



10 November 2022

De Grey Mining Ltd
Ground Floor, 2 Kings Park Road
West Perth WA 6005

Attn. Rod Smith

Re. Hemi Gold Project – Turner River and Site Closure Flood Modelling

Dear Mr. Smith,

The attached report summarises hydrologic and hydraulic modelling efforts undertaken to define the Turner River discharge extents and the flood extents for conceptual closure planning.

The Turner River discharge modelling accounts for approximately 16 GL of discharge into the Turner River over a 3-year period.

Flood models for closure scenarios have been developed for the proposed closure footprint, accounting for both localised rainfall-runoff and flood flows from the Turner River and Yule River up to the Probable Maximum Flood.

Additional details related to the previous hydrologic and hydraulic modelling are available in the accompanying hydrological and hydrogeological report and in the July 2018 Pilbara Gold Project Surface Water Assessment prepared by AQ2.

Please feel free to contact me with any questions or clarifications.

Kind regards,

Krey Price
Director, Surface Water Consulting

1. Introduction

1.1. Project Overview

De Grey Mining Ltd (“De Grey”), a Western Australian based mining company listed on the Australian Securities Exchange (“ASX:DEG”) is seeking to develop the Hemi Gold Project (“Project”) in the Pilbara, some 85 kms from the regional hub of Port Hedland.

This hydrology report by Surface Water Solutions forms part of the De Grey Mining Ltd Hemi Gold Project Pre-Feasibility Study, which assesses the environmental impacts, community interaction, technical requirements, and financial robustness of the Project to a PFS level.

The Project is of a scale that places it in Tier 1 category for gold mine developments. The Project consists of six deposits: Aquila, Brolga, Crow, Diucon, Eagle and Falcon, collectively known as the Hemi deposits. Although the Hemi deposits will provide ore for the Project over a mine life in excess of 10 years, there is also potential for additional resources from regional deposits that may, subject to the outcomes of further studies, be processed at the Hemi processing facility.

The location of the Hemi deposit in relation to the Regional deposits is shown in Figure 1, and the proposed layout of the associated infrastructure is shown in Figure 2.

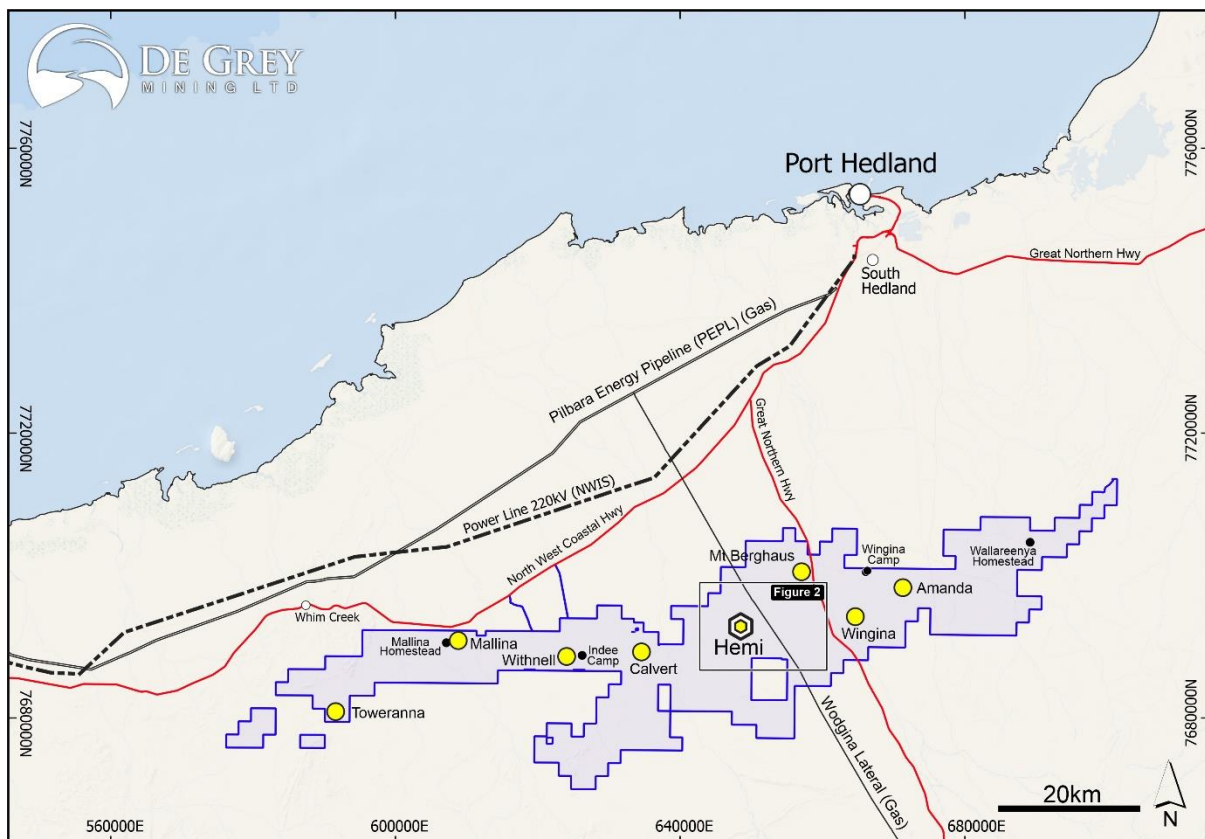


Figure 1 – Hemi Gold Project Location

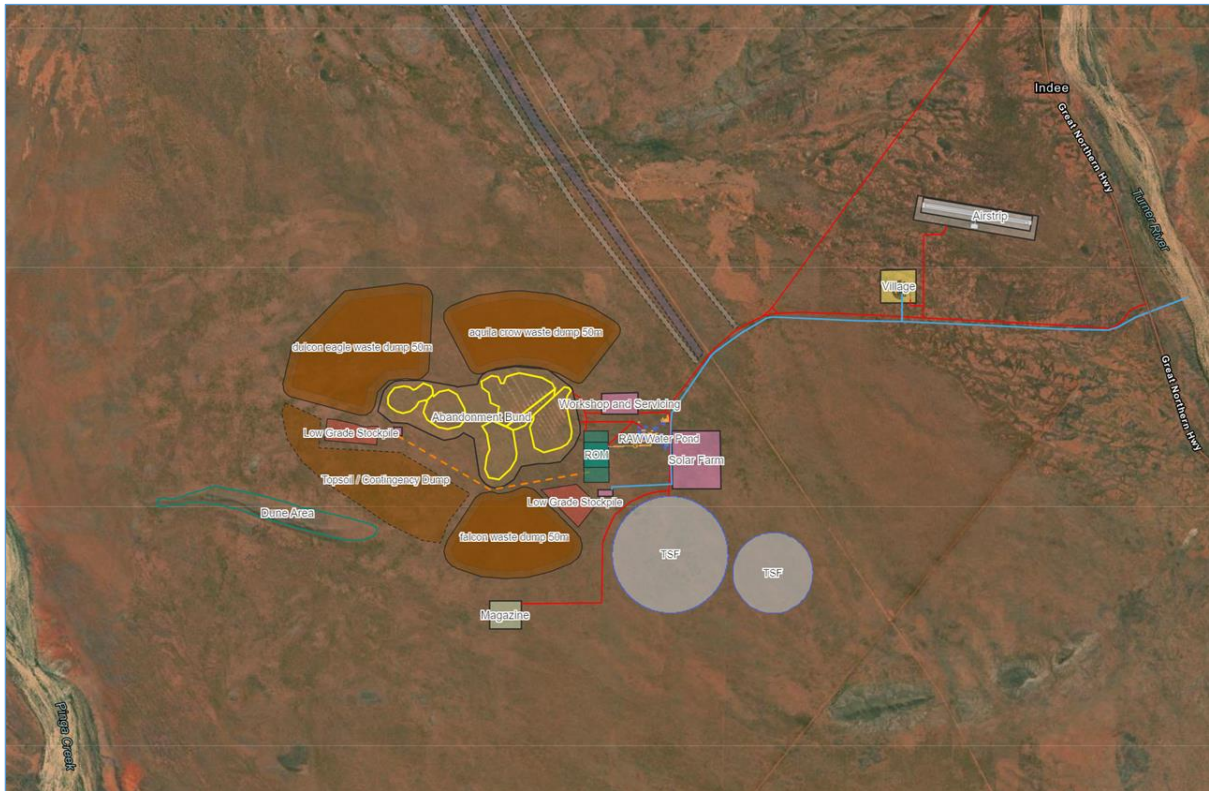


Figure 2 – Hemi Gold Project Infrastructure

The Project comprises the following key components:

- The sequential development of open cut pits at the Hemi deposits for the life of mine
- The construction and subsequent operation of a nominal 10 Mtpa processing facility located adjacent to the Hemi deposits capable of achieving 93% to 95% gold recovery from free milling and semi refractory ores
- Staged construction of tailings storage facilities (TSFs) with a planned capacity for 130 Mt of tailings slurry
- A water supply from the local groundwater aquifer with accompanying groundwater and surface water management infrastructure to facilitate mine dewatering and aquifer diversion
- A village with messing and accommodation capacity for approximately 600 personnel
- An airstrip with capacity for jet aircraft (up to 100 seat capacity)
- A 12 km sealed access road from the Great Northern Highway

The discharge assessment indicates that under the adopted discharge schedule and assumed infiltration rates, discharge will reach the Great Northern Highway crossing during the first year of discharge but will not reach the ocean outlet. Flows cease upstream of the Great Northern Highway during Year 2.

Under maximum discharge conditions, the inundated area covers approximately 5% of the Turner River channel. Using flows around the external site perimeter, it may be beneficial to incorporate bunds and drains that direct runoff into closed pits to accelerate groundwater recharge. These options may be explored further in conjunction with the hydrogeological analyses as the closure designs progress.



1.2. Scope of Work

This report summarises the results of Turner River discharge modelling and concept-level flood modelling for mine closure scenarios.

Turner River discharge modelling accounts for the discharge of approximately 16 GL into the Turner River over a 30-month period. The accompanying figures and electronic geospatial data show conditions during three selected months representative of high, average, and low flow conditions.

Conceptual closure flood model results are presented for the 1% AEP, 1 in 1,000 AEP, and PMF events. Results are presented for external flows; internal drainage will be modelled as proposed layouts are confirmed.

1.3. Previous reports

The following reports were compiled from online sources or provided by De Grey as background material to support this hydrology report:

- Hemi Water Data and Information for Scoping Study Consultants, Geowater, 30 March 2021.
- Pilbara Gold Project Surface Water Assessment, AQ2, July 2018.
- Boodarie Strategic Industrial Area Turner River Flood Study, GHD, 2013.

In addition to the above reports, separate hydrological and hydrogeological reports have been prepared for the Mallina Gold Project by Surface Water Solutions and Geowater. The accompanying reports can be referred to for additional details supporting the findings in this report.



2. Turner River Discharge Model Setup

2.1. Approach

Turner River discharge scenarios were run using a two-dimensional (2D) modelling approach using HEC-RAS 6.3 (USACE 2022), with projected monthly discharge rates applied as an inflow time series at the proposed discharge point in the Turner River.

2.2. Terrain

The underlying terrain for the Turner River was compiled from three data sets, in descending order of priority:

- 3m x 3m resolution LiDAR-based digital elevation model (Aerometrex, 2022)
- 5m x 5m resolution LiDAR-based digital elevation model (GSA, 2001-2015)
- Shuttle Radar Topography Mission (SRTM) 1 arc-second digital elevation model

Figure 3 shows the terrain coverage areas relative to the proposed Hemi Deposit infrastructure. The 2022 data set covers the Turner River from 2 km downstream of the Great Northern Highway crossing (Chainage 23500 in Figure 3) to the discharge point (Chainage 67200). Vertical discrepancies of up to 500 mm are present across the interface zones between the 3-metre and 5-metre data sets. In areas with overlapping coverage, the 2022 3-metre DEM is adopted as the underlying terrain for the Turner River discharge modelling.

The average vertical difference between the available LiDAR-based DEM data and satellite-based SRTM topography is approximately 3 metres. As a result of the substantial discrepancy, the SRTM elevations were adjusted to best match the LiDAR-based terrain elevations at the interface. Representative channels were then applied to the SRTM terrain data using cross sections extracted from the LiDAR-based terrain.

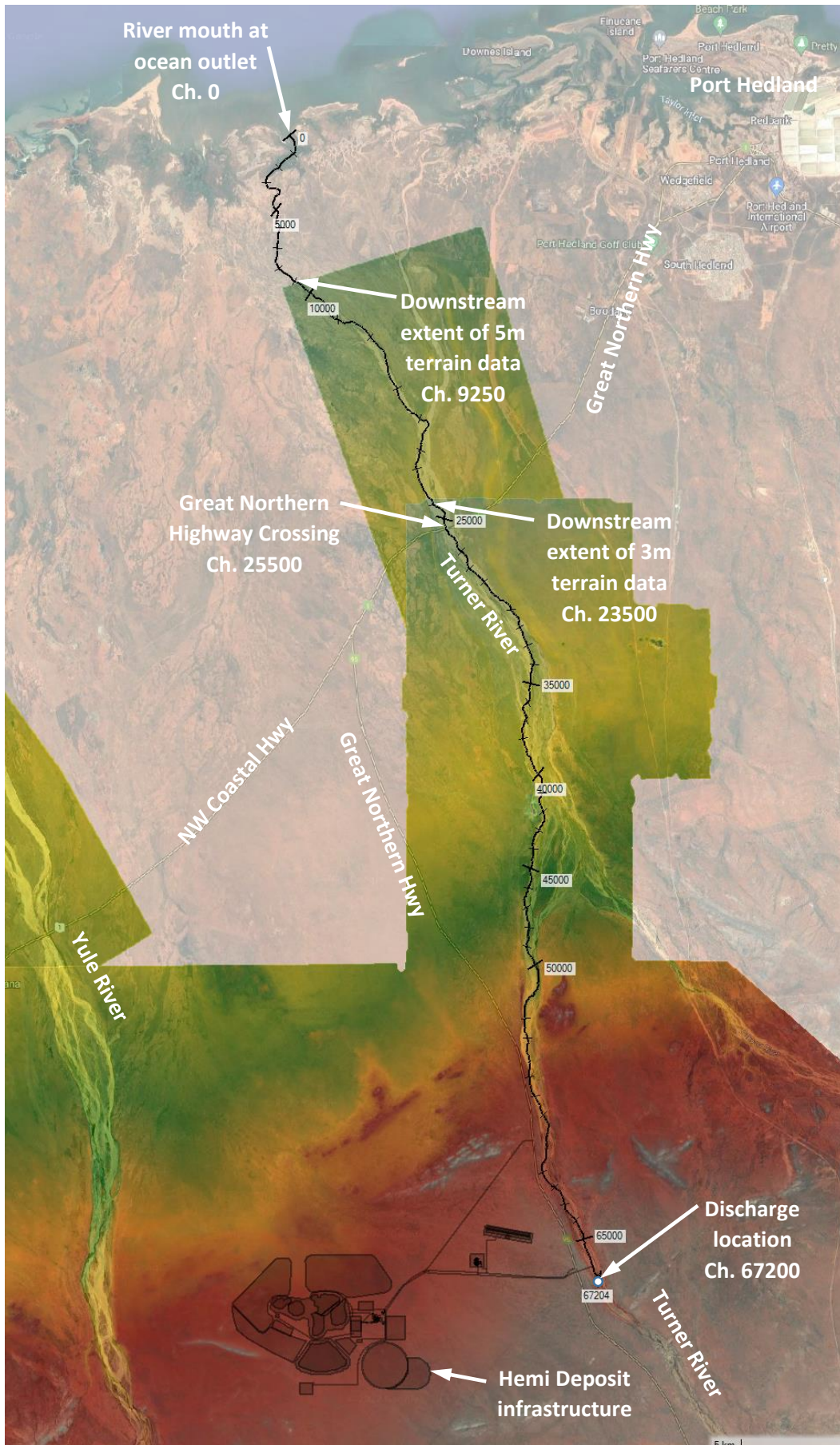


Figure 3 – Yule River and Turner River catchments
Turner River and Site Closure Flood Modelling

2.3. Discharge

Figure 4 shows the proposed discharge point near the Great Northern Highway Mt Dove turnoff, with coordinates displayed in the GDA94 MGA Zone 50 coordinate system (EPSG 28350).

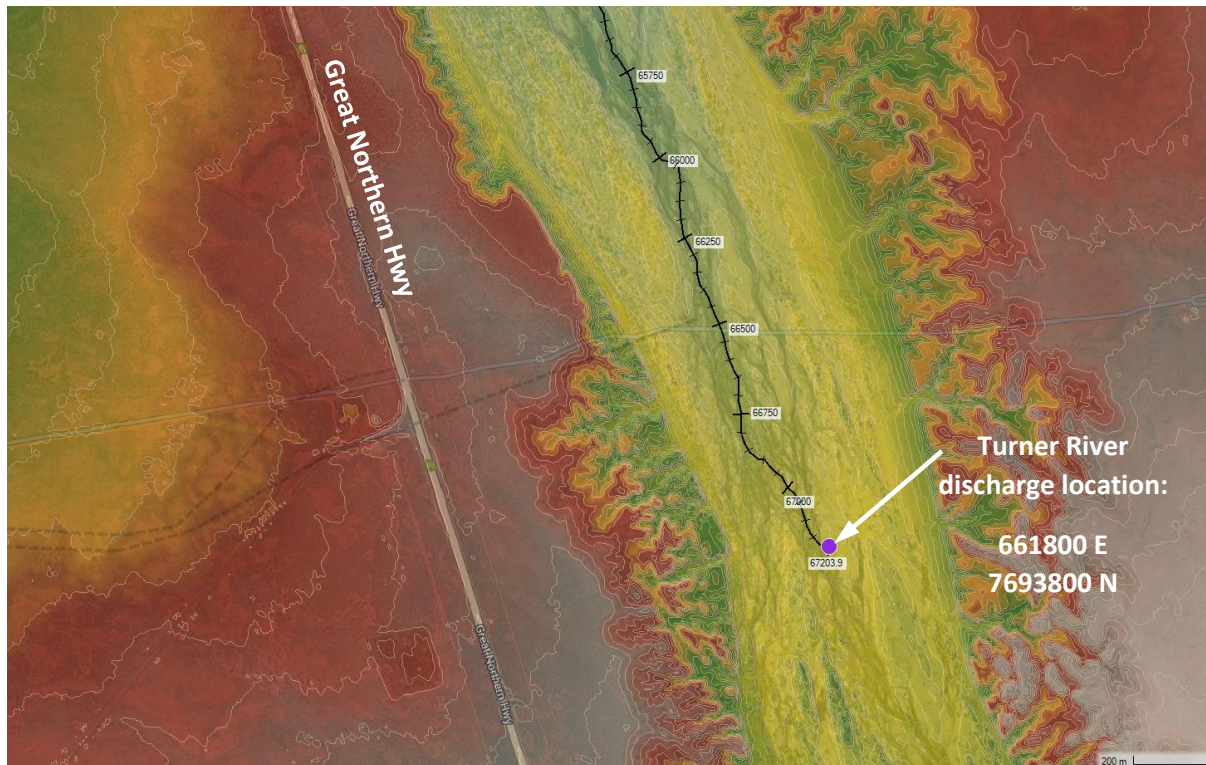


Figure 4 –Turner River discharge location

The proposed discharge volume into the Turner River is 16 GL over a 2½ year period (30 months), with approximately 8 GL discharged during Year 1, 7 GL during Year 2, and 1 GL during Year 3.

Figure 5 shows the estimated monthly inflow rates for the 30-month discharge period, with an average discharge rate of approximately 18 ML/day. The discharge rate is approximately 24 ML/day for the first 21 months of operation, reaching a maximum discharge rate of approximately 24.2 ML/day. The minimum discharge rate of 0.3 ML/day occurs in Month 27 (September of Year 3.)

2.4. Evaporation Losses

Figure 6 shows the estimated monthly evaporation rates corresponding to each month of discharge through the three-year period. The SILO climate database shows an annual class A pan evaporation depth of 3200 mm for the Turner River area. The maximum monthly rate of 364 mm occurs in December, and the minimum rate of 171 mm/month occurs in June. The average monthly evaporation rate is approximately 267 mm.

For the discharge assessment, pan evaporation rates are converted to estimated evaporation in the Turner River using an assumed pan coefficient of 1.3 (ratio of river evaporation to pan evaporation). For use in the hydraulic modelling, evaporation rates are multiplied by the inundated area based on 1-km river reaches and removed from the model using negative inflow boundary conditions.

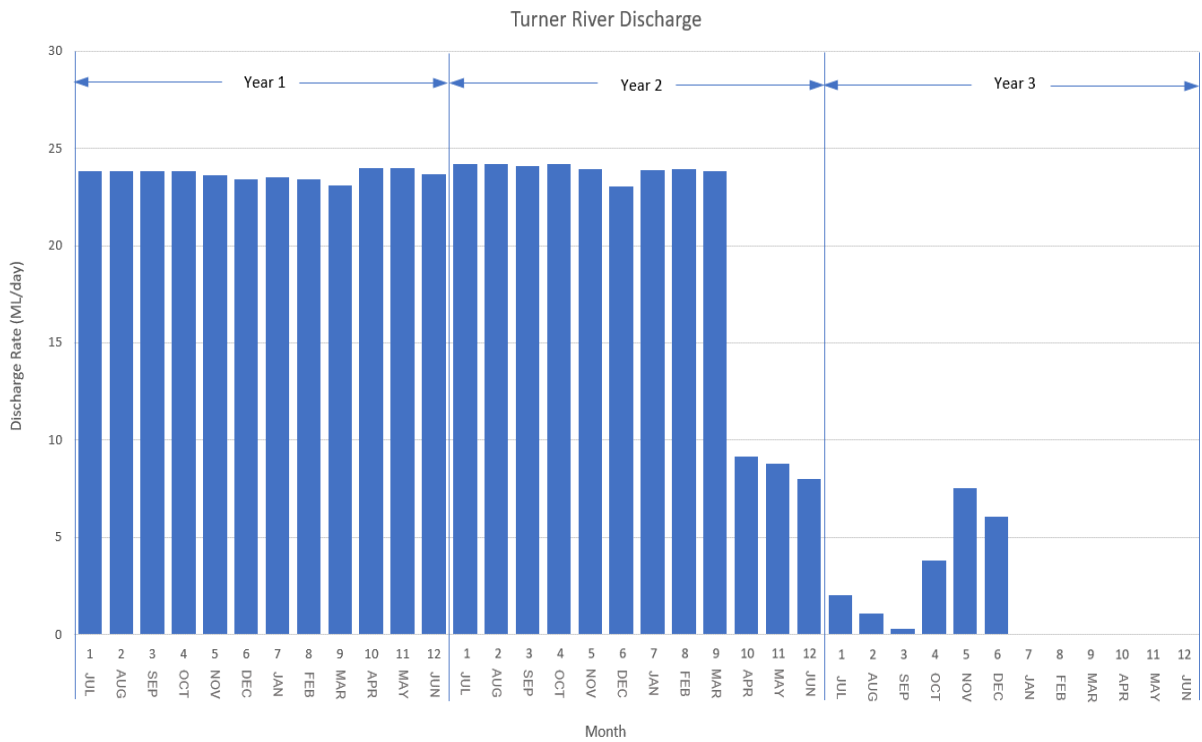


Figure 5 – Monthly Turner River discharge estimates

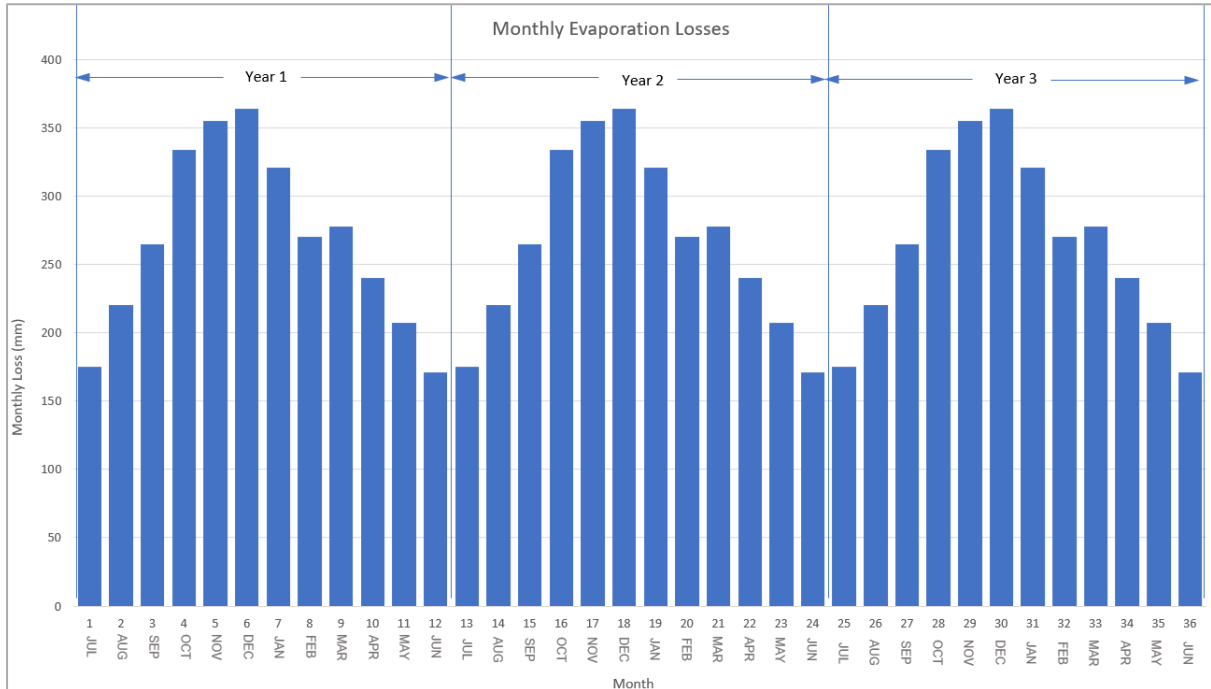


Figure 6 – Monthly evaporation losses

2.5. Infiltration Losses

Table 1 shows the infiltration losses applied to the model, varying with the distance from the discharge point. Infiltration rates are provided by Geowater, as documented in the accompanying hydrogeological report. The one-off volumetric loss applies to available storage above the water table. The volumetric loss varies between 110 and 150 ML per km of river reach.

In addition to the initial volumetric loss, an ongoing infiltration loss rate is applied to the model to account for continuing loss to groundwater. The rate varies from 0.08 to 0.53 ML per day for each km of river reach. Losses were applied as negative inflow time series boundary conditions. Additional details on the infiltration parameters are provided in the hydrogeological reporting under separate cover.

Table 1 – Summary of infiltration losses

River Chainage		One-off volumetric loss	Ongoing infiltration loss
Downstream of Discharge Point (km)	Upstream of Ocean Outlet (km)	ML per river reach km	ML/day per river reach km
0 - 2	65 - 67	140	0.08
2 - 6	61 - 65	130	0.24
6 - 10	57-61	110	0.20
10 - 30	37-57	150	0.24
>30	<37	150	0.53

2.6. 2D Mesh

A two-dimensional (2D) flow area was delineated to cover each of the primary flow paths of the Turner River, including areas with braided or split flows. Figure 7 shows the coverage extents. The 2D flow boundary covers an area of approximately 250 km².

A computational mesh spacing of 10 metres was applied to the low flow channels, with break lines applied along flow paths to orient cell faces with flow directions. Break lines were also applied along crests and grade breaks of existing terrain features. Areas outside of the main channel were assigned a mesh spacing of 50 metres, with interpolated cell sizes applied to intermediate areas.

HEC-RAS recognises the sub-grid terrain resolution within individual computational cells, and the flow transfer calculations between individual grid cells account for the geometry of the underlying surface at the terrain resolution of up to 3 m by 3 m along the Turner River.

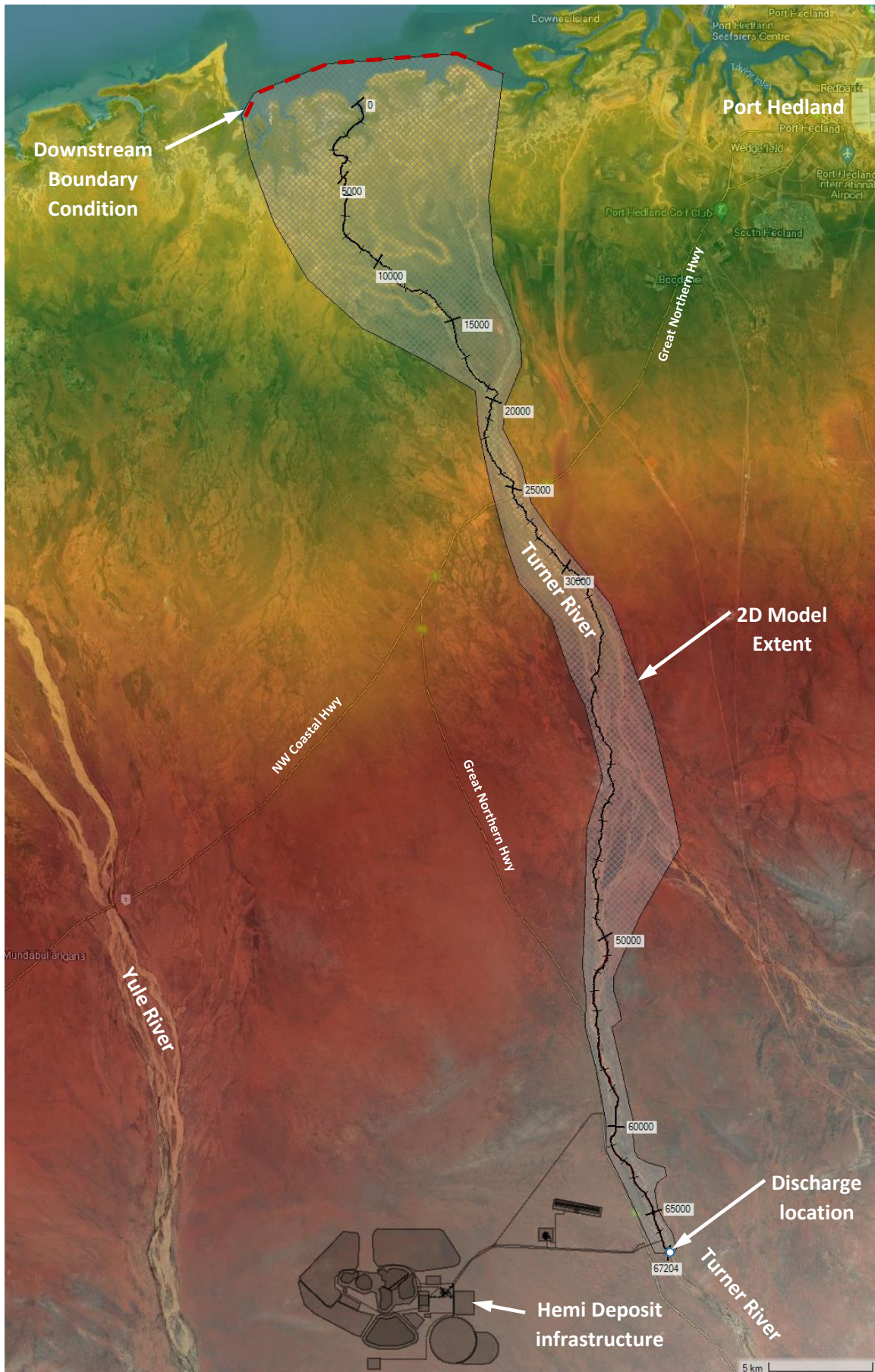


Figure 7 – Extents of model coverage

2.7. Roughness

The maximum discharge rate of 24.2 ML/day corresponds to a flow rate of approximately 0.28 m³/s. Flow directions in this range of flows are highly variable, and the channel is characterised by split flow paths with substantial meandering and braiding. Aerial photography indicates some vegetation within the low flow inundation extents. Figure 8 shows a drone photo of the Turner River near the proposed discharge location on 3 June 2022, with an approximate flow rate of 60 ML/day.

Based on a comparison to tabulated values in Australian Rainfall and Runoff guidance (Ball, et al., 2019), a uniform Manning's roughness coefficient of 0.05 was adopted for the Turner River discharge modelling. This value is slightly higher than typical values for clean, natural channels; however, it is considered appropriate given the relatively shallow flow depths and presence of vegetation.

Sensitivity analyses were applied for roughness coefficients ranging from 0.03 to 0.07. Flow widths corresponding to the range of modelled discharge rates near the discharge point vary by approximately +/- 20% within the applied range of roughness coefficients (Appendix D).



Figure 8 – Turner River looking downstream near proposed discharge location

2.8. Outflow Boundary Conditions

The downstream boundary condition was assigned a steady-state tidal elevation of 0.5 mAHD. Storm surge has not been accounted for in the discharge modelling. Sensitivity analyses indicate that the results upstream of the Great Northern Highway are not sensitive to typical fluctuations in tide levels or to climate change variations in sea level.

2.9. Structures

No culverts or bridge structures were included in the Turner River discharge model except where roadways are reflected in the terrain data. This approach assumes that flow rates are sufficiently low and steady to avoid throttling where raised embankments induce ponding.



2.10. Computational Settings

A 3-month simulation window was applied to the initial model runs to allow steady flow conditions to establish and initial volumetric losses to be absorbed. Subsequent months were modelled with steady inflow rates to establish relationships between discharge and top width along the Turner River until flows ceased or reached the downstream model boundary.

A variable time step was assigned based on a maximum Courant Number of 2.0. Using this option, HEC-RAS selects an adaptive time step based on the assigned computational mesh size and computed velocities. The adopted time step generally ranged between 10 and 20 seconds. Mass balance errors and water surface elevation convergence errors were checked to ensure model stability and that imbalances remained below reasonable thresholds, confirming compliance with Courant Number criteria in the published guidance (USACE, 2022).

The full momentum shallow water equation set was applied to all model runs. Except where otherwise noted, program defaults have been applied to all remaining coefficients, options, tolerances, and model settings.

2.11. Summary of Model Inputs

Table 2 summarises the model parameters applied to each of the runs associated with the Turner River discharge modelling.

Table 2 – Summary of model parameters

Model Parameter	Value
Inflow	Monthly Turner River discharge per schedule
Outflow	Constant ocean stage 0.5 mAHD
Simulation window	3 months
Computational time step	10-20 seconds
Computational mesh grid	10-50 metres
Roughness	0.03 - 0.07
Equation set	Full momentum
DEM grid resolution	3 metres



3. Turner River Discharge Model Results

3.1. Discharge versus Surface Area

Infiltration and evaporation loss rates along the Turner River are related to the surface area of the surface water in the channel. With the assumed loss rates applied, the linear discharge extents of surface water expression over time are primarily dependent on the relationship between discharge and surface area in each reach of the Turner River. This relationship is a function of the topography and bathymetry in the available terrain data sets.

In order to determine the discharge versus surface area relationship, a series of steady inflows ranging from 0.1 ML/day to 50 ML/day was applied to the model. The inundated area associated with each discharge rate was extracted at 5 km intervals along the Turner River and divided by the reach length to compute the average top width for each reach.

Figure 9 shows the average relationship between top width and discharge along the Turner River. As shown in the figure, the minimum top width is approximately 10 m, even for very low flow rates. This effect is related to intermittent ponding along the profile that occurs where the flow path is interrupted by high spots in the terrain. The average top width only decreases below approximately 10 m when upstream inflows are completely cut off and ponding is reduced due to evaporation and infiltration.

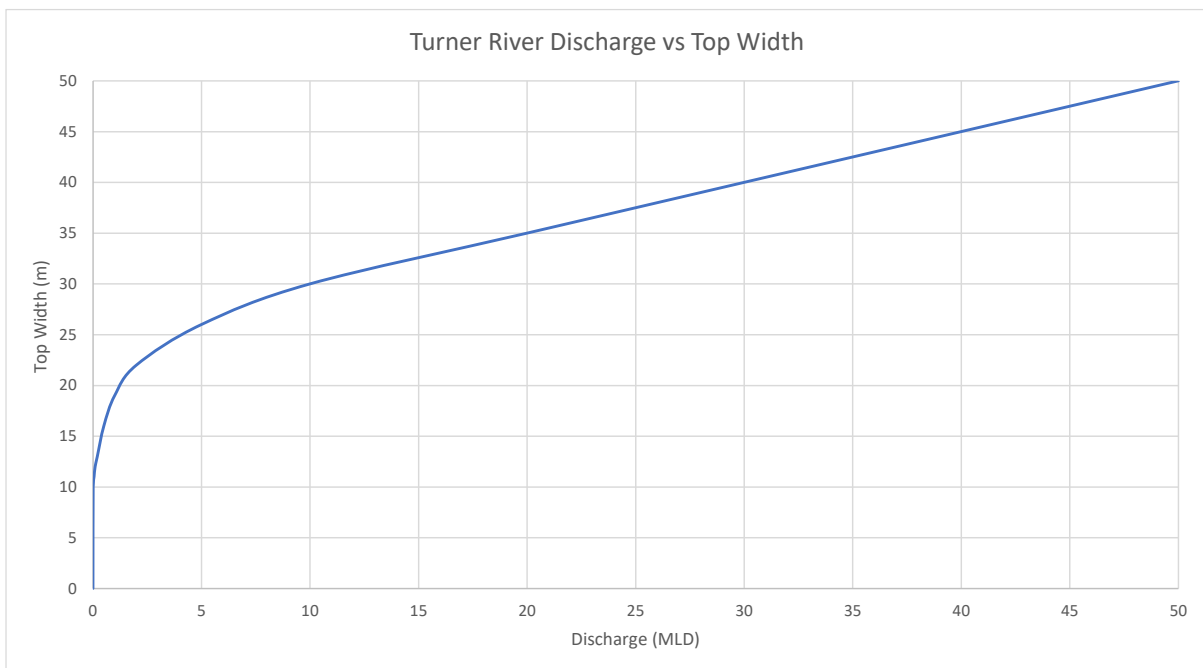


Figure 9 – Discharge versus top width relationship for Turner River

3.2. Discharge versus river chainage

Figure 10 shows the fluctuations in discharge rate over the three-year discharge period. Individual hydrographs are shown at 10-km increments between the discharge point and the Great Northern Highway crossing. As shown in the figure, the initial volumetric loss is reflected as a downstream wetting front progressing at a rate of approximately 5 km per month.

Surface water flows at the Great Northern Highway, located approximately 40 km downstream of the discharge point, begin in Month 8 (February) of Year 1 and cease in Month 10 (April) of Year 2.

No surface water flows were observable in the model at the index sections located 50 km and 60 km downstream of the discharge point. Surface water flows do not reach the coastal outlet, located just over 67 km downstream of the discharge point.

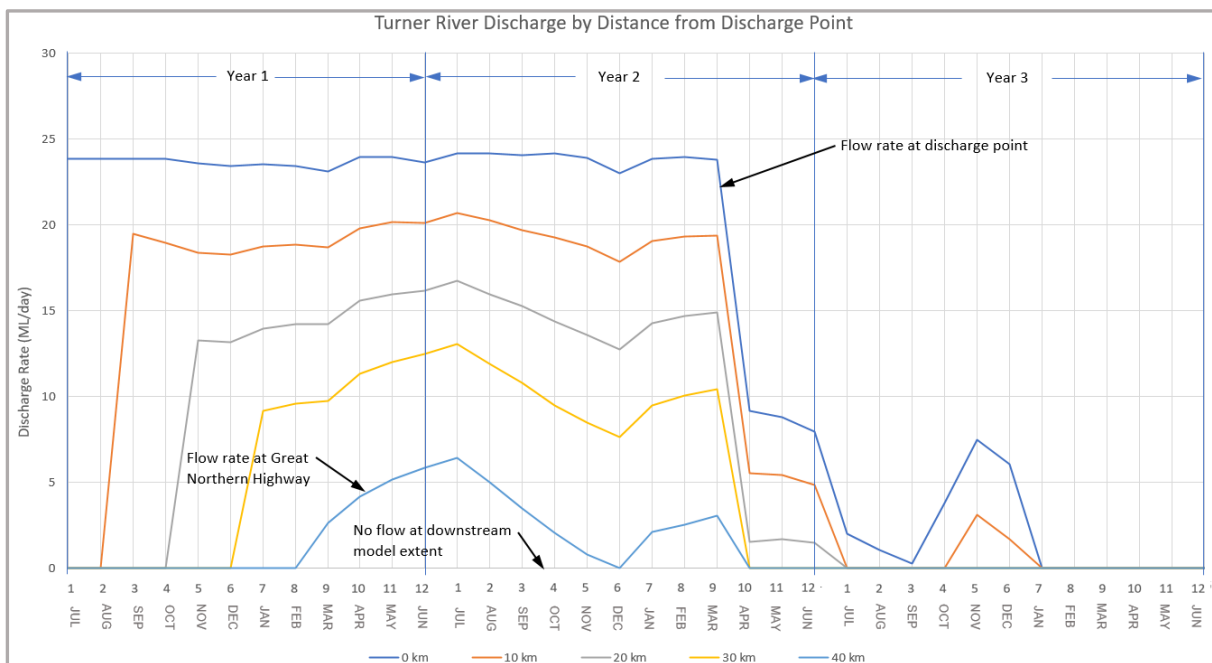


Figure 10 – Flow rate over 3-year discharge period at 10 km increments along Turner River

Figure 11 shows the variation of discharge rate along the Turner River for two selected points in time (at the end of Year 1 and Year 2). Month 12 (June) of Year 1 represents high flow conditions; Month 12 (June) of Year 2 represents low flow conditions. As shown in Figure 11, flows at the Great Northern Highway (approximately 40 km downstream of the discharge point) are approximately 25% of the upstream inflow rate at the end of Year 1; at the end of Year 2, flows cease upstream of the Great Northern Highway.

The horizontal axis in Figure 11 shows the distance downstream of the discharge point, with chainage references listed as distance from the ocean outlet as shown in Figure 3.

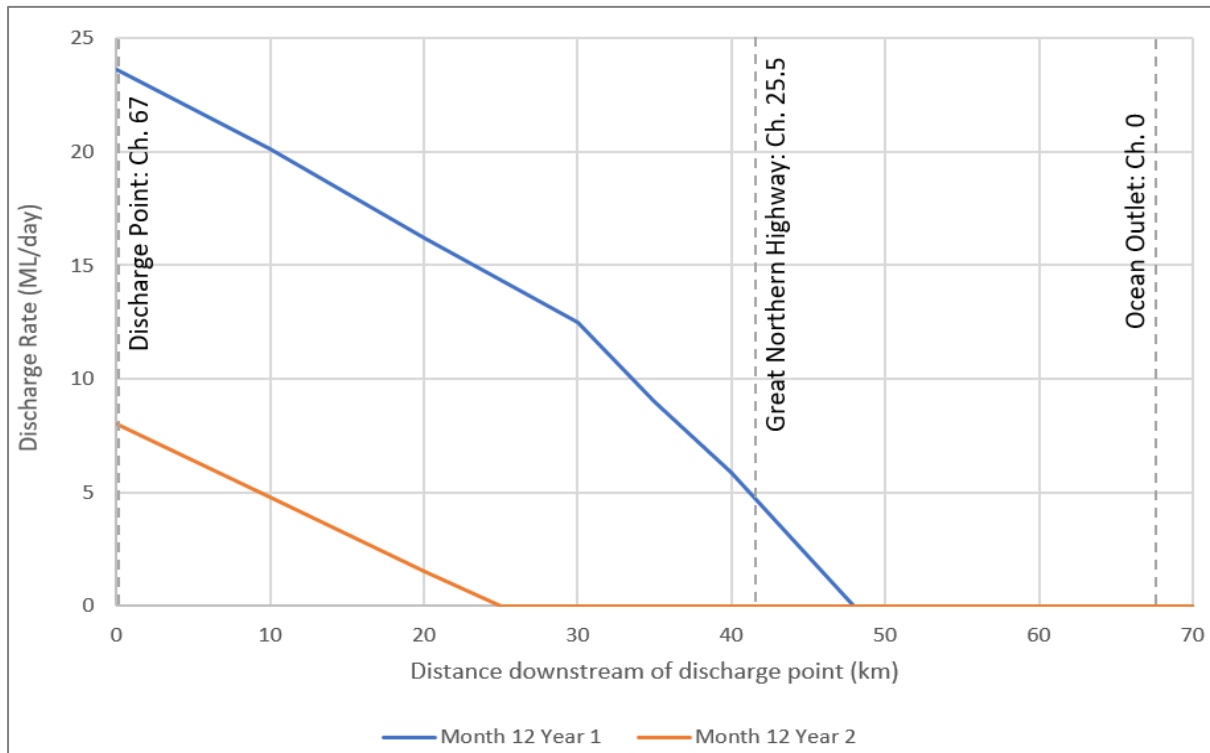


Figure 11 – Flow rate along Turner River for high and low flow periods

3.3. Inundation extents

Figure 12 shows the inundation extents for high and low flow periods in the vicinity of the discharge point. Additional plan view figures are shown for 10-km increments in Appendix C. Differences between flow scenarios are relatively minor, due to the ponding that results from undulations in the underlying topography. Whilst the high discharge rate reflects an approximate four-fold increase over the low discharge rate, for example, the inundated area typically varies by less than a factor of two. A June 2022 flow event in the Turner River resulted in runoff rates equal to approximately twice the maximum modelled discharge rate. A comparison of drone photographs with the modelled results confirmed the approximate distribution of the flow represented by the model results.

3.4. Profiles

Figure 13 shows the longitudinal profile of the Turner River. Water surface elevation profiles are indistinguishable at the scale of the figure; inset figures are shown for 10-km increments in Appendix A for additional resolution. It should be noted that profiles are shown with extreme vertical exaggeration (approximately 500 times).

Figure 14 shows the flow depths along the Turner River for the selected high and low flow scenarios. Flow depths are characterised by intermittent near-zero depths where controlling topography ponds water that spills through and ponds in the next downstream pool. Appendix A includes depth profiles for 1-km and 5-km running averages to reflect depth trends more clearly. Flow depths decrease in the downstream direction, with typical flow depths of 0.4 m in the upstream reaches and 0.1 – 0.2 m in the downstream reaches near the coastal outlet.

Figure 15 shows the velocity profiles along the Turner River for the selected high and low flow scenarios. Average velocities are approximately 0.1 m/s, exhibiting a decreasing trend in the downstream direction.

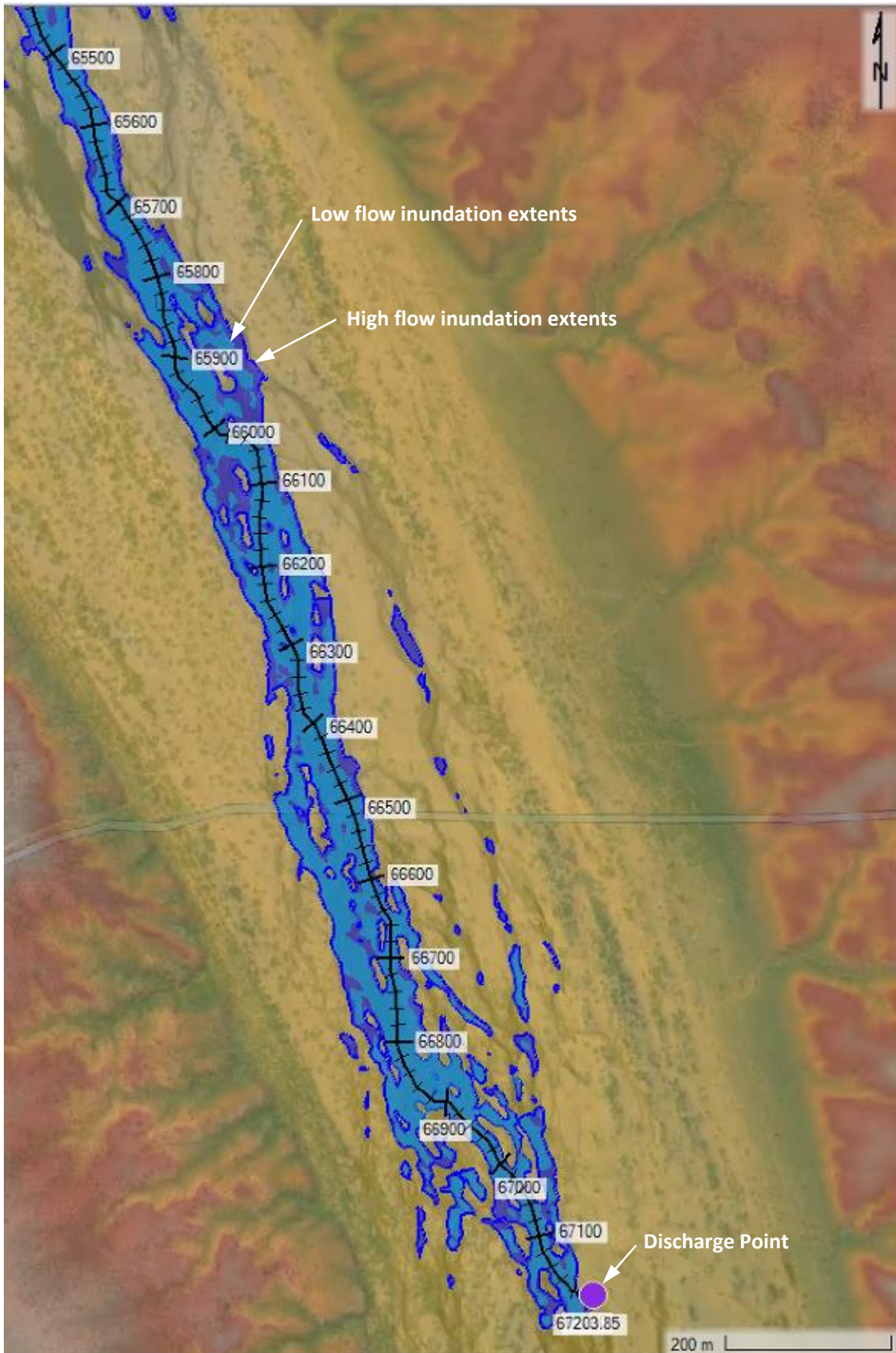


Figure 12 – Flow rate by river chainage for selected high and low flow periods

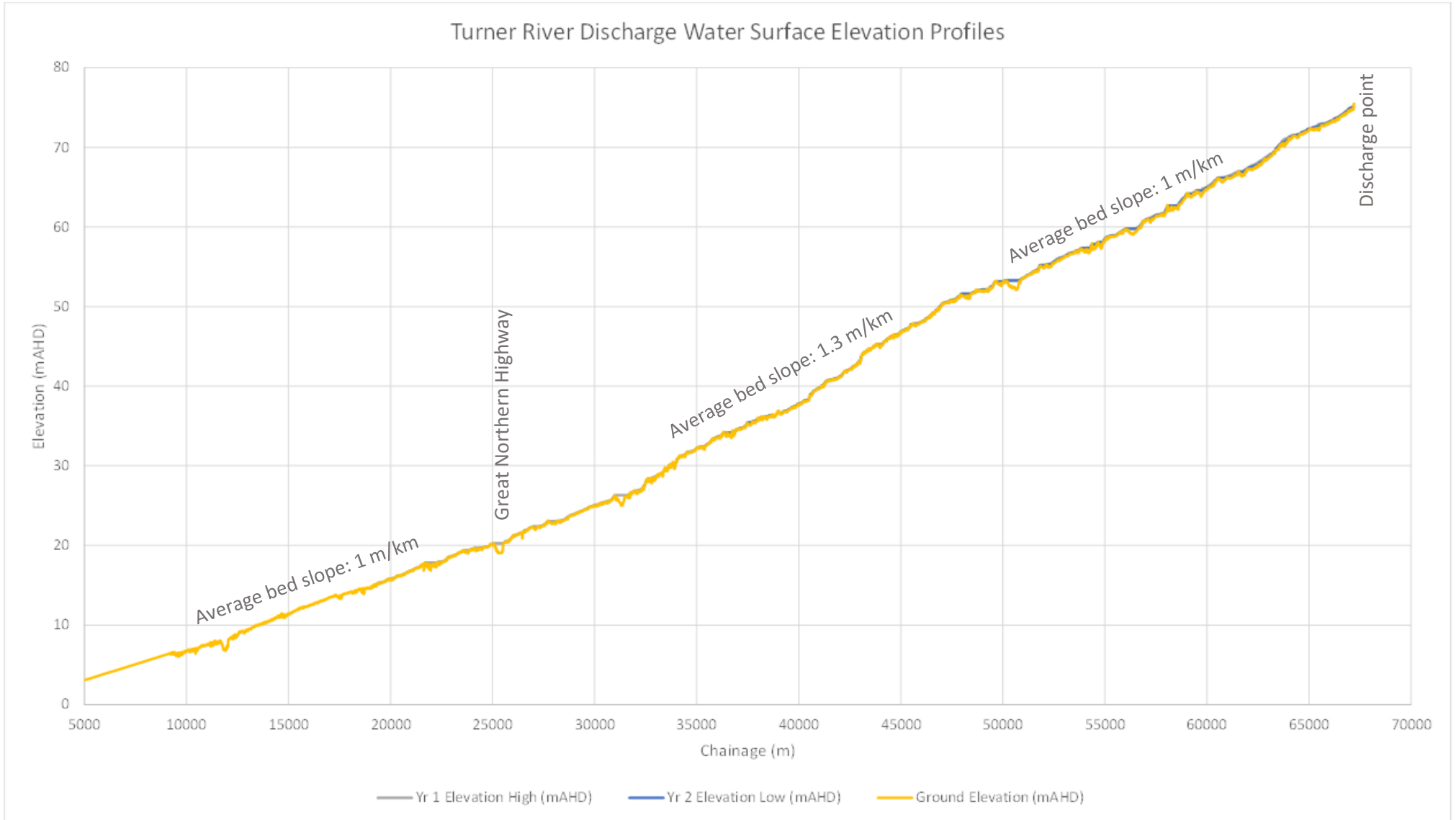


Figure 13 – Water surface elevation by river chainage for selected high and low flow periods

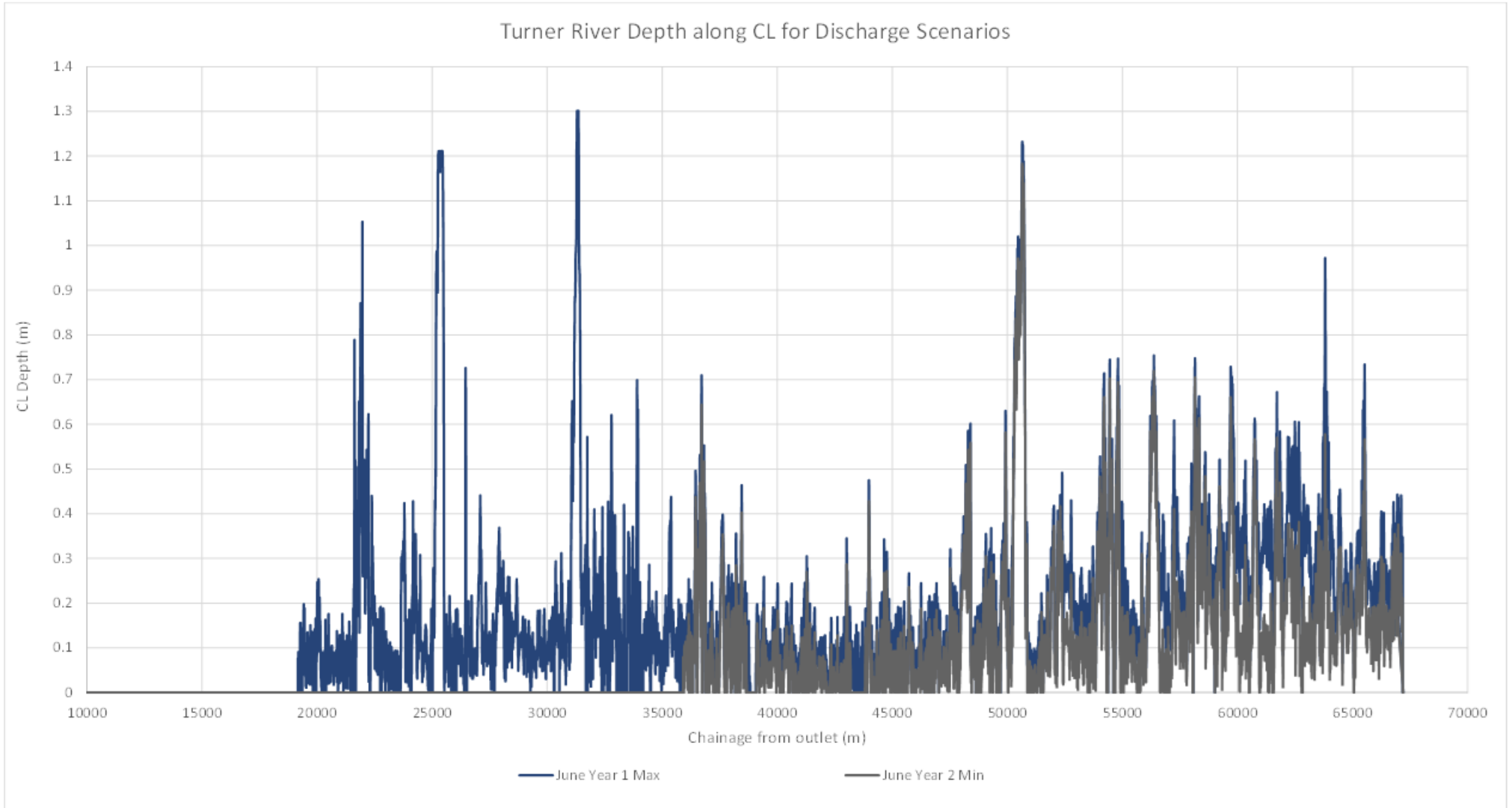


Figure 14 – Flow depth by river chainage for selected high and low flow periods



Turner River Velocity along CL

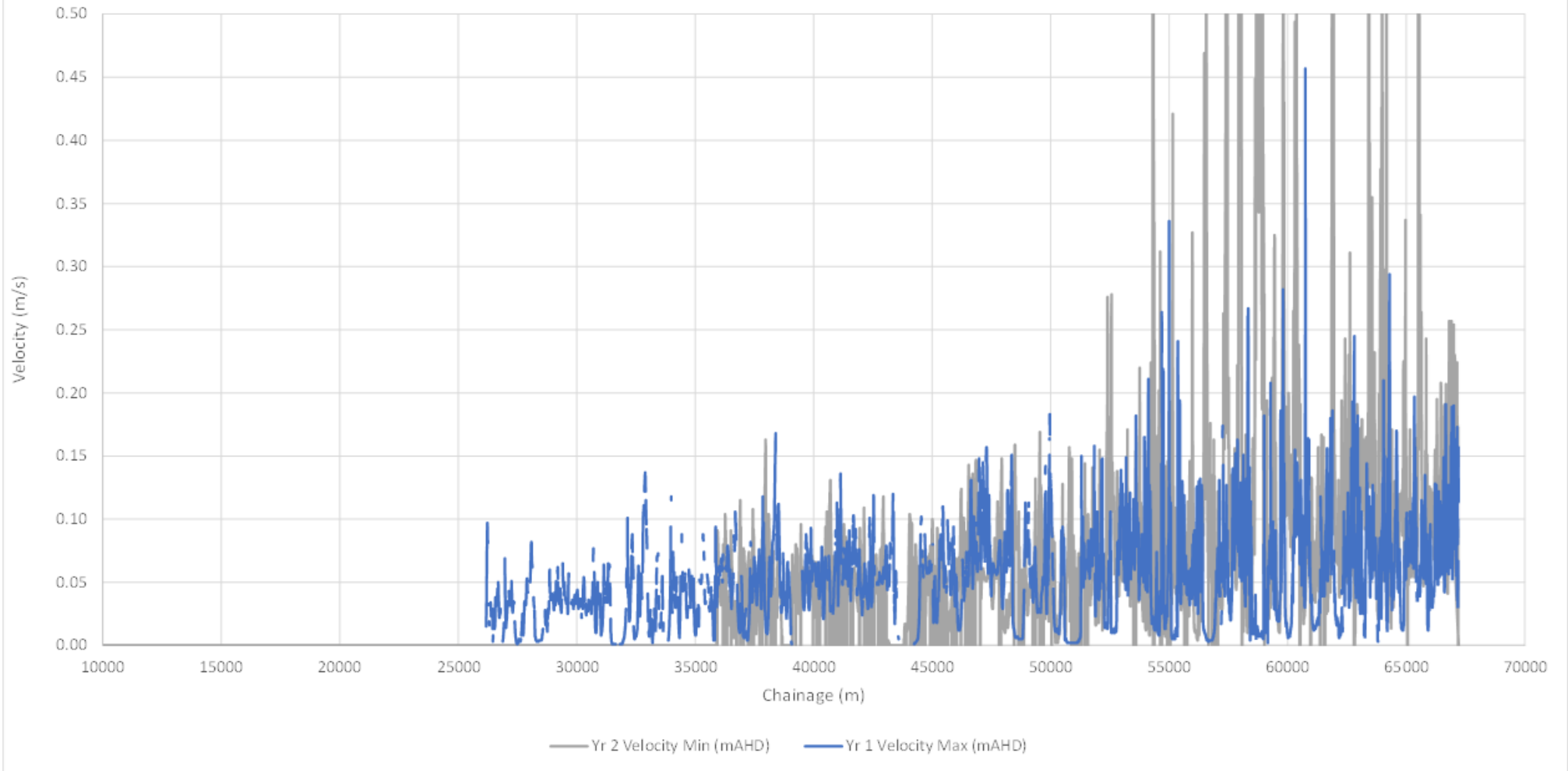


Figure 15 – Velocity by river chainage for selected high and low flow periods

3.5. Cross section results

Figure 16 shows the water surface elevations associated with the high and low flow conditions along a cross section at the discharge point. Maximum flow depths are approximately 300 mm with a top width of approximately 50 m. It should be noted that cross sections are shown with extreme vertical exaggeration (approximately 100 times). At 1:1 scales, the flow depths are not discernible.

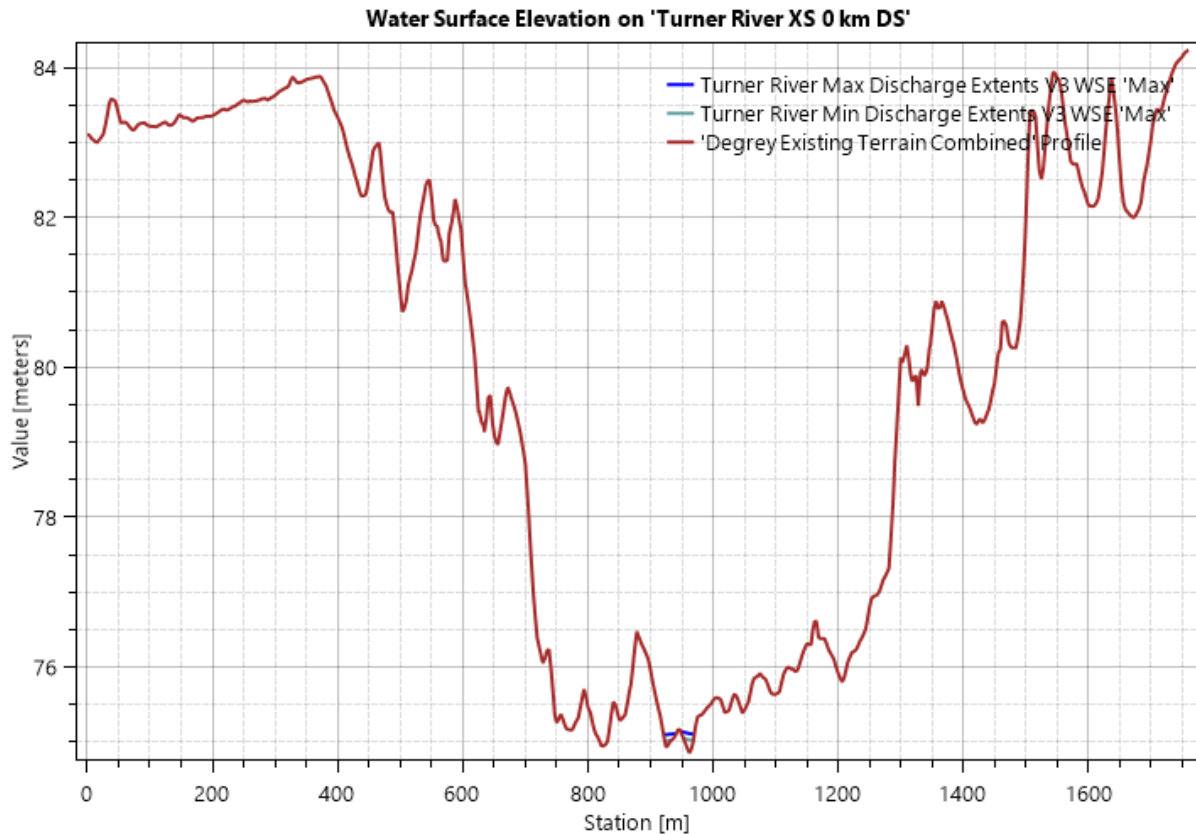


Figure 16 – Cross section at discharge point showing high and low flow water surface

Appendix B shows additional cross sections at 2 km increments for the first 20 km downstream of the discharge point, and at 5 km increments for the remaining Turner River reach. As shown in the figures, the Turner River channel is approximately 1 km wide, with the maximum discharge generally less than 50 m in width.

3.6. Comparison to flood flows

Flood flows are shown for comparison only and have not been accounted for in the discharge modelling. Figure 17 and Figure 18 show typical cross sections with water surface elevations from Turner River discharge compared to flood flows. Top widths of the flood flows are substantially larger than the wetted widths associated with the maximum discharge rate. Figure 19 shows the same comparison in profile view. Figure 20 shows a hypothetical situation with discharge subject to evaporation only (no infiltration).

The maximum discharge (approximately 0.5 m³/s) represents less than 0.1% of the 10% AEP flood flow on a peak flow and volumetric basis.

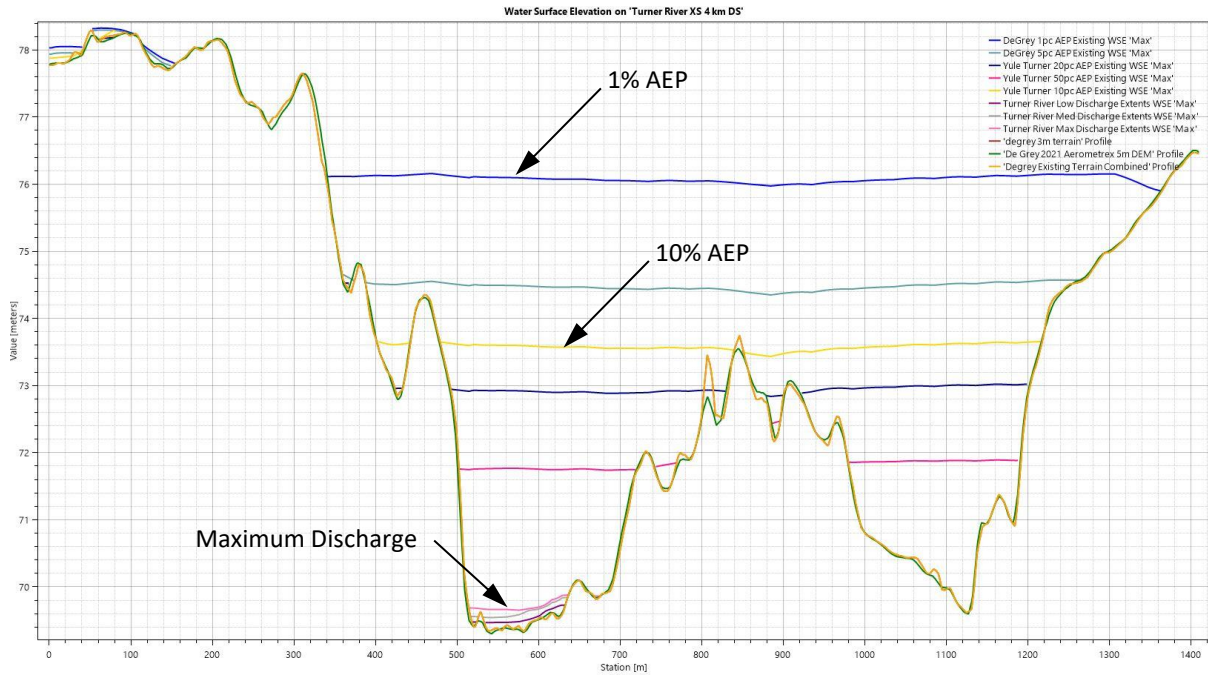


Figure 17 – Cross section 4 km downstream of discharge point showing flood flows

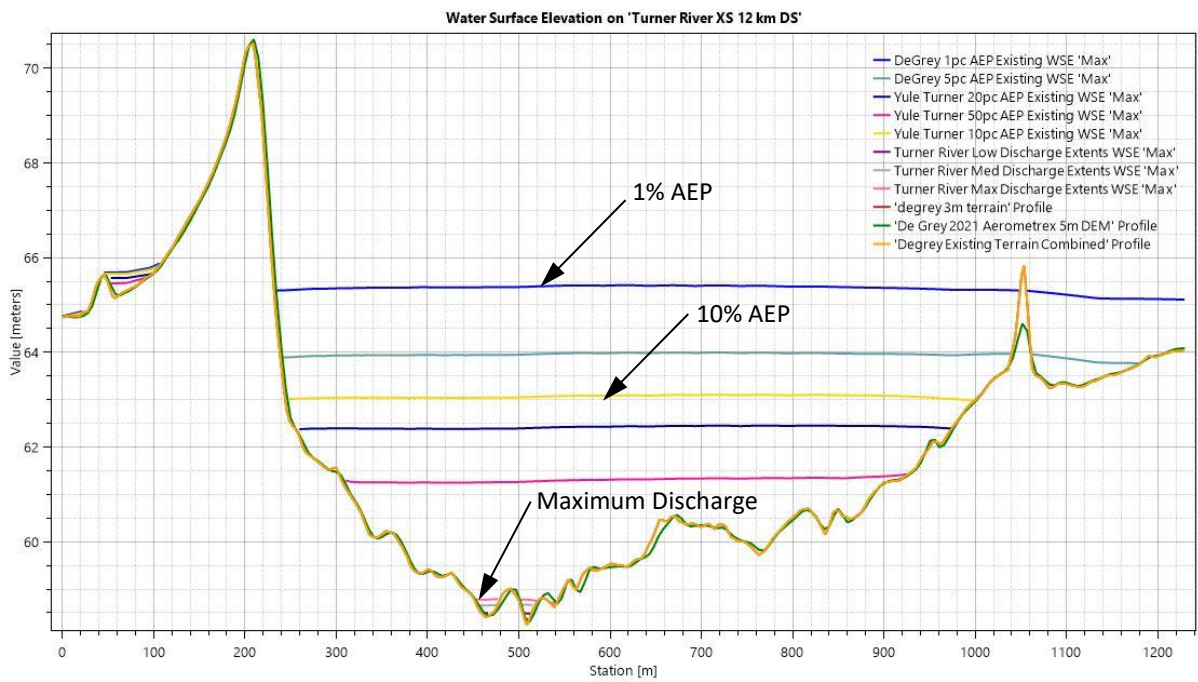


Figure 18 – Cross section 12 km downstream of discharge point showing flood flows

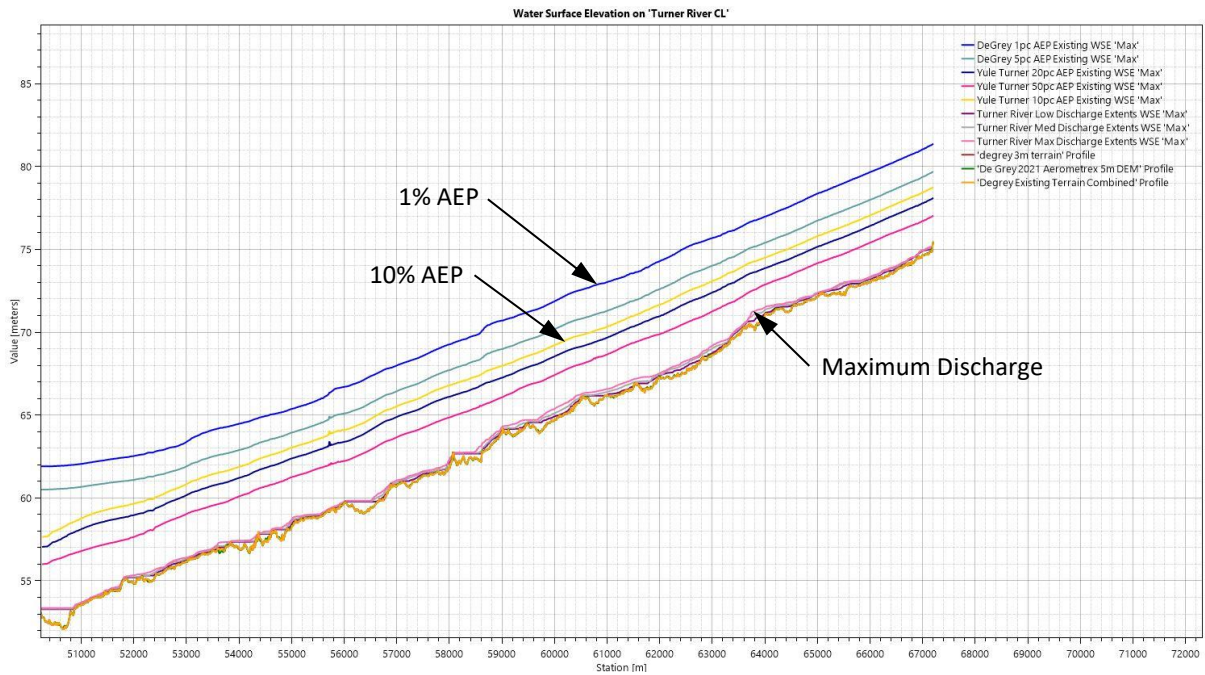


Figure 19 – Water surface elevation profiles for Turner River discharge versus flood flow

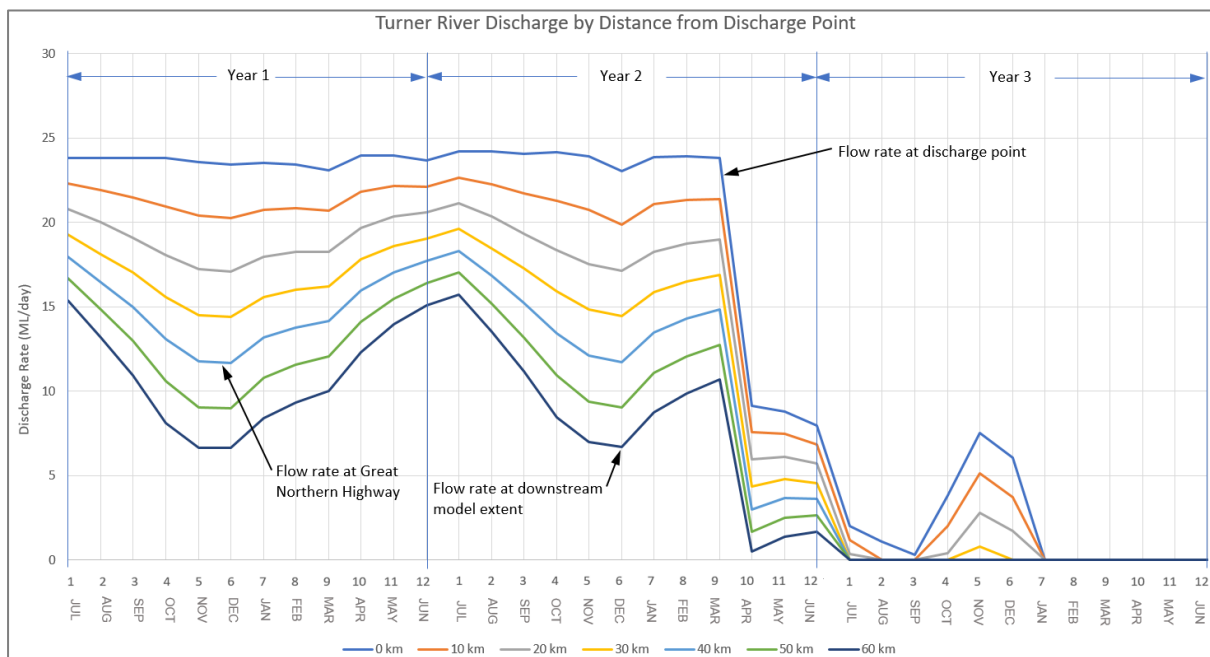


Figure 20 – Discharge rates for evaporation losses only

Appendix D includes sensitivity results for a range of infiltration rates. As shown in the figures in Appendix D, varying the ongoing infiltration rate by +/- 25% results in a shift of approximately +/- 10 km in the downstream wetting front extent.



4. Closure Conditions Flood Model Setup

4.1. Approach

A proposed conceptual closure footprint was added to the flood model. Closure features were separated from external flows with a hypothetical bund to assess maximum impacts from impinging flows. A 2D flood model was used to assess flow depths and velocities for flood events ranging from the 1% AEP to the Probable Maximum Flood (PMF). The model simulates local rainfall and flood flows in the Turner and Yule River. Modelling was conducted using HEC-RAS 6.3.1 (USACE 2022), with all model settings matching those presented in the Mallina Gold Hydrology report (Surface Water Solutions, 2022) except where otherwise noted below.

4.2. Closure Features

Figure 21 shows the closure footprint applied to the flood model. The closure footprint was applied as a perimeter bund that prevents overtopping in all scenarios up to the PMF. The bund was added as a terrain feature to the 3m x 3m resolution DEM described in Chapter 2.

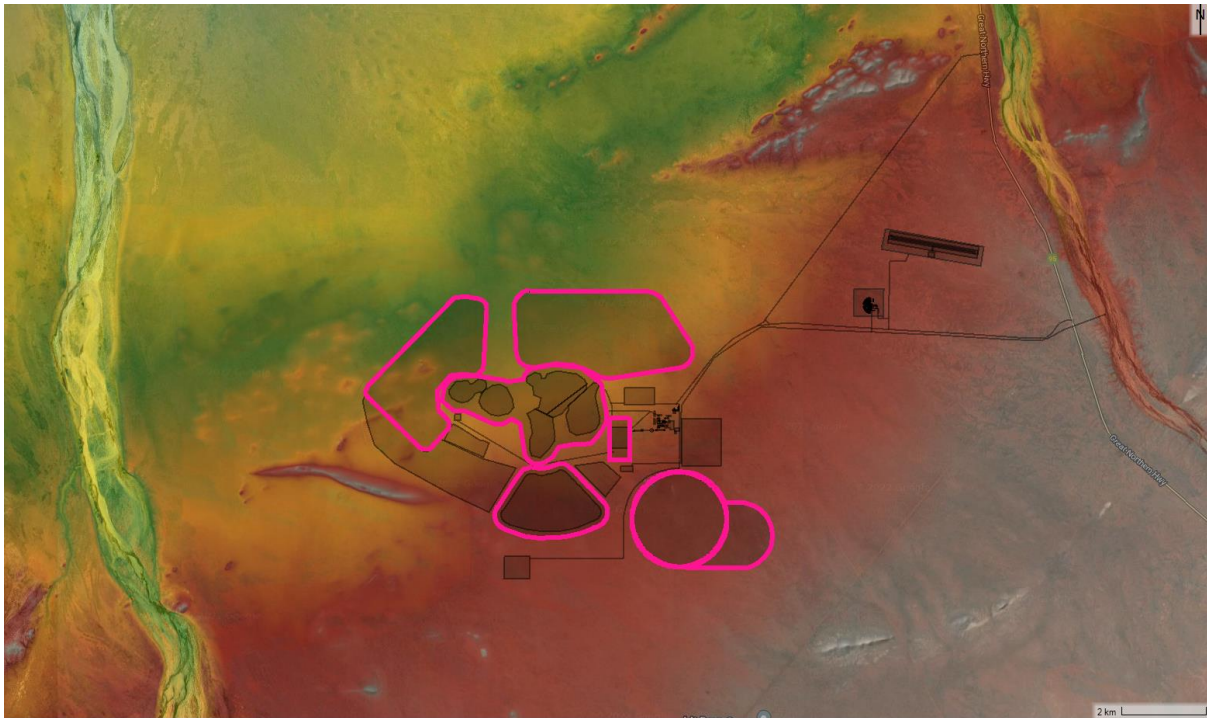


Figure 21 – Closure footprint applied to the flood model

4.3. Precipitation

Precipitation intensities and temporal patterns were extracted from the ARR data hub and Bureau of Meteorology (BoM) Intensity-Frequency-Duration (IFD) data for the Turner River catchment, Yule River catchment, and the local Hemi deposit areas. ARR and BoM data are available for storm events up to the 1 in 2,000 AEP. For less frequent storms, interpolation between the 1 in 2000 AEP event and the probable maximum precipitation (PMP) is required. Figure 22 shows the AEP associated with the PMP for the three modelled catchment areas. The associated AEP varies from approximately 1 in 100,000 to 1 in 4,000,000.

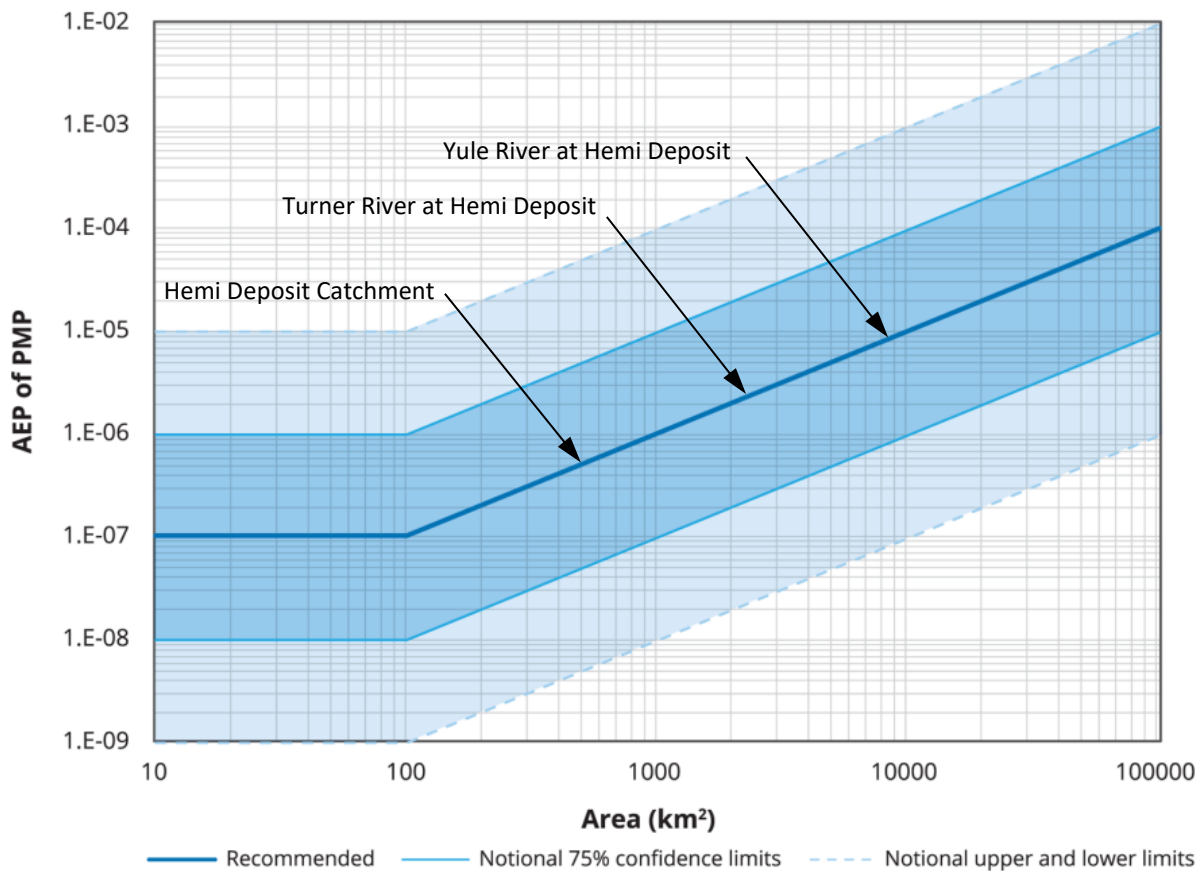


Figure 22 – Annual Exceedance Probability for PMP (Ball, et al., 2019)

PMP precipitation depths for the Yule and Turner River basins were developed using the revised Generalised Tropical Storm Method (GTSMR) (BoM 2005). Precipitation depths for the local catchment area were developed using the Generalised Short-Duration Method (GSDM) (BoM 2003).

The catchments are located within the Coastal Zone. Based on the available topography, a smooth terrain category was applied without an elevation adjustment factor. Relevant moisture adjustment and decay factors were applied to the initial depths, resulting in a 72-hour precipitation depth of 1690 mm for the Turner River and 1400 mm for the Yule River catchment. A 6-hour rainfall depth of 590 mm was estimated for the local catchment. Recommended temporal patterns were applied to the rainfall and the resulting patterns were applied with a rain-on-grid or direct precipitation approach.

Due to the large variation in catchment sizes, coincident PMF events in the local catchment and the Yule and Turner River basins are not considered. Events were modelled separately, and figures represent a composite of the maximum flood condition from each of the three catchments.

Flow rates for the Yule River and Turner River were extracted from a range of sources, as cited in the Mallina Gold Hydrology report. These flow rates were extrapolated using the assigned AEP and compared with the flow rates resulting from the rain-on-grid modelling using the AEP values in Figure 22. The applied flow rates were found to show reasonable correlation in light of the inherent uncertainties presented in the 2022 hydrology report, matching the peak flow rates within 25%.



Figure 23 shows the precipitation depths by annual exceedance probability for selected durations. Figure 24 shows the peak discharge rates for the Yule and Turner Rivers for the same range of annual exceedance probabilities.

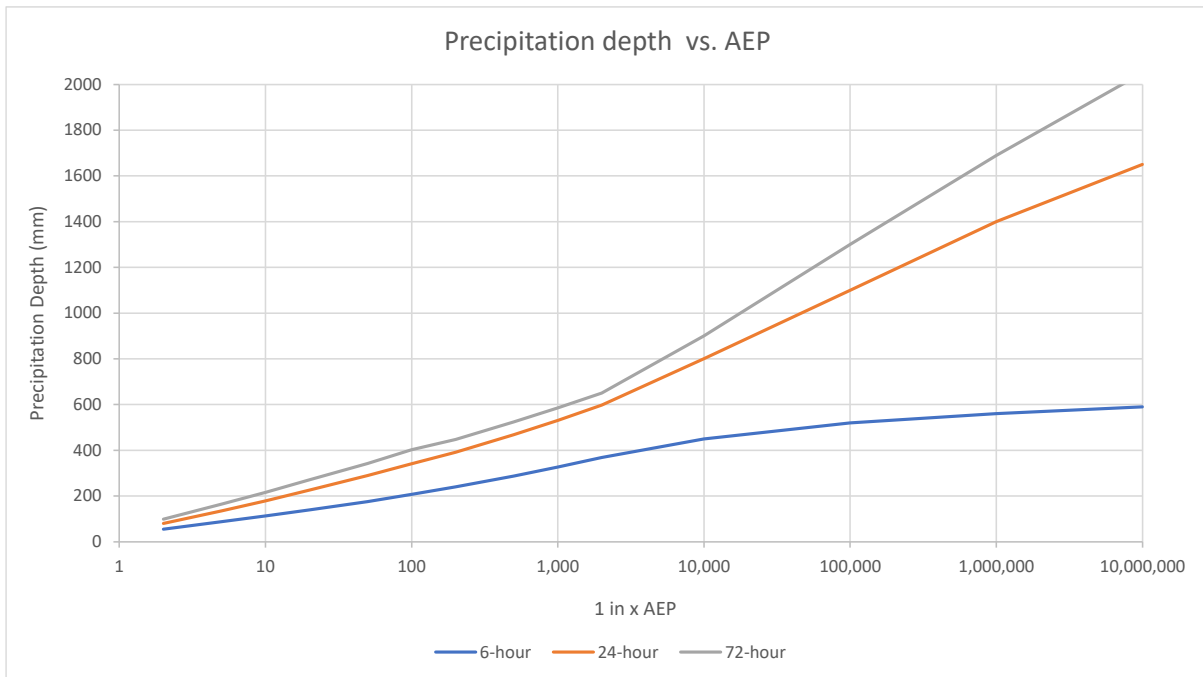


Figure 23 – 6-hour, 24-hour, and 72-hour precipitation depths by AEP

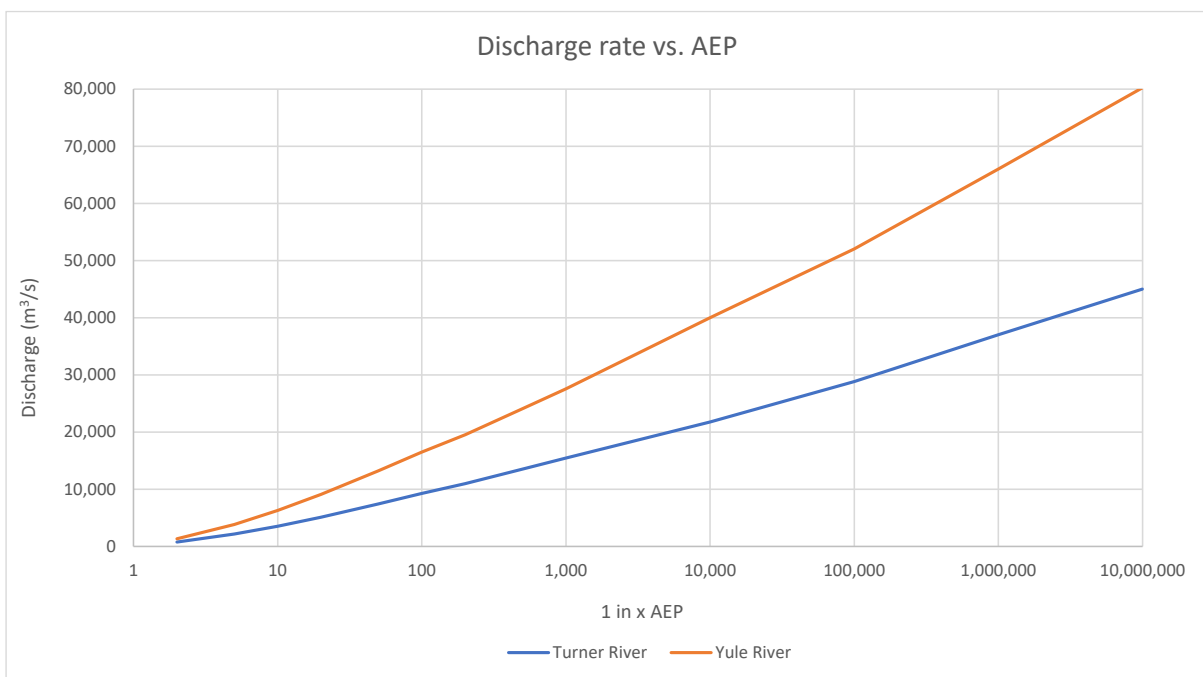


Figure 24 – Turner River and Yule River peak discharge rates by AEP



5. Closure Conditions Flood Model Results

5.1. Inundation Extent

Figure 25 shows the 1% AEP maximum inundation extents with the closure features incorporated. Results are shown with a 5cm display threshold. 1 in 1000 AEP results are shown in Figure 27, and the PMF results are shown in Figure 29.

5.2. Velocity

Figure 26 shows the 1% AEP maximum velocities with the closure features incorporated. 1 in 1000 AEP results are shown in Figure 28, and the PMF results are shown in Figure 30. High velocities (> 2 m/s) are shown in red in the figures. Indicative velocity-based rip rap sizing criteria are included in the Mallina Gold Hydrology report (Surface Water Solutions, 2022). Any crossings or closure features left in place near the river channels may be subject to erosive forces and would require riprap or other scour countermeasures for stability.

Maximum velocities around the site perimeter are generally limited to less than 1 m/s, indicating that rip rap protection would not be required to protect bunds from external flows in the assessed events.

5.3. Profiles

Figure 31 shows the maximum water surface elevations for 1% AEP, 1 in 1000 AEP, and PMF events along the southern perimeter of the closure features. Profile alignments with chainage reference are shown in the plan view figures; profiles are included where the closure features would be subject to impinging flows. Maximum depths are shown in Figure 32, and maximum velocities are shown in Figure 33.

The depths around the external perimeter are indicative of the required bund heights to protect against each event; freeboard would need to be added in keeping with adopted design criteria. Flow depths range up to approximately 3.5 metres in the PMF event; the indicated depths may be reduced with the incorporation of perimeter drains.

Velocities along the profile alignments are indicative of the erosive forces that the external bunds would be subjected to.

Figure 34 shows the maximum water surface elevations for 1% AEP, 1 in 1000 AEP, and PMF events along the eastern perimeter. Maximum depths are shown in Figure 35, and maximum velocities are shown in Figure 36.

5.4. Alternatives

Further design efforts may incorporate closure bunds designed as water-retaining features where flows are to be routed around the outside of the bunds. Closure drains may be incorporated into the closure design to facilitate drainage away from perimeter bunds and prevent long-term standing water following flood events. Closure drainage features would require additional sedimentation and scour assessment to ensure that they can function maintenance-free in the post-closure period. The relative location of the closure features may be adjusted to incorporate any benefits derived from including flow paths between the features.

Rather than routing flows around the external site perimeter, it may be beneficial to incorporate bunds and drains that direct runoff into closed pits in order to accelerate groundwater recharge. These options may be explored further in conjunction with the hydrogeological analyses as the closure designs progress.

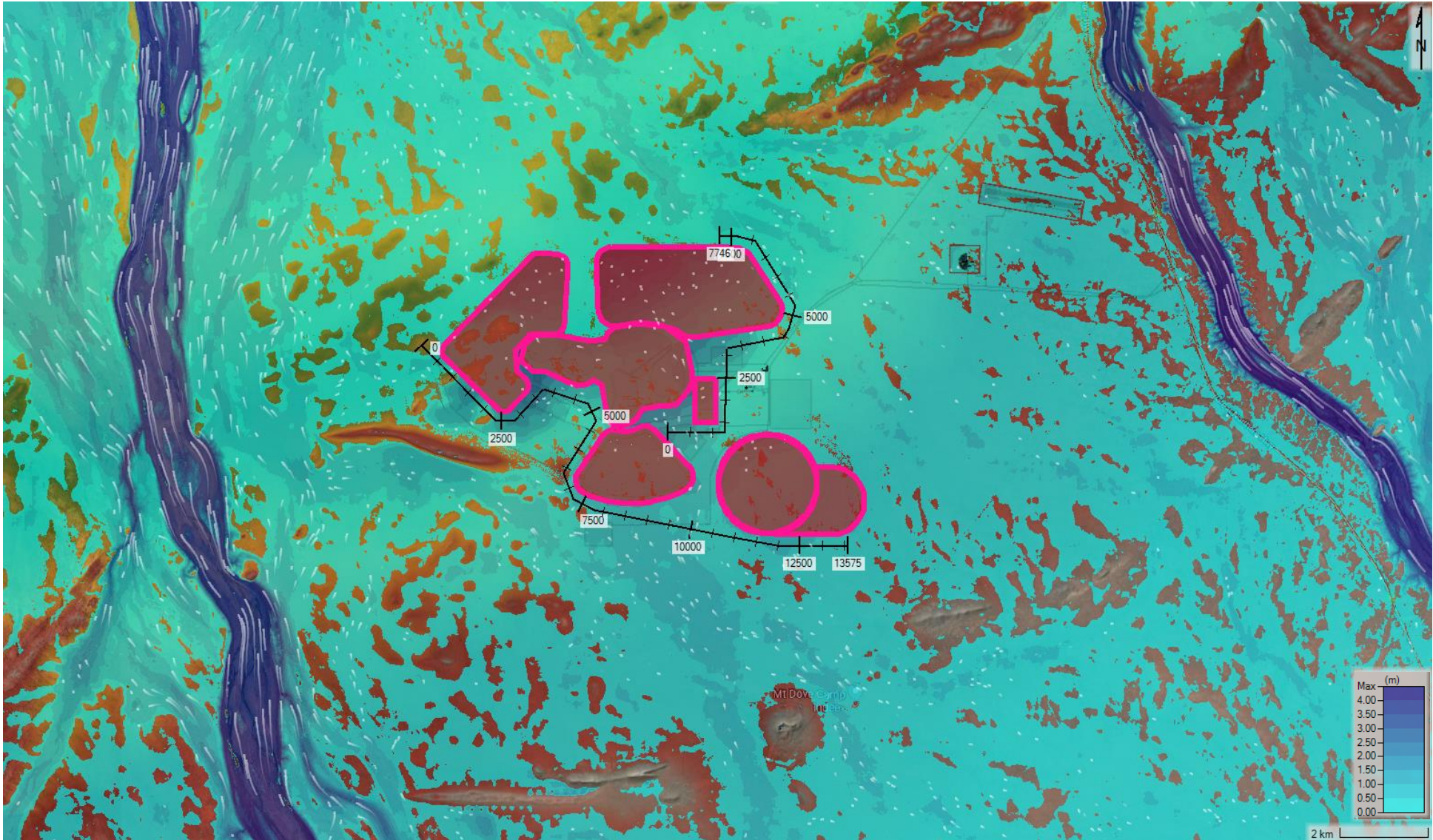


Figure 25 – Maximum 1% AEP water surface elevation for conceptual closure scenario

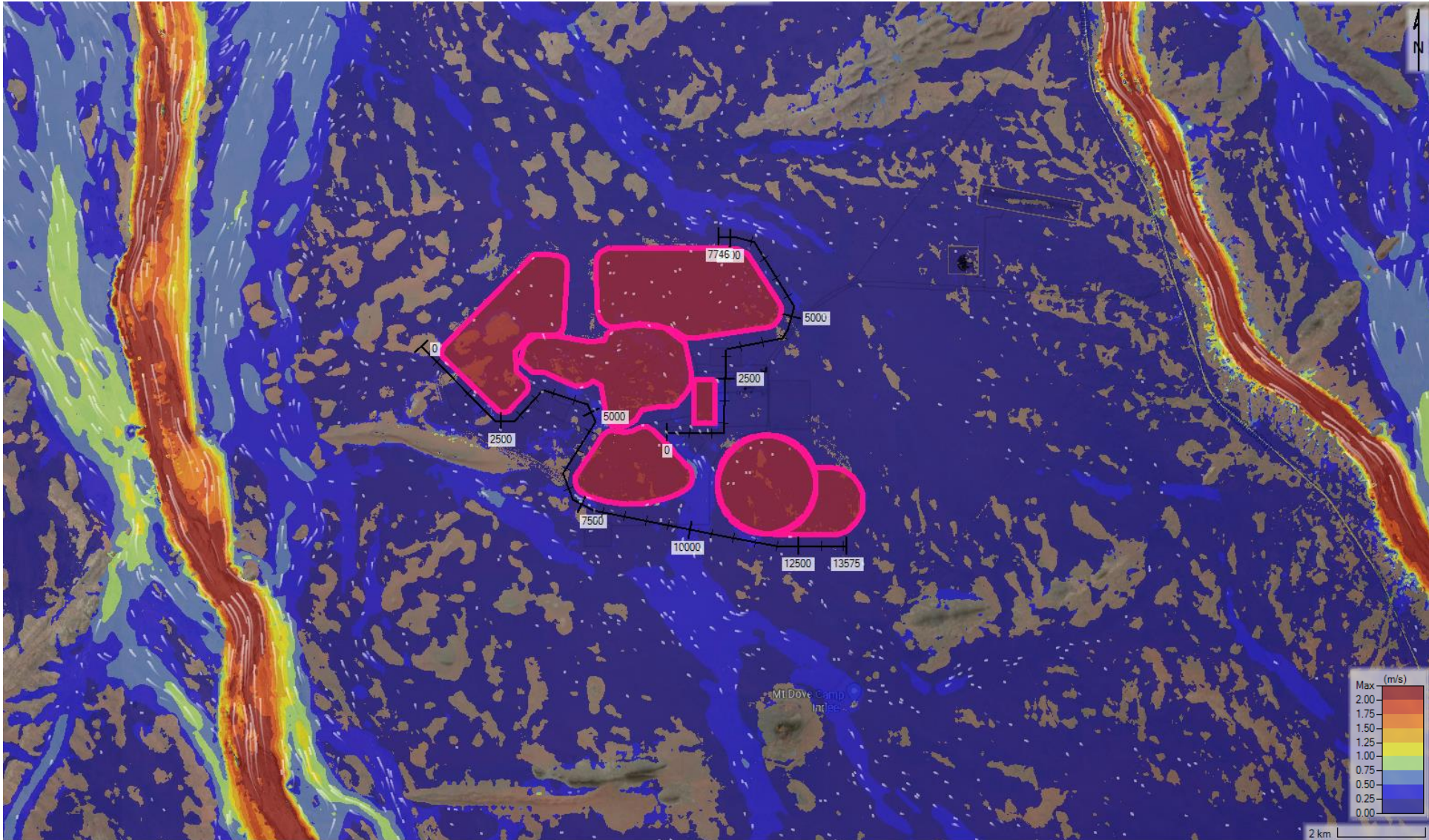


Figure 26 – Maximum 1% AEP velocity for conceptual closure scenario

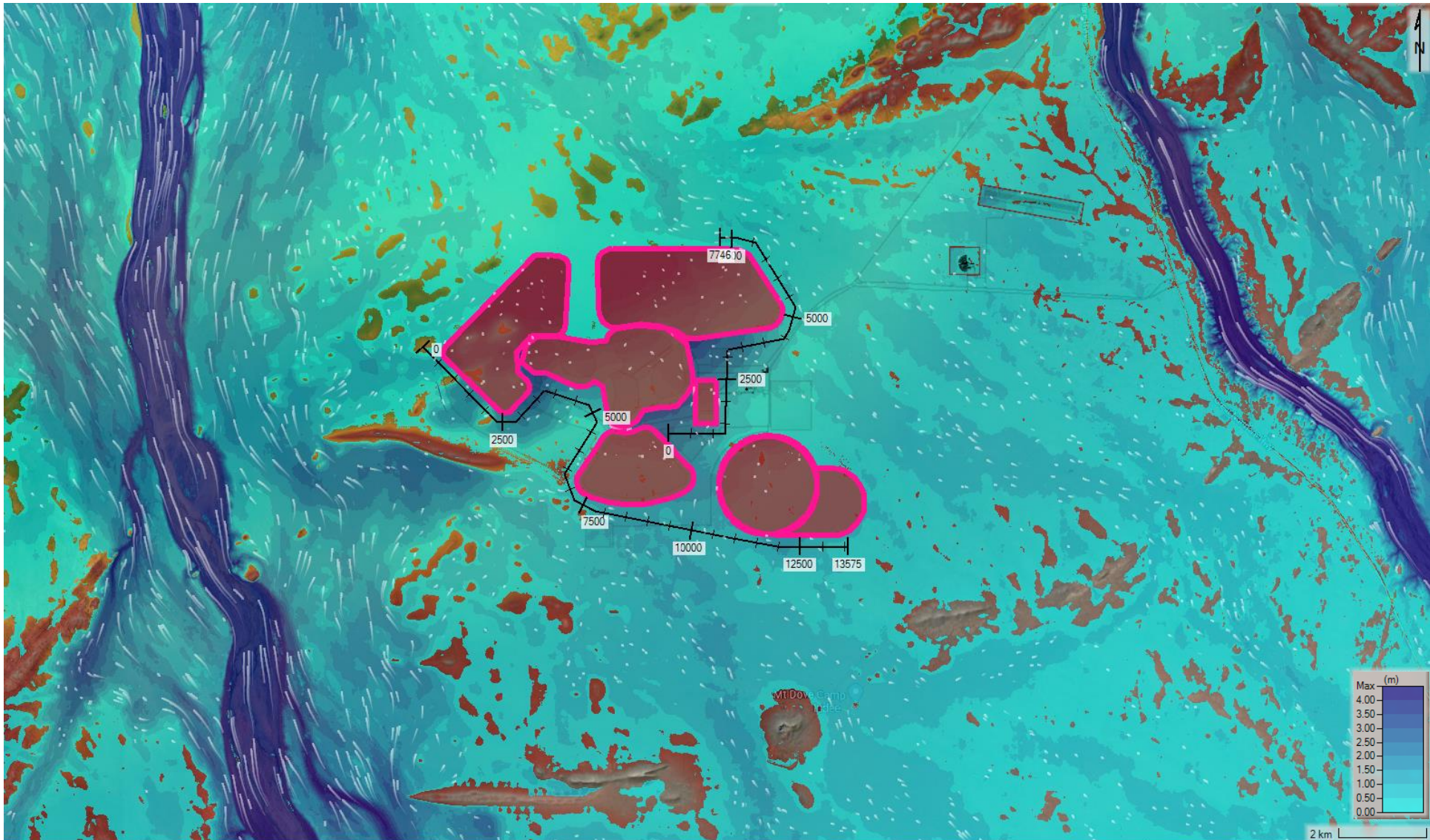


Figure 27 – Maximum 1 in 1000 AEP water surface elevation for conceptual closure scenario

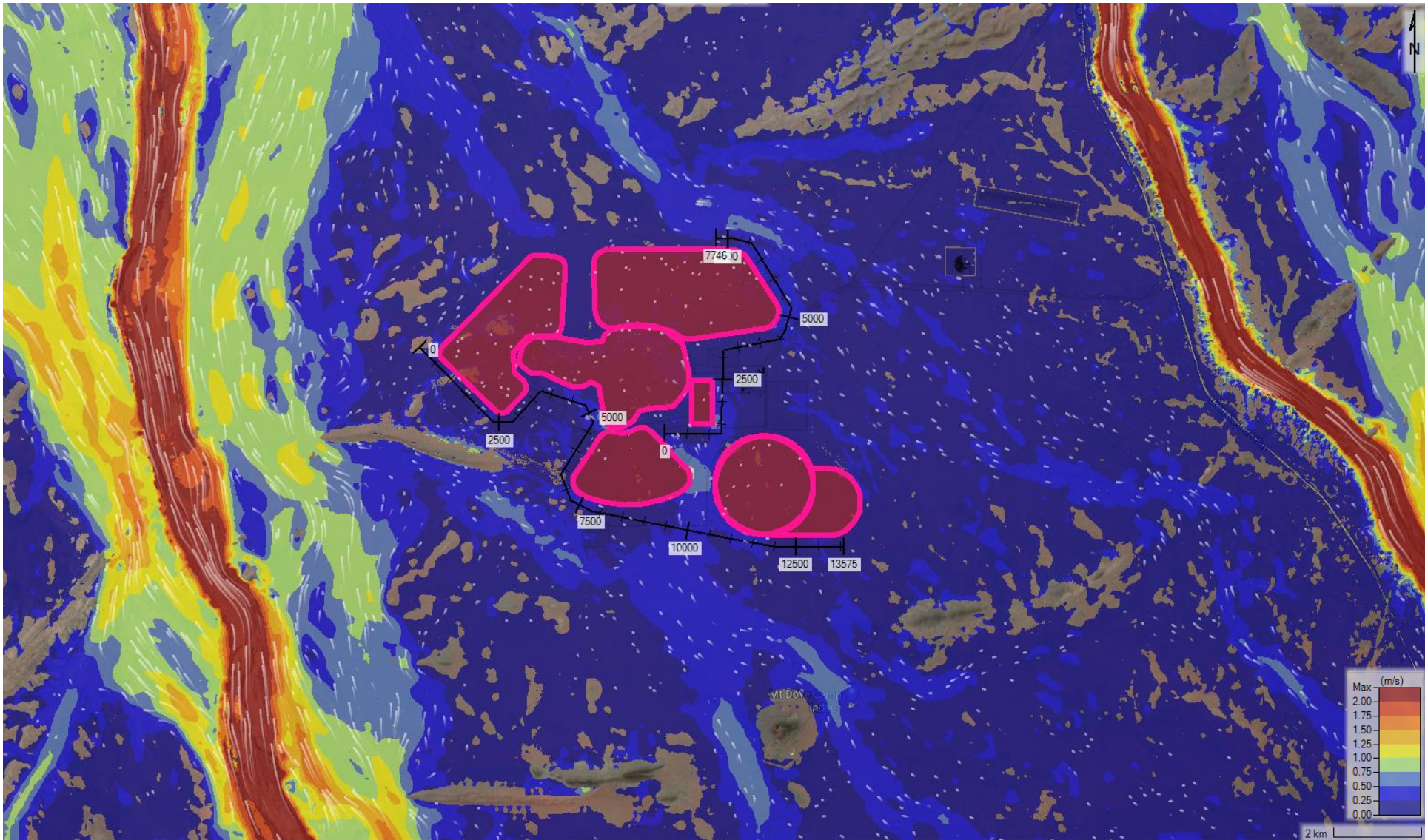


Figure 28 – Maximum 1 in 1000 AEP velocity for conceptual closure scenario

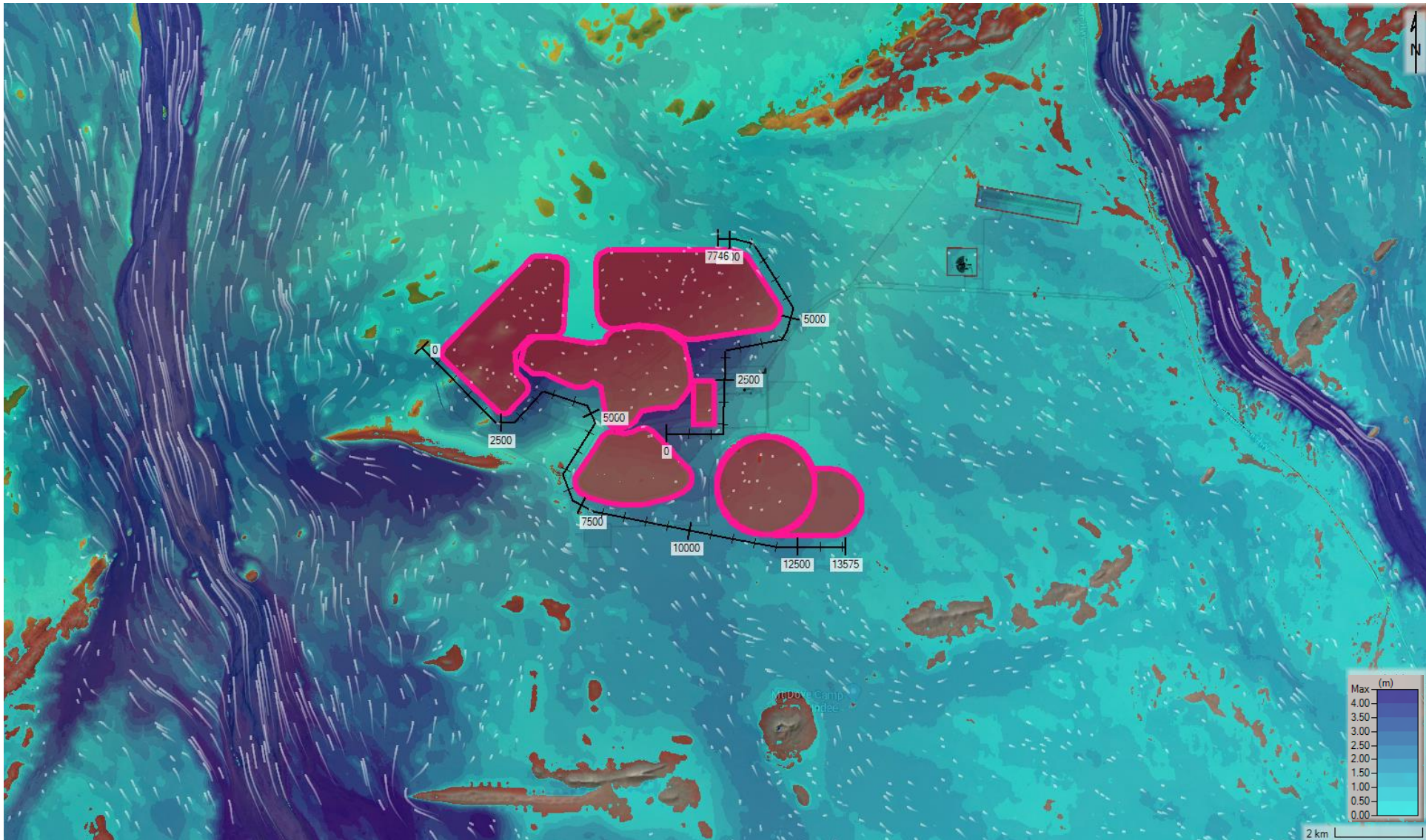


Figure 29 – Maximum PMF water surface elevation for conceptual closure scenario

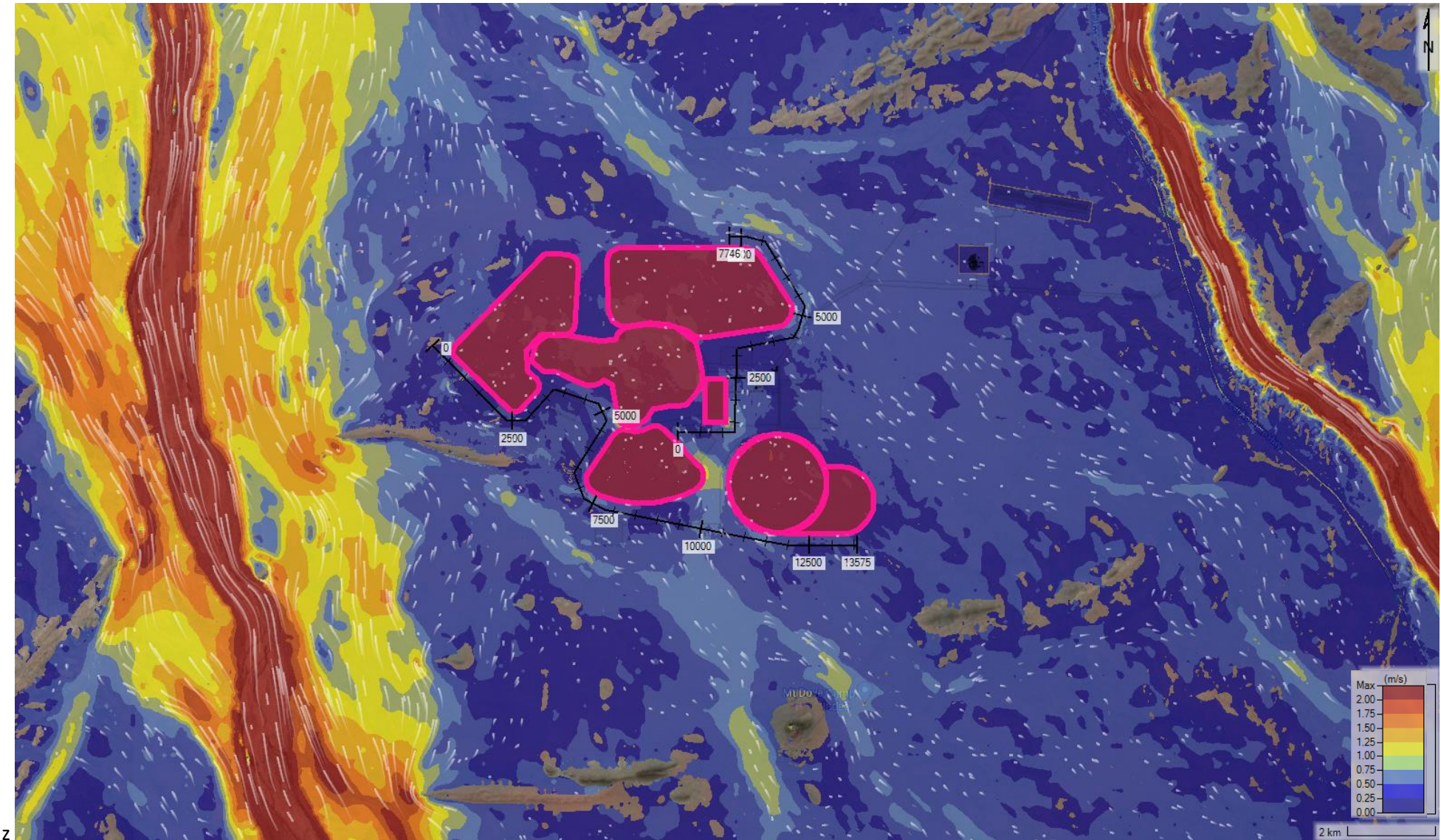


Figure 30 – Maximum PMF velocity for conceptual closure scenario



Water Surface Elevation on 'Southern Perimeter'

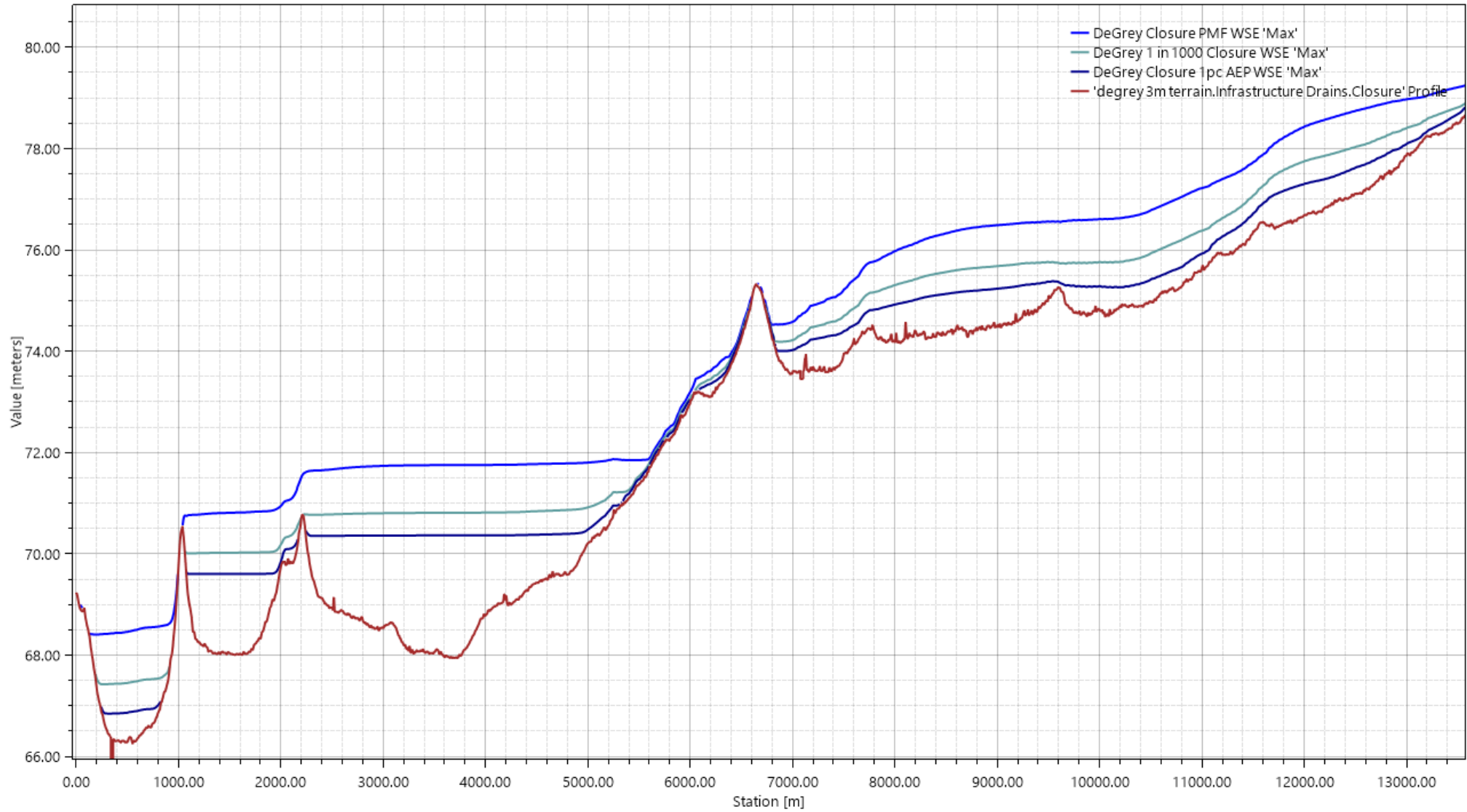


Figure 31 – Maximum water surface elevation profile along southern perimeter



Depth on 'Southern Perimeter'

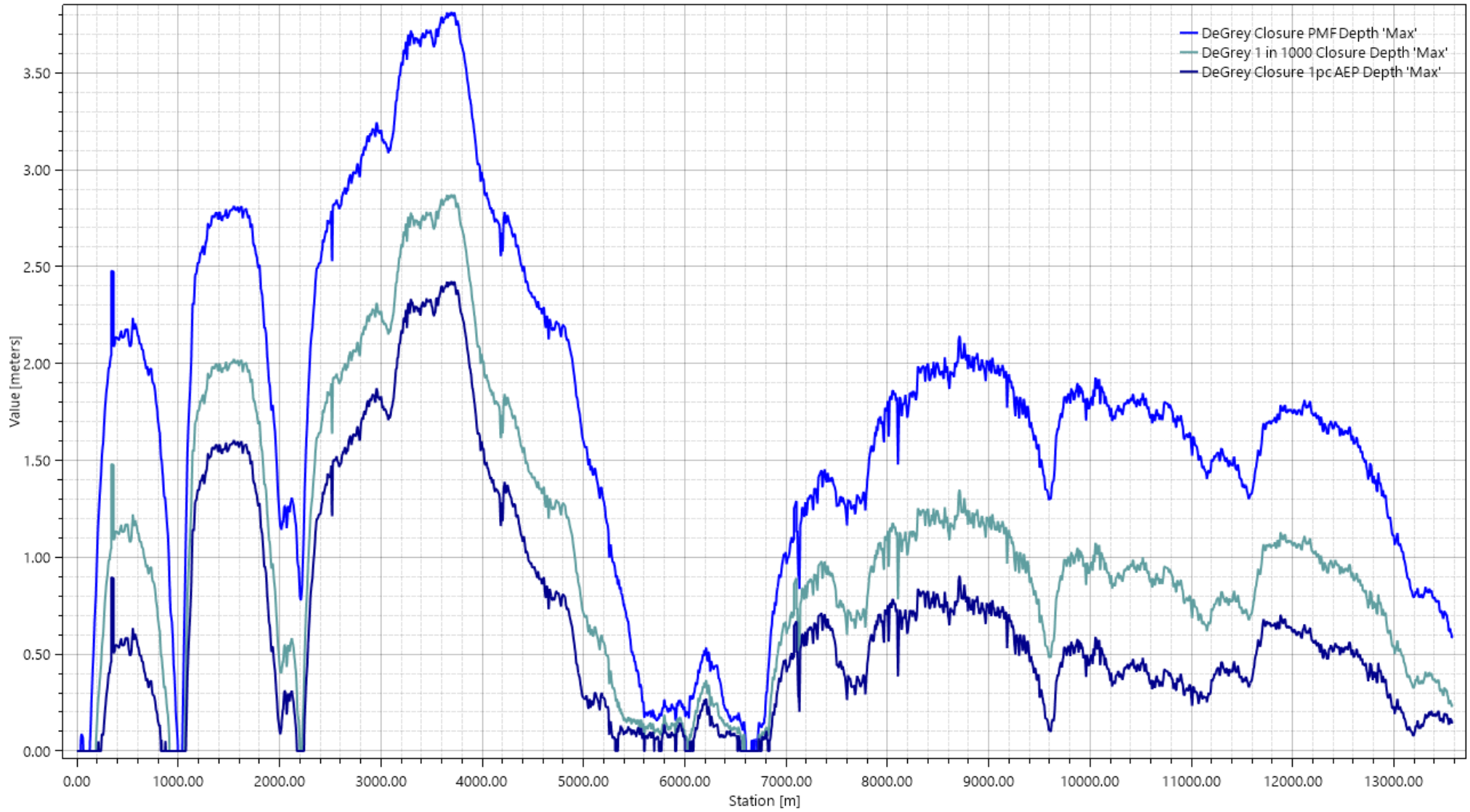


Figure 32 – Maximum depth profile along southern perimeter



Velocity on 'Southern Perimeter'

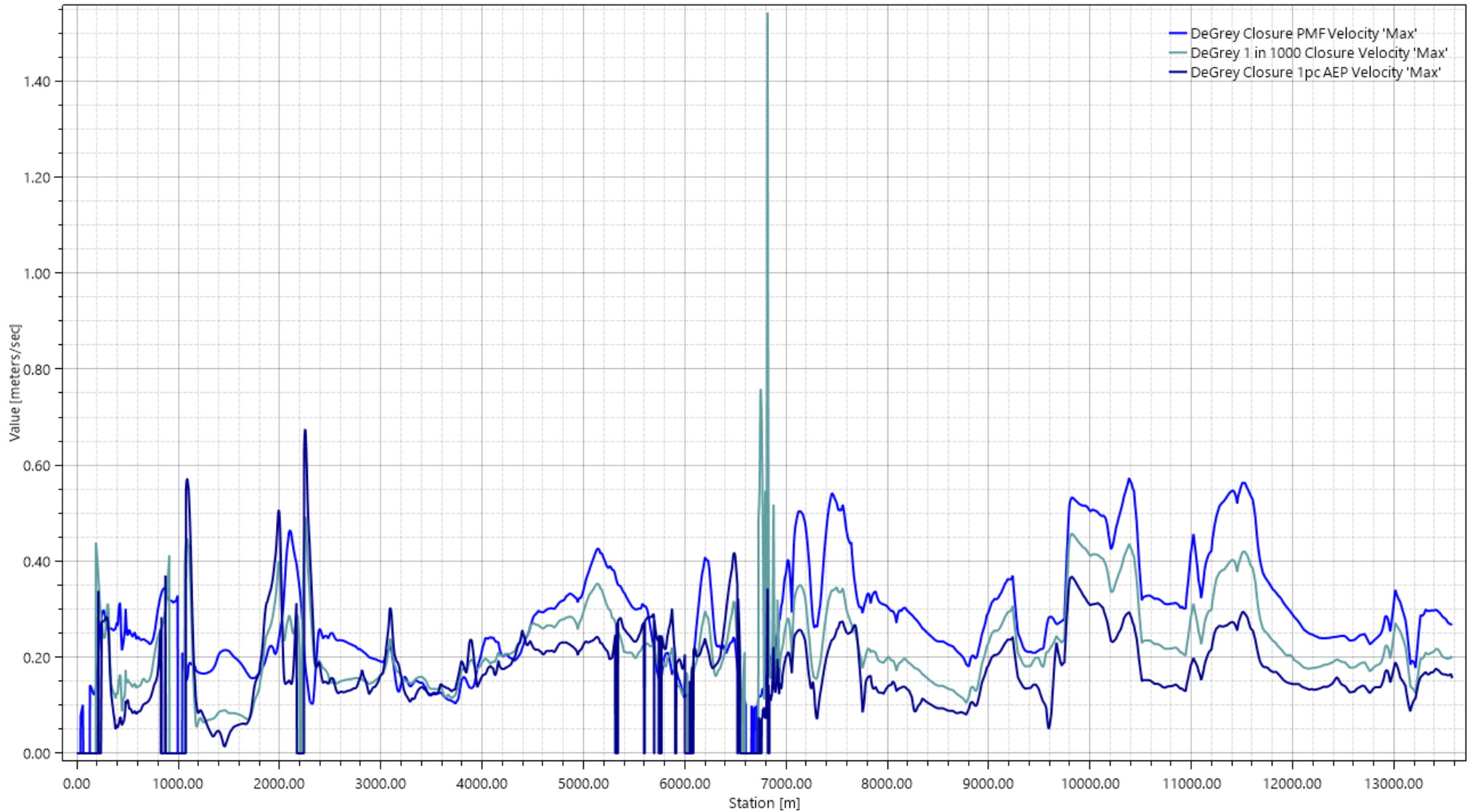


Figure 33 – Maximum velocity profile along southern perimeter



Water Surface Elevation on 'East Perimeter'

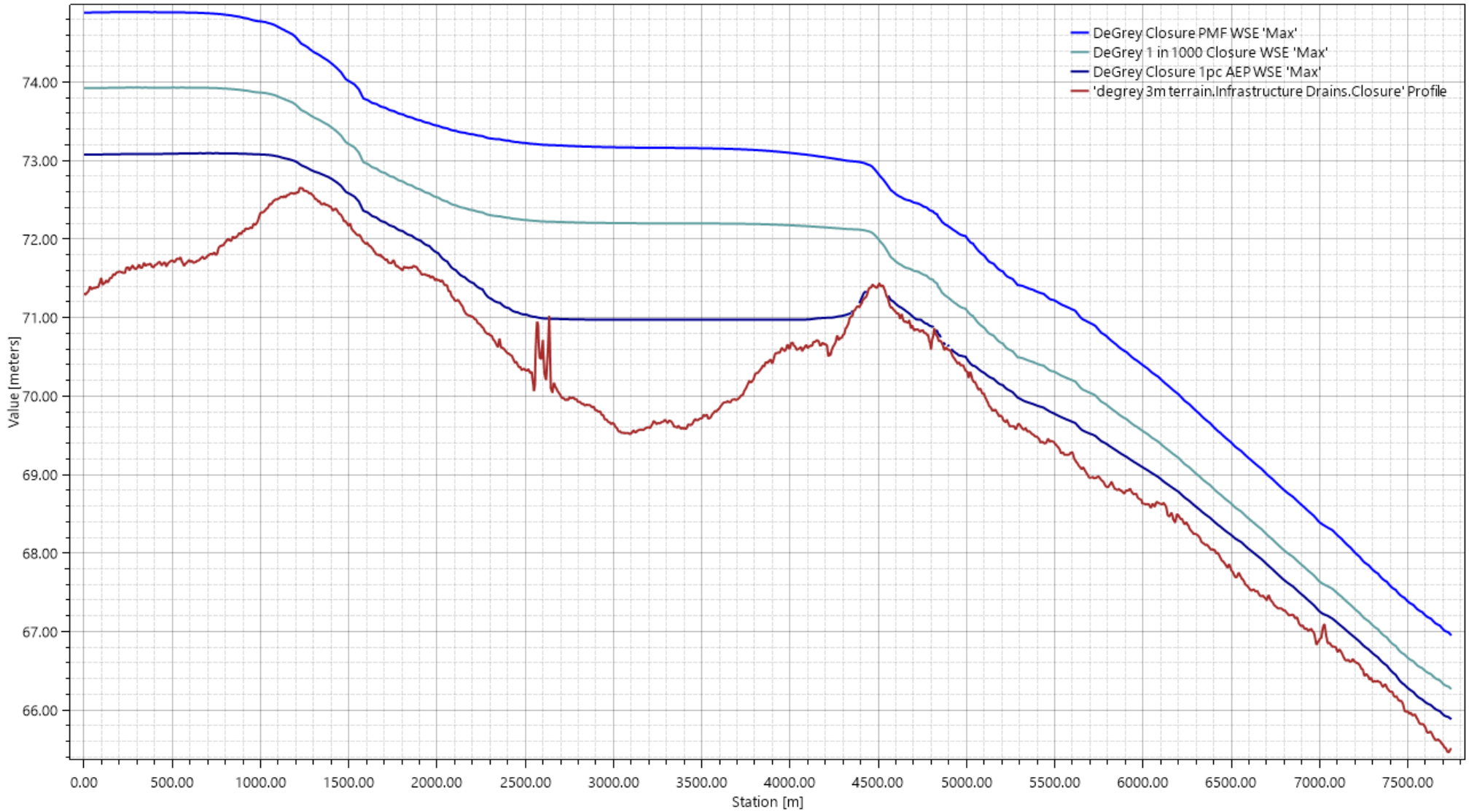


Figure 34 – Maximum water surface elevation profile along eastern perimeter



Depth on 'East Perimeter'

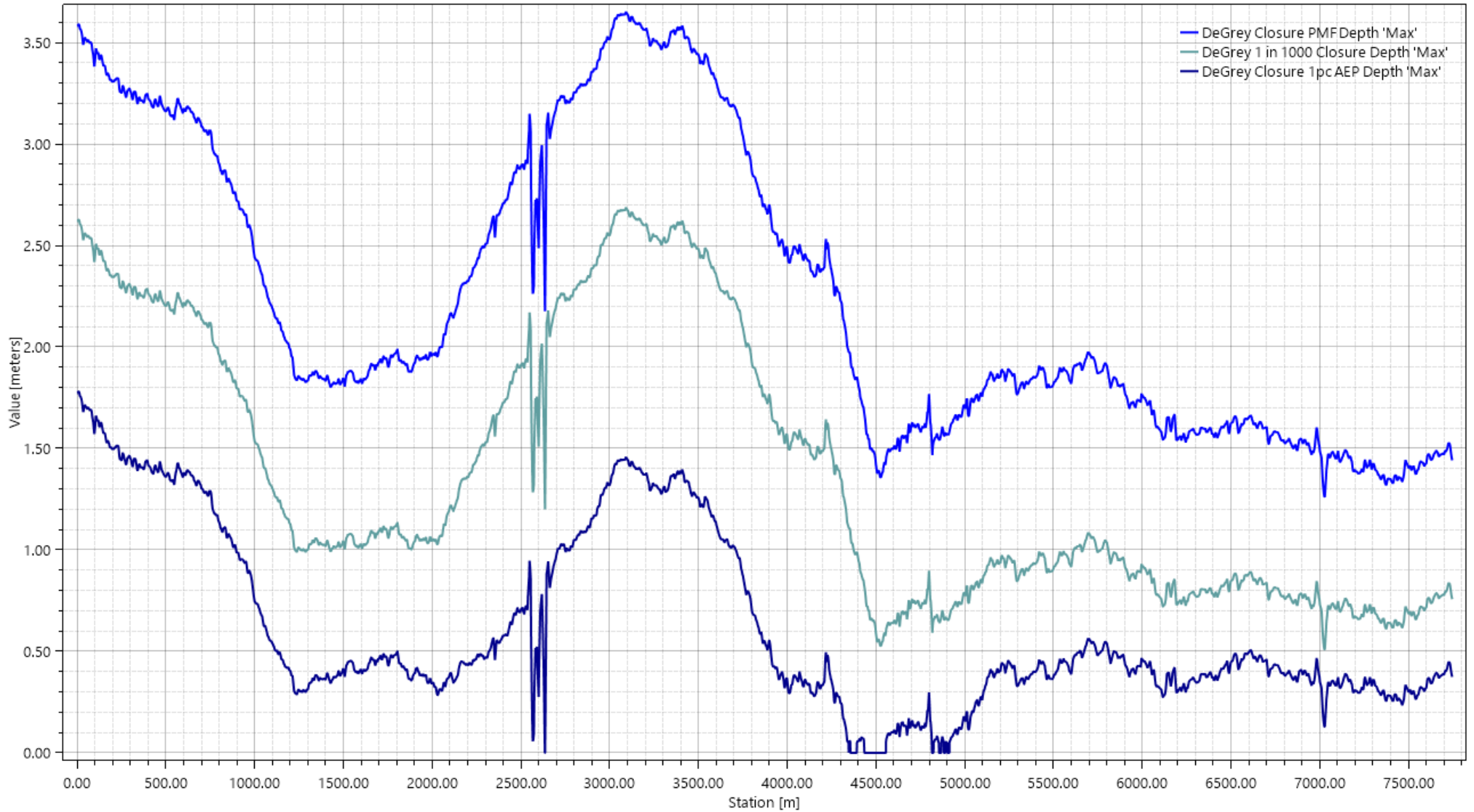


Figure 35 – Maximum depth profile along eastern perimeter



Velocity on 'East Perimeter'

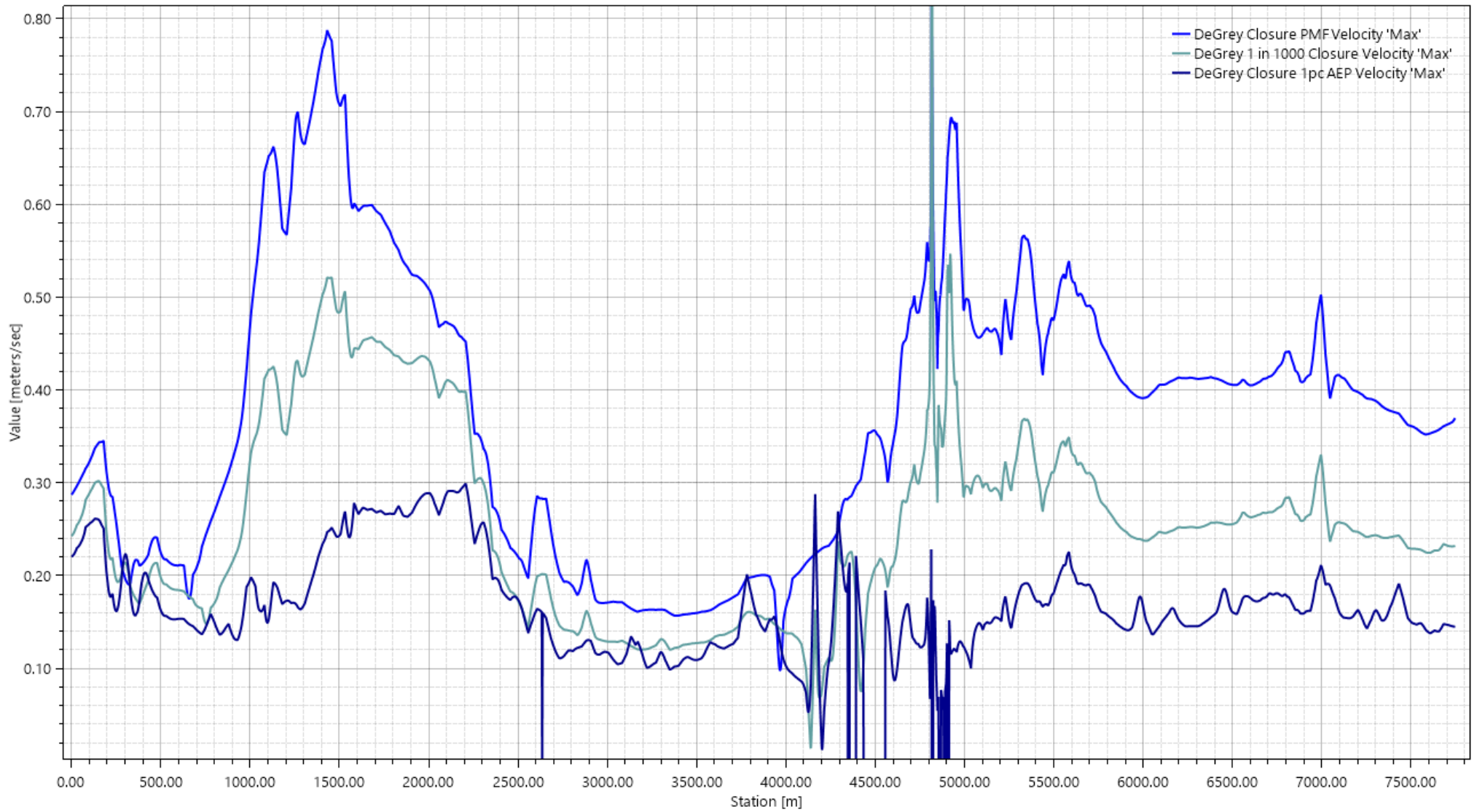


Figure 36 – Maximum velocity profile along eastern perimeter



To increase recovery rates for the groundwater table within post-closure pit voids, surface water runoff that ponds behind the proposed closure bunds could potentially be captured and directed into the pits. Figure 37 shows the cumulative flow volumes associated with a range of storm events from the 50% AEP to the 1 in 1000 AEP event. Flow volumes are extracted where the proposed closure features intercept surface water runoff. For an assumed initial post-closure period of 20 years with the runoff capture in place, a series of events comprising the modelled scenarios may be expected, with the total runoff volume from the selected series being captured in the pits. Surface water runoff from events lower than the 50% AEP would not result in appreciable runoff. A 20-year scenario, for example, may include approximately six 50% events, three 20% events, two 10% events, and one 5% event. The corresponding rise in pit lake level associated with localised runoff can be computed from the available stage vs. storage curves for the pits.

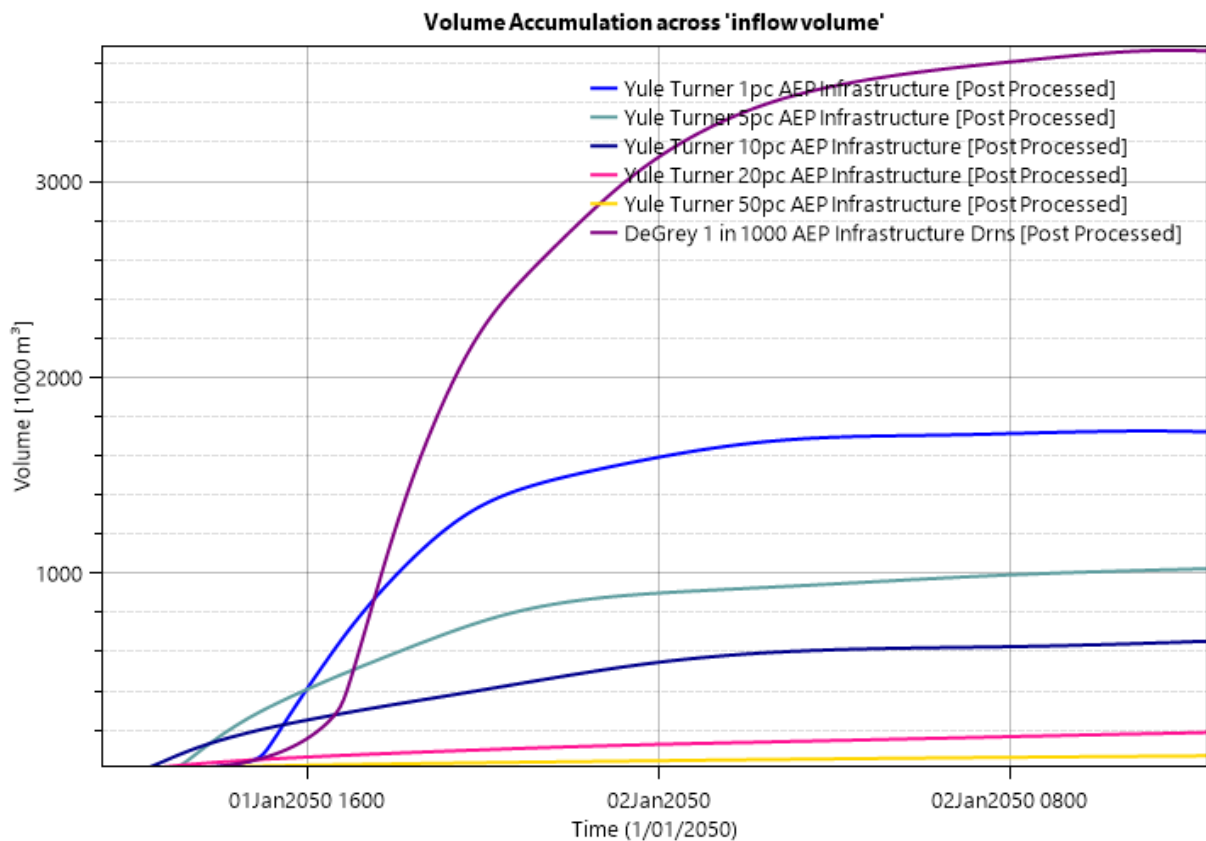


Figure 37 – Turner River and Yule River peak discharge rates by AEP



6. Conclusions

6.1. Limitations

The results presented in this report are limited to the accuracy of the provided terrain data. The models do not account for the presence of any earthworks or features constructed since the acquisition of the terrain data. If more recent or more detailed terrain data become available, the results of this study should be revisited. Some vertical discrepancies are apparent in the interface areas between the available terrain data sources, potentially affecting flood levels by +/- 20cm. These discrepancies do not affect flood levels at the proposed site location.

Due to the limited availability of gauge data, the confidence bands around the hydrological results are very wide, particularly for extreme events. Additional background details on the flow estimation procedures and confidence limits are included in AQ2 2018.

Further closure designs should be undertaken with adherence to all applicable guidelines, including ARR 2019, Western Australia mine closure guidelines, erosion and scouring protection guidelines, and other local guidelines as appropriate.

6.2. Summary and Recommendations

The discharge assessment indicates that under the adopted discharge schedule and assumed infiltration rates, discharge will reach the Great Northern Highway crossing during the first year of discharge but will not reach the ocean outlet. Flows cease upstream of the Great Northern Highway during Year 2.

Under maximum discharge conditions, the inundated area covers approximately 5% of the Turner River channel. Maximum flow rates are less than 0.1% of the 10% AEP flood event on a peak flow and volumetric basis.

Maximum PMF flow depths under the closure scenarios are approximately 3.5 metres. Closure drains may be incorporated to reduce these depths against the external perimeter bunds. Maximum velocities from external flows are relatively low (<0.8m/s) in the 1% AEP flood event, indicating that rip rap or other scour measures are not required.

Depths and velocities in the Yule and Turner River channels are substantial, and the placement of any closure features near the river channels should include adequate scour rock protection and should accommodate potential changes in the morphology of the river bed.

Further closure designs may incorporate closure bunds as water-retaining levees and drains to prevent long-term standing water following flood events. Closure drainage features would require additional sedimentation and scour assessment to ensure that they can function maintenance-free in the post-closure period. The relative location of the closure features may be adjusted to incorporate any benefits derived from providing flow paths between the closure features.

Rather than routing flows around the external site perimeter, it may be beneficial to incorporate bunds and drains that direct runoff into closed pits to accelerate groundwater recharge. These options may be explored further in conjunction with the hydrogeological analyses as the closure designs progress.



7. References

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Appendix A. Water Surface Elevation Profiles

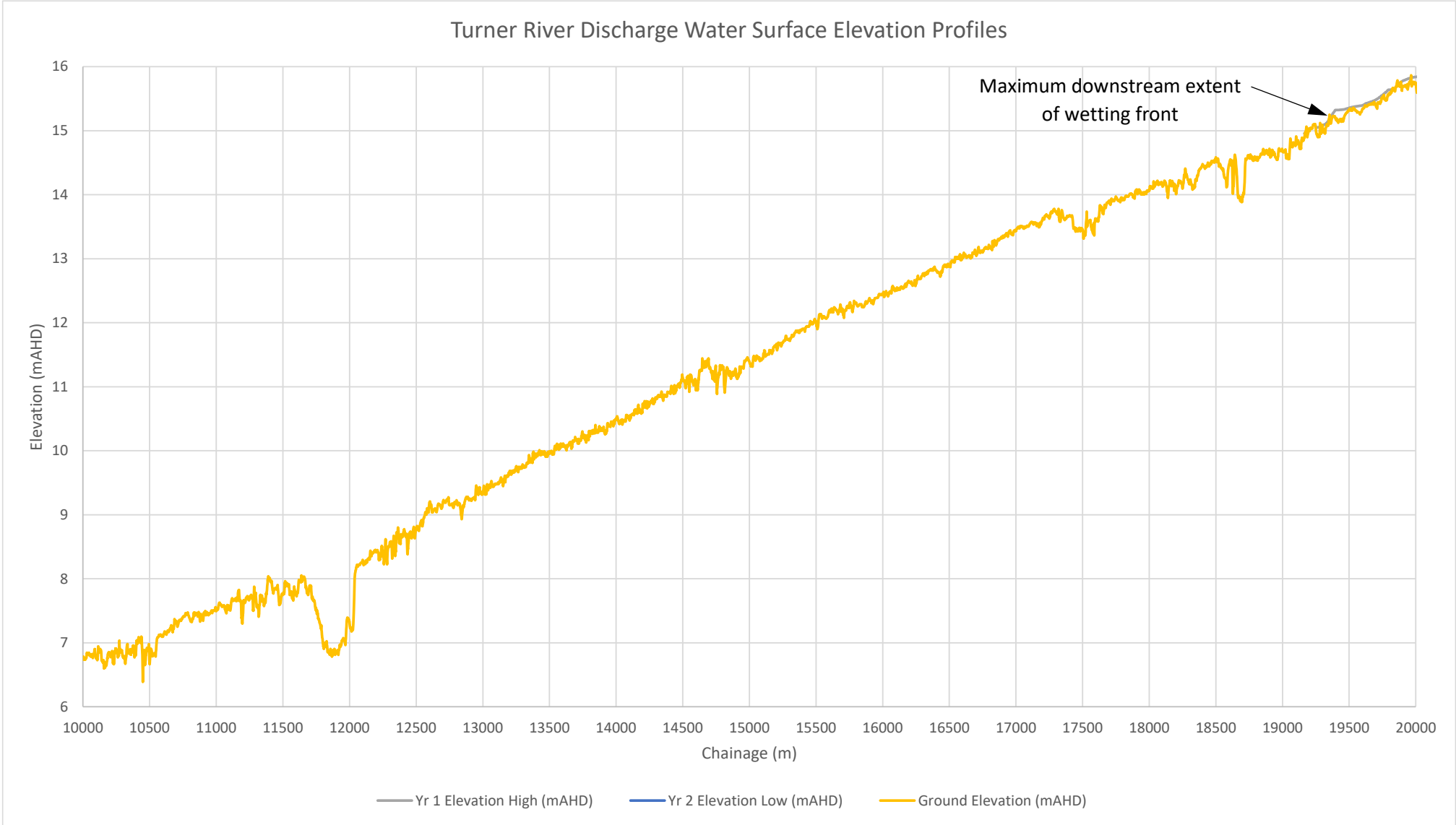


Figure A-1 – Water surface elevation profiles for high and low flows, Chainage 10000-20000

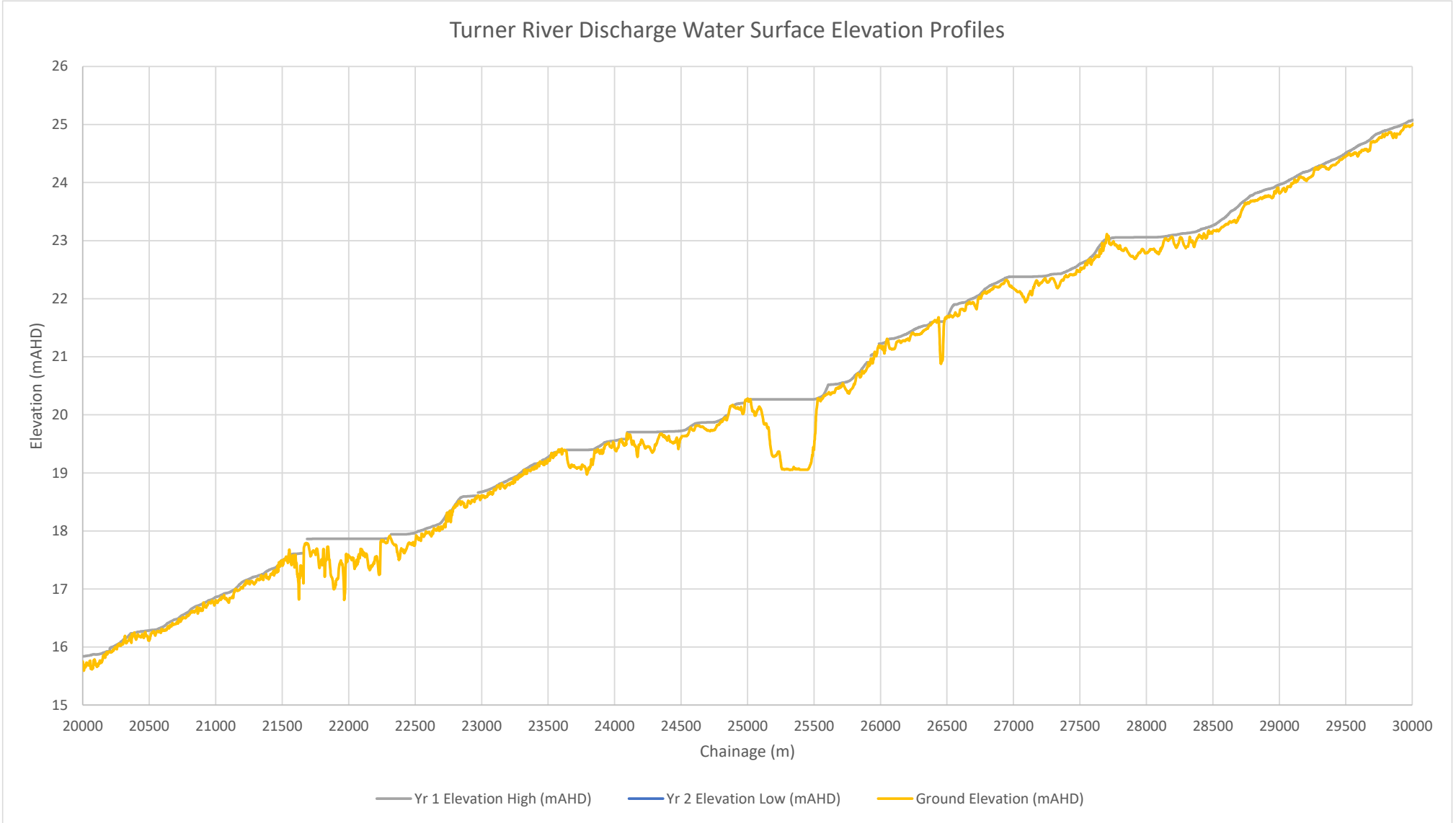


Figure A-2 – Water surface elevation profiles for high and low flows, Chainage 20000-30000

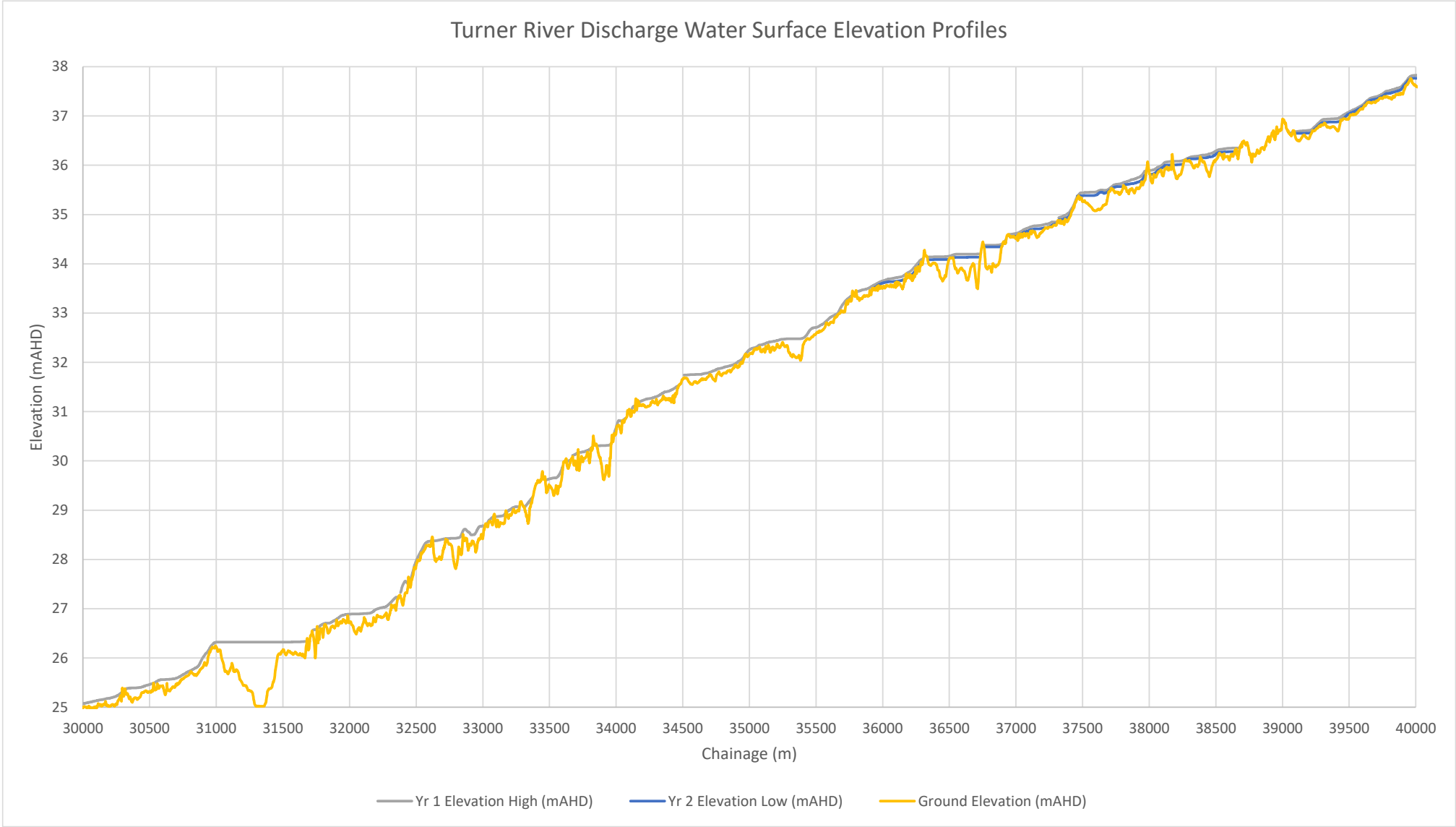


Figure A-3 – Water surface elevation profiles for high and low flows, Chainage 30000-40000

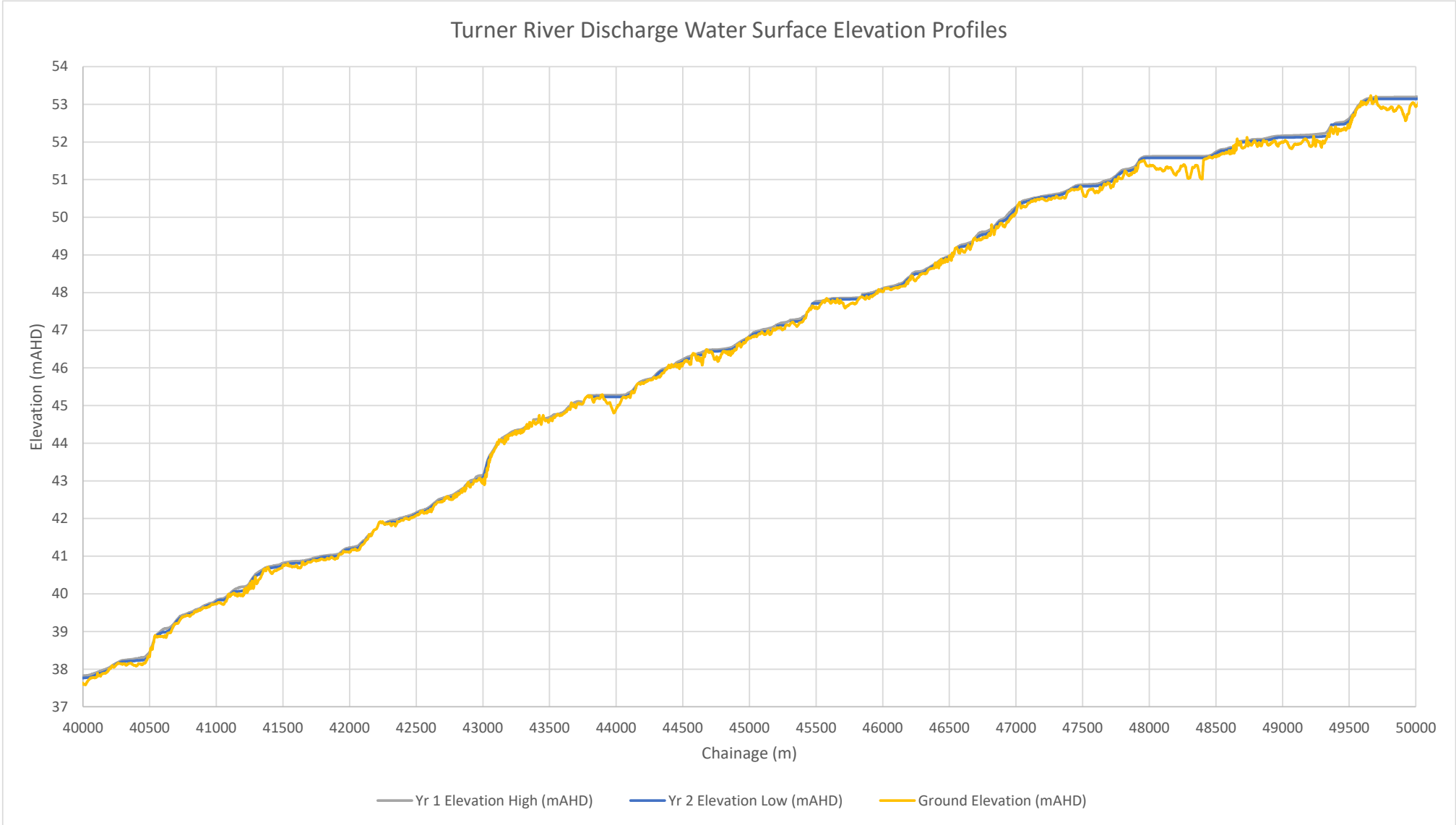


Figure A-4 – Water surface elevation profiles for high and low flows, Chainage 40000-50000

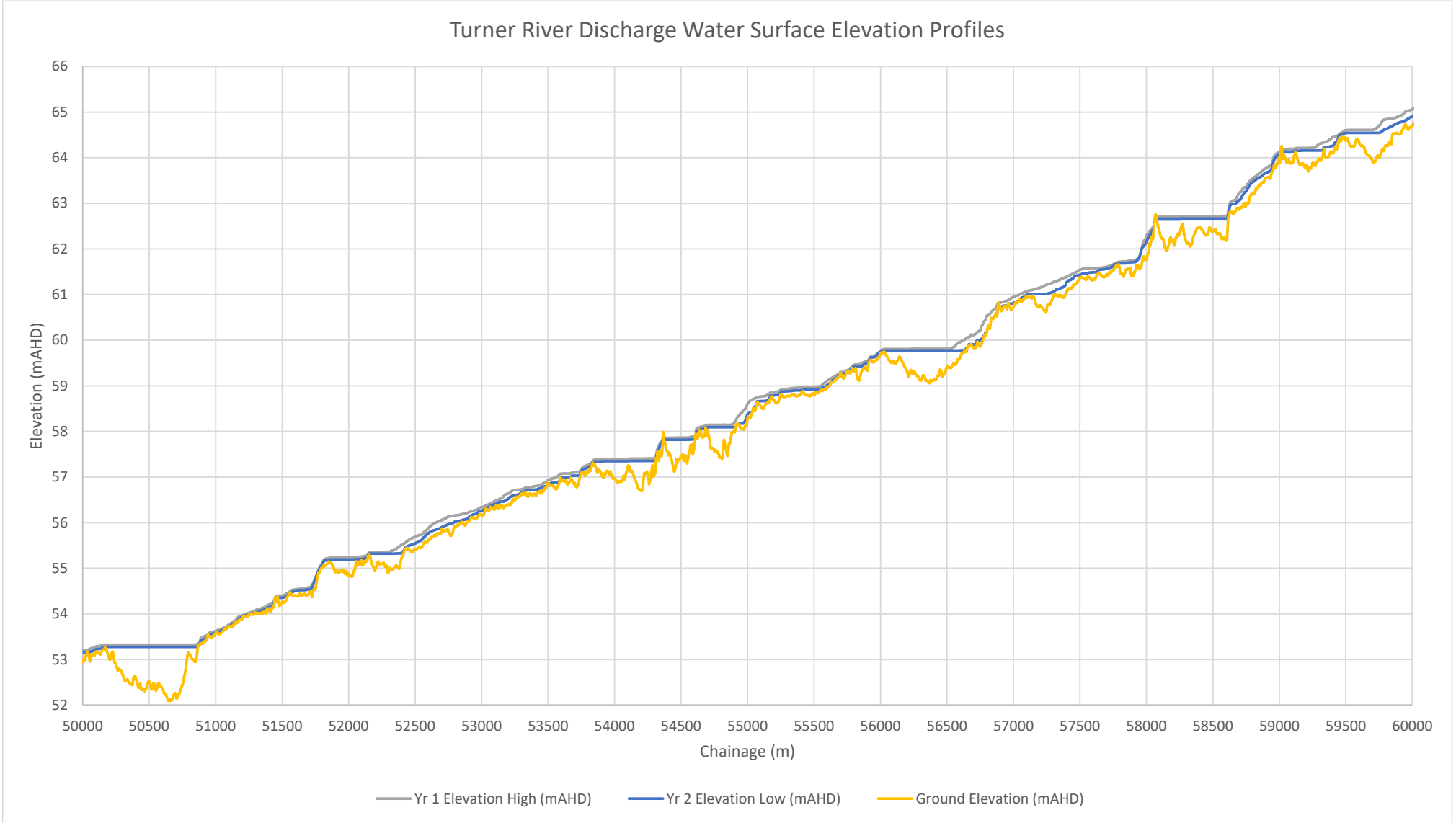


Figure A-5 – Water surface elevation profiles for high and low flows, Chainage 50000-60000

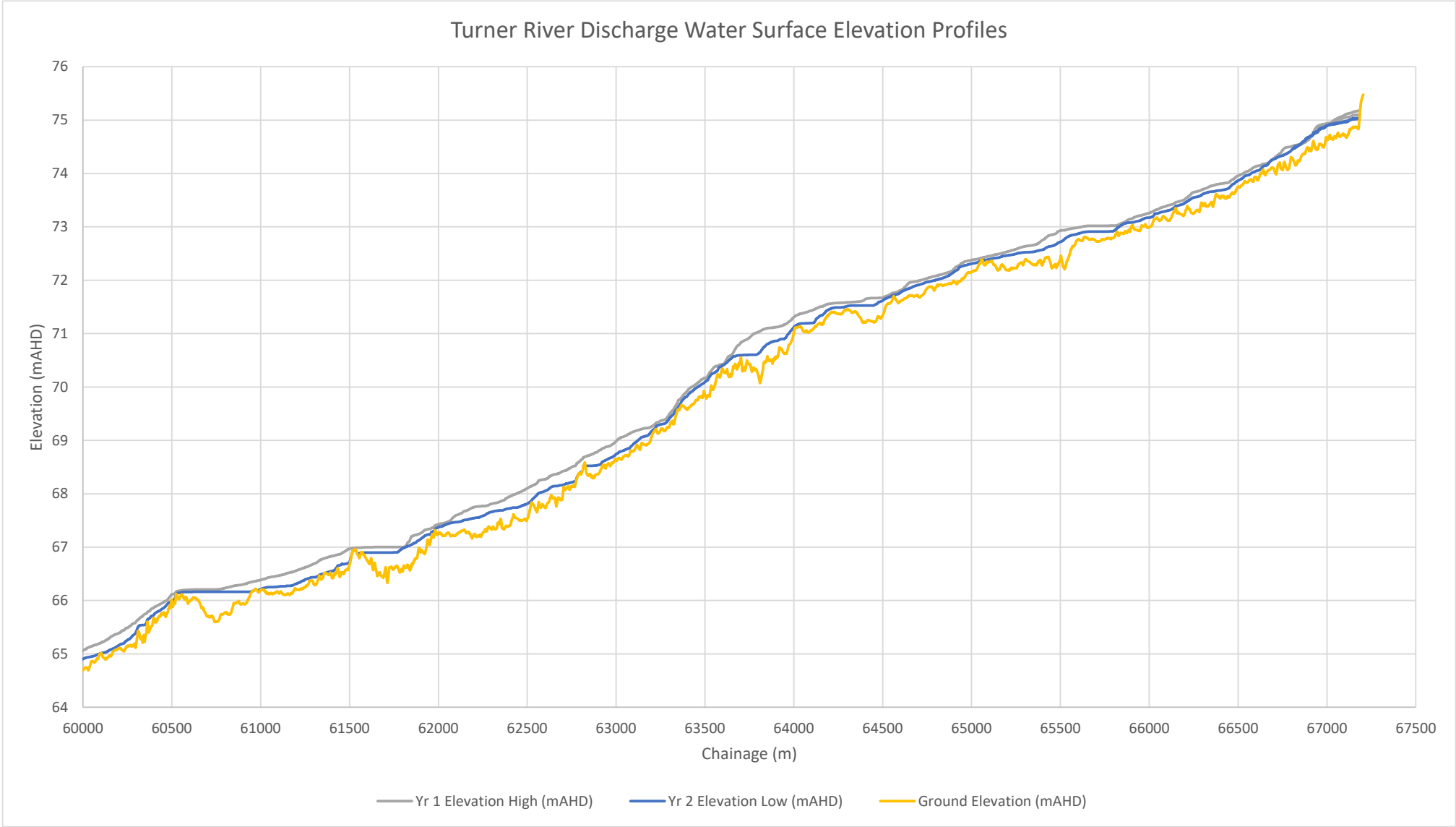


Figure A-6 – Water surface elevation profiles for high and low flows, Chainage 60000-70000

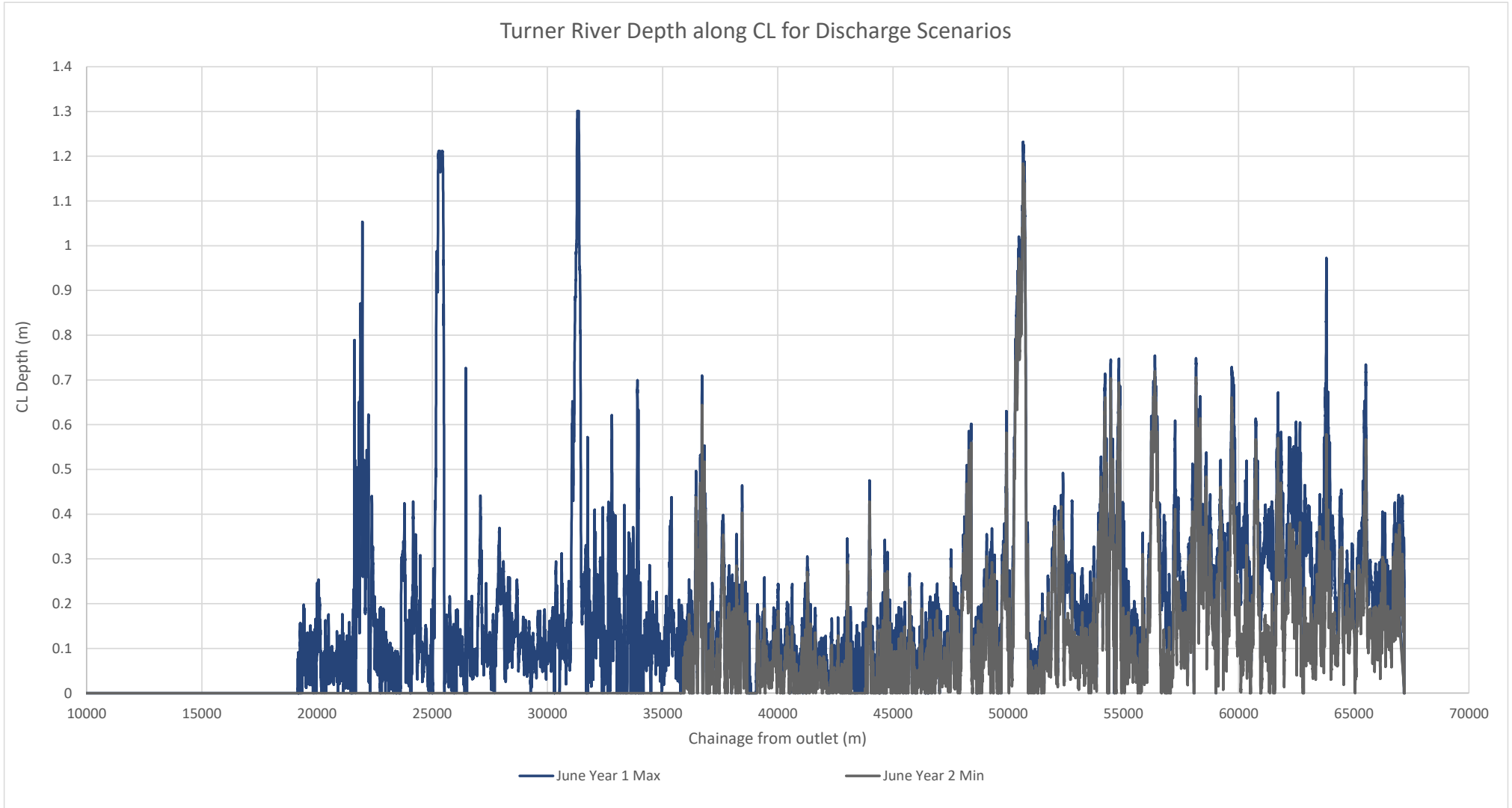


Figure A-7 – Flow depths for high and low flows

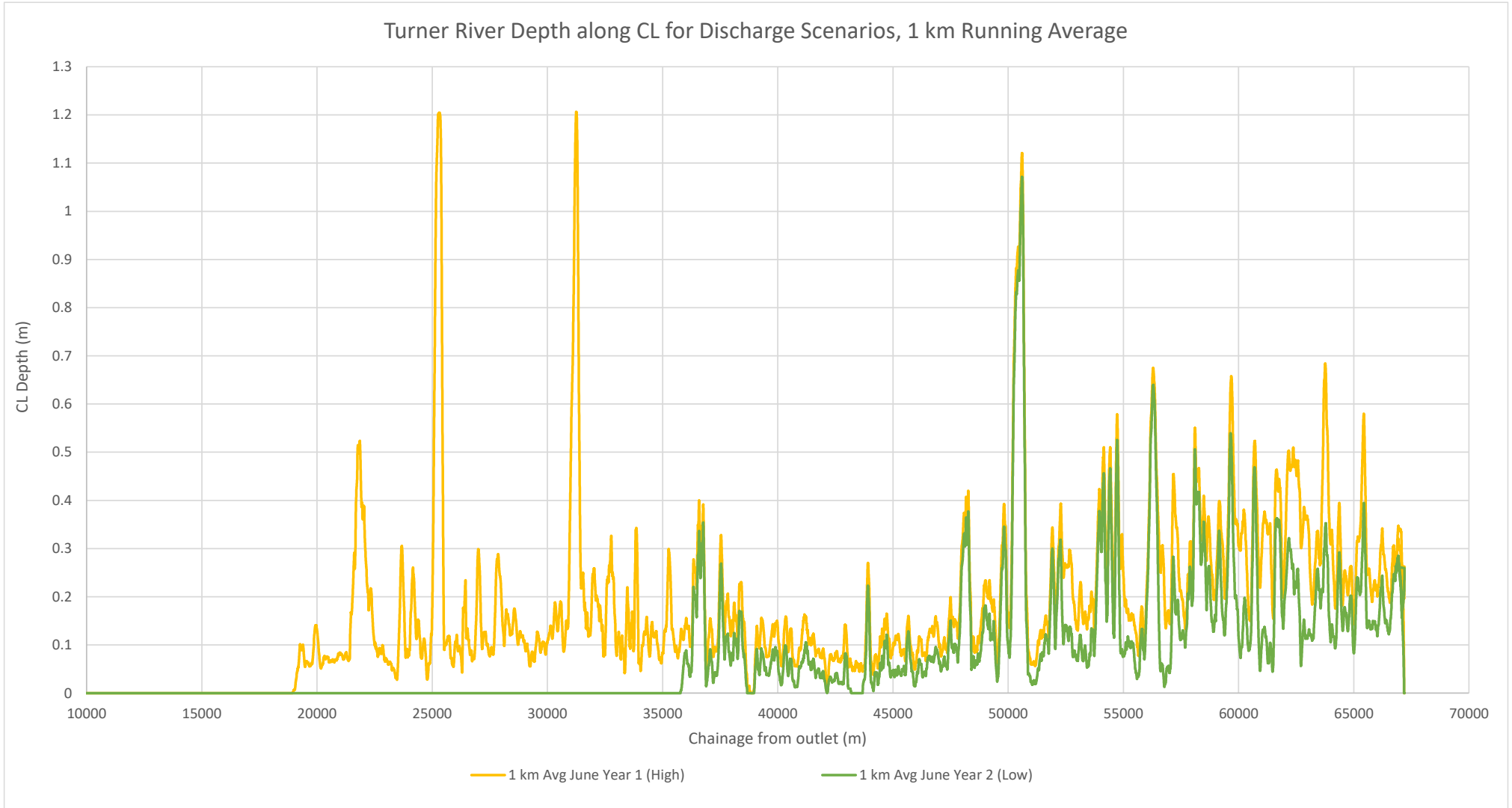


Figure A-8 – Flow depths for high and low flows, 1-km running average

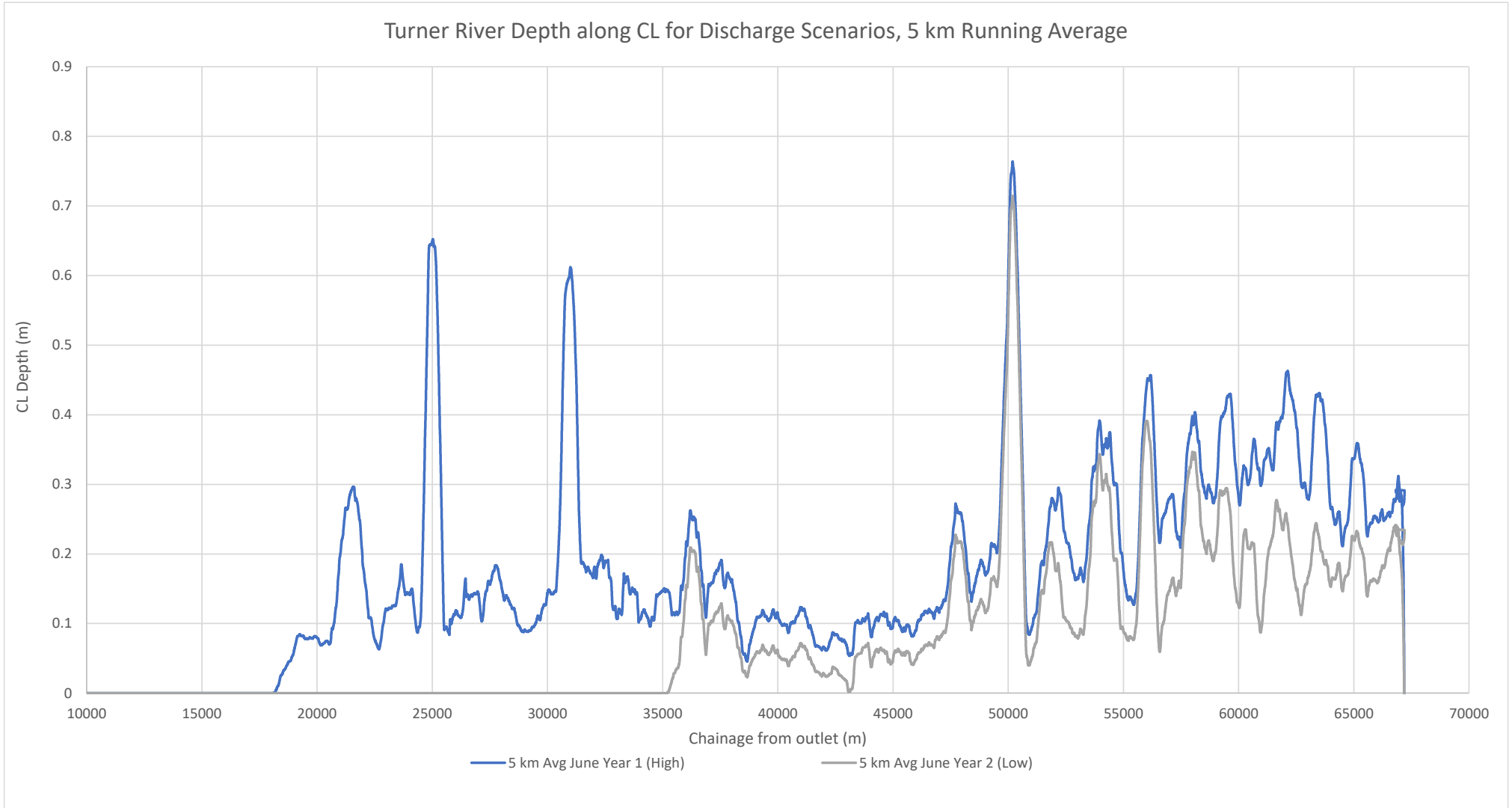


Figure A-9 – Flow depths for high and low flows, 5-km running average



Appendix B. Cross Sections

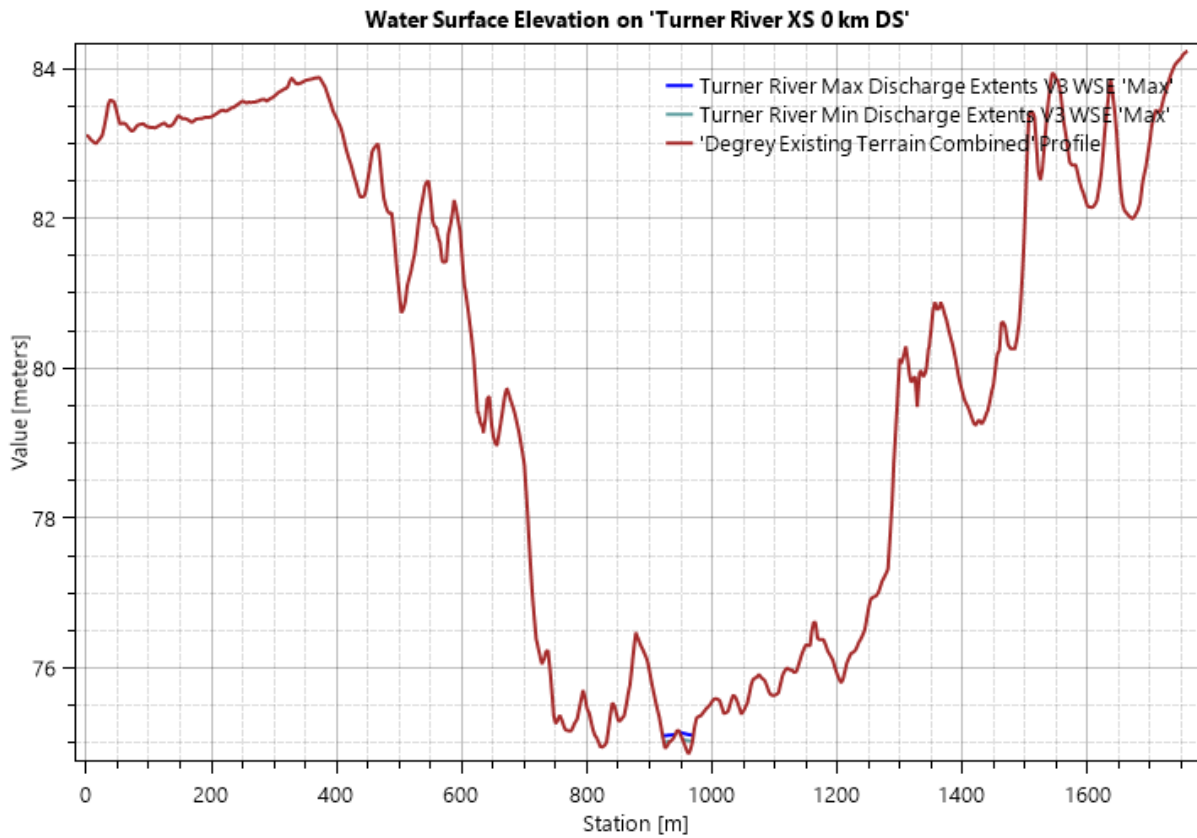


Figure C-1 – Water surface elevations at discharge point

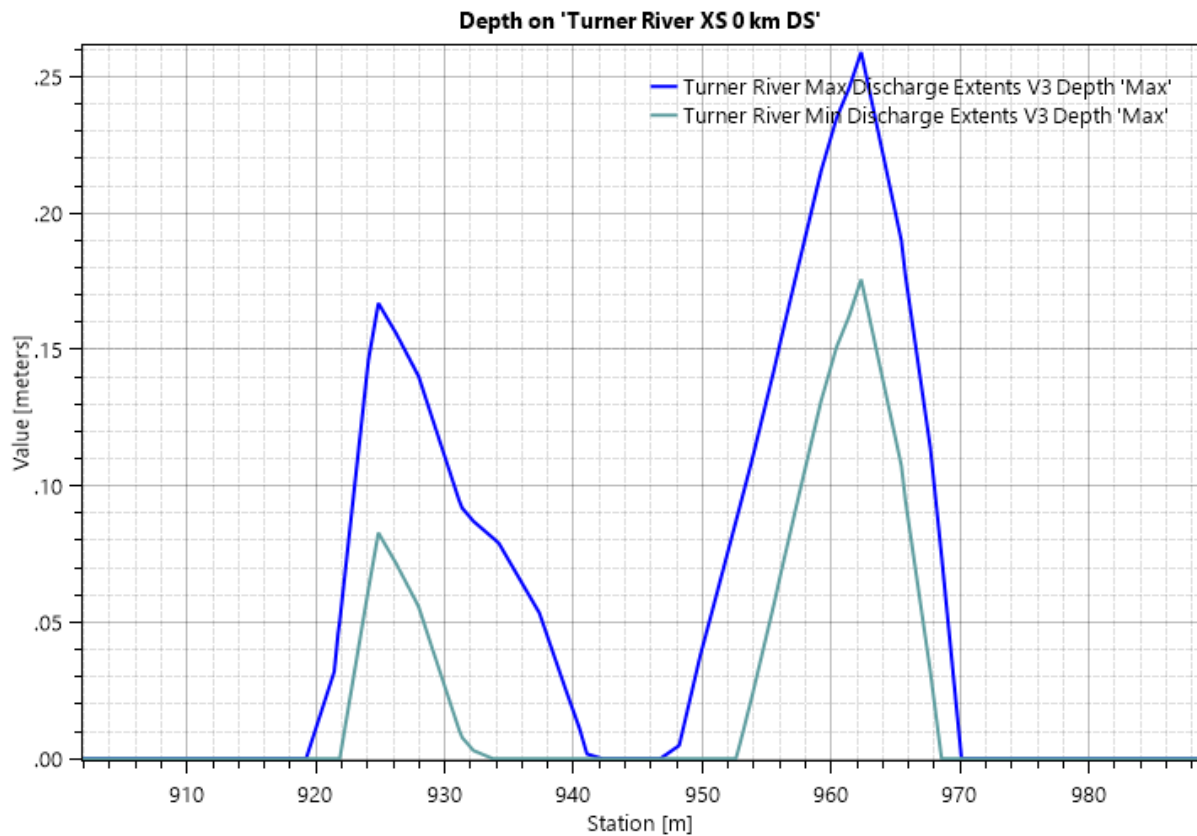


Figure C-2 – Flow depths at discharge point

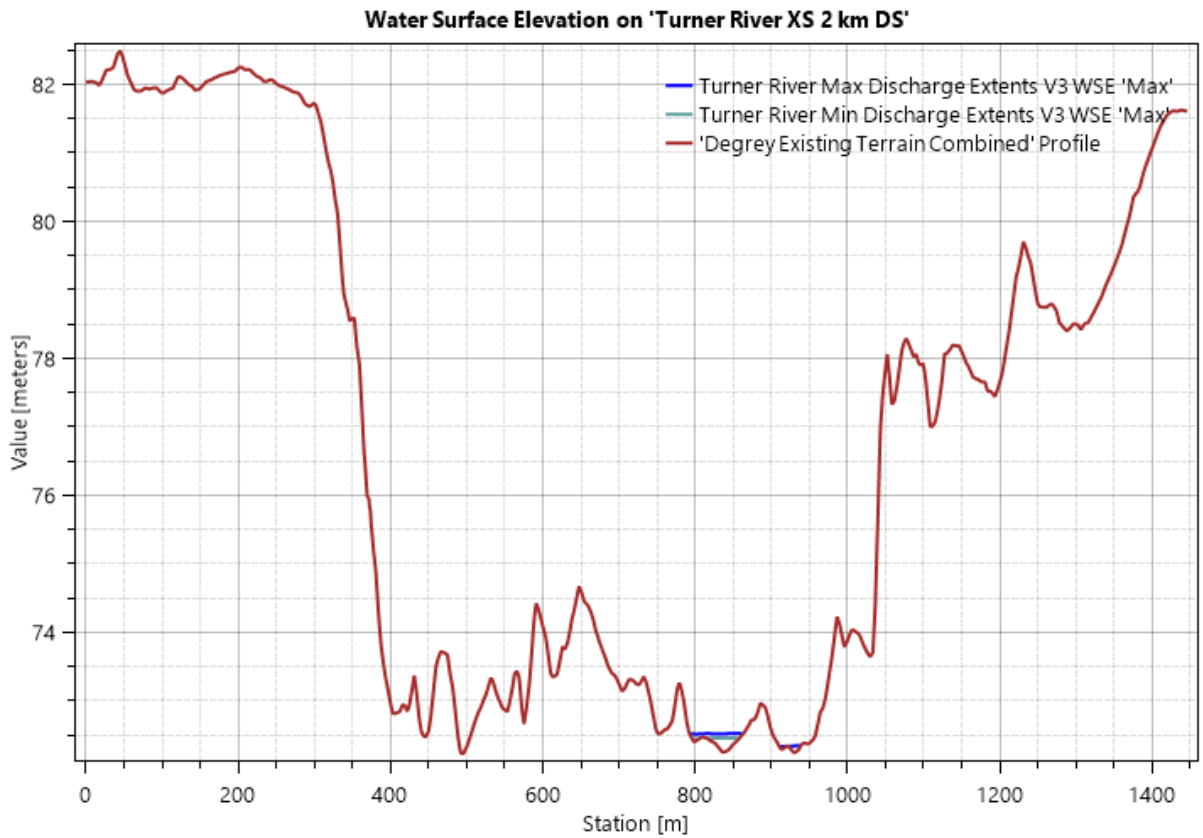


Figure C-3 – Water surface elevations 2 km downstream of discharge point

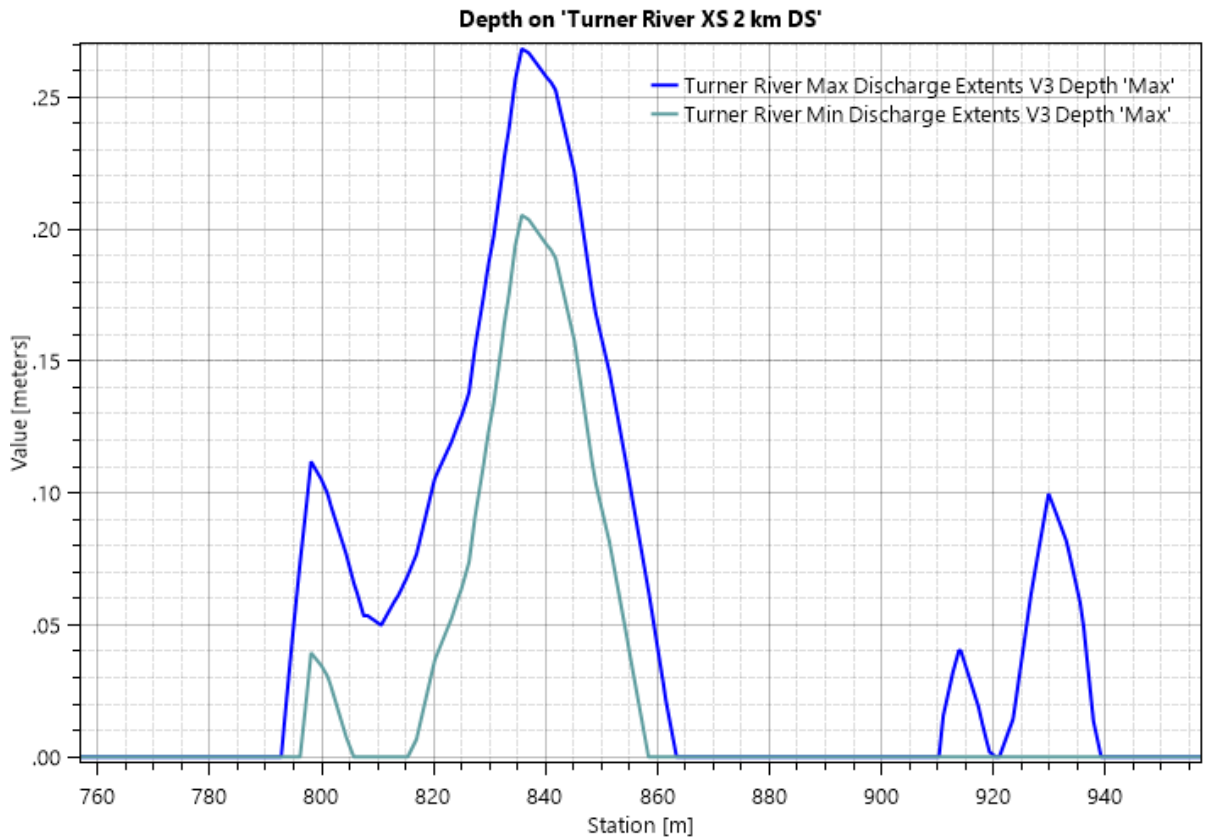


Figure C-4 – Flow depths 2 km downstream of discharge point

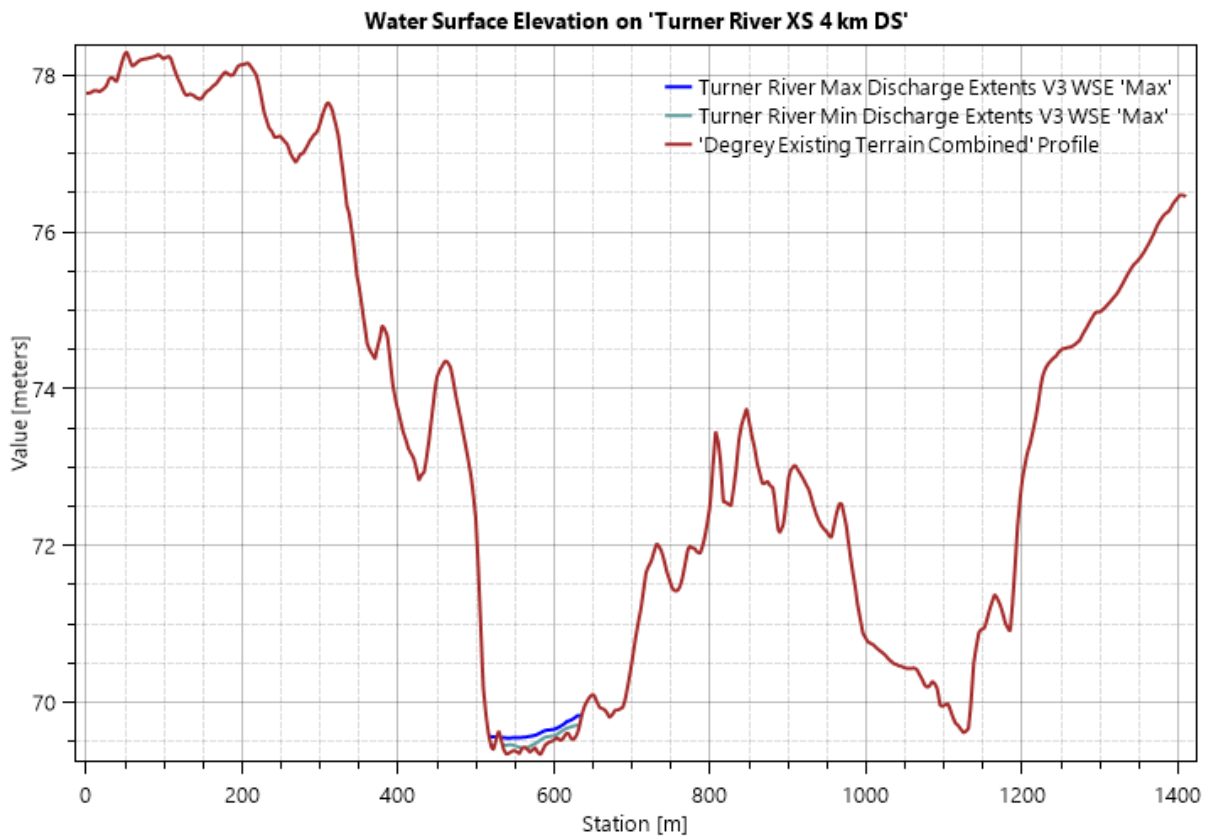


Figure C-5 – Water surface elevations 4 km downstream of discharge point

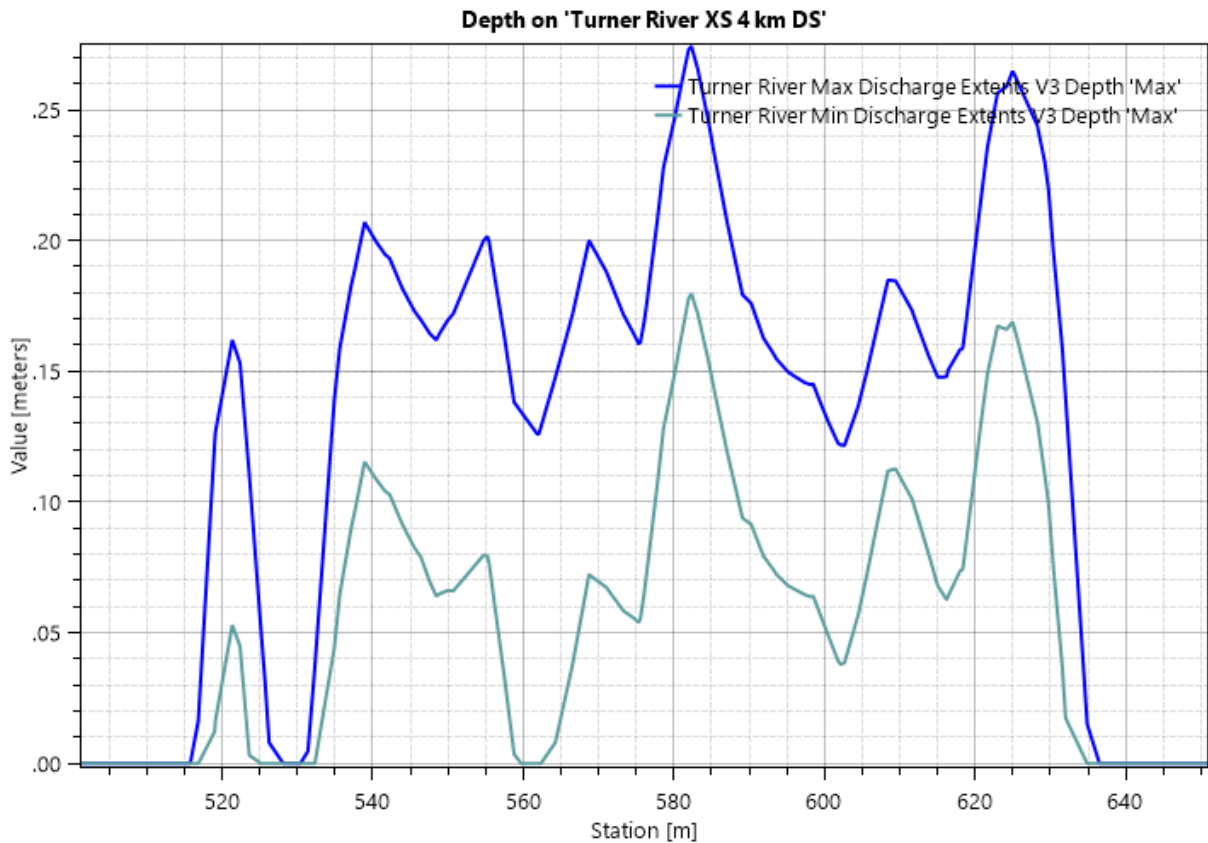


Figure C-6 – Flow depths 4 km downstream of discharge point

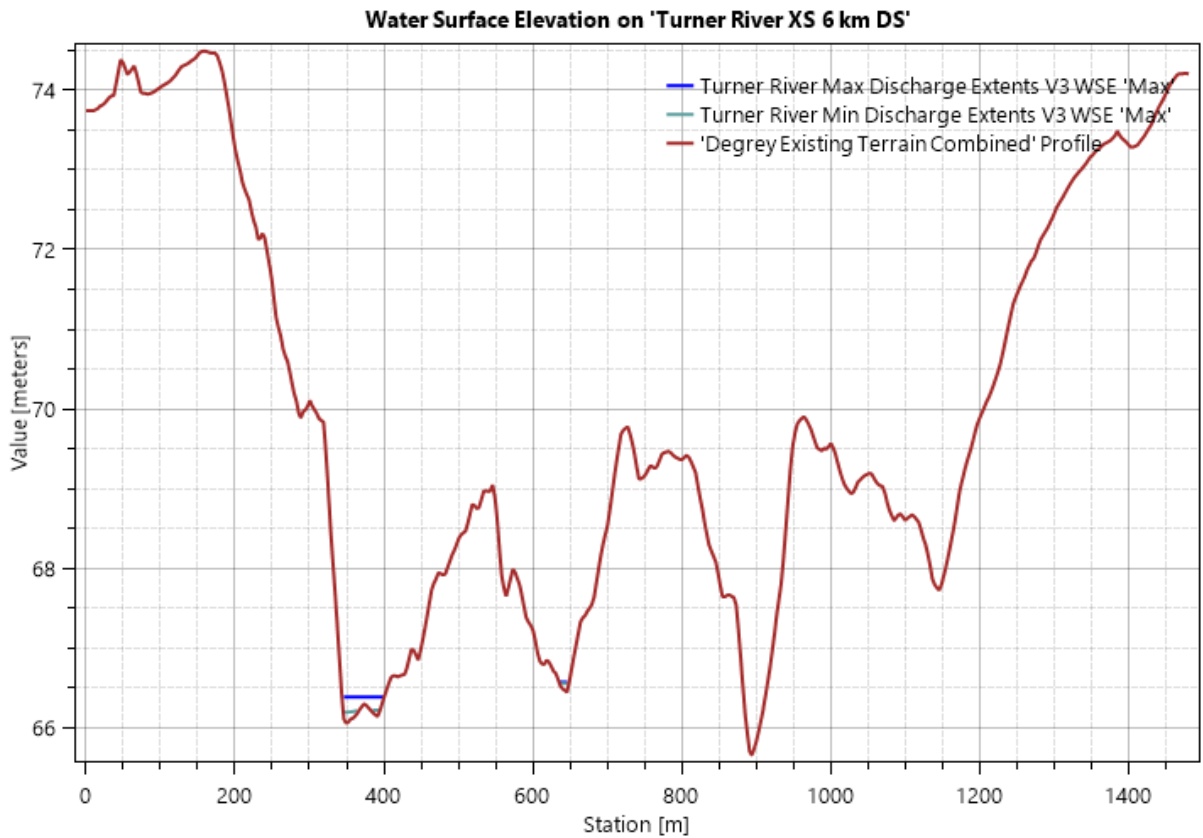


Figure C-7 – Water surface elevations 6 km downstream of discharge point

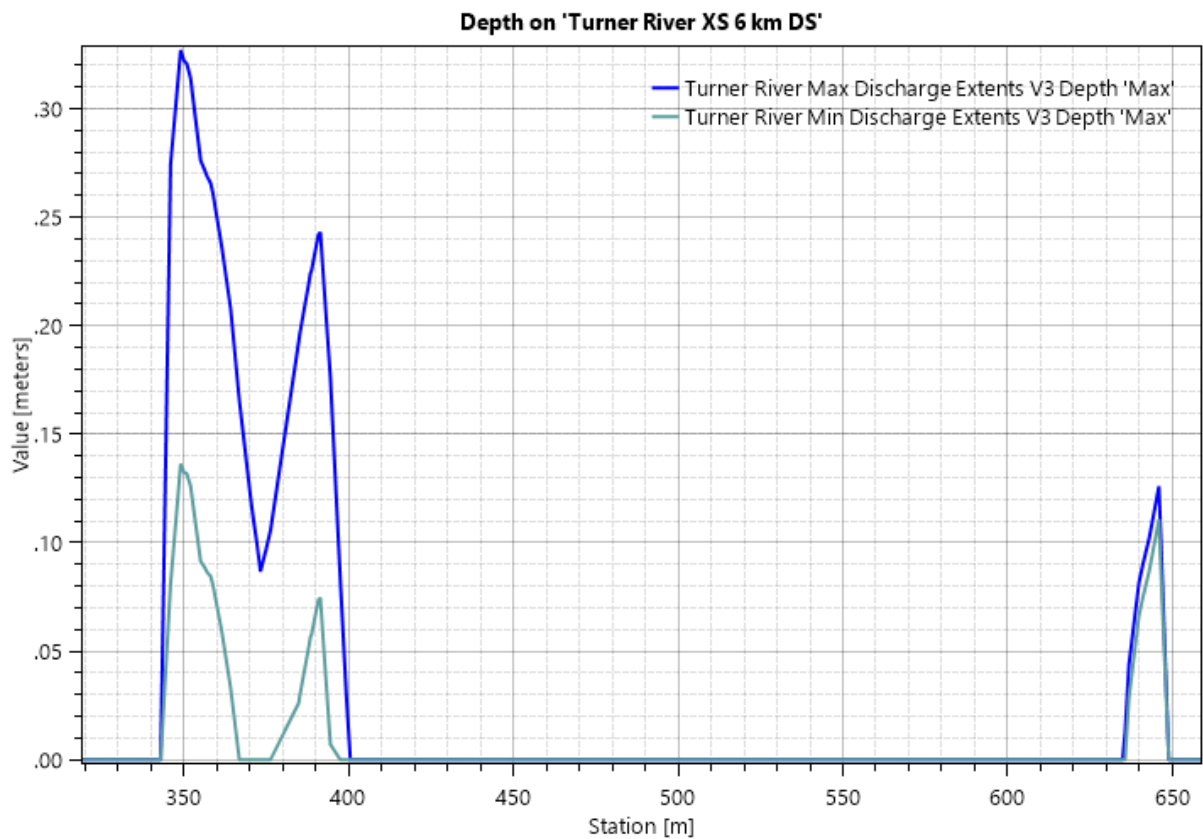


Figure C-8 – Flow depths 6 km downstream of discharge point

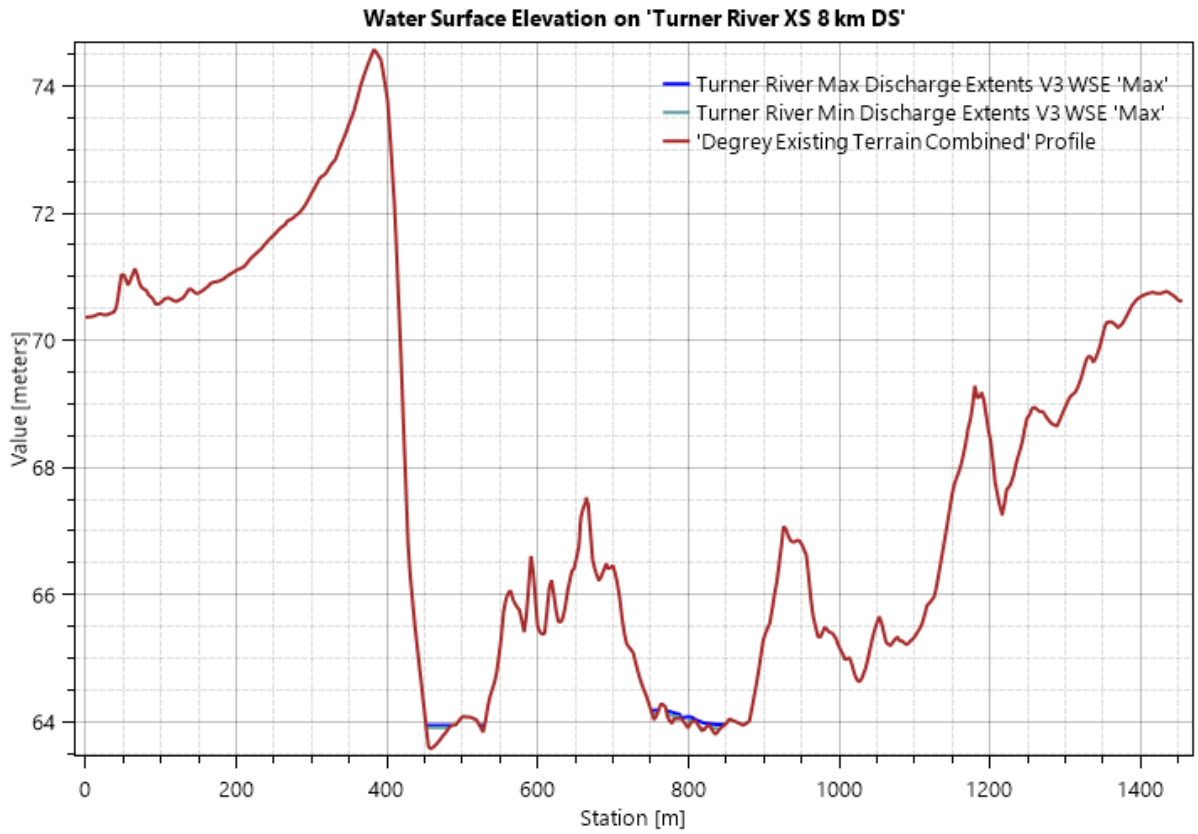


Figure C-9 – Water surface elevations 8 km downstream of discharge point

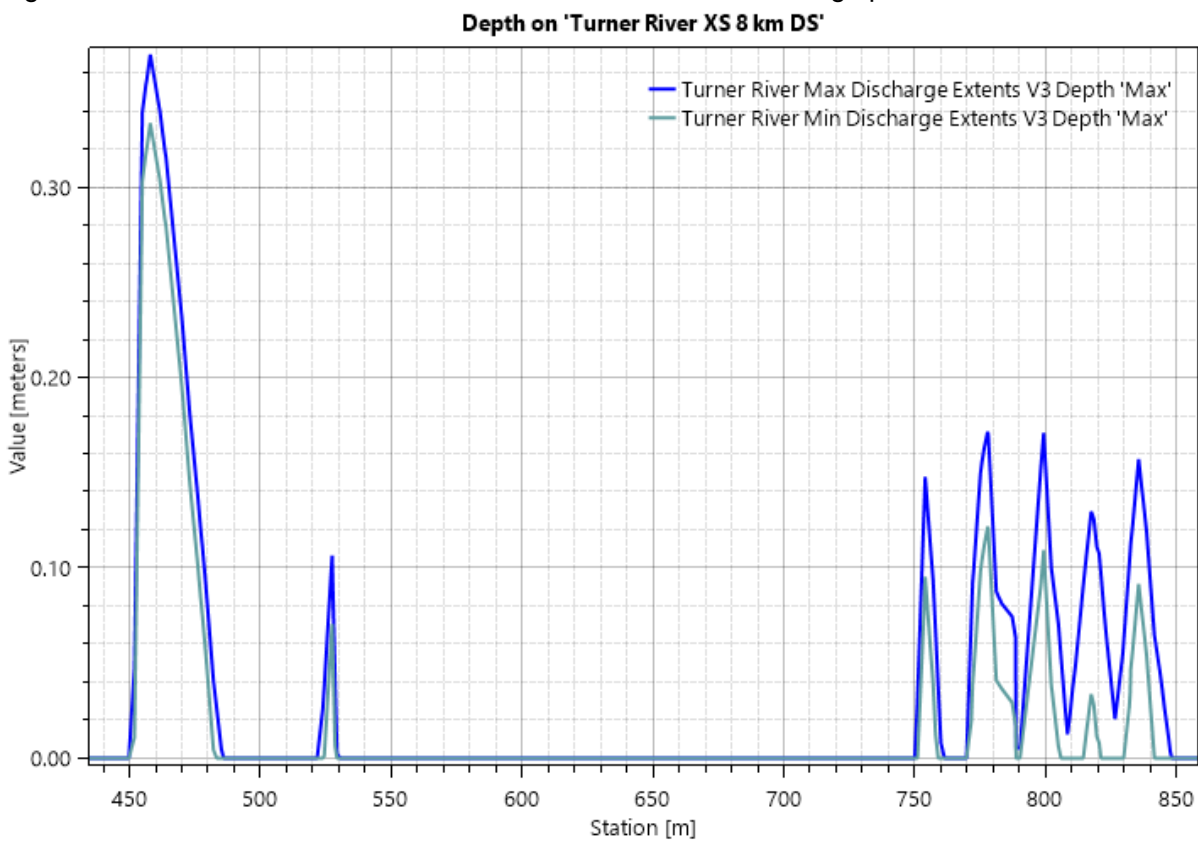


Figure C-10 – Flow depths 8 km downstream of discharge point

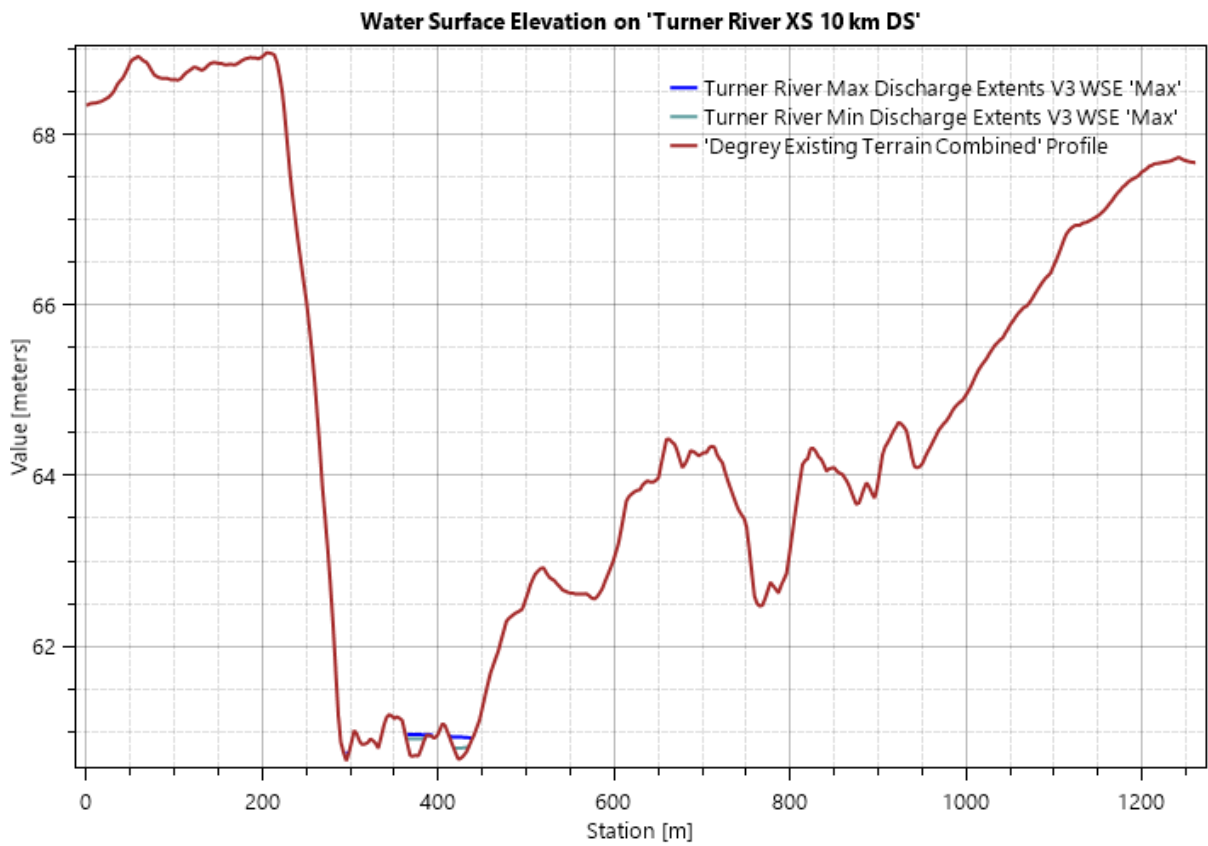


Figure C-11 – Water surface elevations 10 km downstream of discharge point

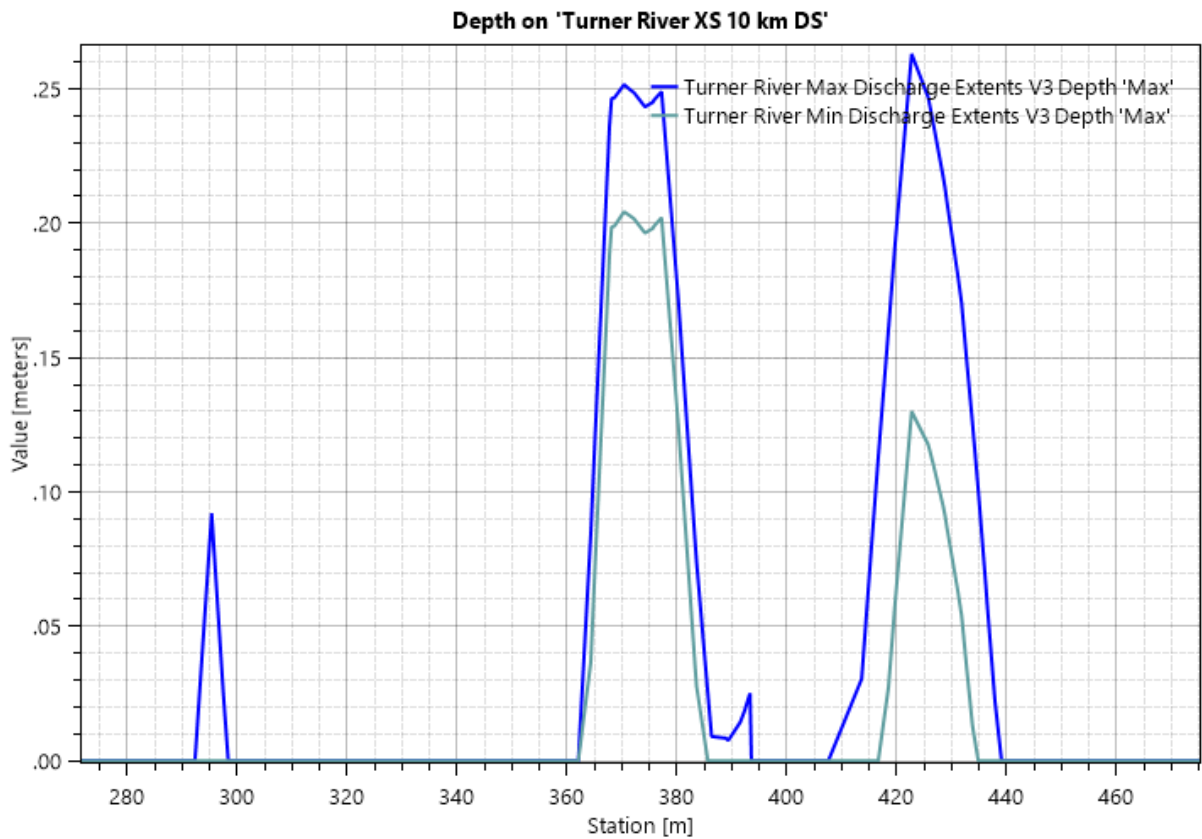


Figure C-12 – Flow depths 10 km downstream of discharge point

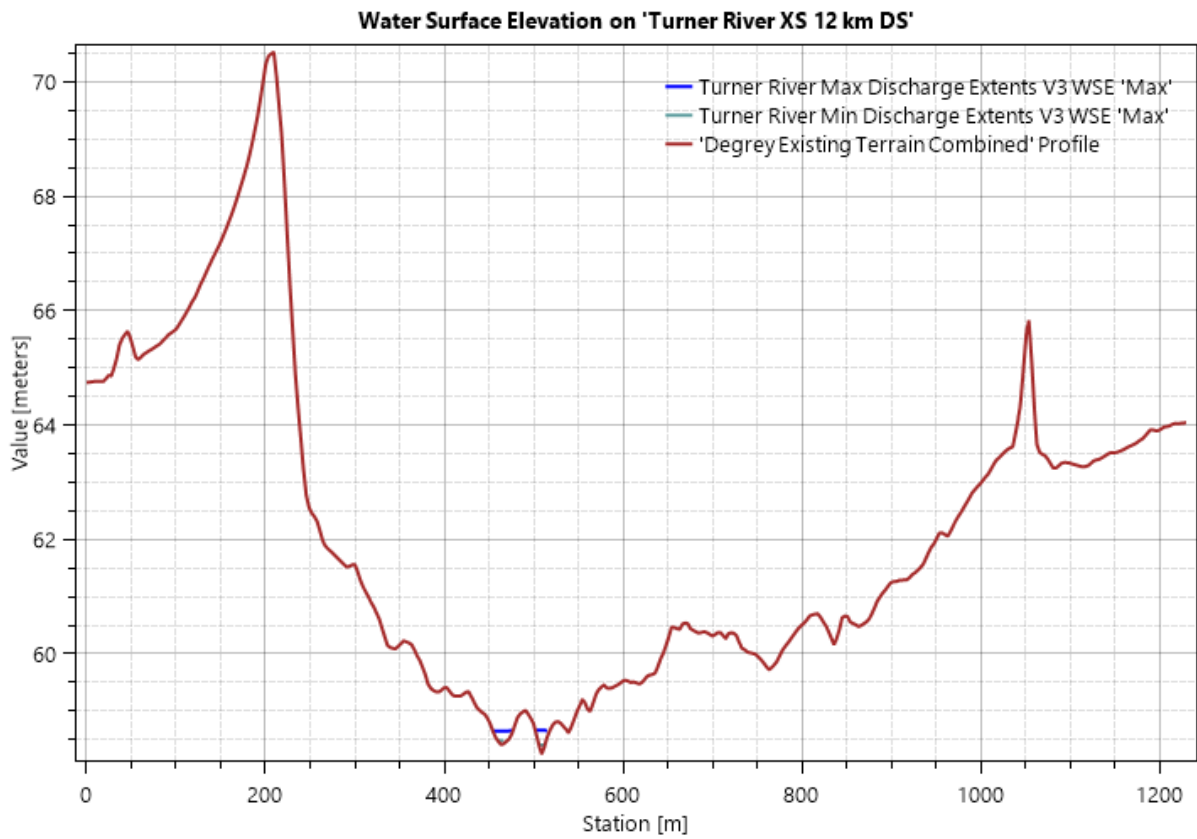


Figure C-13 – Water surface elevations 12 km downstream of discharge point

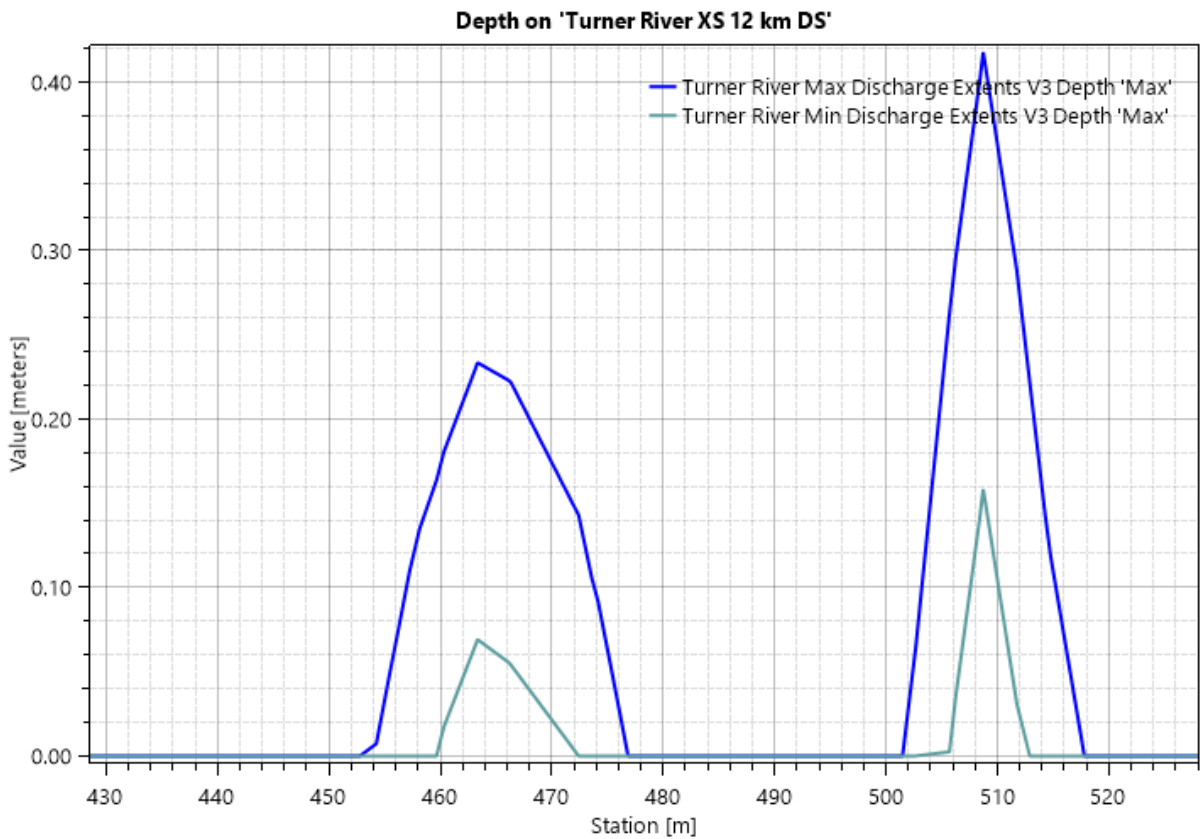


Figure C-14 – Flow depths 12 km downstream of discharge point

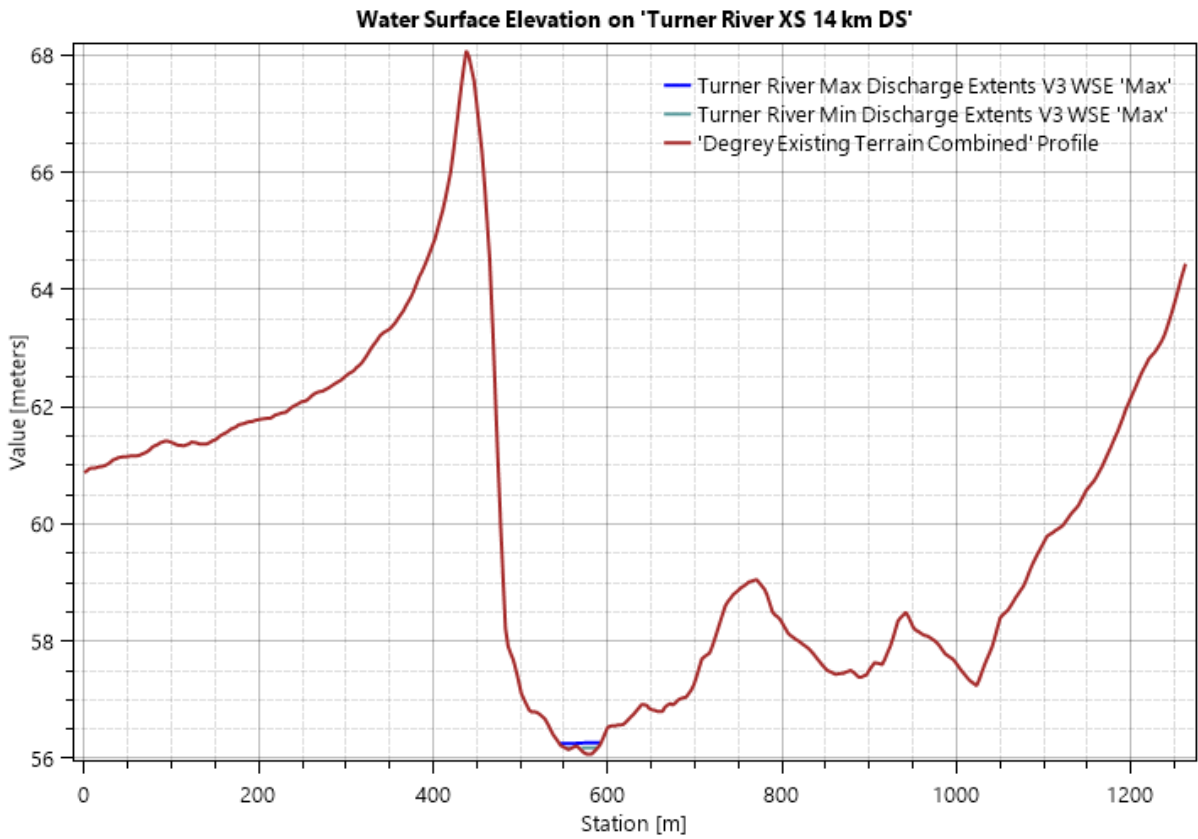


Figure C-15 – Water surface elevations 14 km downstream of discharge point

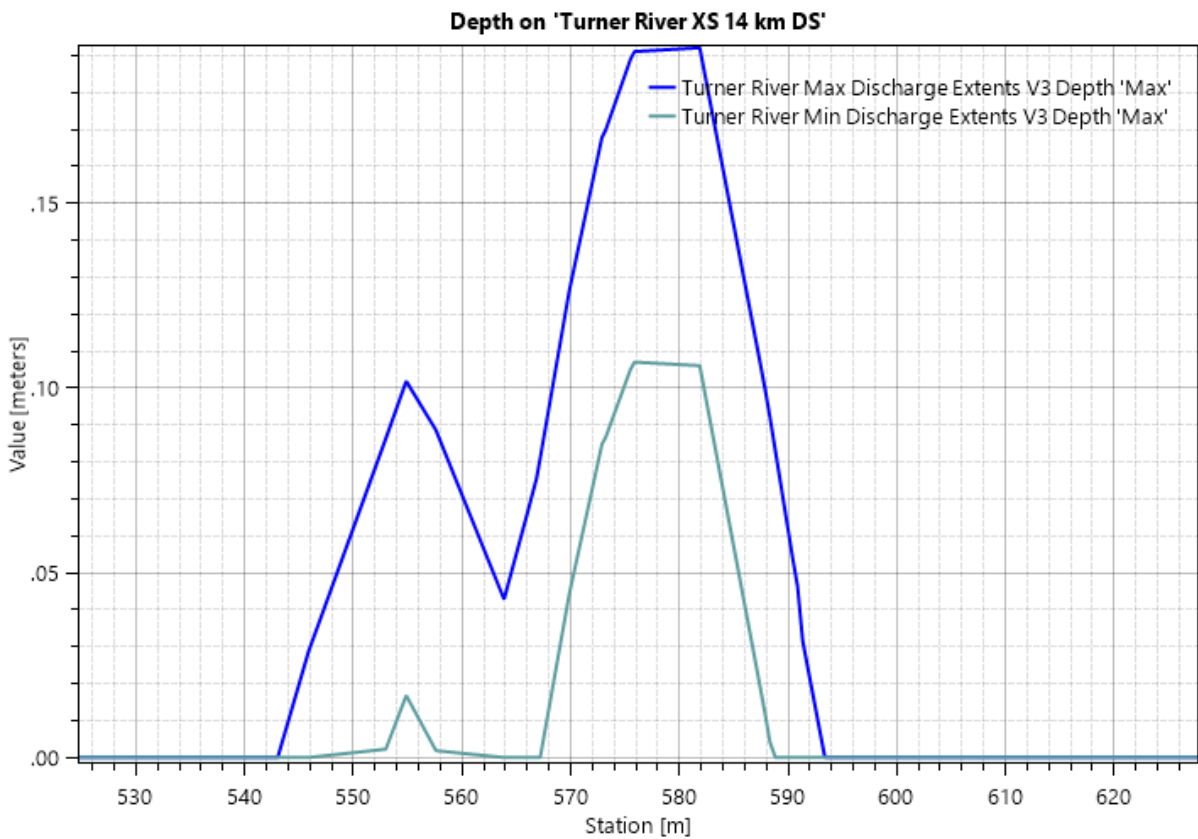


Figure C-16 – Flow depths 14 km downstream of discharge point

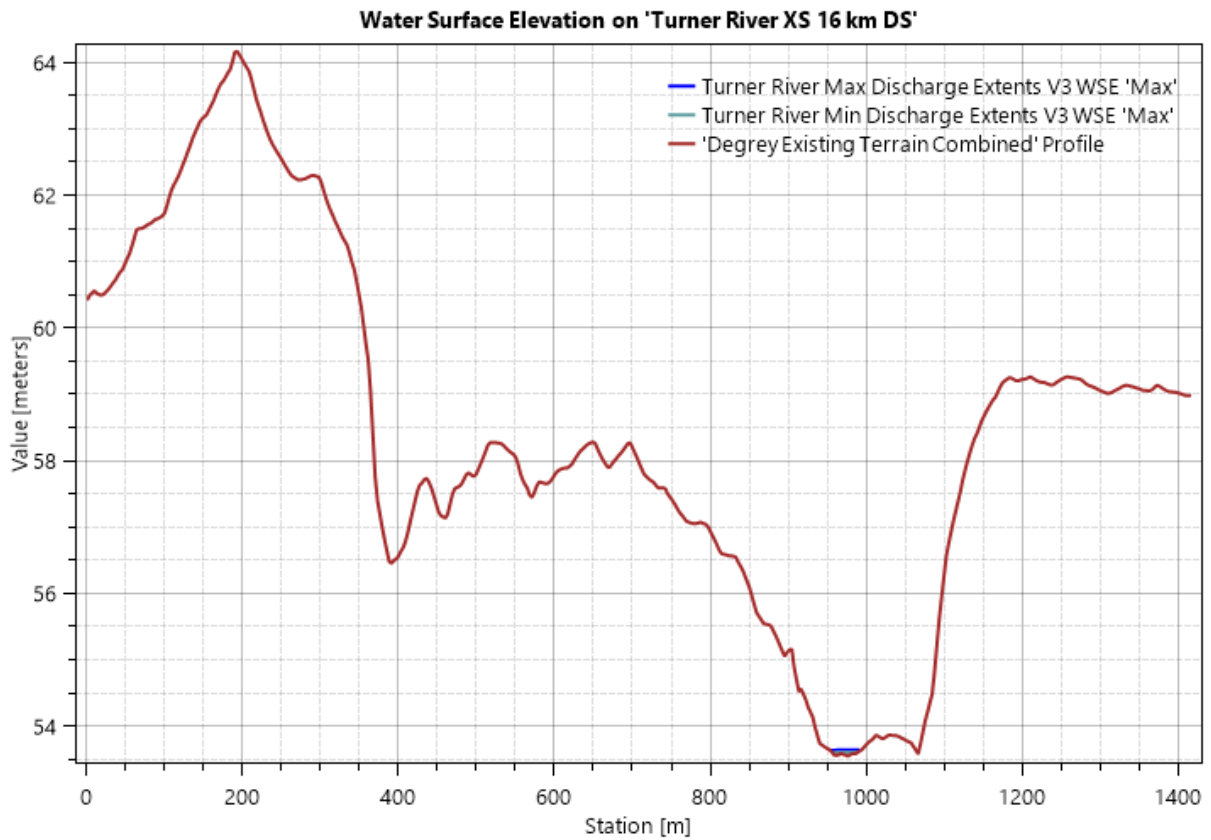


Figure C-17 – Water surface elevations 16 km downstream of discharge point

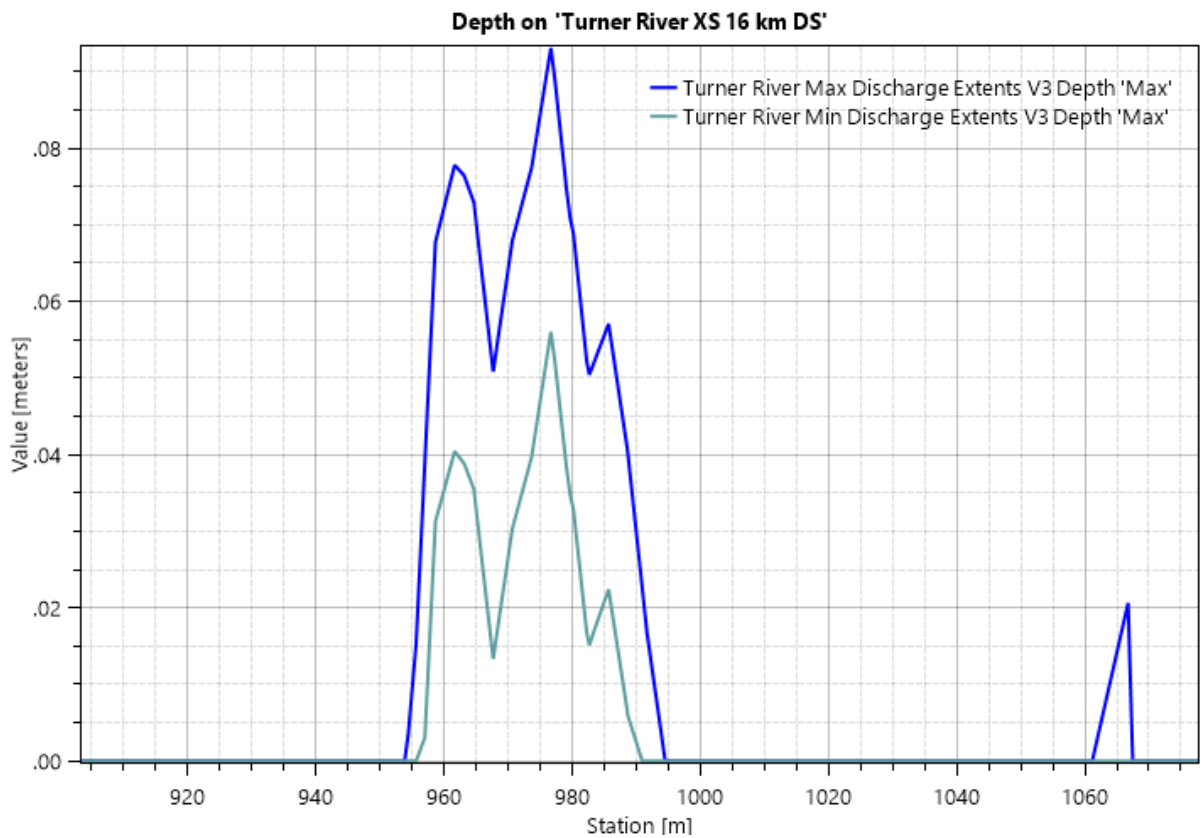


Figure C-18 – Flow depths 16 km downstream of discharge point

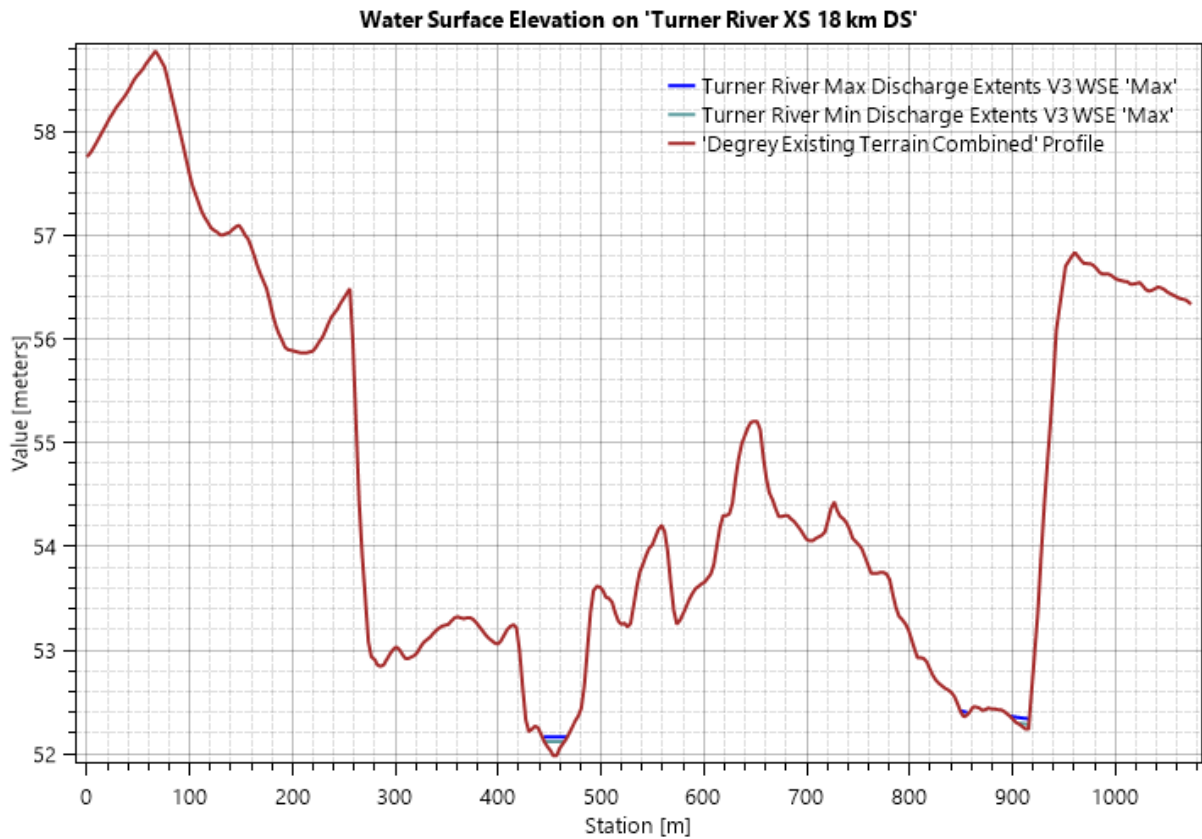


Figure C-19 – Water surface elevations 18 km downstream of discharge point

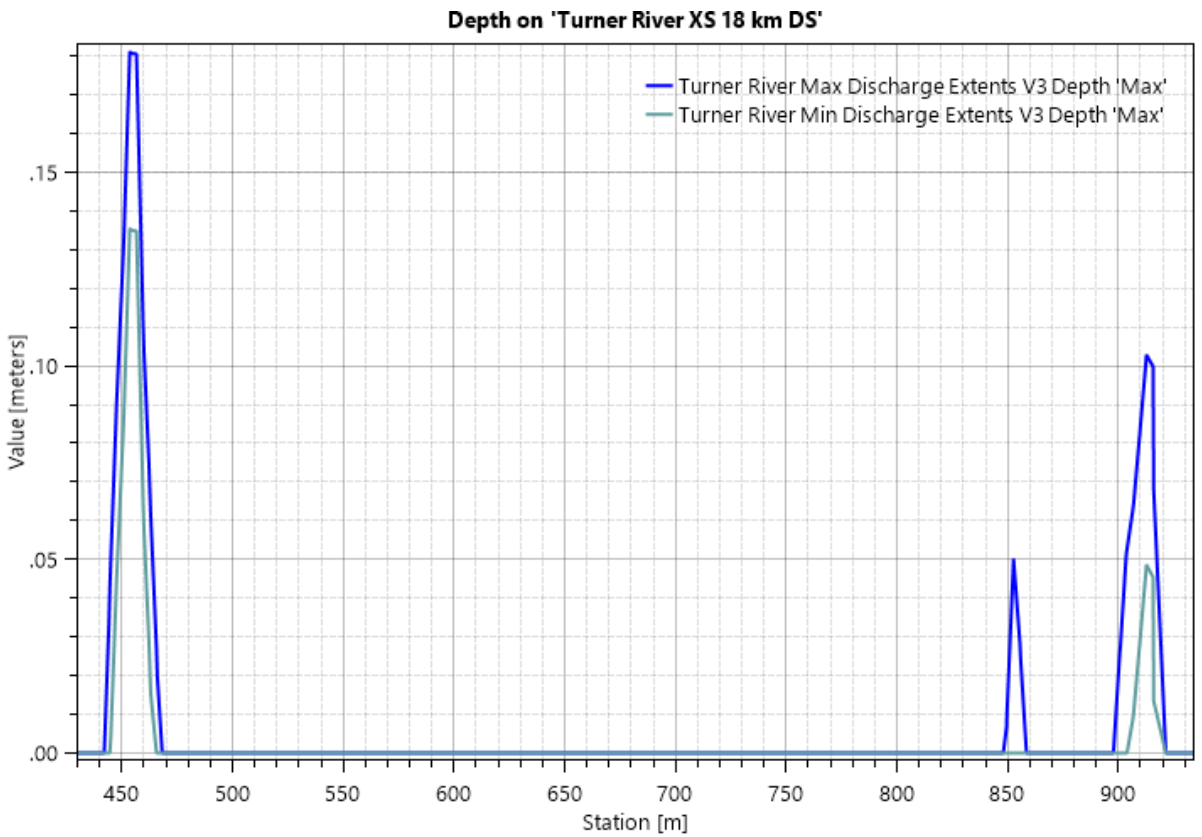


Figure C-20 – Flow depths 18 km downstream of discharge point

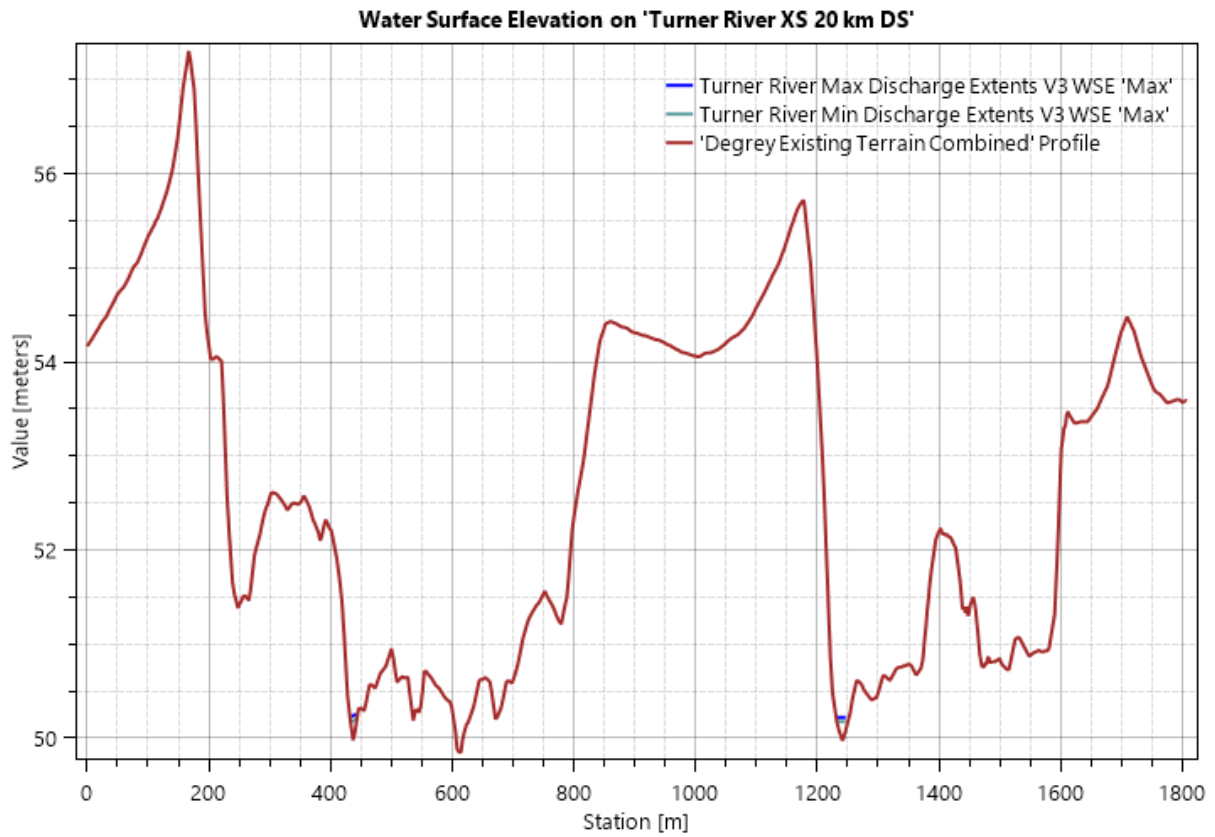


Figure C-21 – Water surface elevations 20 km downstream of discharge point

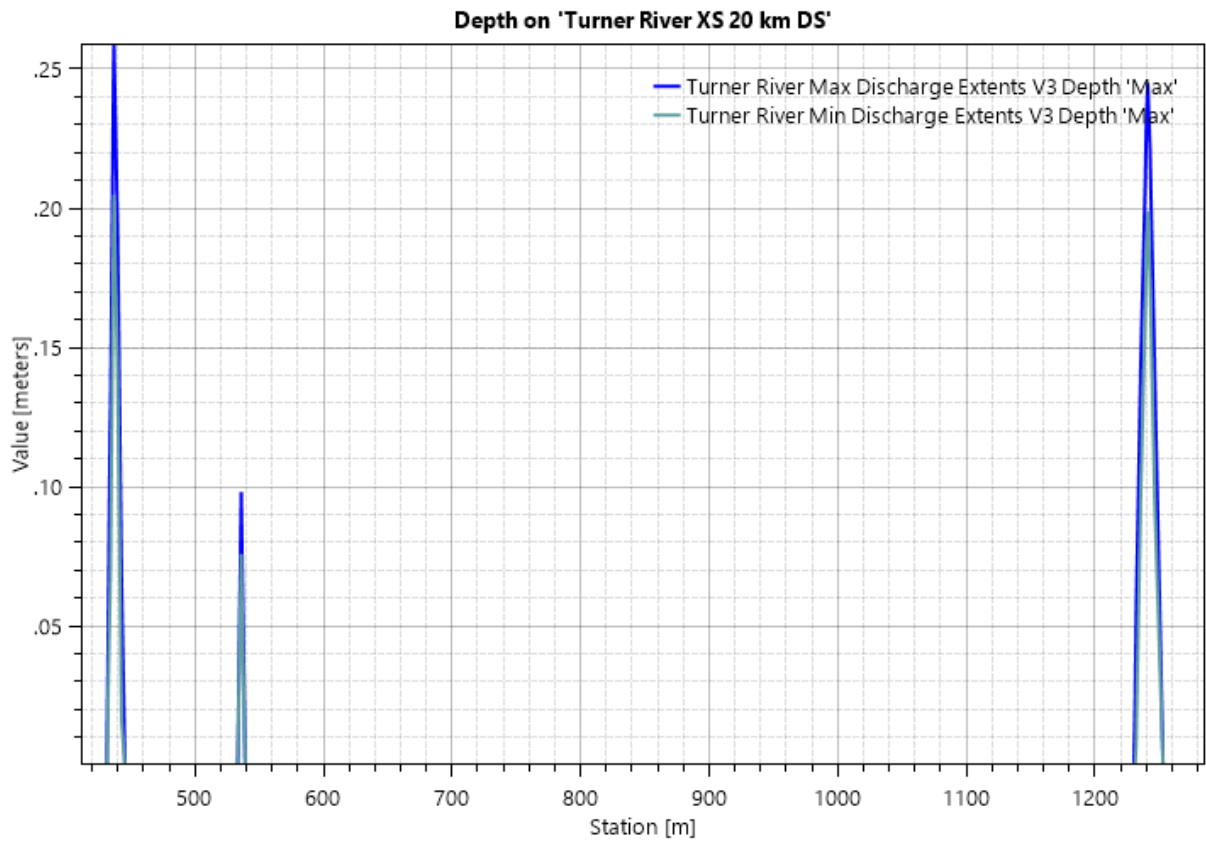


Figure C-22 – Flow depths 20 km downstream of discharge point

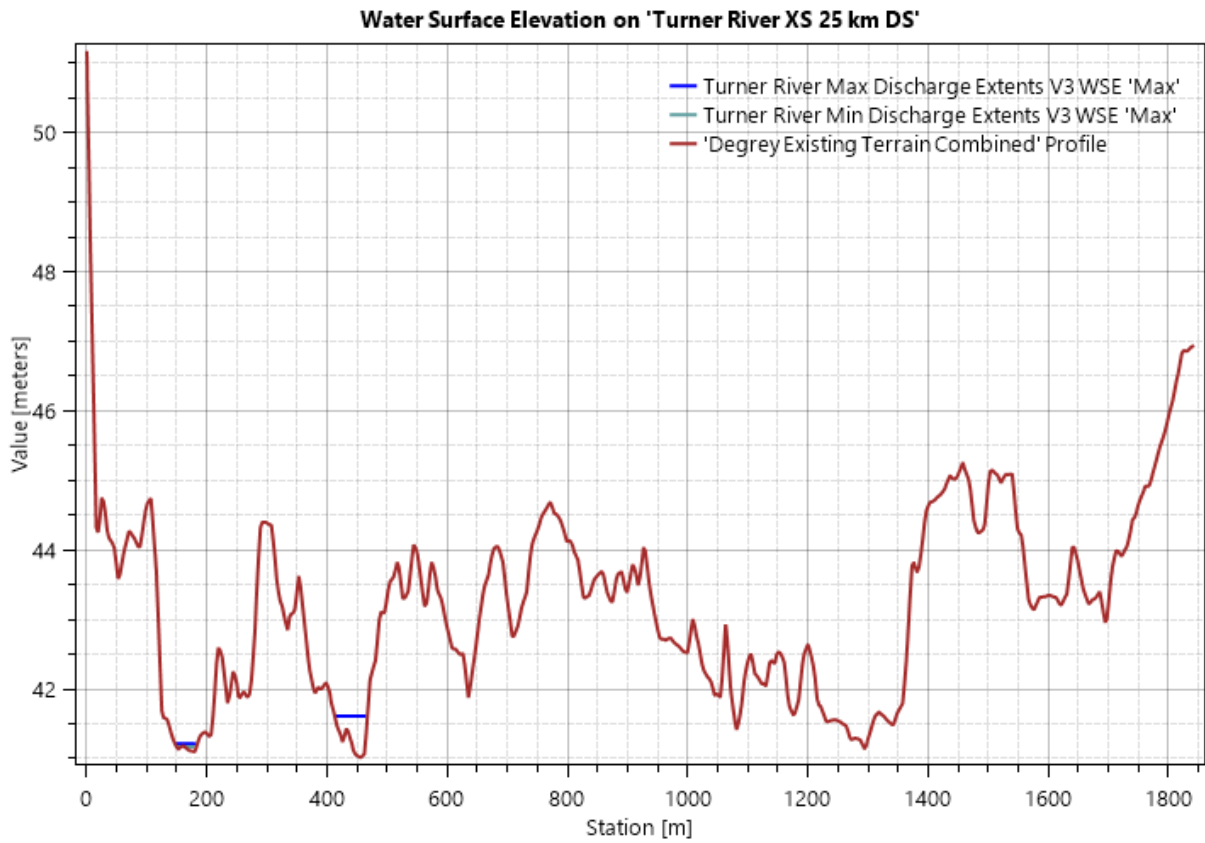


Figure C-23 – Water surface elevations 25 km downstream of discharge point

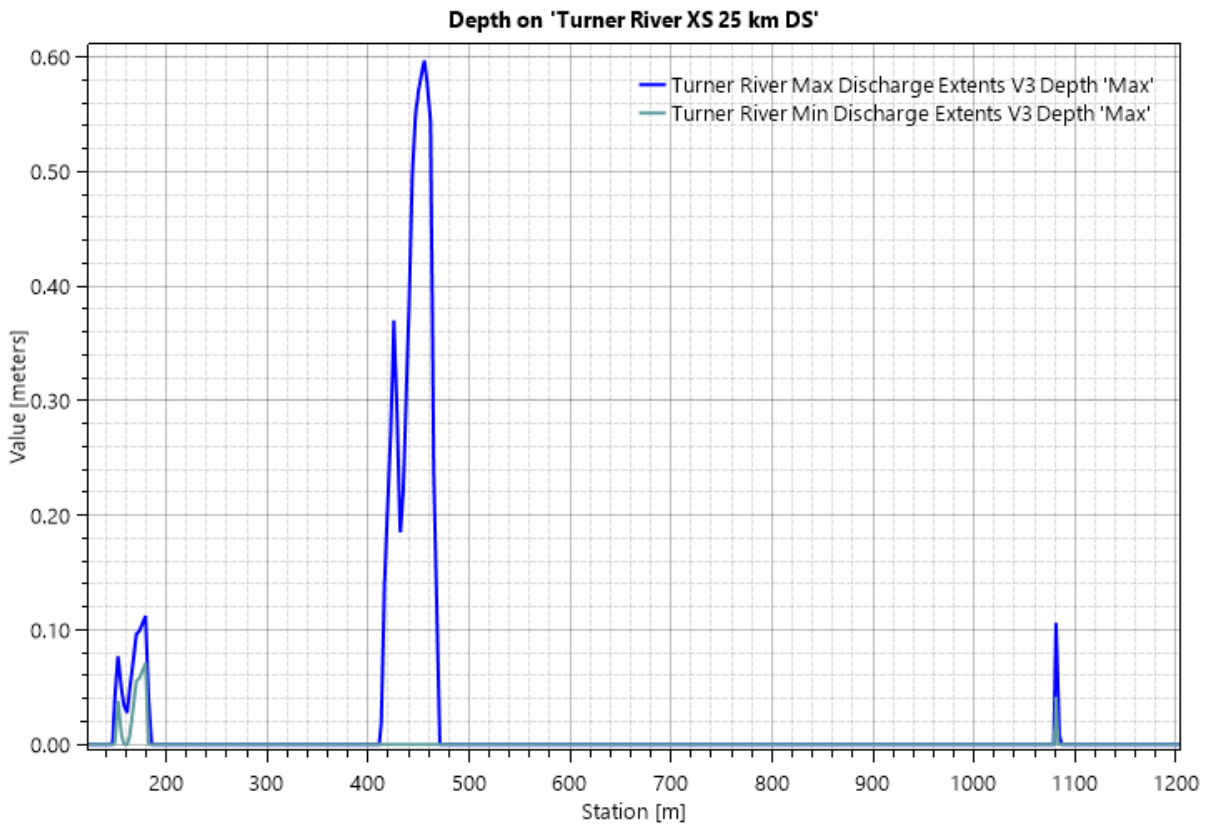


Figure C-24 – Flow depths 25 km downstream of discharge point

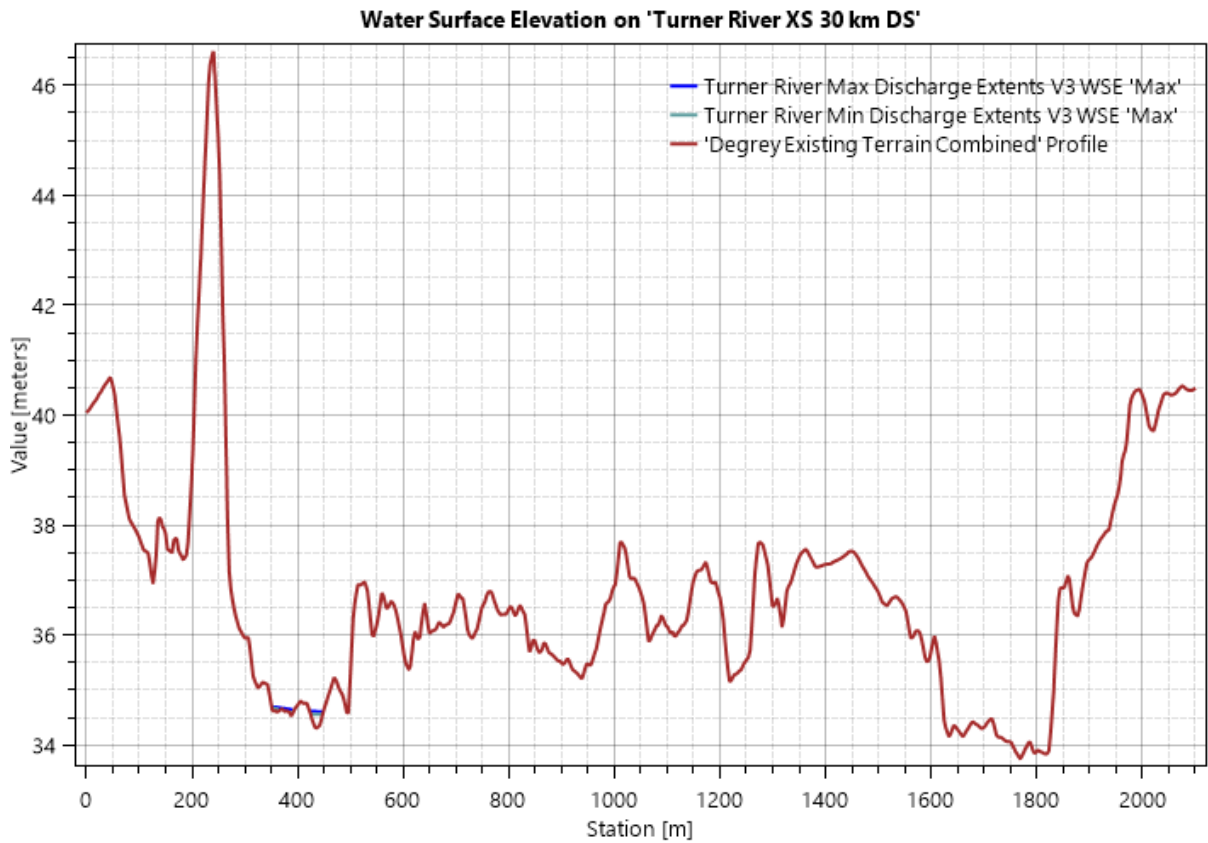


Figure C-25 – Water surface elevations 30 km downstream of discharge point

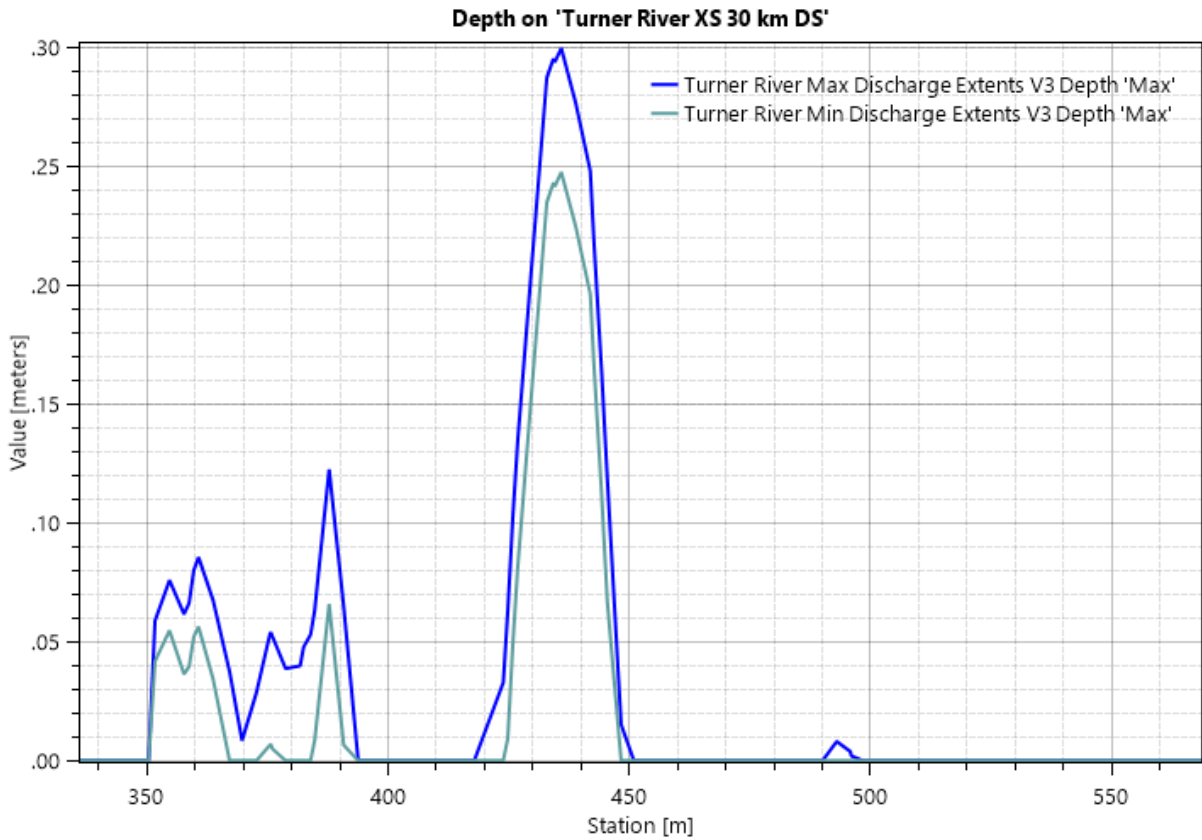


Figure C-26 – Flow depths 30 km downstream of discharge point

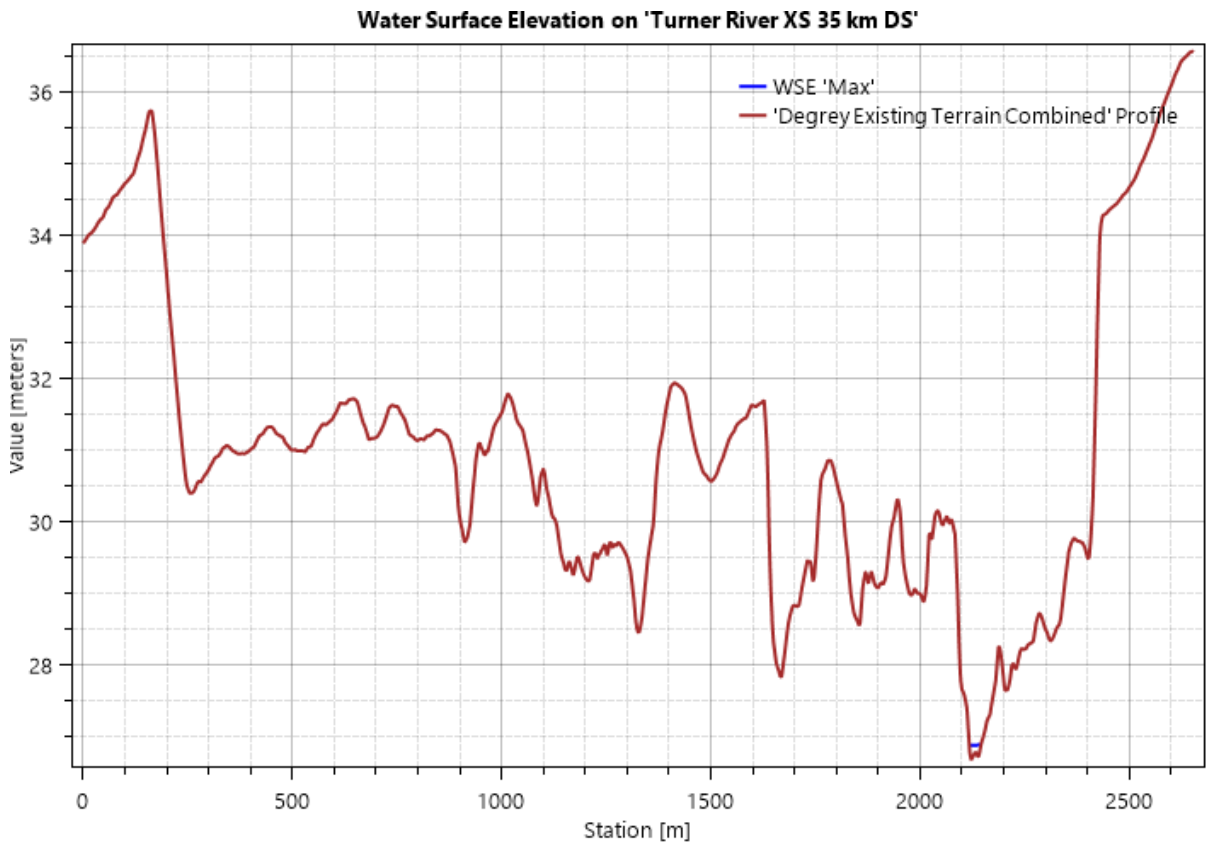


Figure C-27 – Water surface elevations 35 km downstream of discharge point

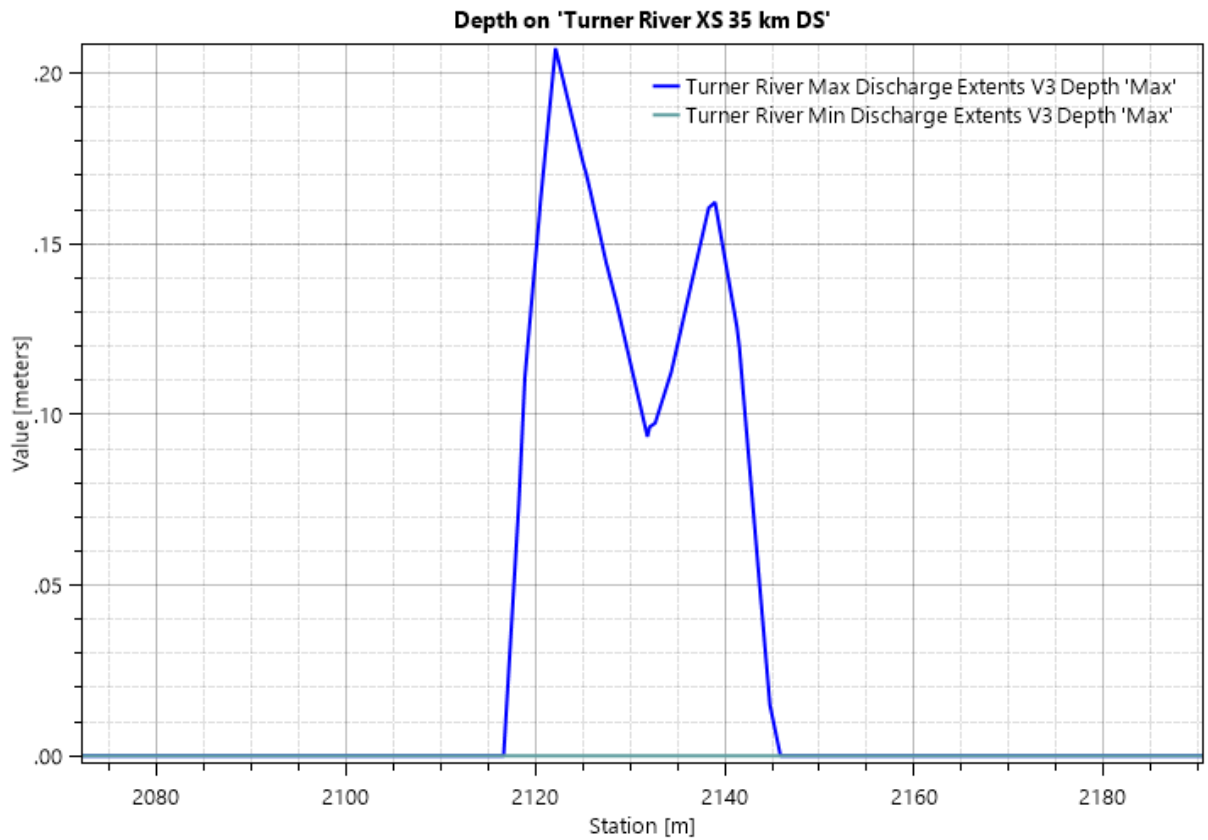


Figure C-28 – Flow depths 35 km downstream of discharge point

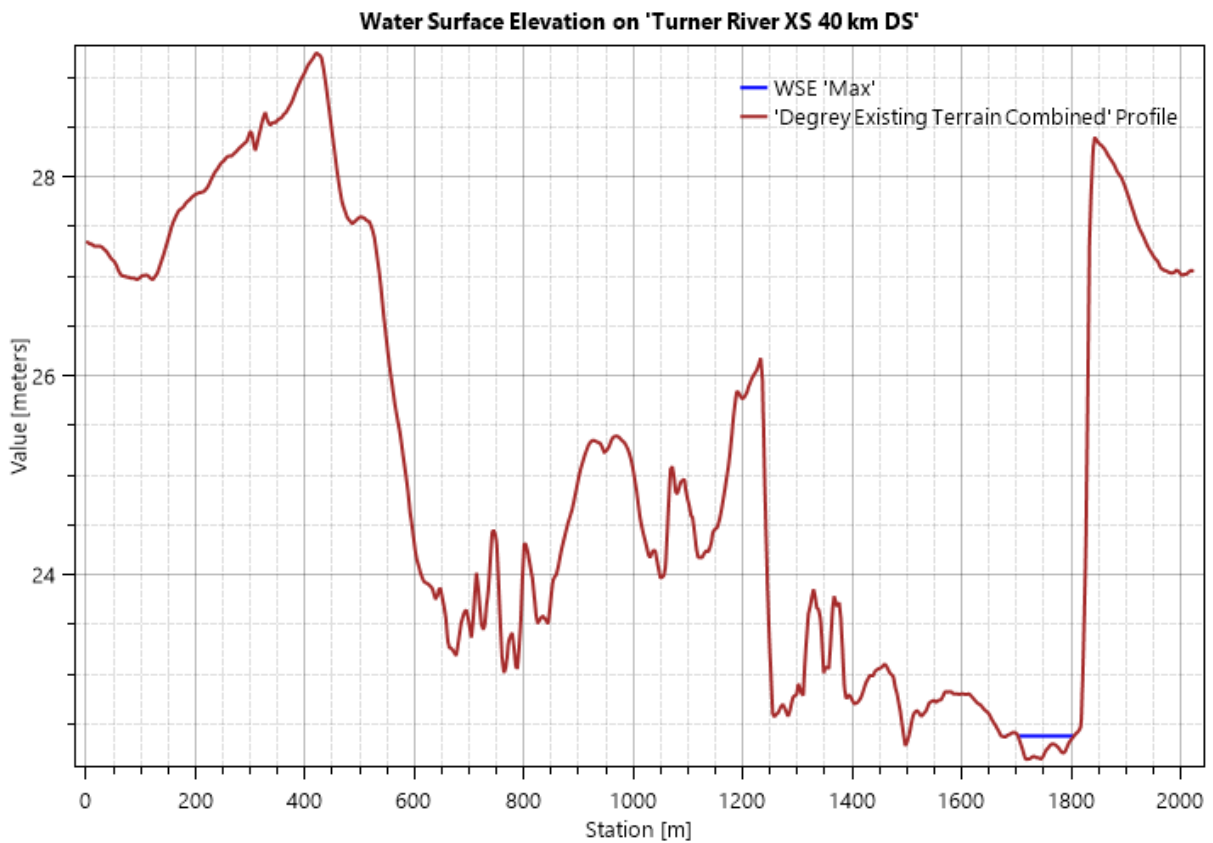


Figure C-29 – Water surface elevations 40 km downstream of discharge point

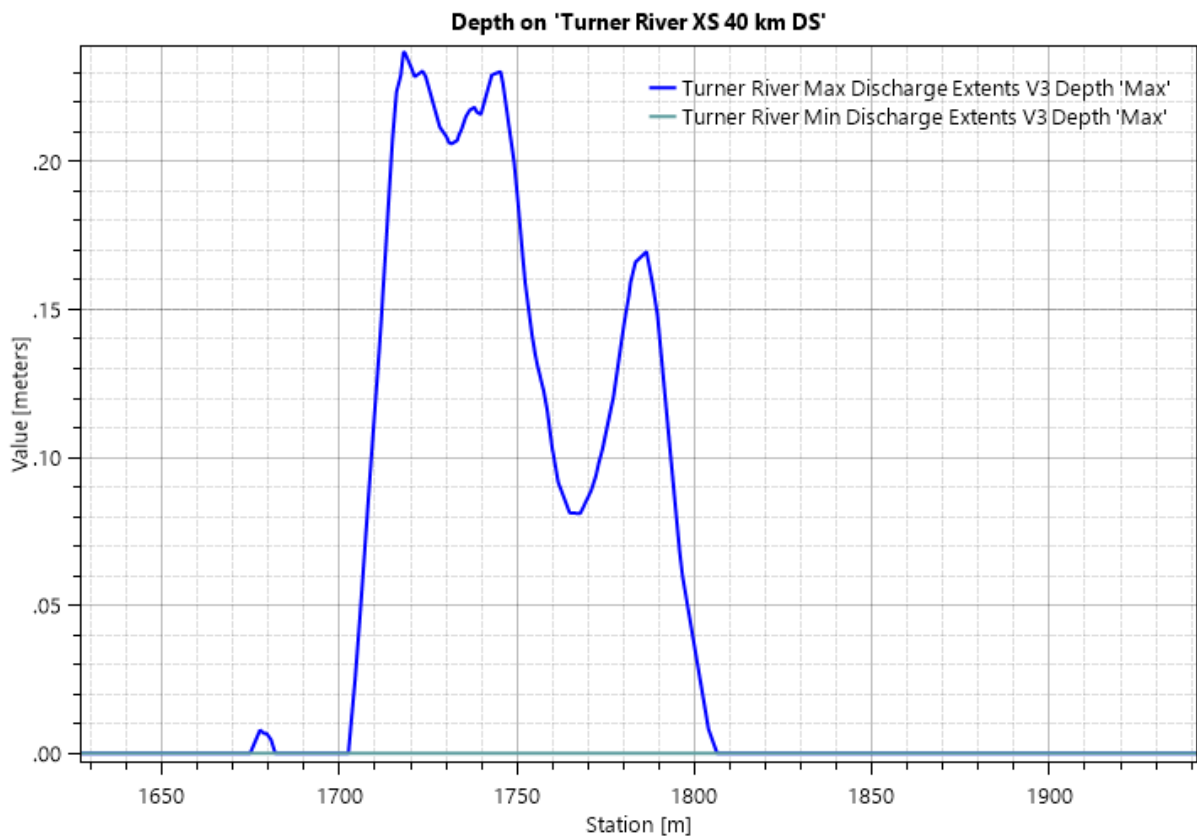


Figure C-30 – Flow depths 40 km downstream of discharge point

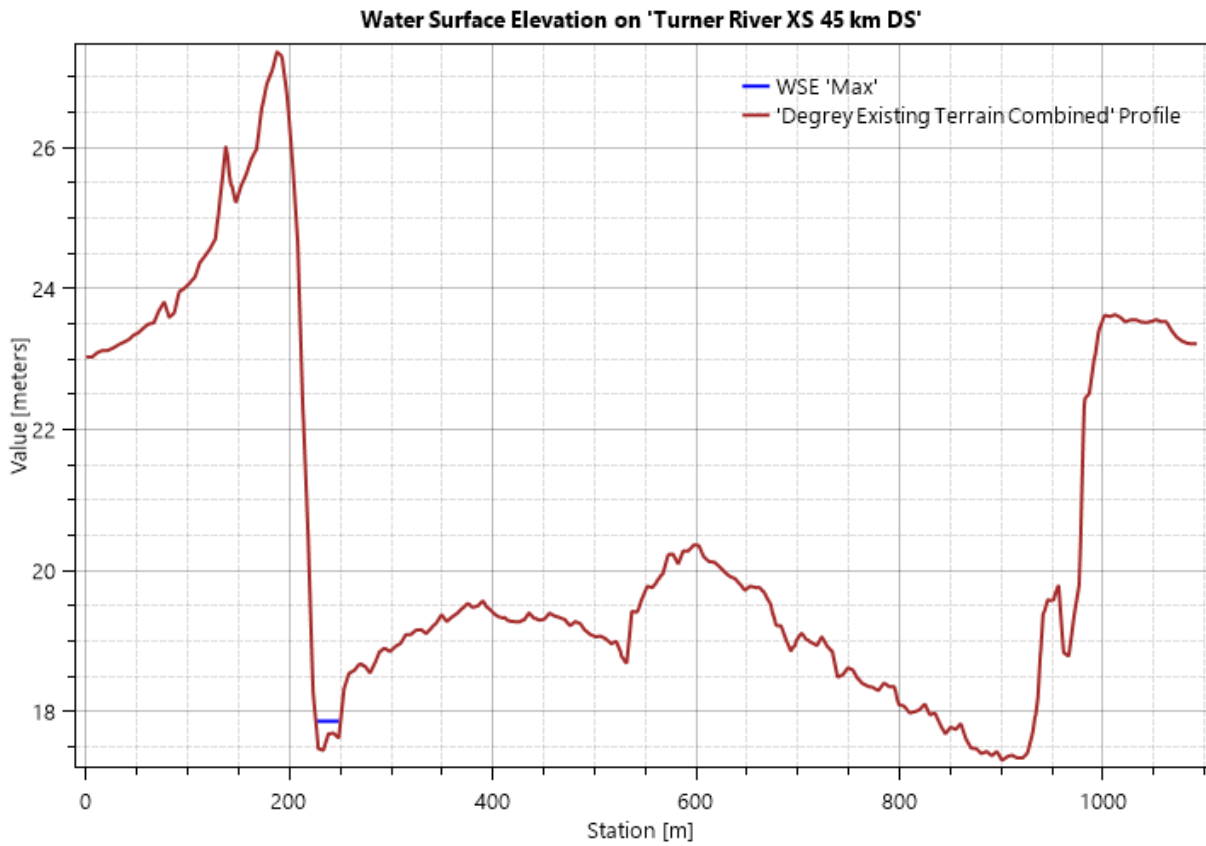


Figure C-31 – Water surface elevations 45 km downstream of discharge point

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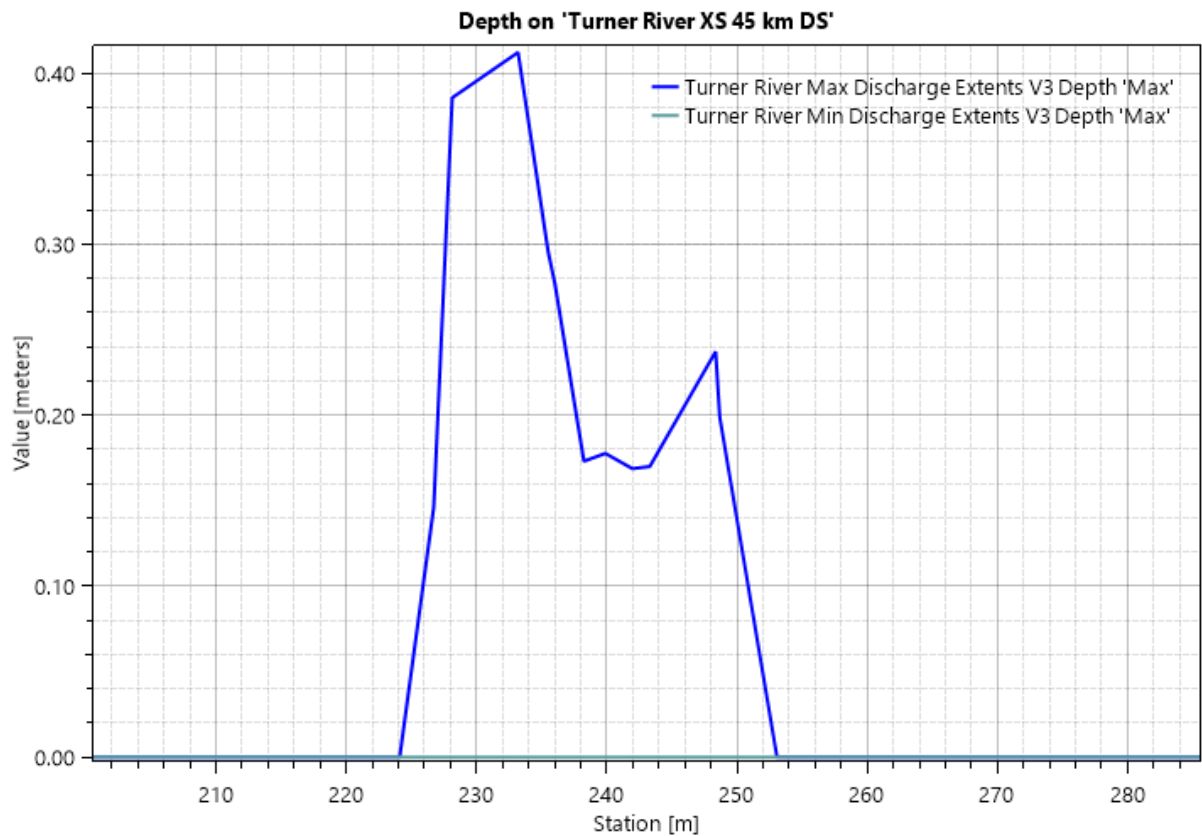


Figure C-32 – Flow depths 45 km downstream of discharge point



Appendix C. Plan View Results

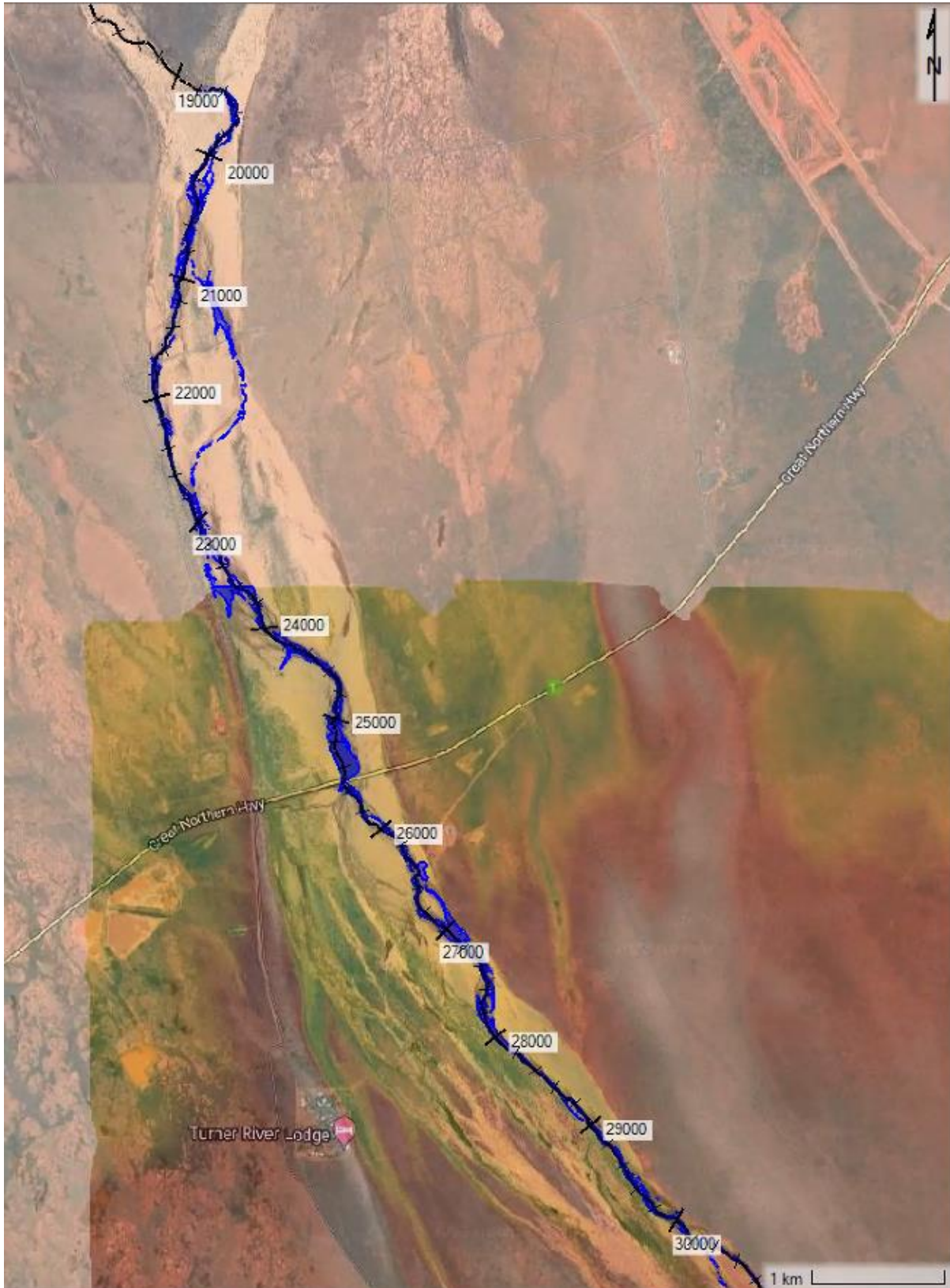


Figure D-1 – High and low discharge inundation extents Chainage 20000-30000



Figure D-2 – High and low discharge inundation extents Chainage 30000-40000

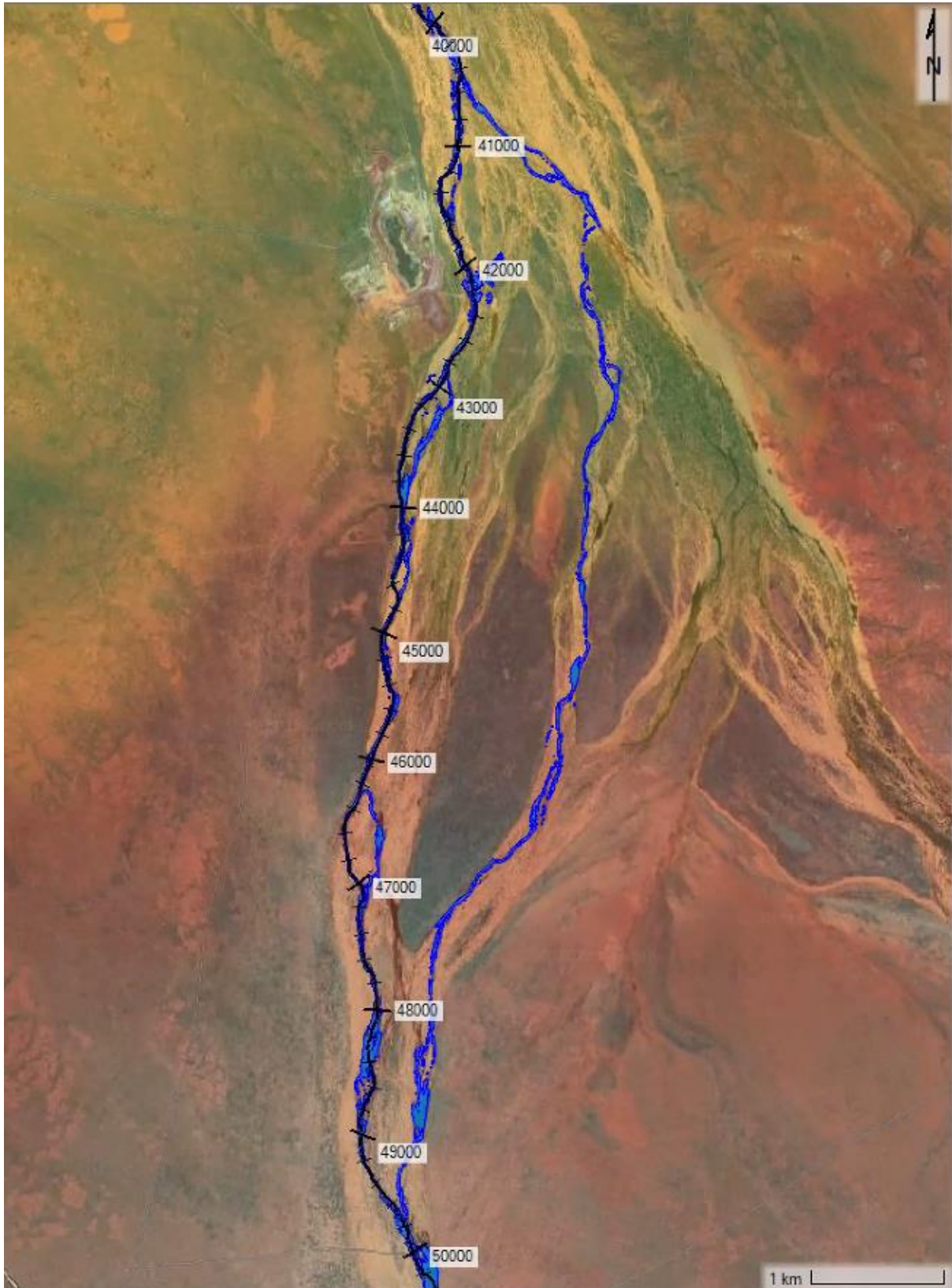


Figure D-3 – High and low discharge inundation extents Chainage 40000-50000

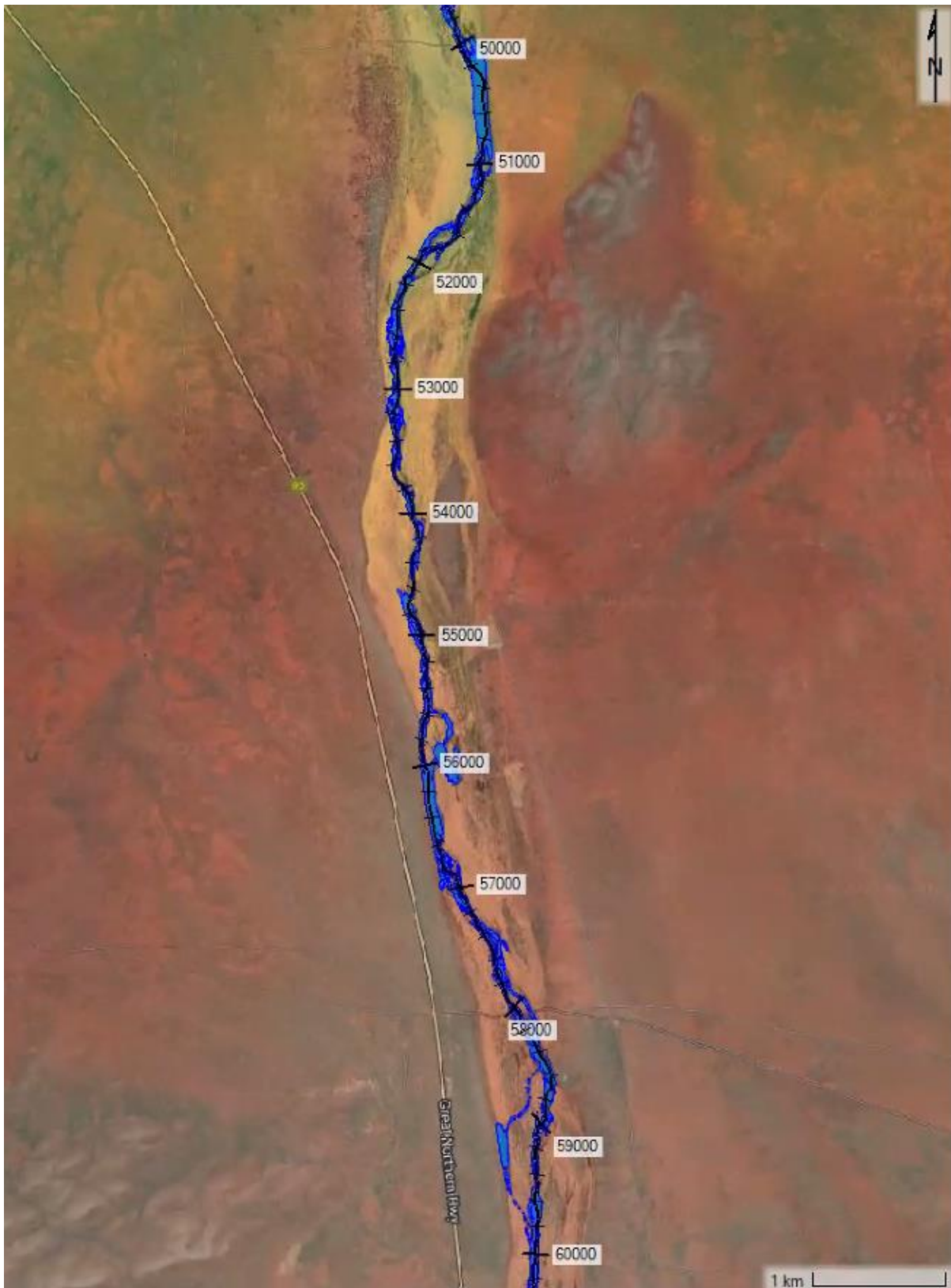


Figure D-4 – High and low discharge inundation extents Chainage 50000-60000

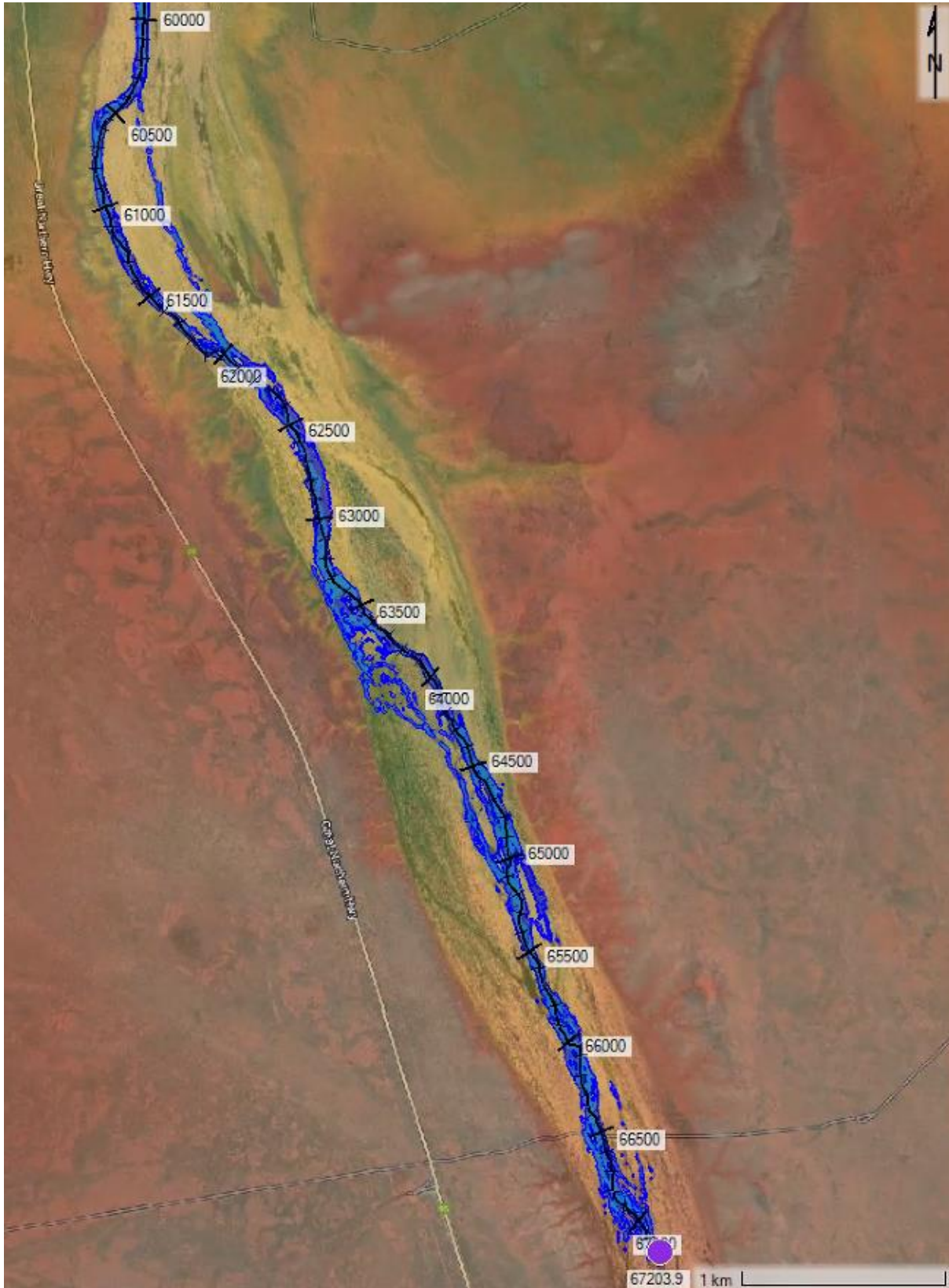
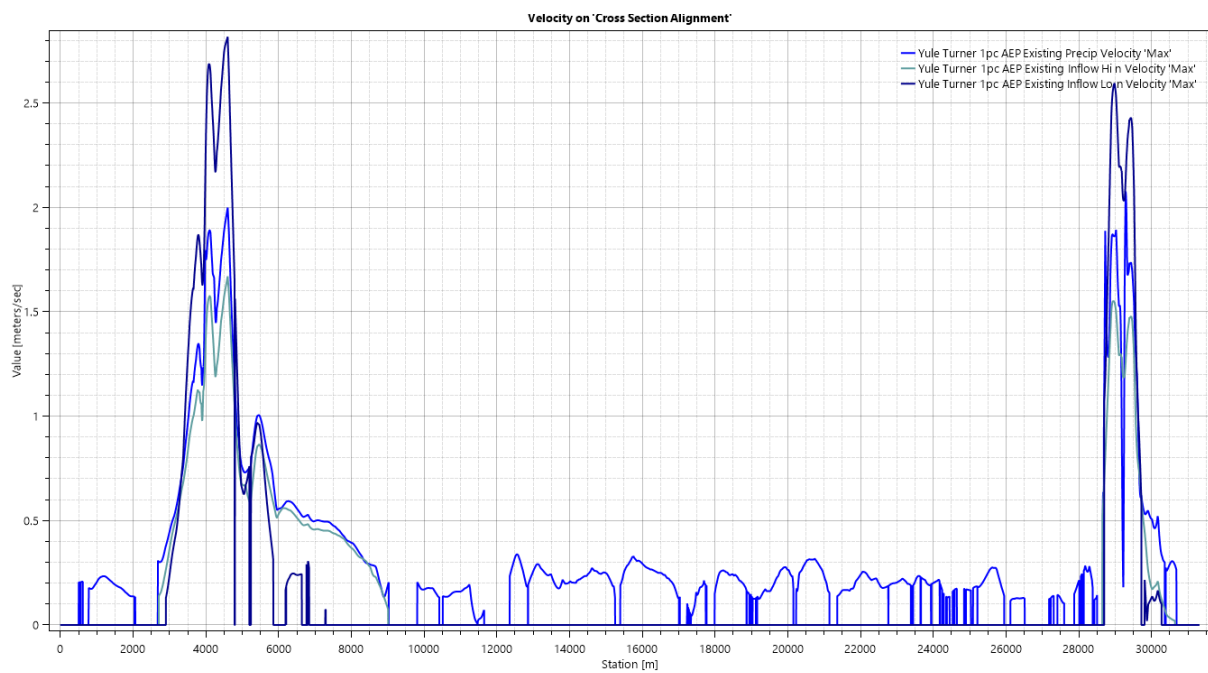
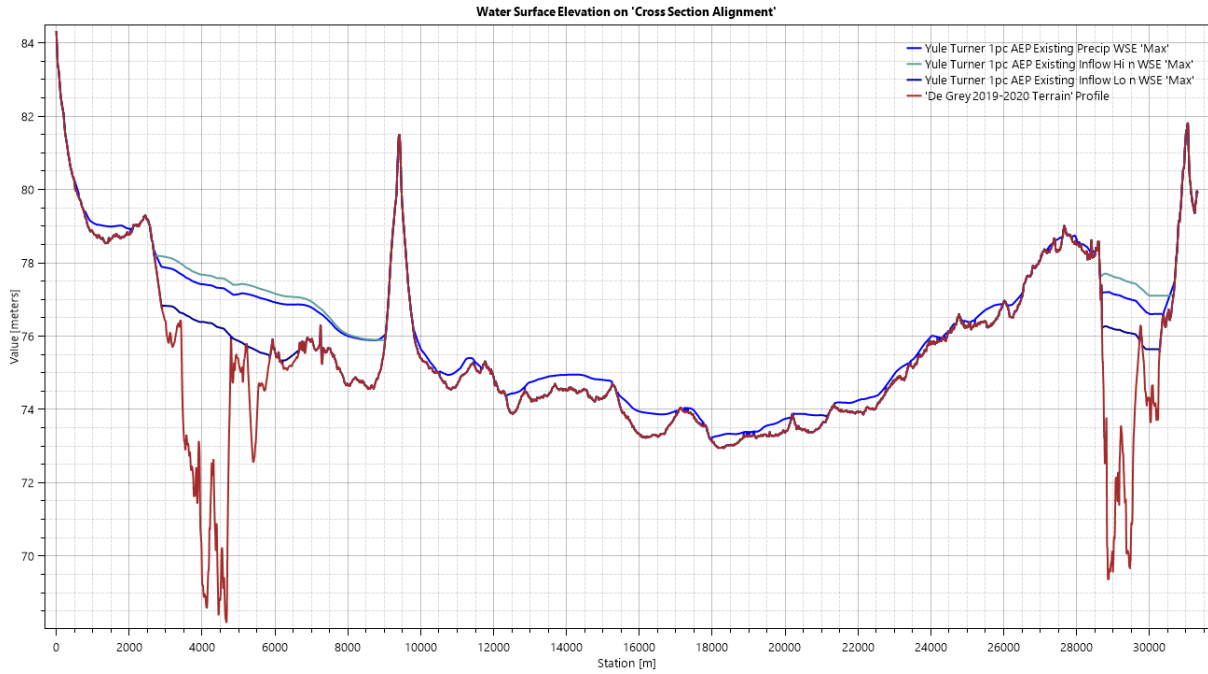


Figure D-5 – High and low discharge inundation extents Chainage 60000-70000



Appendix D. Roughness Sensitivity





Infiltration Sensitivity

