1D MIKE11 model a 2D TUFLOW model and a refined 1D MIKE11 model based on the graphical output of the TUFLOW model. The purpose of creating the three hydrodynamic models was to compare the data output between 1D and 2D modelling and ascertain which method provided a more complete picture of the flooding characteristics for the area. The time spent to produce these models was assessed to determine which model was more time consuming.

Inflow boundary conditions for all modelling were created using the RAFTS hydrologic model and calibrating the model to Rational Method calculations. Discharge hydrographs from the RAFTS modelling were subsequently used directly in all hydrodynamic models.

Hydrodynamic models were produced based on topographic information provided by MRG Water Consulting, aerial photography and boundary conditions.

Topographic information was represented in the simple 1D model based on an inspection of the survey data provided. Cross sections were then entered manually. The topographic condition of the site was represented in the 2D model based on a DEM generated from the survey data. The DEM was created utilising the MapInfo add-on Vertical Mapper. It was found that the DEM creation was sensitive to the scheme used in Vertical Mapper. Cross sections for the refined 1D model were obtained directly from the DEM by the use of an automatic cross section generator in the TUFLOW suite. This provided a time effective means of producing the refined 1D model. However, this time saving process may not be utilised in an industry setting as a DEM would generally not be created if only 1D modelling was intended.

It was found that the most time effective model to create was the TUFLOW model. The key difference in model set up time was the representation of the topographic conditions within the model. It was established that generating a DEM is less time consuming than the creation of cross sections. This is dependent on factors such as survey data being in a format that is easily translated to a DEM. The results indicate that there is a direct correlation between the number of cross sections and model set up time.

The refined 1D hydrodynamic model was found to have inherent instabilities due to the multiple branch linkages within the model. In particular there are significant calculation instabilities at the location where two branches split off a main branch. This is not an indication of a weakness in the MIKE11 modelling but rather it highlights a constraint. In the MIKE11 model set up there must be at least one cross section between locations where multiple flow paths split from a branch. This constraint is not evident in the TUFLOW model of this location as the model was setup utilising the DEM and a fine grid as opposed to cross sections.

The analysis of the data output from the 1D and 2D hydrodynamic modelling shows that generally the 2D model distributes the water more effectively throughout the site. The 2D hydrodynamic TUFLOW model can allow for phenomena such as ponding of water. This is due to the ability for the model to allow for variance within the XY plane of height and roughness. As a result there is a more comprehensive indication of flood characteristics throughout the site. The 2D model in comparison to the 1D hydrodynamic model allows for a greater definition of material roughness and elevation as it is not confined to specific sections.

The TUFLOW print output lines are helpful for an indication of the flood behaviour but the graphical time series output which is viewed through post processing software such as SMS provides a substantially more comprehensive summary of the results. This allows the modeller to see the flooding on the complete site whilst analysing individual elements. However, due to the nature of 1D modelling results can be viewed at specific sections and a complete indication of flood characteristics can be obtained. This is particularly helpful when analysing specific locations. In this respect the 1D model is superior.

Investigation of the output from the storm events with lower discharges demonstrated that if flows are contained within a defined channel a 1D hydrodynamic model is sufficient to obtain flood characteristics. In particular if a more time effective method for cross section determination is employed the 1D modelling would be more valuable due to its ability to have any definition required of the channel.

In conclusion the study comparing 1D and 2D hydrodynamic modelling revealed that when it is anticipated that there will be significant interaction between flow paths or breakout from channels a 2D hydrodynamic model will provide a more comprehensive indication of flood characteristics throughout the study area. In comparison, it was determined that if it is anticipated that flow will be confined within a channel(s) 1D hydrodynamic modelling would provide more valuable results.

6.1 RECOMMENDATIONS

It is recommended that additional study be carried out to further compare 1D and 2D hydrodynamic models the recommended additional features include:

- **A study of a large creek or river system.** Large creek/river systems will have locations where the flow is generally 1D and other locations where there is breakouts and interaction with minor flow paths. A study of this nature would contain elements that highlights the strength of both models and would provide a clearer comparison.
- **Modelling that includes hydraulic structure**. A study that includes hydraulic structures such weirs, bridges and culverts would provide a further comparison of the two models as it would highlight their interaction with these structures.

- **TUFLOW Model with 1D component.** TUFLOW has the capability to nest a 1D component containing cross sections within a 2D grid. This allows definition within areas where flow is generally 1D in nature and accounts for the 2D characteristics of flows outside the bounds of the 1D component. A TUFLOW model set up in such a way would have the strengths of both 1D and 2D modelling. Therefore this would provide an effective comparison of the capabilities of each model.
- **Calibrate to a historic flood event.** Both models could be calibrated to a historic flood event at key locations. The advantage of this would be that flood characteristics at locations other than the calibration points could be compared. Furthermore, the changes in flood characteristics as design discharges are input into the models could be observed. This would provide a stronger comparison as a result of more similarities in the models due to the calibration.

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APPENDICES

Appendix A

Project Specification

University of Southern Queensland

FACULTY OF ENGINEERING AND SURVEYING

ENG 4111/4112 Research Project
PROJECT SPECIFICATION

FOR:	**Jeremy Cox**
TOPIC:	INVESTIGATION OF THE PRACTICALITY AND ACCURACY OF A 2D HYDRODYNAMIC MODEL
SUPERVISORS:	Ian Brodie; Mark Gibson – Director/Hydraulic & Hydrologic Engineer MRG Water Consulting Pty Ltd; Darren Rogers – Director/Hydraulic & Hydrologic Engineer Storm Water Consulting Pty Ltd.
ENROLMENT:	ENG 4111 – S1, 2010; ENG 4112 – S2, 2010
PROJECT AIM:	To investigate the accuracy and practicality of a two-dimensional hydrodynamic model in modelling flood impacts and characteristics on a proposed residential development under both pre and post development conditions.
SPONSORSHIP:	MRG Water Consulting Pty Ltd. Storm Water Consulting Pty Ltd.
CONFIDENTIALITY:	While this research project is solely the work of this student the information and data used is the property of MRG Water Consulting Pty Ltd as this company is the source of information, data, software and all other project specific resources required for this project.

PROGRAMME: **Issue A, 20th March 2010**

1. Research information regarding two-dimensional and one-dimensional hydraulic modelling. Research will be directed towards the applications and practicality of utilising each type of model for open channel and flood plain modelling.
2. Investigate catchment hydrology for the study site and obtain rainfall and statistical peak discharge information for the site and upstream catchment.
3. Obtain discharge hydrographs for key locations at the upstream end of the site to be utilised as boundary conditions in the hydraulic models. Hydrographs will be generated by the use of a hydrologic/flood routing model and the rainfall and peak discharge information obtained in step.
4. Extract cross sections from survey data and civil design information for pre and post development conditions
5. Generate three dimensional surfaces for pre and post development conditions from survey data and civil design information
6. Construct a one-dimensional and two-dimensional hydrodynamic model of the site for pre and post conditions and run for the critical 100 year Average Recurrence Interval discharges/rainfall event.
7. Compare the accuracy and output extent of the one-dimensional and two-dimensional models and draw conclusions as to the need and practicality of each model

As time permits:

8. Propose alternative cross section locations for the one-dimensional model to improve accuracy of output.
9. Investigate coincident flooding and alternative tailwater conditions for both the one-dimensional and two-dimensional models

AGREED: ____________ (Student) UBrodie ____________ (Supervisors)

Date: 31 / 3 /2010 Date: 31 / 03 /2010

Appendix B

Rational Method Calculations

RATIONAL METHOD CALCULATIONS

Project: Parklands Bvd Meridan Plains
Location of Discharge: Point 1
Catchment Condition: Existing Site/Developed upstream
Other Comments:

Time of Concentration	70.5	minutes			
	Rural	Urban		Total	
Sub-Catchment Areas	115.855	42.952		158.81	ha
C10 Runoff Coefficients	0.70	0.80			

ARI (years)	Rainfall Intensity (mm/hr)	Depth (mm)	Fy	Runoff Coefficients Rural	Urban	0	Discharges (cumecs) Rural	Urban	0	TOTAL
1	□□	□2	0□□0	0□□□	0□□□	0□00	□□□□□	2□7□□	0□000	**9.19**
2	□□	□□	0□□□	0□□0	0□□□	0□00	□□770	□□71□	0□000	**12.49**
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10	□□	7□	1□00	0□70	0□□0	0□00	1□□□□□	□□20□	0□000	**20.85**
20	7□	□7	1□0□	0□7□	0□□□	0□00	17□□□□	7□□□0	0□000	**25.03**
□0	□7	102	1□1□	0□□1	0□□2	0□00	22□□□□	□□□□0	0□000	**32.09**
100	□□	11□	1□20	0□□□	0□□□	0□00	2□□0□2	11□0□□	0□000	**37.08**

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Table B1

RATIONAL METHOD CALCULATIONS

Project: Parklands Bvd Meridan Plains

Location of Discharge: Point 1
Catchment Condition: Fully Developed
Other Comments:

Time of Concentration	60.6	minutes			
	Urban	Rural		Total	
Sub-Catchment Areas	93.766	65.041		158.81	ha
C10 Runoff Coefficients	0.80	0.70			

ARI (years)	Rainfall Intensity (mm/hr)	Rainfall Depth (mm)	Fy	Runoff Coefficients Urban	Runoff Coefficients Rural	Runoff Coefficients 0	Discharges (cumecs) Urban	Discharges (cumecs) Rural	Discharges (cumecs) 0	TOTAL
1	□□□□	□0	0□□0	0□□□	0□□□	0□00	□□□□1	□□□7□	0□000	**10.53**
2	□0	□1	0□□□	0□□□	0□□0	0□00	□□□□□	□□□7□	0□000	**14.23**
□	□□	□□	0□□□	0□7□	0□□7	0□00	12□□71	7□□□□	0□000	**20.04**
10	71	72	1□00	0□□0	0□70	0□00	1□7□□	□□□7□	0□000	**23.77**
20	□1	□2	1□0□	0□□□	0□7□	0□00	17□722	10□7□□	0□000	**28.48**
□0	□□	□□	1□1□	0□□2	0□□1	0□00	22□7□□	1□□□17	0□000	**36.58**
100	10□	10□	1□20	0□□□	0□□□	0□00	2□2□□	1□□□□□	0□000	**42.19**

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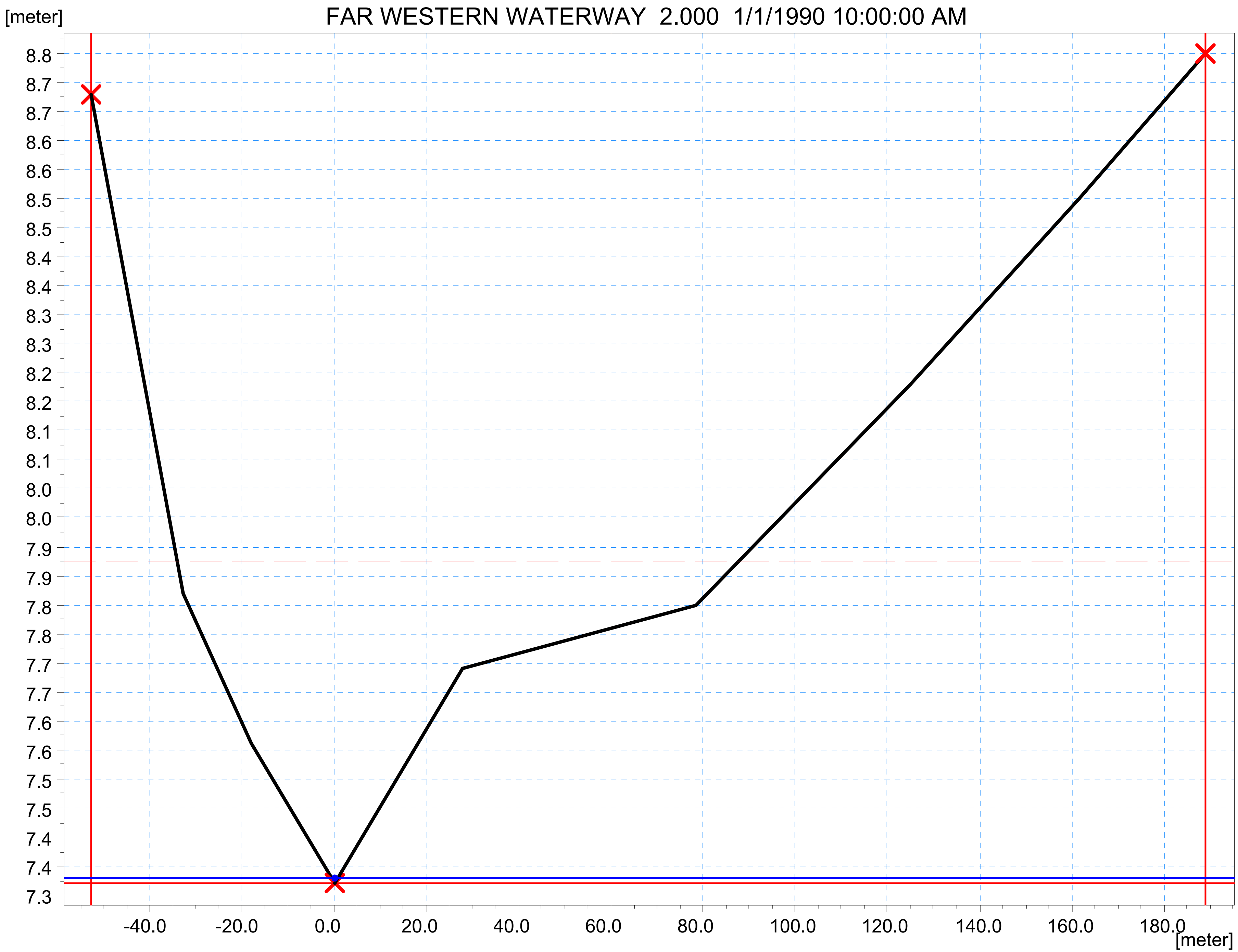
Table B2

Appendix C

1D Cross Section



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8.5
8.5
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8.4
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8.3
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8.2
8.1
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8.0
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7.9
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7.6
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7.4
7.4
7.3
-40.0
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20.0
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120.0
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180.0
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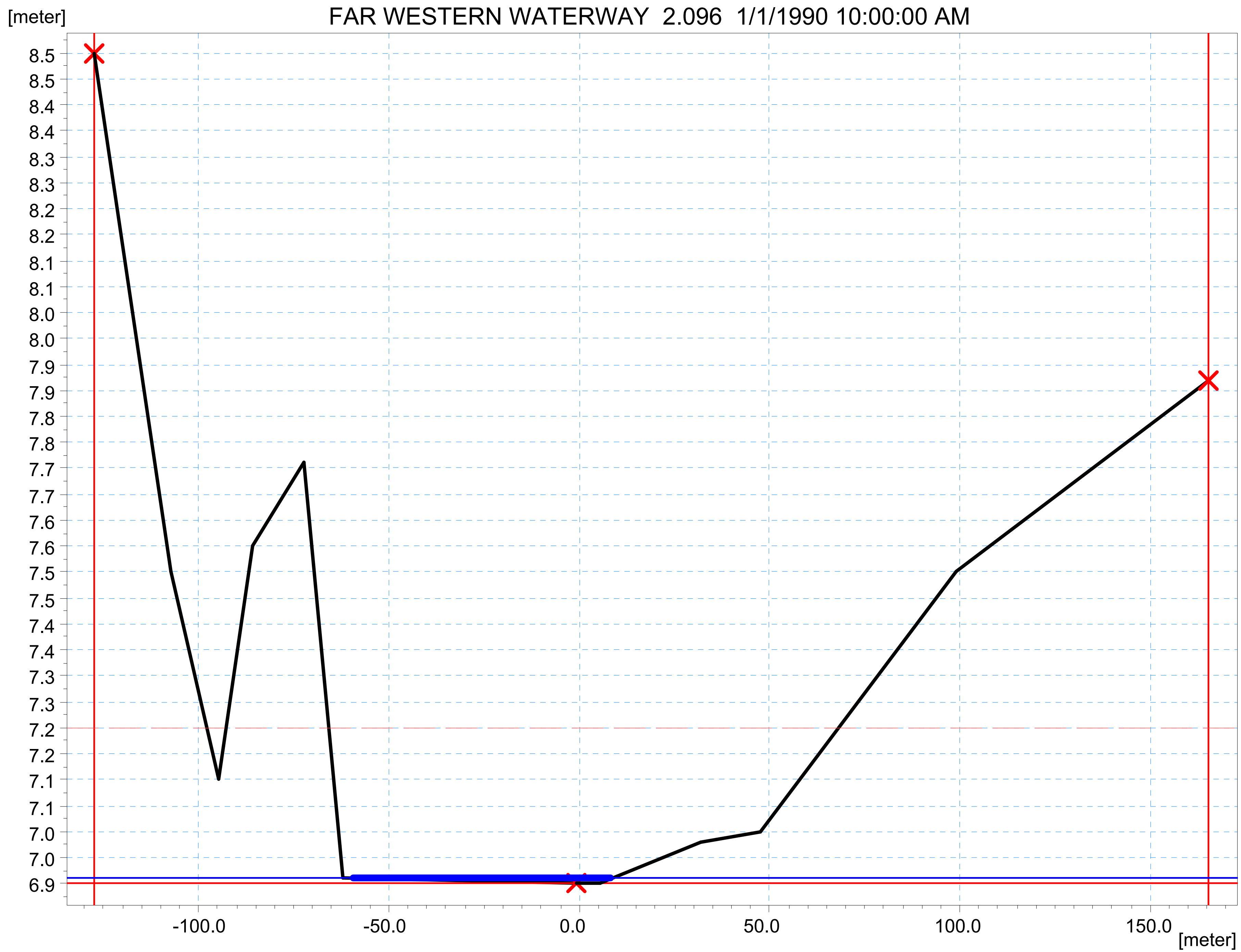

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7.9
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7.3
7.2
7.2
7.1
7.1
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7.0
6.9
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0.0
50.0
100.0
150.0
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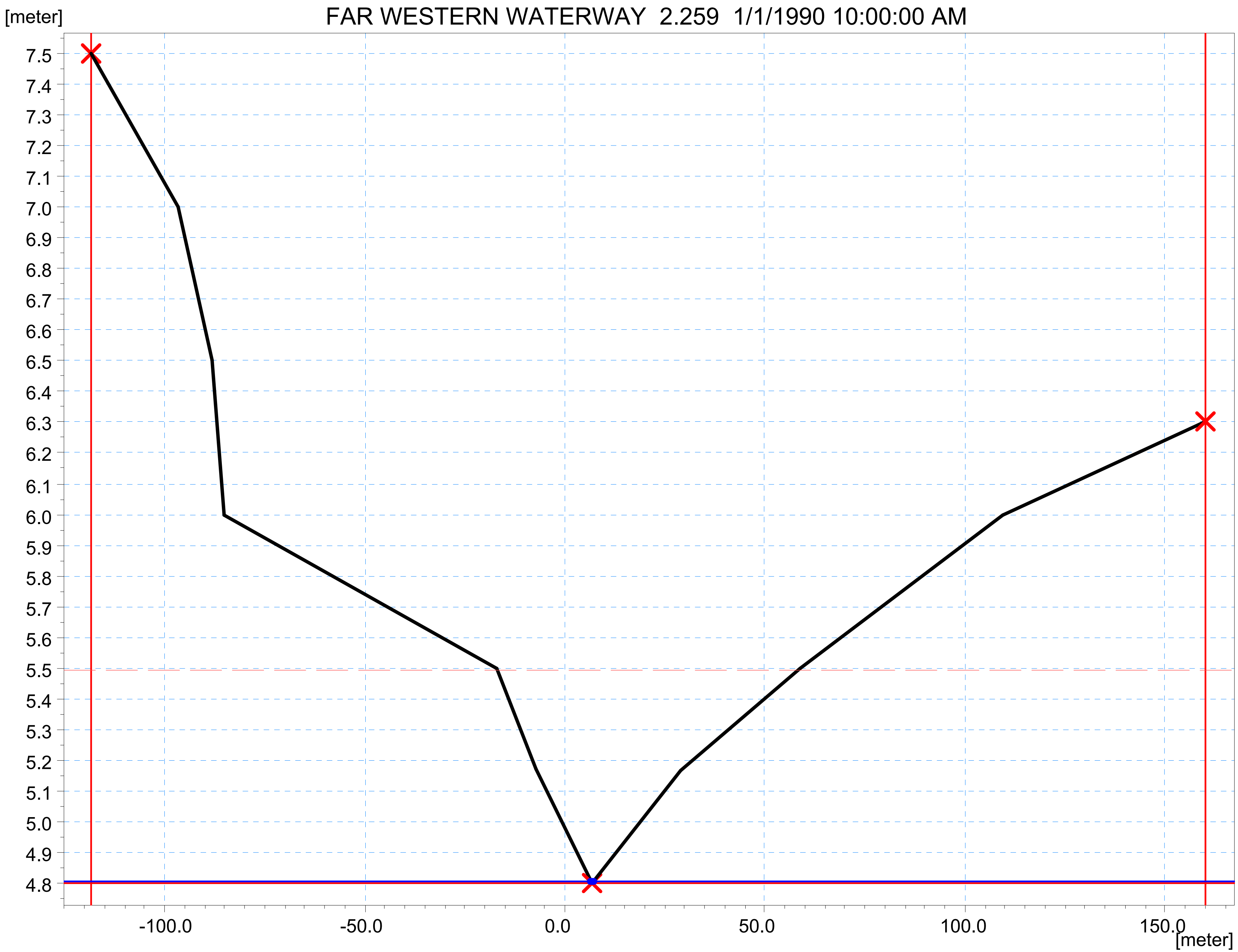

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7.2
7.1
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6.1
6.0
5.9
5.8
5.7
5.6
5.5
5.4
5.3
5.2
5.1
5.0
4.9
4.8
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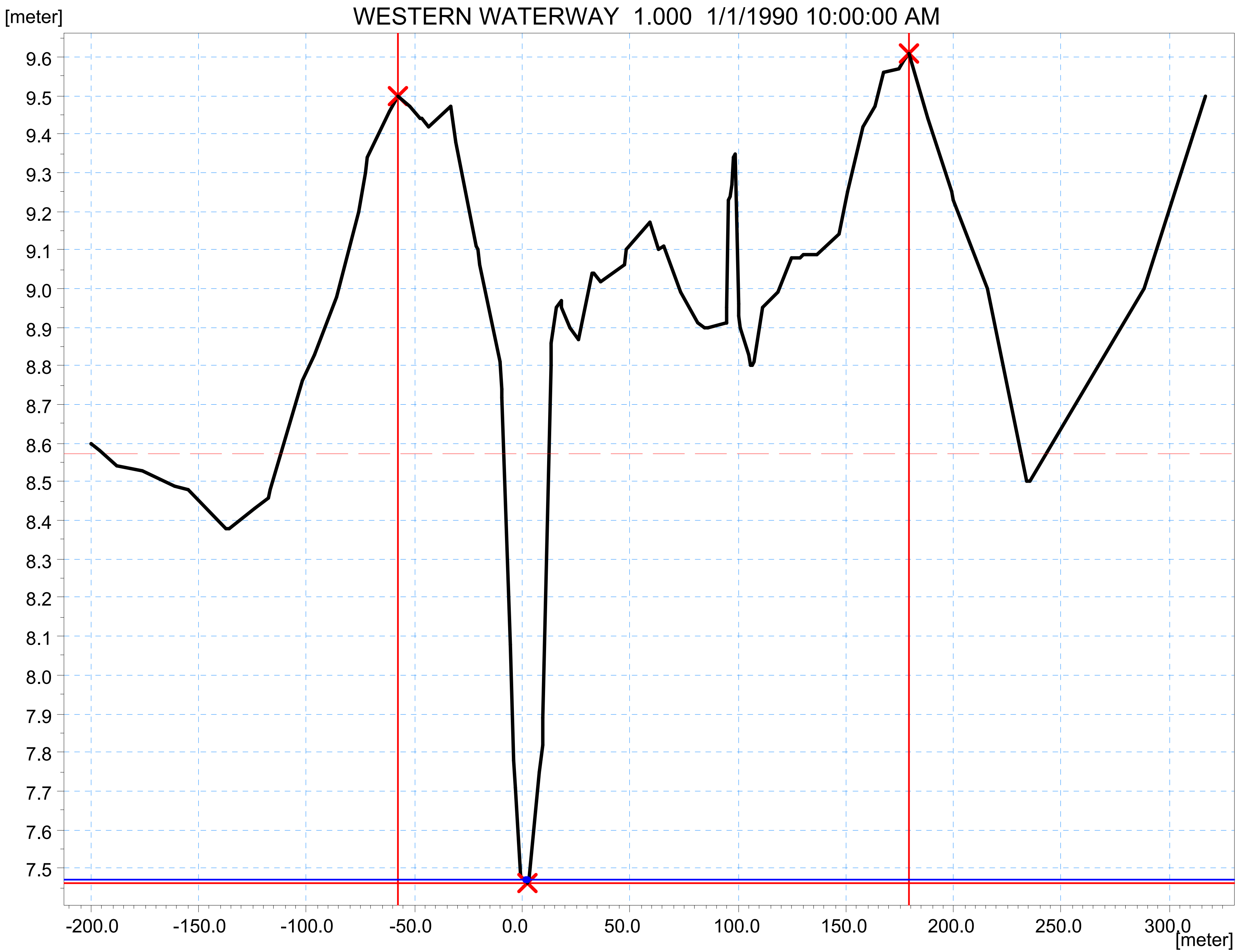
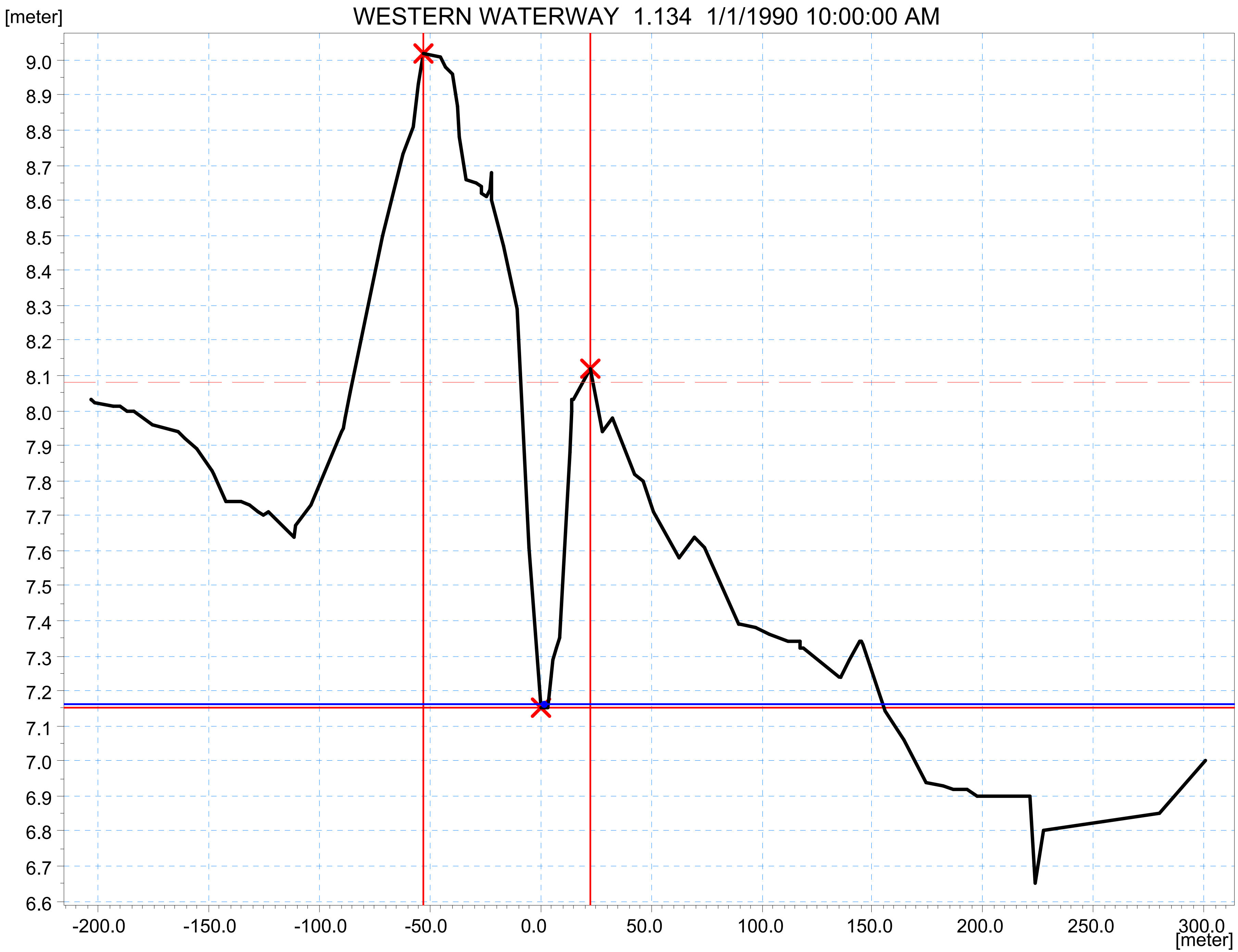
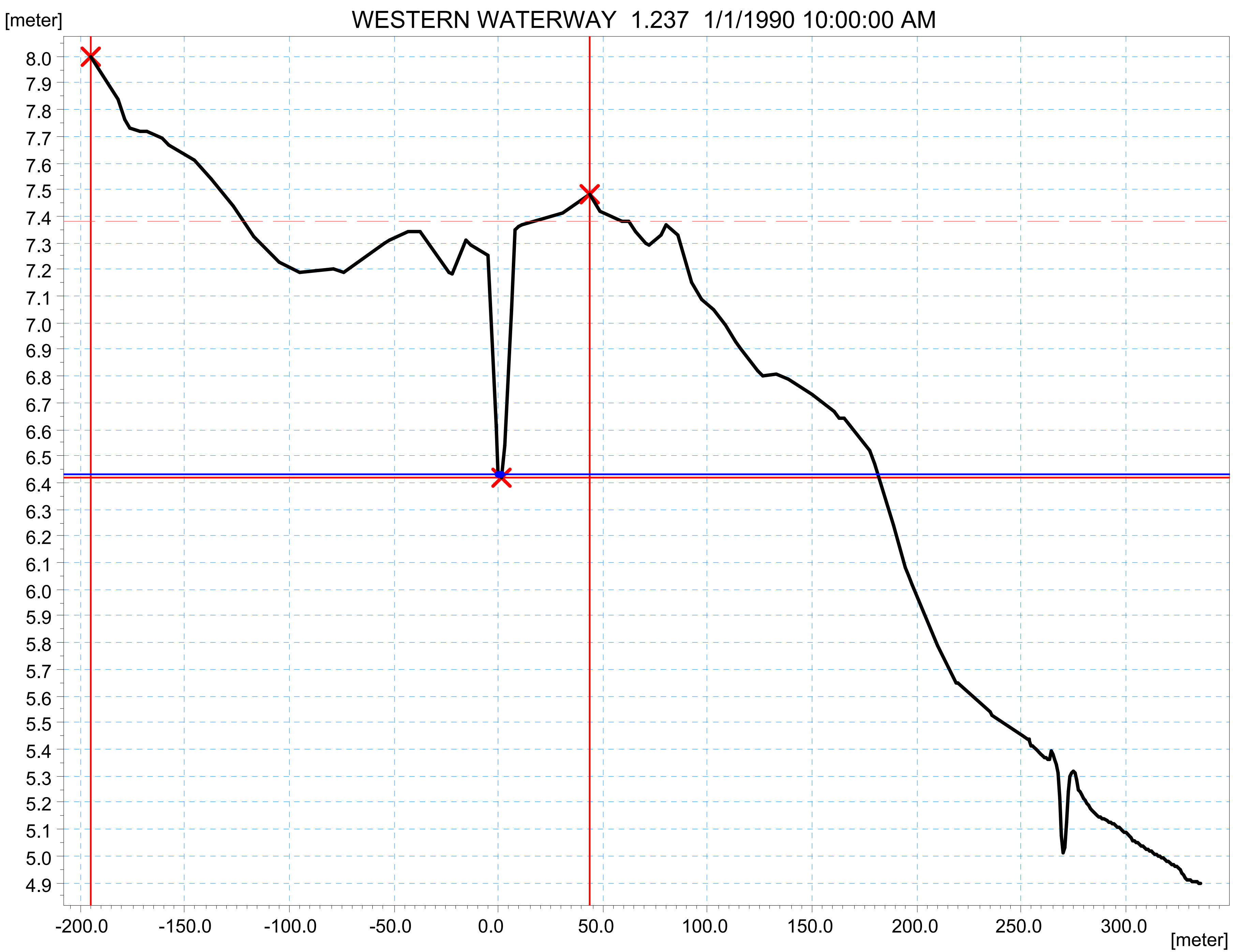
50.0
100.0
150.0
[meter]

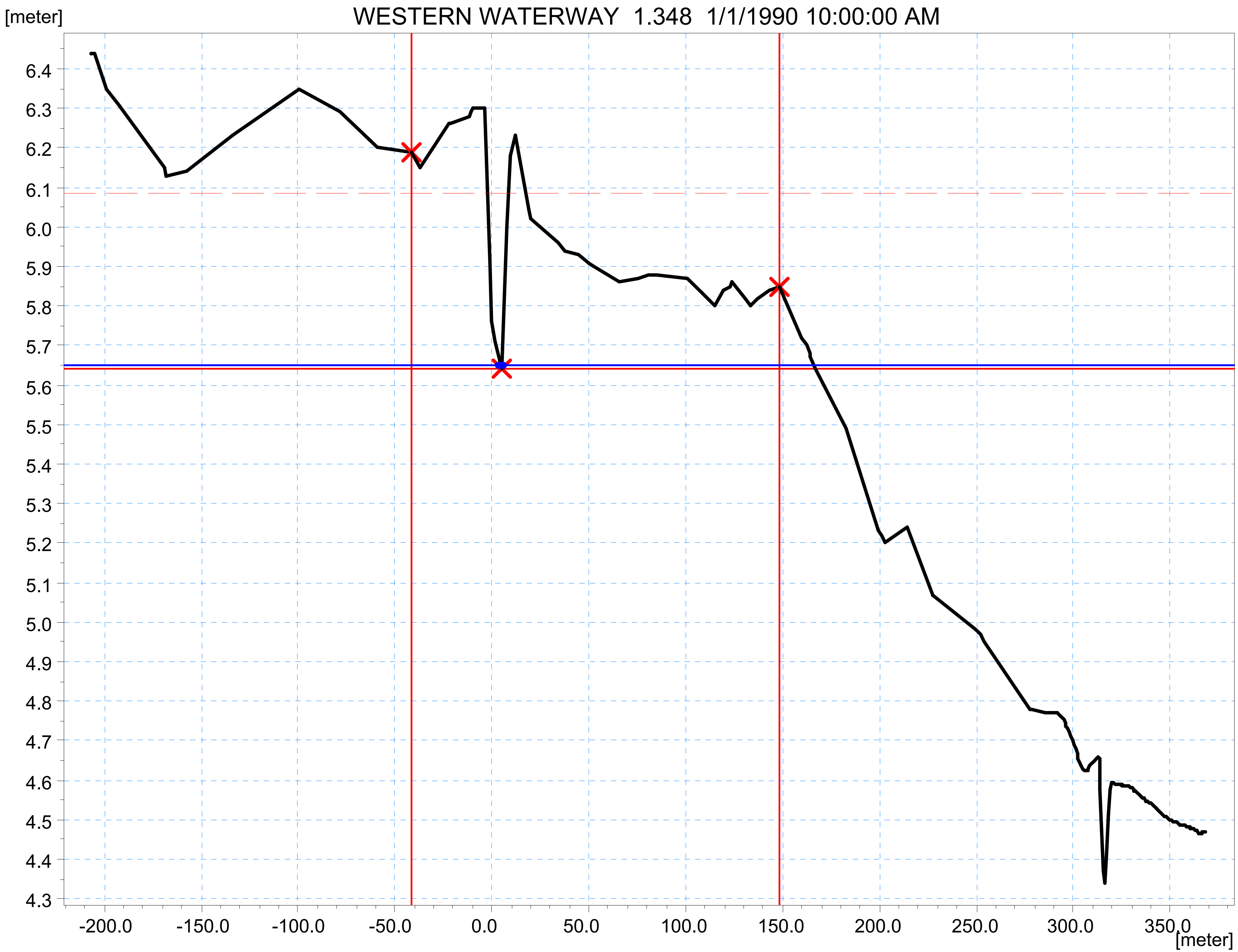


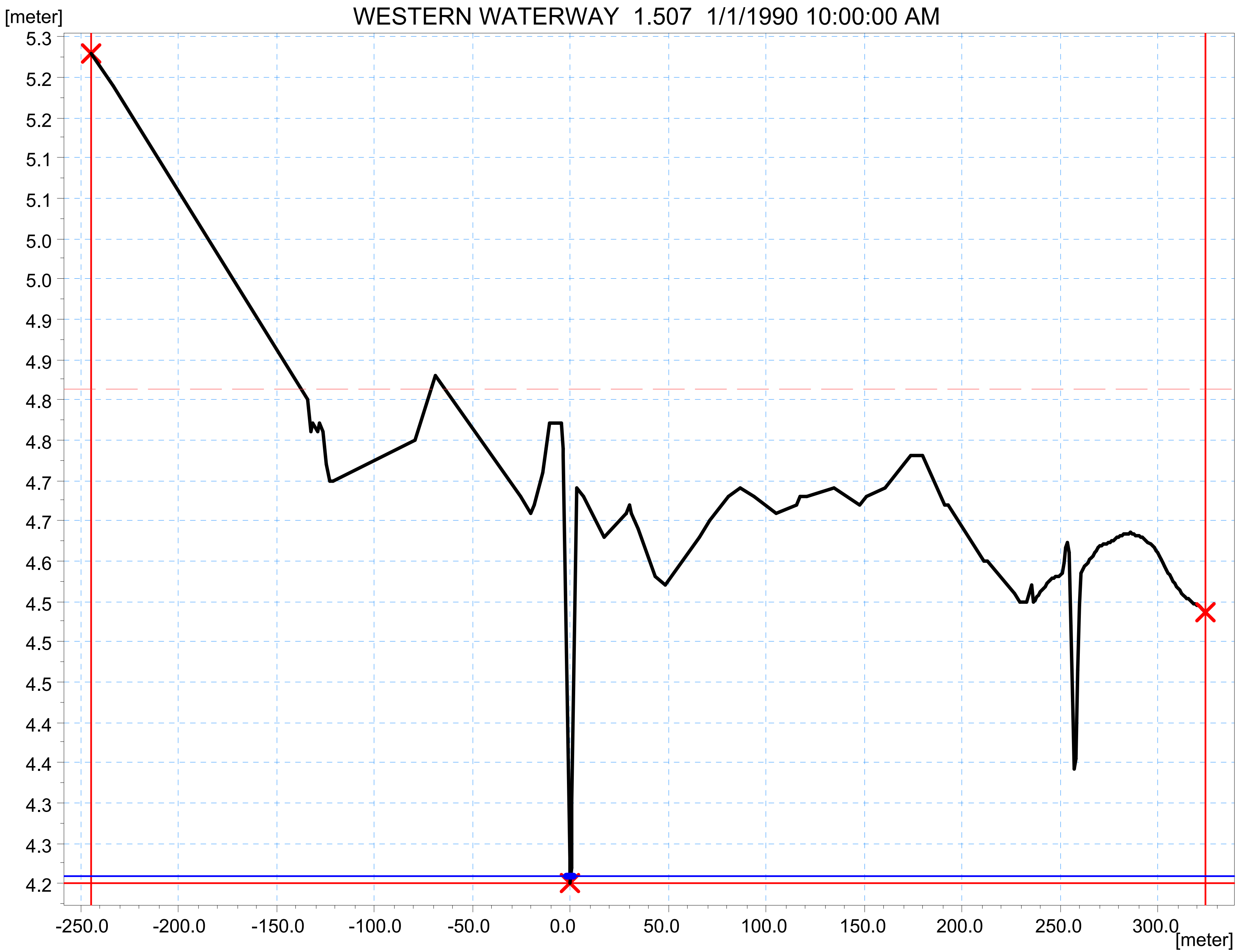
WESTERN WATERWAY 1.000 1/1/1990 10:00:00 AM
[meter]
9.6
9.5
9.4
9.3
9.2
9.1
9.0
8.9
8.8
8.7
8.6
8.5
8.4
8.3
8.2
8.1
8.0
7.9
7.8
7.7
7.6
7.5
-200.0
-150.0
-100.0
-50.0
0.0
50.0
100.0
150.0
200.0
250.0
300.0
[meter]


WESTERN WATERWAY 1.134 1/1/1990 10:00:00 AM
[meter]
9.0
8.9
8.8
8.7
8.6
8.5
8.4
8.3
8.2
8.1
8.0
7.9
7.8
7.7
7.6
7.5
7.4
7.3
7.2
7.1
7.0
6.9
6.8
6.7
6.6
-200.0
-150.0
-100.0
-50.0
0.0
50.0
100.0
150.0
200.0
250.0
300.0
[meter]

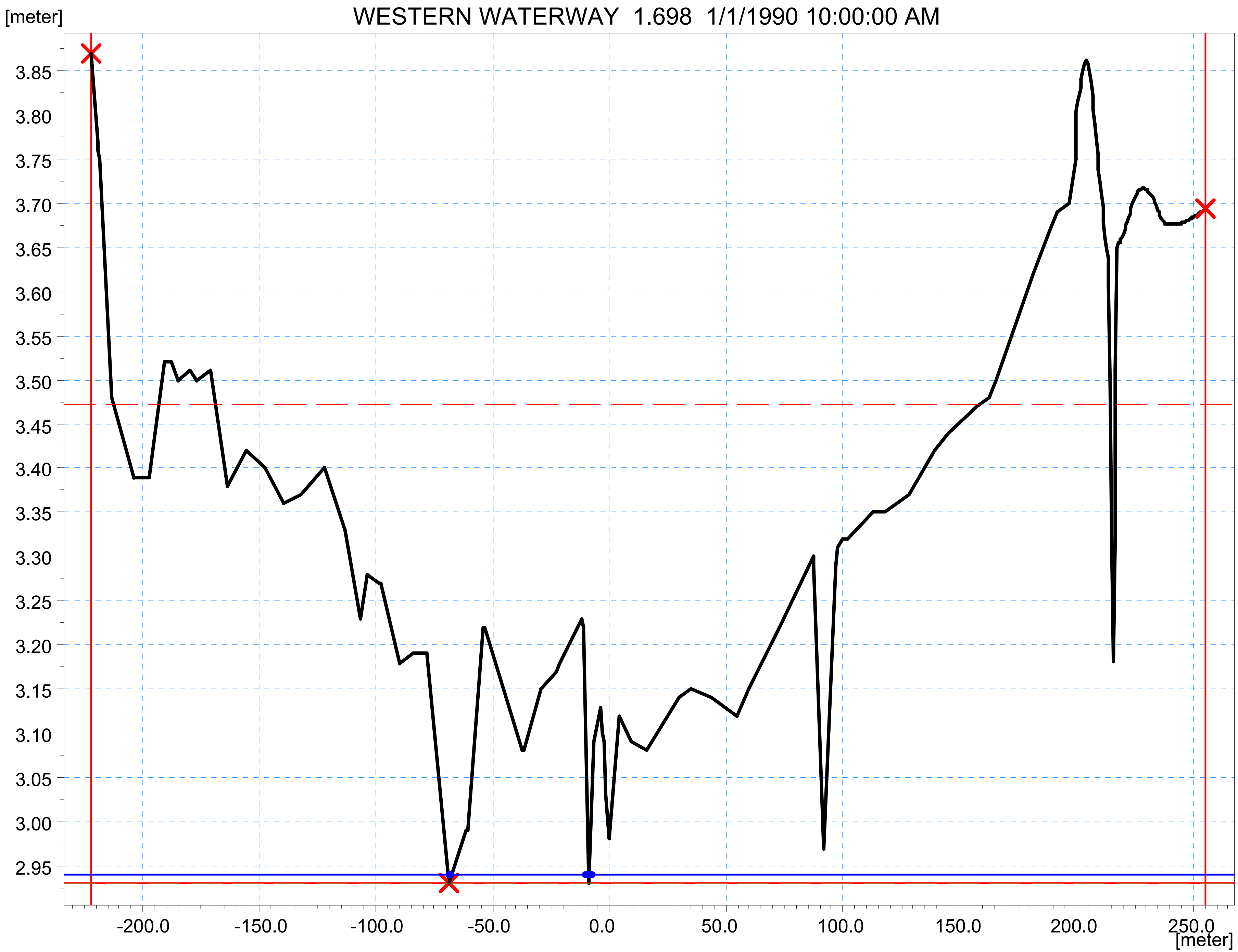


WESTERN WATERWAY 1.237 1/1/1990 10:00:00 AM
[meter]
8.0
7.9
7.8
7.7
7.6
7.5
7.4
7.3
7.2
7.1
7.0
6.9
6.8
6.7
6.6
6.5
6.4
6.3
6.2
6.1
6.0
5.9
5.8
5.7
5.6
5.5
5.4
5.3
5.2
5.1
5.0
4.9
-200.0
-150.0
-100.0
-50.0
0.0
50.0
100.0
150.0
200.0
250.0
300.0
[meter]

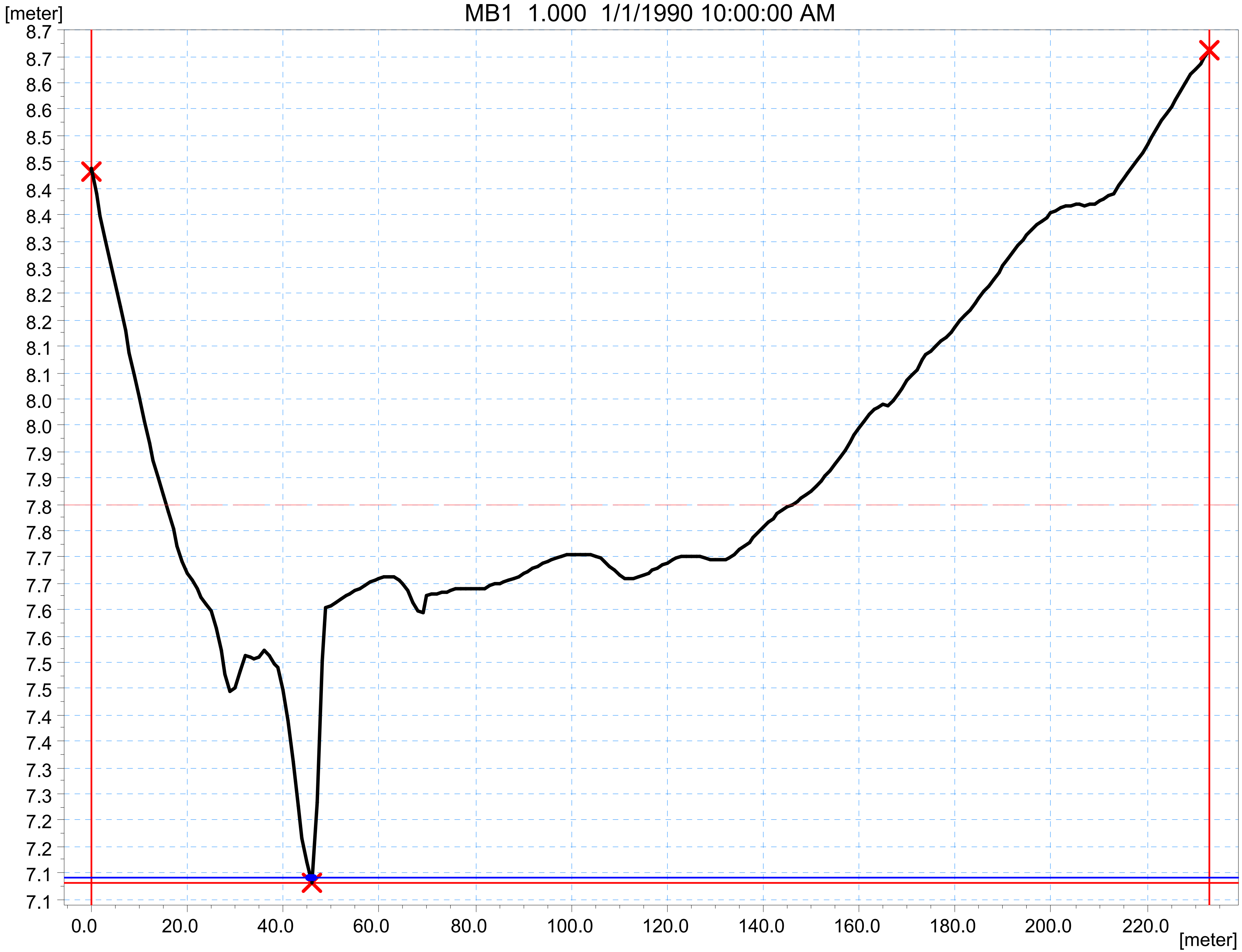

WESTERN WATERWAY 1.348 1/1/1990 10:00:00 AM
[meter]
6.4
6.3
6.2
6.1
6.0
5.9
5.8
5.7
5.6
5.5
5.4
5.3
5.2
5.1
5.0
4.9
4.8
4.7
4.6
4.5
4.4
4.3
-200.0
-150.0
-100.0
-50.0
0.0
50.0
100.0
150.0
200.0
250.0
300.0
350.0
[meter]

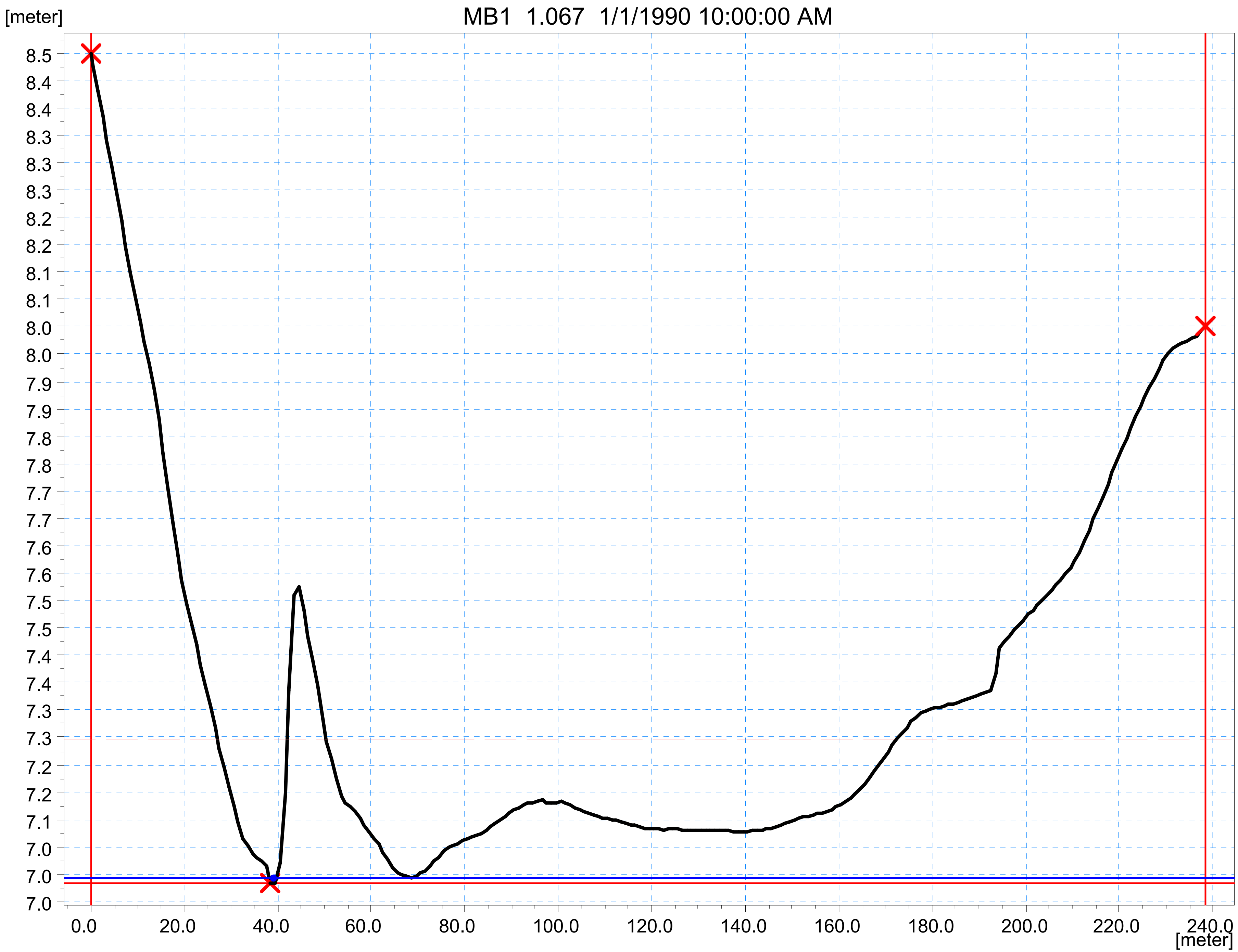


WESTERN WATERWAY 1.507 1/1/1990 10:00:00 AM
[meter]
5.3
5.2
5.2
5.1
5.1
5.0
5.0
4.9
4.9
4.8
4.8
4.7
4.7
4.6
4.5
4.5
4.5
4.4
4.4
4.3
4.3
4.2
-250.0
-200.0
-150.0
-100.0
-50.0
0.0
50.0
100.0
150.0
200.0
250.0
300.0
[meter]

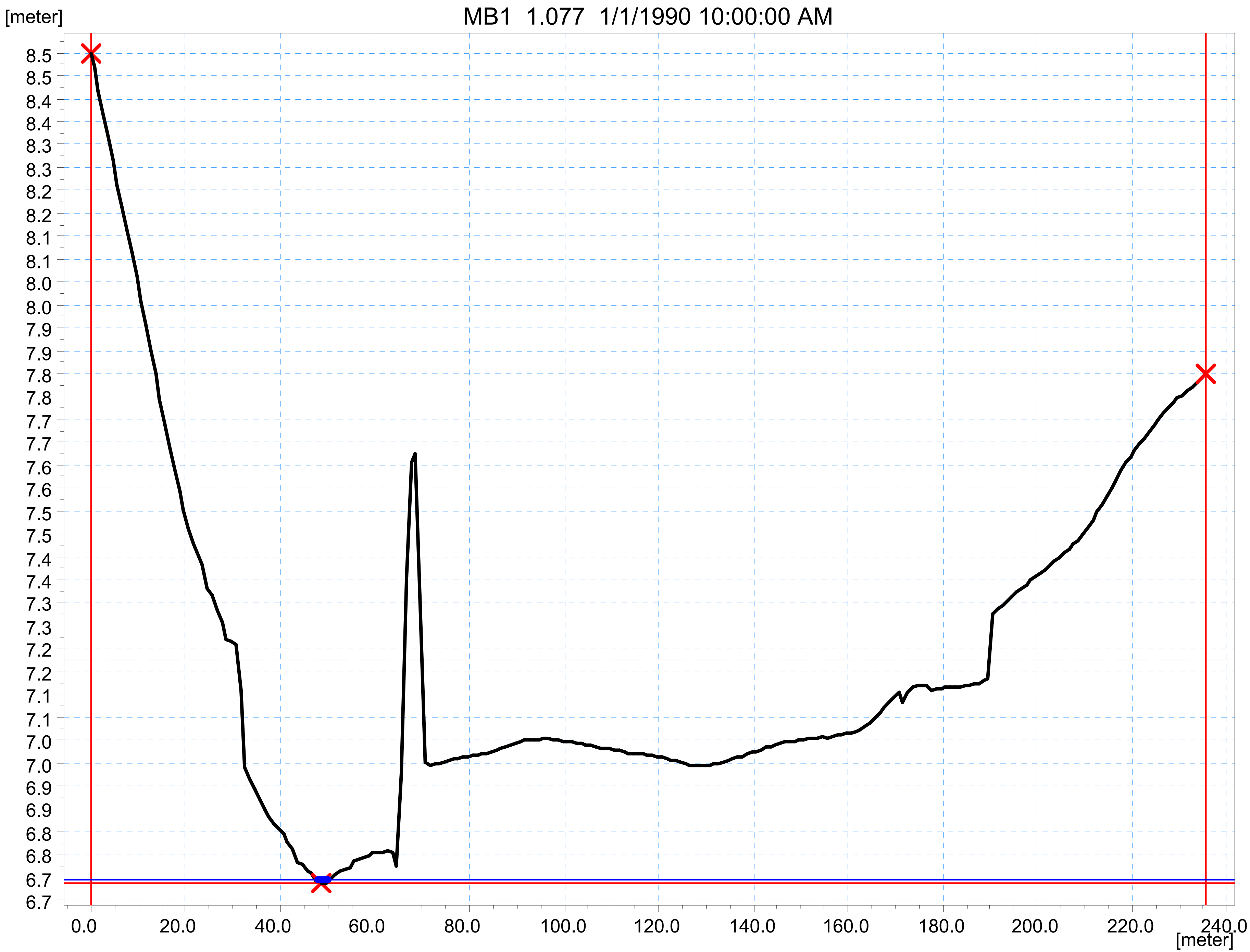


WESTERN WATERWAY 1.698 1/1/1990 10:00:00 AM
[meter]
3.85
3.80
3.75
3.70
3.65
3.60
3.55
3.50
3.45
3.40
3.35
3.30
3.25
3.20
3.15
3.10
3.05
3.00
2.95
-200.0
-150.0
-100.0
-50.0
0.0
50.0
100.0
150.0
200.0
250.0
[meter]

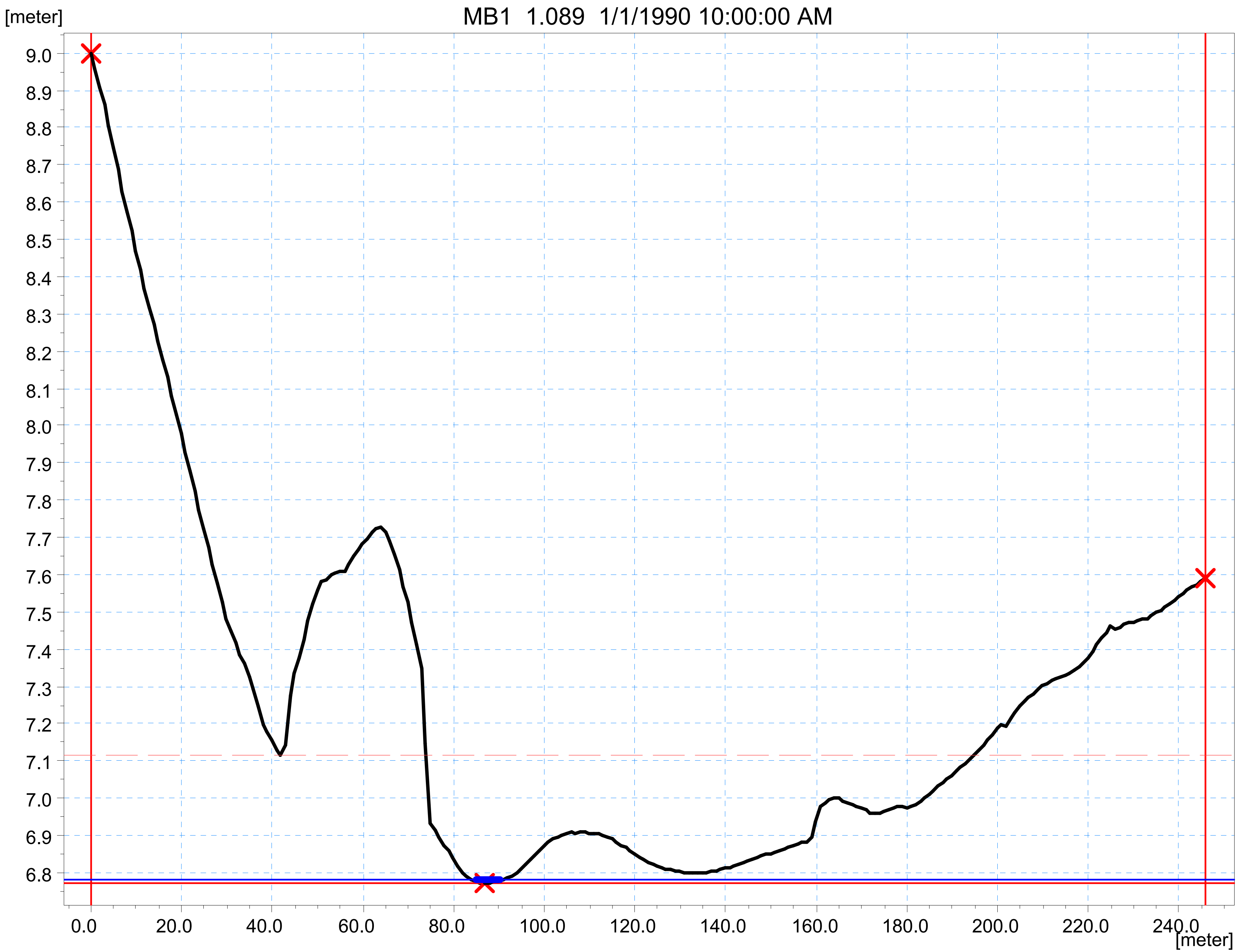

MB1 1.000 1/1/1990 10:00:00 AM
[meter]
8.7
8.7
8.6
8.6
8.5
8.5
8.4
8.4
8.3
8.3
8.2
8.2
8.1
8.1
8.0
8.0
7.9
7.9
7.8
7.8
7.7
7.7
7.6
7.6
7.5
7.5
7.4
7.4
7.3
7.3
7.2
7.2
7.1
7.1
0.0
20.0
40.0
60.0
80.0
100.0
120.0
140.0
160.0
180.0
200.0
220.0
[meter]



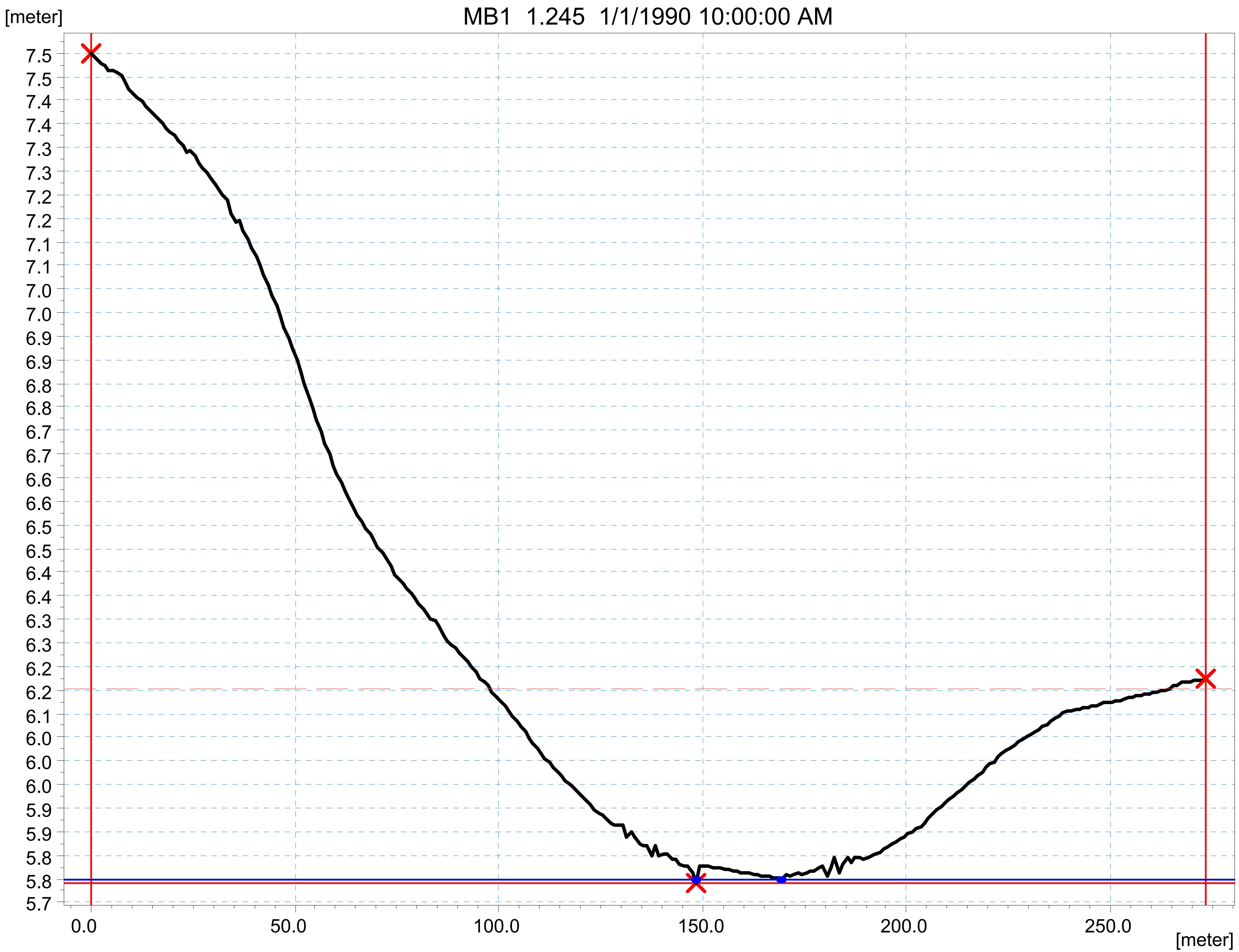
MB1 1.067 1/1/1990 10:00:00 AM
[meter]
8.5
8.4
8.4
8.3
8.3
8.3
8.2
8.2
8.1
8.1
8.0
8.0
7.9
7.9
7.8
7.8
7.7
7.7
7.6
7.6
7.5
7.5
7.4
7.4
7.3
7.3
7.2
7.2
7.1
7.0
7.0
7.0
0.0
20.0
40.0
60.0
80.0
100.0
120.0
140.0
160.0
180.0
200.0
220.0
240.0
[meter]

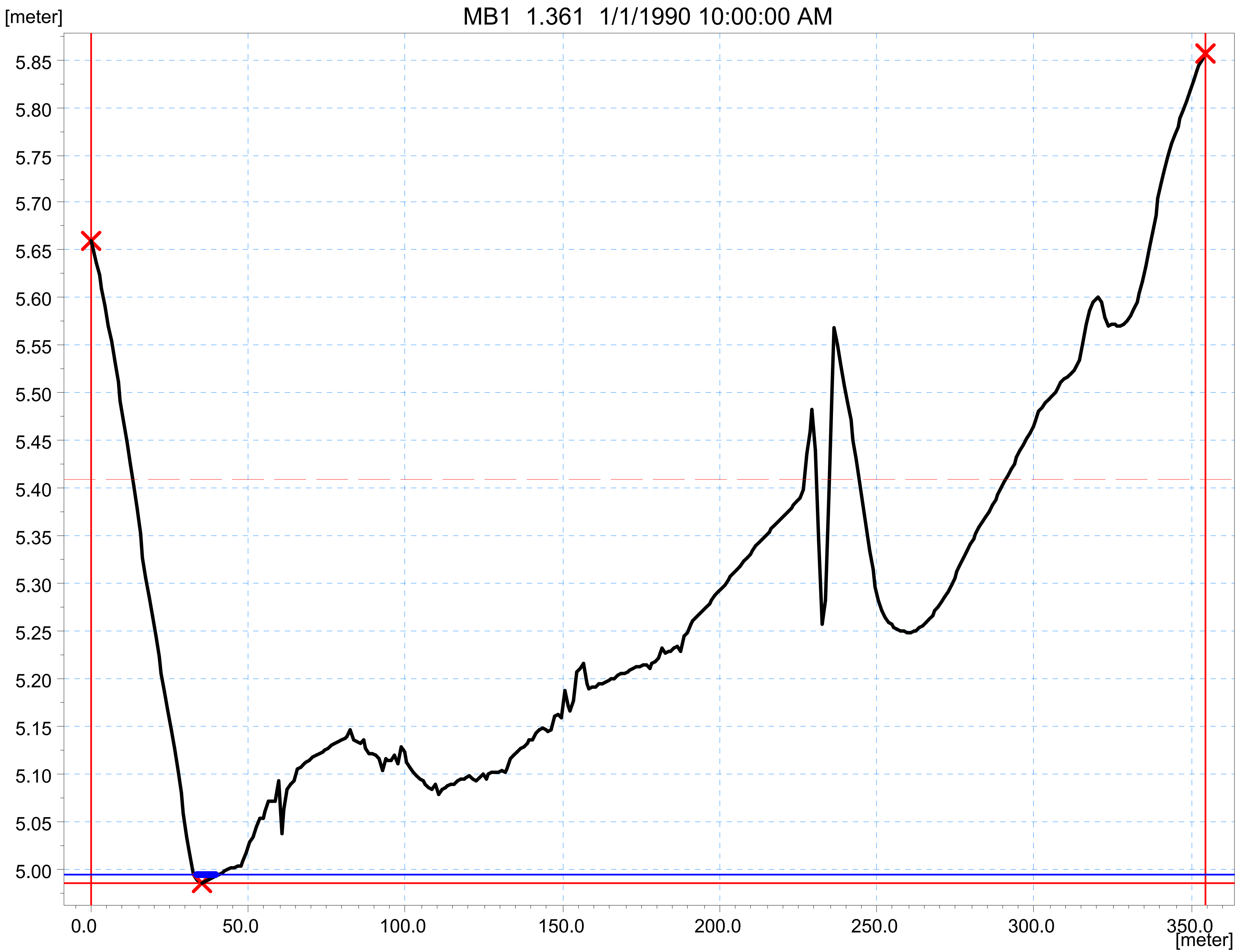


MB1 1.077 1/1/1990 10:00:00 AM
[meter]
8.5
8.5
8.4
8.4
8.3
8.3
8.2
8.2
8.1
8.1
8.0
8.0
7.9
7.9
7.8
7.8
7.7
7.7
7.6
7.6
7.5
7.5
7.4
7.4
7.3
7.3
7.2
7.2
7.1
7.1
7.0
7.0
6.9
6.9
6.8
6.8
6.7
6.7
0.0
20.0
40.0
60.0
80.0
100.0
120.0
140.0
160.0
180.0
200.0
220.0
240.0
[meter]

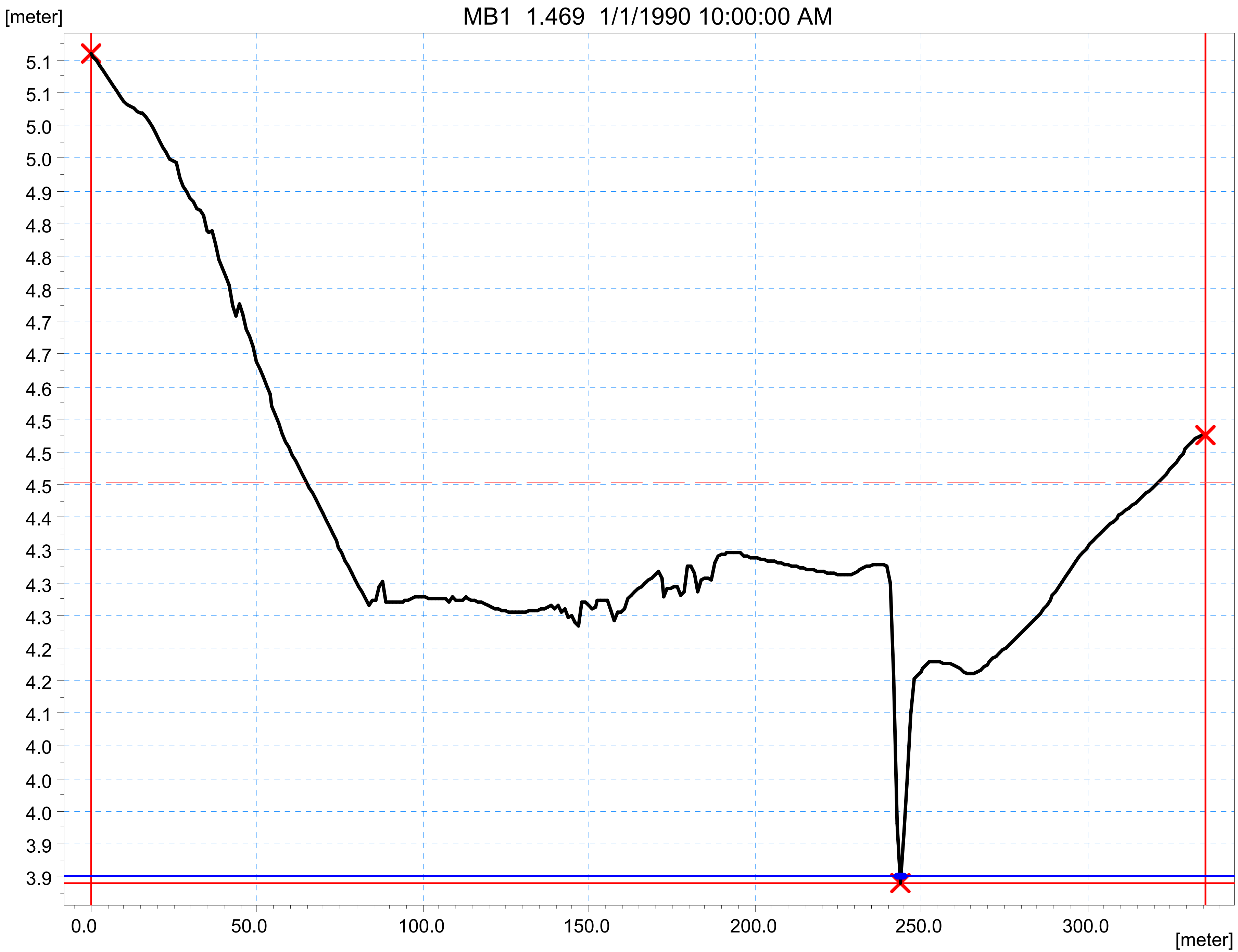


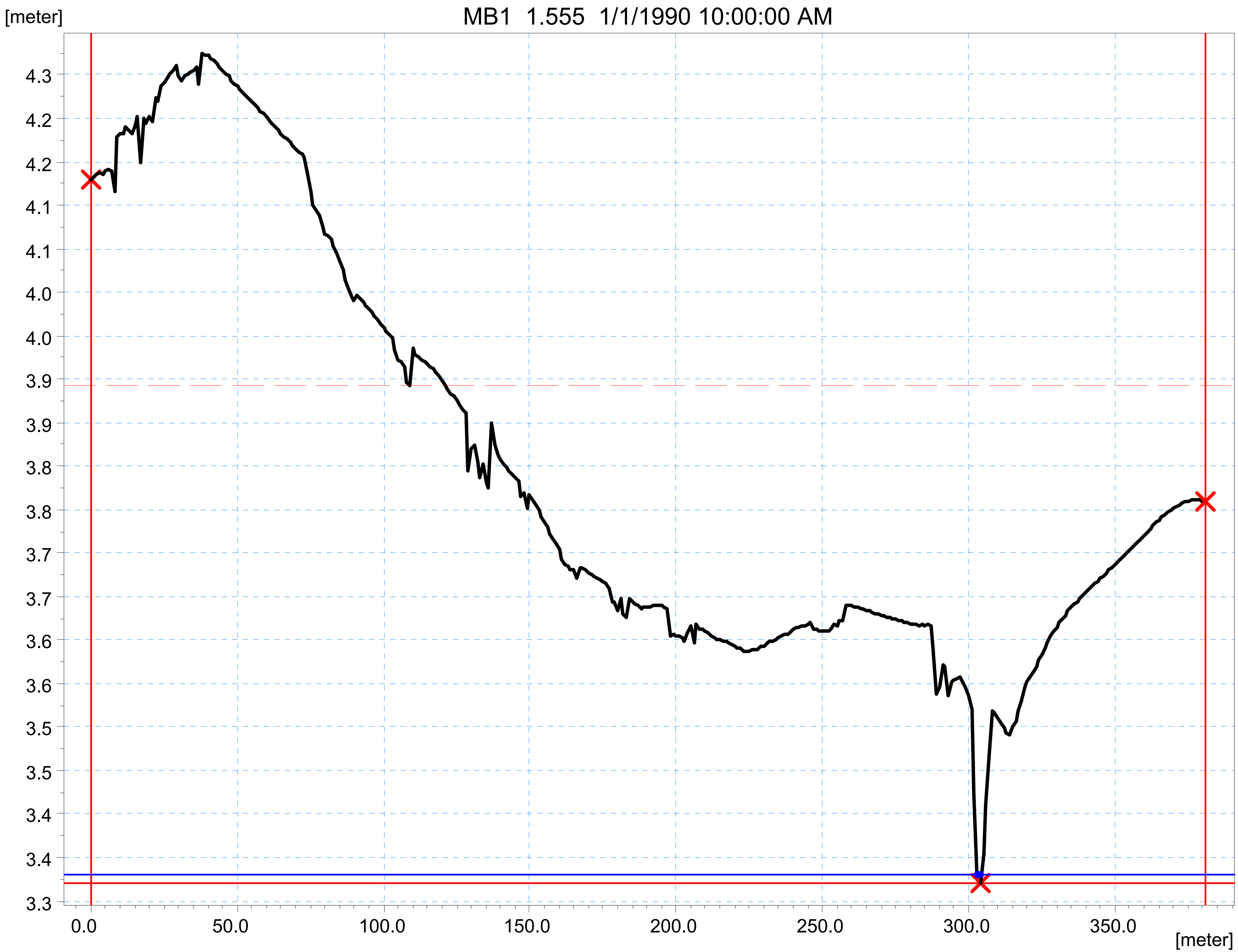
MB1 1.089 1/1/1990 10:00:00 AM
[meter]
9.0
8.9
8.8
8.7
8.6
8.5
8.4
8.3
8.2
8.1
8.0
7.9
7.8
7.7
7.6
7.5
7.4
7.3
7.2
7.1
7.0
6.9
6.8
0.0
20.0
40.0
60.0
80.0
100.0
120.0
140.0
160.0
180.0
200.0
220.0
240.0
[meter]


MB1 1.245 1/1/1990 10:00:00 AM
[meter]
7.5
7.5
7.4
7.4
7.3
7.3
7.2
7.2
7.1
7.1
7.0
7.0
6.9
6.9
6.8
6.8
6.7
6.7
6.6
6.6
6.5
6.5
6.4
6.4
6.3
6.3
6.2
6.2
6.1
6.0
6.0
6.0
5.9
5.9
5.8
5.8
5.7
0.0
50.0
100.0
150.0
200.0
250.0
[meter]



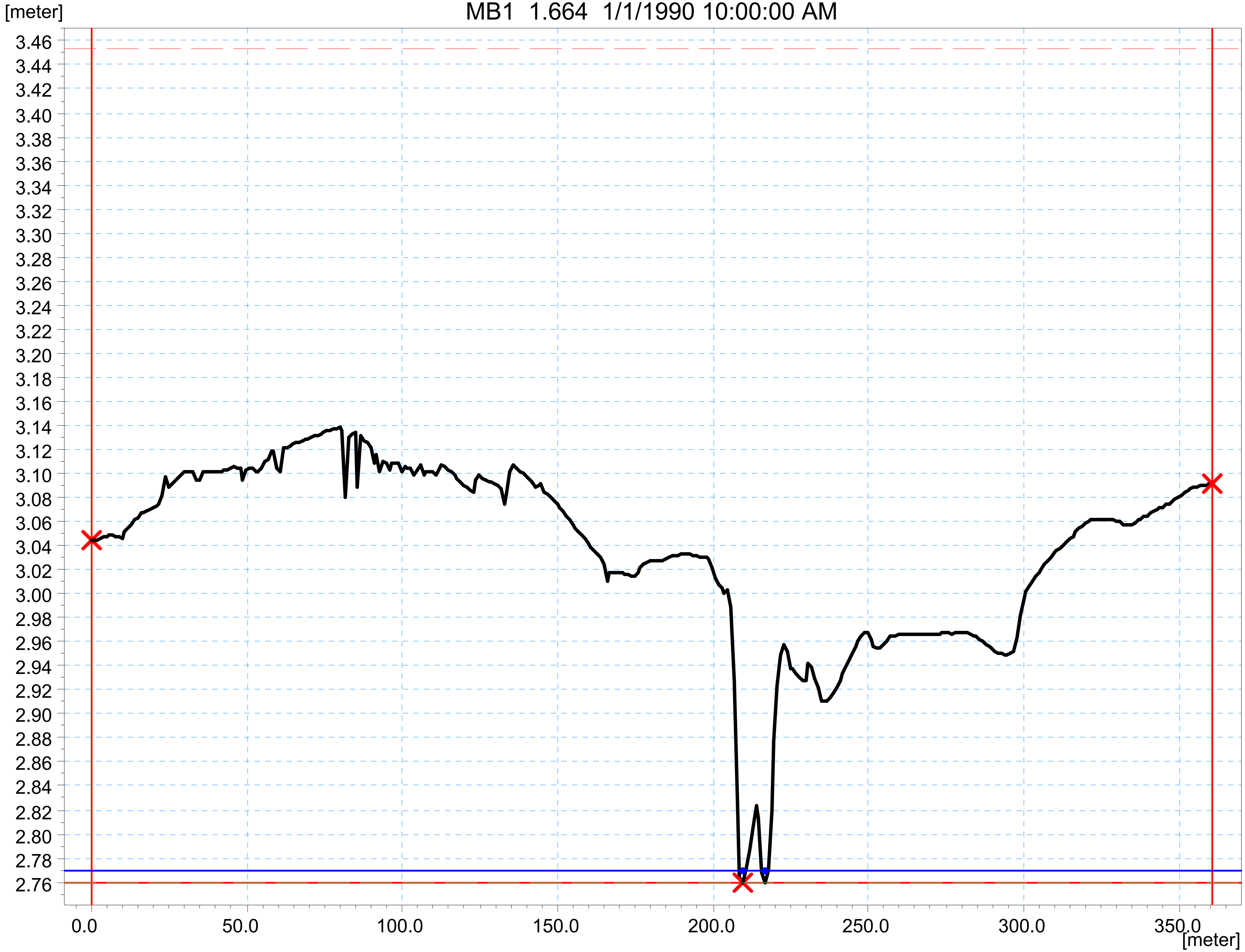
MB1 1.361 1/1/1990 10:00:00 AM
[meter]
5.85
5.80
5.75
5.70
5.65
5.60
5.55
5.50
5.45
5.40
5.35
5.30
5.25
5.20
5.15
5.10
5.05
5.00
0.0
50.0
100.0
150.0
200.0
250.0
300.0
350.0
[meter]

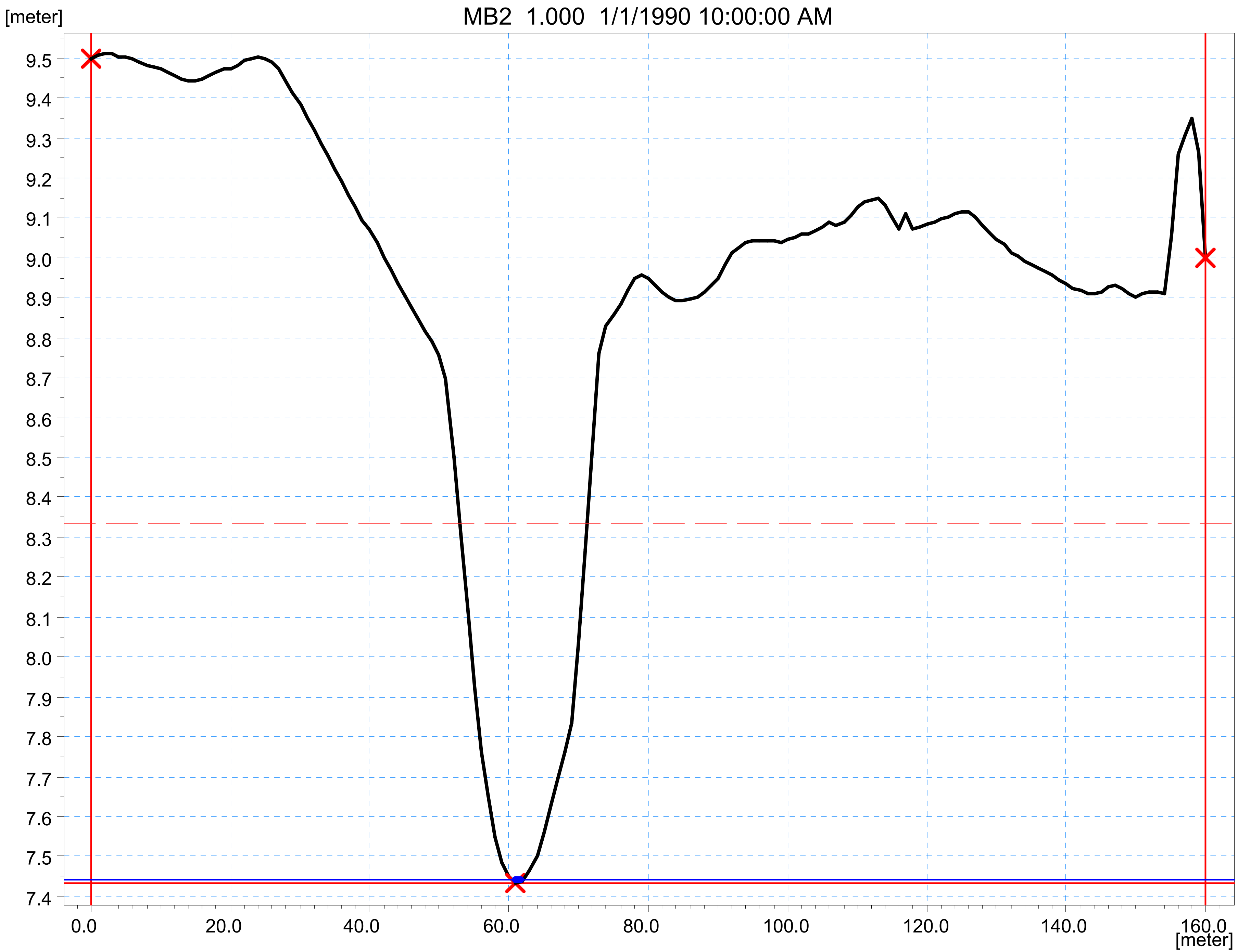

MB1 1.469 1/1/1990 10:00:00 AM
[meter]
5.1
5.1
5.0
5.0
4.9
4.8
4.8
4.8
4.7
4.7
4.6
4.5
4.5
4.5
4.4
4.3
4.3
4.3
4.2
4.2
4.1
4.0
4.0
4.0
3.9
3.9
0.0
50.0
100.0
150.0
200.0
250.0
300.0
[meter]



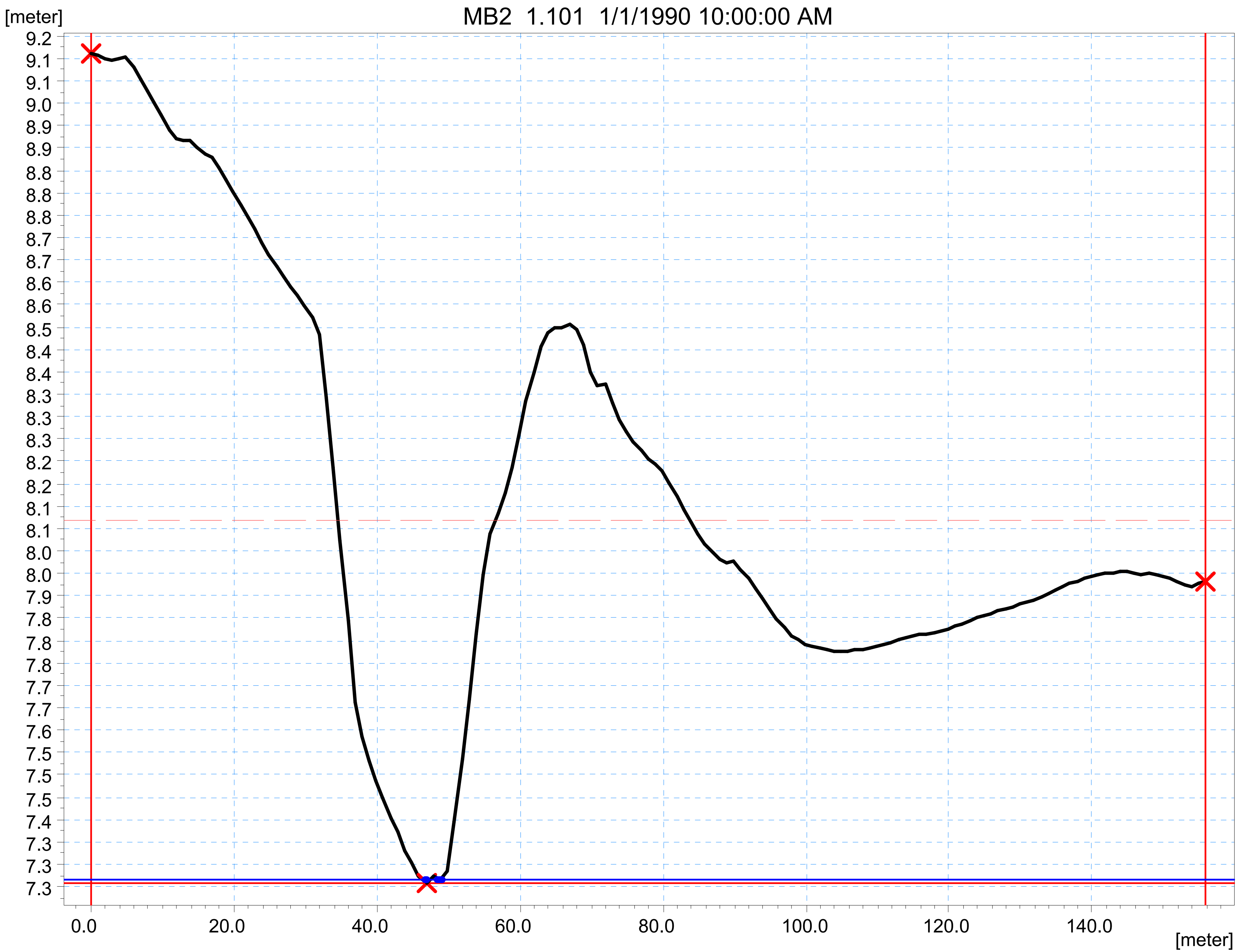
[meter]
MB1 1.555 1/1/1990 10:00:00 AM
4.3
4.2
4.2
4.1
4.1
4.0
4.0
3.9
3.9
3.8
3.8
3.7
3.7
3.6
3.6
3.5
3.5
3.4
3.4
3.3
0.0
50.0
100.0
150.0
200.0
250.0
300.0
350.0
[meter]

MB1 1.664 1/1/1990 10:00:00 AM

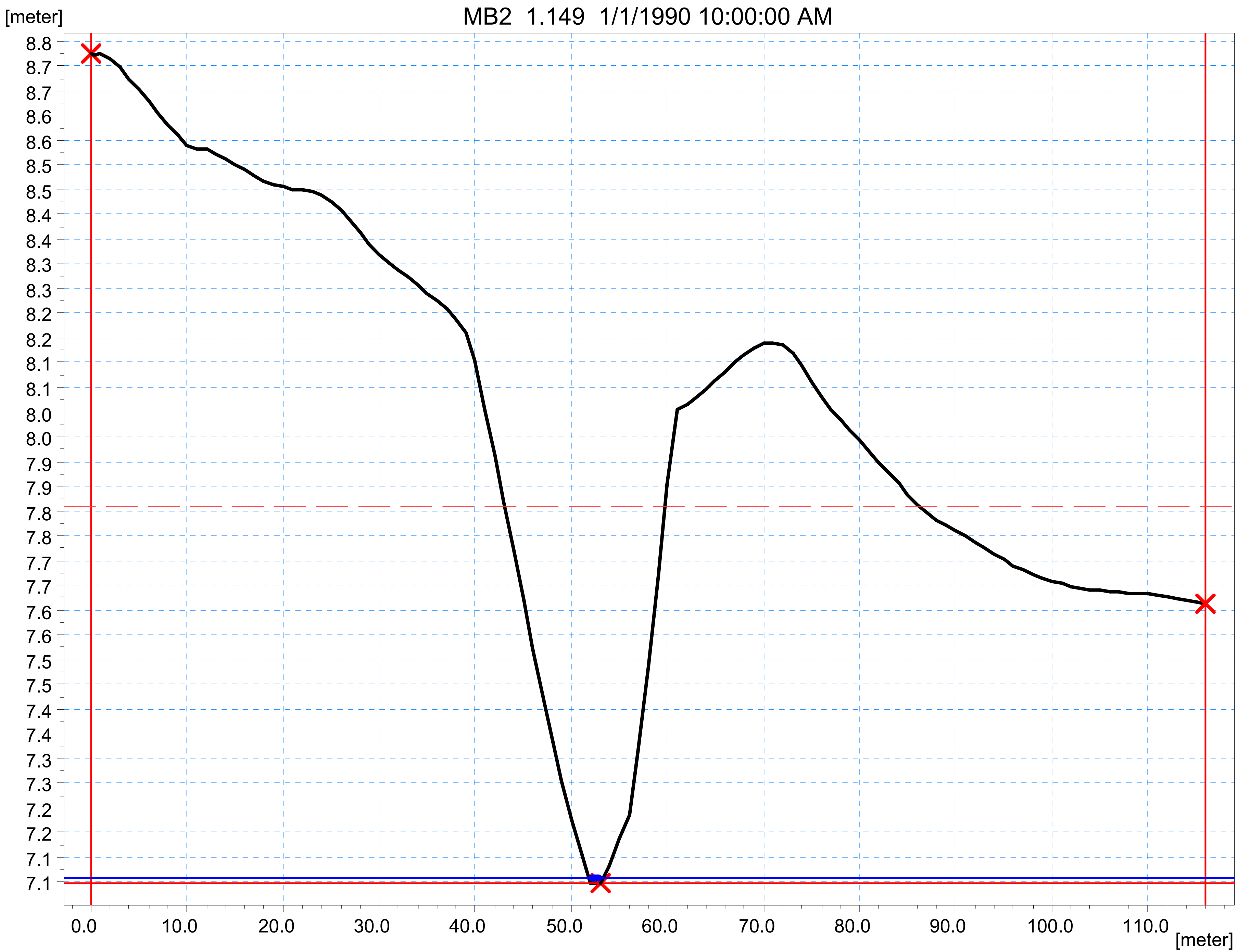




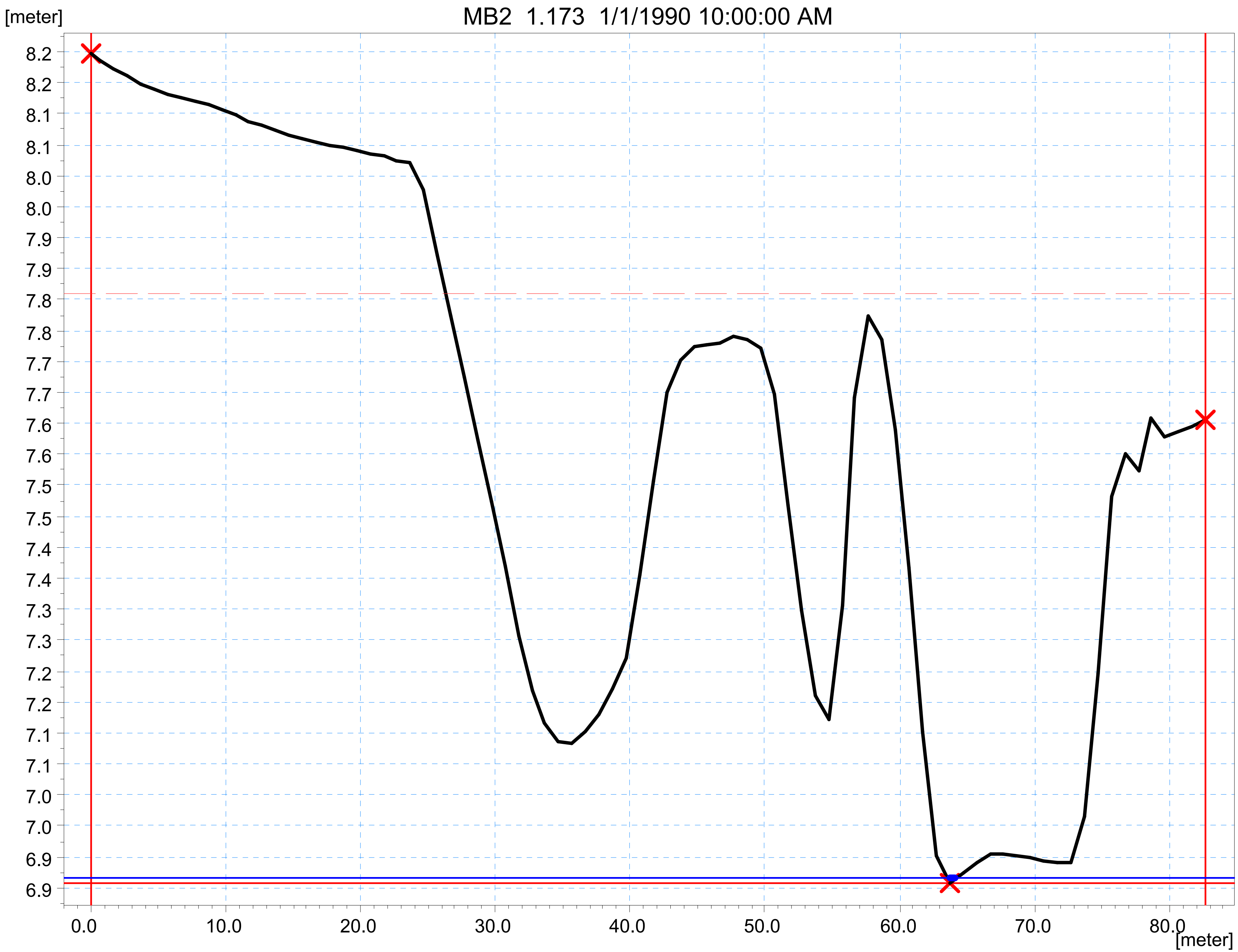
MB2 1.000 1/1/1990 10:00:00 AM
[meter]
9.5
9.4
9.3
9.2
9.1
9.0
8.9
8.8
8.7
8.6
8.5
8.4
8.3
8.2
8.1
8.0
7.9
7.8
7.7
7.6
7.5
7.4
0.0
20.0
40.0
60.0
80.0
100.0
120.0
140.0
160.0
[meter]

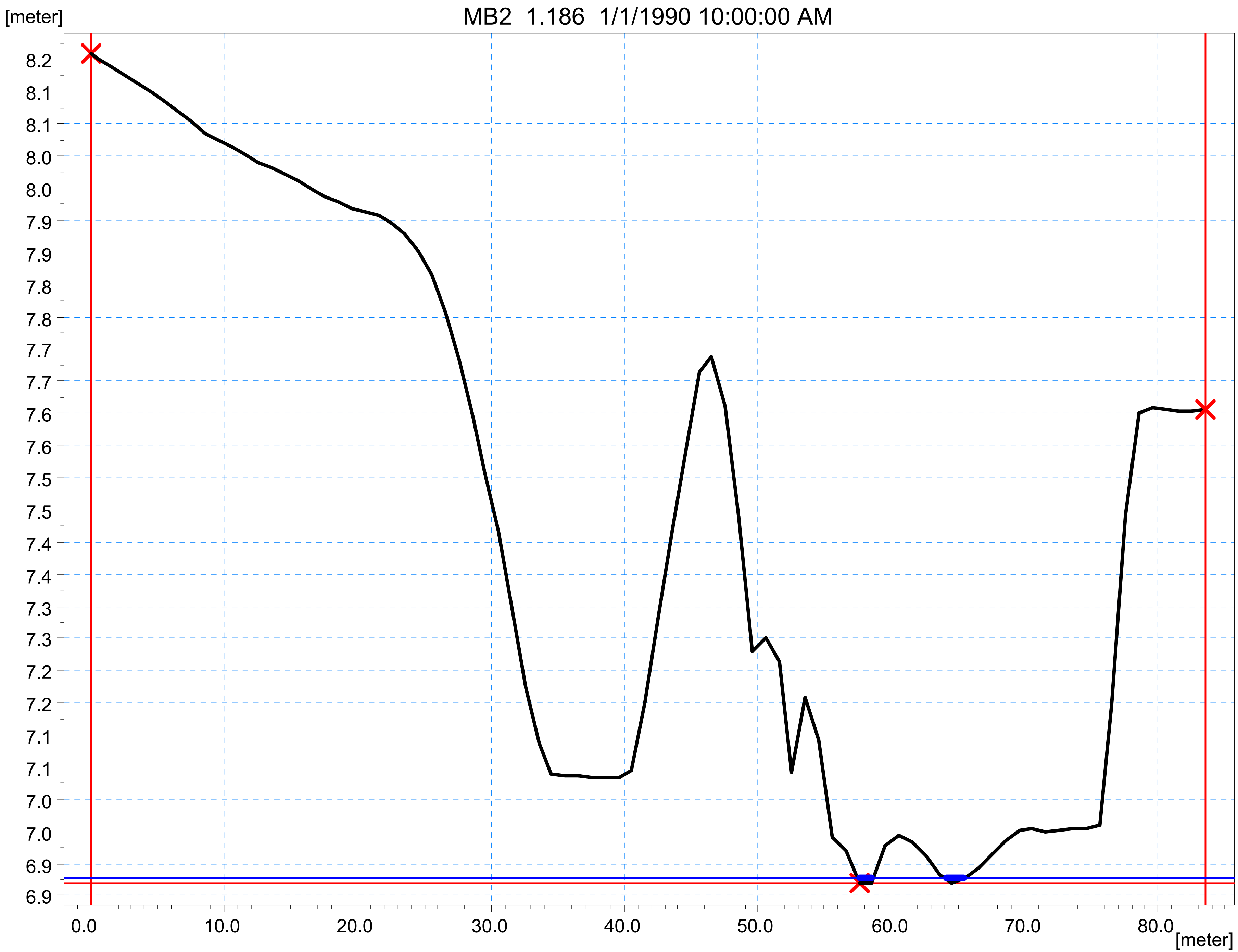


MB2 1.101 1/1/1990 10:00:00 AM
[meter]
9.2
9.1
9.1
9.0
8.9
8.9
8.8
8.8
8.8
8.7
8.7
8.6
8.6
8.5
8.4
8.4
8.3
8.3
8.3
8.2
8.2
8.1
8.1
8.0
8.0
7.9
7.8
7.8
7.8
7.7
7.7
7.6
7.5
7.5
7.5
7.4
7.3
7.3
7.3
0.0
20.0
40.0
60.0
80.0
100.0
120.0
140.0
[meter]

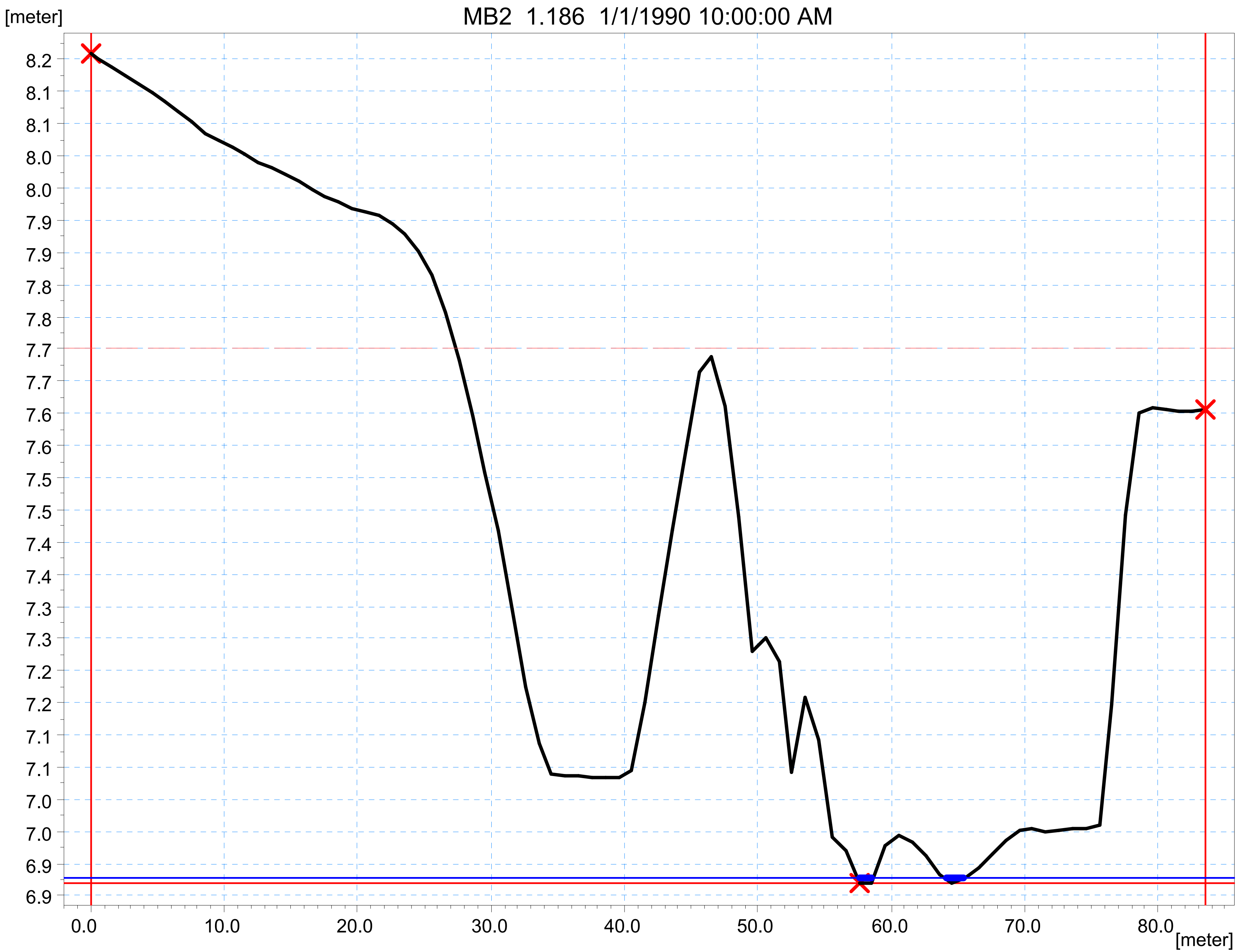


MB2 1.149 1/1/1990 10:00:00 AM
[meter]
8.8
8.7
8.7
8.6
8.6
8.5
8.5
8.4
8.4
8.3
8.3
8.2
8.2
8.1
8.1
8.0
8.0
7.9
7.9
7.8
7.8
7.7
7.7
7.6
7.6
7.5
7.5
7.4
7.4
7.3
7.3
7.2
7.2
7.1
7.1
0.0
10.0
20.0
30.0
40.0
50.0
60.0
70.0
80.0
90.0
100.0
110.0
[meter]

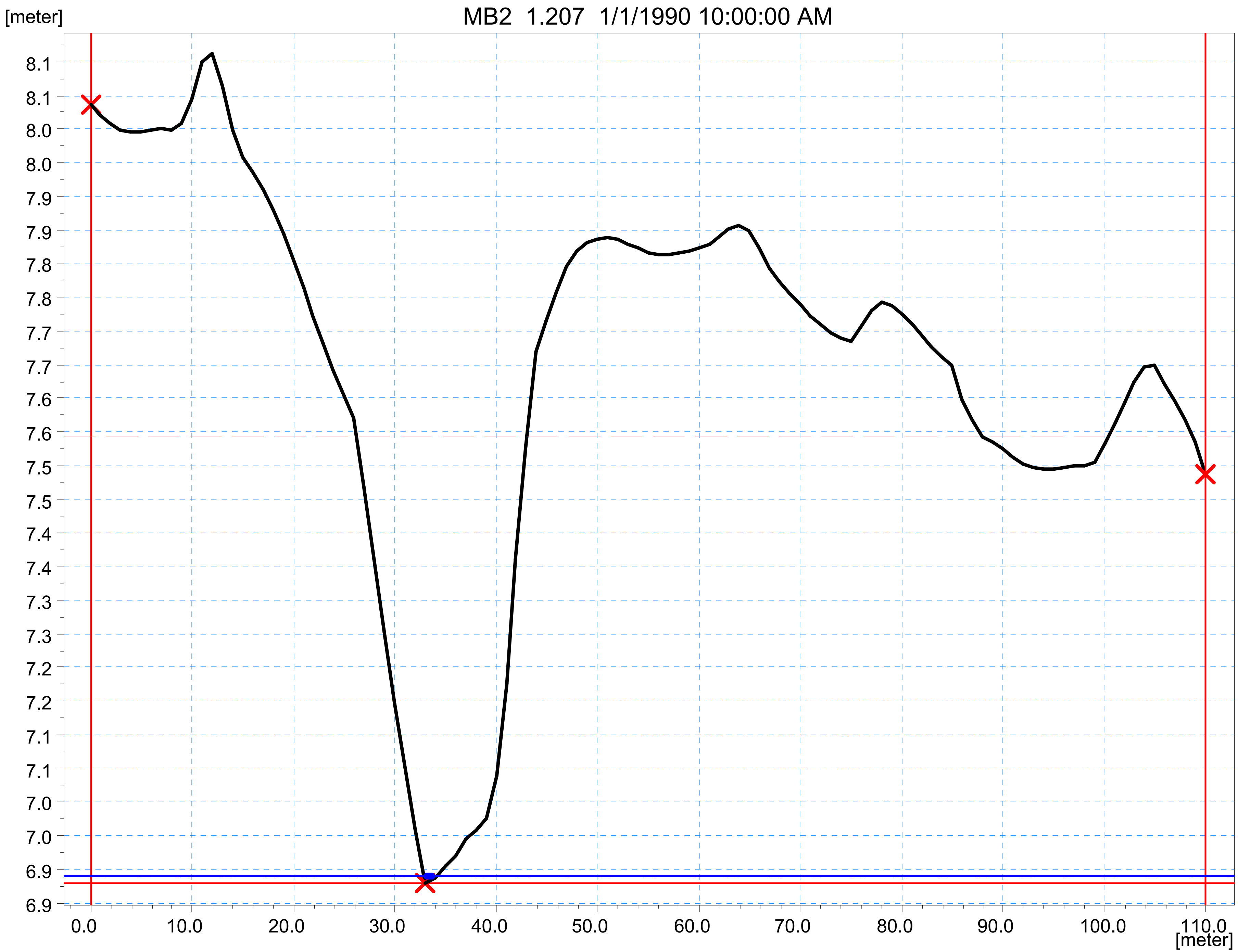

MB2 1.173 1/1/1990 10:00:00 AM
[meter]
8.2
8.2
8.1
8.1
8.0
8.0
7.9
7.9
7.8
7.8
7.7
7.7
7.6
7.6
7.5
7.5
7.4
7.4
7.3
7.3
7.2
7.2
7.1
7.1
7.0
7.0
6.9
6.9
0.0
10.0
20.0
30.0
40.0
50.0
60.0
70.0
80.0
[meter]



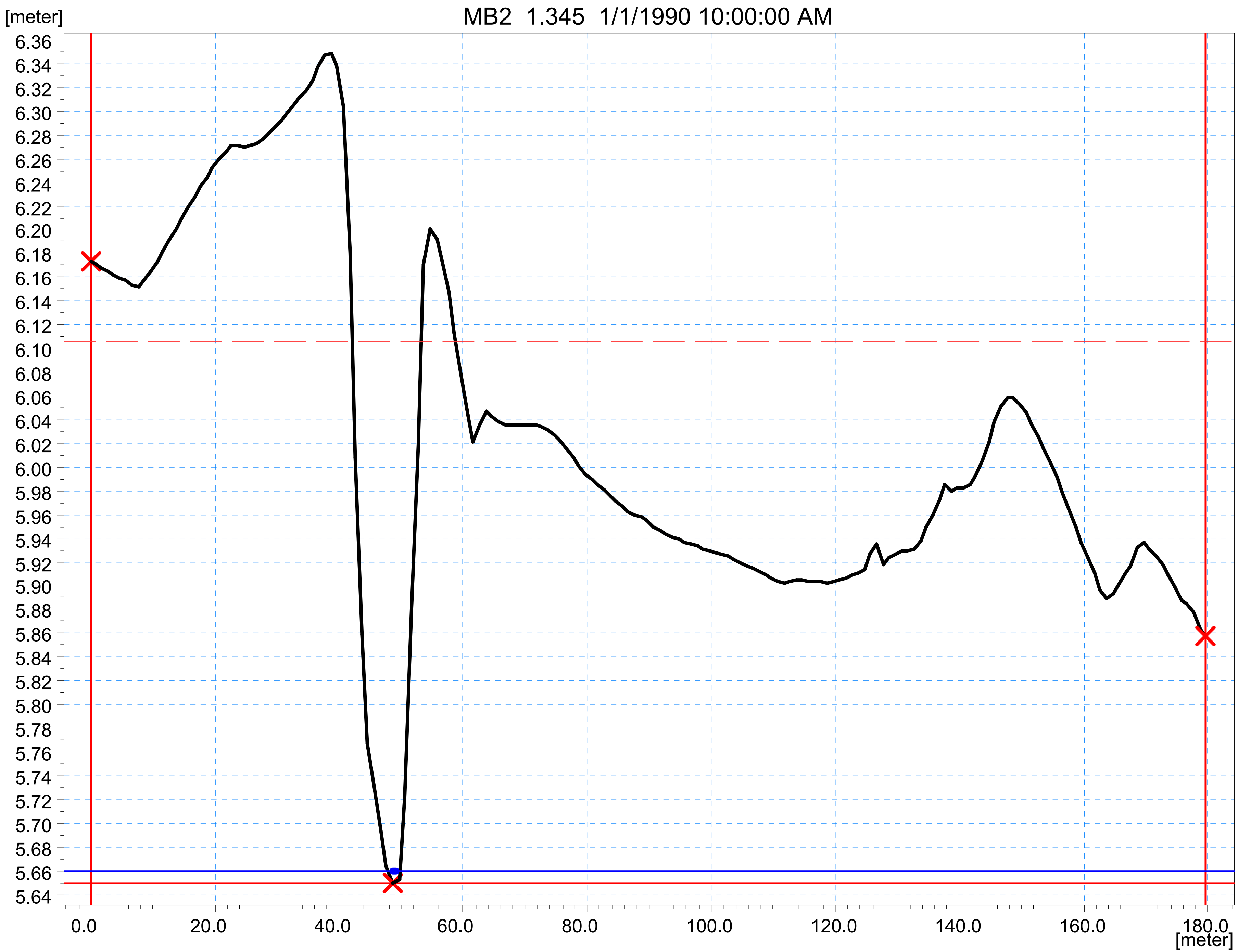
MB2 1.186 1/1/1990 10:00:00 AM
[meter]
8.2
8.1
8.1
8.0
8.0
7.9
7.9
7.8
7.8
7.7
7.7
7.6
7.6
7.5
7.5
7.4
7.4
7.3
7.3
7.2
7.2
7.1
7.1
7.0
7.0
6.9
6.9
0.0
10.0
20.0
30.0
40.0
50.0
60.0
70.0
80.0
[meter]



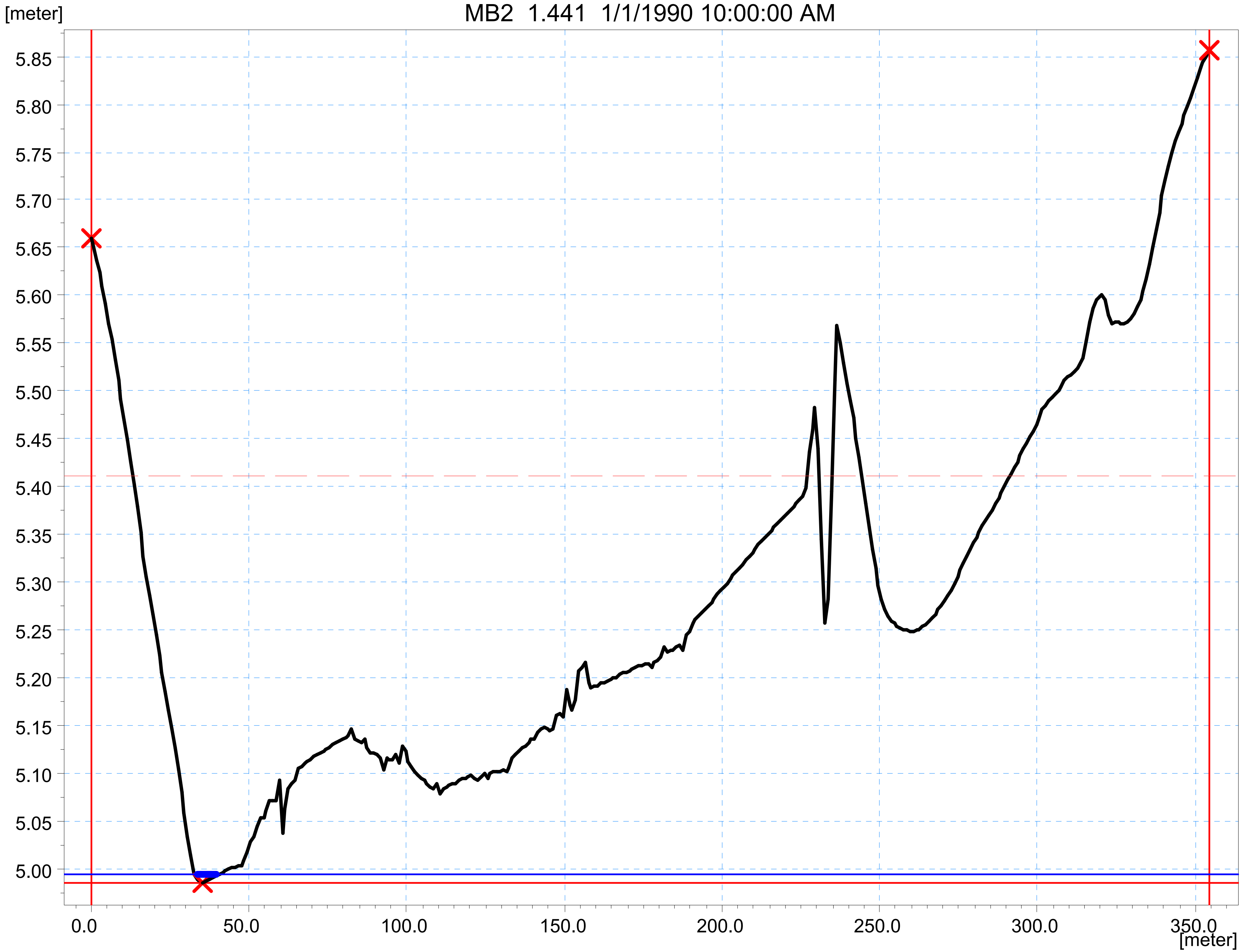
MB2 1.186 1/1/1990 10:00:00 AM
[meter]
8.2
8.1
8.1
8.0
8.0
7.9
7.9
7.8
7.8
7.7
7.7
7.6
7.6
7.5
7.5
7.4
7.4
7.3
7.3
7.2
7.2
7.1
7.1
7.0
7.0
6.9
6.9
0.0
10.0
20.0
30.0
40.0
50.0
60.0
70.0
80.0
[meter]


MB2 1.207 1/1/1990 10:00:00 AM
[meter]
8.1
8.1
8.0
8.0
7.9
7.9
7.8
7.8
7.7
7.7
7.6
7.6
7.5
7.5
7.4
7.4
7.3
7.3
7.2
7.2
7.1
7.1
7.0
7.0
6.9
6.9
0.0
10.0
20.0
30.0
40.0
50.0
60.0
70.0
80.0
90.0
100.0
110.0
[meter]

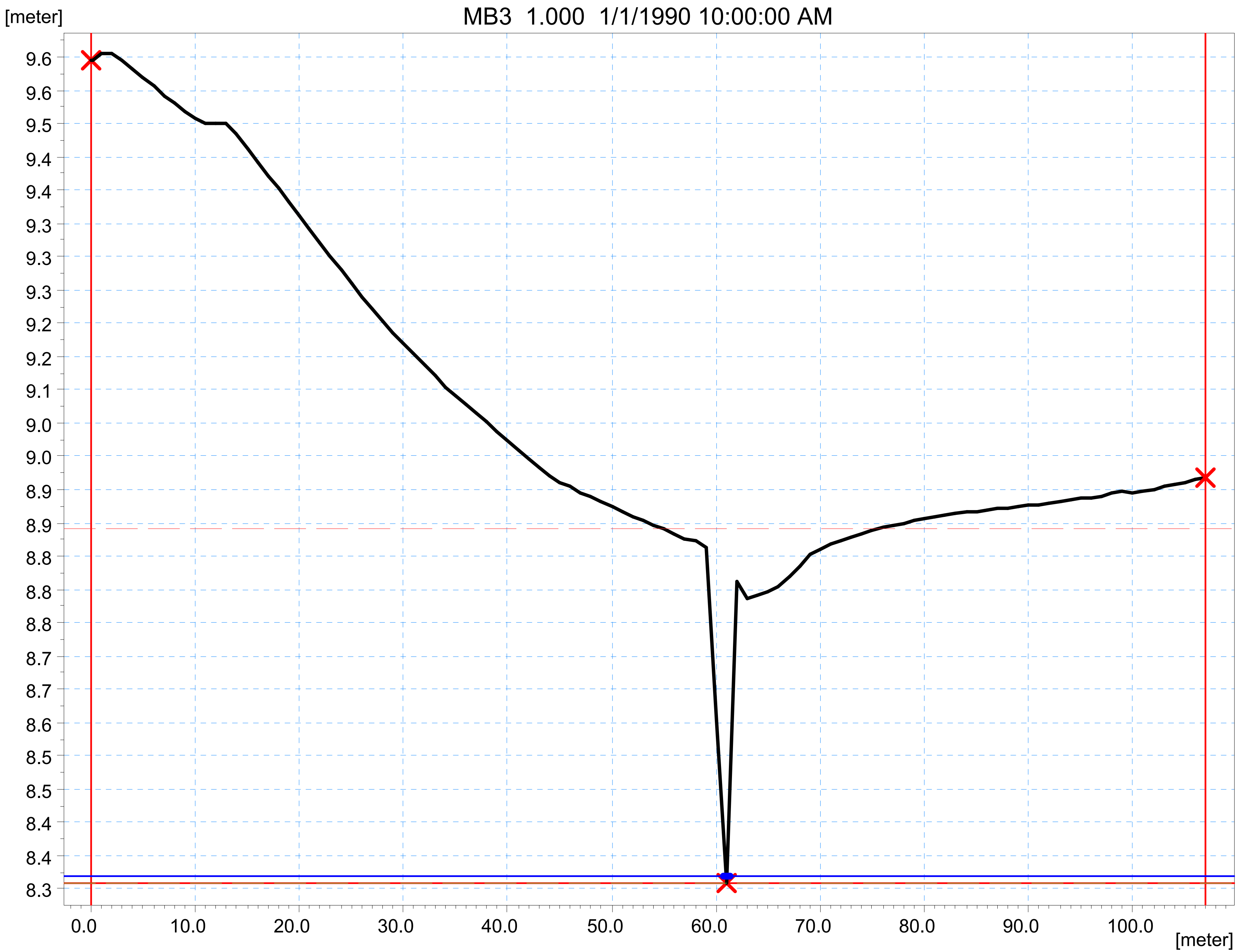
MB2 1.288 1/1/1990 10:00:00 AM
[meter]
6.86
6.84
6.82
6.80
6.78
6.76
6.74
6.72
6.70
6.68
6.66
6.64
6.62
6.60
6.58
6.56
6.54
6.52
6.50
6.48
6.46
6.44
6.42
6.40
6.38
6.36
6.34
6.32
6.30
6.28
6.26
6.24
6.22
6.20
6.18
6.16
6.14
6.12
0.0
20.0
40.0
60.0
80.0
100.0
120.0
140.0
160.0
[meter]



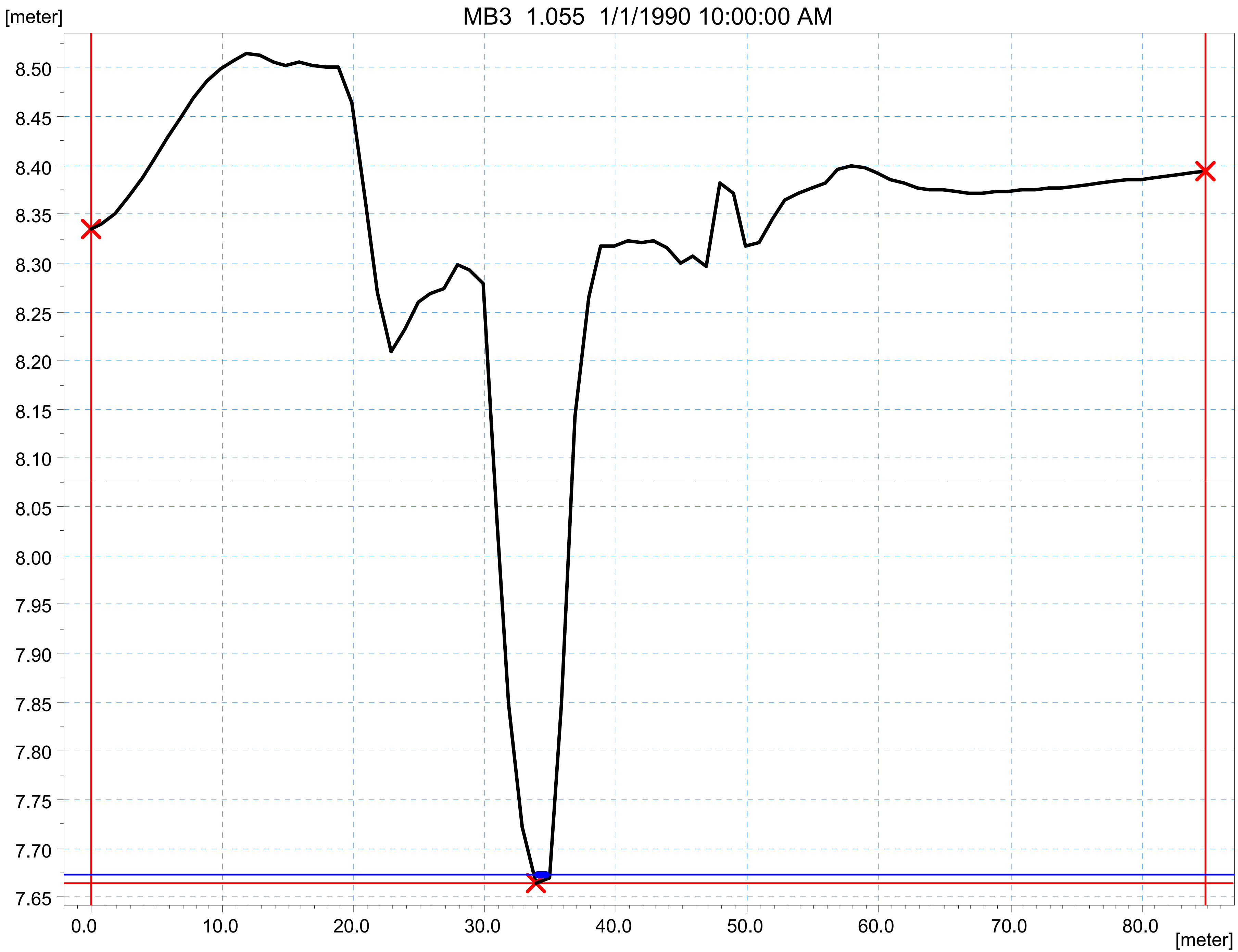
MB2 1.345 1/1/1990 10:00:00 AM
[meter]
6.36
6.34
6.32
6.30
6.28
6.26
6.24
6.22
6.20
6.18
6.16
6.14
6.12
6.10
6.08
6.06
6.04
6.02
6.00
5.98
5.96
5.94
5.92
5.90
5.88
5.86
5.84
5.82
5.80
5.78
5.76
5.74
5.72
5.70
5.68
5.66
5.64
0.0
20.0
40.0
60.0
80.0
100.0
120.0
140.0
160.0
180.0
[meter]



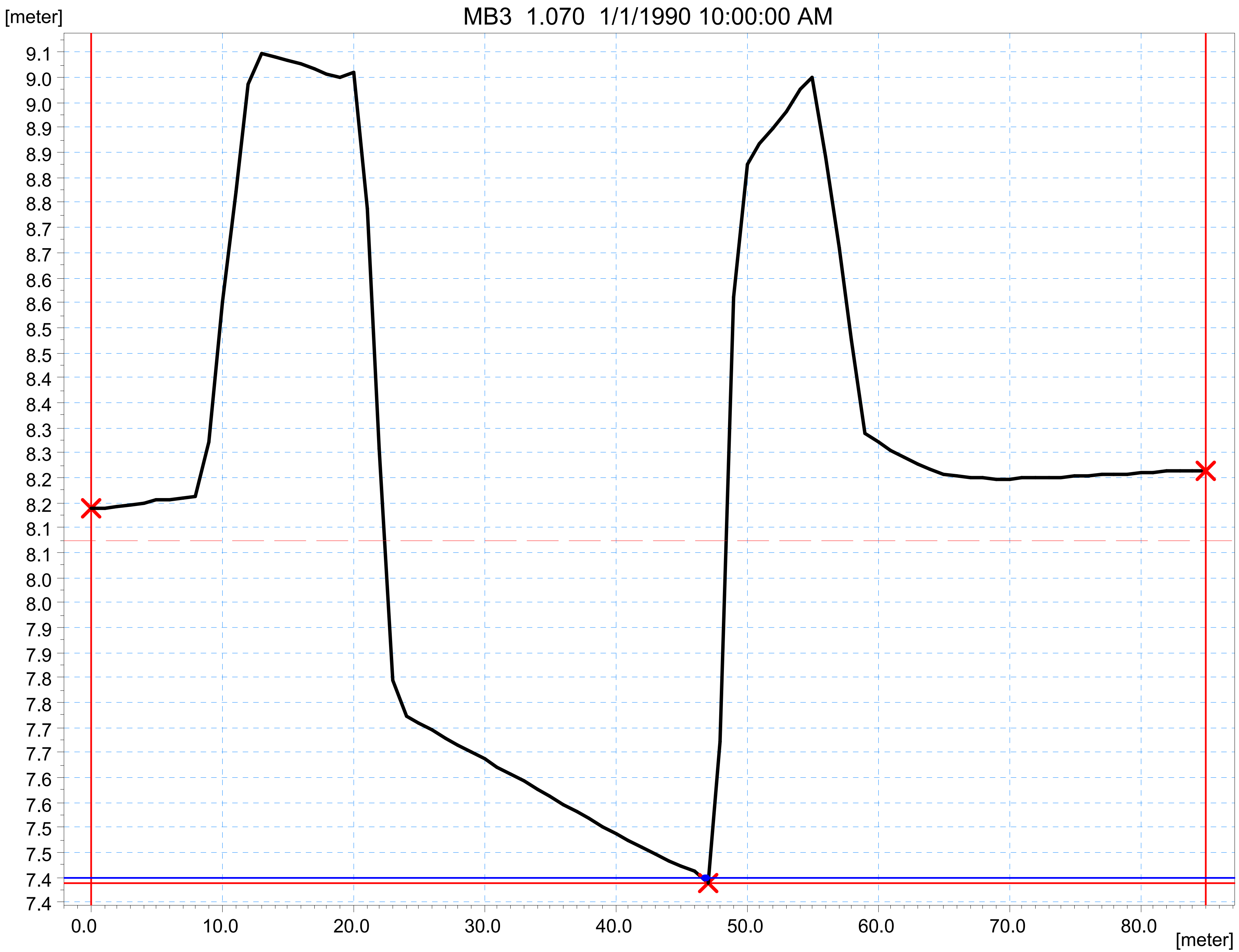
MB2 1.441 1/1/1990 10:00:00 AM
[meter]
5.85
5.80
5.75
5.70
5.65
5.60
5.55
5.50
5.45
5.40
5.35
5.30
5.25
5.20
5.15
5.10
5.05
5.00
0.0
50.0
100.0
150.0
200.0
250.0
300.0
350.0
[meter]



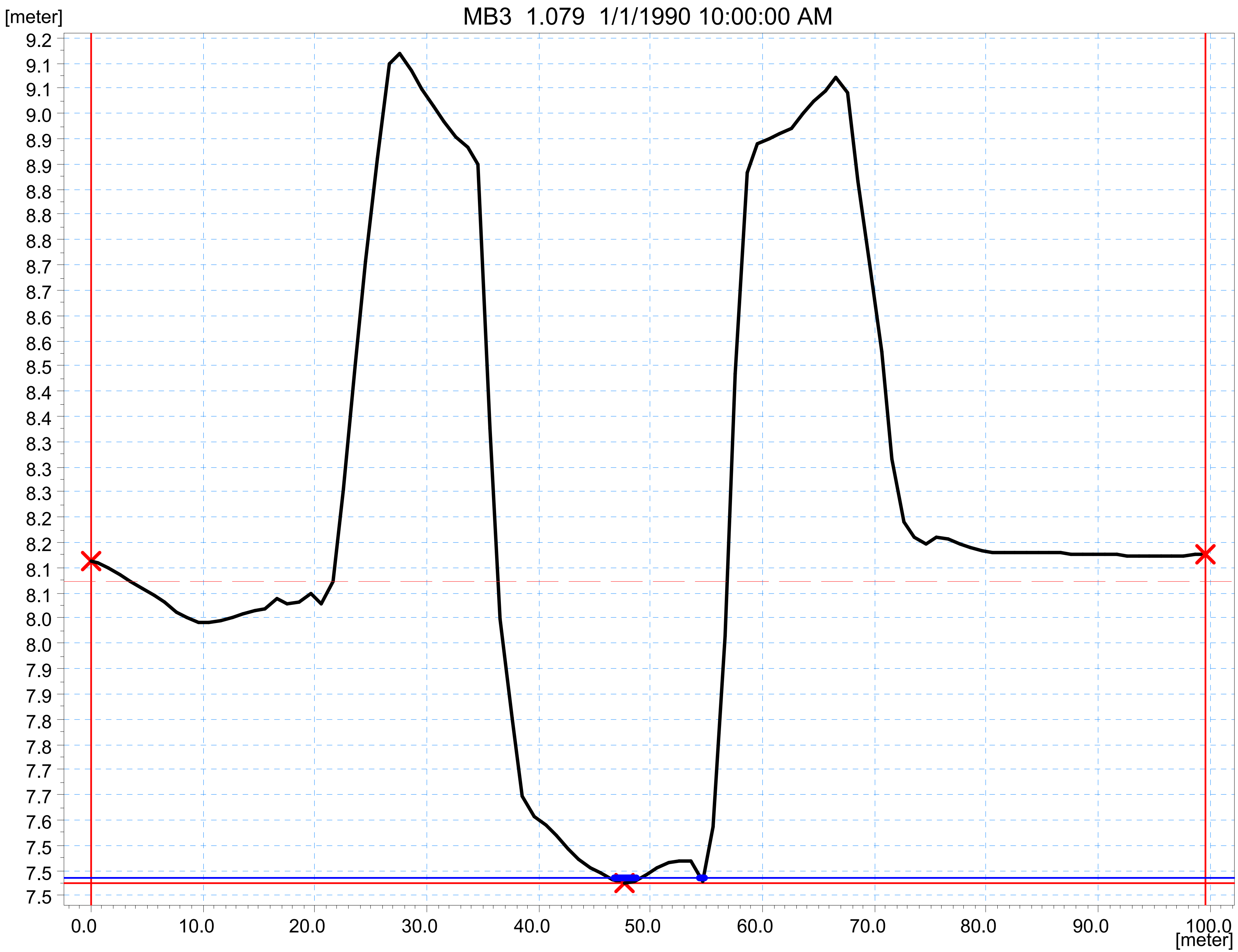
[meter]
MB3 1.000 1/1/1990 10:00:00 AM
9.6
9.6
9.5
9.4
9.4
9.3
9.3
9.3
9.2
9.2
9.1
9.0
9.0
8.9
8.9
8.8
8.8
8.8
8.7
8.7
8.6
8.5
8.5
8.4
8.4
8.3
0.0
10.0
20.0
30.0
40.0
50.0
60.0
70.0
80.0
90.0
100.0
[meter]

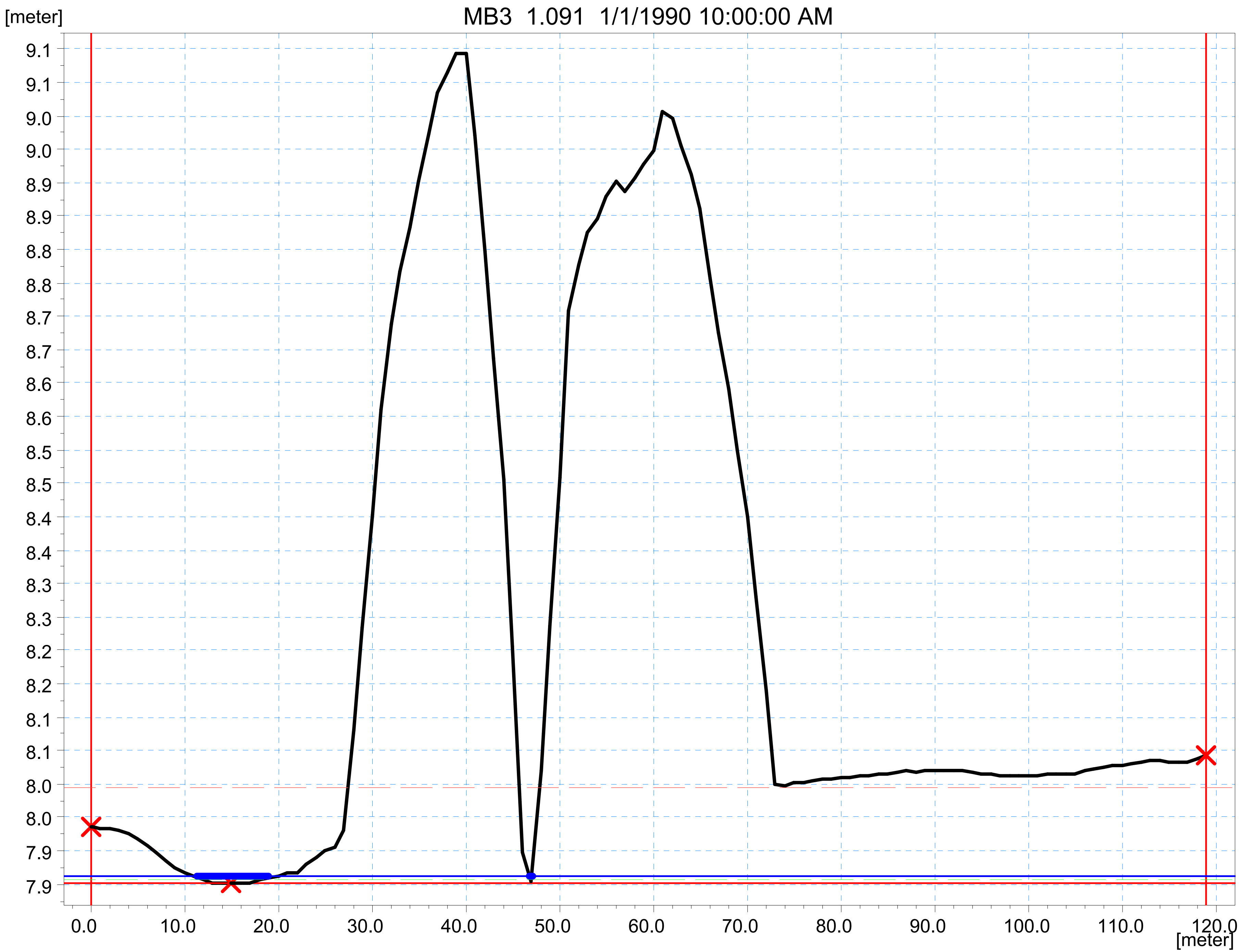


MB3 1.055 1/1/1990 10:00:00 AM
[meter]
8.50
8.45
8.40
8.35
8.30
8.25
8.20
8.15
8.10
8.05
8.00
7.95
7.90
7.85
7.80
7.75
7.70
7.65
0.0
10.0
20.0
30.0
40.0
50.0
60.0
70.0
80.0
[meter]

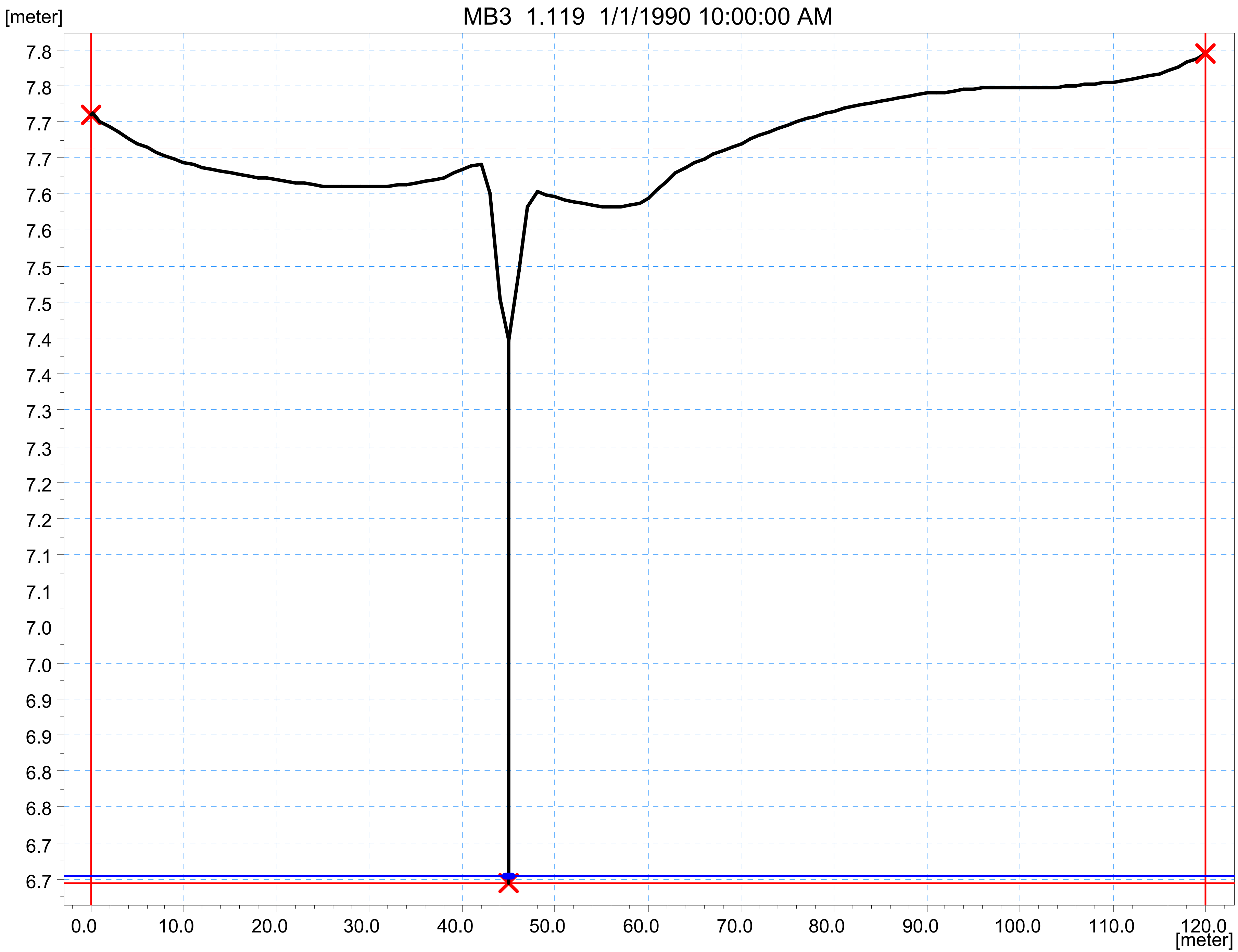


MB3 1.070 1/1/1990 10:00:00 AM
[meter]
9.1
9.0
9.0
8.9
8.9
8.8
8.8
8.7
8.7
8.6
8.6
8.5
8.5
8.4
8.4
8.3
8.3
8.2
8.2
8.1
8.1
8.0
8.0
7.9
7.9
7.8
7.8
7.7
7.7
7.6
7.6
7.5
7.5
7.4
7.4
0.0
10.0
20.0
30.0
40.0
50.0
60.0
70.0
80.0
[meter]

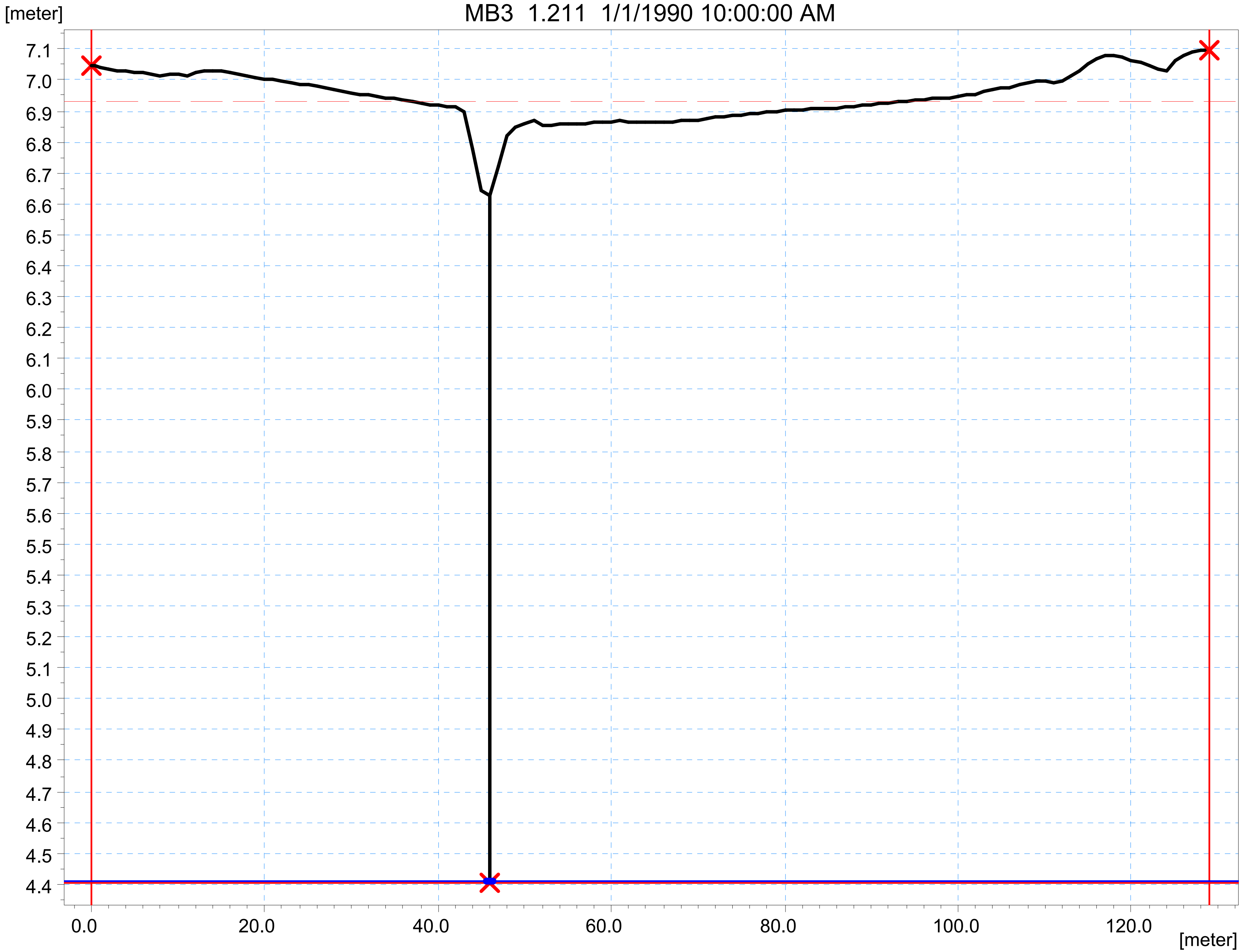

MB3 1.079 1/1/1990 10:00:00 AM
[meter]
9.2
9.1
9.1
9.0
8.9
8.9
8.8
8.8
8.8
8.7
8.7
8.6
8.6
8.5
8.4
8.4
8.3
8.3
8.3
8.2
8.2
8.1
8.1
8.0
8.0
7.9
7.9
7.8
7.8
7.7
7.7
7.6
7.5
7.5
7.5
0.0
10.0
20.0
30.0
40.0
50.0
60.0
70.0
80.0
90.0
100.0
[meter]

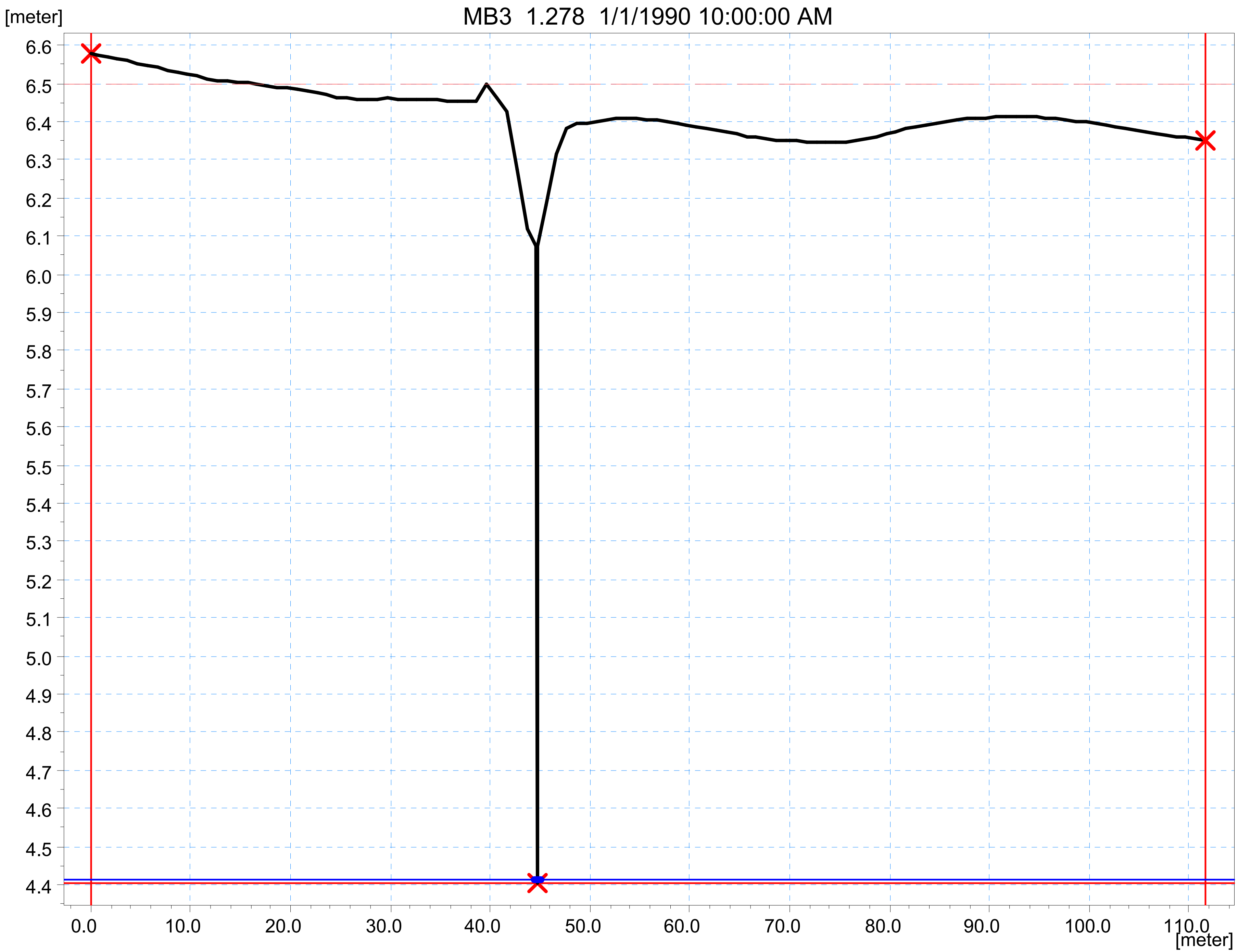


MB3 1.091 1/1/1990 10:00:00 AM
[meter]
9.1
9.1
9.0
9.0
8.9
8.9
8.8
8.8
8.7
8.7
8.6
8.6
8.5
8.5
8.4
8.4
8.3
8.3
8.2
8.2
8.1
8.1
8.0
8.0
7.9
7.9
0.0
10.0
20.0
30.0
40.0
50.0
60.0
70.0
80.0
90.0
100.0
110.0
120.0
[meter]

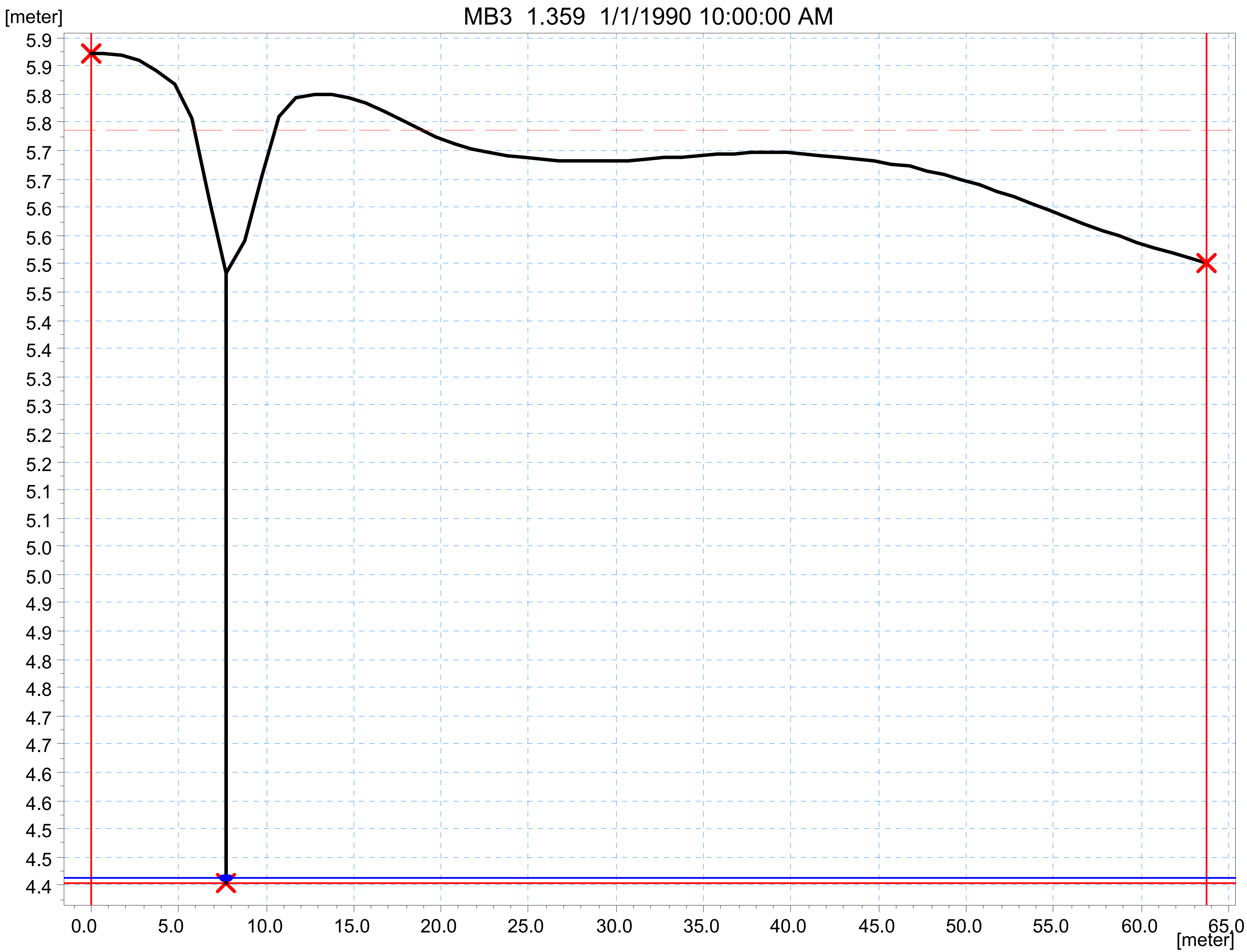


MB3 1.119 1/1/1990 10:00:00 AM
[meter]
7.8
7.8
7.7
7.7
7.6
7.6
7.5
7.5
7.4
7.4
7.3
7.3
7.2
7.2
7.1
7.1
7.0
7.0
6.9
6.9
6.8
6.8
6.7
6.7
0.0
10.0
20.0
30.0
40.0
50.0
60.0
70.0
80.0
90.0
100.0
110.0
120.0
[meter]

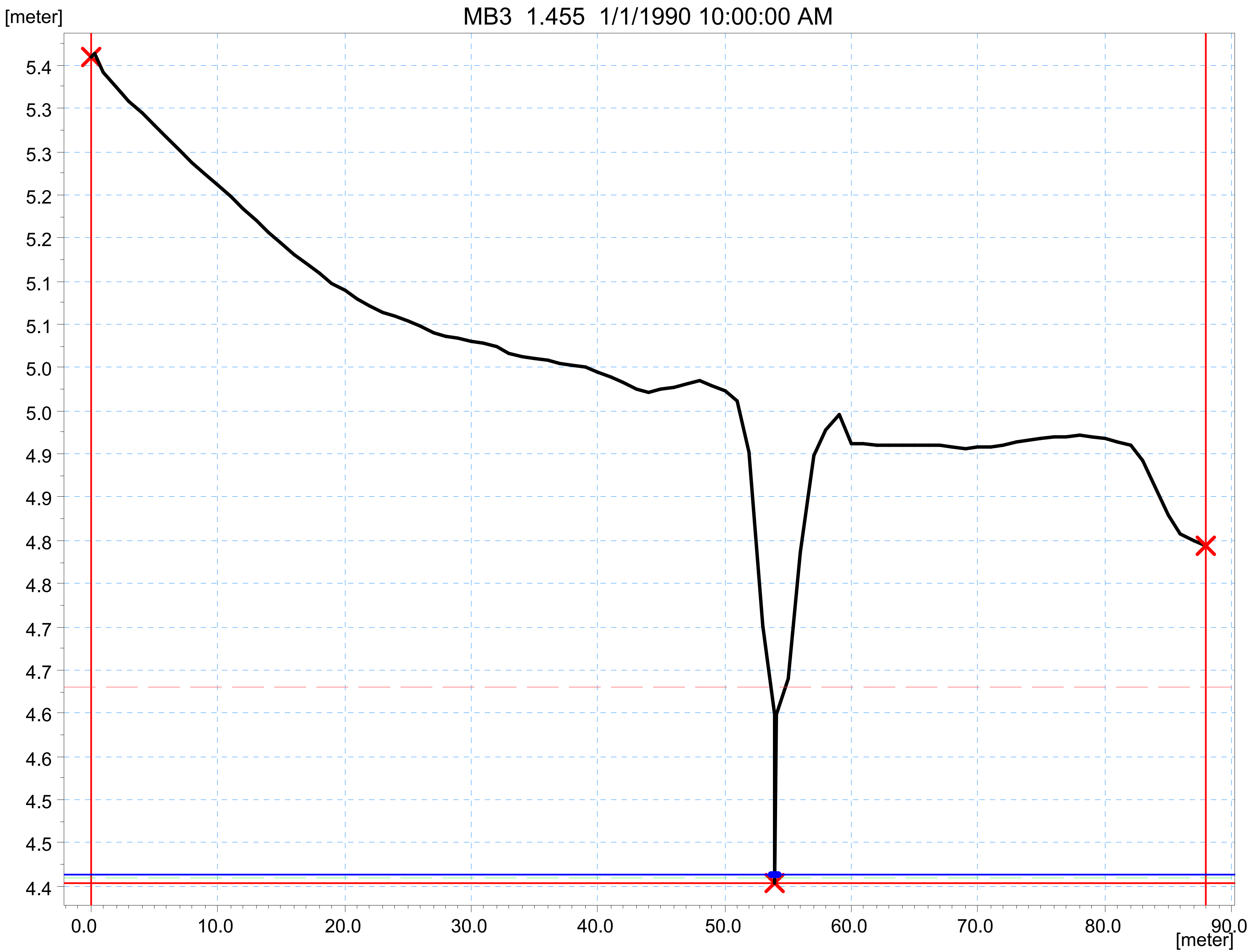

MB3 1.211 1/1/1990 10:00:00 AM
[meter]
7.1
7.0
6.9
6.8
6.7
6.6
6.5
6.4
6.3
6.2
6.1
6.0
5.9
5.8
5.7
5.6
5.5
5.4
5.3
5.2
5.1
5.0
4.9
4.8
4.7
4.6
4.5
4.4
0.0
20.0
40.0
60.0
80.0
100.0
120.0
[meter]



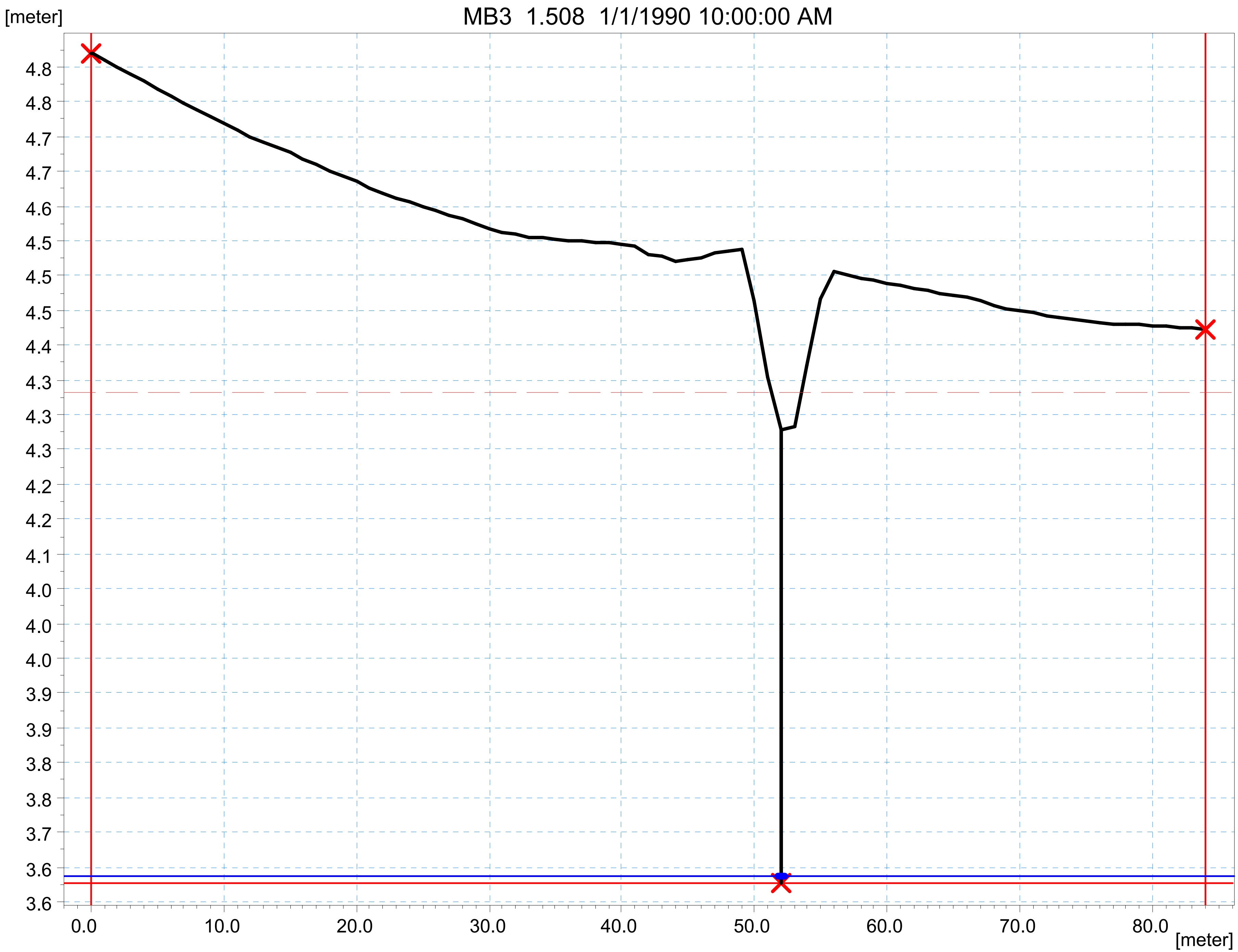
MB3 1.278 1/1/1990 10:00:00 AM
[meter]
6.6
6.5
6.4
6.3
6.2
6.1
6.0
5.9
5.8
5.7
5.6
5.5
5.4
5.3
5.2
5.1
5.0
4.9
4.8
4.7
4.6
4.5
4.4
0.0
10.0
20.0
30.0
40.0
50.0
60.0
70.0
80.0
90.0
100.0
110.0
[meter]

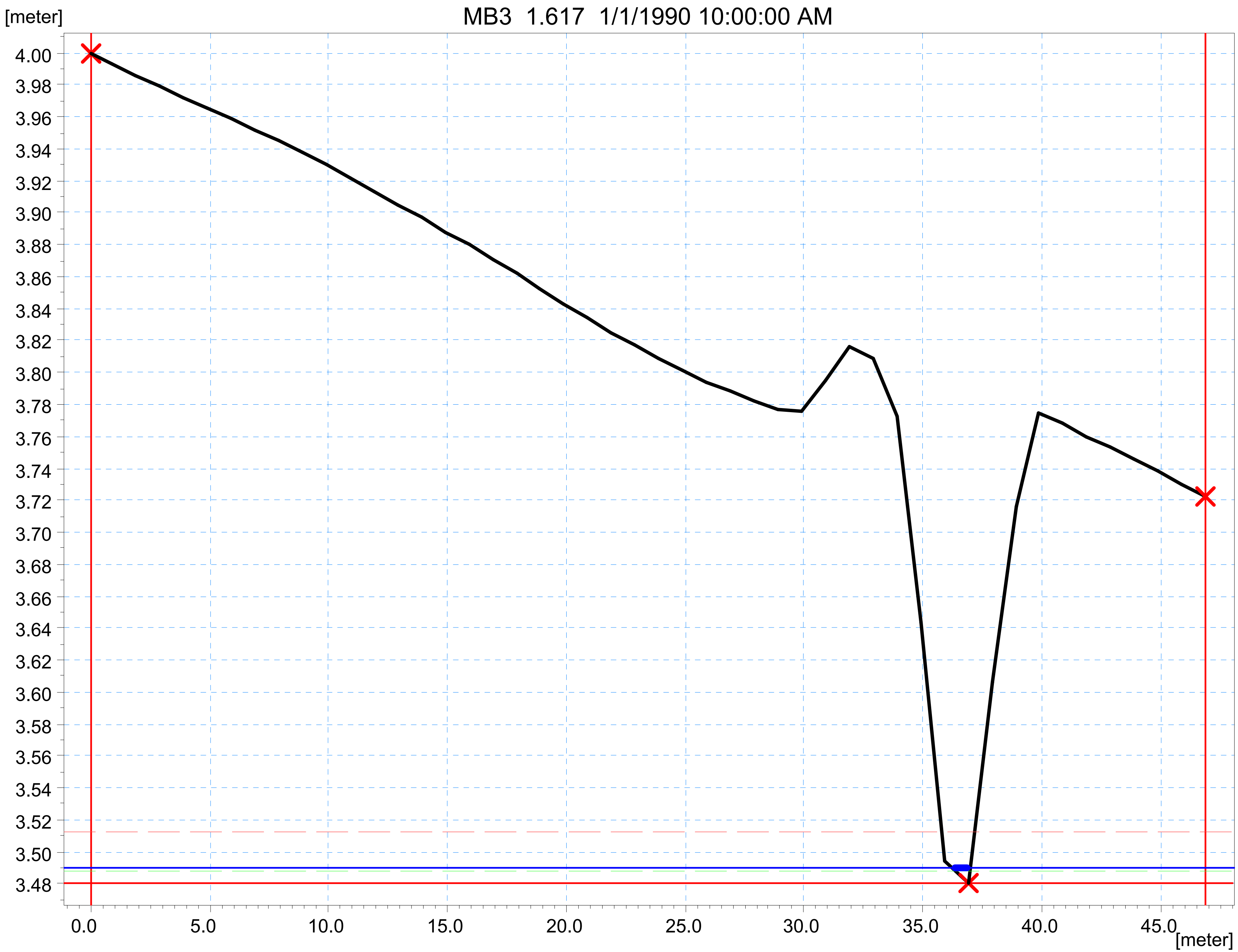


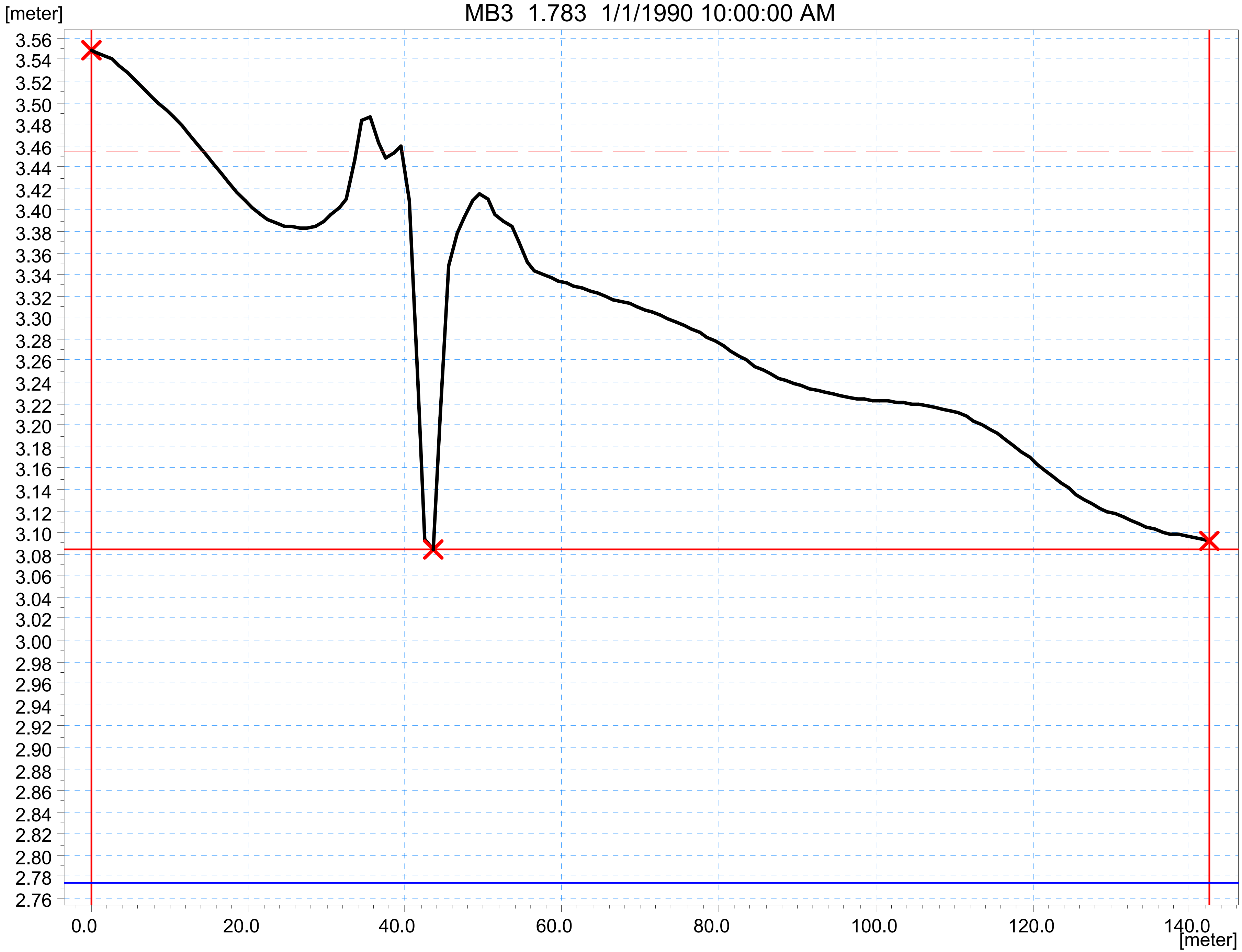
MB3 1.359 1/1/1990 10:00:00 AM
[meter]
5.9
5.9
5.8
5.8
5.7
5.7
5.6
5.6
5.5
5.5
5.4
5.4
5.3
5.3
5.2
5.2
5.1
5.1
5.0
5.0
4.9
4.9
4.8
4.8
4.7
4.7
4.6
4.6
4.5
4.5
4.4
0.0
5.0
10.0
15.0
20.0
25.0
30.0
35.0
40.0
45.0
50.0
55.0
60.0
65.0
[meter]

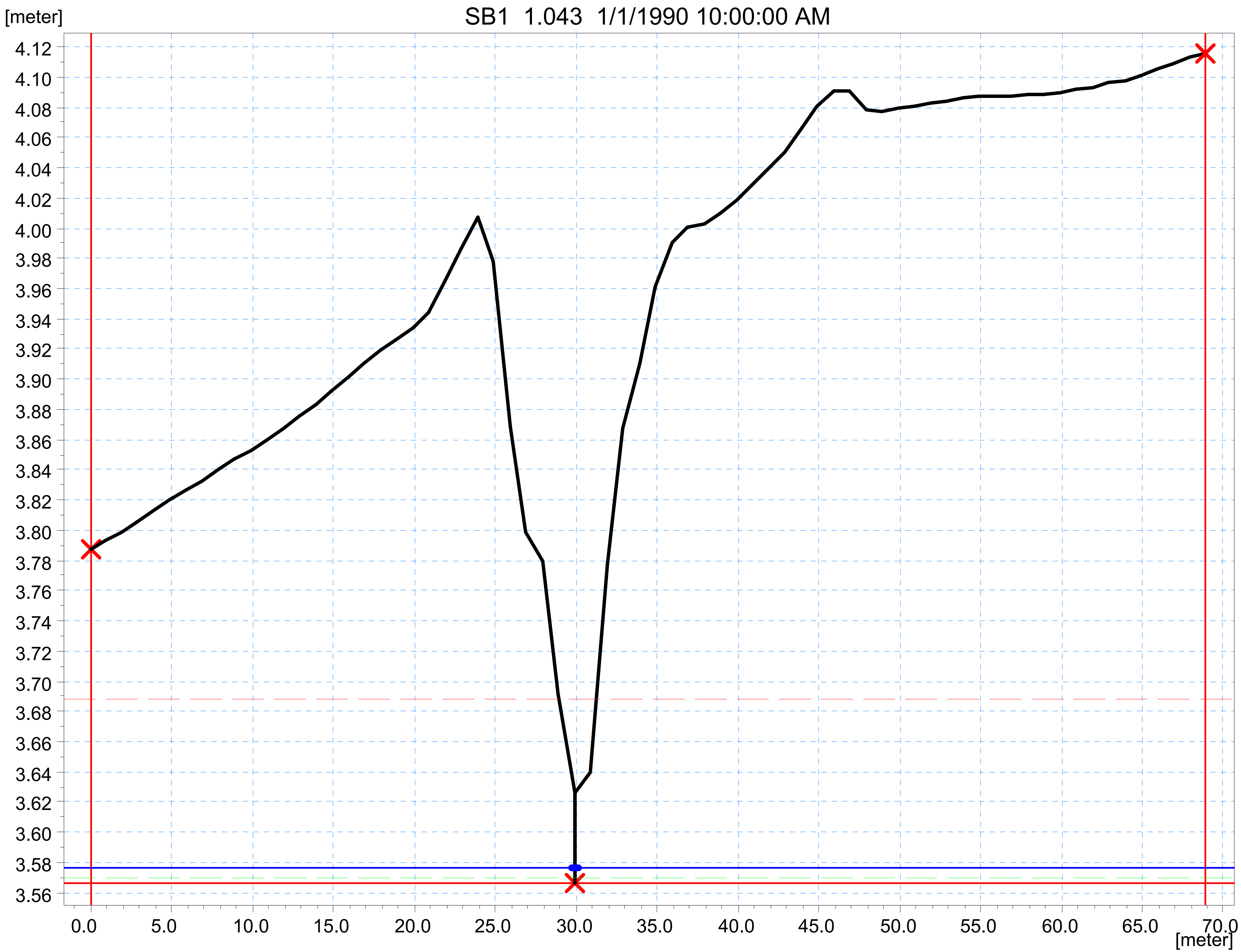


MB3 1.455 1/1/1990 10:00:00 AM
[meter]
5.4
5.3
5.3
5.2
5.2
5.1
5.1
5.0
5.0
4.9
4.9
4.8
4.8
4.7
4.7
4.6
4.6
4.5
4.5
4.4
0.0
10.0
20.0
30.0
40.0
50.0
60.0
70.0
80.0
90.0
[meter]

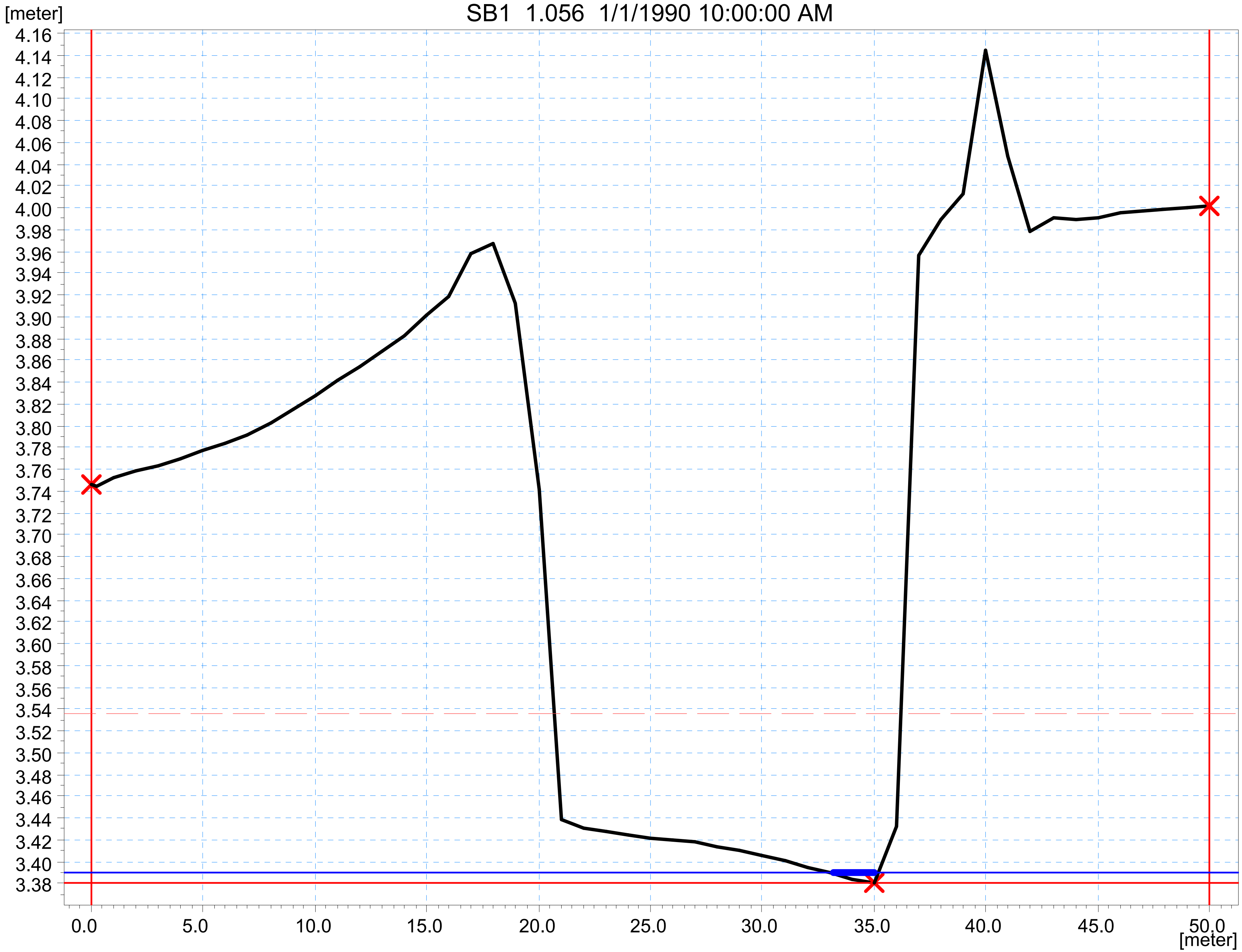

MB3 1.508 1/1/1990 10:00:00 AM
[meter]
4.8
4.8
4.7
4.7
4.6
4.5
4.5
4.5
4.4
4.3
4.3
4.3
4.2
4.2
4.1
4.0
4.0
4.0
3.9
3.9
3.8
3.8
3.7
3.6
3.6
0.0
10.0
20.0
30.0
40.0
50.0
60.0
70.0
80.0
[meter]


MB3 1.617 1/1/1990 10:00:00 AM
[meter]
4.00
3.98
3.96
3.94
3.92
3.90
3.88
3.86
3.84
3.82
3.80
3.78
3.76
3.74
3.72
3.70
3.68
3.66
3.64
3.62
3.60
3.58
3.56
3.54
3.52
3.50
3.48
0.0
5.0
10.0
15.0
20.0
25.0
30.0
35.0
40.0
45.0
[meter]

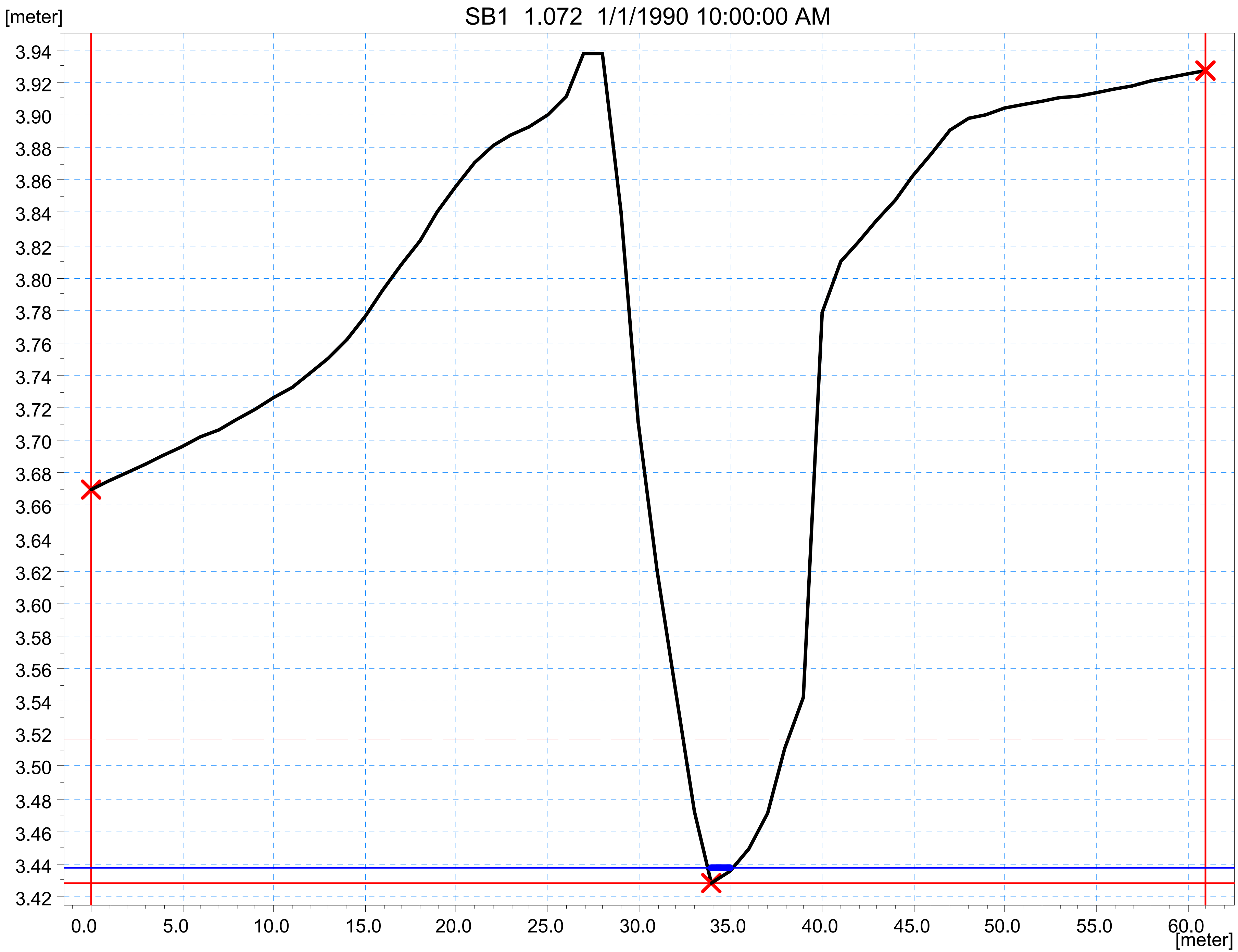

MB3 1.783 1/1/1990 10:00:00 AM
[meter]
3.56
3.54
3.52
3.50
3.48
3.46
3.44
3.42
3.40
3.38
3.36
3.34
3.32
3.30
3.28
3.26
3.24
3.22
3.20
3.18
3.16
3.14
3.12
3.10
3.08
3.06
3.04
3.02
3.00
2.98
2.96
2.94
2.92
2.90
2.88
2.86
2.84
2.82
2.80
2.78
2.76
0.0
20.0
40.0
60.0
80.0
100.0
120.0
140.0
[meter]

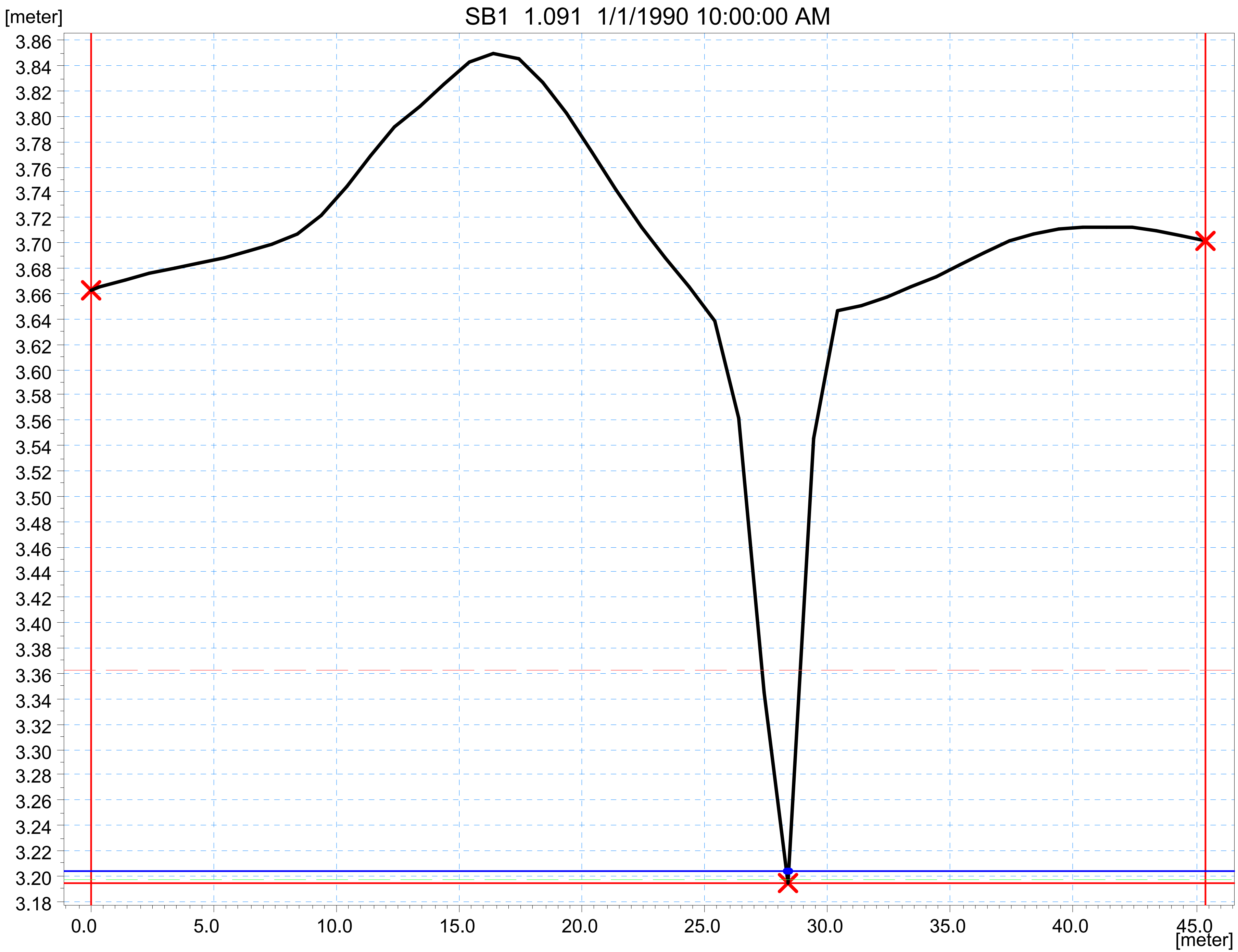


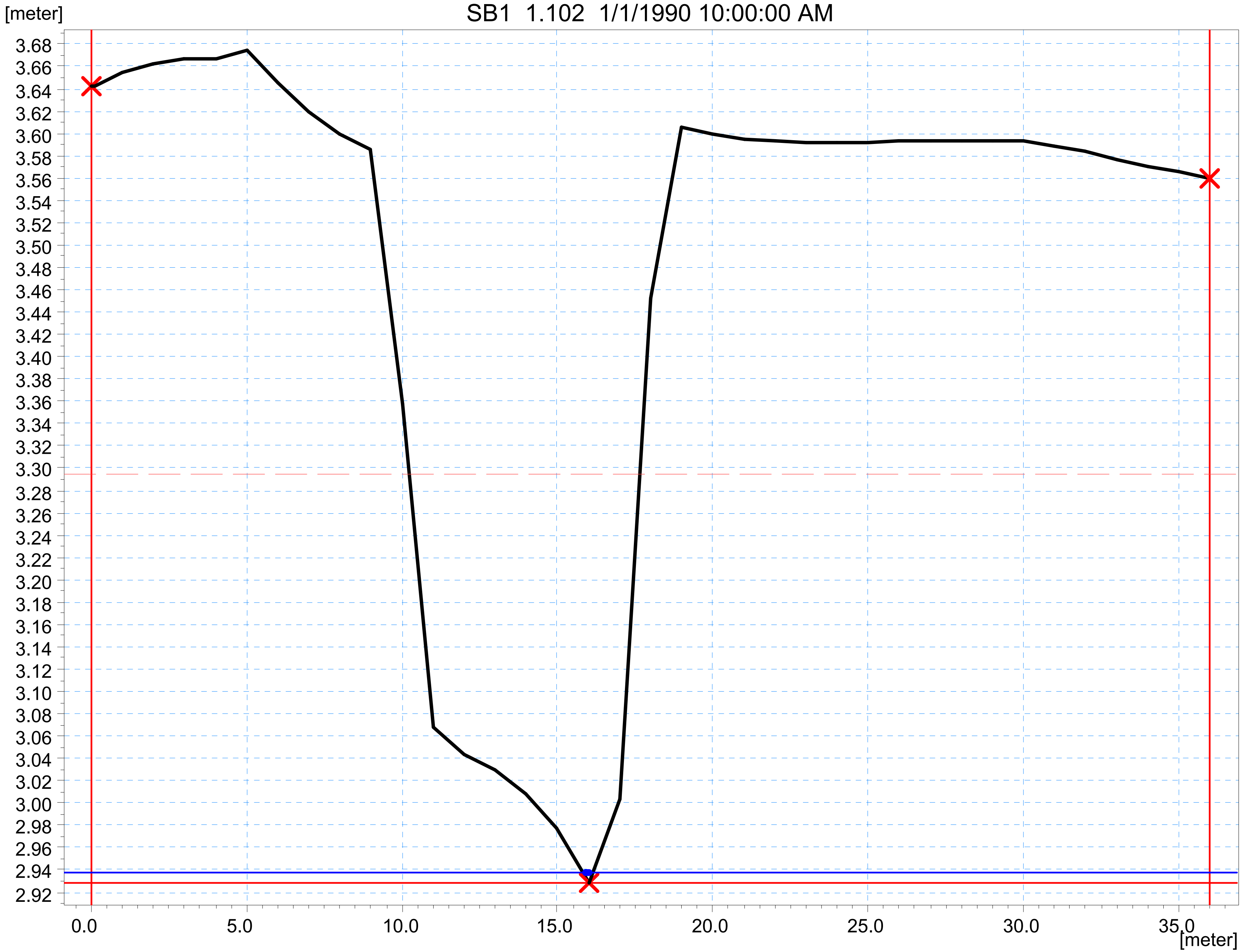
SB1 1.043 1/1/1990 10:00:00 AM
[meter]
4.12
4.10
4.08
4.06
4.04
4.02
4.00
3.98
3.96
3.94
3.92
3.90
3.88
3.86
3.84
3.82
3.80
3.78
3.76
3.74
3.72
3.70
3.68
3.66
3.64
3.62
3.60
3.58
3.56
0.0
5.0
10.0
15.0
20.0
25.0
30.0
35.0
40.0
45.0
50.0
55.0
60.0
65.0
70.0
[meter]



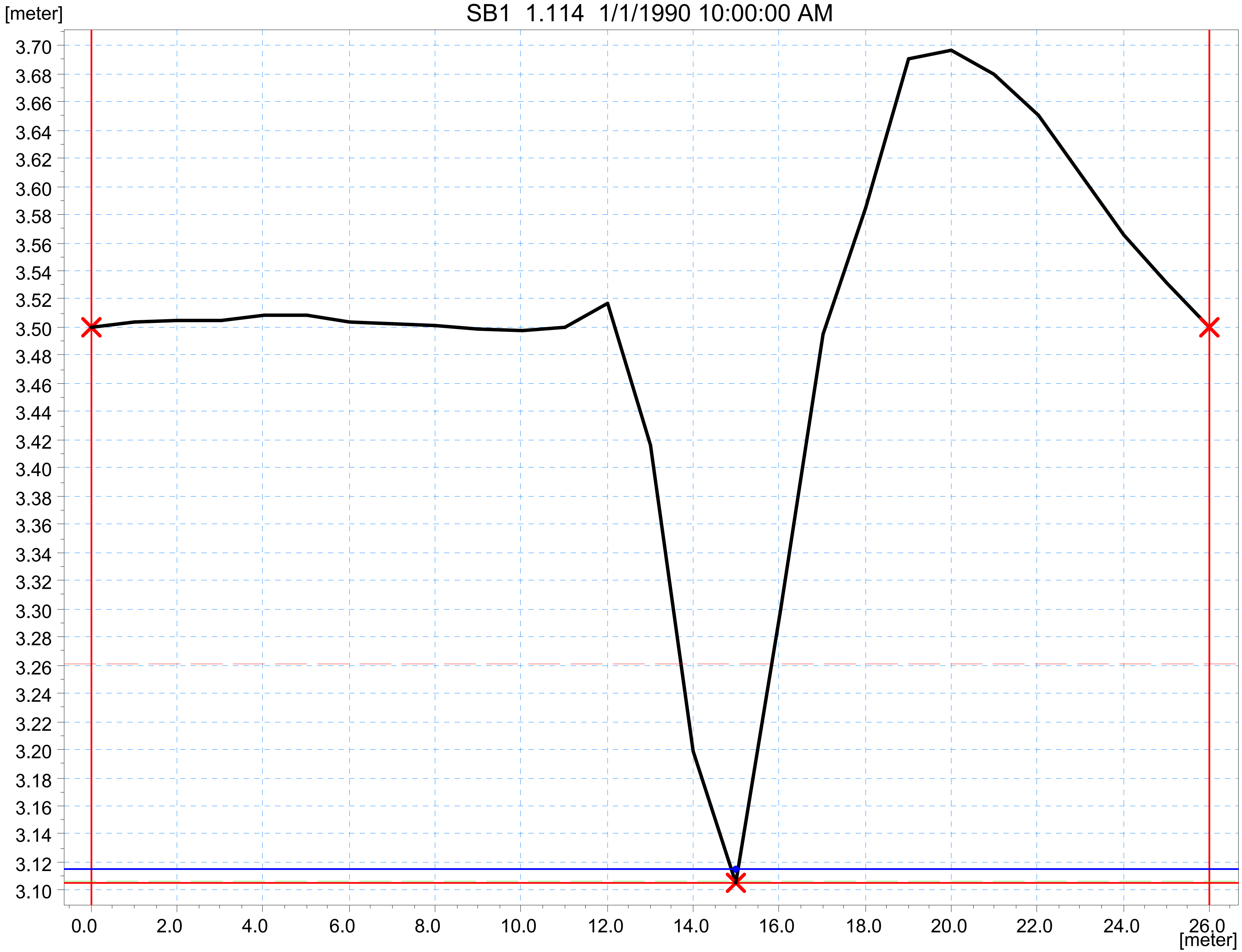
SB1 1.056 1/1/1990 10:00:00 AM
[meter]
4.16
4.14
4.12
4.10
4.08
4.06
4.04
4.02
4.00
3.98
3.96
3.94
3.92
3.90
3.88
3.86
3.84
3.82
3.80
3.78
3.76
3.74
3.72
3.70
3.68
3.66
3.64
3.62
3.60
3.58
3.56
3.54
3.52
3.50
3.48
3.46
3.44
3.42
3.40
3.38
0.0
5.0
10.0
15.0
20.0
25.0
30.0
35.0
40.0
45.0
50.0
[meter]

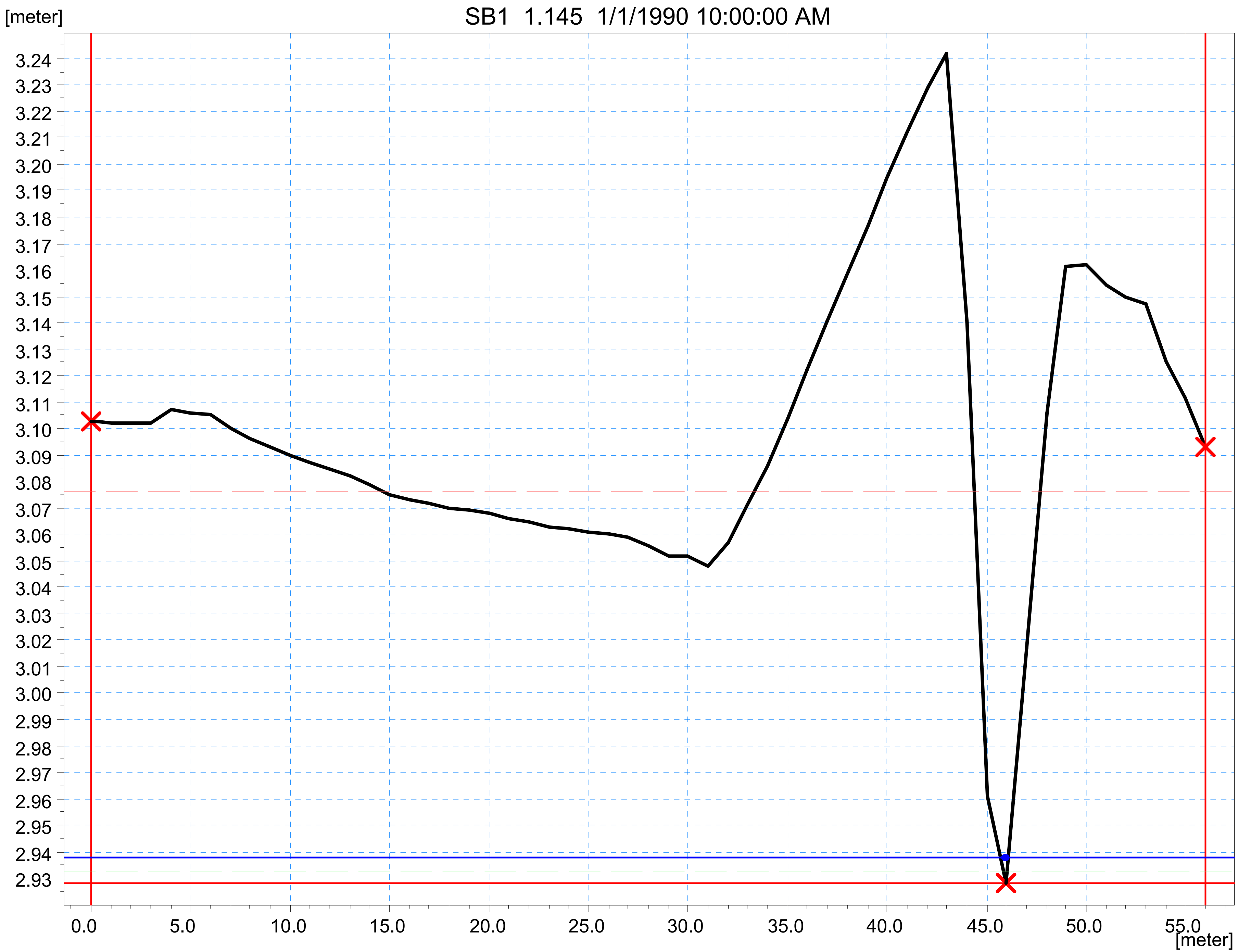

SB1 1.072 1/1/1990 10:00:00 AM
[meter]
3.94
3.92
3.90
3.88
3.86
3.84
3.82
3.80
3.78
3.76
3.74
3.72
3.70
3.68
3.66
3.64
3.62
3.60
3.58
3.56
3.54
3.52
3.50
3.48
3.46
3.44
3.42
0.0
5.0
10.0
15.0
20.0
25.0
30.0
35.0
40.0
45.0
50.0
55.0
60.0
[meter]


SB1 1.091 1/1/1990 10:00:00 AM
[meter]
3.86
3.84
3.82
3.80
3.78
3.76
3.74
3.72
3.70
3.68
3.66
3.64
3.62
3.60
3.58
3.56
3.54
3.52
3.50
3.48
3.46
3.44
3.42
3.40
3.38
3.36
3.34
3.32
3.30
3.28
3.26
3.24
3.22
3.20
3.18
0.0
5.0
10.0
15.0
20.0
25.0
30.0
35.0
40.0
45.0
[meter]

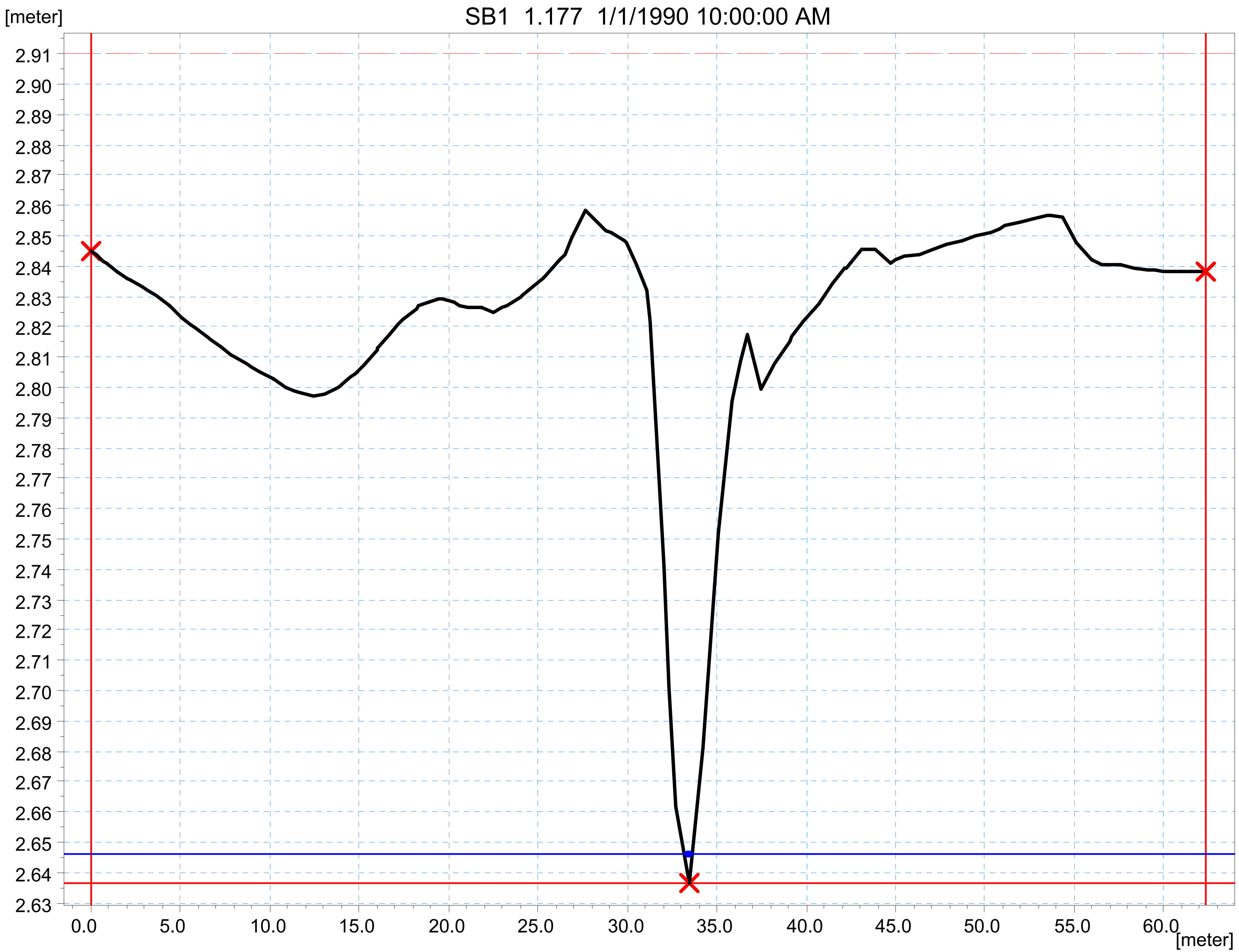


[meter]
SB1 1.102 1/1/1990 10:00:00 AM
3.68
3.66
3.64
3.62
3.60
3.58
3.56
3.54
3.52
3.50
3.48
3.46
3.44
3.42
3.40
3.38
3.36
3.34
3.32
3.30
3.28
3.26
3.24
3.22
3.20
3.18
3.16
3.14
3.12
3.10
3.08
3.06
3.04
3.02
3.00
2.98
2.96
2.94
2.92
0.0
5.0
10.0
15.0
20.0
25.0
30.0
35.0
[meter]

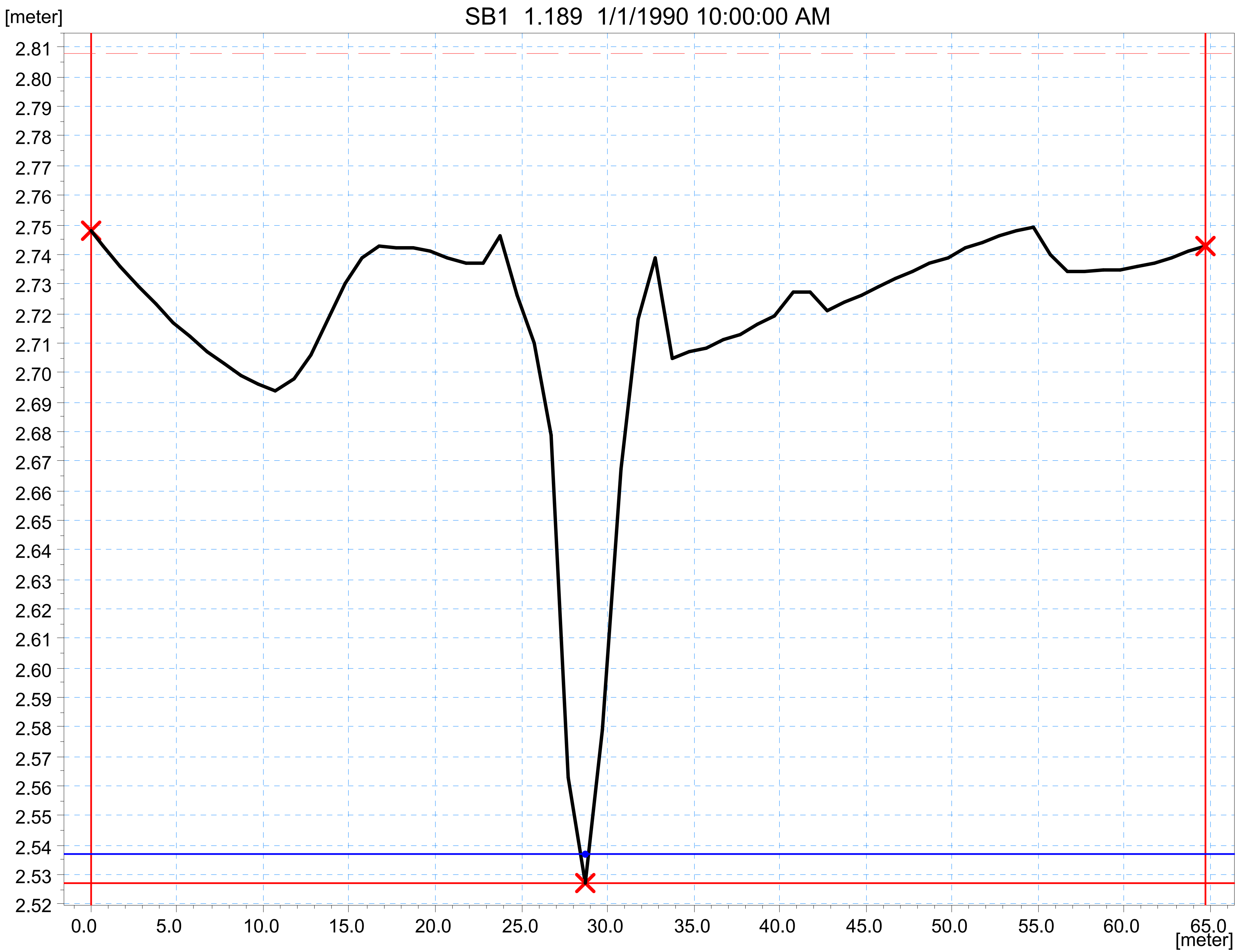

SB1 1.114 1/1/1990 10:00:00 AM
[meter]
3.70
3.68
3.66
3.64
3.62
3.60
3.58
3.56
3.54
3.52
3.50
3.48
3.46
3.44
3.42
3.40
3.38
3.36
3.34
3.32
3.30
3.28
3.26
3.24
3.22
3.20
3.18
3.16
3.14
3.12
3.10
0.0
2.0
4.0
6.0
8.0
10.0
12.0
14.0
16.0
18.0
20.0
22.0
24.0
26.0
[meter]



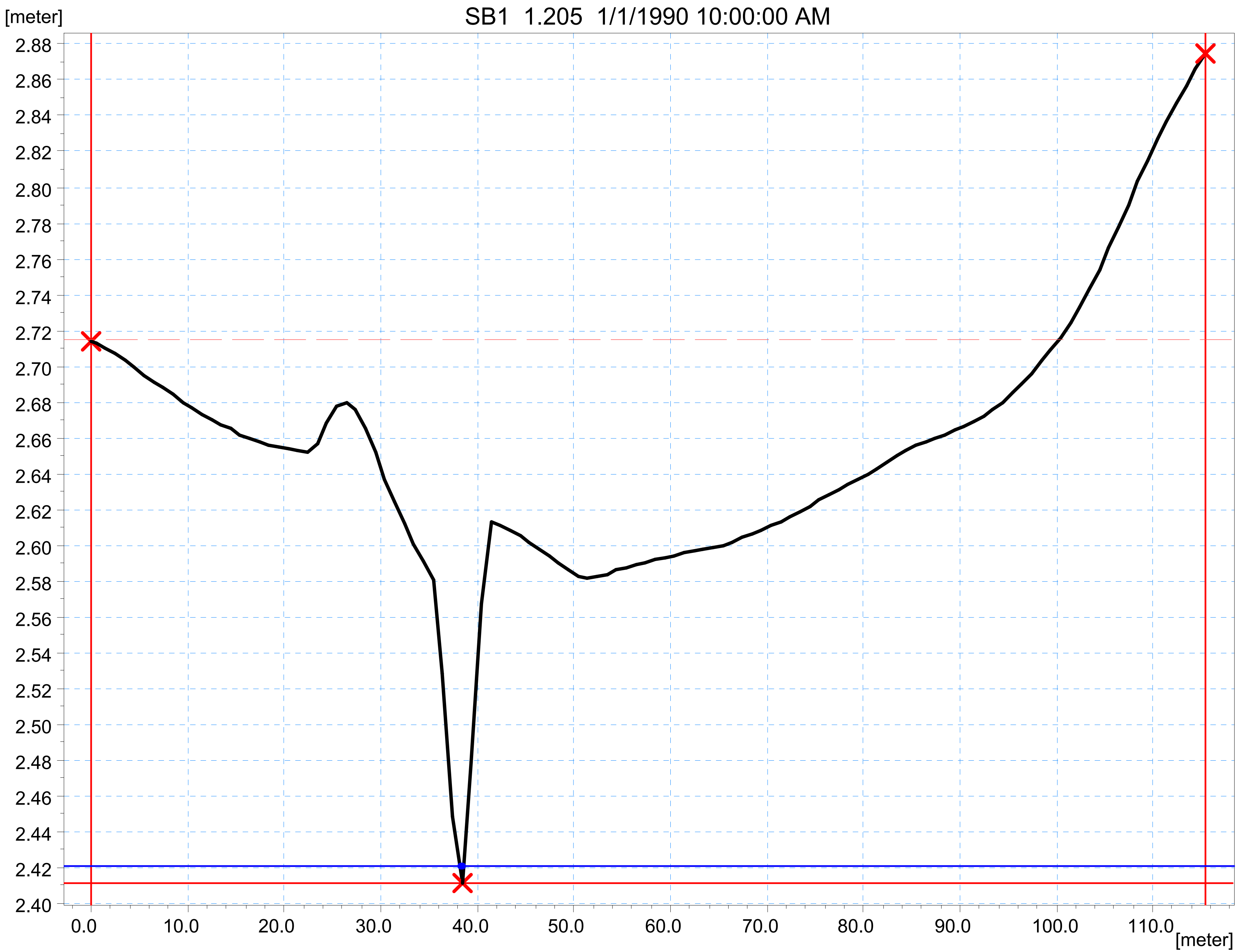
SB1 1.145 1/1/1990 10:00:00 AM
[meter]
3.24
3.23
3.22
3.21
3.20
3.19
3.18
3.17
3.16
3.15
3.14
3.13
3.12
3.11
3.10
3.09
3.08
3.07
3.06
3.05
3.04
3.03
3.02
3.01
3.00
2.99
2.98
2.97
2.96
2.95
2.94
2.93
0.0
5.0
10.0
15.0
20.0
25.0
30.0
35.0
40.0
45.0
50.0
55.0
[meter]

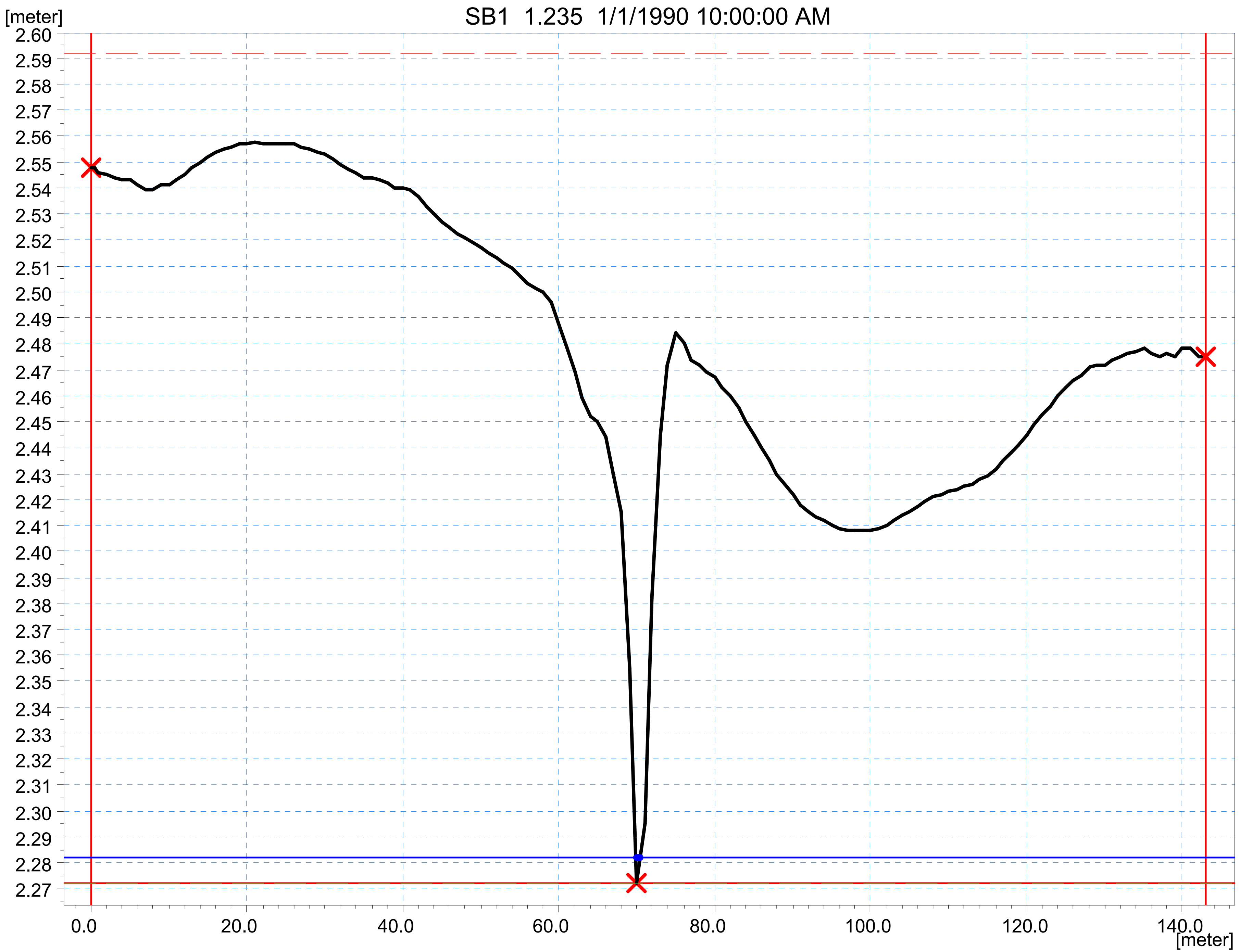


SB1 1.177 1/1/1990 10:00:00 AM
[meter]
2.91
2.90
2.89
2.88
2.87
2.86
2.85
2.84
2.83
2.82
2.81
2.80
2.79
2.78
2.77
2.76
2.75
2.74
2.73
2.72
2.71
2.70
2.69
2.68
2.67
2.66
2.65
2.64
2.63
0.0
5.0
10.0
15.0
20.0
25.0
30.0
35.0
40.0
45.0
50.0
55.0
60.0
[meter]

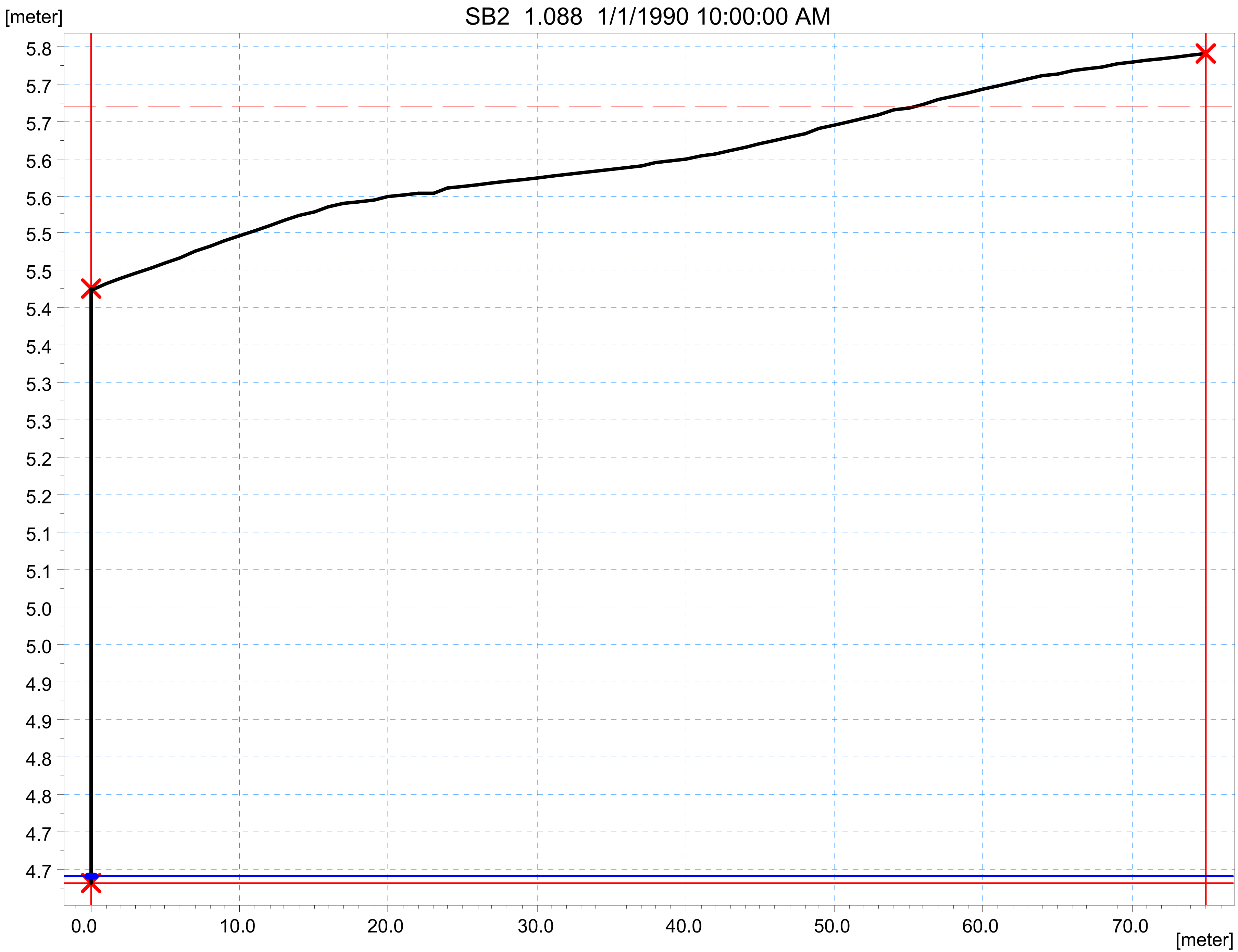


SB1 1.189 1/1/1990 10:00:00 AM
[meter]
2.81
2.80
2.79
2.78
2.77
2.76
2.75
2.74
2.73
2.72
2.71
2.70
2.69
2.68
2.67
2.66
2.65
2.64
2.63
2.62
2.61
2.60
2.59
2.58
2.57
2.56
2.55
2.54
2.53
2.52
0.0
5.0
10.0
15.0
20.0
25.0
30.0
35.0
40.0
45.0
50.0
55.0
60.0
65.0
[meter]

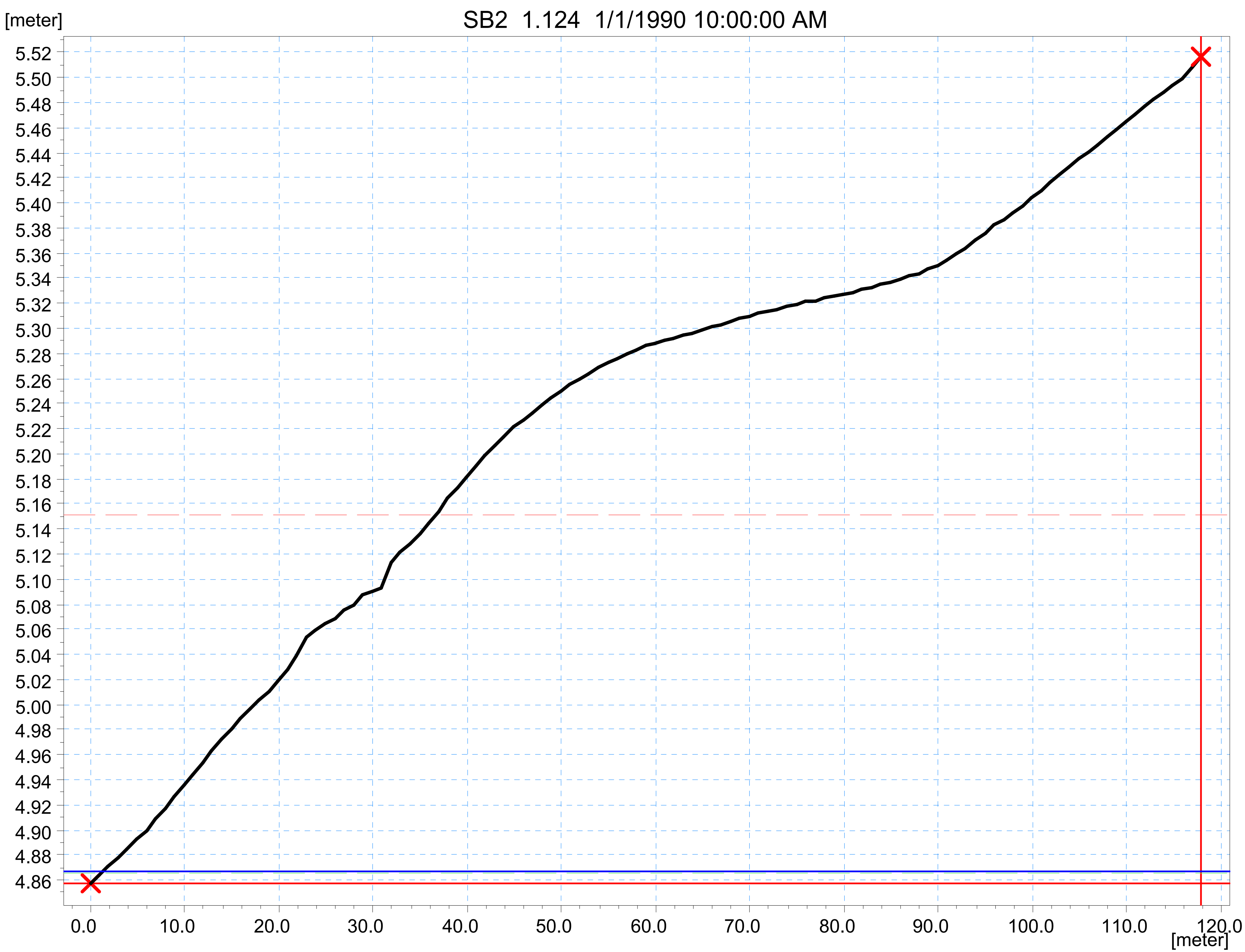

SB1 1.205 1/1/1990 10:00:00 AM
[meter]
2.88
2.86
2.84
2.82
2.80
2.78
2.76
2.74
2.72
2.70
2.68
2.66
2.64
2.62
2.60
2.58
2.56
2.54
2.52
2.50
2.48
2.46
2.44
2.42
2.40
0.0
10.0
20.0
30.0
40.0
50.0
60.0
70.0
80.0
90.0
100.0
110.0
[meter]

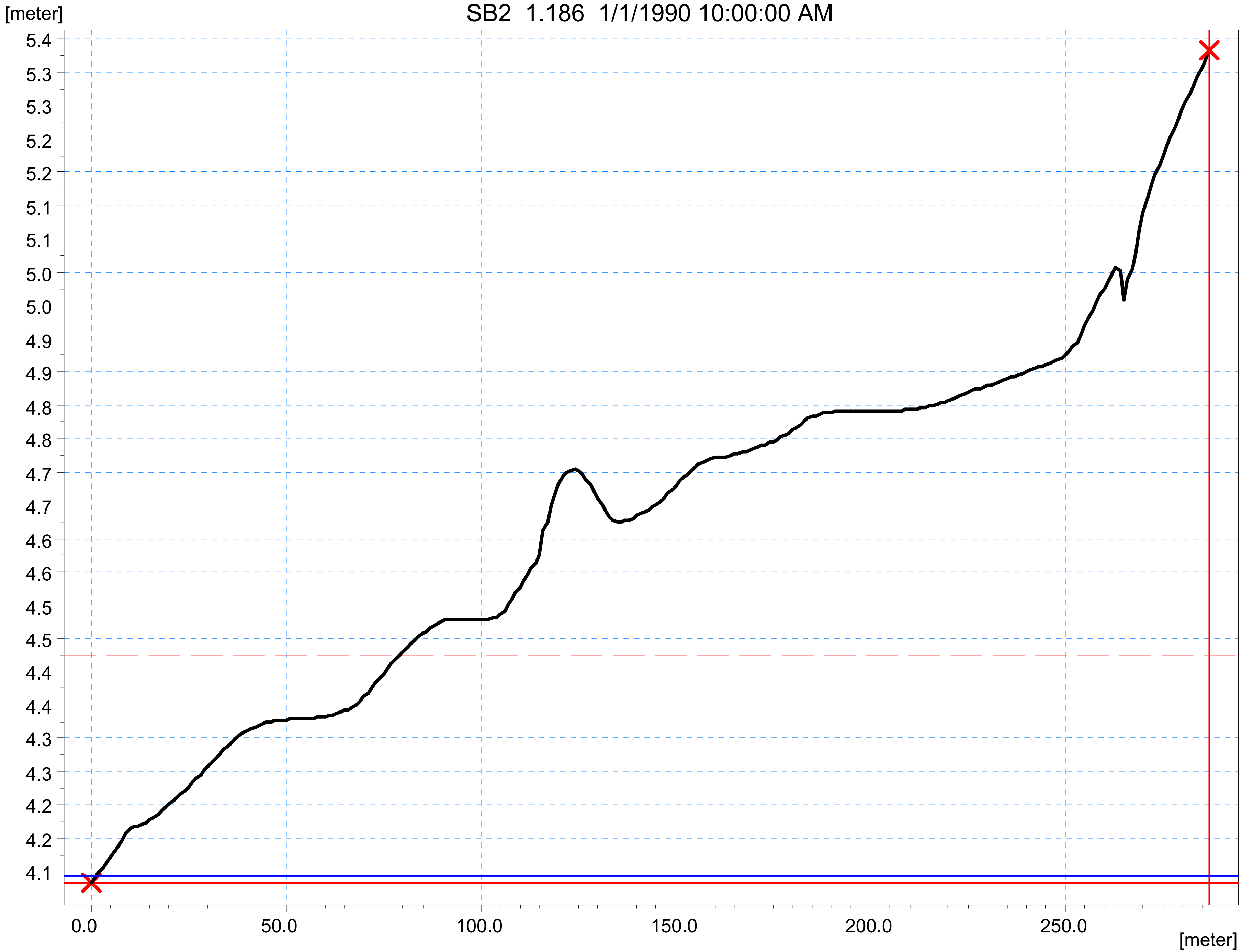


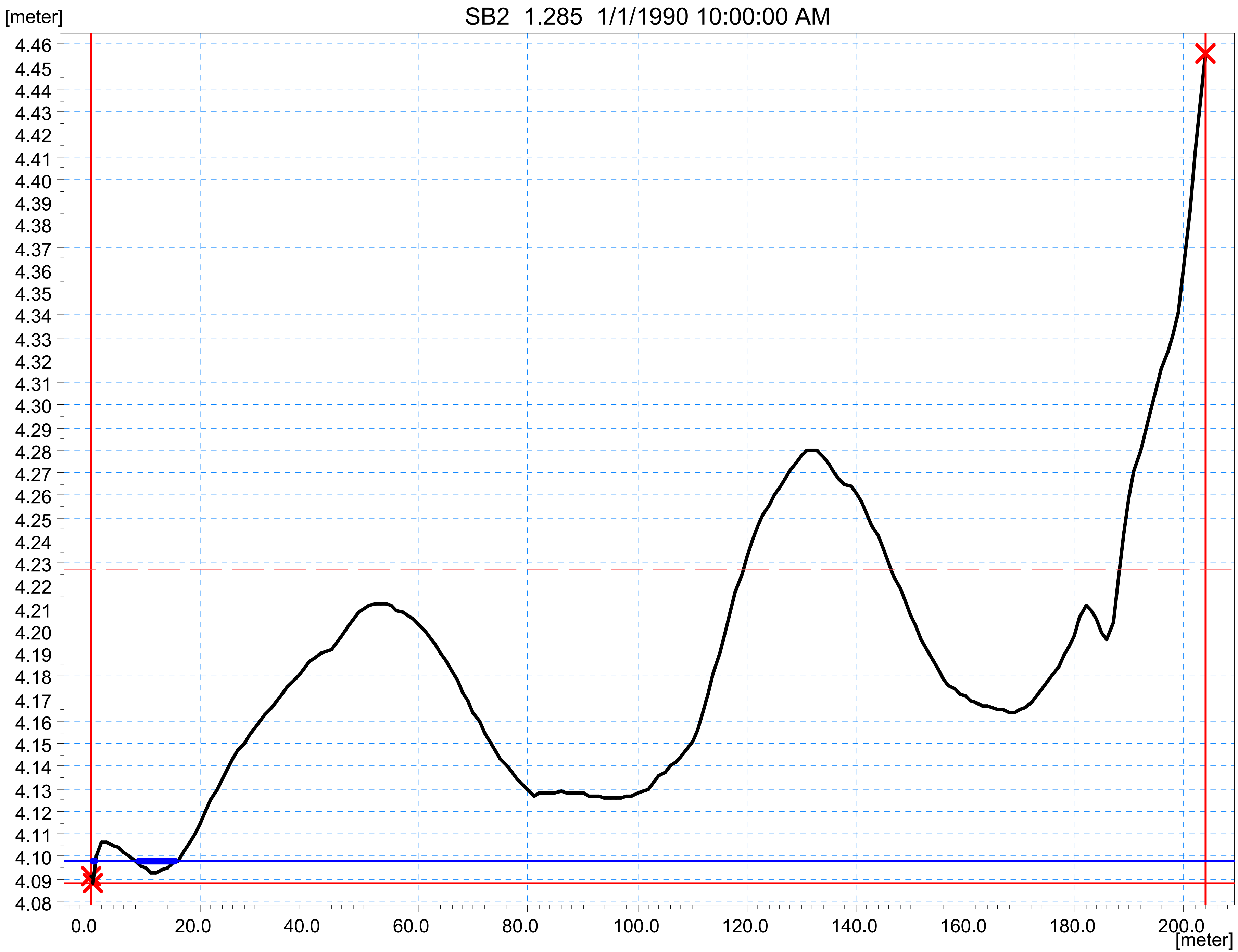
SB1 1.235 1/1/1990 10:00:00 AM
[meter]
2.60
2.59
2.58
2.57
2.56
2.55
2.54
2.53
2.52
2.51
2.50
2.49
2.48
2.47
2.46
2.45
2.44
2.43
2.42
2.41
2.40
2.39
2.38
2.37
2.36
2.35
2.34
2.33
2.32
2.31
2.30
2.29
2.28
2.27
0.0
20.0
40.0
60.0
80.0
100.0
120.0
140.0
[meter]



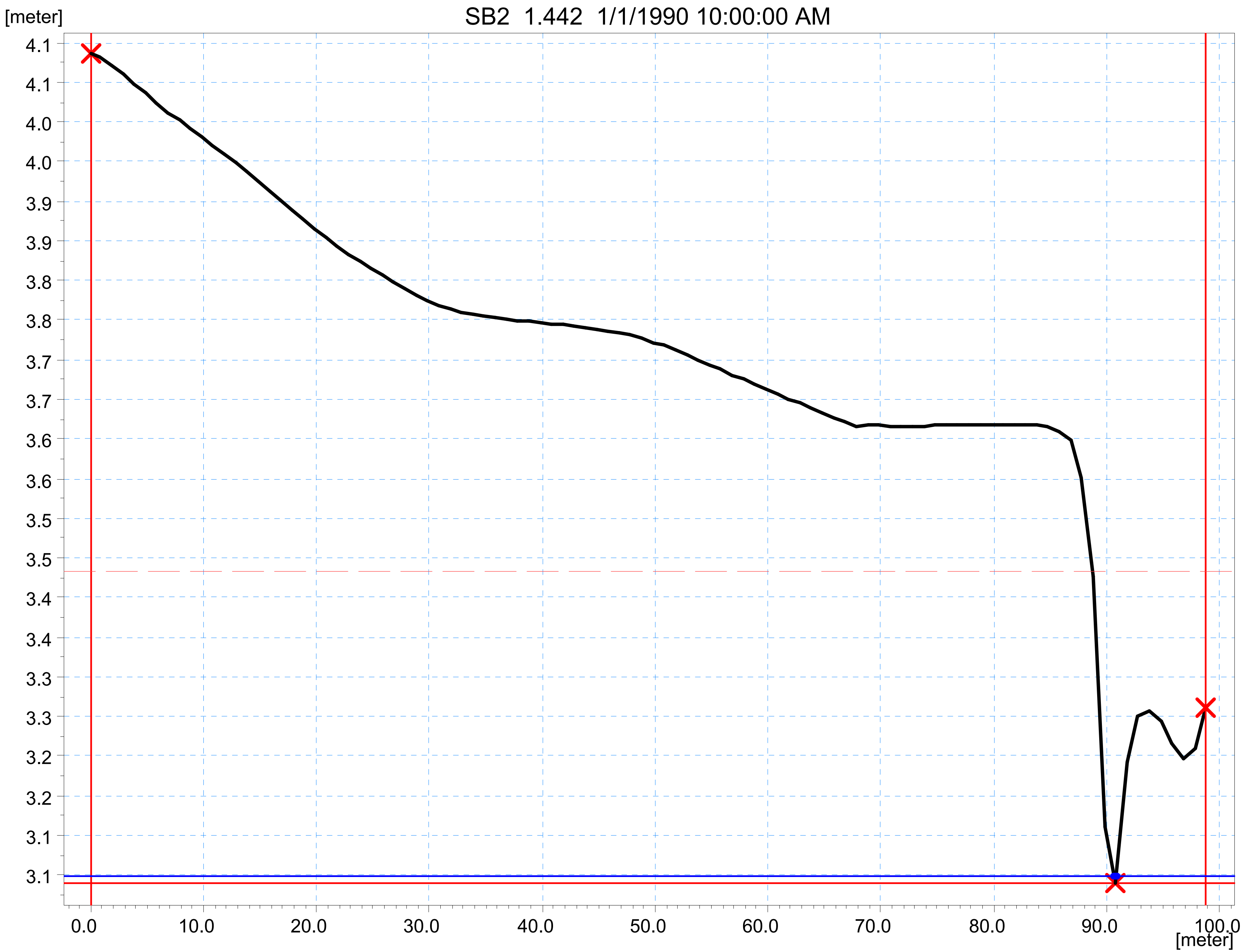
SB2 1.088 1/1/1990 10:00:00 AM
[meter]
5.8
5.7
5.7
5.6
5.6
5.5
5.5
5.4
5.4
5.3
5.3
5.2
5.2
5.1
5.1
5.0
5.0
4.9
4.9
4.8
4.8
4.7
4.7
0.0
10.0
20.0
30.0
40.0
50.0
60.0
70.0
[meter]


SB2 1.124 1/1/1990 10:00:00 AM
[meter]
5.52
5.50
5.48
5.46
5.44
5.42
5.40
5.38
5.36
5.34
5.32
5.30
5.28
5.26
5.24
5.22
5.20
5.18
5.16
5.14
5.12
5.10
5.08
5.06
5.04
5.02
5.00
4.98
4.96
4.94
4.92
4.90
4.88
4.86
0.0
10.0
20.0
30.0
40.0
50.0
60.0
70.0
80.0
90.0
100.0
110.0
120.0
[meter]

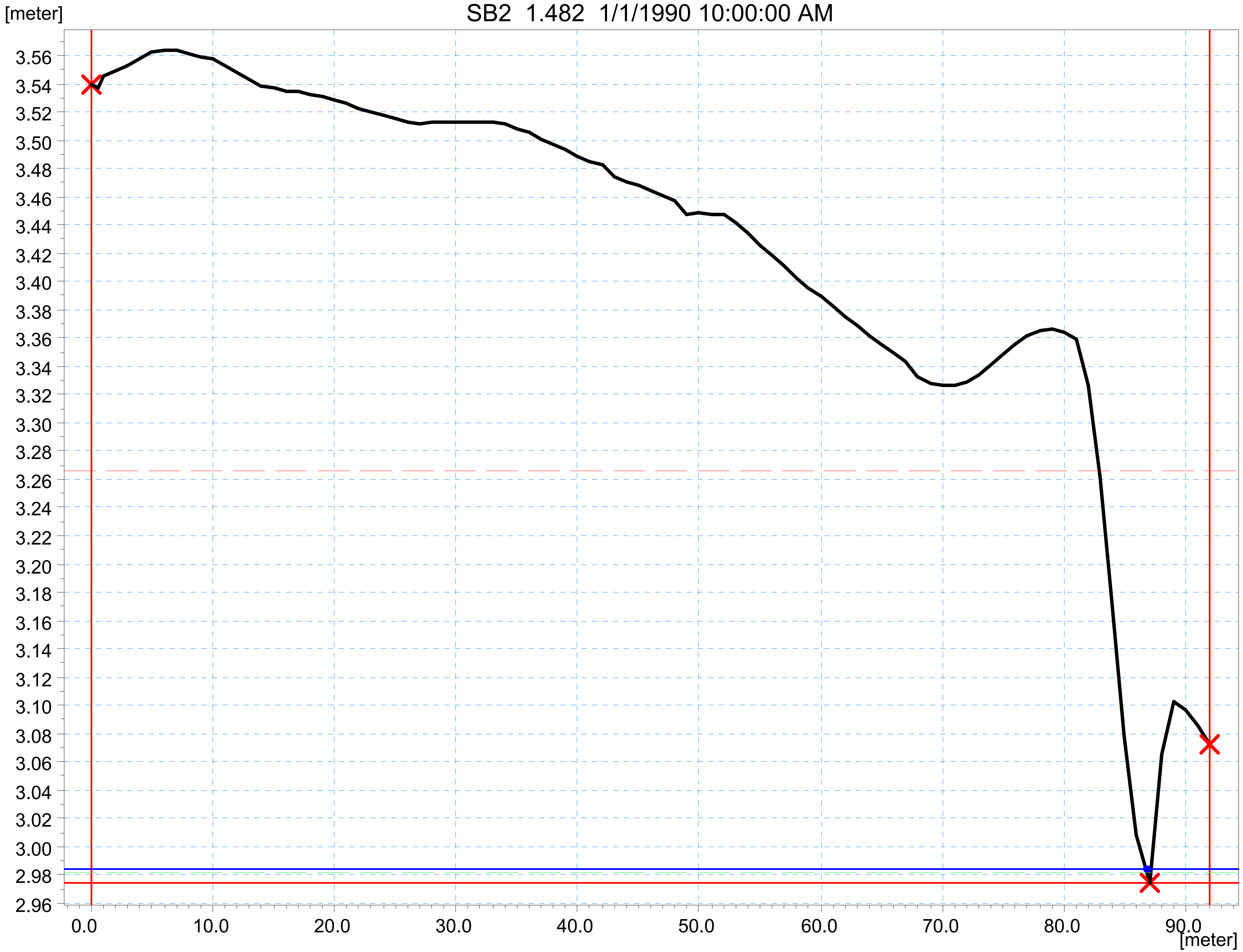

SB2 1.186 1/1/1990 10:00:00 AM
[meter]
5.4
5.3
5.3
5.2
5.2
5.1
5.1
5.0
5.0
4.9
4.9
4.8
4.8
4.7
4.7
4.6
4.6
4.5
4.5
4.4
4.4
4.3
4.3
4.2
4.2
4.1
0.0
50.0
100.0
150.0
200.0
250.0
[meter]



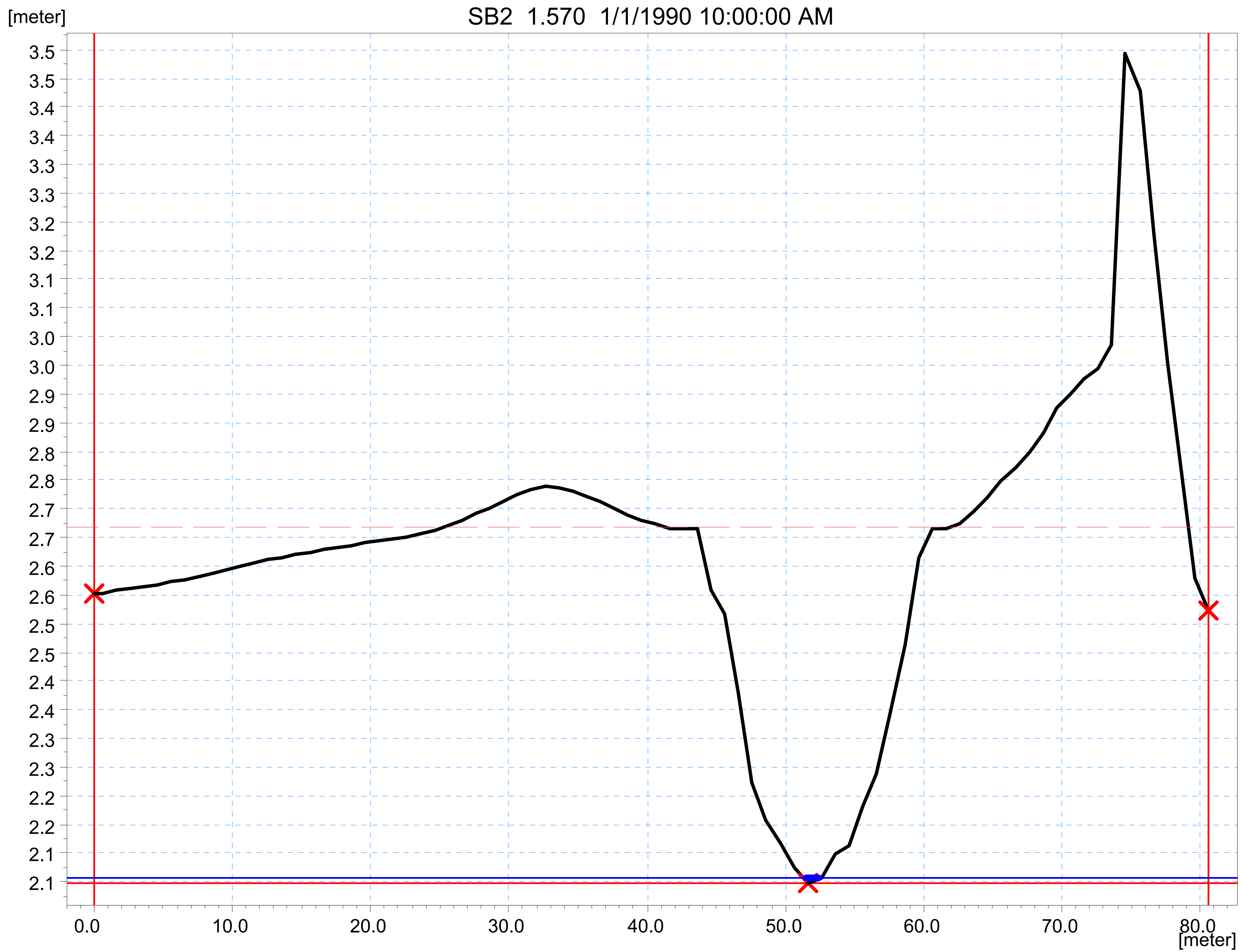
SB2 1.285 1/1/1990 10:00:00 AM
[meter]
4.46
4.45
4.44
4.43
4.42
4.41
4.40
4.39
4.38
4.37
4.36
4.35
4.34
4.33
4.32
4.31
4.30
4.29
4.28
4.27
4.26
4.25
4.24
4.23
4.22
4.21
4.20
4.19
4.18
4.17
4.16
4.15
4.14
4.13
4.12
4.11
4.10
4.09
4.08
0.0
20.0
40.0
60.0
80.0
100.0
120.0
140.0
160.0
180.0
200.0
[meter]



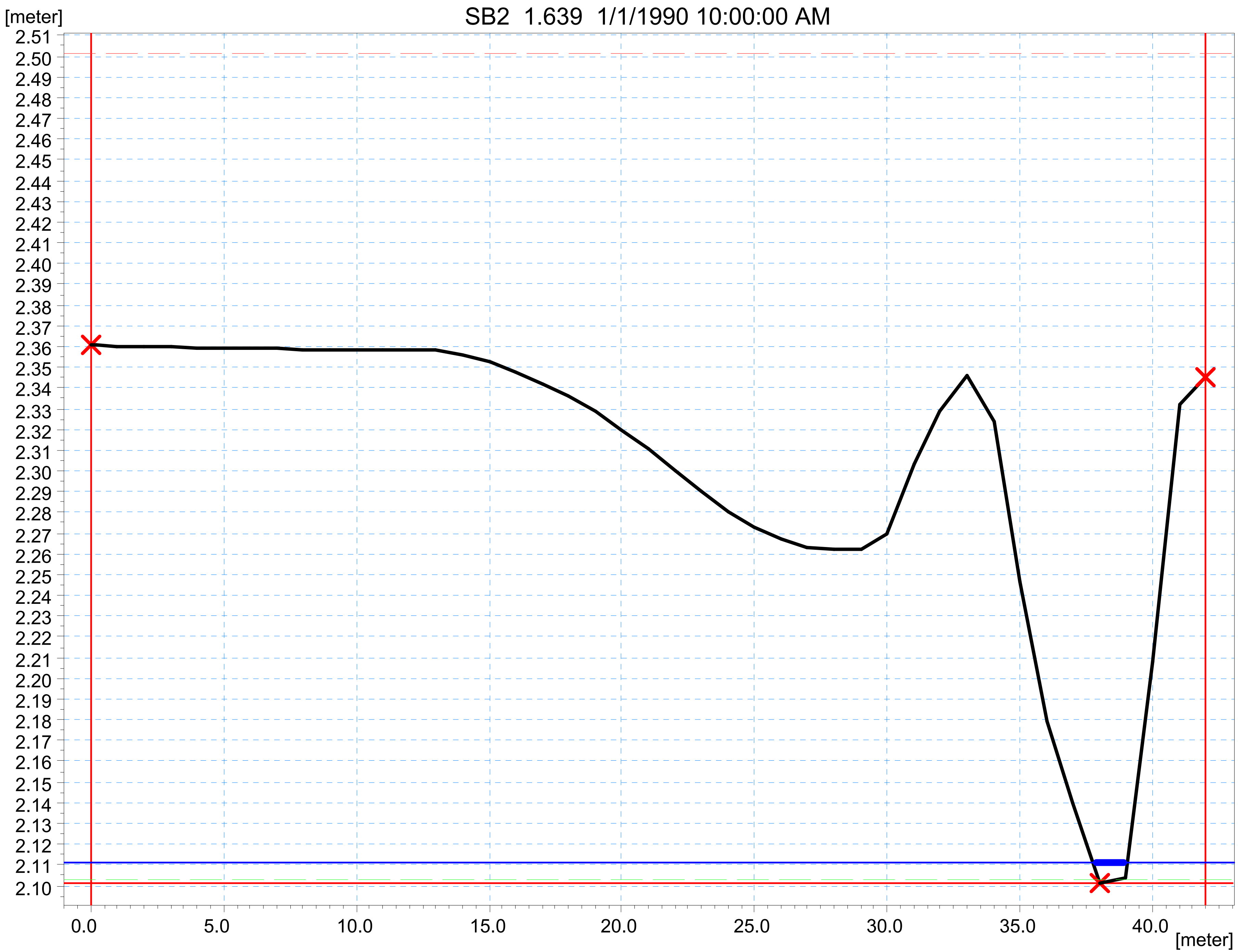
SB2 1.442 1/1/1990 10:00:00 AM
[meter]
4.1
4.1
4.0
4.0
3.9
3.9
3.8
3.8
3.7
3.7
3.6
3.6
3.5
3.5
3.4
3.4
3.3
3.3
3.2
3.2
3.1
3.1
0.0
10.0
20.0
30.0
40.0
50.0
60.0
70.0
80.0
90.0
100.0
[meter]



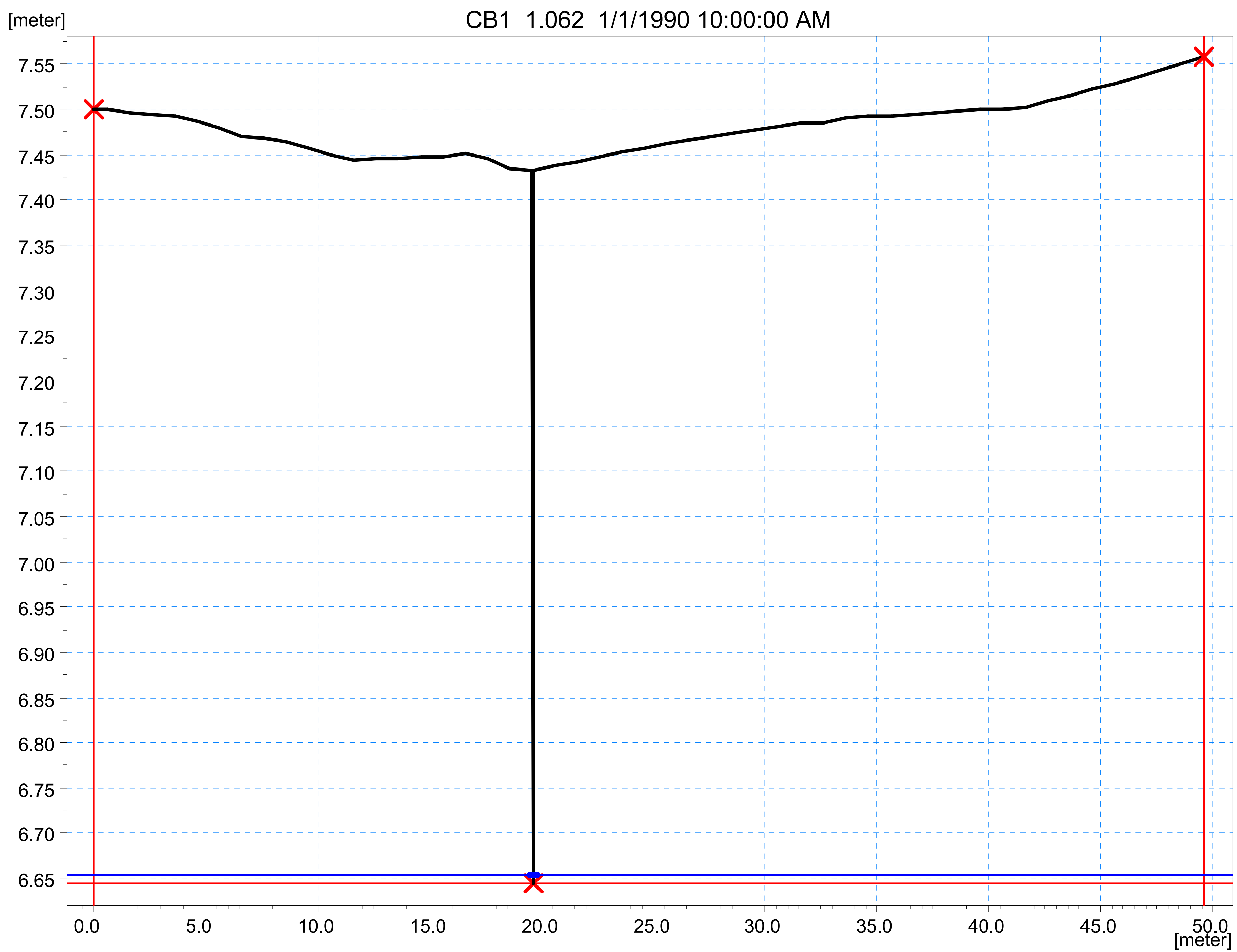
[meter]
SB2 1.482 1/1/1990 10:00:00 AM
3.56
3.54
3.52
3.50
3.48
3.46
3.44
3.42
3.40
3.38
3.36
3.34
3.32
3.30
3.28
3.26
3.24
3.22
3.20
3.18
3.16
3.14
3.12
3.10
3.08
3.06
3.04
3.02
3.00
2.98
2.96
0.0
10.0
20.0
30.0
40.0
50.0
60.0
70.0
80.0
90.0
[meter]



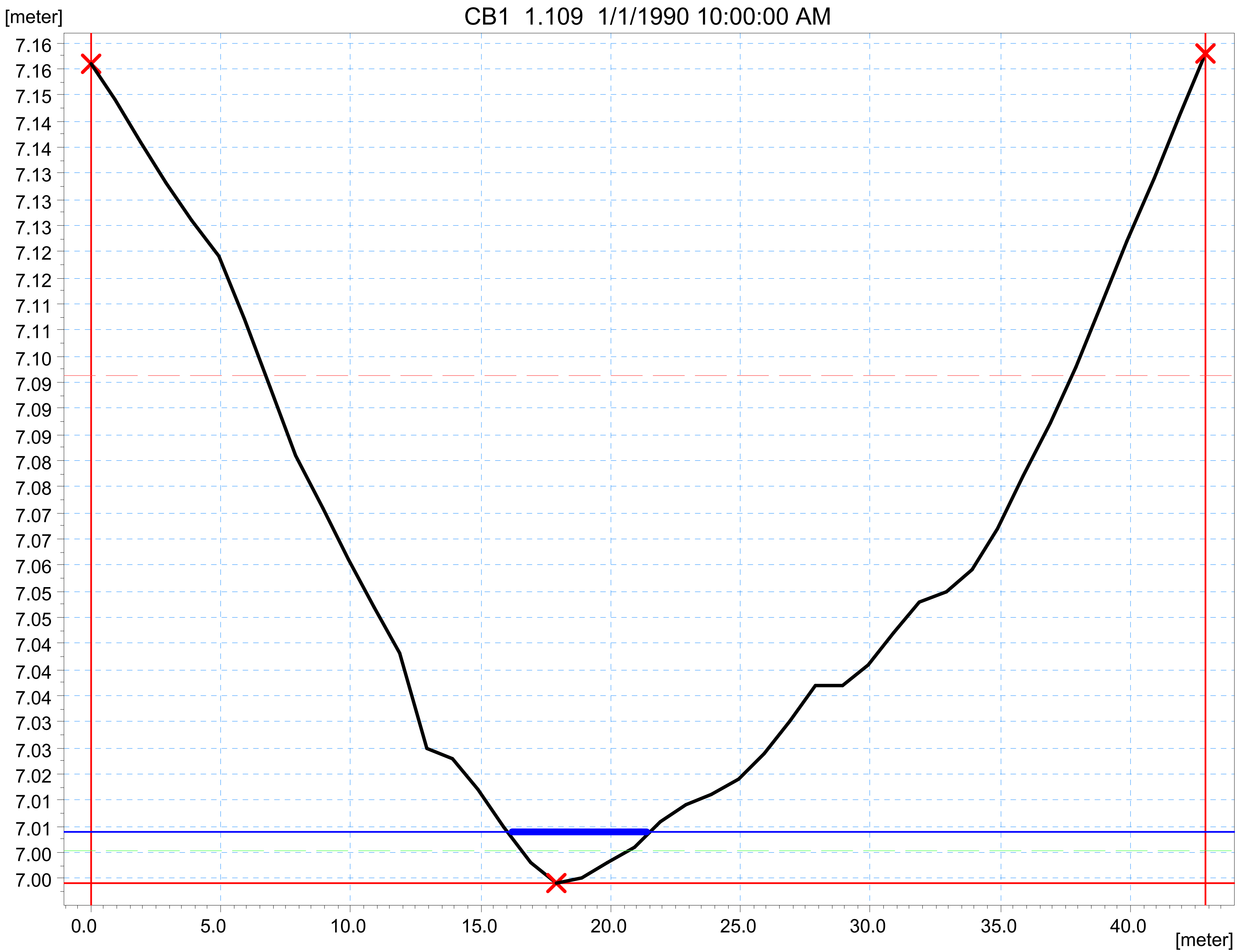
SB2 1.570 1/1/1990 10:00:00 AM
[meter]
3.5
3.5
3.4
3.4
3.3
3.3
3.2
3.2
3.1
3.1
3.0
3.0
2.9
2.9
2.8
2.8
2.7
2.7
2.6
2.6
2.5
2.5
2.4
2.4
2.3
2.3
2.2
2.2
2.1
2.1
0.0
10.0
20.0
30.0
40.0
50.0
60.0
70.0
80.0
[meter]



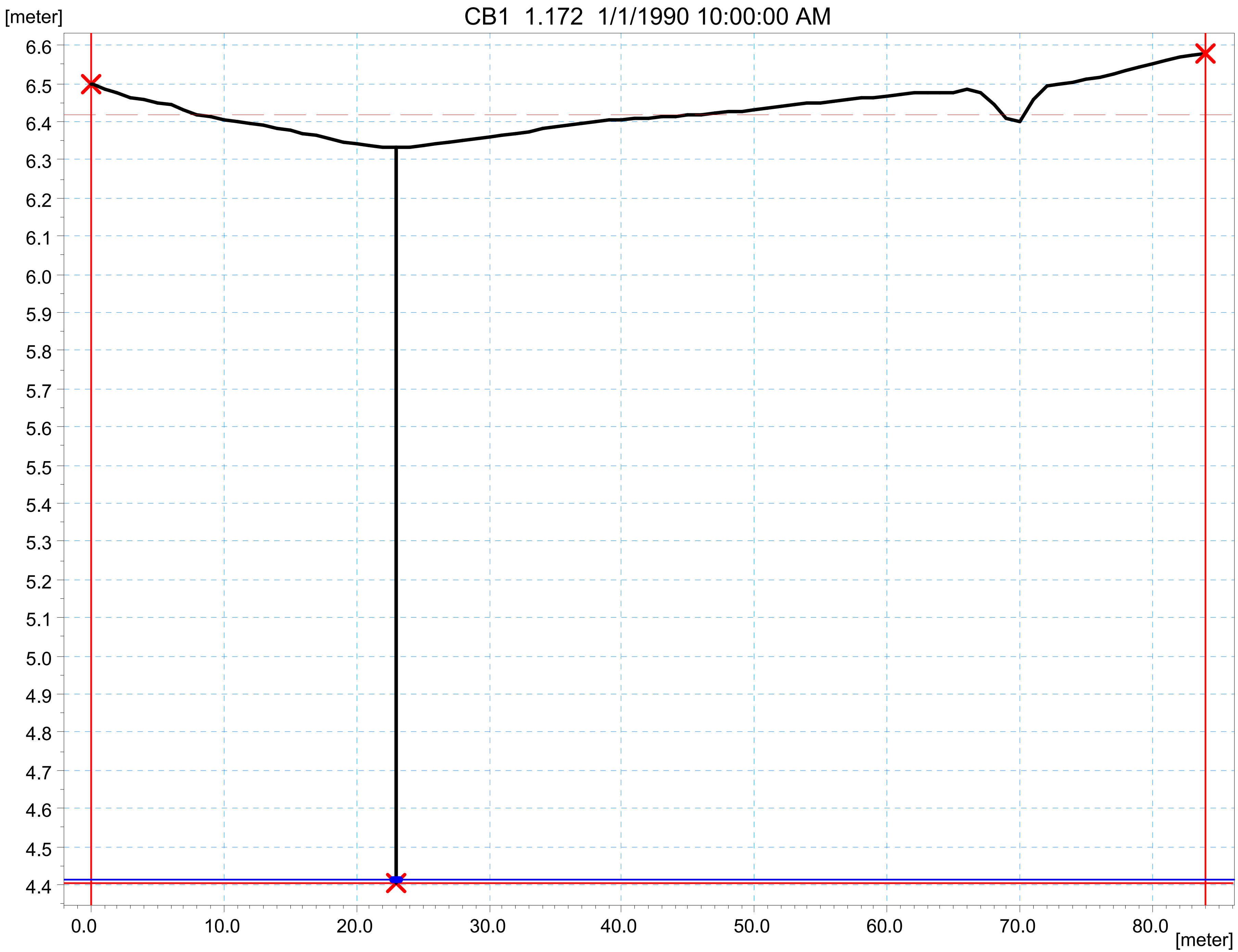
SB2 1.639 1/1/1990 10:00:00 AM
[meter]
2.51
2.50
2.49
2.48
2.47
2.46
2.45
2.44
2.43
2.42
2.41
2.40
2.39
2.38
2.37
2.36
2.35
2.34
2.33
2.32
2.31
2.30
2.29
2.28
2.27
2.26
2.25
2.24
2.23
2.22
2.21
2.20
2.19
2.18
2.17
2.16
2.15
2.14
2.13
2.12
2.11
2.10
0.0
5.0
10.0
15.0
20.0
25.0
30.0
35.0
40.0
[meter]



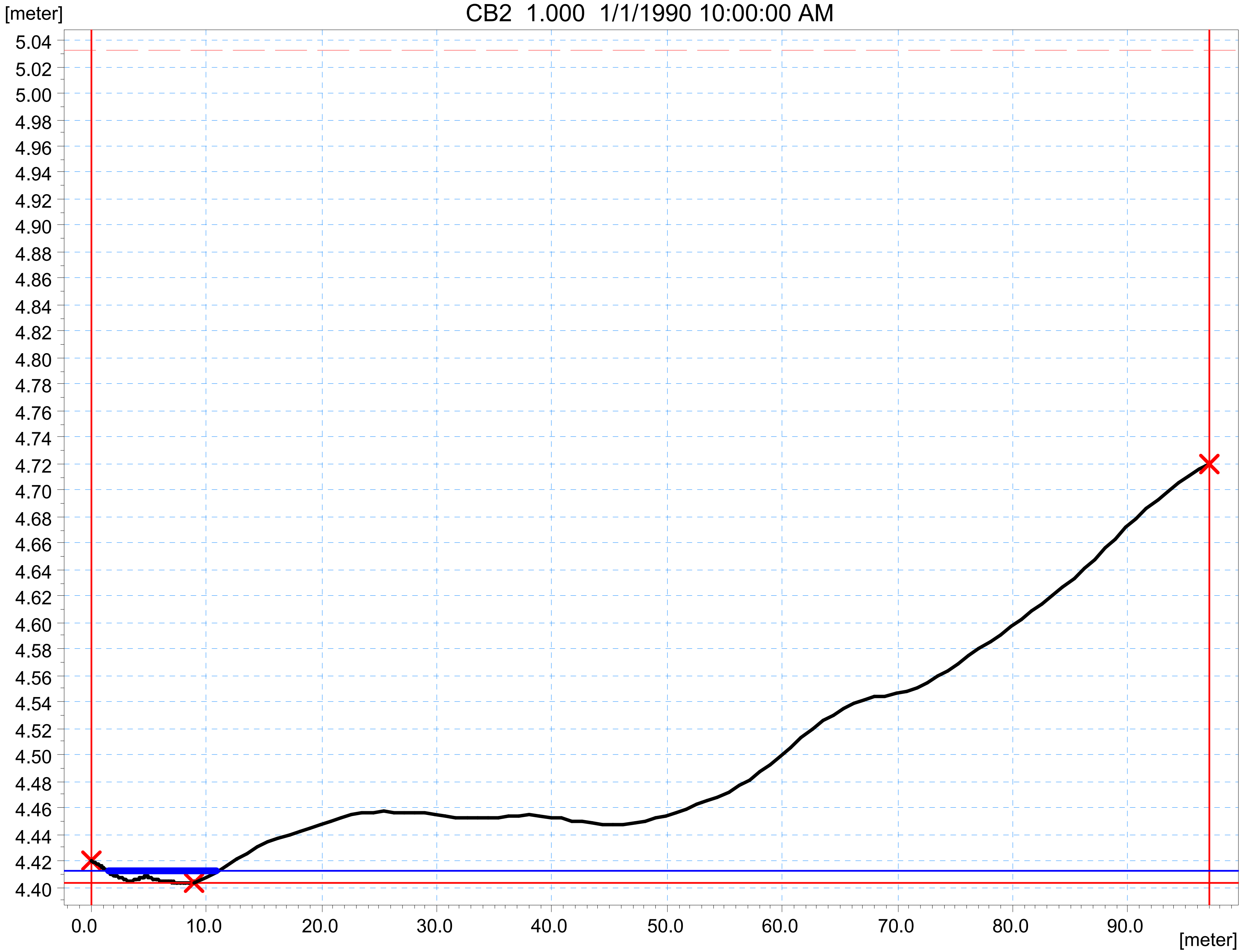
CB1 1.062 1/1/1990 10:00:00 AM
[meter]
7.55
7.50
7.45
7.40
7.35
7.30
7.25
7.20
7.15
7.10
7.05
7.00
6.95
6.90
6.85
6.80
6.75
6.70
6.65
0.0
5.0
10.0
15.0
20.0
25.0
30.0
35.0
40.0
45.0
50.0
[meter]



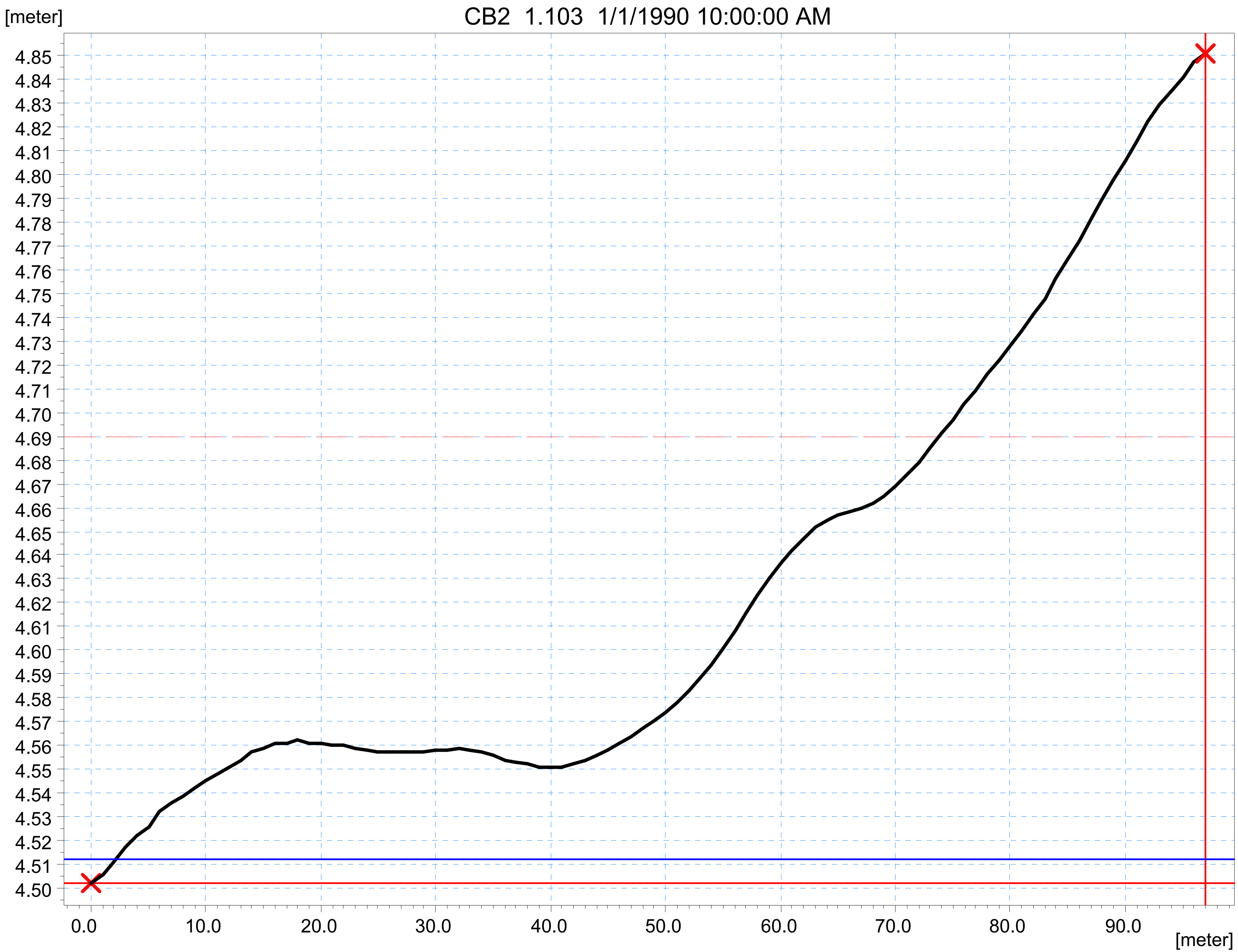
CB1 1.109 1/1/1990 10:00:00 AM
[meter]
7.16
7.16
7.15
7.14
7.14
7.13
7.13
7.13
7.12
7.12
7.11
7.11
7.10
7.09
7.09
7.09
7.08
7.08
7.07
7.07
7.06
7.05
7.05
7.04
7.04
7.04
7.03
7.03
7.02
7.01
7.01
7.00
7.00
0.0
5.0
10.0
15.0
20.0
25.0
30.0
35.0
40.0
[meter]



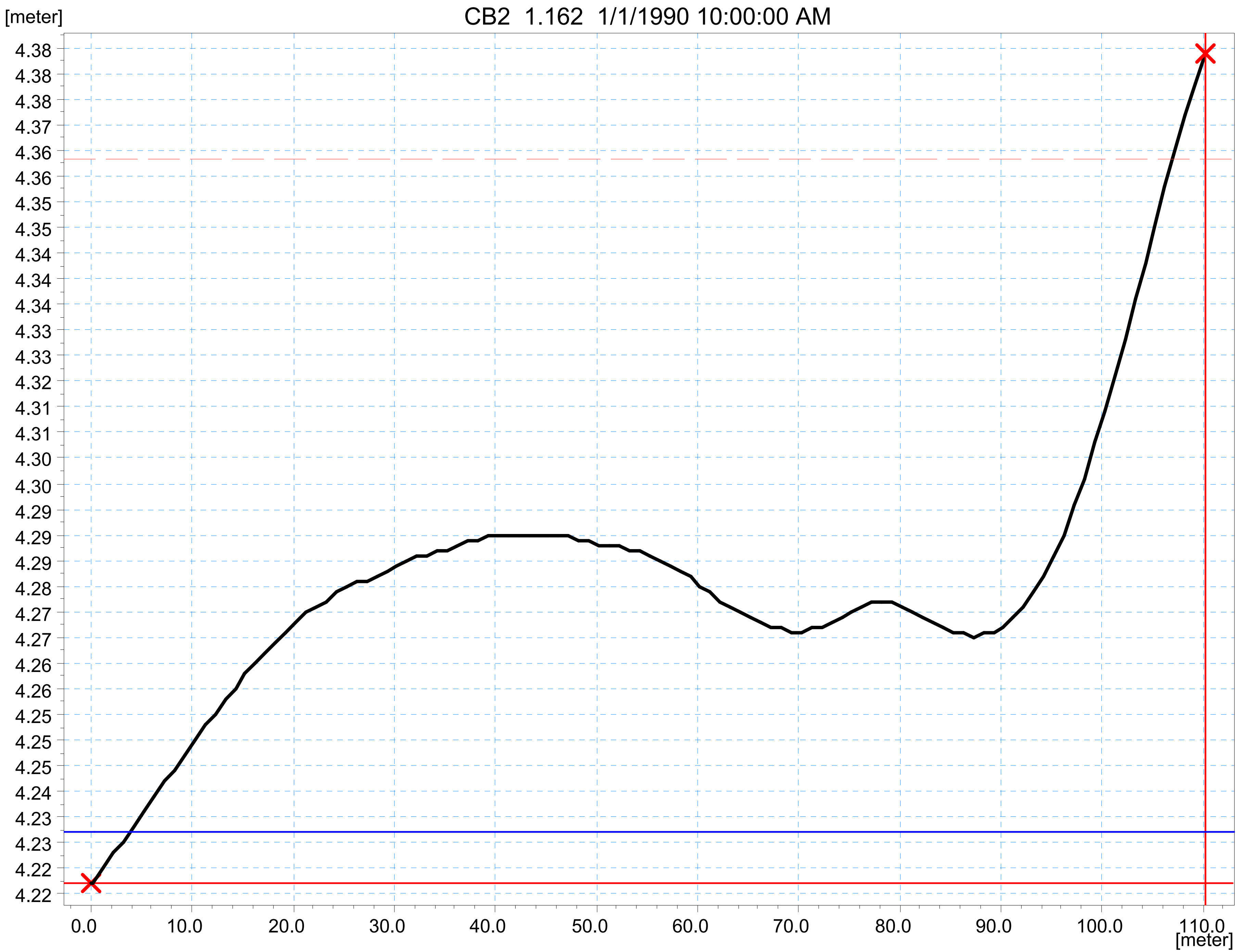
CB1 1.172 1/1/1990 10:00:00 AM
[meter]
6.6
6.5
6.4
6.3
6.2
6.1
6.0
5.9
5.8
5.7
5.6
5.5
5.4
5.3
5.2
5.1
5.0
4.9
4.8
4.7
4.6
4.5
4.4
0.0
10.0
20.0
30.0
40.0
50.0
60.0
70.0
80.0
[meter]

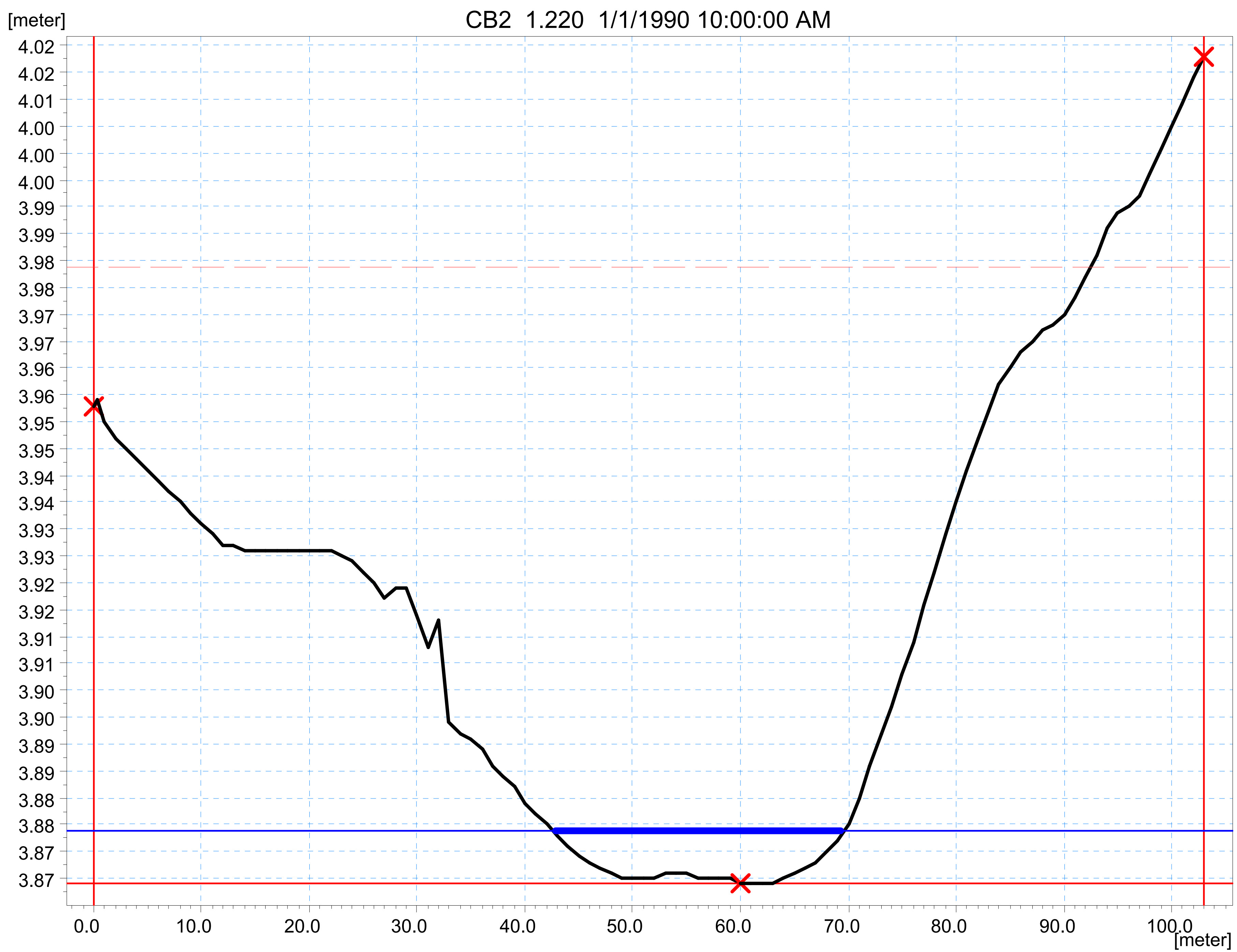


CB2 1.000 1/1/1990 10:00:00 AM
[meter]
5.04
5.02
5.00
4.98
4.96
4.94
4.92
4.90
4.88
4.86
4.84
4.82
4.80
4.78
4.76
4.74
4.72
4.70
4.68
4.66
4.64
4.62
4.60
4.58
4.56
4.54
4.52
4.50
4.48
4.46
4.44
4.42
4.40
0.0
10.0
20.0
30.0
40.0
50.0
60.0
70.0
80.0
90.0
[meter]

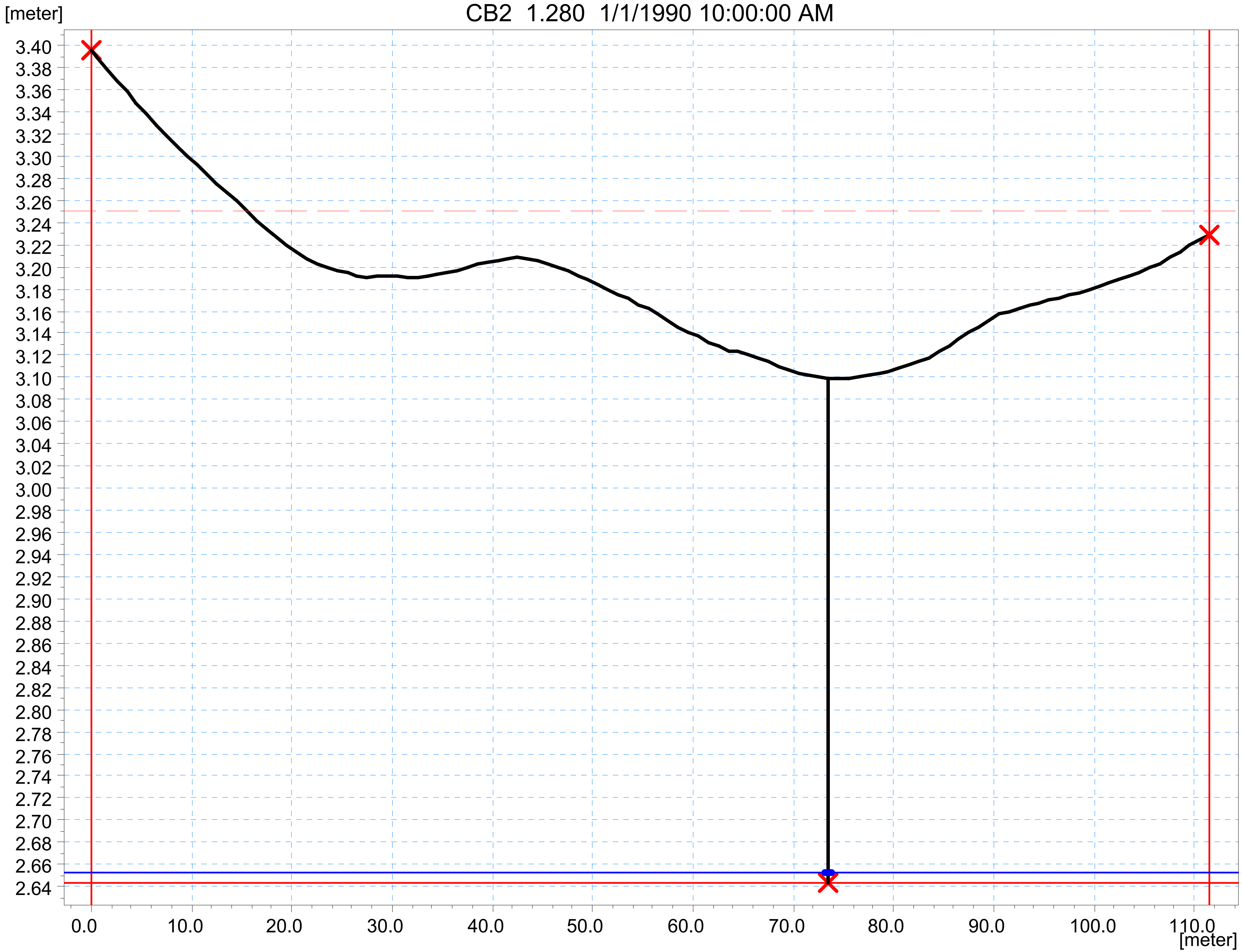


[meter]
CB2 1.103 1/1/1990 10:00:00 AM
4.85
4.84
4.83
4.82
4.81
4.80
4.79
4.78
4.77
4.76
4.75
4.74
4.73
4.72
4.71
4.70
4.69
4.68
4.67
4.66
4.65
4.64
4.63
4.62
4.61
4.60
4.59
4.58
4.57
4.56
4.55
4.54
4.53
4.52
4.51
4.50
0.0
10.0
20.0
30.0
40.0
50.0
60.0
70.0
80.0
90.0
[meter]

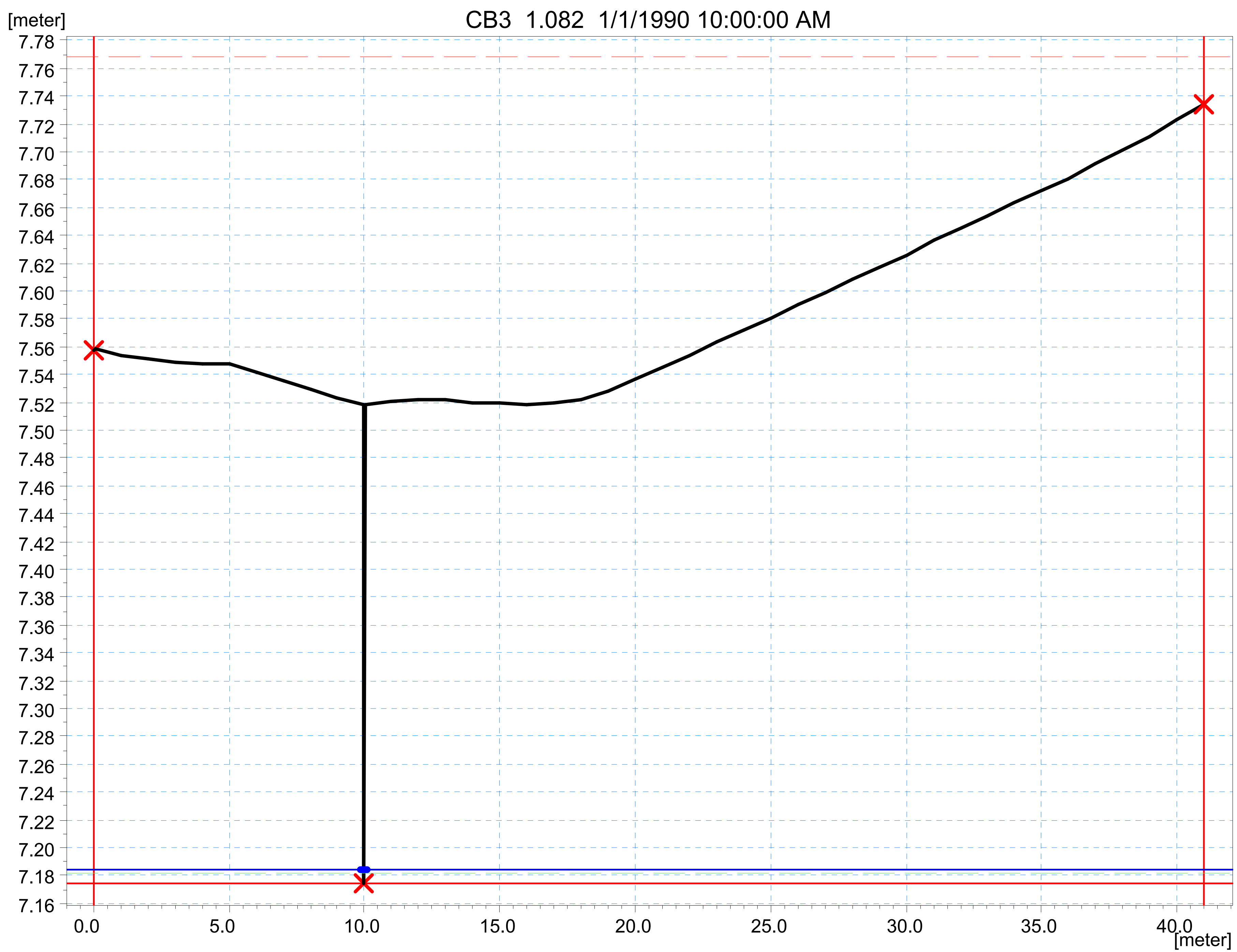

CB2 1.162 1/1/1990 10:00:00 AM
[meter]
4.38
4.38
4.38
4.37
4.36
4.36
4.35
4.35
4.34
4.34
4.34
4.33
4.33
4.32
4.31
4.31
4.30
4.30
4.29
4.29
4.29
4.28
4.27
4.27
4.26
4.26
4.25
4.25
4.25
4.24
4.23
4.23
4.22
4.22
0.0
10.0
20.0
30.0
40.0
50.0
60.0
70.0
80.0
90.0
100.0
110.0
[meter]



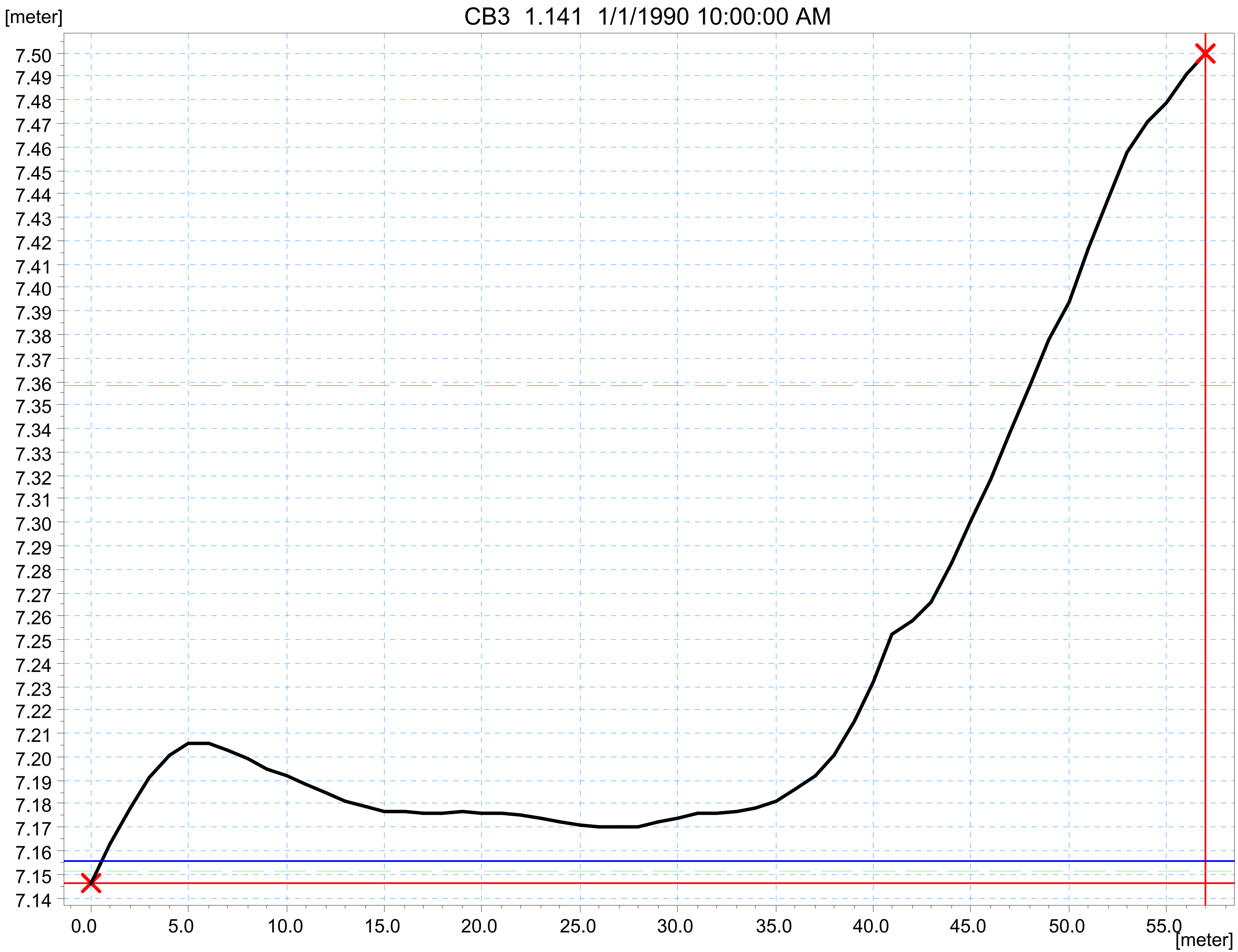
CB2 1.220 1/1/1990 10:00:00 AM
[meter]
4.02
4.02
4.01
4.00
4.00
4.00
3.99
3.99
3.98
3.98
3.97
3.97
3.96
3.96
3.95
3.95
3.94
3.94
3.93
3.93
3.92
3.92
3.91
3.91
3.90
3.90
3.89
3.89
3.88
3.88
3.87
3.87
0.0
10.0
20.0
30.0
40.0
50.0
60.0
70.0
80.0
90.0
100.0
[meter]



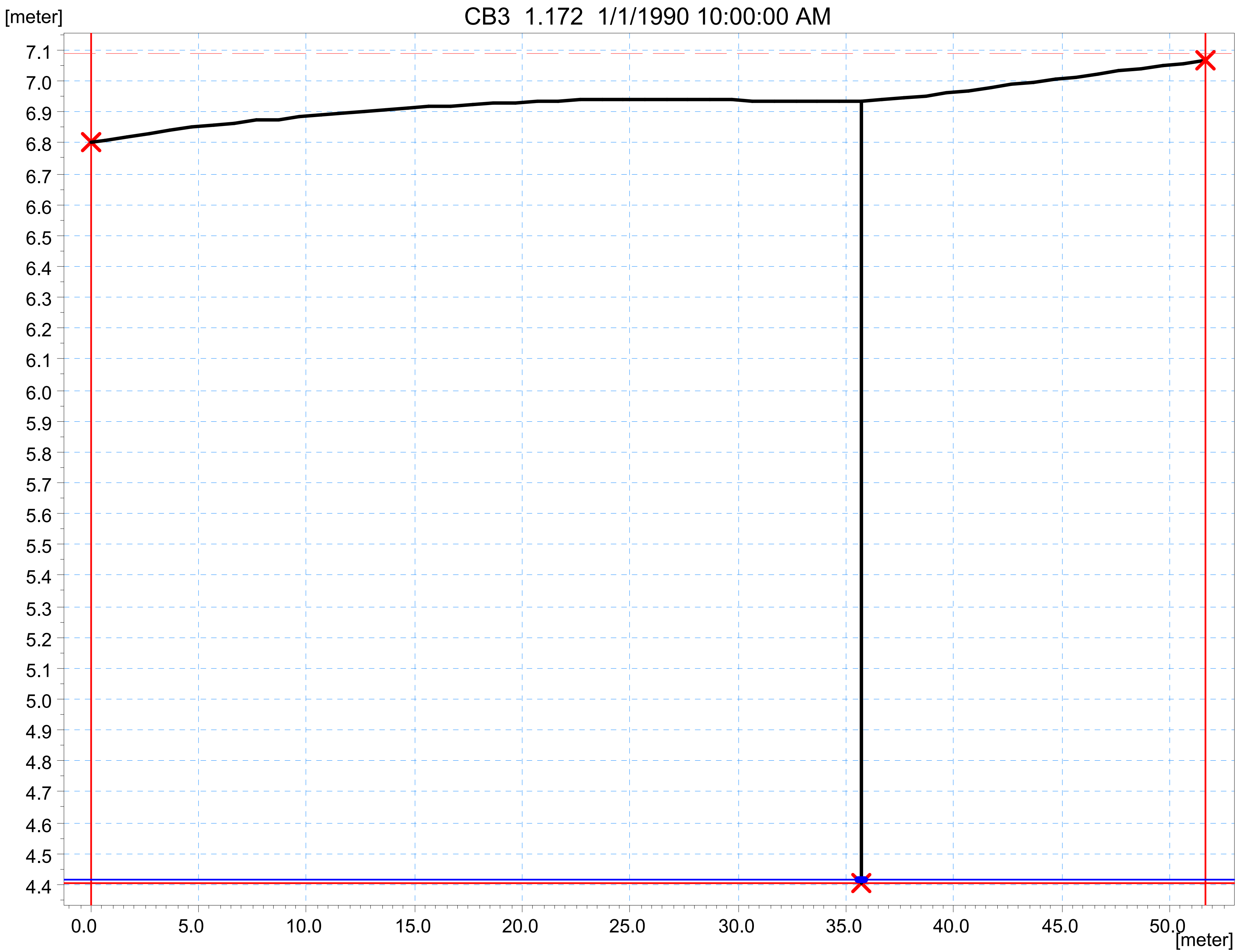
[meter]
CB2 1.280 1/1/1990 10:00:00 AM
3.40
3.38
3.36
3.34
3.32
3.30
3.28
3.26
3.24
3.22
3.20
3.18
3.16
3.14
3.12
3.10
3.08
3.06
3.04
3.02
3.00
2.98
2.96
2.94
2.92
2.90
2.88
2.86
2.84
2.82
2.80
2.78
2.76
2.74
2.72
2.70
2.68
2.66
2.64
0.0
10.0
20.0
30.0
40.0
50.0
60.0
70.0
80.0
90.0
100.0
110.0
[meter]



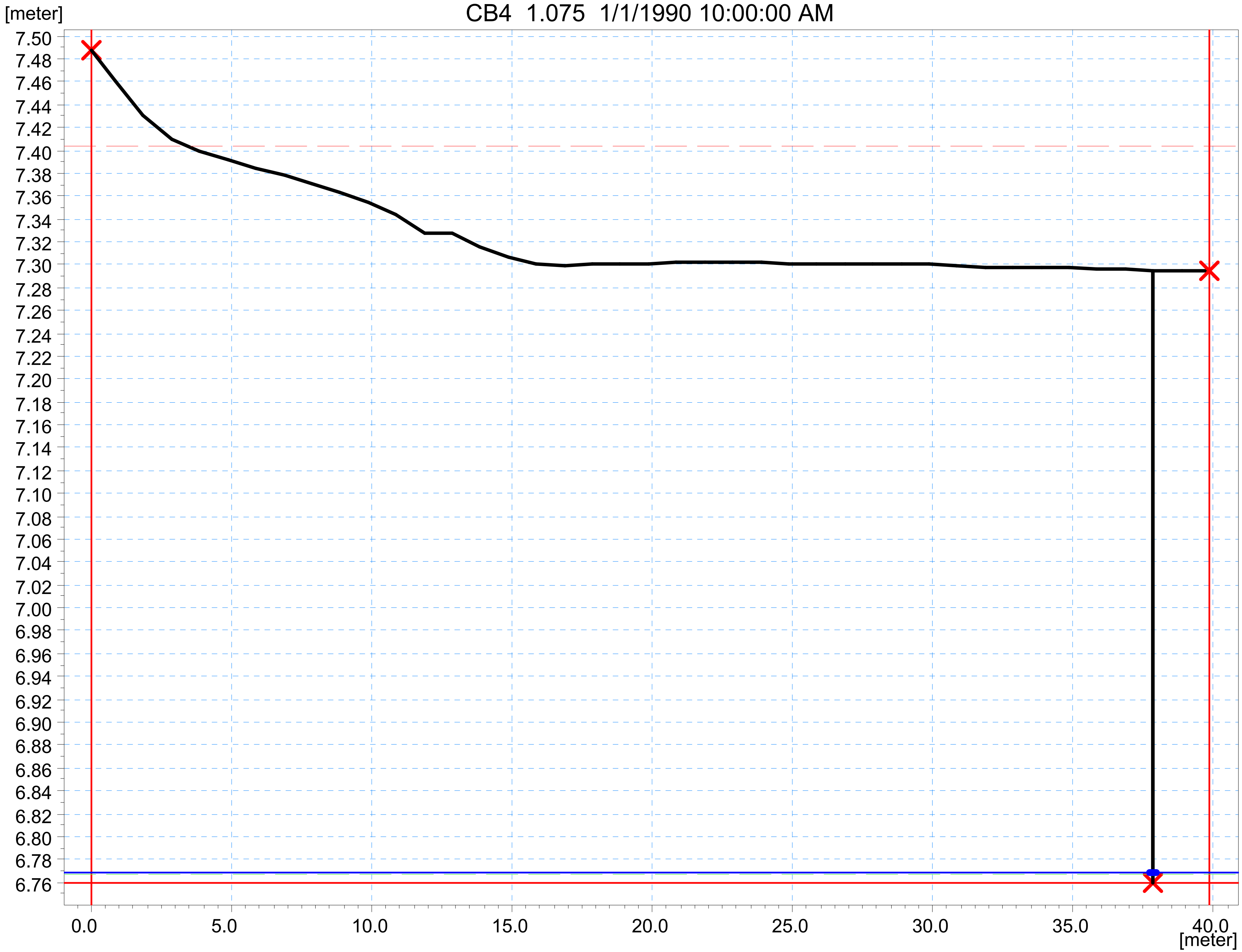
CB3 1.082 1/1/1990 10:00:00 AM
[meter]
7.78
7.76
7.74
7.72
7.70
7.68
7.66
7.64
7.62
7.60
7.58
7.56
7.54
7.52
7.50
7.48
7.46
7.44
7.42
7.40
7.38
7.36
7.34
7.32
7.30
7.28
7.26
7.24
7.22
7.20
7.18
7.16
0.0
5.0
10.0
15.0
20.0
25.0
30.0
35.0
40.0
[meter]



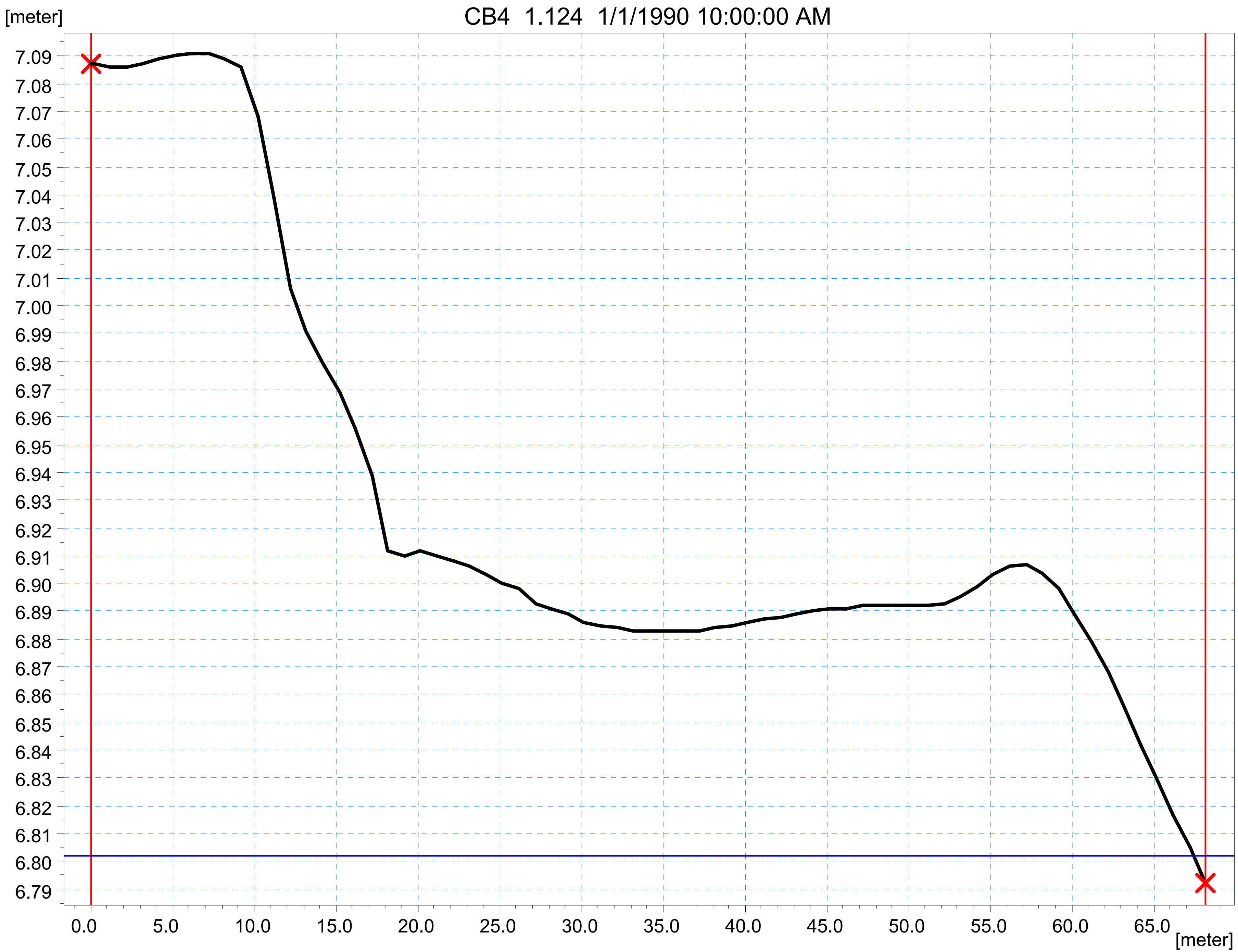
CB3 1.141 1/1/1990 10:00:00 AM
[meter]
7.50
7.49
7.48
7.47
7.46
7.45
7.44
7.43
7.42
7.41
7.40
7.39
7.38
7.37
7.36
7.35
7.34
7.33
7.32
7.31
7.30
7.29
7.28
7.27
7.26
7.25
7.24
7.23
7.22
7.21
7.20
7.19
7.18
7.17
7.16
7.15
7.14
0.0
5.0
10.0
15.0
20.0
25.0
30.0
35.0
40.0
45.0
50.0
55.0
[meter]



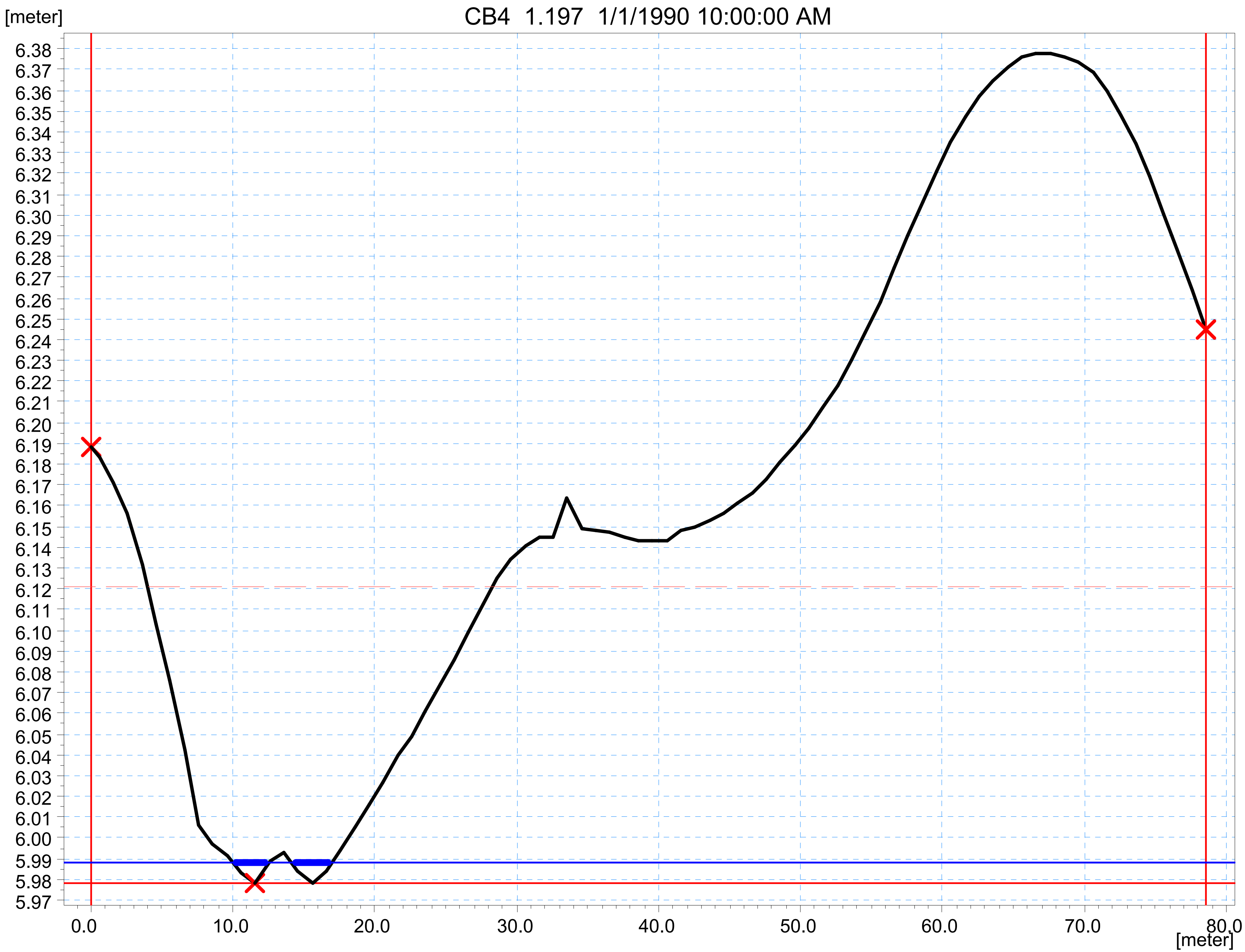
CB3 1.172 1/1/1990 10:00:00 AM
[meter]
7.1
7.0
6.9
6.8
6.7
6.6
6.5
6.4
6.3
6.2
6.1
6.0
5.9
5.8
5.7
5.6
5.5
5.4
5.3
5.2
5.1
5.0
4.9
4.8
4.7
4.6
4.5
4.4
0.0
5.0
10.0
15.0
20.0
25.0
30.0
35.0
40.0
45.0
50.0
[meter]

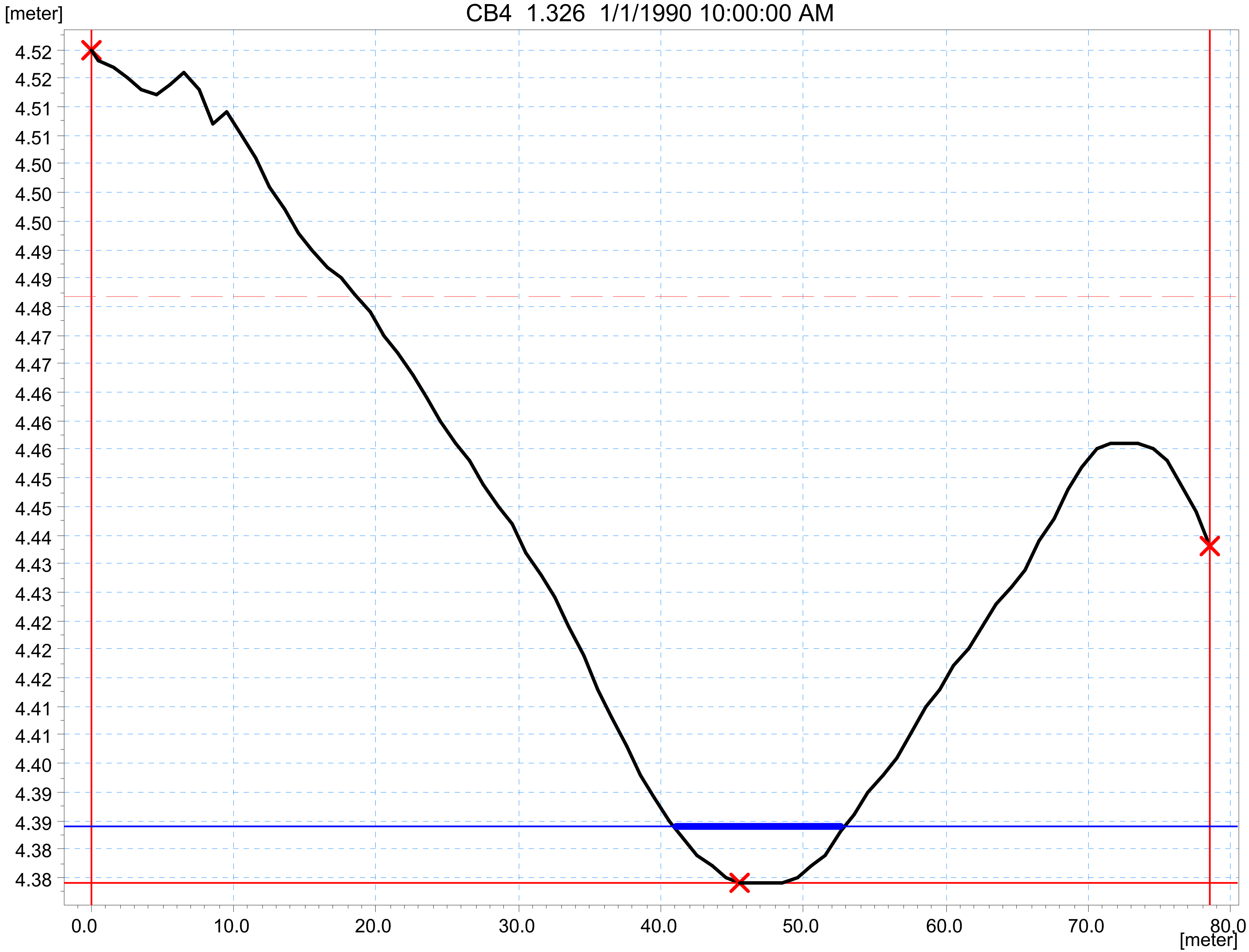


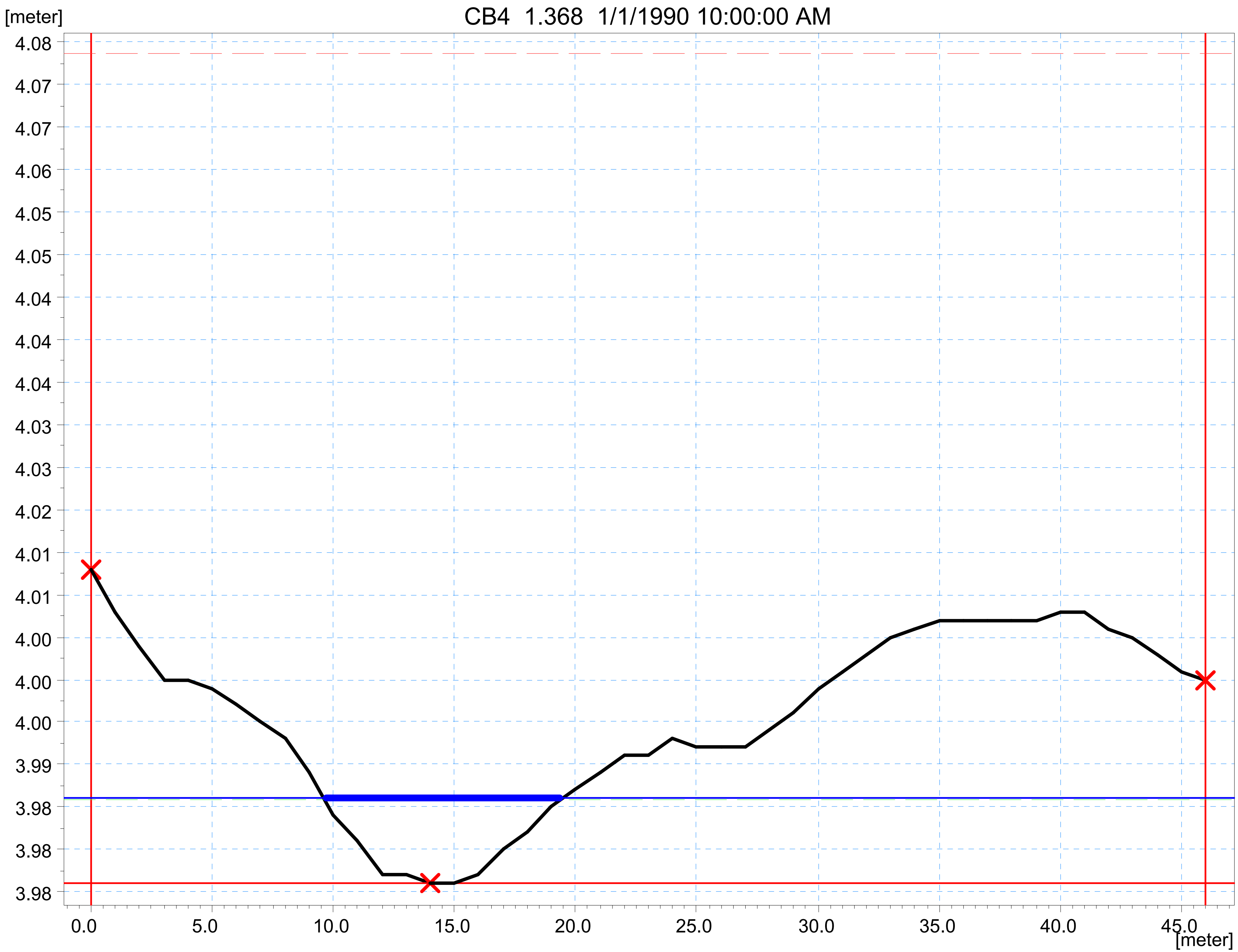
CB4 1.075 1/1/1990 10:00:00 AM
[meter]
7.50
7.48
7.46
7.44
7.42
7.40
7.38
7.36
7.34
7.32
7.30
7.28
7.26
7.24
7.22
7.20
7.18
7.16
7.14
7.12
7.10
7.08
7.06
7.04
7.02
7.00
6.98
6.96
6.94
6.92
6.90
6.88
6.86
6.84
6.82
6.80
6.78
6.76
0.0
5.0
10.0
15.0
20.0
25.0
30.0
35.0
40.0
[meter]



CB4 1.124 1/1/1990 10:00:00 AM
[meter]
7.09
7.08
7.07
7.06
7.05
7.04
7.03
7.02
7.01
7.00
6.99
6.98
6.97
6.96
6.95
6.94
6.93
6.92
6.91
6.90
6.89
6.88
6.87
6.86
6.85
6.84
6.83
6.82
6.81
6.80
6.79
0.0
5.0
10.0
15.0
20.0
25.0
30.0
35.0
40.0
45.0
50.0
55.0
60.0
65.0
[meter]

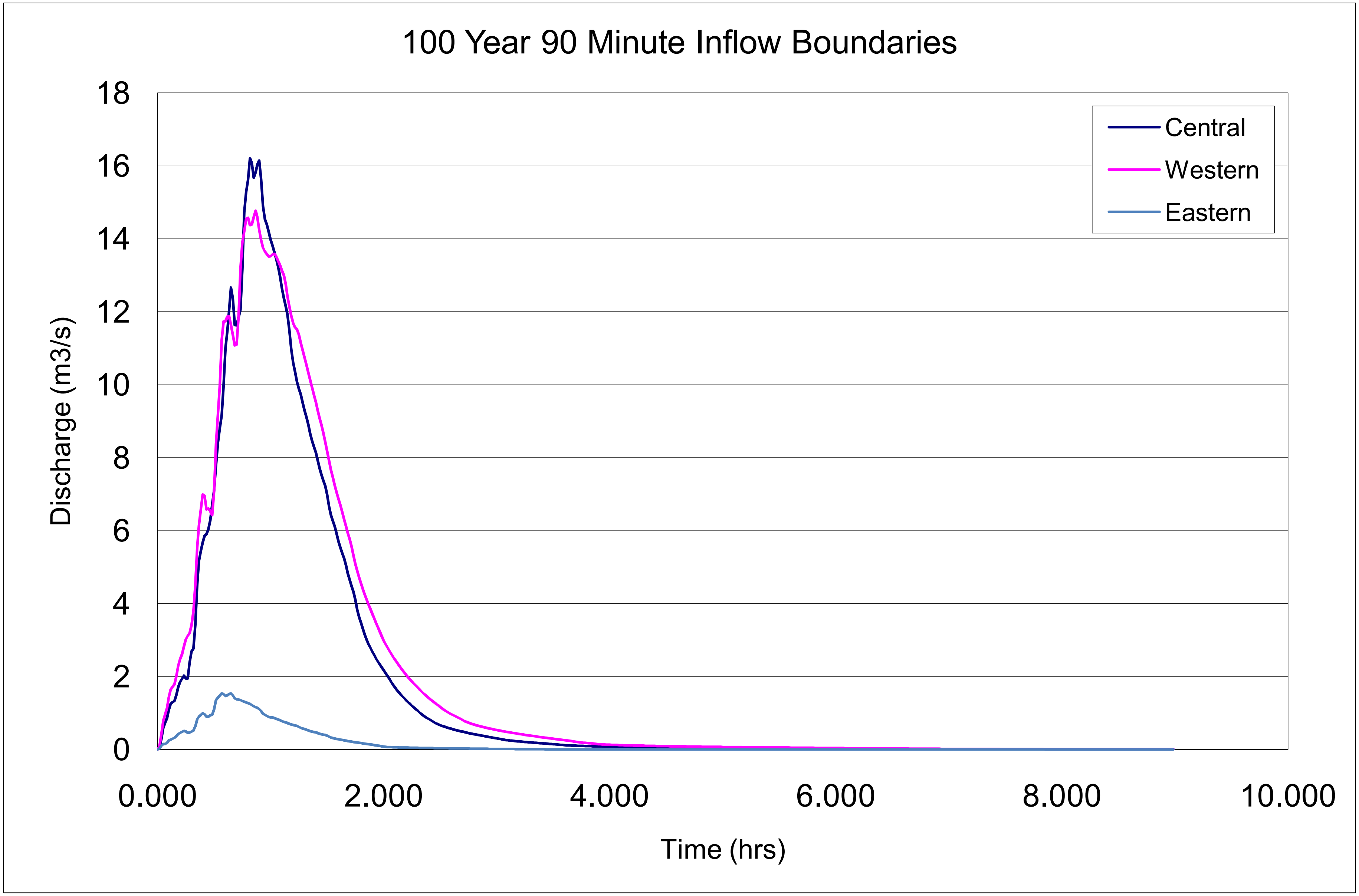

[meter]
CB4 1.197 1/1/1990 10:00:00 AM
6.38
6.37
6.36
6.35
6.34
6.33
6.32
6.31
6.30
6.29
6.28
6.27
6.26
6.25
6.24
6.23
6.22
6.21
6.20
6.19
6.18
6.17
6.16
6.15
6.14
6.13
6.12
6.11
6.10
6.09
6.08
6.07
6.06
6.05
6.04
6.03
6.02
6.01
6.00
5.99
5.98
5.97
0.0
10.0
20.0
30.0
40.0
50.0
60.0
70.0
80.0
[meter]


CB4 1.326 1/1/1990 10:00:00 AM
[meter]
4.52
4.52
4.51
4.51
4.50
4.50
4.50
4.49
4.49
4.48
4.47
4.47
4.46
4.46
4.46
4.45
4.45
4.44
4.43
4.43
4.42
4.42
4.42
4.41
4.41
4.40
4.39
4.39
4.38
4.38
0.0
10.0
20.0
30.0
40.0
50.0
60.0
70.0
80.0
[meter]

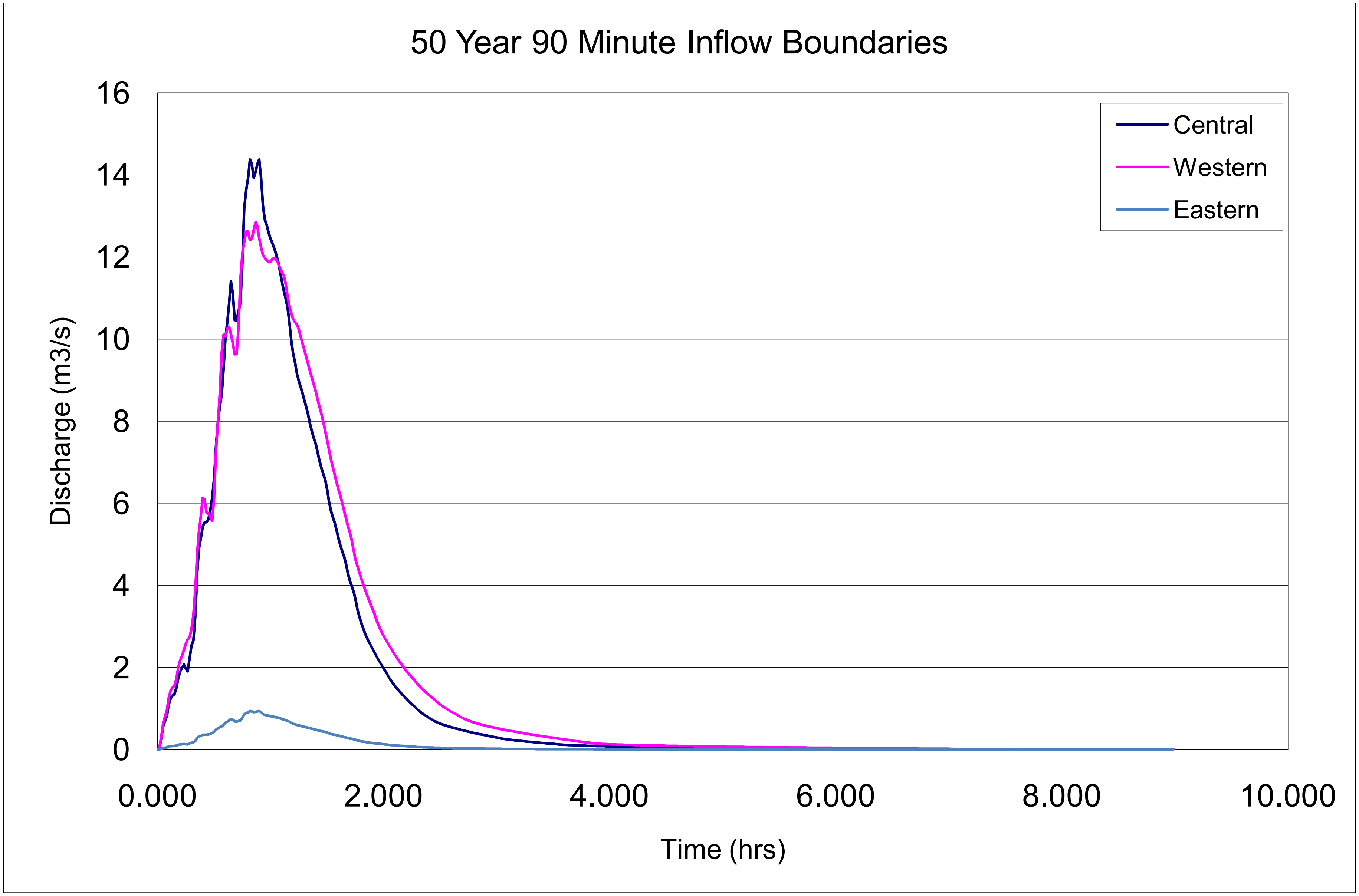

CB4 1.368 1/1/1990 10:00:00 AM
[meter]
4.08
4.07
4.07
4.06
4.05
4.05
4.04
4.04
4.04
4.03
4.03
4.02
4.01
4.01
4.00
4.00
4.00
3.99
3.98
3.98
3.98
0.0
5.0
10.0
15.0
20.0
25.0
30.0
35.0
40.0
45.0
[meter]

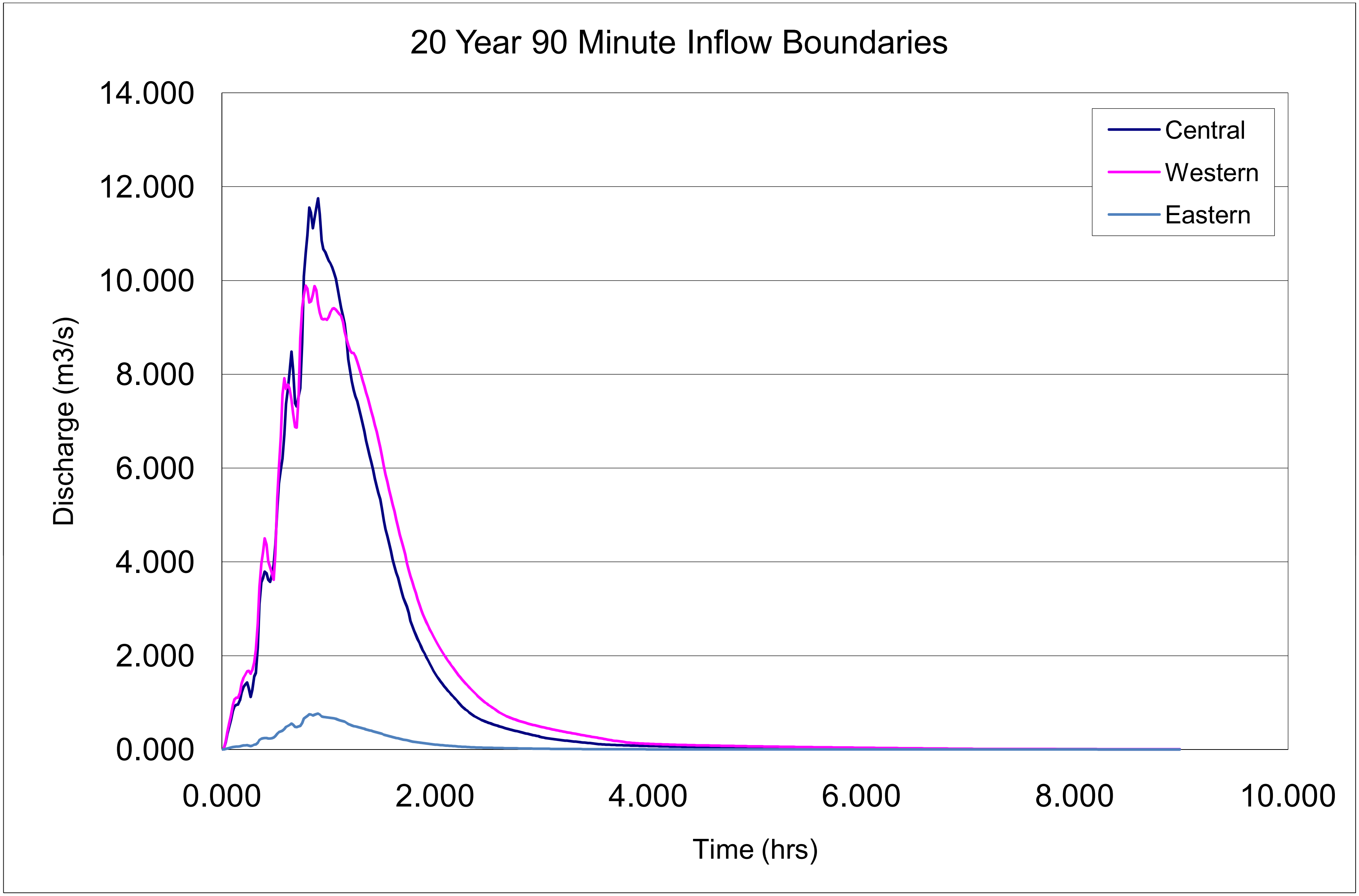
Appendix D

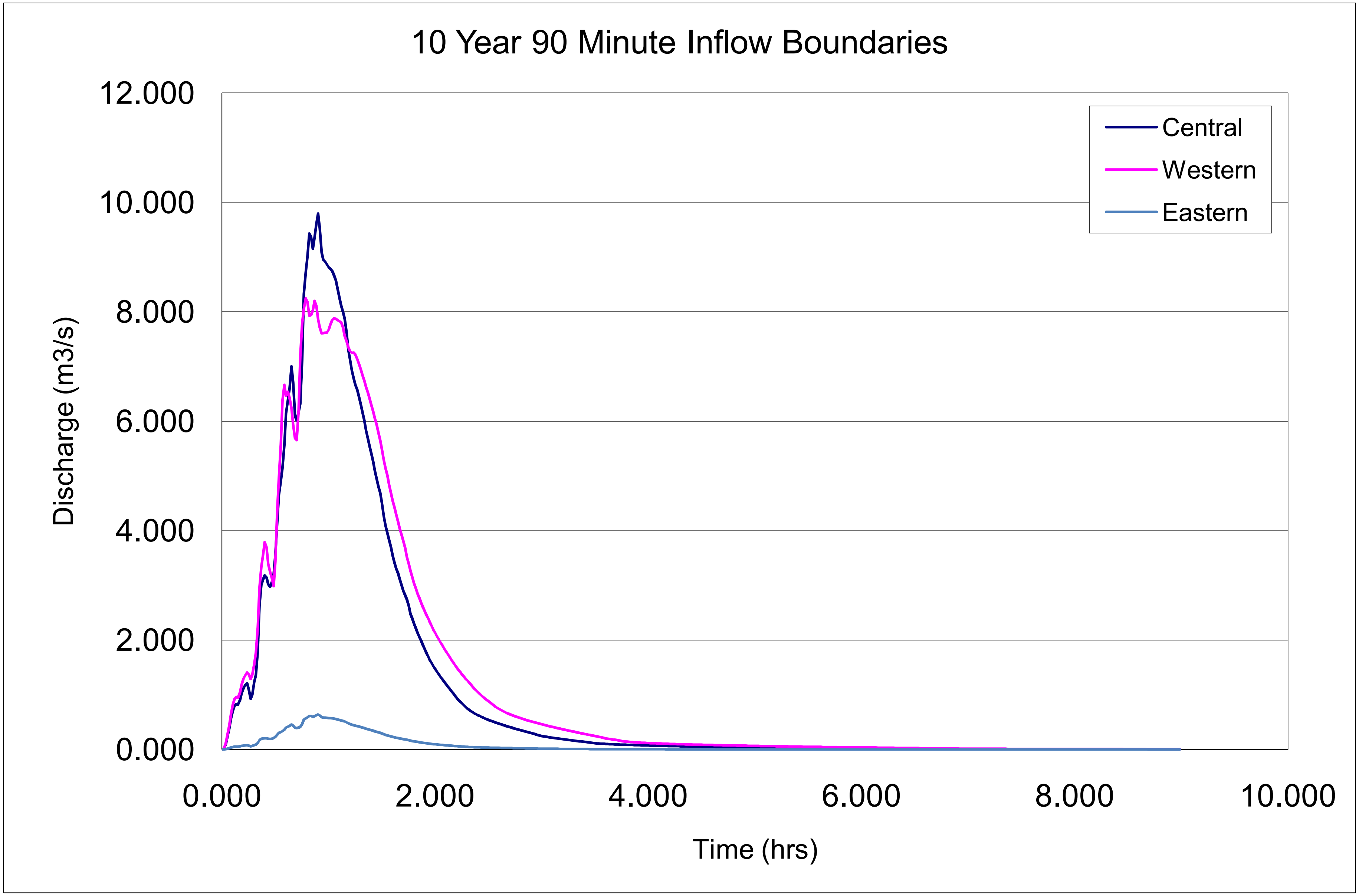
Inflow Boundaries



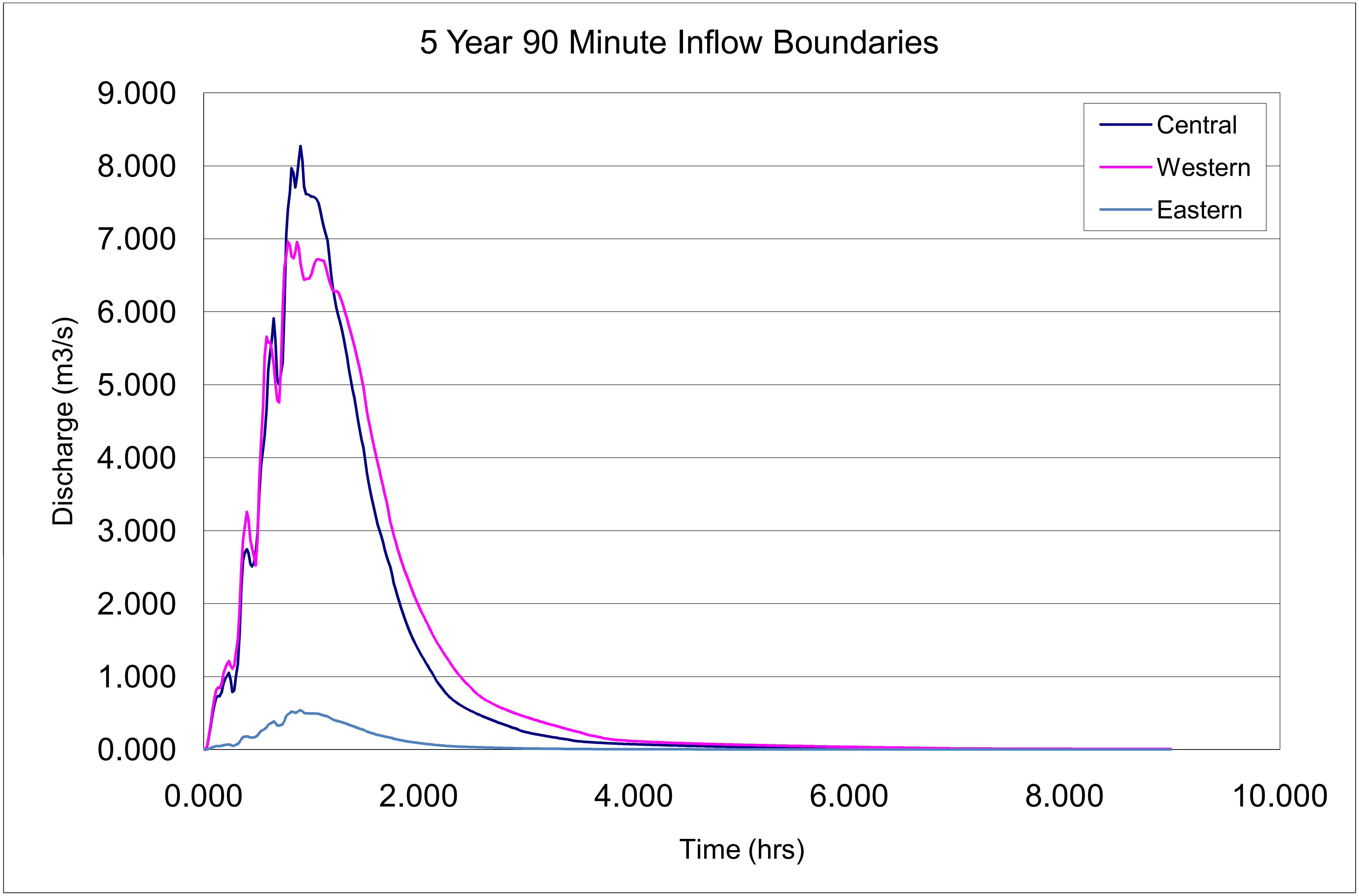
100 Year 90 Minute Inflow Boundaries
Discharge (m3/s)
18
16
14
12
10
8
6
4
2
0
0.000
2.000
4.000
6.000
8.000
10.000
Time (hrs)
Central
Western
Eastern

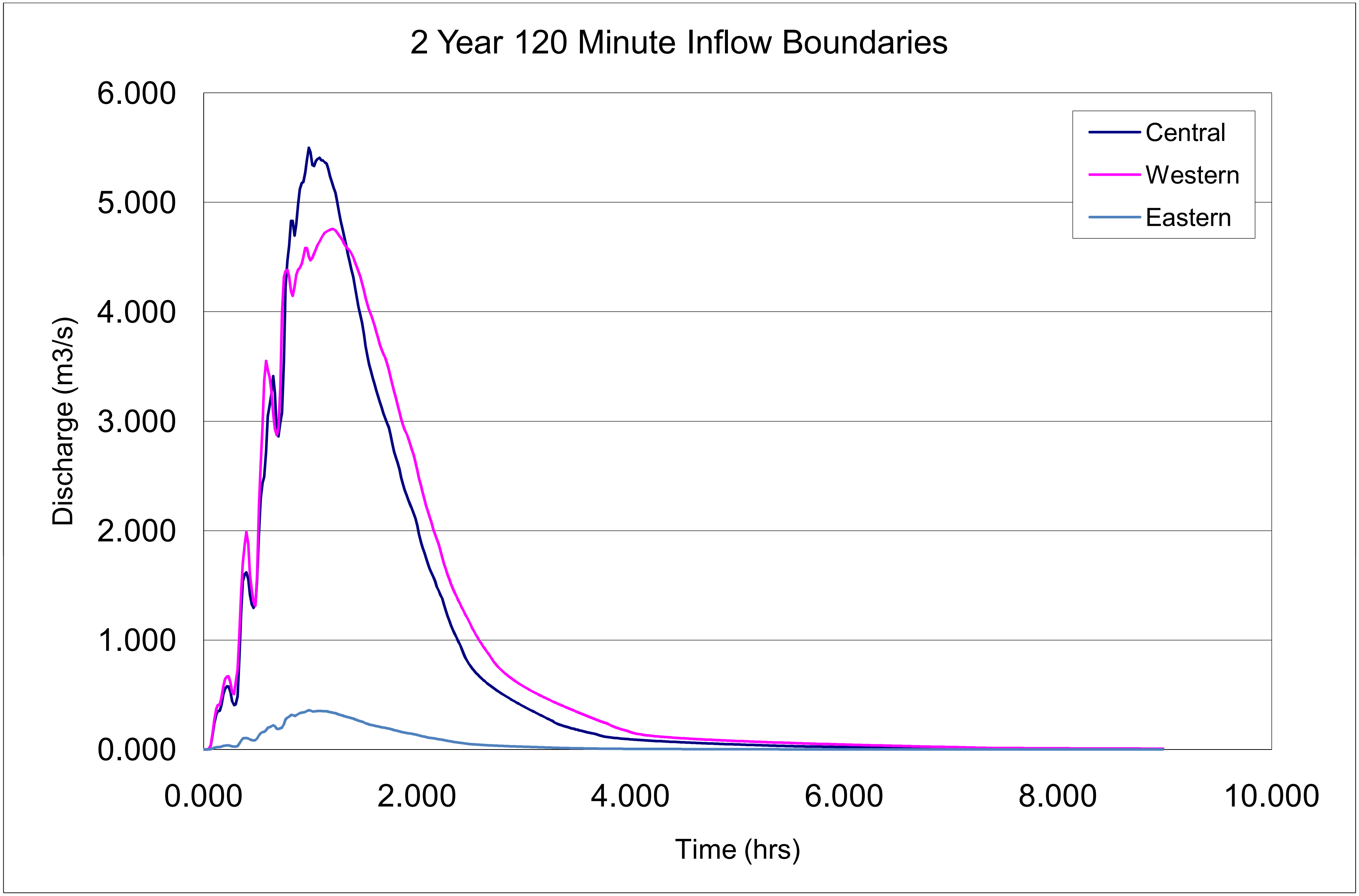

50 Year 90 Minute Inflow Boundaries
Discharge (m3/s)
16
14
12
10
8
6
4
2
0
0.000
2.000
4.000
6.000
8.000
10.000
Time (hrs)
Central
Western
Eastern


20 Year 90 Minute Inflow Boundaries
Discharge (m3/s)
14.000
12.000
10.000
8.000
6.000
4.000
2.000
0.000
0.000
2.000
4.000
6.000
8.000
10.000
Time (hrs)
Central
Western
Eastern

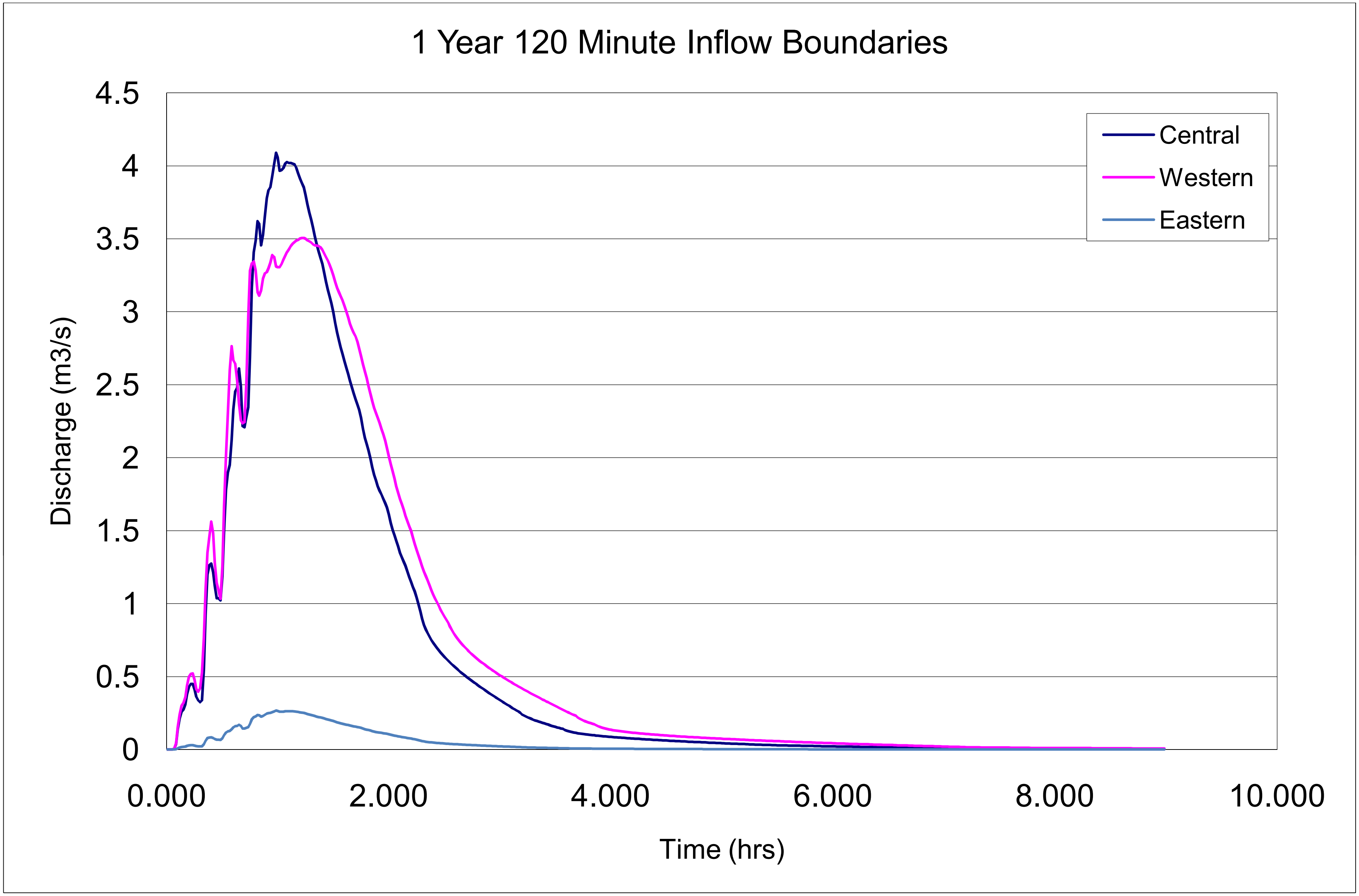


10 Year 90 Minute Inflow Boundaries
Central
Western
Eastern
Discharge (m3/s)
12.000
10.000
8.000
6.000
4.000
2.000
0.000
0.000
2.000
4.000
6.000
8.000
10.000
Time (hrs)


5 Year 90 Minute Inflow Boundaries
Central
Western
Eastern
Discharge (m3/s)
9.000
8.000
7.000
6.000
5.000
4.000
3.000
2.000
1.000
0.000
0.000
2.000
4.000
6.000
8.000
10.000
Time (hrs)



2 Year 120 Minute Inflow Boundaries
Discharge (m3/s)
6.000
5.000
4.000
3.000
2.000
1.000
0.000
0.000
2.000
4.000
6.000
8.000
10.000
Time (hrs)
Central
Western
Eastern


1 Year 120 Minute Inflow Boundaries
Discharge (m3/s)
Time (hrs)
Central
Western
Eastern
4.5
4
3.5
3
2.5
2
1.5
1
0.5
0
0.000
2.000
4.000
6.000
8.000
10.000

Appendix E

Results Comparisons

100 Year 90 Minute Results

Chainage	1D Peak Water Level m AHD	2D Peak Water Level m AHD	Difference m
1000	8.575	8.919	0.344
1134	8.078	8.071	-0.007
1237	7.377	7.304	-0.073
1348	6.087	6.195	0.108
1507	4.814	5.052	0.238
1698	3.472	3.554	0.082
2000	7.875	7.896	0.021
2096	7.198	7.179	-0.019
2259	5.491	6.155	0.664

Chainage	1D Peak Discharges m3/s	2D Peak Discharges m3/s	Difference m3/s	Difference %
1134	17.209	17.278	0.069	0.4%
1237	17.056	15.599	-1.457	-8.5%
1348	16.887	14.639	-2.248	-13.3%
1507	30.429	26.511	-3.918	-12.9%
1698	29.692	27.357	-2.335	-7.9%
2096	14.641	14.625	-0.016	-0.1%
2259	14.236	14.44	0.204	1.4%

1 Year 120 Minute Results

Chainage	1D Peak Water Level m AHD	2D Peak Water Level m AHD	Difference m
1000	7.883	8.695	0.812
1134	7.609	7.754	0.145
1237	6.998	7.056	0.058
1348	5.953	6.164	0.211
1507	4.706	5.015	0.309
1698	3.27	3.414	0.144
2000	7.654	7.784	0.13
2096	7.041	6.996	-0.045
2259	5.272	5.951	0.679

Chainage	1D Peak Discharges m3/s	2D Peak Discharges m3/s	Difference m3/s	Difference %
1134	4.326	4.244	-0.082	-1.9%
1237	4.275	4.213	-0.062	-1.5%
1348	4.273	4.205	-0.068	-1.6%
1507	7.616	7.339	-0.277	-3.6%
1698	7.478	7.415	-0.063	-0.8%
2096	3.502	3.502	0	0.0%
2259	3.471	3.492	0.021	0.6%

1D Section Name	2D Location Name	1D Peak Water Levels m AHD	2D Peak Water Levels m AHD	Difference m
MB1 1000	2D MB1_1	7.798	7.868	0.070
MB1 1067	2D MB1_2	7.244	7.426	0.182
MB1 1077	2D MB1_3	7.175	7.350	0.175
MB1 1089	2D MB1_4	7.114	7.179	0.065
MB1 1245	2D MB1_5	6.151	6.156	0.005
MB1 1361	2D MB1_6	5.410	5.488	0.078
MB1 1469	2D MB1_7	4.452	4.541	0.089
MB1 1555	2D MB1_8	3.892	3.930	0.038
MB1 1664	2D MB1_9	3.454	3.287	-0.167
MB2 1000	2D MB2_1	8.334	8.559	0.225
MB2 1101	2D MB2_2	8.068	8.371	0.303
MB2 1149	2D MB2_3	7.810	8.028	0.218
MB2 1173	2D MB2_4	7.810	7.820	0.010
MB2 1178	2D MB2_5	7.704	7.818	0.114
MB2 1186	2D MB2_6	7.700	7.802	0.102
MB2 1207	2D MB2_7	7.543	7.656	0.113
MB2 1288	2D MB2_9	6.744	6.843	0.099
MB2 1345	2D MB2_10	6.106	6.213	0.107
MB3 1000	2D MB3_1	8.892	8.920	0.028
MB3 1055	2D MB3_2	8.077	8.392	0.315
MB3 1070	2D MB3_3	8.073	8.377	0.304
MB3 1079	2D MB3_4	8.073	8.378	0.305
MB3 1091	2D MB3_5	7.996	8.273	0.277
MB3 1119	2D MB3_6	7.662	7.669	0.007
MB3 1211	2D MB3_8	6.930	6.957	0.027
MB3 1278	2D MB3_9	6.497	6.491	-0.006
MB3 1359	2D MB3_10	5.735	5.800	0.065
MB3 1455	2D MB3_11	4.630	5.088	0.458
MB3 1508	2D MB3_12	4.332	4.629	0.297
MB3 1617	2D MB3_13	3.512	3.886	0.374
MB3 1679	2D MB3_14	3.456	3.429	-0.027
SB1 1043	2D SB1_1	3.688	4.078	0.390
SB1 1056	2D SB1_2	3.536	3.965	0.429
SB1 1072	2D SB1_3	3.516	3.918	0.402
SB1 1091	2D SB1_4	3.363	3.711	0.348
SB1 1102	2D SB1_5	3.295	3.603	0.308
SB1 1114	2D SB1_6	3.261	3.513	0.252
SB1 1145	2D SB1_7	3.076	3.160	0.084
SB1 1189	2D SB1_8	2.808	2.803	-0.005
SB1 1205	2D SB1_9	2.715	2.725	0.010
SB1 1235	2D SB1_10	2.592	2.628	0.036
SB2 1088	2D SB2_1	5.671	5.600	-0.071
SB2 1124	2D SB2_2	5.151	5.310	0.159
SB2 1186	2D SB2_3	4.423	4.776	0.353
SB2 1285	2D SB2_4	4.227	4.205	-0.022
SB2 1442	2D SB2_5	3.433	3.716	0.283
SB2 1482	2D SB2_6	3.266	3.463	0.197
SB2 1570	2D SB2_8	2.667	2.618	-0.049
SB2 1639	2D SB2_10	2.502	2.617	0.115
CB1 1062	2D CB1_1	7.522	7.600	0.078
CB1 1109	2D CB1_2	7.096	7.154	0.058
CB1 1172	2D CB1_4	6.418	6.517	0.099
CB2 1103	2D CB2_1	4.690	4.693	0.003
CB2 1162	2D CB2_2	4.363	4.341	-0.022
CB2 1220	2D CB2_3	3.979	3.962	-0.017
CB2 1280	2D CB2_5	3.250	3.350	0.100
CB3 1082	2D CB3_1	7.769	7.624	-0.145
CB3 1141	2D CB3_2	7.358	7.251	-0.107
CB3 1172	2D CB3_3	7.090	6.962	-0.128
CB4 1075	2D CB4_1	7.404	7.411	0.007
CB4 1124	2D CB4_2	6.949	6.982	0.033
CB4 1197	2D CB4_3	6.121	6.243	0.122
CB4 1326	2D CB4_5	4.482	4.523	0.041
CB4 1368	2D CB4_6	4.074	4.038	-0.036

1D Section Name	2D Location Name	1D Peak Discharges m3/s	1D Peak Discharges m3/s	Difference m3/s	Difference%
MB1 1067	2D MB1_2	14.691	14.634	-0.057	-0.4%
MB1 1077	2D MB1_3	14.648	14.617	-0.031	-0.2%
MB1 1089	2D MB1_4	14.631	14.625	-0.006	0.0%
MB1 1245	2D MB1_5	14.483	14.436	-0.047	-0.3%
MB1 1361	2D MB1_6	14.051	25.546	11.495	81.8%
MB1 1469	2D MB1_7	23.668	25.347	1.679	7.1%
MB1 1555	2D MB1_8	23.472	25.212	1.74	7.4%
MB1 1664	2D MB1_9	20.611	26.793	6.182	30.0%
MB2 1101	2D MB2_2	16.205	16.013	-0.192	-1.2%
MB2 1149	2D MB2_3	12.415	15.409	2.994	24.1%
MB2 1173	2D MB2_4	15.104	13.894	-1.21	-8.0%
MB2 1178	2D MB2_5	38.537	13.177	-25.36	-65.8%
MB2 1186	2D MB2_6	17.1	12.411	-4.689	-27.4%
MB2 1207	2D MB2_7	11.579	11.628	0.049	0.4%
MB2 1288	2D MB2_9	10.77	11.541	0.771	7.2%
MB2 1345	2D MB2_10	10.638	11.403	0.765	7.2%
MB3 1055	2D MB3_2	1.522	1.491	-0.031	-2.0%
MB3 1070	2D MB3_3	1.519	1.473	-0.046	-3.0%
MB3 1079	2D MB3_4	1.516	1.463	-0.053	-3.5%
MB3 1091	2D MB3_5	1.517	1.46	-0.057	-3.8%
MB3 1119	2D MB3_6	1.514	1.449	-0.065	-4.3%
MB3 1211	2D MB3_8	1.477	1.419	-0.058	-3.9%
MB3 1278	2D MB3_9	4.867	1.677	-3.19	-65.5%
MB3 1359	2D MB3_10	3.716	1.49	-2.226	-59.9%
MB3 1455	2D MB3_11	0.187	2.977	2.79	1492.0%
MB3 1508	2D MB3_12	0.171	2.521	2.35	1374.3%
MB3 1617	2D MB3_13	0.1	1.045	0.945	945.0%
MB3 1679	2D MB3_14	0.1	2.31	2.21	2210.0%
SB1 1043	2D SB1_1	0.167	1.52	1.353	810.2%
SB1 1056	2D SB1_2	0.166	1.091	0.925	557.2%
SB1 1072	2D SB1_3	0.164	1.178	1.014	618.3%
SB1 1091	2D SB1_4	0.164	0.851	0.687	418.9%
SB1 1102	2D SB1_5	0.164	0.898	0.734	447.6%
SB1 1114	2D SB1_6	0.162	0.735	0.573	353.7%
SB1 1145	2D SB1_7	0.17	0.769	0.599	352.4%
SB1 1189	2D SB1_8	2.323	1.857	-0.466	-20.1%
SB1 1205	2D SB1_9	2.313	2.45	0.137	5.9%
SB1 1235	2D SB1_10	2.235	2.891	0.656	29.4%
SB2 1088	2D SB2_1	3.577	0.376	-3.201	-89.5%
SB2 1124	2D SB2_2	3.523	0.25	-3.273	-92.9%
SB2 1186	2D SB2_3	3.493	0.213	-3.28	-93.9%
SB2 1285	2D SB2_4	3.169	0.211	-2.958	-93.3%
SB2 1442	2D SB2_5	2.558	0.195	-2.363	-92.4%
SB2 1482	2D SB2_6	2.555	0.177	-2.378	-93.1%
SB2 1570	2D SB2_8	2.558	0.142	-2.416	-94.4%
SB2 1639	2D SB2_10	2.853	0.301	-2.552	-89.4%
CB1 1062	2D CB1_1	0.944	2.202	1.258	133.3%
CB1 1109	2D CB1_2	0.552	1.507	0.955	173.0%
CB1 1172	2D CB1_4	0.549	2.579	2.03	369.8%
CB2 1103	2D CB2_1	2.928	1.3	-1.628	-55.6%
CB2 1162	2D CB2_2	2.695	1.414	-1.281	-47.5%
CB2 1220	2D CB2_3	2.505	1.143	-1.362	-54.4%
CB2 1280	2D CB2_5	2.39	1.436	-0.954	-39.9%
CB3 1082	2D CB3_1	3.774	0.586	-3.188	-84.5%
CB3 1141	2D CB3_2	3.745	0.706	-3.039	-81.1%
CB3 1172	2D CB3_3	3.729	0.547	-3.182	-85.3%
CB4 1075	2D CB4_1	3.268	1.404	-1.864	-57.0%
CB4 1124	2D CB4_2	1.063	1.309	0.246	23.1%
CB4 1197	2D CB4_3	1.046	0.833	-0.213	-20.4%
CB4 1326	2D CB4_5	1.018	0.465	-0.553	-54.3%
CB4 1368	2D CB4_6	0.988	0.396	-0.592	-59.9%

1D Section Name	2D Location Name	1D Peak Water Levels m AHD	2D Peak Water Levels m AHD	Difference m
MB1 1000	MB1_1	7.647	7.750	0.103
MB1 1067	MB1_2	7.149	7.296	0.147
MB1 1077	MB1_3	7.017	7.235	0.218
MB1 1089	MB1_4	6.961	6.996	0.035
MB1 1245	MB1_5	5.964	5.955	-0.009
MB1 1361	MB1_6	5.272	5.452	0.180
MB1 1469	MB1_7	4.346	4.398	0.052
MB1 1555	MB1_8	3.729	3.778	0.049
MB1 1664	MB1_9	3.164	3.169	0.005
MB2 1000	MB2_1	7.973	8.061	0.088
MB2 1101	MB2_2	7.781	7.952	0.171
MB2 1149	MB2_3	7.525	7.710	0.185
MB2 1173	MB2_4	7.447	7.523	0.076
MB2 1178	MB2_5	7.391	7.517	0.126
MB2 1186	MB2_6	7.383	7.507	0.124
MB2 1207	MB2_7	7.268	7.343	0.075
MB2 1288	MB2_9	6.577	6.661	0.084
MB2 1345	MB2_10	6.010	6.167	0.157
MB3 1000	MB3_1	8.619	8.696	0.077
MB3 1055	MB3_2	7.946	8.095	0.149
MB3 1070	MB3_3	7.946	8.096	0.150
MB3 1079	MB3_4	7.946	8.096	0.150
MB3 1091	MB3_5	7.935	8.015	0.080
MB3 1119	MB3_6	7.418	7.593	0.175
MB3 1211	MB3_8	6.055	6.847	0.792
MB3 1278	MB3_9	5.702	6.299	0.597
MB3 1359	MB3_10	5.504	5.708	0.204
MB3 1455	MB3_11	4.462	4.806	0.344
MB3 1508	MB3_12	4.145	4.454	0.309
MB3 1617	MB3_13	3.496	3.687	0.191
MB3 1679	MB3_14	3.144	3.280	0.136
SB1 1043	SB1_1	3.615	3.797	0.182
SB1 1056	SB1_2	3.476	3.797	0.321
SB1 1072	SB1_3	3.475	3.797	0.322
SB1 1091	SB1_4	3.248	3.171	-0.077
SB1 1102	SB1_5	3.166	3.060	-0.106
SB1 1114	SB1_6	3.163	3.114	-0.049
SB1 1145	SB1_7	2.979	2.915	-0.064
SB1 1189	SB1_8	2.586	2.526	-0.060
SB1 1205	SB1_9	2.473	2.453	-0.020
SB1 1235	SB1_10	2.307	2.453	0.146
SB2 1088	SB2_1	5.357	5.421	0.064
SB2 1124	SB2_2	4.889	4.856	-0.033
SB2 1186	SB2_3	4.156	4.080	-0.076
SB2 1285	SB2_4	4.101	4.085	-0.016
SB2 1442	SB2_5	3.060	3.054	-0.006
SB2 1482	SB2_6	2.984	2.984	0.000
SB2 1570	SB2_8	2.080	2.453	0.373
SB2 1639	SB2_10	2.111	2.453	0.342
CB1 1062	CB1_1	7.232	7.430	0.198
CB1 1109	CB1_2	7.014	6.996	-0.018
CB1 1172	CB1_4	4.851	6.331	1.480
CB2 1103	CB2_1	4.568	4.502	-0.066
CB2 1162	CB2_2	4.274	4.222	-0.052
CB2 1220	CB2_3	3.893	3.864	-0.029
CB2 1280	CB2_5	3.013	3.097	0.084
CB3 1082	CB3_1	7.582	7.519	-0.063
CB3 1141	CB3_2	7.232	7.152	-0.080
CB3 1172	CB3_3	6.896	6.802	-0.094
CB4 1075	CB4_1	7.313	7.293	-0.020
CB4 1124	CB4_2	6.829	6.792	-0.037
CB4 1197	cb4_3	6.000	5.979	-0.021
CB4 1326	CB4_5	4.395	4.379	-0.016
CB4 1368	CB4_6	3.988	3.975	-0.013

1D Section Name	2D Location Name	1D Peak Discharges m3/s	1D Peak Discharges m3/s	Difference m3/s	Difference%
MB1 1067	MB1_2	3.504	3.505	0.001	0.0%
MB1 1077	MB1_3	3.501	3.503	0.002	0.1%
MB1 1089	MB1_4	3.501	3.502	0.001	0.0%
MB1 1245	MB1_5	3.488	3.492	0.004	0.1%
MB1 1361	MB1_6	3.444	7.328	3.884	112.8%
MB1 1469	MB1_7	7.003	7.281	0.278	4.0%
MB1 1555	MB1_8	6.969	7.253	0.284	4.1%
MB1 1664	MB1_9	6.651	7.381	0.73	11.0%
MB2 1101	MB2_2	4.057	4.008	-0.049	-1.2%
MB2 1149	MB2_3	3.854	4.007	0.153	4.0%
MB2 1173	MB2_4	4.556	4.008	-0.548	-12.0%
MB2 1178	MB2_5	13.262	4.011	-9.251	-69.8%
MB2 1186	MB2_6	6.26	4.006	-2.254	-36.0%
MB2 1207	MB2_7	3.991	4.003	0.012	0.3%
MB2 1288	MB2_9	3.769	4	0.231	6.1%
MB2 1345	MB2_10	3.752	4.002	0.25	6.7%
MB3 1055	MB3_2	0.266	0.26	-0.006	-2.3%
MB3 1070	MB3_3	0.262	0.254	-0.008	-3.1%
MB3 1079	MB3_4	0.261	0.254	-0.007	-2.7%
MB3 1091	MB3_5	0.261	0.253	-0.008	-3.1%
MB3 1119	MB3_6	0.26	0.253	-0.007	-2.7%
MB3 1211	MB3_8	0.246	0.253	0.007	2.8%
MB3 1278	MB3_9	0.34	0.253	-0.087	-25.6%
MB3 1359	MB3_10	0.11	0.252	0.142	129.1%
MB3 1455	MB3_11	0.1	0.252	0.152	152.0%
MB3 1508	MB3_12	0.1	0.252	0.152	152.0%
MB3 1617	MB3_13	0.1	0.201	0.101	101.0%
MB3 1679	MB3_14	0.1	0.201	0.101	101.0%
SB1 1043	SB1_1	0.1	0.05	-0.05	-50.0%
SB1 1056	SB1_2	0.1	0.038	-0.062	-62.0%
SB1 1072	SB1_3	0.1	0.002	-0.098	-98.0%
SB1 1091	SB1_4	0.1	0.002	-0.098	-98.0%
SB1 1102	SB1_5	0.1	0.001	-0.099	-99.0%
SB1 1114	SB1_6	0.1	0	-0.1	-100.0%
SB1 1145	SB1_7	0.1	0	-0.1	-100.0%
SB1 1189	SB1_8	0.1	0	-0.1	-100.0%
SB1 1205	SB1_9	0.1	0	-0.1	-100.0%
SB1 1235	SB1_10	0.1	0.004	-0.096	-96.0%
SB2 1088	SB2_1	0.1	0	-0.1	-100.0%
SB2 1124	SB2_2	0.1	0	-0.1	-100.0%
SB2 1186	SB2_3	0.1	0	-0.1	-100.0%
SB2 1285	SB2_4	0.1	0	-0.1	-100.0%
SB2 1442	SB2_5	0.1	0	-0.1	-100.0%
SB2 1482	SB2_6	0.1	0	-0.1	-100.0%
SB2 1570	SB2_8	0.1	0.001	-0.099	-99.0%
SB2 1639	SB2_10	0.1	0.044	-0.056	-56.0%
CB1 1062	CB1_1	0.1	0	-0.1	-100.0%
CB1 1109	CB1_2	0.1	0	-0.1	-100.0%
CB1 1172	CB1_4	0.1	0	-0.1	-100.0%
CB2 1103	CB2_1	0.1	0	-0.1	-100.0%
CB2 1162	CB2_2	0.1	0	-0.1	-100.0%
CB2 1220	CB2_3	0.1	0	-0.1	-100.0%
CB2 1280	CB2_5	0.1	0	-0.1	-100.0%
CB3 1082	CB3_1	0.223	0	-0.223	-100.0%
CB3 1141	CB3_2	0.21	0	-0.21	-100.0%
CB3 1172	CB3_3	0.206	0	-0.206	-100.0%
CB4 1075	CB4_1	0.286	0	-0.286	-100.0%
CB4 1124	CB4_2	0.1	0	-0.1	-100.0%
CB4 1197	cb4_3	0.1	0	-0.1	-100.0%
CB4 1326	CB4_5	0.1	0	-0.1	-100.0%
CB4 1368	CB4_6	0.1	0	-0.1	-100.0%