Wearing Street Aquatic and Arts Precinct

City of Onkaparinga

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EXECUTIVE SUMMARY

The City of Onkaparinga is planning upgrades to the Wearing Street Aquatic and Arts Precinct, located at Port Noarlunga. The project is currently in the conceptual planning stage and includes works on the frontage on Onkaparinga River, adjacent to Sauerbier House Arts and Cultural Centre. The area is currently and proposed to continue to be heavily used by the community.

Water Technology was invited by the City of Onkaparinga to undertake a hydrologic and hydraulic assessment of the waterway and prepare a geomorphic assessment of the site. The primary aim of the study is to prepare detailed hydraulic modelling of the local waterway to estimate hydraulic parameters, such as water level and flow velocity, that will inform the geomorphic assessment and overall concept design.

The Wearing Street Aquatic and Arts Precinct is located in Port Noarlunga on the northern bank of Onkaparinga River. The precinct is located immediately downstream of the Saltfleet Street bridge and approximately 1.8km upstream of the river outlet to the St Vincent Gulf. Between Mount Bold Reservoir and the coast, the Onkaparinga River is reasonably sinuous, but very confined through the Onkaparinga River National Park with very little floodplain or room for lateral movement. Once the river exits the hills (from the Main South Road Bridge) the river is laterally unconfined and meanders the remaining distance to the coast.

The study site is located on this meandering, laterally unconfined section of the river. This section of the river is laterally migrating meaning that the channel is moving across the floodplain. This occurs though meander extension where erosion occurs on the outside bank of meander bends due to increased hydraulic energy in that location. At the study site erosion is actively occurring, which is made evident by the undercutting and failure of previous rock and concrete erosion mitigation measures.

Hydrological discharge estimates adopted for the hydraulic modelling of the waterway were based a detailed hydrological study of Onkaparinga River system by the Department for Transport Energy and Infrastructure. The DTEI study reviewed previous hydrological assessments of the river system and included updated hydrological modelling and Flood Frequency Analysis (FFA). The design discharges from the DTEI study have been used as the basis for this analysis.

A new detailed TUFLOW hydraulic model was developed for the lower Onkaparinga River. The TUFLOW model has been used to simulate the existing site conditions for the relevant reach of the lower Onkaparinga River, which tested a range of tidal scenarios. The hydraulic model has been used to estimate peak floodwater depth, velocity, water surface level, and other associated hydraulic outputs required for the geomorphic assessment.

The hydraulic modelling results show velocities along the outer mender bend within the vicinity of the site ranging between 0.6 m/s for 50% AEP flows and 1 m/s for the 1% AEP flow. Velocities in the middle of the channel range between 0.8 m/s and 1.5 m/s. These velocities are not considered high; however, the resulting modelled bed shear stresses of between 6 and 10 N/m^2 are sufficient to move sandy particles. These results support the observations of active erosion and undermining of the previously installed erosion mitigation works.

The currently proposed concept design will leave the beach face and access paths susceptible to erosion. Under base flow and tidal conditions, the beach face is likely to require regular nourishment to maintain its level to which the lower landing of the concrete path is aligned. However, during large events it is likely that substantial erosion will occur along the proposed rock wall. So long as the rock wall is keyed well into the bed this is unlikely to cause damage to the rock wall itself. Likewise, as long as the lower landing of the concrete access path is also keyed in, undermining of the path is unlikely; however, erosion may periodically leave a large step from the landing to the “beach” surface.

Three options are proposed for erosion mitigation at this location:

1. **Implement the currently proposed preferred layout and establish on ongoing maintenance plan:**

   This option is likely to be subject to ongoing and event-based erosion. This will likely necessitate, ongoing maintenance, requiring:

   - Beach nourishment as sand is removed from the beach face.
   - Maintenance to the paths.
   - Event based maintenance to the rock walls.
   - Periodic maintenance to the concrete ramps.

   It is important that rock armouring be designed by an appropriately qualified engineer. RipRap and hydraulic modelling results should be used to determine the rock size and slope for the rock walls. Ideally, concrete should not be used with the RipRap. As the rock settles it is likely that some damage to the concrete will be sustained (cracking etc). Finally, all rock armouring and the lower landing of the ramps should be keyed into the toe of the bank by 4 x the d_{50} diameter (prescribed by RipRap).

2. **A continuous rock wall along the affected area:**

   This option is the most durable and low maintenance option for erosion protection however offers little aesthetic appeal. Access to launch boat craft could be achieved by adding a pontoon or using steel or timber stairs and platforms as described above.

3. **Smaller stepped rock walls in a terraced arrangement.**

   A terraced arrangement may be a balance between the two previous options with stepped beach flats between low lying rock walls. If a terraced beach front is appealing to the community over a high rock wall, the following the following recommendations are made regarding the design:

   - Each terraced wall behind the front wall should be keyed into at least 0.5 metres below the wall in front.
   - Terraced surfaces should have a slight slope equal to approximately 1(h):10(v).
   - Public safety should be considered with regards to the height of each terrace wall. This may result in a requirement for stairs to be added over the rock. These should be steel or timber, but not concrete.
   - If fill is required behind each rock wall, the fill should be track compacted.
   - It is highly likely that some sand will be removed from these surfaces during large events. This will necessitate nourishment efforts from to maintain a beach like aesthetic. This is expected to occur with events larger than 5% AEP.

Given the observed erosion and the hydraulic modelling results, erosion can be expected to continue along this site in the short, medium, and long term. It is considered unlikely that turning the proposed area into an unprotected beach like area is sustainable without maintenance.

If nourishment is cost prohibitive and a “beach like” aesthetic is considered essential, the recommended option is a terraced arrangement as outlined above. However, if ongoing sand nourishment is not an issue, then the proposed concept plan arrangement can be achieved. Under this scenario it is likely that the concrete ramps will sustain damage associated with minor movement of the rock as it settles. As such maintenance of the ramps will also be required. The maintenance effort could be minimised through attention to this aspect through the detailed design.

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1 INTRODUCTION

1.1 Background

The City of Onkaparinga is planning upgrades to the Wearing Street Aquatic and Arts Precinct, located at Port Noarlunga. The project is currently in the conceptual planning stage and includes works on the frontage on Onkaparinga River, adjacent to Sauerbier House Arts and Cultural Centre. The area is currently and proposed to continue to be heavily used by the community.

To assist with the development of the concept plan, Water Technology was invited by the City of Onkaparinga to undertake a hydrologic and hydraulic assessment of the waterway and prepare a geomorphic assessment of the site. The primary aim of the study is to prepare detailed hydraulic modelling of the local waterway to estimate hydraulic parameters, such as water level and flow velocity, that will inform the geomorphic assessment and overall concept design.

Water Technology have previously prepared a memorandum on 11 February 2019 (report number 6518-01_M01v01) which detailed local riverine and oceanic conditions which could cause inundation at the site, undertook an erosion assessment which noted lateral migration of the river channel at the site, and put forward a number of recommendations. The recommendations included keeping the rock revetment, which was proposed to be removed, and to undertake an updated detailed hydrological and hydraulic assessment of the Onkaparinga River at the site and its immediate surrounds. It is understood that the input from the community has noted several issues with the current site, such as access and safety issues particularly with the rocky entrance to the waterway.

1.2 Site Description and Concept Plan

The Wearing Street Aquatic and Arts Precinct is in Port Noarlunga on the northern bank of Onkaparinga River. The precinct is located immediately downstream of the Saltfleet Street bridge and approximately 1.8km upstream of the river outlet to the St Vincent Gulf. A map showing the precinct in the wider area as well as a close up map aerial image of the site is presented in Figure 1-1. A concept plan of the proposed precinct is shown in Figure 1-2.

The Onkaparinga River is a regulated river that runs west from the Mount Bold Reservoir with its headwaters in the Adelaide Hills and as for north as Mount Torrens. Between Mount Bold Reservoir and the coast, the Onkaparinga River is reasonably sinuous, but very confined through the Onkaparinga River National Park with very little floodplain or room for lateral movement. Once the river exits the hills (from the Main South Road Bridge) the river is laterally unconfined and meanders the remaining distance to the coast. The study site is located on this meandering, laterally unconfined section of the river.

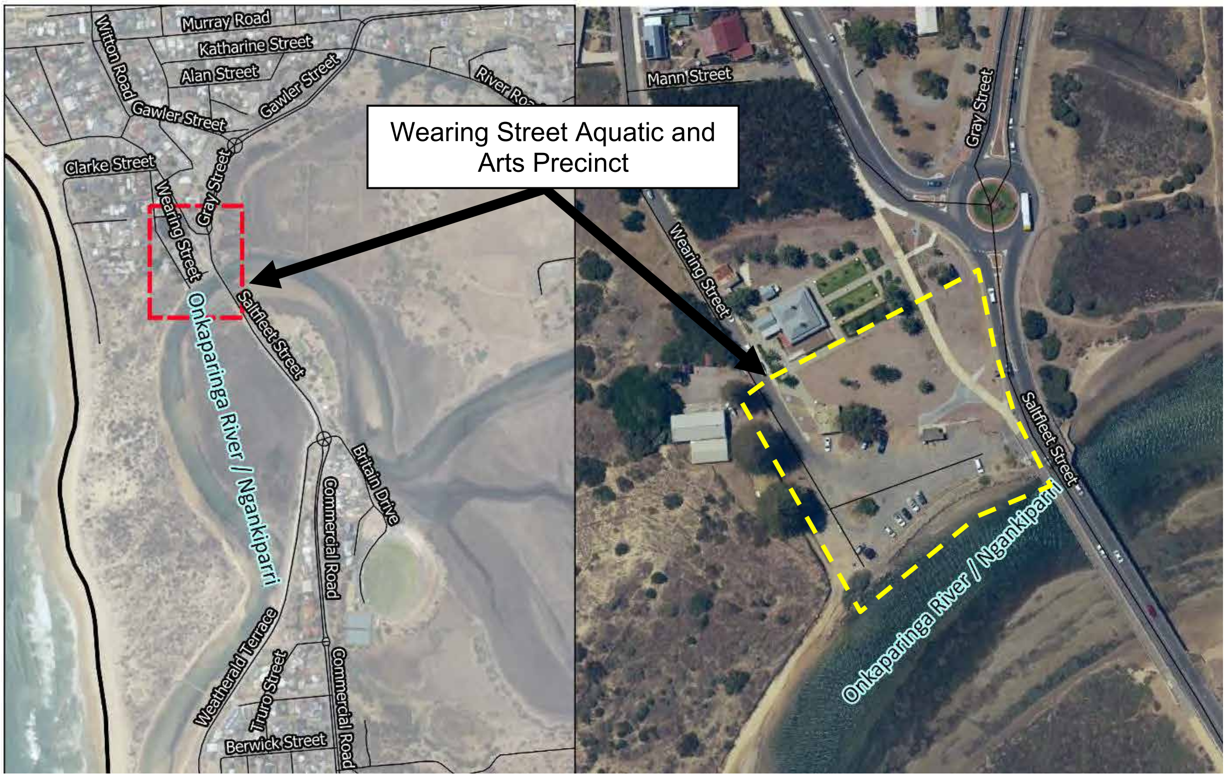


FIGURE 1-1 SITE LOCATION

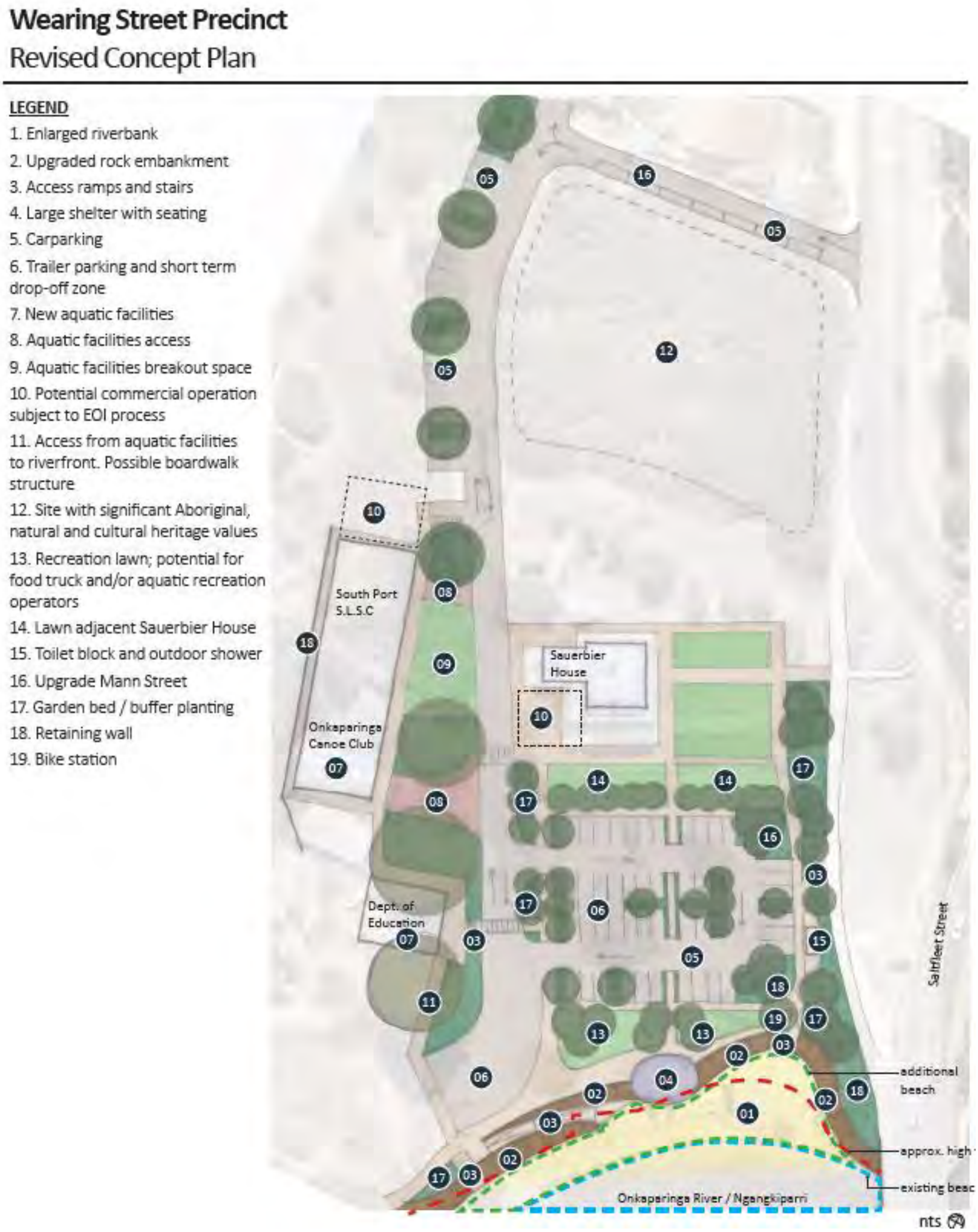


FIGURE 1-2 WEARING STREET AQUATIC AND ARTS PRECINCT – CONCEPT

1.3 Study Methodology

To meet the study aims, the following has been undertaken:

- Site inspection.
  - The site inspection was undertaken by Principal Geomorphologist Dr Michael Cheetham and Principal Hydrologist Geoff Fisher. Geoff is the Project Director who recently inspected the site and is familiar with the area.
- Detailed hydraulic modelling of the lower Onkaparinga River.
  - Hydrological review.
    - A review of available hydrological information was undertaken.
    - Peak design discharges and hydrographs were developed for the lower Onkaparinga River.
  - Hydraulic Model Development and Analysis.
    - A new detailed TUFLOW hydraulic model was developed for the lower portion of the Onkaparinga River.
    - Hydraulic assessment of a range of design flood events up to the 1% AEP for a range of tailwater conditions.
    - Processing and mapping of hydraulic parameters to assist with the geomorphic assessment and concept design. The following outputs were generated for each design event magnitude:
      - Peak depth.
      - Peak velocity.
      - Peak water level.
      - Peak bed shear stress.
      - Peak Stream Power.
- Geomorphological analysis.
    - Provide advice on whether rock protection is advised.
    - Provide advice on either altering certain aspects of the proposed concept design.

The detailed assessment methodology, results of the analysis and brief discussion are included in this report.

2 HYDRAULIC ASSESSMENT

2.1 Hydrology Design Discharge Estimation

2.1.1 Overview

The Onkaparinga River is a complex hydrological system with Mount Bold Dam and Clarendon Weir affecting design discharges magnitudes and timing for the lower Onkaparinga River. KBR (2010) have previously undertaken a detailed hydraulic study of the Seaford Rail Extension, which crosses the Onkaparinga River approximately 2.9 km upstream of the site. The KBR study incorporated hydrological information based on a detailed hydrological study of Onkaparinga River system by the Department for Transport Energy and Infrastructure (DTEI, 2010). The DTEI study reviewed previous hydrological assessments of the river system and included updated hydrological modelling and Flood Frequency Analysis (FFA). The design discharges from the DTEI study have been used as the basis for this analysis.

2.1.2 Design Discharges

The DTEI hydrological study provides estimates of design discharges at Old Noarlunga for the 10% AEP up to Probable Maximum Flood (PMF) design flood events. Old Noarlunga is relatively close to the site in the context of the Onkaparinga River system; therefore, the design discharges for this location are applicable for this study. Design estimates at Old Noarlunga were not provided for events more frequent than the 10% AEP (39% and 18% AEP), however FFA estimates for these frequent events are provided for Clarendon Weir, approximately 22km upstream of Old Noarlunga. Clarendon Weir is too far upstream for these flows to be applicable at the site, therefore the flows for these events at Old Noarlunga were based on the ratio of these events to the 2% AEP discharge at Clarendon Weir. The 2% AEP discharge was adopted as the based discharge as it was the only hydrograph provided for the study, as discussed in the next section (hydrograph estimation) of this report.

The FFA results at Clarendon Weir for the 39%, 18% and 2% AEP design floods are presented in Table 2-1. The fraction of the 2% AEP peak discharge was applied to the 2% AEP peak discharge to estimate peak discharge for these events at Old Noarlunga.

TABLE 2-1 ESTIMATED FFA DISCAHRGE AT CLARENDON WEIR (SOURCE: KBR)

Design Event (AEP)	Peak Discharge (m^3/s)	Proportion of 2% AEP Peak Discharge
39%	86	0.32
18%	120	0.45
2%	268	-

The adopted peak design discharges at Old Noarlunga and as inflow to the hydraulic model are presented in Table 2-2.

TABLE 2-2 ESTIMATED DISCHARGE AT OLD NOARLUNGA (SOURCE: KBR)

Design Event (AEP)	Discharge (m^3/s)
39%	96
18%	134
10%	174
5%	220

Design Event (AEP)	Discharge (m^3/s)
2%	299
1%	388
0.2%	1,029
0.05%	1,705
PMF	5,210

2.1.3 Design Hydrograph Estimation

The design hydrographs from the KBR study were not available for this assessment. Kemp (2010) undertook a coincident storm surge and flood assessment to estimate the likelihood that a storm surge would occur in conjunction with a large riverine flood event. The results indicated that a large storm which could cause a storm surge event would occur around the timing of the peak rainfall around 24-hours prior to the Onkaparinga River peaking through the lower reaches of the system. In the report they included a hydrograph export from their hydrology model for the 2% AEP design flood, which is presented in Figure 2-1. The design hydrographs adopted for this study are based on the provided hydrograph and scaled to the relevant peak discharges shown previously in Table 2-2.

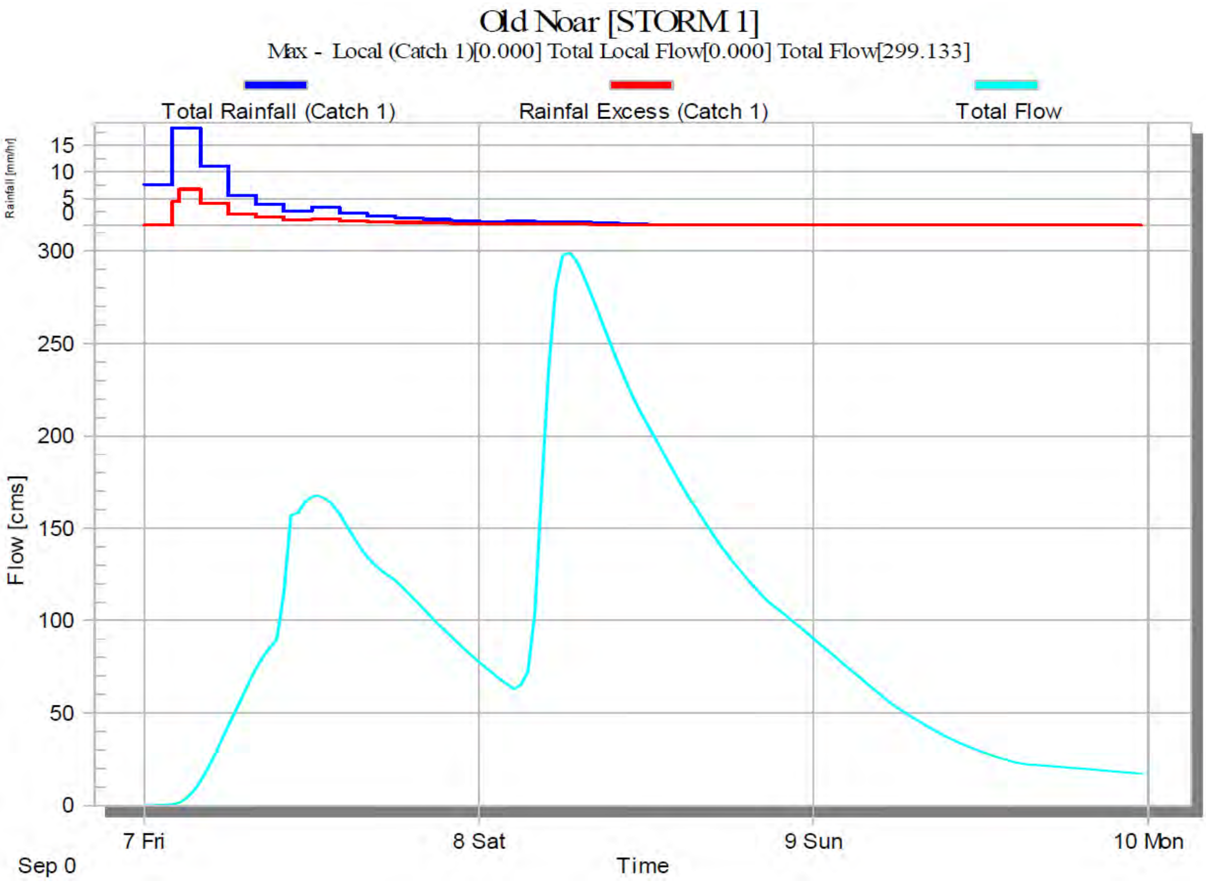


FIGURE 2-1 2% AEP 36-HOUR DESIGN STORM HYDROGRAPH

The adopted design hydrographs for the 39% AEP (2yr ARI) to 1% AEP (100yr ARI) are shown on Figure 2-2. These hydrographs were used as the Onkaparinga River inflow to the hydraulic model.

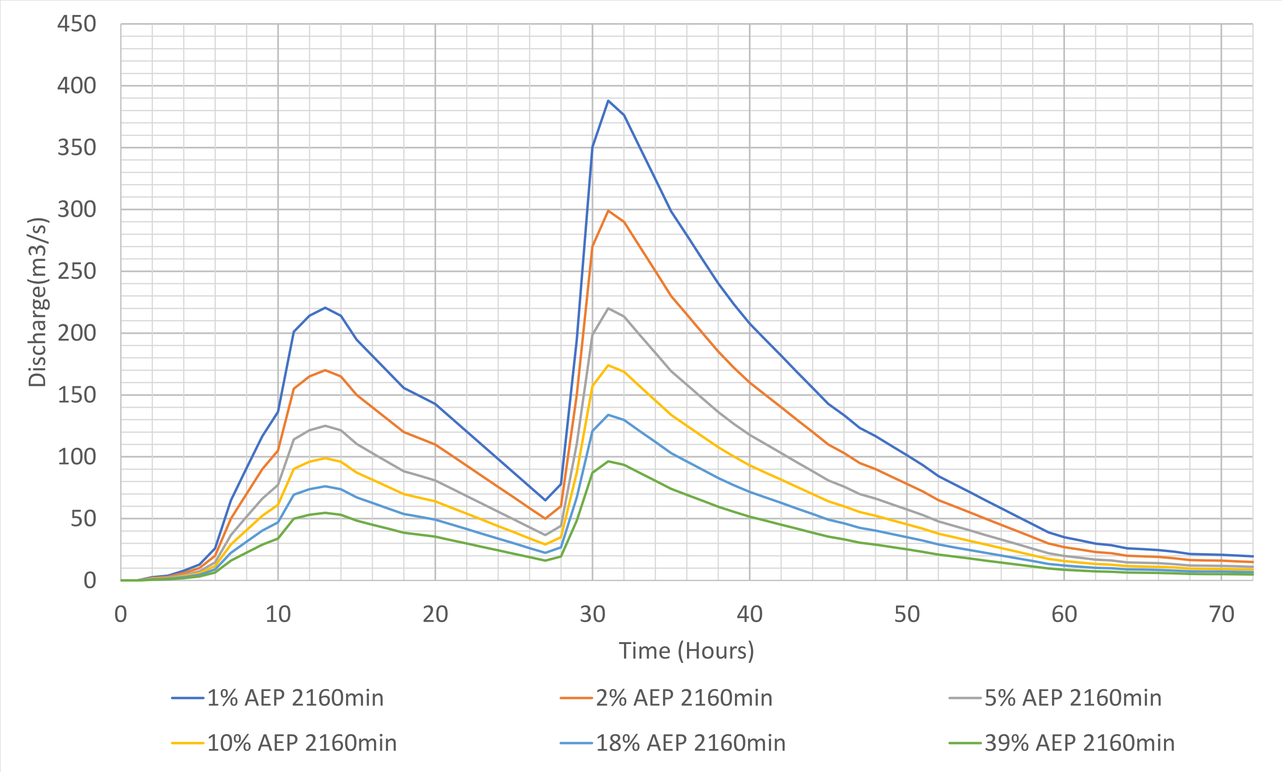


FIGURE 2-2 DERIVED AND ADOPTED HYDROGRAPHS AT OLD NOARLUNGA

2.2 Hydraulic Model Development

2.2.1 Overview

A TUFLOW hydraulic model (build 2018-03-AC) for the lower Onkaparinga River was developed employing the HPC (Heavily Parallelised Compute) solution scheme. TUFLOW is a 1D-2D linked hydraulic modelling software that solves depth-averaged shallow water equations to perform hydraulic simulations. The TUFLOW model developed for this study has been used to simulate the existing site conditions for a reach of the lower Onkaparinga River, the model extent is shown on Figure 2-3. The model boundary was situated a sufficient distance upstream to ensure any inaccuracies associated with boundary effects would not have any impact on the model results at the site. The downstream boundary as set approximately 150m downstream of the river outlet into the St Vincent Gulf. The hydraulic model has been used to estimate peak floodwater depth, velocity, water surface level, and other associated hydraulic outputs required for the geomorphic assessment. This section describes the development of the TUFLOW model.

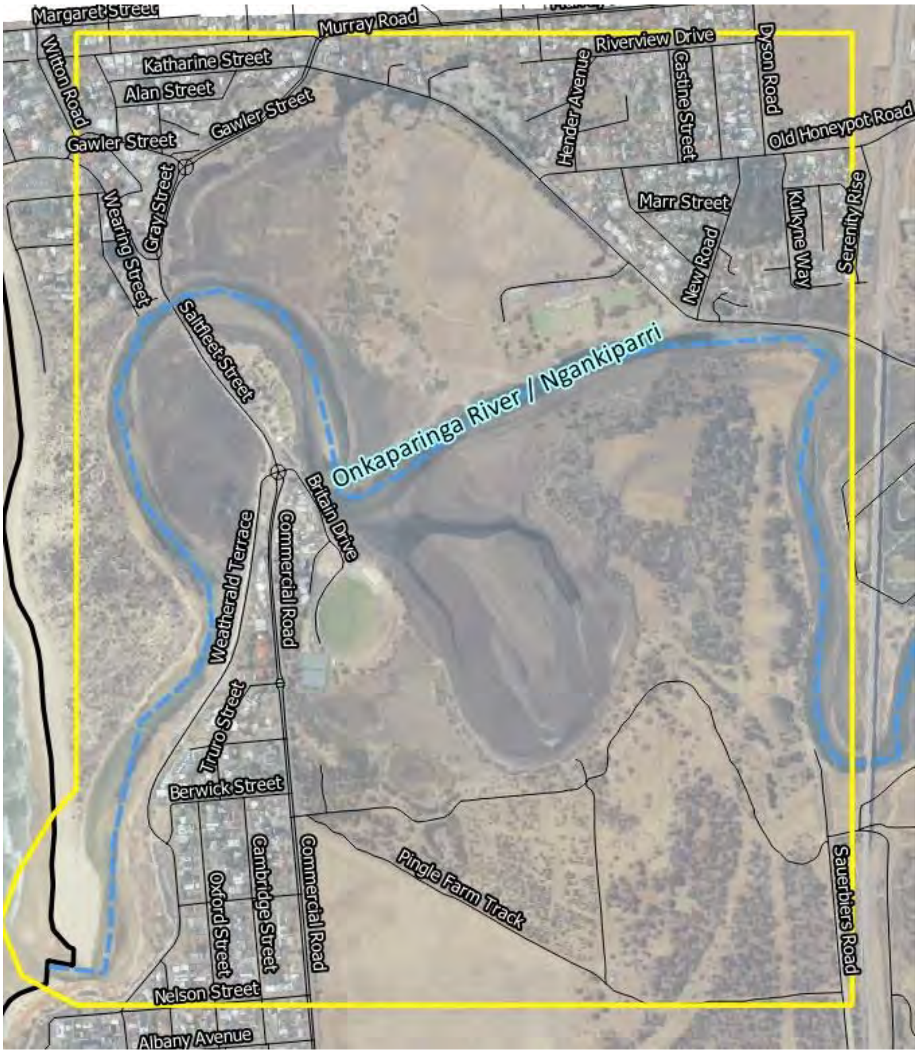


FIGURE 2-3 TUFLOW MODEL EXTENT

2.2.2 TUFLOW Model Topography and Layout

The TUFLOW model topography is based on LiDAR elevation data captured recently in 2018. The LiDAR has been supplemented with bathymetric data from the KRB (2010) study, which was provided by the City of Onkaparinga. While the bathymetry data is somewhat dated, the availability and inclusion of the data in the hydraulic model allows for a more accurate assessment and providing more robust estimates of hydraulic outputs from the modelling. Comparisons of aerial imagery from when the bathymetric data was captured in 2010 and recent aerial imagery from 2020 indicates that the waterway geometry has not significantly changed in this time. If there have been significant changes as a result of erosion or sedimentation, it will affect the hydraulic estimates at the site. This combination of the LiDAR captured in 2018 and bathymetry from 2010 resulted in the most representative available topographic dataset.

The model extent, boundary locations and topography are illustrated in Figure 2-4 with the following comments provided in respect to the selected extent:

- Total TUFLOW model covers a 5.5km reach of the Onkaparinga River and an area of approximately 3.6 square kilometres.
- The model adopted a grid resolution of 5m.

- The downstream boundary was positioned a considerable distance downstream from the site (approximately 1500m) and to specifically include the oceanic tidal conditions at the river outlet.
- The upstream extent was taken a considerable distance upstream from the site (approximately 2000 m) in order to appropriately assess the discharge towards the site.
- The lateral extents were selected to fully contain the flooding in the 1% AEP event.

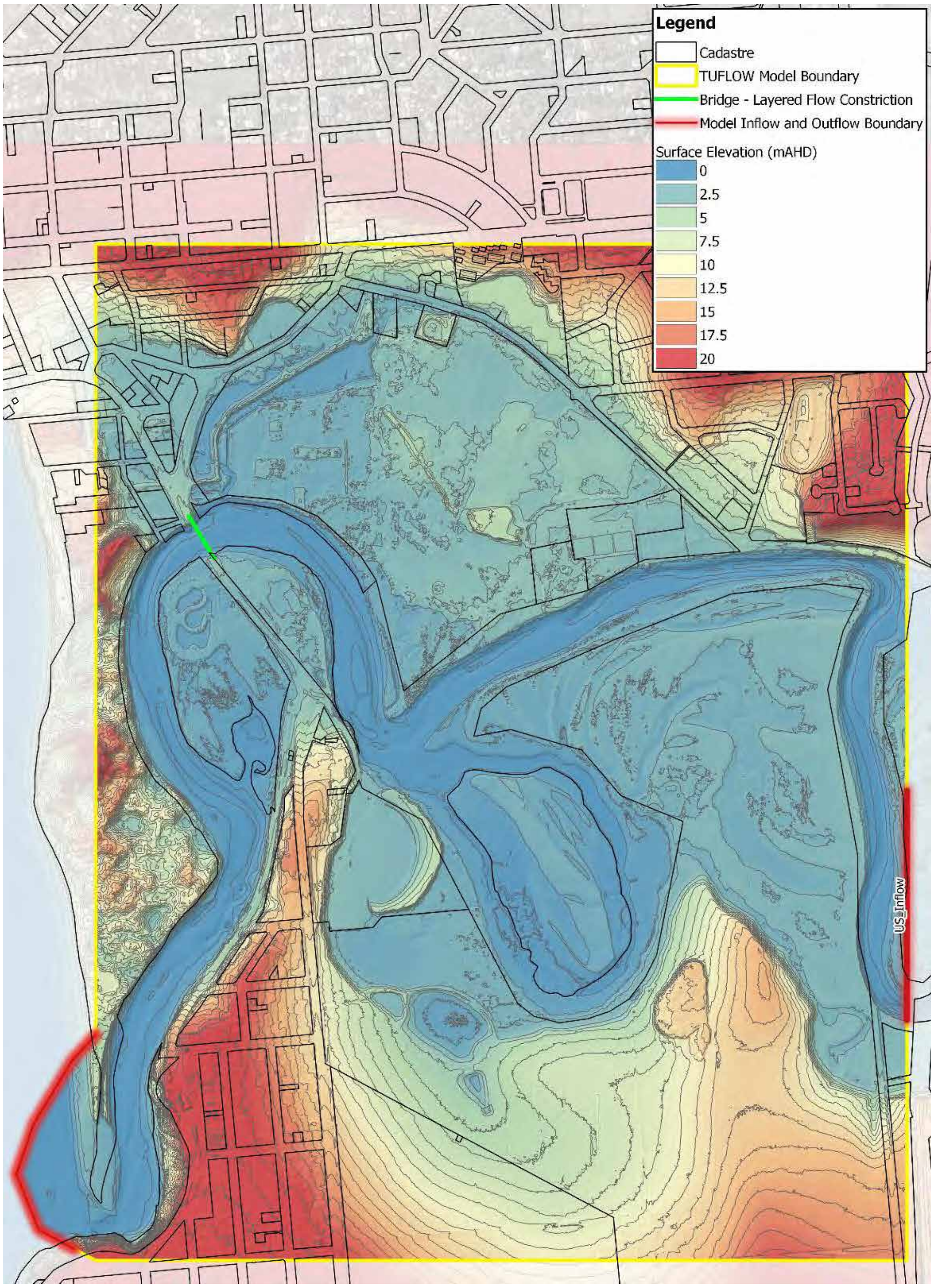


FIGURE 2-4 TUFLOW HYDRAULIC MODEL EXTENT, TOPOGRAPHY, AND KEY FEATURES

2.2.3 TUFLOW Model Structures

The model extent includes the Saltfleet Street bridge, which is immediately upstream of the subject site. This is a significant structure and has been included in the hydraulic model. The bridge was modelled in TUFLOW as a layered flow constriction, which simulates the blockage and head loss based on the bridge geometry such as pier size, pier spacing and deck level. The bridge parameters were estimated from copies of the original design drawings from 1948, a copy of the drawing is shown in Figure 2-5.

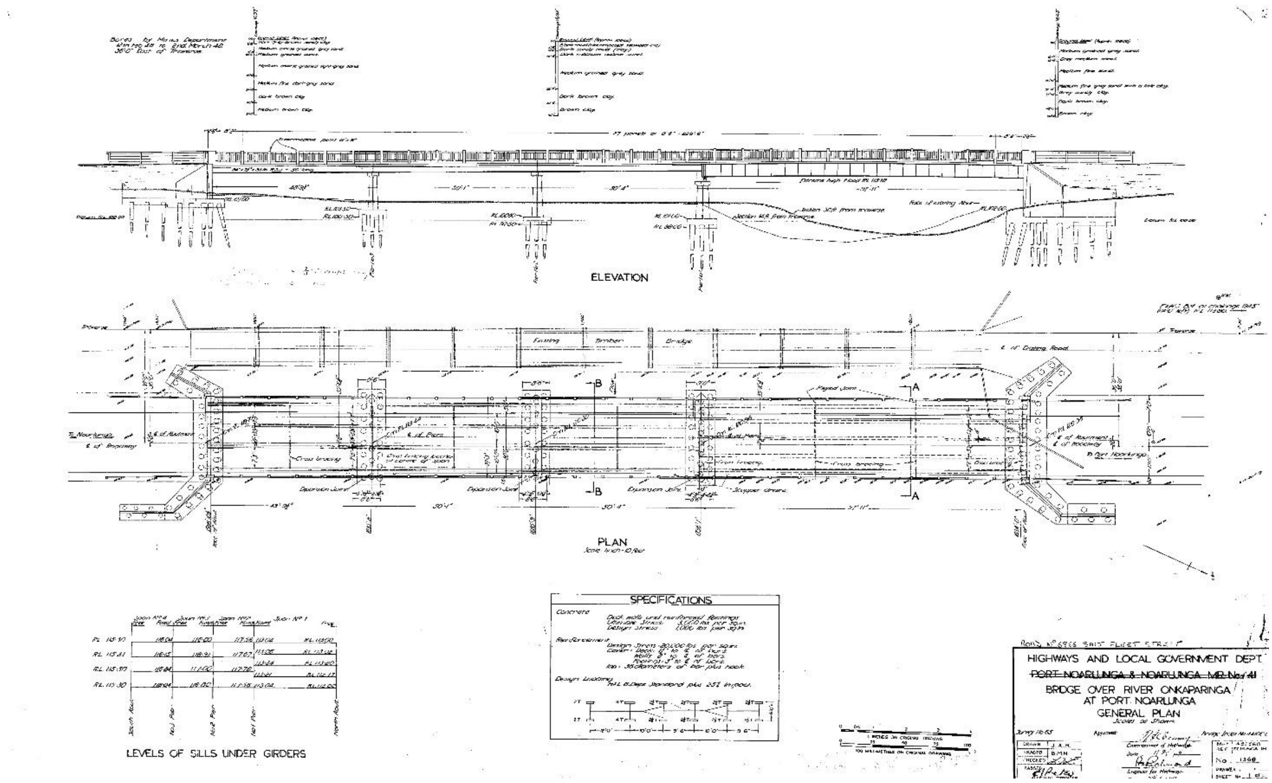


FIGURE 2-5 SALTFEEET STREET BRIDGE SPECIFICATIONS

2.2.4 TUFLOW Model Downstream Boundary Conditions

Hydraulic modelling outcomes, such as design water levels and velocities, can vary based on the adopted tidal boundaries. For this analysis, a range of tidal boundaries were considered and combined to capture the worst case scenario for each of the model outputs. The static tidal levels being considered at this location for this assessment are outlined in Table 2-3. The location of the downstream boundary is shown in Figure 2-4. A review of the results indicated that the tidal boundary only had a minor effect on flood levels and velocities at the site during periods of very high river flow.

TABLE 2-3 TIDAL BOUNDARY CONDITION'S WATER LEVEL

Tidal Boundary Condition	Static Ocean Water Level (mAHD)
Highest Astronomical Tide (HAT)	1.2
Lowest Astronomical Tide (LAT)	-1.3
Mean High Water Springs (MHW)	0.7

2.2.5 TUFLOW Model Hydraulic Roughness

The hydraulic roughness within the model domain was represented using Manning's 'n' values. The adopted land use and associated Manning's 'n' values are shown in Table 2-4. The existing and developed case hydraulic roughness maps are presented in Figure 2-6.

TABLE 2-4 ADOPTED LAND USE TYPES AND ASSOCIATED MANNING'S 'N' ROUGHNESS

Land Use Type	Manning's 'n' Roughness
Road and Verge	0.025
Residential	0.100
Building (Solid)	0.300
Park / Open Space	0.050
Industry / Commercial	0.100
High Density Industrial / Commercial	0.300
Creek / River / Channel	0.020

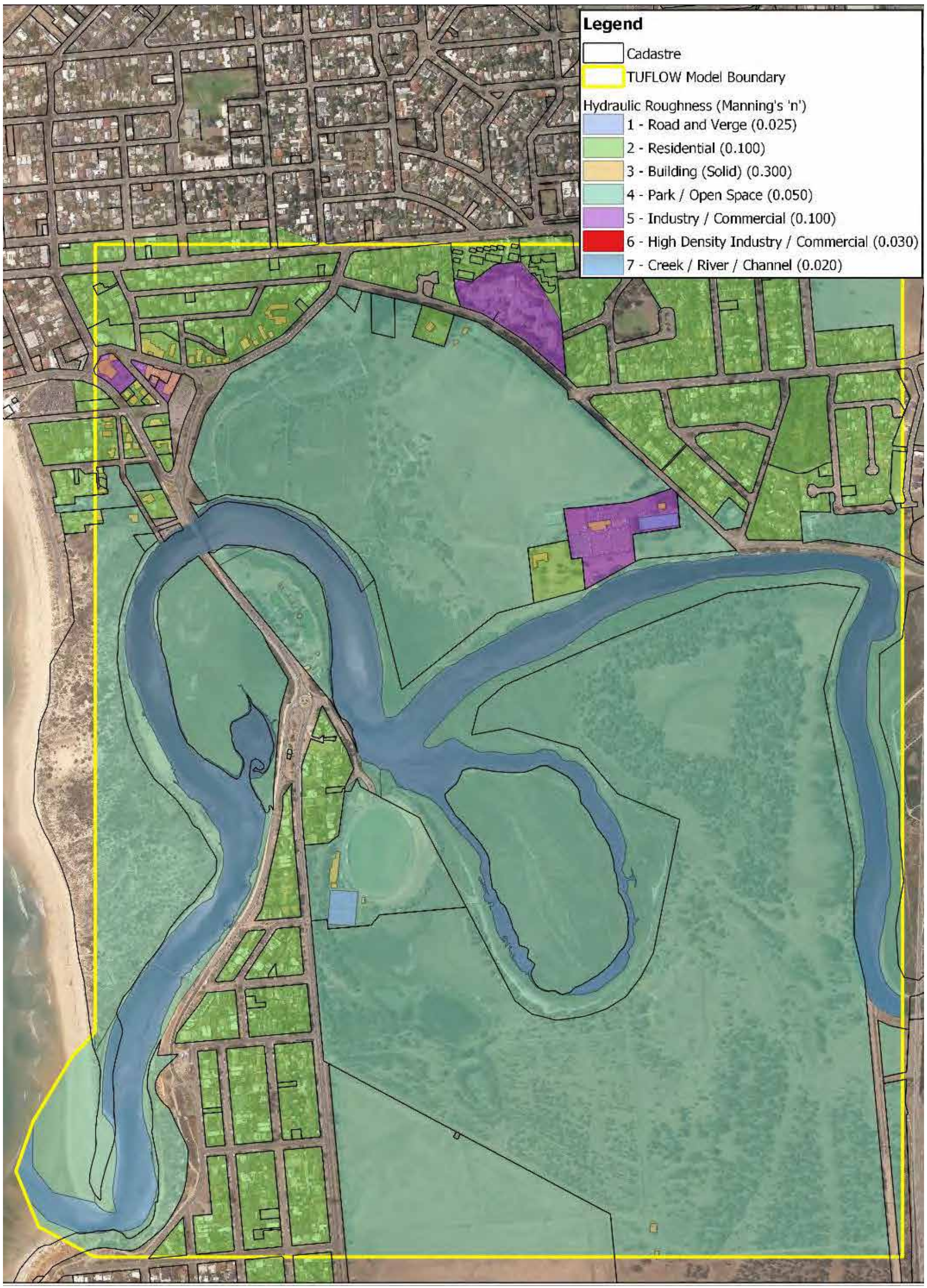


FIGURE 2-6 TUFLOW HYDRAULIC MODEL MANNING'S 'N' ROUGHNESS

2.3 Hydraulic Model Results

2.3.1 Hydraulic Result Maps

The hydraulic model results have been prepared as a series of GIS maps for each of the hydraulic outputs. The flood result for the range of design events assessed are included as a series of maps in the appendices to this report. The maps are arranged in the appendices as follows:

- Appendix A – Maximum Flood Depth.
- Appendix B – Maximum Water Surface Level.
- Appendix C – Maximum Flood Velocity.

- Appendix D – Maximum Stream Power.
- Appendix E – Maximum Bed Shear Stress.

2.3.2 Hydraulic Result Discussion

2.3.2.1 General Description of Flood Behaviour

The Onkaparinga River is a large river catchment of over 500 square kilometres which rises on the slopes of the Mounty Lofty Ranges, descending around 420m over its 88 km course. It is a complex river system with two large storages in the upper catchment at the Mount Bold Reservoir and Clarendon Weir. Much of the non-flood flows at Clarendon Weir are diverted away from the river system to the Happy Valley Reservoir.

Through the lower reaches of the watercourse, the Onkaparinga River changes from a confined waterway with high hills on either side to a wider floodplain and estuary. Within the reach of the Onkaparinga River modelled as part of this study, the waterway generally contains all flood events up to the 20% AEP design flood, with some flooding of wetlands and undeveloped areas. In the 10% AEP design flood, flood waters break out of the river system and onto the floodplain. This causes minor inundation across Saltfleet Street near the Port Noarlunga Playground and fields, to the south of the Saltfleet Street bridge. At the subject site, the 10% AEP design flood begins to inundate the existing carpark. In the larger events, some local roads are inundated with flooding at the site extends further north, surrounding Sauerbier House.

2.3.2.2 Comparison to KBR Study

As this study has adopted the same discharge as the KBR study, the water levels should not vary significantly. The water level simulated at the Saltfleet Bridge was compared to levels documented in the KBR (2010) report and are shown on Table 2-5. The levels for this study have been extracted from the model which adopted a MHWS downstream boundary to match the KBR study. It is unclear whether KBR had extracted the water level upstream or downstream of the bridge, so both have been extracted and are shown in Table 2-5. As shown on the table, the water levels are very similar. Figure 2-7 shows the 1% AEP extent compared to the previous model, which also shows a good correlation between the results. The minor differences in flood level and extent will be due to several factors including different hydraulic modelling packages, base topography, and roughness. The comparison indicates the model is producing reasonable results, which match previous studies.

TABLE 2-5 WATER LEVEL AT SALTFLEET STREET BRIDGE – MHWS DOWNSTREAM BOUNDARY

Design Event	KBR (2010)	Water Technology Upstream Bridge (2020)	Water Technology Downstream Bridge (2020)
5%	2.08	2.16	2.12
2%	2.43	2.52	2.48
1%	2.69	2.86	2.82

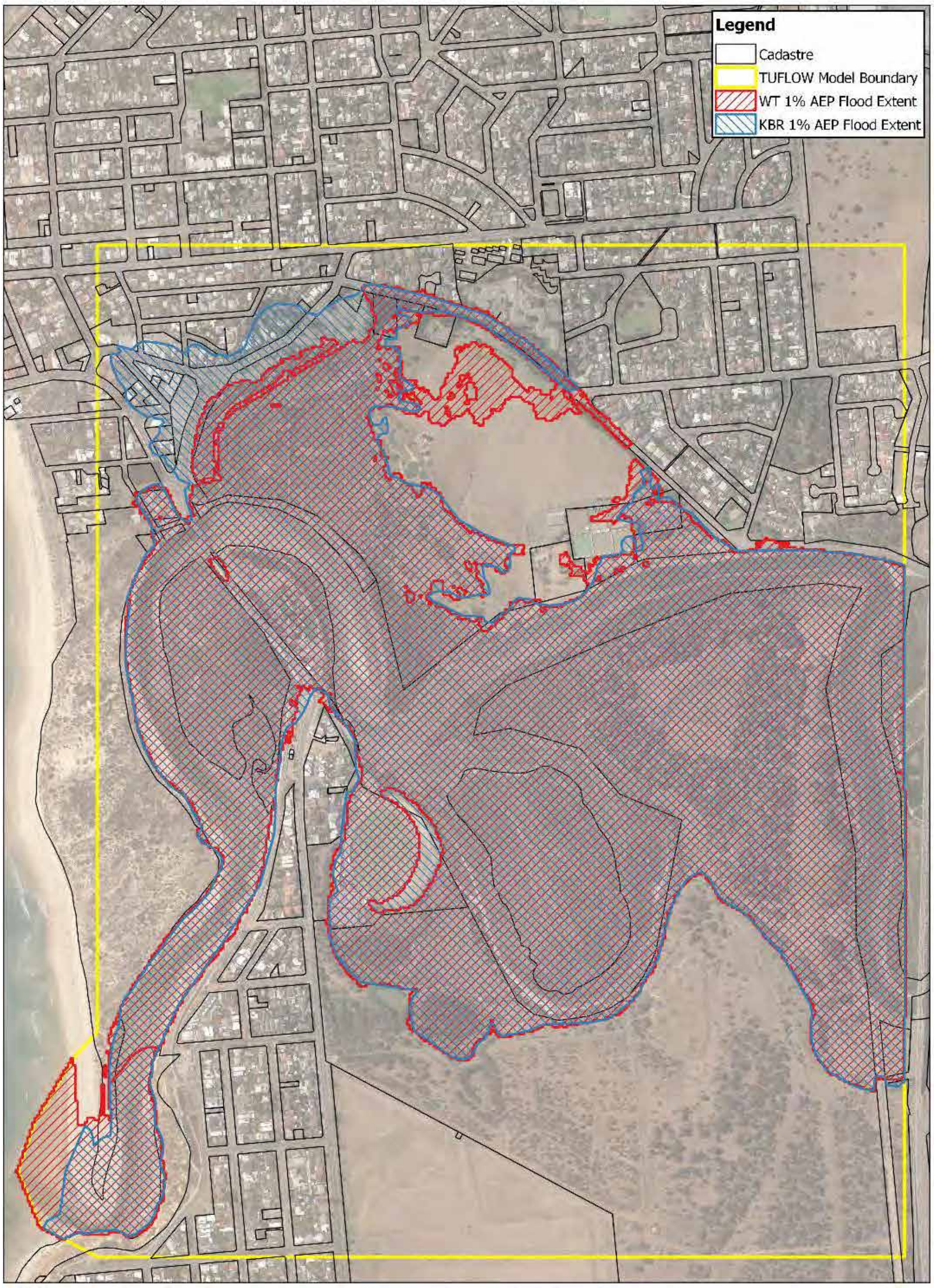


FIGURE 2-7 1% AEP FLOOD EXTENT COMPARISON TO KBR 2010 STUDY

2.3.2.3 Tidal Boundary Impact

The KBR (2010) report noted that an earlier study undertaken by CMPS&F (Egis Consulting Australia) indicated the tide had a minimal effect on flood depths upstream of Saltfleet Street Bridge. A range of tidal boundaries have been adopted for this study from LAT to HAT. A review of the results indicates that in the more frequent events, the 39%, 18%, 10% AEP design floods, the tidal effects reach beyond the Saltfleet Street Bridge and extend to the model inflow boundary. For the rarer events, the 5%, 2% and 1% AEP design floods, the tidal effects are negligible by Saltfleet Street bridge.

3 GEOMORPHIC ASSESSMENT

3.1 Current Condition and Erosion Processes

Within the vicinity of the study area, the Onkaparinga River is a wide and shallow, tidally influence system with a medium-grained sandy bed. Dense and continuous riparian vegetation is essentially absent once the river leaves the National Park. As stated above, this section of the river is laterally migrating meaning that the channel is moving across the floodplain. This occurs though meander extension where erosion occurs on the outside bank of meander bends due to increased hydraulic energy in that location and localised effects such as helical flow. Meander extension is evident on most meander bends, including at the study area, which is located on the apex of such a bend. Upstream of the site rock armouring has been placed to halt this process due to the proximity of Britain Drive and, again, further upstream on both meander bends that come close to River Road (Figure 3-1).

FIGURE 3-1 ROCK ARMOURING AND GROIN PREVENTING EROSION OF OUTSIDE BEND NEAR RIVER ROAD.

At the study site erosion is actively occurring, which is made evident by the undercutting and failure of previous rock and concrete erosion mitigation measures (Figure 3-2). This is also evident immediately adjacent to the study site upstream of the Saltfleet Street Bridge with failed rock armouring. At all locations, this erosion is occurring through fluvial scour as water flows against the outer bank of the bend. Flow is concentrated against the outer bank of the bend resulting in higher velocities and shear stresses (energy required to lift and entrain particles in flow). As such, where meanders are located in laterally unconfined areas (areas where movement of the river is not limited by bedrock) they are prone to erosion. This is a natural phenomenon, however, is often exacerbated by changes to land use and increases in discharge, or removal of vegetation.

FIGURE 3-2 UNDERMINED AND FAILING ROCK ARMOURING AT STUDY SITE.

In this area erosion is occurring over the full length of the meander bend. It is gradual but observations certainly suggest that flows are capable of moving sand, particularly when no depositional features were observed. The hydraulic modelling results show velocities along the outer mender bend within the vicinity of the site ranging between 0.6 m/s for 50% AEP flows and 1 m/s for the 1% AEP flow. Velocities in the middle of the channel range between 0.8 m/s and 1.5 m/s. These velocities are not considered high; however, the resulting modelled bed shear stresses of between 6 and 10 N/m^2 are sufficient to move sandy particles. These results support the observations of active erosion and undermining of the previously installed erosion mitigation works.

Given the observed erosion and the hydraulic modelling results, erosion can be expected to continue along this site in the short medium and long term. As such, it is considered unlikely that turning the proposed area into an unprotected beach like area is sustainable without ongoing maintenance (sand nourishment). Rock armouring in this location is considered necessary to halt erosion and reduce the maintenance costs. Bed shear stress is a function of depth and water surface slope. Given that the water surface slope in this location is very low, depth is the critical factor at the study site. It is therefore likely that terracing may be an achievable sustainable method of incorporating boat launch facilities and “beach like” aesthetics in this location.

3.2 Impacts Related to Proposed Concept Design

The currently proposed concept design (Figure 1-2) will leave the beach face and access paths susceptible to erosion. Under base flow and tidal conditions, the beach face is likely to require regular nourishment to maintain its level to which the lower landing of the concrete path is aligned. However, during large events it is likely that substantial erosion will occur along the proposed rock wall. So long as the rock wall is keyed well into the bed this is unlikely to cause damage to the rock wall itself. Likewise, as long as the lower landing of the concrete access path is also keyed in, undermining of the path is unlikely; however, erosion may periodically leave a large step from the landing to the “beach” surface.

3.3 Options for rock Armouring Arrangement

Three options are proposed for erosion mitigation at this location:

- Implement the currently proposed layout, ensuring the rock and concrete landing are keyed into the ground surface to an appropriate depth, and establish on ongoing beach nourishment and maintenance plan.
- A continuous rock wall along the affected area.
- Smaller stepped rock walls in a terraced arrangement.

These options are discussed in the following sections.

3.3.1 Option 1: Implement Currently Proposed Layout

This option has been developed through Council’s community consultation processes and is Council’s presently preferred option.

As discussed, this option is likely to be subject to ongoing and event-based erosion. This will likely necessitate, ongoing maintenance, requiring:

- Beach nourishment as sand is removed from the beach face.
- Maintenance to the paths.
- Event based maintenance to the rock walls.
- Periodic maintenance to the concrete ramps.

It is important that rock armouring be designed by an appropriately qualified engineer. RipRap and hydraulic modelling results should be used to determine the rock size and slope for the rock walls. Ideally, concrete should not be used with the RipRap. As the rock settles it is likely that some damage to the concrete will be sustained (cracking etc). Finally, all rock armouring and the lower landing of the ramps should be keyed into the toe of the bank by 4 x the d_{50} diameter (prescribed by RipRap) (Figure 3-3).

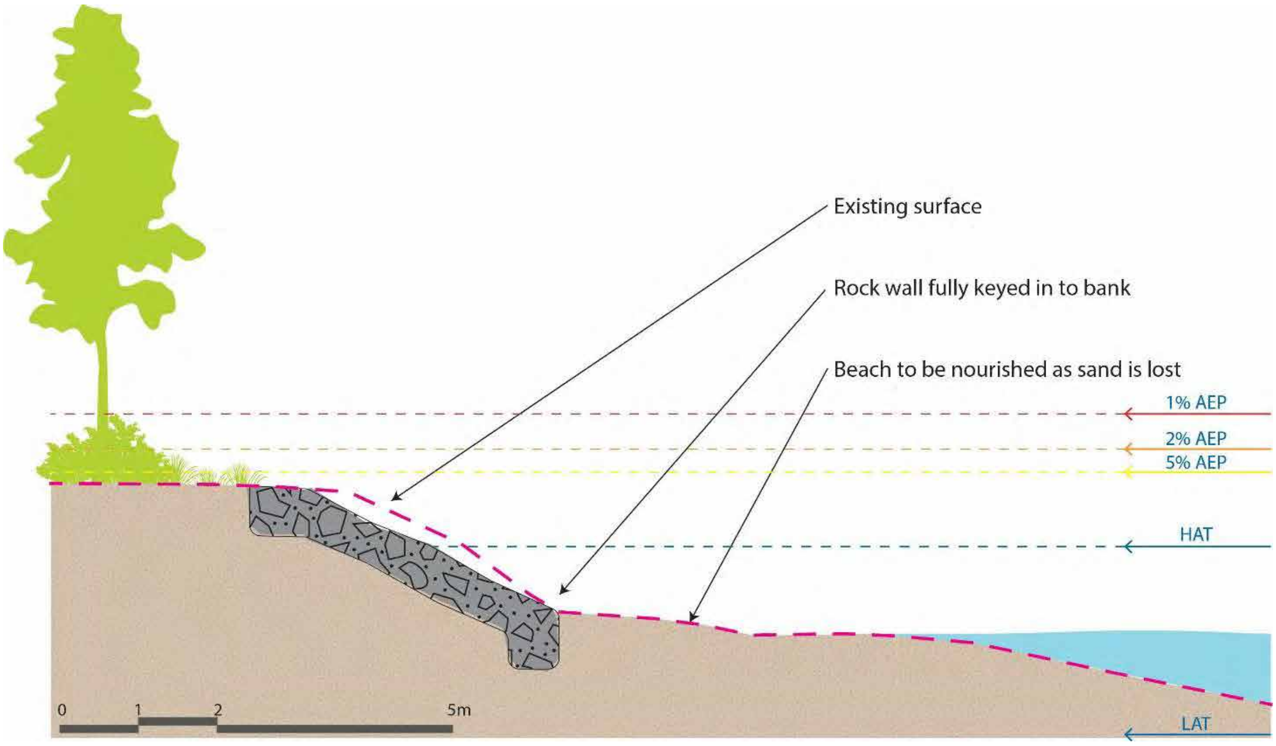


FIGURE 3-3 FEATURES TO SUPPORT THE CURRENT CONCEPT PLAN CROSS-SECTION.

3.3.2 Option 2: Continuous rock wall along the affected area.

This option is the most durable and low maintenance option for erosion protection however offers little aesthetic appeal.

Access to launch boat craft could be achieved by adding a pontoon or using steel or timber stairs / ramps and platforms as described above. This option is shown in Figure 3-4 and Figure 3-5.

It is understood that this option is the least preferred option by the community or Council.

The requirement for a pontoon also would make the logistics for launching craft complicated, particularly given the very heavy usage of the site during the busy summer periods. Hence one of primary objectives for future use of the site would not be met (it would however stabilise the site and minimise ongoing maintenance requirements, and hence was initially included in the assessment process).

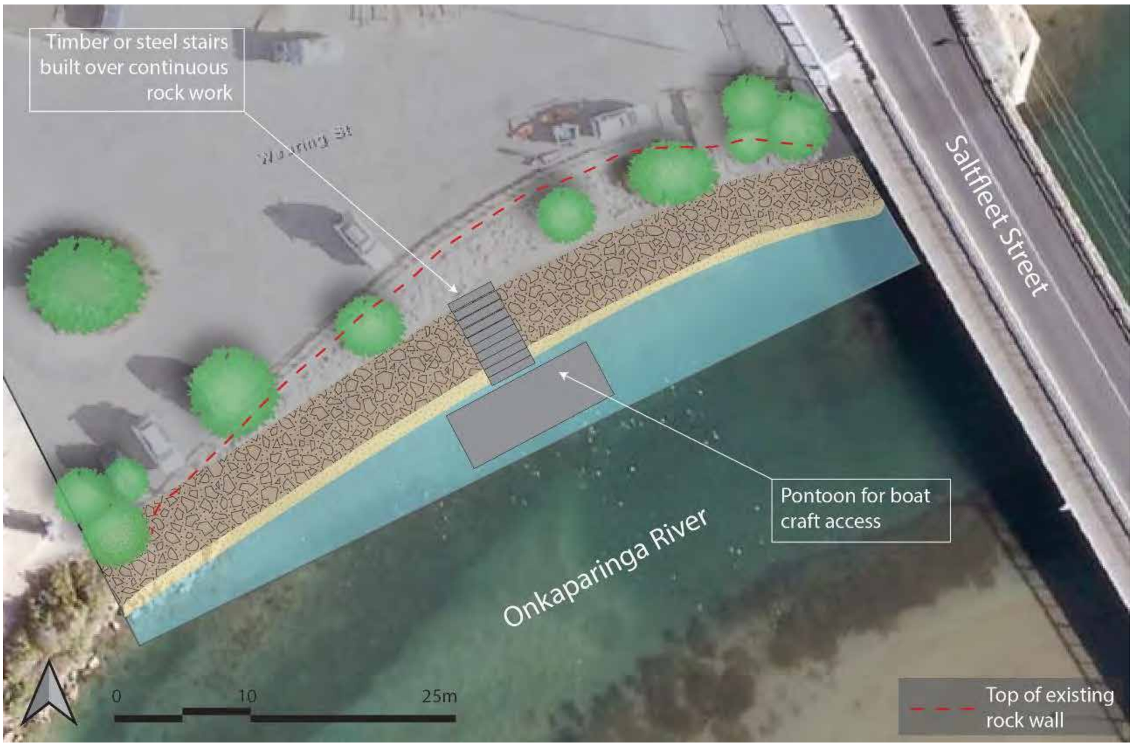


FIGURE 3-4 CONTINUOUS ROCK WALL ALONG THE AFFECTED AREA.

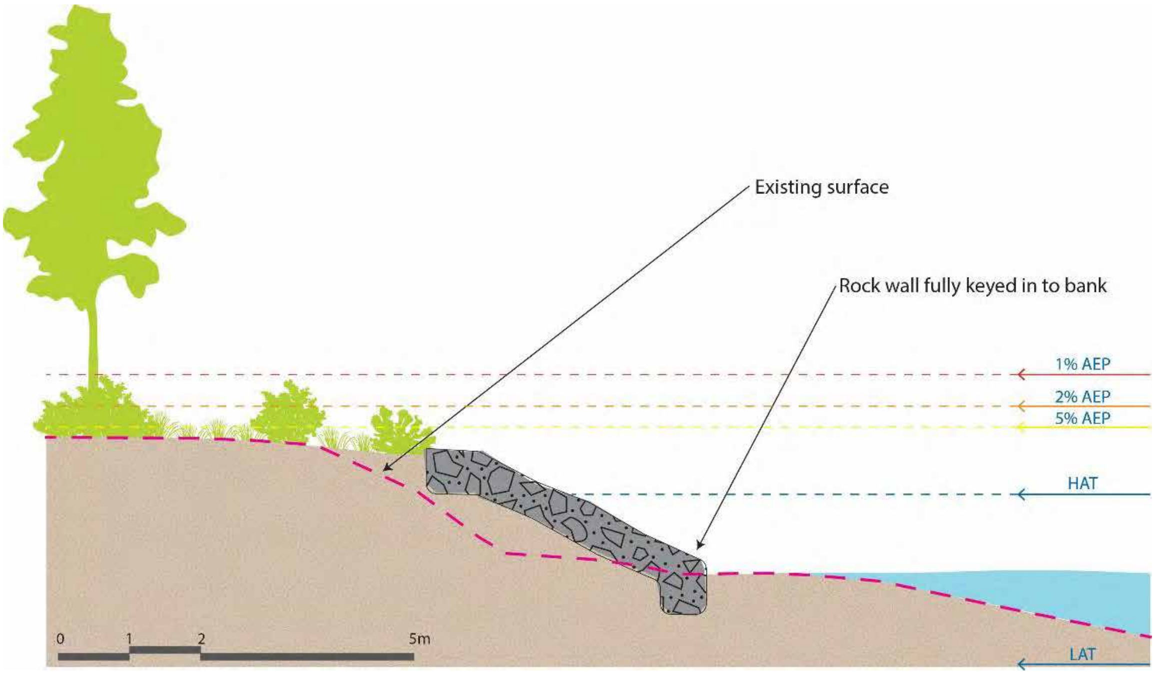


FIGURE 3-5 CONTINUOUS ROCK WALL ALONG THE AFFECTED AREA CROSS SECTION.

3.3.3 Option 3: Smaller stepped rock walls in a terraced arrangement.

A terraced arrangement may be a balance between the two previous options with stepped beach flats between low lying rock walls (Figure 3-6 and Figure 3-7). If a terraced beach front is appealing to the community over a high rock wall, the following the following recommendations are made regarding the design:

- Each terraced wall behind the front wall should be keyed into at least 0.5 metres below the wall in front.
- Terraced surfaces should have a slight slope equal to approximately 1(h):10(v).
- Public safety should be considered with regards to the height of each terrace wall. This may result in a requirement for stairs to be added over the rock. These should be steel or timber, but not concrete.
- If fill is required behind each rock wall, the fill should be track compacted.
- It is highly likely that some sand will be removed from these surfaces during large events. This will necessitate nourishment efforts from to maintain a beach like aesthetic. This is expected to occur with events larger than 5% AEP.

FIGURE 3-6 SMALLER STEPPED ROCK WALLS IN A TERRACED ARRANGEMENT.

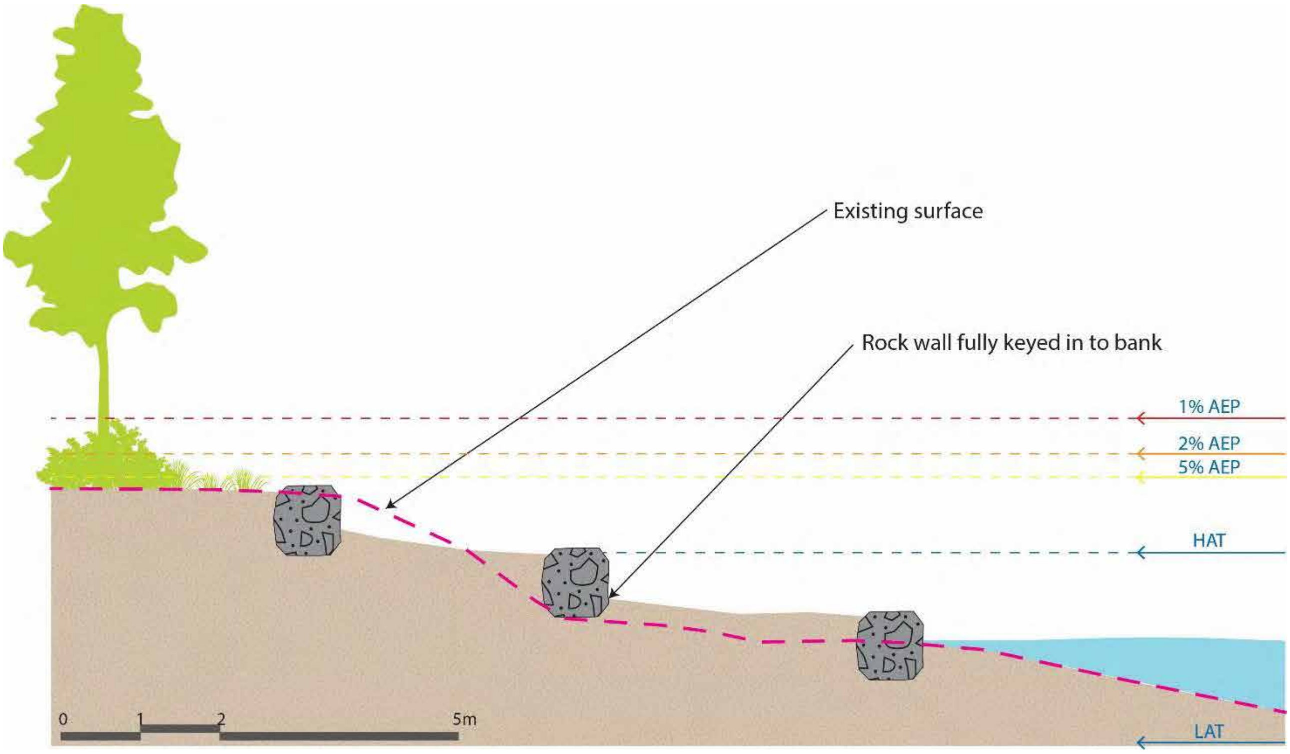


FIGURE 3-7 SMALLER STEPPED ROCK WALLS IN A TERRACED ARRANGEMENT CROSS-SECTION.

3.3.4 Maintenance Associated with Proposed Options

Maintenance requirements for each option will vary. Sand nourishment will require access for a ruck to the top of bank and access for a small (rubber-footed) earth moving machine to the beach face (which is possible using steel or timber ramps). Identified maintenance requirements for each option are listed below:

- **Option 1: Implement currently preferred layout.**
  - Event based monitoring of rock wall and beach face.
  - Frequent beach nourishment as sand is removed from the beach face.
  - Event based removal of slack water silt deposits (event dependant).
  - Medium to long- term maintenance associated concrete ramps.
- **Option 2: Continuous rock wall along the affected area.**
  - Event based monitoring of rock wall.
  - Long- term infrequent maintenance associated with steel or timber ramps or stairs.
- **Option 3: Smaller stepped rock walls in a terraced arrangement.**
  - Event based monitoring of rock wall and beach face.
  - Event based beach nourishment as sand is removed from the beach face.
  - Event based removal of slack water silt deposits (event dependant).
  - Long- term infrequent maintenance associated with steel or timber ramps or stairs.

3.4 Recommendation

If ongoing sand nourishment is not an issue, then the proposed concept plan arrangement can be achieved. Under this scenario it is likely that the concrete ramps will sustain damage associated with minor movement of the rock as it settles. However, if nourishment is cost prohibitive and a “beach like” aesthetic is considered essential, the recommended option is a terraced arrangement as outlined above.

In any of the arrangements it is important that rock armouring be designed by an appropriately qualified engineer. RipRap and hydraulic modelling results should be used to determine the rock size and slope for the rock walls. Concrete should not be used with the RipRap. Additionally, all rock armouring should be keyed into the toe of the bank by 4 x the d_{50} diameter (prescribed by RipRap).

If the proposed concept plan arrangement is maintained, it is likely that erosion of the beach sand erosion may periodically leave a large step from the landing to the “beach” surface. To avoid this scenario the lower landing could be placed at a lower invert, with sand nourishment to a level above and over the landing. As such periodic erosion will simply expose the landing rather than produce a step.

4 SUMMARY

The City of Onkaparinga is planning upgrades to the Wearing Street Aquatic and Arts Precinct, located at Port Noarlunga. The project is currently in the conceptual planning stage and includes works on the frontage on Onkaparinga River, adjacent to Sauerbier House Arts and Cultural Centre. To assist with the development of the concept plan, Water Technology was invited by the City of Onkaparinga to prepare a hydrologic and hydraulic assessment of the waterway and prepare a geomorphic assessment of the site.

A review of hydrologic information available for the Onkaparinga River was undertaken. Design peak discharges and hydrographs were estimated from studies of the catchment and river system by KBR and DTEI. A new detailed TUFLOW model was developed of the lower Onkaparinga River which was used to estimate a range of hydraulic parameters at the site. The results of the hydraulic assessment can be used as guidance regarding flood immunity and flood hazard for the proposed concept design. The hydraulic results were used to inform the geomorphic assessment.

The study area is prone to ongoing erosion and rock armouring is considered necessary to halt lateral migration of the channel. This can be achieved through placement of a continuous rock wall along the affected area or by adding several smaller rock walls in a terraced arrangement to improve aesthetics and usability. However, ramps or any steeply sloped and unprotected surfaces are likely to erode over time, as such, access to the lower intertidal section or terrace surfaces would be better achieved using steel or timber stairs rather than concrete stairs which will be susceptible to undercutting and damage.

Given the observed erosion and the hydraulic modelling results, erosion can be expected to continue along this site in the short, medium, and long term. It is considered unlikely that turning the proposed area into an unprotected beach like area is sustainable without maintenance.

If nourishment is cost prohibitive and a “beach like” aesthetic is considered essential, the recommended option is a terraced arrangement as outlined above. However, if ongoing sand nourishment is not an issue, then the proposed concept plan arrangement can be achieved. Under this scenario it is likely that the concrete ramps will sustain damage associated with minor movement of the rock as it settles. As such maintenance of the ramps will also be required.

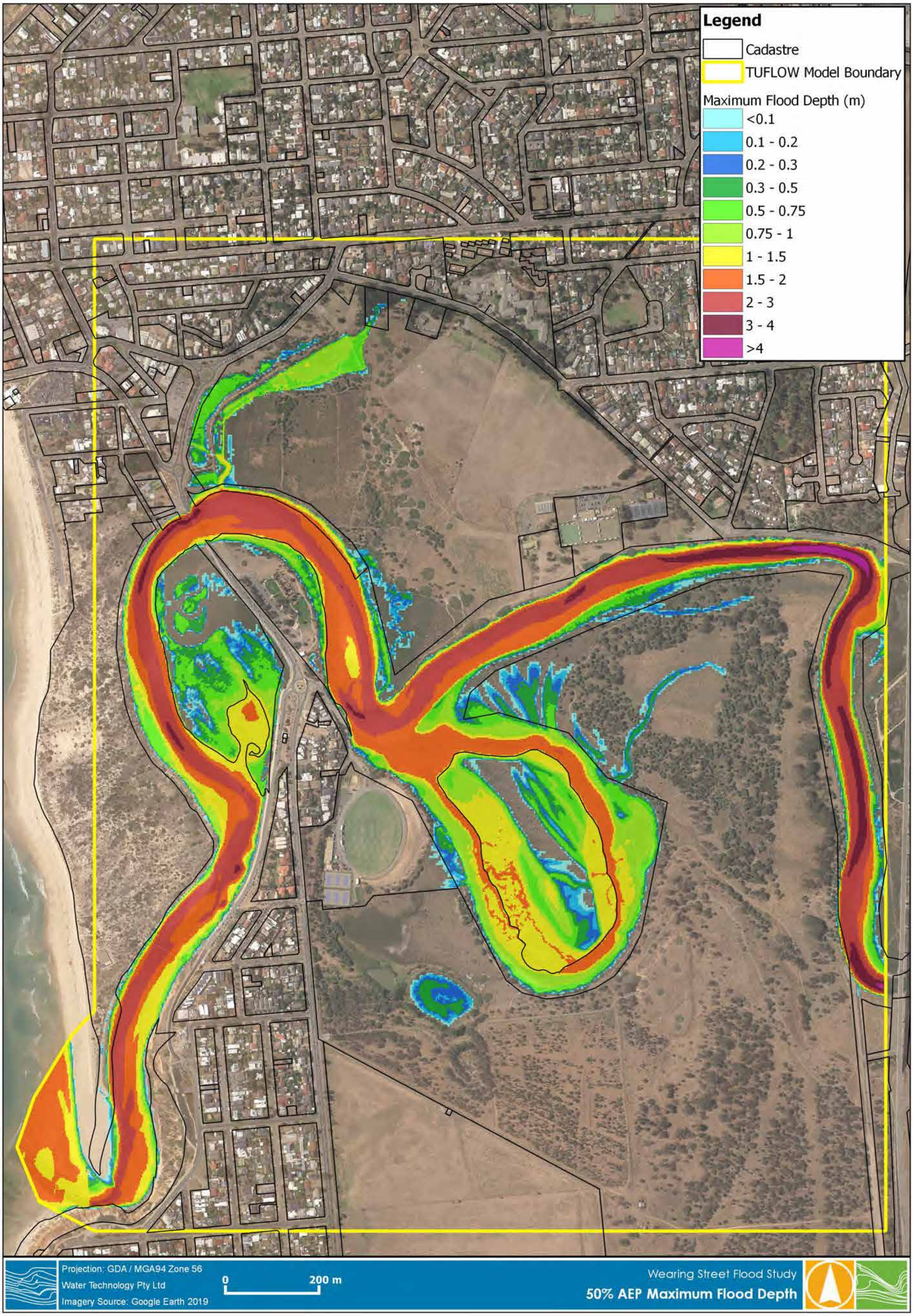
5 REFERENCES

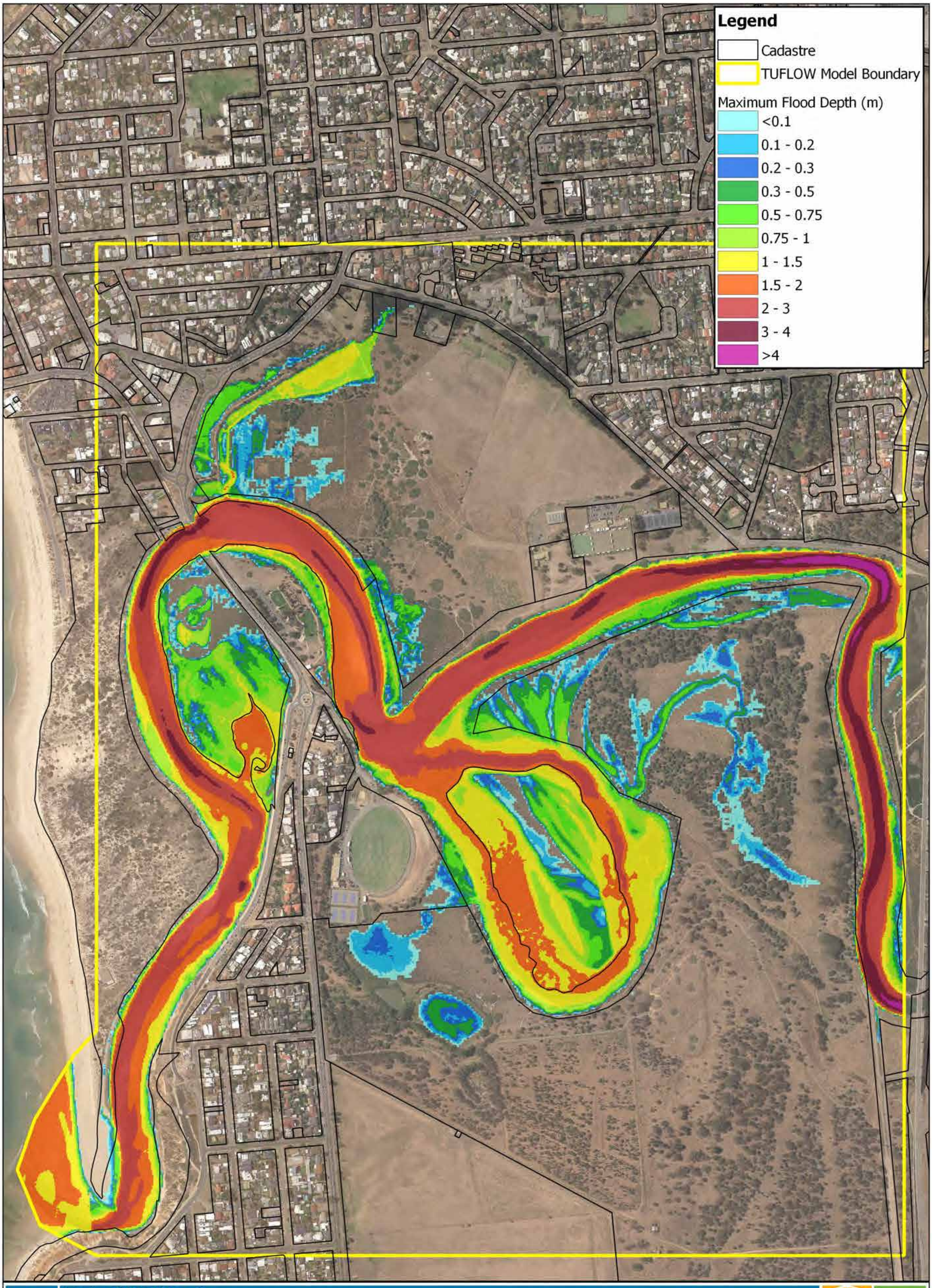
DTEI (2010), Onkaparinga River Downstream of Mount Bold – Hydrology, prepared for the Department of Transport, Energy and Infrastructure, 2010.

KBR (2010), Flood Study for Seaford Rail Extension – Study Report, prepared for the Department for Transport, Energy & Infrastructure, 17 December 2010.

Kemp (2010), Coincidence of Storm Surge and Peak Flow, prepared for the Department of Transport, Energy and Infrastructure, Dr David Kemp, DTEI October 2010.

APPENDIX A
FLOOD DEPTH MAPS

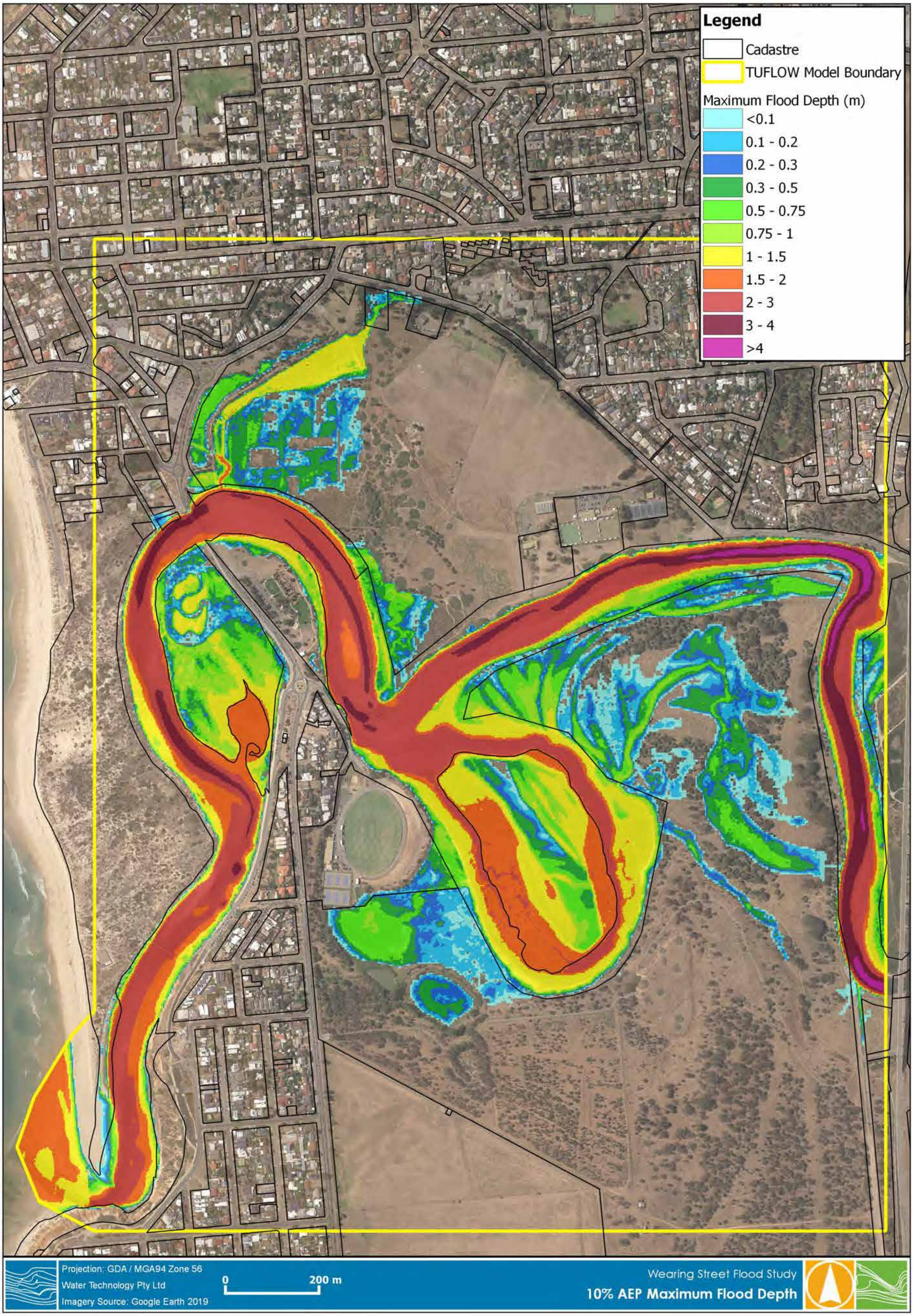

Legend
Cadastre
TUFLOW Model Boundary
Maximum Flood Depth (m)
<0.1
0.1 - 0.2
0.2 - 0.3
0.3 - 0.5
0.5 - 0.75
0.75 - 1
1 - 1.5
1.5 - 2
2 - 3
3 - 4
>4
Projection: GDA / MGA94 Zone 56
Water Technology Pty Ltd
Imagery Source: Google Earth 2019
0
200 m
Wearing Street Flood Study
50% AEP Maximum Flood Depth

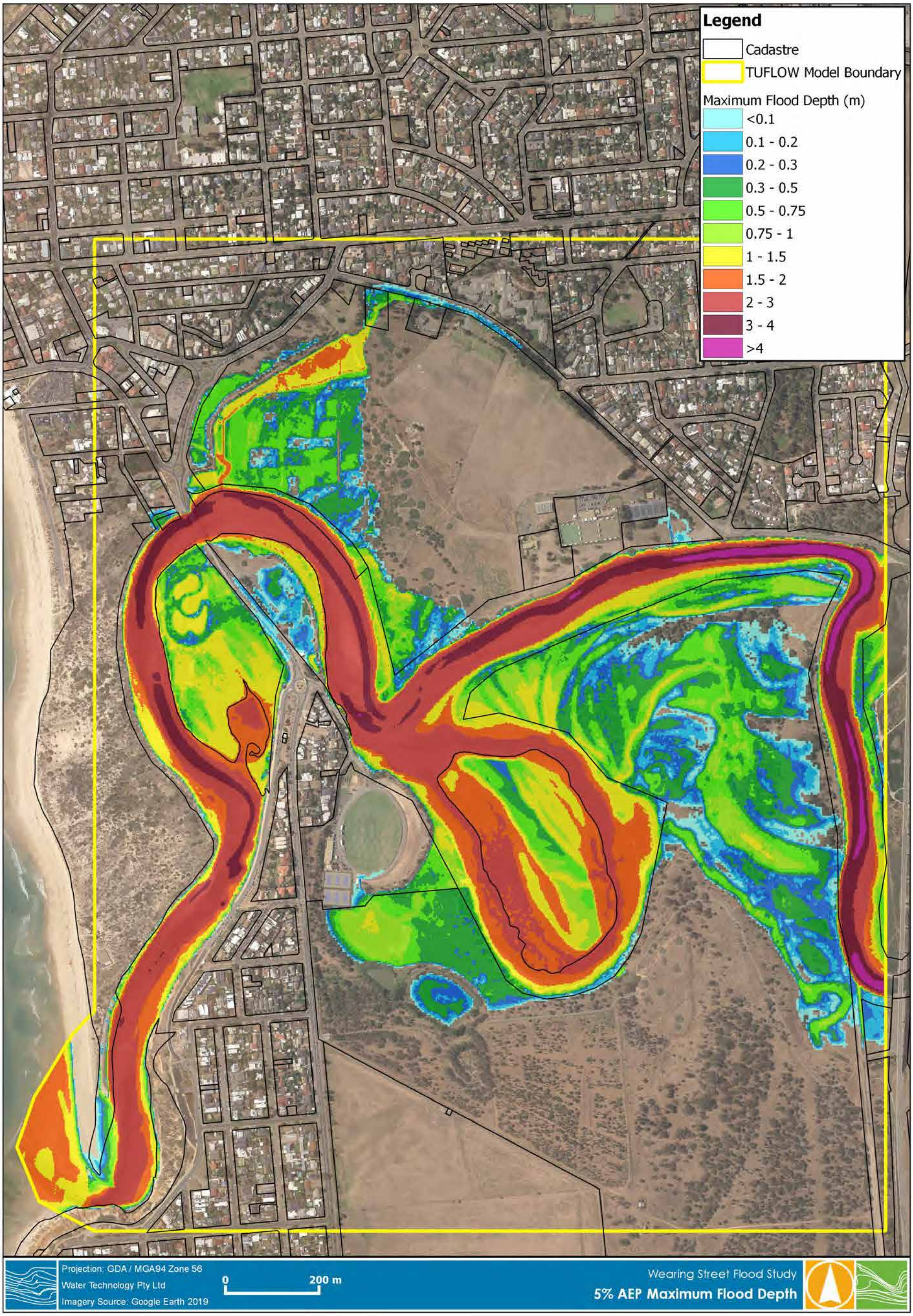


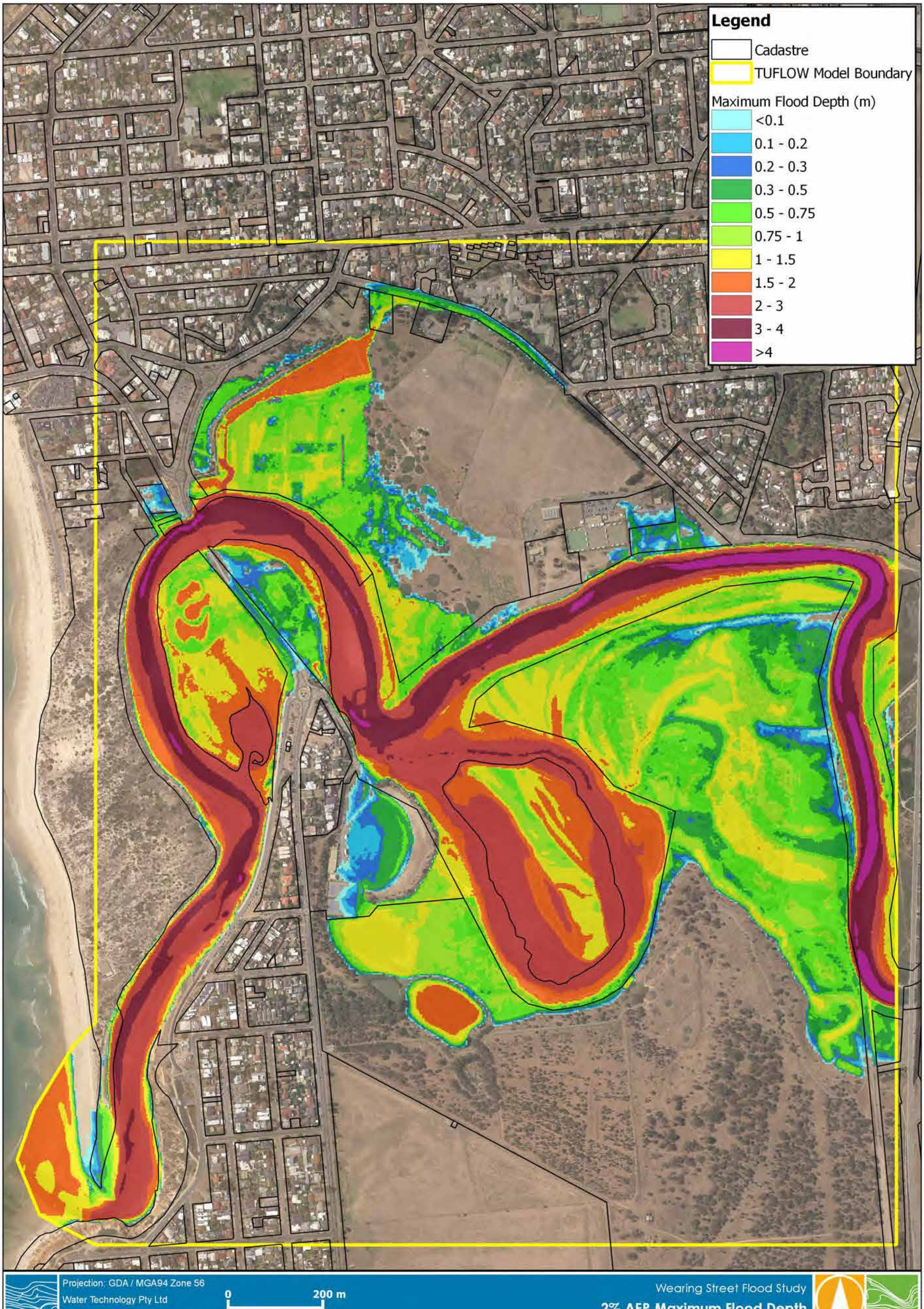
Projection: GDA / MGA94 Zone 56
Water Technology Pty Ltd
Imagery Source: Google Earth 2019

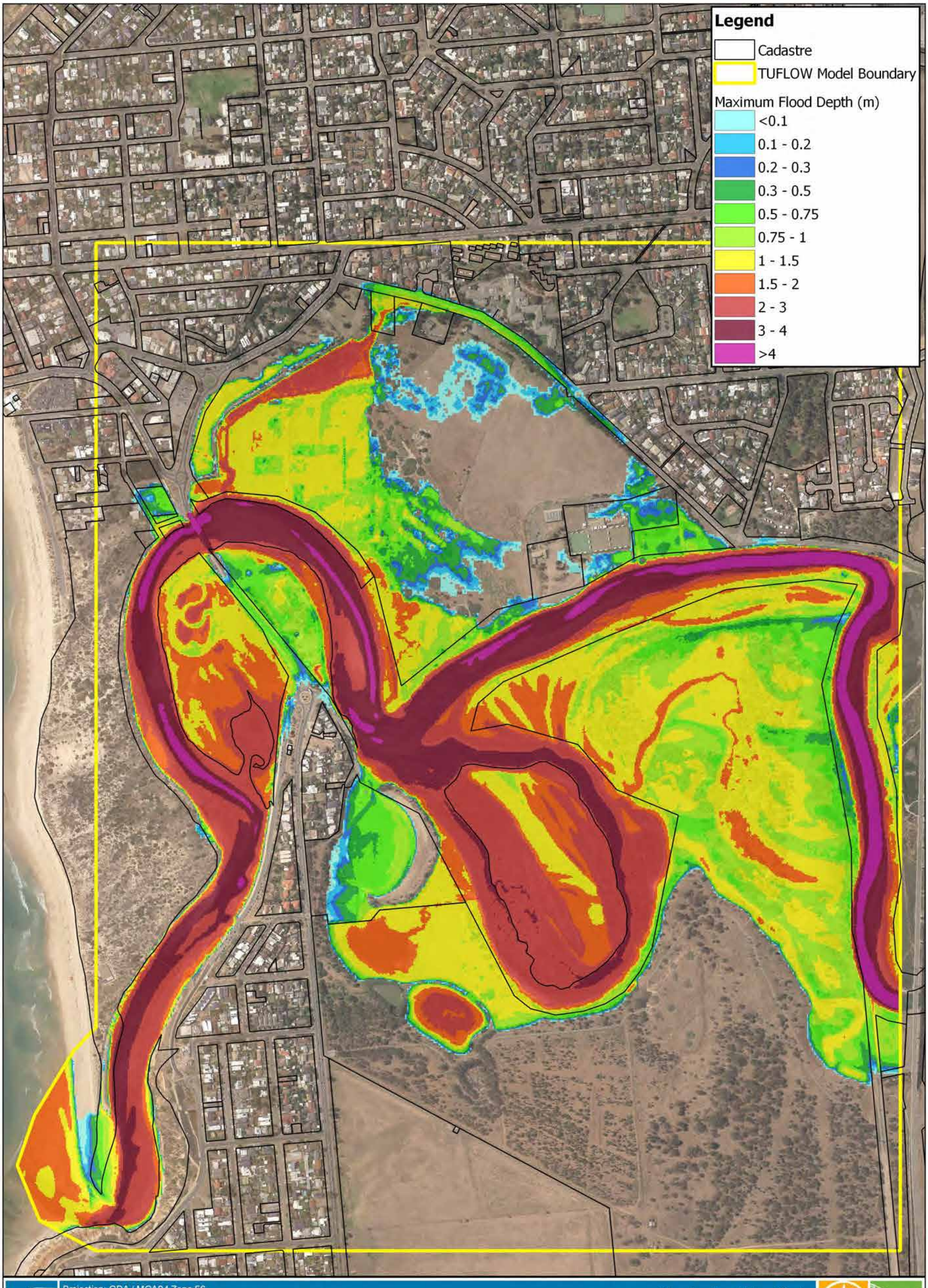
Wearing Street Flood Study

20% AEP Maximum Flood Depth


Legend
Cadastre
TUFLOW Model Boundary
Maximum Flood Depth (m)
<0.1
0.1 - 0.2
0.2 - 0.3
0.3 - 0.5
0.5 - 0.75
0.75 - 1
1 - 1.5
1.5 - 2
2 - 3
3 - 4
>4
Projection: GDA / MGA94 Zone 56
Water Technology Pty Ltd
Imagery Source: Google Earth 2019
0
200 m
Wearing Street Flood Study
10% AEP Maximum Flood Depth


Legend
Cadastre
TUFLOW Model Boundary
Maximum Flood Depth (m)
<0.1
0.1 - 0.2
0.2 - 0.3
0.3 - 0.5
0.5 - 0.75
0.75 - 1
1 - 1.5
1.5 - 2
2 - 3
3 - 4
>4
Projection: GDA / MGA94 Zone 56
Water Technology Pty Ltd
Imagery Source: Google Earth 2019
0
200 m
Wearing Street Flood Study
5% AEP Maximum Flood Depth

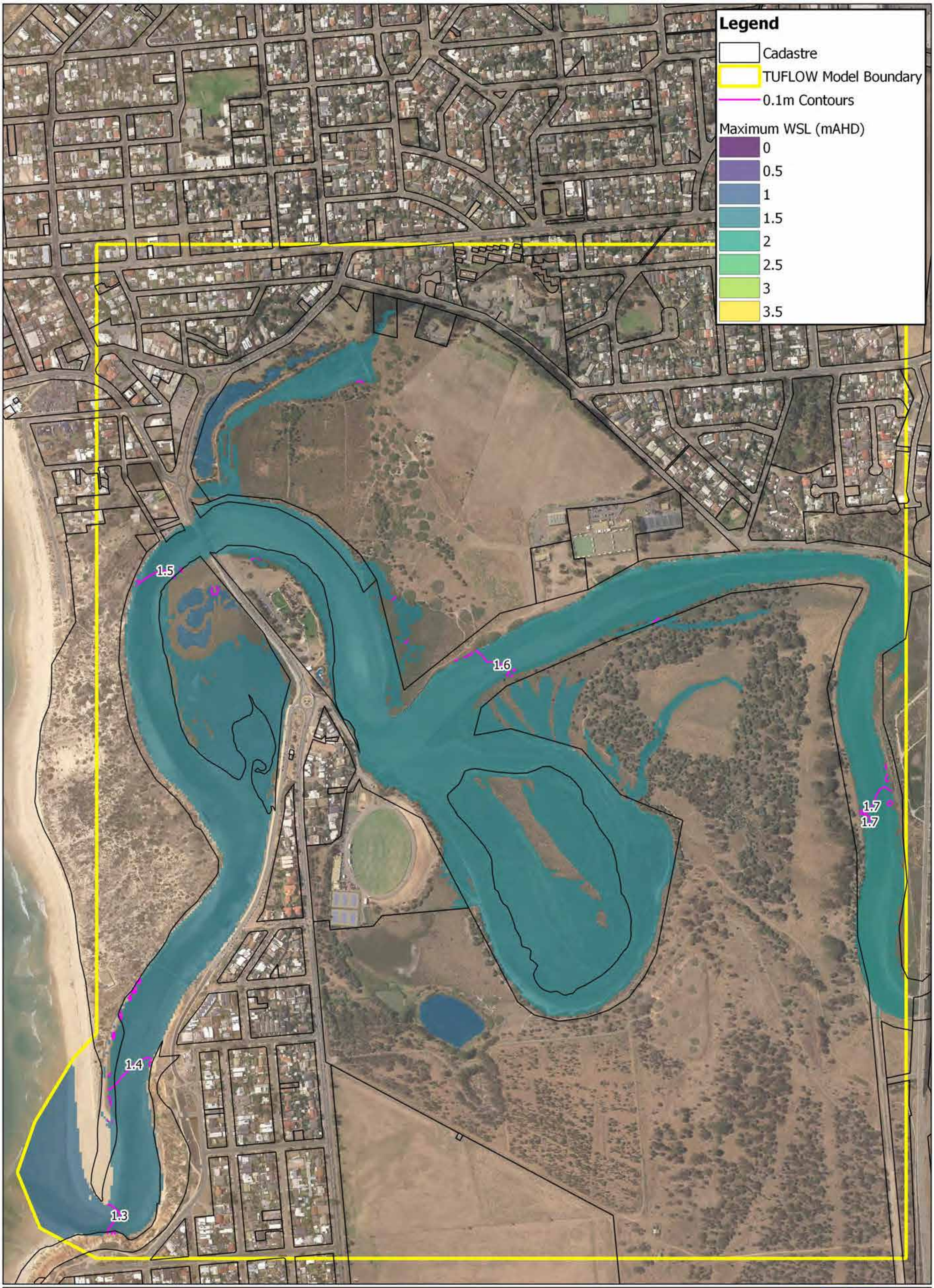

Legend
Cadastre
TUFLOW Model Boundary
Maximum Flood Depth (m)
<0.1
0.1 - 0.2
0.2 - 0.3
0.3 - 0.5
0.5 - 0.75
0.75 - 1
1 - 1.5
1.5 - 2
2 - 3
3 - 4
>4
Projection: GDA / MGA94 Zone 56
Water Technology Pty Ltd
Imagery Source: Google Earth 2019
0
200 m
Wearing Street Flood Study
2% AEP Maximum Flood Depth



Projection: GDA / MGA94 Zone 56
Water Technology Pty Ltd
Imagery Source: Google Earth 2019

Wearing Street Flood Study
1% AEP Maximum Flood Depth

APPENDIX B
FLOOD WATER SURFACE LEVEL MAPS

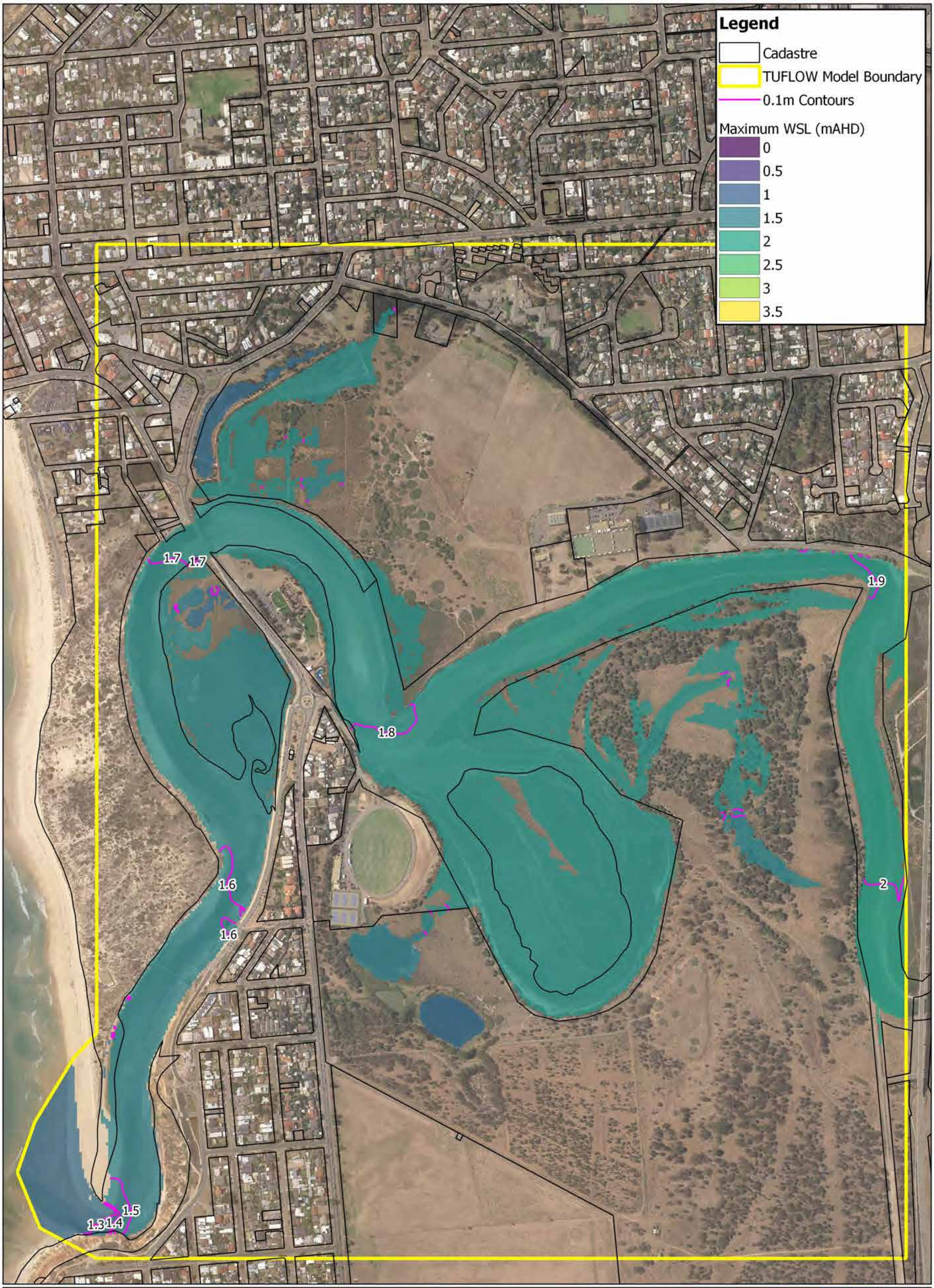


Projection: GDA / MGA94 Zone 56
Water Technology Pty Ltd
Imagery Source: Google Earth 2019

0 200 m

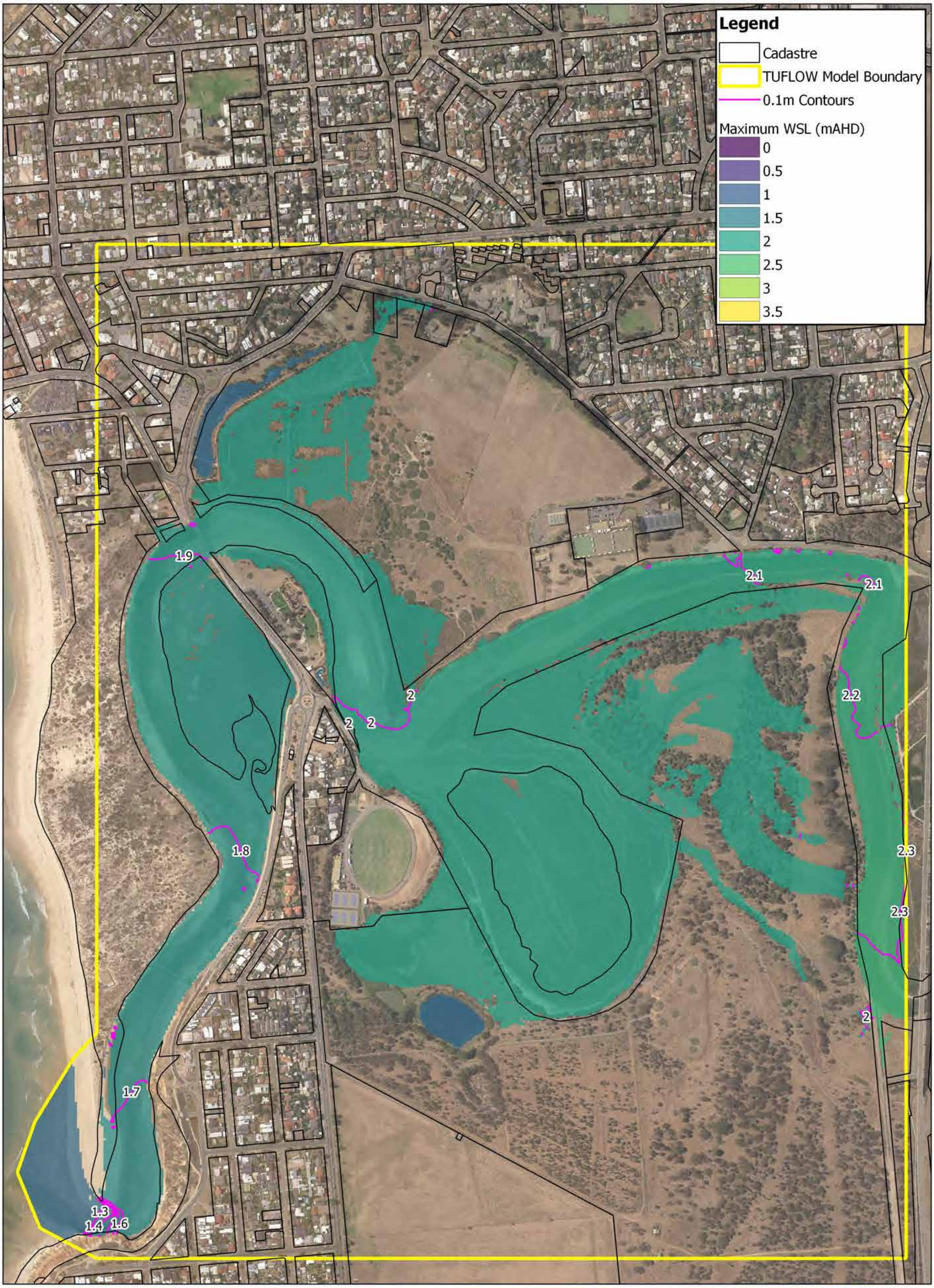
Wearing Street Flood Study

50% AEP Maximum Water Surface Level



Projection: GDA / MGA94 Zone 56
Water Technology Pty Ltd
Imagery Source: Google Earth 2019

Wearing Street Flood Study
20% AEP Maximum Water Surface Level

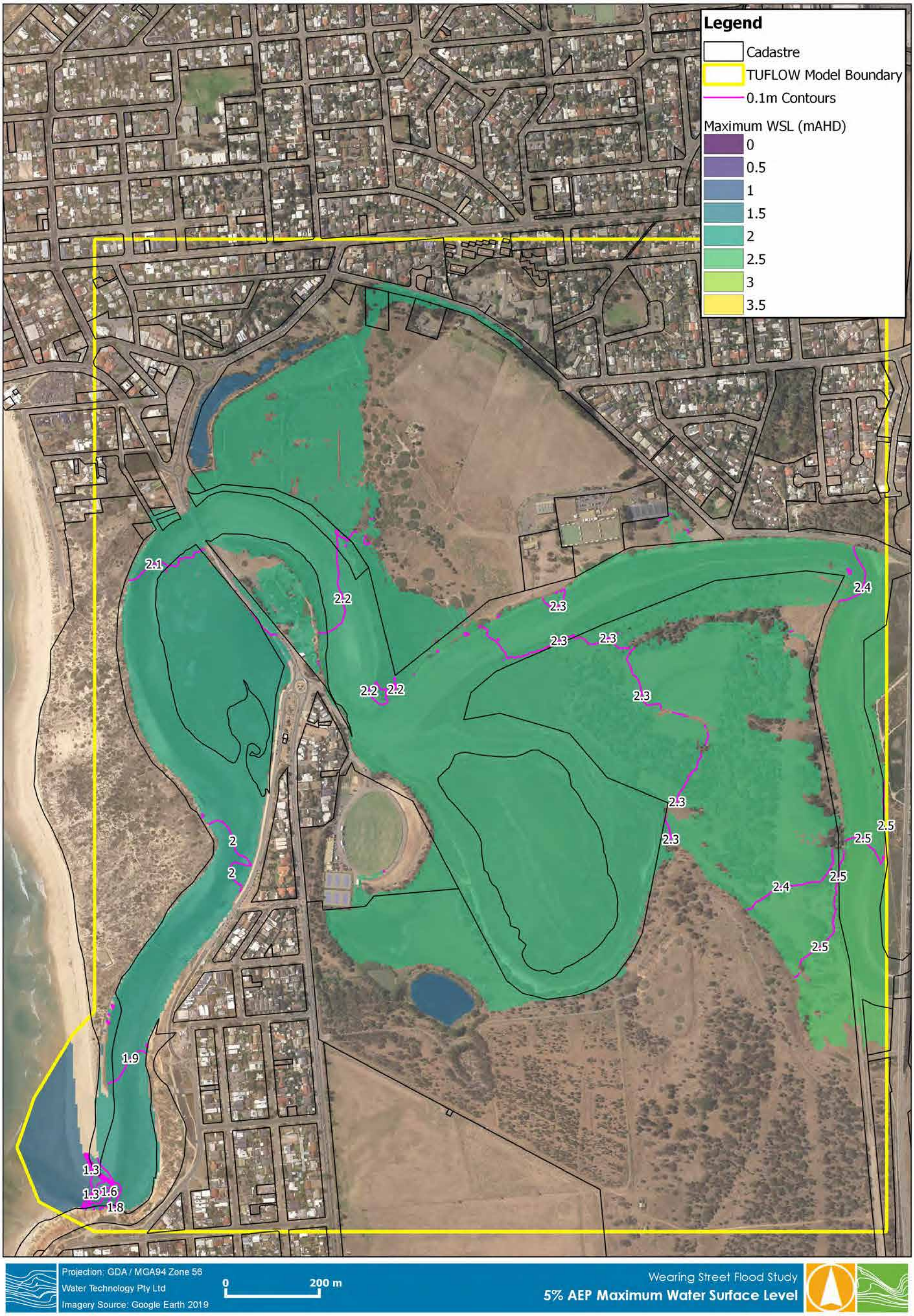


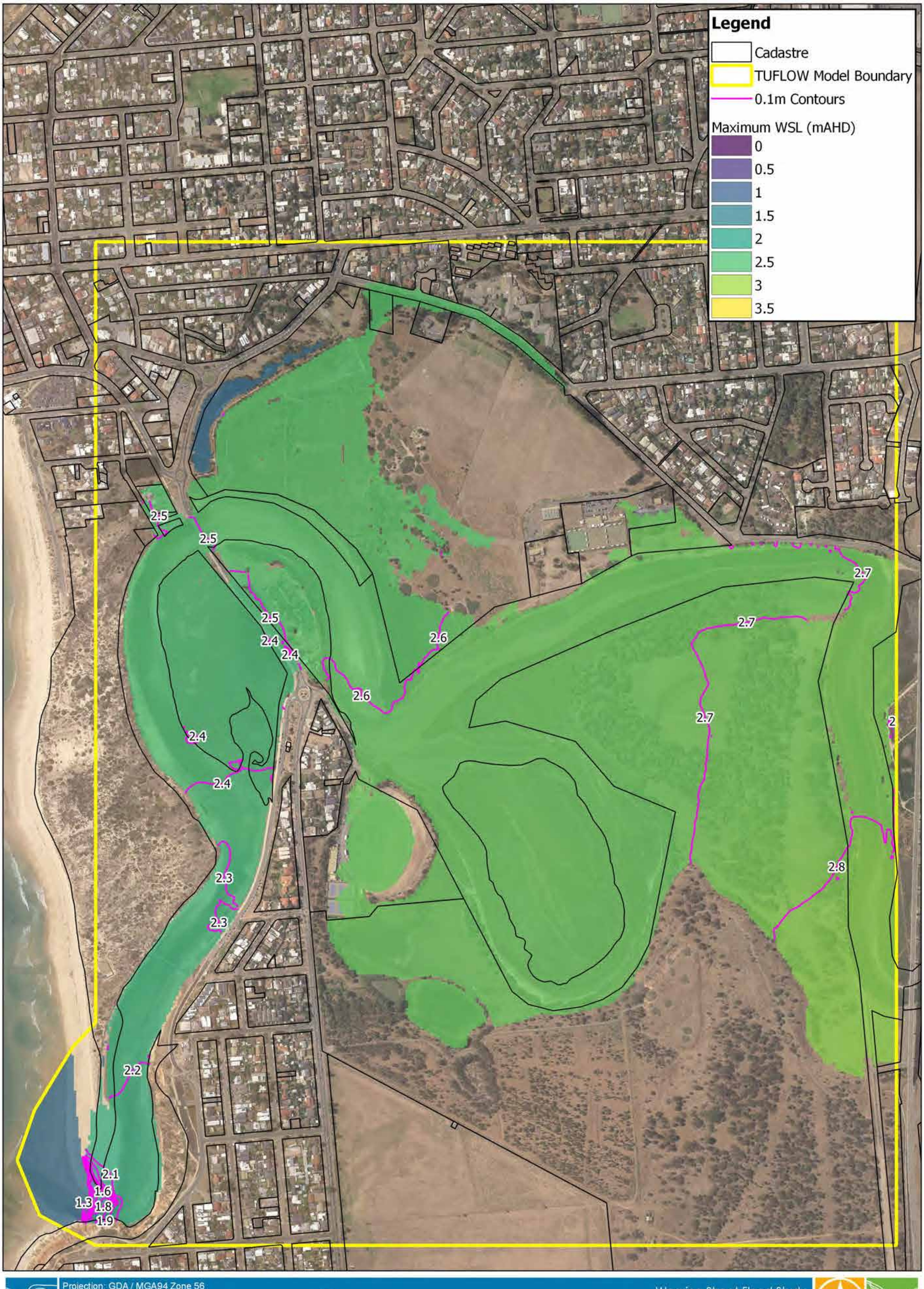
Projection: GDA / MGA94 Zone 56
Water Technology Pty Ltd
Imagery Source: Google Earth 2019

0 200 m

Wearing Street Flood Study

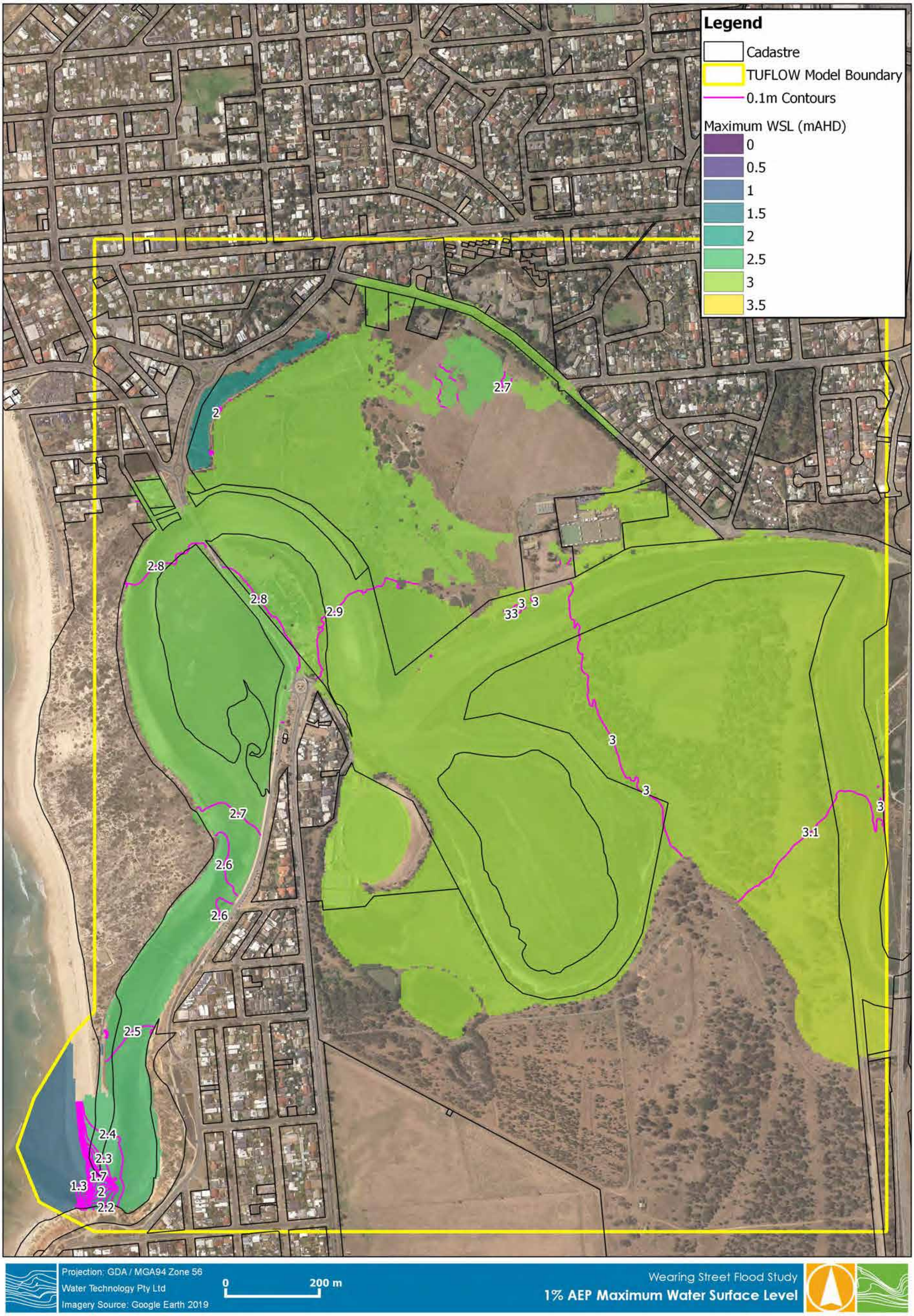
10% AEP Maximum Water Surface Level


Legend
Cadastre
TUFLOW Model Boundary
0.1m Contours
Maximum WSL (mAHD)
0
0.5
1
1.5
2
2.5
3
3.5
2.1
2.2
2.2
2.2
2.3
2.3
2.3
2.3
2.3
2.3
2.4
2.4
2.5
2.5
2.5
2.5
2
2
1.9
1.3
1.3
1.6
1.8
Projection: GDA / MGA94 Zone 56
Water Technology Pty Ltd
Imagery Source: Google Earth 2019
0
200 m
Wearing Street Flood Study
5% AEP Maximum Water Surface Level

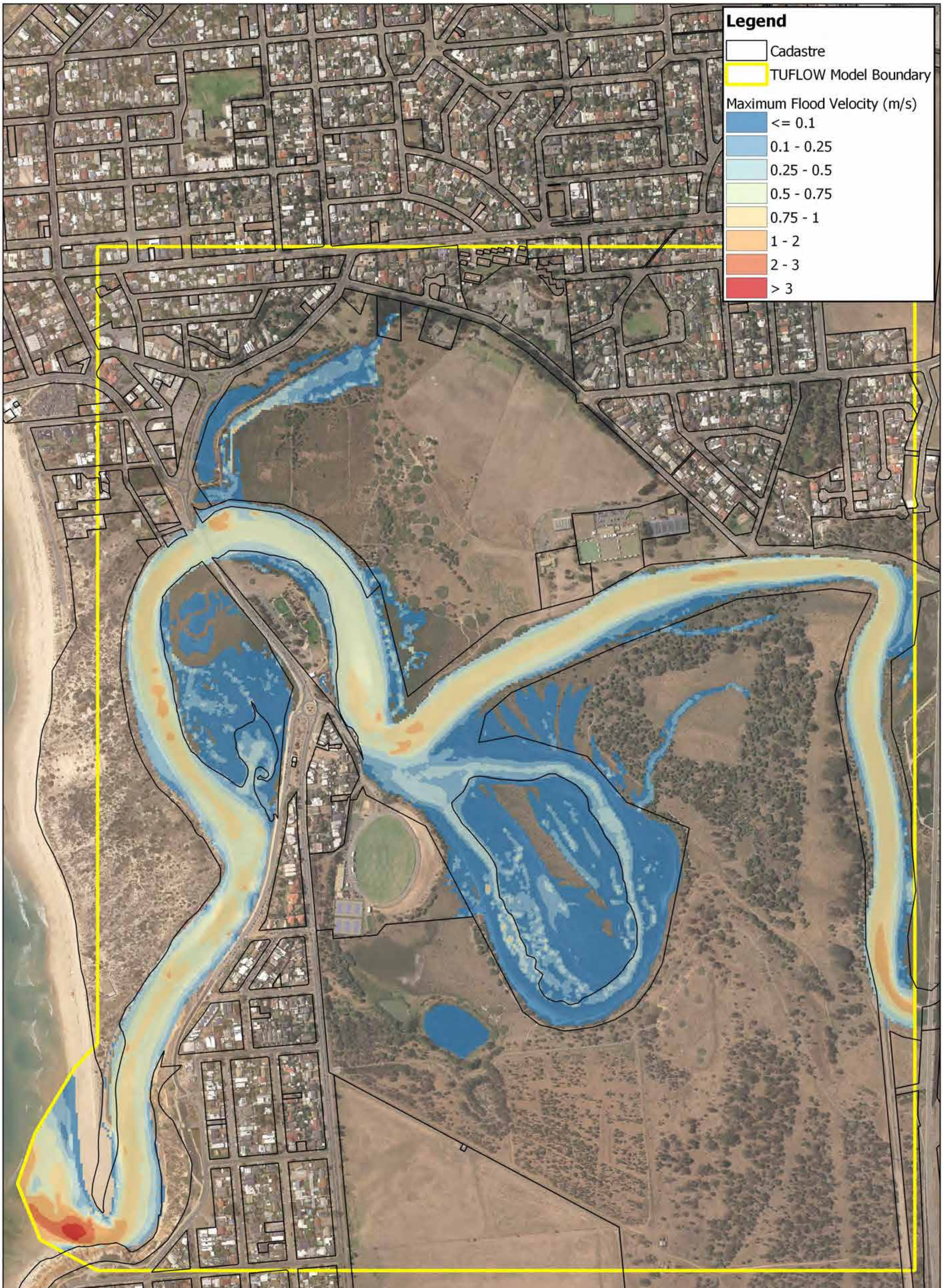


Projection: GDA / MGA94 Zone 56
Water Technology Pty Ltd
Imagery Source: Google Earth 2019

Wearing Street Flood Study
2% AEP Maximum Water Surface Level

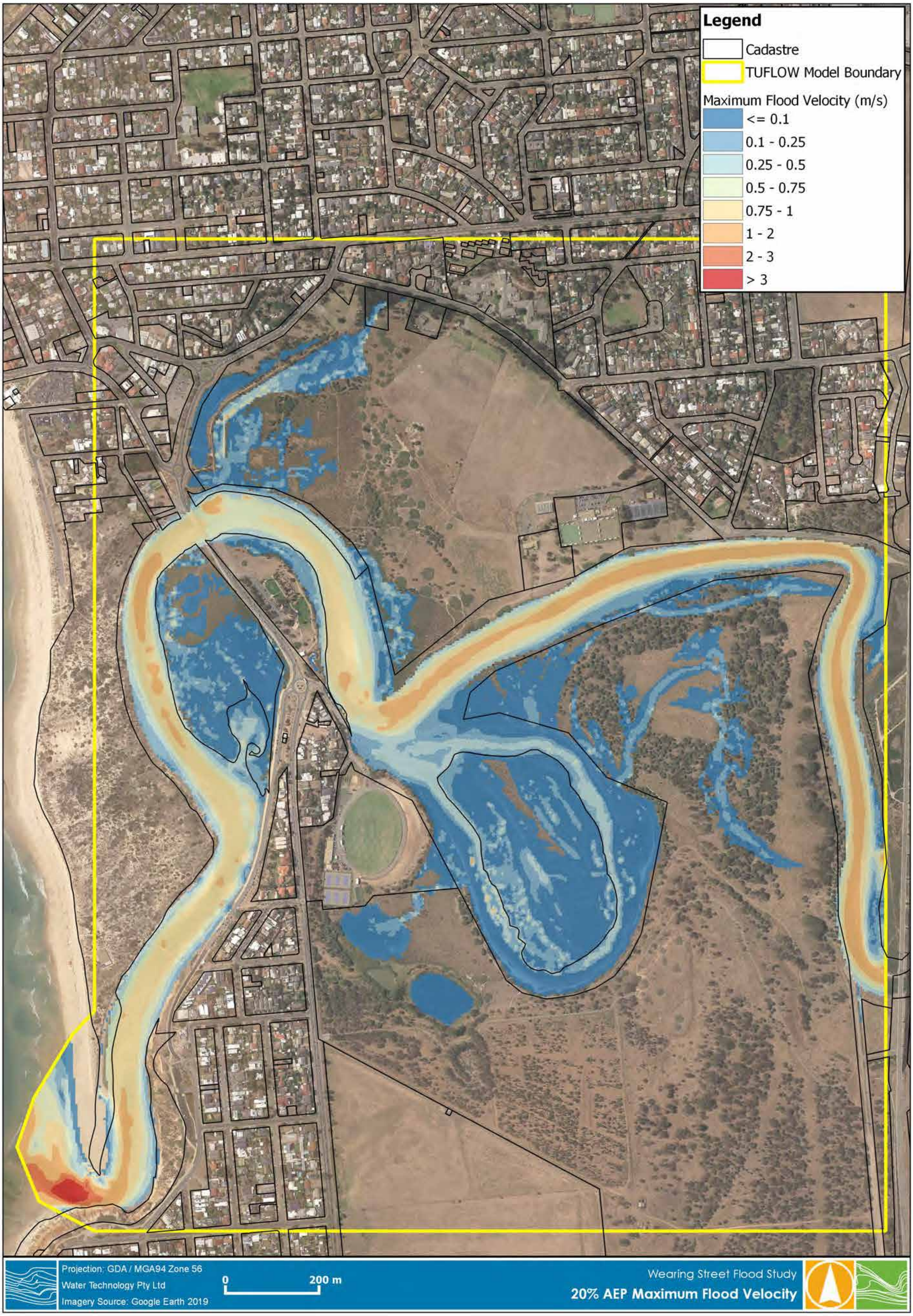

Legend
Cadastre
TUFLOW Model Boundary
0.1m Contours
Maximum WSL (mAHD)
0
0.5
1
1.5
2
2.5
3
3.5
2.7
2
2.8
2.8
2.9
3
3
33
3
3
3
3.1
2.7
2.6
2.6
2.5
2.4
2.3
1.7
1.3
2
2.2
Projection: GDA / MGA94 Zone 56
Water Technology Pty Ltd
Imagery Source: Google Earth 2019
0
200 m
Wearing Street Flood Study
1% AEP Maximum Water Surface Level

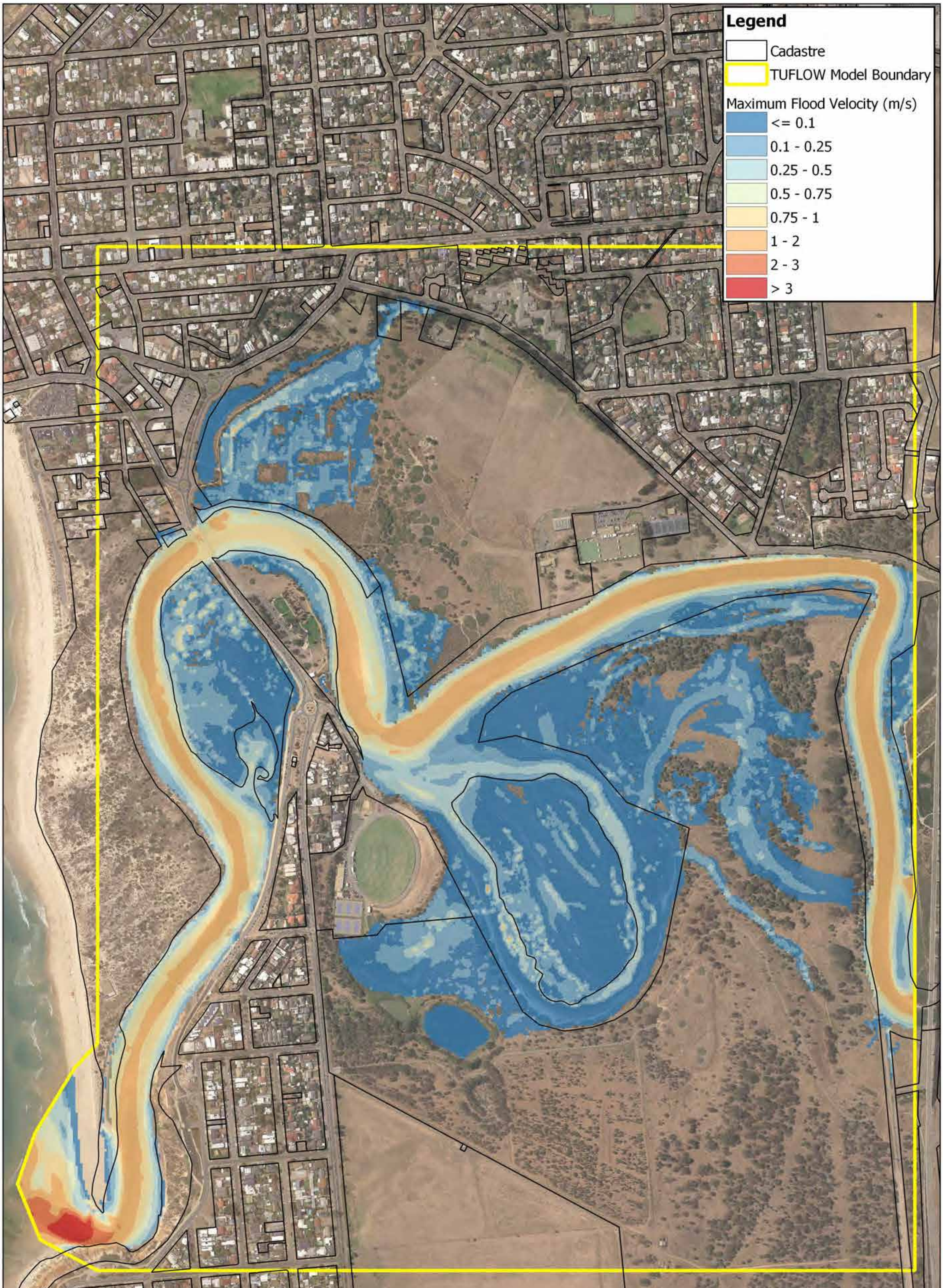
APPENDIX C
FLOOD VELOCITY MAPS



Projection: GDA / MGA94 Zone 56
Water Technology Pty Ltd
Imagery Source: Google Earth 2019

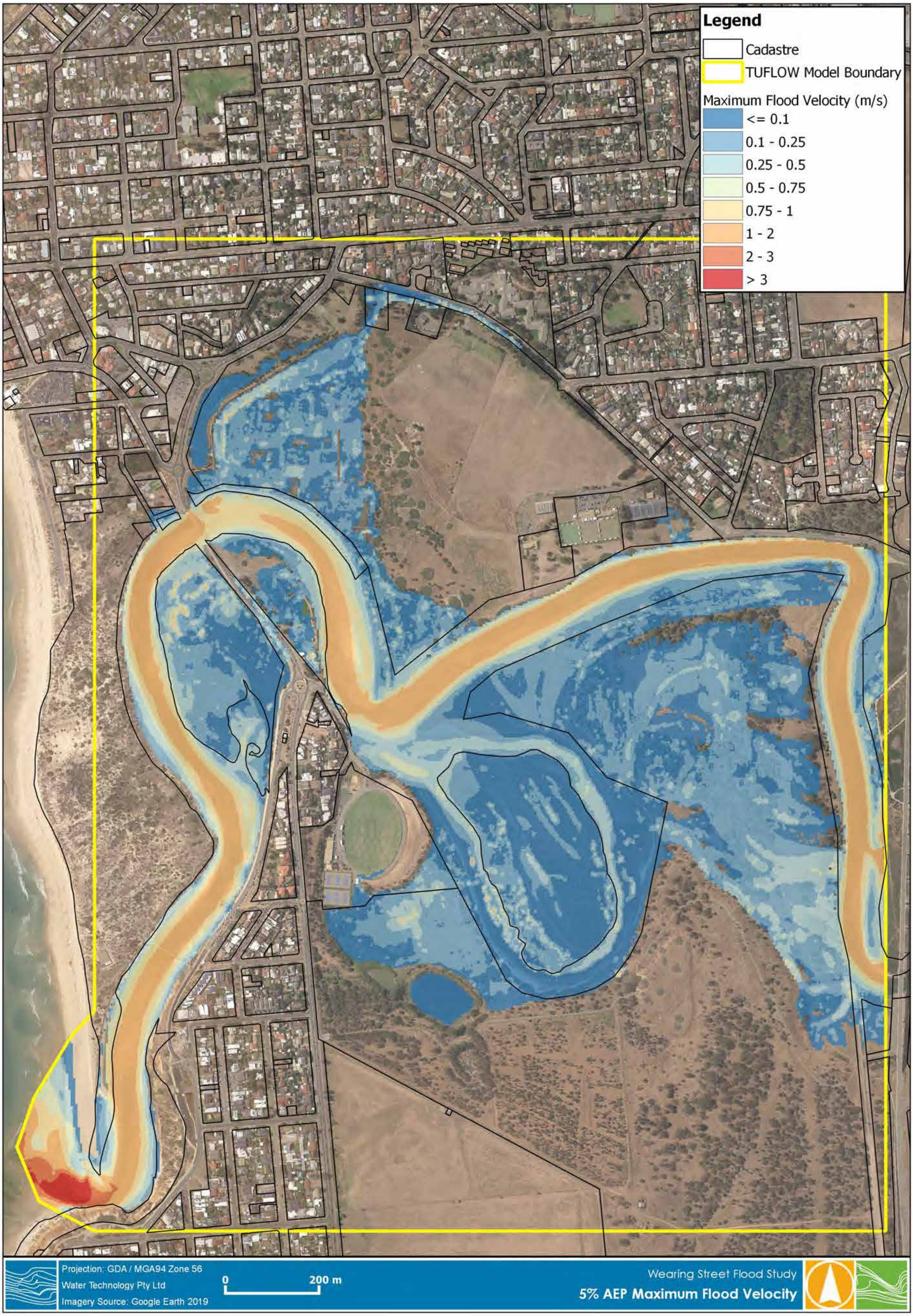
Wearing Street Flood Study
50% AEP Maximum Flood Velocity

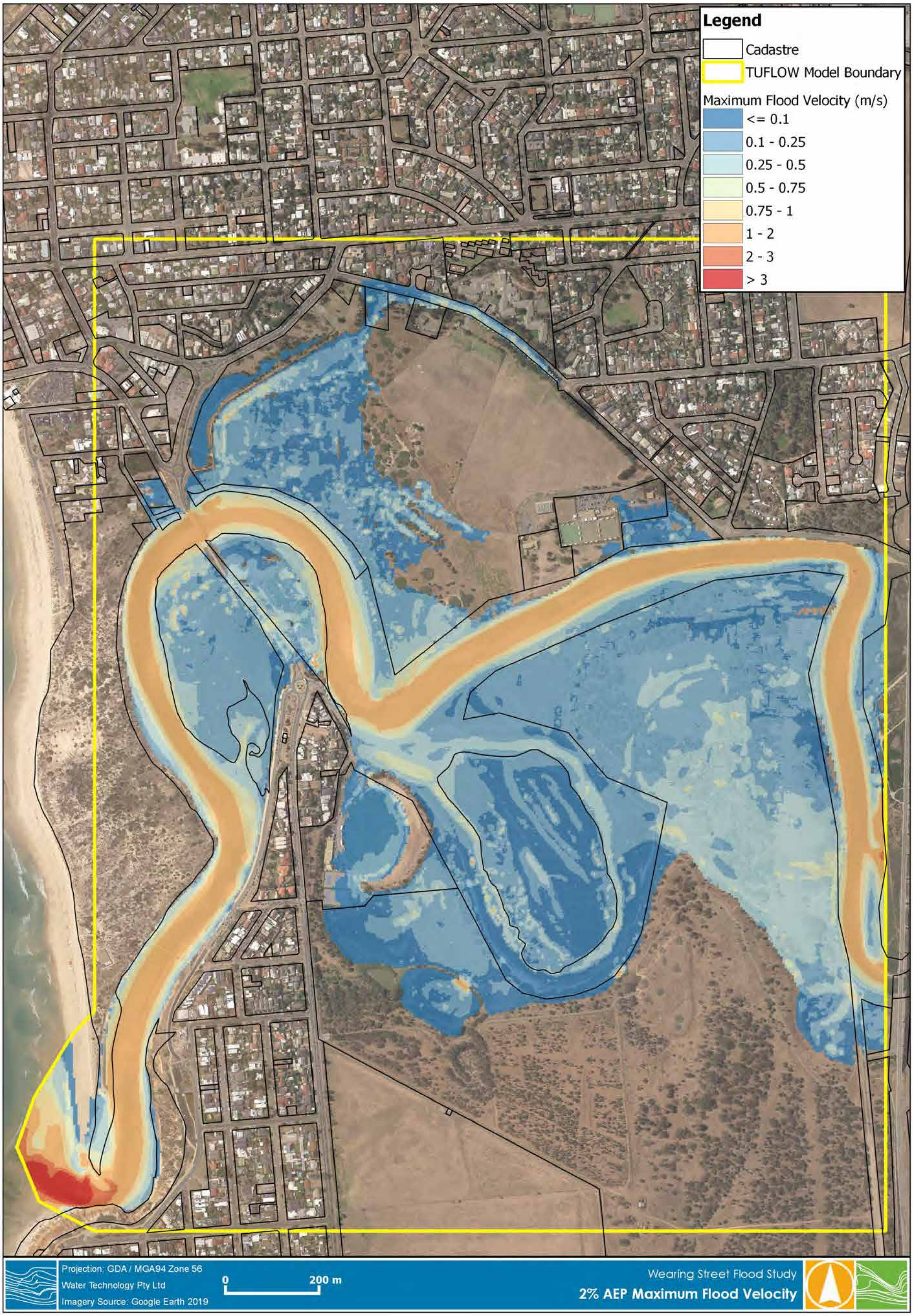

Legend
Cadastre
TUFLOW Model Boundary
Maximum Flood Velocity (m/s)
<= 0.1
0.1 - 0.25
0.25 - 0.5
0.5 - 0.75
0.75 - 1
1 - 2
2 - 3
> 3
Projection: GDA / MGA94 Zone 56
Water Technology Pty Ltd
Imagery Source: Google Earth 2019
0
200 m
Wearing Street Flood Study
20% AEP Maximum Flood Velocity

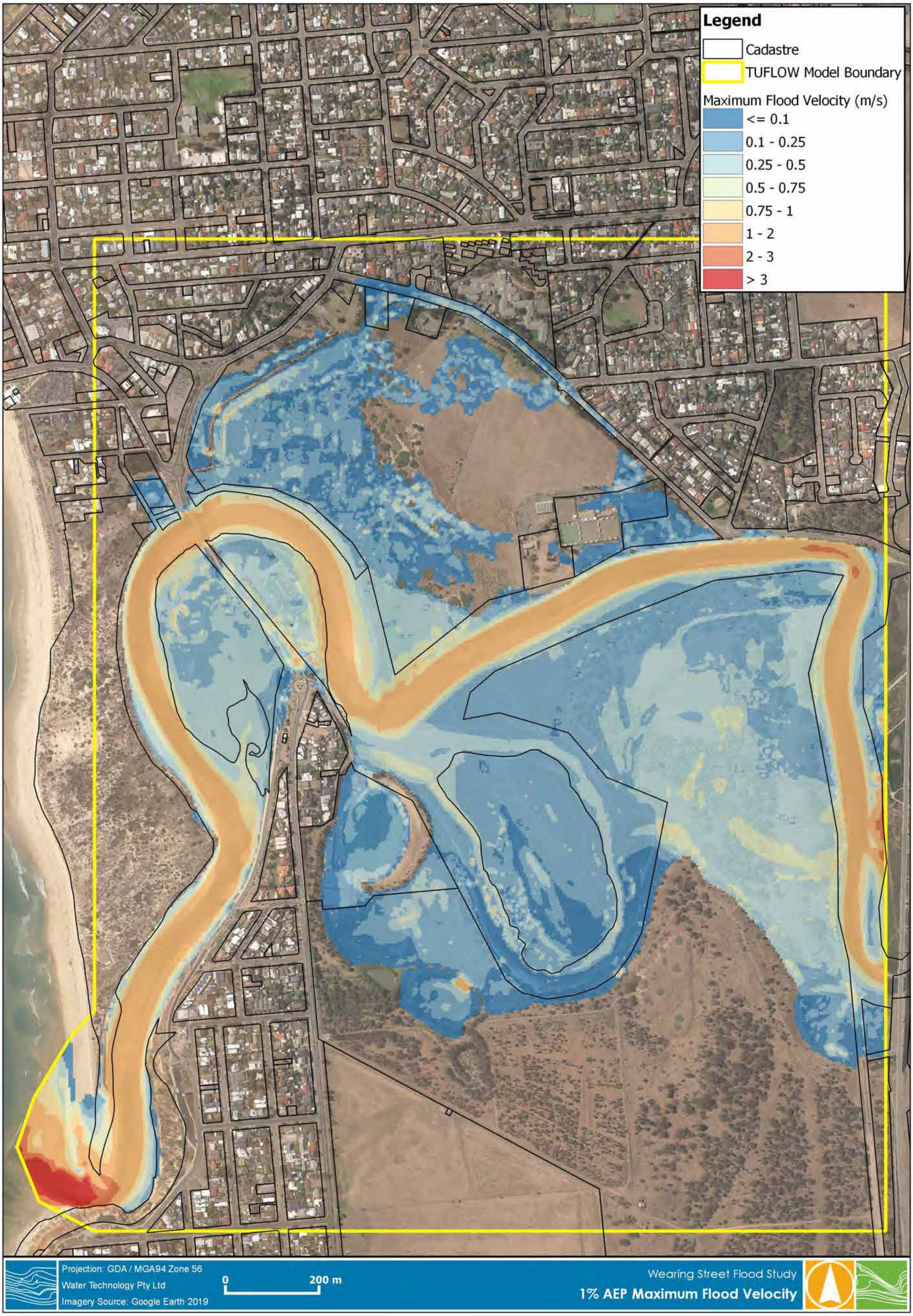


Projection: GDA / MGA94 Zone 56
Water Technology Pty Ltd
Imagery Source: Google Earth 2019

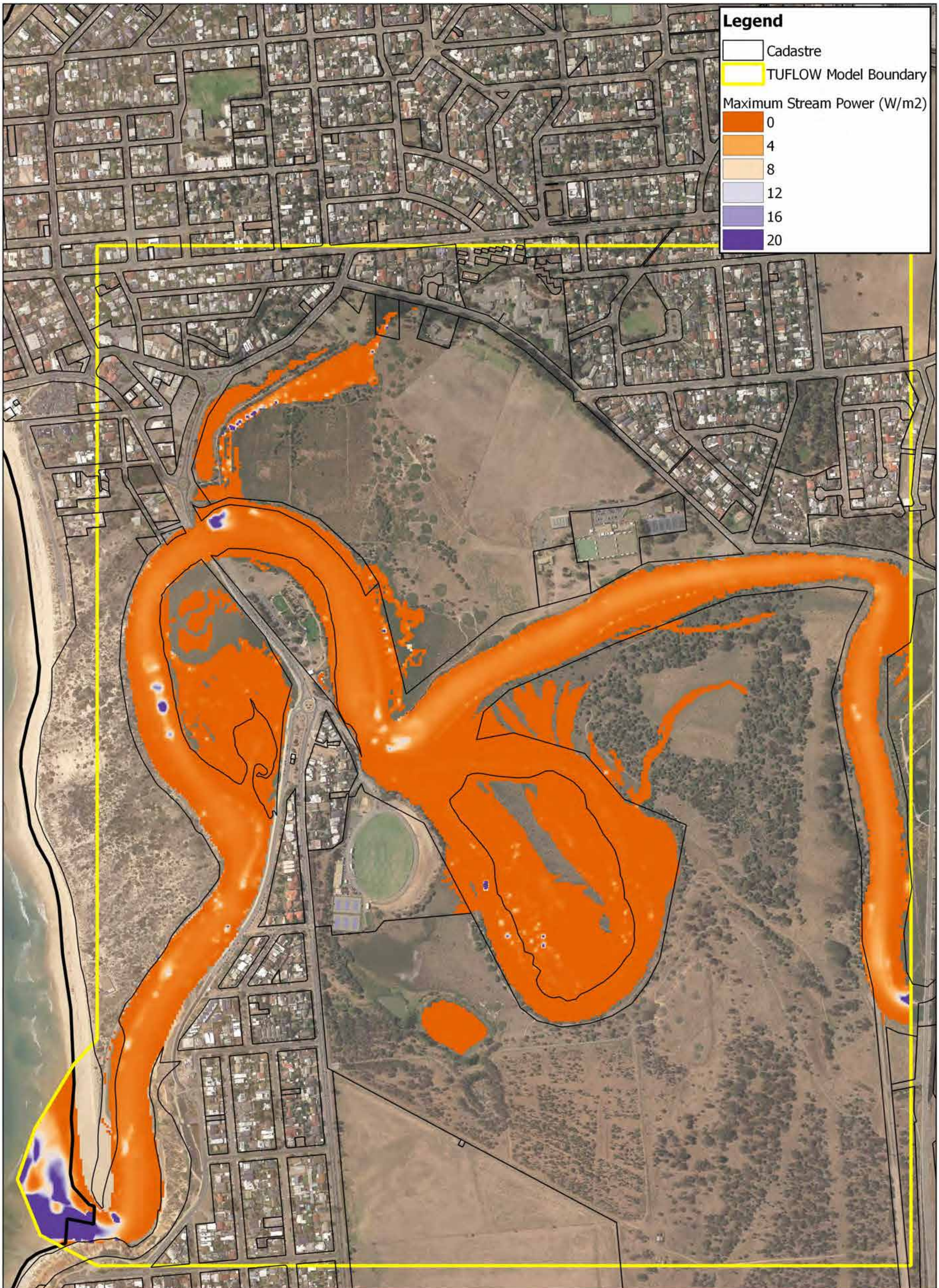
Wearing Street Flood Study
10% AEP Maximum Flood Velocity


Legend
Cadastre
TUFLOW Model Boundary
Maximum Flood Velocity (m/s)
<= 0.1
0.1 - 0.25
0.25 - 0.5
0.5 - 0.75
0.75 - 1
1 - 2
2 - 3
> 3
Projection: GDA / MGA94 Zone 56
Water Technology Pty Ltd
Imagery Source: Google Earth 2019
0
200 m
Wearing Street Flood Study
5% AEP Maximum Flood Velocity


Legend
Cadastre
TUFLOW Model Boundary
Maximum Flood Velocity (m/s)
<= 0.1
0.1 - 0.25
0.25 - 0.5
0.5 - 0.75
0.75 - 1
1 - 2
2 - 3
> 3
Projection: GDA / MGA94 Zone 56
Water Technology Pty Ltd
Imagery Source: Google Earth 2019
0
200 m
Wearing Street Flood Study
2% AEP Maximum Flood Velocity


Legend
Cadastre
TUFLOW Model Boundary
Maximum Flood Velocity (m/s)
<= 0.1
0.1 - 0.25
0.25 - 0.5
0.5 - 0.75
0.75 - 1
1 - 2
2 - 3
> 3
Projection: GDA / MGA94 Zone 56
Water Technology Pty Ltd
Imagery Source: Google Earth 2019
0
200 m
Wearing Street Flood Study
1% AEP Maximum Flood Velocity

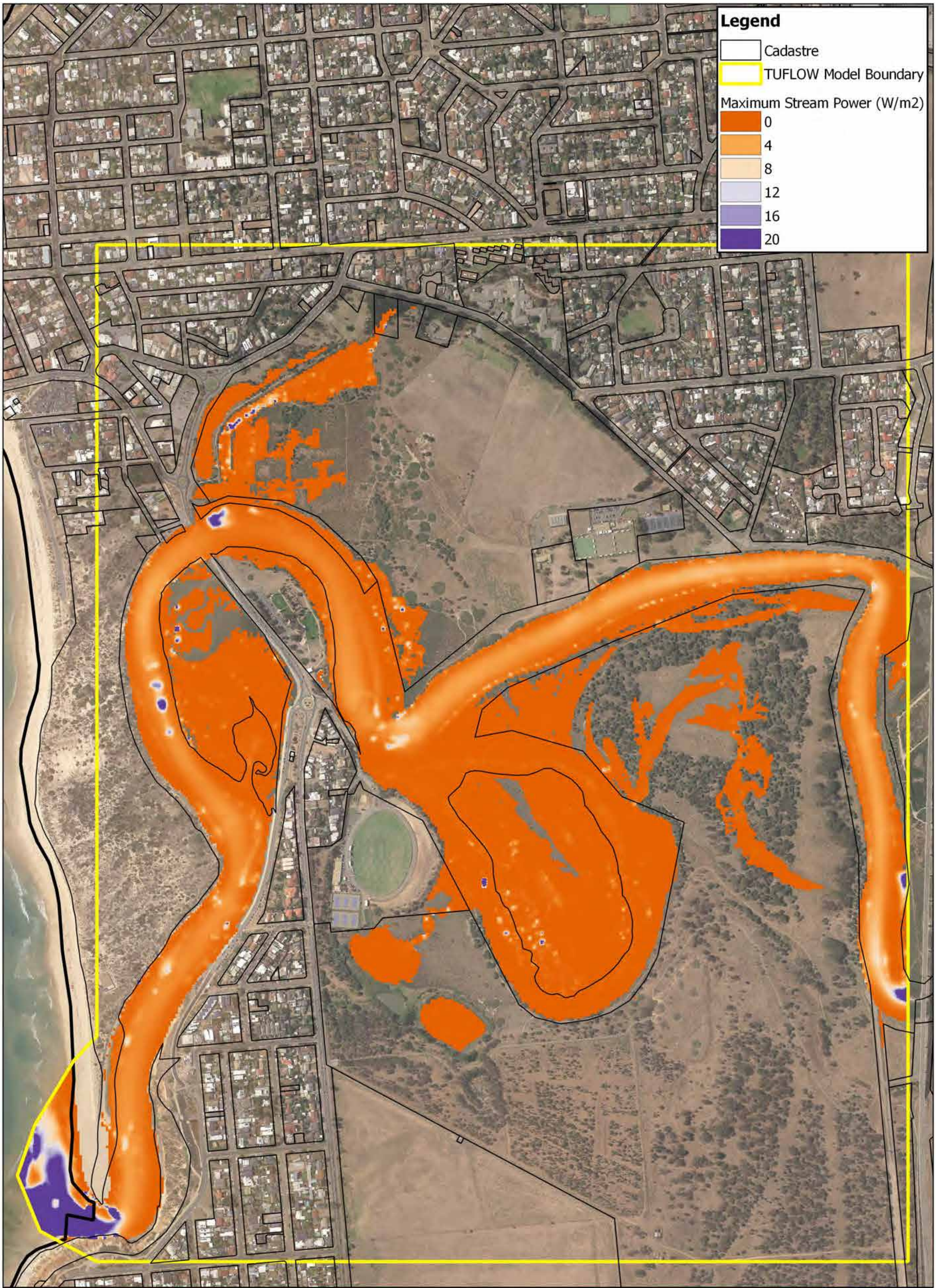
APPENDIX D
FLOOD STREAM POWER MAPS



Projection: GDA / MGA94 Zone 56
Water Technology Pty Ltd
Imagery Source: Google Earth 2019

0 200 m

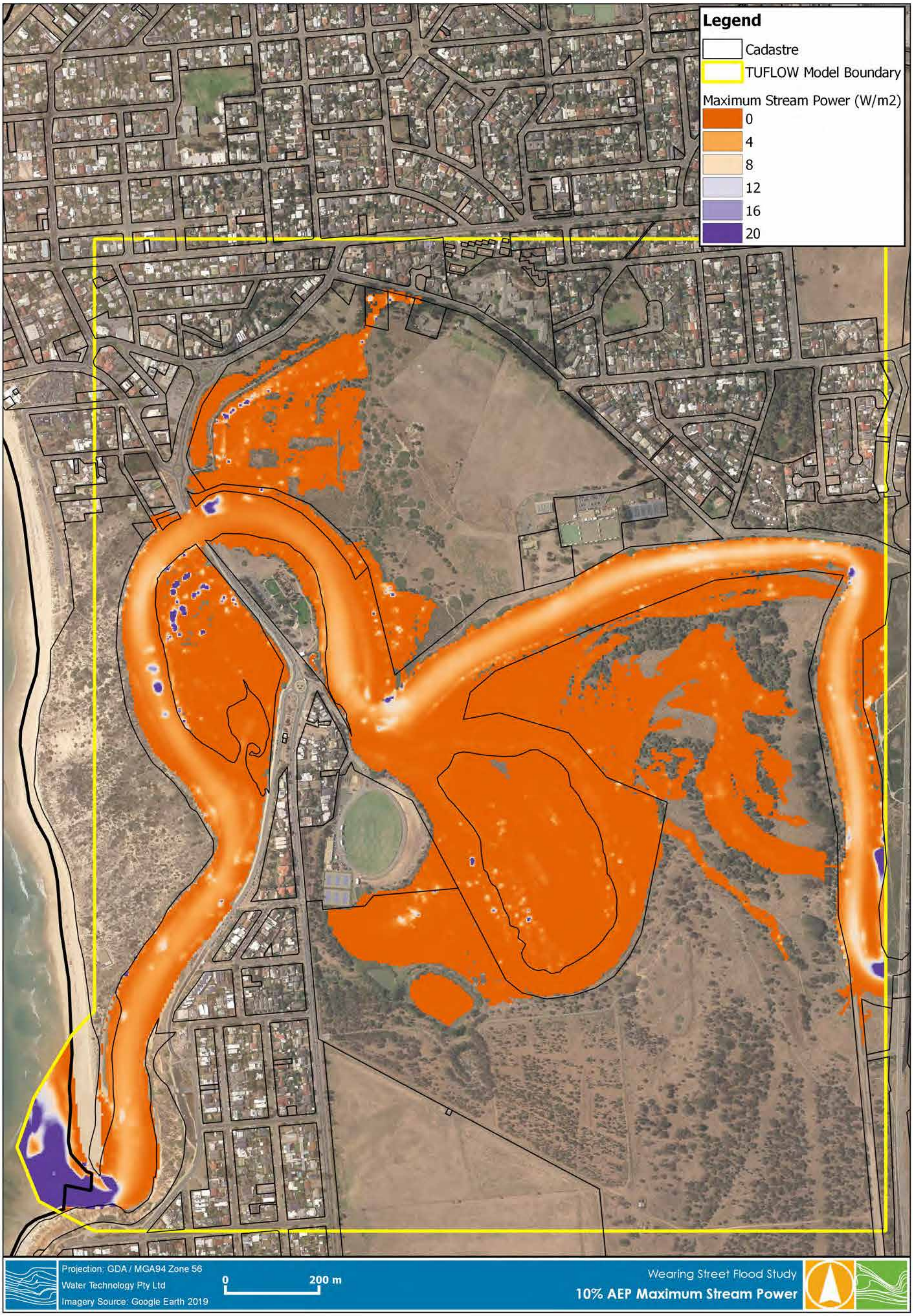
Wearing Street Flood Study
50% AEP Maximum Stream Power



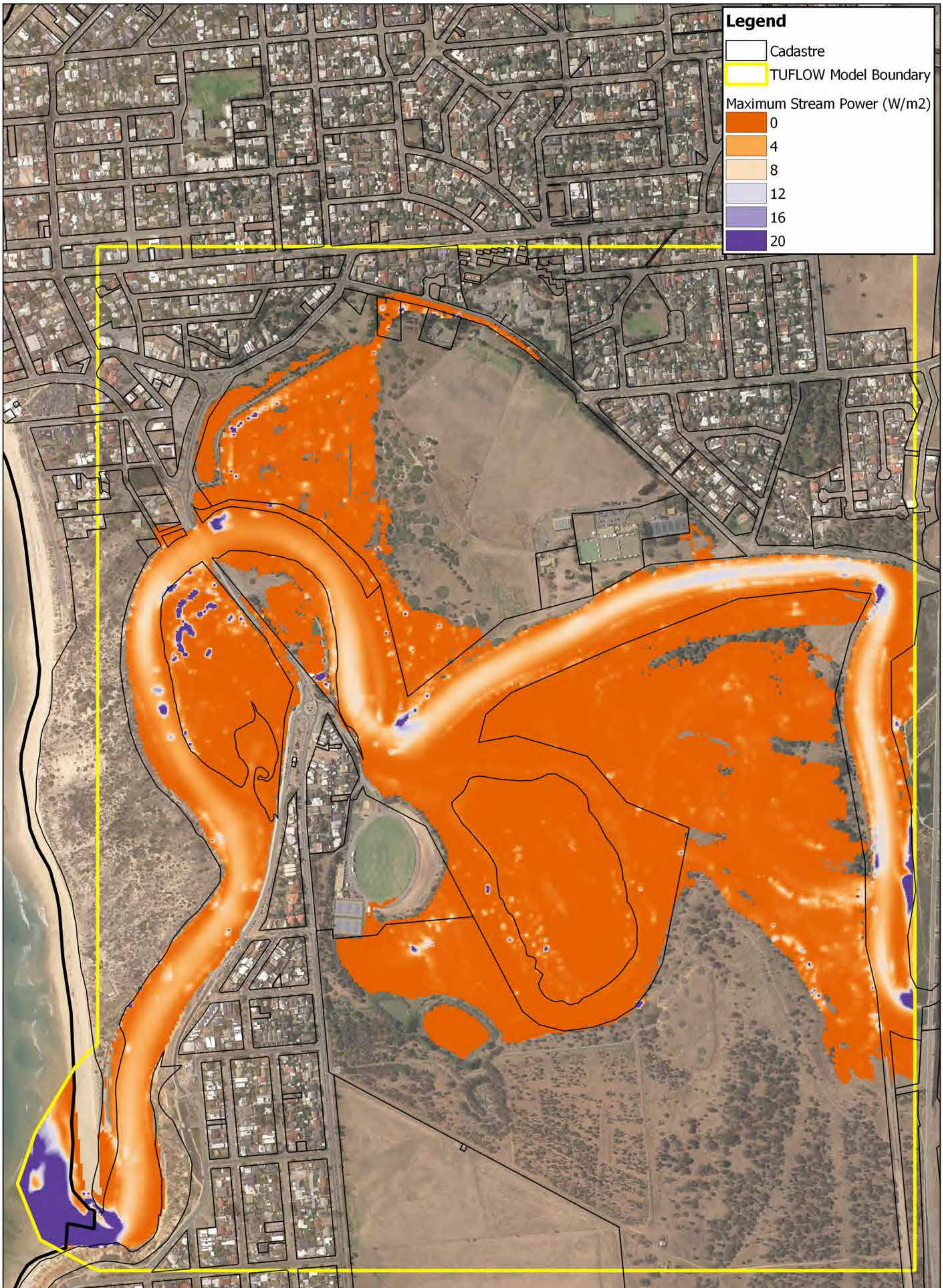
Projection: GDA / MGA94 Zone 56
Water Technology Pty Ltd
Imagery Source: Google Earth 2019

Wearing Street Flood Study

20% AEP Maximum Stream Power



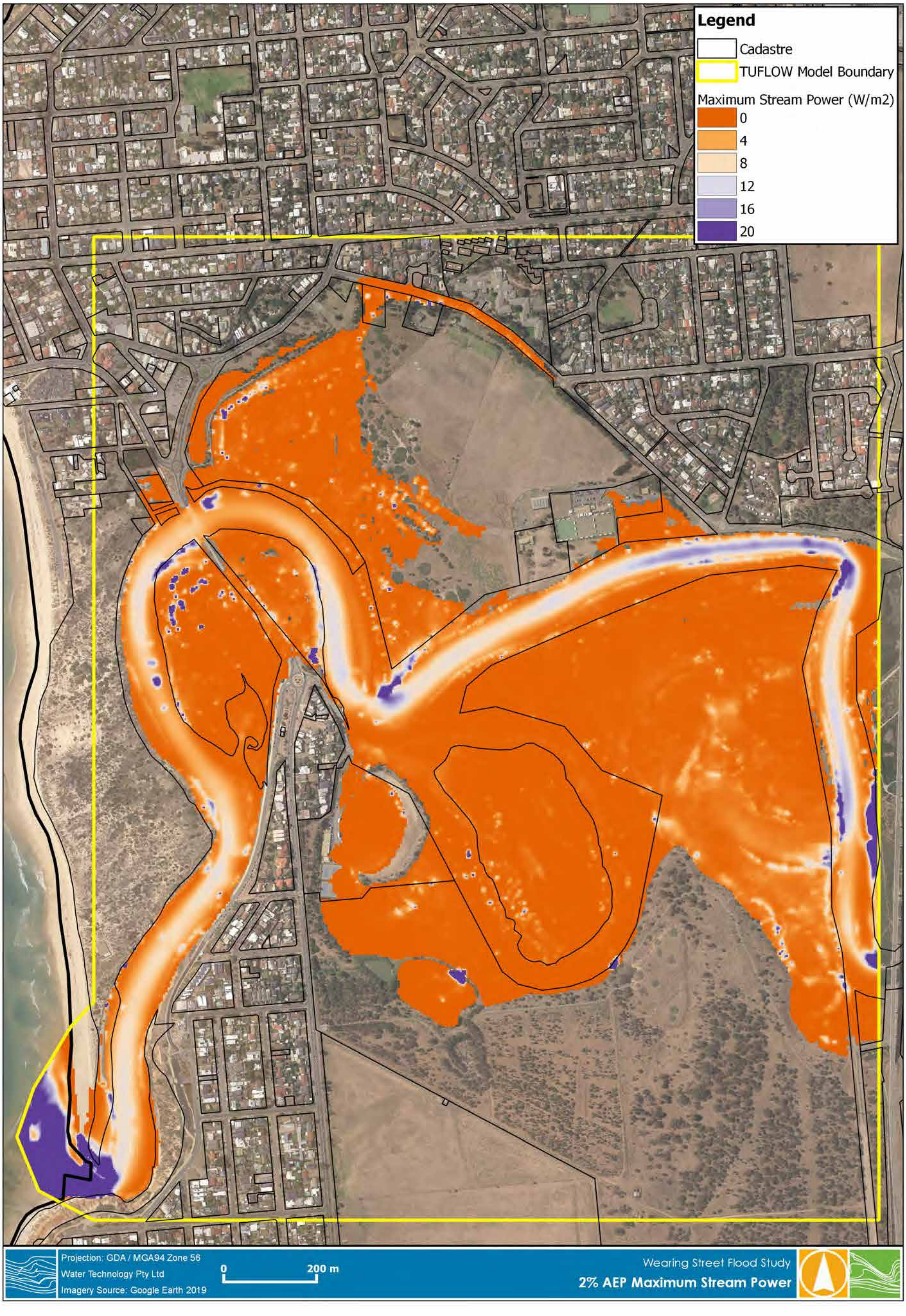
Legend
Cadastre
TUFLOW Model Boundary
Maximum Stream Power (W/m2)
0
4
8
12
16
20
Projection: GDA / MGA94 Zone 56
Water Technology Pty Ltd
Imagery Source: Google Earth 2019
0
200 m
Wearing Street Flood Study
10% AEP Maximum Stream Power

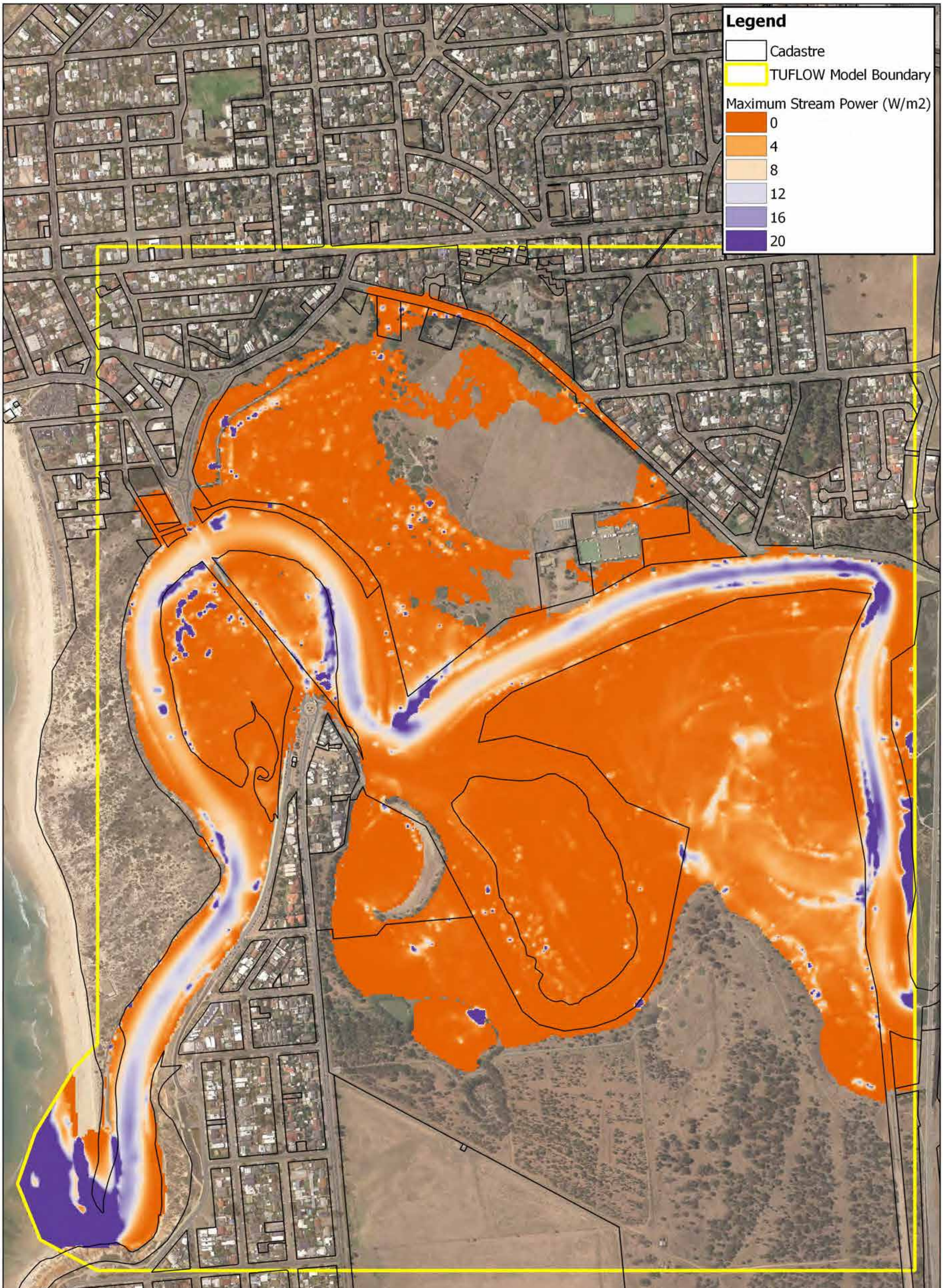


Projection: GDA / MGA94 Zone 56
Water Technology Pty Ltd
Imagery Source: Google Earth 2019

Wearing Street Flood Study

5% AEP Maximum Stream Power


Legend
Cadastre
TUFLOW Model Boundary
Maximum Stream Power (W/m2)
0
4
8
12
16
20
Projection: GDA / MGA94 Zone 56
Water Technology Pty Ltd
Imagery Source: Google Earth 2019
0
200 m
Wearing Street Flood Study
2% AEP Maximum Stream Power

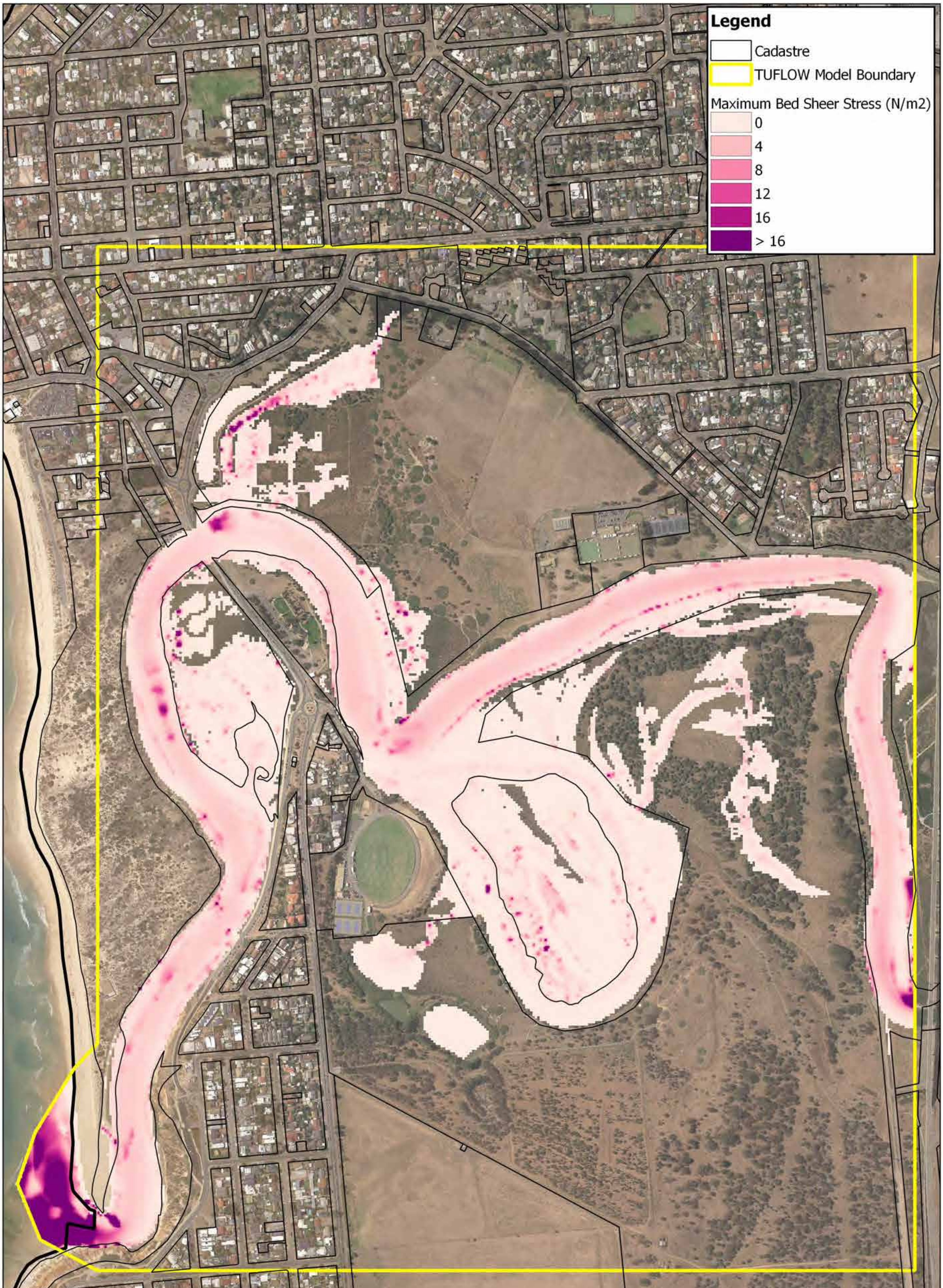


Projection: GDA / MGA94 Zone 56
Water Technology Pty Ltd
Imagery Source: Google Earth 2019

Wearing Street Flood Study
1% AEP Maximum Stream Power

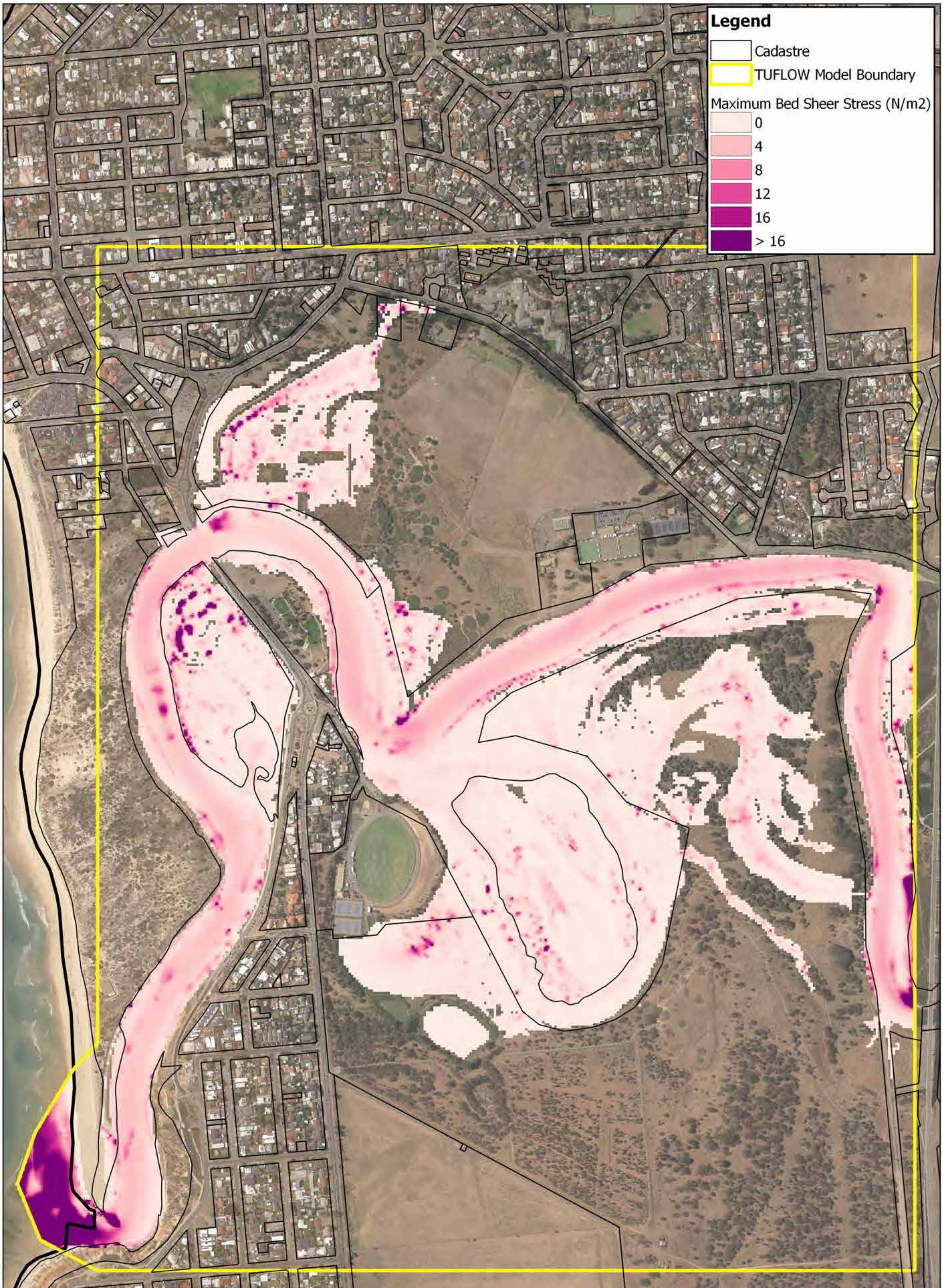
APPENDIX E
FLOOD BED SHEAR STRESS MAPS


Legend
Cadastre
TUFLOW Model Boundary
Maximum Bed Sheer Stress (N/m2)
0
4
8
12
16
> 16
Projection: GDA / MGA94 Zone 56
Water Technology Pty Ltd
Imagery Source: Google Earth 2019
0
200 m
Wearing Street Flood Study
50% AEP Maximum Bed Shear Stress



Projection: GDA / MGA94 Zone 56
Water Technology Pty Ltd
Imagery Source: Google Earth 2019

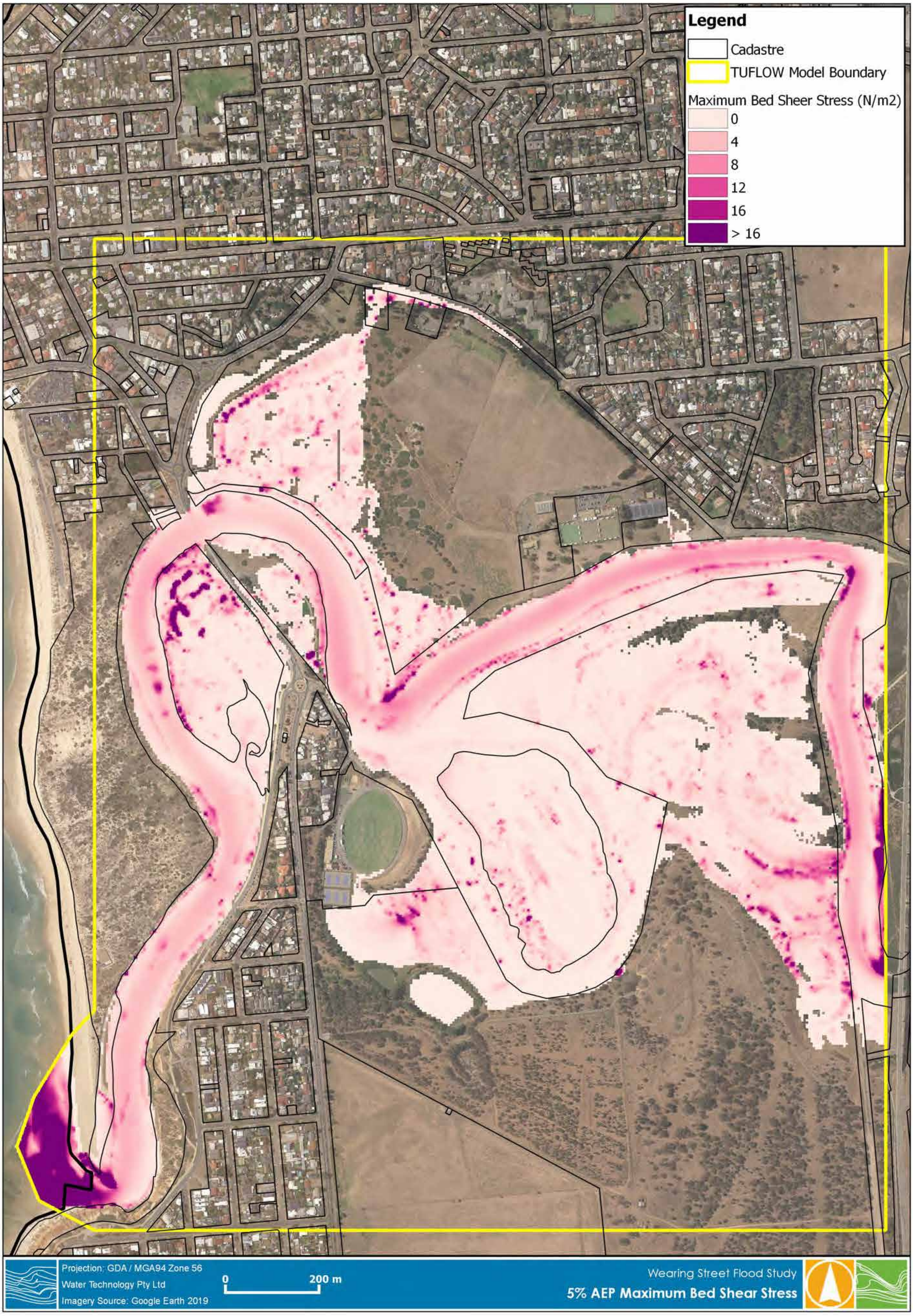
Wearing Street Flood Study
20% AEP Maximum Bed Shear Stress

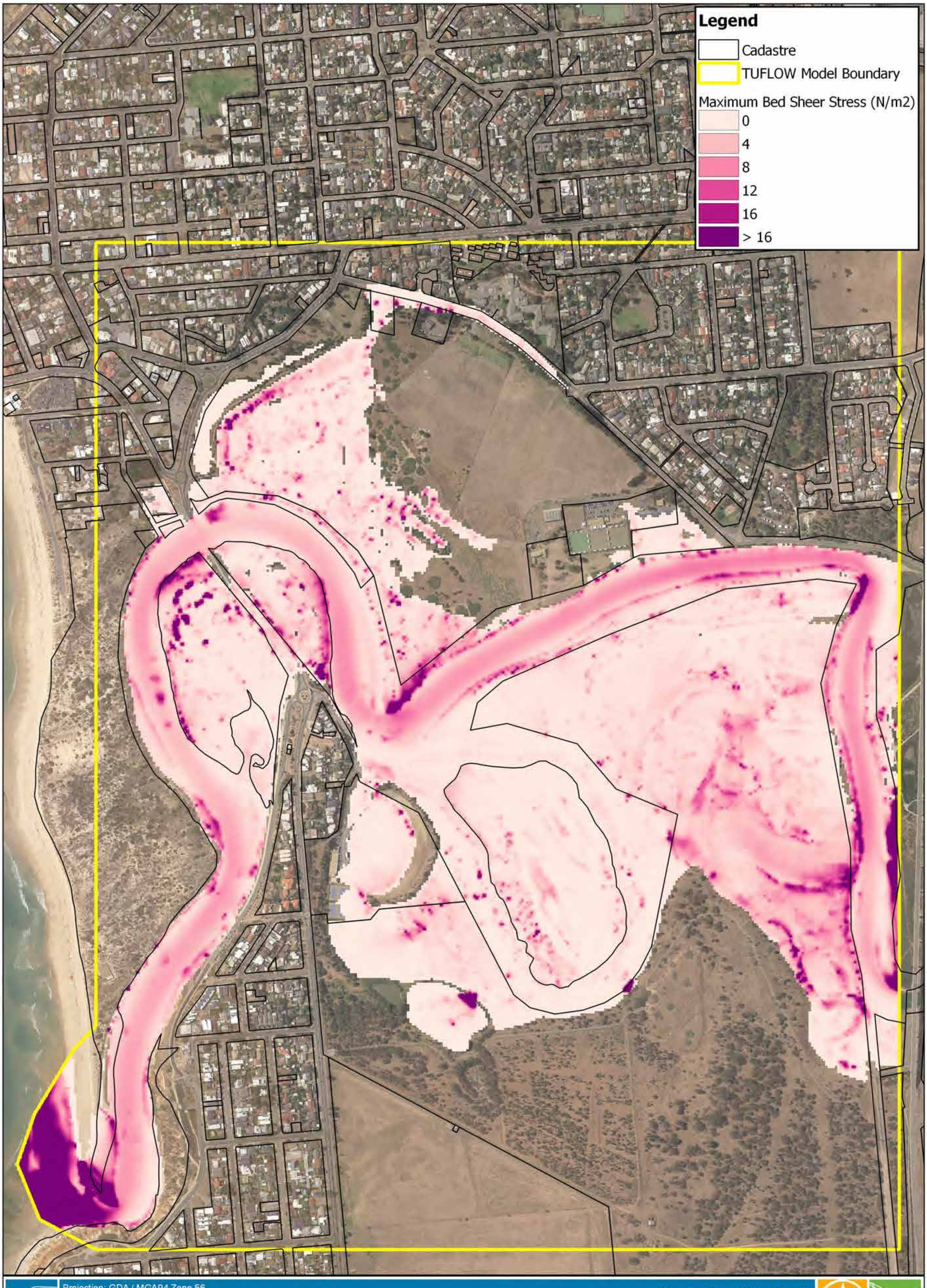


Projection: GDA / MGA94 Zone 56
Water Technology Pty Ltd
Imagery Source: Google Earth 2019

Wearing Street Flood Study

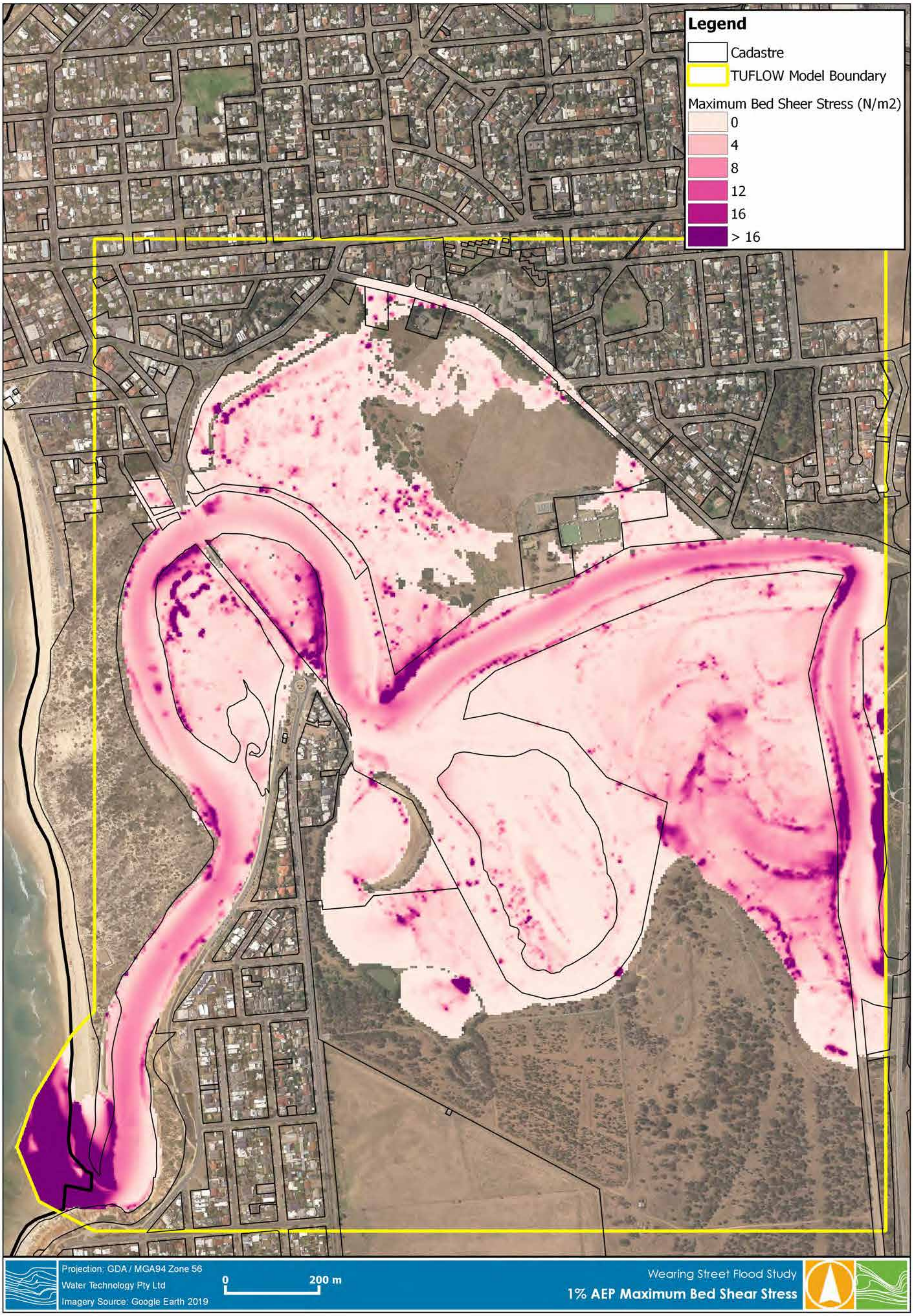
10% AEP Maximum Bed Shear Stress


Legend
Cadastre
TUFLOW Model Boundary
Maximum Bed Sheer Stress (N/m2)
0
4
8
12
16
> 16
Projection: GDA / MGA94 Zone 56
Water Technology Pty Ltd
Imagery Source: Google Earth 2019
0
200 m
Wearing Street Flood Study
5% AEP Maximum Bed Shear Stress



Projection: GDA / MGA94 Zone 56
Water Technology Pty Ltd
Imagery Source: Google Earth 2019

Wearing Street Flood Study
2% AEP Maximum Bed Shear Stress


Legend
Cadastre
TUFLOW Model Boundary
Maximum Bed Sheer Stress (N/m2)
0
4
8
12
16
> 16
Projection: GDA / MGA94 Zone 56
Water Technology Pty Ltd
Imagery Source: Google Earth 2019
0
200 m
Wearing Street Flood Study
1% AEP Maximum Bed Shear Stress

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