Tomago wetland hydrological study - Kooragang Nature Reserve

THE UNIVERSITY OF NEW SOUTH WALES

water
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Manly Vale N.S.W. Australia

TOMAGO WETLAND HYDROLOGICAL STUDY, KOORAGANG NATURE RESERVE

by

W C Glamore, K M Hawker and B M Miller

Technical Report 2005/28
September 2005

THE UNIVERSITY OF NEW SOUTH WALES
SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING
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1. INTRODUCTION

The Water Research Laboratory (WRL) at the University of New South Wales was commissioned by the NSW National Parks and Wildlife Services (NPWS) to undertake numerical modelling works associated with rehabilitating the Tomago Wetland within and adjacent to Kooragang Nature Reserve (Figure 1.1). The overall goal of the Tomago Rehabilitation Project, of which this study forms a component, is to reinstate tidal flows at the Tomago Wetlands to restore shorebird night roosting habitat and enhance fish nursery, while maintaining rainwater drainage off of neighbouring properties and ensuring that tidal influence is contained within the project boundary.

To achieve this goal, a range of investigations have been undertaken including previous preliminary hydrodynamic modelling (Patterson Britton and Partners, 2004). Following these projects, an unplanned trial inundation occurred in late December 2004 – early January 2005 when two 1500 mm floodgate flaps were vandalised and tidal inundation occurred across the site. During this event, on-ground and aerial inspections of the inundation extent were undertaken. NPWS considered that the amount of water within the site would be sufficient for the project's ecological goals, while not requiring extensive engineering works to protect private properties.

NPWS commissioned WRL to use the information from the unplanned trial inundation and other available information to assist in achieving the projects goal. This specific study's primary aim was to develop a hydrodynamic model that could effectively simulate the trial conditions. With this established model a number of sensitivity tests and scenarios could be undertaken to determine the extent of works required to permit tidal inundation, the location of these works, the sensitivity to various drain designs and numerical coefficients, and the influence of tidal inundation on site drainage. The findings from these tasks are given within this report.

The report is divided into 5 Sections. Following this introduction, Section 2 provides a description of the modelling approach, including a brief description of the available data and a conceptual model of the field processes. Section 3 describes the model development and qualitative calibration steps. Section 4 details the scenario testing, including a number of tidal flushing scenarios with internal control structures. Finally, Section 5 provides a summary of the results and a series of recommendations for future works.

2. PROCESS UNDERSTANDING AND CONCEPTUALISATION

To ensure that the numerical model would accurately represent the hydraulic processes and make meaningful predictions regarding tidal restoration works across the site, a range of steps were undertaken to understand the hydrodynamic processes at the site. This understanding was then incorporated within a conceptual model of the site and subsequently used to parameterise the numerical model.

2.1 Field Reconnaissance

On August 2nd 2005, a site visit was undertaken by Dr William Glamore of the Water Research Laboratory in conjunction with Mr Michael Murphy (NPWS) and Ms Peggy Svoboda (Hunter-Central Rivers Catchment Management Authority). The site visit included an inspection of the ring drain levee, the floodgate culverts, a range of internal drainage canals and various site features. A photo record and various measurements were taken throughout the day to assist in the model development.

2.2 Data Analysis

A range of data was provided to undertake the study. This primarily included site spot height data, GIS background layers and some files related to the previous preliminary modelling exercise. Upon inspection it was determined that other than some boundary condition information (mean tidal levels), the preliminary modelling files were of limited use to this study. Further, inspection of the topographic data revealed gaps in the datasets which were overcome via manual inspection and integration using ArcGIS. Other information, including aerial photos of the site and previous reports, were assessed with regards to the calibration process.

2.3 Hydraulic Calculations

The flow hydraulics of the long culverts which currently drain the site are complex and require specialised routines within the numerical model. A specific hydraulic relationship was established to predict discharge through the floodgates for any number of open floodgates relative to the tide, independent of the number of culverts that are draining the site (i.e. two floodgates open to tidal flushing and 9 culverts draining the system). The flow through each culvert was optimised to best represent the site conditions and these findings were calibrated against analytic equations and using culvert specific hydraulic programs. More information regarding the culvert hydraulics is provided in Section 3.1.4.

2.4 Conceptual Model Development and Implementation

The study site (Figure 1.1) is a large low-lying floodplain dissected by a series of drainage canals of varying capacity. In its current state, two series of 1500 mm diameter circular floodgates (5 in the western portion of the site and 4 in the eastern) provide drainage of the site (approximately 9,000,000 m^2). One-way floodgates attached to the nine (9) floodgates currently restrict tidal waters entering from the Hunter River and, as such, the water level in the drainage network is continually maintained at/near low tide levels. Inflows to the drainage network include surface and groundwater drainage from upland areas, rainfall, and surface and groundwater drainage directly from the site. Detailed hydrology of the catchment has not been undertaken.

Based on the current understanding of the system, flow within the drainage network is predominately uni-directional, although some leakage through the floodgates may enable bi-directional flow. Further, any water that becomes ponded within the floodplain area, either by rainfall or tidal flushing, typically fills or drains out via preferential channels. The site inspection highlighted these preferential flow areas, which are typically either depressions in the levee banks or small drains without floodgates.

Based on the above understanding of the site a numerical model was parameterised. Model development and calibration is described in Section 3.

3. MODEL SET-UP AND CALIBRATION

Based on the available data and the conceptual model presented above, it was determined that a 1-Dimensional (1-D) numerical model with off-channel storage would best represent the hydrodynamics of the site. A 1-D model is preferable considering:

- the gaps in topographic data
- the importance of matching internal levels and flow control
- the minimal leakage through elements
- the ability to maintain storage-volume relationships
- the quick run times which permits a wide range of sensitivity and scenario testing.

The commercially available hydrodynamic suite of RMA models were employed for the study. RMAGEN was employed during the mesh construction, RMA2 (v. 6.5) was used for model runs and RMAPLT for data visualisation. Extracted data was also plotted in ArcGIS (v. 9.0).

This section describes the steps taken during model development, including model parameterisation, qualitative calibration tasks and a range of sensitivity tests. Scenario testing is discussed in Section 4.

3.1 Model Set-up

3.1.1 Available topography

The topography at the site was established by the analysis of ground survey point data, supplemented by aerial photographs and ArcGIS layers defining the location of the main site features (e.g. drainage layout), as provided by NPWS. The digital elevation map (DEM) was developed in ArcGIS to estimate the ground surface levels across the site, by interpolating the ground survey point data (Figure 3.1). The digital elevation map was subsequently used to calculate the water level-volume relationship of the site.

3.1.2 Mesh

A 1-D mesh of the site was created in RMAGEN. The mesh consists of a network of 1-D elements representing the main drainage channels (i.e. the north-south drain, the ring drain, and their major connecting drains), connected by junctions to 1-D elements containing overbank storage. These storage areas were developed using ArcGIS and represent the overbank floodplain areas.

The elevations of the nodes forming the mesh were set equal to the invert levels of the drains, as provided by NPWS. The elevations of the overbank storage areas were defined according to the ground surface tin.

Results (water elevations, velocities) from model simulations could be extracted at any node in the mesh for analysis. Figure 3.2 shows the 1-D mesh with nodes labelled where results have been extracted for presentation in this report.

3.1.3 Boundary Conditions

An input elevation boundary condition was applied at the east and west gate entrances of the site to simulate the tidal fluctuations (Figure 3.3). Two tides were applied during the model simulations: a mean tide with an amplitude of 1 m, oscillating between ± 0.5 metres Australian Height Datum (m AHD); and an event tide from January 2005 which fluctuates between an elevation of approximately 0.90 m AHD and - 0.80 m AHD (as determined from tidal harmonics for Sydney).

Constant flow rates were applied as boundary inflows at two nodes (Figure 3.2) in the northern (central and eastern) extents of the site where it was desired to simulate inflow rates (i.e. rainfall, drainage etc.). Constant flow rates of 0, 2.5 and 5 m^3/s were applied on the boundary nodes. These values are based on estimates of either rainfall across the site (using IFD calculations) or long term drainage, however, hydrological investigations have not been undertaken.

Manning's *n* coefficients for the drainage channel network were selected based on field observations, literature and professional judgement. A range of coefficients were applied to the mesh, from 0.025 in the main North-South (NS) and ring drains to 0.050 in the drains connected to off-channel storage and 0.070 in the minor connecting drains. These Manning's *n* coefficients are similar to preliminary modelling studies of the site.

3.1.4 Hydraulics

As described in Section 2.3, a specialised flow control structure routine was developed and incorporated into the RMA2 model to simulate any desired combination of open and closed floodgates. The long culverts (17 m) were predicted to operate so that inflow is calculated as $Q = H^{3/2}$, or $Q = (H_1 - H_2)^{0.5}$ in drowned conditions, which was conceptualised into the rates shown in Figure 3.4. These flow control structures were applied at the east and west entrances to simulate four (4) closed gates at the east entrance, and five (5) gates at the west entrance. For the various scenario runs, two (2) gates at the west entrance were open to tidal inundation of the wetland (as occurred during the unplanned tidal flushing event).

An assessment of the culvert hydraulics at the entrances was undertaken using an urban drainage hydraulic simulation model STNET (as detailed in Ball, 1997). Using this program and the RMA results, a range of head- and tail-water conditions was simulated to establish suitable coefficients for the resultant flow equations. These tests have provided confidence that the hydraulics of the flow control structures applied at the entrances are consistent with that expected and observed at the site, however, no site specific information is available to calibrate the hydraulic results.

3.2 Model Calibration and Sensitivity Tests

Limited data is available to calibrate the model. Aerial photographs of the site from the January 2005 flood event when two (2) floodgates were removed from the west gated entrance were available, and assisted in determining the extent of inundation and the hydraulic behaviour of the open culverts at the entrance, but did not provide any quantitative data on water levels or velocities.

Lacking quantitative field data for model calibration, a range of tests were undertaken to determine how the model responds to a range of conditions. Level-volume relationships were compared for the model storage versus the ArcGIS digital elevation map to ensure the correct volume relationships. Figure 3.5 shows that the level-volume relationship of the mesh compares closely (generally within 5-10%) with that derived from the DEM maps.

Additional steps to test the model's sensitivity to a range of environmental and modelling conditions, in lieu of actual calibration data, are outlined below.

3.2.1 Influence of Drain Roughness

Test runs were conducted to understand the influence of drain roughness (Manning's *n*) on the hydrodynamics of the site. A falling elevation boundary condition was applied at the east and west entrances, and the response of selected locations across the mesh to draining was assessed. Manning's *n* coefficients considered suitable for the site were applied as detailed in Section 3.1.3. These values were increased for the second run to 0.050/0.075 for the major/minor drains respectively, to simulate increased drain roughness. Water elevation results of the two drain roughness tests are shown in Figures 3.6 and 3.7.

Figure 3.8 shows the response of these models to drainage by examining the total volume of water stored over time. These results depict that drainage through the model is reliant on several factors and that changes in drain roughness can have a significant influence on the efficiency of the drainage network (the flow rate of water through the drains). These tests also illustrate the importance of accurate field data to calibrate the model.

In the absence of calibration data, the selected Manning's *n* coefficients of 0.025 in the major (NS and ring) drains, and 0.050 in the drains connected to overbank storage, were applied for the subsequent scenario modelling. It is important to note that these coefficients were based on professional judgement and, as shown above, any alterations to the Manning's n value would have a significant impact on the model results.

3.2.2 Influence of Catchment Inflow on Water Levels With No Tidal Flows

To test the model's sensitivity to inflow rates and thus gain an understanding of the average drain water levels prior to restoring the tide, a series of sensitivity tests were undertaken with different inflow rates. Throughout this report 'inflow' into the model is used to describe catchment runoff from upland drainage as noted in Figure 1.1. In contrast, tidal flows are associated with removing tidal gates from culverts located at the eastern or western gates.

In the scenarios described within this section, all floodgates were closed to tidal flows at the east and west entrances, and a range of constant inflows entering the northern extent of the model (Figure 1.1) were applied. The January 2005 tide was applied at the boundary and influenced the flow of water discharging from the site via the gates.

Figures 3.9 and 3.10 show the results for constant inflows into the catchment of 2.5 m^3/s and 5 m^3/s respectively. The average water levels established over the site when an inflow of 2.5 m^3/s is approximately - 0.2 m AHD, and slightly higher (~ 0 m AHD) when the inflow is increased to 5 m^3/s. These water levels are an indication of the natural background levels in the drain and suggest the range of likely water levels within the drain prior to the site vandalism. Additional field information would assist in calibrating these "pre-flooding" conditions.

3.2.3 Influence of Inflows with Tidal Restoration (January 2005 Event)

Sensitivity tests were also run to assess the model's ability to simulate the January 2005 flooding event and to determine the influence of tidal flows entering the site (i.e. simulating the removal of floodgates) versus upland inflows to the site (i.e. rainfall/runoff) during this period.

Figure 3.11 shows the results of the January 2005 event with the spring tide applied, two (2) floodgates removed from the west entrance and no additional inflow to the catchment. Figure 3.12 shows the results with the same model setup, but with the addition of a constant flow of 2.5 m^3/s entering the site. Figure 3.13 shows the total volume of water stored over time for the two simulations. As expected, the water levels across the site are increased by inflow to the site, and subsequently, the total volume of water in the system is increased.

Opening the culverts to tidal flows results in elevated water levels across the site, and allows for overbank flooding of the floodplain (Figure 3.14). The average water level established at the site with a constant inflow of 2.5 m^3/s is increased from approximately -0.2 m AHD when the gates are closed, to approximately 0.1 m AHD when two (2) floodgates are removed. This change in average drain water elevation is worth noting, considering its likely impact on drainage of the site.

In an attempt to calibrate the model, the above simulation was compared against photos of the site taken during the January 2005 event. The model results were shown to qualitatively represent the conditions and accurately represent surface flooding extent. While this information provides for a basic qualitative calibration, additional data would provide a more reliable representation of the site. Lacking additional information, the sensitivity and calibration tests depicted above provided the only means for calibrating the model.

4. SCENARIO MODELLING

Following the calibration and sensitivity testing presented above, the model was employed to simulate a range of scenarios. These scenarios included (i) modifications to the drainage network, such as the installation of internal floodgates and removing connectivity between drains, (ii) the influence of stormwater drainage on drainage and tidal flushing and (iii) altered boundary conditions to simulate controlled tidal flushing. Based on these scenarios a range of recommendations was developed.

4.1 Scenario 1: Existing Conditions

Rationale: To establish a model of baseline conditions at the site. The results from this scenario can then be compared with the results from subsequent scenarios.

Description: A mean tide, as shown in Figure 3.3, was applied at the tidal boundaries. Flushing is allowed via two (2) floodgates at the west entrance. There is no inflow (from rainfall runoff) to the site.

Discussion: The results are shown in Figure 4.1. The average water level at the site is approximately 0.1 m AHD. The results agree well with previous modelling runs and provide a means for comparison under different conditions.

4.2 Scenario 2: Restricted Flow From The West Entrance to The Ring Drain

Rationale: To assess the effect of restricting tidal flow in the ring drain, and in particular, how this will influence the extent/impact of flooding on upstream properties that drain into the eastern boundary of the site.

Description: The mean tide is applied and flushing is allowed via two (2) floodgates at the west entrance. Floodgates are applied to the culverts located on the ring drain between the east and west gates to prevent flow eastward along the ring drain. There is no inflow (from rainfall/runoff) to the site.

Discussion: The results are shown in Figure 4.2. The water elevations in the ring drain show tidal fluctuations due to flow connectivity between the EW and other minor drains and the ring drain, however, there is a significant time delay in rising water levels in the ring drain. The water elevations in the ring drain (nodes 204, 197) and its associated floodplain (nodes 270, 298, 285) are significantly lower than those in Scenario One, with an average water elevation of −0.15 m AHD.

These results indicate that the tailwater condition established by the tidal flushing can significantly influence water levels and drainage in the ring drain. By installing floodgates on the ring drain between the east and west entrances, the potential for flooding in the north-east extent of the site would be significantly reduced, however, these results require validation by field experiment results.

4.3 Scenario 3: Restricted Flow From The West Entrance To The Ring Drain, With Inflow To The Catchment

Rationale: To compare with Scenario 2 and to assess the influence of drainage along the ring drain when tidal flows are restored. Inflow into the system simulates what may happen following rainfall events or through upland drainage.

Description: As in Scenario 2, but with the addition of a constant 2.5 m^3/s inflow to the northern extent of the site as a boundary condition.

Discussion: The results from this scenario are shown in Figure 4.3. The average water elevation in the ring drain has increased compared to Scenario 2, due to the inflow from rainfall. The drainage efficiency from the floodplain storage areas to the ring drain is reduced, and the water level at node 285 remains fairly constant at approximately 0.1 m AHD, whereas in Scenario 2 the water drained from this area with the falling tide.

These results provide a useful comparison to Scenario 2 and simulate the influence of drainage on the system. In the absence of site data, it cannot be determined how accurately this flow rate represents real world conditions.

4.4 Scenario 4: All Flow Connections Removed Between The NS And Ring Drains, With No Inflow To The Catchment

Rationale: To assess the effect of removing all connections between the ring drain and the tidal flows from the western gates and to determine how this will influence the extent/impact of flooding on upstream properties that drain into the north eastern boundary of the site.

Description: As for Scenario 2, but with the removal of all flow connections between the NS and ring drains. This can be achieved by establishing a levee along the west bank of the ring drain and installing a set of floodgates along the minor drains.

Discussion: The results are shown in Figure 4.4. The water elevation in the ring drain is controlled by the elevation of the low tide. Since there is no upland inflow, the water level in the ring drain falls to the elevation of the low tide (average water level of -0.5 m AHD).

Land users in the north east of the site with hydraulic connections to the ring drain would have significant protection from tidal flooding by this configuration.

Figures 4.5, 4.6 and 4.7 show the results at nodes 279, 204 and 298, for Scenarios 1, 2 and 4, respectively. There is no significant change to water elevation at node 279, which is in the floodplain connected to the NS drain. The water elevation at node 279 is increased slightly in Scenario 4, when flow from the NS drain is prevented from entering the ring drain. At nodes 204 and 298 on the ring drain, the effect of each management scenario on water elevation is significant.

4.5 Scenario 5: Controlled Tidal Levels

Rationale: To assess the effect of limiting the maximum tide elevations on water levels in the NS drain and the extent of flooding of properties upstream of the NS drain.

Description: As for Scenario 4, and limiting the mean tide signal at the entrances to simulate the use of automated controlled gates, such as SmartGates. Maximum mean tide water elevations of 0.1 m AHD and 0.2 m AHD were simulated.

Discussion: These results are shown in Figures 4.8 and 4.9 for maximum mean tide elevations of 0.1 m AHD and 0.2 m AHD, respectively. The control gates act to significantly lower both the maximum and average water level in the NS drain and its associated floodplain.

Figures 4.10 and 4.11 show the likely site inundation at water elevations of 0.1 m AHD and 0.2 m AHD, respectively, based on the DEM and with the assumed controls in place for this scenario (i.e. floodgates between the east and west entrances, a levee around the ring drain and floodgates on selected minor drains between the NS and ring drain).

The results show that limiting the maximum elevation of the tide would reduce the impact of tidal flooding on properties upstream of the NS drain, whilst still allowing tidal flushing and inundation of the NS drain and the floodplain.

5. SUMMARY AND RECOMMENDATIONS

The Water Research Laboratory (WRL) was commissioned by the NSW National Parks and Wildlife Services (NPWS) to develop a hydrodynamic model of the Tomago Wetlands, in and adjacent to the Kooragang Nature Reserve in the lower Hunter River estuary (Figure 1.1). The aim of the study is to assess the impact of restoring tidal flushing at the site with regards to meeting environmental (night roosting site for migratory birds, fish nursery) and drainage (no decrease in drainage efficiency) criteria. Tidal restoration is likely to occur via the modification of one (or a series) of tidal floodgates that currently provide drainage to the site. Additional internal modifications to the on-ground infrastructure may be required to ensure the tidal waters are contained within the desired areas.

A 1-Dimensional (1-D) hydrodynamic model, using the RMA suite of models, was developed for the study. Vandalism at the site during December 2004 – January 2005, allowed for a period of unplanned tidal inundation. The qualitative data obtained by NPWS from this period was used as a basic calibration for the model. A range of sensitivity tests were also undertaken to ensure the model results provided a reasonable estimate of site conditions. Further, a numerical subroutine of the RMA source code was adapted to allow flow through the culverts (both bi-directional and/or uni-directional) and this was checked against analytical equations and specialised hydraulic programs.

The sensitivity tests undertaken highlighted the importance of the model calibration process and the need for targeted field data collection programs. Figures 3.6 to 3.8 depict that the model responds well to changes in drain roughness (Manning's *n*) but that field measurements are required to confirm the drainage coefficients. Figures 3.9 and 3.10 highlight the influence of inflow into the system to depict how the drainage system may be operating in its current state under both low flow and flood periods. As a final sensitivity test, the unplanned tidal flushing event was simulated and water elevations across the site were extracted and compared against the available qualitative information. The model was shown to be in good agreement with the available data, although this calibration is only sufficient for broad scale estimates of site hydrodynamics.

Scenario tests were undertaken using the model described above to determine if the goals of the restoration study could be achieved. A baseline test using a long-term average tidal cycle, with flushing through two (2) open floodgates at the west entrance, was first run to provide a means for comparison (Figure 4.1). Subsequent scenarios were then run to determine the optimal method of restoring tidal flows via the western gate without adversely influencing upstream drainage. Figures 4.2 and 4.3 show that floodgating off the culverts between the eastern and western floodgates decreased the impact of tidal flushing on properties that drain into the eastern boundary. This was further improved by removing any connectivity between drains (Figure 4.4).

To reduce the impact on properties upstream of the North-South drain it was proposed that the river water levels (i.e. tidal flows) could be controlled using an automated floodgate structure. Two different maximum water levels, 0.1 m AHD and 0.2 m AHD, were then simulated and these results (as shown in Figures 4.8 – 4.11) indicate that by limiting the tidal elevations the flooding objectives could be achieved without unduly impacting upstream sites. This could be further improved by the installation of a series of one-way floodgates, which restrict tidal waters, along the North-South drain and by providing breaks in the levee bank for the rising water to enter the floodplain area. These levee breaks could take the form of reverse floodgates that allow water into the site under hydrostatic pressure but restrict water flowing out of the system.

Restricting tidal flows by the use of automated floodgates also reduces the total volume of water within the system and thus, maintains the Tomago Wetlands as a suitable flood retention basin within the Hunter Valley Flood Scheme (WBM, 2004). Based on the level – volume relationship (maximum elevation of the ring levee of 1.3 m AHD) filling the site to 0.1 m AHD removes 2.7 % capacity by volume, whereas filling the site to 0.2 m AHD removes 5.62 % of storage capacity. Conversely, by allowing uncontrolled tidal flushing a spring tide of say 0.9 m AHD would remove 57.7 % of storage capacity. Furthermore, an automated floodgate with telemetry (such as a SmartGate) would allow for the system to be remotely operated and as such, could be instructed to remain closed if flooding is forecasted. An automated controlled gate system could also have a pluviometer directly connected to the operating system to alter the systems position (i.e. when it should be open or closed) based on on-site rainfall readings.

Based on the above findings a range of recommendations are suggested below:

1. **Pilot study-** a series of controlled trial openings should be undertaken over several tidal cycles to collect the required data necessary to (i) calibrate the hydrodynamic model, (ii) determine exactly where and of what type of on-ground infrastructure work is necessary and (iii) to provide a better understanding of the culvert hydraulics and future modifications required. All data recorders should be installed at least one week prior to and proceeding the floodgate opening to record the antecedent and recovery conditions. Measurements should be targeted towards obtaining flow (bi-directional) and level information so that discharge can be calculated. At a minimum, water level and/or flow velocities should be calculated at five locations: (i) in and around the opened culverts,

(ii) within the North-South drain, (iii) at two key spots on the floodplain and (iv) within the ring drain near the eastern property boundary.

The information from the pilot study (or studies) could then be used to (i) verify the model for future modelling exercises, (ii) determine the exact location of on-ground works, and (iii) assess the structures required to control tidal flushing.

2. **On-ground works**- While the pilot study would be critical to finalising large scale on-ground works, key infrastructure could be modified prior to the study. Of key concern are the culverts which connect the eastern and western floodgates and the series of culverts which drain the western area of the North-South drain. Site works to floodgate these culverts could commence immediately. Other works such as drain clearing and levee construction/rehabilitation could also commence.

3. **Data collection**- It is recommended that data collection across the site should commence as soon as possible. Data collection should include basic water quality parameters (pH, salinity or electrical conductivity, etc) and physical parameters including water levels and velocities. As detailed in the Pilot Study recommendations above, collection and analysis of field data measurements will provide several key findings. It is important to note that as an ancillary benefit, tidal flushing may improve drain water quality. This is important as surface water with a high content of soluble iron and fines could form highly reactive monosulphide deposits if held in ponds with a reducing condition (as may be created by the tidal flushing). The targeted collection of data would help in highlighting potential risks, while also providing evidence of any improvements in long-term conditions.

4. **Automated floodgate control-** Based on the above findings it appears that some type of control structure should be designed and constructed for the site to control the quantity of water permitted with the system. Any system which is considered for the site should satisfy the 10 essential criteria set out in Glamore (2003). There may be significant advantages in installing a system that can be remotely controlled via telemetry due to site access. The design of the floodgate control system should be reassessed following the pilot study.

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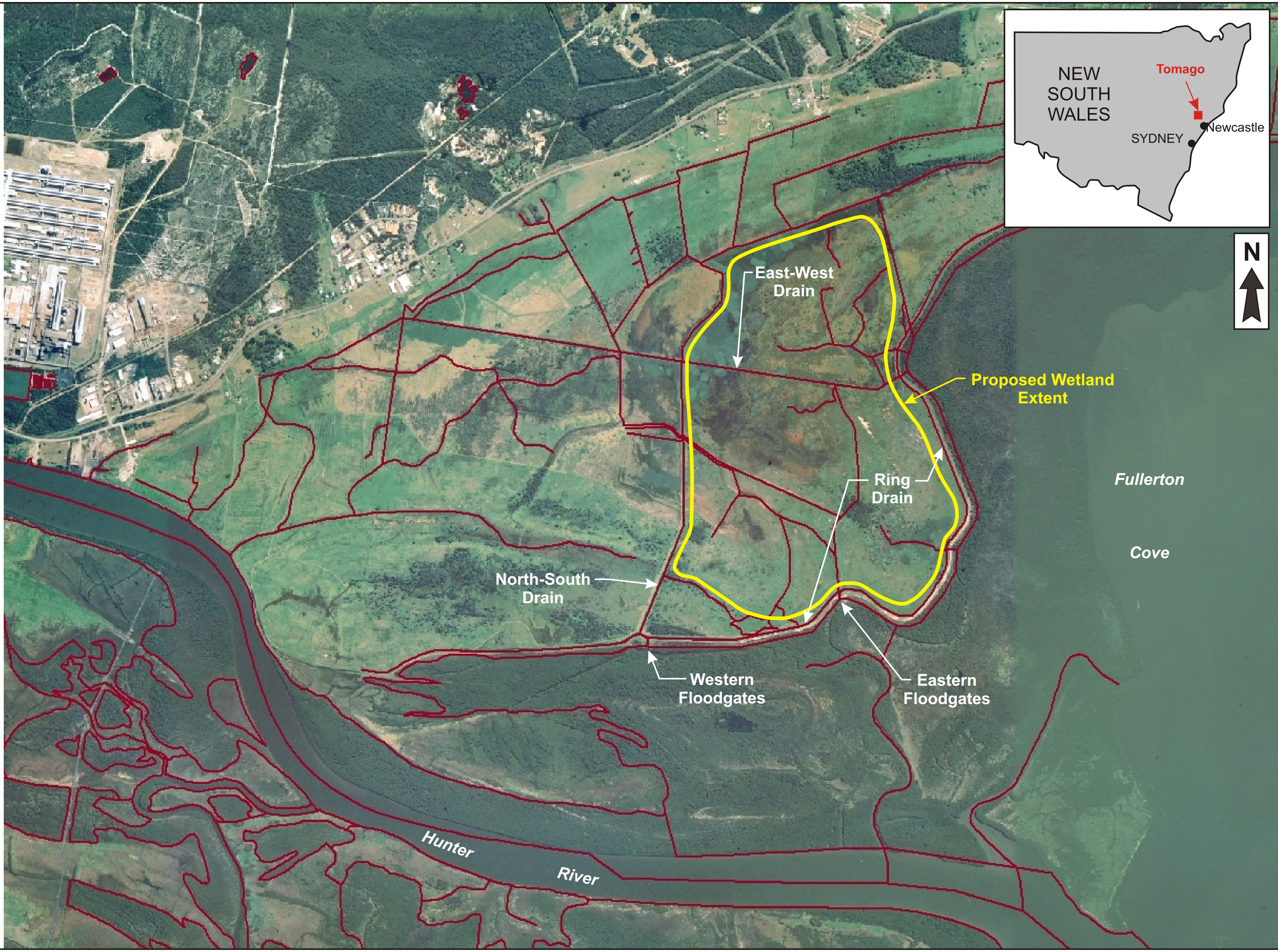

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STUDY DOMAIN

Figure
1.1

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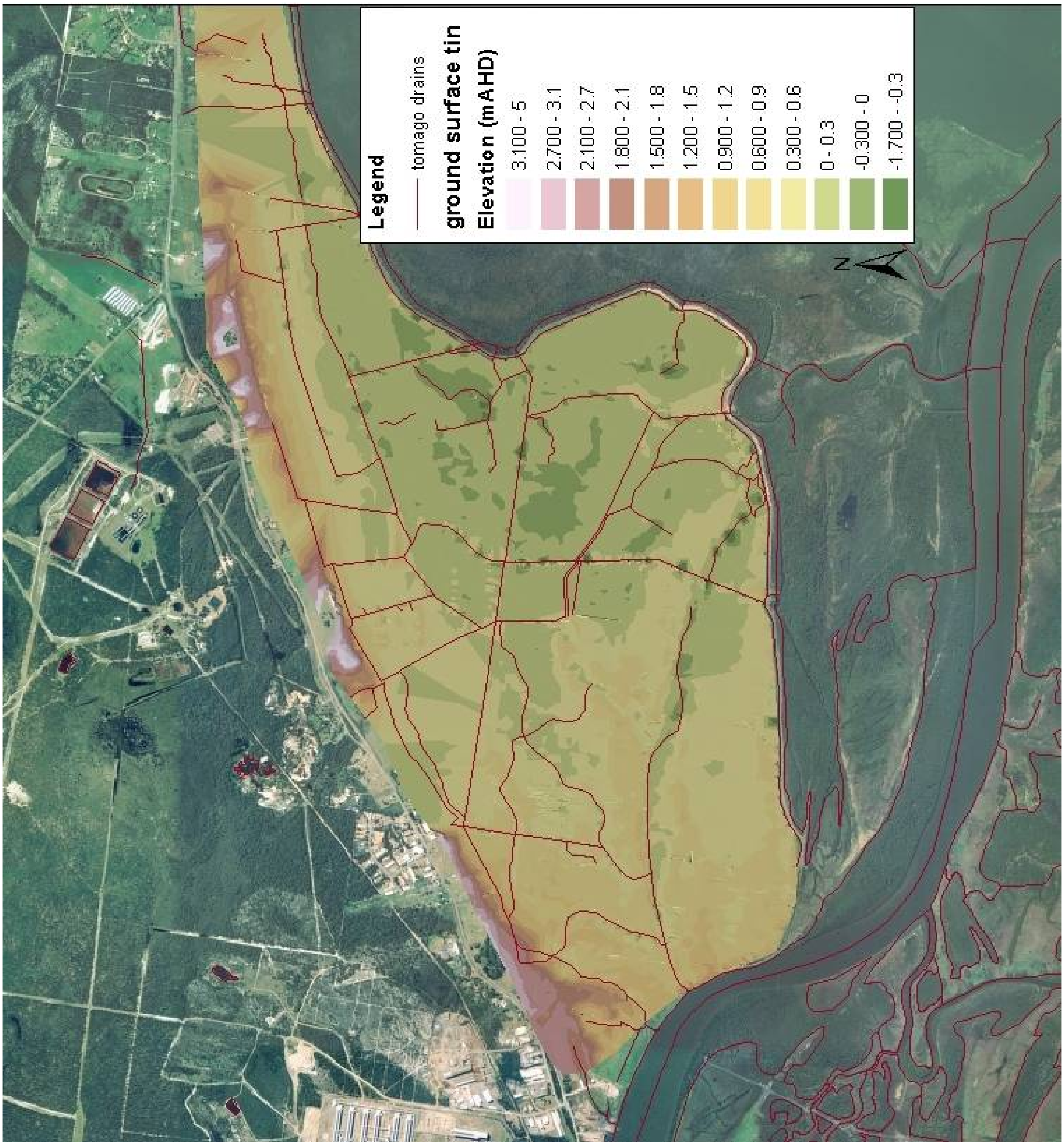
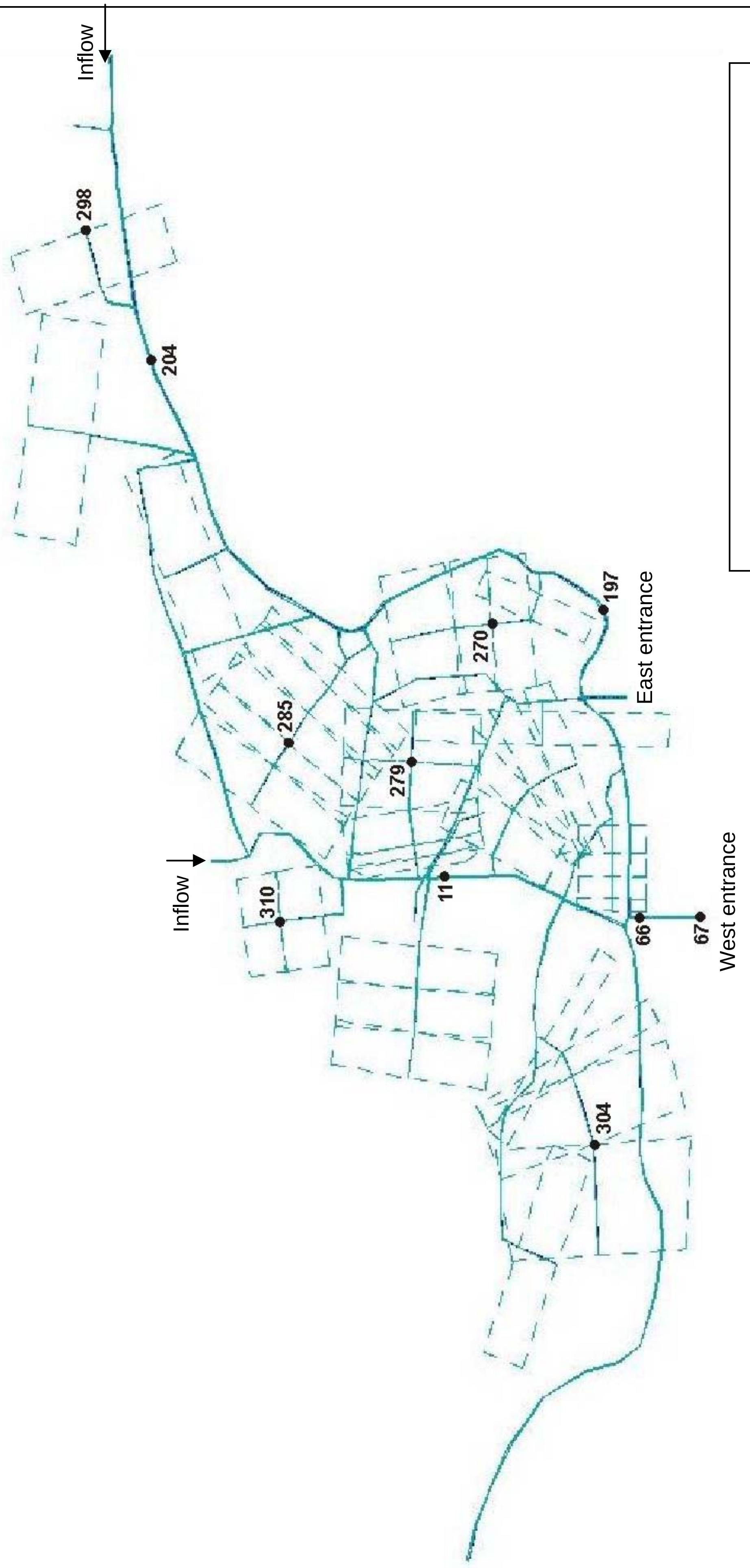


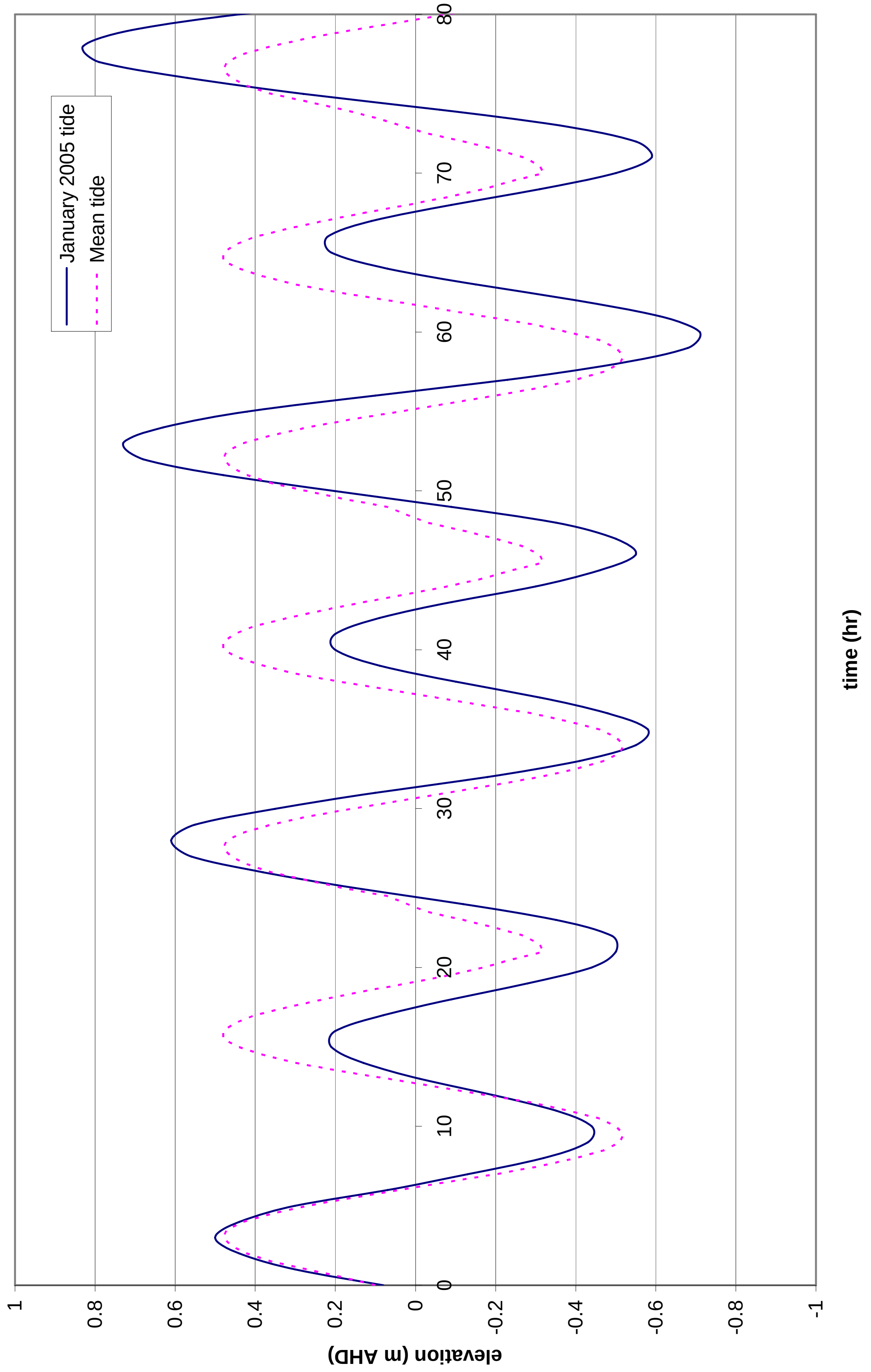
WRL Report No. 2005/28	DIGITAL ELEVATION MAP OF THE SITE	Figure 3.1



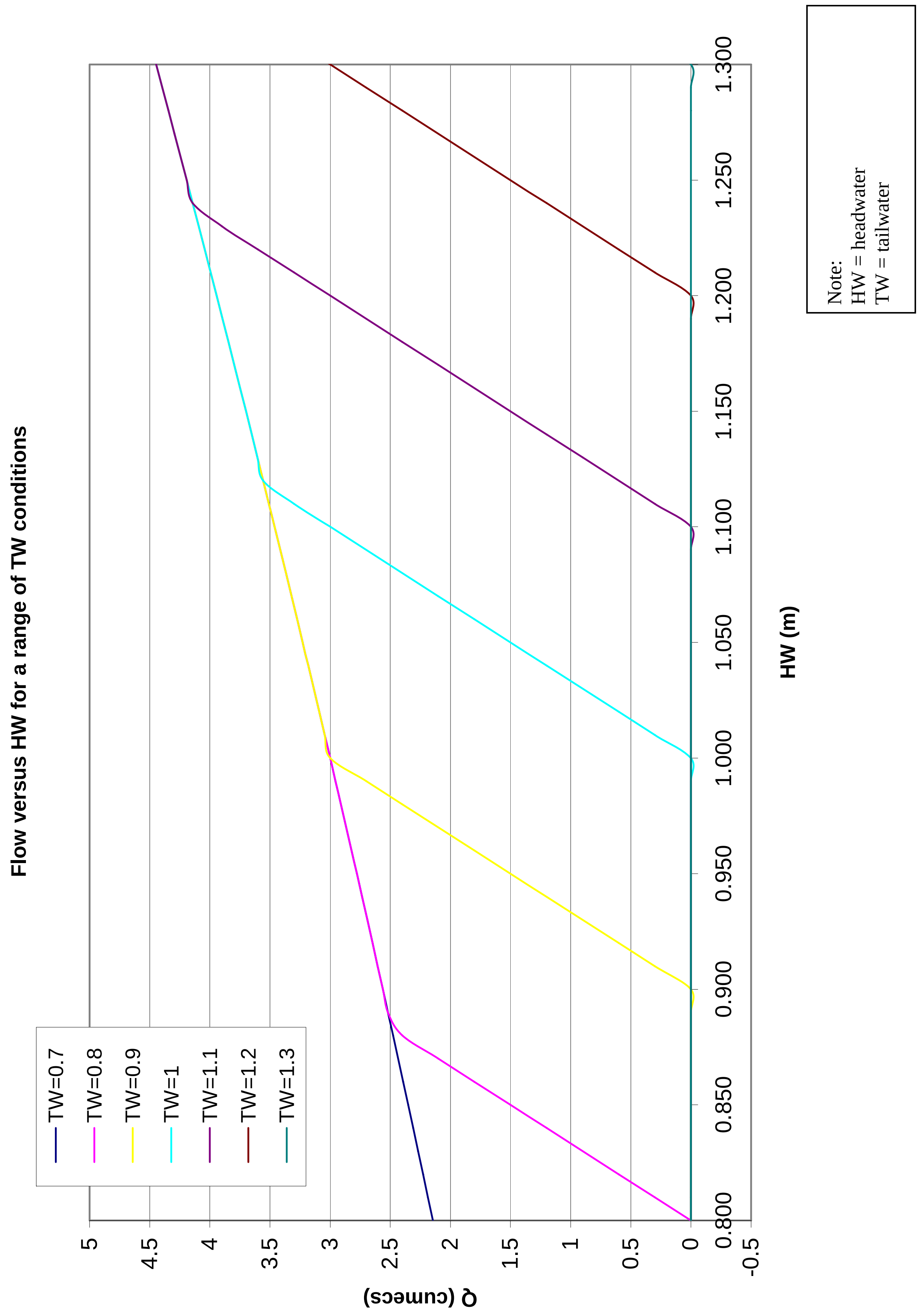
Note: The marked points represent nodes within the mesh which are used in subsequent figures. These points were chosen to represent conditions across the floodplain. Additional locations can be selected.

1-D MESH WITH LOCATIONS OF EXTRACTED DATA

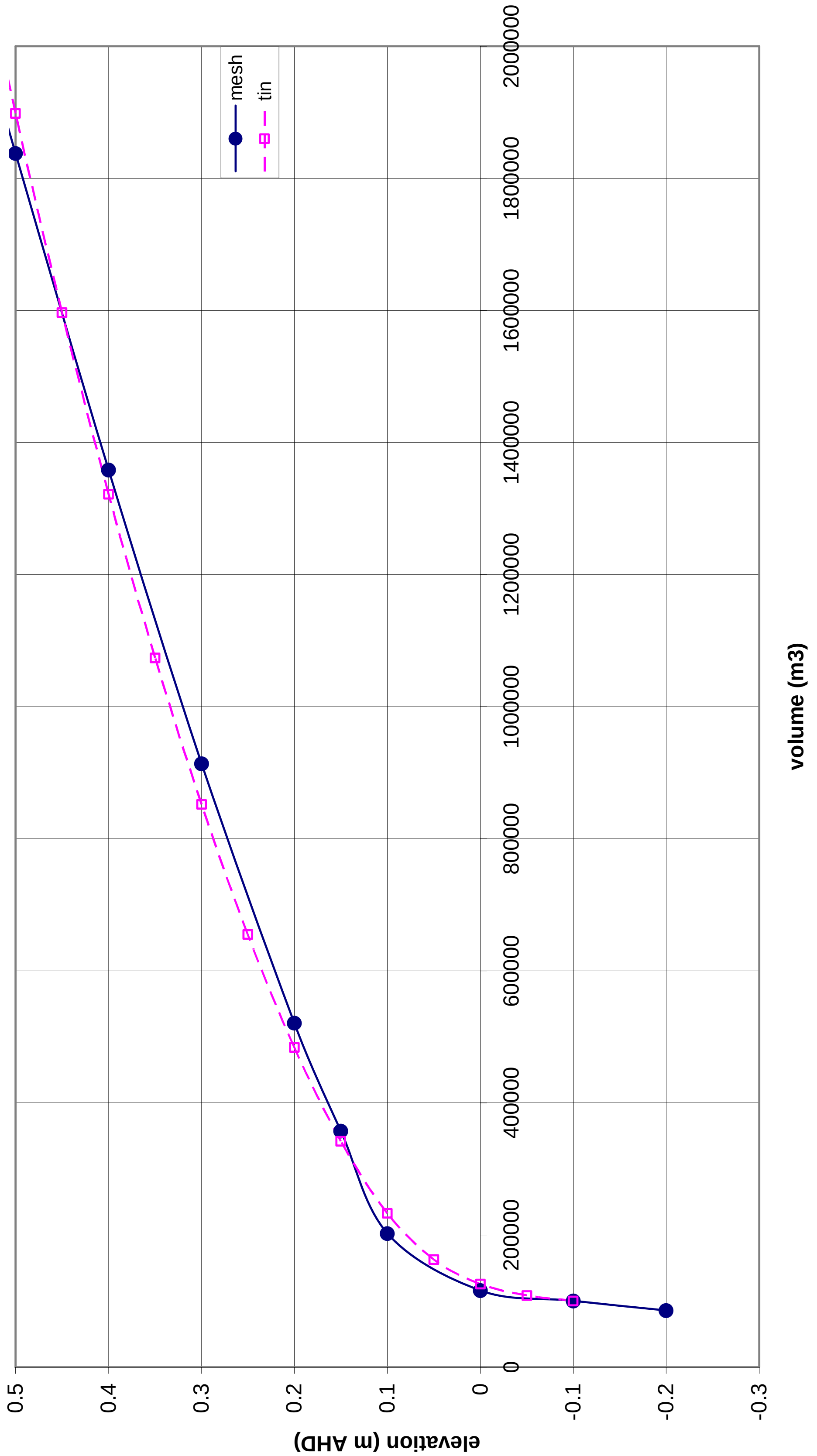
Figure 3.2



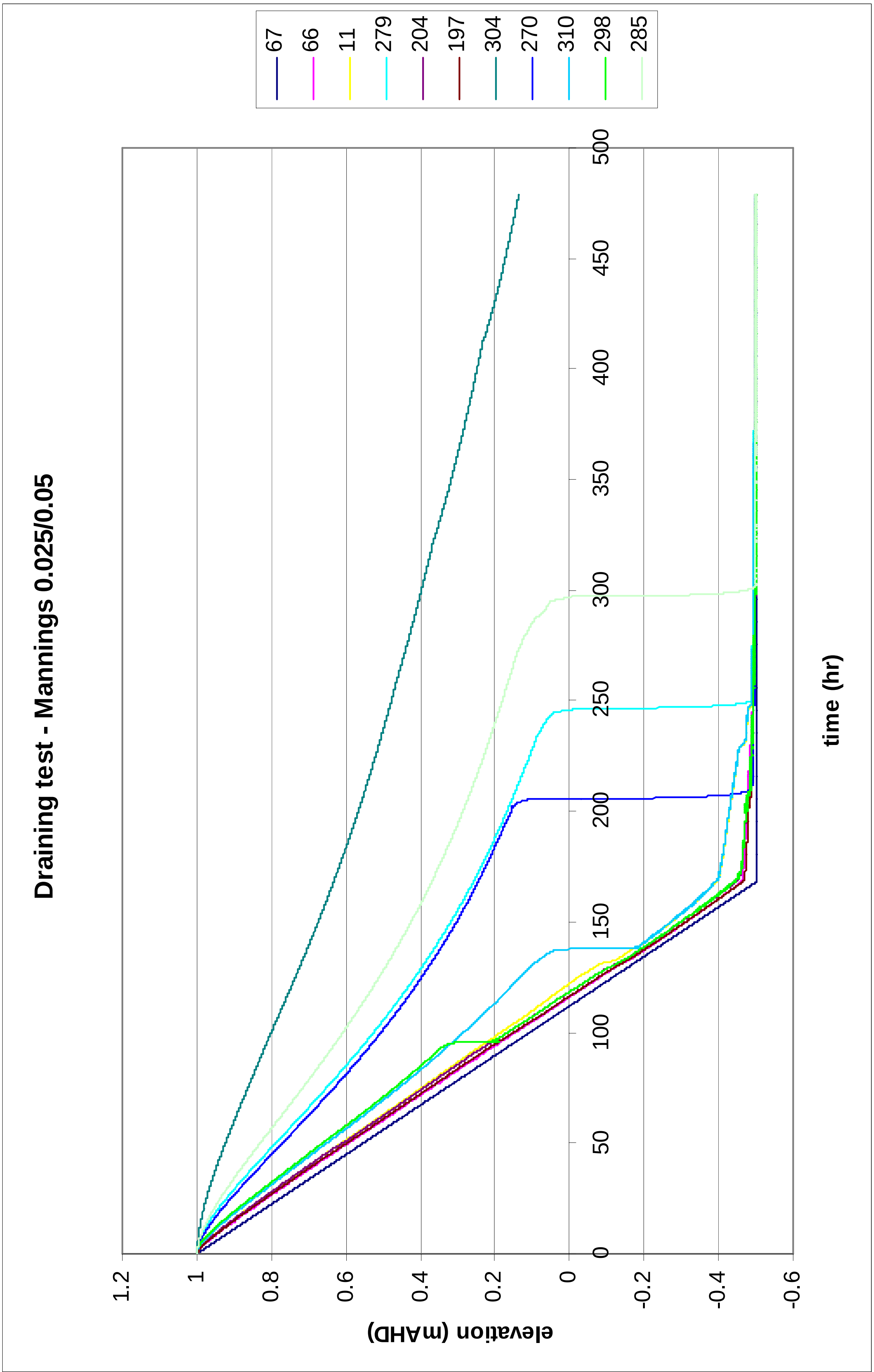
WRL Report No. 2005/28	TIDAL BOUNDARY CONDITIONS	Figure 3.3



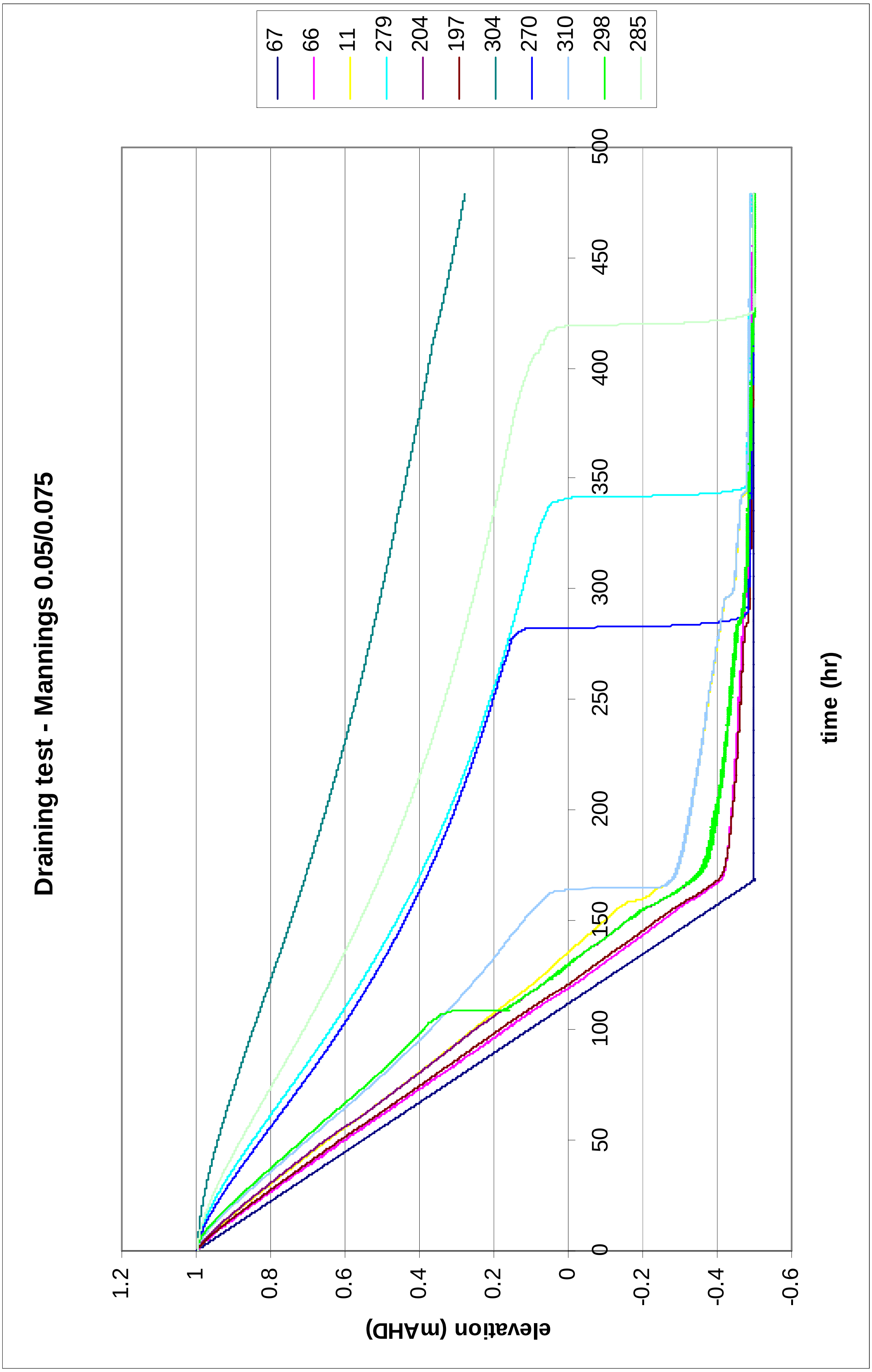
WRL Report No. 2005/28	HYDRAULIC RELATIONSHIP	Figure 3.4



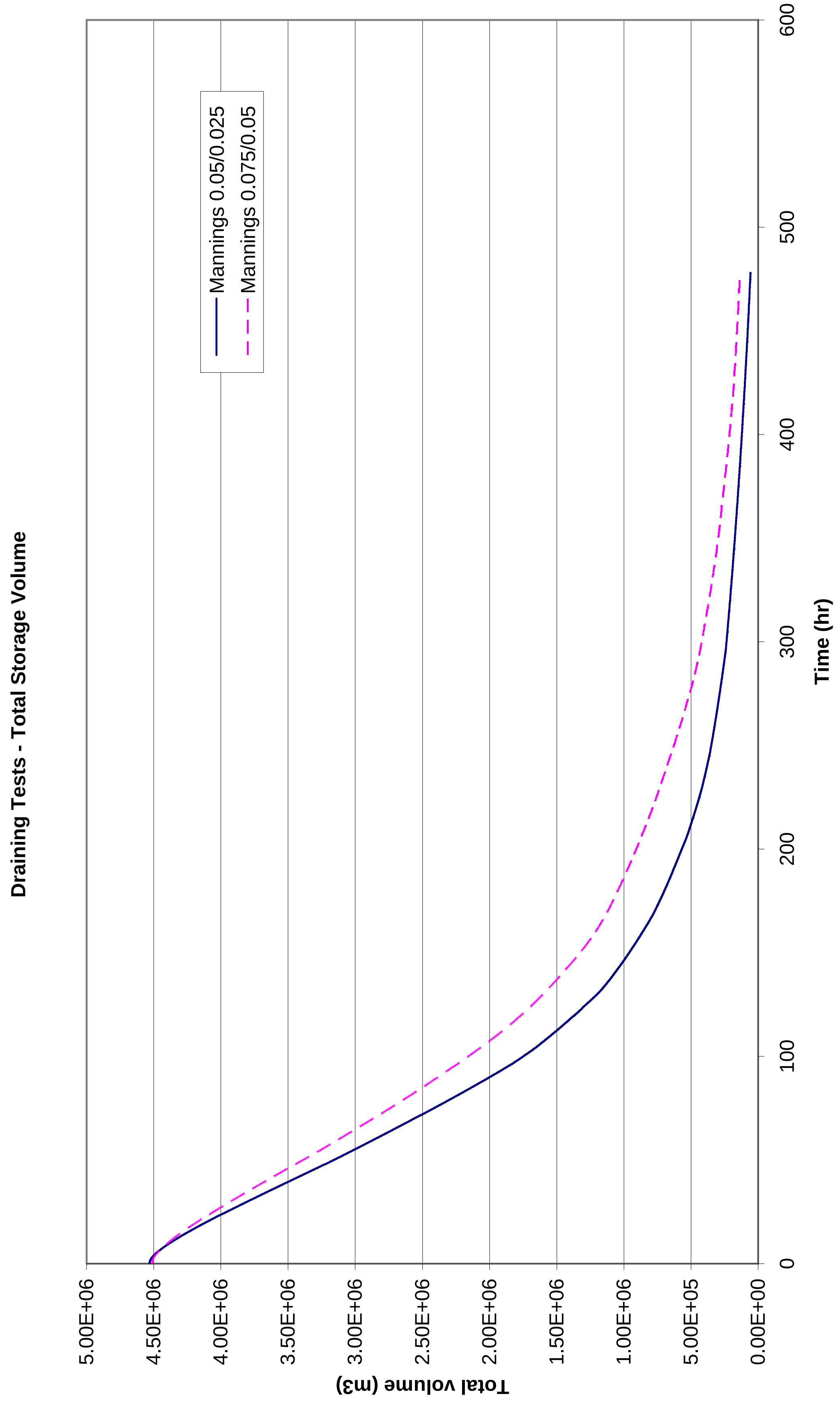
WRL Report No. 2005/28	LEVEL – VOLUME RELATIONSHIP	Figure 3.5



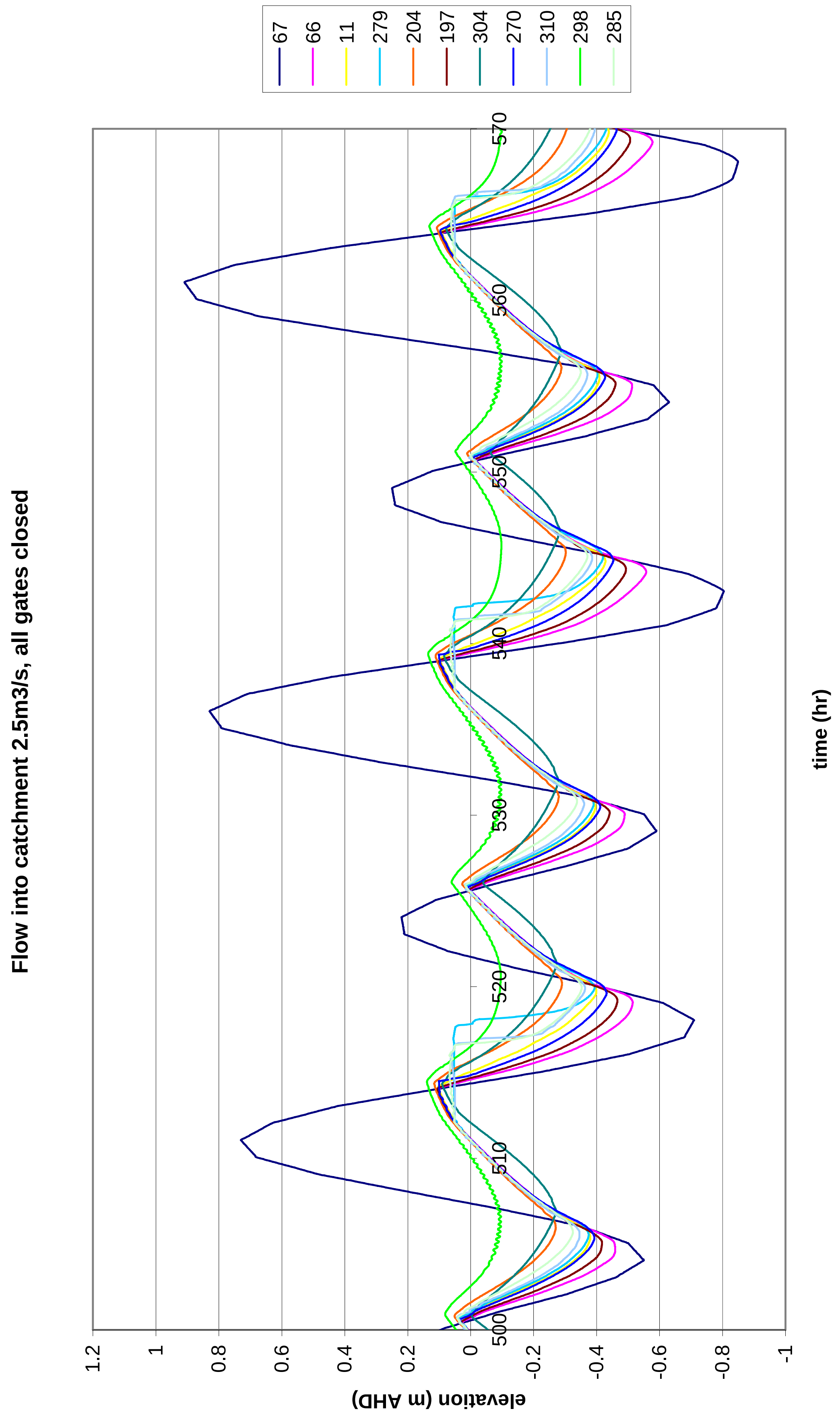
WRL Report No. 2005/28	INFLUENCE OF DRAIN ROUGHNESS MANNING'S *n* OF 0.025/ 0.05 IN MAJOR/ MINOR DRAINS	Figure 3.6



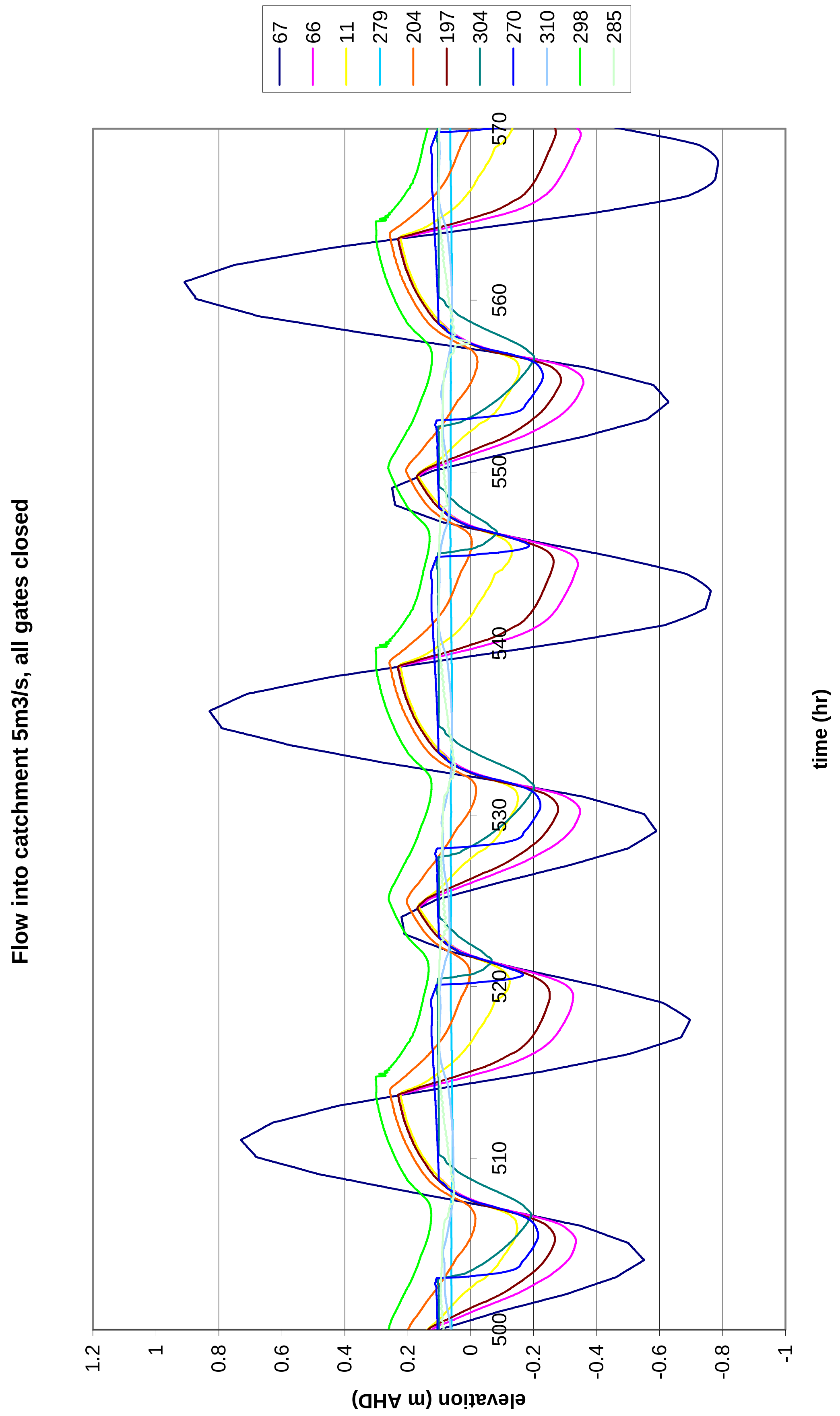
WRL Report No. 2005/28	INFLUENCE OF DRAIN ROUGHNESS MANNING'S *n* OF 0.05/ 0.075 IN MAJOR/ MINOR DRAINS	Figure 3.7



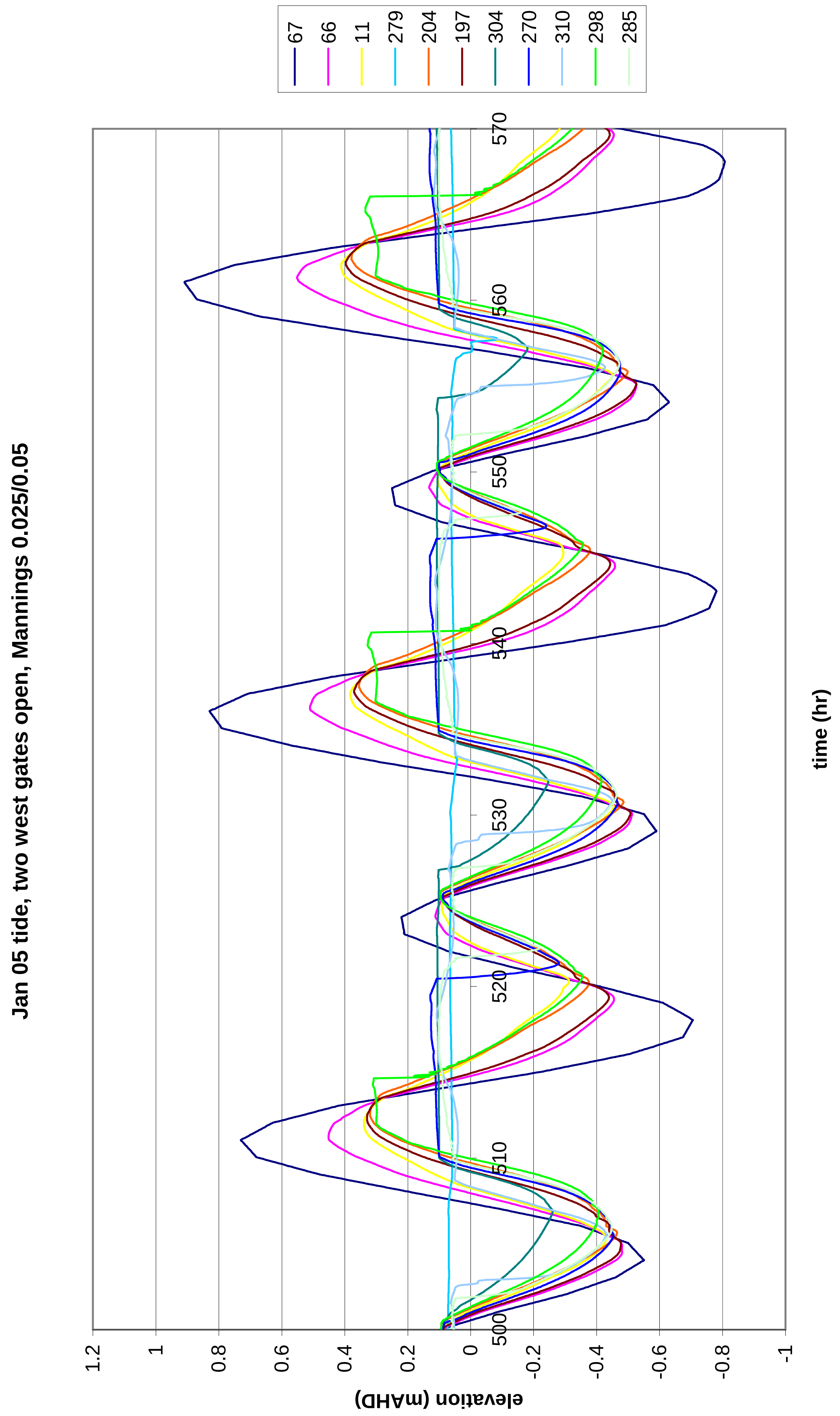
WRL Report No. 2005/28	INFLUENCE OF DRAIN ROUGHNESS COMPARISON OF TOTAL VOLUME IN STORAGE	Figure 3.8



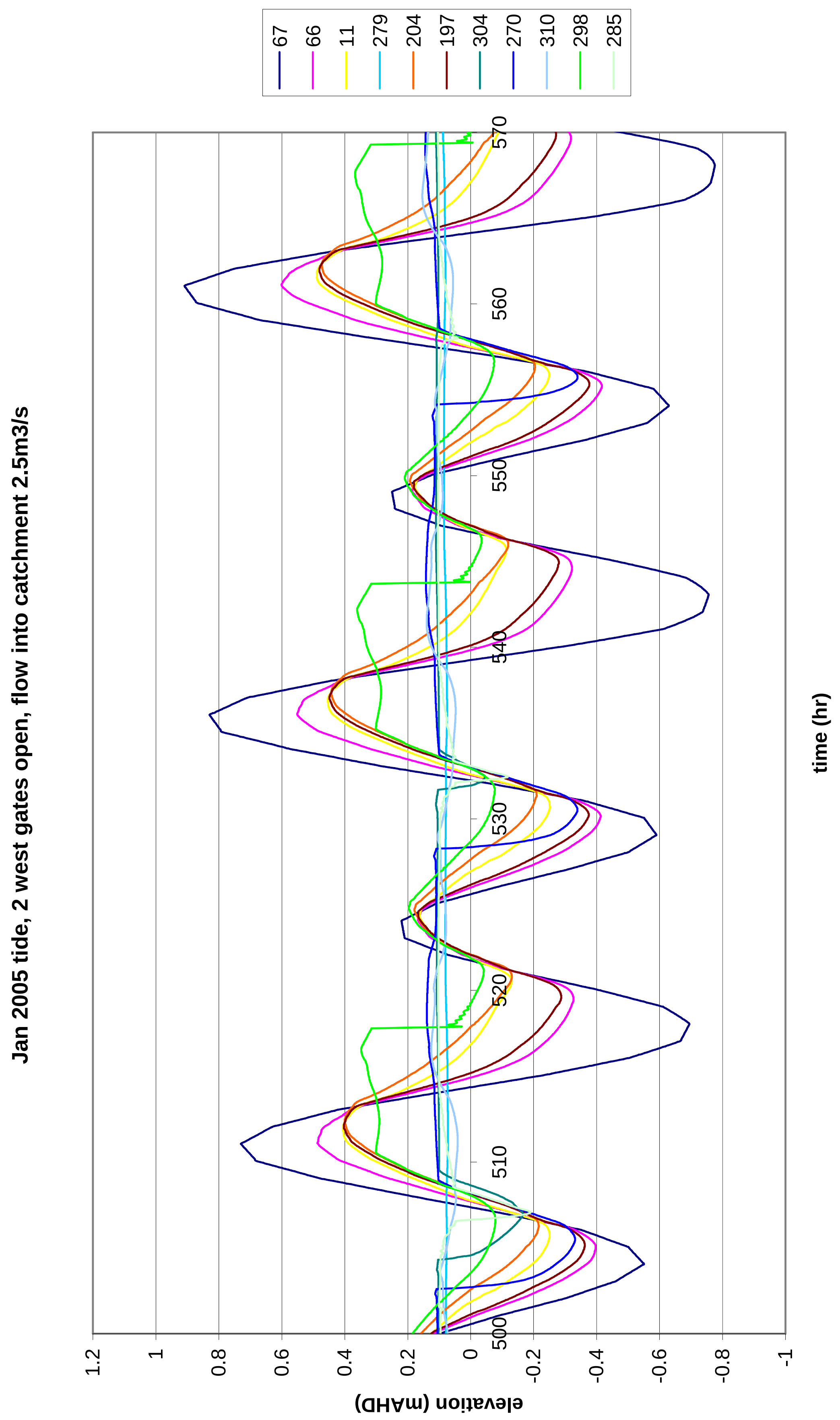
WRL Report No. 2005/28	JANUARY 2005 TIDE, ALL FLOOD GATES CLOSED, INFLOW OF 2.5 m^3s^{-1}	Figure 3.9



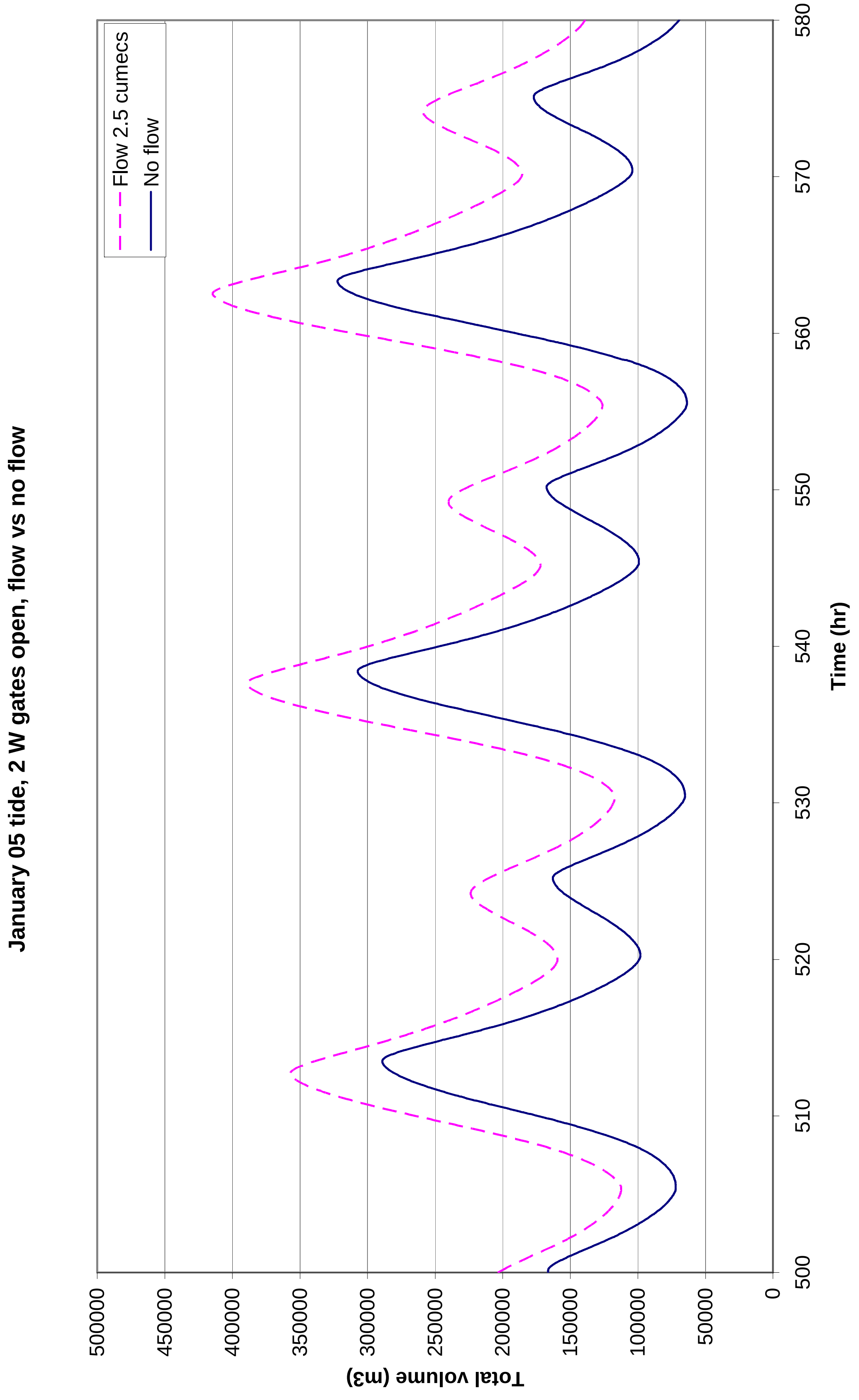
WRL Report No. 2005/28	JANUARY 2005 TIDE, ALL FLOOD GATES CLOSED, INFLOW OF 5 m^3s^{-1}	Figure 3.10



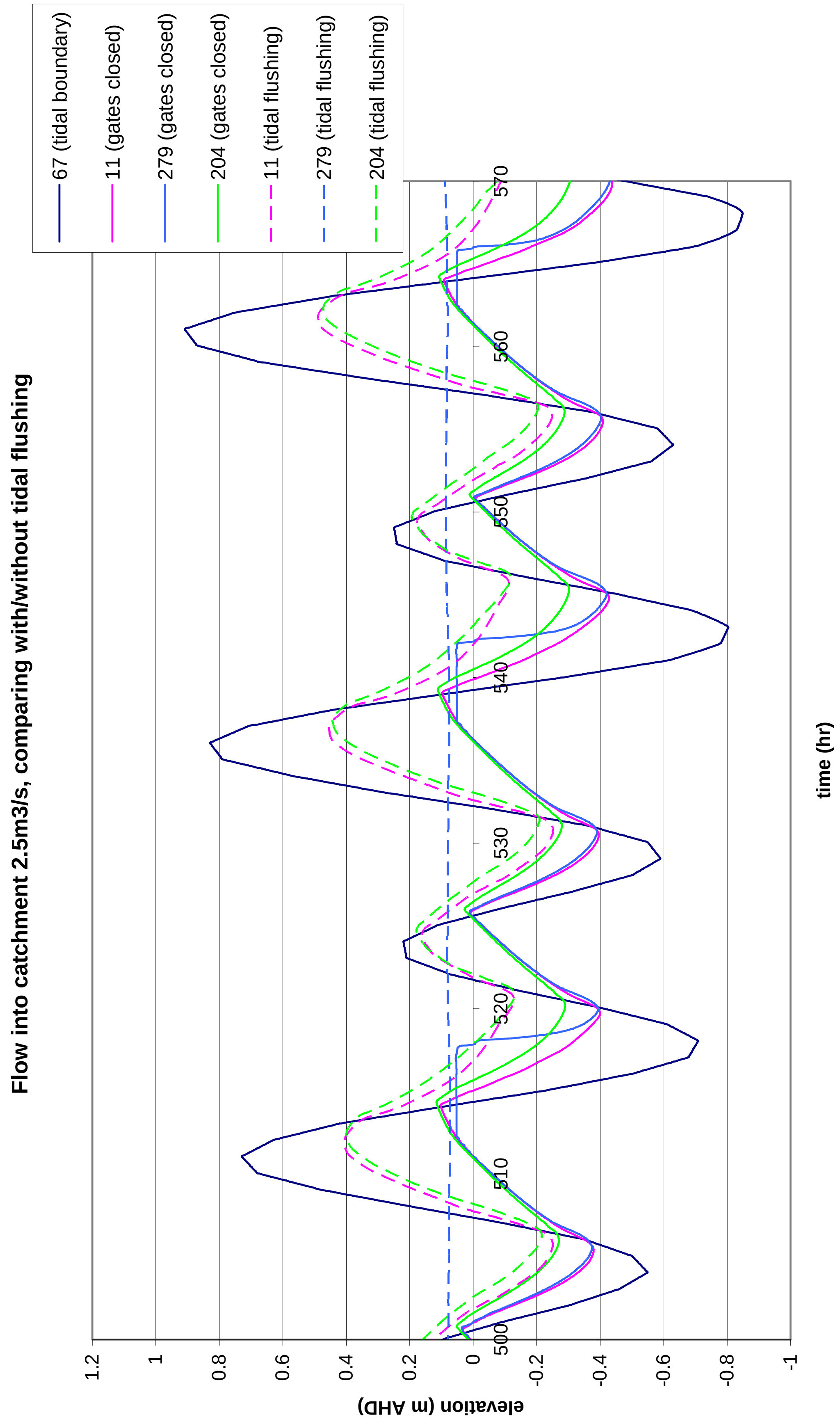
WRL Report No. 2005/28	JANUARY 2005 TIDE, TWO GATES OPEN ON THE WEST ENTRANCE, NO INFLOW TO THE CATCHMENT	Figure 3.11



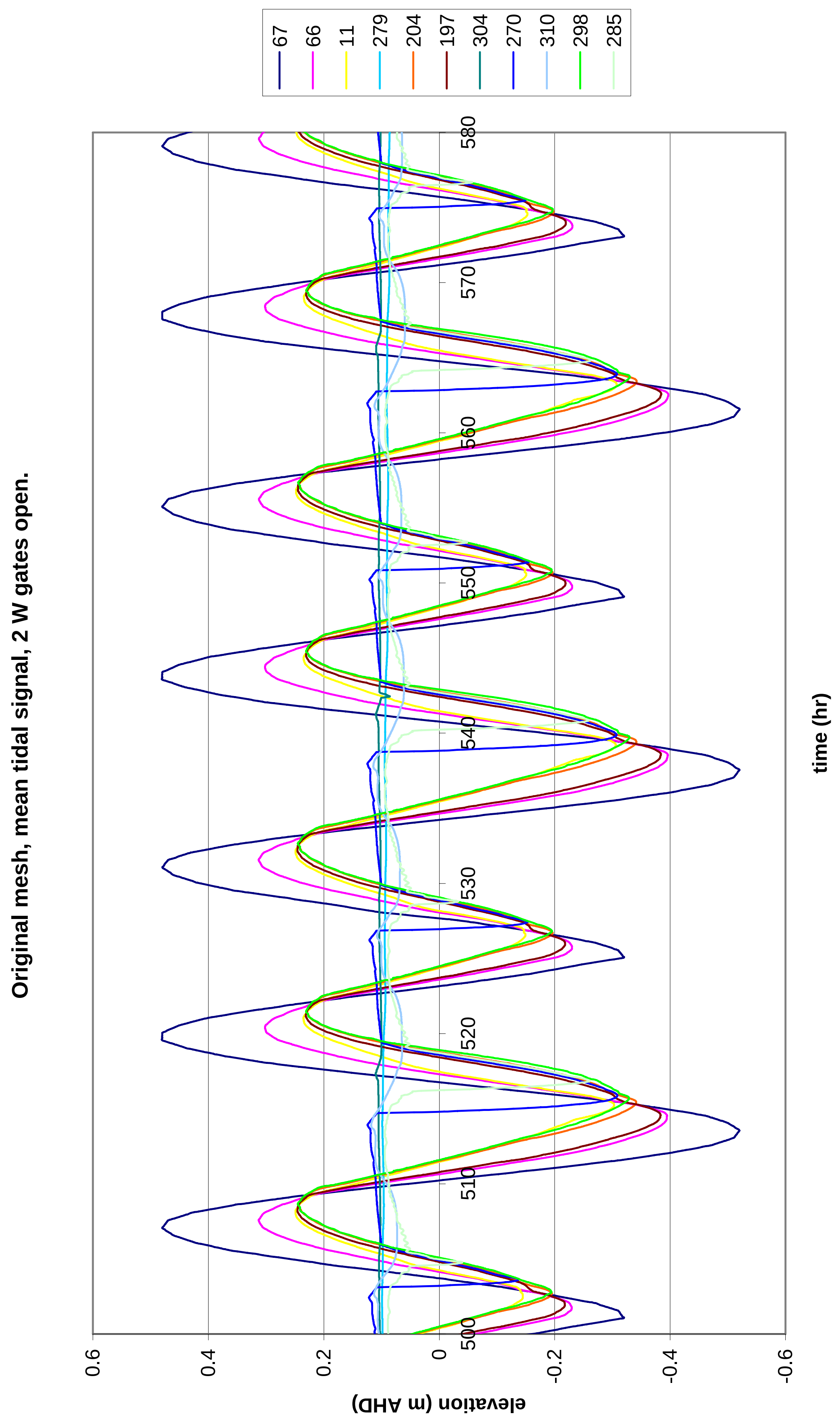
WRL Report No. 2005/28	JANUARY 2005 TIDE, TWO GATES OPEN AT THE WEST ENTRANCE, INFLOW OF 2.5 m^3s^{-1}	Figure 3.12



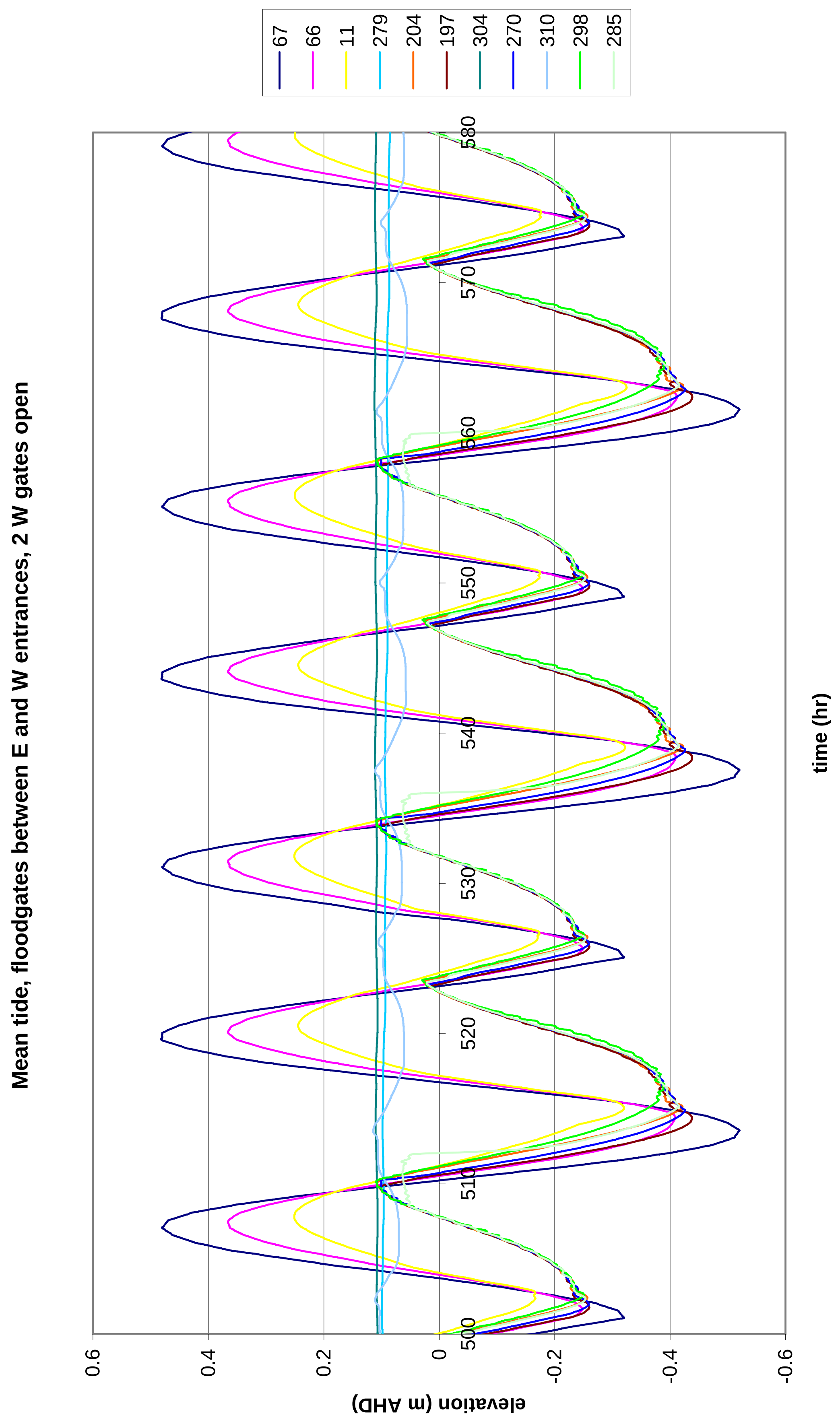
WRL Report No.2005/ 28	COMPARISON OF TOTAL VOLUME IN STORAGE TIDAL FLOODING – NO INFLOW VERSUS 2.5m^{3}s^{-1} INFLOW	Figure 3.13



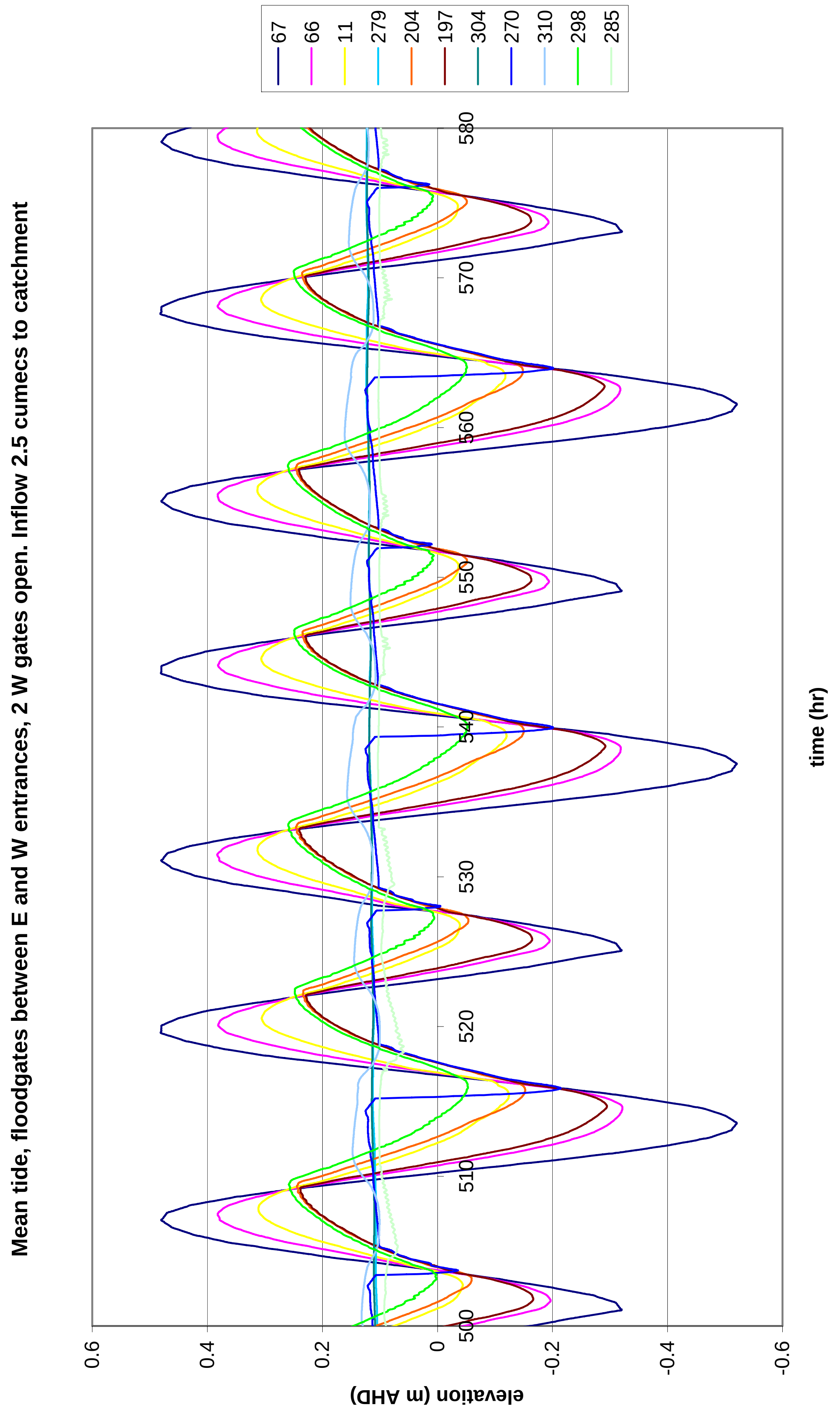
WRL Report No.2005/28	COMPARISON OF DRAIN WATER LEVELS BETWEEN EXISTING AND TIDAL FLUSHING SCENARIOS (NODES 11, 204, 279)	Figure 3.14



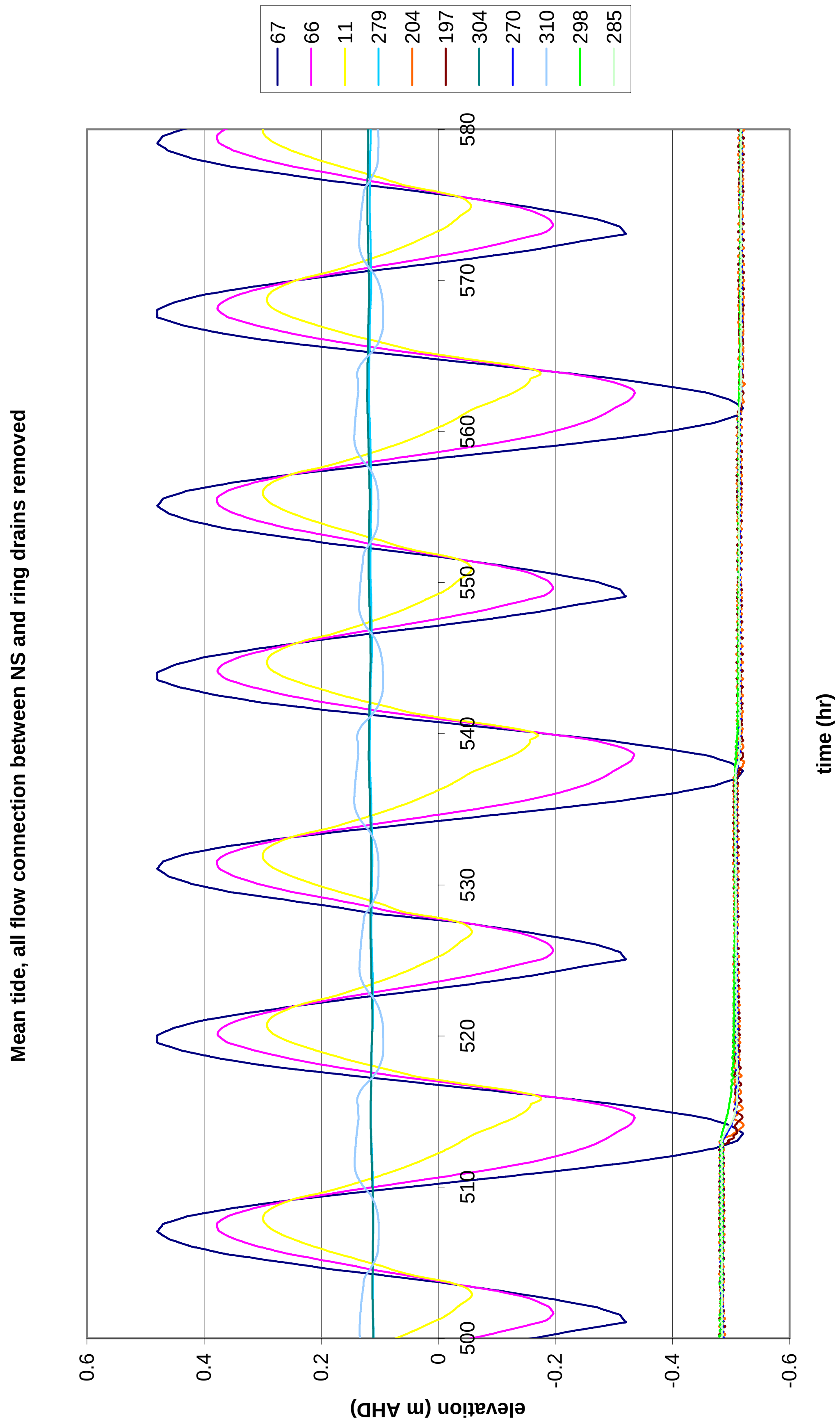
WRL Report No.2005/28	SCENARIO 1: BASESELINE CONDITIONS	Figure 4.1



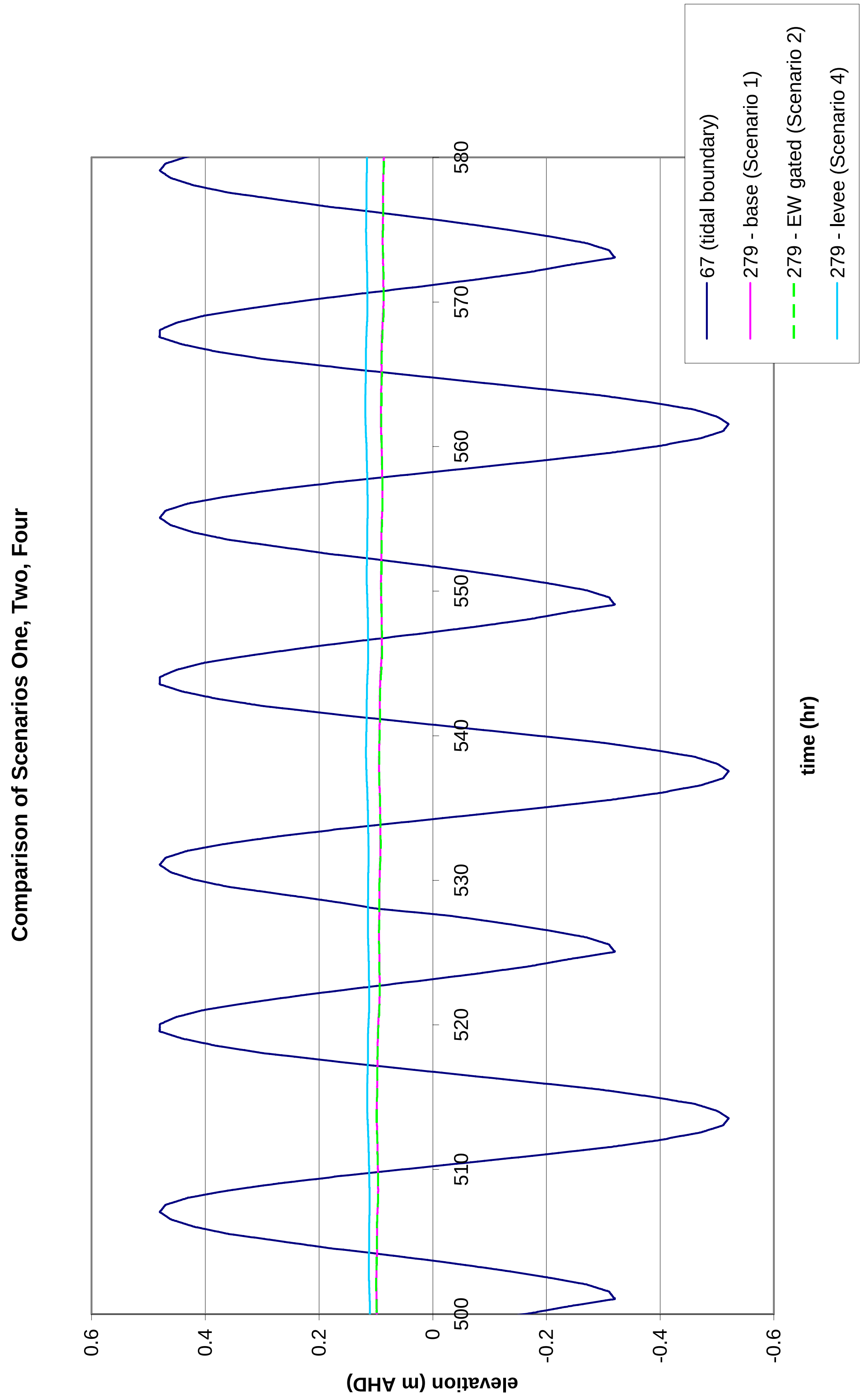
WRL Report No.2005/28	SCENARIO 2 FLOODGATES BETWEEN THE EAST AND WEST ENTRANCES, NO INFLOW TO THE CATCHMENT	Figure 4.2



WRL Report No.2005/28	SCENARIO 3 FLOODGATES BETWEEN THE EAST AND WEST ENTRANCES, INFLOW OF $2.5\ m^3s^{-1}$ TO THE CATCHMENT	Figure 4.3

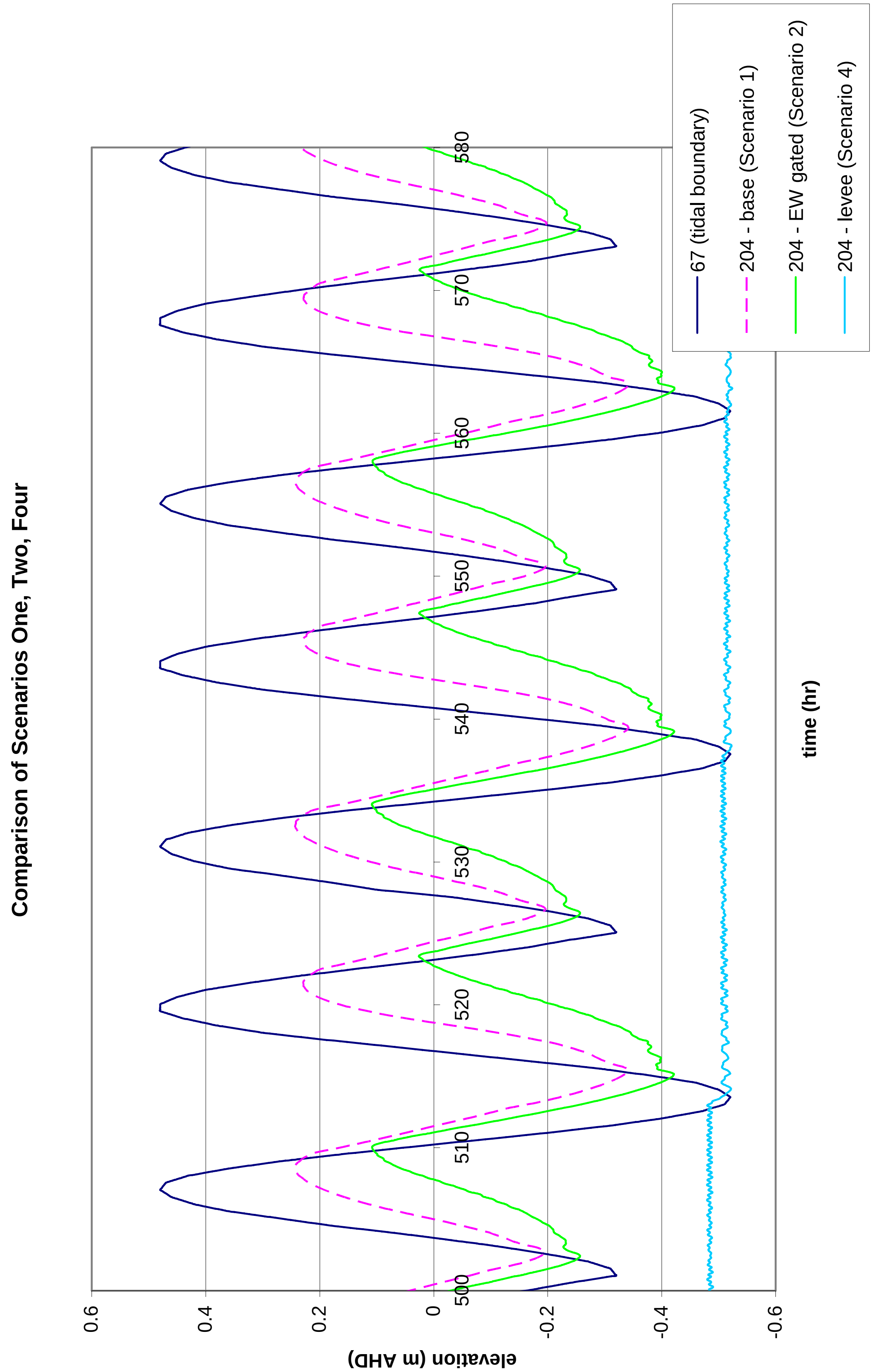


WRL Report No.2005/28	SCENARIO 4 FLOODGATES INSTALLED BETWEEN THE EAST AND WEST ENTRANCES, AND ALL FLOW CONNECTIONS BETWEEN NS AND RING DRAINS REMOVED	Figure 4.4

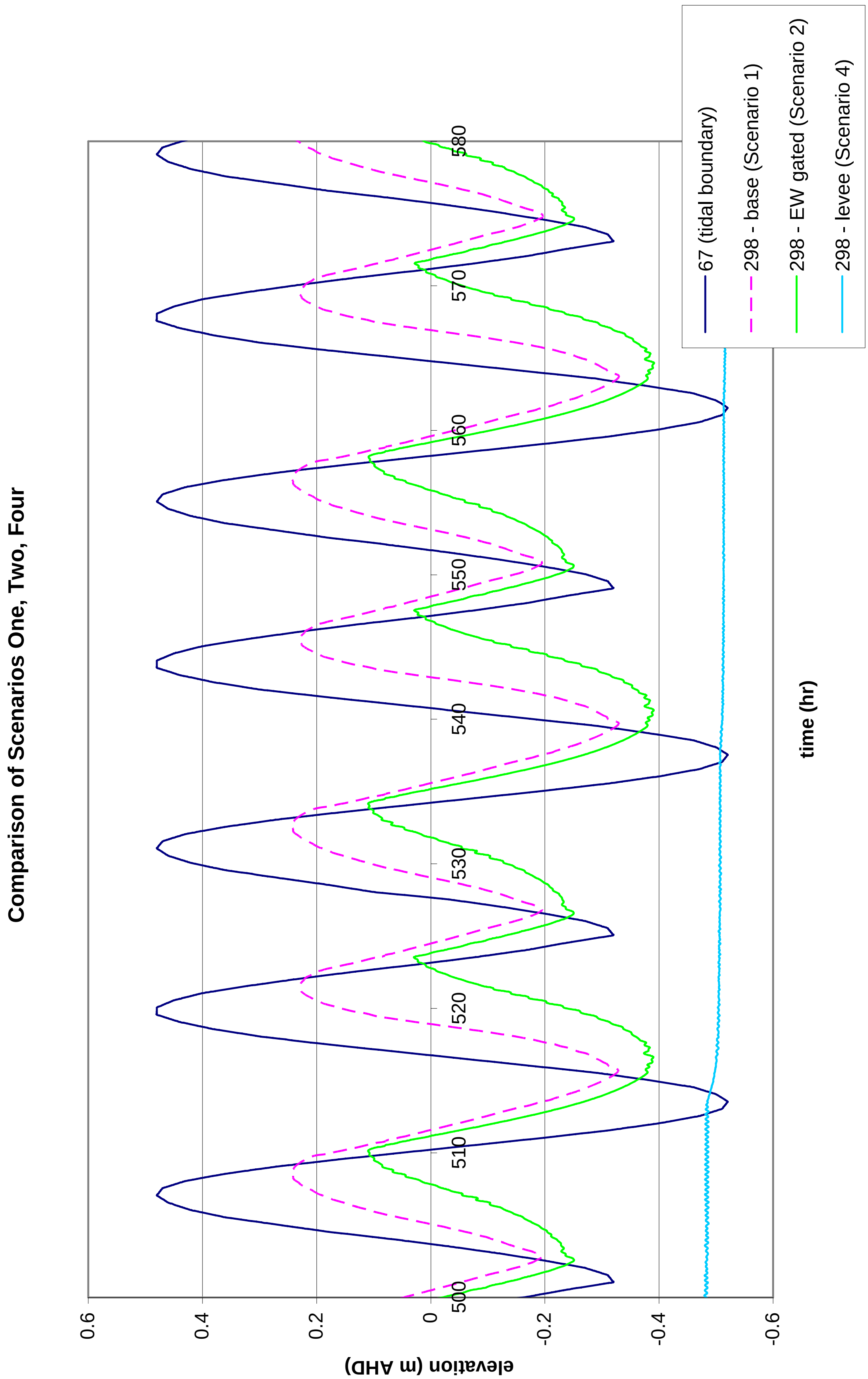


Note: The results of Scenarios 1 and 2 are overlaid

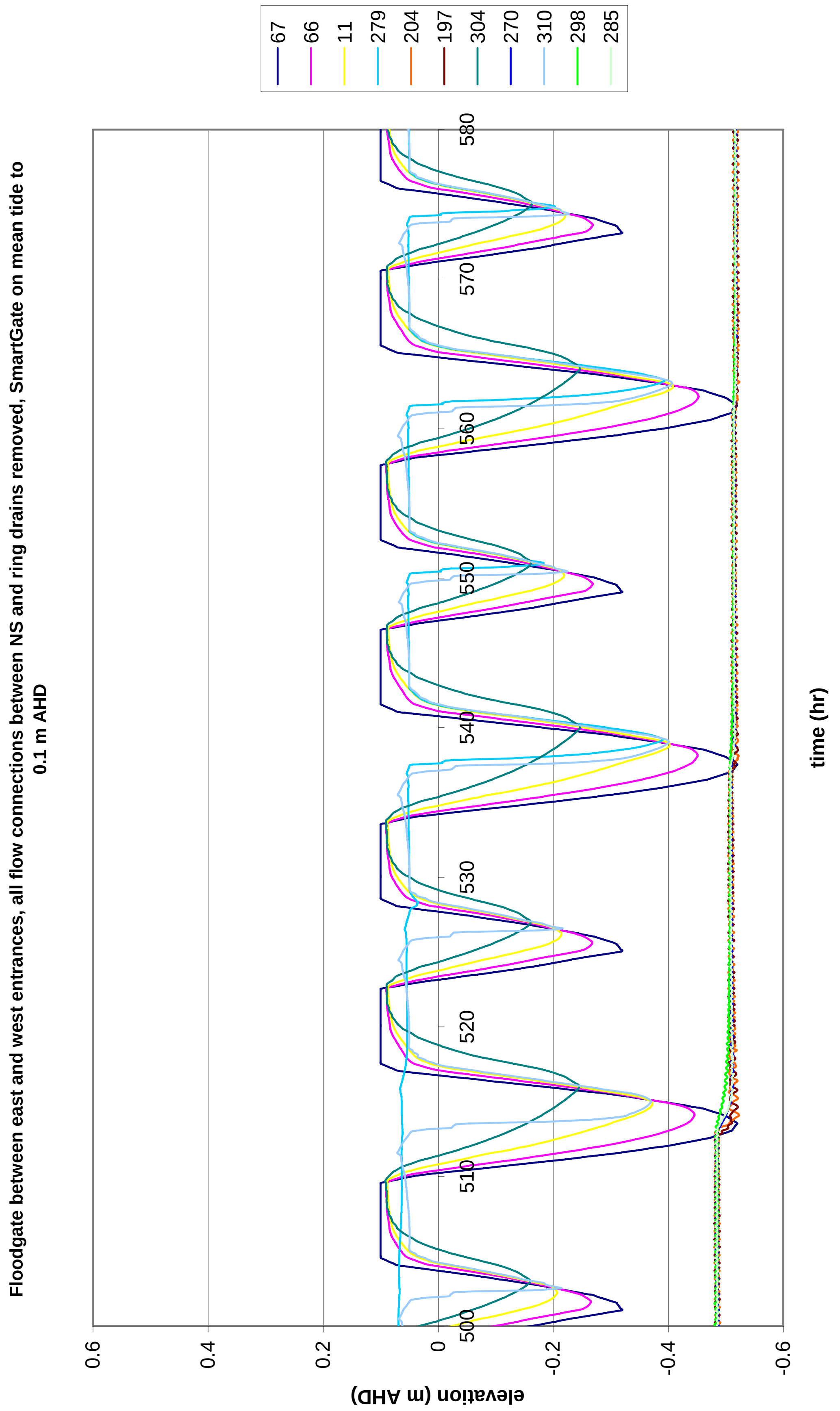
WRL Report No.2005/28	RESULTS AT NODE 279 COMPARISON OF SCENARIOS 1, 2 AND 4	Figure 4.5



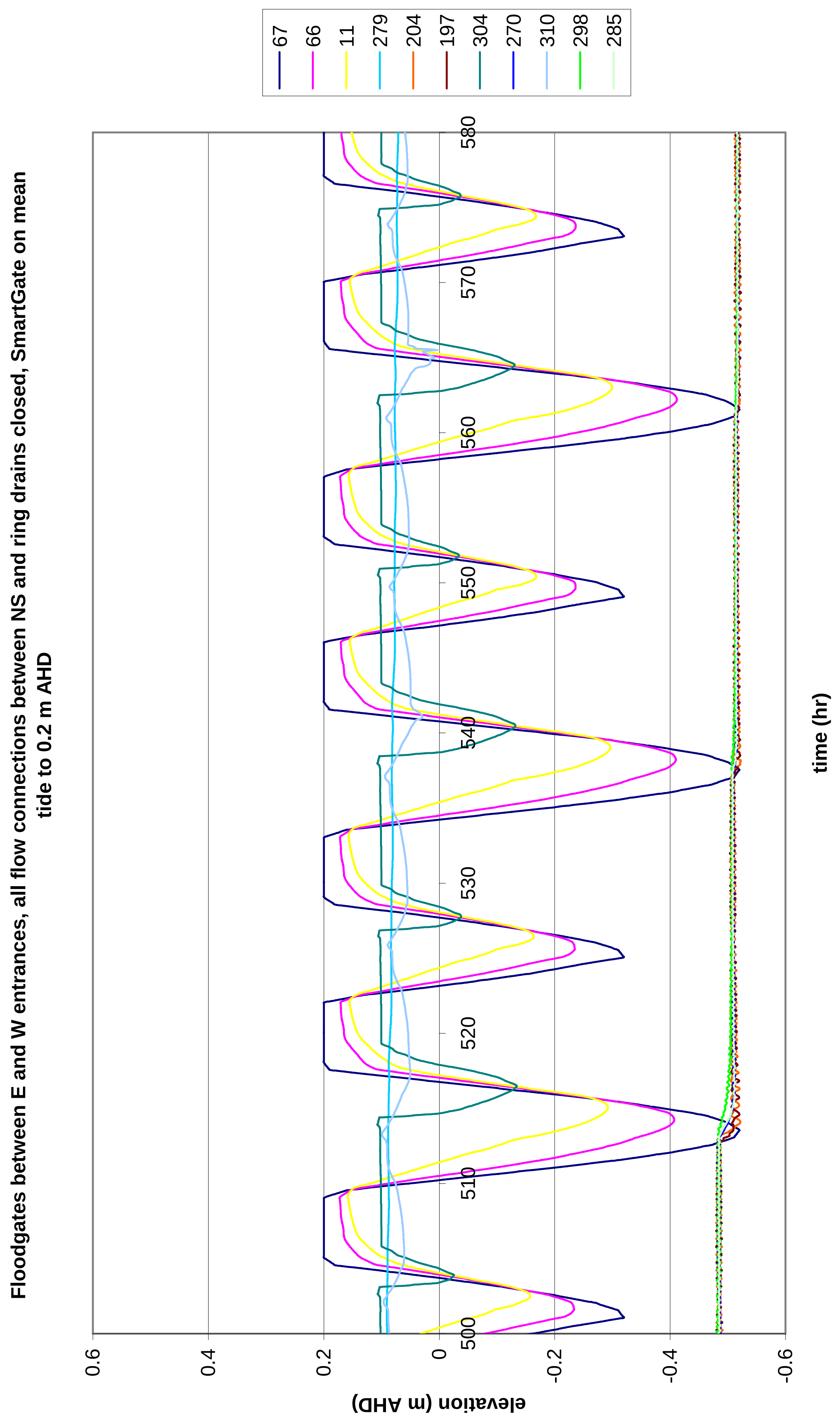
WRL Report No.2005/28	RESULTS AT NODE 204 COMPARISON OFSCENARIOS 1, 2 AND 4	Figure 4.6



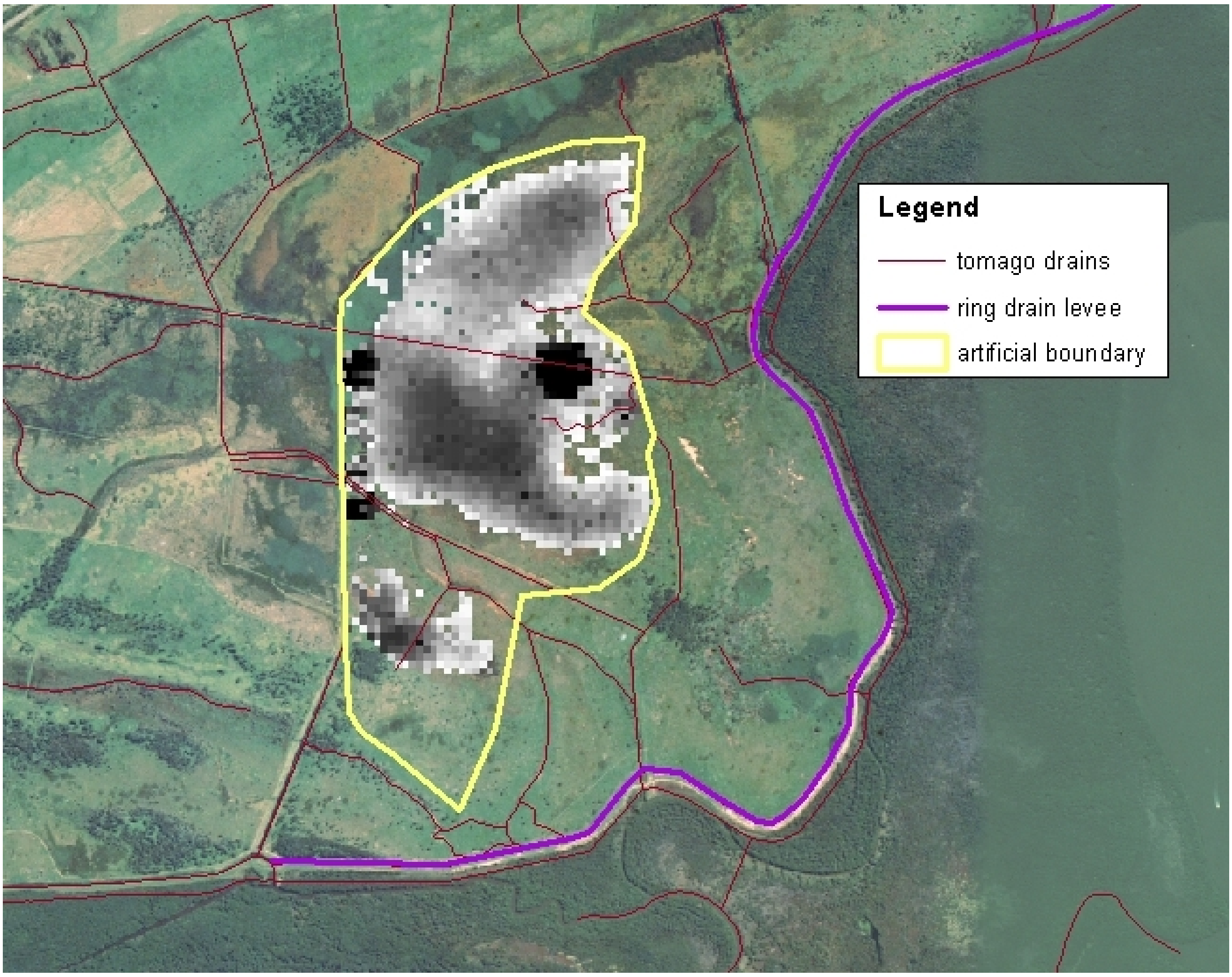
WRL Report No.2005/28	RESULTS AT NODE 298 COMPARISON OF SCENARIOS 1, 2 AND 4	Figure 4.7



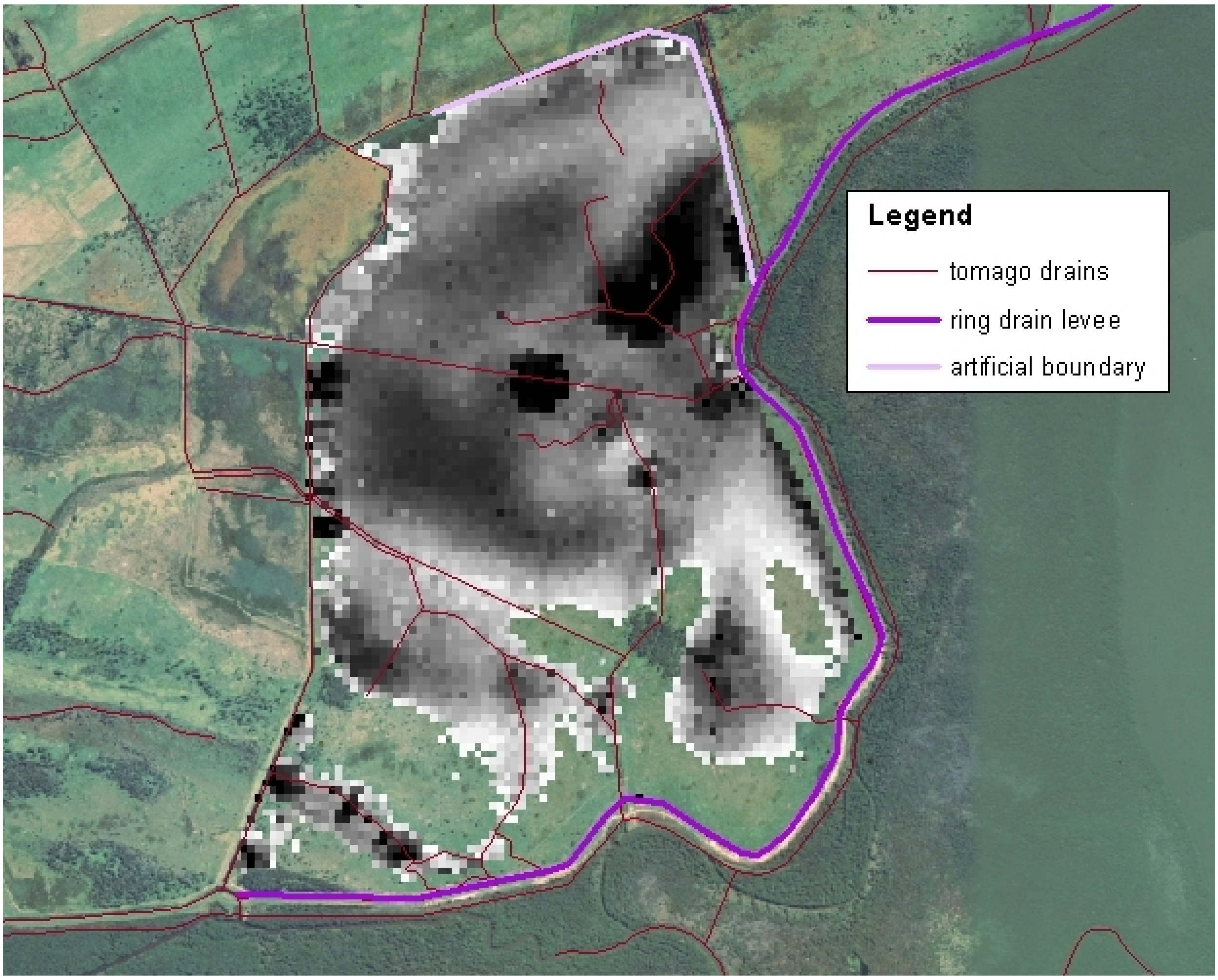
WRL Report No.2005/28	SCENARIO 5 CONTROL GATES INSTALLED TO RESTRICT THE MAXIMUM TIDAL ELEVATION TO 0.1 m AHD	Figure 4.8



WRL Report No.2005/28	SCENARIO 5: CONTROL GATES INSTALLED TO RESTRICT THE MAXIMUM TIDAL ELEVATION TO 0.2 m AHD	Figure 4.9



WRL Report No.2005/28	SCENARIO 5 LIKELY EXTENT OF SITE INUNDATION AT A WATER ELEVATION OF 0.1 m AHD	Figure 4.10



WRL Report No.2005/28	SCENARIO 5: LIKELY EXTENT OF SITE INUNDATION AT A WATER ELEVATION OF 0.2 m AHD	Figure 4.11