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ANNA BAY DRAINAGE STUDY: REDUCING ACID WATER IMPACTS VIA WATER CONTROL STRUCTURES AND TIDAL RESTORATION

by

W C Glamore and C D Wasko

Technical Report 2009/12
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1. INTRODUCTION

Anna Bay is located approximately 60 km north of Newcastle on the mid-north coast of New South Wales (NSW) in Port Stephens. The Anna Bay catchment and surrounds is presented in Figure 1. The catchment is low-lying and consists of a drainage network to facilitate removal of water from the catchment. Floodgates on the end of the drainage system prevent tidal intrusion.

The Anna Bay drainage network is presented in Figure 2. Acid Sulphate Soils (ASS) occur naturally throughout the Anna Bay catchment. Decreases in the water table due to the drainage network has led to pyrite oxidation and the subsequent generation and transport of acidic by-products into the surface water system.

In 2008, the NSW Department of Primary Industries (DPI) undertook a comprehensive site assessment and monitoring program to determine the extent of acidic sulphate soils and to assess the hydrology and elevation of the site. This assisted in identifying ASS remediation techniques (NSW Department of Primary Industries, 2008). One of the techniques suggested was the installation of a low-set tilting weir that would decrease the surface-groundwater flow gradient by maintaining higher surface water levels in the drain and hence, reduce ASS oxidation and transport. Glamore (2008) recommended a self-tilting weir that would maintain an elevated drain water level in dry conditions, but would have a reduced invert level in wet conditions. This design would meet the conflicting requirements of maintaining an elevated water level to reduce ASS oxidation and transport while ensuring the drains capability to remove surface water during rainfall events. Furthermore, a modified floodgate design was also proposed to neutralise acidic surface water prior to discharging into Port Stephens. Both the weir structure and the modified floodgate have the potential to influence the drainage characteristics of the Anna Bay catchment.

WRL was commissioned by the NSW Department of Primary Industries and Port Stephens Council to develop a numerical (i.e. computer) model of the surface water dynamics to assess the impact of the two proposed remediation actions on catchment drainage. This report outlines how this model was constructed, calibrated and subsequently used to simulate the influence of the two options (i.e. weir and modified floodgate) on the surface water dynamics. The model used available water level data during wet and dry periods to simulate surface water levels, flow distributions and velocities. The existing condition was simulated using available data. Each remediation option was then simulated to determine how the action impacted the surface water dynamics. The combined impact of both a weir and a modified floodgate was also tested. These simulations were then compared with the existing conditions.

This report is divided into 6 main sections. Following this introduction, Section 2 highlights the modelling approach undertaken. This section explains each chronological step undertaken in numerical model development from data collection and assimilation through to developing, calibrating and implementing the model to assess the two proposed remediation actions. Following this overview of the modelling approach, Section 3 presents the field data used in model implementation. Section 4 then details the numerical model itself. The implementation of the model, the boundary conditions used and model calibration are all discussed. Subsequent scenario results are presented and interpreted to assess the remediation techniques. Section 5 discusses these findings based on the results presented in Section 4. Section 6 concludes the report by highlighting key findings and recommendations.

2. MODELLING APPROACH

The following modelling approach was undertaken to ensure that the computer model would accurately represent Anna Bay's hydrology and could be used as a tool for assessing the comparative impact of the ASS remediation options. Each step discussed below is further detailed in the subsequent sections of this report.

Field Data Collection

The development of an accurate numerical model is largely dependent on adequately representing the site geometry. All field data used in the model was supplied by DPI. The data included an aerial laser survey (LiDAR) of the catchment, numerous cross-sections of the main drains in the Anna Bay drainage network, surveys of all relevant water control structures in the system, and water levels recorded at three sites along the drainage system for a period of approximately 6 weeks. Further tidal water level data from Port Stephens was used for comparison to Anna Bay water level data and was sourced from the Manly Hydraulics Laboratory (MHL).

Data Analysis and Interpretation

The water level data provided by DPI was assessed and compared to water level gauges in Port Stephens to gain a better understanding of the system. All data was assessed for its integrity and professional judgements were performed when determining the use and application of the data. A complete description of the data analysis and interpretation is provided in Section 3.

Model Development and Implementation

The MIKE Flood numerical model by DHI Software was chosen for the Anna Bay drainage model. MIKE Flood was chosen as it applies a finite difference scheme (as opposed to finite element scheme) for its numerical solution. This means that the MIKE model is guaranteed to conserve mass both locally and globally, which is essential when modelling wetting and drying, such as in the overland flow situation of the Anna Bay catchment.

MIKE Flood is a 1-D/2-D hybrid model that has the ability to dynamically link 1-D and 2-D model elements. MIKE-11 was used to model the drainage system. MIKE-11 is a 1-D flow model that simulates one-dimensional flow through a channel as well as flow through hydraulic structures such as culverts and weirs. This model is ideally suited to modelling the Anna Bay drainage system as it is dominated by one-dimensional flow. MIKE-21 (2-D) was used to model overland flow in the catchment. MIKE-21 is a 2-D flow model and is widely used in Australia for flood and inundation studies where two-dimensional flow dominates and stratification of flow is not present. MIKE-21 uses a finite difference scheme that allows cells to be either wet or dry and consequently conserves mass between elements ensuring model results are accurate. LIDAR data assisted in determining the elevation between the 1-D and 2-D model elements (i.e. bank heights).

Model Calibration and Parameterisation

Model calibration was achieved by comparing the water level data collected by DPI with the model results at the relevant location. Three separate stations were monitored, (i) downstream of the floodgate, (ii) upstream of the floodgate and (iii) at a low point approximately 2.3 kms upstream of the floodgate. The location of the floodgates and the low point is shown in Figure 2. The downstream water level was used as a boundary condition. Inflow into the model and Manning's *n* (a representation of channel roughness) were both adjusted to best represent the measured water level upstream of the floodgate and at the low point.

Modelling Scenarios

After developing and calibrating the model, simulations were performed to assess the suitability of the following ASS restoration strategies:

- A low-set tilting weir
- A modification to the floodgates that allows tidal flushing
- A combination of both options.

These strategies were assessed in dry and wet conditions. The primary goal was to determine the impact each option had on the drainage and water levels within the Anna Bay catchment (in comparison to the existing conditions).

3. FIELD DATA

All the field data used in developing and calibrating the numerical model was supplied by DPI, with further data for validating the water levels sourced from Manly Hydraulics Laboratory (MHL). This section reports analysis of the data obtained by WRL for the use in its numerical model set-up and testing.

3.1 Topography

Two data sets were provided by DPI for the construction of the geometry and topography of the 1-D and 2-D numerical models. The general topography of the Anna Bay catchment was provided via the LiDAR survey on a 2 m grid as shown in Figure 3. The geometry of the 1-D numerical model was sourced from cross-sectional profiles measured by DPI at selected locations along the drainage system. These profiles were surveyed by DPI using a laser level referenced to a benchmark with a known Australian Height Datum (AHD). The location of each profile was measured using a hand-held GPS (nominal accuracy ±5 m), with each location presented in Figure 4.

Based on the methodology used the accuracy of each level measured in the profile is likely to be within 50 mm. Note that this accuracy also applies to the control structure survey and the water level data.

3.2 Water Control Structures

All water control structures in the Anna Bay drainage system are presented in Figure 5. This does not include small structures that lead into the primary drainage system. Similar to the cross-sections surveyed by the DPI, all invert levels were surveyed using a laser level referenced to a known benchmark, with the location of each structure identified using a hand-held GPS. Table 1 presents the location, type, invert level and size of each structure as surveyed.

Table 1
Details of Control Structures in Anna Bay Drainage System

Structure Name	Location (WGS 84)	Size	Invert Level (m AHD)
Wallis Creek (Anna Bay) Floodgate	32º45'47.39"S 152º03'28.00"E	3 cells, each 1.55 m (high) × 1.8 m (wide)	SW through to NE: -0.718, -0.72, -0.728
Ferntree (1)	32º46'28.98"S 152º04'54.22"E	Box culvert: 0.6 m (high) × 3 m (wide)	0.57
Ferntree (2)	32º46'30.19"S 152º05'02.32"E	Box culvert: 0.6 m (high) × 3 m (wide)	0.798
Ferntree (3)	32º46'30.86"S 152º05'08.92"E	Box culvert: 0.6 m (high) × 3 m (wide)	0.974
Ferntree (4)	32º46'31.59"S 152º05'11.94"E	Culvert consisting of two 1.05 m diameter pipes	North to South: 1.742, 1.737
Ferntree (5)	32º46'34.06"S 152º04'53.03"E	Culvert consisting of one 0.9 m diameter pipe	1.005
Ferntree Drain Culvert	Passes under Nelson Bay Road	Culvert consisting of four 1.05 m diameter pipes	-0.22
Nelson Bay Road Culvert	32º45'57.21"S 152º04'56.46"E	2 cells, each 1.5 m (high) × 3.4 m (wide)	SW through to NW: -0.489, -0.672
Port Stephens Drive Culvert	32º45'51.08"S 152º03'59.11"E	One box cell 1.48 m (high) × 2.7 m (wide) and four 1.35 m diameter pipes	North to South: -0.705, -0.57, -0.52, -0.503, -0.497
Bennett (entering Diemar)	32º45'33.87"S 156º05'33.66"E	Culvert consisting of two 0.9 m diameter pipes	East to West: -1.14, -1.09
Bennett (entering Back Drain)	North of confluence of Back Drain and Bennett Drain	Culvert consisting of two 1.3 m diameter pipes	NW though to SE: -0.527, -0.667

3.3 Anna Bay Drain Water Level Data

Half-hourly water level data was provided by DPI for the period from 11th of August 2008 to 26th of September 2008 (a total of 47 days) for the following three sites and is presented in Figure 6:

- Immediately downstream of the floodgates
- Immediately upstream of the floodgates
- 2.3 km upstream of the floodgates at a low point immediately downstream of the Nelson Bay Road culvert located on the Main Drain (Figure 2).

Figure 6 presents these data as time series plots. As shown in Figure 6, the water level measured at the low point 2.3 km upstream of the floodgates does not fall below -0.05 m AHD or exceed 0.74 m AHD. As the water control structures downstream of this site (Table 1) are unlikely to induce these maximum and minimum levels, it was assumed that this data was incorrect. This is further illustrated in Section 4.2 where calibration of the numerical model is discussed.

3.4 Port Stephens Water Level Data

The water level for the Tomaree gauge at Port Stephens was supplied by MHL. This site is located on the southern head at the entrance to Port Stephens (Figure 1). A comparison of the record of water levels measured by DPI downstream of the floodgates with that recorded at Tomaree is presented in Figure 7. It is clear that the water level in the Anna Bay drainage system is elevated compared to the tide entering the harbour. This is typical of other estuarine tributaries and often results in the dampening of the tidal signal. More discussion on the boundary conditions is provided in Section 4.

4. NUMERICAL MODEL

As discussed in Section 2, the modelling approach applied MIKE Flood with a one-dimensional model of the drainage system dynamically linked to a two-dimensional model of the overbank (floodplain) flow area. In this section a detailed description of the model implementation, model calibration and results of all the scenarios tested are presented as proposed to mitigate and remediate the acid sulphate soils.

4.1 Model Implementation

The major drainage network in the Anna Bay catchment was simulated as a series of linked flow branches each capable of transmitting one-dimensional (1-D) flow. The flow through this system is dependent on the drain geometry and the resistance to flow (e.g. amount of vegetation) across the bed/banks of the drains. The drain geometry was sourced from the cross-sections surveyed by DPI. The 1-D network of drains requires the geometry to be specified at every point in the system. When cross-sections were not surveyed at the end of a drain, the nearest cross-section was used to represent the geometry at that point. If a cross-section was not available at a drainage junction, the model geometry was linearly interpolated from adjoining cross-sections. Figure 8 shows the outline of the 1-D mesh representing the drainage network. Each structure in the drainage network is also shown, but it is worth noting that structures at the very ends of the drainage system were not included in the numerical model as they have been represented as boundary conditions (as discussed in Section 4.2). The resistance to flow in the network is simulated using Manning's *n*, a measure of channel roughness and its resistance to flow. This was used as a calibration parameter and is discussed in Section 4.3.

The two-dimensional model topography was sourced directly from the LiDAR data. The 2 m gridded LiDAR survey was re-sampled at a 5 m resolution in ArcGIS and imported into MIKE-21. A 5 m model grid was chosen to discretise the Anna Bay catchment. This ensured computational efficiency and represented the major landform features. This grid spacing resulted in approximately 600,000 grid points with simulation times of approximately 24 hours for a 14 day simulation.

The MIKE-21 model allows for computational points outside the scope of interest to be removed from the computation through the use of a maximum simulated height. Figure 9 shows the 2-D model overlain by the 1-D model shown in Figure 8. To improve computational efficiency and minimise model instability, the low-lying points outside the modelling domain (i.e. outside the Anna Bay drainage catchment) were artificially removed in ArcGIS.

The 1-D and 2-D models were coupled at 5 m intervals to better match the 2-D resolution and improve model robustness and stability. As such, the 1-D MIKE model had computational points interpolated along its branches every 5 m to ensure a one-to-one coupling of the two models. The models were coupled using digitised bank heights from the LiDAR survey. At each point a manual elevation was selected from the LiDAR data as best representing the bank height.

The 1-D/2-D model presented in Figures 8 and 9 shows the extent of the drain network and overbank drainage system. It is recognised that some small drains that potentially fill the drainage system may not have been represented in the 1-D model. A field reconnaissance of the catchment provided an overview of all large drains (>3 m wide). Based on this reconnaissance these major drains have been included as 1-D channels. These large drains were shown to effectively simulate the in-stream drainage hydraulics during dry periods.

All drains not included in the 1-D drainage network are effectively included in the 2-D model as surface depressions. However, if these small drains were filled with water when the LiDAR survey was undertaken they may not be accurately represented in the model. By not representing the full capacity of these small drains, the model conservatively represented flow paths under wet conditions (i.e. the drainage system will drain more quickly in real world conditions). Further, since the main purpose of the model was to compare the hydrologic impact of the proposed structures versus the existing hydrologic conditions, the model geometry was deemed fit for the desired purpose.

4.2 Boundary and Initial Conditions

A tidal water level was applied at the outlet of the drainage system. For all simulations this water level was sourced from the DPI water level gauge located downstream of the floodgate. An inflow was applied at the upstream extent of each drain. The amount of inflow applied was equivalent to a base flow volume during dry periods (further discussed in Section 4.3).

An initial fixed water level condition was applied in the 1-D drains. For the dry scenarios this was equivalent to the starting water level of the tide and was equal to 0.29 m AHD. For the wet scenarios this was 0.75 m AHD, approximately equivalent to the highest water level recorded in the drain system over the period of recording. For the 2-D model, an initial water level was applied as an elevation condition across the entire modelling domain.

4.3 Model Calibration

The one-dimensional model was calibrated to two parameters; the total amount of inflow applied at the start of each drain and the model roughness coefficient, Manning's *n*. The adopted period was calibrated for the first two weeks of the data record (from the 11th to the 25th of August 2008). As little to no rainfall was recorded in this period it was deemed representative of a 'dry' period.

Initial model runs indicated that water levels in the model were sensitive to the applied inflows at the upstream boundaries. As such, the model was initially calibrated to flow by adjusting the inflow rate for each drain. A number of scenarios were run to achieve the desired maximum water level. The best match for the available date was 0.02 m^3/s. This flow was distributed to each of the drains as inflow scaled by the length of each drain. Table 2 presents the flow applied at each upstream boundary. Note that the Main Drain and Ferntree Drain both have two boundary conditions, and the flow presented in Table 2 was divided equally with half applied at each of the boundaries.

Table 2
Flow Boundaries

Drain Name	Flow (m^3/s)
Bennett	0.03
Back	0.04
Main	0.07
Ferntree	0.06

Once the inflows were determined, Manning's *n* was used to fine-tune the model calibration. This involved comparing the upstream floodgate water level to the water level measured at the low point in the system and replicating the lag present between the peaks and troughs in the two water level time series. Further, the rising limb of the water level is not constant and increasing Manning's *n* provided good agreement between field data and model results. Table 3 presents the values chosen. Note that no data was available to calibrate Manning's *n* in the 2-D section of the model and a value of 0.033 was adopted. This is consistent with the average ground cover conditions observed across the Anna Bay floodplain.

Table 3
Manning's *n*

Flow Type	Manning's *n*
1-D	0.060
2-D	0.033

Calibration results are presented in Figure 10 from the 11th to the 22nd of August 2008. The results indicate that immediately upstream of the floodgates the water level is well represented. The peaks and general trends (i.e. phasing) of the water level are well matched. The slight over-estimate of the troughs could be the result of several factors including the misrepresentation of the cross-section at the discrete point where measurement took place or a poor estimation of the tidal boundary due to relatively long intervals between recorded measurements. Nonetheless, the calibration shows good agreement with the available data and indicates that not only is the model representative of the drainage system, but the general volume of the drainage system has been reasonably represented.

The low point location (Figure 2) is not as well represented by the model, however it is not possible to accurately replicate the data provided. This is most likely related to the problems with the field data as has been discussed previously (Section 3.3). The water level peaks at this point are slightly underestimated and this could be the result of calibrating flow to the floodgate. Regardless, the modelled peaks and the rates of water level increase/decrease are generally within 50 mm of the observed data. This implies model accuracy is similar to that of the data presented in Section 3.1 (±0.05 m). The calibration confirms that the model is sufficiently accurate to assess the difference between the water level in the system with and without the ASS remediation techniques to an accuracy of 50 mm.

4.4 Model Scenarios

The modelled scenarios are summarised below and are presented in Table 4. A total of 9 different scenarios were modelled to simulate the effect of two remediation techniques. Both remediation techniques were simulated individually and in conjunction with each other. Scenarios were also performed with no remediation techniques in place to allow comparison between the current situation and that with the structures in place.

The proposed weir has been placed at the location shown in Figure 11, downstream of the confluence of Back Drain and Main Drain and upstream of the Nelson Bay Road culvert. The coordinates of the weir in MGA-56 roughly correspond to an Easting of 414 140 and

Northing of 6 374 262. The weir has been modelled as a broad crested weir across the entire width of the cross-section of Main Drain with no head wall to restrict flow above its crest. The height of the weir crest was chosen to ensure a water level height preventing the exposure of the sulfidic soil layer (NSW Department of Primary Industries, 2008). The collapsed weir height was constrained by the practical ability of construction of a tilting weir. In practice, to achieve performance of the tilting weir, the head walls would be constructed outside the existing channel extents, with a tilting mechanism operated by floats and activated as the water level in the drains responds to a storm event. Further details of similar such structures can be obtained from Rampano (2009). Each model scenario assumed that the structure was either standing or collapsed.

Representing the modified floodgate involved inserting a two-way culvert into the floodgate with a size of 200 mm wide by 200 mm tall with an invert level of -0.5 m AHD to allow tidal flushing of the system.

Table 4
Modelled Scenarios

Scenario Number	Simulation	Dry/ Wet	Modelling Approach
1	Existing site	Dry	The drainage system is simulated for 2 weeks of dry conditions with spring tides
2a	Site with weir at 0.1 m AHD	Dry	As per scenario 1 with weir at 0.1 m AHD
2b	Site with weir at 0.0 m AHD	Dry	As per scenario 1 with weir at 0.0 m AHD
3	Site with modified floodgate	Dry	As per scenario 1 with modified floodgate
4	Site with weir at 0.1 m AHD and modified floodgate	Dry	A combination of scenario 2a and 3
5	Existing site with high water level	Wet	The drainage system is simulated for 1 week of wet conditions starting with an elevated tide
6	Existing site with high water level and collapsed tilting weir at -0.3 m AHD	Wet	As per scenario 5 with weir at -0.3 m AHD
7	Existing site with high water level and modified floodgate	Wet	As per scenario 5 with modified floodgate
8	Existing site with high water level and both collapsed tilting weir and modified floodgate	Wet	A combination of scenario 6 and 7

Scenarios 1 to 4 in Table 4 are dry period simulations and have been modelled using the primary 1-D drain model as no overbank flow (flooding) occurs and any flow in the smaller channels, which are only modelled in 2-D, is inconsequential to the results. Scenarios 5 to 8 in Table 4 are wet period simulations and primarily differ from the dry period scenarios for the following three reasons:

- 2-D flow is modelled as well as 1-D flow
- The initial water level is set higher; including flooding of overbank areas
- A worst case elevated tide is used, compared to the spring tide in the dry scenario.

Results are presented for each of the scenarios as a time series of the water level at several indicative locations in the drainage system. The locations for these results are presented graphically in Figure 12 and summarised in Table 5. Each location is referenced by an ID consisting of an abbreviation of the drain name and its chainage along the system. Note that the points selected are not at the extreme upper reaches of the system as the rise in the bed level becomes dominant and there is no measureable impact related to the remediation technique upstream of these locations.

Table 5
Location of Dry Scenario Results

Location ID	Nearest Drain Profile	Drain Name	Location Chainage[1] (m)	Reason for Selection
Ben-1300	BenD1	Bennett	1300	Representative of water level in Bennet Drain
BD-1000	BD5	Back	1000	Representative of water level in Back Drain
MD-1580	MD16	Main	1580	Immediately upstream of the floodgates
MD-4200	MD7	Main	4200	Immediately upstream of weir and downstream of confluence of Main Drain, Back Drain and Ferntree Drain
MD-6550	MD1	Main	6550	Representative of water level in upper reaches of Main Drain

[1] Note that the chainage is measured from the upstream end of each drain, except for Main Drain where the chainage is measured from the Port Stephens outlet.

4.4.1 Dry Scenarios

The dry period scenarios were modelled from 11th of August to the 25th of August 2008 to simulate a fortnight of dry conditions. The boundary conditions were used as described above with the model driven by the water level measured downstream of the floodgate over the period modelled (applied at the outlet to the drainage system).

Figures 13 to 17 present the results for the dry period scenarios at the five locations described in Table 5. Each figure indicates the bank heights of the nearest cross-section profile surveyed. The same vertical scale has been applied to all figures so comparison can be made between locations. The results are presented from the 12th of August.

The simulated water level in Bennett Drain for all five dry scenarios (see Table 4) is presented in Figure 13, with the nearest surveyed bank heights (profile BenD1) also drawn. For existing site conditions the modelled water level is shown to fluctuate between approximately +0.05 m AHD and -0.20 m AHD. The water level rises as water flows into the system from the surrounding catchment (surface and groundwater) and then recedes as the tide falls and the floodgates open.

In addition to the existing condition, each scenario testing the remediation techniques is also plotted. Scenario 2a, plotted as blue triangles, shows an almost constant water level at approximately 0.14 m AHD. This water level is a consequence of the weir being placed in the system at 0.1 m AHD. Note the water level remains higher than the level of the weir as baseflow is entering the system upstream of the weir. Scenario 2b is plotted in purple triangles and shows the resulting simulation of a weir set at 0.0 m AHD. The water level for this case never falls below 0.04 m AHD and, although oscillating slightly as a result of the lower weir height remains relatively constant. Scenario 3, shown in red crosses, presents the simulated water level in Bennett's drain as a result of opening the floodgates via a 200 mm × 200 mm culvert. Note that for this case the water level is slightly elevated as a result, however, this increase is no more than 0.05 m. Note also the culvert has no adverse effect on the rate of drainage as the trough in the water level is not elevated compared to the existing case. The final dry scenario, shown on Figure 13, is drawn in green diamonds and simulates a combination of the two remediation techniques (weir and open floodgate). The culvert and the weir cause the water level to occasionally elevate above the water level modelled for the weir only case, however, as a whole remains relatively constant at the previously modelled height of 0.14 m AHD. All the modelled heights stay well within the limits of the bank heights which were surveyed at 1.1 m AHD and 0.55 m AHD, respectively.

Figure 14 presents the modelled water level in Back Drain upstream of the proposed weir. The water level in Back Drain behaves very similarly to Bennett's Drain (shown in Figure 13). As a result of the weir, the modelled water levels are relatively constant; 0.14 m AHD for the 0.1 m AHD weir and 0.04 m AHD for the 0.0 m AHD weir, respectively. Again a culvert in the floodgates makes little difference elevating the water level no more than 0.05 m AHD. When both remediation techniques are in place the culvert is the dominant control structure in the system with limited water level fluctuations.

The model results demonstrate that the water downstream of the proposed weir behaves significantly differently to upstream of the weir. Figure 15 presents the modelled water level just upstream of the floodgate. As this is further downstream in the network the water level fluctuates significantly for Scenario 1, for the existing site, approximately +0.1 m AHD to -0.3 m AHD. The modelled peak water level at this location is increased for each of the remediation techniques, the greatest increase occurring when both remediation techniques are in place, and the smallest when just the culvert is in place. The maximum peak water level is increased by approximately 0.1 m, with the minimum water level relatively unaffected. This indicates that in dry conditions the drainage rate is likely to be unaffected by the remediation options tested.

Figures 16 and 17 present the modelled water level in the Main Drain, just upstream of the proposed weir and in the upper reaches of the Main Drain. The simulated water level is almost identical to those presented in Figures 13 and 14 at locations upstream of the proposed weir. Note again that the water level remains relatively constant above the weir height (when the proposed weir is in place), and the open floodgate allows the water levels to fluctuate slightly higher than previously detailed. With either or both the remediation techniques in place, the water level remains significantly below the bank height.

4.4.2 Wet Scenarios

The wet period scenarios were modelled for a period of one week using a recorded tidal water level gauge that was elevated by rainfall to ensure that drainage of the system represented wet conditions. The boundary condition used for the water level at the outlet of the drainage system was measured downstream of the floodgate from the 6th of September to the 13th of September 2008 (see Figure 6). The initial water level for all the wet scenarios was 0.75 m AHD. This was approximately the highest water level recorded in the data provided. It was found that it was not necessary to run the model for more than one week as by that stage the water level had drained to a lower level that was completely controlled by the boundary conditions applied. Note that the inflows applied to the system were identical to those in the dry scenarios, as it was assumed that the groundwater inflows were unlikely to vary significantly over a duration of one week after a storm event. This assumption may underestimate the flow that would have occurred into the system based on the elevated downstream levels, however, it is not possible to ascertain these inflows accurately as some of the inflow will be from water seeping out of the ground and some will be overland flow that has been included in the 2-D model. Distinguishing between these flows, the volume of the flows, and how they vary in time is beyond the scope of this investigation. The applied boundary conditions were, however, considered adequate for the primary purpose of providing a comparison of the network under the influence of the different proposed remediation techniques.

Scenario 5 is the first of the wet scenarios modelled. It represents the existing site with an initially high floodplain water level. The system is allowed to drain under the influence of the external tide for a period of 7 days. The upper panel of Figure 18 presents the initial water level. This is not necessarily the entire area that would be inundated if a wet event occurred, but it does show the depth of water throughout Anna Bay if a water height of 0.75 m AHD was measured uniformly across the site. This water depth is the initial water depth used for all the wet scenarios.

As a benchmark, it has been assumed that the key pasture crops in the area are able to survive for five days before they are significantly harmed by inundation. Therefore the resultant water depth after 5 days of model simulation for the existing site is presented in the lower panel of Figure 18. As discussed in Section 4.1 this is not the exact water depth expected at the site for several reasons. First, the LiDAR survey does not penetrate water, so any depths presented in portions of the model simulated only in 2-D will be depths above the standing water that was present when the LiDAR survey took place. Second, the LiDAR survey is spaced on a 2 m grid, and the finite difference grid used in the 2-D model used a 5 m spacing. The consequence of this change in resolution is that some small drains or depressions which form the critical path for surface water conveyance and drainage may not have been represented in the LiDAR, or in the model. Regardless of these assumptions, Figure 18 is representative of the areas and amount of drainage that could be expected after 5 days, if a high initial water level was observed across the Anna Bay catchment. As shown, the majority of the water has left the drainage system after 5 days. Any water remaining is mainly due to the inability of water to drain out of the system as it is isolated in unconnected depressions. It can be expected that this remaining water would likely infiltrate into the soil in a 5 day period and this is not represented in this model.

As the model simulates the general trend of runoff from the Anna Bay floodplain, it allows for comparison between different remediation techniques. Figures 19 to 23 present the water level for all four wet scenarios (Scenarios 5 to 8) at the locations described in Table 5. The first 12 hours from each scenario are omitted as the model is stabilising at this time. For all the locations presented the difference between the water level observed in the drains is negligible. The water level is slightly elevated due to the presence of the modified floodgate, but with the elevated tide and weir collapsed below the water level, the weir has little to no effect on water level.

After 5 days there is no appreciable difference between the water depth for Scenarios 6, 7 and 8 as compared to Scenario 5 in which the existing site was modelled and hence inundation maps for the remediation techniques are not presented for every case. If those maps were to be presented they would appear identical to the bottom panel of Figure 18. The only differences observed are ±0.025 m which is within the accuracy of the model and data provided. It can be observed that after 5 days, none of the remediation techniques have affected the drainage efficiency of the Anna Bay floodplain. This is expected, as the weir is below the water surface, and the modified floodgate may aid drainage and allow additional flow out of the system.

5. DISCUSSION OF RESULTS

The aim of the remediation techniques is twofold. First, a self-tilting weir would ensure that during dry conditions an elevated water level is maintained. This would reduce the exposure of ASS to atmospheric oxygen and decrease acid transport from the groundwater. Second, a modified floodgate will allow some tidal water to enter the system and neutralise acidic surface water. Both actions have been designed to minimise the impact on water levels particularly during wet periods.

Currently the drainage network is controlled by the floodgate both in dry and wet conditions. The floodgates prevent tidal inflows into the drainage network from downstream. However, water entering the drainage network from upstream of the floodgate due to surface and groundwater flows are allowed to flow out towards Port Stephens as the tide falls. The dry scenarios show that the installation of a tilting weir will result in slightly elevated water levels upstream of the weir to a height just above the weir crest. Installation of a modified floodgate results in slightly higher peaks in water level in the network. Both the remediation measures hence perform as required; slightly elevating the water level in the upper reaches of the network and allowing a small amount of the tidal prism to enter the system. As a result of the remediation measures being installed, the system, though still being governed by the floodgates, will now have two additional controls introduced; the weir and the modified floodgate.

A wet event has been represented by setting an initially high water level and allowing the system to drain under the influence of the elevated tidal signal. There is no appreciable difference after five days in the amount of surface water remaining in the Anna Bay catchment between the existing site and when the remediation techniques are in place. This suggests that after 5 days the amount of water drained from the overland areas of the Anna Bay catchments is not affected by the remediation techniques. Sensitivity analysis was performed on the possibility of the weir not collapsing to the modelled height of -0.3 m AHD, however, no change was observed for the overland drainage. Hence in a wet event the weir and modified floodgate do not control the behaviour of the drainage network.

The Anna Bay drainage network is designed to efficiently transport water in a wet event from Anna Bay to Port Stephens. As assessed, these simulated remediation techniques have no appreciable affect on the capacity of the drainage network during wet periods.

In calibrating and simulating the Anna Bay network some additional key observations have been made. First, it does not appear that any of the structures in the system are choking the flow and preventing efficient drainage of the system. The time series presented for the wet scenarios show no appreciable flow restrictions through the system. This would suggest that increasing the size of any of the culverts in the system would be unlikely to have a significant impact on the drainage of the system, especially over the time frame of several days. However, as headloss is directly related to velocity, flow constrictions may occur if higher velocities are encountered.

Second, in reference to Figure 7, it is evident that the tidal signal is elevated compared to that observed in Port Stephens. This suggests that although the drains are cleared upstream of the floodgate no dredging is performed downstream of the floodgate. Although drain clearing decreases the resistance to flow, if the water level at the floodgate was lowered a greater improvement in drainage could be expected. The water levels in Figure 7 suggest that there is sediment in the drains between the floodgate and the outlet to Port Stephens. This is resulting in an elevated tidal signal preventing the floodgate from working to maximum efficiency. If such a measure was to be pursued then the effectiveness of the Anna Bay drainage network could be significantly enhanced.

6. CONCLUSIONS

The Water Research Laboratory was commissioned by the New South Wales Department of Primary Industries to develop a numerical model of the surface water dynamics to assess the impact of proposed remediation actions for the Anna Bay site. A coupled 1-D model representing the primary drainage network, and 2-D model representing overland flow and minor drains, was developed using the MIKE Flood. The model was developed using survey data provided by DPI, which included a LiDAR survey of the entire catchment and drain profiles surveyed at several locations throughout the drain network. The model was calibrated to water level measured over a two week period of little to no rainfall.

Several scenarios were simulated in the numerical model, including base case scenarios to establish a set of typical operating conditions for the drainage network in both dry and wet periods. The remediation techniques were then implemented and the model was rerun to investigate the effect of these measures on the drainage network.

In dry conditions the proposed weir results in slightly elevated water levels upstream, while in wet conditions this weir has no effect on the overland drainage of Anna Bay catchment after 5 days. The proposed modified floodgates allow tidal water to enter the system, and this results in a higher peak water level in the system than naturally occurs in dry times. However, this increase is relatively small, in the order of centimetres, with the largest increase of approximately 10 cm occurring at the floodgates. In wet events, after 5 days the same amount of water has drained off from the overland system as when the floodgate was not modified.

It is recommended that if the efficiency of the Anna Bay drainage network to convey water from Anna Bay to Port Stephens is a concern, cleaning of the drain leading from the floodgate to Port Stephens should be investigated. Clearing of this portion of the system may result in more efficient drainage but it is also likely to increase acid production and transport unless the weir and floodgate are installed.

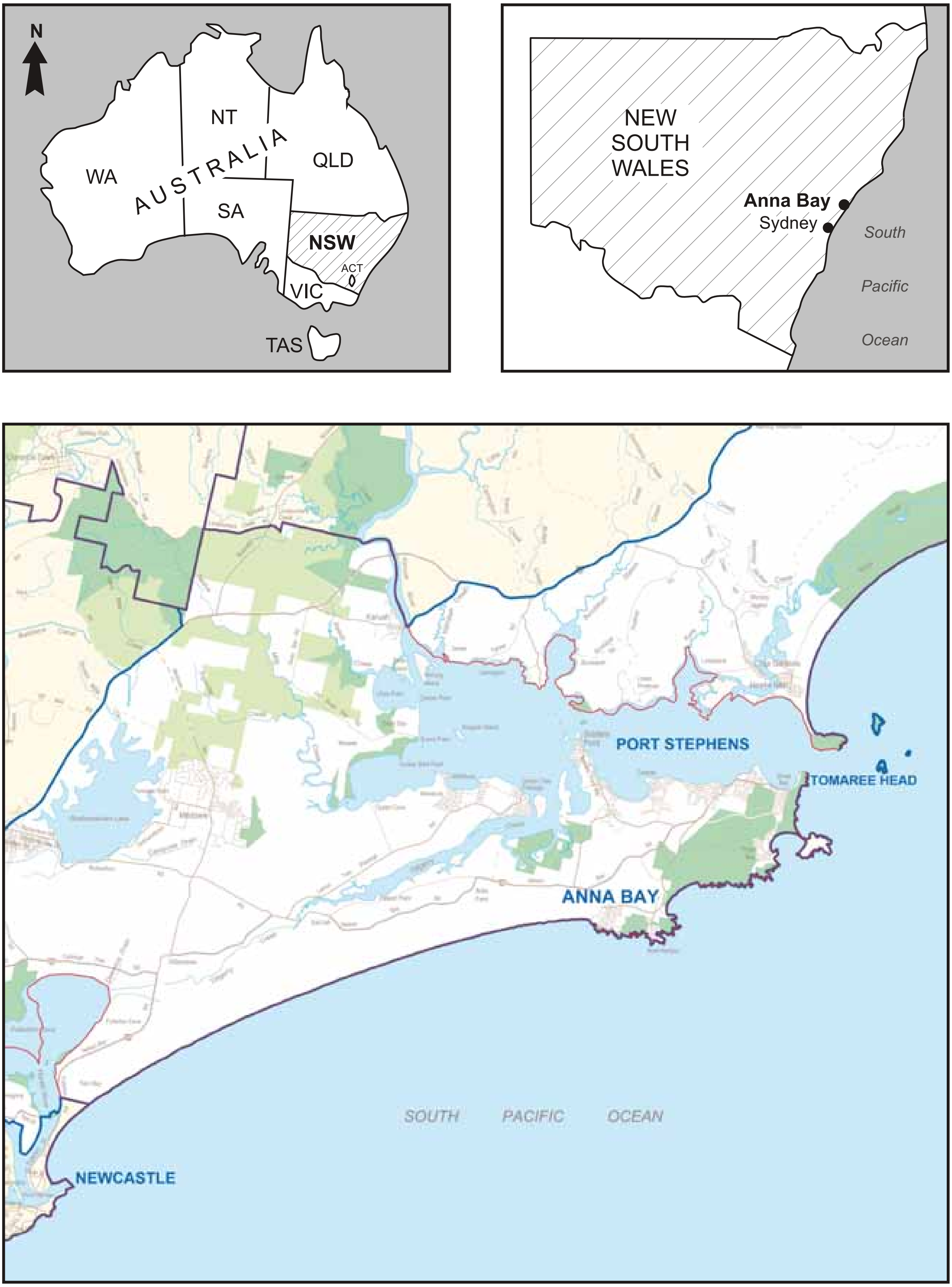
The work presented in this report presents the surface water impacts of implementing remediation works at the Anna Bay site. Water quality modelling scenarios using either PHREEQC or the water quality module MIKE AD can be considered to further optimise the possible remediation works by modelling the buffering capacity of possible salt intrusion. If additional information on the hydrology of Anna Bay was to become available it would be strongly recommended that the scenarios presented in this report be repeated using the available data.

7. REFERENCES

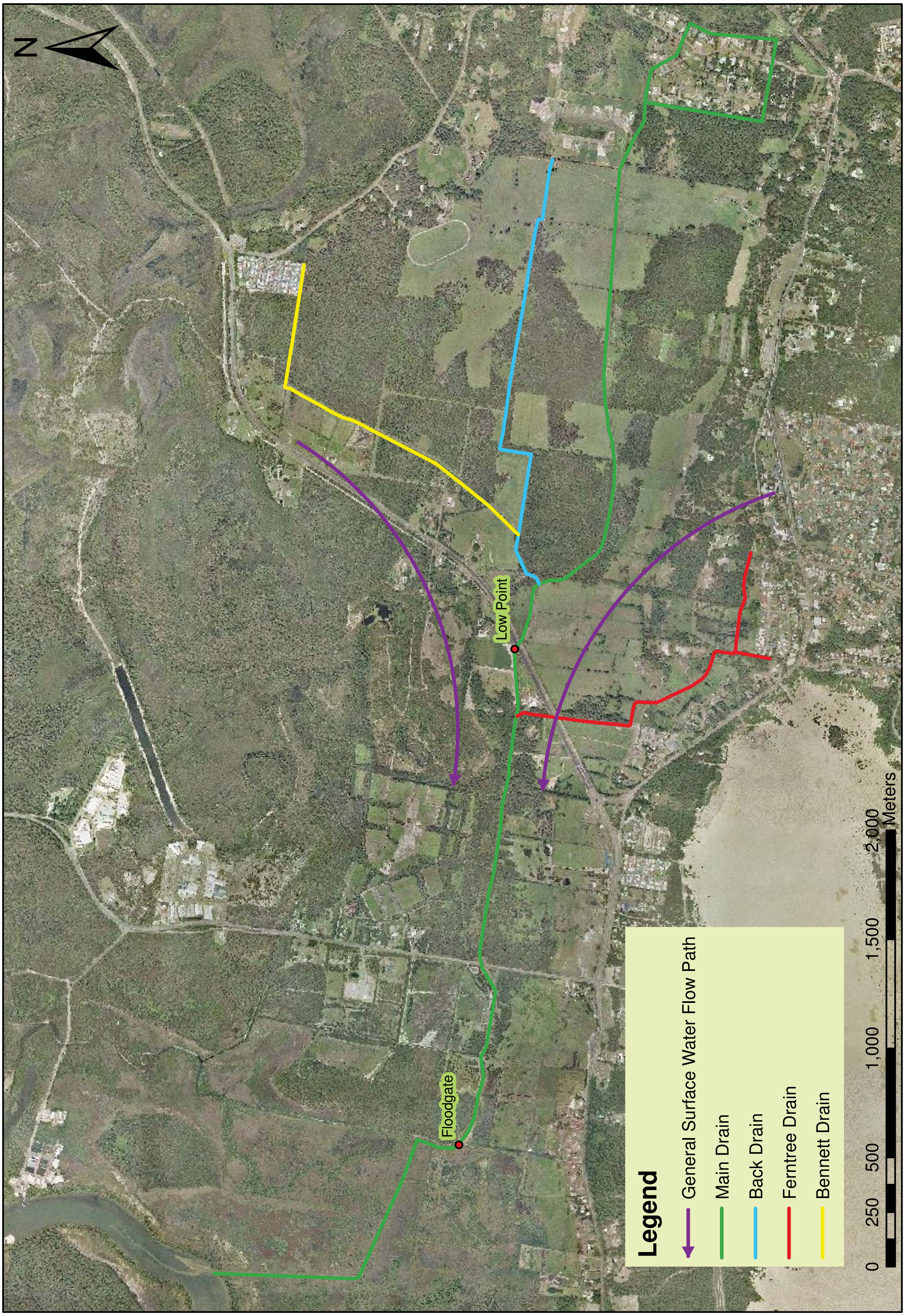
Glamore, W C (2008), “Preliminary Assessment of Anna Bay ASS Remediation Plan”. *WRL Letter Report* No 08087.01 wcg.20081021.

NSW Department of Primary Industries (2008), “Acid Sulfate Soils Assessment in the Anna Bay Catchment”. Report to the Hunter-Central Rivers Catchment Management Authority. Department of Primary Industries (Aquatic Habitat Rehabilitation), Port Stephens.

Rampano, B (2009), “Water Control Structures: Designs for natural resource management on coastal floodplains”. NSW Department of Industry and Investment (Aquatic Habitat Rehabilitation), Port Stephens.



WRL Report No. 2009/12	LOCATION OF ANNA BAY CATCHMENT	Figure 1 08087.01.cdr



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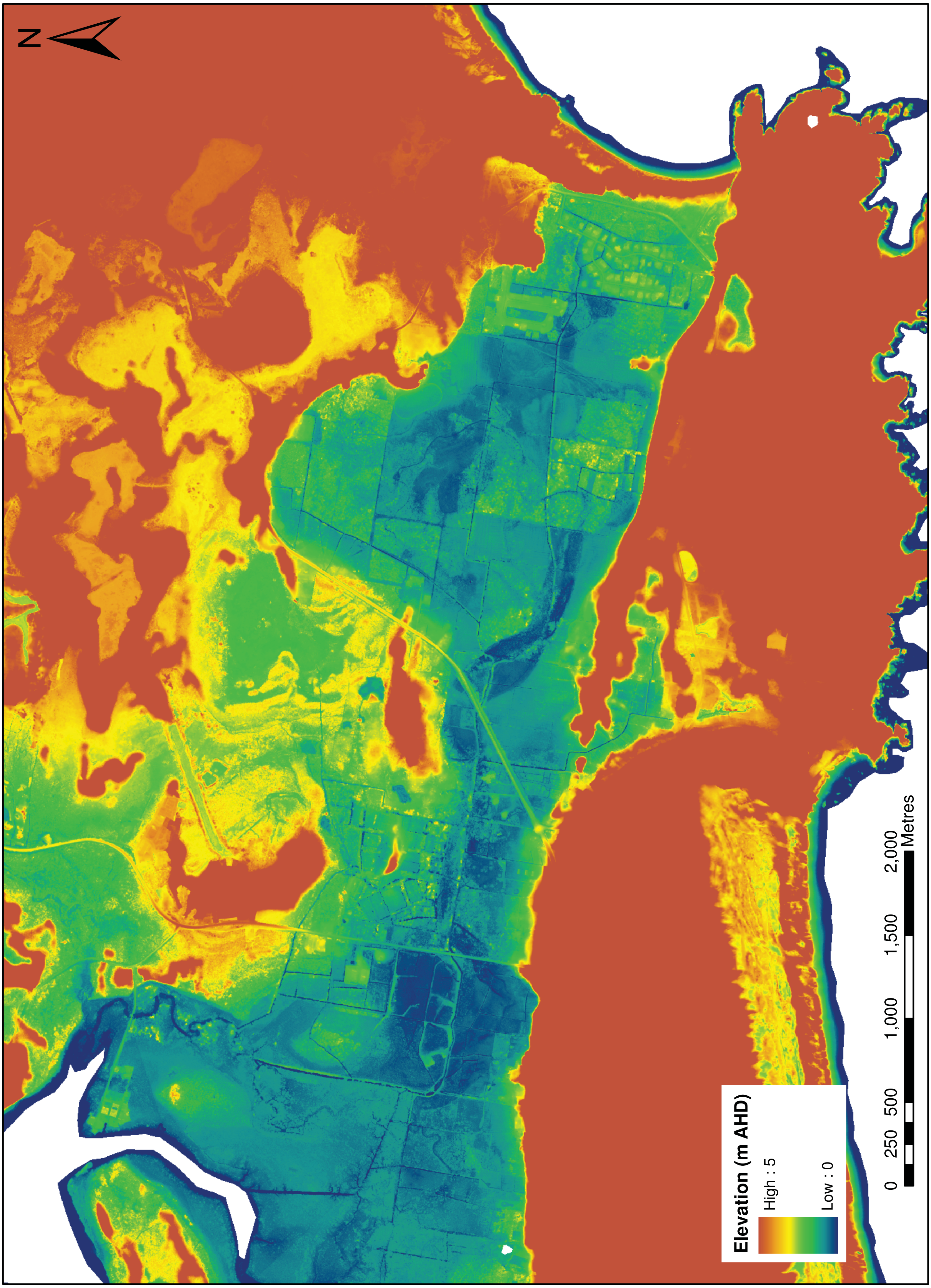
ANNA BAY DRAINAGE NETWORK

Figure

2

Figure02.mxd



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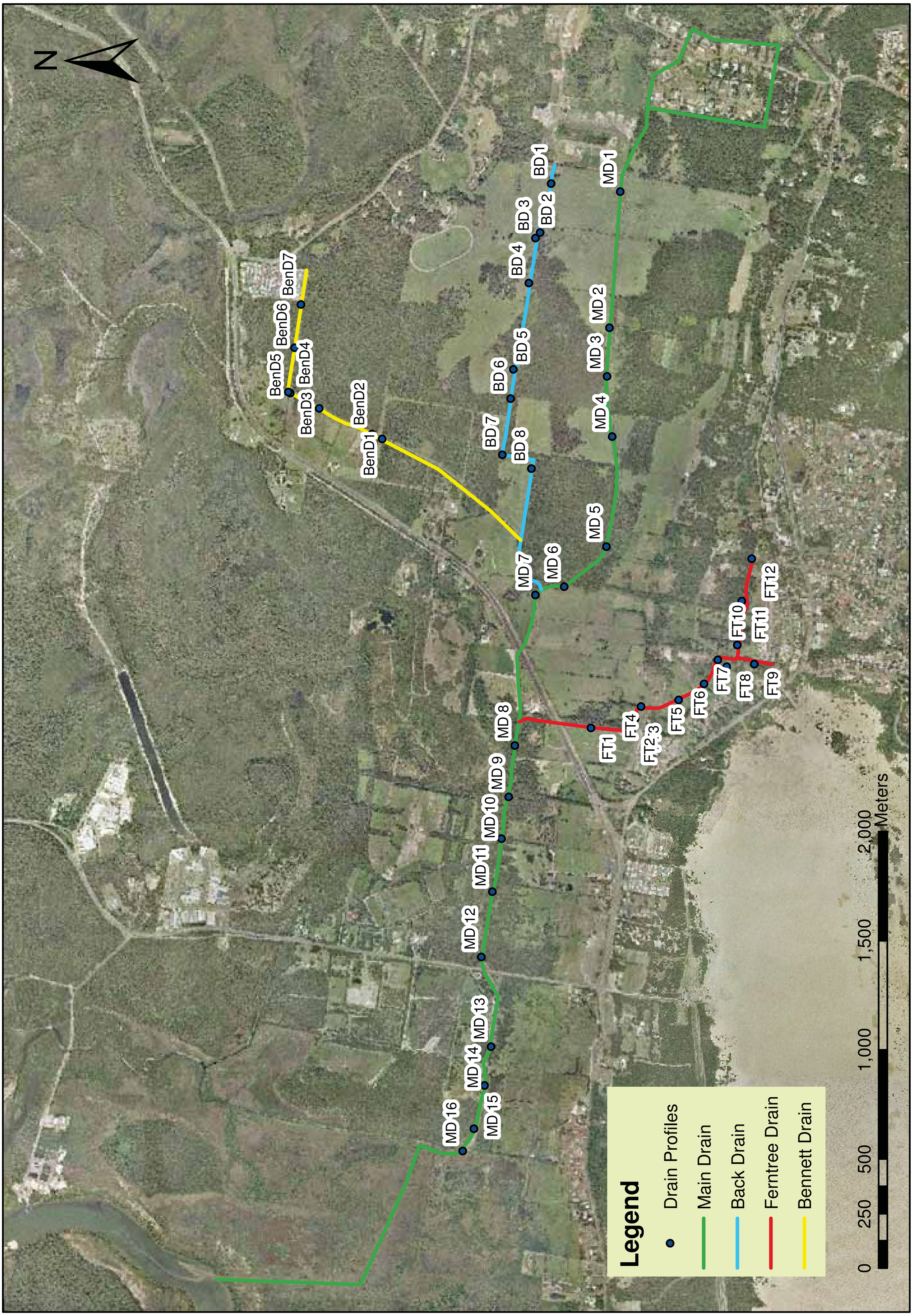
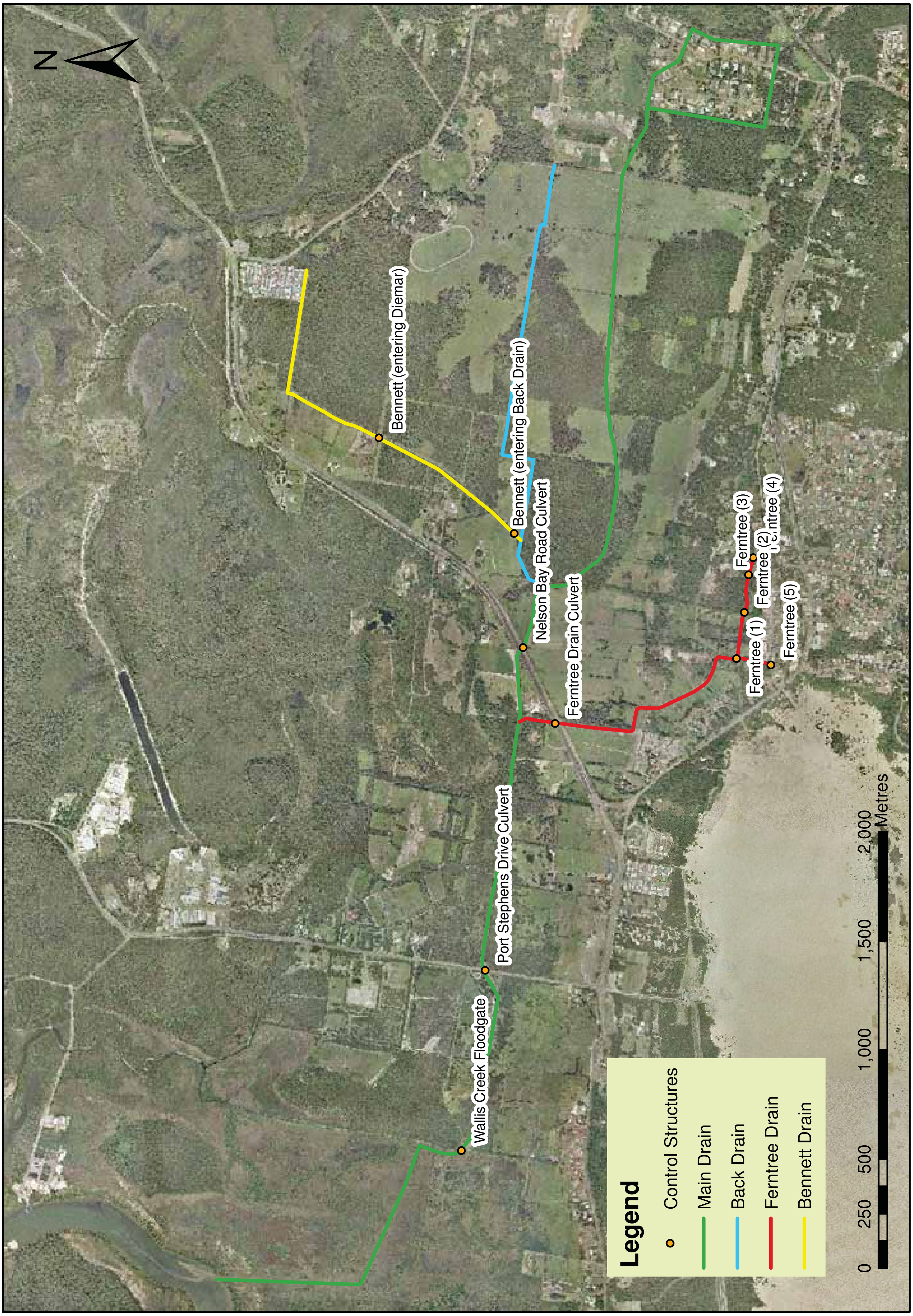
ANNA BAY LiDAR SURVEY

Figure 3

Figure03.mxd



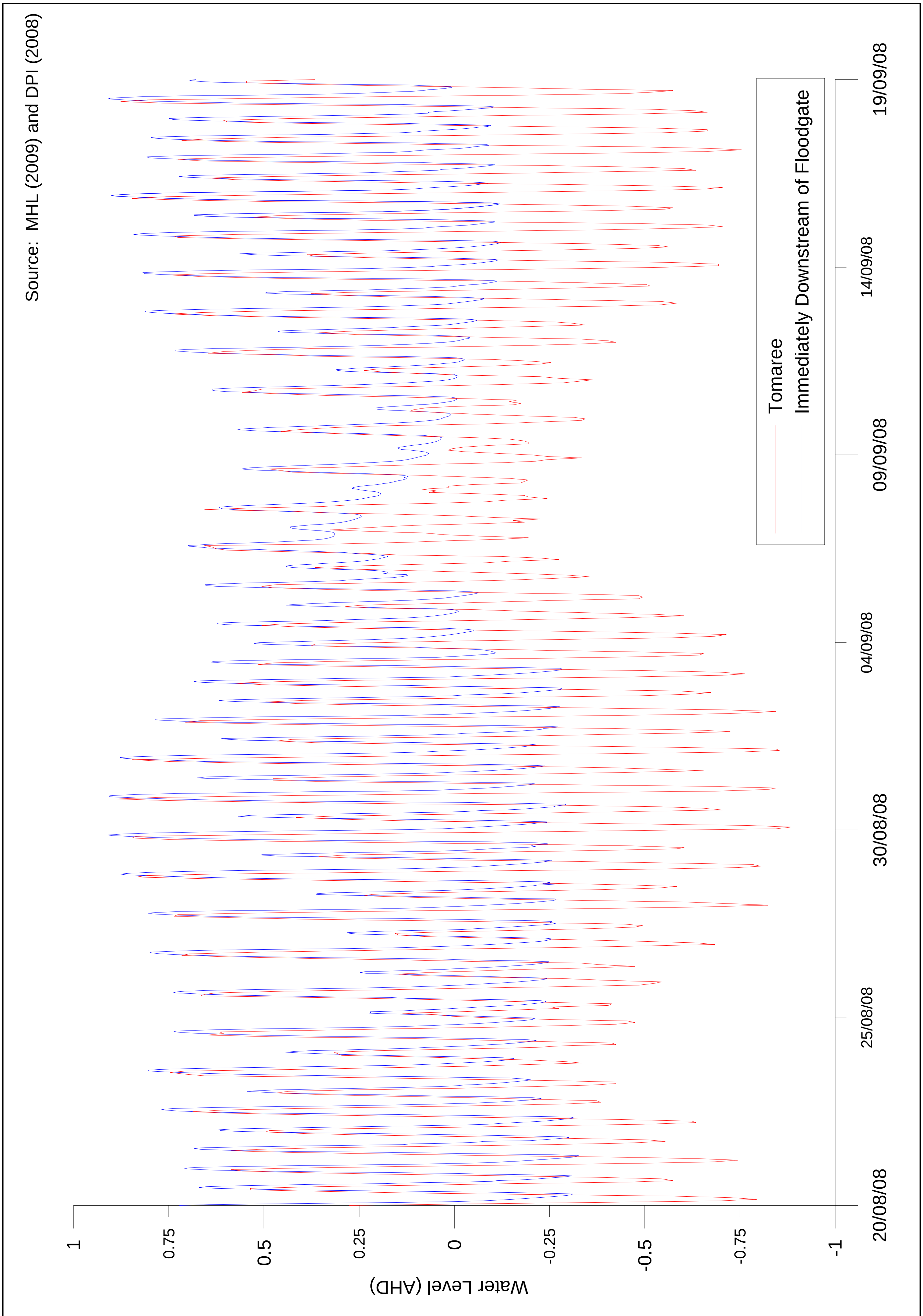
WRL Report No. 2009/12	LOCATION OF DRAIN PROFILES	Figure 4 Figure04.mxd



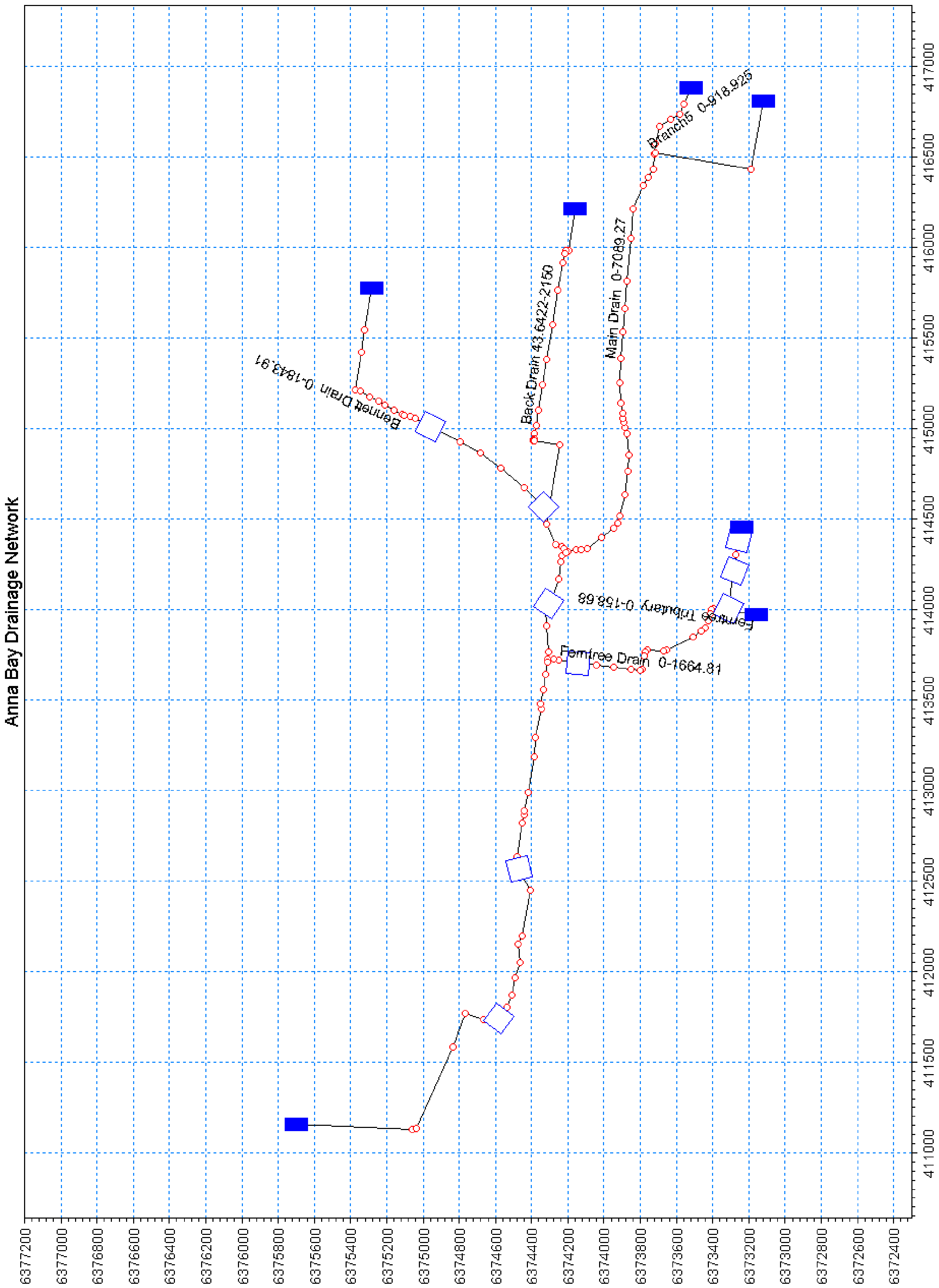
WRL Report No. 2009/12	LOCATION OF LARGE CONTROL STRUCTURES IN PRIMARY DRAINAGE NETWORK	Figure 5

Figure05.mxd

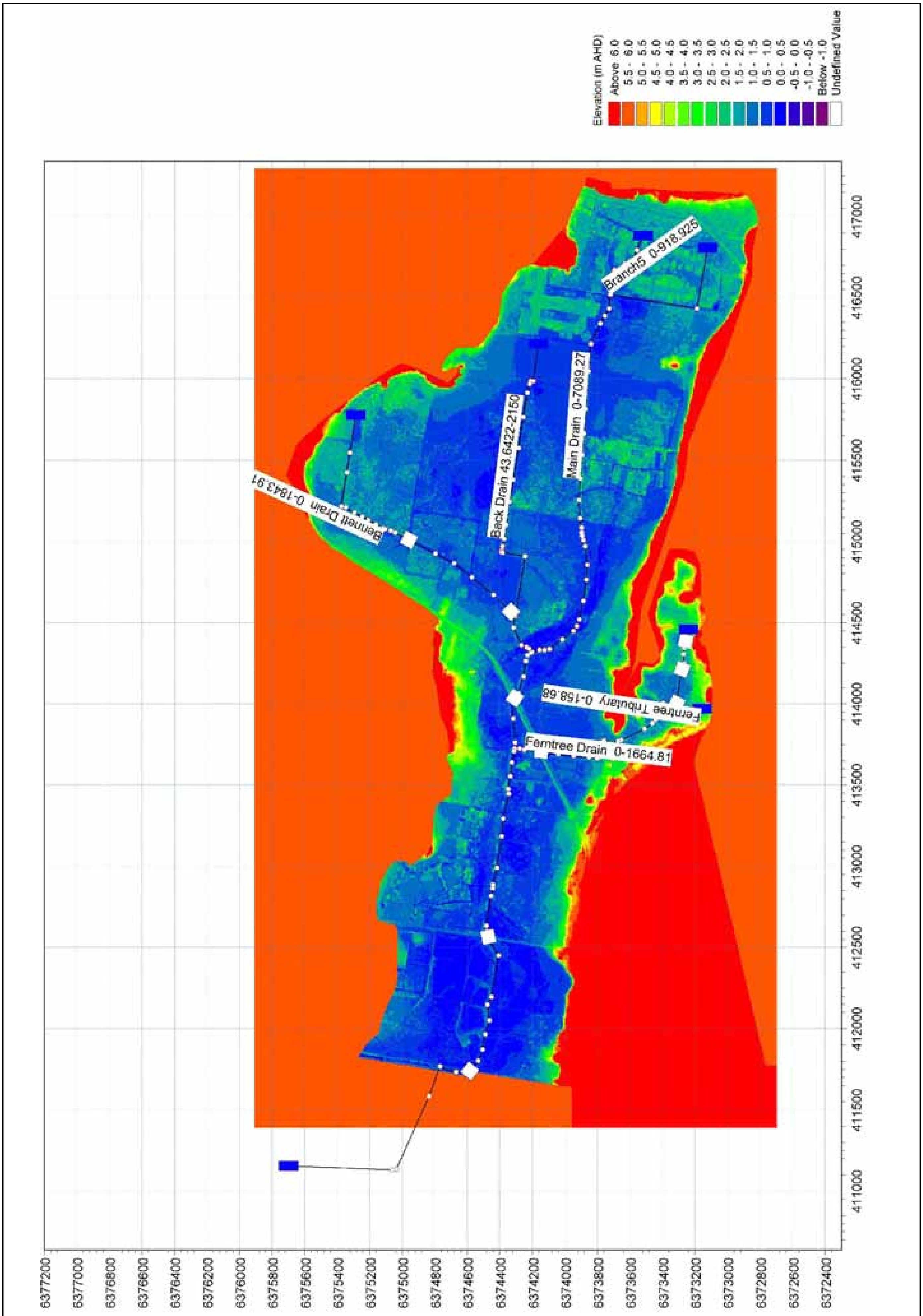
WRL Report No. 2009/12	WATER LEVEL DATA FOR ANNA BAY DRAIN SYSTEM	Figure 6



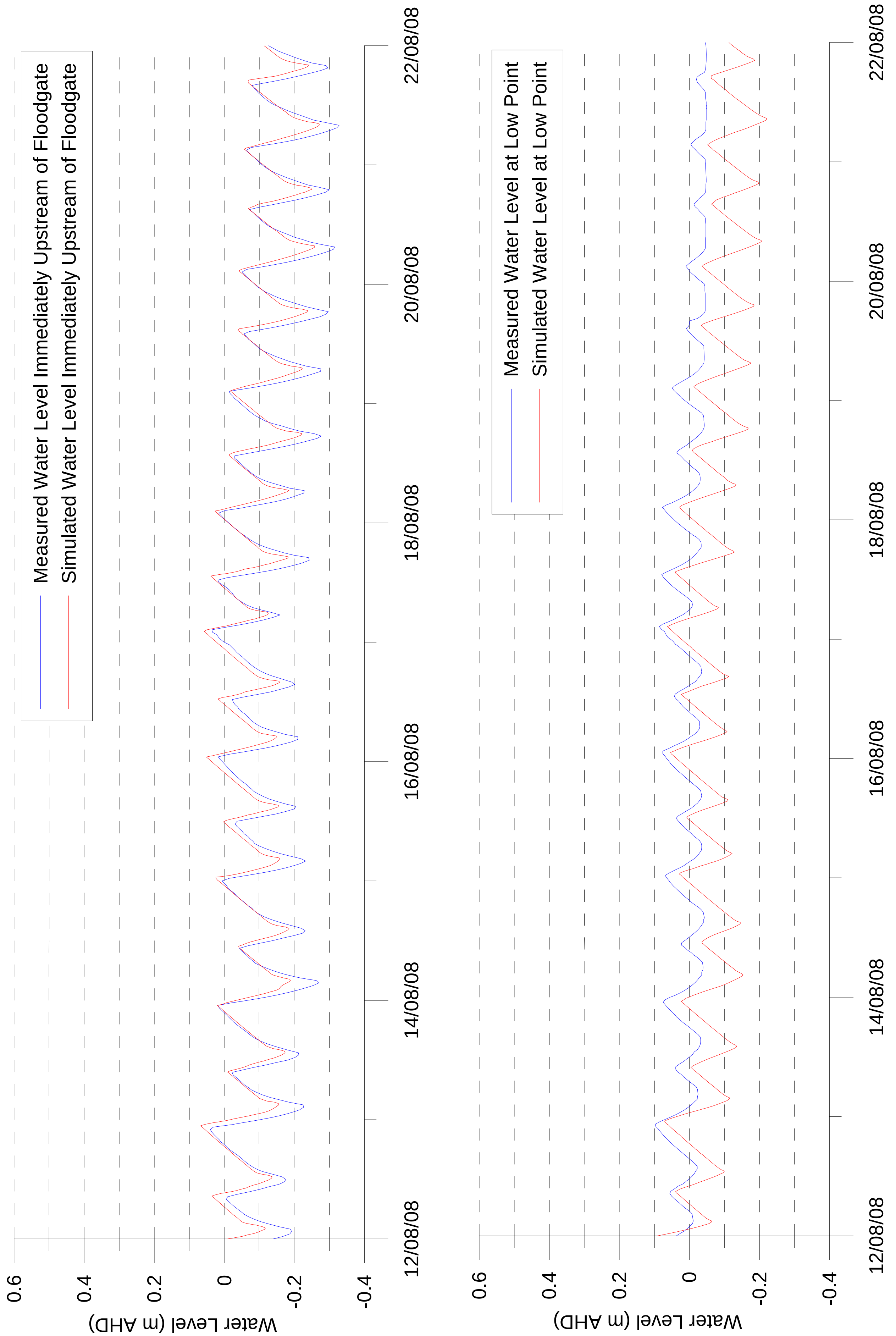
WRL Report No. 2009/12	WATER LEVELS DOWNSTREAM OF FLOODGATE COMPARED TO WATER LEVELS AT TOMAREE	Figure 7



WRL Report No. 2009/12	ONE DIMENSIONAL MODEL OF ANNA BAY DRAINAGE NETWORK	Figure 8



WRL Report No. 2009/12	COUPLED 1D-2D ANNA BAY MODEL	Figure 9



WRL Report No. 2009/12	MEASURED AND SIMULATED WATER LEVELS UPSTREAM OF FLOODGATE AND AT LOW POINT	Figure 10

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LOCATION OF PROPOSED WEIR

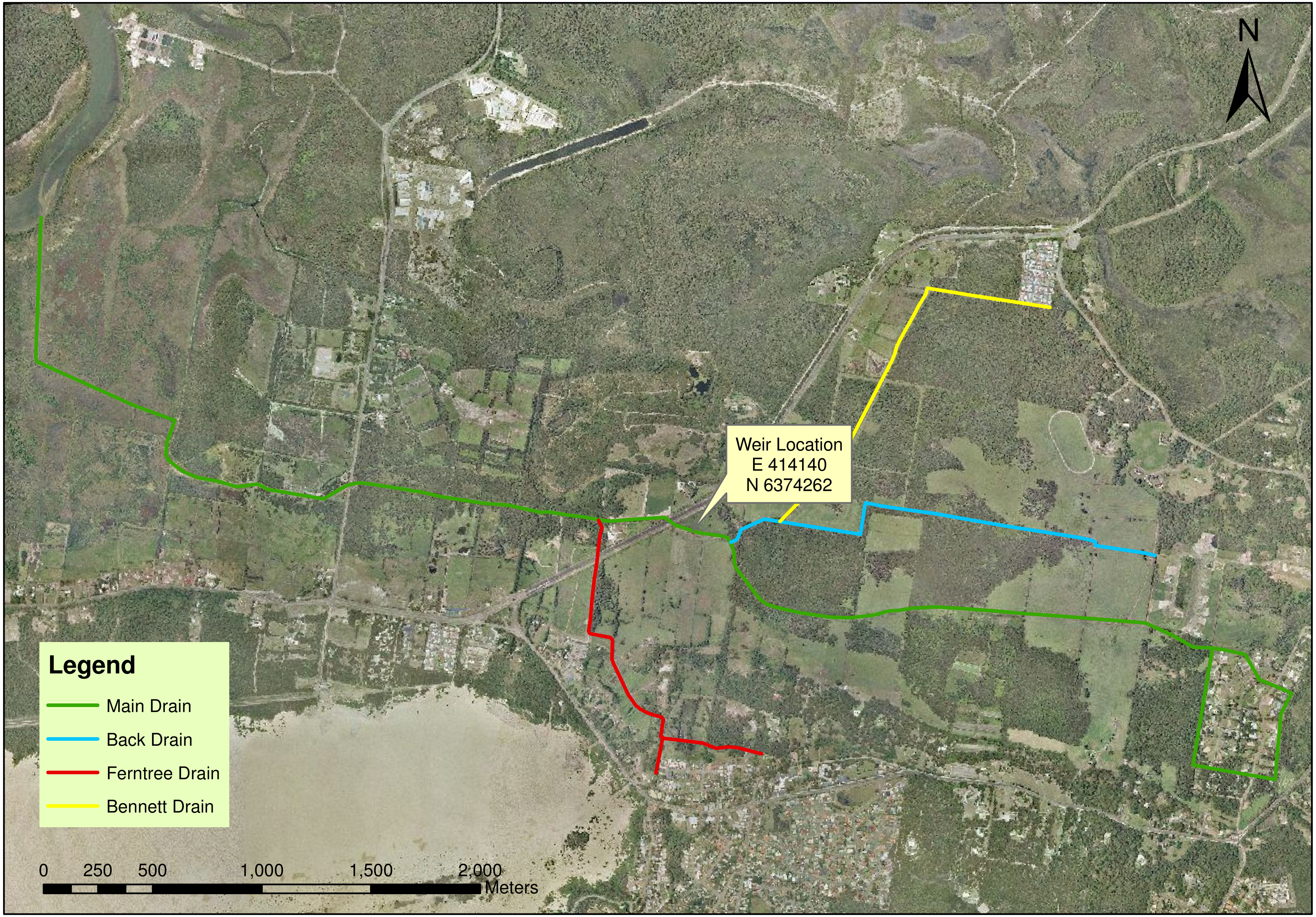

Figure 11

Figure11.mxd

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LOCATION OF DRY SCENARIO RESULTS

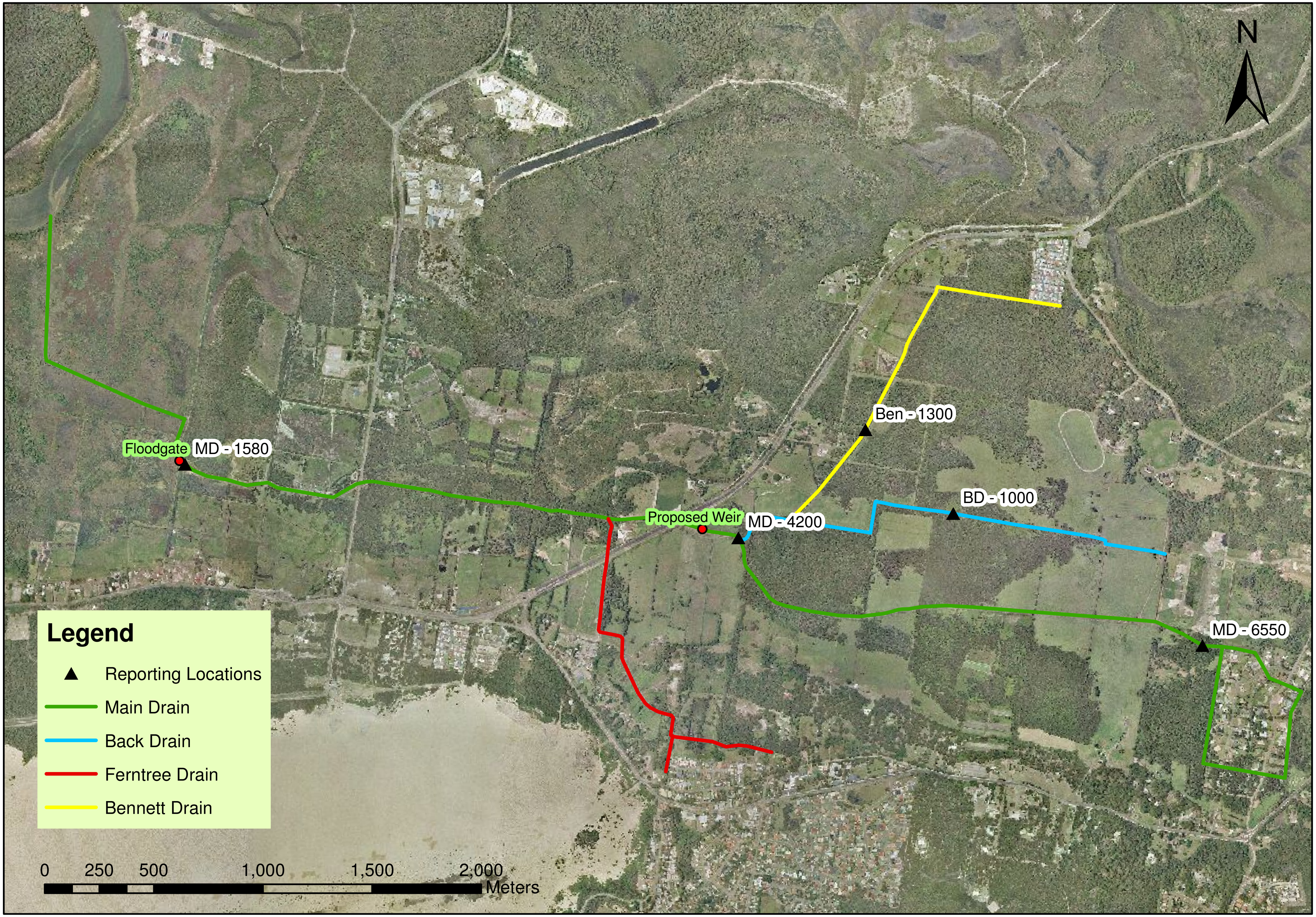
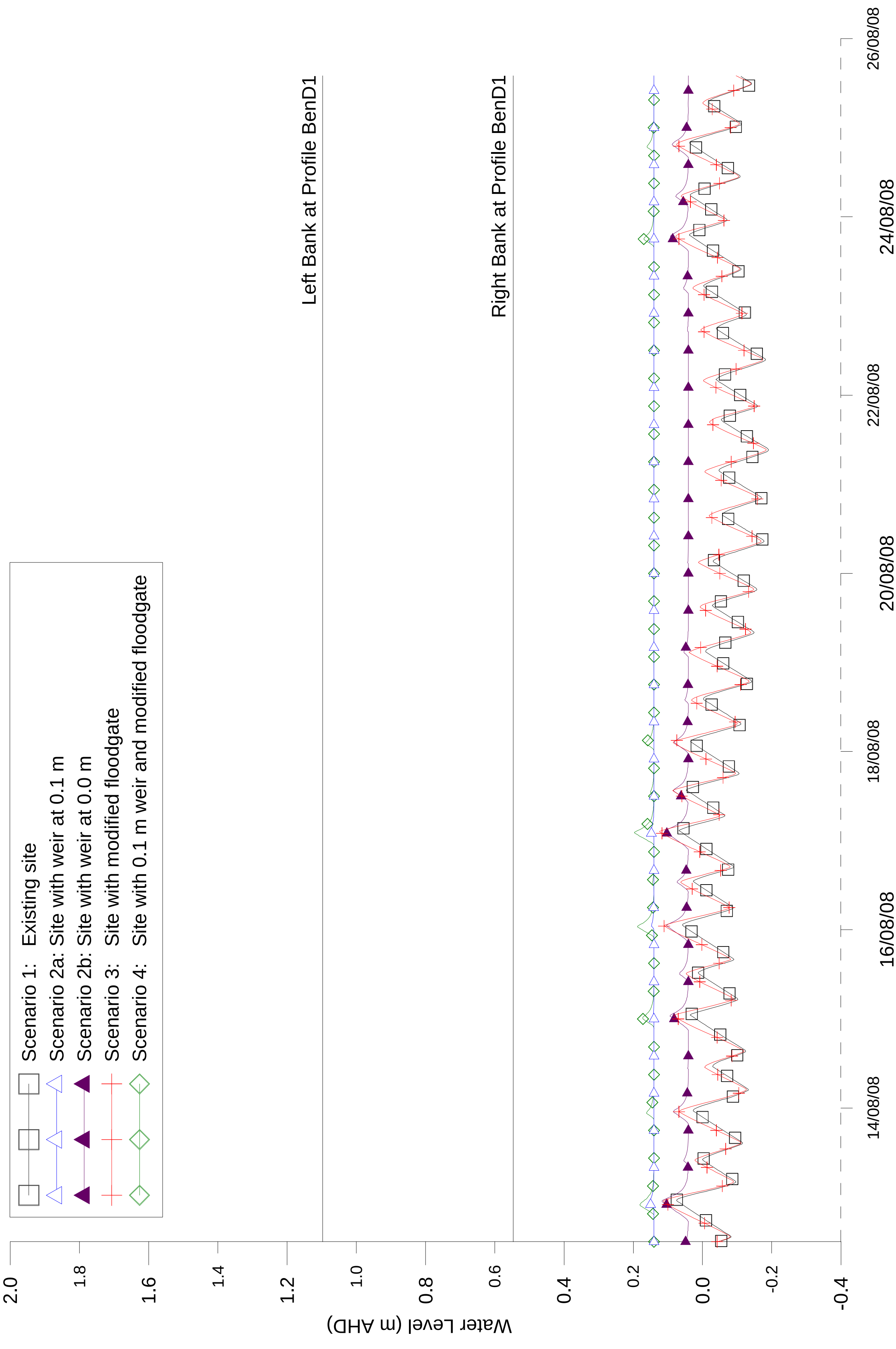


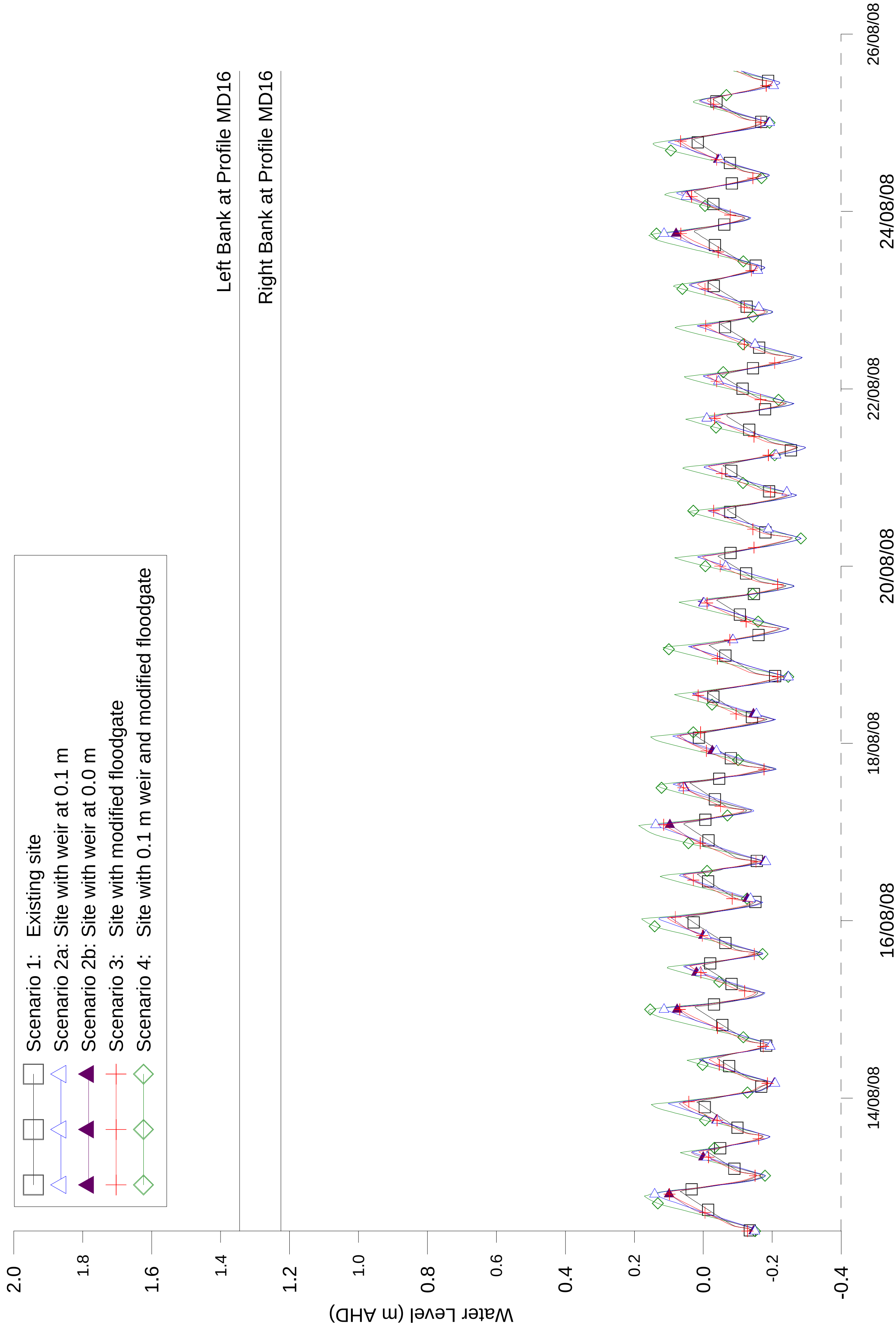
Figure 12

Figure12.mxd

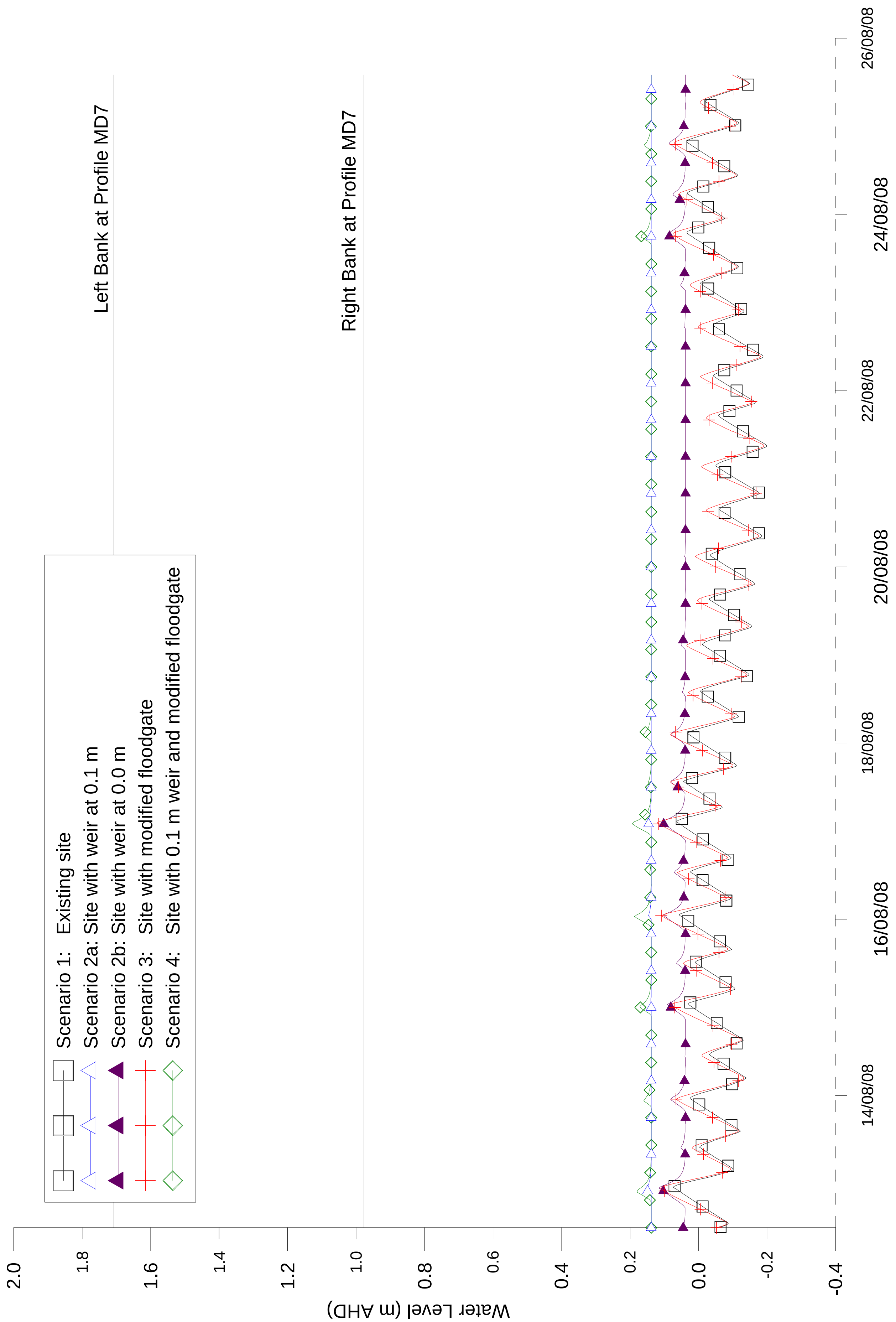


WRL Report No. 2009/12	SIMULATED WATER LEVEL FOR DRY PERIOD SCENARIOS AT Ben-1300	Figure 13

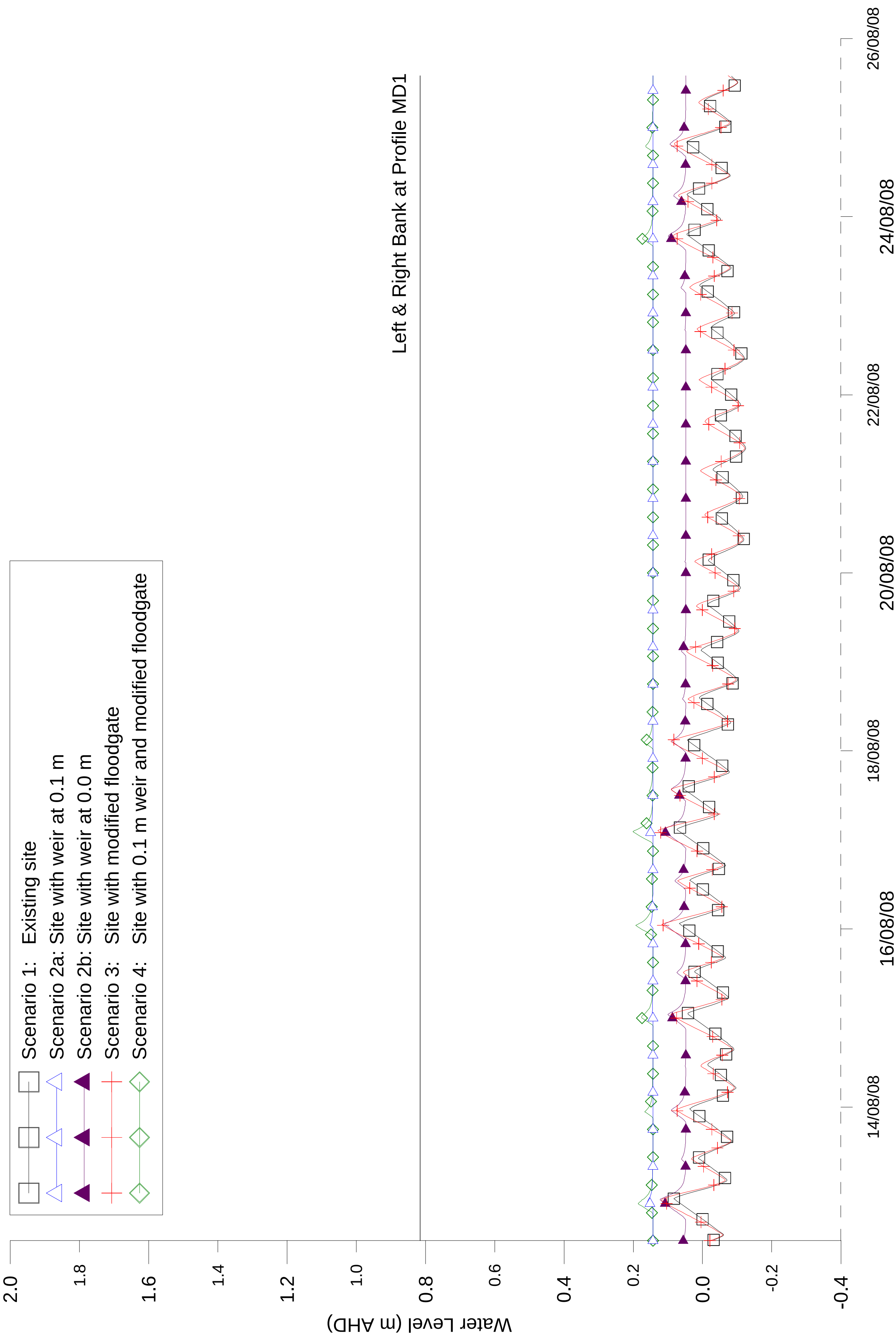
WRL Report No. 2009/12	SIMULATED WATER LEVEL FOR DRY PERIOD SCNEARIOS AT BD-1000	Figure 14



WRL Report No. 2009/12	SIMULATED WATER LEVEL FOR DRY PERIOD SCENARIOS AT MD-1580	Figure 15



WRL Report No. 2009/12	SIMULATED WATER LEVEL FOR DRY PERIOD SCENARIOS AT MD-4200	Figure 16



WRL Report No. 2009/12	SIMULATED WATER LEVEL FOR DRY PERIOD SCENARIOS AT MD-6550	Figure 17

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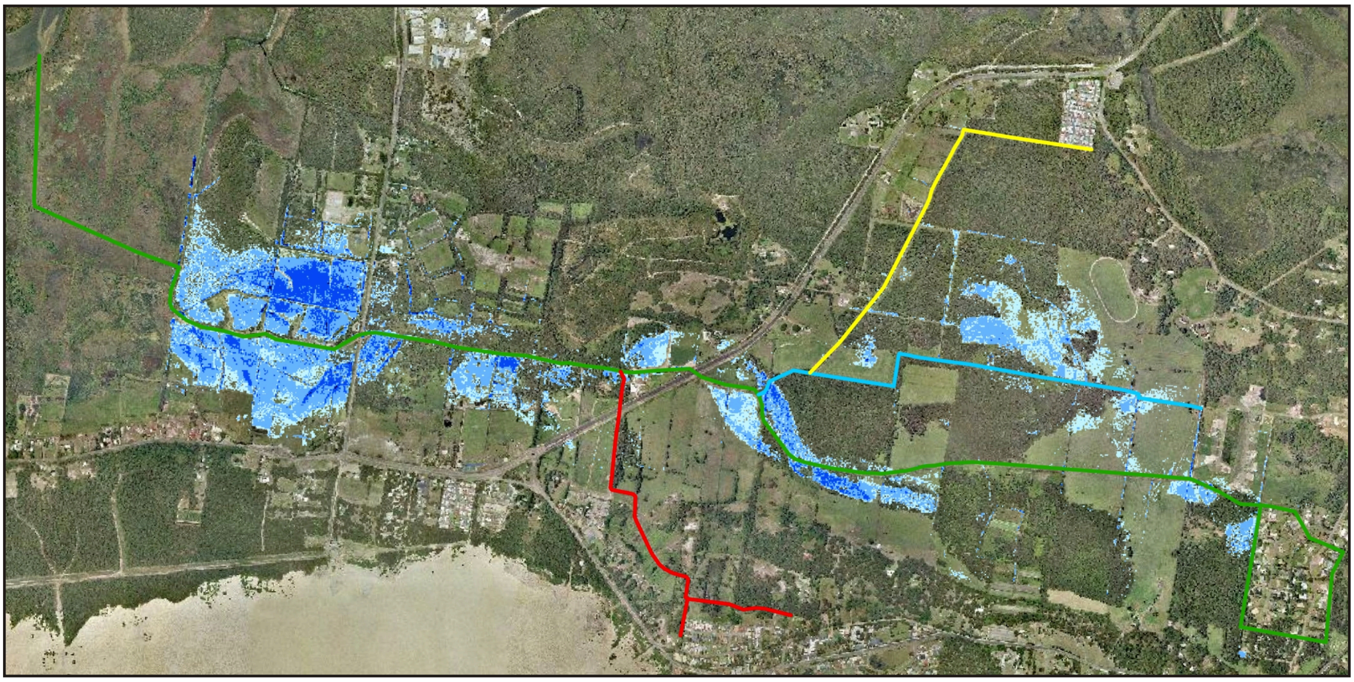
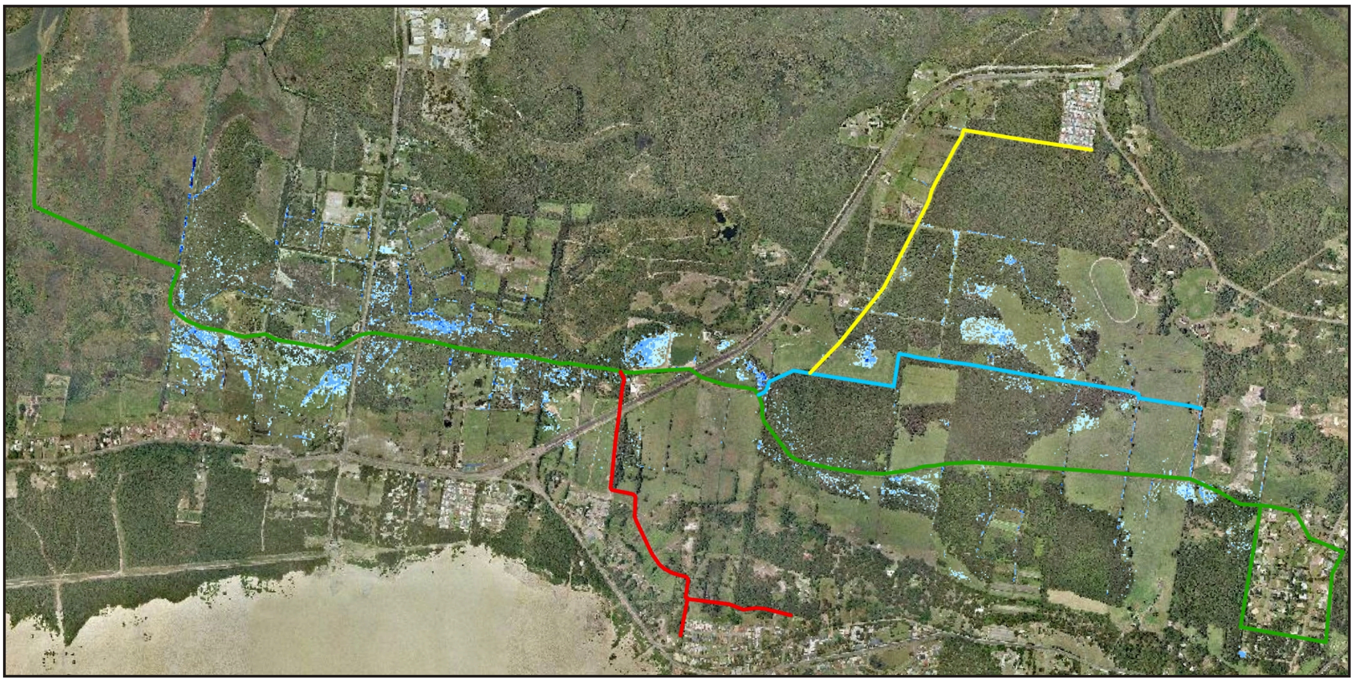


Initial Water Depth for Wet Period Simulations (Existing Site)



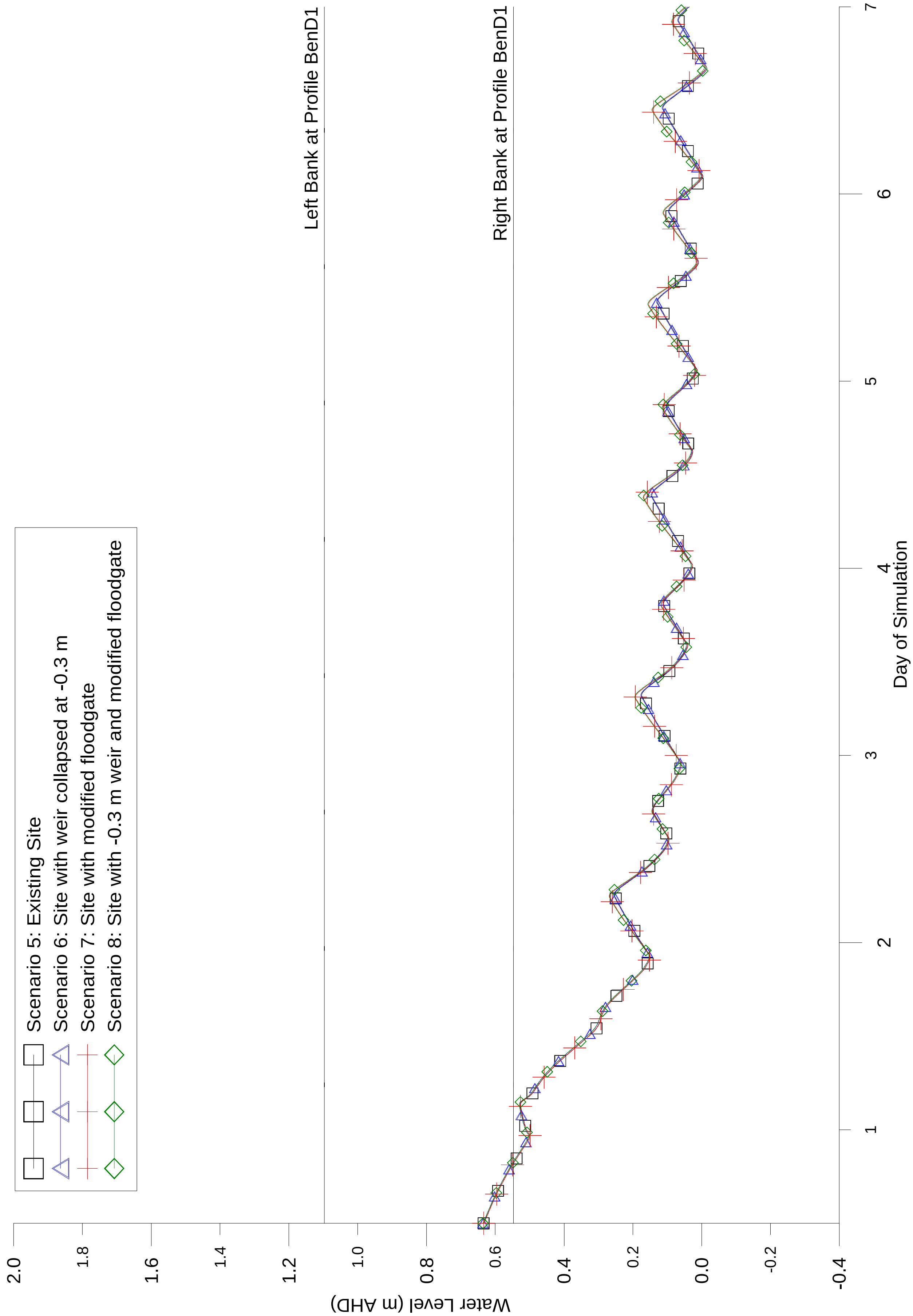
Water Depth After 5 Days (Existing Site)

Water Depth (m)

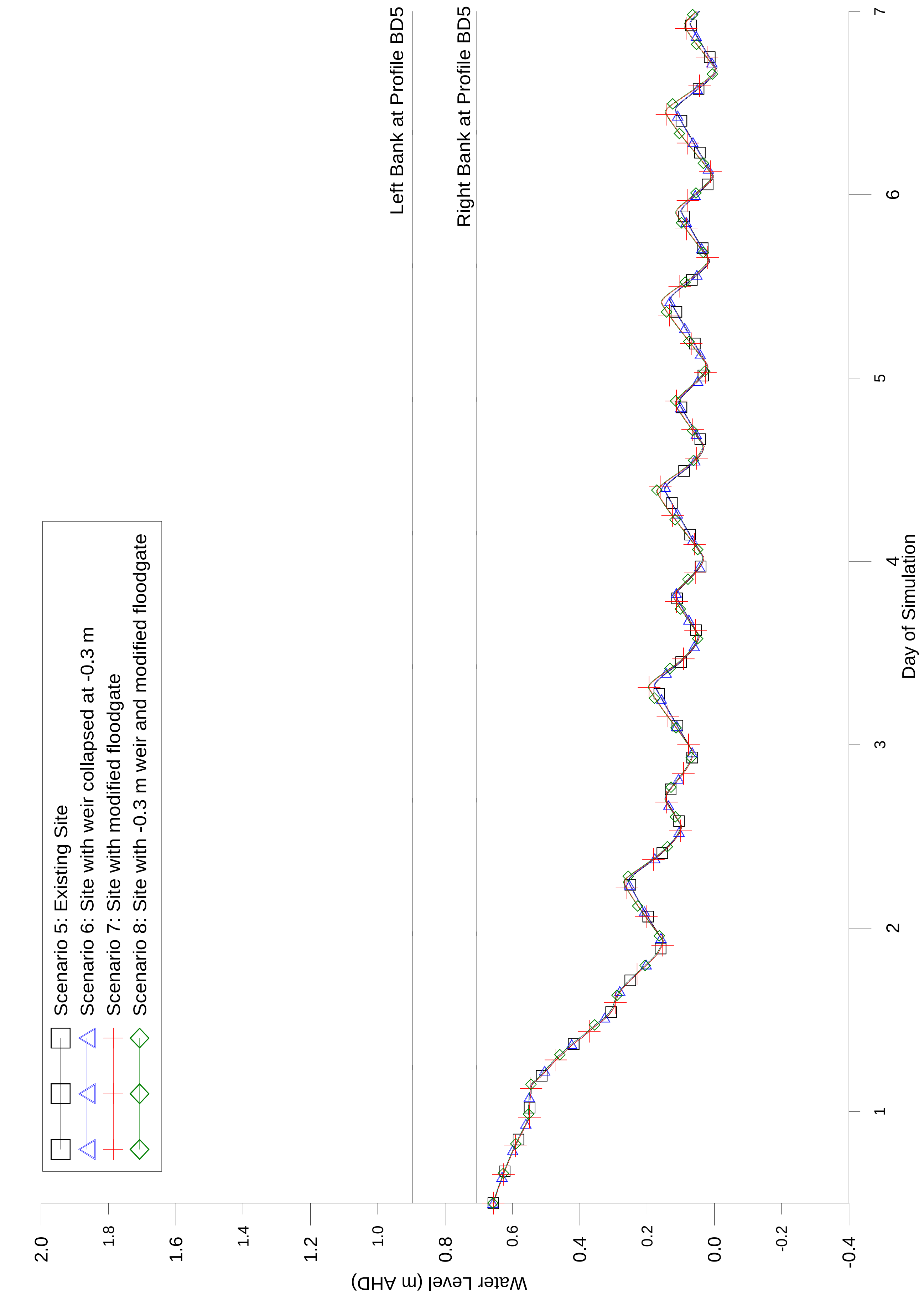
- 0.15 - 0.25
- 0.25 - 0.5
- 0.5 - 0.75
- 0.75 - 1

0 250 500 1,000 1,500 2,000 Metres

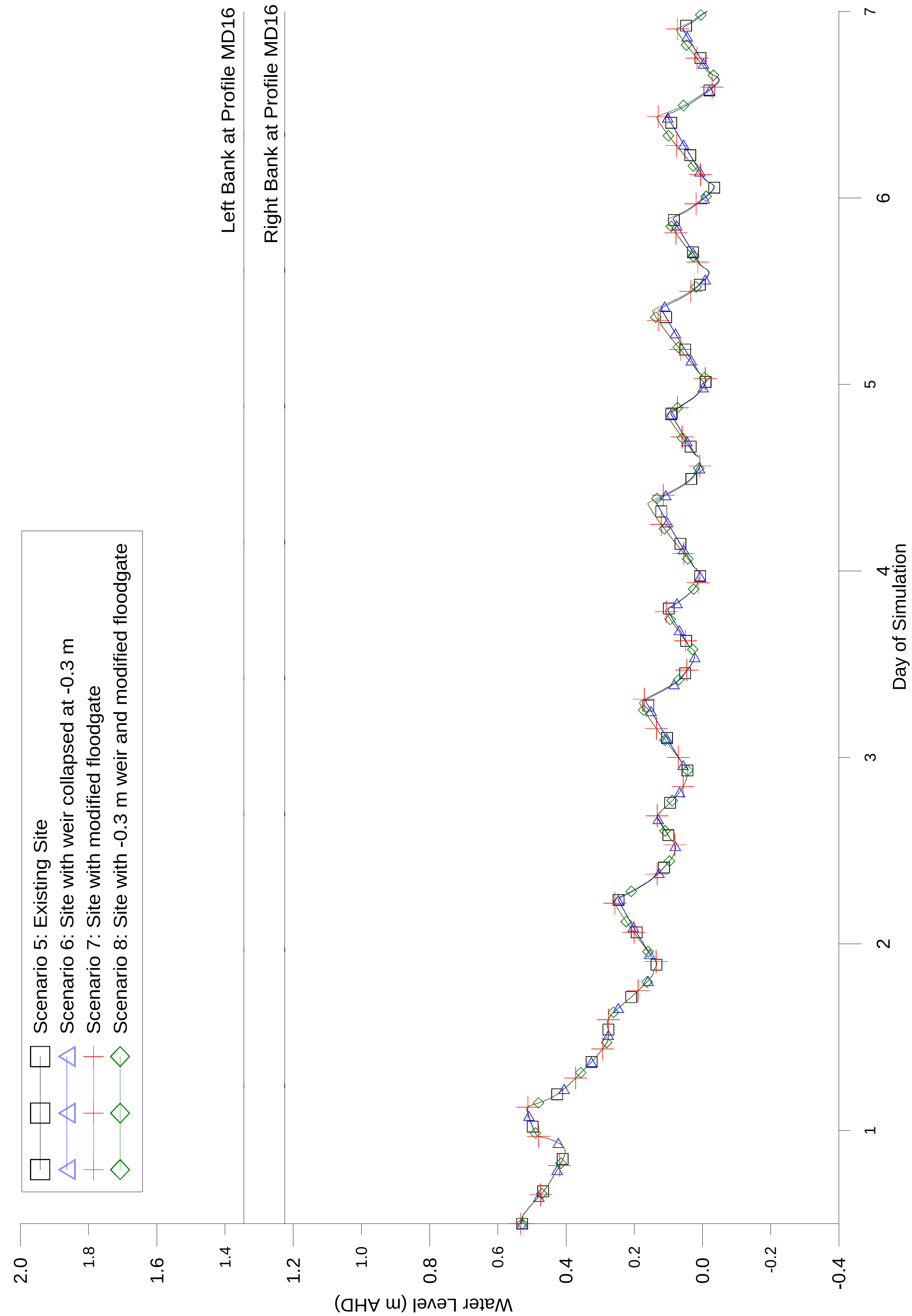
WRL Report No. 2009/12	MODELLED INITIAL WATER DEPTH AND WATER DEPTH AFTER 5 DAYS FOR EXISTING SITE	Figure 18 08087.18.cdr



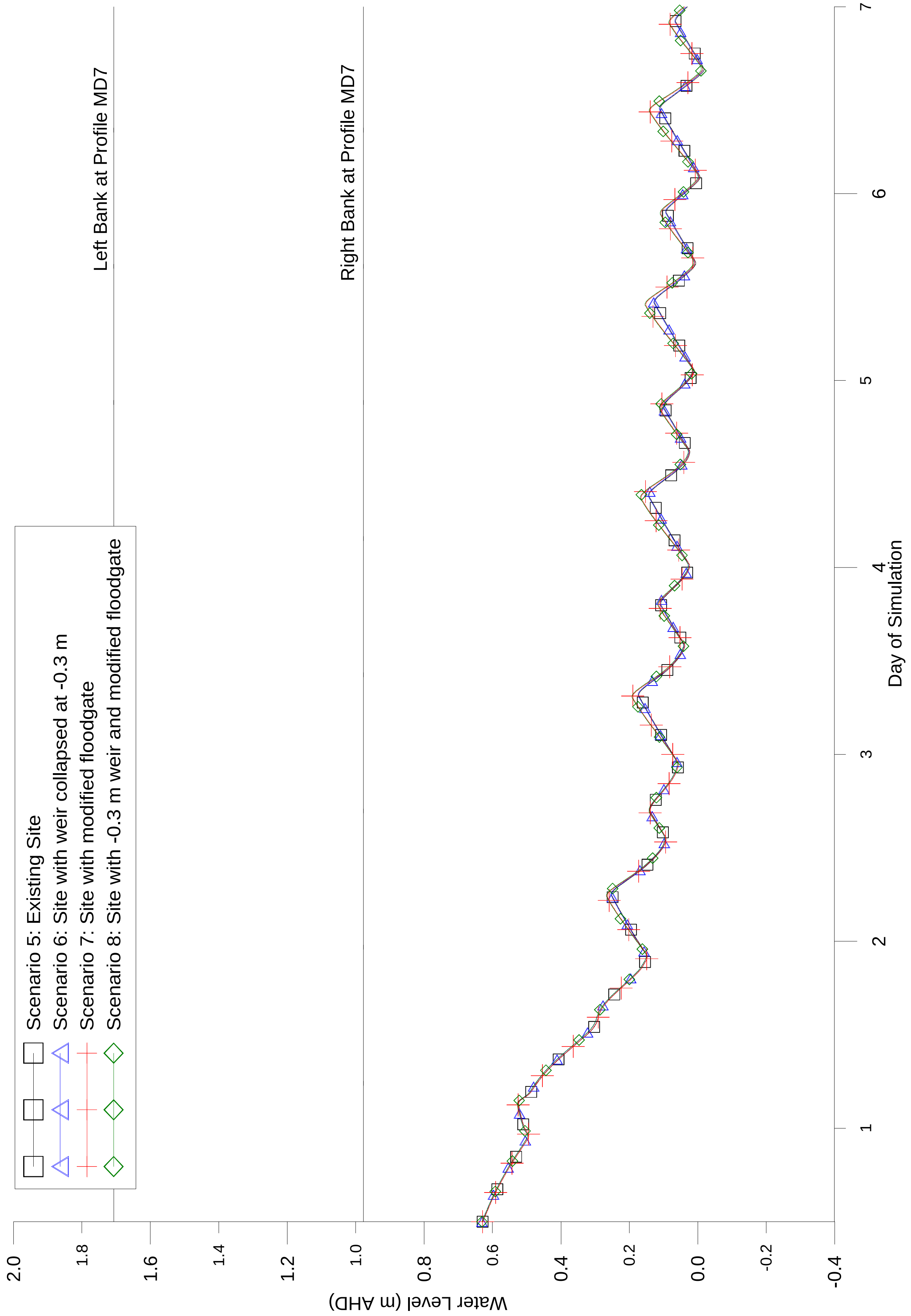
WRL Report No. 2009/12	SIMULATED WATER LEVEL FOR WET PERIOD SCENARIOS AT Ben-1300	Figure 19



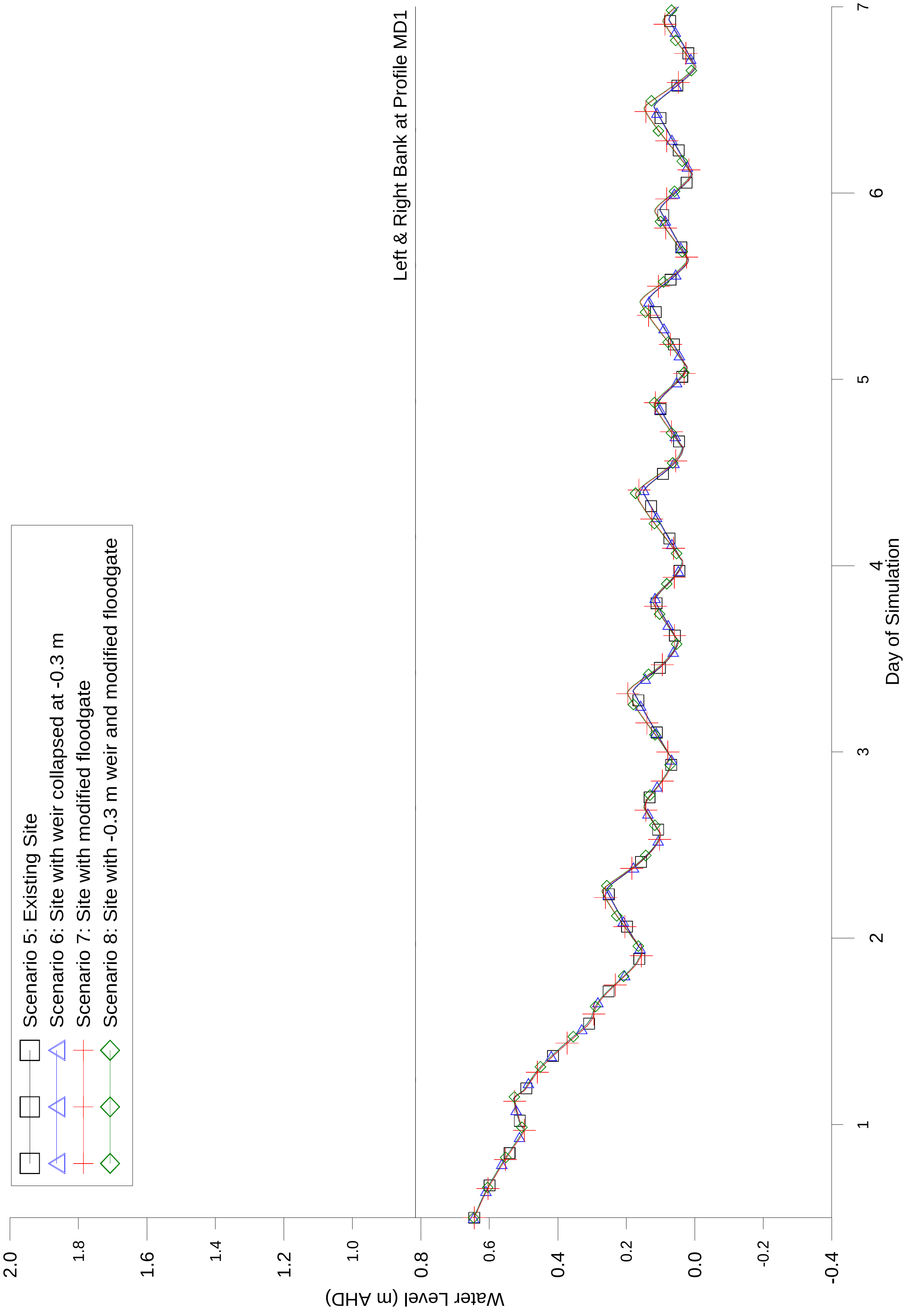
WRL Report No. 2009/12	SIMULATED WATER LEVEL FOR WET PERIOD SCENARIOS AT BD-1000	Figure 20



WRL Report No. 2009/12	SIMULATED WATER LEVEL FOR WET PERIOD SCENARIOS AT MD-1580	Figure 21



WRL Report No. 2009/12	SIMULATED WATER LEVEL FOR WET PERIOD SCENARIOS AT MD-4200	Figure 22



WRL Report No. 2009/12	SIMULATED WATER LEVEL FOR WET PERIOD SCENARIOS AT MD-6550	Figure 23