Northeast Business Park MIKE21 Flood Study

May, 2008

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Executive summary

Northeast Business Park Pty Ltd is proposing to develop a 326 ha multiuse precinct on 760 ha of privately owned land located at Nolan Drive, Morayfield. This degraded site is a former pine plantation on the southern banks of the Caboolture River near Burpengary. The development will have a marine industry and business focus and provide new public access to the riverfront.

The terms of reference for this Flood Study were set at a meeting on the 10 August 2005 attended by Trefor Jones and Leanne Salter of the Caboolture Shire Council (CSC), Northeast Business Park Pty Ltd and Parsons Brinckerhoff. At this meeting the flood plain management policy and the stormwater quality requirements were discussed.

This investigation details the floodplain modelling for the proposed Northeast Business Park. Modelling was undertaken using the MIKE21 software package developed by the Danish Hydraulics Institute. The outcomes of the modelling have been assessed against CSC's two main floodplain management conditions:

- no net loss of flood storage across the development site
- no resultant increase in flood levels over adjoining properties.

Model scenarios contained in this report are:

- Base Case — developed to determine the existing condition peak flood levels throughout the floodplain. This case represents the existing floodplain topography as surveyed in October 2005. Model calibration and verification was undertaken with the base case against three historical events (1972, 1989 and 1991). Model sensitivity, model fitness and a mass balance were also assessed. Overall the MIKE21 model is a good representation of the lower Caboolture River floodplain and comparison against the 1994 flood model results shows an improvement in the calibration model and the verification models. Therefore the model is appropriate to assess development within the floodplain.
- Development Case — represents the proposed development with flood mitigation works. This development case includes the cut and fill plan as supplied by Northeast Business Park Pty Ltd (Drawing 0304 SK36, issue SD04, dated 30 July 2007 Ref 20430-10D).

The preferred mitigation case consists of:

- north by-pass channel — cut to 1.5 m AHD, grass managed
- Raft Creek — cut to 2.0 m AHD, grass managed
- south by-pass channel — cut to 1.5 m AHD, grass managed
- six earth diversion banks — three near the marina, two on the eastern boundary, one in the north-western section.

It is estimated that the total earthworks (as cut) for the by-pass channels in the preferred mitigation scenario 699,000 m^3. This does not include the six earth diversion banks as design of these structures will be undertaken during the detailed design phase.

The preferred mitigation case shows overall reductions in the peak water levels for the 100 year ARI events across the flood plain. This is due to the flood mitigation works that increase the conveyance through the development site and therefore reduce the flood conveyance through the northern section of the lower Caboolture River floodplain (north of the Caboolture River).

The changes in the flow velocities within Caboolture River due to the flood mitigation works are insignificant when compared to the base case velocities. As expected the navigation channel has the most impact on river velocities.

Overall the proposed works represent a net benefit for the community in terms of flooding. The peak flood levels will be lowered in much of the surrounding flood plain with localised peak flood level increases occurring only within the site boundary or in locations where existing infrastructure will not be impacted.

There is an increase in floodplain storage within the development boundaries in the order of 1.4 million m^3.

The following recommendations are made:

- the preferred mitigation strategies be adopted to minimise afflux associated with the proposed development in accordance with CSC's requirements
- the detailed design of any structures (bridges, culverts, etc) that are proposed within the floodplain (over, under, or through) will need to be appropriately modelled to assess the impacts on flood levels
- the maintenance of the grass managed areas is essential to the flood mitigation proposed in this study. These areas must be designed such that the vegetation/land cover/land use relate to a Manning's n roughness value of 0.035. Deviations from this value may need to be remodelled
- structural input is recommended for the design of the earth diversion banks to avoid 'washouts' and therefore compromise the flood mitigation proposed.

1. Introduction

Northeast Business Park Pty Ltd is proposing to develop a 326 ha multiuse precinct on 760 ha of privately owned land located at Nolan Drive, Morayfield. This degraded site is a former pine plantation on the southern banks of the Caboolture River near Burpengary. The development will have a marine industry and business focus and provide new public access to the riverfront.

The Northeast Business Park is located immediately downstream of the Bruce Highway and is within the study area of the 1994 Flood Study ("Caboolture Flood Study comprising Caboolture River, King John Creek, Lagoon Creek", prepared by Australian Water Engineering (AWE), April 1994). AWE investigations indicated that the flood levels for the upstream end of the site is 7.88 m AHD (Bruce Highway Bridge) down to 2.47 m AHD at the confluence of King John Creek and the Caboolture River.

Figure 1-1 shows the proposed development locality and boundary.

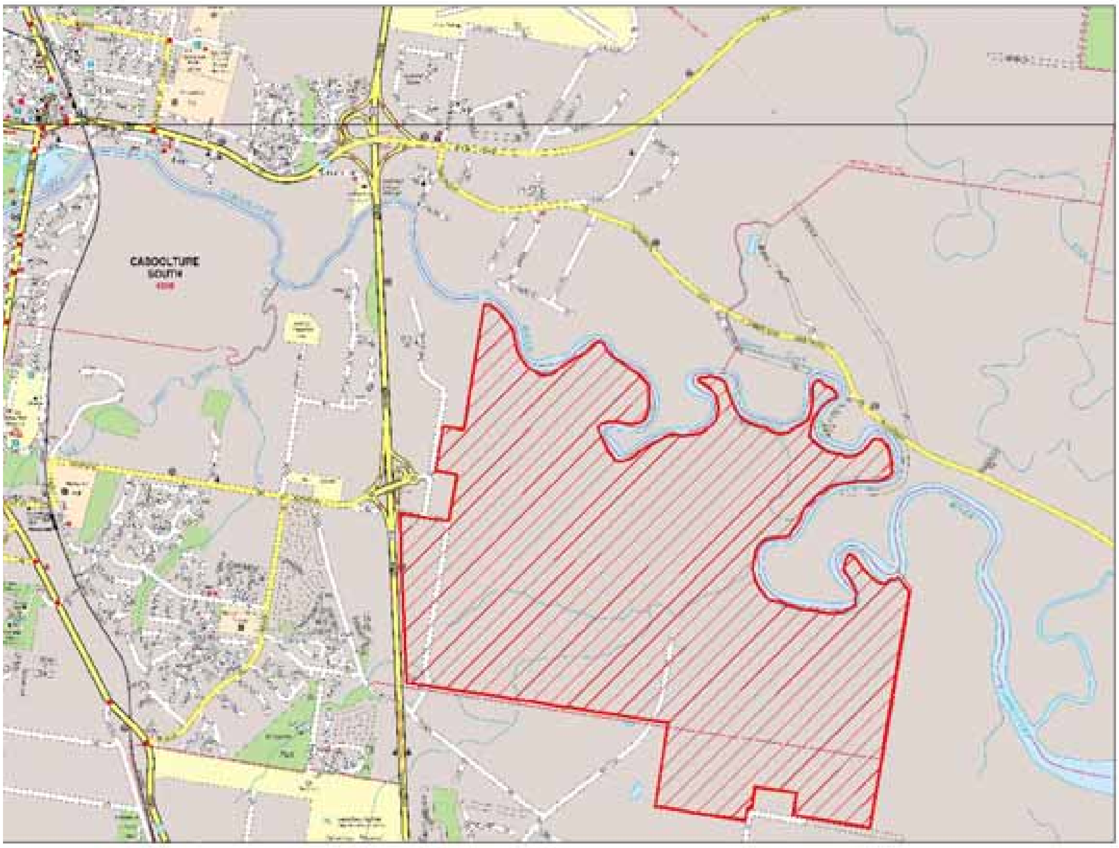


Figure 1-1: Location of Northeast Business Park

1.1 Study objectives

The primary objectives of this report are as follows:

- to provide Northeast Business Park Pty Ltd with advice showing the potential impact of the proposed earthworks plan over the development site, subject to Council's requirements of no adverse impact over adjoining properties
- to provide Northeast Business Park Pty Ltd with recommendations for any further flood mitigation strategies required to meet Council's requirements.

1.2 Background

The terms of reference for this Flood Study were set at a meeting on the 10 August 2005 attended by Trefor Jones and Leanne Salter of the Caboolture Shire Council (CSC), Northeast Business Park Pty Ltd and Parsons Brinckerhoff (PB). At this meeting the flood plain management policy and the stormwater quality requirements were discussed. Prior to the modelling work being undertaken, CSC was consulted to ensure that the flood model met their requirements.

The previous 1994 flood modelling by AWE has provided an acceptable basis for the determination of broad scale flood level prediction and broad scale flood inundation mapping. However, the schematisation of the AWE EXTRAN model of the Caboolture River downstream of Captain Whish Bridge illustrates the complexity of the flood flow patterns expected in the area (Figure 1-2).

One-dimensional (1D) (quasi-2D) models such as the AWE model require all flow paths to be pre-determined at model setup stage, thus requiring assumptions of expected flood behaviour over a range of flow magnitudes. In these models the floodplain is represented as a series of connected 1D links. Each 1D link is defined by a series of cross section spaced at intervals along the link. The accuracy of the model is governed by how well the cross section represents the shape of the waterway and how well the links represent the flow paths. As this site is relatively flat and flow paths are not clearly defined a 2D model is expected to provide a more accurate representation of the floodplain. Therefore this study adopted a two-dimensional (2D) flood modelling approach utilising MIKE21 (developed by the Danish Hydraulics Institute).

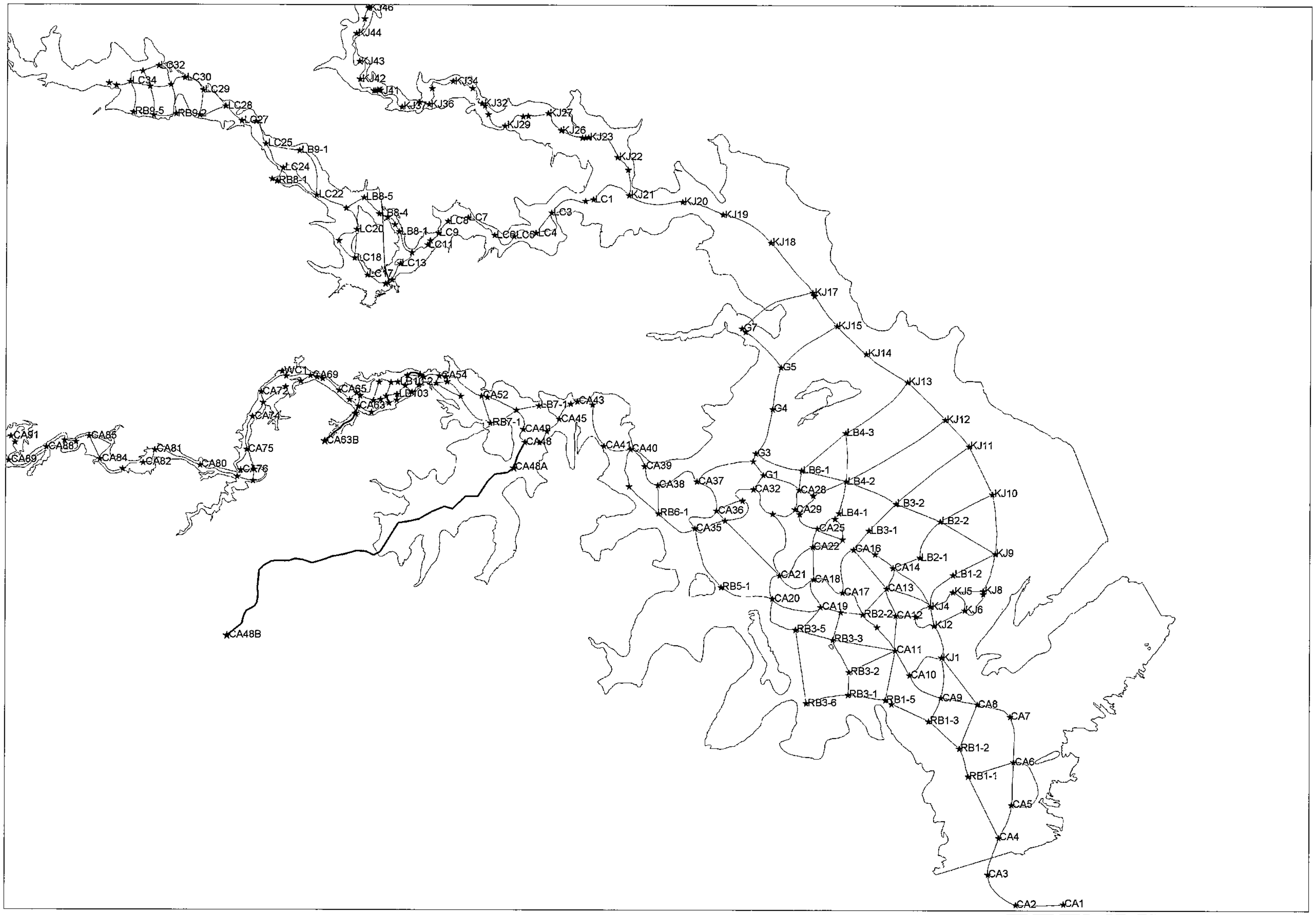
Discussions with CSC indicated that the 1994 Flood Study is the current flood model for use in Council's planning procedures. As such, there should be good correlation between the 1994 model and the MIKE21 model. Any significant differences between the models would need to be explained to a reasonable standard.

Trefor Jones of CSC was contacted on the 13 September to confirm that freshwater is the dominant flow at the development site. The initial tidal boundary condition for all model scenarios was set at the Mean High Water Springs (MHWS), which is 0.81 m AHD. The flood study methodology was provided to Council on 4 June 2007 outlining the adopted tidal boundary and the process the flood study would follow. The adoption of the MHWS tidal boundary provided a more conservative representation of the tidal conditions as the MHWS is the long term average of the heights of two successive high waters when the range of tide is greatest, at full and new moon. This was the basis of the October 2007 flood study (2138171B-RPT001-B:ag) that formed part of the planning application MCU-2002-1079 and MCU-2004-1420.

CSC supplied comments regarding the October 2007 flood study in December 2007. These comments were based on an independent review of the flood study.

The review comments required refinement of the calibration and verification models, in particular the 1972 event, and the inclusion of a tidal boundary similar to that adopted in the 1994 AWE flood study. The tidal boundary (as reported in this report) is a sinusoidal tide peaking at 2.3 m AHD with a 12 hour period. This is descried in Sections 3.1 and 4.3 and represents a 1 in 100 year ARI tidal event. The tidal boundary provides a better representation of the floodplain for the calibration and verification models and therefore was adopted for the base case and development scenarios. Recorded tidal levels were used for the calibration and verification models.

All issues raised by the independent reviewer have been addressed in this flood report (2138171B-D:ag, May 2008).



PRELIMINARY ISSUE

REV	DESCRIPTION	DATE	APPROVED
A	PRELIMINARY ISSUE		

Scale	A3 ORIGINAL

		Date
Drawn	A.G.	08/08
Designed	A.G	08/08
Drawing Check		
Design Check		
Project Approval		

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Client	Job No
NORTHEAST BUSINESS PARK PTY LTD.	2138171B

Project	Drawing No
NORTHEAST BUSINESS PARK AWE MODEL SETUP	Figure 1.2

Cad File	Rev
Figure 1.2.dwg	B

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1.3 Previous investigations

Australian Water Engineering (AWE) previously undertook flood plain modelling of the Caboolture River catchment for the CSC in April 1994. The AWE report entitled 'Caboolture Flood Study' comprising the Caboolture River, King John Creek and Lagoon Creek' details the investigations associated with that study. That investigation and key results are summarised as follows.

- A hydrologic model of the entire Caboolture River catchment was developed using the RAFTS software package. Inflow hydrographs for the catchment determined by the AWE investigations were used in this study.
- A hydraulic model of the Caboolture River floodplain, including King John Creek and Lagoon Creek was constructed using the EXTRAN software package. An estimation of the flood behaviour throughout the catchment was investigated using this model.
- The 1-D model was calibrated using three historical events: February 1972, April 1989 and December 1991. The calibrations of the hydrologic and hydraulic models were satisfactory and were generally able to reproduce the observed discharges and flood levels with acceptable levels of accuracy. However, the 1D model does not take into account lateral variations, which are expected to be significant over the study site.
- The effect of high ocean levels in Moreton Bay is generally limited to the lower 5 km or 6 km long floodplain reach upstream of the mouth of the Caboolture River. Upstream of these lower reaches, flooding is due to stormwater runoff rather than high tides.
- The flood inundation maps produced indicated that extensive areas downstream of the Bruce Highway will be inundated by floodwaters during the 10 year, 50 year and 100 year ARI flood events indicating that the location of Northeast Business Park will need a detailed flood report as part of the planning application.

1.4 Caboolture Shire Plan

The Design and Development Manual (Part A - Roadworks and Stormwater Drainage) — Draft, April 2005 — sets out the criteria for submission of operational works drawings required by Council. The document aims to give supplementary information to the CSC Planning Scheme, and therefore is focused on infrastructure development rather than flood studies. However, the document refers to flood models and/or floodplains as follows.

Section 8.9 Minimum Flood Immunity Levels (see Table 1-1) contains the following information.

Table 1-1: Minimum flood immunity levels from CSC Design and Development Manual

Location	Minimum Design Allotment Levels for Urban Zones or Level of Flood Free Area in Rural and Rural Residential Zones
Adjacent to River, Creek or Waterway	Calculated 100 year ARI ultimate flood levels + 300 mm freeboard
Adjacent to Engineered Channels	Calculated 100 year ARI ultimate flood levels + 300 mm freeboard
In areas affected by tidal water	Adopted 100 year ARI storm surge level + 300 mm freeboard (the adopted 100 year ARI storm surge is 2.3 m AHD. This value incorporates greenhouse effects)
Adjacent to roads and overland flow paths	Calculated 100 year ARI ultimate flood levels + 50 mm freeboard

The minimum flood immunity level for the proposed developed areas will therefore be the 100 year flood level plus 300 mm.

Section 8.17 Open Channels states that the requirements of Queensland Urban Drainage Manual (QUDM) Section 8 shall apply. In addition to QUDM, the following criteria shall also apply:

"All hydrologic and hydraulic calculations for the purpose of determining ultimate flood levels and development fill and flood levels shall be based on the 100 year ARI flows for a fully developed catchment and a fully vegetated waterways corridor using minimum Manning's n of 0.15, unless otherwise approved by Council." The adopted roughness values are discussed in Section 4.2.

1.5 Caboolture Shire Council Flood Plain Management Policy 803/02

This document details the policy for managing re-zoning or sub-division applications.

For residential zones the document states:

- alteration of site contours, including filling, may be undertaken subject to no net loss of flood storage across the subject land for all storm events up to and including the 1 in 100 year event
- the determination of flood storage is to be by computer model based on pre and post development field contour surveys.

For rural zones the document states:

- subdivision of floodable land will only be approved for rural zoned properties where each of the proposed parcels of land has an area of land in its natural state prior to any earthworks being carried out which satisfies additional criteria (refer to Appendix A).

For zones other than Residential, Rural Residential or Rural, the document states:

- subdivision applications will be considered on the circumstances of the individual proposals. Such proposals are subject to additional criteria (refer to Appendix A).

2. Existing environment

The site is adjacent to the Caboolture River estuary and large parts of the site are located within the floodplain. Tidal and freshwater wetlands occur throughout the lower areas of the site. One natural waterway traverses the site, along with several constructed channels.

Vegetation has been largely cleared from the terrestrial areas. The site was last used as a softwood plantation and prior to that was variously grazed and cropped, including sugar cane (4Site Natural Solutions, 2004).

Natural vegetation generally occurs in the low lying areas of the site, including drainage lines, freshwater swamps, tidal creeks and the banks of the Caboolture River.

Soils generally have a sandy loam surface, and across the site fall into three categories — red massive, deep yellow massive and deep grey poorly drained soils. They vary from well drained to poorly drained, and parts of the site have also been identified as being subject to potential acid sulfate soils (4Site Natural Solutions, 2004). This is discussed in further detail in the Geological Report undertaken by J.E. Siemon (September 2005).

2.1 Topography

The site slopes north-east from the Bruce Highway towards the Caboolture River which forms the northern site boundary. Ground levels vary between 1.5 and 5.0 m Australian Height Datum (AHD) and small hills rise up to 14 m and 17.5 m AHD along the southern and western boundaries.

Within the site is one natural waterway (Raft Creek) and several constructed channels. Raft Creek enters the site approximately 600 m to the east of the south-western site corner and flows in a northeast direction towards the Caboolture River (4Site Natural Solutions, 2004). A large constructed channel traverses the site in an east-northeast direction to flow into the Caboolture River. This channel begins in an adjoining property past the western border.

Stormwater runoff generally flows to the waterways on site where it is directed to the Caboolture River. Significant catchment areas external to the development boundary convey overland stormwater flows through the site to the Caboolture River. Due to the relatively flat topography low lying areas on the southern part of the site are poorly drained with minor ponding of water occurring after significant rainfall events (4Site Natural Solutions, 2004).

Low lying areas adjacent to the Caboolture River are inundated during high tides. This has been highlighted by the presence of marine vegetation within these areas, comprising tidal mangroves and salt marsh communities.

3. Methodology

2D modelling allows the entire topography of the floodplain to be described and modelled. The flow paths do not need to be predefined, because the model determines the flow distributions based on water levels and ground levels at each time step in the model run. 2D modelling therefore provides a more accurate determination of the extent, magnitude and direction of flood flows and impacts on flood associated with development of the site.

In summary, the methodology adopted for this study was as follows:

- prepare base case MIKE21 model:
  - develop base case topographical model
  - incorporate roads and Council's river cross sections (bathymetry) into topographical model
  - prepare roughness model based on aerial photography
- calibrate and verify MIKE21 model against recorded historical events (1972, 1989 and 1991)
- run base case model for 100 year ARI event, based on hydrology extracted from 1994 AWE flood model
- determine critical 100 year ARI flood level envelope , based on combined flood inflows plus downstream tidal surge level
- incorporate proposed cut and fill option into development case
- run development case for the 100 year ARI
- compare flood levels before and after development
- prepare flood mitigation cases and re-run development case to check that no adverse flood impact are generated on adjacent properties
- assess sensitivity of the model to changes in roughness values.

3.1 Boundary conditions

The inflows used in the 2D model were extracted from the AWE EXTRAN model, the locations of which are described in the next section.

The downstream boundary condition for the design case was derived from the CSC report (AWE, Section 4.4.4). The adopted 100 year ARI tidal boundary is a sinusoidal tide peaking at 2.3 m AHD with a with a 12 hour period. This includes a 0.3 m rise in ocean water level for climate change.

For calibration and verification cases, the downstream tidal boundary was extracted from the EXTRAN model at the appropriate location and represents the recorded tide level.

3.2 Tools

The following tools were used to develop the flood model:

- XPSWMM and XPRafts to extract the 1994 flood model data
- Acad – the master plan was provided in this format and the report figures were generated in this format
- 12D – the bathymetry, terrain, and MIKE21 grid were all generated using 12D and then exported to x y z format. The model results for the earthworks calculations were provided as 12D models
- DHI software – MIKE21 processor, toolbox programs
- PB 'in-house' DHI programs – suite of tools developed for pre and post processing MIKE21 models.

3.3 Post-processing

Post processing was undertaken using a suite of in-house tools specifically generated to extract results from MIKE21. These are based on the Mike Zero and MIKE21 toolkit programs, however can be executed outside the DHI user interface. The following are all generated as part of these programs:

- water surface levels
- water depths
- velocities
- Froude numbers
- Courant Friedrich Levy ratio
- model noise
- afflux.

Microsoft Excel is also utilised to generate long section plots of:

- water surface levels
- inflow hydrographs
- river profiles.

4. Data used

4.1 Topographical data

The hydraulic model was developed using the following topographical data sources.

General topography — aerial survey presented on a 4.6 m estimated point density from AAMHATCH dated October 2005. Superfluous points not adding to the terrain definition within 0.15 m were removed. This data was also used to provide details of the roads throughout the floodplain. The digital data documentation is contained in Appendix B.

Bathymetric survey — Mapping & Hydrographic Surveys supplied detailed bathymetric survey of the Caboolture River from Beachmere (Caboolture River mouth) to the Caboolture Weir. The survey was undertaken in 2006–2007. This processed data was integrated with the above terrain data and mesh geometry was developed with a grid spacing of 10 m. The grid spacing of 10 m was chosen to provide an acceptable level of model accuracy, whilst also enabling acceptable model run time.

The topography map in Figure 4-1 shows the adopted base case model topography.

4.2 Bed friction data

The bed friction was developed using aerial photos from Studio Tekton (2005 & 2007), CSC (1999–2000) and Department of Natural Resources and Mines' MAPVIEW Aerial Photography, version 2.2.0, build 9 (1997 - 2004). The base values are shown in Table 4-1.

Table 4-1: Base value roughness derived from aerial photography

Land Use	Manning's n	MIKE21 roughness (=1/n)
Main Floodplain	0.08	12.5
River	0.035	28.57
Roads	0.03	33.3
Mangroves	0.16	6.25
Urban area	0.15	6.67
Forest	0.12	8.3
Rougher floodplain	0.09	11.11

The roughness map in Figure 4-2 shows the base value case model friction.

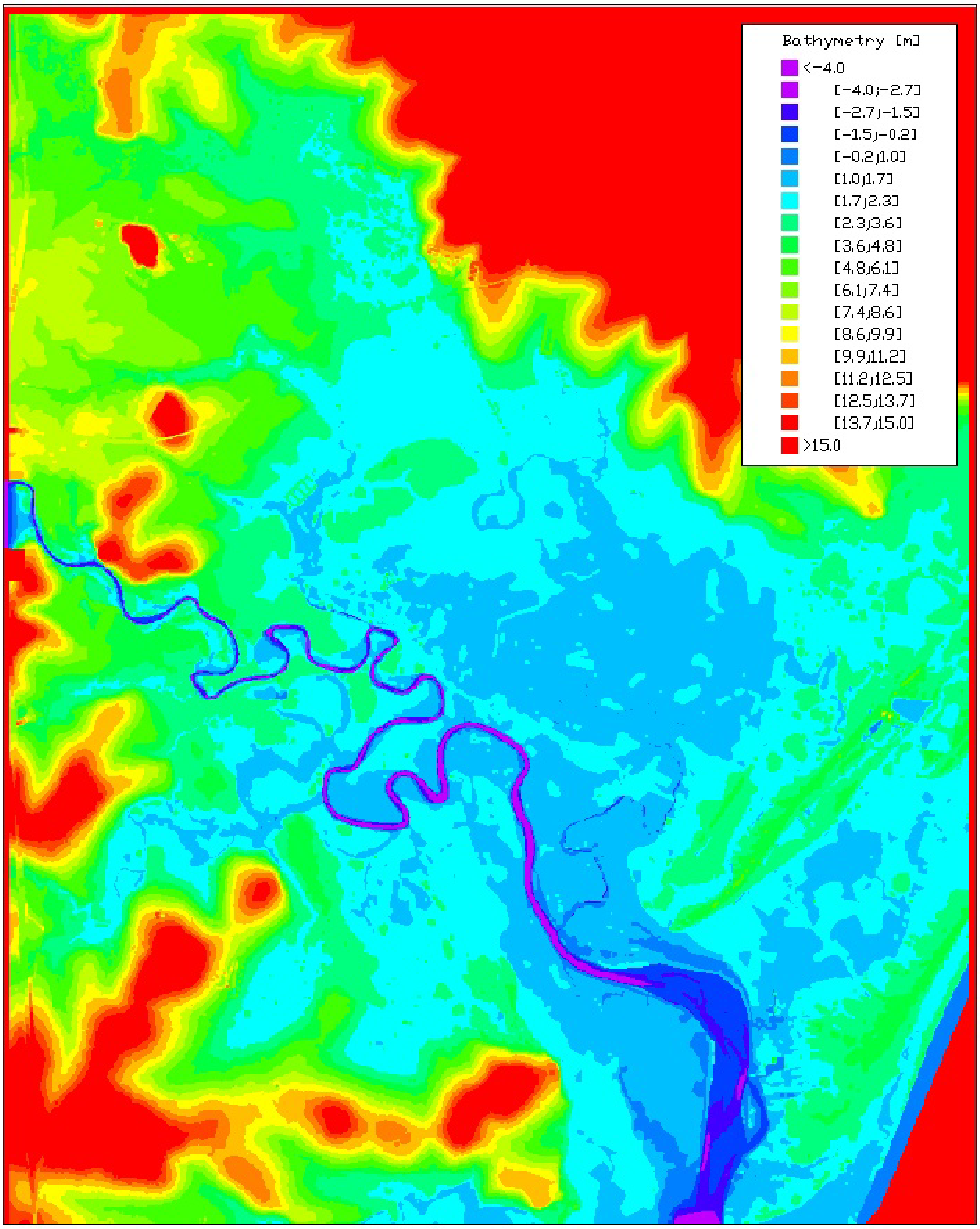

Figure 4-1: Base case topography

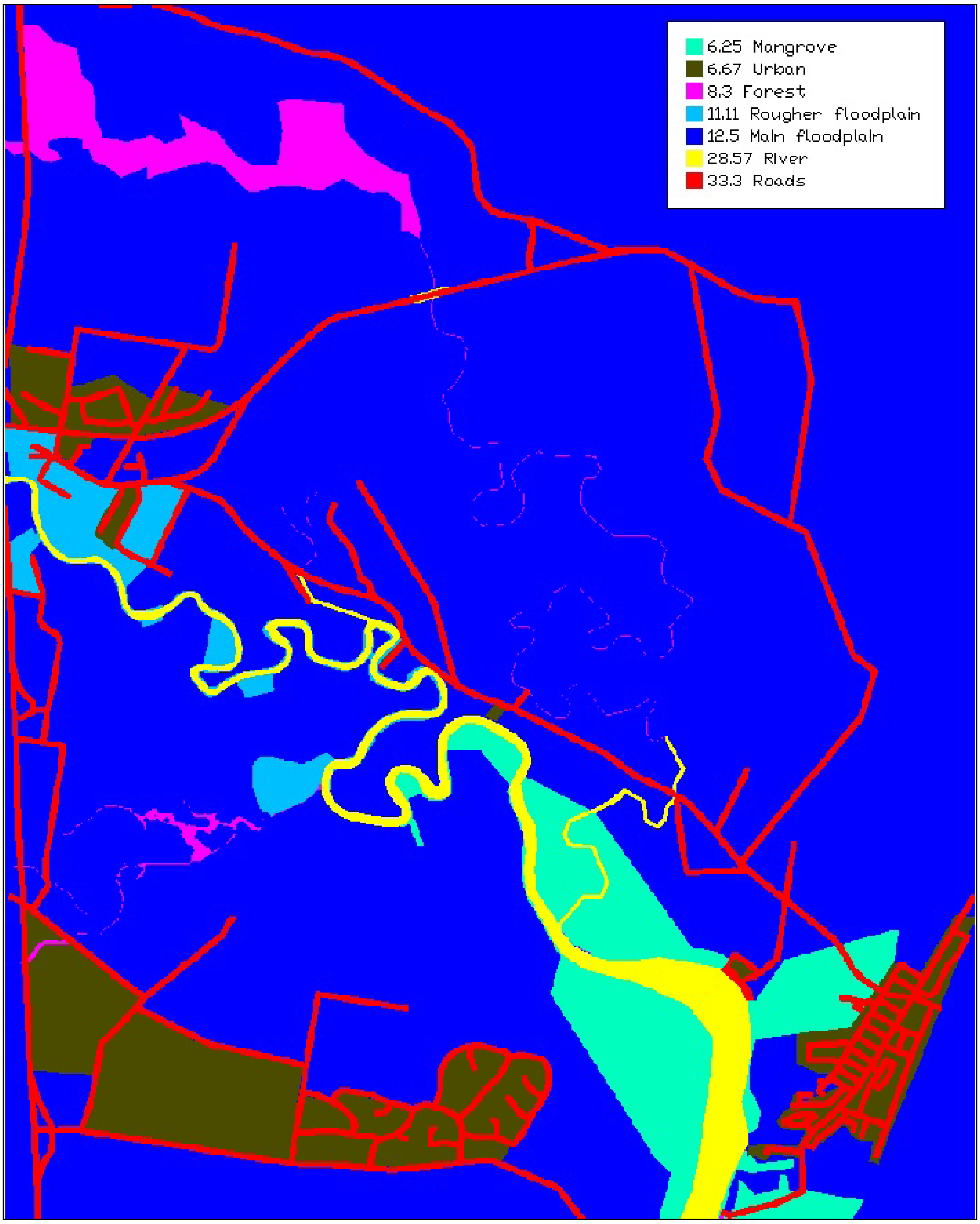

Figure 4-2: Base case friction map

4.3 Boundary conditions

The 1994 AWE hydrological model was used to determine the flow hydrograph in the flood model. The hydrological model was not reviewed or updated. Table 4-2 details the peak inflows used for the 1 in 100 year ARI event and the approximate location in the MIKE21 model grid. Figure 4-3 presents the approximate location of the inflow points within the model and Figure 4-4 presents the flood hydrographs adopted from the 1994 study as inflows for the 1 in 100 year ARI event.

Table 4-2: Peak discharges at model inflow locations

Location	Inflow type	100 yr flow (m^3/s)	Historical Feb. 1972 flow (m^3/s)	Historical Apr. 1989 flow (m^3/s)	Historical Dec. 1991 flow (m^3/s)	MIKE21 grid location (j, k)
CA 43- Caboolture River at Caboolture Township (modelled as a boundary condition)	Boundary condition	1395	1062	863	885	0, 616 - 0, 623
LC1-Lagoon Creek – Upper Catchment (modelled as a source)	Point source	247	197	174	174	7,894
KJ23-King John Creek – Upper Catchment (modelled as a source)	Point source	73	62	37	41	5,980
CA 29	Point source	62	32	25	16	277,671
CA 20	Point source	101	45	49.5	41	89,268
CA 7	Point source	33	14	11	9	673,244
RB6_1	Point source	40	19	17	12	38,426
KJ 19	Point source	60	35	27	16	343,767
KJ 13	Point source	49	26	21	12	496,682
KJ 10	Point source	50	27	21	13	695,476

Note: All flows are extracted from the AWE 1994 flood study

The western boundary of the MIKE21 model was the Bruce Highway. Therefore the upper Caboolture River floodplain was not modelled. However, Lagoon Creek and King John Creek downstream of the Bruce Highway were included in the model domain as boundary conditions. The local inflows were modelled as point sources.

In the calibration cases the downstream boundary was modelled as a time dependant water level. The values were extracted from the water level at node CA4 from the 1994 EXTRAN model. The EXTRAN models included a downstream observed tidal boundary.

The downstream condition for the design and mitigation cases was modelled as a time dependant water level with a period of 12 hours and amplitude of 2.3 m from the mean sea level. This is described in greater detail in Section 6.2.

The initial water surface for all models was set at 0.0 m AHD. This allows the areas in the model that are below 0.0 m AHD to be 'wet' at the start of the model simulation.

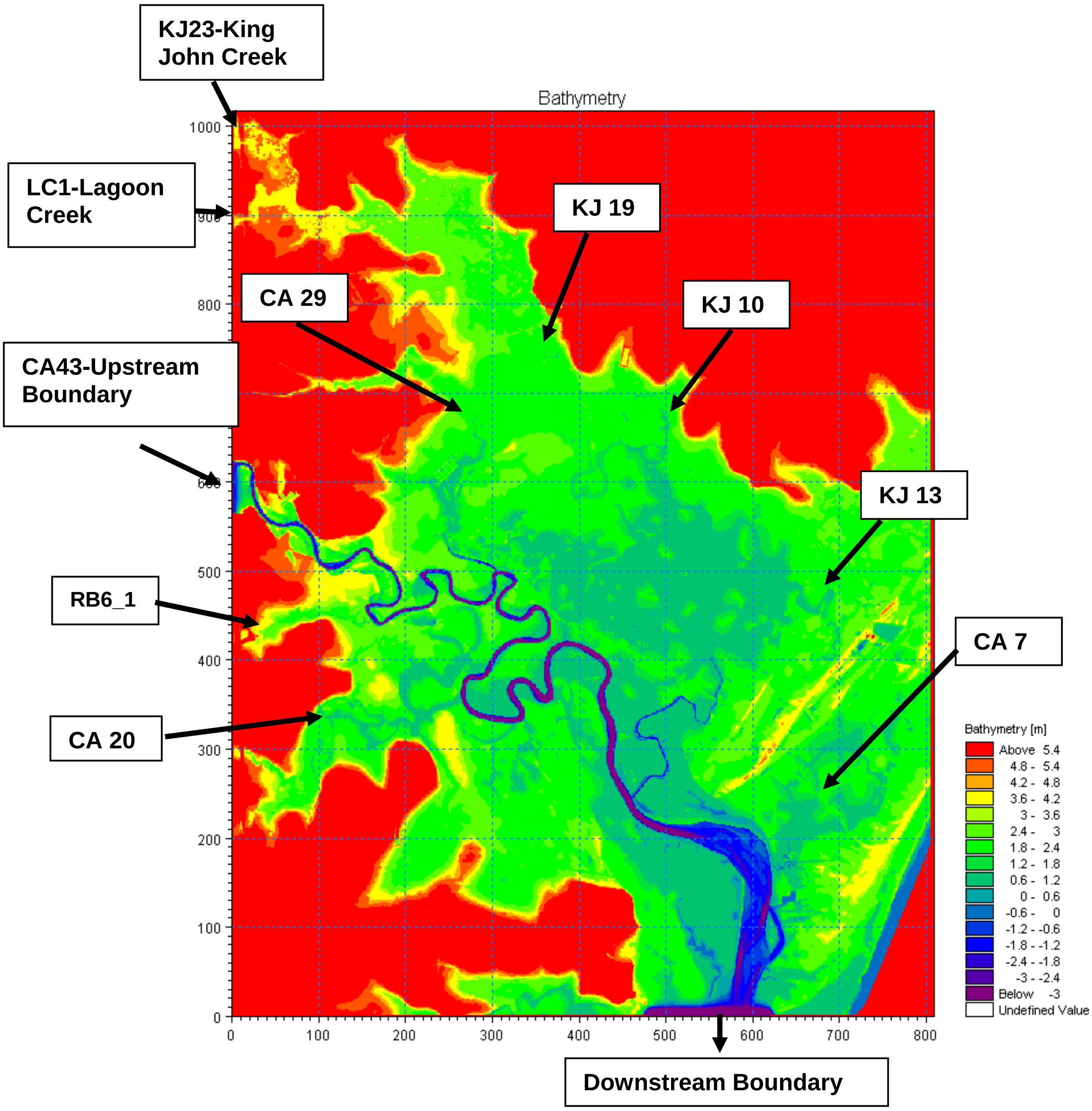

Figure 4-3: Inflows location and title

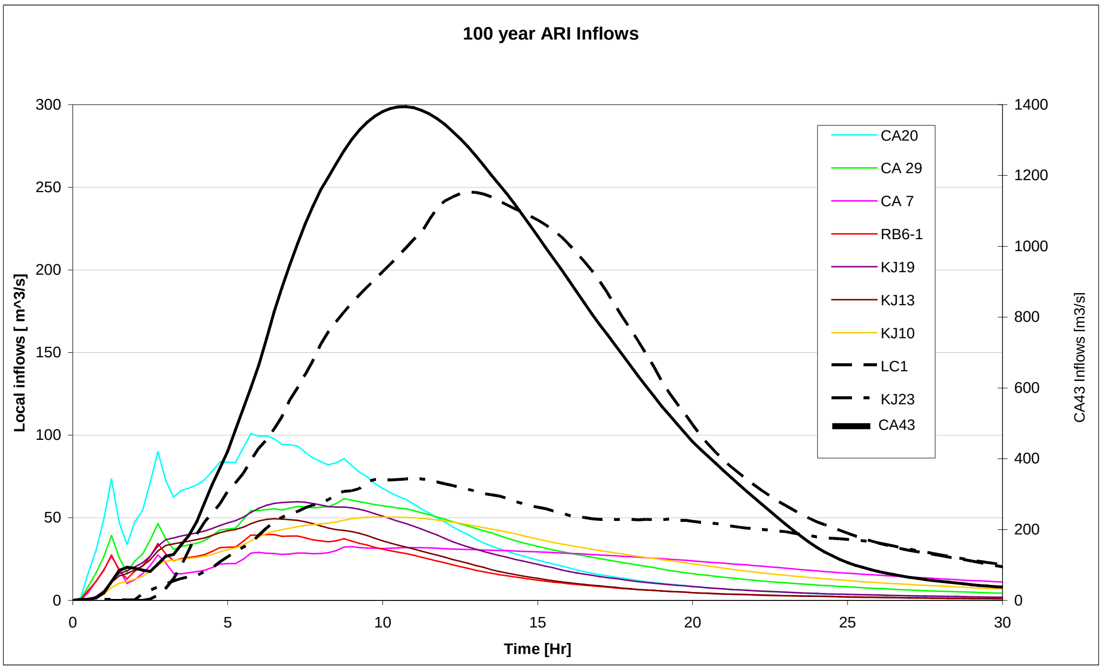

Figure 4-4: Adopted inflow hydrographs for the 100 year ARI design event

5. Model calibration and verification

Model calibration and verification was undertaken using three historical events, as detailed in the 1994 study:

- February 1972 — thought to be in the order of a 15 to 40 year ARI event
- April 1989 — thought to be in the order of a 10 to 20 year ARI event
- December 1991 — thought to be in the order of a 15 to 20 year ARI event

The December 1991 event was used to calibrate the model while the February 1972 and April 1989 events were used to verify the model.

The 1994 hydrological model contained the flow hydrographs of the three events at the upstream end of the flood model. A simulation of each historical flood event was undertaken using these flows with the base case model as described above.

Table 4-2 details the peak inflows used and the approximate location in the MIKE21 model grid.

5.1 Model calibration – 1991 event

The MIKE21 model was calibrated against the recorded flood level of the 1991 event. Roughness values were adjusted and the resultant water surface levels were compared with the recorded data.

The Caboolture River, Lagoon Creek and King John Creek inflows hydrographs for the 1991 event were derived from the 1994 hydrological model and are presented in Figure 5-1.

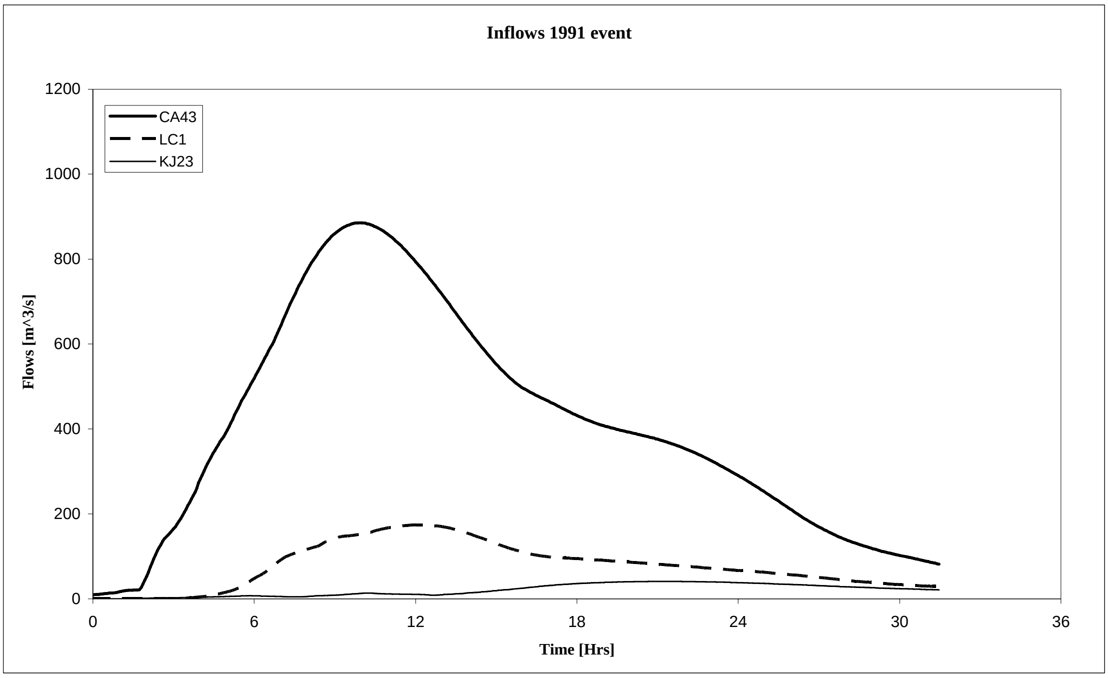

Figure 5-1: Main Inflows hydrograph for the 1991 event

The results of the calibration are presented in Figure 5-2 and represent a long section along the Caboolture River. There is a good fit between the calibration model results and the recorded data. The adopted roughness values derived from the calibration model and subsequently used in the base case modelling are presented in Table 5-1.

Table 5-2 (Section 5.1.5) presents a numerical summary of all the calibration and verification models.

Table 5-1: Base value roughness derived from aerial photography

Land Use	Manning's n	MIKE21 roughness (=1/n)
Caboolture River	0.035	28.57
Roads	0.03	33.33
Floodplain	0.08	12.5
Mangroves	0.16	6.25
Urban area	0.15	6.66

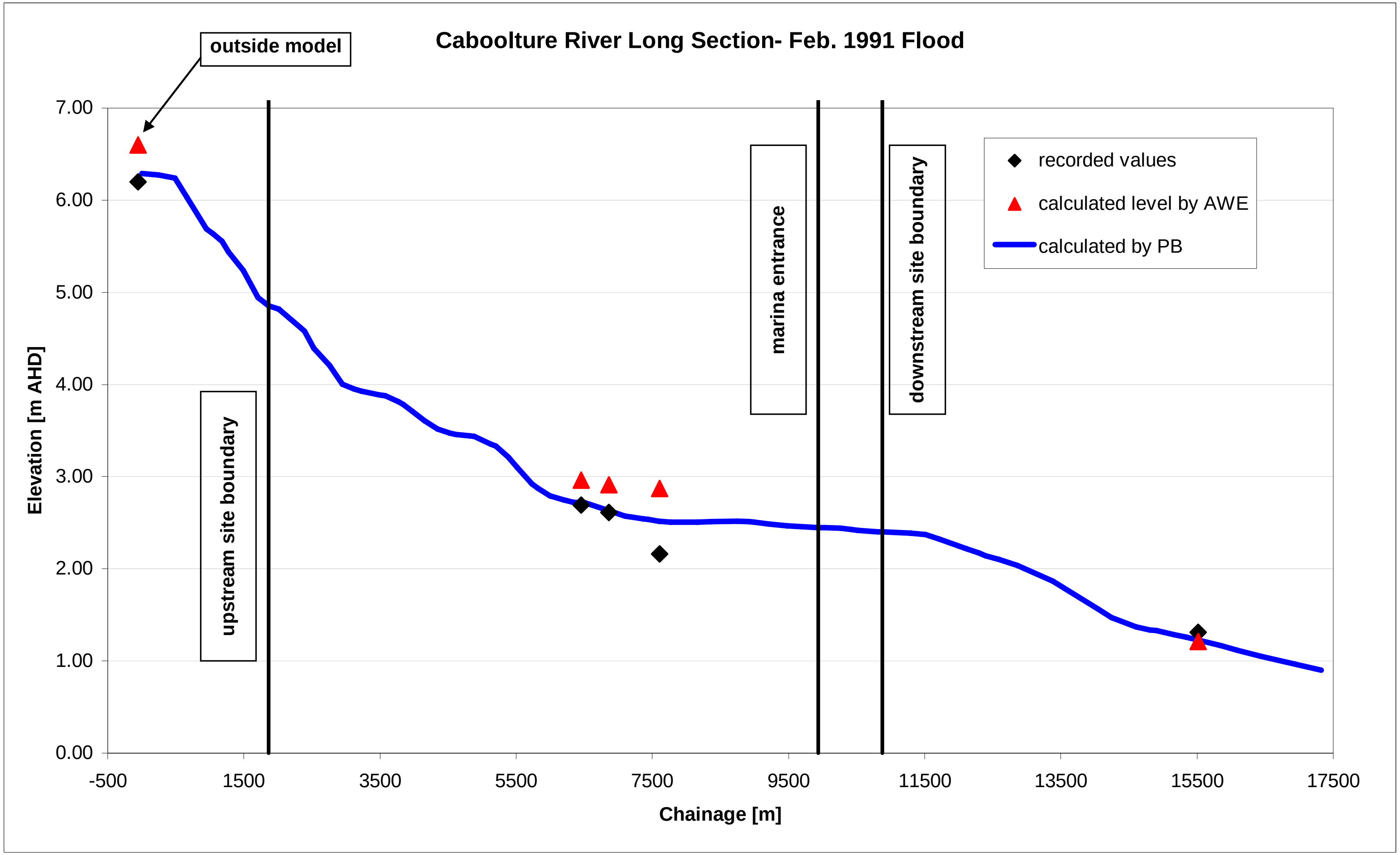

Figure 5-2: Water surface long section for the 1991 event

5.2 Model verification – 1989 event

The Caboolture River, Lagoon Creek and King John creek inflows hydrographs for the 1989 event were extracted from the 1994 hydrological model. These inflows are shown in Figure 5-3.

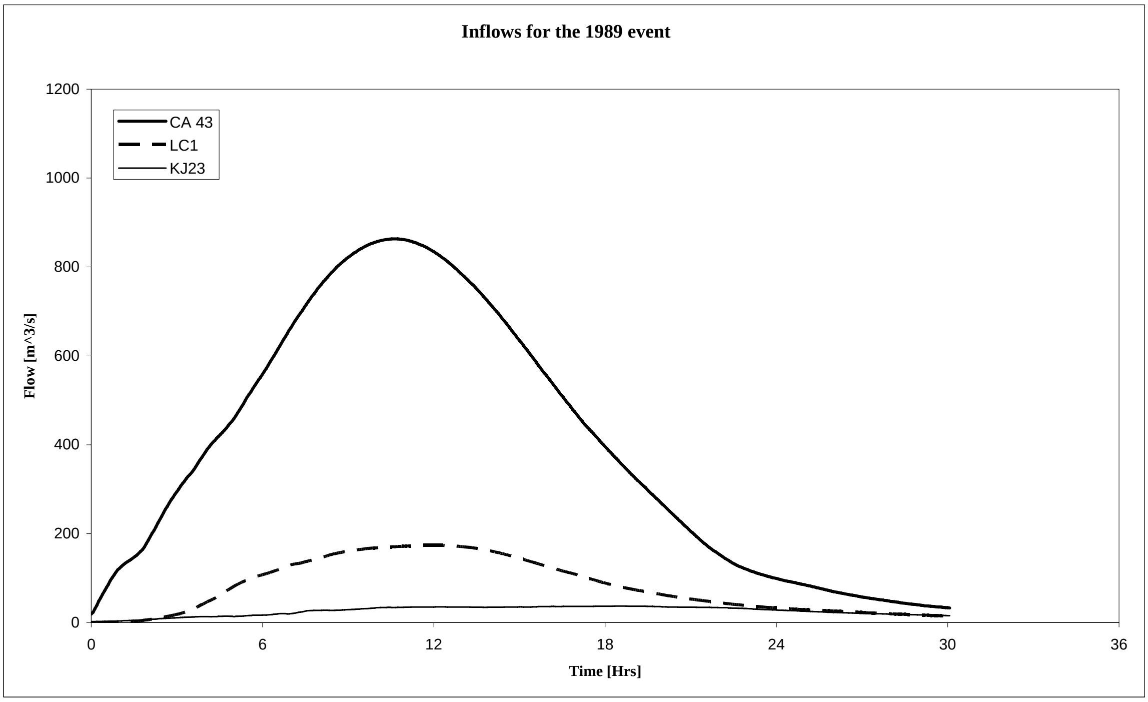

Figure 5-3: Main Inflows hydrographs for the 1989 event

Figure 5-4 presents a long section through the Caboolture River presenting the maximum water surface level for the 1989 event. Table 5-2 (Section 5.1.5) presents the numerical analysis for this event. The modelled results and the recorded levels are very similar.

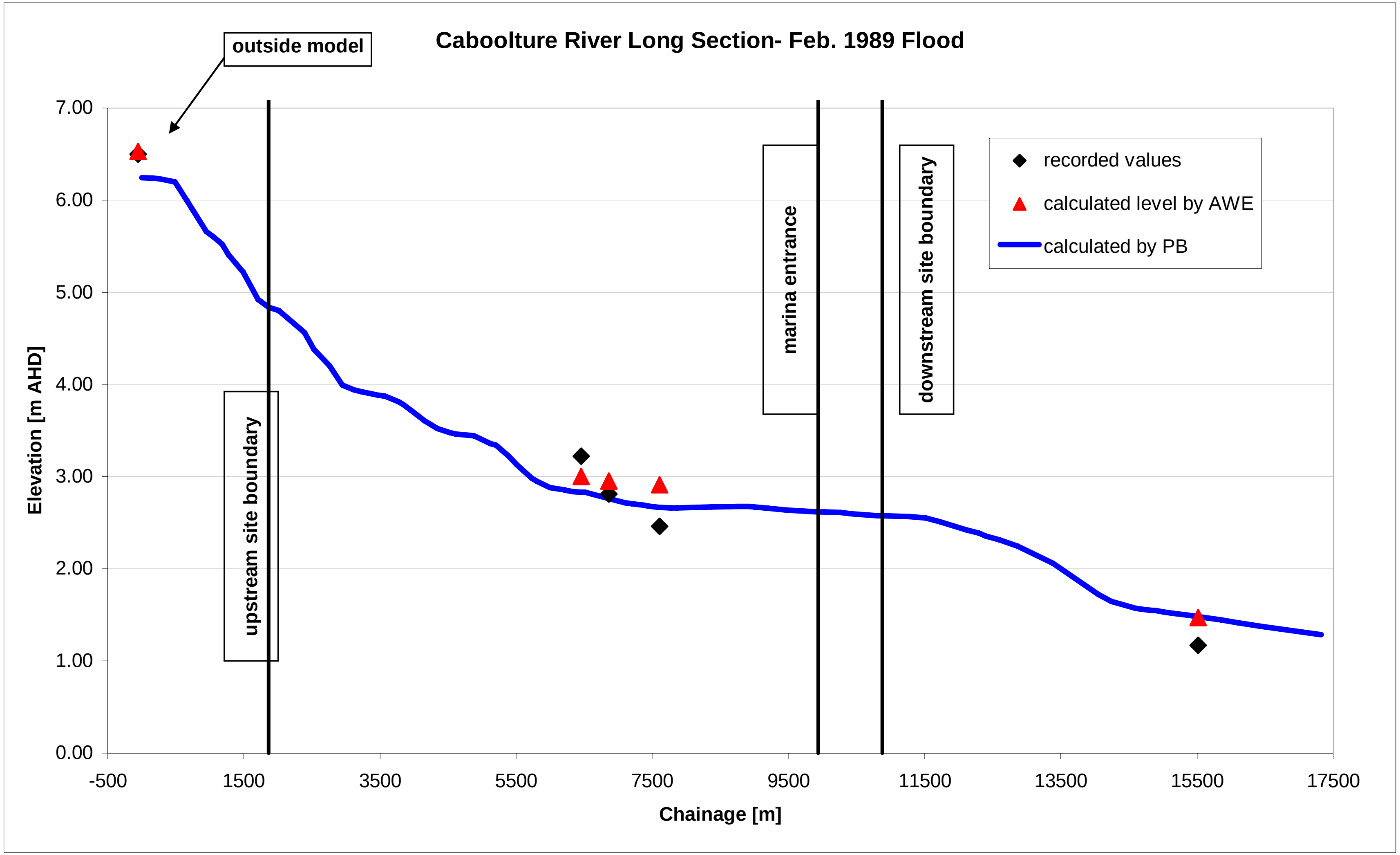

Figure 5-4: Water surface long section for the 1989 event

5.3 Model verification – 1972 event

The Caboolture River, Lagoon Creek and King John creek inflows hydrographs for the 1972 event were extracted from the 1994 hydrological model. These inflows are shown in Figure 5-5.

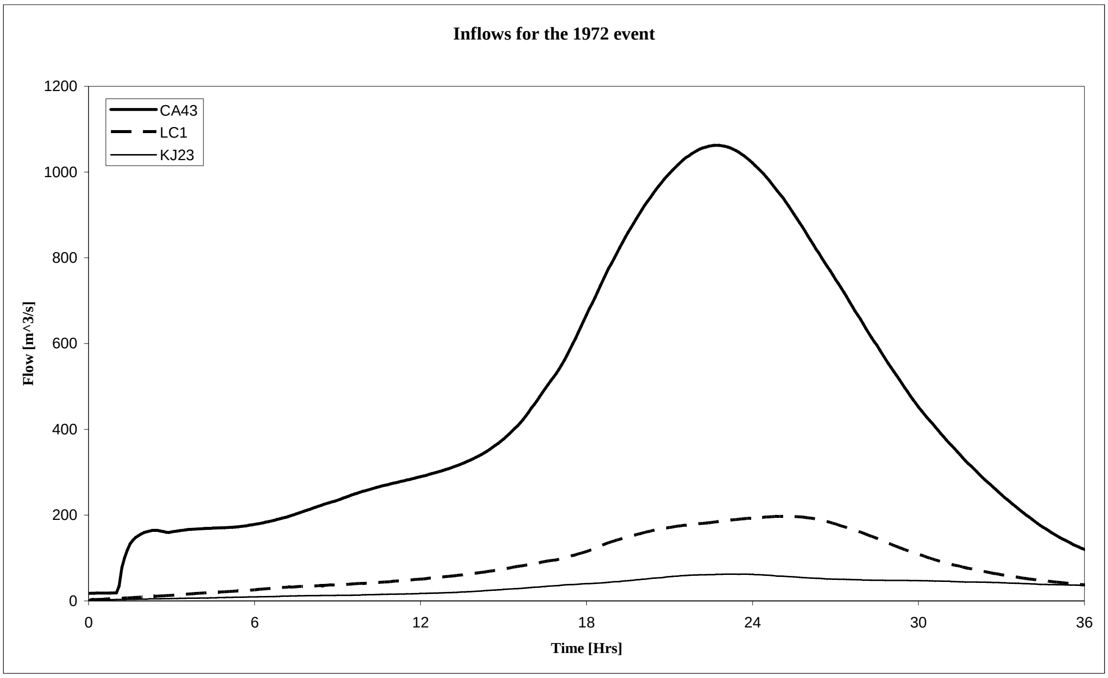

Figure 5-5: Main Inflows hydrographs for the 1972 event

Figure 5-6 presents the long section through the Caboolture River providing the maximum water surface level during that event. Table 5-2 (Section 5.1.5) presents the numerical analysis for the 1972 event.

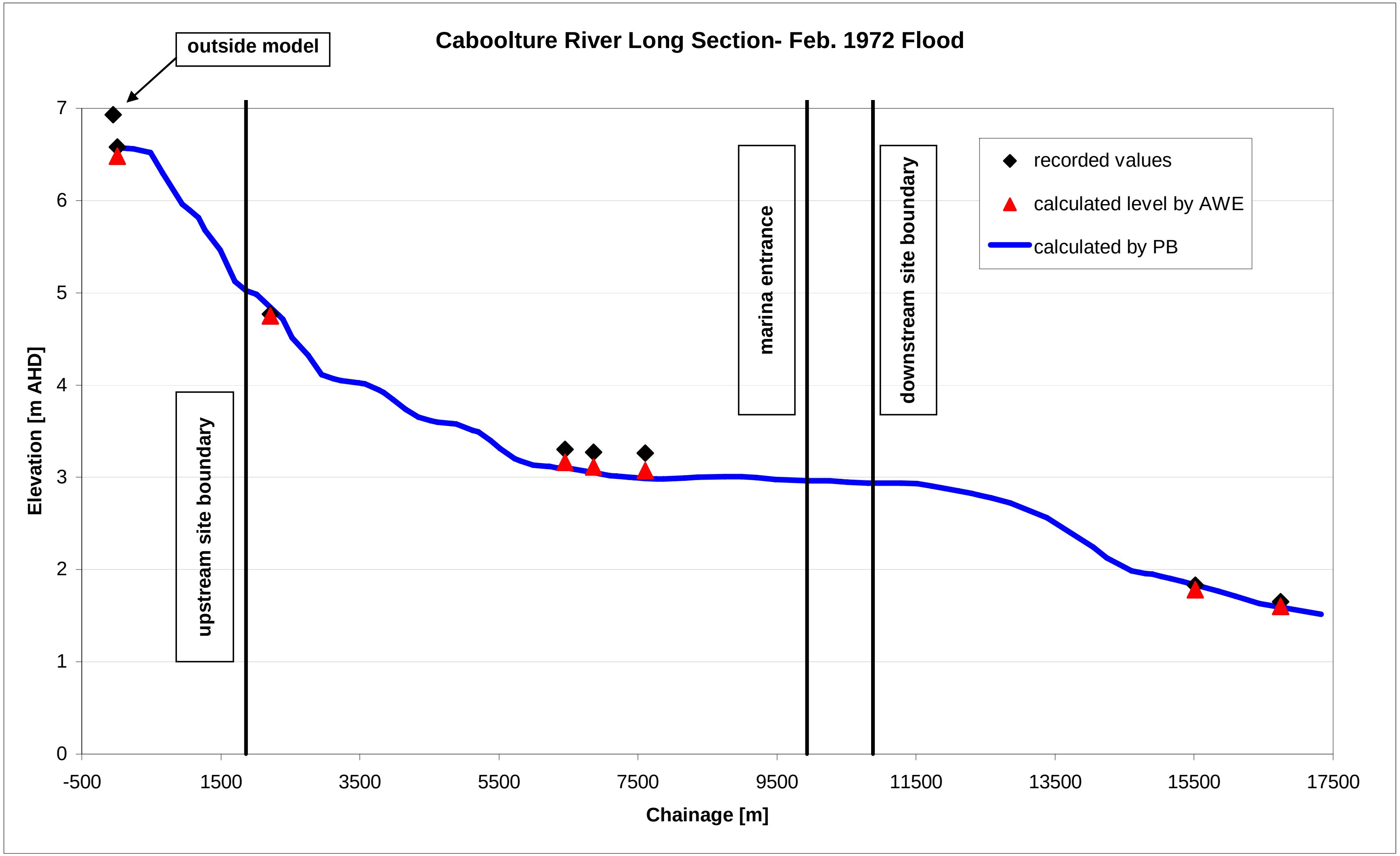

Figure 5-6: Water surface level for the 1972 event

5.4 Recorded data discussion

The discrepancies between modelled and recorded flood data occur for a number of reasons. The field measurements of maximum flood levels are generally taken from flood marks and accumulations of flood debris giving a point estimate of water levels reached during the flood, which could be affected by wave action and temporary blockages, among other factors.

It should be noted that the floodplain has probably changed over time between each of the historic events and the present day, with differences likely in terms of geometry, land usage and vegetation. The models used in this analysis were developed from the latest available topographical data and do not necessarily represent the catchment at the time of the historic event. This will account for some of the discrepancies between modelled and recorded flood levels.

The correlation between recorded and modelled data shown in Table 5-2 and shown in Figure 5-2, Figure 5-4 and Figure 5-6 are considered to be acceptable for modelling a catchment of this size. The MIKE21 base case model gives a good overall reproduction of the February 1972, April 1989 and December 1991 flood events, and as such can be used confidently to optimise the master plan in terms of floodplain management.

5.5 Numerical analyses

Table 5-2 presents the estimated results of the three historical events for the MIKE21 model and the EXTRAN model.

Along the Caboolture River the standard deviation across the three events for the EXTRAN model is about 0.26 m while the standard deviation for the MIKE21 model is about 0.21 m.

The MIKE21 model is therefore statistically slightly more accurate than the EXTRAN model in the estimation of flood levels along the Caboolture River.

Table 5-2: Calibration summary

	EXTRAN node location	CA44 Bruce Highway U/S	CA43 Bruce Highway D/S	CA37/38 Lawrence Street	CA28 Beachmere Goong	CA25/26 Riversleigh Road	CA23/24 Beachmere Monty	CA7 Baker Flat Road	CA5 Whiting Street	Standard deviation of differences
Approximate chainage		N/A	0	2,011	6,332	6,508	7,437	15,329	16,438	
Observed	WSL 1972	6.93	6.58	4.77	3.3	3.27	3.26	1.83	1.65	
	WSL 1989	6.5			3.22	2.81	2.46	1.17		
	WSL 1991	6.2			2.69	2.61	2.16	1.31		
Calculated 1994 (AWE)	WSL 1972	7.12	6.48	4.75	3.16	3.11	3.07	1.78	1.6	
	WSL 1989	6.53			3	2.95	2.91	1.47		
	WSL 1991	6.6			2.96	2.91	2.87	1.21		
Differences 1994 (AWE)	WSL 1972	0.19	-0.1	-0.02	-0.14	-0.16	-0.19	-0.05	-0.05	
	WSL 1989	0.03			-0.22	0.14	0.45	0.3		0.26
	WSL 1991	0.4			0.27	0.3	0.71	-0.1		
Calculated 2008 (PB)	WSL 1972	N/A	6.57	4.76	3.10	3.05	2.99	1.87	1.63	
	WSL 1989	N/A	6.24	4.61	2.83	2.76	2.67	1.50	1.38	
	WSL 1991	N/A	6.29	4.62	2.72	2.63	2.52	1.26	1.05	
Differences 2008 (PB)	WSL 1972		-0.01	-0.01	-0.20	-0.22	-0.27	0.04	-0.02	
	WSL 1989				-0.39	-0.05	0.21	0.33		0.21
	WSL 1991				0.03	0.02	0.36	-0.05		

6. Base case model

The following section contains the model results for the 1 in 100 year ARI event.

6.1 Base case modelling results

The resultant water levels for the base case are shown in Figure 6-1 and range from 2.3 m to 8 m. Flow vectors shown in this figure are indicative of the wide floodplain and demonstrates the spreading of flood water that occurs downstream of Captain Whish bridge. The majority of velocities shown are less than 1.0 m/s; however, within sections of the main Caboolture River velocities exceed 2.0 m/s.

Figure 6-2 presents the base case flood depths. The maximum depth within the floodplain is 4 m. As expected the depth within Caboolture River the depth is greater than 4 m.

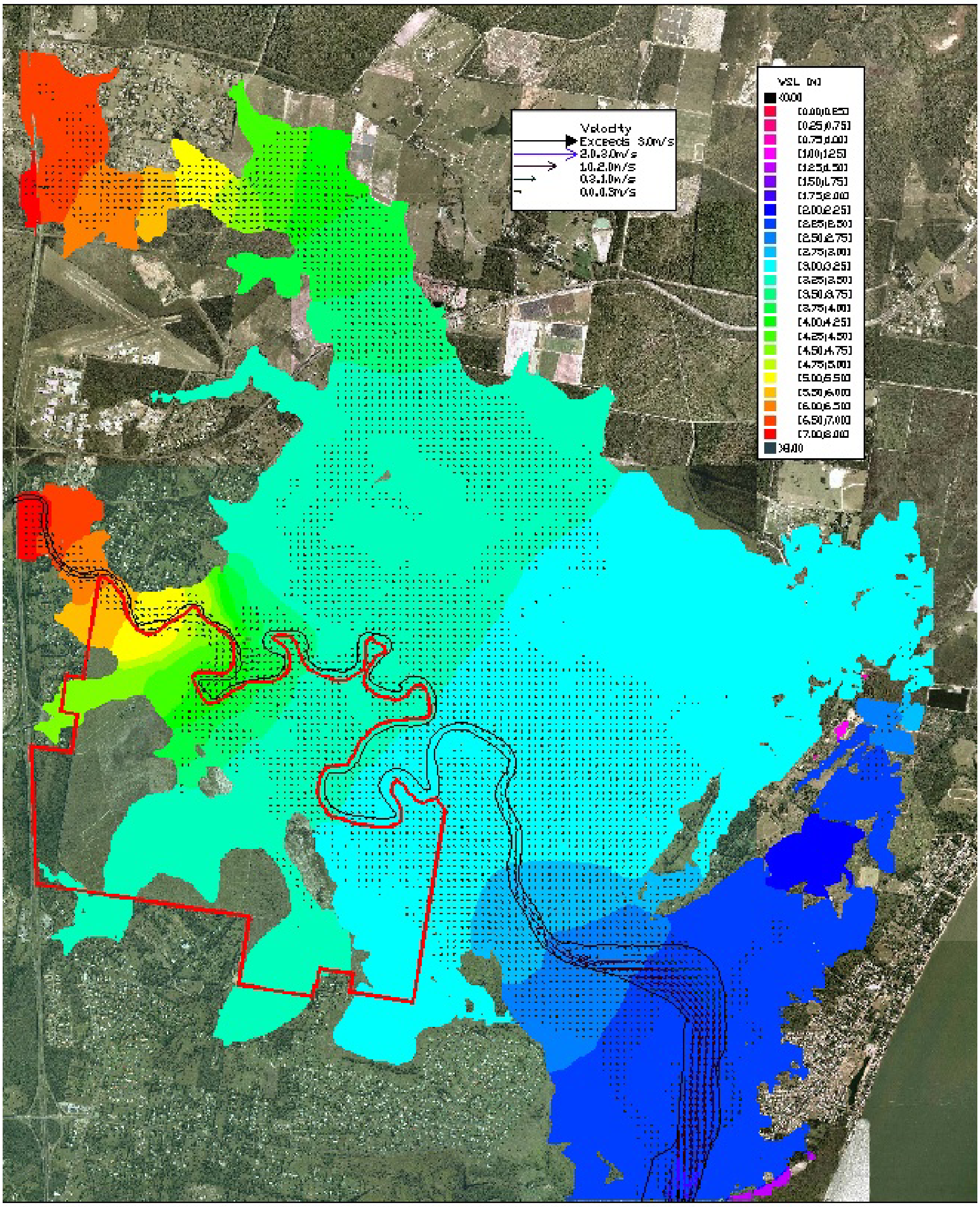

Figure 6-1: 100 year Base case water surface level

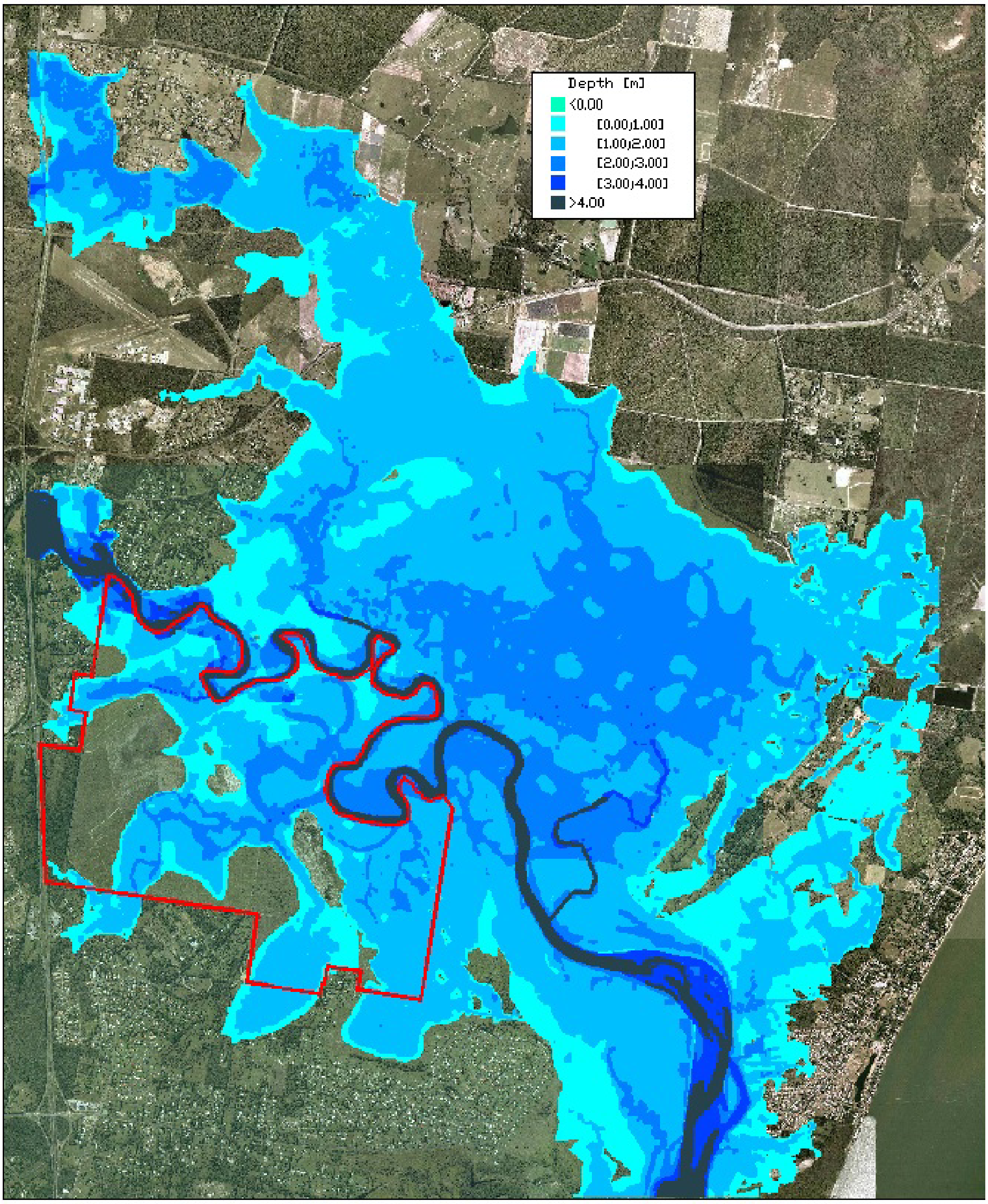

Figure 6-2: 100 year Base case flood depth

6.2 Model sensitivity

To assess the sensitivity of the model, changes in the downstream boundaries condition and roughness values were evaluated.

Figure 6-3 shows the four tidal boundary conditions which were evaluated, the timing of each tidal pattern is offset by three hours from the previous tide timing, therefore producing the four following boundaries: tide-0h ; tide-3h ; tide-6h; tide-9h.

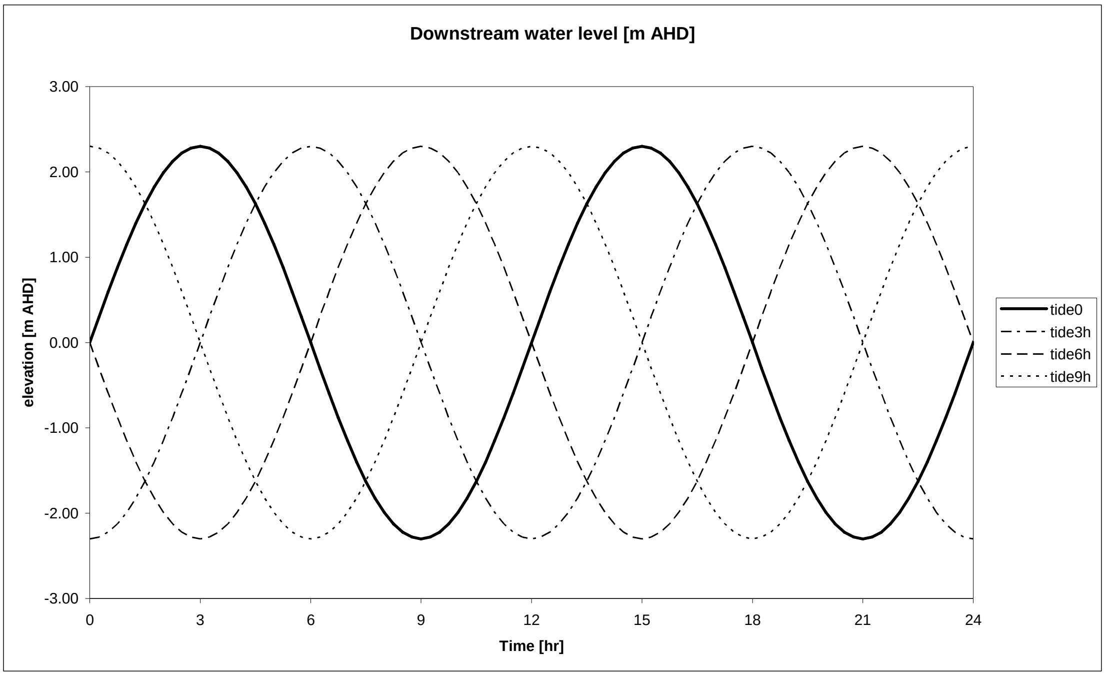

Figure 6-3: Tidal downstream boundaries assessed

Figure 6-4 presents the water surface level long sections for the four downstream tidal boundary cases. Changes to the downstream boundary condition did not make a significant impact on water surface levels at the proposed site. The maximum absolute difference at the development site is less than 0.05 m when comparing the tide-0h to the other tidal boundary conditions.

Therefore Tide-0h is the downstream tidal boundary cycle that has been adopted for the rest of this study as it globally produces the highest water surface level in the model domain.

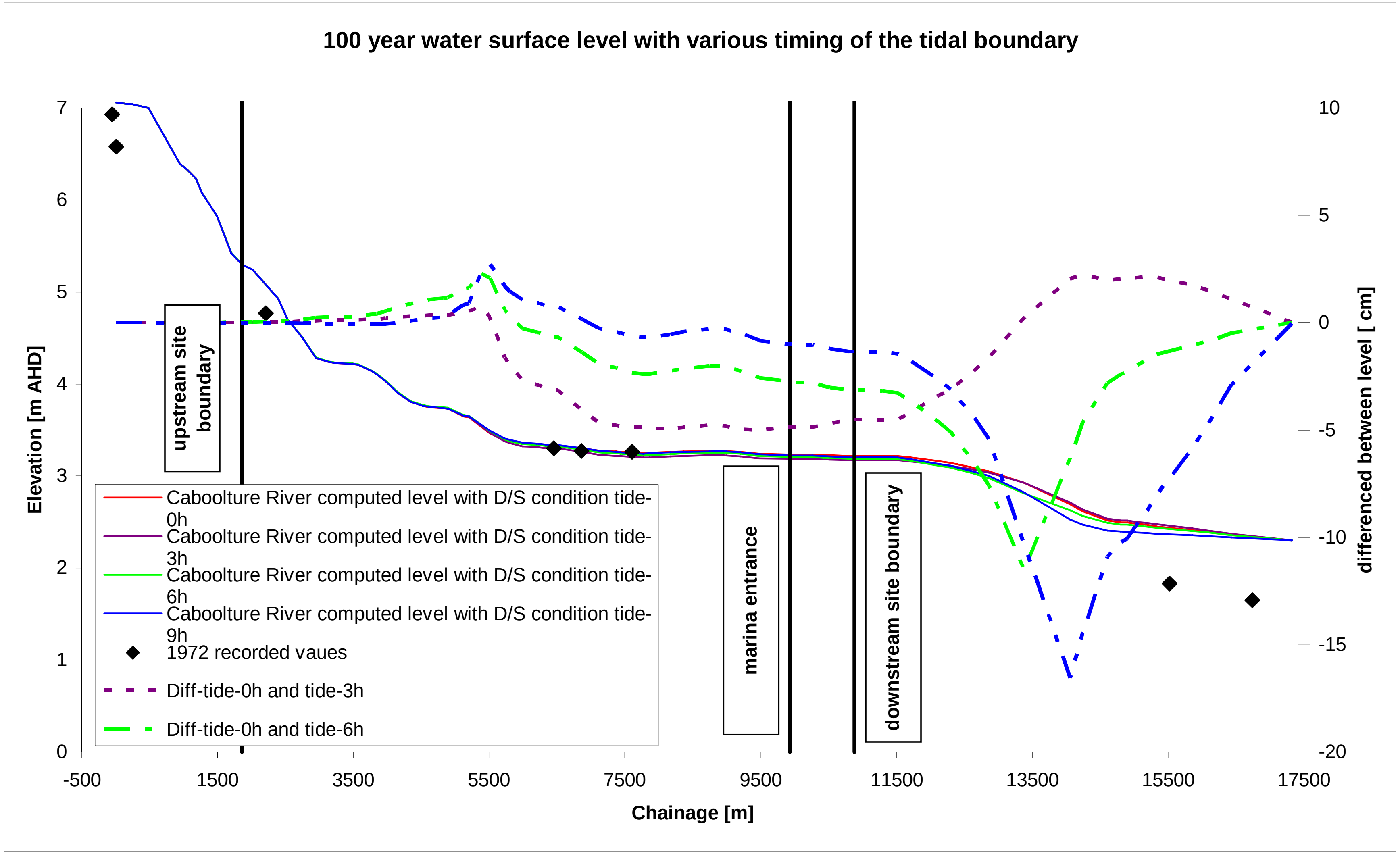

Figure 6-4: Longitudinal profile of water surface elevations due to changes of downstream boundary condition.

Figure 6-5 presents the water surface long sections for three cases where the roughness values were altered as presented in Table 6-1. Figure 6-5 shows that changes to the roughness values do not make a significant impact on the water surface level at the site of the proposed development with the maximum absolute difference less than 0.12 m.

Table 6-1: Manning's n values applied in the calibration runs

	Floodplain	River	Road	Mangrove	Multiplier
Base case	0.08	0.035	0.03	0.16	**1**
Roughness -20%	0.064	0.028	0.024	0.12	**0.8**
Roughness +20%	0.096	0.042	0.036	0.19	**1.2**

The results of the sensitivity assessment have revealed that the modelled water surface levels are not overly sensitive to small changes in the downstream boundary conditions or small global changes in roughness values.

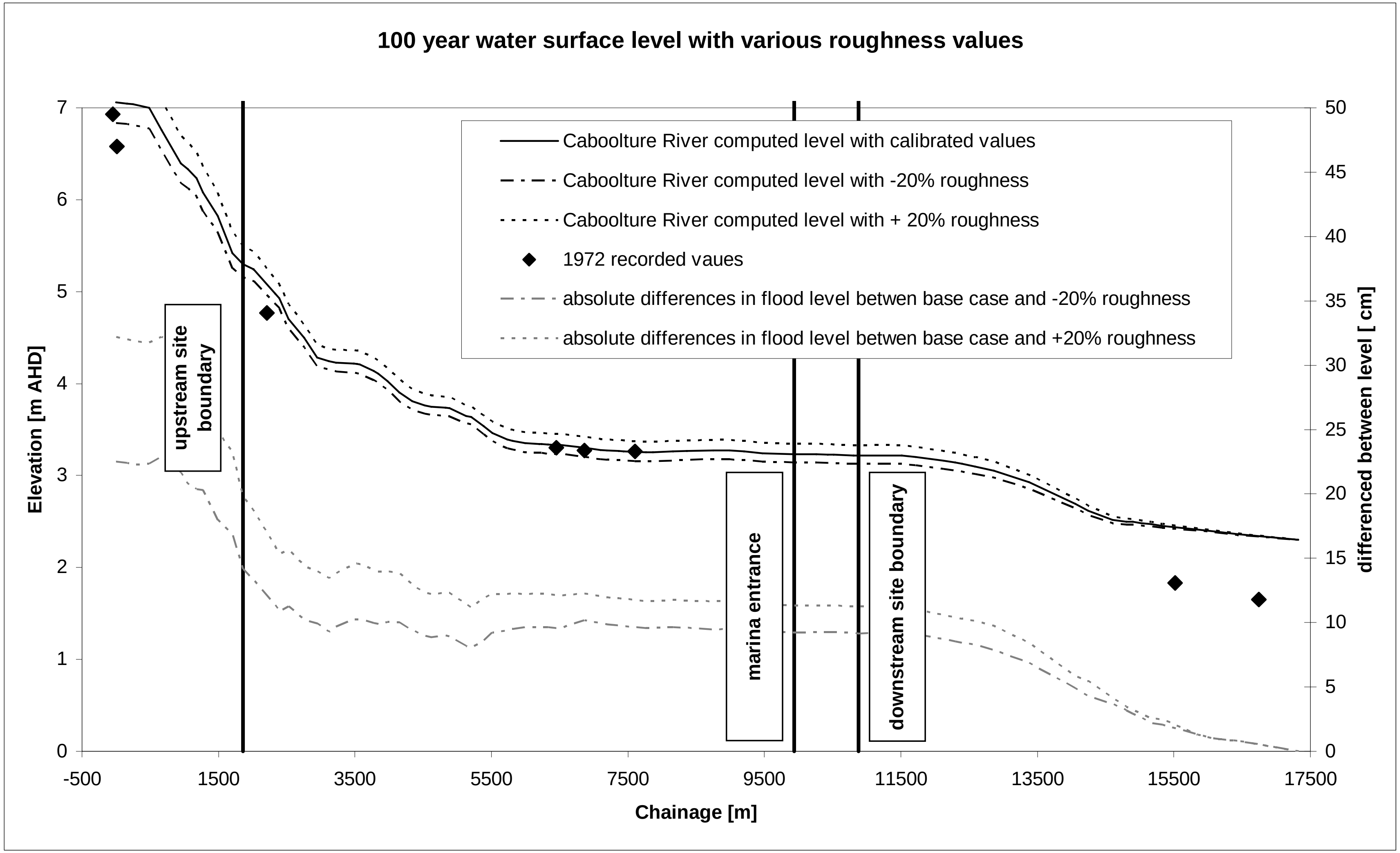

Figure 6-5: Water surface long section with changing roughness condition

6.3 Model fitness

Model Fitness is illustrated by Figure 6-6, Figure 6-7 and Figure 6-8, where the maxima of the following parameters are described throughout the model domain:

1. Froude number
2. Courant Friedrichs Levy condition (CFL)
3. Signal variance or noise in the model

The Froude numbers indicate sub-critical flow through the model domain, with the maximum not exceeding 1.0. Consequently the scenario being simulated is consistent with the model formulation, particularly with respect to the flow being in sub-critical regime

The MIKE21 solution scheme is centred (on average) in time and space finite difference solver. Consequently, there are no implicit limits on CFL except that temporal and spatial scales are resolved.

The finite difference grid is 10 m, which is considered adequate to model all significant flow paths, and in particular at the area of interest. The CFL is less than 1.20 and according to the work of Abbot et al. (1981) the behaviour phase is stable and reasonable for CFL <10.

Therefore the model behaviour is within the acceptable range of CFL.

Small numerical oscillations were created as part of the numerical calculation within the MIKE21 engine. The numerical amplitude of this noise can be compared to a wave of similar energy, as the signal variance is a measure of energy.

To produce an afflux map with 1 cm accuracy, the afflux must be within ± 0.5 cm. Thus the pre- and post-development water surface results need to have accuracy within ±0.25 cm. From Figure 5-6 it can be seen that the noise in the model is within this tolerance.

Therefore, the model is representative of the floodplain in terms of model fitness.

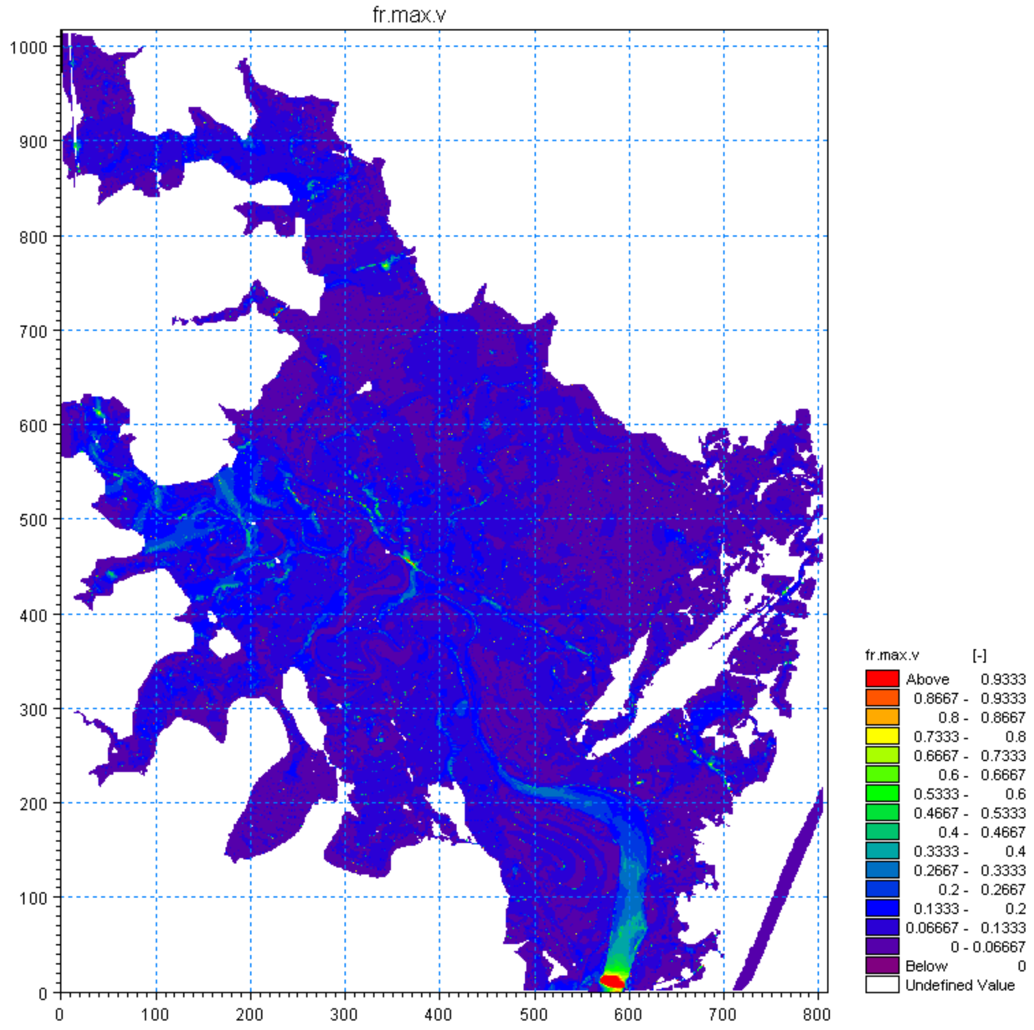

Figure 6-6: Froude map for the 100 year base case model

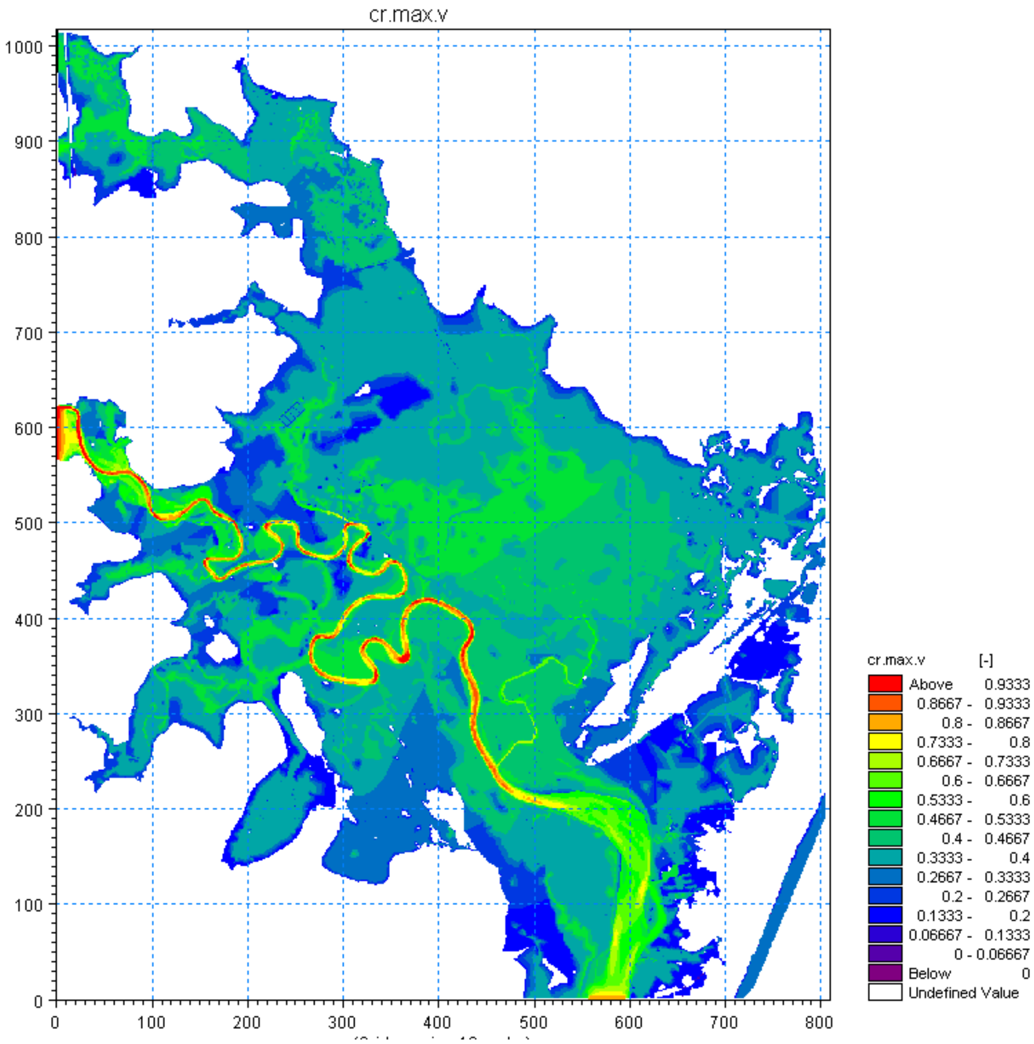

Figure 6-7: Courant map for the 100 year base case model

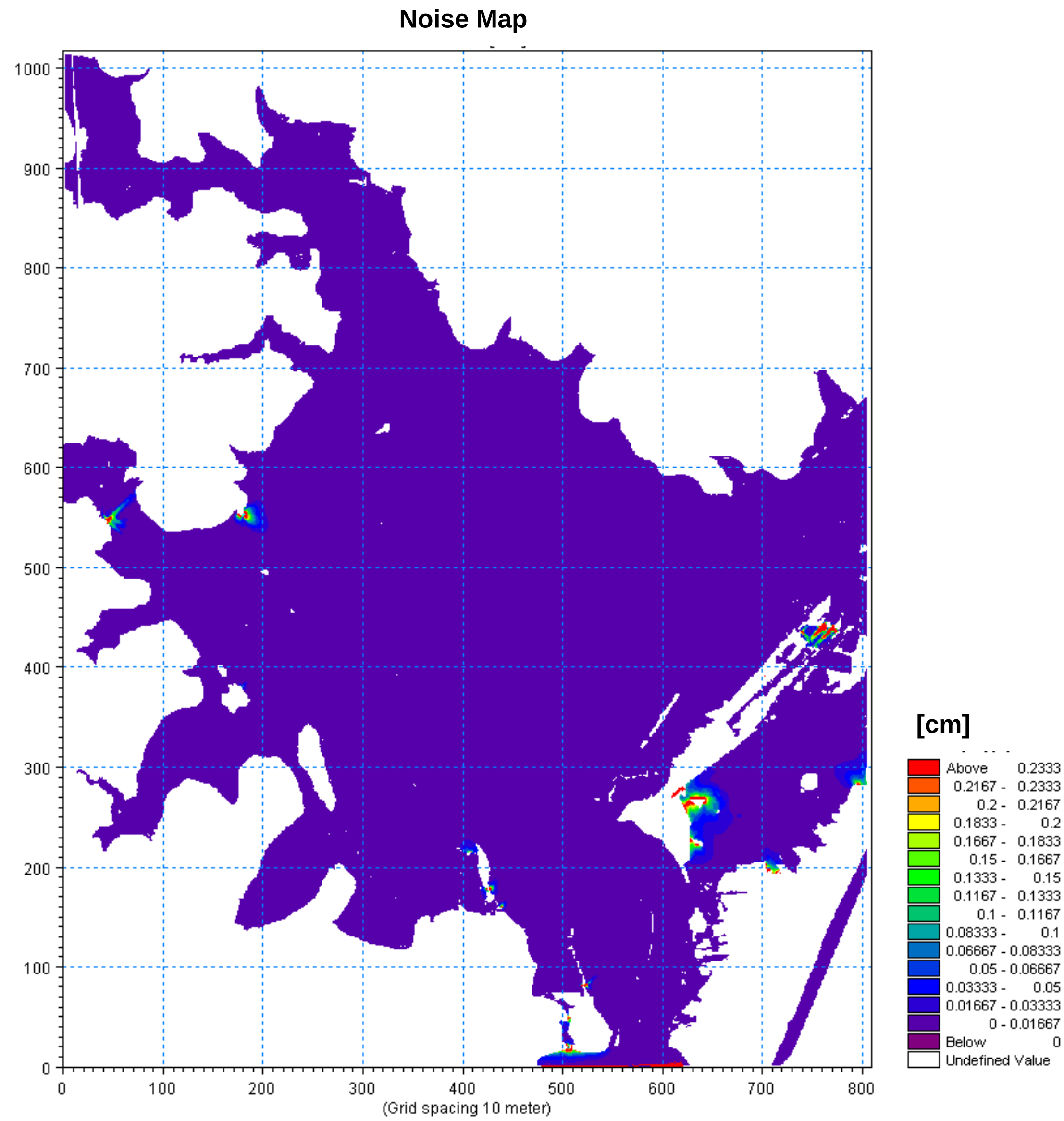

Figure 6-8: Noise map for the 100 year ARI event with steady-state flow

6.4 Mass balance

To check the validity of the MIKE21 model an investigation of the mass balance was also undertaken. This is a relationship between the inflow and outflow volume and represents the theoretical mass gain in the model domain.

This theoretical mass gain was then compared to the actual mass gain measured in the domain. The difference between these two values represents the absolute mass gain error.

Figure 6-9 presents the absolute mass gain error, and the relative mass gain error against the inflow volume for the 100 year base case model. The mass balance investigation shows that the model gains 2% of the total mass in the model domain.

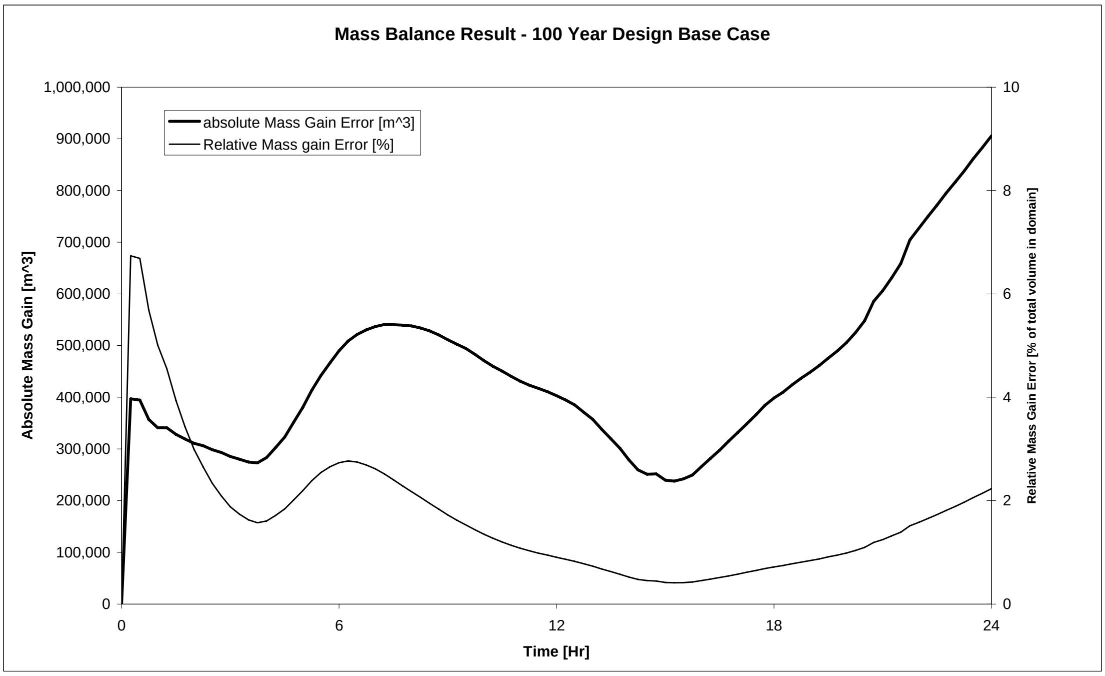

Figure 6-9: Mass balance result for the 100 year design base case

7. Design flood events

7.1 Un-mitigated development case model

To determine the impacts of the proposed development, the base case model terrain was amended as per the cut and fill diagram provided by Northeast Business Park Pty Ltd. (Appendix C - Drawing 0304 SK36, issue SD04, dated 30 July 2007 Ref 20430-10D). The alterations made reflect the earthworks associated with the proposed development.

The schematic in Figure 7-1 shows the un-mitigated development scenario. Those areas within the development boundaries that need to be above the 100 year ARI peak flood level (e.g. commercial, residential or industrial) are shown (cross-hatched).

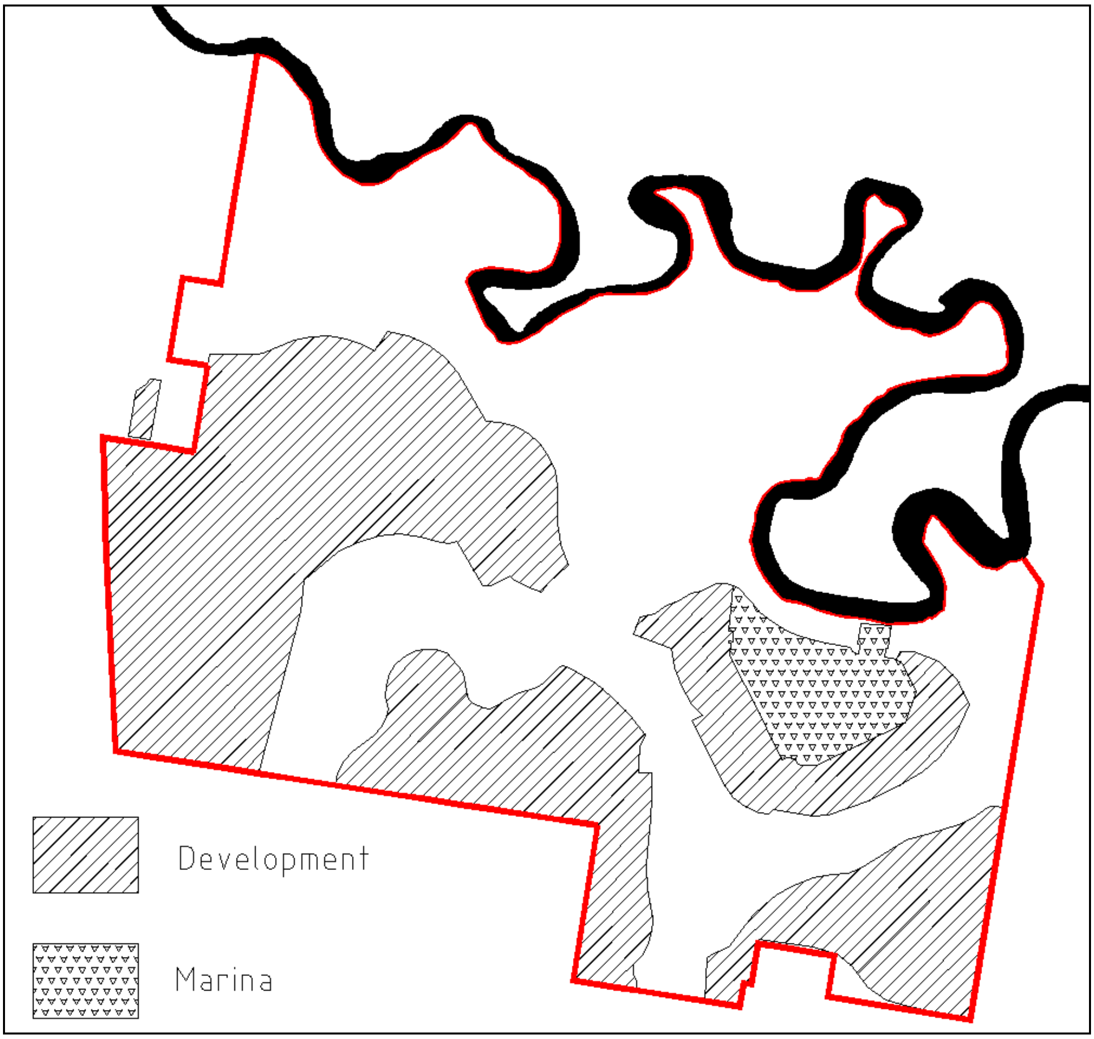

Figure 7-1: Schematic of the changes to the base case for the un-mitigated case

In addition to the development site cut and fill earthworks, a section of the Caboolture River will be dredged to suit the navigational requirements. The dredged section will be roughly trapezoidal in shape, with a base width of 40 m (minimum), a bed level of -4.25 m AHD and 1:3 side slopes. The upstream end of the dredging will be the upstream point of the navigational section of the river (approximately E502671, N6999503). The downstream end of the dredging in the model is the downstream model boundary. The actual downstream extent of the dredging is beyond the model boundaries. This was incorporated into the river bathymetry for the un-mitigated and mitigated scenarios. Figure 7-2 shows the impact of the dredging on the river bed.

The bed level of the marina basin was set at -3.5 m AHD.

Bed friction values and inflows remain the same as the base case scenario.

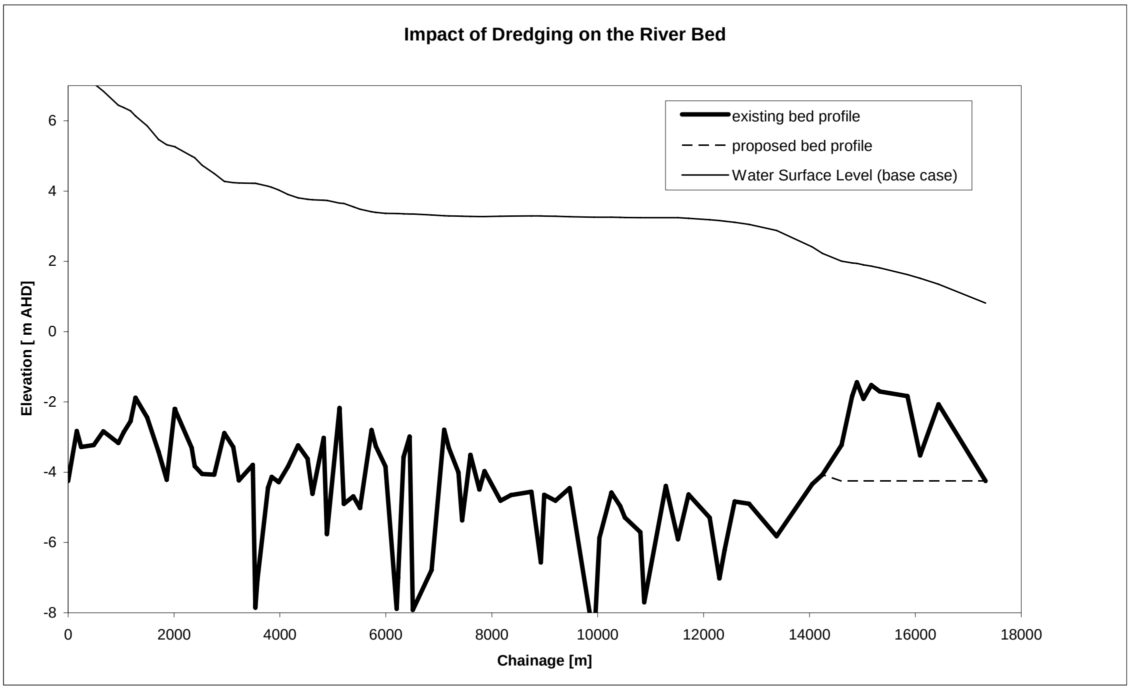

Figure 7-2: Effects of dredging on the river bed

7.1.1 Un-mitigated development case-model results

The proposed un-mitigated case produces high afflux across the flood plain. The impact is particularly significant to the north-east of the development site as shown in Figure 7-3. The development is shown to force the flood water towards the northern side of the Caboolture River. These results show that mitigation measures are required to reduce the impact of the high affluxes.

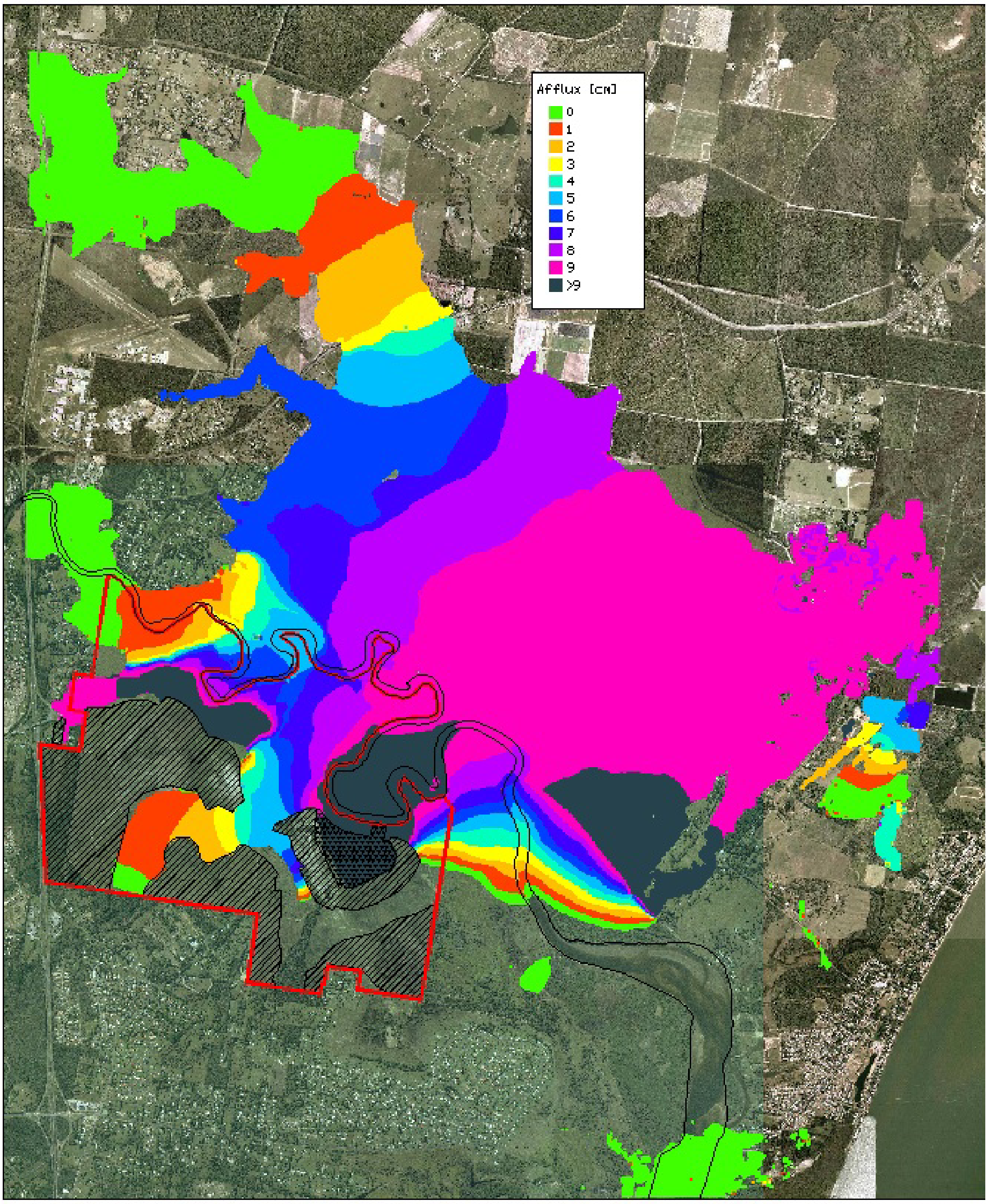

Figure 7-3: Afflux map for the un-mitigated case

7.2 Mitigated development case

In addition to the changes to the un-mitigated development model described in Section 7.1, there is a need to mitigate the increased peak flood levels outside the development boundary due to the proposed works. This is a requirement of the CSC Shire Plan.

The shape of the storage is dictated by the development master plan layout and by constraints associated with development near to or adjacent to rivers and creeks. For the Caboolture River, no development can occur within 100 m of the top of bank. For Raft Creek this distance is reduced to 80 m.

The mitigation philosophy to offset the increase in peak flood levels outside the development site is based on the following two criteria:

- increase flow conveyance through the proposed development
- construct earth diversion banks to help direct the flow through the site and away from sensitive areas.

The inclusion of a detention basin to attenuate flood waters was not considered for the following reasons:

- a large volume of water will need to be stored before the detention basin could have a significant effect on the large volumes of flood water from the Caboolture river system
- land restrictions relating to the large volume needing to be stored
- depth restrictions requiring the detention basin to remain above the tidal limit will force the basin to be shallow and have limited impact.

Based on these principles and the development and environmental constraints, Figure 7-4 shows the general location of the flood mitigation elements within the development that will be optimised within the development site.

The following section describes each flood mitigation element, of which a summary is presented in Section 7.2.6.

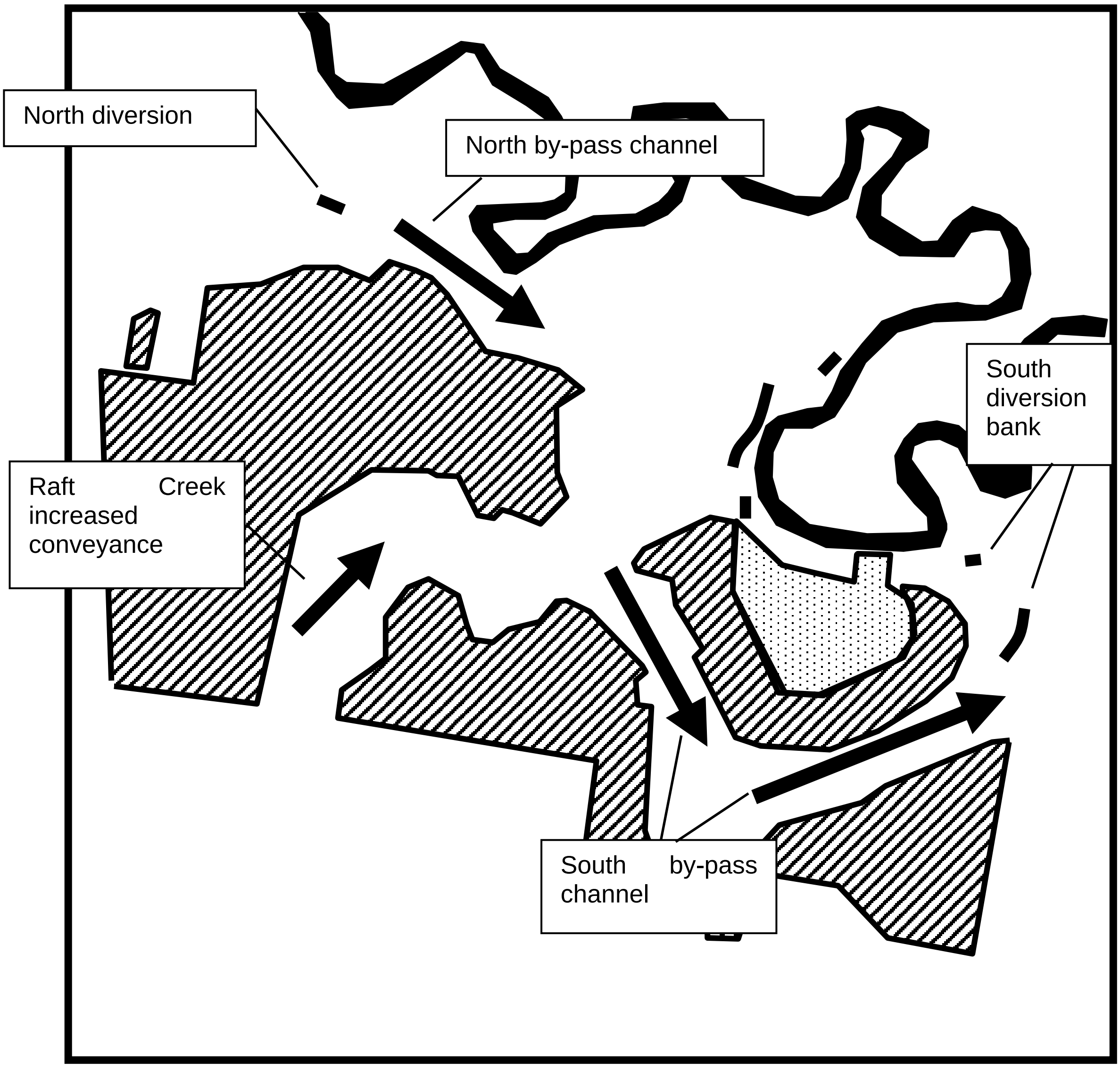

Figure 7-4: Schematic of the mitigation philosophy

7.2.1 Details – mitigation options for the north by-pass channel

Earthworks within the north by-pass channel will reduce the afflux on the north side of the proposed development. The topography changes (before and after development) for this mitigation option are shown in 7-5. The area within the black box shows the extent of earthworks required.

The objective of this mitigation is to increase the conveyance on the south side of the river and convey the water towards the south-east side of the proposed development thus the afflux upstream of the development is reduced. The approximate volume which needs to be cut to reduce the natural ground to a height of 1.5 m AHD within this area is approximately 160,000 m^3.

Manning's n roughness of the ground was reduced from 0.08 to 0.04 within the boundaries of the north by-pass channel.

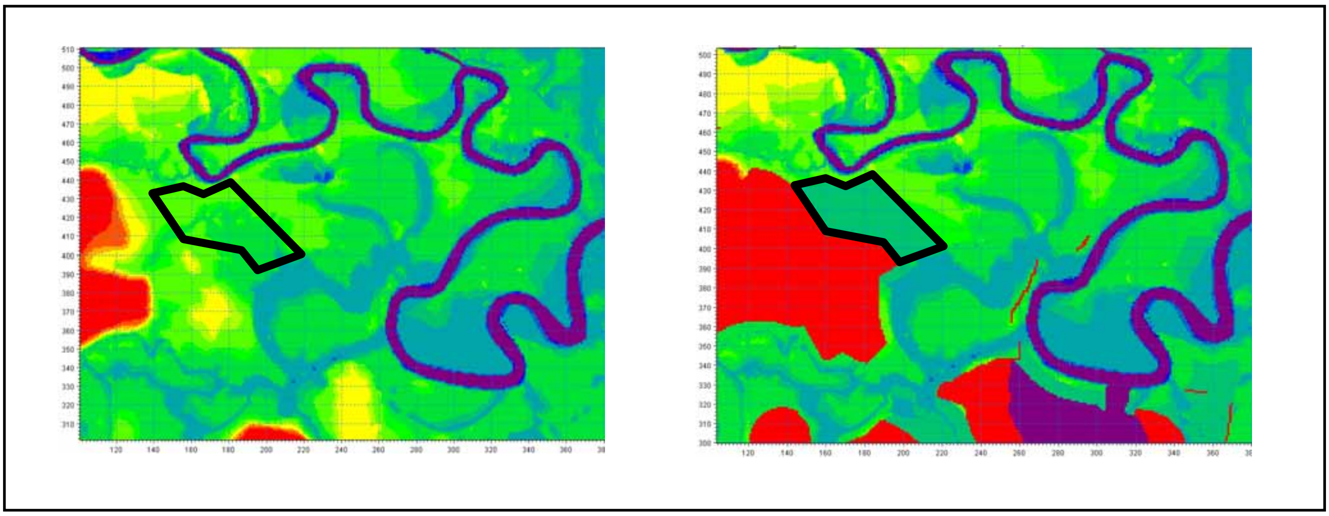

Figure 7-5: Proposed North Channel by-pass (un-mitigated and mitigated cases)

7.2.2 Details – mitigation options for earth diversion banks

Earth diversion banks are required at four locations within the development site. Figure 7-6 shows the location of these diversion banks.

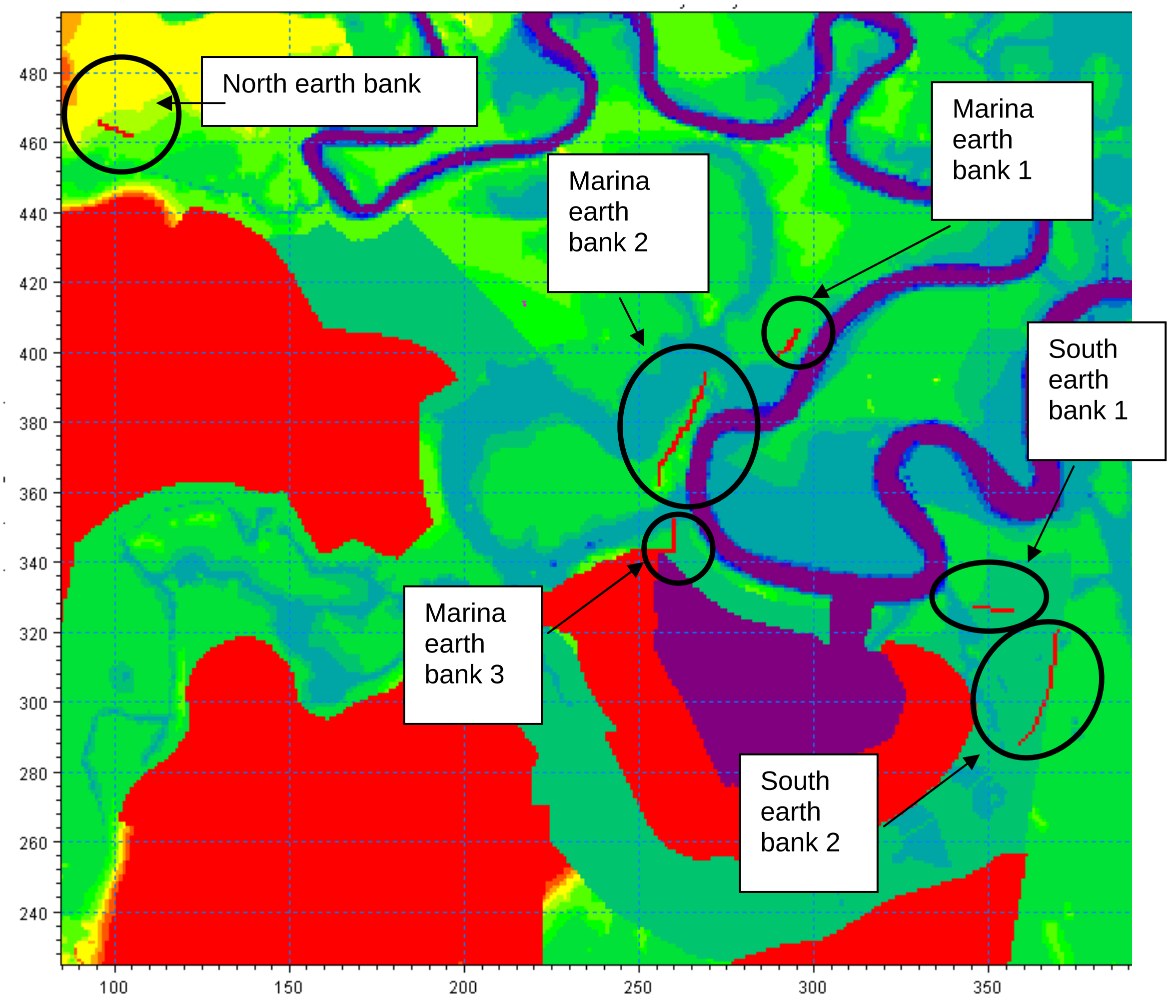

Figure 7-6: Proposed earth diversion embankments

The north earth bank is needed to prevent afflux on the west side of the development while the three marina earth banks are required to prevent affluxes north of the marina.

The south earth banks prevent increased peak flood levels at the downstream boundary.

The earth diversion banks will be designed such that they are a minimum of 0.3 m above the 1 in 100 year ARI flood level, with one in four sides. The final design of these earth banks will require structural input.

7.2.3 Details – mitigation options for south by-pass channel

The flow conveyance on the south side of the river needs to be enhanced wherever possible. An important flow route exists south of the proposed marina. The topography changes (before and after development) for this mitigation option are shown in Figure 7-7. The area within the black box shows the extent of earthworks and the location of the south by-pass channel mitigation.

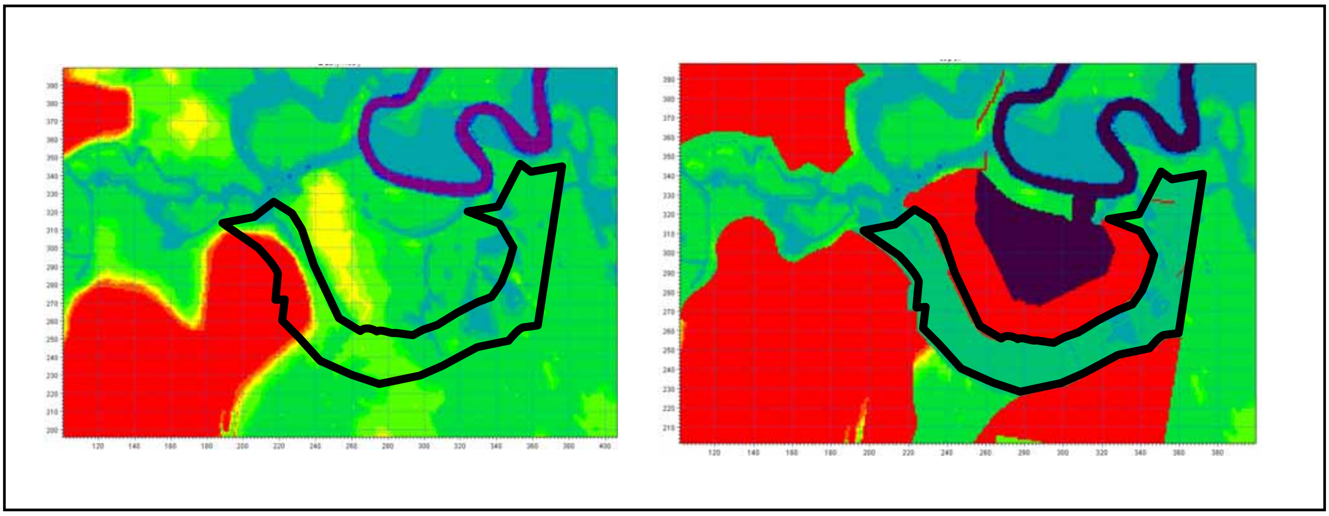

Figure 7-7: Proposed south by-pass channel mitigation (unmitigated and mitigated cases)

Land within the south by-pass channel will be cut to 1.5 m AHD. The Manning's n roughness coefficient varies from 0.08 to 0.04 depending on the mitigation requirements (refer Table 5-1.).

The volume of natural ground which needs to be removed to reach a level of 1.5 m AHD is approximately 436,000 m^3.

7.2.4 Detail – mitigation options in Raft Creek area

A section in the southern parts of Raft Creek (within the Development's boundaries) is constricted and increases the peak water levels. This area is shown in Figure 7-8. The offset is a cut parallel to Raft Creek.

The volume of ground that needs to be cut to a height of 2.0 m AHD is 103,000 m^3.

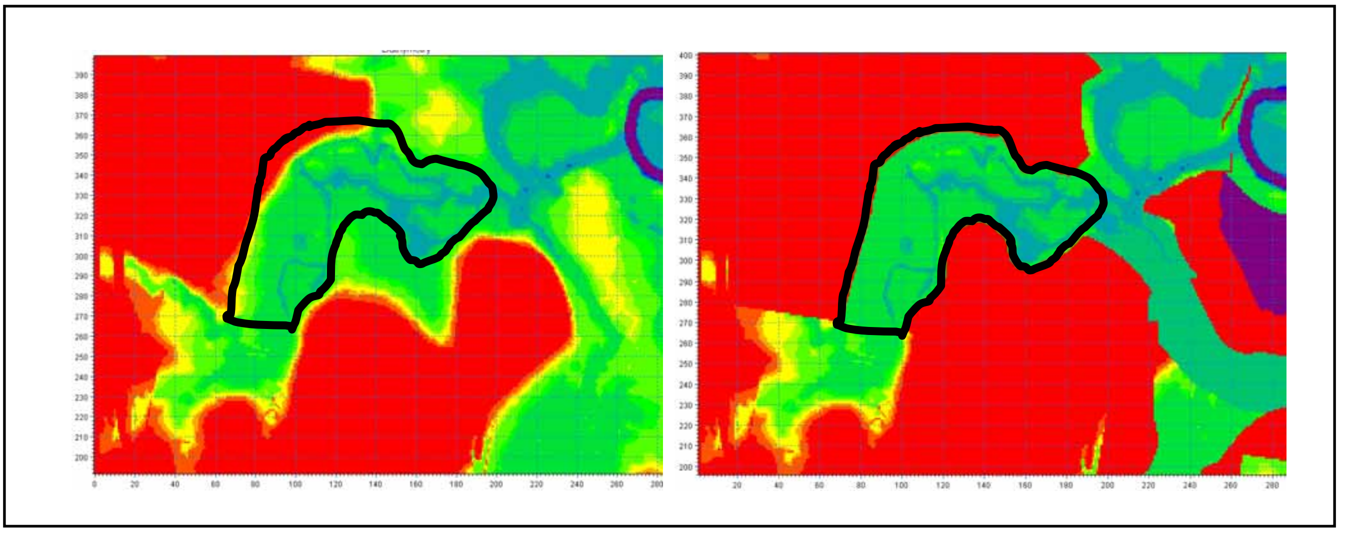

Figure 7-8: Proposed mitigation in Raft Creek area (un-mitigated and mitigated cases)

7.2.5 Details – grass managed areas

Figure 7-9 presents the area where grass management is required. In these areas the roughness is decreased from 0.08 to 0.04.

This decrease would represent a change to a smoother ground surface where the grass is maintained at a much lower level such as the type of grass on a golf course or sports ground.

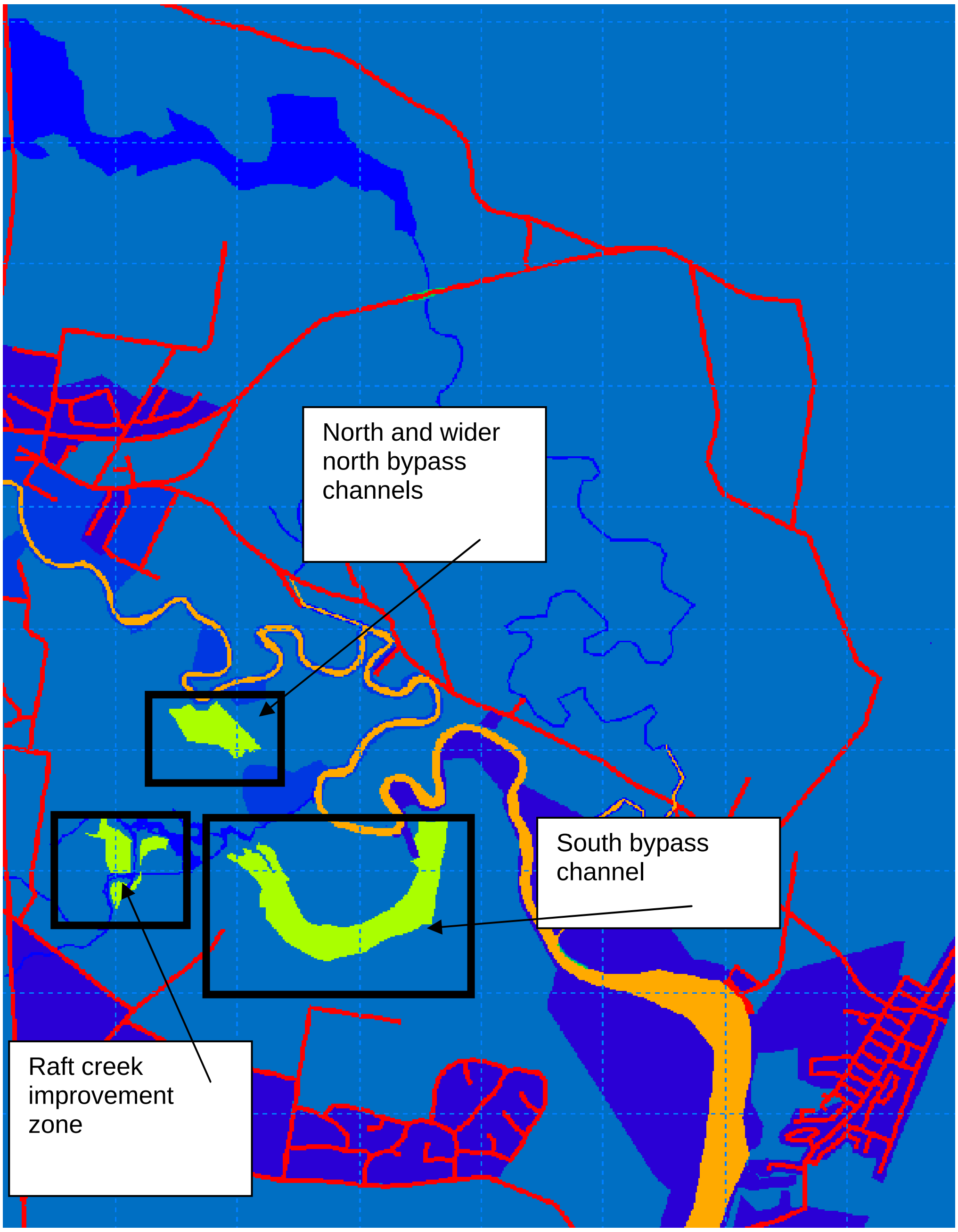

Figure 7-9: Proposed grass managed area with reduced Manning's n

7.2.6 Summary – mitigation case

The following summarises the preferred mitigation case undertaken for this study as well as an estimate of the volume of earthworks required:

- north by-pass channel: reduced roughness, cut to 1.5 m (160,000 m^3).
- south by-pass channel: reduced roughness, cut to 1.5 m (436,000 m^3).
- Raft Creek-improvement: reduced roughness, cut to 2 m (103,000 m^3)
- total volume of cut: 699,000 m^3.
- six earth diversion banks — three near the marina, two at the eastern boundary and one in the north-western section (earthworks not included in above cut volume).

7.3 Preferred mitigation case

The following section provides the results for the preferred mitigation case for the 100 year ARI event only.

7.3.1 Afflux

Figure 7-10 presents the afflux for the preferred mitigation case. The afflux is considerably reduced within the floodplain. CSC's floodplain guidelines are met as there is no afflux outside the development boundary.

In this case all proposed excavated areas cut (north by-pass channel, wider north by-pass channel, south by- channel and raft channel) have been modelled with a reduced roughness as per Figure 7-9.

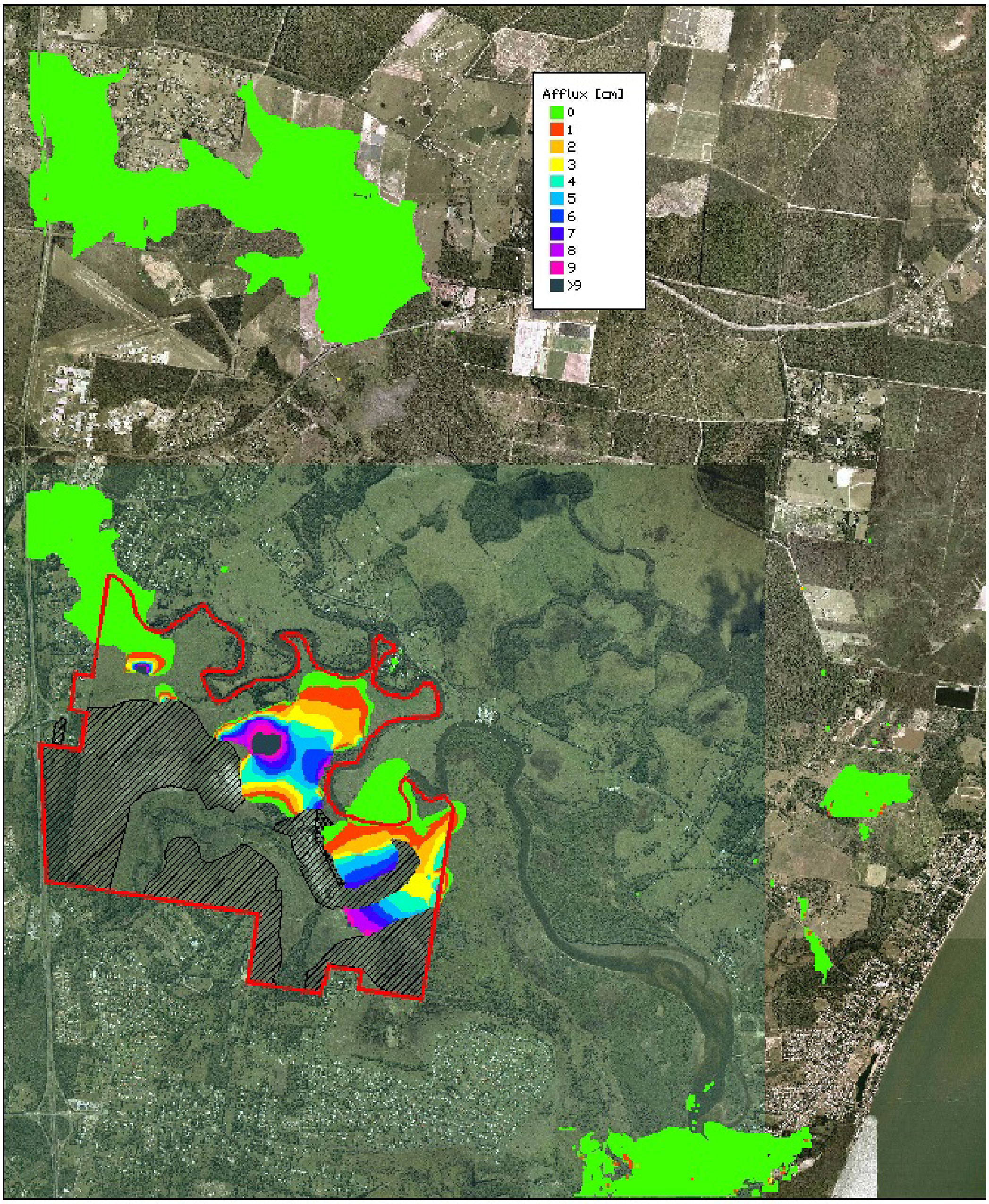

Figure 7-10: Preferred mitigation case afflux map

7.3.2 Water surface levels

The maximum water surface level and maximum flow velocity for the preferred mitigation case are shown in Figure 7-11. The water surface elevations range from 2.3 m AHD to 7.5 m AHD. The majority of velocities shown are less than 1.0 m/s; however, within sections of the main Caboolture River velocities exceed 2.0 m/s.

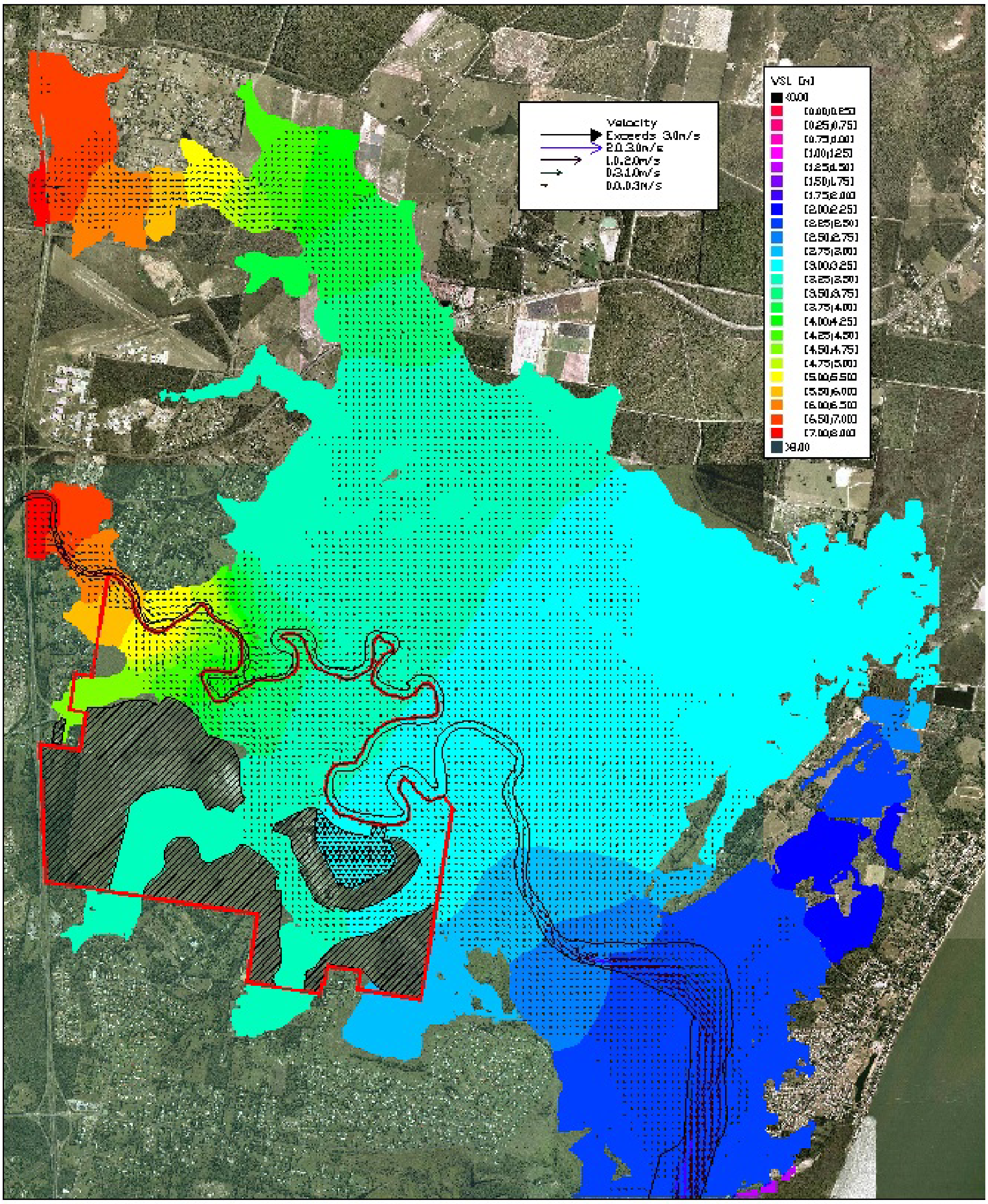

Figure 7-11: Maximum water surface level and velocity for the preferred mitigation case

Figure 7-12 presents the long section of the water surface levels for the existing, un-mitigated and the preferred mitigated case. The comparison of the three cases shows very little difference in water surface levels.

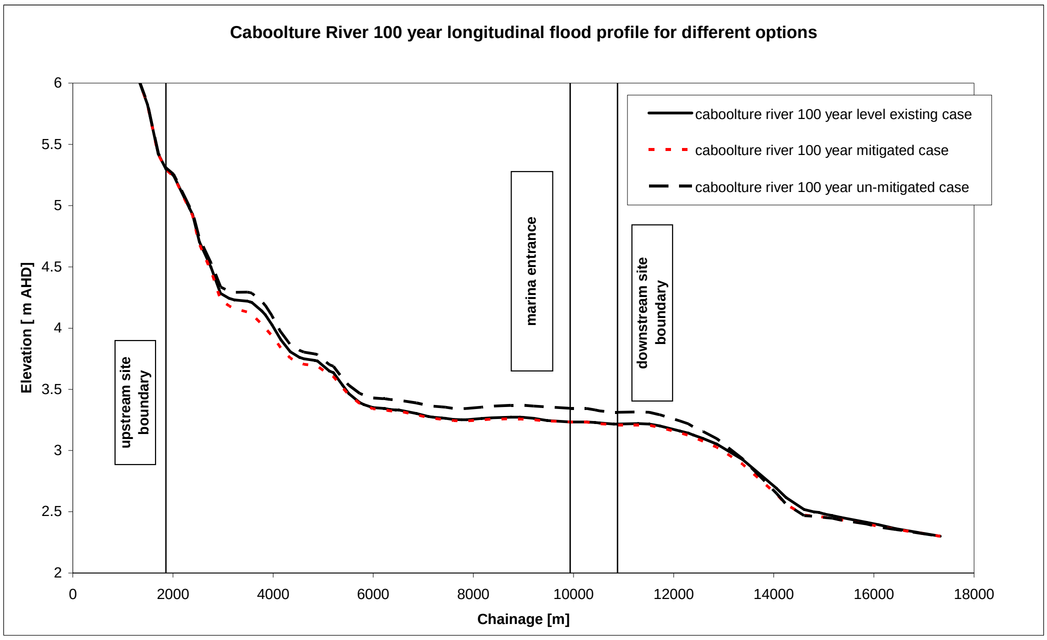

Figure 7-12: Water surface level long section

7.3.3 Depth

Figure 7-13 presents the preferred mitigation case flood depths. The maximum depth within the floodplain is 4 m. The depth within Caboolture River the depth is greater than 4 m.

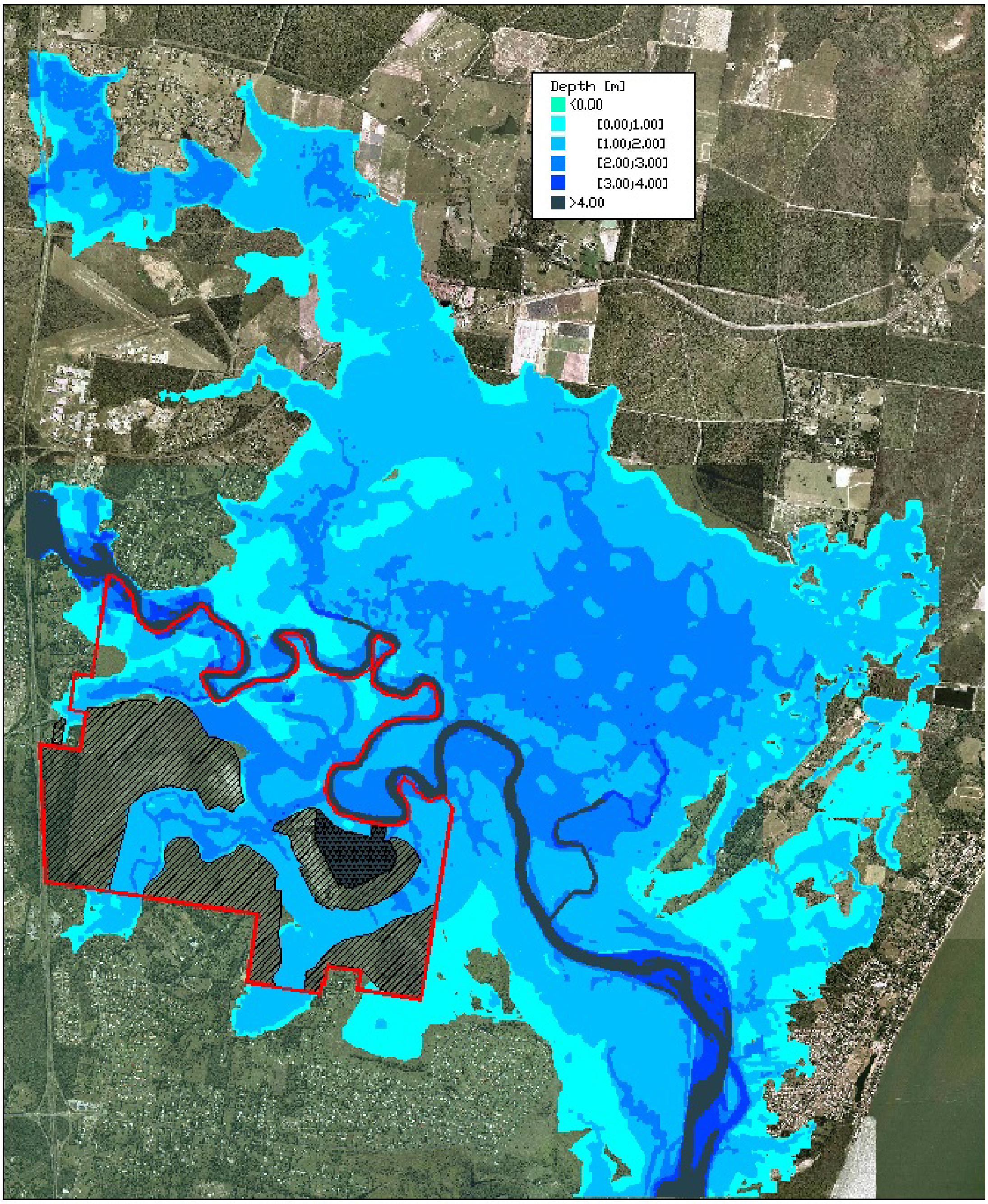

Figure 7-13: Water depths for the preferred mitigation case

7.3.4 Flow patterns over the proposed site

The flow patterns over the site need to be understood such that suitable scour protection can be designed to protect areas subject to high velocities. The velocity and volume of water going through the site are presented in this section.

To assess the flow patterns on the site, the volume and velocity of flow were extracted from the modelling results of the preferred mitigation case at four locations, as shown in Figure 7-14. These locations were selected as the flow was significantly constricted at this site thus providing the highest flow velocity.

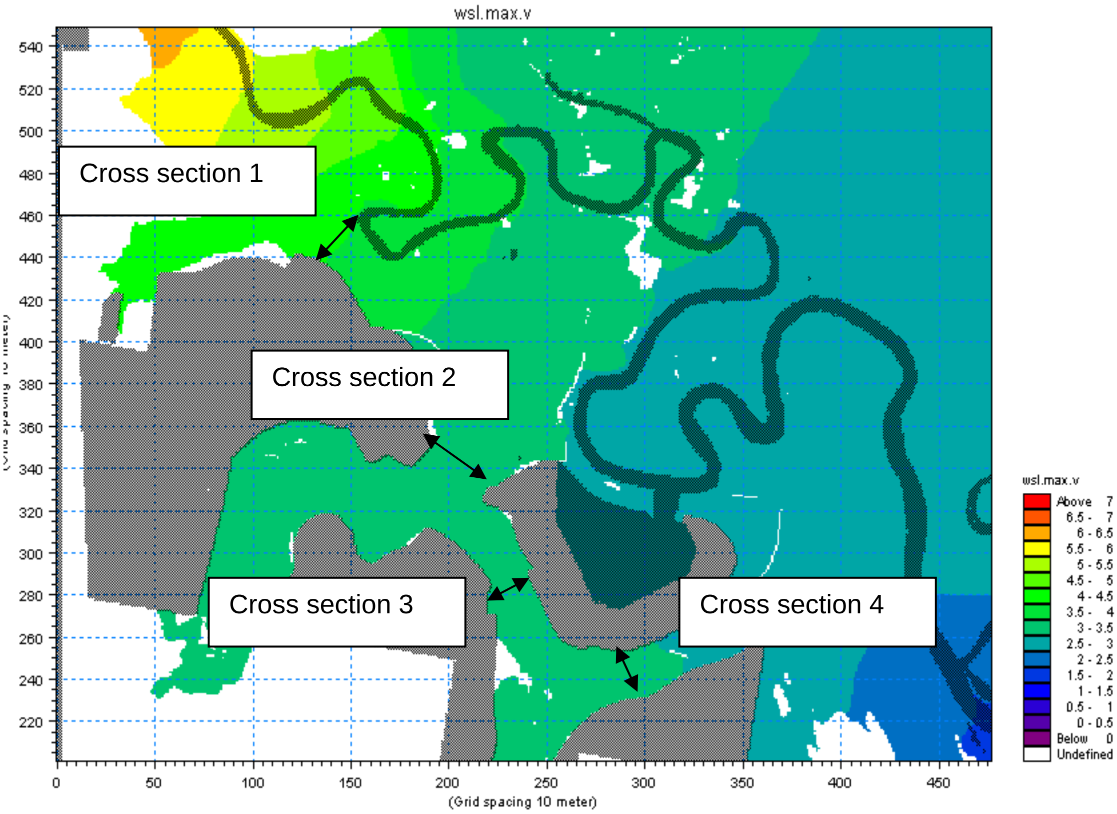

Figure 7-14: Flow volume and speed cross section locations

Figure 7-15 presents the flow velocity at each cross section location for each time step of the flood model. The speed is relatively small and never exceeds 0.8 m/s. Therefore the soil in the proposed by-pass channels should not be prone to erosion. The spike at cross section three is most likely due to local inflows from Raft Creek coming through the cross section before the peak of the Caboolture River flows. Regardless, the largest speed predicted at cross section three occurs at approximately nine hours.

Figure 7-16 presents the flow volume at each cross section location for each time step of the flood model. As expected cross section one has the highest peak discharge. The peak flows have spread throughout the floodplain somewhat and therefore have reduced in magnitude at the other cross sections. Cross sections three and four have similar discharges due to the similarity of preferred mitigation works: similar ground elevations, roughness values and flow areas.

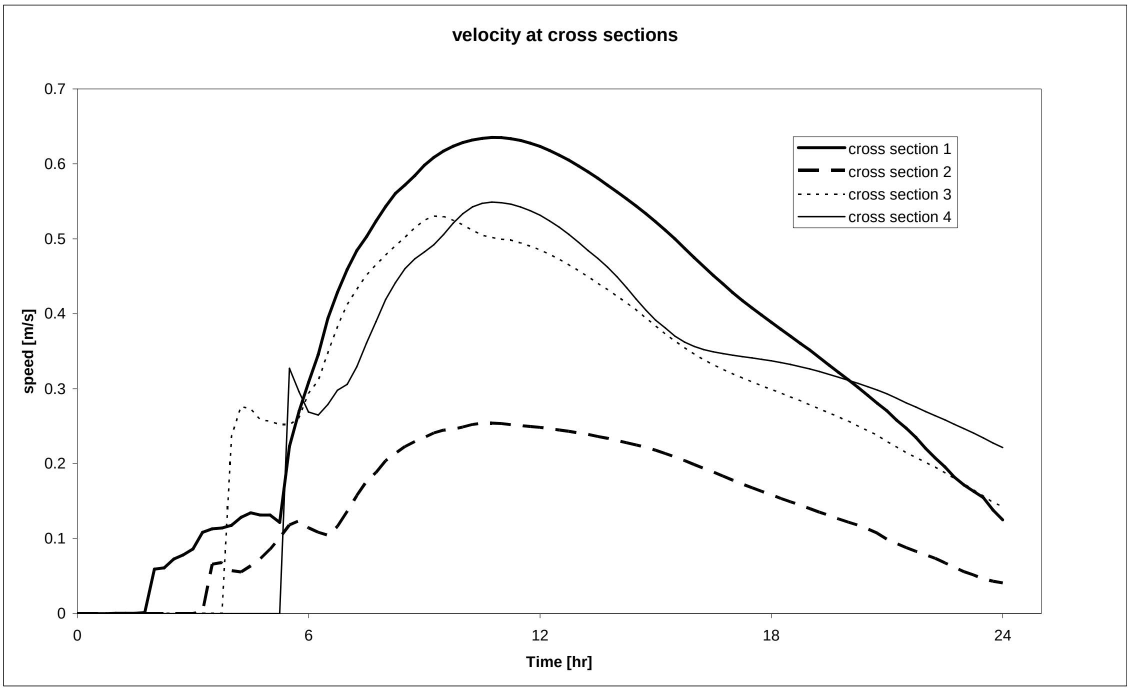

Figure 7-15: Velocity at cross sections

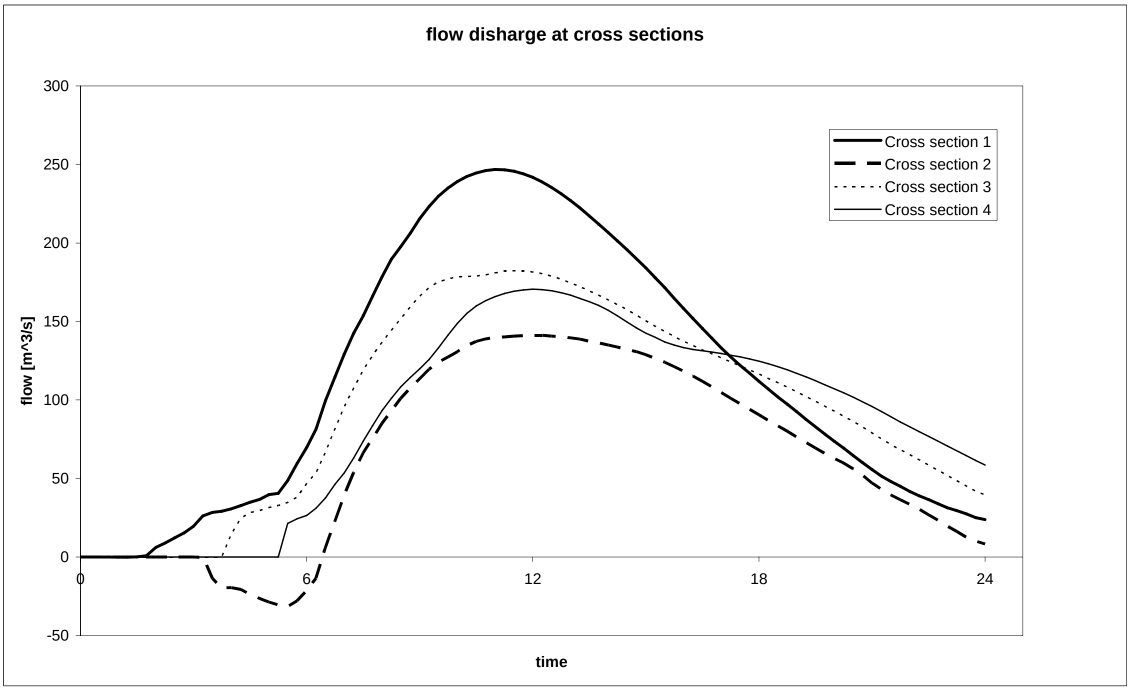

Figure 7-16: Flow discharge at cross sections

7.3.5 Flow velocities in Caboolture River

Figure 7-17 shows the velocities along the centreline of the Caboolture River. The figure shows that the velocities are generally maintained between the pre-development and post-development scenarios. The exception is the increase in velocity within the navigation channel at the downstream end of the model.

Some scour would naturally be expected for the 100 year flood event. The impact of the development on the velocities in the channel is not significant.

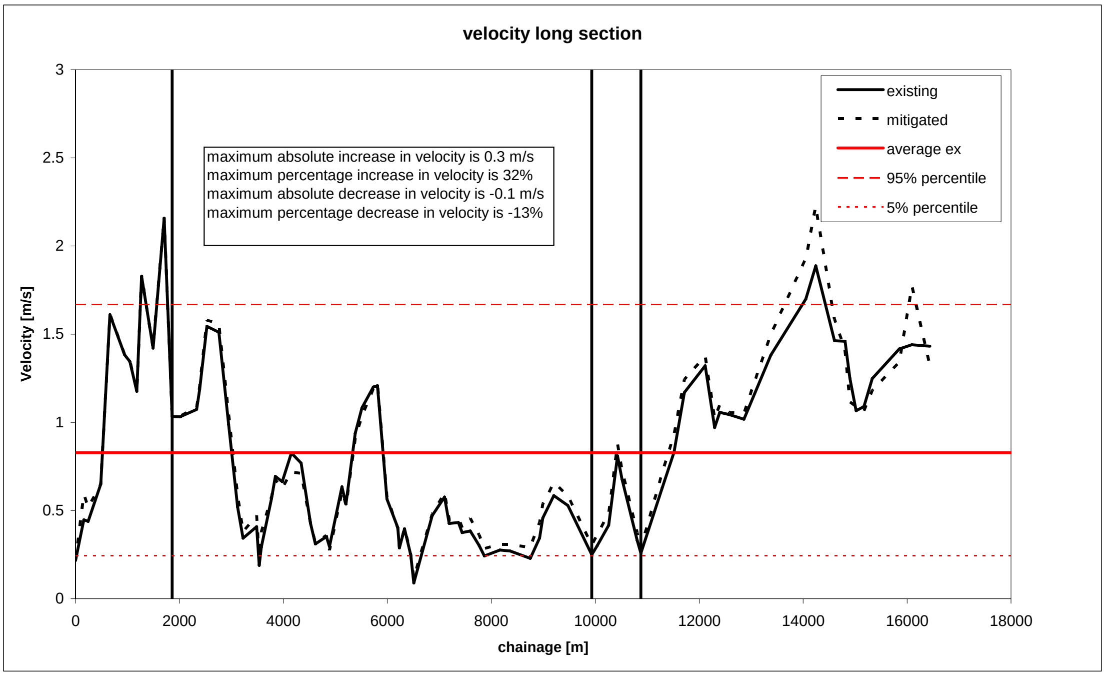

Figure 7-17: Longitudinal section of water velocity along the Caboolture River (m/s)

7.4 Design details of the proposed earth diversion banks

Table 7-1 presents the height at which the earth diversion banks needs to be set. This table also shows the flow velocity at which the bank would have to be protected in order to prevent erosion and scour.

Table 7-1: Details of proposed earth diversion banks

Earth diversion bank	Ground [m AHD]		Maximum WSL [m AHD]		Maximum velocity [m/s]		Height of earth diversion banks above ground with 300mm freeboard [m]	
	US	DS	US	DS	US	DS	US	DS
North	4.2	3.2	4.7	4.5	0.8	0.5	0.8	1.6
Marina 1	2	2	3.3	3.2	0.4	0.1	1.6	1.5
Marina 2	1.2	2	3.4	3.2	0.5	0.5	2.5	1.5
Marina 3	1	2	3.3	3.2	0.5	0.2	2.6	1.5
South 1	1.5	1.5	3.2	3.1	0.2	0.4	2	1.9
South 2	1.5	2	3.2	3.1	0.35	0.2	2	1.4

7.5 Net benefits for wider floodplain

The preferred flood mitigation as described above has a net benefit to the wider floodplain. Figure 7-18 present the reduction in peak flood levels for the 1 in 100 year ARI event. The increased conveyance through the development site by use of earth diversion banks, grass management and additional earthworks reduces the flood risk to the wider community.

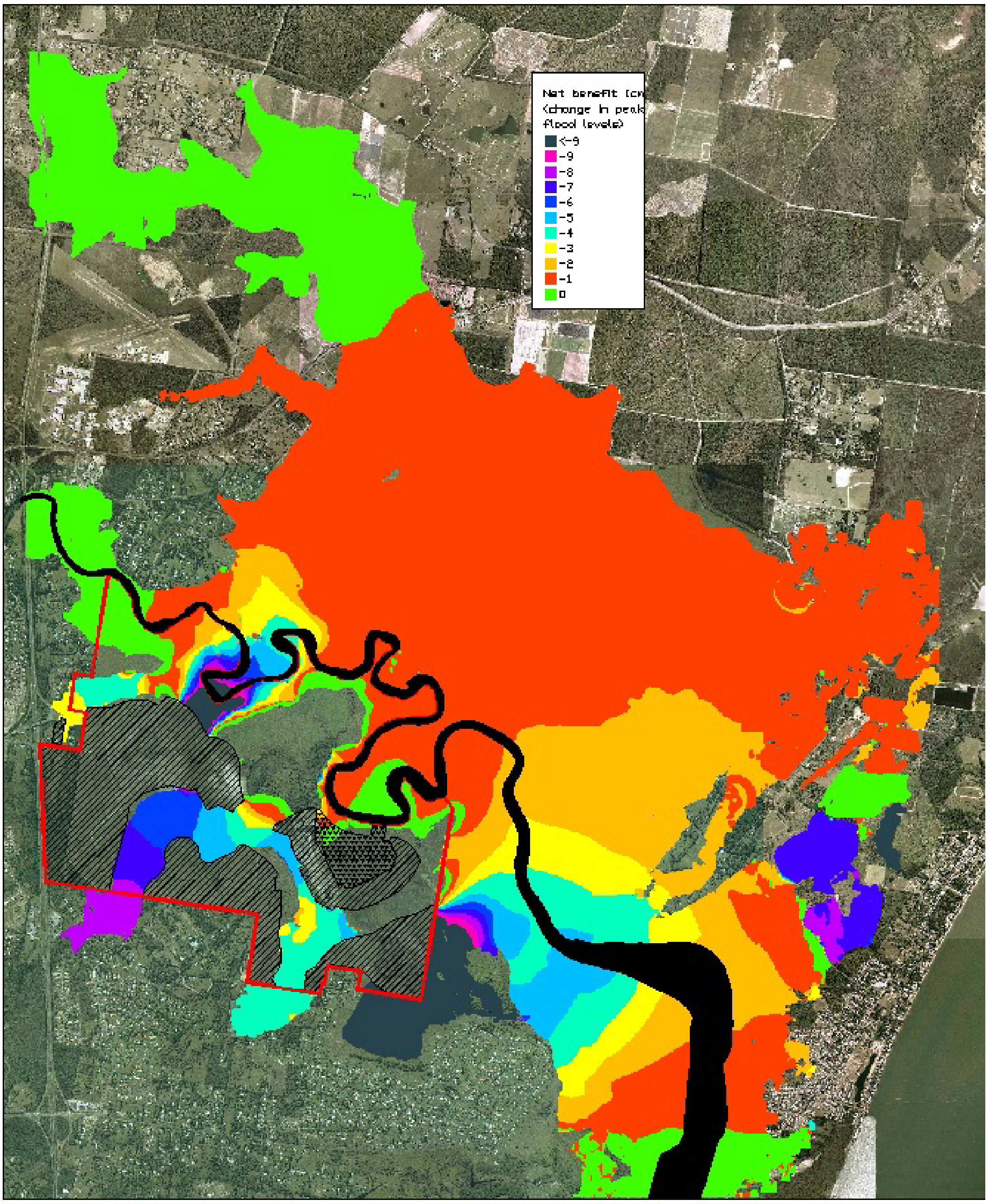

Figure 7-18: Net benefit map showing decrease in peak flood levels

7.6 Flood plain storage calculations

A simple earthworks model was developed using 12d to show that there is no net loss of floodplain storage for the preferred mitigation case. The topography and water surface levels of the basecase and the preferred mitigation case were triangulated in 12d and a simple cut/fill calculation undertaken for the area within the development boundary. The results are:

- base case floodplain storage = 7,844,562 m^3
- preferred mitigation scenario floodplain storage = 9,268,352 m^3

Therefore the development scenario provides an additional 1,423,790 m^3 in floodplain storage.

8. Conclusions and recommendations

The purpose of this flood study is to provide floodplain information for the planning application that includes the development of Northeast Business Park; 760 ha of industrial, commercial, parkland and future residential land use within CSC. The conclusions and recommendations made in this report are only applicable to the floodplain within and immediately surrounding the area of the proposed development.

8.1 Base case model

The following remarks are made in relation to the base case flood model:

- calibration was undertaken against the 1991 event
- verification was undertaken against the 1972 and 1989 historical events with a good match between measured and modelled water surface level
- the flood model is not sensitive to changes in downstream boundary conditions within the context of the development site
- the model fitness assessments based on Froude numbers, Courant Friedrichs Levy ratio and model noise, demonstrate the stability of the model
- the maximum relative mass gain error is insignificant at 2 % of the total mass in the domain.

Therefore the MIKE21 flood model can be used confidently to simulate the flow across the floodplain, providing a tool to assess the flood mitigation requirements.

8.2 Proposed development

The development case includes the cut and fill plan as per master plan (Drawing 0304 SK36, issue SD04, dated 30 July 2007 Ref 20430-10D), and includes a 40 m wide dredged navigable channel downstream of the fish habitat area.

The flood model results showed that the un-mitigated master plan increased the peak flood levels for the 1 in 100 year ARI event across the majority of the floodplain. Therefore flood mitigation was required as per CSC's floodplain policy.

Flood mitigation was required to offset the increased peak flood levels outside the development site and was based on two principles:

- increase flow conveyance through the proposed development
- construct earth diversion banks to help direct the flow through the site and away from sensitive areas.

The preferred mitigation case presented in this report includes a combination of earth diversion banks and additional land cuts. The flood mitigation elements were located in four distinct areas within the development: North by-pass channel, wider north by-pass channel, Raft Creek and the southern by-pass channel.

The preferred mitigation case consists of:

- north by-pass channel — cut to 1.5 m AHD, grass managed
- Raft Creek — cut to 2.0 m AHD, grass managed
- south by-pass channel — cut to 1.5 m AHD, grass managed
- six earth diversion banks — three near the marina, two on the eastern boundary, one in the north-western section.

It is estimated that the total earthworks (cut) for the flood mitigation is 699,000 m^3. This does not include earthworks required for the earth diversion banks.

The preferred mitigation case shows overall reductions in the peak water levels for the 100 year ARI events across the flood plain. This is due to the flood mitigation works that increase the conveyance through the development site and therefore reduce the flood conveyance through the northern section of the lower Caboolture River floodplain (north of the Caboolture River).

The changes in the flow velocities within Caboolture River due to the flood mitigation works are insignificant when compared to the existing case velocities. As expected the navigation channel has the most impact on river velocities.

Overall the proposed works represent a net benefit for the community in terms of flooding. The peak flood levels will be lowered in much of the surrounding flood plain with localised peak flood level increases occurring only within the site boundary or in locations where existing infrastructure will not be impacted.

8.3 Recommendations

Adopt the preferred mitigation strategies to minimise afflux associated with the proposed development in accordance with CSC's requirements

The detailed design of any structures (bridges, culverts, etc) that are proposed within the floodplain (over, under, or through) will need to be appropriately modelled to assess the impacts on the floodplain.

The maintenance of the grass managed areas is essential to the flood mitigation proposed in this study. These areas must be designed such that the land use relates to a Manning's n roughness value of 0.04 which correspond to well maintain grassed areas. Deviations from this value will need to be remodelled.

Structural input is recommended for the design of the earth diversion banks to avoid 'washouts' and therefore compromise the flood mitigation proposed.

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Appendix A

Caboolture Shire Council
Flood Plain Management Policy
803/02

CABOOLTURE E0012

COUNCIL POLICY No: 803/02

FLOOD PLAIN MANAGEMENT

RESOLUTION:

The calculated average recurrence interval event (ARI) is a calculated flood caused by any body of water which rises as the result of a calculated event or the joint probability of a combination of calculated events including storm water runoff, storm/cyclonic surge, tide or other event.

1. FOR REZONING CONTROL

An application for rezoning will not be approved unless the applicant can demonstrate that the minimum requirements for subdivision control can be met in the zone into which the subject land is proposed to be placed.

2. FOR SUBDIVISION CONTROL

(a) Residential Zones

(i) Subdivision of land below the calculated 100 year ARI flood contour will not be approved.

(ii) Council may permit works to achieve the criteria set in Clause 2(a)(i) subject to the following:-

- Alteration of site contours, including filling, may be undertaken subject to no net loss of flood storage across the subject land for all storm events up to and including a 1 in 100 year event.
- The determination of flood storage is to be by computer model based on pre and post development field contour surveys.
- Where site contours are amended such work is to be undertaken in such a manner that adjoining properties remain free draining and with no resultant increase in flood levels.

(b) Rural Residential Zones A, B and C.

(i) within each allotment at least three thousand (3,000) square metres in one parcel which is above the calculated fifty (50) year ARI flood contour prior to alteration of the natural ground profile and

(ii) within each allotment at least one thousand (1000) square metres in one parcel with a minimum depth or width of twenty five (25) metres, included in the area of land in (i) above, which is above the calculated one hundred (100) year ARI flood contour prior to alteration of the natural ground profile.

(iii) the area above the 100 year flood contour [item (ii) refers] must front onto a dedicated road and have a minimum frontage to the road of 10 metres.

(iv) Creeks or watercourses having defined bed and banks are not permitted to traverse Rural Residential B and C allotments.

(v) The area occupied by creeks and watercourses having defined bed and banks plus a minimum distance of 10 metres from the "top-of bank" are to be contained within drainage reserves external to Rural Residential B and C allotments.

(vi) The determination whether or not a creek or watercourse has defined bed and banks and the determination of the "top-of-bank" is to be to the satisfaction of the Shire Engineer.

(vii) Where construction works not approved by the Shire Engineer are undertaken to alter the shape of a creek or watercourse prior to determination of items (iv), (v) and (vi) the area below the flood contour associated with a 10 year storm event plus a minimum distance of 10 metres from the 10 year flood contour is to be contained within a drainage reserve external to allotments.

(c) Rural Zone – Subdivision of floodable land will only be approved for rural zoned properties where each of the proposed parcels of land has an area of land in its natural state prior to any earthworks being carried out which satisfies all the following requirements –

(i) At least one thousand (1000) square metres in each parcel with a minimum depth or width of twenty five (25) metres above the calculated one hundred (100) year ARI flood contour;

(ii) has a slope not steeper than one (1) vertical to six (6) horizontal before undertaking any earthworks;

(iii) each parcel must front onto a dedicated road or be connected to a dedicated road by a constructed access which is above the calculated five (5) year ARI flood contour and the construction of which does not raise the flood levels on the adjoining parcels of land.

(iv) each parcel must exhibit a means of egress to a high ground retreat from the area specified in Clause (c) (i).

(d) Zones other than Residential, Rural Residential or Rural

- Subdivision applications will be considered on the circumstances of the individual proposals. Such proposals other than those to accommodate existing lawful buildings should be capable of complying with the following guidelines:-

(i) All parcels of land formed by subdivision should be capable of having the floor level of any building constructed above the calculated one hundred (100) year ARI flood contour for habitable buildings and above the calculated fifty (50) year ARI flood contour for non-habitable buildings and with a maximum height of floor levels above natural ground level of one (1) metre;

(ii) the construction of such buildings or the filling of land adjacent the buildings must not restrict the flow of floodwater, significantly increase the flood levels or create drainage problems on adjacent parcels of land;

(iii) Where filling of land will not restrict the flow of flood waters, significantly increase flood levels or create drainage problems on adjacent parcels of land, Council may permit the filling of land to meet the above requirements where the natural ground level is within one (1) metre of the calculated one hundred (100) year ARI flood contour. All fill batters must be less than one (1) vertical to ten (10) horizontal.

3. FOR BUILDING APPLICATION CONTROL

(a) In areas affected by flood water, where the construction of such buildings is allowed "As of Right" in the zone in which the land is situated:-

(i) The floor level of habitable rooms must be not lower than the higher of:-

(1) 300mm above the highest recorded flood level as determined by Council; or

(2) above the calculated one hundred (100) year ARI flood level where such level has been determined by Council.

(ii) The floor level of non-habitable buildings (garages, carports, farm sheds etc.,) may be constructed at or below the highest recorded flood level as determined by Council.

(b) In areas affected by tidal water:

(i) The floor of habitable rooms must not be lower than RL 2.3 metres AHD.

(ii) The minimum ground or floor level of RL 2.0 metres AHD to be provided to non-habitable buildings.

(iii) Septic Trench Installation.

Septic trenches are to be constructed so as to have a minimum surface RL of 2.0 metres AHD.

Where filling of land is necessary to facilitate the construction of a septic trench installation, only an evenly graded clean sand fill is to be used.

The following discretions may be exercised:

1. For minor additions to an already existing building, Council may approve a floor level on a non-habitable building at or above RL 1.7 metres AHD.

2. Septic trench surface levels of RL 1.7 metres AHD may be approved as follows:

   (a) Where a septic system is being added to an existing building or,

   (b) Where filling of the subject land to increase the surface level to RL 2.0 metres AHD or higher may create drainage and seepage problems on adjacent parcels of land.

The discretion given in (1) and (2) will be subject to the following conditions:

   (a) Owner signing a Statutory Declaration stating that the owner is aware of the possibility of tidal/storm surge flooding.

   (b) The property notes for the subject property being noted so that future purchasers will be aware of the problem prior to purchase.

3. In those instances where filling of the subject land to raise the surface level of septic trenches is required and Council considers that such filling may:

   (i) Restrict the flow of floodwaters, tidal water or stormwater, or,

   (ii) Increase flood levels of adjacent parcels of land, or,

   (iii) Create drainage and seepage problems on adjacent parcels of land.

then Council may refuse the application or require that plans be amended to demonstrate how sanitary and sullage wastes are to be disposed of to the satisfaction of Council.

(c) In areas where Council has determined fill levels in accordance with a master drainage scheme:

   (i) The floor level of habitable rooms must be not lower than the recommended minimum height of 225mm above the determined fill level for the subject property

   (ii) The floor level of non-habitable rooms must be at or above the determined fill level for the subject property.

   (iii) Septic Trench Installation

Septic trenches are to be constructed so that the surface level of the trenches are at or higher than the determined fill level of the subject property.

Where filling of land is necessary to facilitate the construction of a septic trench installation, only an evenly graded clean sand fill is to be used.

The following discretion may be exercised:

(1) For minor additions to an already existing building, Council may approve a floor level on a non-habitable building at or above RL 1.7 metres AHD.

(2) Septic trench surface levels of RL 1.7 metres AHD and higher may be approved as follows:

(a) Where a septic system is being added to an existing building or

(b) Filling of the subject land to increase the surface level to the determined fill level or higher may create drainage and seepage problems on adjacent parcels of land.

The discretion given in (1) and (2) will be subject to the following conditions:

(a) Owner signing a Statutory Declaration stating that the owner is aware of the possibility of drainage problems.

(b) The property notes for the subject property being noted so that future purchasers will be aware of the problem prior to purchase.

(3) In those instances where filling of the subject land to raise the surface level of septic trenches is required and Council considers that such filling may -

(i) Restrict the flow of floodwaters, tidal water or stormwater, or,

(ii) Increase flood levels on adjacent parcels of land, or

(iii) Create drainage and seepage problems on adjacent parcels of land,

Then Council may refuse the application or require that plans be amended to demonstrate how sanitary and sullage wastes are to be disposed of to the satisfaction of Council.

(d) Where filling of land is necessary to facilitate the construction of a concrete slab on ground type building to the levels specified in Clauses (a)(i) and (ii), (b)(i) and (ii) c(i) and (ii) above and Council considers that such filling may:

(i) restrict the flow of floodwaters, tidal water or stormwater or,

(ii) increase flood levels on adjacent parcels of land, or,

(iii) create drainage and seepage problems on adjacent parcels of land,

then Council may refuse the application or require that plans be amended to delete such filling and provide for the building floor level to be elevated above the natural ground level to comply with Clauses (a)(i), (b)(i) and (c)(i). In this case, the supporting structure must be designed to minimise the effects on d(i) and d(ii) above where this is relevant.

Appendix B

AAMHatch Digital Data
Documentation

DIGITAL DATA DOCUMENTATION

NORTH EAST BUSINESS PARK

DIGITAL TERRAIN DATA (CABOOLTURE REGION)

VOLUME 210131701NOB

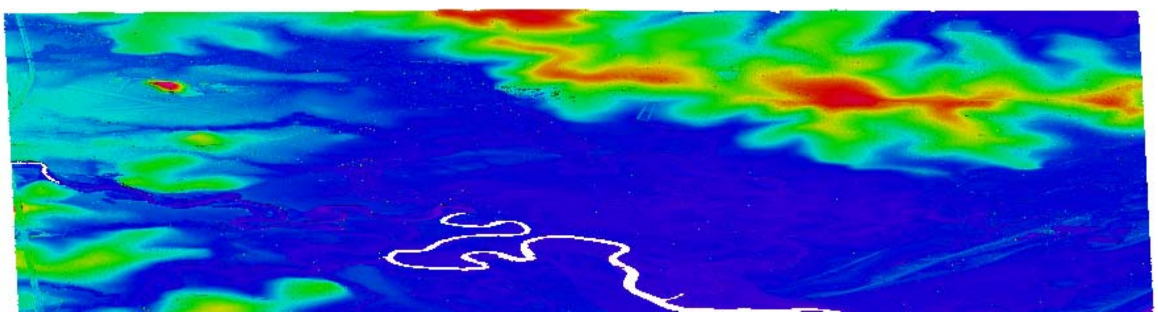
Summary

Project

Airborne Laser Scanning was collected over the Caboolture region between 23rd September 2005 and 14th October 2005. Data was collected without incident over approx. 8262 ha.

Data

Data files on this volume include;
Thinned ground points (XYZ) in space separated ASCII Files.



CONTENTS	Page Nos.

1. PROJECT REPORT

Acquisition: Airborne Laser Scanning (ALS) data was acquired from a fixed wing aircraft between 23rd September 2005 and 14th October 2005.

Ground Support: GPS base station support was provided by Landcentre VRS01 Woolloongabba without incident. The ground check points acquired by Jones Flint & Pike allowed an assessment of the accuracy of the ALS data.

Data Processing: Reduction of the ALS data proceeded without any significant problems. Laser strikes were classified as ground points and non-ground points were removed using a single algorithm across the project area. Manual checking and editing of the data classification against intensity imagery further improved the quality of the terrain model.

Data Presentation: The data provided on this volume has been supplied in accordance with a specification agreed with the primary client. Subsequent users experiencing difficulties in handling the data should please contact AAMHatch to arrange a more appropriate data presentation

Further Issues: There are no further issues to report.

2. DATA INSTALLATION

Data format	:	Space delimited ASCII
Number & type of media	:	One 650MB CD ROM
Number of files on media	:	30 GRD files (XYZ), 1 tile_system.DXF file and README.PDF
Data formatted on	:	19.10.2005
Disk volume	:	210131701NOB
AAMHatch Job Manager	:	Kerry Eastwick Ph 07 3620 3111

README FILE

This document (README.PDF) is provided as an Acrobat file in this volume.

To open the file, double click on the PDF file to activate Acrobat Reader Software.

Adobe Acrobat Reader may be downloaded from:
http://www.adobe.com/products/acrobat/readstep2.html

LOADING NOTES

Data may be copied using a file copy utility such as Windows Explorer or similar.

FILE SIZES AND NAMES

Data is provided in tiles 2km by 2km to the following filenaming convention:

eg. C4966996.grd	C-	project abbreviation
	496 -	coordinate easting (in thousands) of south west tile corner
	6996 -	coordinate northing (in thousands) of south west tile corner
	.grd -	classified as “Thinned ground”

A list of the files contained on this volume is provided in Section 7.

SAMPLE LISTING

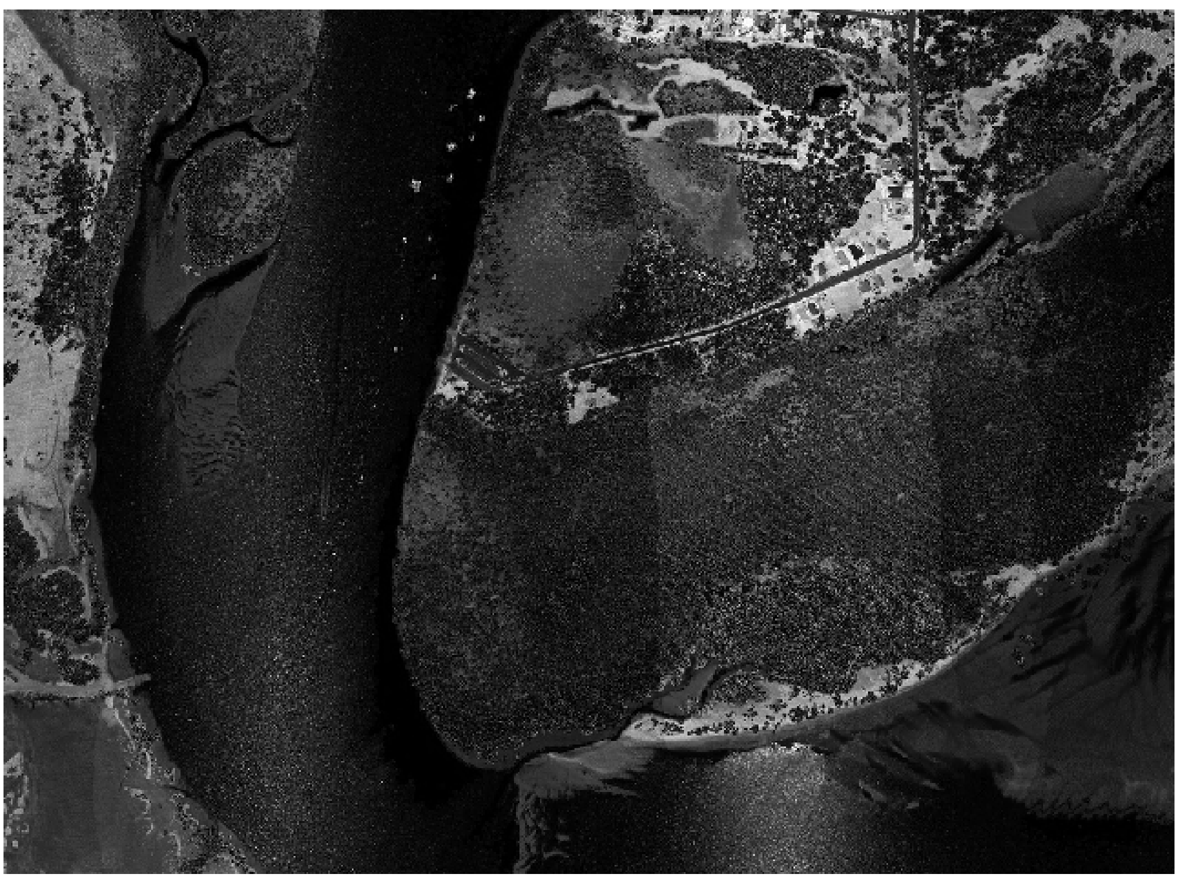
E	N	RL
497608.240	6998446.580	16.628
497610.250	6998446.590	16.848
497611.270	6998446.600	17.088
497616.240	6998446.570	16.668
497625.210	6998446.570	16.828
497628.210	6998446.590	17.168
497643.110	6998446.560	16.778
497648.070	6998446.560	16.878
497651.070	6998446.560	16.928
497661.010	6998446.550	16.939

etc.

3. ADDITIONAL SERVICES

AAMHatch can perform the following additional services on the data contained on this volume if required:

Change horizontal datum	:	to AMG or other local grid
Alter geoid modeling	:	by transforming ALS data to fit orthometric survey heights
Improve data classification	:	by tailoring parameters to suit regional variations
Further classification	:	Assist building identification by further classifying non-ground strikes
Data thinning	:	to remove superfluous points not adding to the terrain definition
Data subset	:	by dividing the data into different tiles or polygons
Data presentation	:	by creating contours, profiles, perspectives, flythroughs, colour-coded height plots etc.
Ground truthing	:	by comparing the ALS terrain model with extra independent height data
Data gridding	:	to convert the measured spot heights into a regular grid
Extra data	:	Extra data was collected beyond that supplied on this volume.
Intensity Image	:	Greyscale image created from laser's intensity returns. (sample below)



4. METADATA

DATA CHARACTERISTICS

Characteristic	Description
Format	Space delimited ASCII
Size	4800000 data points (approximate)
Captured terrain model	0.9m average point separation
Supplied terrain model	4.6m estimated point density, separated into ground & non-ground
Data thinning	Points not contributing to the terrain definition within 0.15m removed
Laser footprint size	0.24m

REFERENCE SYSTEMS

	Horizontal	Vertical
Datum	GDA94	AHD
Projection	MGA Zone 56	N/A
Geoid Model	N/A	Ausgeoid98
Reference Point	Landcentre	Landcentre
	6959847.6515 E	49.3481 RL
	503483.9814 N	
Survey Control	PSM103234	1.977 RL
	504511.795E 6998595.975N	

Note: On 01-01-2000, Australia formally changed its reference spheroid from AGD to GDA94, and its map grid from AMG to MGA. MGA coordinates are approximately 200m different from AMG. For more details including definitions of GDA compliance and GDA compatibility, visit : http://www.aamhatch.com.au/papers/GDA_Comp.pdf

GDA ***This data is GDA-compliant***

SOURCE DATA

	Source	Description	Ref No	Date
Laser scanning	AAMHatch	70,000 Hz	2101317	23/09-14/10.05
GPS base data	AAMHatch	Static GPS	2101317	23/09-14/10.05
Base Stn coords	Landcentre	Published Value	2101317	23.09.05
Test points	JF+P	Total station	2101317	10.10.05

EXPECTED ACCURACY

Project specifications and technical processes were designed to achieve data accuracies as follows:

	Measured Point	Derived Point	Basis of Estimation
Vertical data		0.15	Deductive estimate
Vertical data	0.113		Comparison with 143 test points
Horizontal data		<0.37	Deductive estimate (1/3000 flying height)

Notes On Expected Accuracy

- Values shown represent standard error (68% confidence level or 1 sigma), in metres
- "Derived points" are those interpolated from a terrain model.
- "Measured points" are those observed directly.
- Accuracy estimates for terrain modeling refer to the terrain definition on clear ground. Ground definition in vegetated terrain may contain localised areas with systematic errors or outliers which fall outside this accuracy estimate
- Laser strikes have been classified as "ground", based upon algorithms tailored for major terrain/vegetation combinations existing in the project area. The definition of the ground may be less accurate in isolated pockets of dissimilar terrain/vegetation combinations.

LIMITATIONS OF DATA

- Features depicted are as shown on the legend.
- The definition of the ground under trees may be less accurate.

DATA VALIDATION

- Ground data in this volume has been compared to 143 test points obtained by field survey and assumed to be error-free. The test points were distributed in 1 group across the mapping area and located on clear ground. Comparison of the test points with elevations interpolated from measured data resulted in:
  Standard Error (RMS): 0.113m

- Data classification has been manually checked and edited against any available imagery.

USE OF DATA

- Intended use : Planning, Conceptual Design

5. CONDITIONS OF SUPPLY

The data in this volume has been commissioned by **NORTH EAST BUSINESS PARK**.

The data in this volume is provided by AAMHatch Pty Limited (AAMHatch) to **NORTH EAST BUSINESS PARK** under AAMHatch standard Terms of Engagement, which provide a license for **NORTH EAST BUSINESS PARK** to access and use the data only for the project and explicit purpose for which it is provided. AAMHatch retains ownership of all Intellectual Property Rights in relation to this data or modifications, enhancements or subsets of this data. The data must not be sold, lent or distributed to any other party; and used subject to the following conditions:

1. This file (README.PDF) is always stored with the unaltered data contained in this volume.

2. The data is not altered in any way without the approval of AAMHatch. The data may be copied from this file to another.

3. The data is not used for purposes beyond that explicitly agreed in the description of the Services provided by AAMHatch.

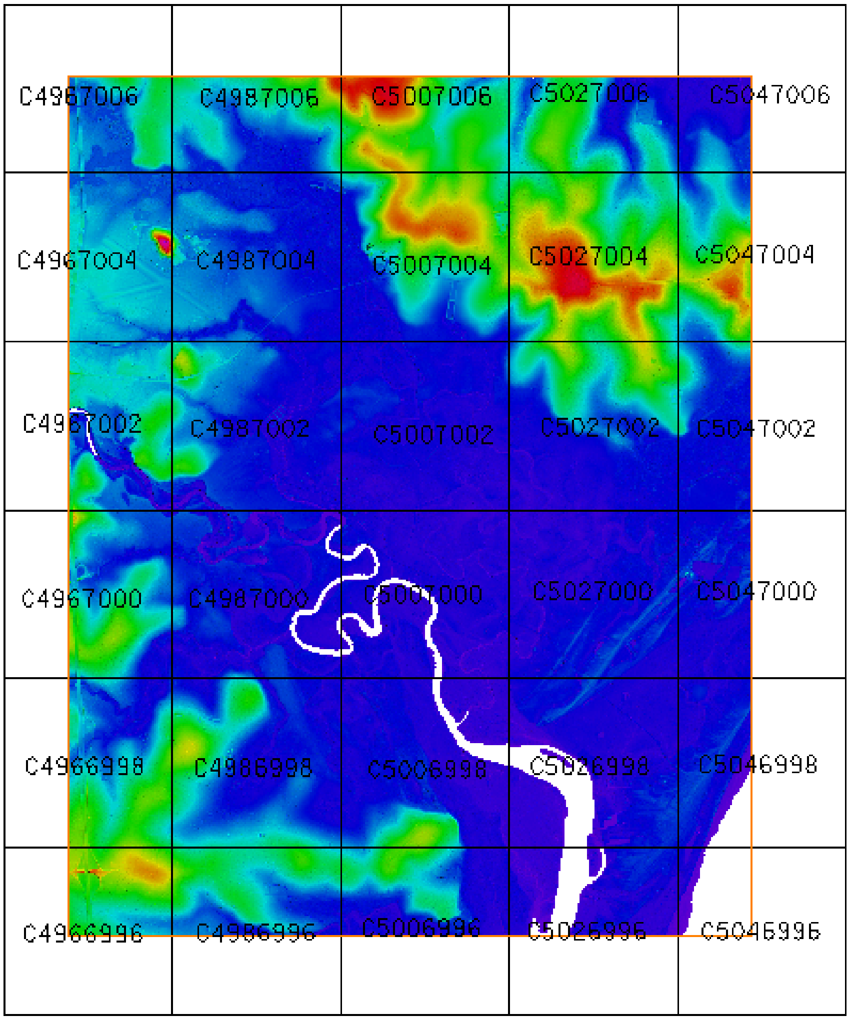
Any breach of these conditions will result in the immediate termination of the license issued by AAMHatch, and **NORTH EAST BUSINESS PARK** will indemnify AAMHatch from all resulting liabilities.

Any problems associated with the information in the data files contained in this volume should be reported to:

AAMHatch Pty Limited

16 Julia St,
FORTITUDE VALLEY QLD 4006
Telephone (07) 3620 3111
Facsimile (07) 3620 3133
Email info@aamhatch.com.au
Web www.aamhatch.com.au

6. VALIDATION PLOT



7. FILES SUPPLIED

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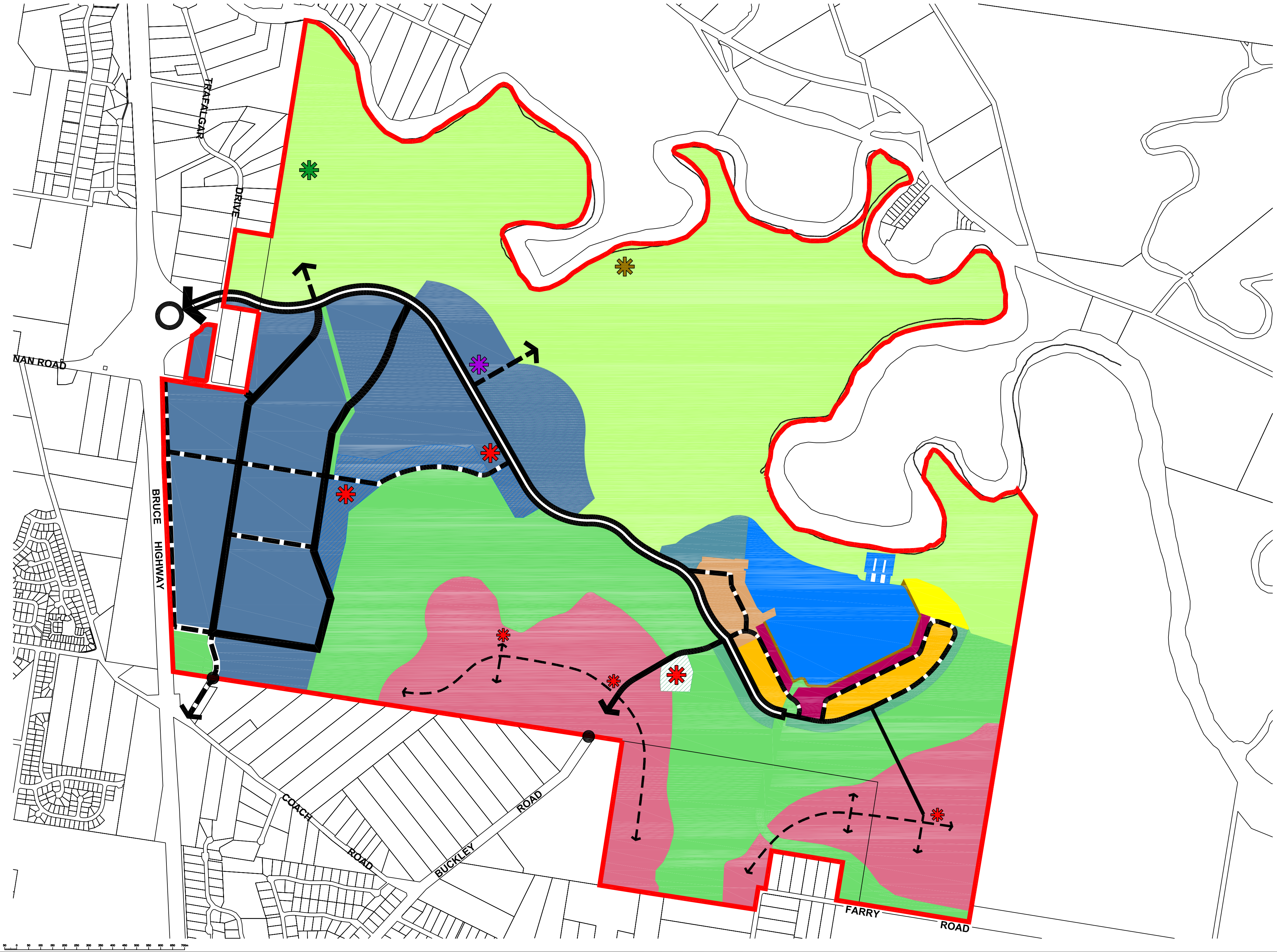
Appendix C

Northeast Business Park
Cut and Fill Drawing



LEGEND

- STRUCTURE PLAN BOUNDARY
- 1(1) MIBA ESPLANADE
- 1(2) MIBA CORE
- 1(3) MIBA HIGHWAY
- 1(4) MIBA SOUTH
- 1(5) MIBA MARINE INDUSTRY
- 2(1) MARINA BASIN
- 2(2) SHIPYARD
- 2(3) MARINA CENTRE
- 2(4) MARINA RESIDENTIAL
- 2(5) HOTEL
- 2(6) MARINA APARTMENTS
- 2(7) MARINA PAVILION
- 2(8) GOLF RESIDENTIAL
- 3(1) RESIDENTIAL
- 3(2) RESIDENTIAL
- 3(3) RESIDENTIAL
- 4(1) GOLF CLUB
- 4(2) GOLF COURSE
- 4(3) OPEN SPACE
- COMMUNITY NODE
- HERITAGE PARK
- SPORTING FIELDS
- EDUCATION & TRAINING
- PRIMARY ROUTE
- COLLECTOR
- MINOR COLLECTOR
- LOAD LIMITED THRESHOLD

CLIENT

NORTHEAST BUSINESS PARK

PROJECT

PROPOSED DEVELOPMENT

NOLAN DRIVE
MORAYFIELD
STRUCTURE PLAN

Level Datum	
Origin	
Date	30 JULY 2007
Surveyed	
Comp By.	KCH
DWG Name.	20430STRUCTURE
Local Authority	CABOOLTURE SHIRE COUNCIL
Job Reference	20430
Scale	1:5000@A0
Plan Reference	**20430-10D**

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