Government of **Western Australia**
Department of **Water and Environmental Regulation**

East of Kwinana flood modelling and drainage study

Supporting local water management and future development

Department of Water and Environmental Regulation

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Cover photograph: Drone photo looking south with Duck Pond on the left and Peel Main Drain on Kwinana Freeway's right (F. Tong).

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Summary

The East of Kwinana and Pinjarra-Ravenswood planning investigation area (PIA) was identified in the Perth and Peel @ 3.5 million frameworks (Western Australian Planning Commission 2018) as requiring further investigation of drainage and flood risk. In 2019, the Department of Planning, Lands and Heritage (DPLH) asked the Department of Water and Environmental Regulation (the department) to conduct a land capability assessment based on flooding. The assessment will inform DPLH's analysis of East of Kwinana and Pinjarra-Ravenswood PIA.

This report should be read in combination with the department's report titled *East of Kwinana and Pinjarra-Ravenswood planning investigation areas: Flood risk management land capability assessment* (flood risk management land capability assessment; DWER, 2021).

This report is a technical document that outlines the development and application of the East of Kwinana flood models. The objective is to understand the flood behaviour under existing catchment land use and potential development scenarios through floodplain mapping. The process and decision-making that underpins the modelling scenarios are documented in the flood risk management land capability assessment (DWER, 2021).

This report outlines the development and application of the East of Kwinana flood models and includes the following key components:

- an overview of the catchment hydrology
- an outline of the hydrological study to determine the inputs and boundary conditions for the hydraulics modelling
- an introduction to the development of hydraulic models
- a discussion on the hydraulic modelling for the current land-use and watercourses at critical locations within the study area (pre-development modelling); this component involves the use of the flood models for floodplain mapping and flood behaviour investigation
- an investigation on the hydraulic modelling under potential development scenarios (post-development modelling).

1 Introduction

The East of Kwinana area was identified as one of the sectors in the East of Kwinana and Pinjarra-Ravenswood planning investigation area (PIA) within the Perth and Peel @ 3.5 million frameworks' suite of land-use plans, released by the Western Australian Planning Commission in 2018.

One of the considerations for the PIA is inundation and flood risk management. The Department of Water and Environmental Regulation (the department) is the lead agency in floodplain management in Western Australia (WA). The Department of Planning, Lands and Heritage (DPLH) sought the department's advice on drainage and flooding within the PIAs.

The East of Kwinana sector (East of Kwinana) is about 20 km south of the Perth CBD and about 5 km east of Rockingham (Figure 1-1). The catchment area covers the Swan Coastal Plain east of the Kwinana Freeway between Byford and Mundijong and extends into the Darling Scarp in the east.

The PIA surrounding area is within the Swan Coastal Plain (McArthur and Bettenay, 1974) and is characterised by flat terrain (except a small portion on the west side) and high groundwater tables. The area is known to experience groundwater inundation and periodic flooding. Water Corporation has constructed a network of drains within the study area. The objective of this network is to drain the inundated regions within 72 hours of rainfall (Hall, 2015a). However, the drains are not designed as primary flood mitigation works. There are elevated banks along the major drains with unknown geotechnical stability which should not be relied upon to withstand significant flooding (Marillier, 2015).

East of Kwinana is covered by two existing flood studies by the department for the Birrega and Oaklands Catchment Hall (2015a) and the North East Baldivis Catchment (Marillier, 2015) (Figure 1-2). These two models were the basis of this study and merged into a more sophisticated, larger East of Kwinana model. The East of Kwinana study:

- uses approaches and parameters consistent with the 2019 edition of Australian rainfall and runoff: A guide to flood estimation (ARR; Commonwealth of Australia)
- covers the entire East of Kwinana sector of the PIA and the contributing catchments
- uses a consistent model resolution for East of Kwinana
- uses enhanced flood modelling software to enable increased model resolution to better represent existing overland flow paths and channels on the floodplain and possible development scenarios within the PIA.

This work (Table 1-1) developed a regional flood model to study the flood behaviour for most of East of Kwinana (Figure 1-1). The model is based on the ARR released by Ball et al. (2019) and a flexible mesh framework. Variable grid resolution was employed with a mesh of acceptable resolution defining the watercourses. A river model was used to simulate the flows inside significant drains. The coupled flood-modelling platform was used to understand the flood behaviour of East of Kwinana. The regional model compared flood behaviours of several conceptual post-development scenarios with that of flood behaviours modelled under existing development. This identifies any impact on flood risk to areas upstream and downstream of East of Kwinana.

This study also develops a local flood model focusing on North East Baldivis north of Mundijong Road, referred to as PIA-Baldivis North in Figure 1-1. The design of the local model was to enable post-development options to be stamped onto the existing bathymetry. A finer resolution than that of the regional model was adopted in the smaller local model, which increased flexibility to represent the post-development drainage within East of Kwinana. The level of detail for defining drainage features within the local model may be helpful in modelling additional options for the area at future planning stages.

Table 1-1 Framework of flood modelling (this report) for the East of Kwinana

Aims	**Investigation**	**Section**
Model development	Model conceptualisation, data collation, hydrology study and hydraulic modelling	§2–§5
PIA scenario 1 flood modelling: North East Baldivis – North of Mundijong Road	○ Pre-development base case ○ Development Option A: Maximise land above the 1% annual exceedance probability (AEP) flood with an engineered levee ○ Development Option B: Maximise land above the 1% AEP flood with flood waterways ○ Development Option C: Maximise land above the 1% AEP flood with an industrial land option	§6 §7.2 §7.4 §7.6
PIA scenario 2 flood modelling: North East Baldivis – South of Mundijong Road	○ Pre-development base case ○ Development Option: Maximise land above the 1% AEP flood with flood waterways	§8.1 §8.2

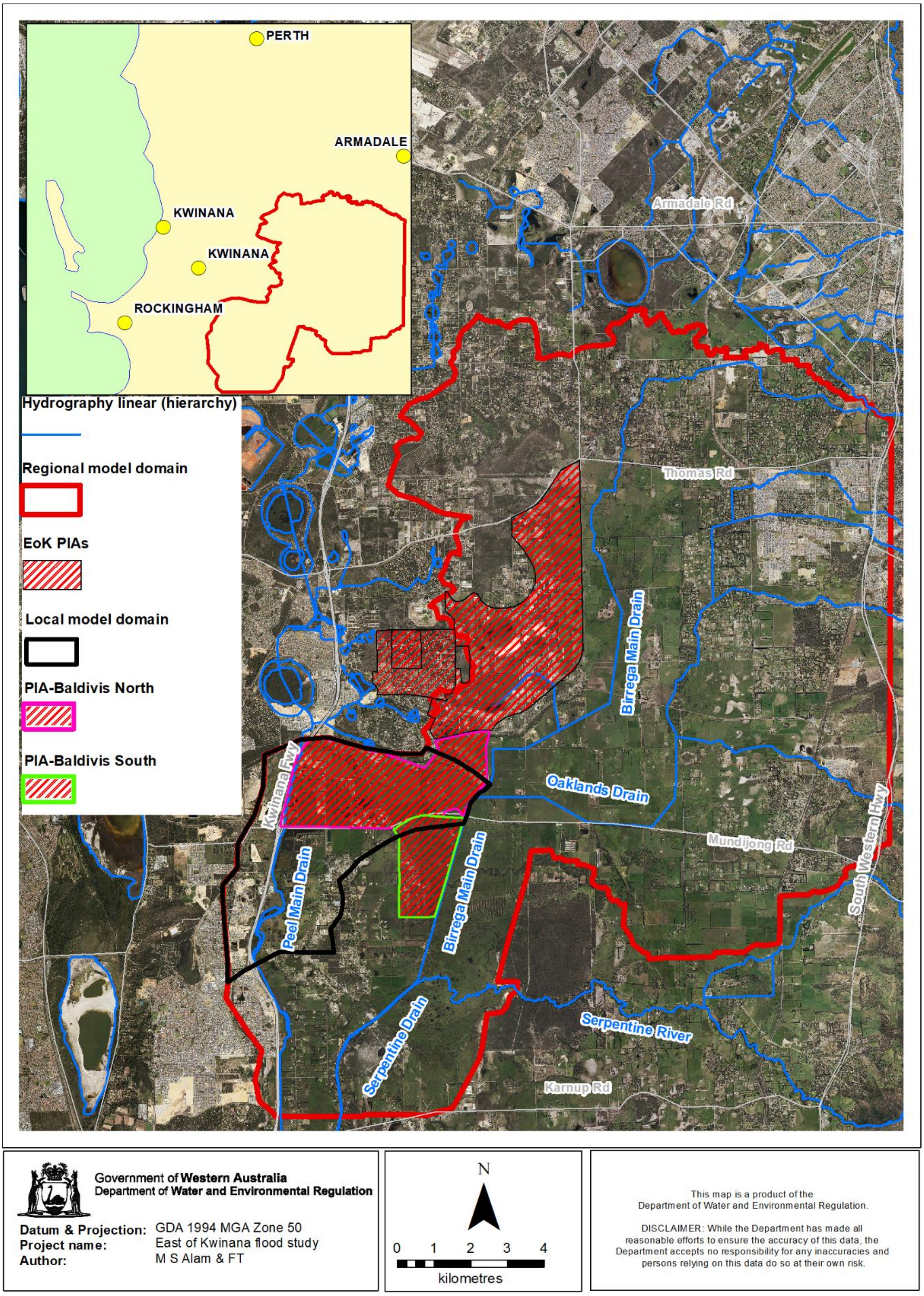


Figure 1-1 Location of the East of Kwinana flood study areas (regional and local models boundary)

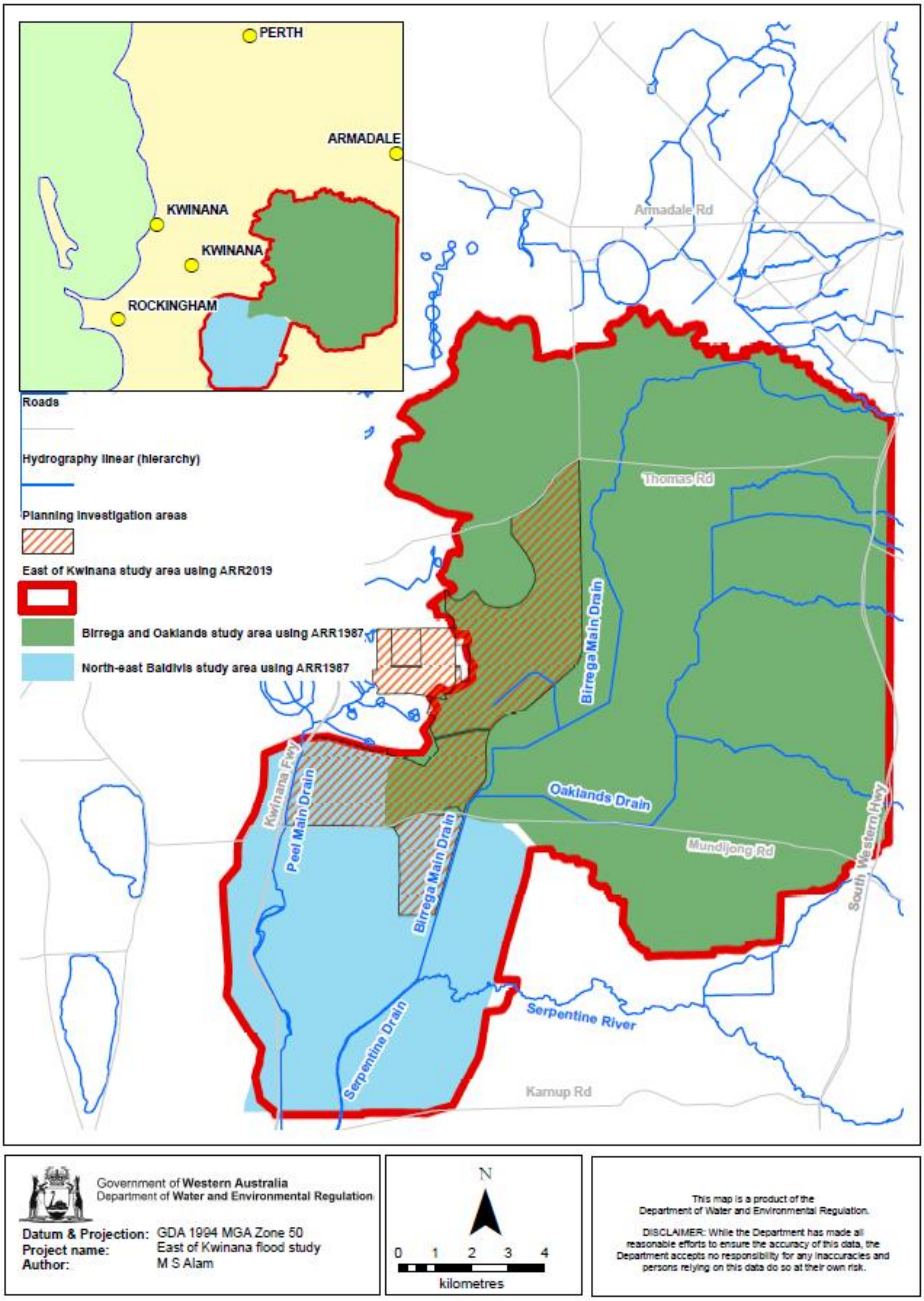


Figure 1-2 Location of the existing flood-model areas and the current study area

1.1 Purpose

This study aims to understand the design flood behaviour under existing catchment land-use to assess potential development plans. The purpose of the modelling work is to inform decision-making at the regional and subregional planning level as part of East of Kwinana, including North East Baldivis north of Mundijong Road (PIA-Baldivis North) and North East Baldivis south of Mundijong Road (PIA-Baldivis South) in Figure 1-1.

The modelling will provide information on flooding for DPLH to consider as part of a broader land capability assessment. The objective is to indicate how much land could be developed in East of Kwinana based on different land-uses (and potential flood standards) and form part of comparative analysis to develop this location versus other possible locations.

Through investigations of different development scenarios for North East Baldivis north of Mundijong Road and North East Baldivis south of Mundijong Road in Figure 1-1, the modelling work aims to answer two critical questions from a flooding perspective:

1. Can the precinct be developed without unacceptable impacts on neighbouring land?
2. How much land can be provided at an appropriate flood standard for the land-use that is being sought?

The flood modelling focus is designed to broadly assess the amount of land able to be developed with flood protection and is not aimed at detailed drainage design or district-level planning.

This study indicates the incremental risks and benefits of additional engineering works and the potential to increase the amount of land with flood protection through engineering optimisation at the district planning stage.

1.2 Approach

The ARR (Geoscience Australia, 2019) is a national guideline document and software for estimating design flood characteristics in Australia. The department followed these guidelines to model the flood risk to the PIA by:

- assessing catchment characteristics and collating data
- hydrologic assessments
    - hydraulic modelling
    - calibration
- design scenarios – existing land-use, possible development scenarios for the PIA
- recommendations.

This report includes:

- Sections 1 and 2 provide an overview of the catchment hydrology and available data, including a literature review on relevant previous studies, data collection on rainfall, streamflow, terrain and land-use, and groundwater and structure.

- Section 3 studies the area's hydrology to determine the inputs and boundary conditions for the hydraulics modelling. This section presents essential information related to decision-making on land-use at the regional level at East of Kwinana . The data also forms part of the land capability assessment.
- Section 4 introduces the development of the regional and local hydraulic models, including the mesh, boundary conditions, rainfall and discharge inputs, model calibration and validation. This component involves the development of flexible mesh hydraulic models and a river model for the study area, including:
  - the collection data on the dimension of bridges, crossings, culverts, and other structures
  - the collection of rainfall, water level and flow datasets for the gauging stations and surveyed and anecdotal peak flood levels within the study to calibrate the hydraulic model
  - the development of a MIKE 21 FM model for the floodplain and MIKE 11 river network model for significant rivers and drains, and their integration if appliable.
- Section 5 discusses the hydraulic modelling based on the current land-use and watercourses and presents flow at critical locations. It aims to understand the present flooding constraints within East of Kwinana. It provides an overview of the land capability and potential flood standards and forms part of comparative analysis on flooding behaviour at different locations. This component involves the use of the flood models for:
  - floodplain mapping for a range of design events including the 10% AEP, 5% AEP, 1% AEP and 0.5% AEP design storm events
  - flood behaviour in the event of a potential spoil-bank breach under existing land-use
  - assessment of using a smaller, local flood model with inputs from a broader regional model.
- Section 6 aims to estimate the amount of land with flood protection that could realistically be achieved based on flooding constraints at the PIA-North East Baldivis north of Mundijong Road (Figure 1-1). It involves:
  - preparing indicative percentage of developable area to facilitate possible development within the PIAs
  - ensuring existing land and development outside the regions is not detrimentally impacted.
- Section 7 aims to estimate the developable area based on flooding constraints at the land of North East Baldivis south of Mundijong Road precinct.

1.3 Catchment and drainage

The study area for the hydraulic model covers 232.7 km^2. It sits within the Swan Coastal Plain (McArthur and Bettenay, 1974), bounded by the Darling Scarp in the east, the Wungong catchment to the north, the Kwinana Freeway to the west, and the Bollard Bulrush Swamp to the south. The land is characterised by sandy soils and flat terrain, with extensive floodplains incised by deep and narrow rural drains, significant waterlogging in winter and flood risks in some areas (Hall, 2015a).

The broader study area includes the catchments of the upper Peel Main Drain, the Birrega Main Drain, the Oaklands Main Drain, and the Serpentine River/Drain. Figure 1-3 shows the hydrological features of the study area.

Birrega Main Drain

The Birrega Main Drain is a significant waterway for most study areas (Figure 1-2 and Figure 1-3). The drain starts from the northeast border and runs about 27 km before merging into the Serpentine Drain at a location 5 km from the southern border.

The Birrega Main Drain was carefully documented by Hall (2015a) through field assessments. Based on that study, the main features are:

- It begins as a 1m-deep offtake of the Wungong Brook just downstream of the South Western Highway and expands to a massive drain at Hopkinson Road (about 2 m deep and 10 m wide).
- It expands to a width of 40 m at Mundijong Road.
- The upper reaches are heavily vegetated and do not have significant levee or spoil banks adjacent to the channel. The drain has banks of unequal height on the left and right sides downstream of Orton Road. The term of spoil banks is used to refer to these existing banks.
- The spoil banks have a series of breaks that allow lateral flow from adjacent channels, which link the drain to flood storage areas.
- During the 1987 flood event, a breakout around Duck Pool resulting in flood flows in a westerly direction through North East Baldivis north of Mundijong Road precinct towards the Peel Main Drain.

Oaklands Main Drain

The Oaklands Main Drain initially runs north south, then flows eastward at Mundijong Road before merging into the Birrega Main Drain. Hall (2015a) documented the main features of the Oaklands Main Drain.

- It receives lateral inflows from catchments with headwaters in the Darling Scarp, through Manjedup Brook, Cardup Brook and Beenyup Brook, where they traverse the Swan Coastal Plain in an east–west direction and discharge to the Oaklands Main Drain.
- Most of the north–south section of Oaklands Main Drain has a western levee significantly higher than the eastern levee.

- This drain expands to about 25 m at its downstream end, where it discharges to the Birrega Main Drain about 600 m north of Mundijong Road.

Peel Main Drain

Several studies investigated the Peel Main Drain (PMD), including SKM (2010b) and Marillier (2015). The entire catchment of the Peel Main Drain sits on the Swan Coastal Plain. It includes various land-uses, with the lower section of the catchment comprised primarily of rural residential and grazing land.

The PMD catchment follows a series of wetlands and includes urban residential, native vegetation, horticultural and rural residential land-uses. Soils within the area are predominantly sandy; however, wetland and alluvial sediments along the drain can have high clay content in some locations. The main features of the PMD drain (Marillier, 2015), in the perspective of this flood study, are:

- The upper section of the Peel Main Drain is external to the hydraulic study area, with a catchment area of 58 km^2. The hydraulic model includes the middle section of the Peel Main Drain. The Peel Main Drain eventually joins the Serpentine River around 3 km south of Karnup Road.
- The PMD intersects a series of wetlands, including Mandogalup Swamp, the Spectacles Wetlands, Alcoa Wellard Wetlands and Maramanup Pool. These wetlands intersect shallow groundwater in winter, receive a baseflow contribution in most winter events, and provide extensive storage areas along the drain.
- The Peel Main Drain has a shallow hydraulic grade within the hydraulic model area, dropping roughly 2 m.
- These two factors (storage area and hydraulic grade) effectively reduce peak flows, increase rainfall response times, and lower peak flow velocities. As such, rainfall events that produce broad peak flows in waterways with their headwaters in the scarp may make only a small response in the Peel Main Drain.

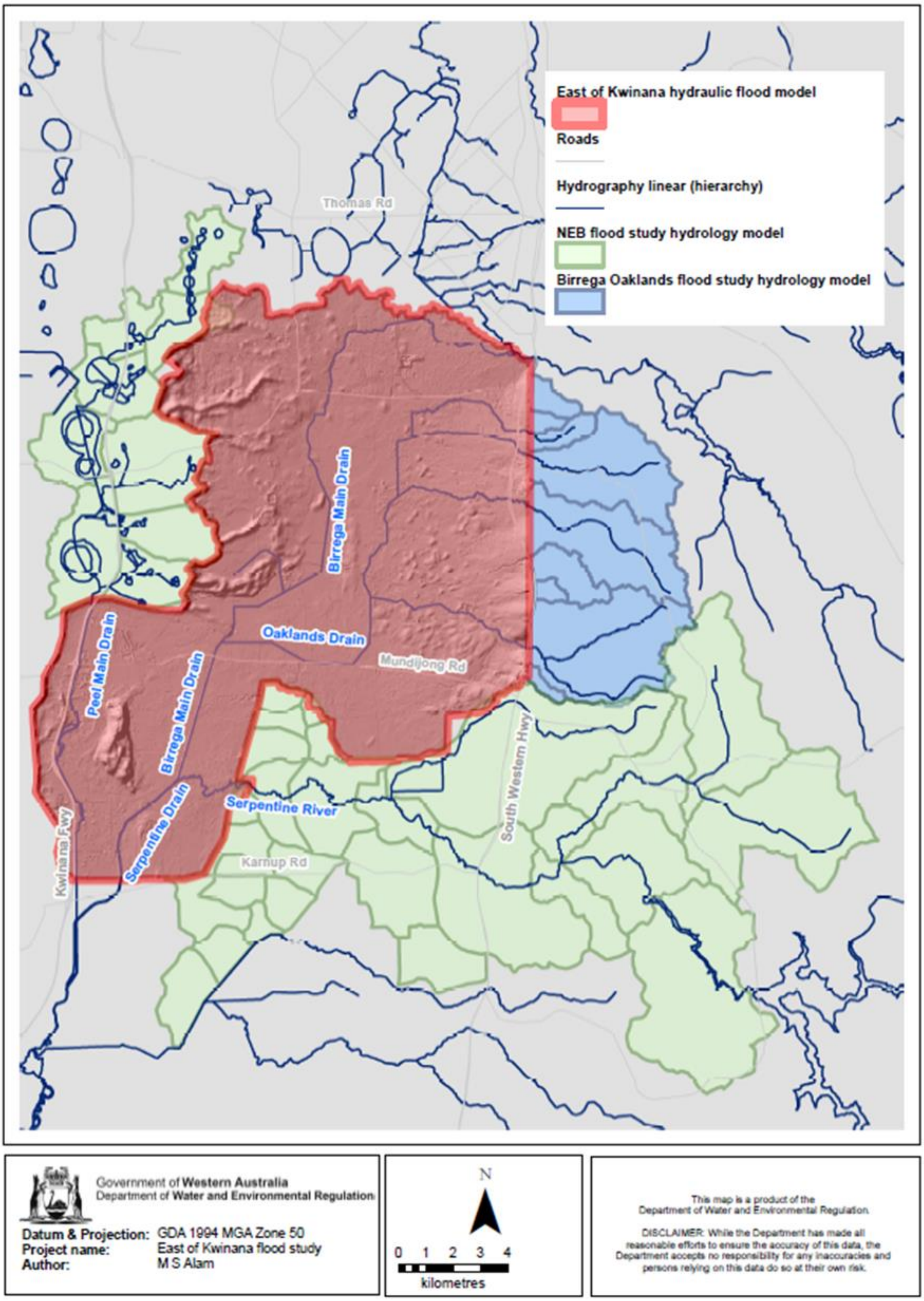


Figure 1-3 The East of Kwinana hydraulic model and the contributing upper North East Baldivis and Birrega and Oaklands hydrology catchment domains

1.4 Literature review

Many essential investigations on flooding and hydrology have been undertaken in areas that intersect with East of Kwinana. Table 1-2 lists some of these studies chronologically.

Table 1-2 Flood and hydrology studies that interact with the study area of the present report

Authors (year)	Area	Title and summary
Durrant and Bowman (2004)	WA	**Title**: Estimation of Rare Design Rainfalls for Western Australia: Application of the CRC-FORGE Method **Summary**: The report derived seasonal and annual design rainfall estimates from an AEP of 1 in 50 to 1 in 2000 and for durations of between 24 and 120 hours. It was based on applying the approach of CRC-FORGE (Cooperative Research Centre - Focussed Rainfall Growth Estimation) to WA. The CRC-FORGE approach was a regional frequency analysis technique developed by the UK Institute of Hydrology for Catchment Hydrology.
Brookes (2008)	Byford town (about 15 km^2)	**Title**: Byford townsite drainage and water management plan **Summary**: This study presented a floodplain management strategy for Byford and was carried out by SKM on behalf of the then Department of Water. The study identified floodway and flood fringe areas by two-dimensional modelling. The floodplain management plan includes flood mitigation focused on managing potential flooding impacts on the site and the immediate neighbouring land and drainage infrastructure.
Brookes (2009)	Jandakot and Peel Main Drain (about 59 km^2)	**Title**: Jandakot drainage and water management plan. **Summary**: The study developed a 1D hydraulic model of the Peel Main Drain using InfoWorks CS. Design rainfall events for 1, 3, 6, 12, 24, 48 and 72-hour durations were simulated using 2-, 10- and 100-year average recurrence interval (ARI) events. Results were used in the design of the stormwater drainage system.

Authors (year)	Area	Title and summary
SKM (2010b)	Catchments of Serpentine River, Birrega Main Drain, and Peel Main Drain	**Title**: Serpentine River Floodplain Management Study – Flood Modelling Report. **Summary**: A hydraulic model of the area was developed using MIKE 21 with a 1 m grid of elevation data points and a grid resolution of 12 m. The study did not explicitly model hydraulic structures. Inflows to the hydraulic model were developed using the RORB hydrologic models. The hydraulic model was used for flood mapping for the 10, 25, 100- and 500-year ARI events. The study found some drains have less than 100-year ARI capacity. Therefore, a levee break scenario was investigated, in which the western levees for the Serpentine and Birrega main drains were removed. It showed a significant increase in the extent of inundation in these two areas.
SKM (2010a)	Serpentine, Baldivis, Karnup and Keralup area	**Title**: Serpentine River Floodplain Management Study – Floodplain Management Strategy. **Summary**: This Floodplain Management Strategy was based on detailed hydrologic and hydraulic modelling and floodplain mapping for the 100-year ARI flood event. The report identified floodway and flood fringe areas and based on the inundation feature of the floodplain, recommended a management strategy for the proposed development.
Marillier et al.(2012)	Lower Serpentine area	**Title**: Lower Serpentine hydrological studies – Conceptual model report. **Summary**: The study developed a conceptual model of groundwater and surface water within the Serpentine study area, which: a) described the local hydrology and climate; b) developed a geological model of the study area; c) defined the aquifer systems and major hydrogeological processes; and d) provided a numerical water balance that includes all significant groundwater and surface water processes.

Authors (year)	Area	Title and summary
Marillier, Hall and Kretschmer (2012)	Lower Serpentine area	**Title**: Lower Serpentine hydrological studies – Model construction and calibration report. **Summary**: This report develops a regional transient surface water and groundwater model over the Lower Serpentine study area. The model was constructed with the MIKE SHE modelling framework, using geological, hydrogeological, hydrological, soil and land-use information. The model simulates rainfall and evapotranspiration, unsaturated zone, saturated zone, channel flow, overland flow and abstraction. Calibration of the Leederville and Rockingham aquifers achieved a scaled root mean square of 3.8% and an average absolute error of 0.80 m.
Marillier, Hall and Kretschmer (2015)	Lower Serpentine area	**Title**: Lower Serpentine hydrological studies – Land development, drainage, and climate scenario report. **Summary**: This final report of three discussed the scenario modelling on climate and land development scenarios at a regional scale. Three scenarios were implemented in the model: future climate scenarios, development scenarios and subsurface drainage scenarios.

Authors (year)	Area	Title and summary
Hall (2015a)	Birrega and Oaklands main drains catchments (about 185 km^2)	**Title**: Lower Serpentine hydrological studies – Conceptual model report. **Summary**: A coupled 1D-2D hydraulic model in MIKE FLOOD was developed for a 176 km^2 area. The study also consisted of flood frequency analysis and RORB modelling for external inflows. The model used the direct-rainfall technique to simulate cross-catchment flows and rainfall-runoff within the hydraulic model domain. The model achieved a satisfactory calibration at the Birrega flow gauging station. The hydraulic model was used to simulate 5, 10-, 20-, 100- and 500-year ARI events for 6, 12, 24, 36, 48 and 72-hour. A levee failure scenario in which the left and right levee banks on the Birrega and Oaklands main drains were removed was also simulated. The 100 year and 500-year ARI events showed that the western levee bank of the Birrega Main Drain would overtop near Duck Pool, redirecting flow through the North East Baldivis area towards the Peel Main Drain.
Marillier (2015)	North East Baldivis and the Peel Main Drain (about 58 km^2)	**Title**: Lower Serpentine hydrological studies - Conceptual model report. **Summary**: The hydraulic model was built using the MIKE FLOOD modelling package, including a 2D hydraulic model coupled with a 1D hydraulic model. The calibrated model was used for simulating design floods for the 5, 10-, 20-, 100- and 500-year ARI events, for durations of 6, 12, 24, 36, 48 and 72-hour. A levee fail scenario was simulated, assuming that the western bank along the Birrega and Serpentine main drains were absent. The 100-year 24-hour simulation found water overtopping from the Birrega Main Drain at Duck Pool. Detailed floodplain mapping was presented.

1.5 Flood models

The MIKE FLOOD software, developed by DHI (Danish Hydraulic Institute), was used in the study, because of its proven performance in Water Engineering. MIKE FLOOD simulates river–floodplain interaction, integrated urban drainage, river modelling and, urban flood analysis. It dynamically couples one-dimension (1D) river model (MIKE 11) and two-dimension (2D) floodplain (MIKE 21) modelling techniques into one single tool.

MIKE 11 is a one-dimension river model simulating flow and water level, water quality and sediment transport in inland water bodies.

MIKE 21 is a two-dimension model simulating inland flooding, overland flows, waves, sediments and, ecology in coastal areas.

A range of modules and methods in MIKE FLOOD were used including a flexible mesh overland flow solver and the rain-on-grid approach. The latter applies rainfall directly on the 2D grid. Once the rain is applied to a catchment domain's grid cells, accurate overland flow routing is possible within the 2D fully dynamic hydraulic model, using shallow water equations. The 2D shallow water equations for overland flow comprise both conservation of mass and conservation of momentum (Johnson, 2013; Hall, 2015b).

2 Model conceptualisation

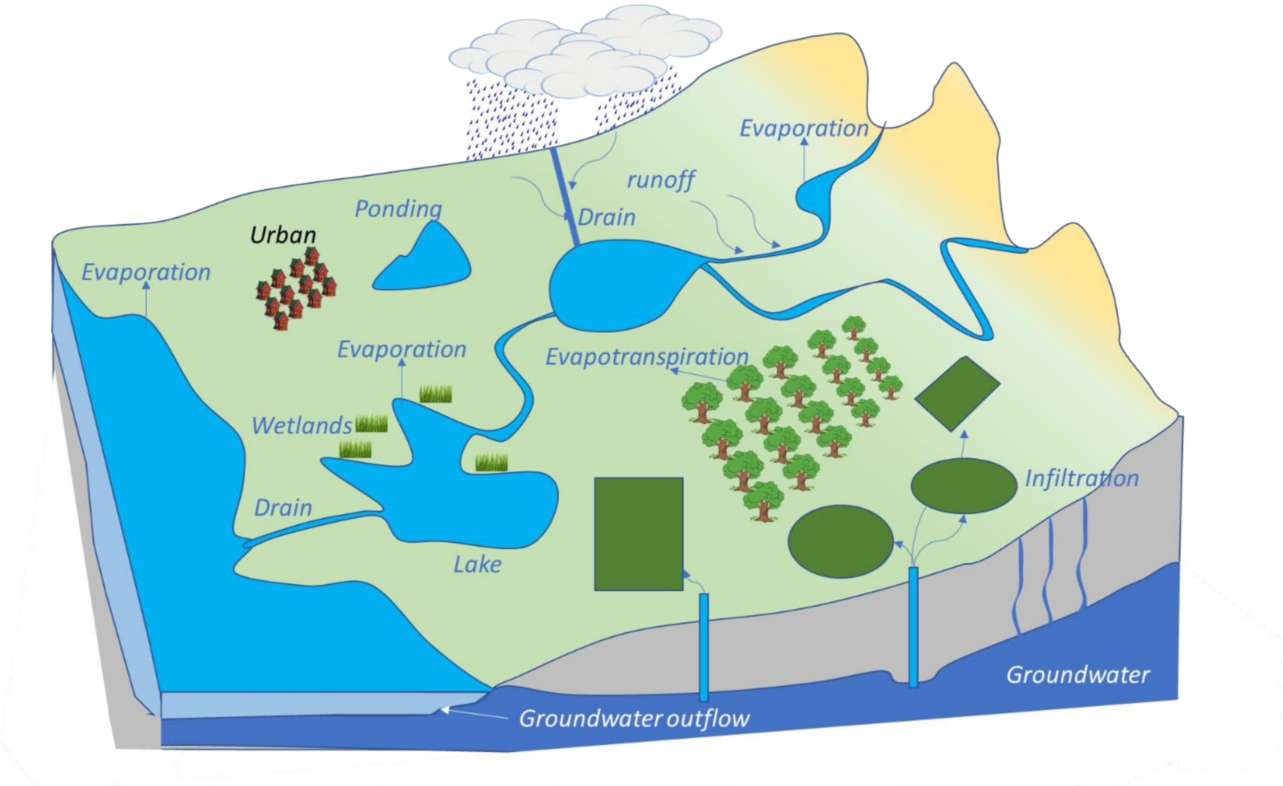


Figure 2-1 The East of Kwinana flood model conceptualisation (Cloud, Town, Forest, Grass based on various sources)

Flooding because of heavy rainfall occurs predominately when the waterways cannot timely drain away the amounts of rain that are falling (Geoscience Australia website). For East of Kwinana, factors that can contribute to flooding include:

- rainfall: its volume, spatial distribution, intensity, and duration over the catchment
- conveyance capacity: the capacity of the drains, waterways, lakes, and other storages to carry runoff
- topography
- spoil banks
- catchment conditions, including the groundwater water level and how wet the catchment is before rainfall
- ground cover.

Rainfall

The volume, intensity, and duration of rainfall affect runoff within a catchment. Infrastructure design, including stormwater drains and flood mitigation levees, rely on estimated rainfall depth for a specified probability. This study employs the latest ARR (2019) to derive design rainfalls in the modelling. It should be noted that design rainfalls are not natural or observed rainfall events; they are probabilistic values representing nature.

Conveyance capacity

The Peel Main Drain, Oaklands Main Drain, Birrega Main Drain are the main waterways within the study area. These are heavily modified drainage channels that are operated and maintained by Water Corporation as a rural drainage network. Although this drainage network may provide some flood mitigating benefits, it is not designed or constructed to provide flood protection. The function of the drainage network is to drain water ponding on rural land within 72 hours of a rainfall event. Observations from past floods and previous flood studies have shown that the rural drainage network should not be relied upon to protect adjacent areas from flooding (Hall, 2015a; Marillier, 2015).

In addition, the Birrega Main Drain (north-south) is not aligned with the natural direction of flow (east-west), which has increased the complexity of flooding. In particular, when the conveyance capacity of the Birrega Main Drain is exceeded, floodwaters will overflow into the North East Baldivis precinct, which then flows westward naturally.

Spoil banks

Both banks of the Birrega Main Drain are expected to be under extra pressure in significant flood events induced by floodwater's lateral impact, since the Birrega Main Drain (north-south) is not aligned with the natural direction of flow (east-west). The finite carrying capacity of the Birrega Main Drain result in water levels approaching and sometimes exceeding the levels of one (or both) of the spoil banks. In these infrequent flood events, the spoil banks are under increased pressure and susceptible to possible failure by overtopping and/or erosion. Anecdotal information of flooding in 1987 confirm spoil bank failures occurred during this event (Hall, 2015a; Marillier, 2015). The exact timing and location of spoil bank failures is difficult to predict with confidence. However, there is an increased likelihood of failure at locations where the conveyance capacity of the Birrega Main Drain is exceeded. One such location is the Birrega Main Drain near the confluence with the Oakland Drain, where floodwaters overflow into the North East Baldivis north of Mundijong Road precinct, which then flows eastward naturally.

Spoil banks adjacent to the main drains are an important feature of the floodplain in the Birrega, Oaklands and Peel Main Drain catchments. The design, construction and maintenance of the spoil bank system indicate that an uncontrolled failure of the banks in a large flood event is likely which will cause a rapid change in flooding behaviour following the natural east-west flow of the land.

Topography

The catchment contributing to flows in East of Kwinana contains hilly regions in the east with well-defined channels that drain onto the Swan Coastal Plain. The channels of the coastal plain have flatter slopes and a lower carrying capacity resulting in overland flows between channels and into cascading storages. East of Kwinana locates in the middle of these cascading storages. Flooding in this area tends to have a lower velocity and is related to both the peak flow generated in the upper areas and the volume of runoff generated both within the hills and on the coastal plain. and represents the difference between faster upstream flood water arriving in this area versus the ability of the downstream channels to convey this water further downstream.

Catchment conditions

East of Kwinana sits within the Birrega, Oaklands and Peel floodplains on the Swan Coastal Plain. There is a mix of sands and clays, and depth to groundwater is shallow across the majority of the area. Much of the lower catchment to the west, the natural landscape can be characterised by flat terrain with large natural surface depressions that pond water seasonally. These areas are also referred to as palusplain.

Wet catchment conditions may temporarily (soon after rainfall) or permanently (seasonal inundation) increase groundwater levels and reduce the capacity of the rainfall to infiltrate the soils of the coastal plain. Wetter conditions promote greater potential runoff in subsequent rainfall events.

Ground cover

In general, vegetation and plants can help to intercept and absorb water and reduce runoff, while urban areas (roofs and roads) tend to rapidly transport rainfall to groundwater (i.e. soakwells) or surface water systems and may reduce evapotranspiration leading to increases in groundwater levels.

Model development

The RORB runoff and streamflow routing program is applied to generate channel flows from the hilly regions in the east, where the flow channels are well defined, but quality Light Detection and Ranging (LiDAR) data is not available. The MIKE FLOOD model is employed to model on land and channel flows to the west of the catchment. This model can vary loss rates depending on soil type and areas of ponding, which is considered suitable for modelling the complex overland flow behaviour on the flat slopes of the Swan coastal plain. The model also allows for the channel and overland flows to be linked but modelled separately. This is particularly useful given the importance of the conveyance in the relatively small drainage channels compared to the overland flood flows occurring over a broader coastal plain.

3 Data collation

Available data related to the hydraulic models is presented in this section, including rainfall, streamflow, terrain, land-use, groundwater and structure.

3.1 Rainfall

Several rainfall gauges have recorded sub-daily rainfall within the study area, but the length of the record varies. The Dog Hill rainfall gauge (509295, department-operated) is the only site situated within the hydraulic model domain that provides rainfall for events in 2017 and 1987 that have been used for calibration/validation of the hydraulic model. It recorded 72 mm and 86 mm rainfall in the 2017 and 1987 events, respectively. The data from this site was used within the hydraulic model domain.

The location of all rainfall gauges within the model domain and surrounding area and associated recorded rainfall totals are displayed in Figure 3-1 and Figure 3-2 and listed in Table 3-1. The observed rainfall totals for the 2017 event are relatively consistent across the model domain and the contributing catchments to the north and east. However, the observations illustrate that the rainfall in the foothills to the east of the model domain recorded significantly more rainfall in the 1987 event.

Table 3-1 Rainfall stations and their records in the 2017 and 1987 events

Event	Site	Total rainfall (mm)
2017	Dog Hill (509295)	72
2017	Mt Curtis (509271)	60
2017	Anketell (009258)	76
1987	Dog Hill (509295)	86
1987	Mt Curtis (509271)	134
1987	Mire Seldom Seen Creek (509270)	137
1987	Seldom Seen Creek (509269)	124
1987	Hopelands Road (509387)	70
1987	Rockingham Post Office (009036)	71

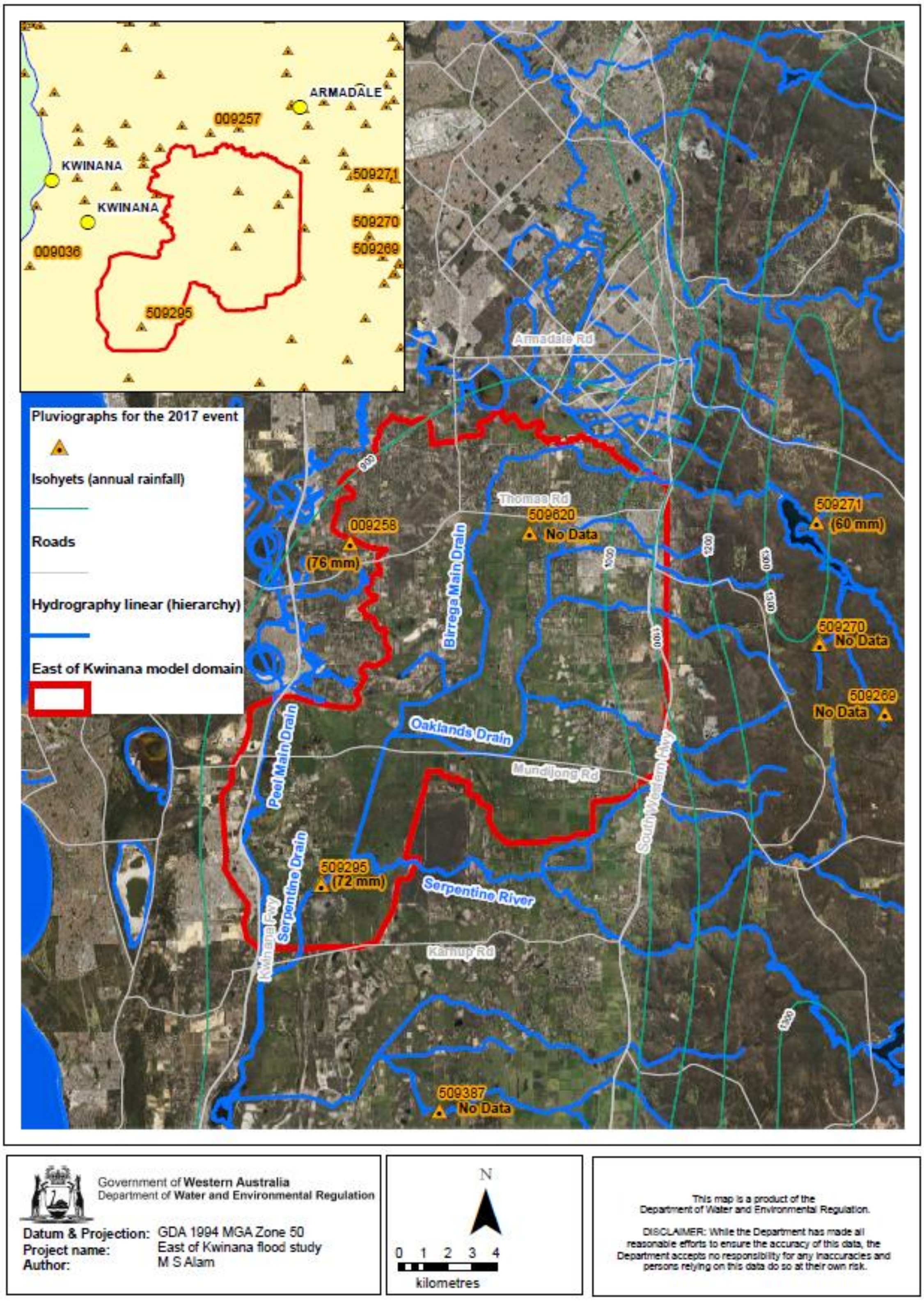


Figure 3-1 Pluviographs used in the hydrology and hydraulic models for the 2017 calibration event

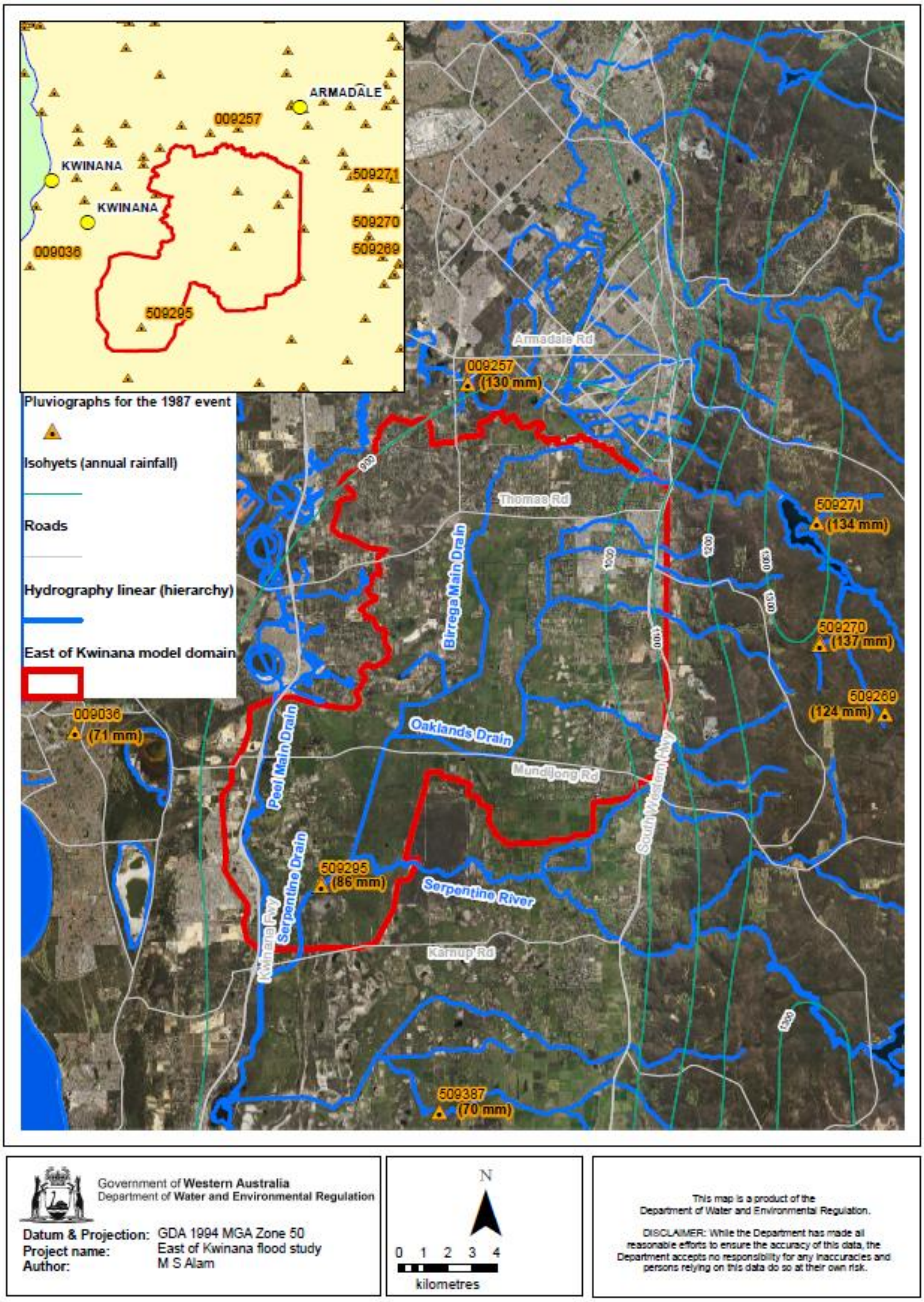


Figure 3-2 Pluviographs used in the hydrology and hydraulic models for the 1987 validation event

3.2 Streamflow data

The department operates four streamflow gauging stations within the model domain and a fifth near the south-west outflow location (Figure 3-3). The data from the Mundijong Road station (614130) on the Birrega Main Drain and the Dog Hill station (614030) gauging station on the Serpentine River could be used in comparing modelled hydrographs (both stage and discharge) for the 2017 event. However, previous studies questioned the accuracy of discharge values at some sites, including Dog Hill (SKM, 2010b; Hall, 2015a). There is no record available at the Lightbody Road (614129) gauge on the Oaklands Main Drain for the 2017 or 1987 events.

3.3 Terrain data

LiDAR data are available for the proportion of the catchment located on the Swan Coastal Plain. A representation of the extent of the LiDAR coverage is shown in Figure 3-4.

These data were captured on 25 February 2008 by Fugro Spatial Solutions Pty Ltd for the Department of Water and have a point density of 1 point per square metre and an accuracy of 0.15 m at 67% confidence. We used LiDAR to develop the bathymetric layer for the 2-D overland flow hydraulic model, develop the cross-sections of waterways used in the 1-D channel flow hydraulic model, and develop internal sub-catchments for the flood study area.

The use of LiDAR is considered good industry practice and is fit for regional flood modelling over such large areas. The stated accuracy of the LiDAR is a minimum standard and does not reflect the actual accuracy. The site is considerably cleared of vegetation, and verification of the LiDAR using ground survey has been undertaken. Critical features like drainage channels/spoil banks have been adjusted based on additional surveys and site inspections.

3.4 Land-use data

The Department of Water developed land-use data was for the region based on Landgate cadastre (2008) with DLI aerial imagery (2008) and LiDAR non-ground returns to determine vegetation extent. Land-use data was necessary for resistance categories in hydraulic modelling and regional parameterisation of hydrologic models.

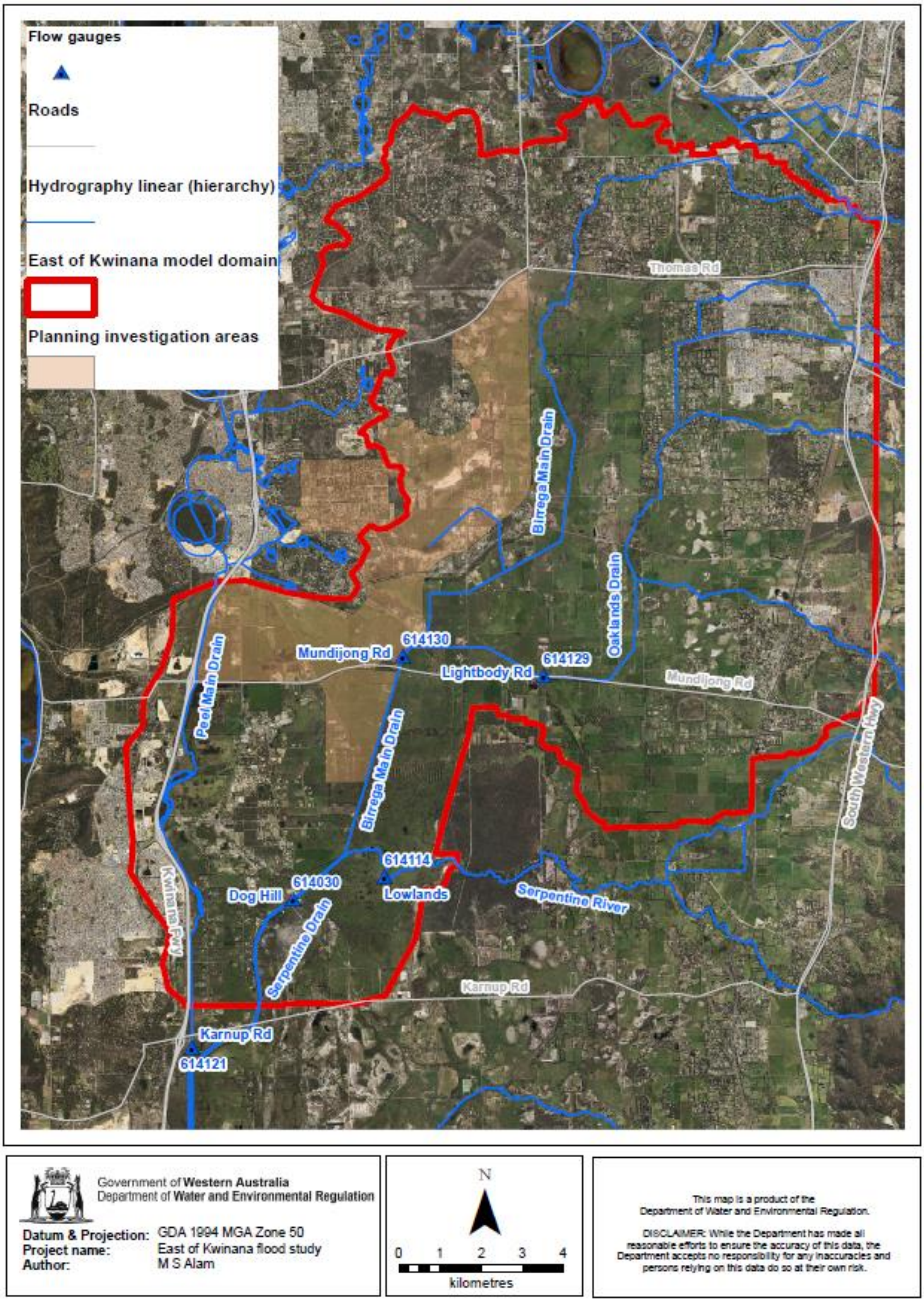


Figure 3-3 Streamflow gauges related to the hydraulic model domain

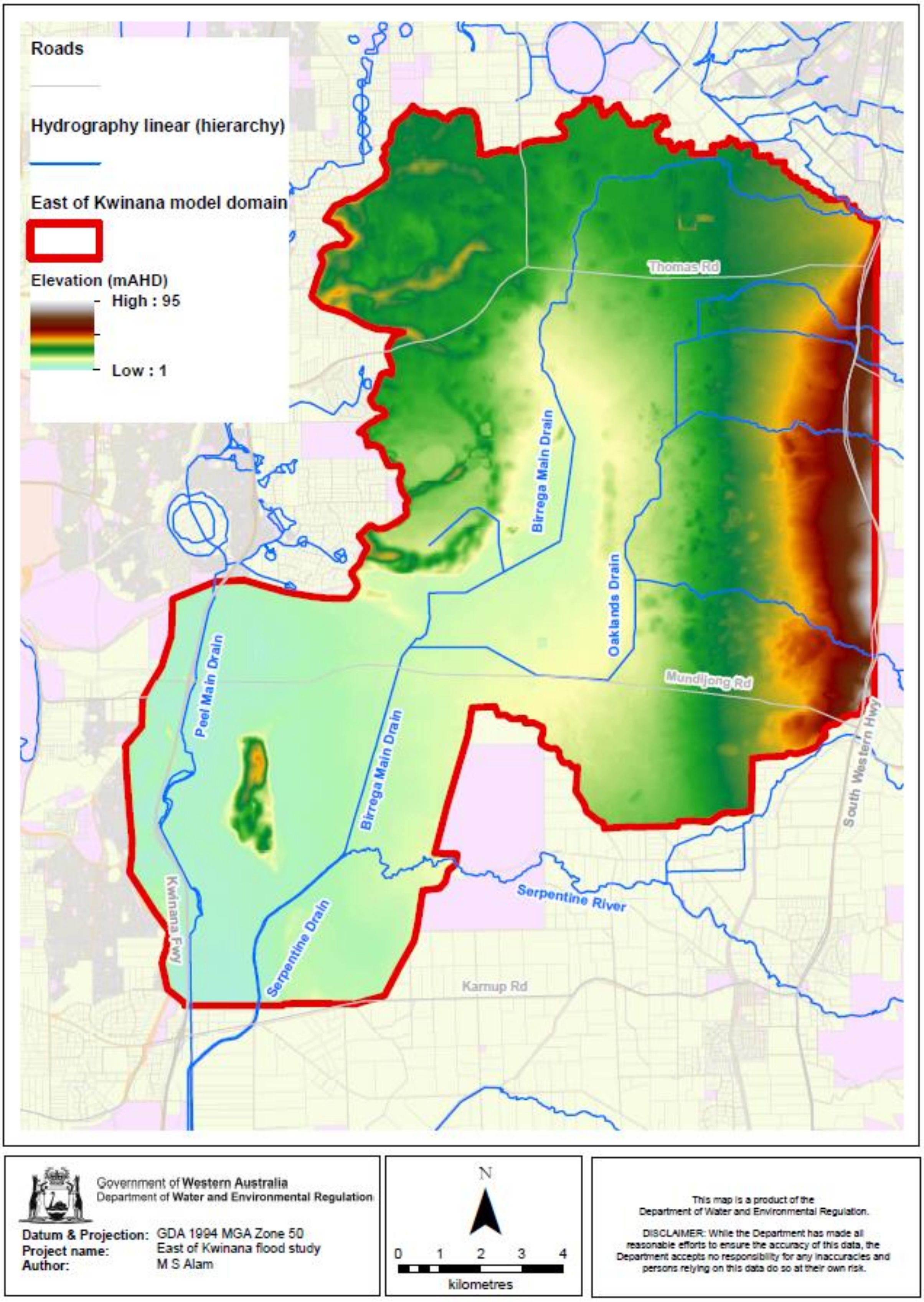


Figure 3-4 Model topography map developed from the department's 1 m LiDAR dataset

3.5 Groundwater inundation data

The department has developed the Lower Serpentine MIKE SHE model (Marillier, Hall and Kretschmer, 2012) to identify groundwater inundation areas, providing reliable groundwater level information and estimate drainage within the study area. Many parts of the study area are prone to shallow groundwater, resulting in regular winter inundation, even in dry years. Therefore, this flood study considers the groundwater component by identifying areas that should be regarded as impervious because of inundation. Section 5.2.4 will further discuss inundation data as part of the infiltration setting, and Figure 5-7 shows the groundwater inundation surfaces used in the flood modelling.

3.6 Structures data

In the hydraulic model construction, the culverts, bridges, and road elevations were considered from two previous models (Hall, 2015a; Marillier, 2015), as described in Section §1.4. No new surveys were carried out on structures compared to the above two studies, except a brief field trip to confirm certain culverts' configurations. In total, the hydraulic model considers more than 30 structures, including many large culverts and bridges along the Birrega, Oaklands and Peel main drains.

4 Hydrology studies

This section outlines the time-dependent inputs for the hydraulic models, including:

- the design rain that falls uniformly on model grids, and,
- the hydrograph that flows into the domain from boundaries.

Both rainfalls and inflows are critical inputs for the East of Kwinana flood models. The volume of water from precipitation typically makes up 60% of the total volume balance of water in the regional flood model, and the rest is from the inflows. However, this estimation depends on specific cases in the East of Kwinana modelling.

The present study's design rainfall was generated by the guidelines set in the newest ARR (2019) (Ball J et al., 2019).

For the regional hydraulic model, it requires inflow hydrographs at 17 locations, including

- MIKE11 inflows at two locations
- MIKE21 'source points' at 15 locations.

As the local hydraulic model requires outflow from the regional model, it also indirectly requires all these inflow hydrographs.

4.1 Design rainfall for regional and local models

4.1.1 Catchment for rainfall generation

We generated the design rainfall following the guidelines set in the updated ARR (2019) (Ball J et al., 2019) through the ARR data hub (2020).

We generated rainfalls from a catchment around the east of the Kwinana modelling area for the regional and local models. Figure 4-1 shows the extent of the catchment.

The selected catchment was deemed appropriate for estimating design flows within the PIAs (North East Baldivis north of Mundijong Road), including contributing catchments of the Peel Main Drain, the Birrega Main Drain and the Oaklands Main Drain. The size of the catchment is about 305 km^2.

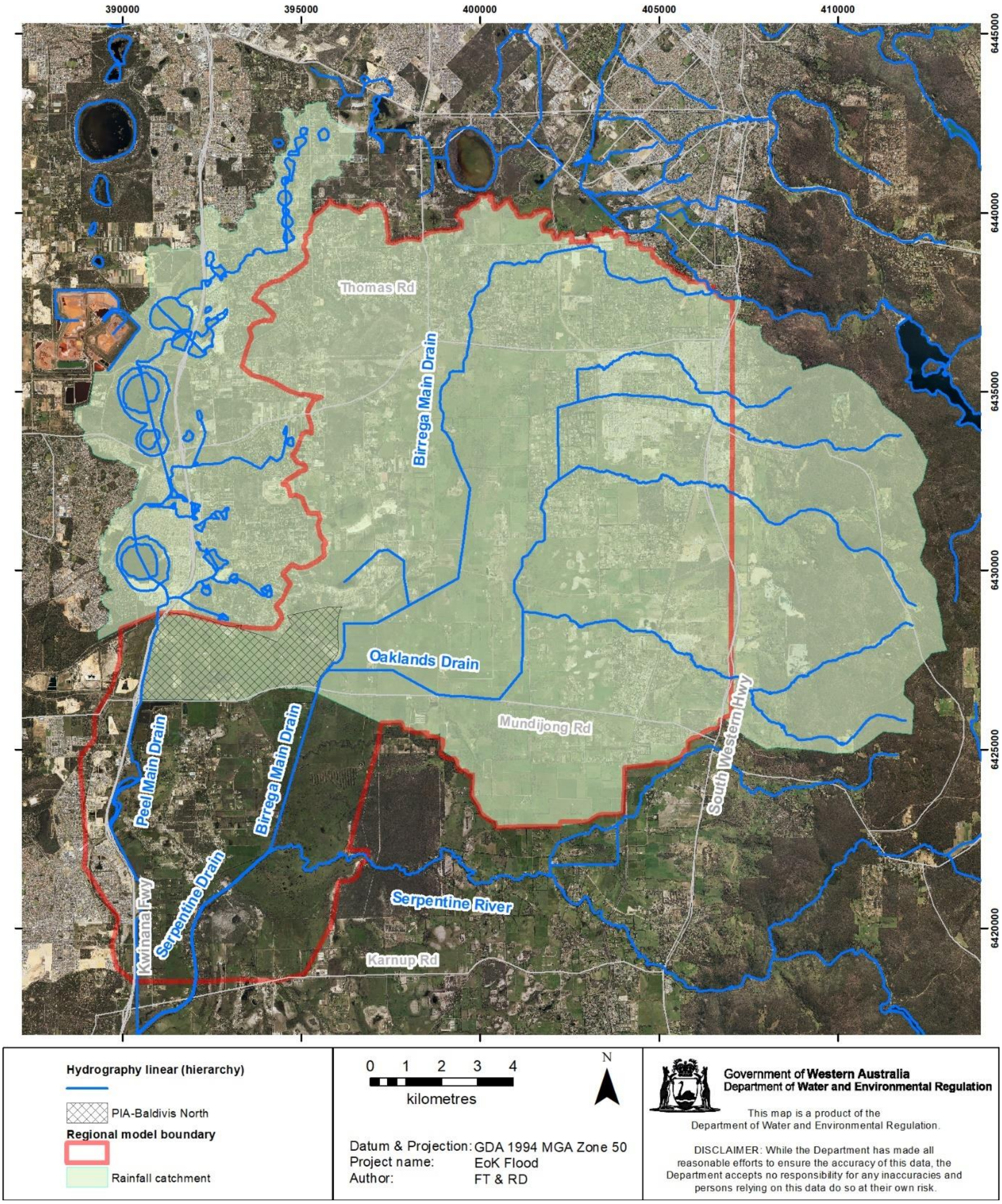


Figure 4-1 The catchment used to generate the design rainfalls North East Baldivis north of Mundijong Road (grid), the hydraulic model and the key linear hydrography linear

4.1.2 Design rainfalls

The design rainfalls used in the present hydraulic model are calculated from:

Design rainfall = Areal reduction factor × Intensity frequency duration.

These terms are explained in the following sections.

Areal reduction factor

Design rainfall depths generated from ARR2019 are based on rainfall Intensity Frequency–Duration (IFD) data related to specific-point data in a catchment rather than the whole catchment area (Ball J et al., 2019). However, the application of these point-based rainfalls would overestimate the total volume of rain if applied uniformly over the catchment since the design rainfall intensities at a point are not representative of the average rainfall intensity across the catchment (Hall, 2015a; Ball J et al., 2019).

An areal reduction factor (ARF) is derived and applied to rainfall depths used for design flood estimates (Hall, 2015a) to account for rainfall variation across a catchment. ARF is the ratio between the design values of average areal rainfall and point rainfall (Ball J et al., 2019), computed for the same duration and AEP.

Table 4-1 shows the ARF for the regional model based on the catchment, as shown in Figure 4-1. For completeness, the ARFs are given for seven AEPs and nine durations, calculated based on ARR 2019 (Ball J et al., 2019, pp. 62–64). It is seen the ARF effect is most pronounced for smaller AEP and shorter duration events (top-right with bright red in Table 4-1, and less prominent for larger AEP and more extended duration rainfall events (bottom-left with bright green in Table 4-1). This observation is in agreement with discussions in Hall (2015a).

Table 4-1 ARF for the regional model

AEP / duration	20%	10%	5%	2%	1%	0.50%	0.20%
3 hour	0.817	0.798	0.779	0.754	0.734	0.715	0.690
6 hour	0.868	0.858	0.848	0.835	0.825	0.815	0.802
9 hour	0.893	0.887	0.881	0.874	0.868	0.863	0.855
12 hour	0.904	0.898	0.893	0.885	0.879	0.874	0.866
18 hour	0.921	0.917	0.913	0.907	0.903	0.899	0.894
24 hour	0.938	0.935	0.933	0.930	0.927	0.925	0.922
36 hour	0.946	0.944	0.941	0.938	0.936	0.934	0.930
48 hour	0.951	0.949	0.947	0.944	0.941	0.939	0.936
72 hour	0.958	0.956	0.953	0.951	0.948	0.946	0.943

Design rainfall depth

Rainfall Intensity–Frequency–Duration (IFD) date was based on ARR 2019 (Ball J et al., 2019) and generated from ARR data hub (2020) by the shapefile of the area as given in Figure 4-1. Table 4-2 presents the rainfall depth for several AEPs and durations. These data were obtained by multiplying the IFD design rainfall depths with the corresponding ARF. The same information in Table 4-2 is illustrated in Figure 4-2 to visually compare the size of rainfall depth as a function of rainfall durations.

Table 4-2 Design rainfall depths (mm) for the regional model (and the local model without the engineered levee)

AEP duration	20%	10%	5%	2%	1%	0.50%	0.20%
3 hour	30.7	35.0	39.3	45.4	50.2	56.5	64.7
6 hour	41.9	48.4	55.5	65.8	74.6	85.6	101.0
9 hour	49.5	57.8	66.7	80.1	91.2	106.1	126.6
12 hour	55.2	64.6	74.6	89.4	102.0	118.8	141.2
18 hour	64.0	75.0	86.6	104.4	119.2	137.6	163.6
24 hour	71.1	83.2	96.1	115.3	130.8	150.8	178.8
36 hour	80.3	93.4	107.3	126.7	143.2	160.6	186.1
48 hour	87.2	100.6	114.6	134.9	150.6	166.2	189.1
72 hour	97.7	112.8	126.8	146.4	161.2	176.0	197.2

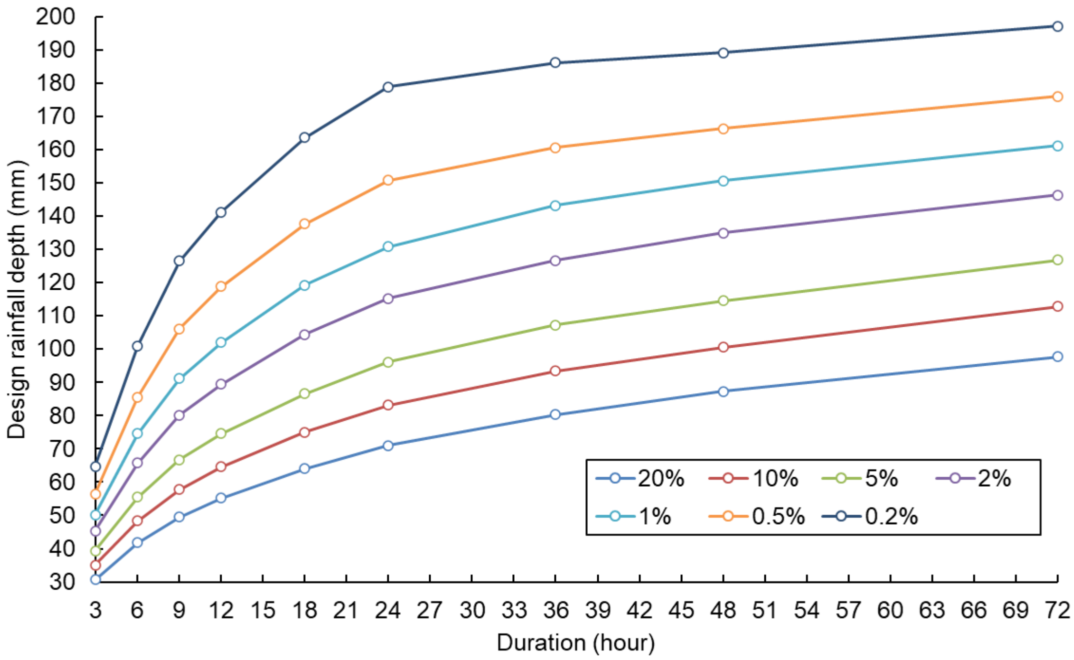


Figure 4-2 Design rainfall depths (mm) for the regional model at different AEP events.

Temporal patterns

In ARR 2019, there are different sets of areal temporal patterns. For relatively large catchment areas, ranging from 100 km^2 to 40,000 km^2, ARR 2019 recommends areal based temporal patterns. ARR 2019 also recommends point patterns for modelling smaller catchments, including catchment areas less than 75 km^2.

Areal temporal patterns from S-SW Flatlands (West) was adopted considering the catchment area of about 305 km^2 (Figure 4-1). The catchment area lies between the 200 km^2 and 500 km^2 areal based temporal patterns and the temporal patterns at 200 km^2 were used in flood modelling inputs.

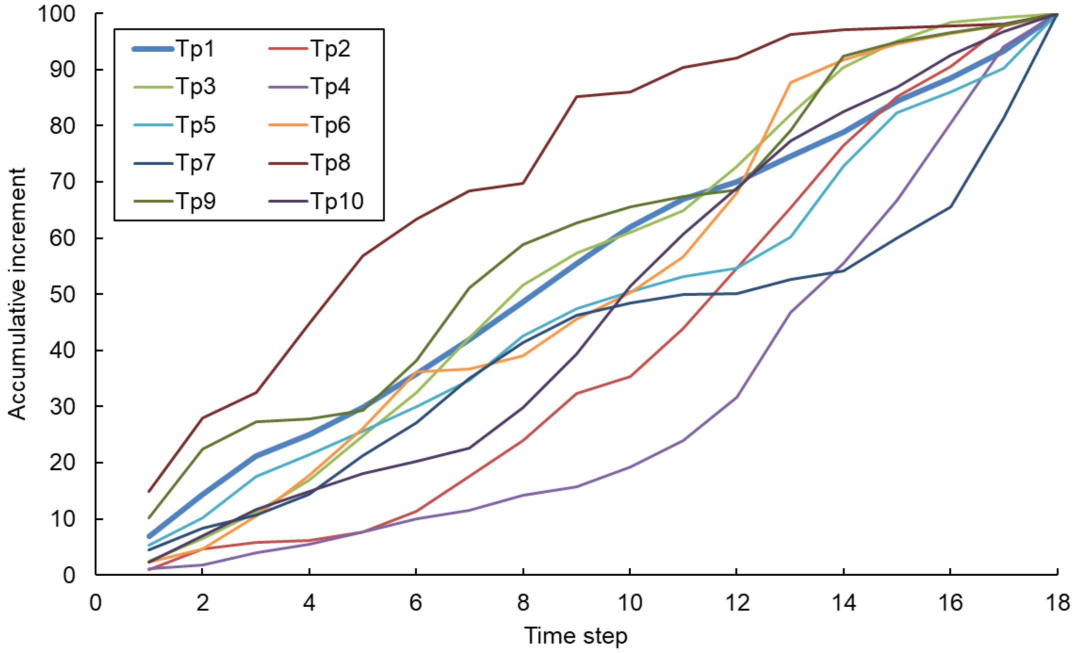


Figure 4-3 Temporal patterns of rainfall for the regional model at 18-hour design events (based on ARR2019, Region: S-SW Flatlands (West) and Area: 200 km^2)

Areal temporal patterns are consistent across all event likelihoods of the same duration. That is, the same duration design rainfall event at any AEP shares the same temporal pattern. Figure 4-3 shows the temporal patterns for 18-hour rainfall events. As discussed later, temporal patterns of precipitation can significantly affect the peak discharge of computed design floods. It may also affect the critical duration because the way rain falls on a catchment affects how floodwater accumulates in the area.

However, different temporal patterns exist for each storm duration. Selected temporal patterns at 12-hour, 24-hour and 36-hour rainfall events are reproduced according to ARR 2019 in Appendix A. For temporal patterns at other durations, please refer to ARR data hub (2020).

4.2 Design hydrology for regional and local models

4.2.1 RORB models

RORB is a general runoff and streamflow routing program used to calculate flood hydrographs from rainfall and other channel inputs (Laurenson, Mein and Nathan, 2010). The RORB model was used to calculate the design hydrographs at 17 locations. The program requires a data file to describe the stream network's particular features being modelled and run interactively (Laurenson, Mein and Nathan, 2010). RORB software version 6.45 was used in the present study.

4.2.2 Design inflows

We generated the inflow hydrographs at 17 locations, as shown in Figure 4-4, including

- MIKE11 inflows at two locations: the Peel Main Drain at the southern edge of Bollard Bulrush Swamp and the Serpentine River along the edge of the Lowlands bushland.
- MIKE21 'source points' at 15 locations: nine along the top-side eastern boundary between Thomas Road and Watkins Road and six on the low-side eastern border south of Mundijong Road.

The inflows were calculated at each rainfall condition; that is, at four AEPs (0.5%, 1%, 5% and 10%) and four durations (12-, 18-, 24 and, 36-hour) using the relevant temporal patterns.

Table 4-3 summarises the peak discharge from RORB at all locations for the 10 temporal patterns relevant to the 1% AEP 18-hour design rainfall event. Examples of the hydrographs for catchment E07 are displayed in Figure 4-5. E07 represents Manjedal Brook's discharge upstream of the South Western Highway bridge (Hall, 2015a).

Despite the local flood model only covering a portion of these areas, it requires outflow from the regional model, which indirectly requires all these inflow hydrographs.

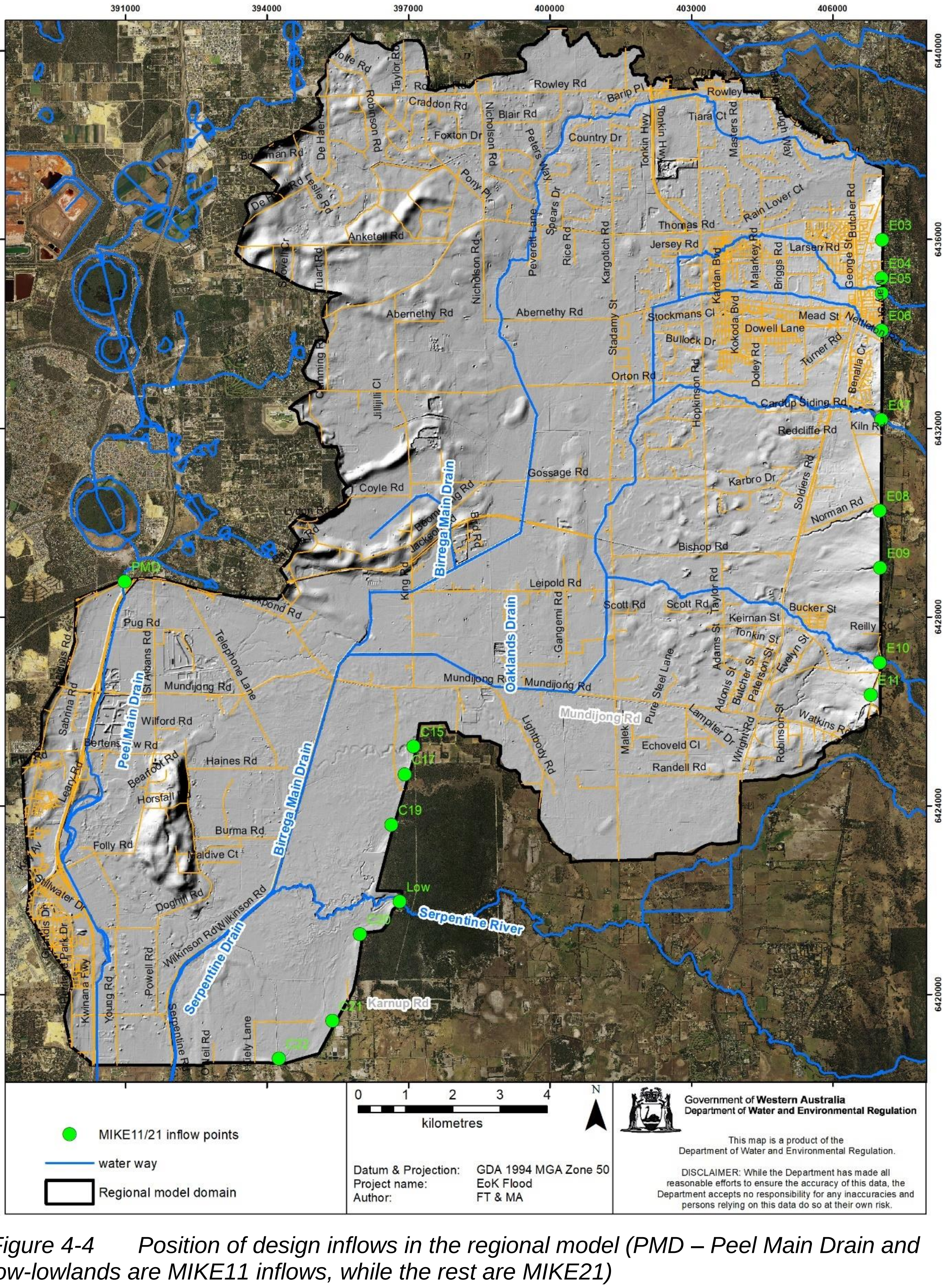


Figure 4-4 Position of design inflows in the regional model (PMD – Peel Main Drain and low-lowlands are MIKE11 inflows, while the rest are MIKE21)

Table 4-3 Peak discharges (m^3/s) for all inlets at 10 temporal patterns (Tp) of the 1% AEP 18-hour duration rainfalls

Tp inlets	**Tp1**	**Tp2**	**Tp3**	**Tp4**	**Tp5**	**Tp6**	**Tp7**	**Tp8**	**Tp9**	**Tp10**
E03	3.18	4.52	3.99	5.70	4.65	6.92	7.63	6.28	5.34	4.65
E04	1.56	2.11	1.86	2.62	2.11	3.38	3.67	2.99	2.61	2.19
E05	2.04	3.08	2.70	3.88	3.14	4.51	5.01	4.08	3.44	3.11
E06	8.58	13.72	11.32	15.99	10.57	14.79	15.32	13.44	11.06	12.97
E07	11.98	18.90	15.87	22.86	14.52	21.25	21.20	19.29	15.26	17.67
E08	3.90	5.85	5.10	7.44	6.06	8.56	9.50	7.85	6.56	5.94
E09	0.96	1.31	1.13	1.63	1.33	2.10	2.27	1.81	1.61	1.34
E10	11.42	17.28	14.04	21.03	12.44	17.58	16.28	17.13	13.07	15.95
E11	1.04	1.42	1.23	1.77	1.43	2.28	2.46	1.98	1.75	1.46
Lowlands	70.66	85.71	79.70	89.46	74.16	81.45	73.43	78.87	73.72	81.18
Cat15	2.11	2.72	2.55	3.22	2.33	3.04	2.92	2.55	2.54	2.42
Cat17	1.10	1.70	1.40	2.10	1.37	1.92	1.93	1.65	1.44	1.55
Cat19	1.97	2.96	2.42	3.52	2.26	3.18	2.97	2.69	2.32	2.65
Cat20	1.02	1.58	1.29	1.95	1.26	1.78	1.79	1.53	1.33	1.43
Cat21	2.74	3.30	3.12	3.84	2.95	3.54	3.56	3.02	3.10	2.96
Cat27	8.96	10.19	9.72	10.64	9.36	9.76	9.76	9.23	9.24	9.67
PMD	8.51	9.57	9.19	9.77	8.73	9.11	9.28	8.90	8.90	9.27

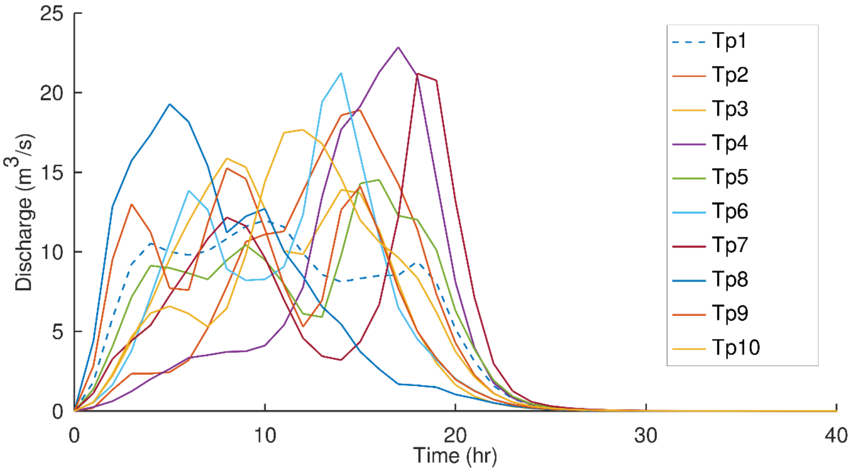


Figure 4-5 RORB Hydrographs at E07 for 1% AEP 18-hour duration rainfall at different temporal patterns

4.3 Design inputs for the local model with a full-height levee option

To prevent discharges to the North East Baldivis north of Mundijong Road precinct, one of the post-development options is to build a levee along the Birrega Main Drain at the Duck Pond location (Option A in Section 7). Under this condition, the engineered levee effectively blocks all overflow from the Birrega Main Drain at 1% AEP at that location to the North East Baldivis north of Mundijong Road.

From a surface water point of view, the precipitation collected by the Birrega and Oaklands main drains will not affect the North East Baldivis north of Mundijong Road precinct. Therefore, building a full-height levee alters the catchment characteristics by cutting the North East Baldivis north of Mundijong Road precinct from its natural drainage basin.

Following the guidelines set in the newest ARR guidelines – ARR (2019) (Ball J et al., 2019) through the ARR data hub (2020), the design rainfalls and their temporal patterns are dependent on the location and area of the catchment of interest. The design inputs with the full-height engineered-levee option are different from those discussed in sections 4.1 and 4.2.

This study tested both design inputs based on different catchment areas for the Option A local model. No significant difference was found in the flooding characteristics from these design inputs.

5 Hydraulic modelling

The hydraulic models were built using DHI's MIKE Flood 2019 package, following two previous studies (Hall, 2015a; Marillier, 2015). The MIKE Flood dynamically couples a two-dimensional (2D) hydraulic model with a one-dimensional (1D) river model into one single tool.

- The 2D component (MIKE 21) was employed to model overland inundation, rainfall-runoff, and infiltration/evaporation, including simulating overland flows, flooding extent across the study area, and determining water levels at various locations.
- The 1D component (MIKE 11) was applied to model the drains, culverts, and bridges within the study area, including water level and discharge inside river channels, allowing water interchanges to and from the overland flows.

This chapter discusses:

- the overview of the regional and local models
- the construction of the MIKE21 model
- the construction of the MIKE11 model
- the coupling of the MIKE21 and MIKE11 models
- model calibration.

5.1 Regional and local flood models

Two flood models were built for the study – a regional and a local. The regional model refers to the study area covered by red boundaries in Figure 5-1. This model covers most of the East of Kwinana sector.

The local model refers to the domain covered by the green boundaries, focusing on the North East Baldivis north of Mundijong Road precinct. We studied several post-development options in Chapter 7 with the local flood model.

Table 5-1 Overview of the regional and local models

Model	Area (km^2)	Element no.	Avg. cell	Inputs
Regional	232.7	972,946	239.2 m2	ARR2019, RORB
Local	24.3	715,865	33.9 m2	ARR2019, RORB, Regional model

The local model focuses on flooding for about 10% of the regional model; therefore, the local model employs relatively fine grids to obtain detailed information for the North East Baldivis north of Mundijong Road precinct (Figure 5-1).

Table 5-1 compares some generic information from the regional and local models. Despite the significant difference in the modelling areas, the two models' element numbers are close, leading to a significantly smaller average cell area in the local model than that in the regional model. As will be outlined later, the local model also requires inflows from the regional model at the confluence of the Birrega Main Drain and the Oaklands Main Drain.

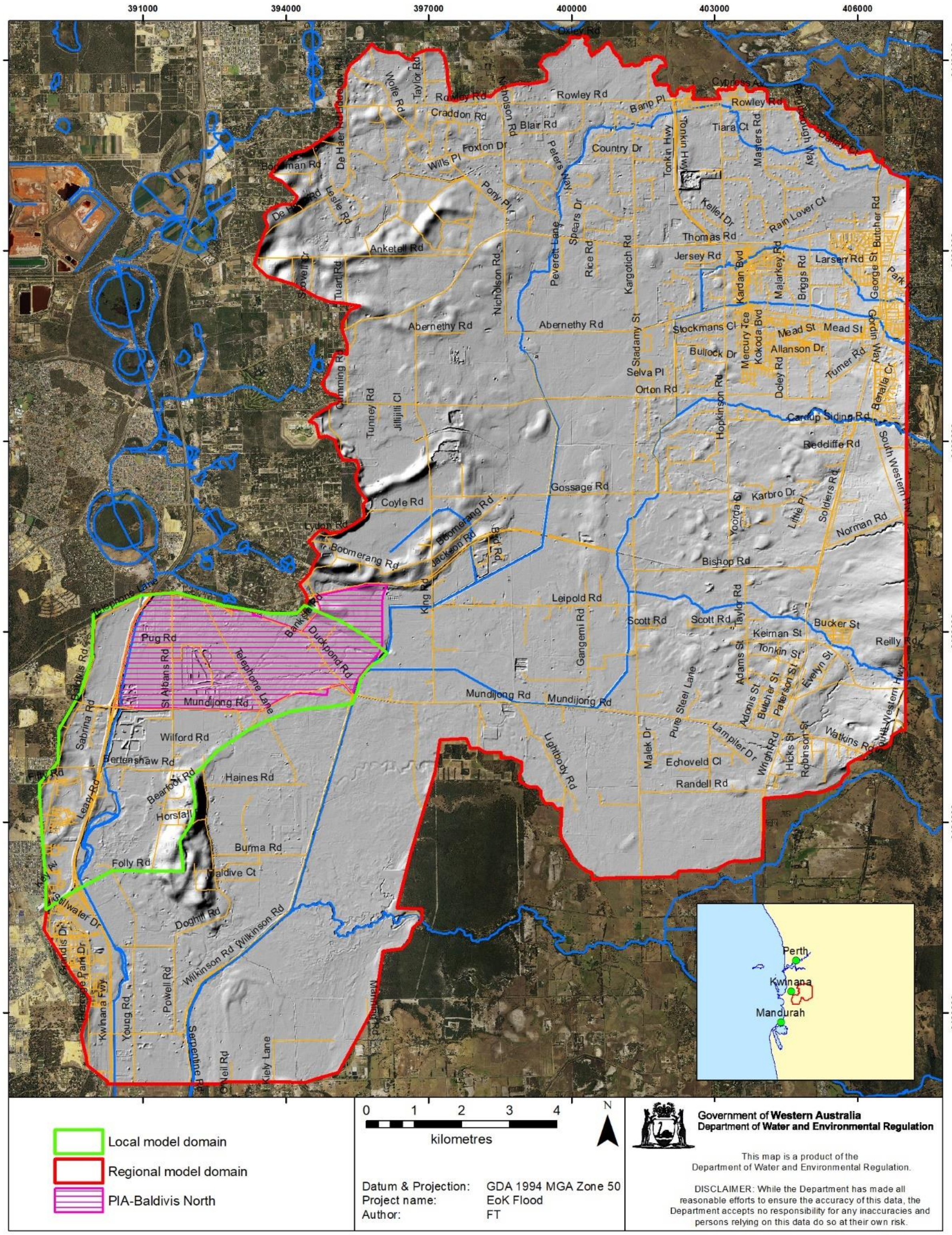


Figure 5-1 Area covered by the regional (red boundary) and local (green boundary) models. The insert on the bottom corner indicates the location of the regional model relative to Perth and Kwinana.

5.2 Regional MIKE 21 FM model construction

MIKE 21 Flow Model FM solves water level and flows in a two-dimensional (2D) environment. The 2-D model domain includes the floodplain and some small channels.

MIKE 21 FM provides a numerical solution for the 2-D incompressible Reynolds averaged Navier-Stokes equations (DHI, 2019b). The modelling code implemented the finite volume based flexible mesh scheme. The calculation involves employing the model with pre-defined inputs and boundary conditions, including the rainfall on grids, spatially distributed hydraulic roughness, infiltration, and inflows from the domain boundary.

5.2.1 Flexible mesh

The built-in mesh generator tool (DHI, 2019c) available in MIKE 21 FM was used to generate a mesh file. Based on the resolution requirements, multiple polygons were adopted to develop mesh with different densities. Polygons are available to define the mesh's local properties, such as maximum element size and the shape of elements (triangular or quadrangular). Polygons were also employed to exclude significant rivers and drains, which were considered in MIKE 11.

The flexible mesh solution approach involves discretising the model domain with a variable resolution. A triangular mesh was considered for the minor channels as quadrangular elements could not represent them because of unclear channel definition. This approach allows flexibility to accommodate higher density mesh with a more significant computational effort in the area of interest, especially for the small low-flow channels that require a fine mesh scale. Figure 5-2 shows an example of mesh resolution and geometry considered for this study. The maximum element sizes are 750 m^2 and 180 m^2 for the upper and lower half of the model domain. The minor channels originating from the scarp within the upper half of the region have a maximum element size of 120 m^2.

The mesh file also considers implementing the topographic data and the boundary locations within the model domain. The model topographic data was from the department’s LiDAR dataset (Figure 3-4). The mesh generator tool calculates an interpolated topographic surface based on the LiDAR dataset (DEM). Figure 5-2 also shows the interpolated topographic layer for a part of the model domain. The white portion of the layer represents watercourses excluded from the 2-D calculation (Oaklands Main Drain). They were simulated in MIKE 11 using the hydrodynamic module.

The built-in mesh tool generated 972,946 triangular elements with 491,147 nodes for simulating overland flows based on the above criteria. The overall model topography map was in the MIKE 21 FM simulation is displayed in Figure 5-3.

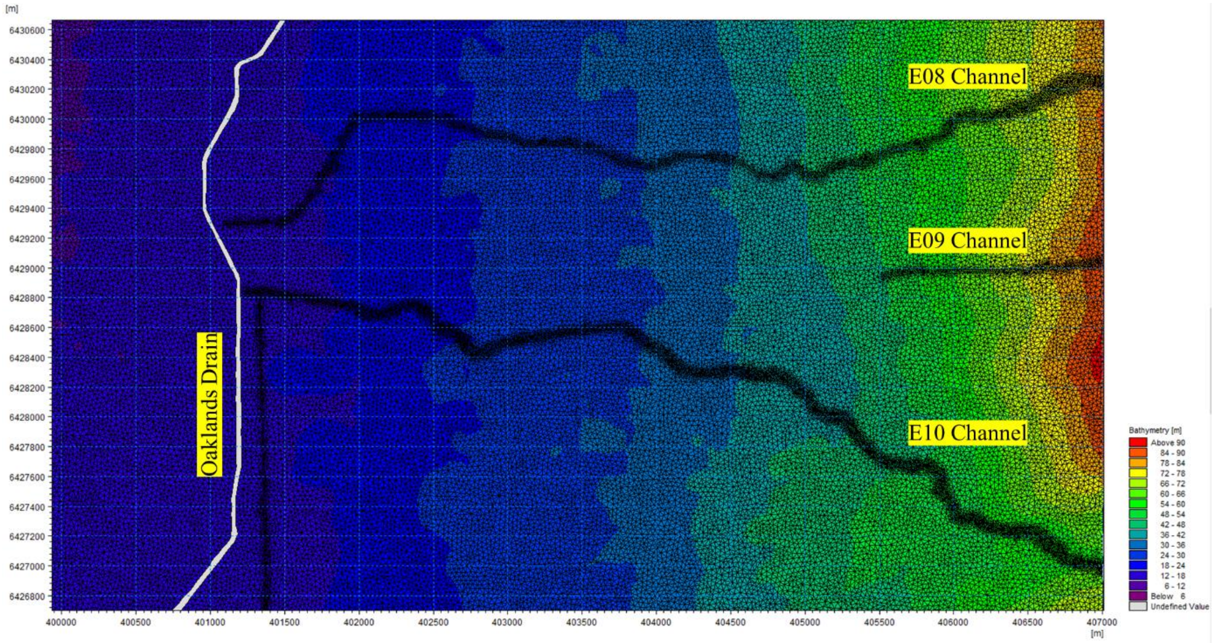


Figure 5-2 Example of mesh showing different mesh resolution/geometry. Here, the pre-development mesh is overlaid on pre-development DEM levels.

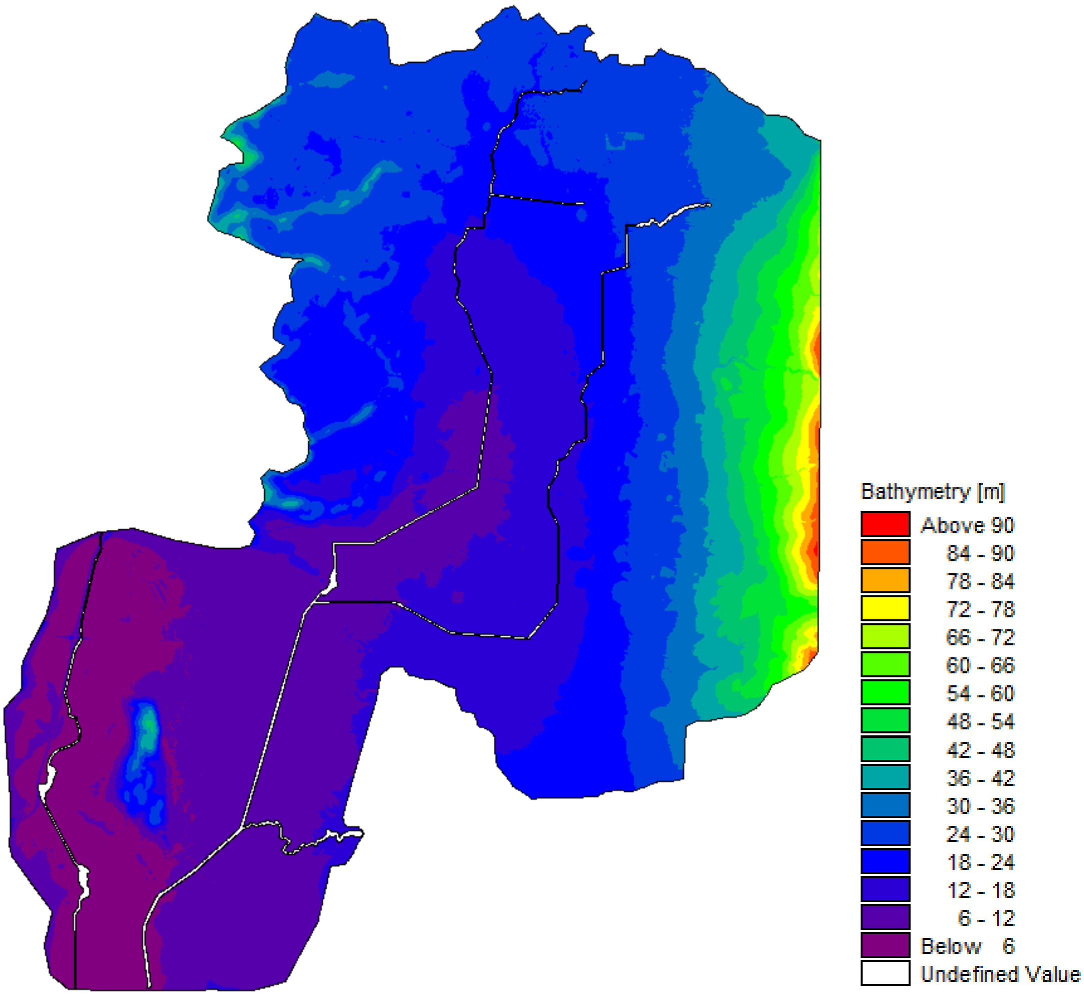


Figure 5-3 MIKE 21 FM model topography

5.2.2 Rainfall

Rainfall was assumed to be spatially homogenous within the domain and was applied as gross precipitation. A time series of rainfall files were assigned in the model set up for a flood event, and it was applied to the entire 2-D model domain. Given the very flat topography, this method is appropriate for the study area (Marillier, 2015).

5.2.3 Roughness

A spatially distributed hydraulic roughness (Manning's number) map was developed based on three broard land-use classifications:

- urban residential and roads/open (40)
- cleared pasture (20)
- vegetated (12.5).

Figure 5-4 presents the colour-coded roughness values. These were reported as Manning's M, which is the inverse of Manning's n.

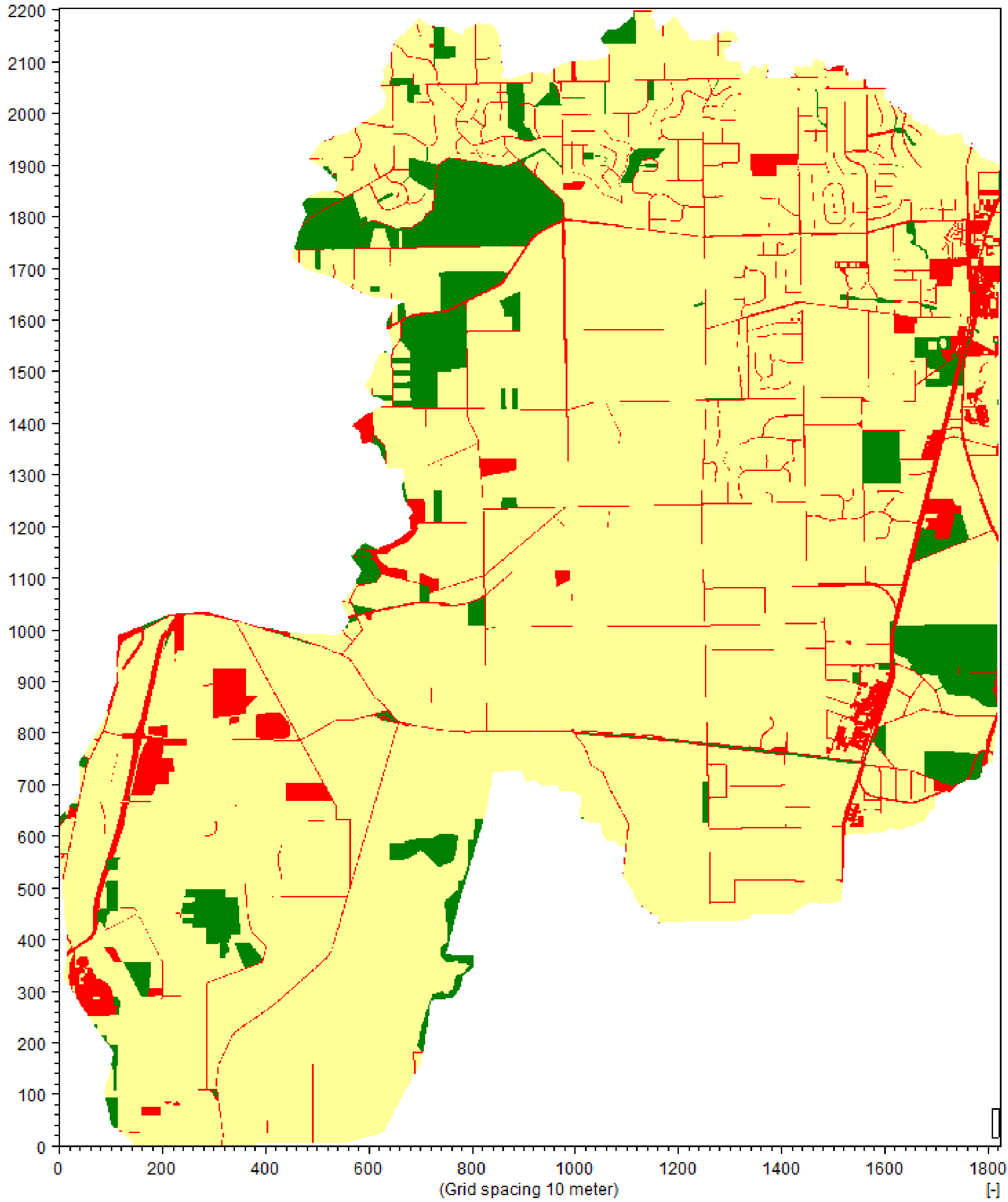


Figure 5-4 Colour coded resistance map-Manning's number 40 (red), 20 (yellow) and 12.5 (green) were used roads/open, pasture and vegetation areas

5.2.4 Infiltration

Infiltration was modelled within the hydraulic model using distributed infiltration rates. The study area was divided into three categories, each with a different infiltration rate (mm/day), which are:

- sand (70 mm/day)
- clay (5 mm/day)

- inundated (0 mm/day).

The soils were broadly classified into two categories (Sand and Clay) based on the Department of Primary Industries and Regional Development soil map unit database. The sandy phases of the Pinjarra, Spearwood and Bassendean soil groups were amalgamated into a single category termed 'sand', allowing some losses through infiltration from the direct rainfall.

The second category included the poorly drained phases of the Pinjarra group. It is termed 'clay' which are characterised by higher clay and organic content and is generally located in depressions.

Figure 5-7 shows the infiltration map that was considered for this flood study. An infiltration rate of 70 mm/day and 5 mm/day was considered for sand and clay soil types, respectively. The rates are considered physically plausible and are consistent with previous flood modelling undertaken on Swan Coastal Plan catchments (Hall, 2015a, p. 54; Marillier, 2015, p. 57). The adopted values did not require further calibration as a satisfactory calibration was achieved for the recent 2017 and July 1987 events. The July 1987 event is equivalent to a 2% AEP event.

Figure 5-7 also shows the inundation areas (dark blue), which were derived from the average groundwater levels (1981–2010) from the Lower Serpentine regional groundwater model (Marillier, 2015). Infiltration for the inundated area was 0.

5.2.5 Inflows from the RORB model

Discharge from the RORB model was introduced within MIKE 21 using 15 source points, including:

- nine locations along the top-side eastern boundary of the model between Thomas Road and Watkins Road, and
- six places on the low-side eastern border south of Mundijong Road.

The locations of the source points are shown in Figure 4-4.

5.2.6 Antecedent condition

An initialisation simulation was carried out to develop initial conditions for the MIKE21 model. A rainfall depth of 20 mm was applied over 12 hours with a further 24 hours simulated after the rainfall event, as shown in Figure 5-5. The initialisation of all design simulations in MIKE21 used overland water depth from the final timestep of this simulation.

This storm was generated using the rainfall pattern and magnitude from a recorded event at Dog Hill.

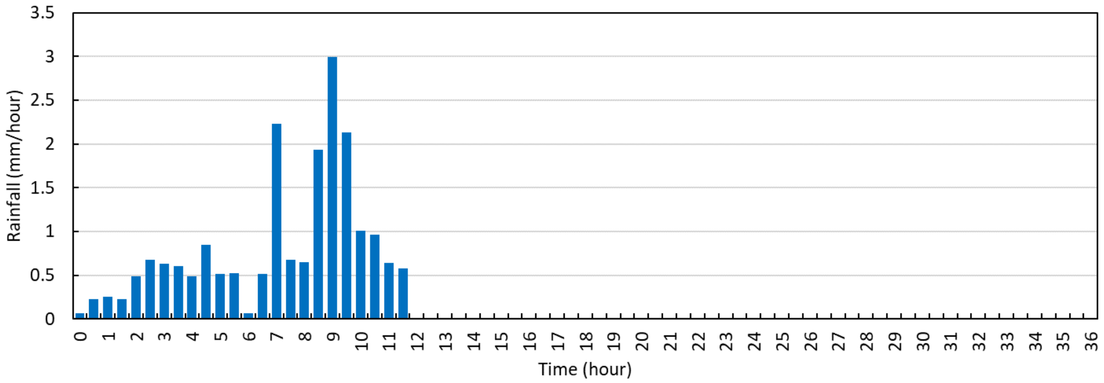


Figure 5-5 Rainfall for initial condition runs

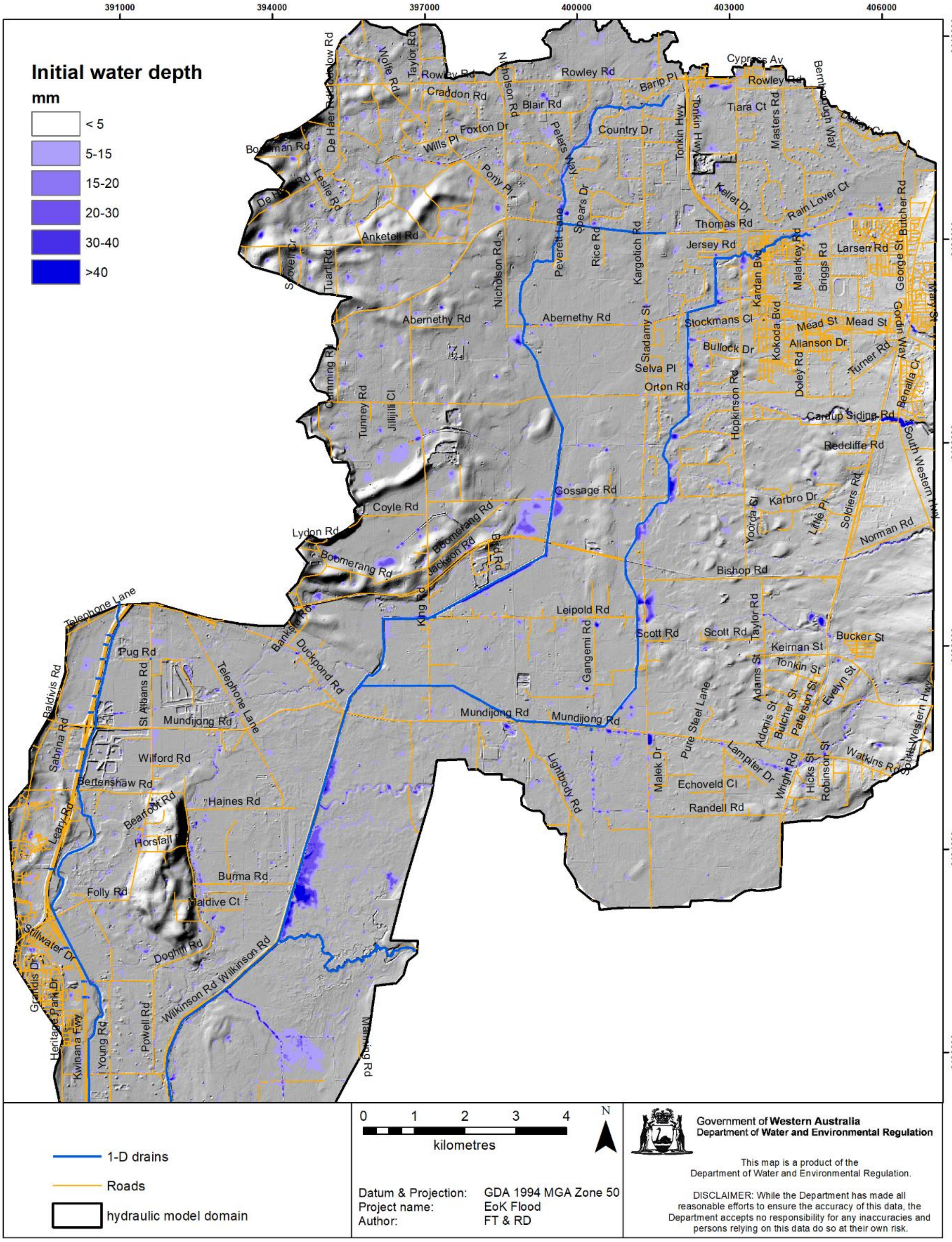


Figure 5-6 Overland water depth for all design and validation runs

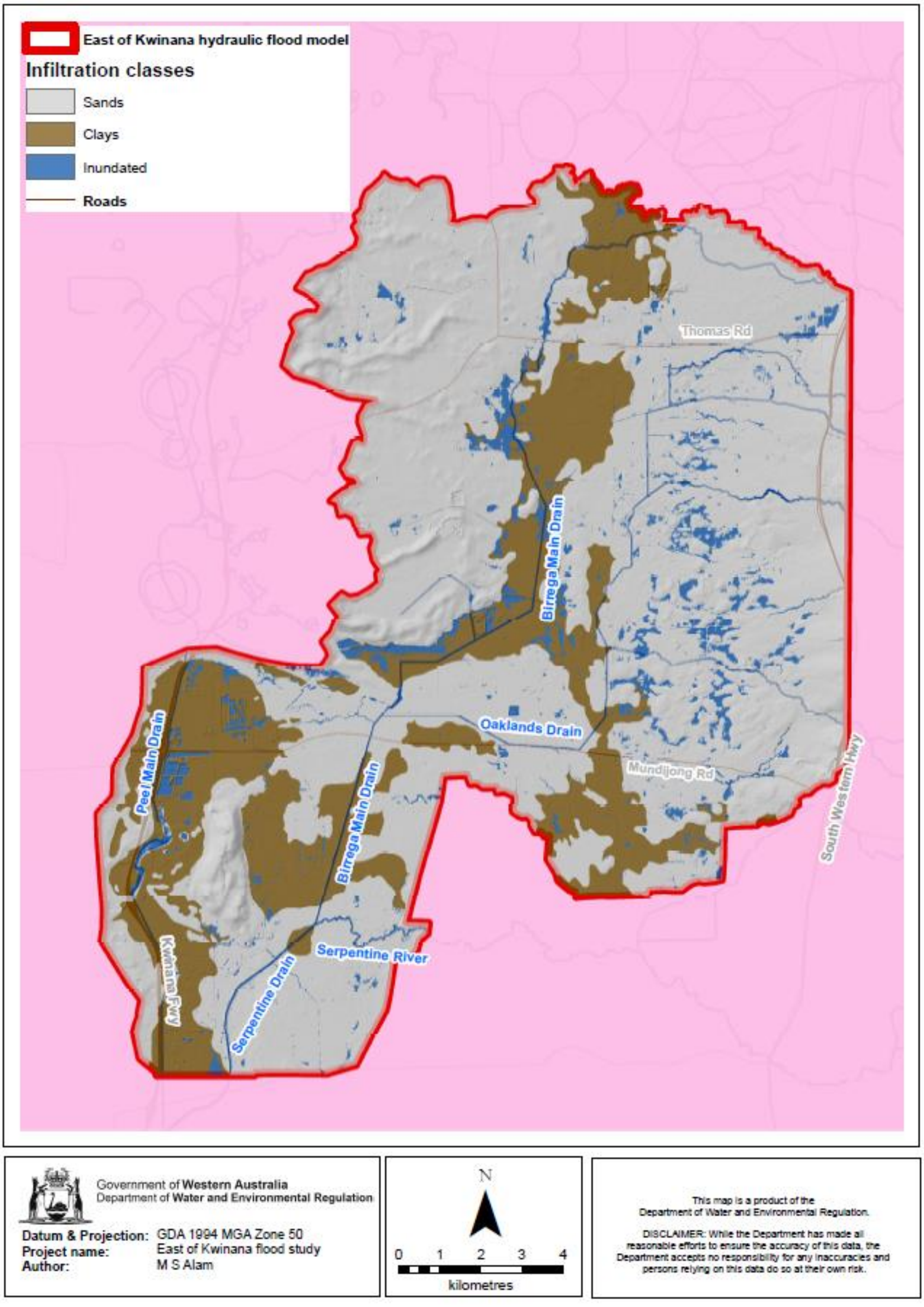


Figure 5-7 Infiltration map considered for the calibration and design runs; an infiltration rate (mm/day) of 70 (grey), 5 (umber) and 0 (blue) was considered for sand, clay and groundwater inundation areas

5.3 Regional MIKE 11 model construction

MIKE 11 was used to represent a one-dimensional (1D) model domain where major rivers and drains were modelled. The hydrodynamic module uses the vertically integrated Saint-Venant equations to solve flows and water levels (DHI, 2019a).

Within the study area, the Birrega, Oaklands, and Peel main drains are typically between 10 and 30 m wide, generally smaller than the MIKE 21 mesh resolution. To accurately capture the flow and water level, these channels were simulated using MIKE 11. The MIKE Flood package can couple the MIKE11 and the MIKE 21 models and exchange water between the river/channel and model topography. Hydraulic structures (culverts and bridges) were also included in the 1D model.

5.3.1 Network

The GIS data was used to define the drain centreline for the MIKE11 waterways within the study area.

5.3.2 Cross-section

Similar to previous studies (Hall, 2015a; Marillier, 2015), MIKE HYDRO was used to capture the MIKE 11 network, cross-sectional information for all channels within the study area. There are a few steps:

- Cross-section locations were defined manually about every 50 m, using a 1 m LiDAR dataset to ensure that all significant channel and levee structure changes were captured.
- MIKE HYDRO was then used to extract cross-sectional levels directly from the LiDAR in the 1D model. Figure 5-8 shows the MIKE 11 Network with the cross-section extraction locations.
- The cross-section and network files were imported into MIKE 11, and every cross-section was checked manually for geometric and conveyance errors and to define left and right levee banks.
- Finally, additional cross-sections were interpolated if necessary, ensuring spatial equivalence between the MIKE 11 cross-sections and the grid-spacing in MIKE 21.

For the spoil bank fail scenarios, the right bank of the Birrega Main Drain was removed for the drains. The MIKE 11 cross-sections were modified so that the right bank height corresponded to the elevation of the LiDAR immediately to the west of the levee bank.

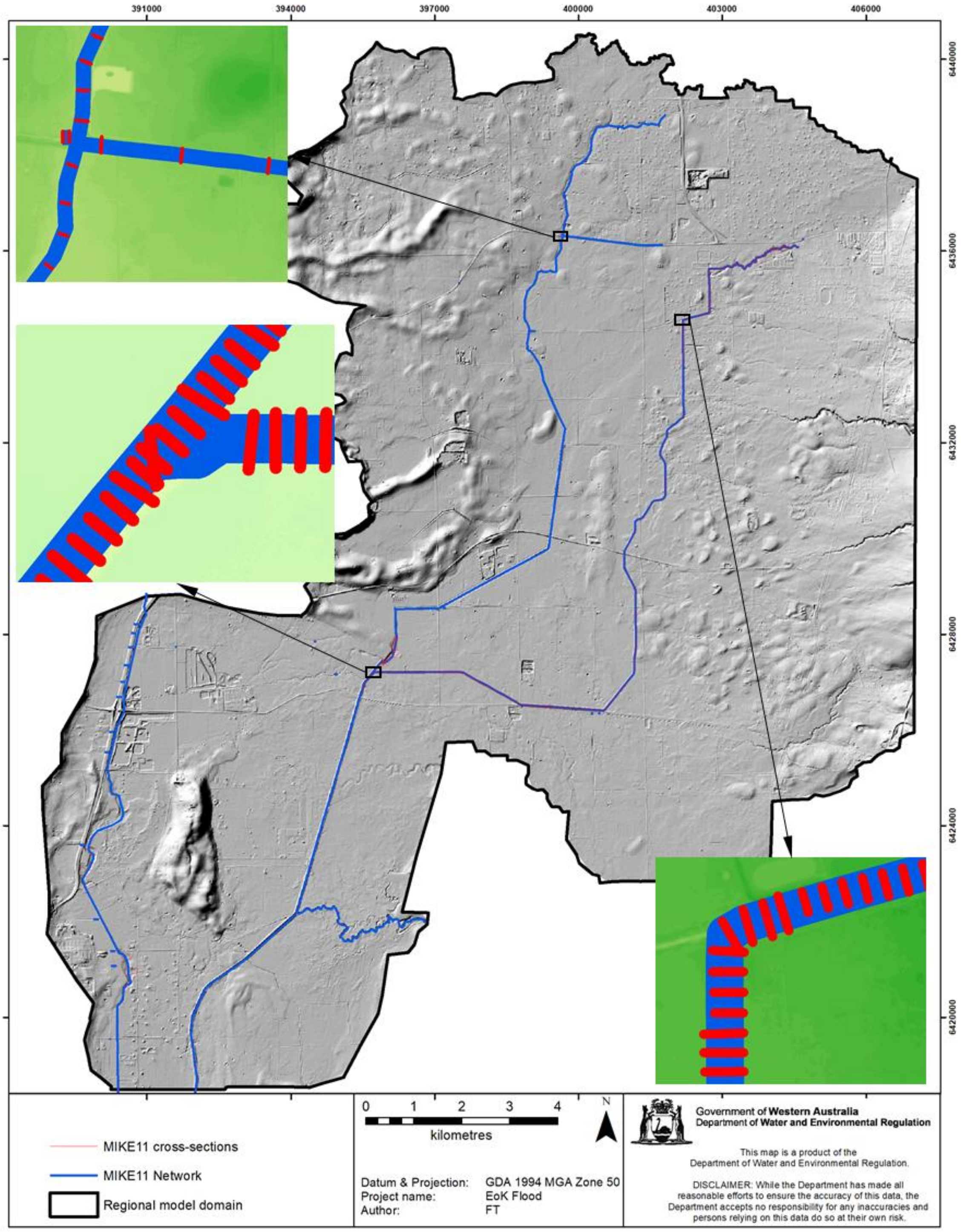


Figure 5-8 MIKE 11 Network and Cross-sections– insets show the cross-sections at three locations

5.3.3 Boundary data and Initial conditions

An initial water level in the 1-D waterways was necessary to simulate baseflow conditions. The initial water level was taken from the model 'hot-start' described in §5.2.6. The hot-start was simulated with 20 mm of rainfall in 12 hours and then allowed to drain for a day. This run had a constant inflow of 0.5 m^3/s in Birrega Main Drain, Oaklands Main Drain, Serpentine River and 0.1 m^3/s in Thomas Road Drain.

The water depth in each of the main drainage channels at the end of the hot start simulation used as the initial conditions in the calibration and design events is generally less than 50 cm, and is shown in Figure 5-9 (Birrega Main Drain upstream) and Figure 5-10 (Oaklands Main Drain), Figure 5-11 (Birrega Main Drain downstream and Serpentine Main Drain).

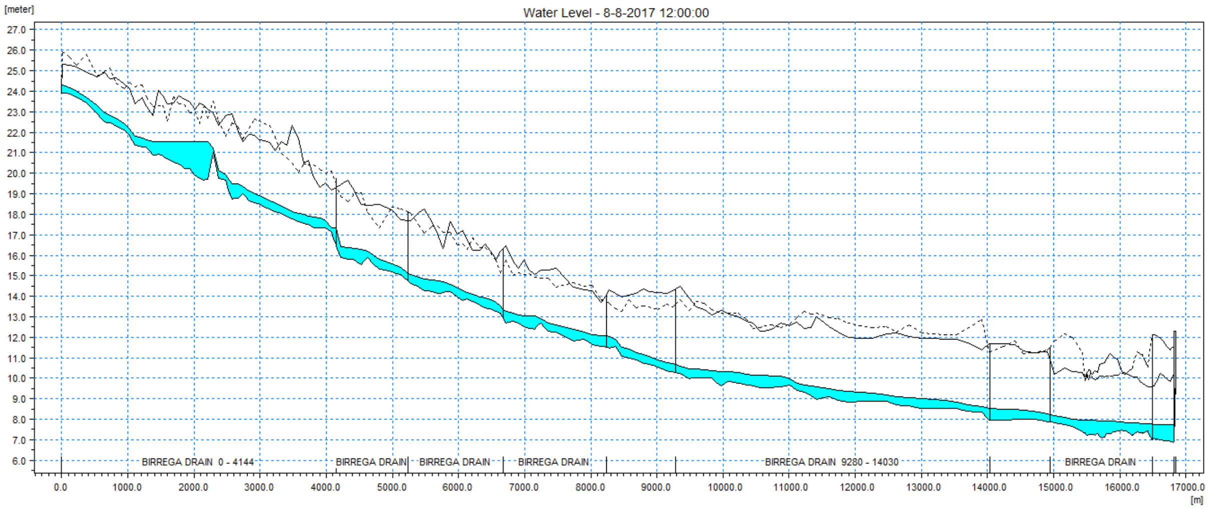


Figure 5-9 Initial water level for the Birrega Main Drain before its confluence with the Oaklands Main Drain

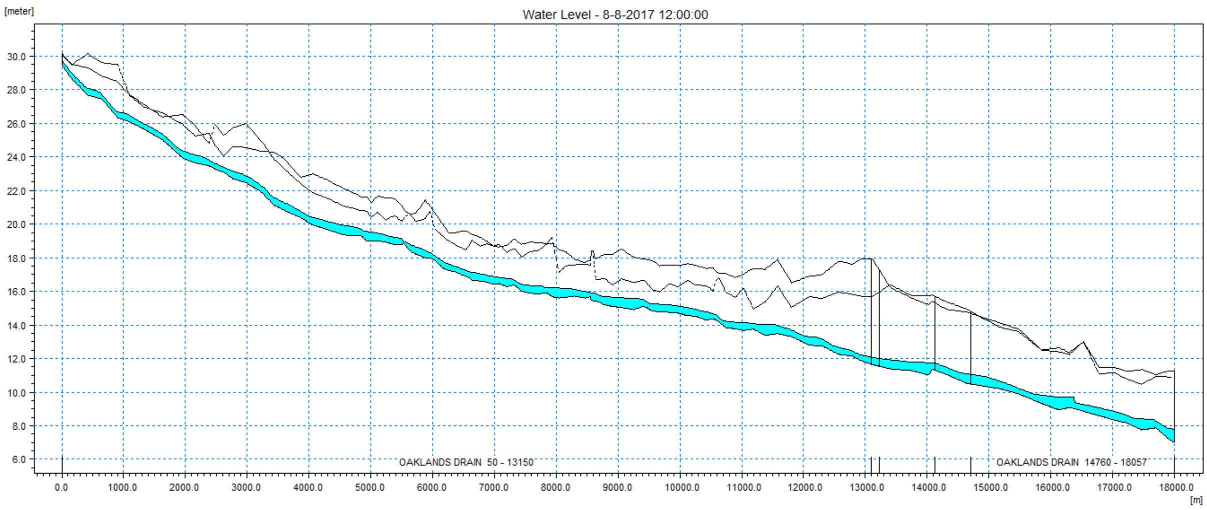


Figure 5-10 Initial water level for the Oaklands Main Drain before its confluence with the Birrega Main Drain

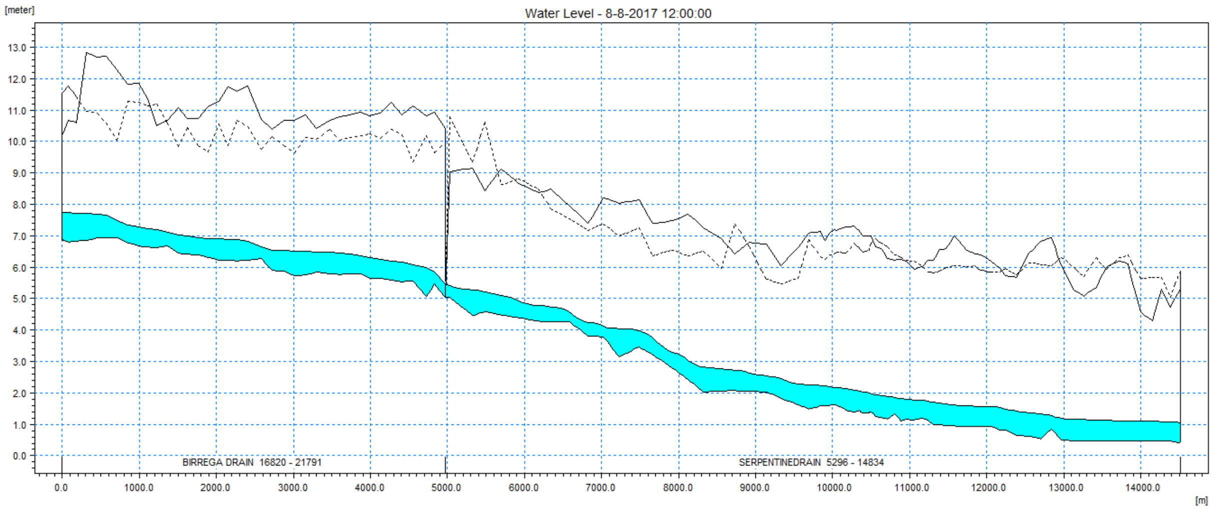


Figure 5-11 Initial water level for the Birrega Main Drain after its confluence with the Oaklands Main Drain, and for the Serpentine Drain.

5.4 MIKE FLOOD

A fully coupled 1D-2D hydrodynamic interaction modelling technique was adopted to transfer water volume between channels and the overland. The MIKE 21 FM model (representing floodplain and some selected low-flow channels) was coupled with the MIKE 11 model (representing major rivers and drains and hydraulic structures) in a MIKE FLOOD environment.

The models were run using a graphical processing unit (GPU) architecture. The GPU accelerates models running on the central processing unit (CPU) by offloading some of the time consuming and compute-intensive codes. As the GPU has massively parallel computing power, it significantly boosts model performance by reducing computational time. The 2019 release version was used in developing models.

5.5 Regional model calibration and validation

Model calibration and validation is an essential process for any flood modelling, as it demonstrates confidence in the hydraulic model's ability to reproduce known flooding behaviour.

The data for several rainfall and streamflow gauges is available for a recent flow event in August 2017. This event did not significantly flood the broader floodplain, but the major drains were running at near full capacity. Consequently, this event's information was considered suitable to calibrate the roughness within the 1D components of the model.

A more significant event occurred in July 1987 (~2% AEP), for which there was anecdotal information that the major drains breached and peak flood levels at five locations within the floodplain. Gauged streamflow information was only available at the Serpentine Drain at Dog Hill (614030) site for the July 1987 event. The anecdotal flood levels and gauged data at Dog Hill were used to validate the model parameters.

Calibration involved adjusting the following parameters within the model:

- The gridded Manning's M values in MIKE 21 FM,
- The Manning's M values in MIKE 11 network,
- The gridded infiltration rates in MIKE 21 FM.

5.5.1 Calibration to the 2017 event

The model calibration for the 2017 event consists of two components:

- Calibration of channel roughness at Dog Hill considering a MIKE 11 flow network (1D alone)
- Calibration of overland roughness and infiltration considering the East of Kwinana MIKE FLOOD model (1D and 2D coupled).

Previous studies in this area have found that channel roughness is the most sensitive parameter in a hydraulic model. To calibrate channel roughness coefficient (Manning's M) alone, the water level datasets for the upstream gauges on the Birrega Main Drain (614130) and Serpentine River at Lowlands (614114) were input directly to the model. They routed to the Serpentine River Dog Hill (614030) site. Figure 5-12 and Figure 5-13 show the flow network used for the roughness study. Modelled inflows for the small area between the gauges were included in the modelling. In the west of the study area, the northern inflows and flows within the Peel Main Drain model domain were also included.

The Manning's M value was varied from 15–40, and the resulting modelled discharge hydrograph at the location Dog Hill gauge on the Serpentine River was extracted and compared to the observations. Figure 5-14 shows that a Manning's M value of 30 best replicated the observed hydrograph. The previous North East Baldivis flood study considered varying roughness values (Manning's M) of 25 and 33, depending on channels characteristics (clean, straight or winding reaches with pools). The calibrated value of M in this study falls within the range considered for the previous research by Marillier (2015).

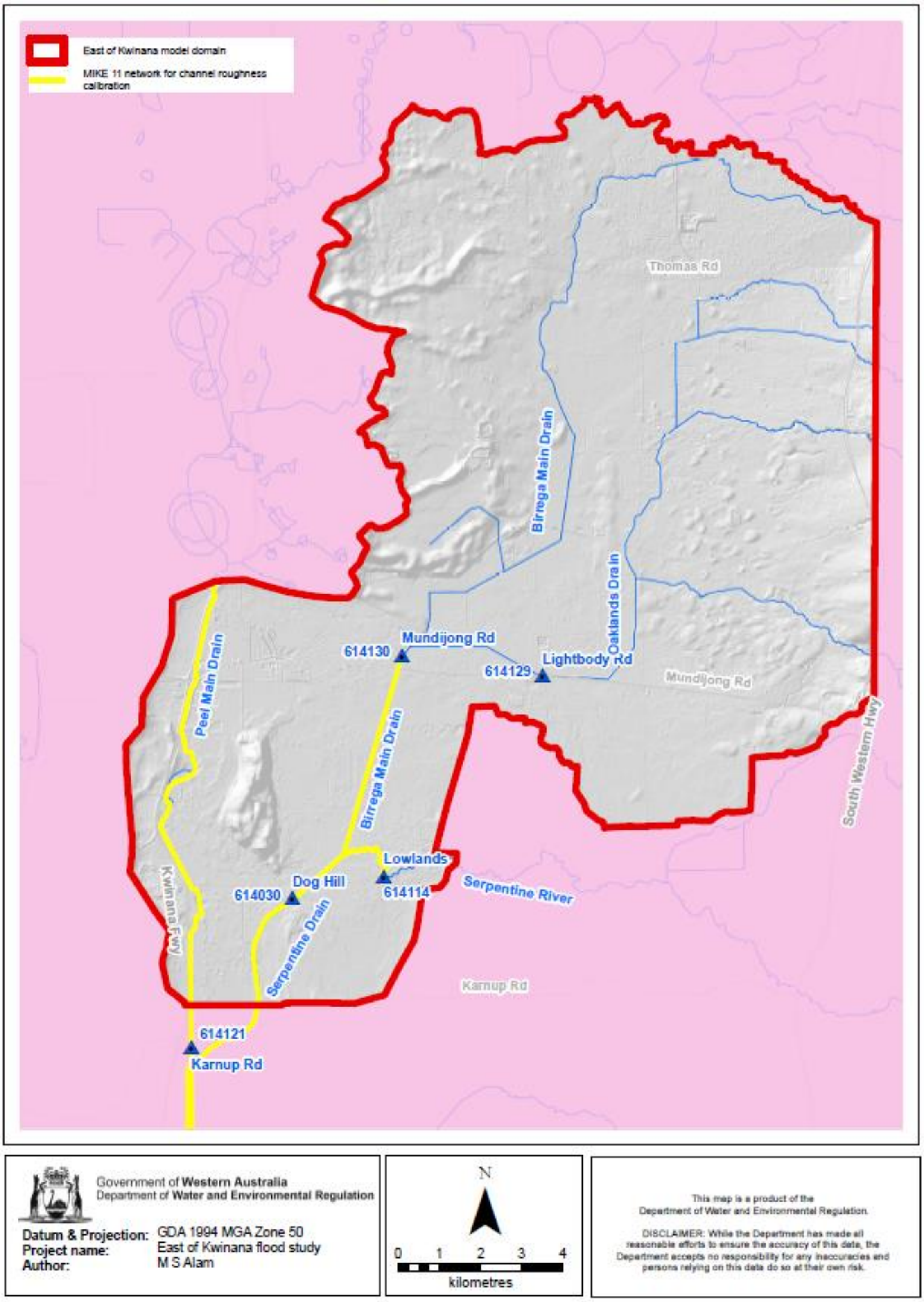


Figure 5-12 Channels considered for the calibration of channel roughness coefficient

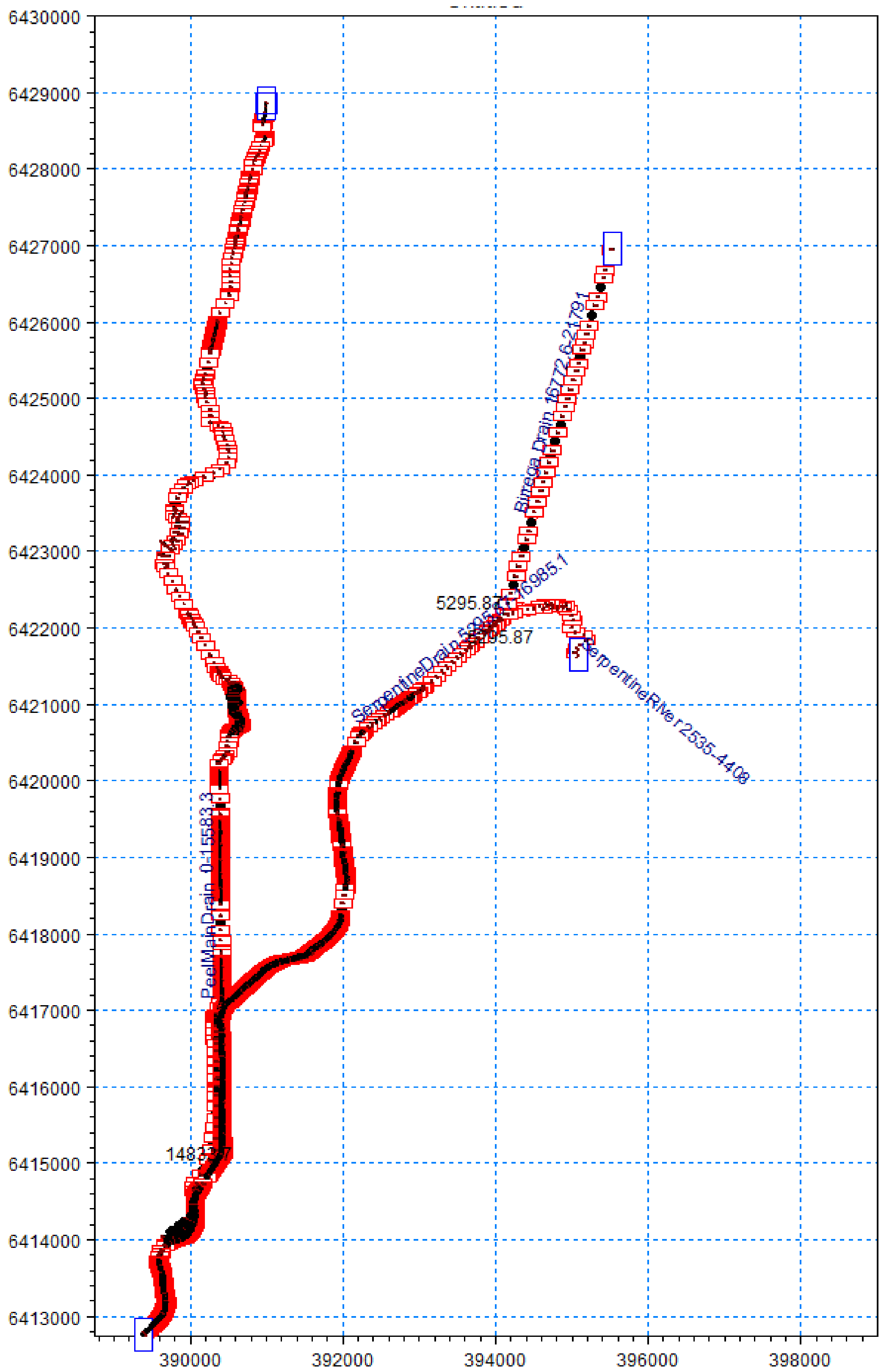


Figure 5-13 Flow network as seen from MIKE11 considered for the channel roughness coefficient's calibration (Source: MSA 2020)

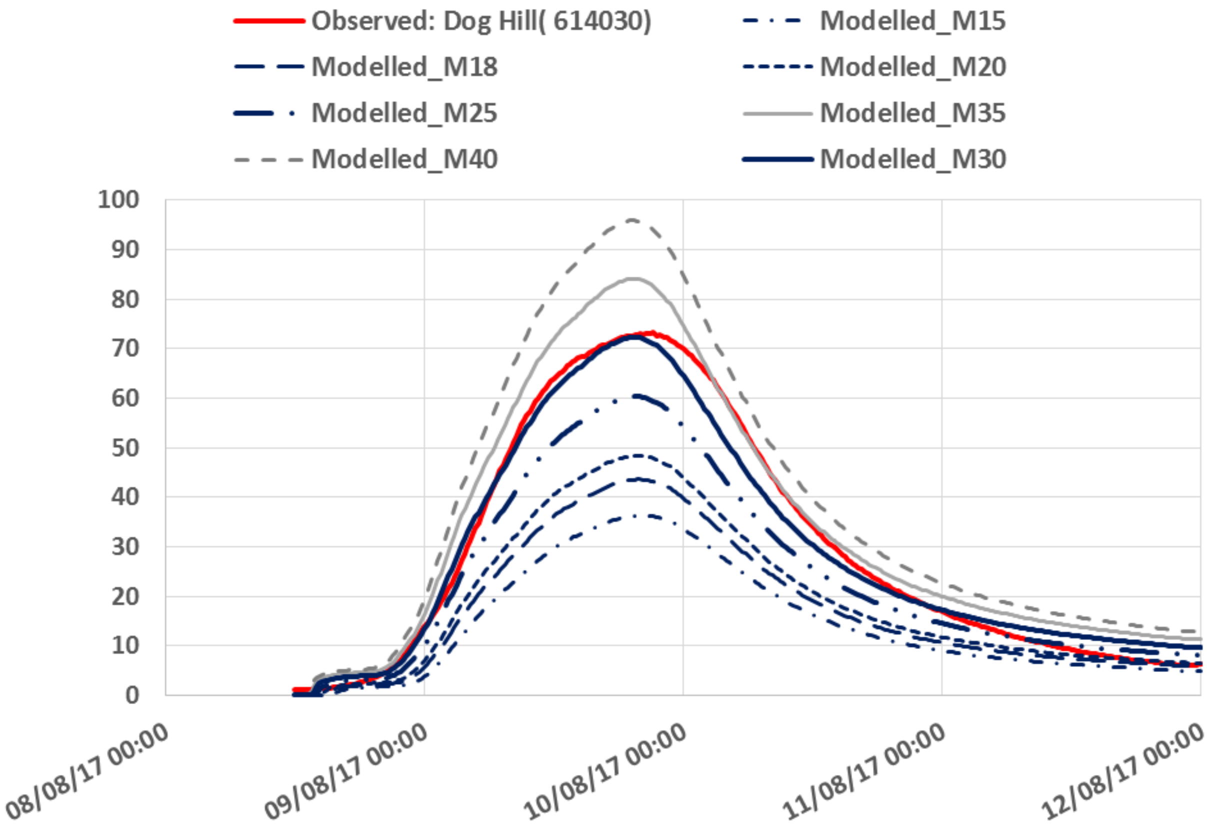


Figure 5-14 Calibration of channel roughness at Dog Hill site using Manning's M for the 2017 event (Source: MSA 2020)

5.5.2 Validation of the 1987 event

The 1987 event is considered to have an estimated likelihood of about 2% AEP for the gauging station on the Birrega Main Drain at Mundijong Road (Hall, 2015a, p. 24). The observed water levels for this event were higher than the 2017 event, and considerably more flooding was observed on the floodplain. The Shire of Serpentine Jarrahdale collected peak water level information at five locations (Figure 5-15).

The 1987 event rainfall information and modelled inflows for this event were applied, and the Manning's M for the drains represented within the MIKE 11 component of the model was set to the calibrated value of 30. The overland roughness parameter and the infiltration rates for soils were varied in the MIKE 21 FM model to reproduce the observed levels. The peak flood levels within the catchment were successfully reproduced using an overland roughness value ranging from 12.5 to 40 depending on land-use (refer to Figure 4.4) and losses of 70 mm/hour and 5 mm/hour for sand and clay, respectively.

The ability of the model to reproduce observed flow behaviour both within the channel and in the floodplain provides greater confidence in the model's ability to simulate the flood behaviour for large flood events (2% AEP to 1% AEP).

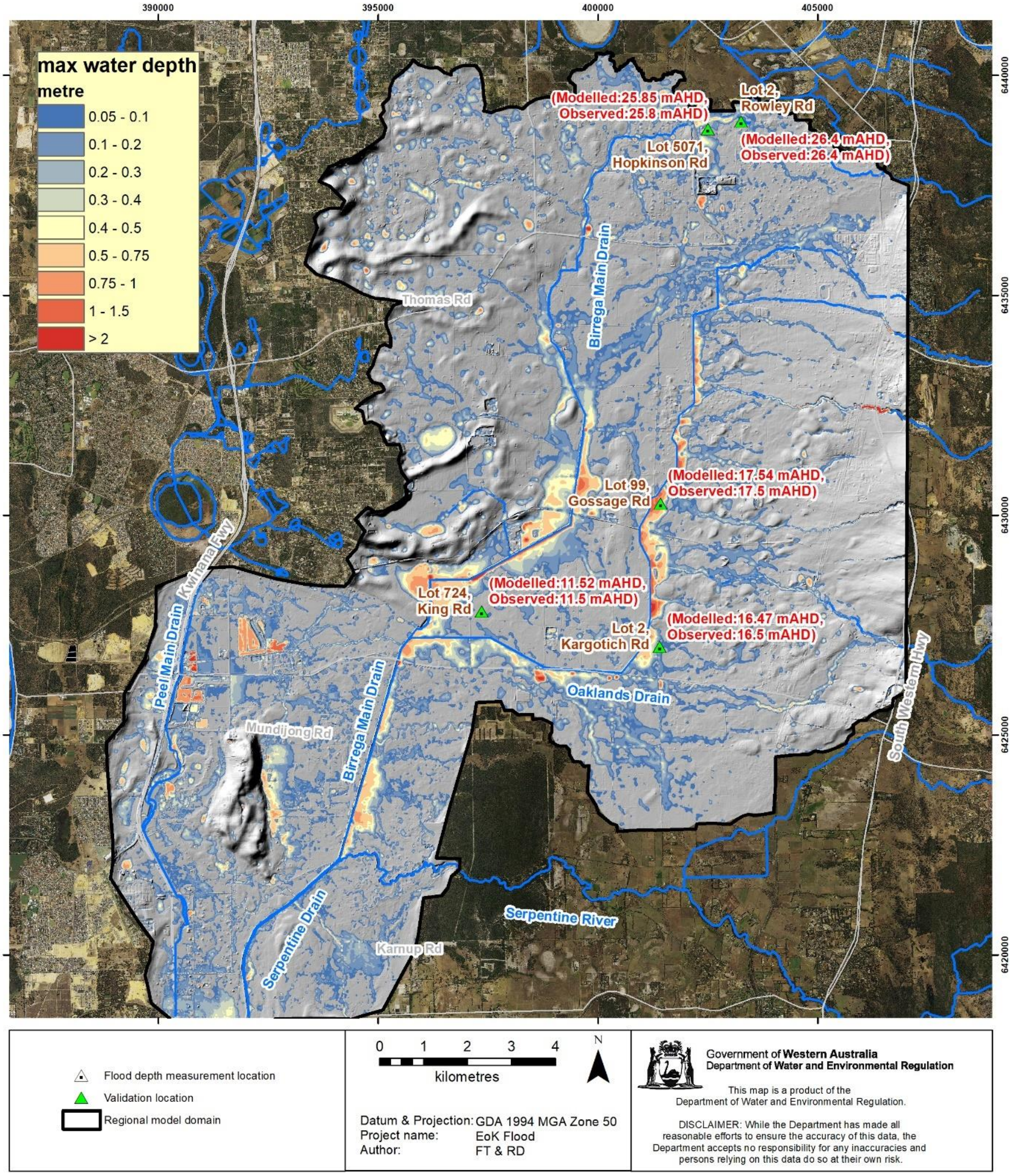


Figure 5-15 Observed versus modelled water levels for the 1987 flood event, along with the flood mapping from the simulation

5.6 Local flood model

The construction of the local flood model was the same process as the regional model outlined in section 5.2. For the pre-development and some post-development simulations, both models share the same model inputs, including rainfall and inflow conditions (as discussed in sections 4.1 and 4.2), and the same roughness (Figure 5-4) and infiltration (Figure 5-7). This information is not repeated here.

The section outlines:

- the aim of building a local flood model, and
- model setups for the local flood model.

The model validation for the local flood model will be present in section 6.3.

5.6.1 Aim of the local flood model

One of the local model goals was to enable post-development scenarios to be stamped onto the existing bathymetry layout for the North East Baldivis north of Mundijong Road precinct. To achieve this, we built a local flood model with a level of detail much finer than the resolution of the regional model. The more satisfactory resolution of the local model leads to better representation of the existing land surface (including road crests, bunds, etc.) and therefore increased confidence in flood behaviour.

5.6.2 MIKE 11 elements

A much finer mesh element was used in the local flood model where drains and waterways are present (Figure 5-16), and therefore, in most cases, MIKE 11 model was not employed in the local model. This is further justified as most of the 1D elements included in the regional model at the North East Baldivis north of Mundijong Road precinct convey minimal discharges in events up to 1% AEP. The Peel Main Drain, on the other hand, was resolved using an even more fine 2D mesh element than other areas (Figure 5-16).

The comparison of the bathymetry of Peel Main Drain at selected locations between the local and the regional model is given in Figure G-14.

This set-up also enables a broad range of post-development options (refer to Section 7) to be run using a single model for comparison. In these post-development options, drains and culverts may differ between development scenarios. For this reason, 1-D elements in the local model, including the Peel Main Drain, are modelled in the MIKE 21 model with sufficiently fine mesh. Using MIKE 11 model would otherwise lock them in position.

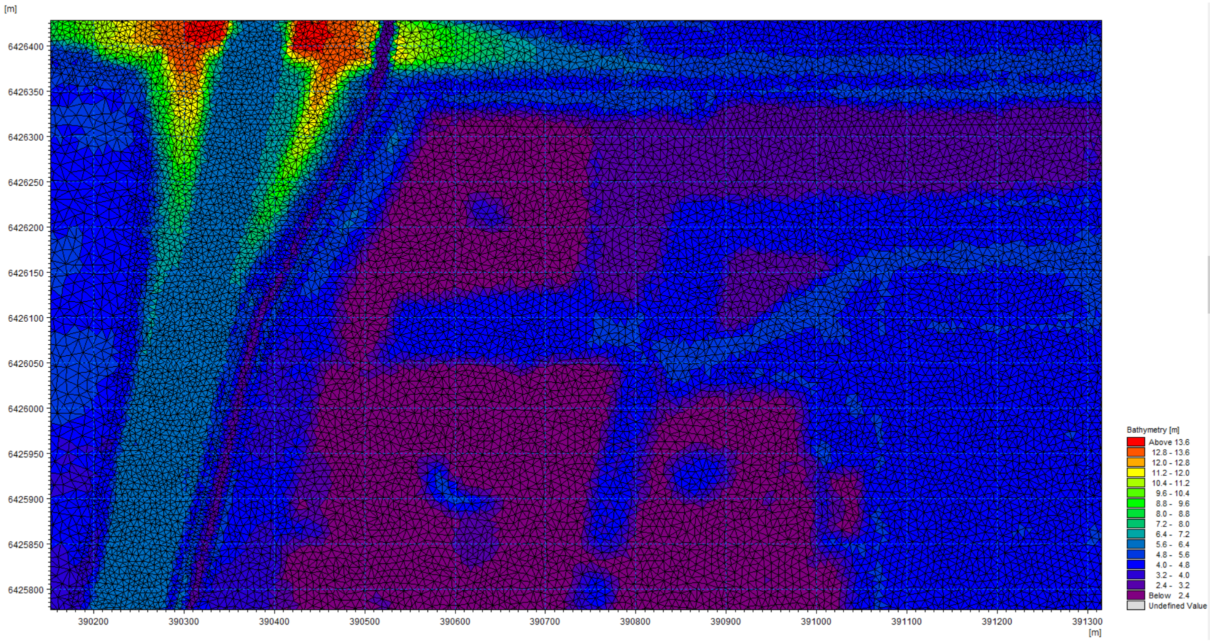


Figure 5-16 Example of mesh showing fine resolution at the corner of Mundijong Road and Kwinana Freeway in the local model

5.6.3 Boundary of the local model

The boundary of the local flood model was carefully selected to be a catchment divide, based on the terrain elevation and the 1987 modelling flood levels from the regional model.

As shown in Figure 5-17, the flood model boundary was chosen along topographical ridges on mountainous land. However, on flat ground, the boundary was determined to cross insignificant ponding areas where water interchange along the edge was negligible.

Upstream of the North East Baldivis north of Mundijong Road precinct, the Birrega Main Drain was chosen as the boundary while cutting out a small portion of the PIA sector area around the Banksia Road. As discussed later, the Birrega Main Drain's potential breakouts were modelled by a source inflow condition in the local model.

Downstream of the North East Baldivis north of Mundijong Road precinct, Folly – Safety Bay Road as the outlet boundary, about 4 km south of Mundijong Road, leaving enough flood basin to minimise the boundary impact. A sensitivity check on this boundary of the Peel Main Drain outlet at Folly Road is given in section 6.3.

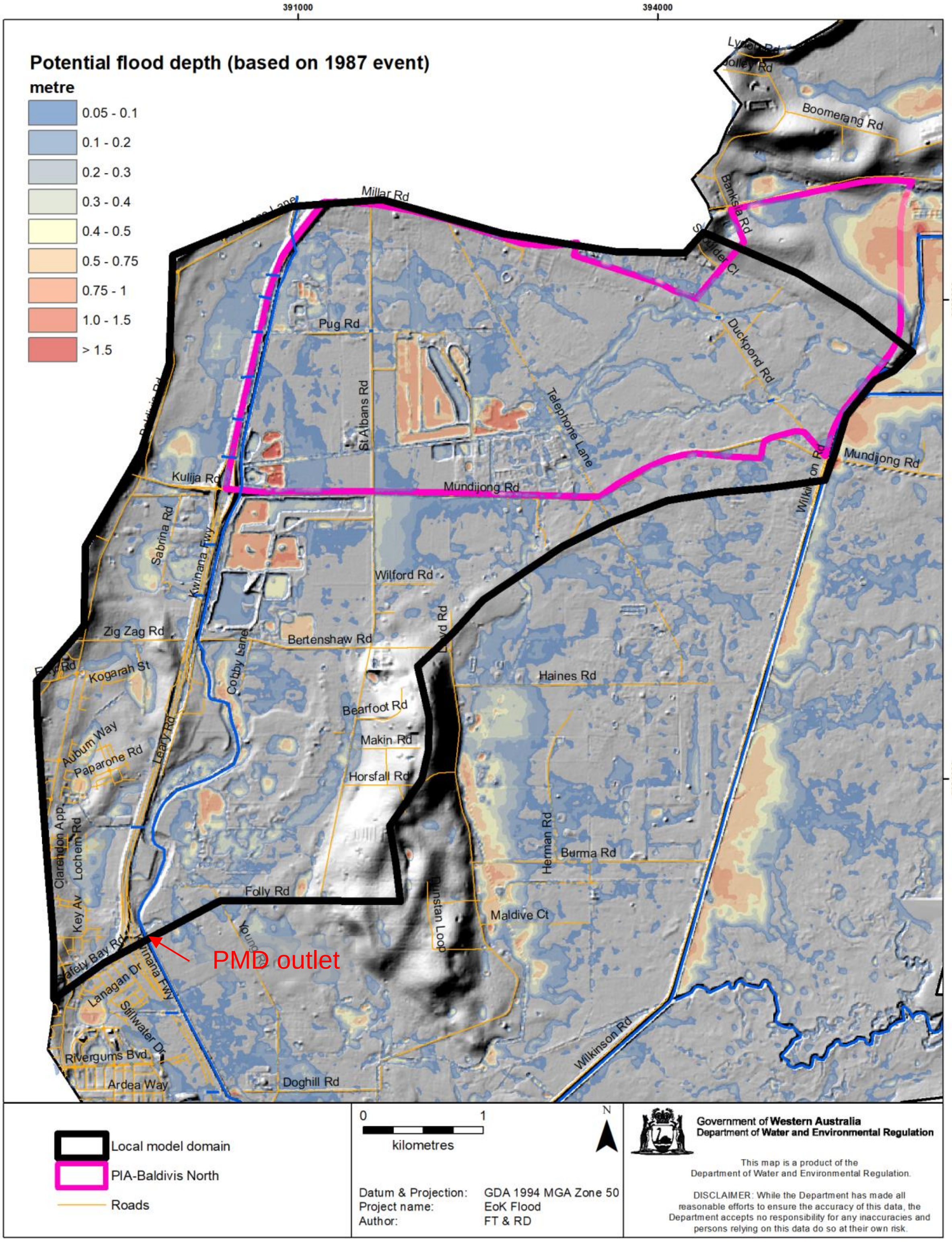


Figure 5-17 The local flood domain and the North East Baldivis north of Mundijong Road precinct and the 1987 modelling flood water level from the regional model

6 North East Baldivis north of Mundijong Road: Pre-development

The results from two hydraulic models were assessed to investigate the flood behaviour (i.e. flood extent, flood depth, discharges, etc.). The results reported in this chapter are from the regional model, covering an area of 232.7 km^2 and a local model focusing on flooding within an area of 24.3 km^2.

For the regional model pre-development modelling:

- Design rainfalls were applied to the coupled hydraulic model for the 10%, 5% AEP, 1% AEP and 0.5% AEP events, for durations of 12-, 24-, 36- and 48-hour.
- The impact of spoil bank fail scenarios, referred to as 'levee fail' in Hall (2015a, p. 73), was one of this study's main priorities. Several potential bank failure scenarios were investigated at 1% AEP, where banks were removed from the bathymetry at locations deemed likely to fail along the Oaklands Main Drain and the Birrega Main Drain.
- Model results are presented in several forms, including cross-sectional discharges and simulated maps of flooding levels.
- Detailed flood mapping of simulated maximum water levels and flood extent based on the combined maximum water depth is presented.
- Peak flood levels and discharge for the Oaklands Main Drain and sections of the Birrega Main Drain are described.

For the local model pre-development modelling:

- Design floods were simulated for the 1% AEP events, only for the critical duration (18-hour).
- One of the spoil bank fail scenarios along the Birrega Main Drain was presented.
- Model results were compared with that of the regional model.

6.1 Pre-development base case for comparison

The pre-development base case establishes the flooding risk under the current rural land-use. It allows for understanding how water in the catchment behaves (both inside and outside East of Kwinana). The pre-development base case was established to compare with post-development scenarios and any future land-use change proposals.

The area is known to experience groundwater inundation and periodic flooding. Water Corporation has constructed a network of drains to drain the inundated regions within 72 hours of rainfall (Hall, 2015a). However, the drain networks are not designed as primary flood mitigation works.

The potential for spoil bank failure was assessed considering common failure mechanisms known to practitioners and identified in documents including the *Levee management guidelines* (Victoria 2015) and the *Flood emergency planning for disaster resilience handbook* (Australian Government, 2017).

The spoil bank assessment identified the most likely locations where failures could occur and were validated against information available from the 1987 flood. Spoil banks on the Birrega Main Drain adjacent to North East Baldivis north of Mundijong Road are likely to fail during a 1% AEP flood event.

A range of failure mechanisms were considered including flood waters overtopping the spoil banks in a 1% AEP event. There is insufficient freeboard between the 1% AEP water level in the Birrega Main Drain and the top of the spoil bank on the drain adjacent to North East Baldivis north of Mundijong Road to conclude that it will not overtop. The spoil bank has not been designed to withstand overtopping indicating that an uncontrolled failure is likely.

The lack of appropriate design, construction certification and a continuous and long-term maintenance and surveillance program further confirm that the spoil banks should not be relied on for flood protection in a 1% AEP flood. One-off investigations such as geotechnical assessment can easily miss latent defects that are likely to initiate failure and do not provide a substitute for a long-term maintenance and surveillance program.

- From the assessment above, the pre-development base case includes spoil bank failure along the Birrega Main Drain adjacent to the North East Baldivis north Mundijong Road precinct. The detailed spoil bank failure scenarios are discussed in §6.2.3 and §6.2.4.

6.2 Regional model results

6.2.1 Critical durations

A study area's critical duration is defined as the representative duration of the design rainfall that produces the maximum peak floodwater depth for much of the area.

In the present case, this was investigated by detailed flood mapping from four selected duration runs from the design events; That is, the 12-, 18-, 24- and 36-hour durations. These durations were selected based on previous studies. Studies (SKM, 2010b; Hall, 2015b; Marillier, 2015) have suggested the critical durations for the 1% AEP event at many locations in the area are around 24-hour.

Design event at 1% AEP

The 1% AEP critical duration was determined using a composite flooding map in Figure 6-1. At each 5 m×5 m grid, the map displays the maximum flood depth at that grid from the 12-, 18-, 24- and 36-hour durations. The median temporal pattern was used in the modelling at each duration.

There are two steps in generating Figure 6-1.

I. For each of the four rainfalls (i.e. the rainfalls in Figure 6-4), a map of peak flood depth is generated from the gridded MIKE 21 results files (5 m×5 m grid).

II. The maximum flood level at each grid was calculated to determine an overall maximum flood map from these four rainfalls. Therefore, this maximum flood map loses temporal information but is the highest possible flood level from the four 1% AEP events.

The critical rainfall duration was determined that was representative and produces the maximum peak floodwater depth for much of the area.

For the 1% AEP, the critical duration for low-level drains (based on MIKE 21) east of Hopkinson Road is mostly 12-hour. Most areas on both sides of Oaklands and Birrega main drains, the North East Baldivis north of Mundijong Road precinct, and some ponding area west of Nicholson Road extending up to the west side of the hydraulic domain, have a critical duration of 18-hour. Towards the downstream of the catchment, especially south of Mundijong Road and west of Wilkinson Road, the critical duration increases to 24-hour. The 36-hour duration is only critical in some relatively sparsely distributed ponding areas (local depressions), mostly disconnected from main flow paths.

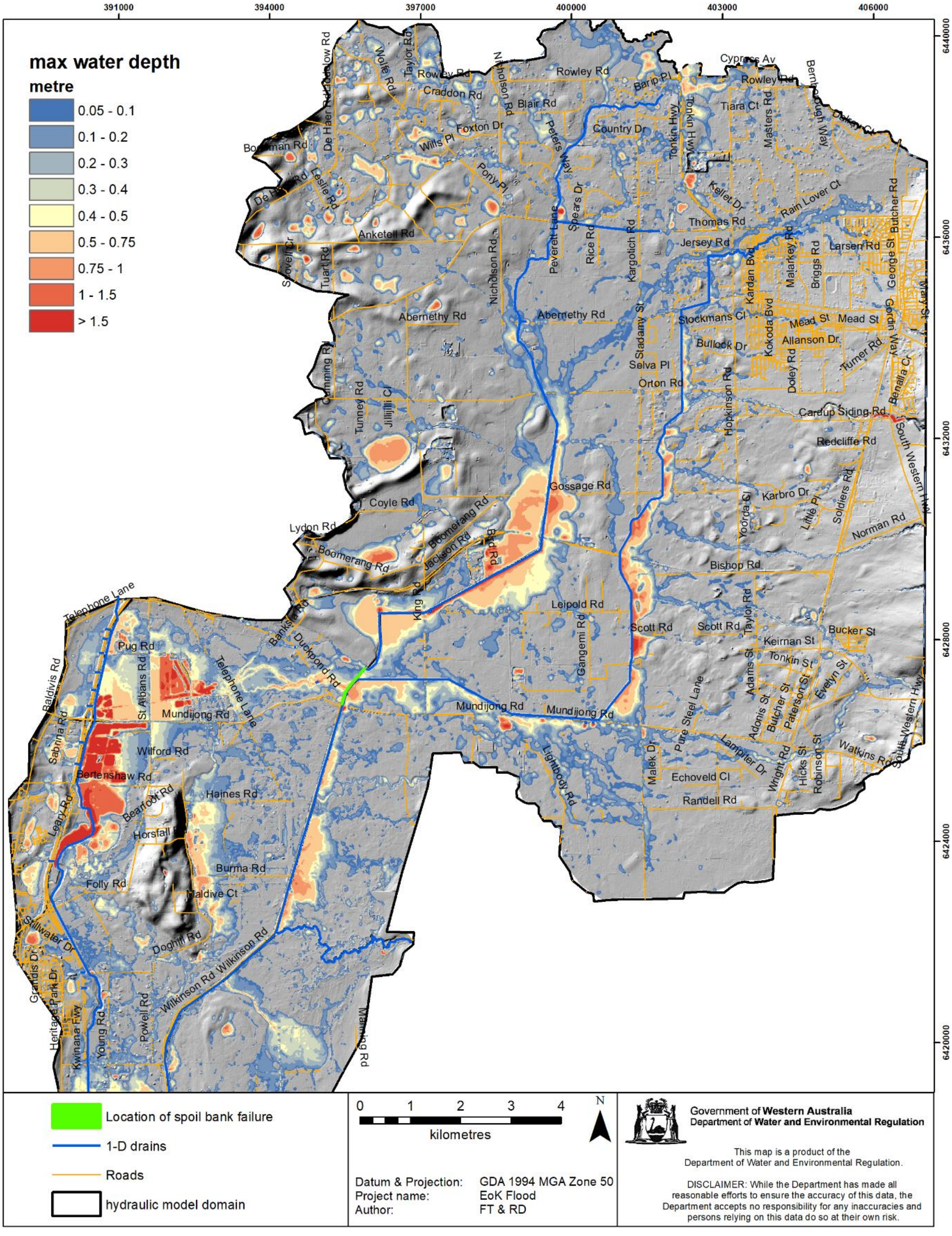


Figure 6-1 Flood mapping from 1% AEP runs (12, 18, 24 and 36-hour durations) at the spoil bank fail 100% scenario, as discussed in section 6.2.3. Each of the rainfalls was deemed to be evenly distributed.

The critical duration for the 1% AEP is deemed 18-hour for the whole area of interest. This is based on two reasons:

- The 18-hour rainfall produces maximum flooding water depth for much of the area inside the domain.

- The 18-hour rainfall produces a representative peak flooding water depth for almost all North East Baldivis north of Mundijong Road precinct.

Figure 6-2 sampled the discharge from the Birrega Main Drain at the Duck Pond (refer to DP in Figure 6-11) to enter the North East Baldivis north of Mundijong Road precinct. It further demonstrated that 18-hour is critical as the discharge from the 18-hour design rainfall is relatively large.

This temporal pattern of rainfall and inflows at 1% AEP (18-hour Tp1) was used for model inputs in the following study unless otherwise specified.

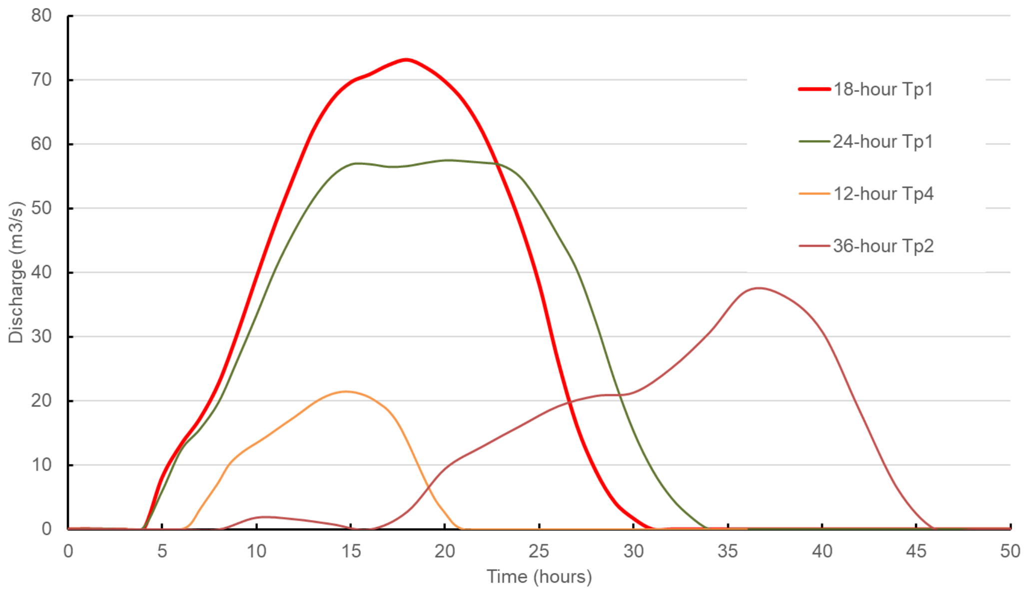


Figure 6-2 Sampled discharges from Birrega Main Drain into North East Baldivis north of Mundijong Road precinct under different rainfall temporal patterns, Tp1 to Tp10, for the 1% AEP 18-hour event

Design event at other AEPs

Similarly, the critical durations at three other AEP events were determined. The critical durations for 10%, 5% AEP and 0.5% AEP rainfalls are all found to be 18-hour. This is consistent with the fact that the same temporal patterns were used in these design rainfalls following ARR data hub (2020), which are based on the areal temporal pattern from S-SW Flatlands (west).

The corresponding flood maps and critical-duration maps are given in the Appendix, which are:

- For 0.5% AEP, the maximum flood map from the four duration runs is given in Figure B-6.
- For 5% AEP, the maximum flood map from the four durations runs is given in Figure D-10.
- For 10% AEP, the maximum flood map from the four durations runs is given in Figure E-11.

6.2.2 Median temporal pattern

The median temporal pattern (Tp) is the one that produces the median flooding conditions within the model area at a location of interest. Selected temporal patterns at certain AEPs are given in Appendix A.

In this study, the temporal rainfall pattern that produces the median discharge from the Birrega Main Drain at the location of Duck Pond (refer to DP in Figure 6-11) was deemed the critical location for defining the median temporal pattern. The discharge here would enter the North East Baldivis north of Mundijong Road precinct. Some justifications for this breakout event are discussed in §6.2.3.

Design event at 1% AEP

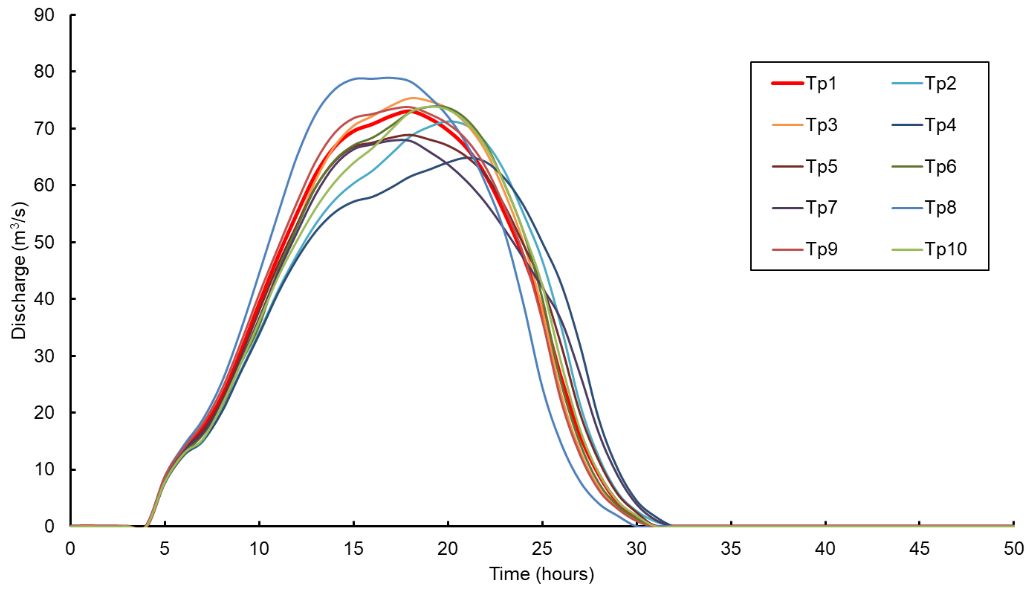


Figure 6-3 Sampled discharge at Duck Pond from the Birrega Main Drain under different rainfall temporal patterns, Tp1 to Tp10, for the 1% AEP 18-hour event

Figure 6-3 gives an example of the sample discharges from 1% AEP 18-hour rainfalls. The sampled discharges correspond to the 10 different temporal patterns of the 18-hour design rainfall.

The maximum discharge varies from 64.8 m^3/s (Tp4) to 78.9 m^3/s (Tp8), and the median is about 73.4 m^3/s. This difference is significant, considering these discharges were induced by the same amount of rainfall over 18 hours.

There is also a time lag between when the discharge reaches its maximum. For instance, Tp4 peaks after 21 hours, while Tp8 peaks about four hours earlier, at 17 hours.

The temporal distribution of the rainfall influences these changes. Tp8 is mostly front-loaded (Loveridge, Babister and Retallick, 2015, p. 28), where most of the rain was concentrated in the first nine hours, while Tp4 is mostly back-loaded rainfall event.

The discharge resulting from Tp1 rainfall with a maximum value at 73.1 m^3/s is the closest to the 10 discharges' median values and was selected to be the median temporal pattern for the 18-hour duration rainfall.

A similar process was followed to determine the median pattern out of 10 for other durations and AEPs. For example, based on 40 simulation runs for the 1% AEP design events, median temporal patterns of the selected durations are deemed as:

- 12-hour Tp4
- 18-hour Tp1
- 24-hour Tp1
- 36-hour Tp2

These temporal patterns are given in Figure 6-4 (Note: The 40 Tps shown in Appendix A).

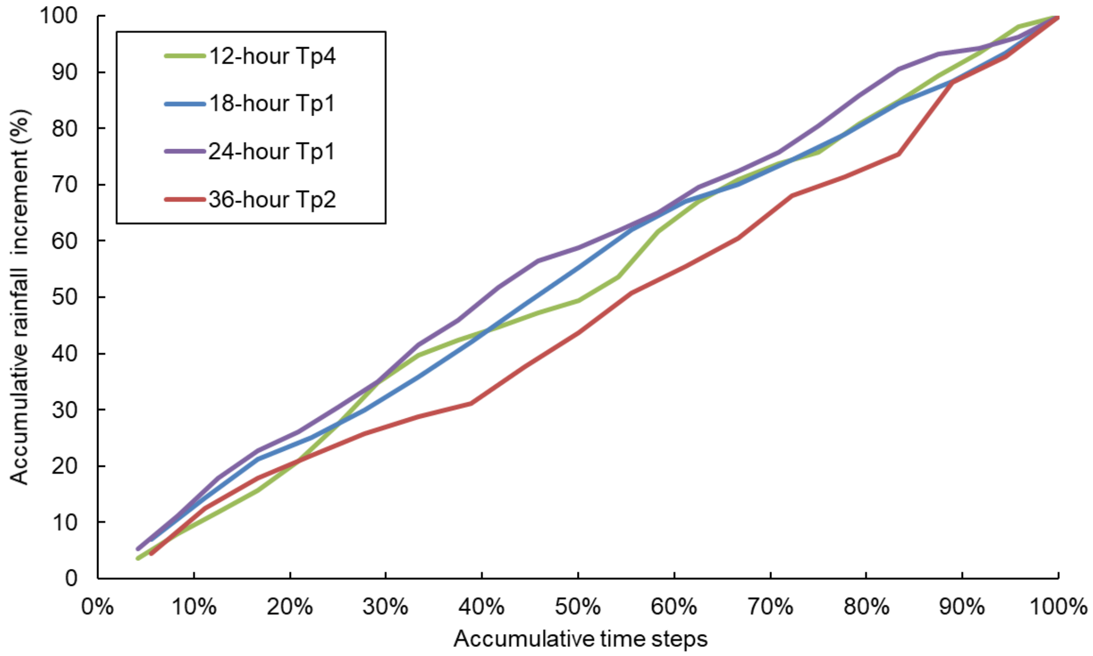


Figure 6-4 Temporal patterns deemed to be the median ones at 12-, 18-, 24- and 36-hour rainfall durations

It is worth noting that the median temporal pattern for the 18-hour duration does not seem to change with the frequency of rainfall events, including the 0.5% AEP and 5% AEP. For more frequent rainfall events, such as the 10% AEP rainfalls, by definition, it changes to Tp5 or Tp6. However, for the 10% AEP conditions, the difference of maximum discharges at this location between Tp1 and Tp5/6 is within 1 m^3/s, which is deemed insignificant to the design flood modelling outcomes.

6.2.3 Spoil bank failure scenarios

The department has detailed the spoil bank failure mechanisms in the flood risk management land capability assessment (DWER, 2021). The flooding within the North East Baldivis north of Mundijong Road precinct is driven by flood waters breaching the spoil bank on the Birrega Main Drain and flowing in a westerly direction, following the natural fall of the land. The floodway through the precinct is a critical consideration for possible downstream development. Flooding of the North East Baldivis north of Mundijong Road precinct is affected by the likelihood and size of breach of the adjacent spoil bank.

The potential for spoil bank breach is considered likely under major flood events (including 1% AEP) for many reasons:

- There were observed breaches during an event in 1987. Anecdotally, the bank breached at this location and on the Oakland Main Drain near Scott Road, resulting in a significant loss in bank height (Hall, 2015a, p. 66). The 1987 rainfall is thought to be ~ 2% AEP event (Hall, 2015a, p. 24).
- The spoil banks are not engineered levees, and their geotechnical conditions are unknown but possibly unstable in high flows (Marillier, 2015, p. 75).
- Under the 1% AEP design event, the modelling completed in the Birrega catchment (Hall, 2015a) concluded that a complete bank failure was possible in this area.
- In the present study, the results of applying design rainfall and inflow to the regional model indicate that the spoil banks within the model domain are overtopped (or close to) at several locations, including the Birrega Main Drain near its confluence with the Oaklands Main Drain (as shown later in Figure 6-7).
- Advice from the asset owner.
- On ground conditions.
- Lack of appropriate historical maintenance and surveillance.

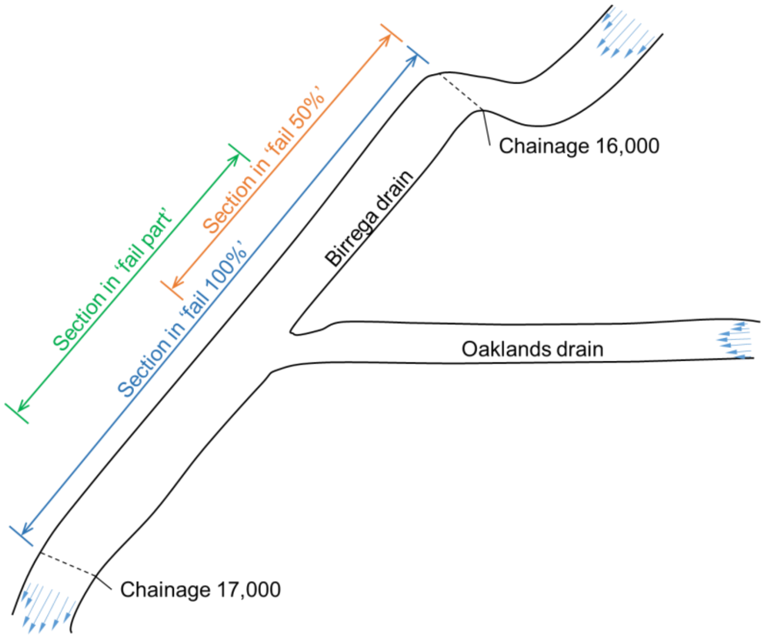


Figure 6-5 A schematic diagram on the Birrega Main Drain spoil bank failure scenarios

Since it is impossible to predict the exact location of spoil bank failure, three failure scenarios along the right bank of Birrega Main Drain at its confluence with the Oaklands Main Drain were investigated, along with investigations upstream and downstream of this location.

Figure 6-5 illustrates these scenarios at the confluence of the Birrega Main Drain and the Oaklands Main Drain.

- **Spoil bank fail 100%:** failure was considered over a length of about 1 km (Figure 6-6) on the right bank of Birrega Main Drain near its confluence with Oaklands Main Drain. This was simulated by adjusting the right bank to the level of the surrounding plain. This failure mode was named spoil bank fail 100%.
- **Spoil bank fail 50%:** failure was considered over the upstream 50% of the right bank (compared to the whole bank as discussed in spoil bank fail 100%). This was simulated by adjusting the about 500 m bank to the level of the surrounding plain. This failure mode was named spoil bank fail 50%.
- **Spoil bank fail part:** failure was considered over the central part of the right bank. About 410 m spoil bank (Figure 6-6) was adjusted to the surrounding plain's level. This failure mode was named spoil bank fail part.

These failure scenarios along the right bank of the Birrega Main Drain were investigated in conjunction with another two scenarios at this location:

- **Spoil bank intact**: the existing spoil bank was kept at its current condition without any height adjustment.
- **Partial flood mitigation (**Build levee to 10.5 m Australian Height Datum**):** A levee is constructed to a height of 10.5 m AHD over the whole length in question (Figure 6-6). The aim was to hold water within the main drain under 5% AEP and allow some overflow under 1% AEP.

The heights of the right bank in these five cases are given in Figure 6-6. The line ‘spoil bank intact’ shows the existing bank's level at its maximum points digitised from the LiDAR dataset (DEM).

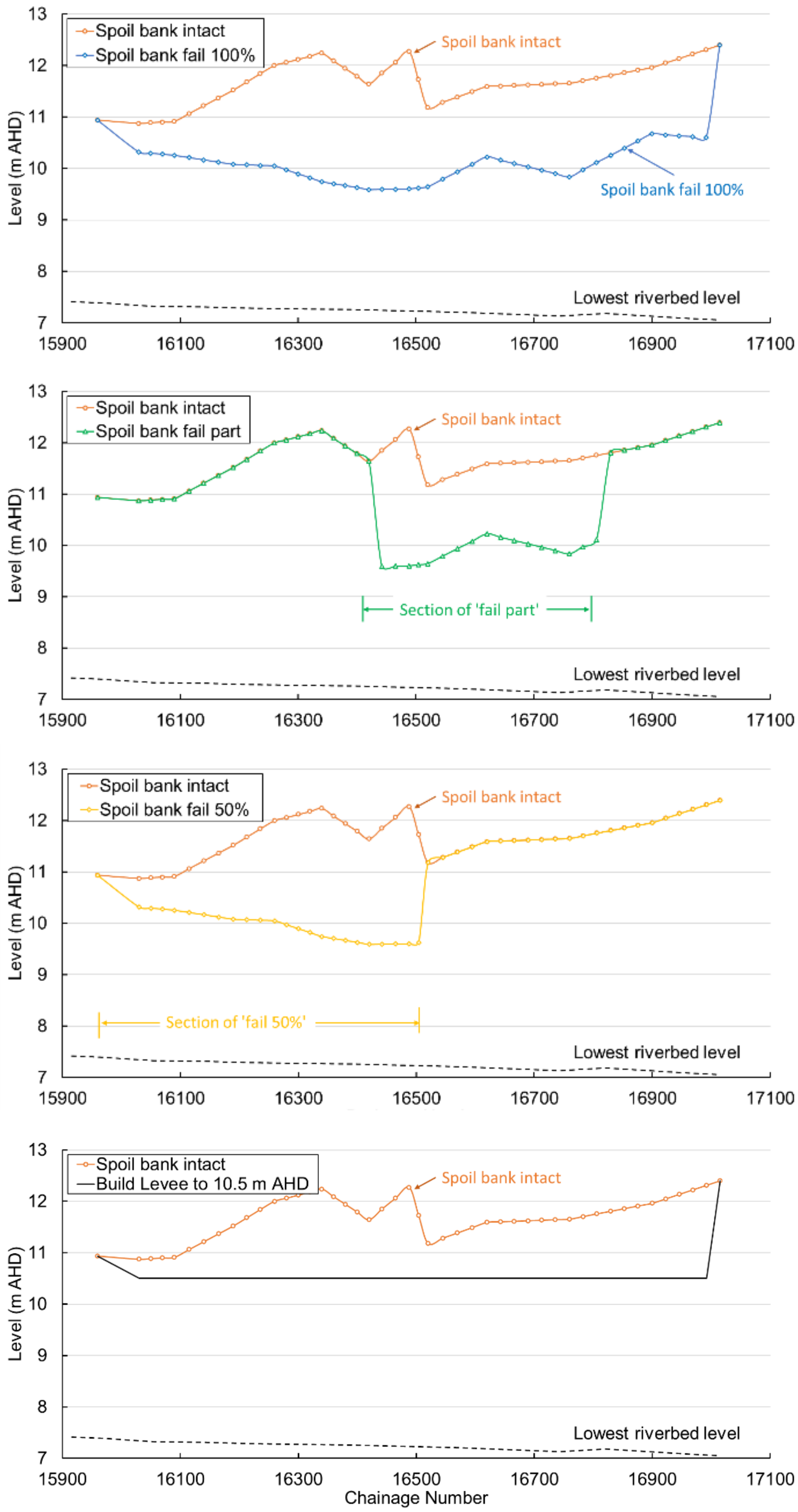


Figure 6-6 Comparison of the heights of the right bank of Birrega Main Drain in different spoil bank scenarios

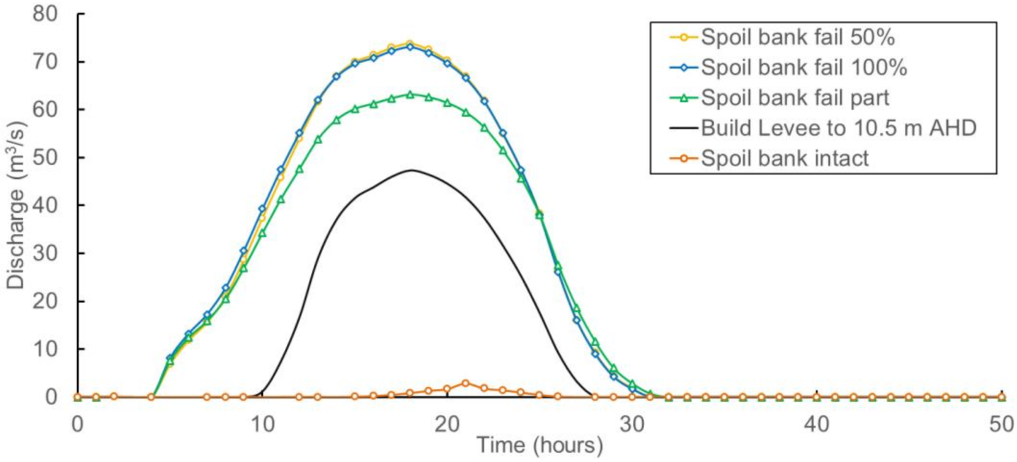


Figure 6-7 Sampled discharges from Birrega Main Drain to the North East Baldivis north of Mundijong Road precinct from five spoil bank failure scenarios (1% AEP 18-hour Tp1)

Figure 6-7 and Figure 6-8 compare the discharges from five simulation runs because of these five spoil bank conditions. Figure 5-7 compares the flows sampled along a cross-section immediately downstream of the spoil bank failure location (DP in Figure 6-11). Figure 6-8 illustrates the discharge inside the Birrega Main Drain sampled at six sections before and after its confluence with the Oaklands Main Drain.

The discharge sensitivity from a breach in the Birrega Main Drain shows that the three failure modes of the Birrega Main Drain do not significantly affect the discharges through a breach at the Duck Pond Road. Some main observations from 1% AEP 18-hour rainfall are:

- The discharge is relatively independent of location and length of bank failure. The maximum discharge ranges from 63.1 to 73.7 m^3/s, a mere 14% change despite a significant difference in size/location of bank failure.
- Building an engineered levee over a length of about 1 km on the right bank of the Birrega Main Drain to 10.5 m AHD would decrease the maximum discharge to 47.3 m^3/s but still has a detrimental impact on neighbouring properties, both upstream and downstream (Section 7.3.3).
- A maximum rate of about 60 m^3/s flows from the Oaklands Main Drain to the Birrega Main Drain.
- In Spoil bank fail 50%, the water flows upstream inside the Birrega Main Drain (chainage 16477) at a maximum rate of -22.7 m^3/s, likely because of the push of water from the Oaklands Main Drain.
- In the spoil bank failure event, the riverine flooding outside the drain (shown later in Figure 6-9) also provides floodwater through the breach. This is evidenced by comparing the sum of discharges flowing out of the drain (Figure 5-7) and that at the downstream chainage 16825 (Figure 6-8). In the event of Spoil bank fail 50%, the total discharge is 10.7 gigalitres (GL), 6% larger than that of the Spoil bank fail intact scenario (1% AEP). This water balance check indicates water outside the drain can be diverted into the North East Baldivis north of Mundijong Road precinct through the bank failure.

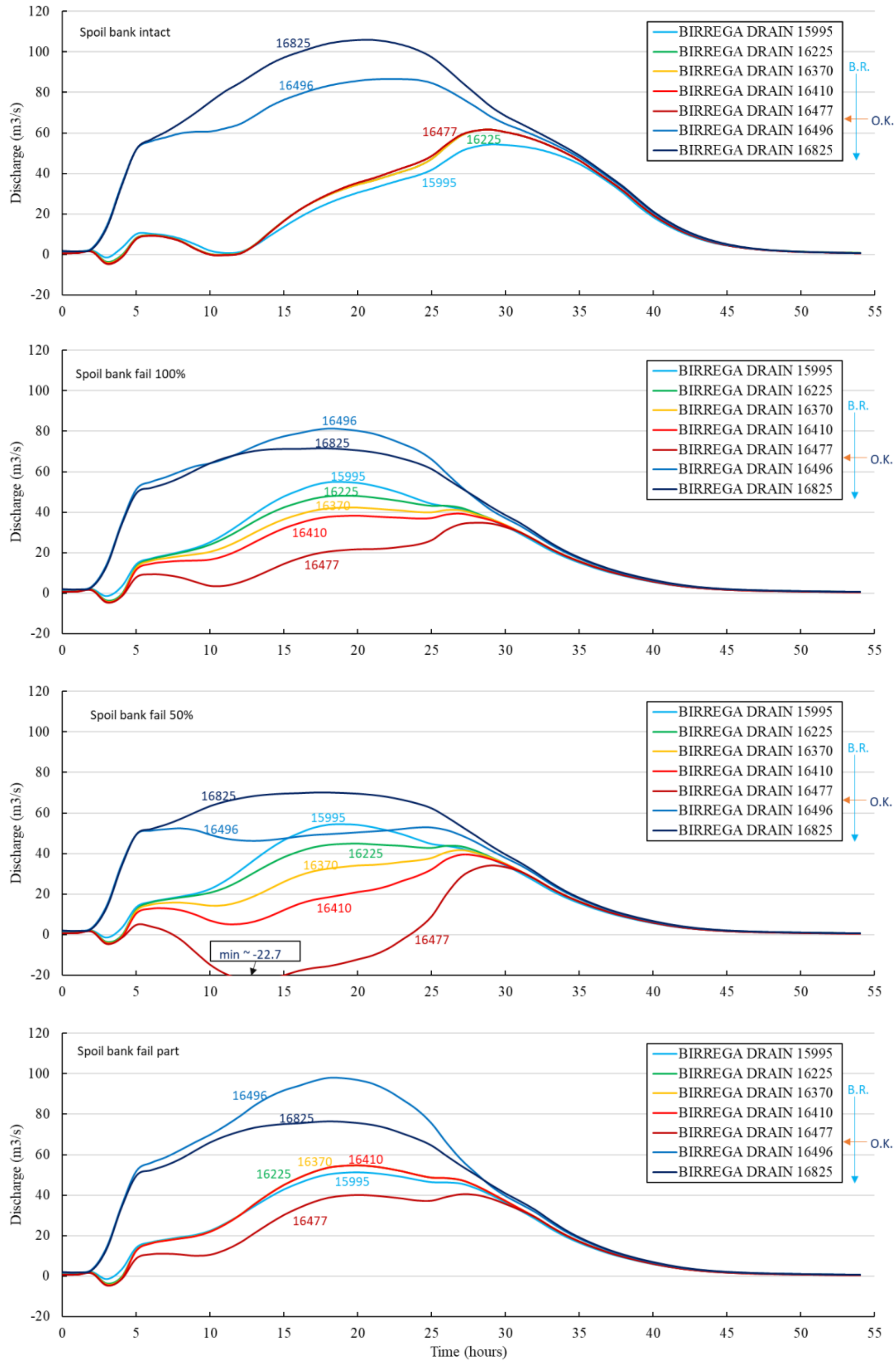


Figure 6-8 Comparison of hydrographs at eight cross-sections of Birrega Main Drain (B.R.) around its confluence with Oaklands Main Drain. Birrega chainage numbers are shown. Oaklands intersects with Birrega between chainages 16477 and 16496.

6.2.4 Spoil bank failure locations

The impact of potential breakouts located upstream and downstream of the Duck Pond Road (DP) were further investigated by three scenarios.

Here, we inspected breaches in addition to the scenario of Spoil bank fail 100%. These three scenarios are detailed in Table 6-1. In particular, the failures of the Oaklands Main Drain (before Scott Road and after Kings Road) are to investigate the impact of upstream spoil bank fail to those discussed in Figure 5-6; while the scenario of Birrega Main Drain near Haines Road is to examine the effect of the downstream spoil bank fail. These locations were chosen based on anecdotal breaches during the 1987 event.

The maximum discharges in Table 6-1 suggest failures of the Oaklands Main Drain from the upstream area, such as at the Scott Road area where breaching was observed in 1987, allowed more water to reach the Birrega Main Drain and increased the discharge from the breach at the Duck Pond location. Therefore, these two scenarios increase the maximum releases to the North East Baldivis north of Mundijong Road precinct.

On the other hand, failures of the Birrega Main Drain from the downstream, such as at Birrega Main Drain near Haines Road, had a minor impact on the location of Duck Pond.

Table 6-1 List of three bank failure scenarios, where the drain banks were lowered in addition to those in the Spoil bank fail 100%

Failure scenarios	**Fail chainages** (In addition to Spoil bank fail 100%)	**Max. discharges at DP** (m^3/s)
Spoil bank fail 100%	(See Figure 5-6)	73.1
Oaklands Main Drain before Scott Road	Oaklands Chainages 10725-10915 (right bank)	78.2
Oaklands Main Drain after Kings Road	Oaklands Chainages 17796-18057 (left bank)	77.3
Birrega Main Drain near Haines Road	Birrega Chainages (left bank) 18320-18580 and 19400-19840	69.8

To conclude, the above sensitivity check on the breaches upstream, downstream, and at the Duck Pond location shows different failure modes did not significantly affect the discharges through a breach at the Duck Pond location near the North East Baldivis north of Mundijong Road precinct. The overflows at this location under 1% AEP 18-hour rainfall range from 63.1 m^3/s (Spoil bank fail part) to 78.2 m^3/s (Oaklands Main Drain before Scott Road), a change that is deemed small considering the various locations and lengths of bank breaches.

For this reason, the scenario of Spoil bank fail 100% is considered representative and adopted as the pre-development condition for the North East Baldivis north of Mundijong Road precinct (Scenario 1 - Table 1-1).

6.2.5 Flooding level and floodplain mapping

Maps of flooding depth from design runs and the floodplain mapping under 1% AEP rainfalls are presented in this section.

Individual flooding maps

The flooding map at the critical median condition (1% AEP 18-hour Tp1 rainfall) for the pre-development case (spoil bank fail 100%) is given in Figure 6-9.

Three other flooding maps from the following individual design runs are given in the Appendix, and maps from different scenarios could be provided on request by the department.

- spoil bank intact (with 1% AEP 18-hour Tp1 rainfall) in Figure C-7.

The sensitivity of flooding to the choice of the temporal pattern selected is shown by:

- 1% AEP 18-hour Tp4 back-loaded rainfall (with spoil bank fail 100%) in Figure C-8
- 1% AEP 18-hour Tp8 front-loaded rainfall (with spoil bank fail 100%) in Figure C-9.

Floodplain extent mapping

Floodplain extent represents the model results of the maximum flood levels for certain design events. The aim is to show the worst flooding condition based on a composite maximum flood level from different modelling runs.

The floodplain extent mapping from 1% AEP rainfalls was prepared in Figure 6-10 from eight simulation runs, which are:

- five modelling runs on the spoil-bank conditions as given in Section 6.2.3, including spoil bank intact and spoil bank fail 100%
- three modelling runs at non-critical 1% AEP rainfalls; that is, the duration of 12-hour Tp4, 24-hour Tp1 and 36-hour Tp2.

Figure 6-10 shows the floodplain extent by a composite maximum flood level based on a combination of four event durations and five spoil bank scenarios. The maximum flood level modelled for each grid cell was calculated from the model outputs and are displayed at a 5 m×5 m grid. No other interpolation or smoothing of the results for the flood extent mapping has been done as part of this project, except that the water level lower than 0.05 m is not shown.

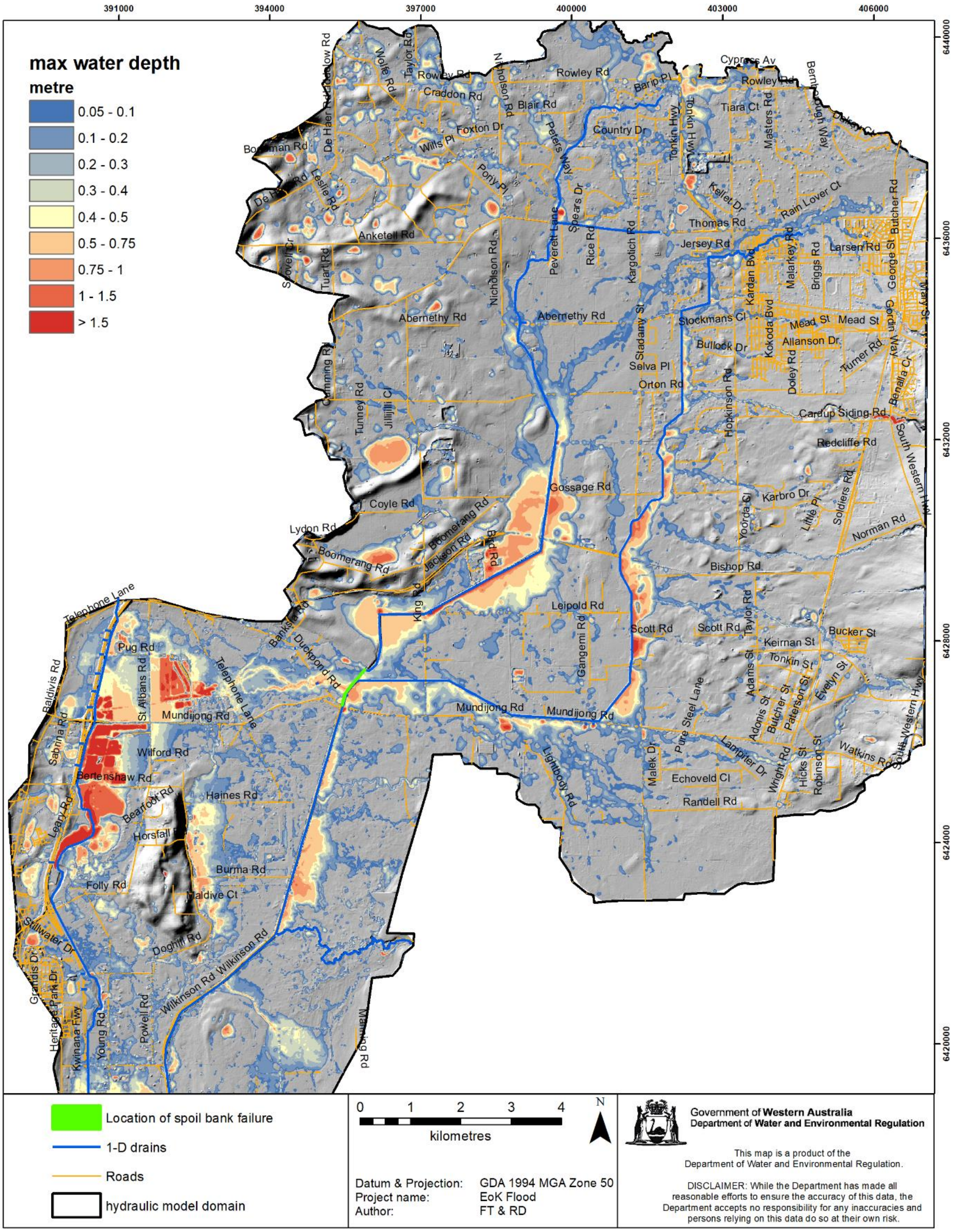


Figure 6-9 Max water depth at the 1% AEP 18-hour Tp1 inputs for the pre-development case (spoil bank fail 100%)

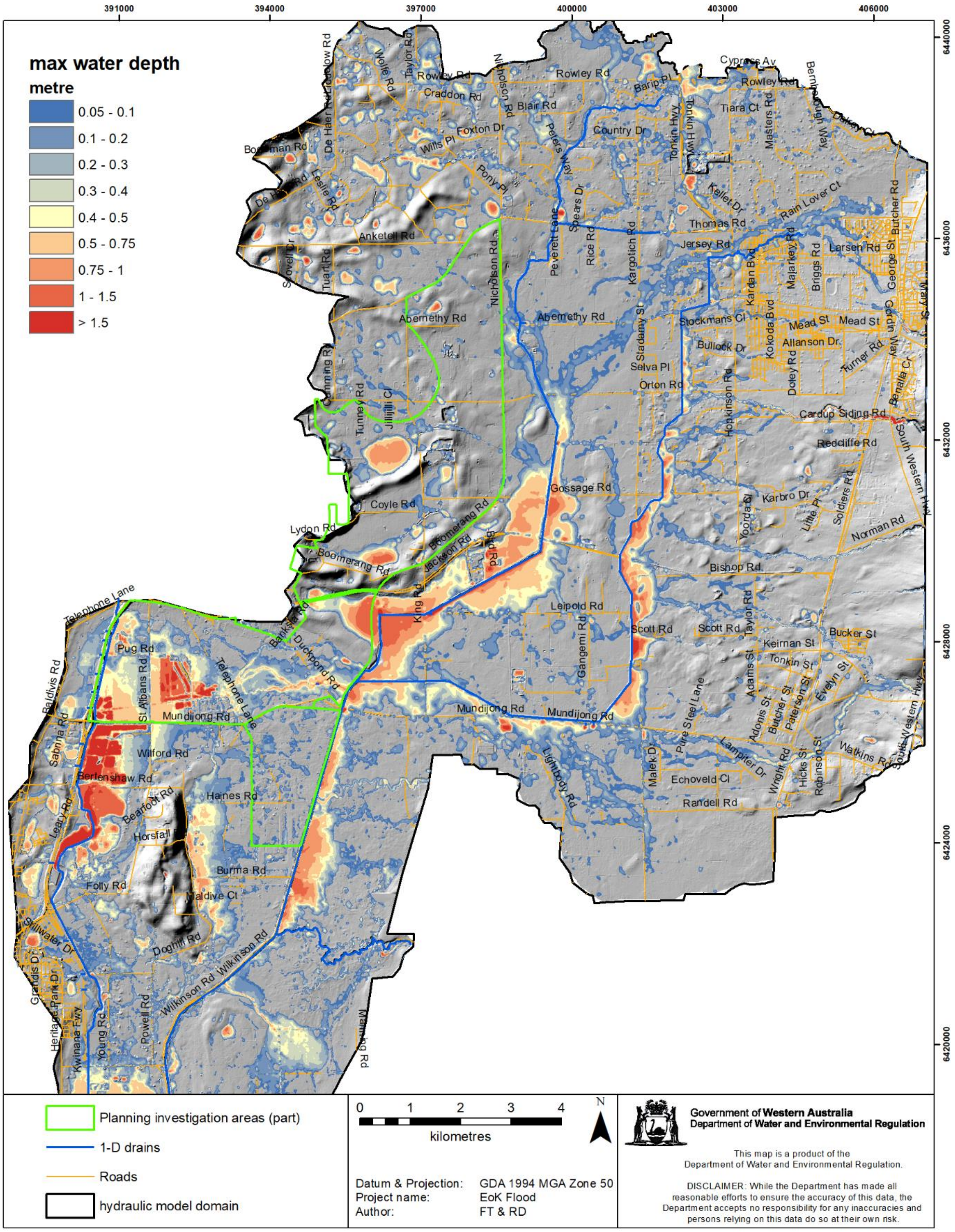


Figure 6-10 Floodplain extent mapping from 1% AEP design runs. This map was produced by a total of eight simulated water levels, as detailed in the text. Part of the PIAs covered by the present study is shown.

Flooding mechanisms

The above flooding maps illustrate distinct flooding characteristics, which is discussed through four flooding mechanisms. Previous studies, including Marillier (2015, pp. 73–75), have demonstrated the area is prone to flooding resulting from direct rainfall, riverine flooding, overtopping and, failure of existing drain banks.

In the following, we discussed the floodplain extent by identifying the four flooding mechanisms throughout the area. It should be noted that a clear cut on what mechanism leads to flooding in a location is not possible. This is because the floodwater concentrated in an area might be induced by more than one flooding mechanism.

I. Pluvial or direct rainfall (surface flood)

Heavy/intense rainfall may result in flooding and ponding of water locally, which is referred to as pluvial food or surface flood. Rainfall leads to many depressions in the study area, where the terrain is typically flat. For that reason, these depressions are disconnected from significant flow paths or drains, and the area usually remains inundated until excess water infiltrates or evaporates.

Among many ponding areas sparsely distributed across the entire region, direct rainfall leads to significant floodwaters which tend to concentrate at the following locations:

- on both sides of Tonkin Hwy north of Thomas Road
- along a diagonal direction in the rectangle area bounded by Thomas Road, Hopkinson Road, Orton Road and the Birrega Main Drain
- on the northern side of Oaklands Main Drain east of Hopkinson Road
- down the hillside at the corner of King Road and Orton Road (part of PIA)
- a large proportion on the western side of the Kwinana Freeway
- on the northern part of the area bounded by Malek Dr, Mundijong Road, Lightbody Road and boundary of the hydraulic model domain
- North of Boomerang Road (part of PIA)
- the area to the east of Dog Hill Road
- the strip of land between the Peel Main Drain and Serpentine Main Drain, close to the southern border of the hydraulic model.

Figure 6-10 shows that a pluvial flood is typically shallow; generally, less than 30 cm deep, but can be up to 1 m in some areas where the land is low.

II. Riverine flooding and overtopping (fluvial flood)

Riverine flooding occurs when prolonged and excessive rainfall causes a river/drain to exceed its capacity. This is because the water concentrated adjacent to its bank discharge timely.

Overtopping occurs when the height of the water inside a drain/river exceeds the level of the bank, and water flows over the top to subsequently flood the land (drain/river loses water). It usually affects smaller downstream drains/rivers.

Both above types of flooding typically occur near riparian and are referred to as fluvial flooding.

Previously studies, including Hall (2015a) and Marillier (2015), have concluded that the Birrega, Serpentine, Oaklands, and Peel main drains might receive too much water from the surrounding plain to timely discharge without exceeding the level of the drain bank.

Some locations, as shown in Figure 6-10, are prone to fluvial flooding, including:

- the area adjacent to the left bank (western/southern) of Oaklands Main Drain south of Abernethy Road to its confluence with Birrega Main Drain
- both sides of Birrega Main Drain south of Abernethy Road to its confluence with Oaklands Main Drain (part of the PIA)
- left bank of Birrega Main Drain south of Mundijong Road to its confluence with Serpentine River.

Figure 6-10 shows that the river flood in the above area is more profound than the surface flood as discussed earlier, especially around the drains where there is sufficient gradient to concentrate flow from nearby land and smaller flow paths.

III. Spoil bank failure

The force of floodwater from the Oaklands Main Drain and instability of the spoil banks might contribute to the potential bank damage near the confluence of the Birrega and Oaklands main drains.

By comparing Figure 6-9 (spoil bank fail 100%) with Figure C-7 (spoil bank intact), we can observe that the damage of bank failure flooding can be widespread. A much greater area would be flooded or experience a significant increase in floodwater depth in spoil bank fail 100% scenario than in spoil bank intact scenario. The fail scenario also results in:

- Directing water from the Birrega Main Drain westwards overtopping Duck Pond Road, Telephone Lane, St Albans Road, and then into Peel Main Drain and southwards to the south (much of the North East Baldivis north of Mundijong Road precinct).
- A significant increase in the depth and extent of flooding to the area south of the Mundijong Road surrounded by Mundijong Road, Kwinana Freeway, Folly Road, and St Albans Road.

It is noted that the flooding in the land in between the Birrega and Peel main drains is a combination of runoff from direct rainfall, overtopping and spoil bank failure. However, the latter would play a significant role in many places, particularly within the PIA area.

6.2.6 Discharges sensitivity to rainfall frequency

Sample flow statistics from four representative cross-sections, were used to extract peak discharge and event volume from the MIKE 21 (and MIKE 11 if appliable) results.

Figure 6-11 shows the locations of these cross-sections included in the discharge calculations. Three of the cross-sections were located along/around the Birrega Main Drain, where overflows were present. At each cross-section location, the design runs with the critical duration and the median temporal pattern were extracted. The peak discharge and associated event volume are reported for the 18-hour event in Table 6-2.

Caution should be given when interpreting results for shallow overland flow paths, where the scale of modelling may be too coarse to provide realistic results. In the 1% AEP events, drain capacities are exceeded, and flow is no longer constrained within the cross-section extent.

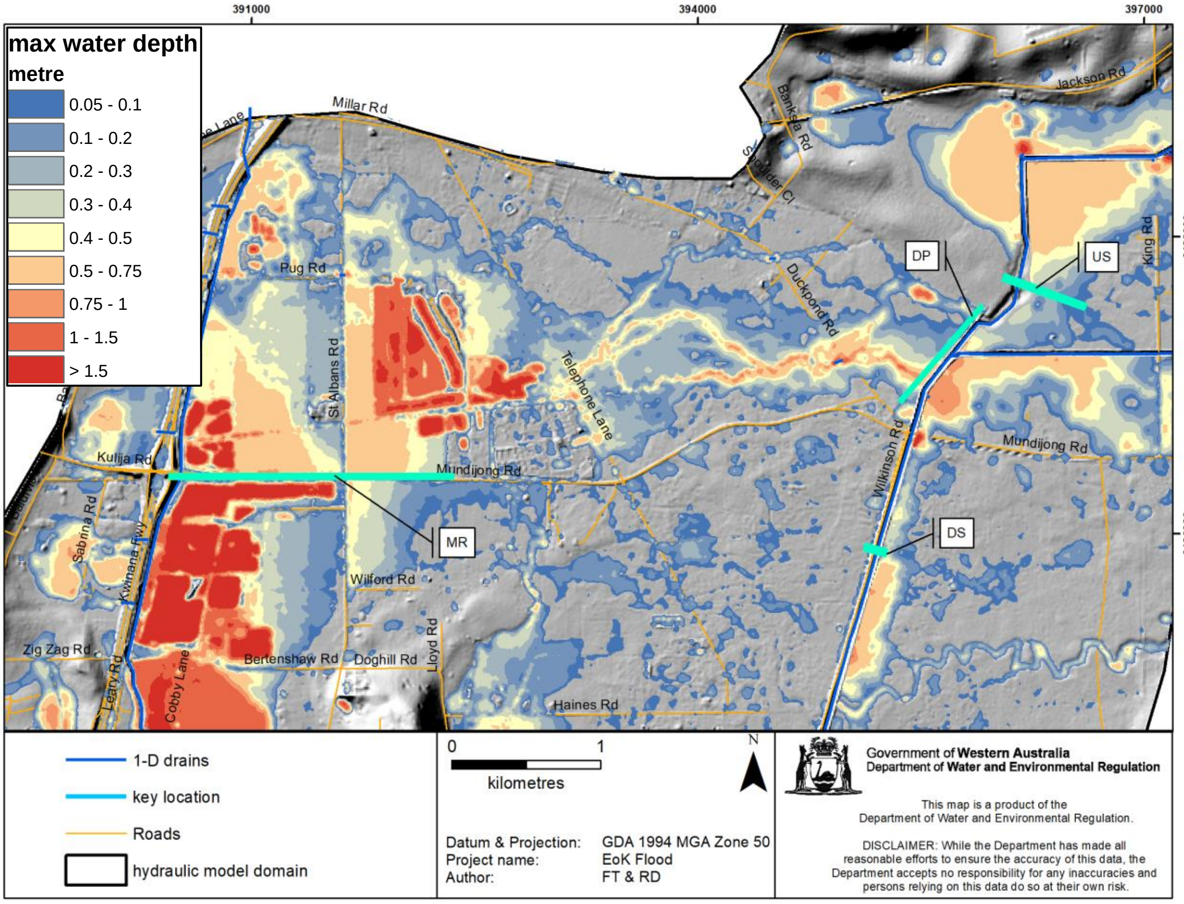


Figure 6-11 Location of the cross-sections for flow statistics (max water depth is from Figure 6-9)

The cross-section at Duck Pond (DP cross-section) illustrates the flows coming from the Birrega Main Drain in the spoil bank failure event (spoil bank fail 100%). Under this scenario, peak flows through the DP section would reach 96.4 m^3/s at 0.5% AEP, 73.1 m^3/s at 1% AEP and 27.9 m^3/s at 5% AEP, respectively. The total event volume of water discharged would be 5.51 GL under 0.5% AEP, 4.02 GL under 1% AEP and 1.25 GL under 5% AEP, respectively.

A large proportion of this water would eventually overflow Mundijong Road (MR cross-section). The peak discharges at the Mundijong Road section, including the flow inside the Peel Main Drain, are 92.1 m^3/s under 0.5% AEP, 65.9 m^3/s under 1% AEP and 15.2 m^3/s under 5% AEP, respectively. The total event volume of water discharged would be 5.45 GL under 0.5% AEP, 3.65 GL under 1% AEP and 0.90 GL under 5% AEP, respectively.

Table 6-2 Flow statistics at four cross-sections from the 18-hour Tp1 rainfalls (the critical duration and median temporal pattern). The discharge and event volume are from MIKE21 and MIKE11.

ID	Peak discharge (m^3/s)			Total volume (GL)		
	5% AEP	**1% AEP**	**0.5% AEP**	**5% AEP**	**1% AEP**	**0.5% AEP**
DP	27.9	73.1	96.4	1.25	4.02	5.51
US	28	55.5	78.3	2.00	4.29	5.75
DS	59.1	72.7	79.8	4.60	6.75	7.58
MR	15.2	65.9	92.1	0.90	3.65	5.45

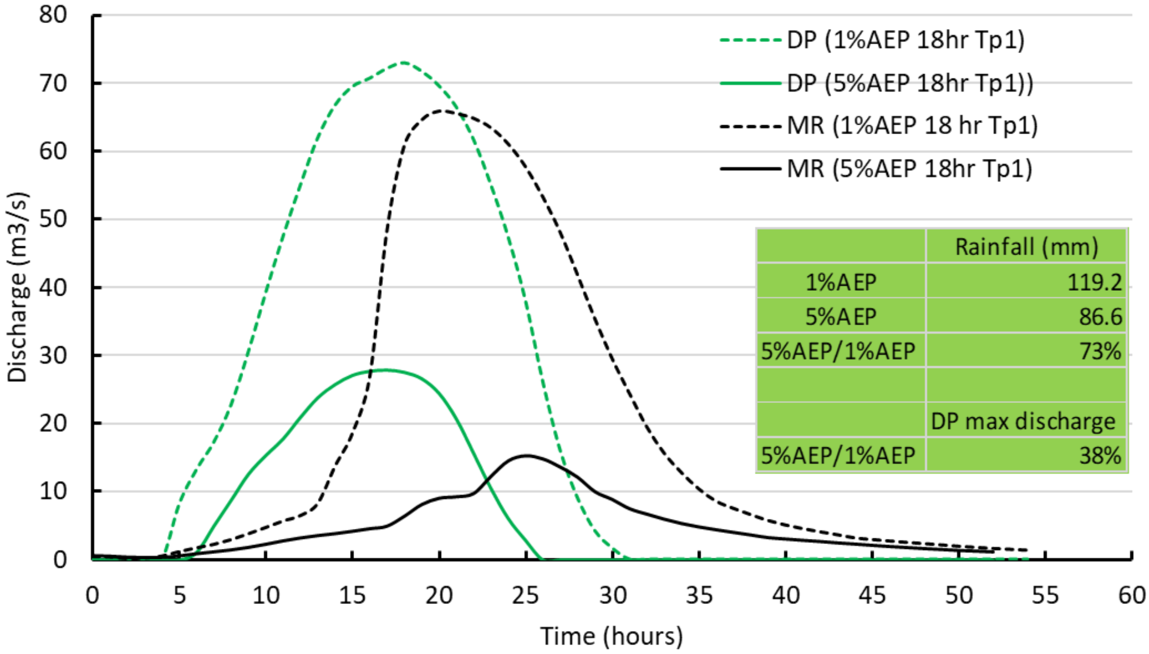


Figure 6-12 Comparison between the discharges from 1% AEP and 5% AEP rainfalls at two locations

The significant difference in peak discharge and volume between the DP and MR sections illustrates the size of floodplain storage. This is evident in the increased flood extent and deeper (pink and red shading) shown in Figure 6-11 north of Mundijong Road and west of Telephone Lane.

The cross-sections at US and DS indicate the flows of the Birrega Main Drain at locations both upstream and downstream of the site of spoil bank failure. Under 1% AEP, peak flows would be 55.5 m^3/s and 72.7 m^3/s through these locations (including riverine flooding), respectively.

The spoil bank failure scenario substantially decreases the discharge through DS cross-sections. In the event of spoil bank intact, the peak flows would reach 110 m^3/s at DS.

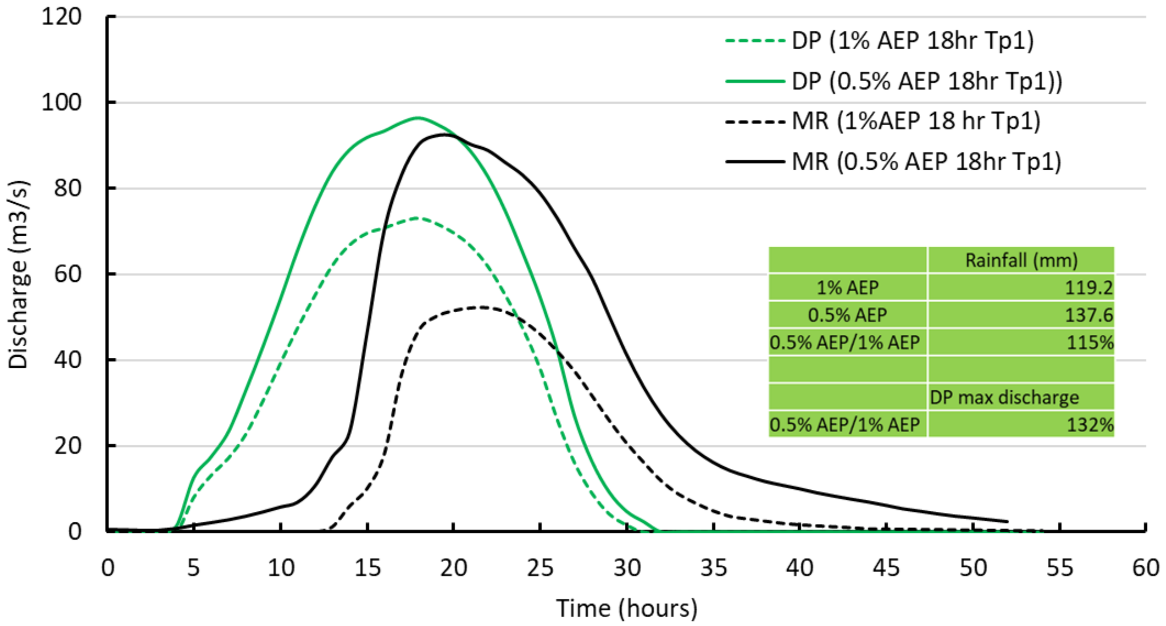


Figure 6-13 Comparison between the discharges from 1% AEP and 0.5% AEP rainfalls at two locations

Figure 6-12 and Figure 6-13 illustrates the time-history of discharges of DP and MR at the 5% AEP, 1% AEP and 0.5% AEP. Two features to highlight are:

- From 1% AEP to 5% AEP, the design rainfall depth decreases 27%, while the discharge at DP reduces 62%.
- From 1% AEP to 0.5% AEP, the design rainfall depth increases 15%, while the discharge at DP rises 32%.

The disproportionate change is because of the storage of the catchment upstream DP. Much of the 5% AEP rainfall was absorbed by the floodplain storage, while the extra rainwater of 0.5% AEP compared to 1% AEP likely ends up as streamflow.

6.2.7 Sensitivity analysis on roughness, losses, and inundation

In addition, a sensitivity check at 1% AEP was carried out on the chosen overland roughness, infiltration and inundation areas. For roughness and infiltration, the sensitivity analysis involved applying values 10% lower or higher than the values listed in §5.2.3 and §5.2.4 over the whole model area. The results of the sensitivity tests were analysed for discharges at key locations and impacts on flood levels where the flood depth exceeded 50 mm.

Discharges from the sensitivity analysis were compared at the Duck Pond cross-section. This cross-section has a total contributing catchment area of over 300 km^2.

The sampled hydrographs are given in Figure 6-14. When the roughness is increased by 10%, the maximum discharge at this location decreases by 4.7%; while when the roughness is decreased by 10%, the maximum discharge at this location increased by 3.0%.

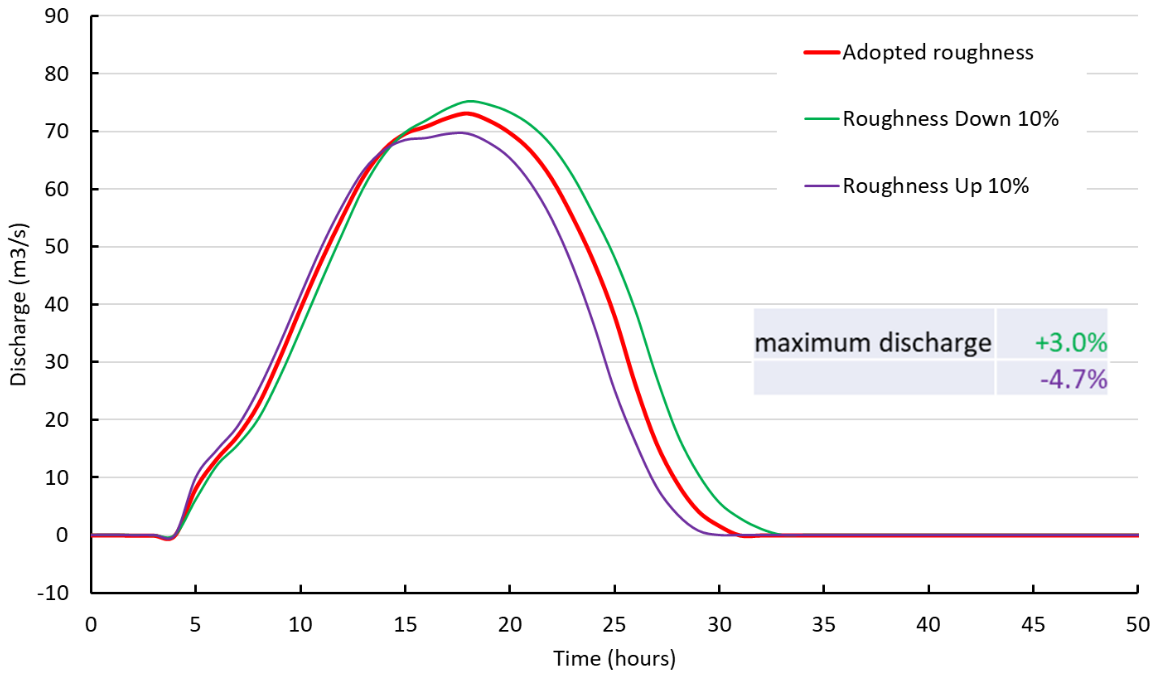


Figure 6-14 Sensitivity check on hydraulic roughness at 1% AEP 18-hour Tp1 by comparing the discharge at DP (refer to Figure 6-11 for the location)

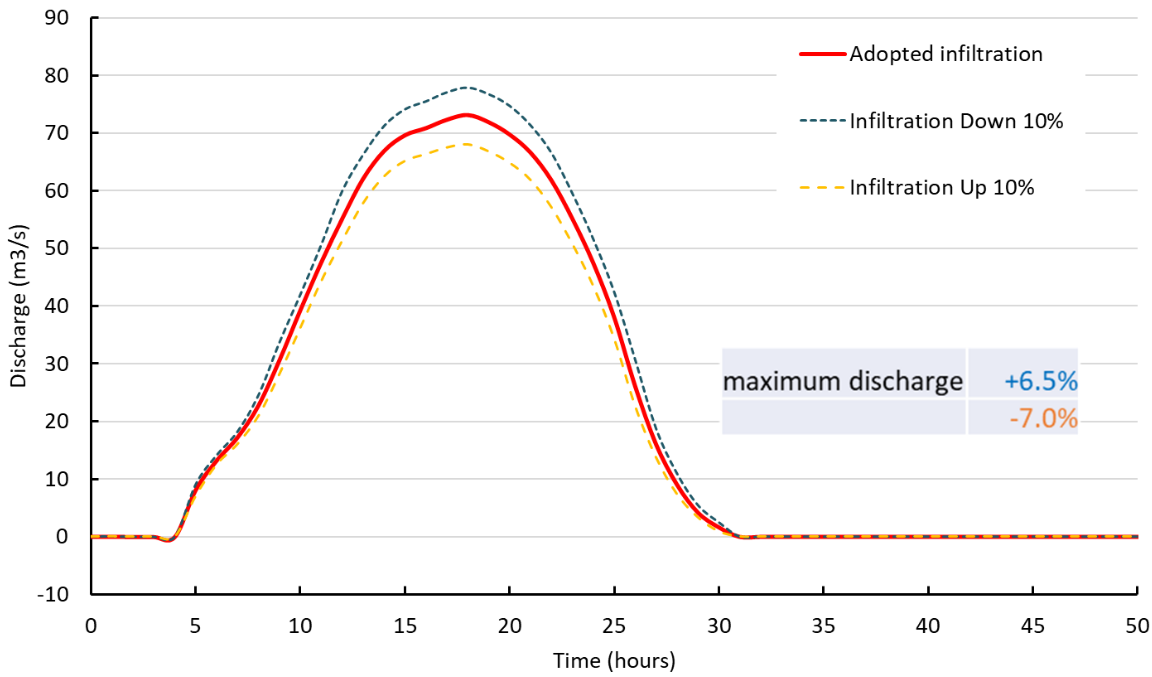


Figure 6-15 Sensitivity check on infiltration at 1% AEP 18-hour Tp1 by comparing the discharge at DP (refer to Figure 6-11 for the location)

Table 6-3 summarises the sensitivity of the regional flood model in comparison to the inflows from the Birrega Main Drain and the average change to flood levels for the North East Baldivis north of Mundijong Road precinct. The results across the broader regional model displayed less sensitivity than the results below.

The analysis showed that the flood modelling results were not sensitive to the tested parameters at the regional level or within the North East Baldivis precinct.

Table 6-3 Predevelopment model sensitivity to roughness, losses and inundation

Sensitivity case	**Percentage change to inflows from the Birrega**	**Average change to flood levels**
Roughness sensitivity		
Roughness up 10%	-5%	-12 mm
Roughness down 10%	+3%	+12 mm
Losses Sensitivity		
Infiltration up 10%	-7%	-15 mm
Infiltration down 10%	+6%	+16 mm
Inundation sensitivity		
No groundwater inundation	-5%	-2 mm
Annual average maximum groundwater level inundation	+2%	+6 mm

6.3 Local model results

6.3.1 Validation and comparison

The existing flooding behaviour of the North East Baldivis north of Mundijong Road precinct was also modelled using the local model. Here, the same rainfalls as discussed in Sections 6.2.1 were used in the local model; that is, the rainfall based on an areal reduction factor for the entire catchment to Mundijong Road (including the upper Peel Main Drain and Birrega and Oaklands main drains catchments, as shown in Figure 4-1).

As determined from the regional model, simulations were carried out for the 1% AEP critical median event (i.e. 18-hour Tp1). An inflow resulting from spoil bank failure of the Birrega Main Drain (as given in Figure 6-7 at spoil bank fail 100%) was applied as a source input near the location of spoil bank fail (Figure 6-11). The inflow at the Peel Main Drain was kept the same as the regional model.

The output from the local flood model simulation at 1% AEP is given in Figure 6-16. This flood map was obtained from the local flood model using the same rainfall and inflows as the regional flood model at 1% AEP 18-hour Tp1 rainfall. The discharge from the regional model because of spoil bank fail 100% was applied in the modelling. The equivalent model output from the regional model is given in Figure 6-9. The distribution of the floodwater in Figure 6-16 is consistent with the outcome from the regional model (in Figure 6-9), as both cases share the same rainfall and streamflow inputs.

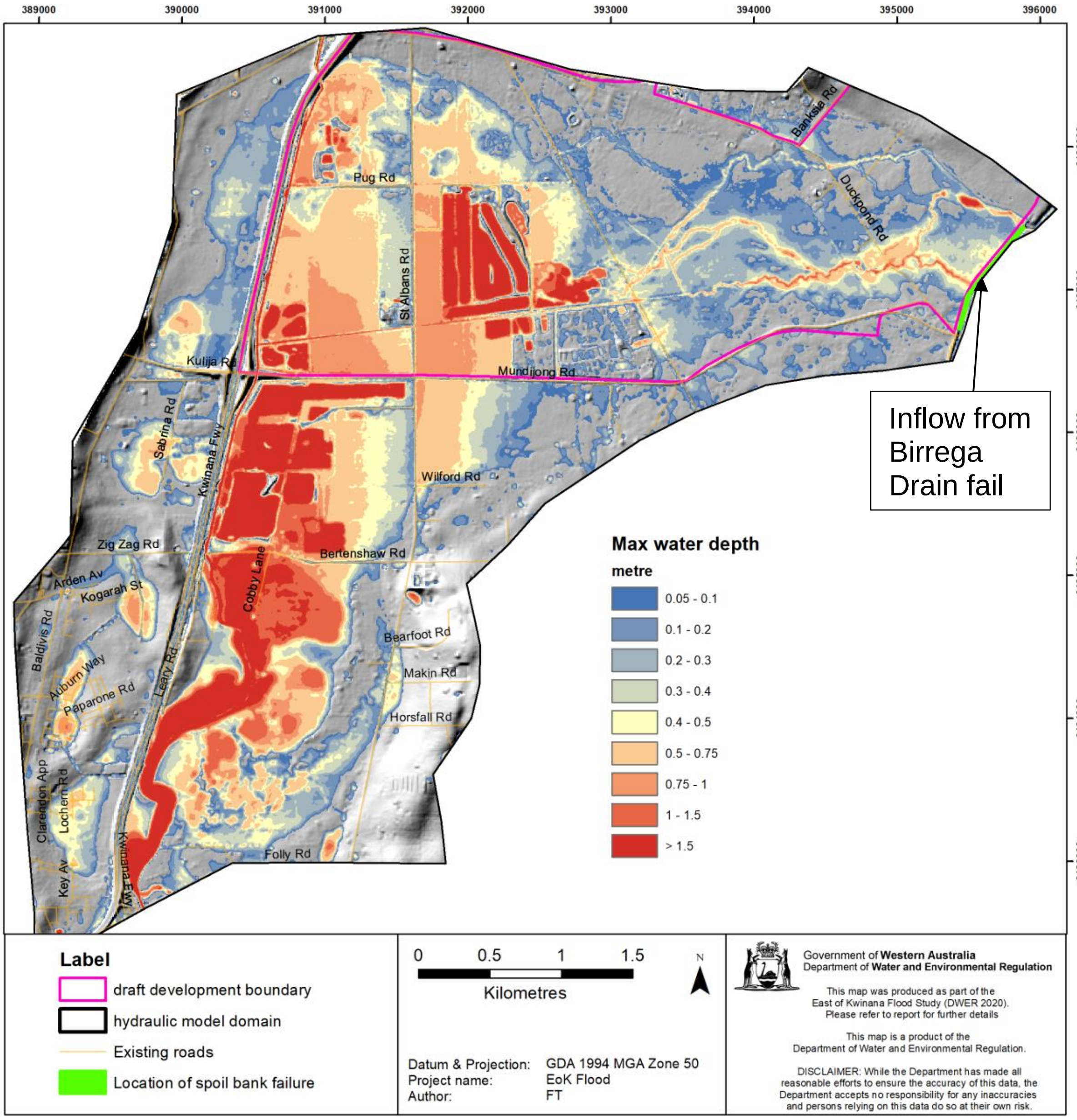


Figure 6-16 Maximum flood water depth from the Local model with spoil bank fail 100%

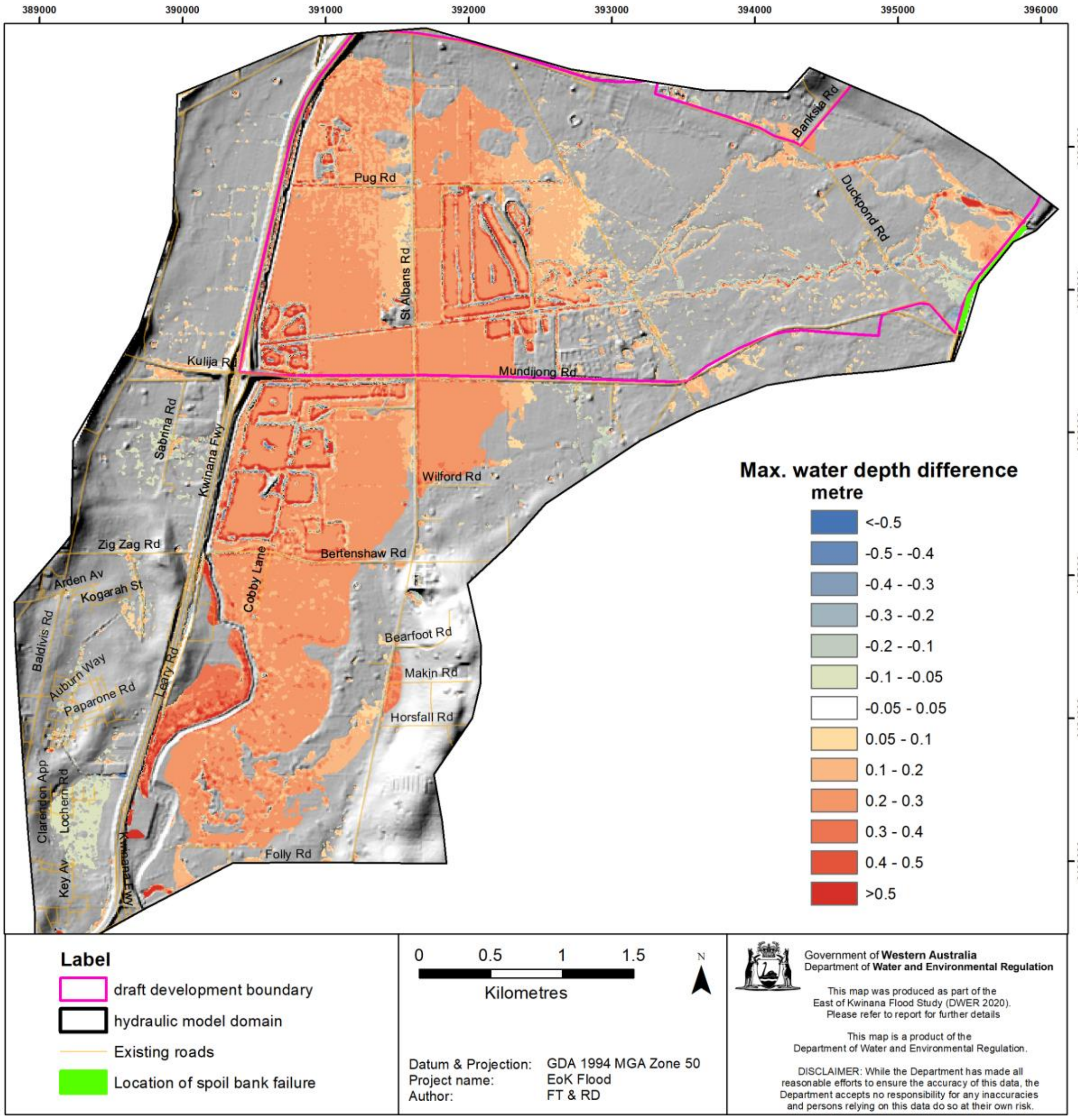


Figure 6-17 The difference in floodwater depth between the local (Figure 6-16) and the regional models (Figure 6-9). The depth between ±0.05 m was not shown.

Figure 6-17 compares the floodwater depth between the local and the regional models. It further demonstrates the consistency between the model results. However, because of the fine mesh used in the local model, which resolves road crest and the floodplain better, the modelled floodwater depth from the local model, in some areas, is 20 - 30 mm more significant than that of the regional model. This is profound upon the floodwater reaching the St Albans Road. In addition, the floodwater inside many natural waterways is also deeper in the local flood model, including the waterways east of Telephone Lane. This increase is because the fine mesh increases the resolution of the channel of waterways.

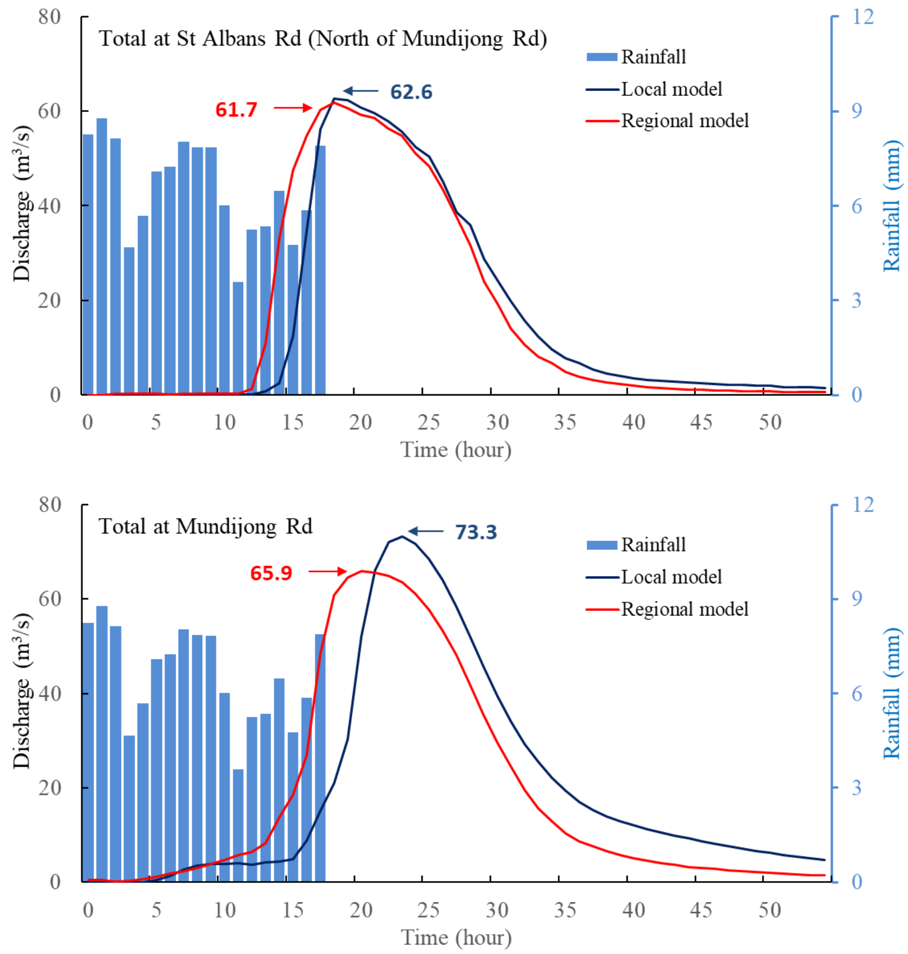


Figure 6-18 Comparison of the total discharges over St Albans Road (top) and Mundijong Road (bottom)

A comparison between the cross-sectional discharges from the local and regional models are given in Figure 6-18 sampled along the St Albans Road and Mundijong Road.

The discharges from both models are relatively consistent at the St Albans Road (North of Mundijong Road), with slightly larger values from the local model. However, the local flood model also predicted a two-hour delay in the hydrograph. The reason is that the fine mesh used in the local model better resolves the floodplain around Duck Pond and the road crest of the St Albans Road (as shown in Figure 5-19), so that the floodwater must be fulfilled the storage before overtopping the St Albans Road.

The local flood model also predicted slightly larger peak discharges than the regional flood model at the Mundijong Road (sampled at MR as shown in Figure 6-11). The local model's time delay in the hydrograph has increased to about three hours for the same reason discussed above. The hydrograph from the local model also appears to be peaky, indicating that once the floodwater water reaches the level of the Mundijong Road, it overflows quickly. For this reason, flood depths from the local model are higher than those from the regional model on both sides of the Mundijong Road.

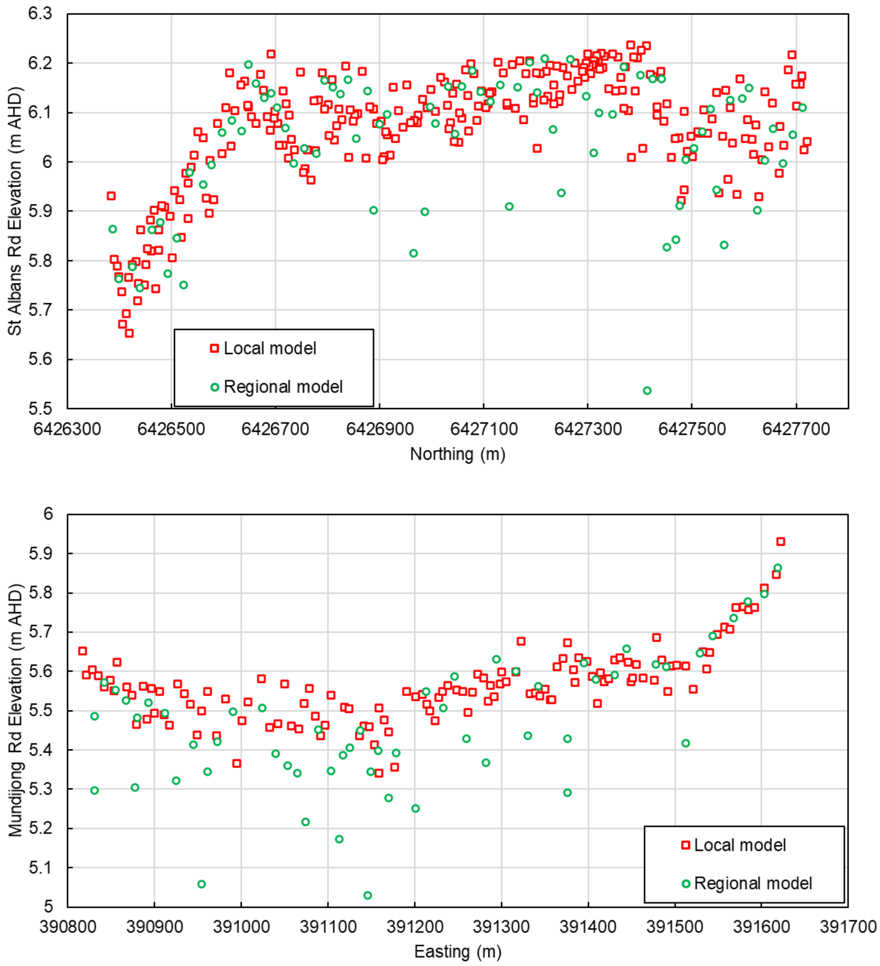


Figure 6-19 Comparison of node bathymetry sampled along the St Albans Road (top) and Mundijong Road (bottom) between the local and the regional model

Figure 6-19 compares the node bathymetry between the local and regional models sampled along the St Albans Road and Mundijong Road. These heights were sampled from triangle (flexible) meshes used in the models. Note that much more nodes were used in the local area flood model, while the bathymetry is more uniformly distributed without apparent low-outlier values, as appeared in the regional flood model.

6.3.2 Impact of the downstream boundary

The Peel Main Drain crosses the downstream boundary of the local flood model (Figure 5-17), and a sensitivity check on the boundary conditions was carried out. The Peel Main Drain outlet is located at Folly Road, about 4 km south of Mundijong Road. The aim in choosing this location far downstream from the North East Baldivis north of Mundijong Road precinct is to leave enough floodplain in the model to minimise the boundary impact on the modelling outcomes.

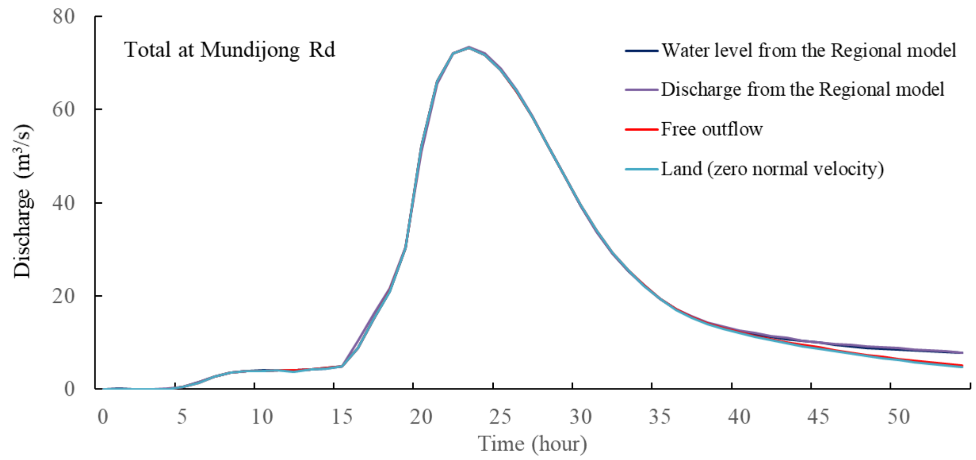


Figure 6-20 Comparison of the total discharges over the Mundijong Road from the local flood model by different boundary conditions

Four different boundary conditions of the Peel Main Drain outlet were considered:

1. Time-varying water level history based on the modelling of the regional model at the same location with the same rainfall condition (at 1% AEP 18-hour Tp1).
2. Time-varying discharges based on the modelling of the regional model at the same location.
3. Free outflow.
4. Land with a zero normal velocity.

Figure 6-20 compares the overflow at Mundijong Road from modellings utilising these four outlet conditions. The discharges are almost identical, which only differs slightly after 45 hours into the simulation. We concluded that the chosen downstream boundary has no impact on the flooding on the North East Baldivis north of Mundijong Road precinct. Despite that, discharge from the regional model is deemed a physical boundary condition and chosen in the following simulation.

7 North East Baldivis north of Mundijong Road – Post-development (Scenario 1)

A few development options were proposed on North East Baldivis north of Mundijong Road (see Figure 7-1). Through the investigation, this work aims to answer the following two questions from a flooding perspective:

Can the precinct be developed without an unacceptable impacts to neighbouring land?

How much land can be provided at an appropriate flood standard for the land-use that is being sought?

The flooding modelling focus is on the amount of land that can be developed with an appropriate flood standard, based on the flooding mechanisms in the pre-development base case. The incremental risks and benefits of additional engineering works and the potential to increase the developable area through engineering optimisation at the district-level land planning stage is also investigated.

The flood risk management land capability assessment *(*DWER, 2021) has been prepared by the department for a more detailed discussion on the post-development options and the department's recommendations for each option (including flooding consideration). For this reason, this report puts more emphasis on modelling techniques and flooding behaviour.

7.1 Development option overview

The flood risk management land capability assessment *(*DWER, 2021) has detailed the development options, which is briefly summarised in Table 7-1 and Figure 7-1. The hypothetical options involve raising some land to a level above certain flood levels. It is not the purpose of the present work to justify land raise or discuss its impact. Instead, it is to carry out flooding analysis with the engineered bathymetry for the flooding constraints. Table 7-1 shows an overview of the three groups of options for development.

Table 7-1 Overview of the development options and the numerical model

Options	Main characteristic	Model
Option A	The option of a full height engineered levee on the east bank near the confluence of the Birrega Main Drain and the Oaklands Main Drain.	Regional model and Local model
Option A+	As above but allowing a maximum of 17 m3/s discharge into Option A terrain under 1% AEP event.	Regional model
Option B	No levee option (spoil bank failure) with land elevated above 1% AEP flood level.	Local model (with inflow from Birrega Main Drain)
Option B+	The same as above, but adjusting the land set aside for floodplain storage	Local model (with inflow from the Birrega Main Drain)

Options	Main characteristic	Model
Option C	No levee option with elevated land above 1% AEP flood level for residential purpose and above 5% AEP flood level for industrial	Local model (with inflow from the Birrega Main Drain)

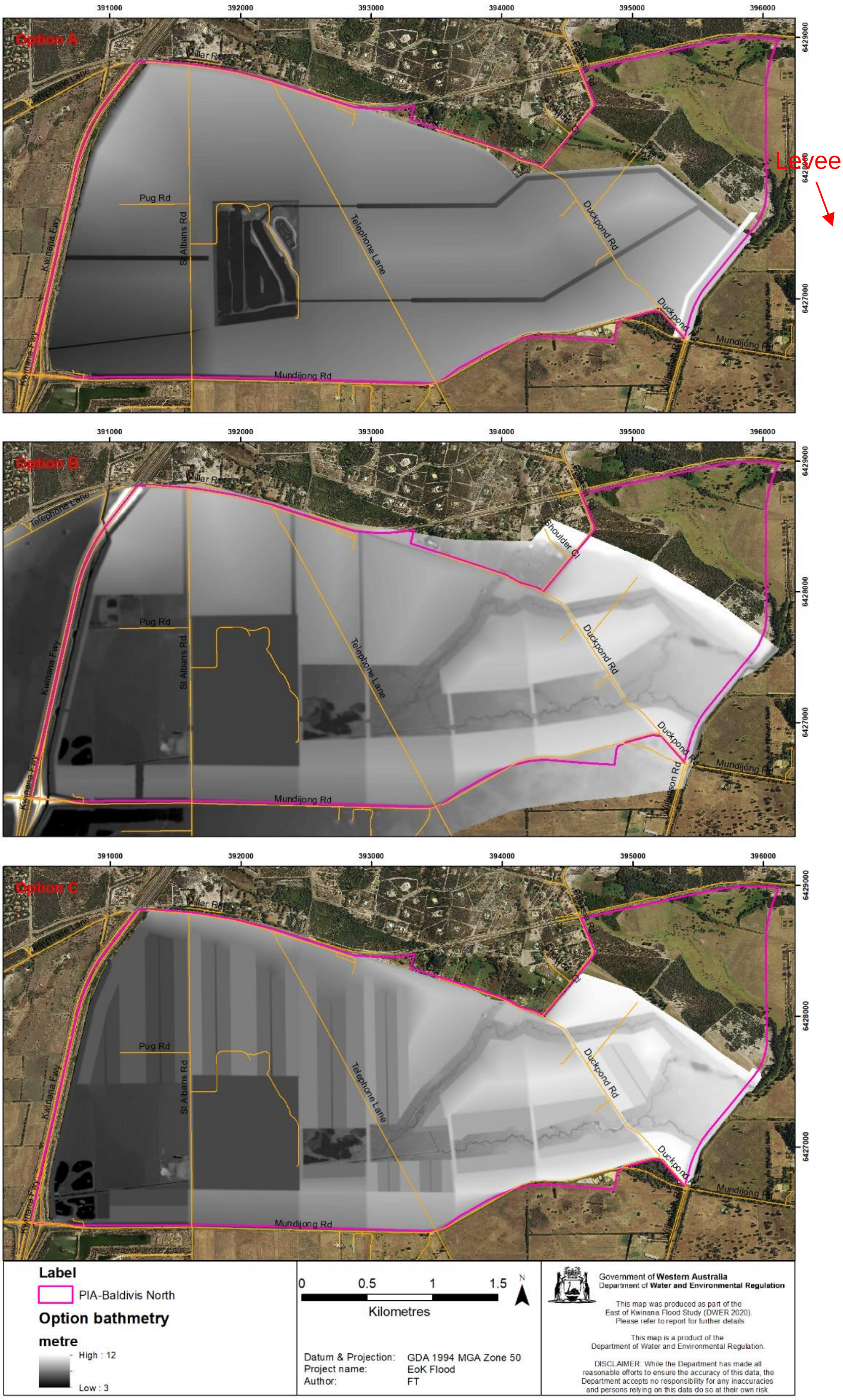


Figure 7-1 Hypothetical bathymetry of selected options for the development scenarios

In Options A and A+, an engineered levee is planned on the right bank near the confluence of the Birrega Main Drain and the Oaklands Main Drain to maximise land-use inside the North East Baldivis north of Mundijong Road precinct. The levee in Option A would block all overflows under 1% AEP, while the levee in Option A+ allows a maximum of 17 m^3/s to overflow from the Birrega Main Drain.

Options B, B+ and C are designed with sufficient waterways for potential spoil bank breaches of the Birrega Main Drain. Option C also includes comparing different land-uses, including residential above the 1% AEP flood versus industrial uses with a varying flood stand (for example, 1% AEP for buildings and a lower standard for others). The local model was used for these options with an inflow along the Birrega Main Drain from the regional model.

7.2 Scenario 1 Option A

As detailed in the flood risk management land capability assessment (DWER, 2021), the main feature of Option A is to install an engineered levee on the right bank of the Birrega Main Drain around its confluence with the Oaklands Main Drain (see Figure 7-1).

This engineered levee's full height is 12 m AHD, and the length is about 1200 m. In the event of 1% AEP rainfall, the engineered levee would block all overflow of the Birrega/Oaklands main drains from entering the North East Baldivis north of Mundijong Road precinct. At the condition of spoil bank intact (Figure 6-6), the maximum flood level at this location is about 11 m AHD at 1% AEP.

The flooding of Option A used both the local and regional hydraulic models. The regional hydraulic model determines Option A’s impacts on land locating both upstream and downstream.

7.2.1 Flood mapping at 1% AEP

The flood mapping for Option A was prepared in Figure 7-2 under the 1% AEP 18-hour rainfall.

The flood mapping is given at a 5 m×5 m grid. No interpolation or smoothing of the result has been done, except that water levels lower than 0.05 m are excluded from the map to eliminate very shallow areas of flooding. The main flooding features from Figure 7-2 are:

- Small areas are prone to water ponding, including places surrounding the Water Ski Park. The ponding depth is generally smaller than 0.4 metres. This type of flooding occurs locally and is generated by overland sheet flow from direct rainfall. Since the flooding areas do not connect to any drains, they will remain inundated until excess water infiltrates or evaporates. This flooding can be potentially amended by increasing the land gradient to concentrate flow, potentially to a drain.
- The design drain appears to work sufficiently in convey the rainfall water. However, on the west side of St Albans Road, the flooding adjacent to the South Drain (as named in Figure 6-4) is because of overflow from the South Drain and can be classified as riverine flooding. It occurs because the drain receives excessive flow and the water level inside the drain exceeds the bank's height. The area of riverine

flooding appears to be larger than that of the flooding area from direct rainfall. Increasing the drain capacity would probably amend the riverine flooding issue.

- The flooding south of Mundijong Road is much less severe than the pre-development case (Figure 6-10). The engineered levee effectively blocks all discharge from the Birrega Main Drain, leading to limited water, if there is, overflows Mundijong Road.

To summarise, the flooding extent from 1% AEP demonstrates that the flood in Option A is largely constrained within the designed drains and water storage. The South Drain, close to the Peel main drain is recommended to be amended by increasing the drain capacity. Option A generally leads to a positive impact on flooding to the downstream catchment (south of the Mundijong Road). However, Option A's effect on the upstream catchment should be investigated (section 7.2.3).

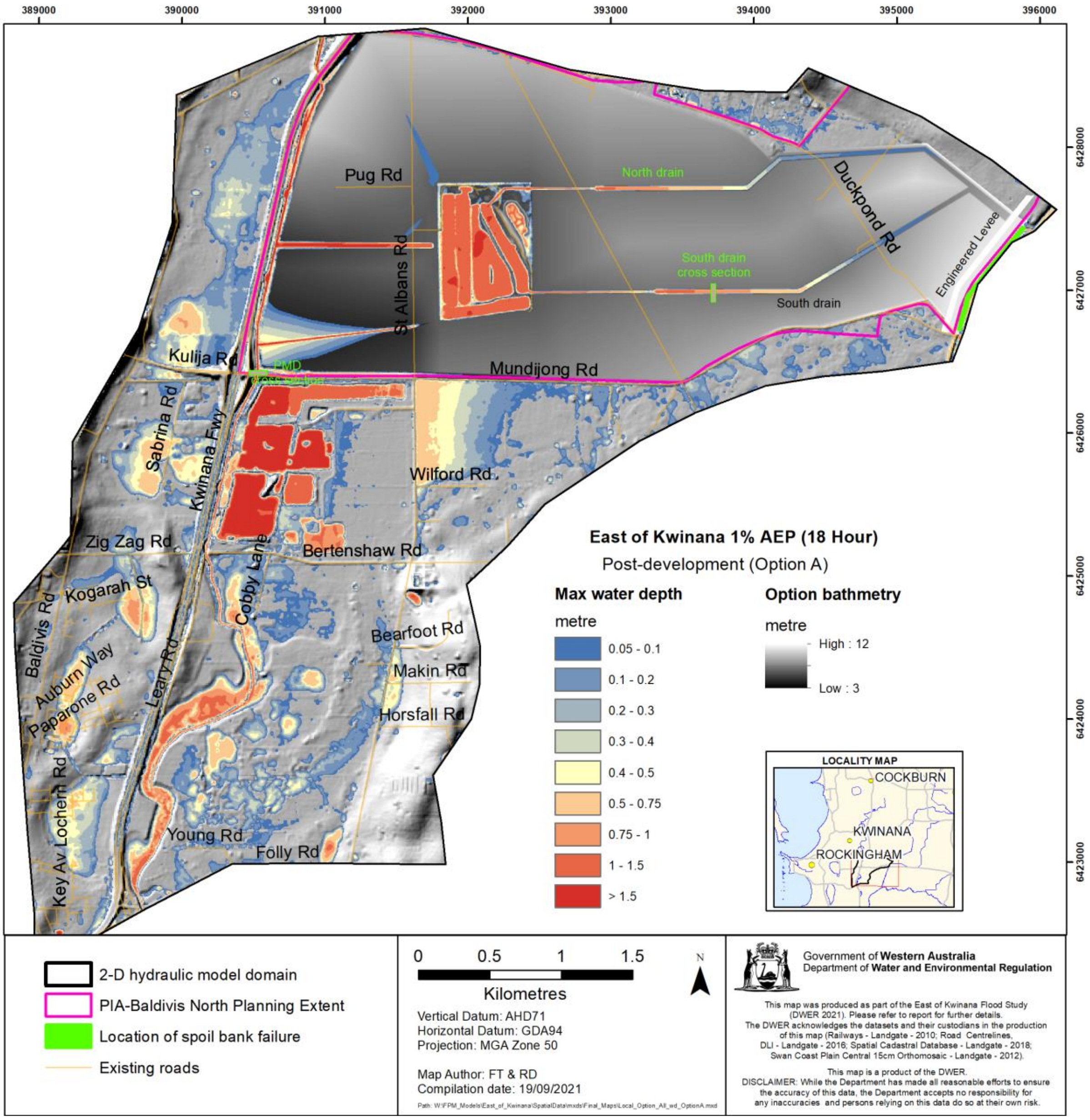
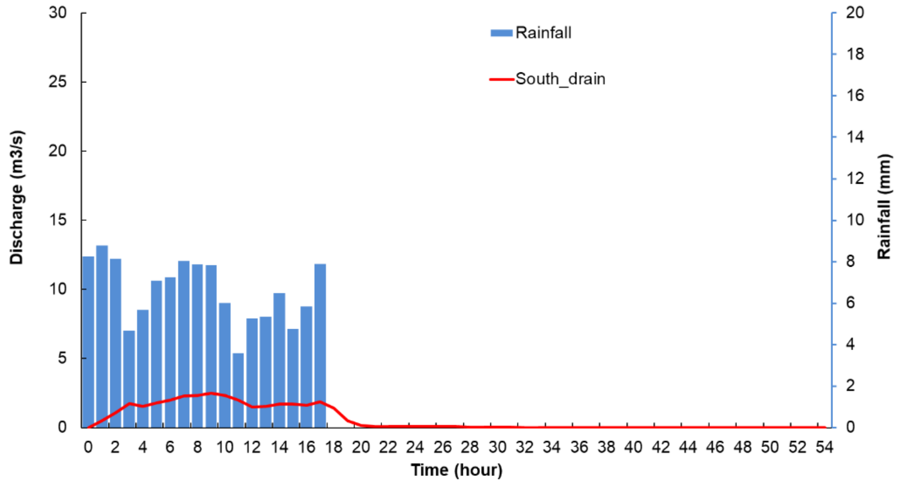


Figure 7-2 Flood mapping for Option A from the 1% AEP 18-hour Tp1 design rainfall

7.2.2 Discharge hydrograph of Option A

Figure 7-3 samples the peak drain discharges at two cross-sections from MIKE21 modelling. These sites are located at the South Drain and the Peel Main Drain (PMD), as illustrated in Figure 7-2.



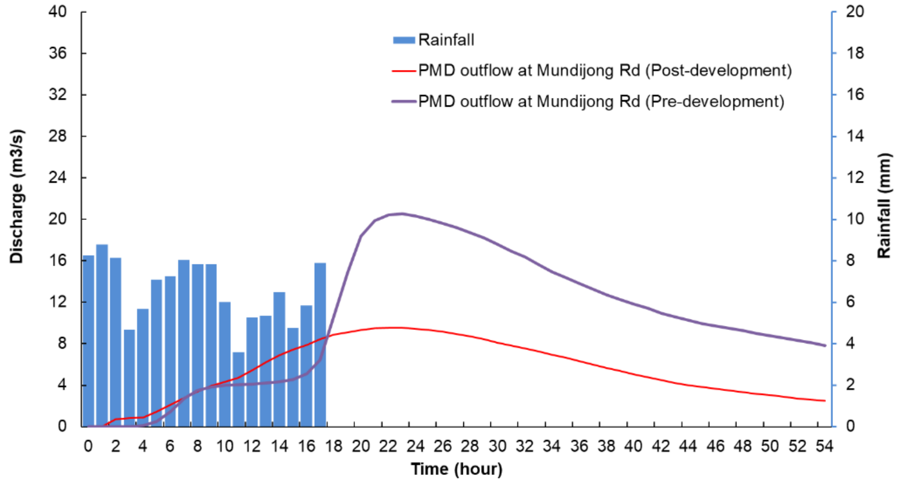


Figure 7-3 Hydrographs measured at the South Drain and the Peel Main Drain at Mundijong Road (see the locations in Figure 7-2)

At the South Drain, the maximum peak discharge is less than 3 m^3/s, generated locally from the elevated land from the 18-hour rainfall.

For the Peel Main Drain at Mundijong Road, the peak post-development discharge is only about half of the pre-development value (10 m^3/s compared to 20 m^3/s). It demonstrates the impact of locally generated runoff in Option A is small. It is worth noting that the Mundijong Road is free from flooding in Option A, inundated in many locations in the pre-development condition.

7.2.3 Impact of Option A

We employ the regional model to study Option A's impact on flooding outside the North East Baldivis north of Mundijong Road. We investigated the effect of an engineered levee on the land locating to its upstream.

A regional flood model for Option A was developed. We overlapped Option A's bathymetry to the pre-development regional model (as discussed in Chapter 6). The resultant regional model maintains all model features, except that the MIKE 11 structures inside North East Baldivis north of Mundijong Road were removed because of bathymetry changes.

Figure 7-4 compares the differences in flood levels between the regional Option A model and the regional pre-development model at 1% AEP 18-hour Tp1. Both hydraulic modellings were carried out with the same inputs.

Upstream the engineered levee, Figure 7-4 demonstrates that a significant area experiences increased flood levels. Here, we highlight two areas.

- Both sides of the Birrega Main Drain between Jackson Road and Oaklands Main Drain experience a level of increase in water depth between 0.3 - 0.5 metres. The water depth difference reaches up to 0.55 metres.
- A strip region adjacent to the left bank of the Birrega Main Drain south of Oaklands Main Drain also experiences a floodwater increase. Here, the levels are between 0.05 to 0.3 metres.

The increases in water depth in the above areas are mainly because of enhanced riverine flooding. The engineered levee constrains floodwater from flowing into the North East Baldivis north of Mundijong Road precinct, leading to the existing drains exceeding their capacity. Option A’s impact is detrimental, considering a large area experiences a significant increase in flood levels.

Figure 7-4 demonstrates a significant area south of Mundijong Road experiences a decrease in flood levels. This area covers most of the Alcoa Wellard Wetlands between St Albans Road and Kwinana Freeway and the Perth Wake/Aqua Park north of Wilford Road. For the same reason, the decrease in water levels is because of the engineered levee, as it blocks floodwater from the Birrega Main Drain from entering the North East Baldivis north of Mundijong Road precinct area.

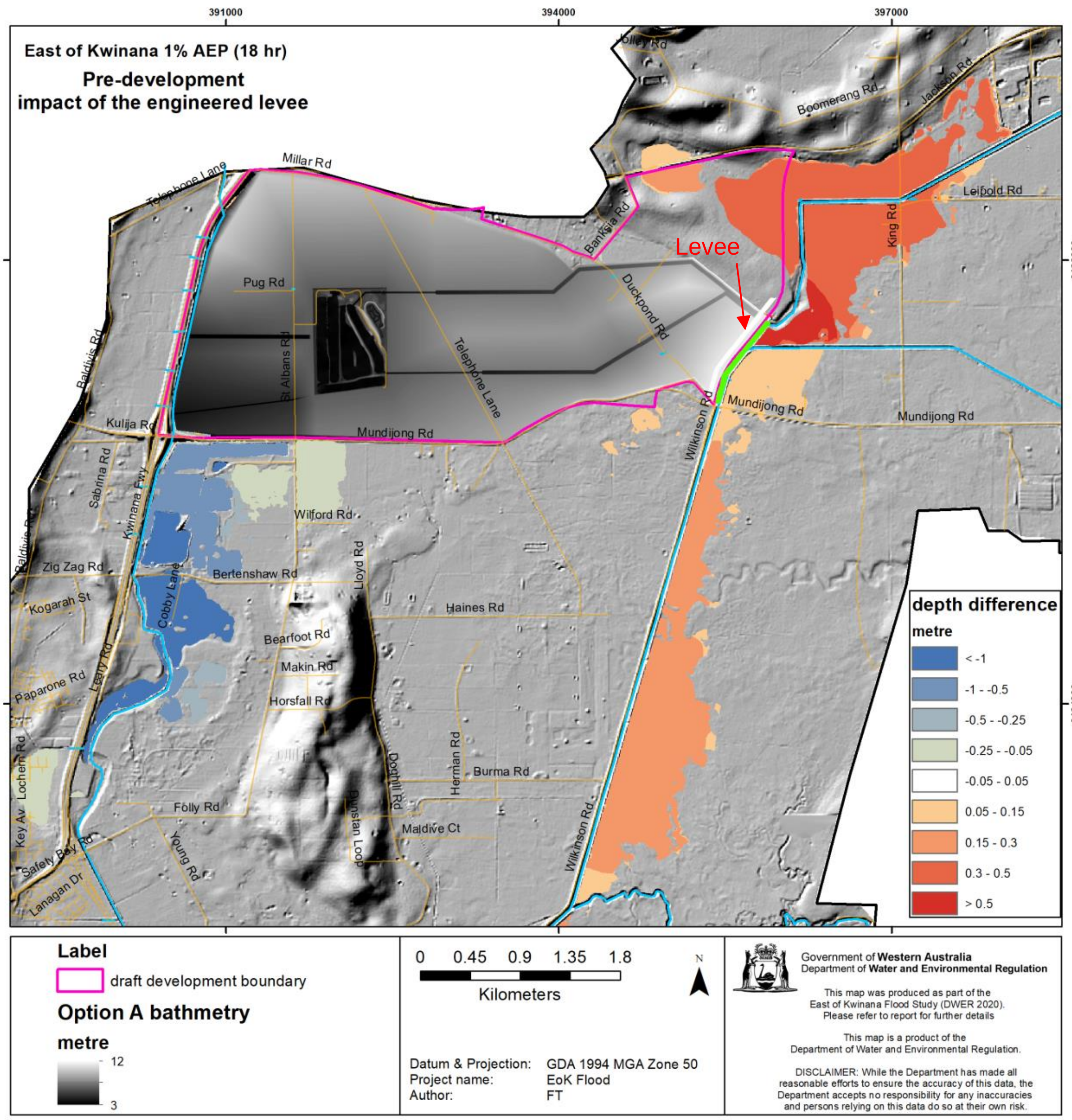


Figure 7-4 The engineered levee's impact at the Birrega Main Drain in Option A to the outsides (the upstream and downstream areas)

7.2.4 Discussion on design inputs with an engineered levee

In Option A, the engineered levee effectively blocks all overflow from the Birrega Main Drain (under the 1% AEP rainfall). Building a full-height levee cuts the North East Baldivis north of Mundijong Road precinct from its natural drainage basin, altering the catchment characteristics from the surface water point of view. However, the groundwater system is expected to be less influenced by the engineered levee. Water collected by the Birrega and Oaklands main drains, and the upstream floodplain will still affect the North East Baldivis north of Mundijong Road precinct through the groundwater system.

Despite that, we have tested design inputs for the local model of Option A based on a catchment of 70.3 km^2 by excluding the Birrega and Oaklands main drains' catchment area.

No significant difference was found in the flooding characteristics from these design inputs compared to sections 7.2.1, and the result was omitted in this report.

7.3 Scenario 1 Option A+

Considering the detrimental impact of Option A, Option A+ is designed to allow a maximum of 17 m^3/s discharge from the Birrega Drain under 1% AEP but maintains the bathymetry of Option A. Details of Option A+ is presented in the flood risk management land capability assessment *(*DWER, 2021).

Option A+ modelling is achieved by incrementally lowering part of the engineered levee adjacent to the North drain within North East Baldivis north of Mundijong Road precinct (Figure 7-1, Option A), which is not detailed here.

Therefore, the adjusted engineered levee in Option A+ cannot block all discharges from the Birrega Main Drain at 1% AEP. The engineered levee does not entirely alter the catchment characteristics. The regional model was employed for flooding analysis with inputs discussed in Section 4.

The Option A+ regional model maintains all the pre-development model features (including the mesh size), except that the MIKE 11 structures inside the North East Baldivis north of Mundijong Road precinct area were deleted because of bathymetry changes.

7.3.1 Drain discharge

The hydrograph sampled at the South Drain is shown in Figure 7-5, along with the time history of overflows at the engineered levee in Option A+. About half of the overflow is diverted to the South Drain in Option A+. Because of their proximity, the two hydrographs share a similar shape; but only one to two hours delay in the flows.

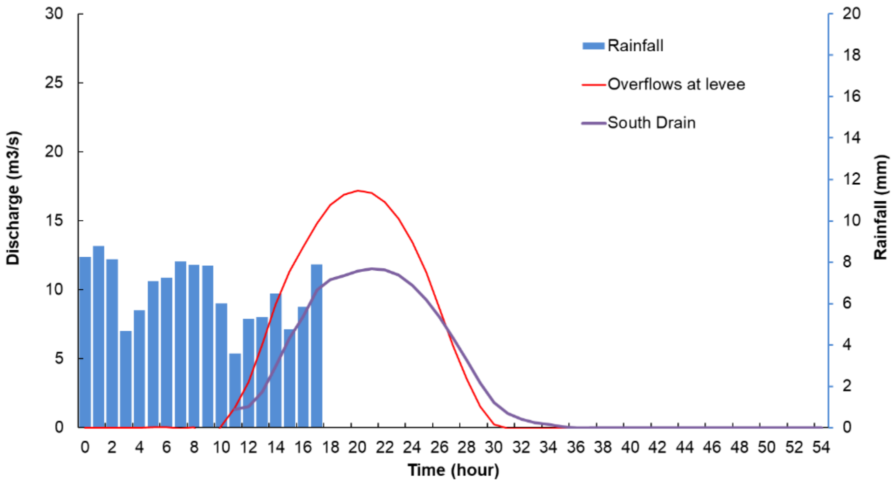


Figure 7-5 Hydrographs at the Levee and South Drain in Option A+

Table 7-2 compares the maximum discharges between Option A and A+. Once again, it highlights the levee's impact on the flows inside the drains in Option A.

Table 7-2 Comparison of the maximum discharges at selected cross-sections under 1% AEP critical rainfalls

Cross section	**Option A** (m^3/s)	**Option A+** (m^3/s)	**Pre-development** (m^3/s)
At the levee (downstream)	0	17.2	73.1
South Drain	2.5	11.5	N/A
North Drain	2.0	5.6	N/A

7.3.2 Flood level

Figure 7-6 shows the maximum flooding water depth of Option A+ at 1% AEP 18-hour Tp1 rainfall and inflows. The flooding carries many similarities to Option A's flood mapping, as discussed in Figure 6-4, including the South Drain's overflows downstream close to the Peel Main drain. A discussion on the flood levels for Option A+ is therefore not detailed.

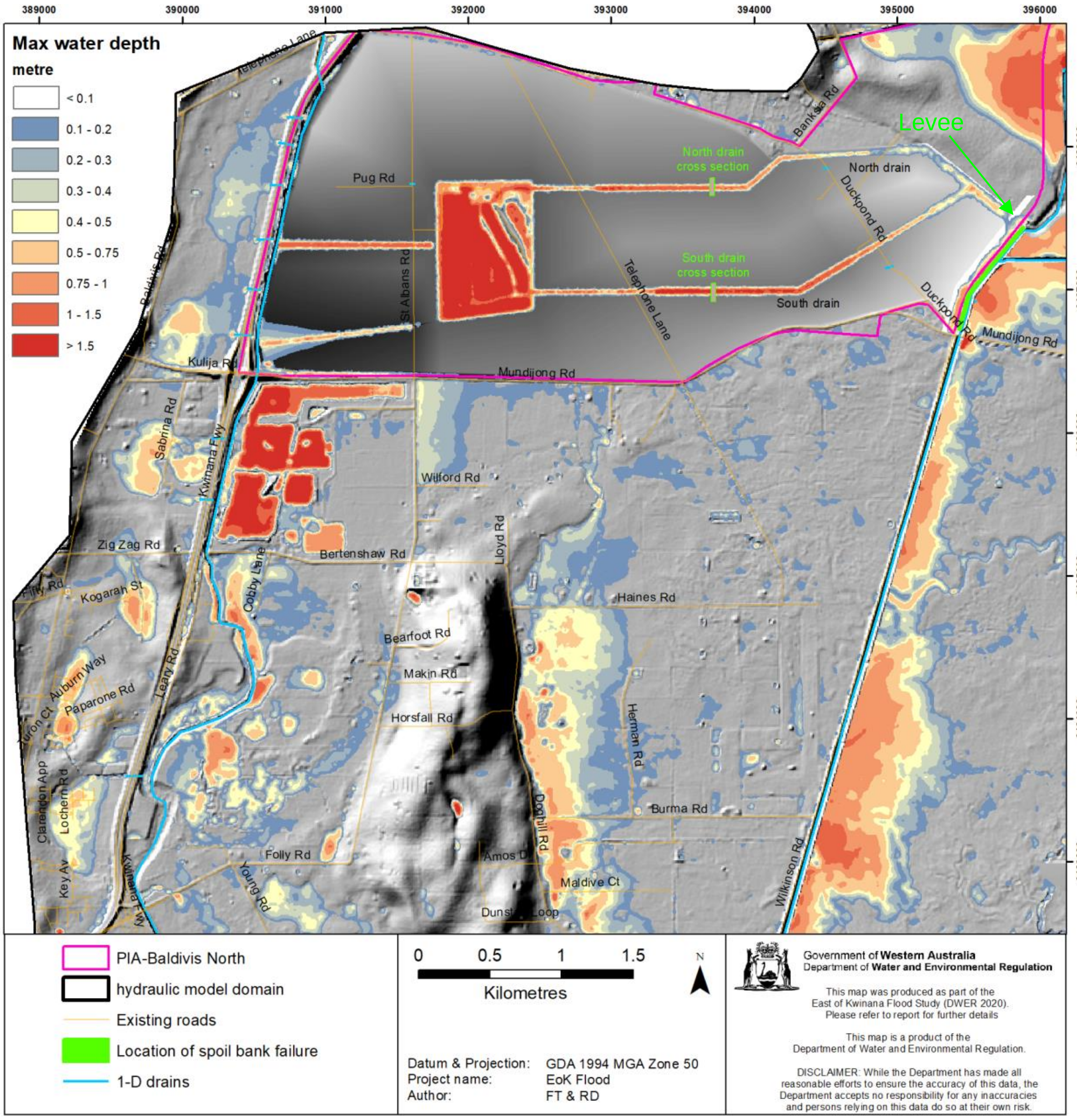


Figure 7-6 Maximum flooding water depth in Option A+ under 1% AEP 18-hour rainfall (based on the regional model). Option A+ is designed to allow a maximum of 17 m^3/s discharge from the Birrega Main Drain under 1% AEP.

7.3.3 Impact of Option A+

The impact of Option A+ to flooding extent outside of the North East Baldivis north of Mundijong Road precinct is studied by comparing the differences of flood levels between Figure 7-6 (Option A+) and Figure 6-9 (pre-development). The difference in flooding water depth is shown in Figure 7-7.

Similar to Figure 7-4, a significant area experiences an increase in flood levels, including both sides of the Birrega Main Drain in between Jackson Road and the Oaklands Main Drain (mostly between 0.3 ~ 0.5 m), and the area adjacent to the left bank of the Birrega Main Drain south of the Oaklands Main Drain (mostly between 0.15 ~ 0.3 m). Compared to Option

A, the areas experiencing deeper floodwaters than the pre-development are similar; however, the increase in Option A+ is lower than 0.5 m.

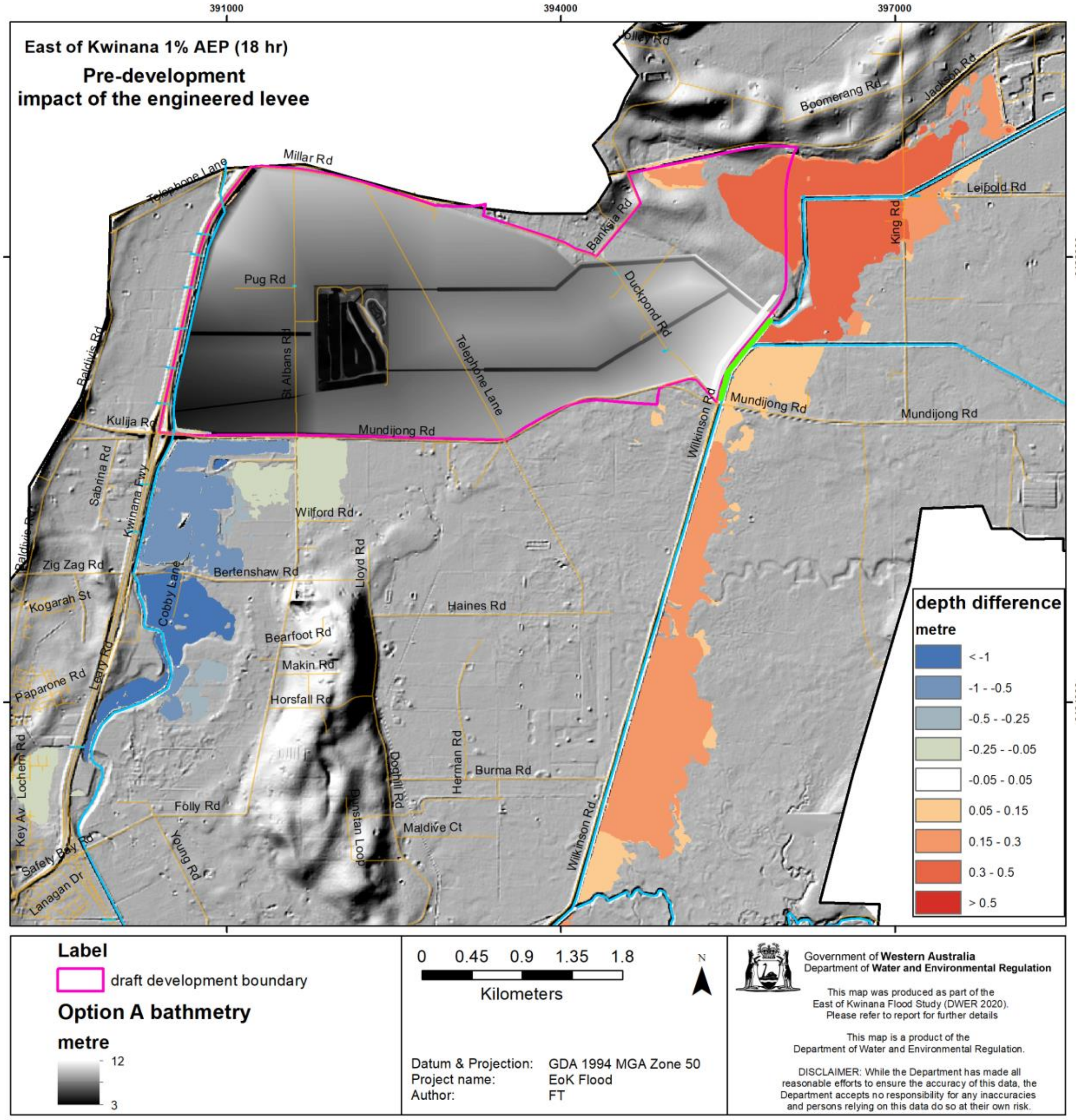


Figure 7-7 The engineered levee's impact on the floodplain by allowing a maximum of 17 m^3/s discharge into Option A terrain

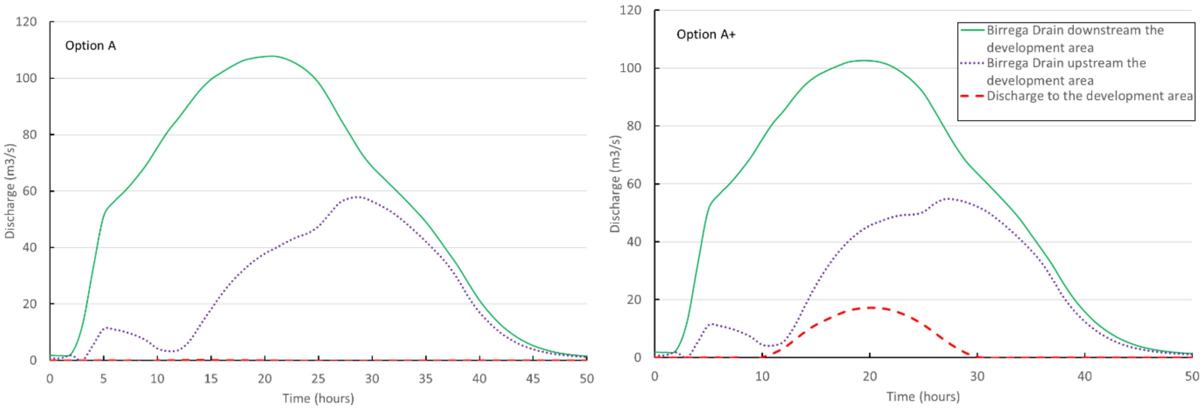


Figure 7-8 The engineered levee's impact on Birrega Main Drain by allowing a maximum of 17 m^3/s discharge into Option A terrain

Figure 7-8 compares the hydrographs measured at two cross-sections of Birrega Main Drain adjacent to the North East Baldivis north of Mundijong Road precinct (, immediately upstream and downstream the area (chainages 15865 and 17200). It shows the difference between Option A and A+ because of overflow over the designed levee.

A water balance calculation based on Figure 7-8 demonstrates the water overflows the engineered levee is from both the floodplain (as discussed in Figure 7-7) and the Birrega Main Drain. Overflows to the North East Baldivis north of Mundijong Road precinct would lessen the riverine flooding around the Birrega Main Drain. The discharge at the downstream location is slightly smaller in Option A+ (102.6 m^3/s) than in Option A (107.8 m^3/s).

7.4 Scenario 1 Option B

The difference between Option A and Option B was that Option A aimed to exclude external flooding from the North East Baldivis north of Mundijong Road, while Option B sought to accommodate flooding that can be reasonably expected under the pre-development conditions. Details of Option B is presented in the flood risk management land capability assessment *(*DWER, 2021).

Under the requirement of spoil bank failure with 1% AEP design rainfall, Option B intends to develop over half of the land for residential purposes. This plan is achieved by selectively raising existing land above the 1% AEP flood level but preparing sufficient waterways for potential flooding, as illustrated in Figure 7-9.

Option B was investigated by the local hydraulic model, with inflows from the regional spoil back fail 100% scenario at the failure location (Figure 7-9). The inflow hydrograph is given in Figure 6-7. The regional rainfalls and inflows generated from sections 4.1 and 4.1.2 were used in the modelling.

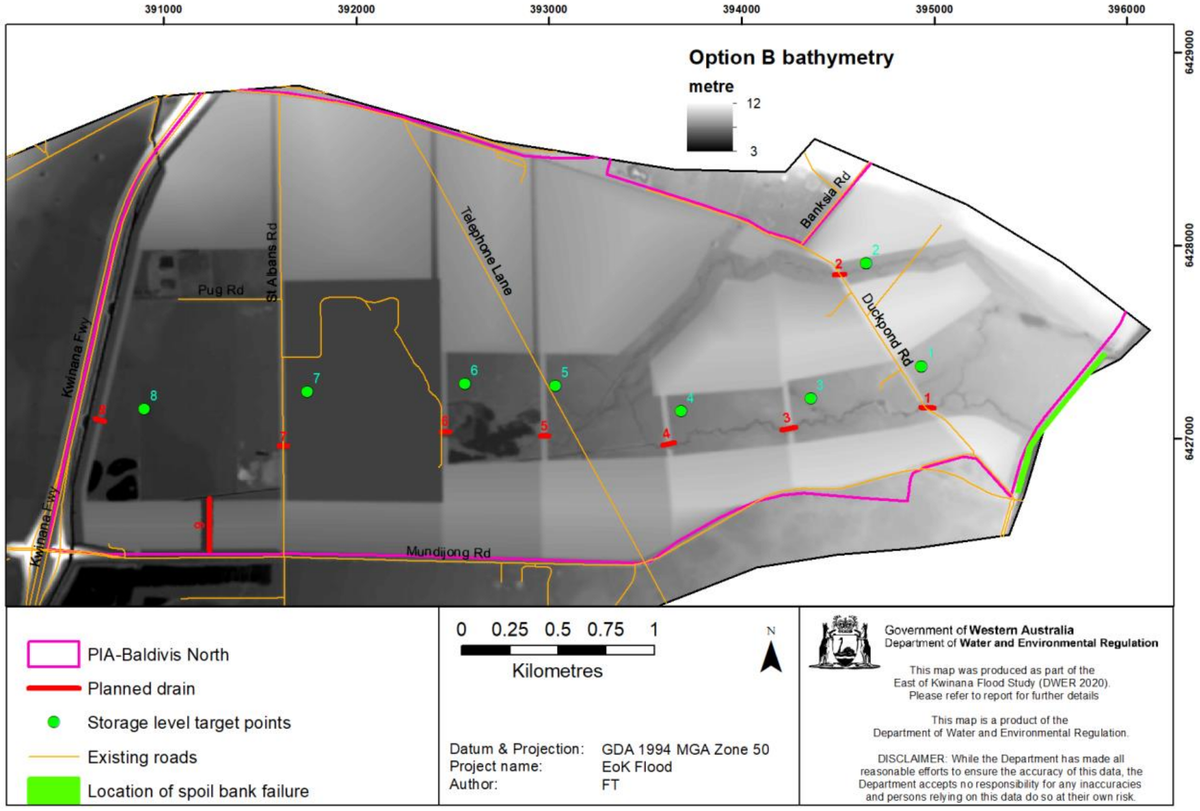


Figure 7-9 Hydraulics control channels (red) to divert floodwaters and flood level gauging points (green) to control storage levels

Option B's floodwater is controlled by nine channels (Figure 7-9) in the hydraulic modelling. These channels divert the floodwaters resulting from spoil bank fail 100% of the Birrega Main Drain. The design principles of these drains are:

- the residential land in any block is not flooded under 1% AEP design rainfall

- the new roads connecting those lands are not flooded under 1% AEP rainfall condition
- the total outflow from the North East Baldivis north of Mundijong Road precinct to the Peel Main Drain or at Mundijong Road is like the pre-development condition.

By iteratively changing the size of drains, which is not discussed in the present report, these drains' final dimensions are given in Table 7-3. These values were achieved by iteratively adjusting the channel sizes by not flooding the adjacent land and roads

Table 7-3 Characteristics of drains designed in Option B to divert floodwater

Channel	**Width (indicative)** (m)	**Depth** (m)	**Bottom level** (m AHD)	**Upstream target level** (m AHD)
Channel 1	35.0	1.30	8.90	10.20 (point 1)
Channel 2	1.5	2.25	7.95	10.20 (point 2)
Channel 3	30.0	1.35	8.25	9.60 (point 3)
Channel 4	24.0	1.65	7.25	8.90 (point 4)
Channel 5	28.5	1.45	6.35	7.80 (point 5)
Channel 6	36.0	1.65	5.35	7.00 (point 6)
Channel 7	25.5	1.50	5.30	6.80 (point 7)
Channel 8	9.0	1.25	4.95	6.20 (point 8)
Channel 9	12.6	1.20	5.00	6.20 (point 8)

7.4.1 Flooding map of Option B

The maximum flooding water depth for Option B under both 1% AEP and 5% AEP (18-hour Tp1) are given in Figure 7-10 and Figure 7-11, respectively. These maps were obtained by calculating the maximum water depth over 54 hours of simulation and then displayed at a 5 m×5 m grid. The main flooding features are:

- all elevated land is above the 1% AEP flood level.
- Floodwater above 0.75 m is observed in most areas in the designed drains.
- overflows to the Peel Main Drain appears to be small.
- flood water distribution downstream (south of Mundijong Road) appears to be similar to the pre-development scenario (Figure 6-9).

As expected at 5% AEP rainfall, Figure 7-11 shows a lower level of floodwater depth than the 1% AEP, although the flooding map bears many similarities to Figure 7-10.

The flooding maps demonstrate floodwater in Option B is constrained by the drains inside the draft bathymetry. Option B leads to a similar impact to downstream catchment compared to the pre-development, and therefore, Option B is feasible from a flooding perspective.

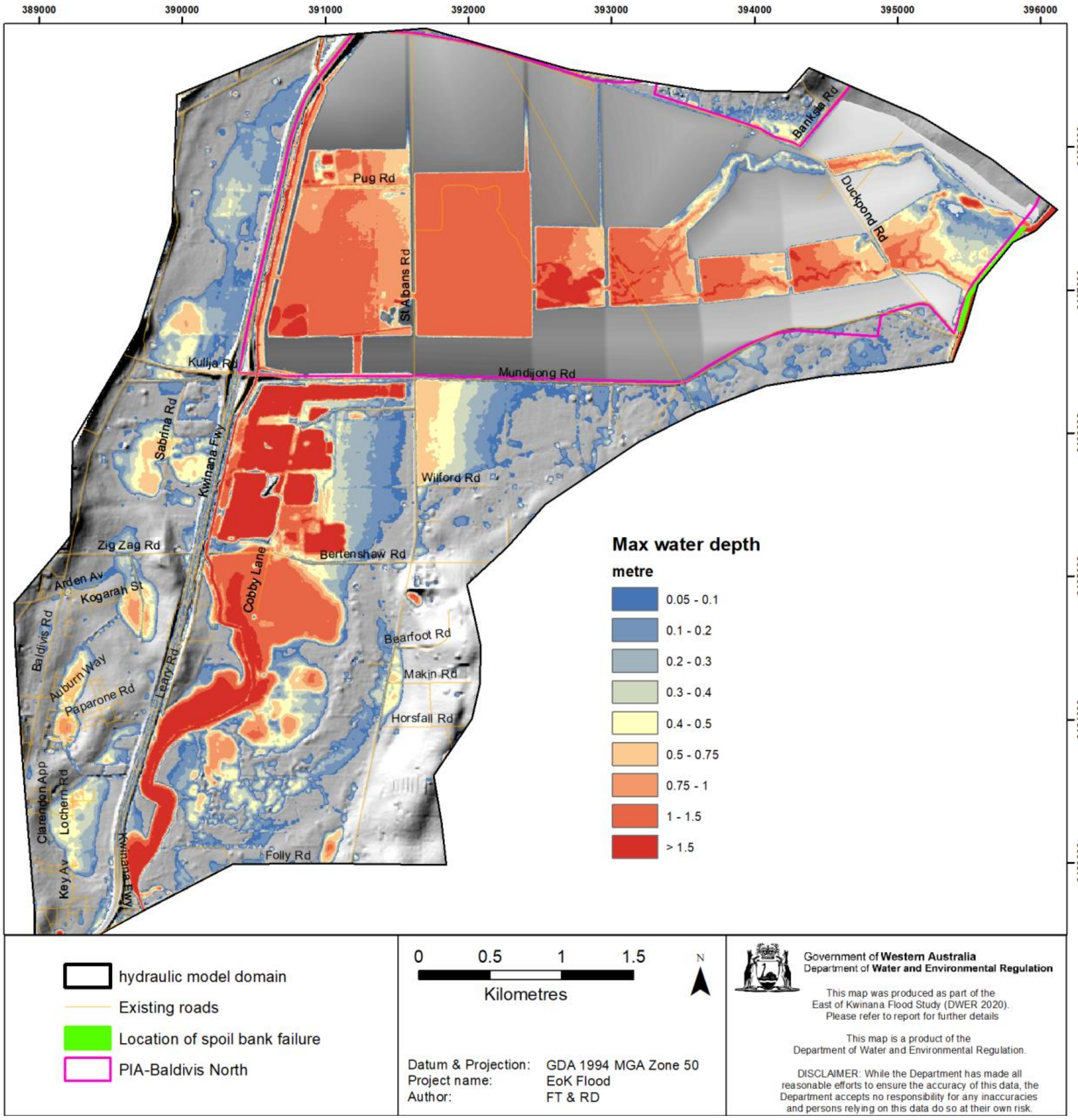


Figure 7-10 Maximum flooding water depth in Option B under 1% AEP 18-hour rainfall

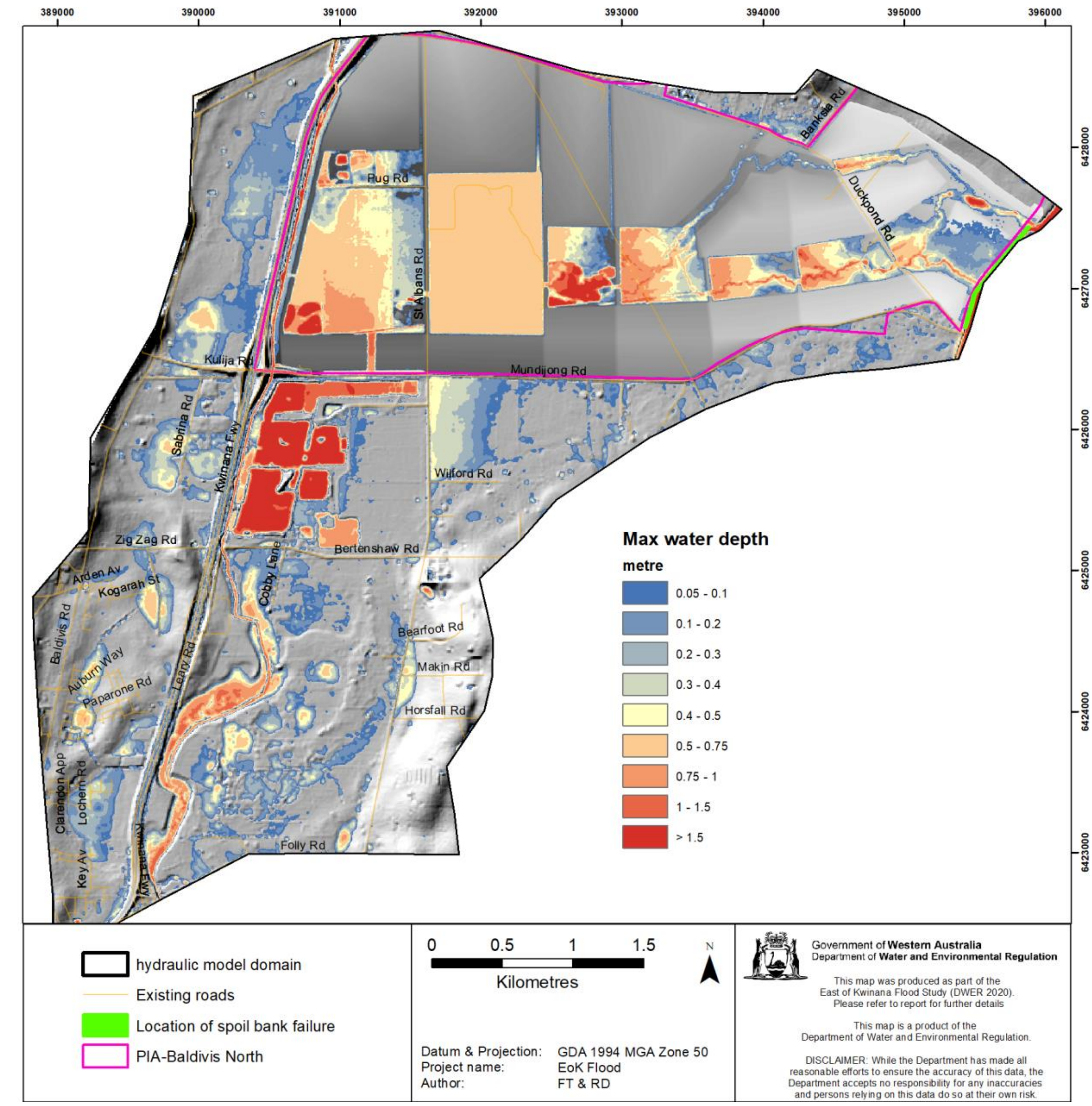


Figure 7-11 Maximum flooding water depth in Option B under 5% AEP 18-hour rainfall

7.4.2 Flood water elevation of Option B

Figure 7-9 presents the hourly records of floodwater surface elevation in Option B at eight sampling locations. The hourly records indicate a cascade of increases in water levels upstream to downstream as floodwater flows eastward point 1 experience water level increases five hours after the simulation start, while point 8 did not experience an apparent increase until 16 hours into the simulation.

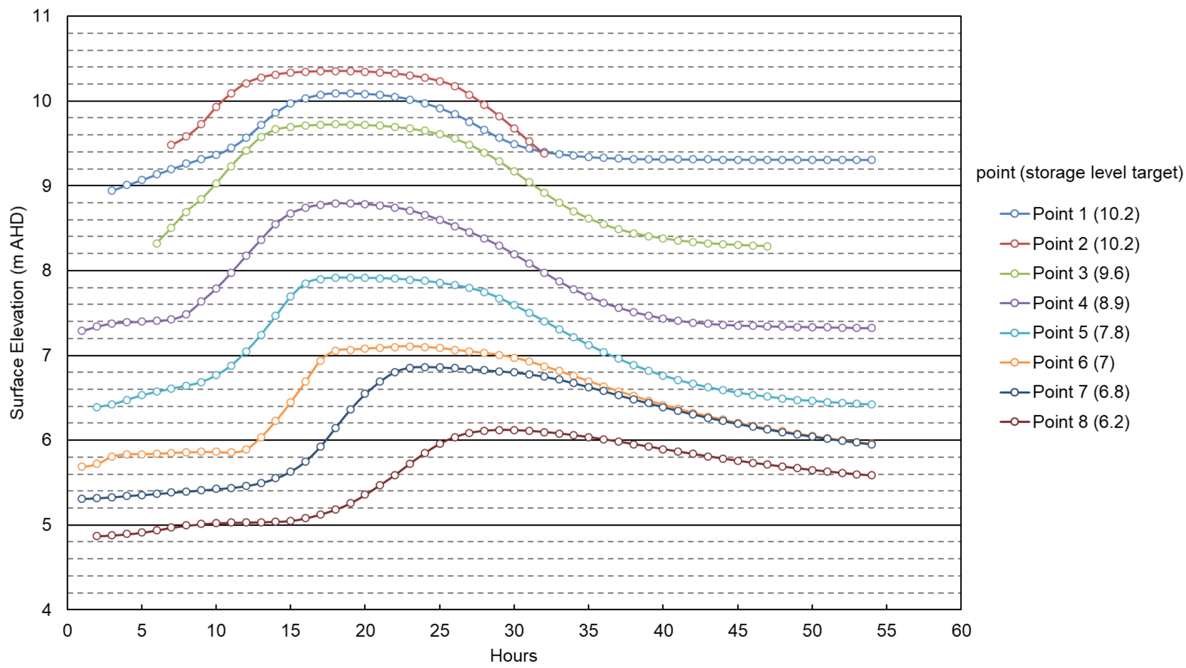


Figure 7-12 Hourly records of floodwater surface elevation in Option B at different sampling locations (point location are given in Figure 7-9)

7.4.3 Discharge hydrograph of Option B

Three representative discharge hydrographs from Option B are given in Figure 7-13, along with the hourly rainfalls. These hydrographs were obtained by sampling Option B's flow rates at North East Baldivis north of Mundijong Road precinct to the Peel Main Drain (total flow to Peel Main Drain), at the Mundijong Road, and the flow rate of Peel Main Drain at Mundijong Road. The peak flow at these three locations occurs about the same time, and they are 60.9 m^3/s at Mundijong Road, 25.1 m^3/s for the Peel Main Drain at Mundijong Road and 21.8 m^3/s for the total flow from North East Baldivis north of Mundijong Road precinct to the Peel Main Drain.

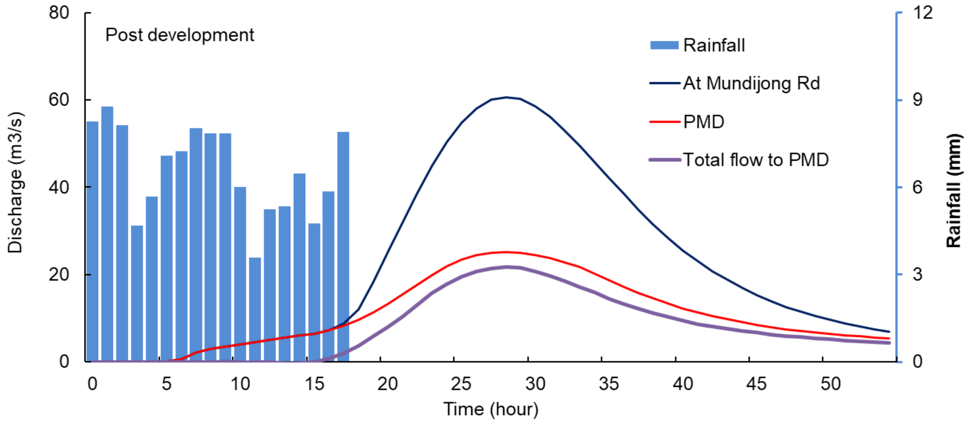


Figure 7-13 Selected discharge hydrographs in Option B under 1% AEP 18-hour duration rainfall

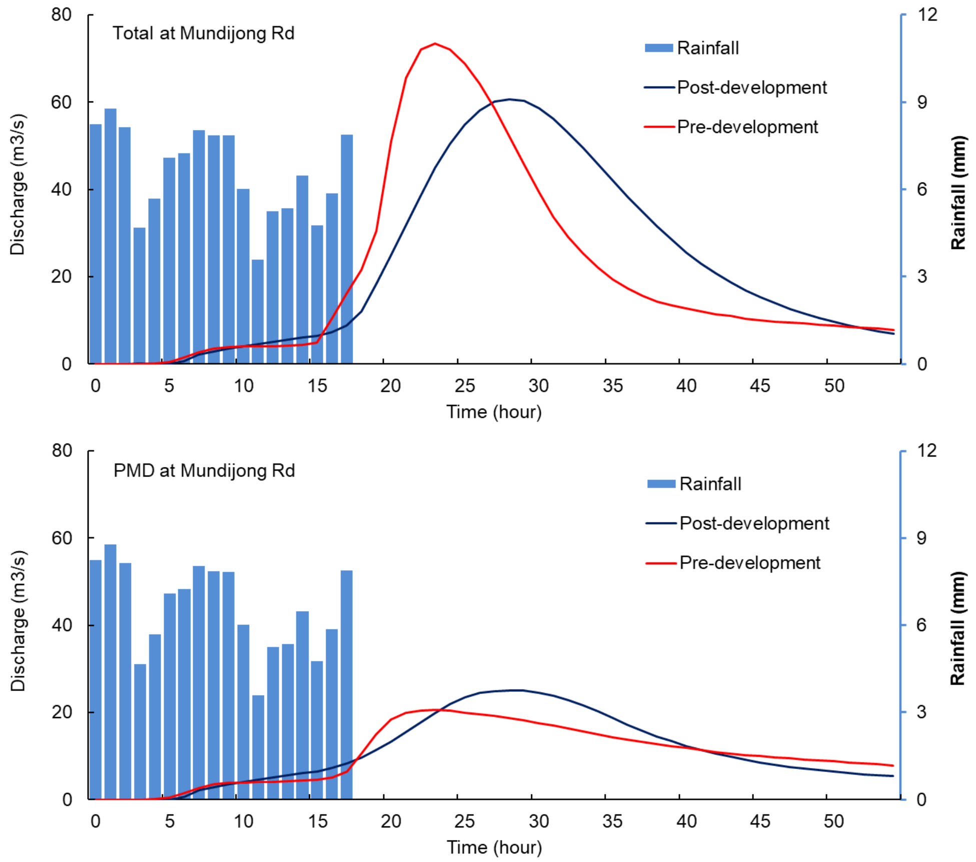


Figure 7-14 The impact of Option B to flow at Mundijong Road and Peel Main Drain

Figure 7-14 compares the discharge hydrographs between post-development and pre-development at two cross-sections; that is, the discharges at Mundijong Road and the Peel Main drain at Mundijong Road. The discharge from post-development is similar to or smaller than the pre-development case, demonstrating the impacts of development to flood water discharges at these locations are acceptable from a flooding water control point of view.

7.5 Scenario 1 Option B+

As detailed in the flood risk management land capability assessment (DWER, 2021), a revised version of Option B was developed, which included the following modifications:

- Provide a formalised spillway for discharge over Mundijong Road
- Reinstate the Cell 7 height target of 6.2 m AHD.
- Maintain floodplain storage
- Provide additional developable area.

The Option B+ hydraulic model is otherwise the same as Option B, except the bathymetry, as shown in Figure 7-15.

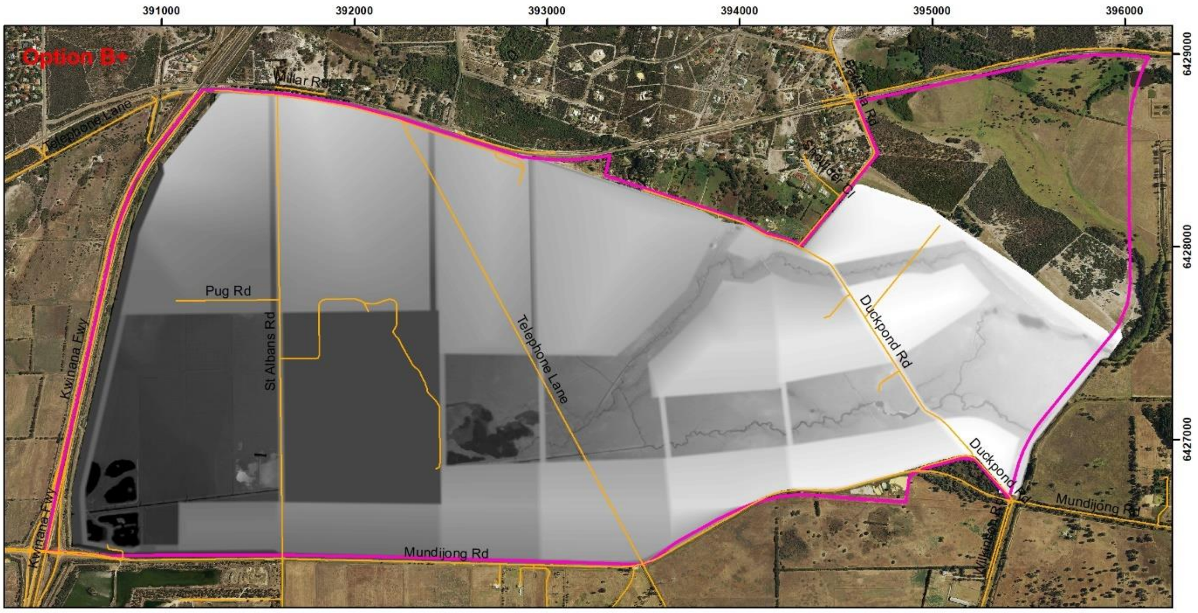


Figure 7-15 Hypothetical bathymetry of Option B+ for the development scenario

7.5.1 Flooding map of Option B+

The maximum flooding water depth of Option B+ at 1% AEP and 5% AEP critical duration is shown in Figure 7-16 and Figure 7-17, respectively. Their distribution is similar to what is discussed for Option B, including areas inside and outside the North East Baldivis north of Mundijong Road precinct. However, because of increased land-use, the maximum water depth in Option B+ is slightly more profound than that in Option B. For this reason, a detailed description of the flooding characteristics is not repeated here.

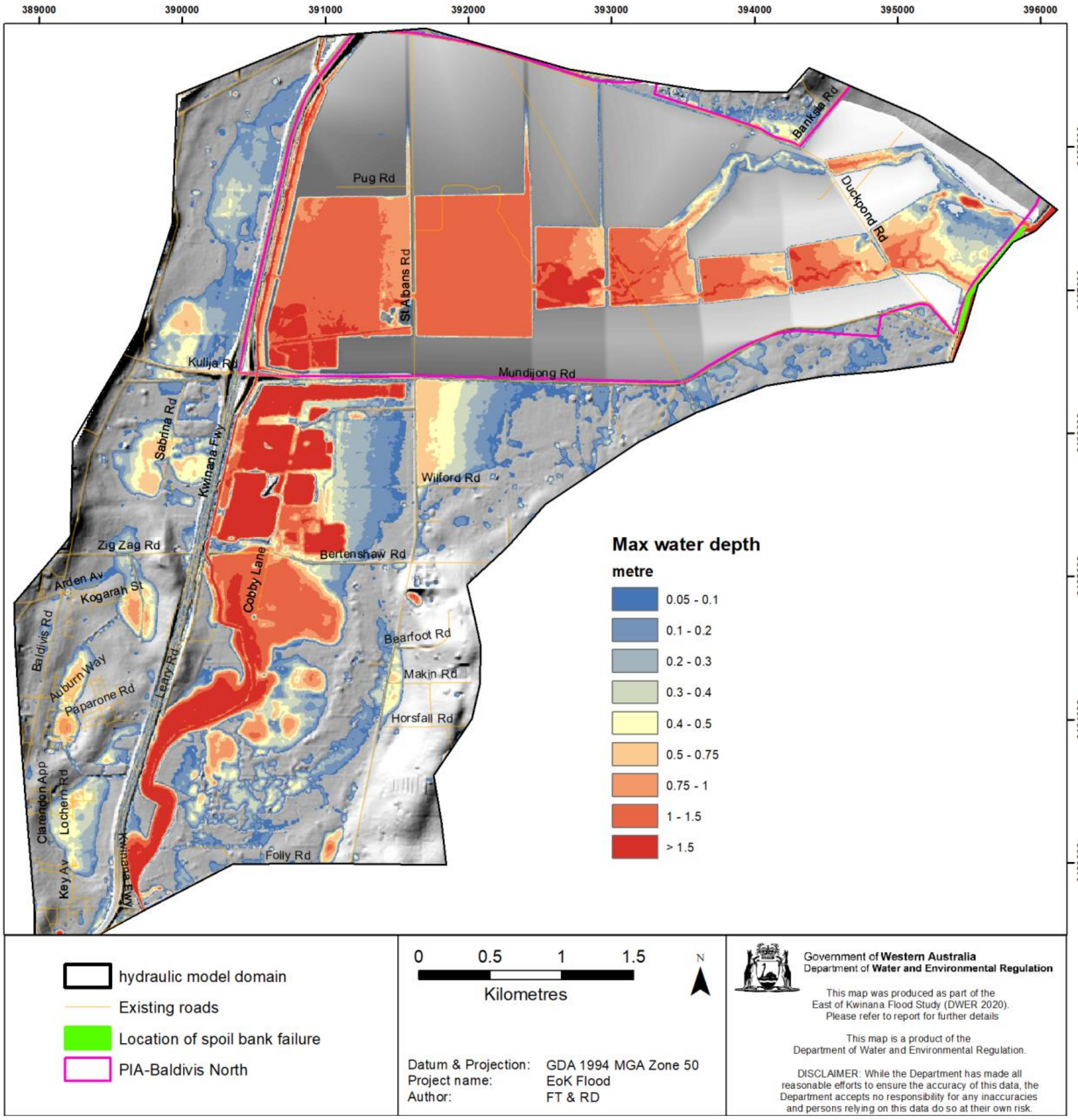


Figure 7-16 Maximum flooding water depth in Option B+ under 1% AEP 18-hour rainfall

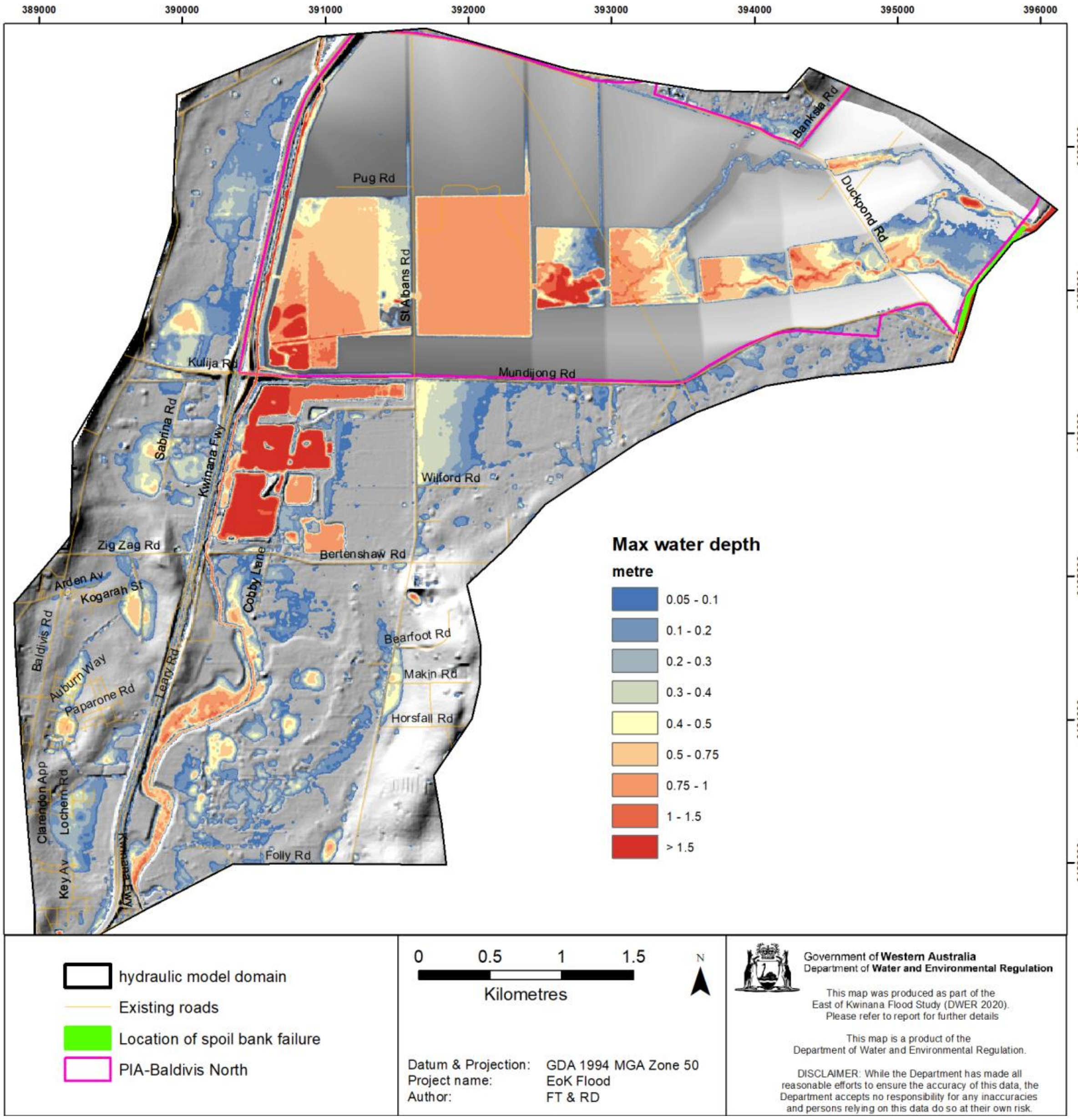


Figure 7-17 Maximum flooding water depth in Option B+ under 5% AEP 18-hour rainfall

7.5.2 Discharge hydrograph of Option B+

Like Option B, three representative discharge hydrographs from Option B+ are given in Figure 7-18 and compared with pre-development in Figure 7-19. The peak discharges are slightly larger than those in Option B but are still similar to or smaller than those in the pre-development case.

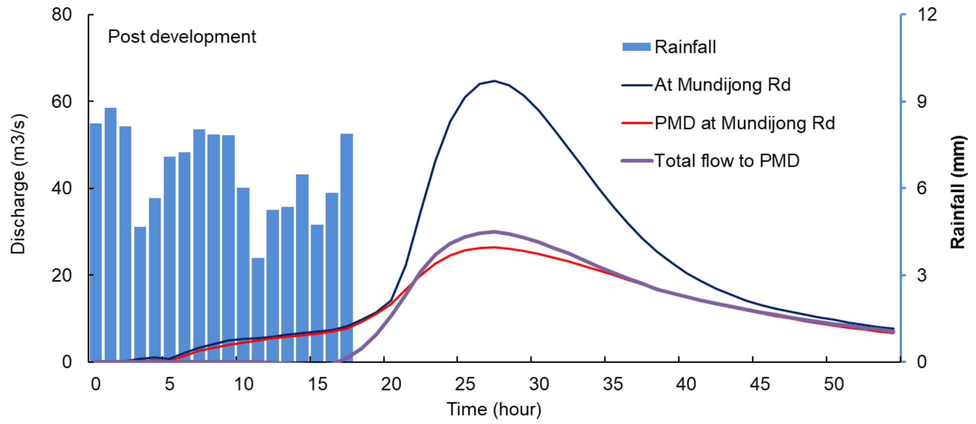


Figure 7-18 Selected discharge hydrographs in Option B+ under 1% AEP 18-hour duration rainfall

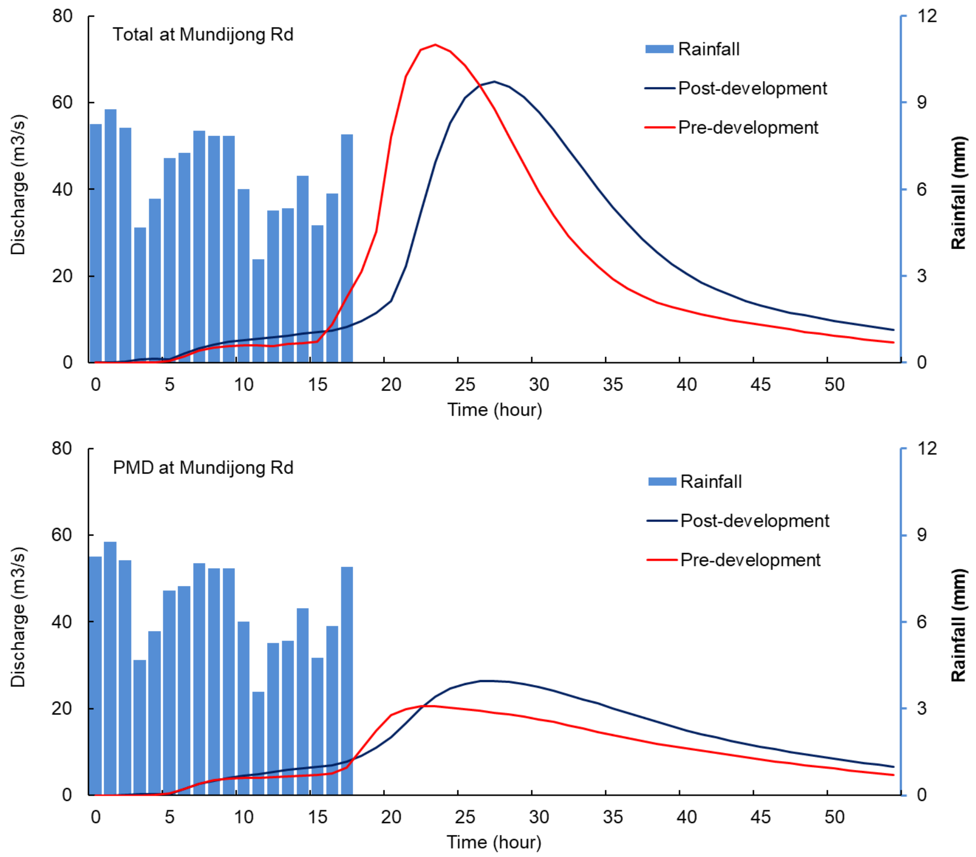


Figure 7-19 The impact of Option B+ to flow at Mundijong Road and Peel Main drain

The flooding map and discharge hydrographs demonstrate Option B+ is feasible. Besides, more land is potentially available for development.

7.5.3 Impact of Option B+

The impact of Option B+ on flooding outside of the North East Baldivis north of Mundijong Road precinct is studied by comparing flood levels between post-development flooding in Figure 7-16 (Option B+) and the pre-development condition in Figure 6-9. The difference in flooding water depth is shown in Figure 7-20, where the maximum downstream flood water depth decreases in a significant area because of the development. This is consistent with the overflow hydrograph at Mundijong Road, as discussed in Figure 7-19. On the other hand, the flood waterways left inside the North East Baldivis north of Mundijong Road precinct primarily experience floodwater depth increase because of the hypothetical engineered work.

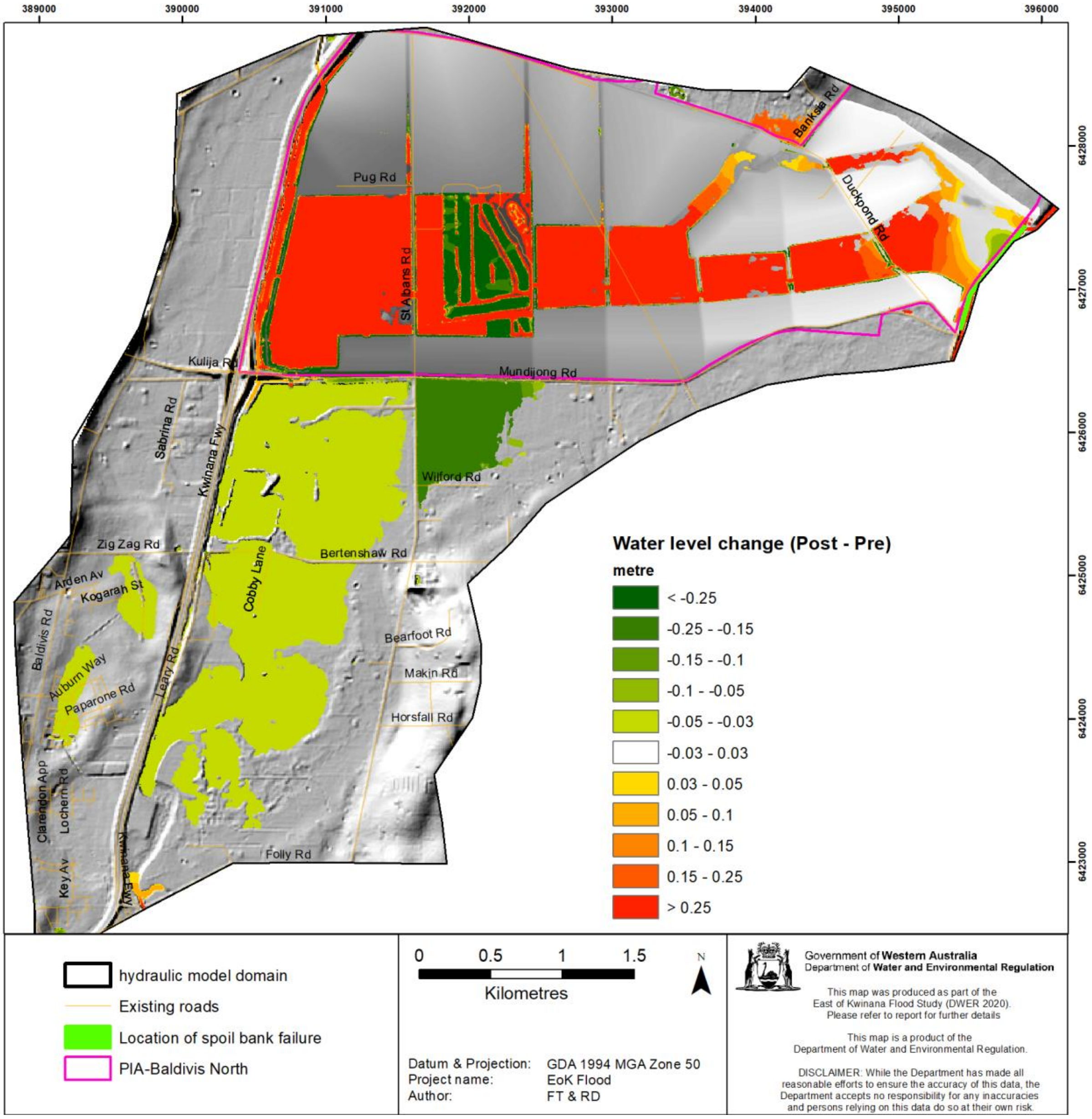


Figure 7-20 The impact of Option B+ to flooding outside the North East Baldivis north of Mundijong Road precinct

7.6 Scenario 1 Option C

The flood risk management land capability assessment (DWER, 2021) detailed that the Option C scenario considered residential and industrial land-uses. The bathymetry of Option C is given in Figure 7-21. The land intended for development is raised according to two criteria:

- for residential land-use, the terrain was raised to above the 1% AEP flood level (high areas of ground are depicted by white in Figure 7-21)
- For industrial land-use, it was raised to above the 5% AEP flood level (low areas of ground are depicted in-grey in Figure 7-21).

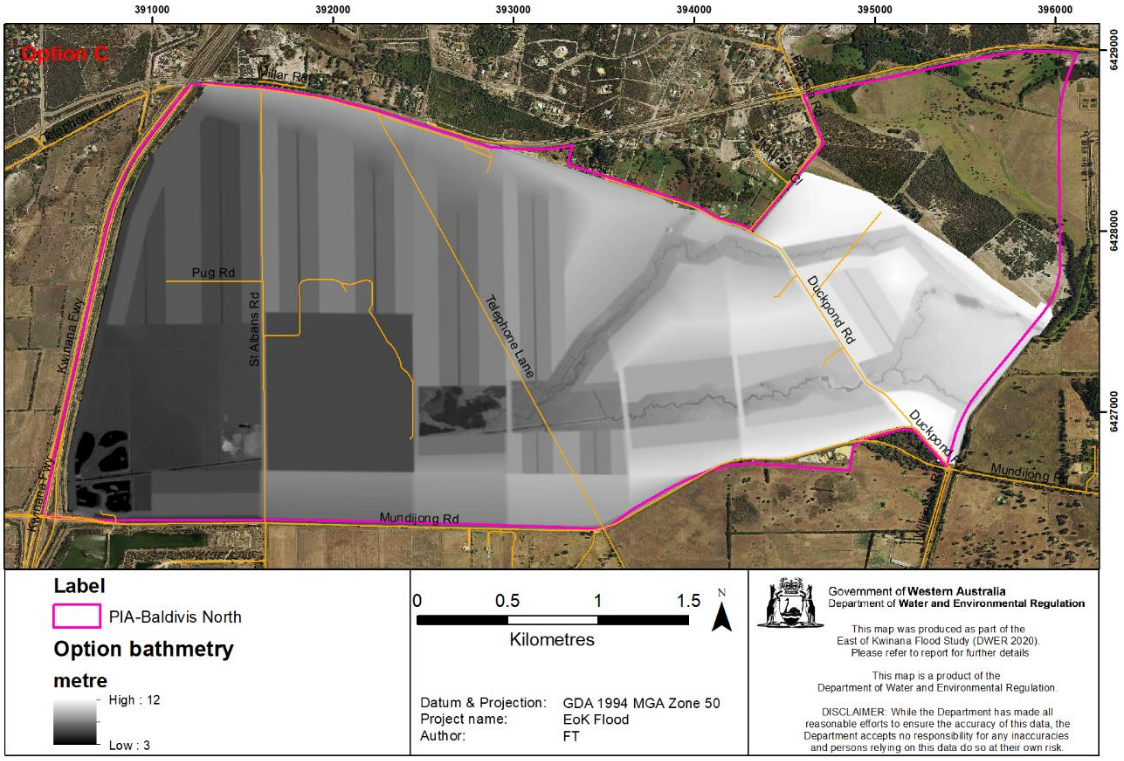


Figure 7-21 Land-use categories in Option C

By iteratively adjusting the channel sizes (similar to what was discussed in Option B), the water levels were controlled to not exceed the land elevations according to the two criteria listed above.

7.6.1 Flooding map of Option C

The maximum flooding water depths for Option C under both 1% AEP and 5% AEP are given in Figure 7-22 and Figure 7-23, respectively. These maps were obtained by calculating the maximum water depth over 54 hours of simulation results and then displayed at a 5 m×5 m grid. Water levels lower than 0.05 m are excluded from the map to eliminate very shallow flooding areas. The main flooding features from Figure 7-22 are:

- All elevated land for residential use is above the 1% AEP flood level.
- All elevated land for industrial use is below the 1% AEP flood level, where the floodwater depth is between 0.5 m ~ 0.75 m.
- Flood water above 0.75 m is observed in most areas in the designed drains.
- Flood water distribution downstream (south of Mundijong Road) appears similar to the pre-development scenario (Figure 6-16).

As expected at 5% AEP rainfall, Figure 7-23 shows a lower level of floodwater depth for most areas than that of the 1% AEP. However, some land intended for industrial use, including the land near Telephone Line, are submerged by floodwater at 5% AEP. The water depth is mostly less than 0.1 metres. The condition can be improved by raising the land in question slightly higher or optimising the channel dimensions inside the domain. However, no further simulations were attempted to improve the flood condition at the 5% AEP condition.

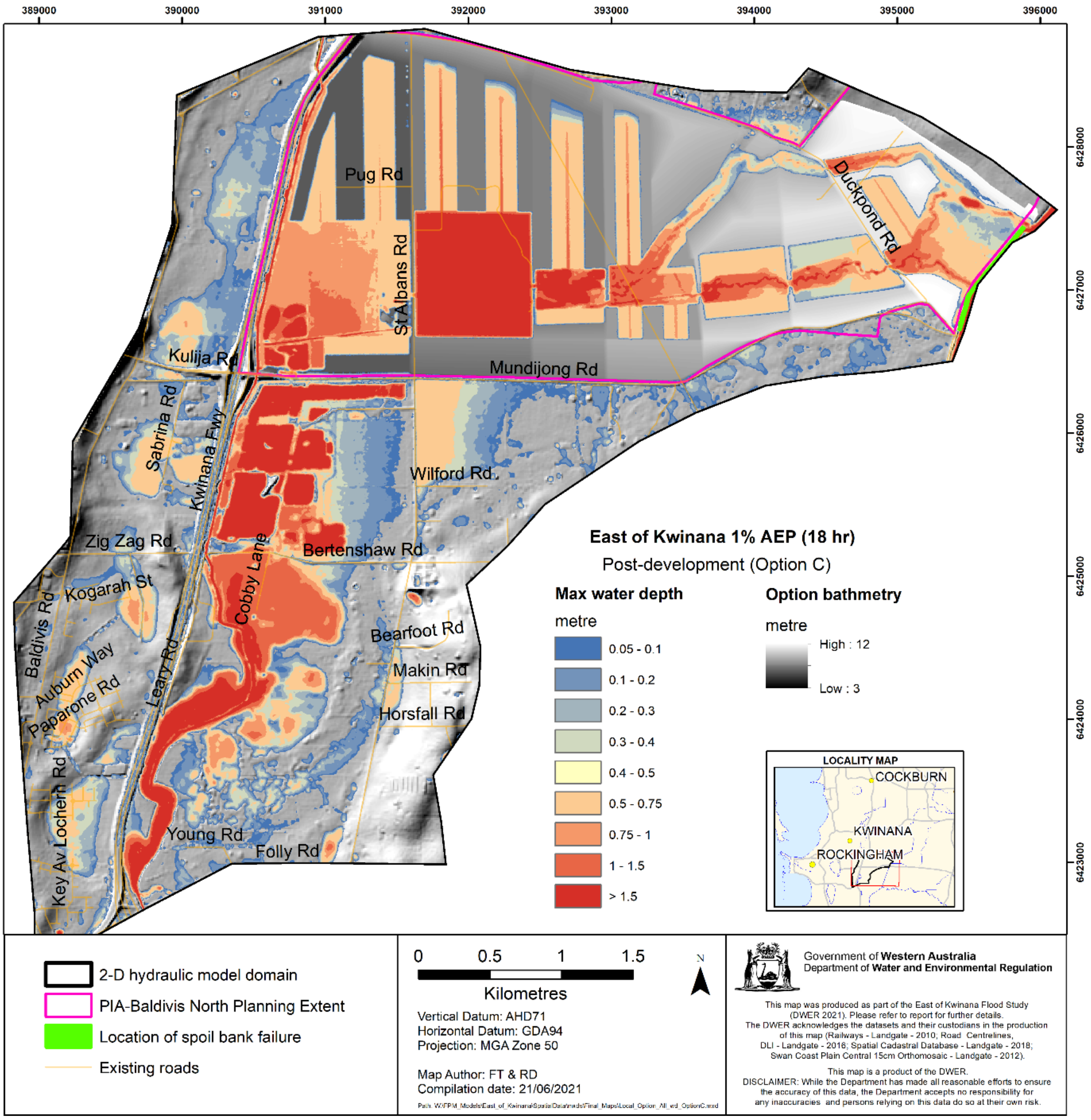


Figure 7-22 Maximum flooding water depth in Option C under 1% AEP 18-hour rainfall. Existing roads are also shown inside the North East Baldivis north of Mundijong Road precinct.

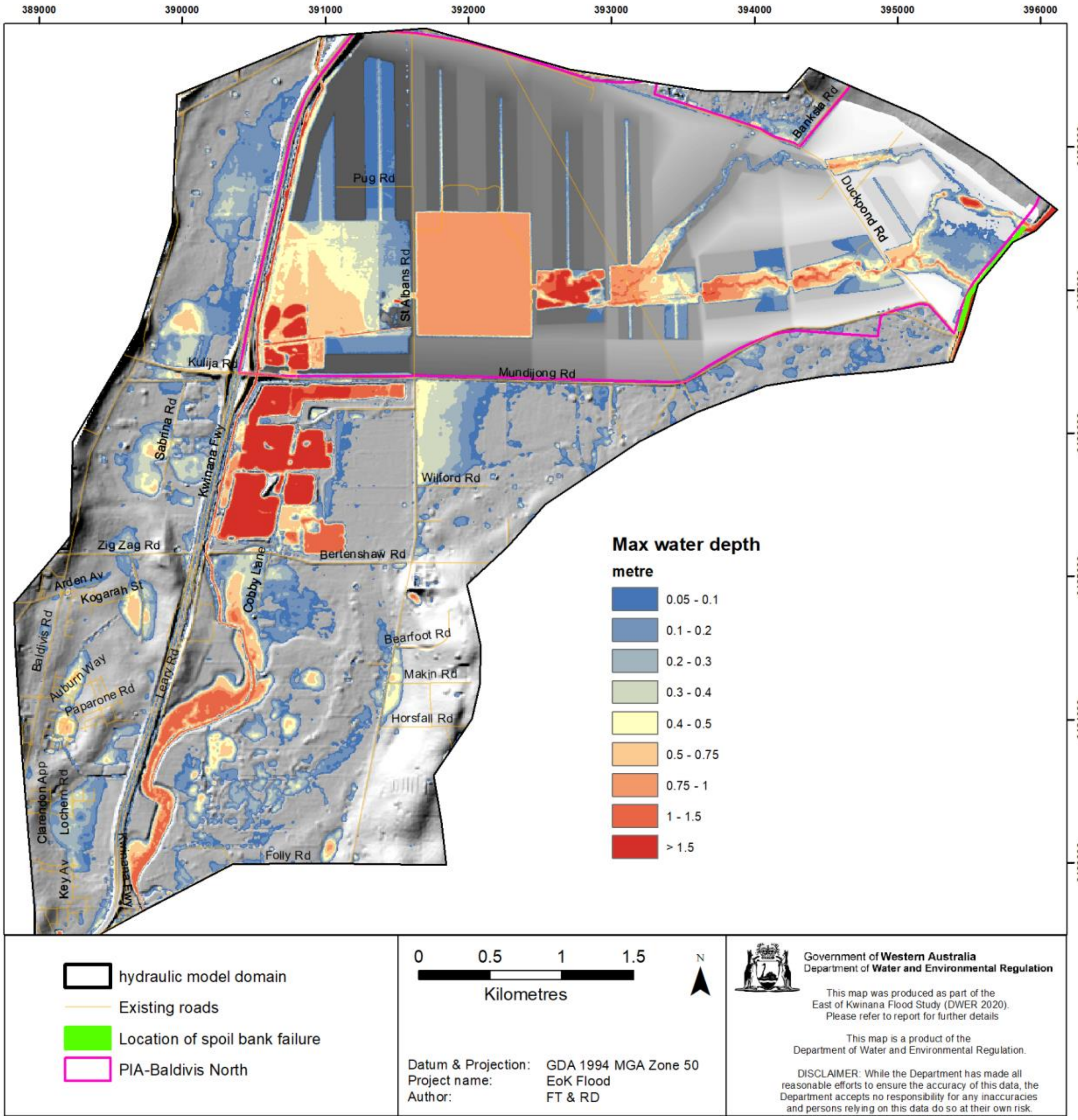


Figure 7-23 Maximum flooding water depth in Option C under 5% AEP 18-hour rainfall

7.6.2 Discharge hydrograph of Option C

Representative discharge hydrographs at 1% AEP from Option C are given in Figure 7-24, along with the hourly rainfalls.

These hydrographs were obtained by sampling the flow rates along the Mundijong Road and the flow rate of the Peel Main Drain at Mundijong Road. The peak flow at these three locations occurs about the same time, and they are 62.1 m^3/s at Mundijong Road and 29.5 m^3/s for the Peel Main Drain at Mundijong Road. The measured overflow from the North East Baldivis north of Mundijong Road precinct to the Peel Main Drain was very small since Option C essentially connected the last floodwater storage near Kwinana Freeway to the Peel Main drain.

Figure 7–24 also compares the discharge hydrographs between post-development and pre-development at these two cross-sections in Option C.

At Mundijong Road, the peak flow from post-development is smaller than in pre-development, decreasing from 73.3 m^3/s to 62.1 m^3/s. The hydrograph of Option C also shows a time delay compared to the pre-development starting from 20 hours into the simulation. Both are because of Option C's cascade storages in the bathymetry (Figure 7-23). However, the total volume of water passing Mundijong Road during this period is 4.3 GL for the post-development case, slightly higher than the 3.9 GL for the pre-development.

For the Peel Main drain at Mundijong Road, the peak discharge from post-development is 29.5 m^3/s, compared to 20.5 m^3/s for the pre-development. The event volume at this location is 2.8 GL (as compared to 1.9 for the pre-development).

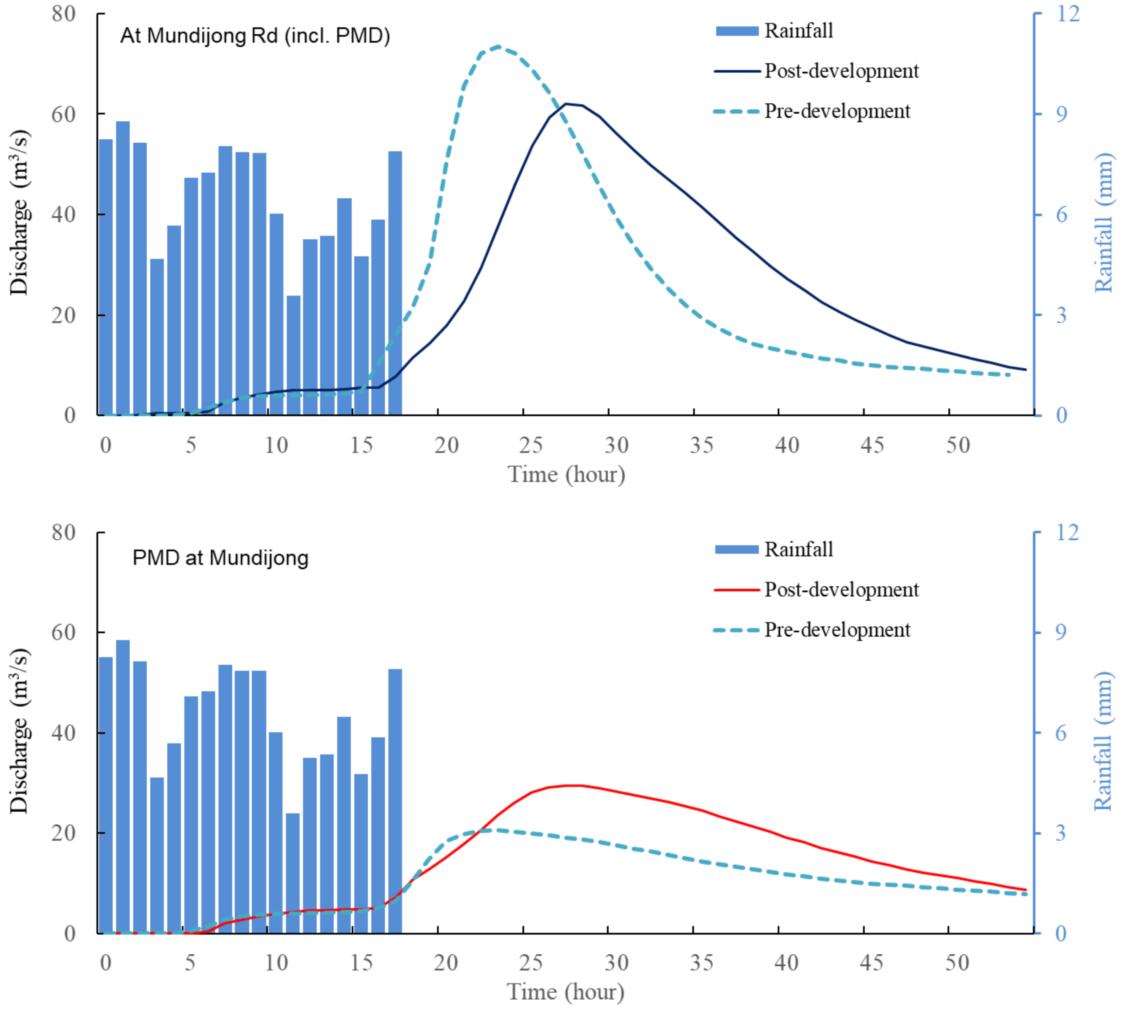


Figure 7-24 The impact of Option C to flow at Mundijong Road and Peel Main drain under 1% AEP 18-hour rainfall

7.6.3 Impact of Option C

The flooding difference map between the post-development water depth in Figure 7-22 and the pre-development one demonstrates flood water in Option C is constrained by the drains.

Option C leads to a slightly positive impact on downstream catchment compared to the pre-development.

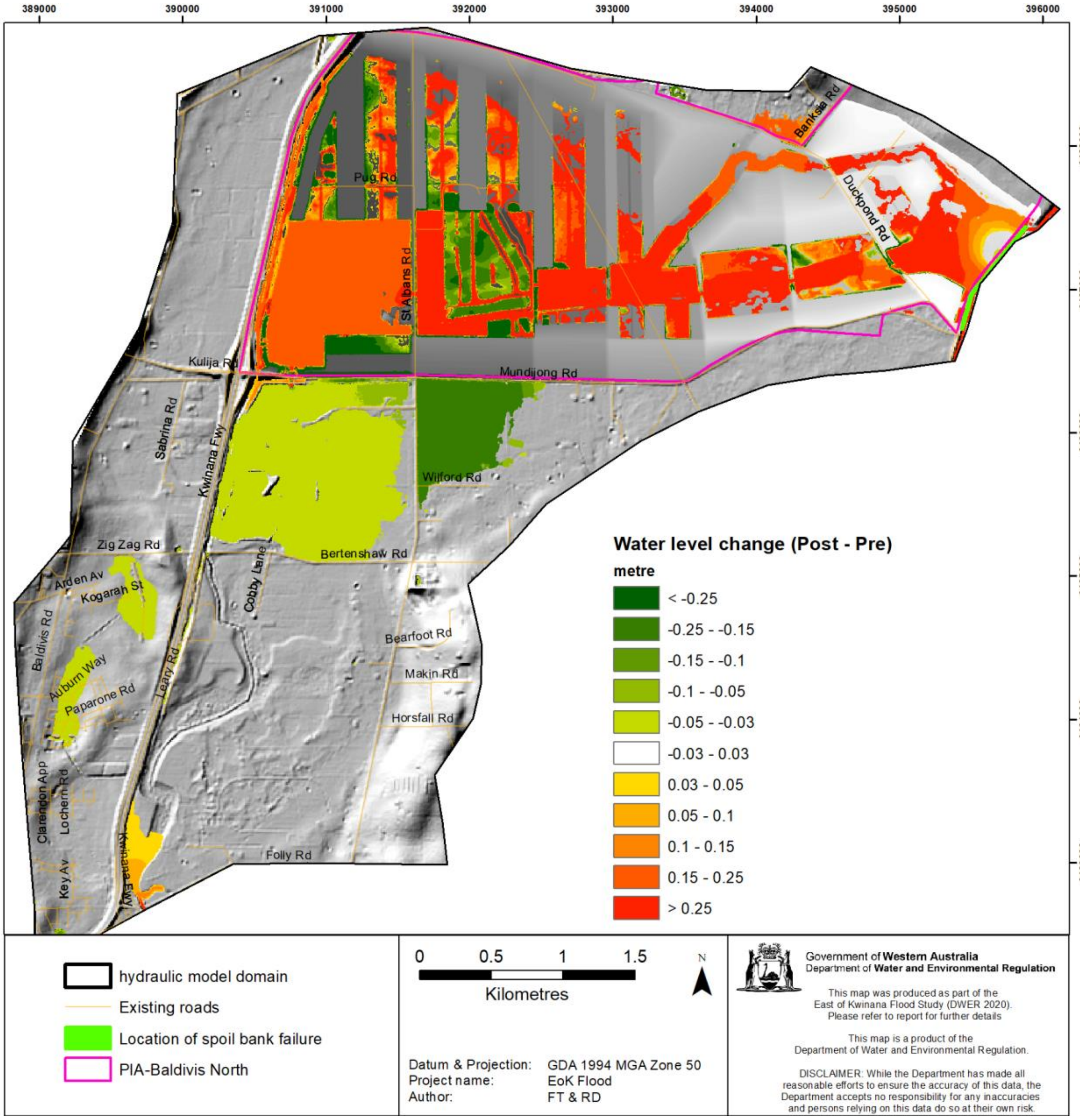


Figure 7-25 The impact of Option C on flooding outside the North East Baldivis north of Mundijong Road precinct

8 North East Baldivis south of Mundijong Road – Pre- and post-development (Scenario 2)

The flood risk management land capability assessment (DWER, 2021).report has detailed the option of development on North East Baldivis south of Mundijong Road precinct (the PIA-Baldivis South, see Figure 8-1). Like what discussed in Section 6, the modelling aims to answer, from a flooding perspective, the following two questions:

1. Can the precinct be developed without unacceptable impacts to neighbouring land?
2. How much land can be provided at an appropriate flood standard for the land-use that is being sought?

Figure 8-1 shows an overview of the proposed development. The option raises the land above the 1% AEP flood levels subject to spoil bank failure scenarios adjacent to the Birrega Main Drain.

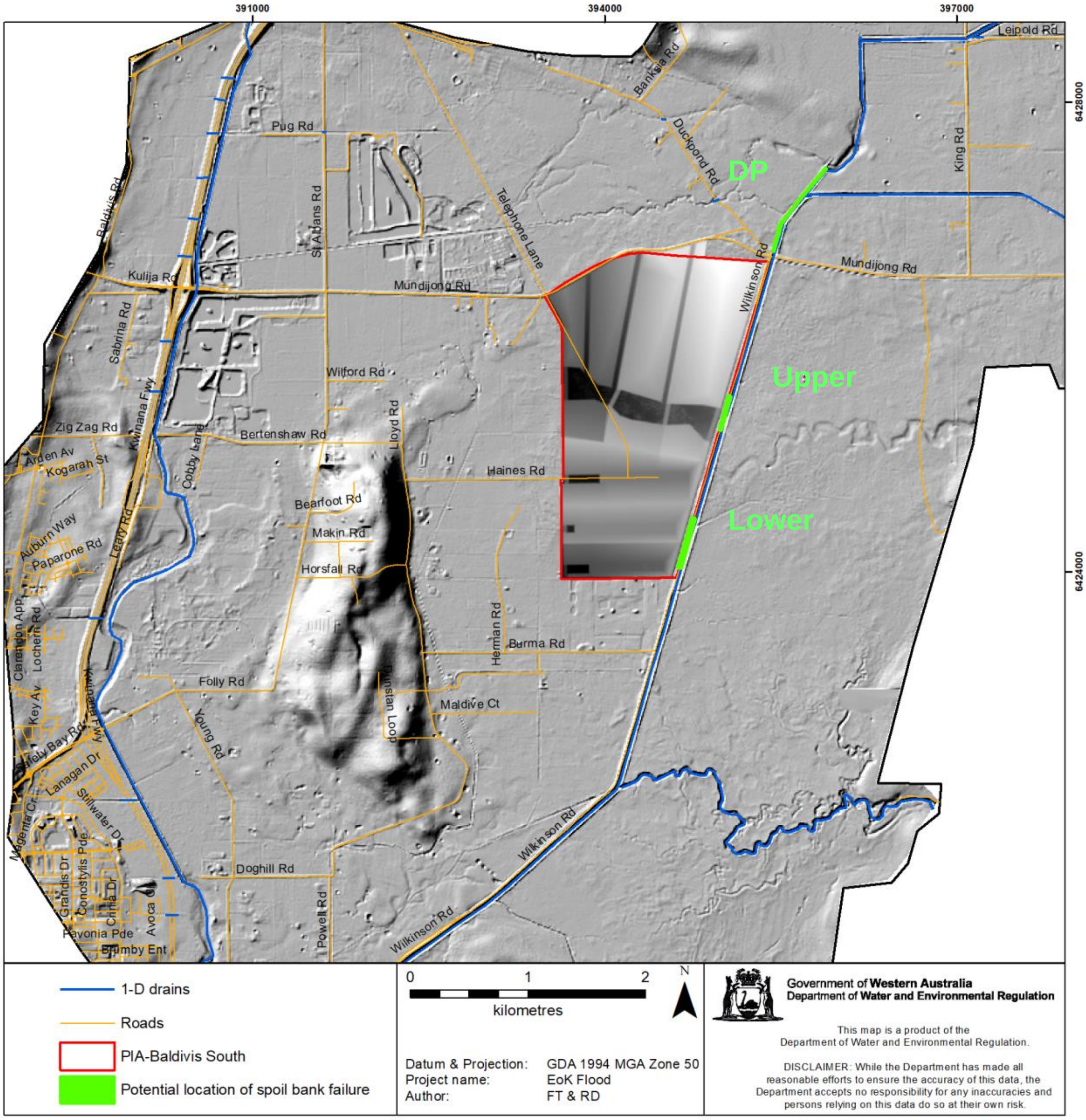


Figure 8-1 Hypothetical bathymetry of the options for the development scenario on North East Baldivis south of Mundijong Road precinct along with susceptible locations of spoil bank failure

8.1 Scenario 2 pre-development base case

The location of the spoil bank failure adjacent to the North East Baldivis south of Mundijong Road precinct was carefully investigated.

The department's spoil bank failure assessment concluded that two locations were likely to fail along the Birrega Main Drain south of Mundijong Road, as shown in Figure 8-1. The characteristics of these locations are:

- upper section: about 260 m bank (from chainage 18320 to 18580)
- lower section: about 440 m bank (from chainage 19400 to 19840).

Secondly, we investigated the discharges from the Birrega Main Drain to the North East Baldivis south of Mundijong Road precinct with an absent right bank at these locations, along with the flood levels in this area.

A combination of scenarios of spoil bank failure was conducted, which are given in Table 8-1.

Table 8-1 Overview of spoil bank failure scenarios along the Birrega Main Drain south of Mundijong Road

Case	North of Mundijong Road	South of Mundijong Road	
	DP	Upper	Lower
0 (Intact)	Intact	Intact	Intact
1 (Upper fail)	Intact	Fail	Intact
2 (Lower fail)	Intact	Intact	Fail
3 (Both fail)	Intact	Fail	Fail
4 (Triple fail)	Fail	Fail	Fail

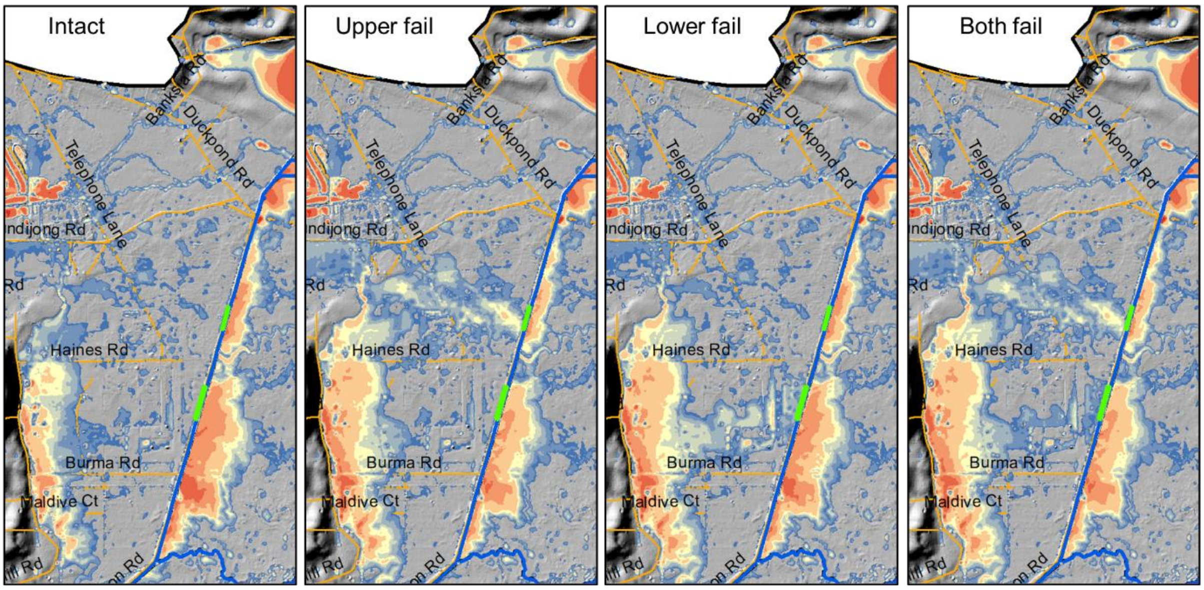


Figure 8-2 The 1% AEP flood levels around the North East Baldivis south of Mundijong Road precinct from different spoil bank failure scenarios

Figure 8-2 shows the flood maps from four scenarios, focusing on the North East Baldivis south of Mundijong Road precinct. In contrast, Figure 8-3 compares the depth difference of the flood levels between the respective failure scenarios and the Intact scenario. Some highlights from Figure 8-2 and Figure 8-3 are:

- all spoil bank fail scenarios lead to an increase in the downstream flood water depth; however, the growth is not uniformly distributed.
- all spoil bank fail scenarios would lessen the flooding near the left bank of the Birrega Main Drain near the breakout
- the upper fail is more severe than the lower fail regarding downstream flooding

- the differences between upper fail and both fail is minor regarding downstream flooding.

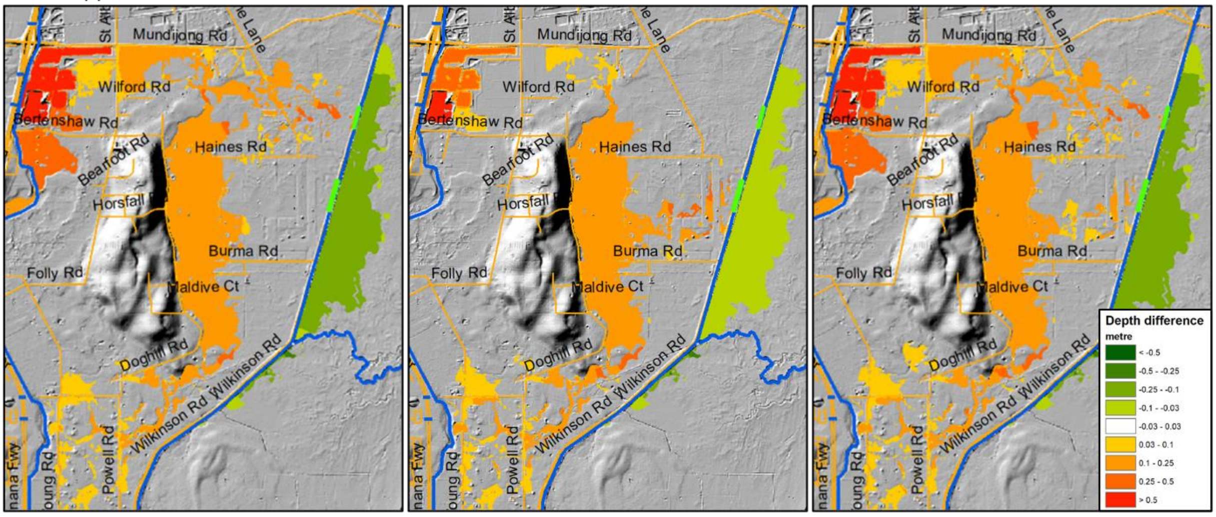


Figure 8-3 The flood difference maps between the failure scenarios and the Intact case (refer to Table 8-1 for all cases)

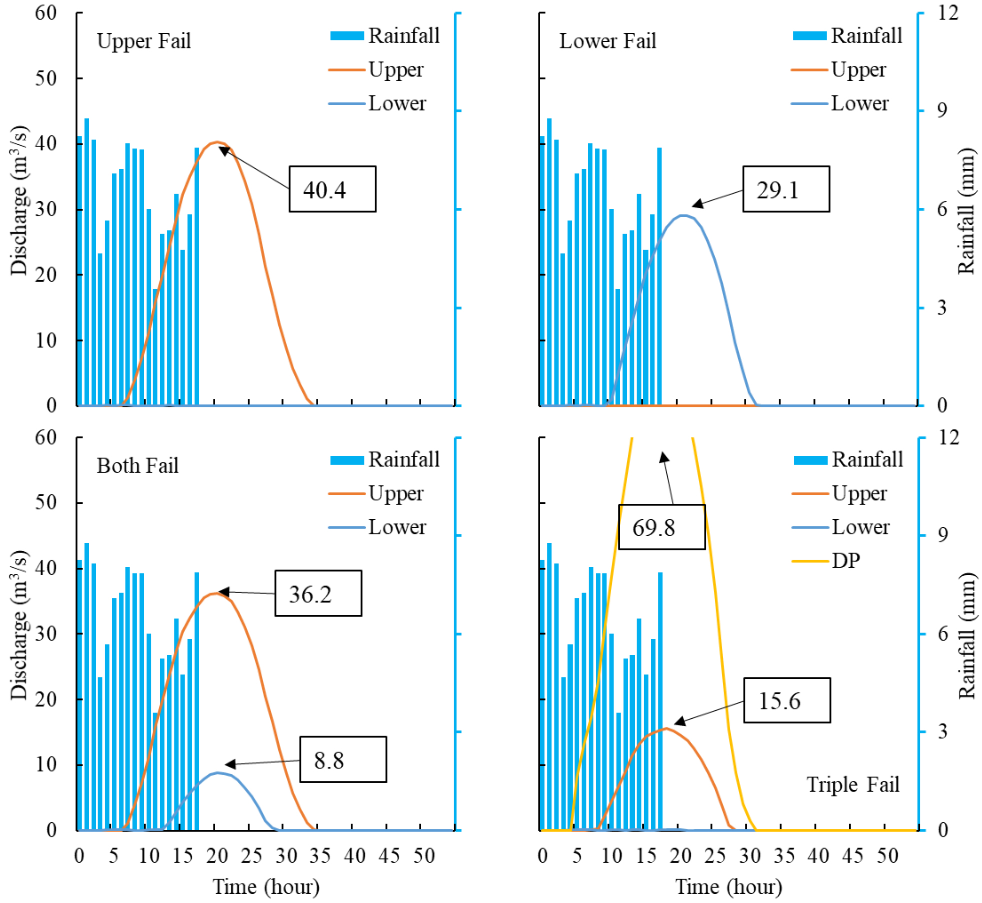


Figure 8-4 Discharges from the Birrega Main Drain measured immediately downstream the respective potential spoil bank failure locations (refer to Table 8-1 for all cases)

Figure 8-4 compares the discharges from different spoil bank fail scenarios. It demonstrates that the Upper fail scenario would introduce the most severe flooding, discharging a maximum of 40.4 m^3/s from the Drain, larger than that in the Lower fail scenario at 29.1 m^3/s, only slightly smaller than the maximum discharge from Both fail at 45.0 m^3/s. This is despite the much shorter failure length of the spoil bank in the Upper fail scenario.

Based on these results, the pre-development base case includes spoil bank failure along the Birrega Main Drain adjacent to the North East Baldivis south of Mundijong Road precinct as defined by the Upper fail scenario.

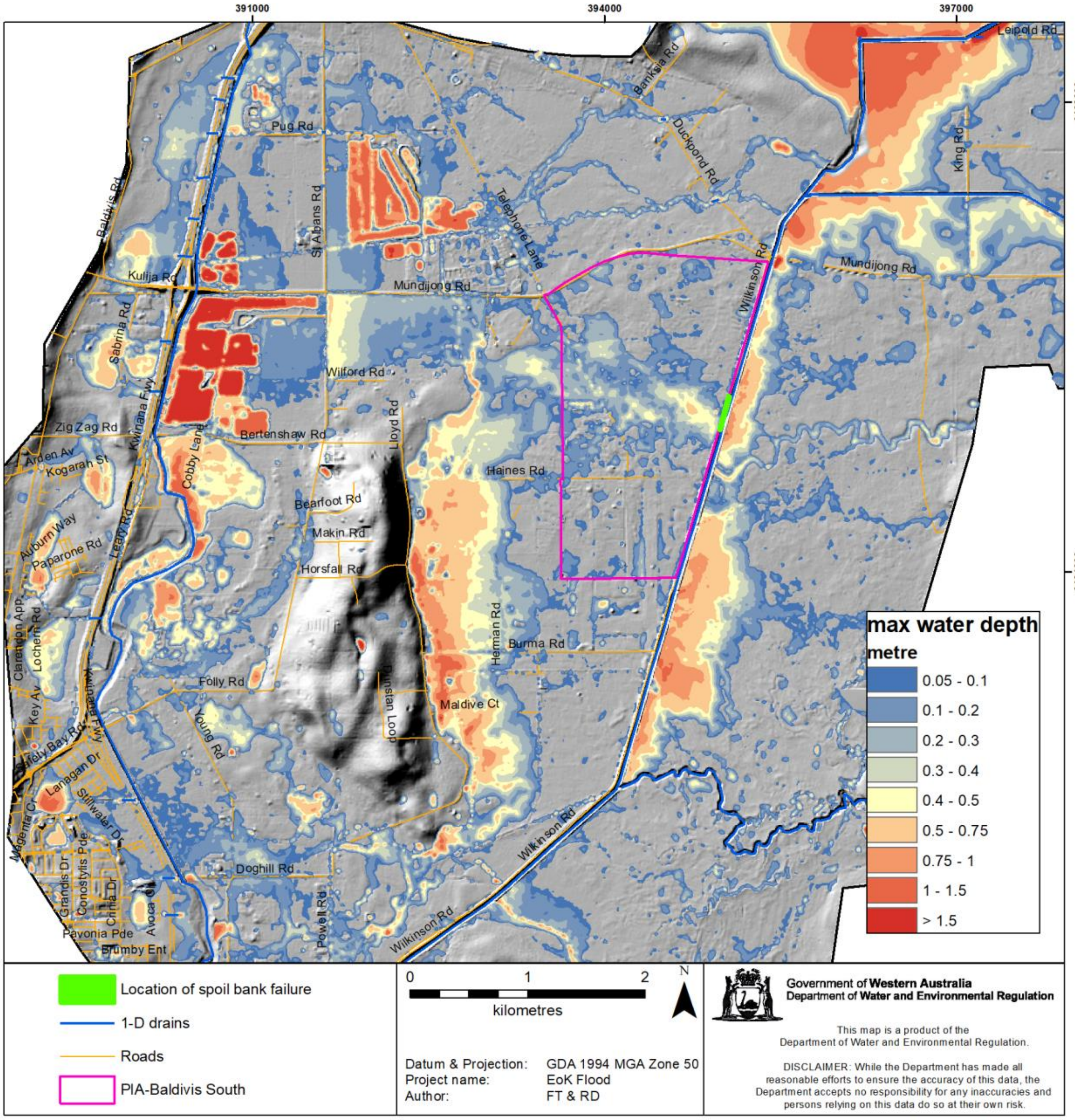


Figure 8-5 Maximum pre-development flooding water depth focusing on North East Baldivis south of Mundijong Road precinct under 1% AEP 18-hour Tp1 rainfall

Figure 8-5 illustrates the pre-development flooding map resulting from the Upper fail scenario at the 1% AEP 18-hour Tp1 rainfall. About one-third of the North East Baldivis south of

Mundijong Road precinct area is flooded with above 0.05 m floodwater, but the water depth is generally lower than 0.5 m.

8.2 Scenario 2 post-development option

Based on the above flooding modelling from the pre-development condition, an option was developed on the North East Baldivis south of Mundijong Road precinct (see Figure 8-1) by raising the land accordingly.

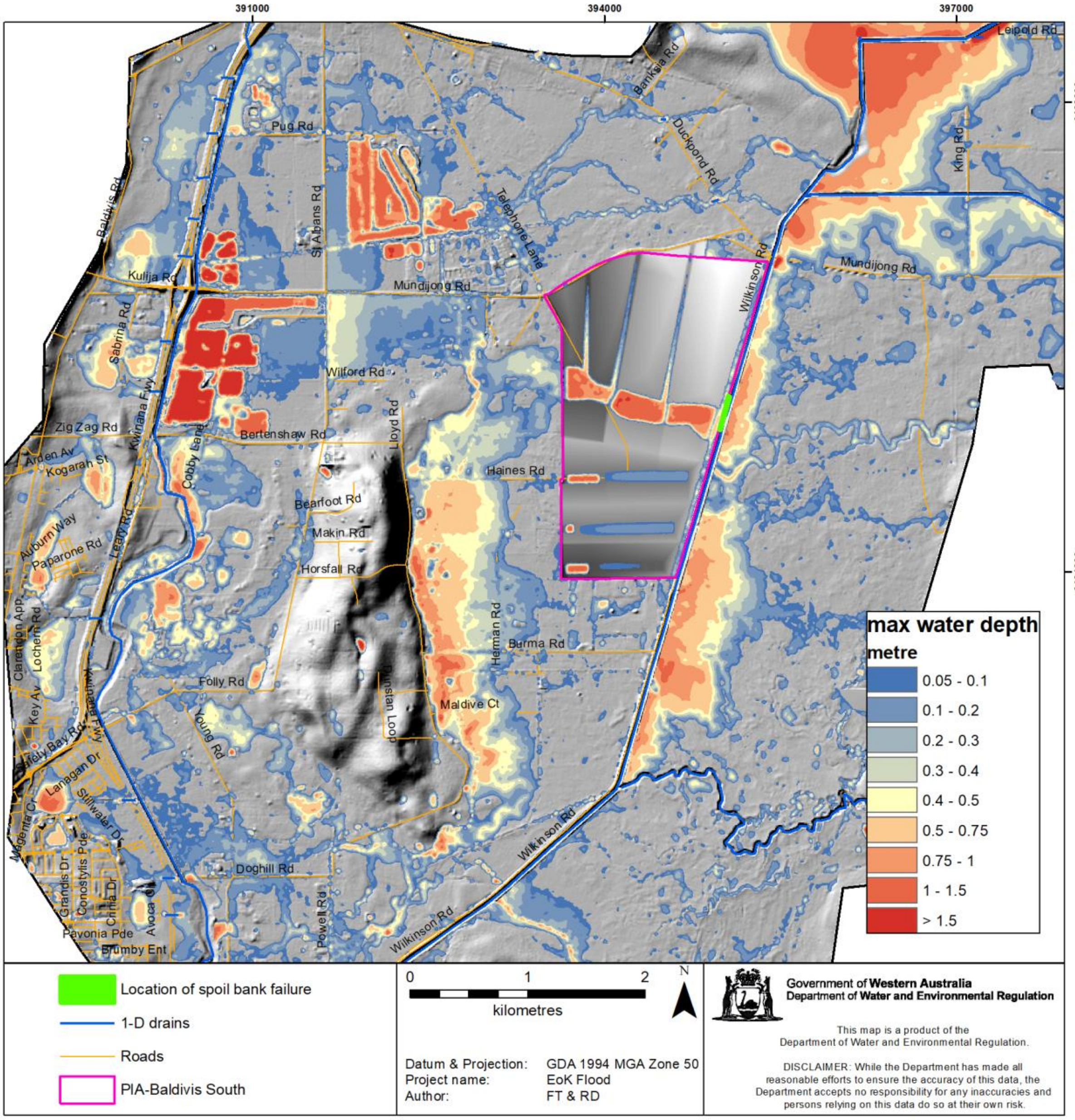


Figure 8-6 Maximum post-development flooding water depth focusing on North East Baldivis south of Mundijong Road precinct under 1% AEP 18-hour Tp1 rainfall

Figure 8-6 illustrates the post-development flooding map resulting from the Upper fail scenario at the 1% AEP 18-hour Tp1 rainfall. Figure 8-6 suggests the designed east-westward channel sufficiently stores or diverts water from the breakout and that most of the raised land is free from 1% AEP flooding, except some shallow ponds of water.

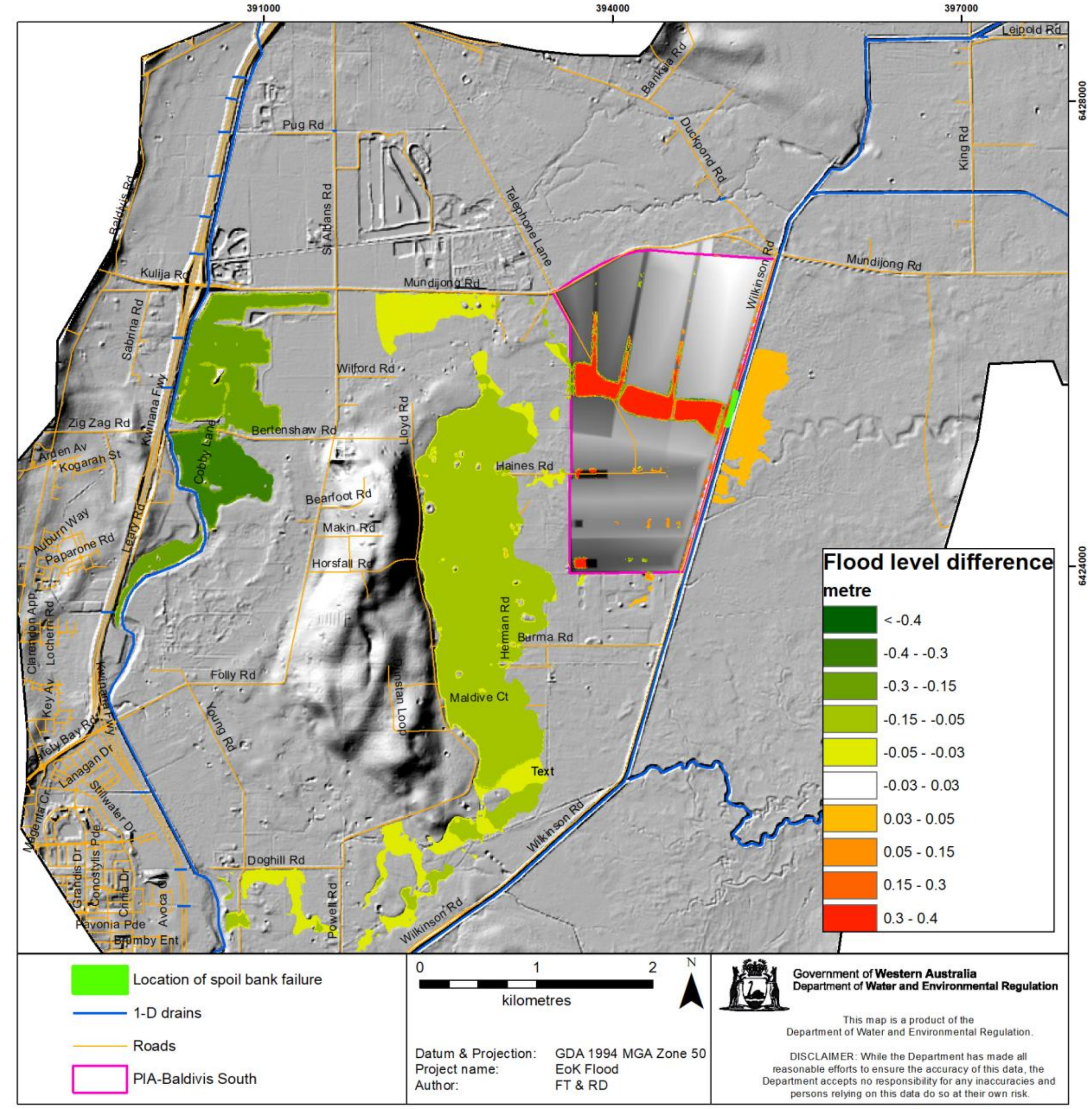


Figure 8-7 Flood level difference between the post- and pre-development conditions as given in Figure 8-6 and Figure 8-5

Figure 8-7 shows the difference of floodwaters between Figure 8-6 and Figure 8-5, the post- and pre-development conditions. It highlights:

- a significant area on the east of Dog Hill would experience a 0.05 ~ 0.15 m decrease in floodwater depth
- a noticeable area bordered by Mundijong Road, Kwinana Freeway, Folly Road, and St Albans Road would experience a decrease over 0.15 m in floodwater depth
- dan area on the left bank near the spoil-bank failure location would experience an increase of less than 0.05 m in floodwater.

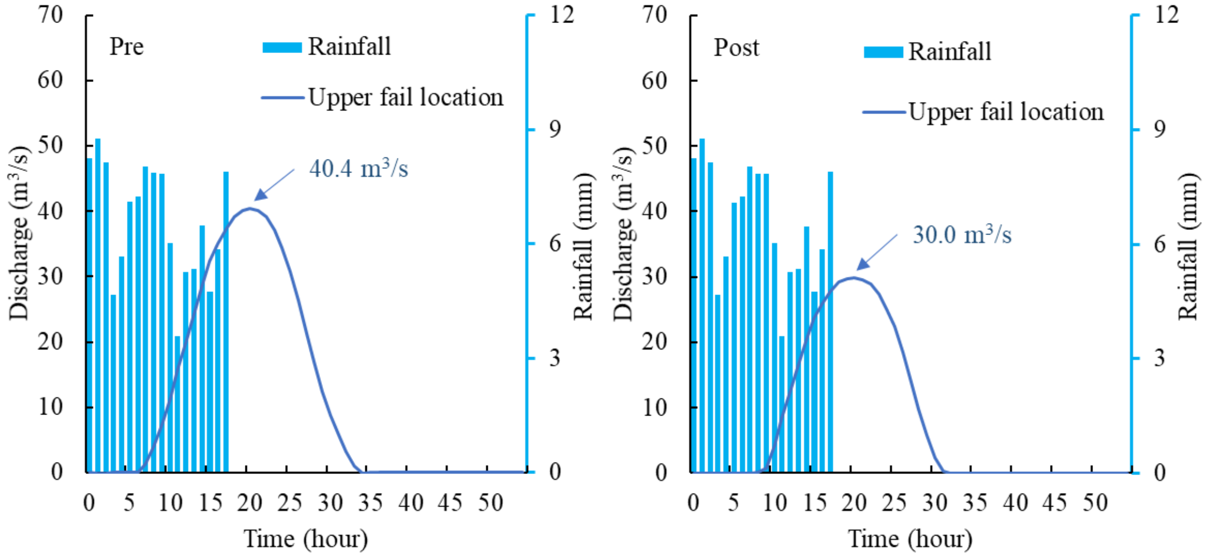


Figure 8-8 Comparison between discharges from the Pre- and Post-development conditions measured at the Upper fail location

There are two reasons for the changes in floodwater:

- the designed east-westward channel stores some floodwater (Figure 8-7) from flowing downstream
- the development option decreases the maximum discharge from the Birrega Main Drain from 40.4 m^3/s to 30.0 m^3/s.

9 Climate change

While climate change is widely accepted, its impacts on flooding in Australia are not readily quantifiable, even with the best available information. The current industry practice in flood prediction under climate change is more speculative than definitive.

ARR 2019 acknowledges that extreme daily rainfall intensity and frequency has changed in Australia linked with global warming since the 1970s. Still, this change in rainfall pattern and intensity is based on short-term observations. Projections analysed by CSIRO and Australian Bureau of Meteorology (2007) found that the highest 1% of daily rainfalls tends to increase in the north of Australia and decrease in the south, with widespread increases in summer and autumn. For structure or infrastructure design, ARR 2019 recommends the Interim Climate Change Guideline and more detailed local studies with increased AEP rainfall depth to be applied to the design standard (including Probable Maximum Flood, PMF).

On the other hand, ARR 2019 points out that warming temperature will likely increase potential evapotranspiration and evaporation, resulting in disproportionate decreases in runoff because of a disconnection between surface and groundwater. Therefore, increases in design rainfall will not always result in increases in flood estimates.

Climate change was indirectly considered in this study through the model sensitivity study through two aspects:

- the inflows to East of Kwinana under different design rainfalls (section 6.2.6).
- the discharges from the adjacent drains to East of Kwinana under different spoil bank failure scenarios (section 6.2.3 and section 6.2.4).

The modelling of 1 in 200 (0.5%) AEP events can be used to inform potential changes in 1% AEP flows and water levels should climate change increase rainfall intensities and runoff. Similarly, modelling of 1 in 50 (2%) events can be used to inform possible reductions in 1% AEP values should climate change reduce rainfall intensities and /or runoff. The modelling of a broad range of design events between 10% AEP and 1 in 2000 (0.05%) AEP allows for similar semi-quantitative approach to testing the impact of potential climate change over the range of design events, not just the 1% AEP event.

The possibility of climate change effecting the flows and water levels also could affect the possibility of spoil bank failures. The modelling of scenarios with and without failures provides information on upper and lower bounds for these two scenarios for the design events.

10 Conclusion

This report presents regional and local flood models for East of Kwinana to assess the existing flood behaviour and how it may change under conceptual development scenarios.

Flood maps were generated by the regional model covering 232.7 km^2 on the Swan Coastal Plain, bounded by the Darling Scarp, the Wungong catchment to the north, the Kwinana Freeway, and the Bollard Bulrush Swamp. The local flood model presents the flooding condition focusing on the North East Baldivis north of Mundijong Road precinct with fine grids to enable post-development options to be stamped onto the existing bathymetry.

Some conclusions about the existing flood behaviour are as follows.

- Potential spoil-bank failure is high at the confluence of the Birrega Main Drain and the Oaklands Main Drain in the 1-in-100-year flood event. The department has determined spoil-bank failure as the base case to which any future development proposal is compared.
- The breakout discharges to the North East Baldivis north of Mundijong Road are relatively independent of the location or length of spoil-bank failure, and a representative discharge is considered
- Modelling outcomes are broadly consistent between the local and the regional flood models. Still, we demonstrated that the local model is slightly more reliable as it better resolves the overland flow paths.

The department has prepared the flood risk management land capability assessment (DWER, 2021), detailing recommendations on development options. From a flooding perspective, findings for North East Baldivis north of Mundijong Road precinct are:

- Installing an engineered levee at the potential spoil-bank failure locations (Option A and Option A+) would cause detrimental impacts along the Birrega Main Drain. Much land up to 5 km upstream and downstream of the potential failure locations would experience increased riverine flooding.
- Leaving enough flood waterways but selectively raising the land can make 55% land above the 1-in-100-year flood level (Option B and Option B+). The resultant flooding impacts the upstream and downstream land in a similar way to the pre-development condition.
- Alternatively, retaining flood waterways but raising land according to 1-in-100-year and 1-in-20-year flood standards would potentially make a total of 65% land available for development without detrimental impacts to others.

With two-thirds of the North East Baldivis south of Mundijong Road precinct deemed above the 1-in-100-year flooding constraint in the pre-development condition, over 90% of the land could be free of 1-in-100-year flooding through certain engineered work.

Appendix A Temporal patterns of rainfall

This section shows the design rainfalls' temporal patterns for selected durations generated based on ARR 2019.

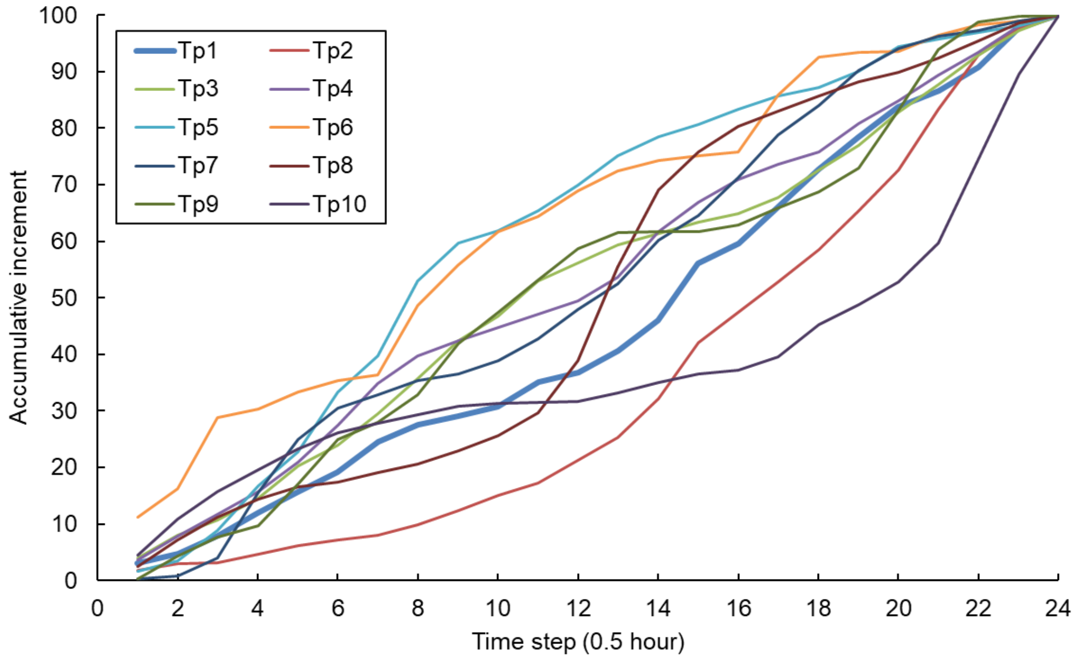


Figure A-1 Temporal patterns of rainfall for the regional model at the 12-hour event (based on ARR 2019, Region: S-SW Flatlands (West) and Area: 200 km²)

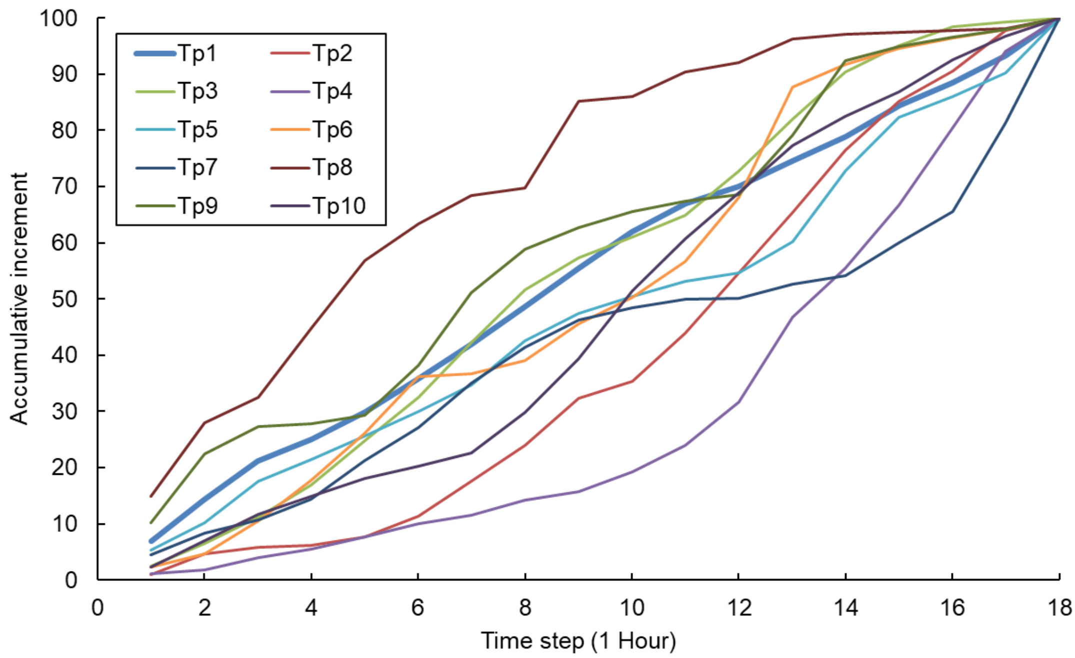


Figure A-2 Temporal patterns of rainfall for the regional model at the 18-hour event (based on ARR 2019, Region: S-SW Flatlands (West) and Area: 200 km²)

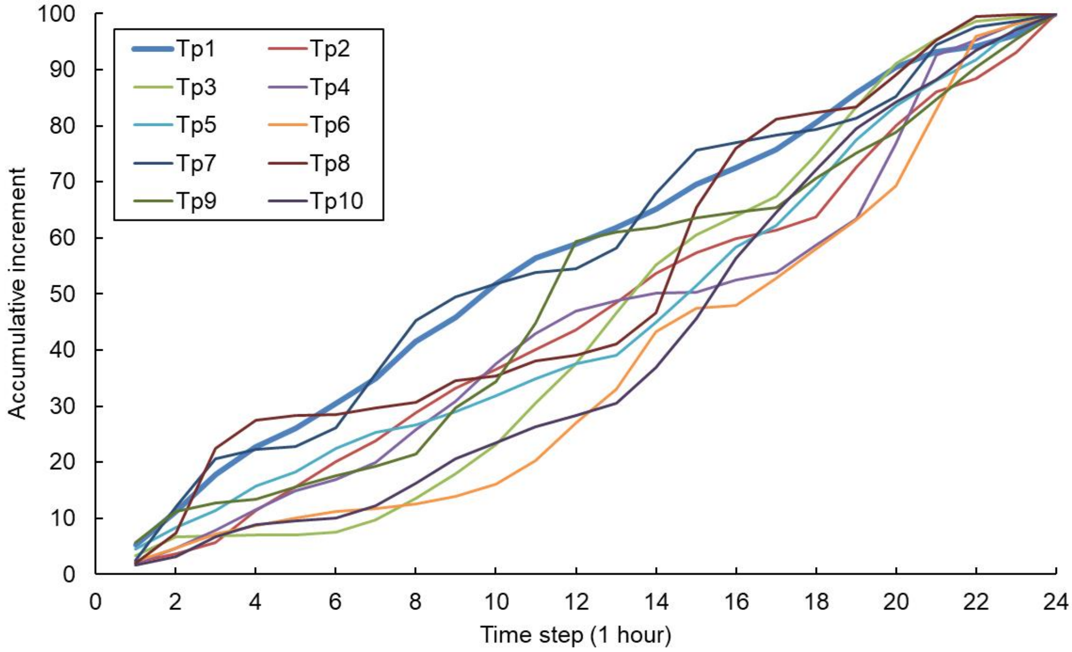


Figure A-3 Temporal patterns of rainfall for the regional model at the 24-hour event (based on ARR 2019, Region: S-SW Flatlands (West) and Area: 200 km^2)

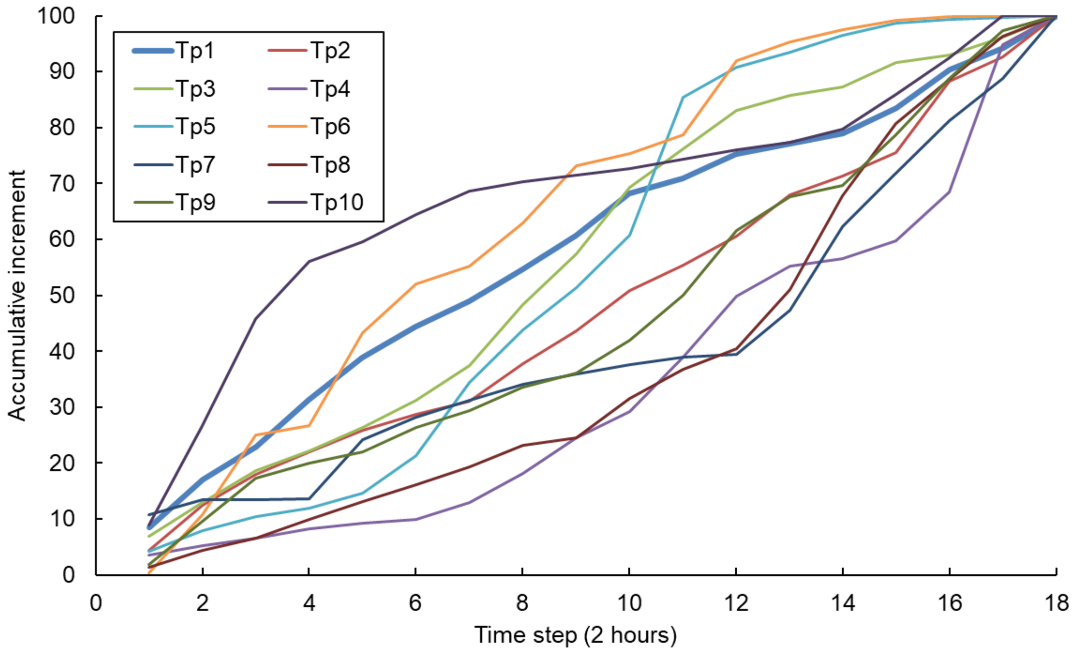


Figure A-4 Temporal patterns of rainfall for the regional model at the 36-hour event (based on ARR 2019, Region: S-SW Flatlands (West) and Area: 200 km^2)

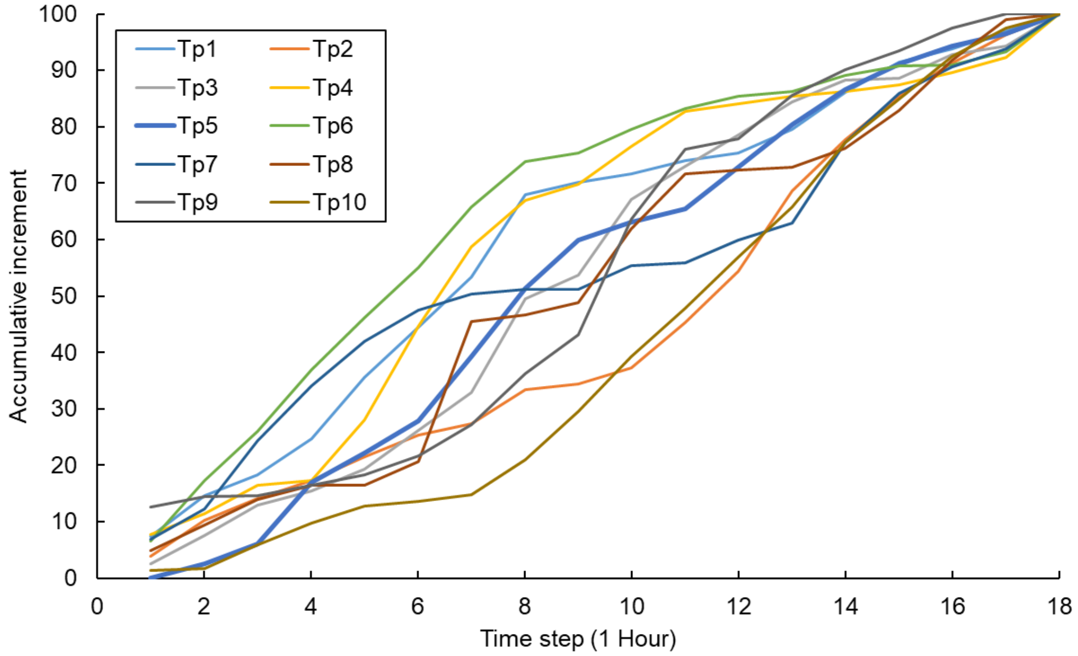


Figure A-5 Temporal patterns of rainfall for the 18-hour event (based on ARR2019, Region: S-SW Flatlands (West) and Point based rare)

Appendix B 0.5% AEP flooding maps

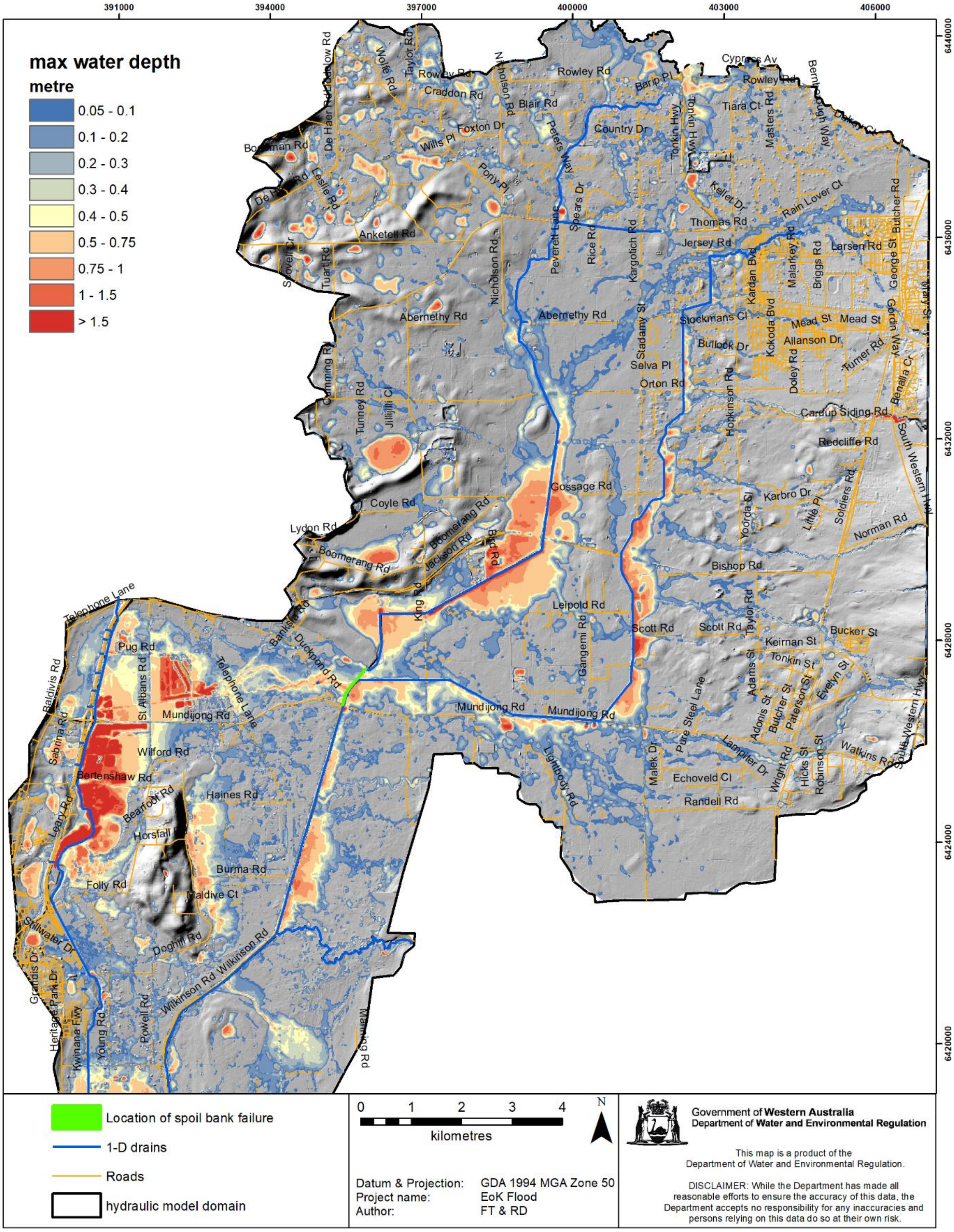


Figure B-6 Max. water depth at 0.5% AEP from 12-, 18-, 24- and 36-hr rainfalls with evenly distributed temporal patterns, which was generated by calculating the maximum floodwater depth at each grid

Appendix C 1% AEP individual flooding maps

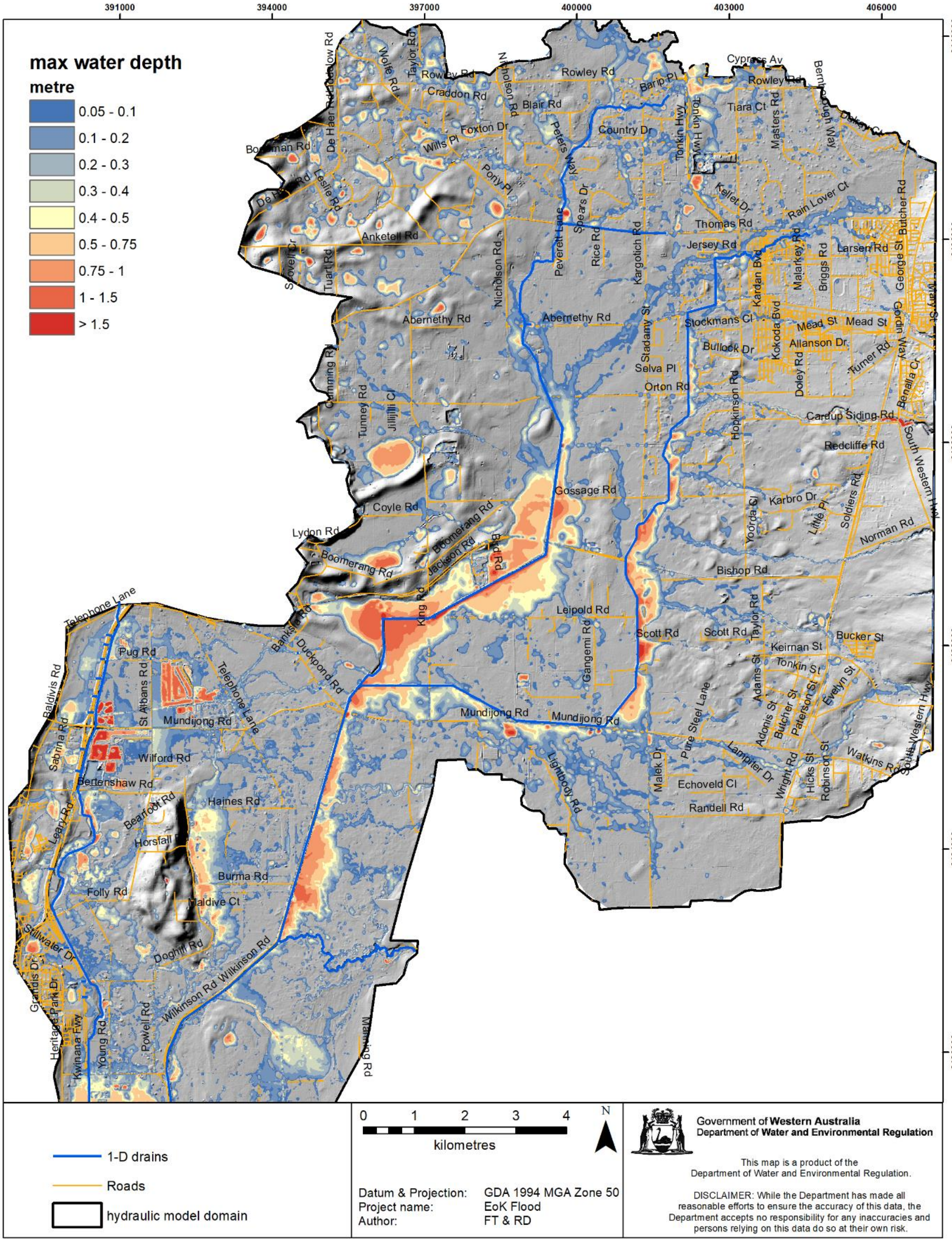


Figure C-7 Maximum flooding water depth with spoil bank intact (at 1% AEP 18-hour Tp1 rainfall)

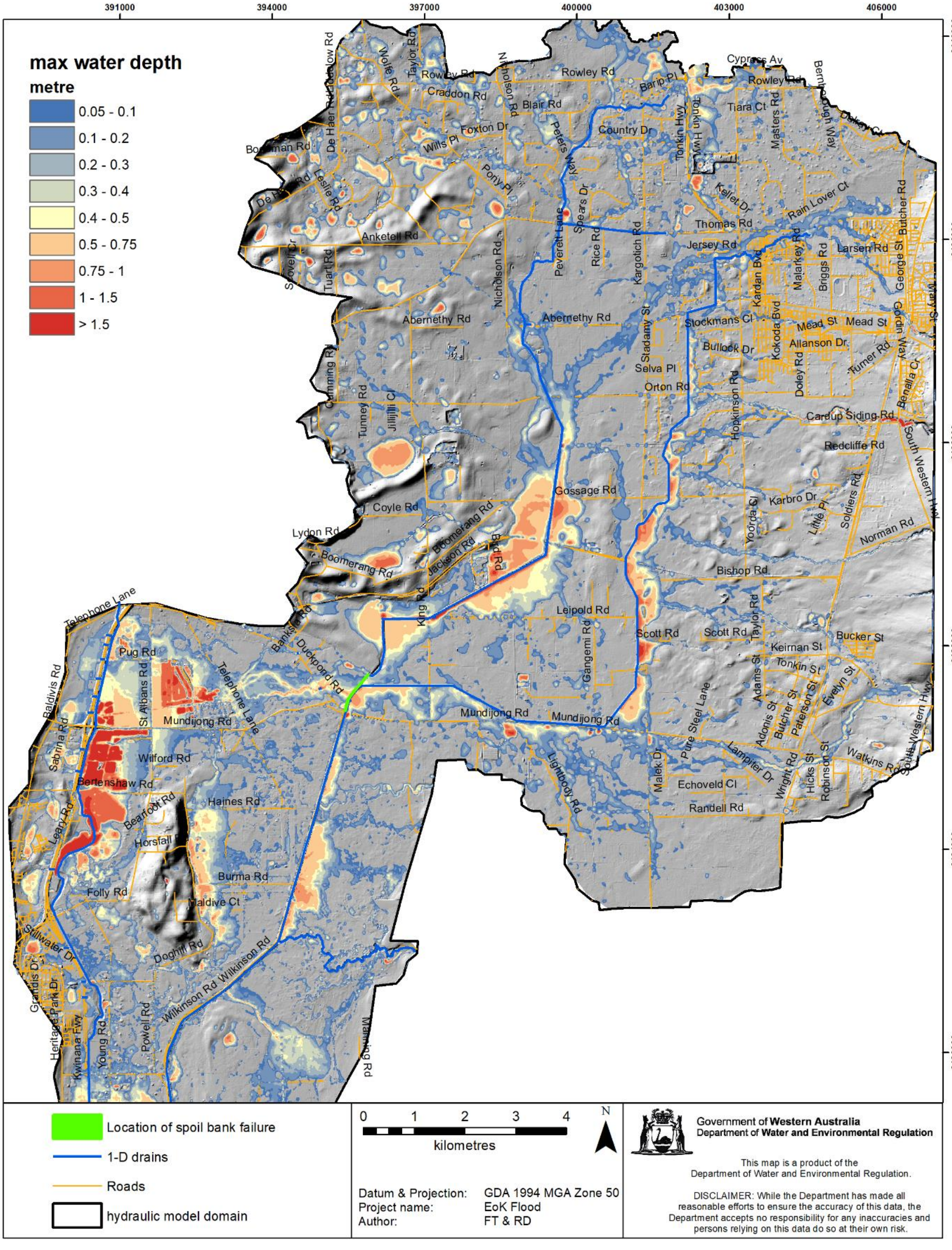


Figure C-8 Maximum flooding water depth at 1% AEP 18-hour Tp4 back-loaded rainfall (with spoil bank fail 100%)

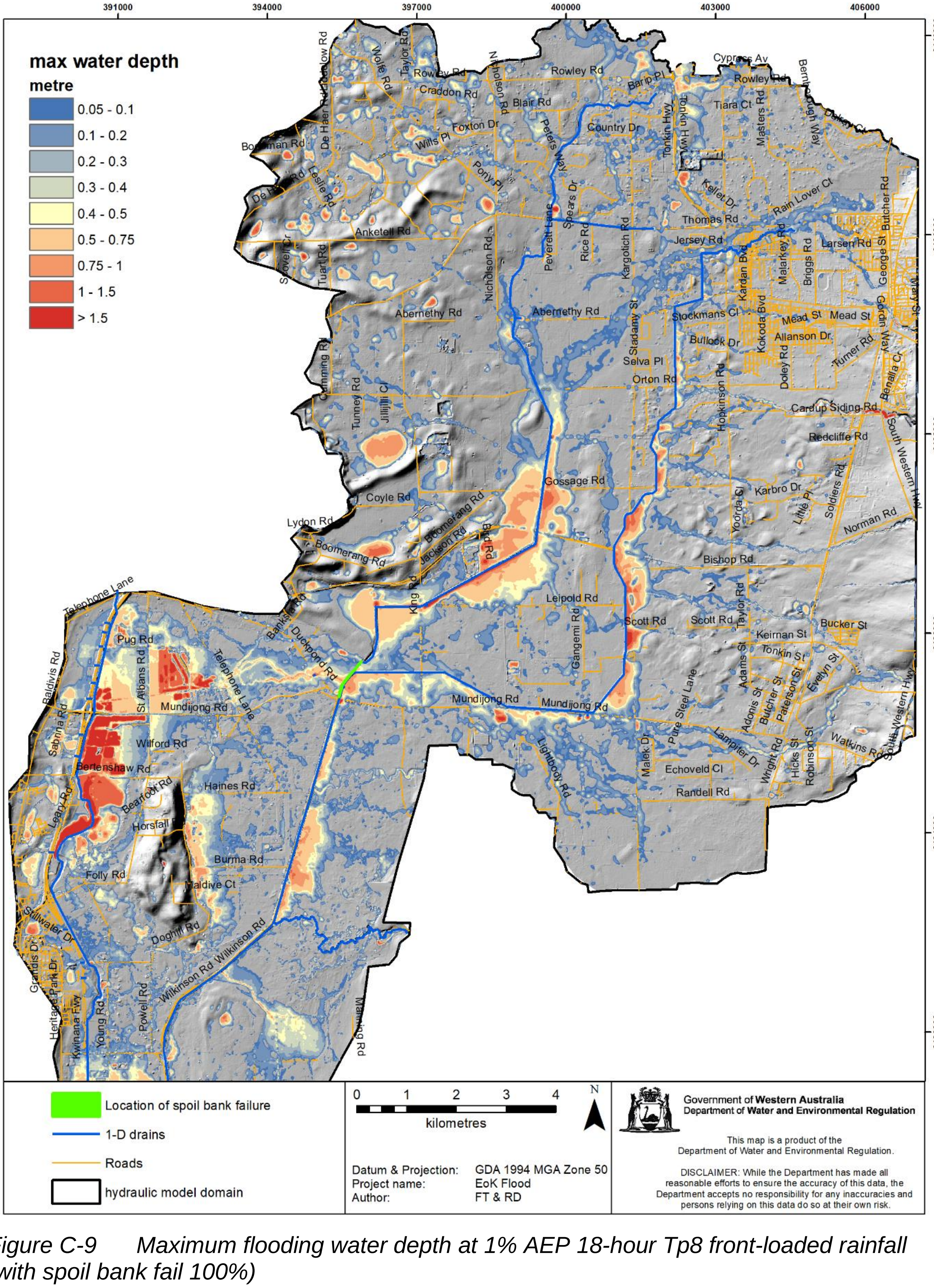


Figure C-9 Maximum flooding water depth at 1% AEP 18-hour Tp8 front-loaded rainfall (with spoil bank fail 100%)

Appendix D 5% AEP flooding maps

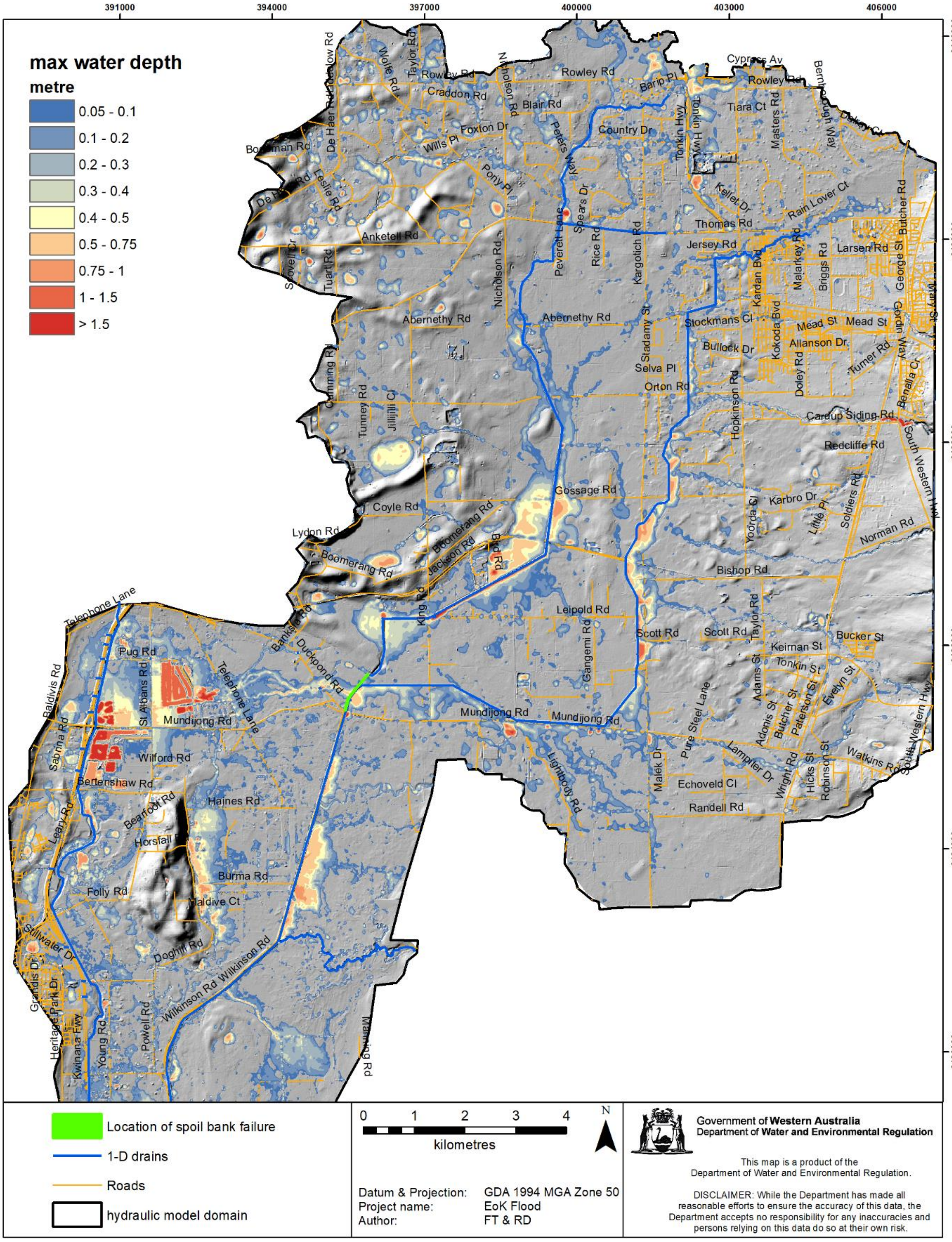


Figure D-10 Max. water depth at 5% AEP from 12-, 18-, 24- and 36-hour rainfalls with evenly distributed temporal patterns

Appendix E 10% AEP flooding maps

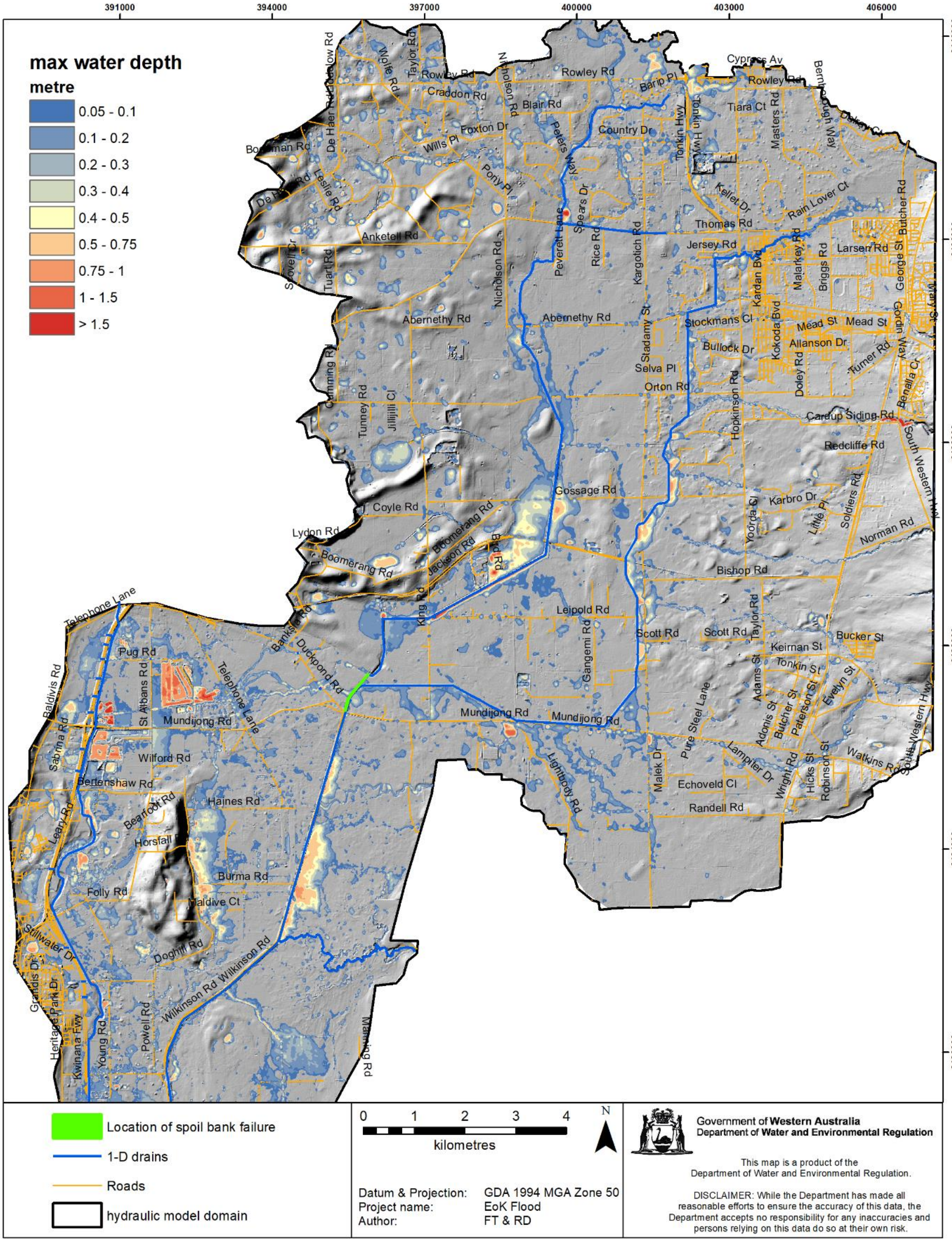


Figure E-11 Max. water depth at 10% AEP from 12-, 18-, 24- and 36-hour rainfalls with evenly distributed temporal patterns

Appendix F Flooding maps of spoil bank fail scenarios

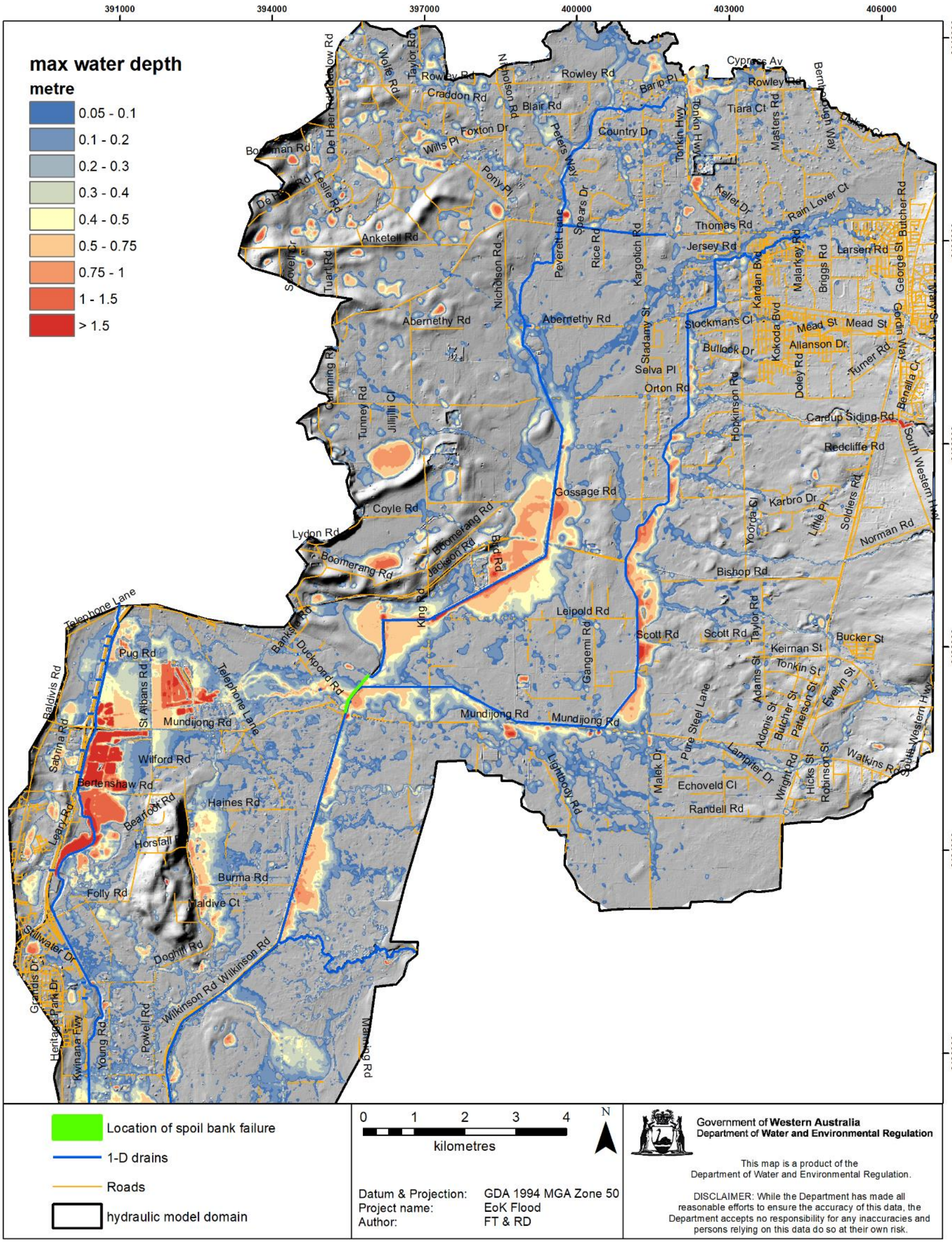


Figure F-12 Maximum flooding water depth with spoil bank fail part scenario (at 1% AEP 18-hour Tp1 rainfall)

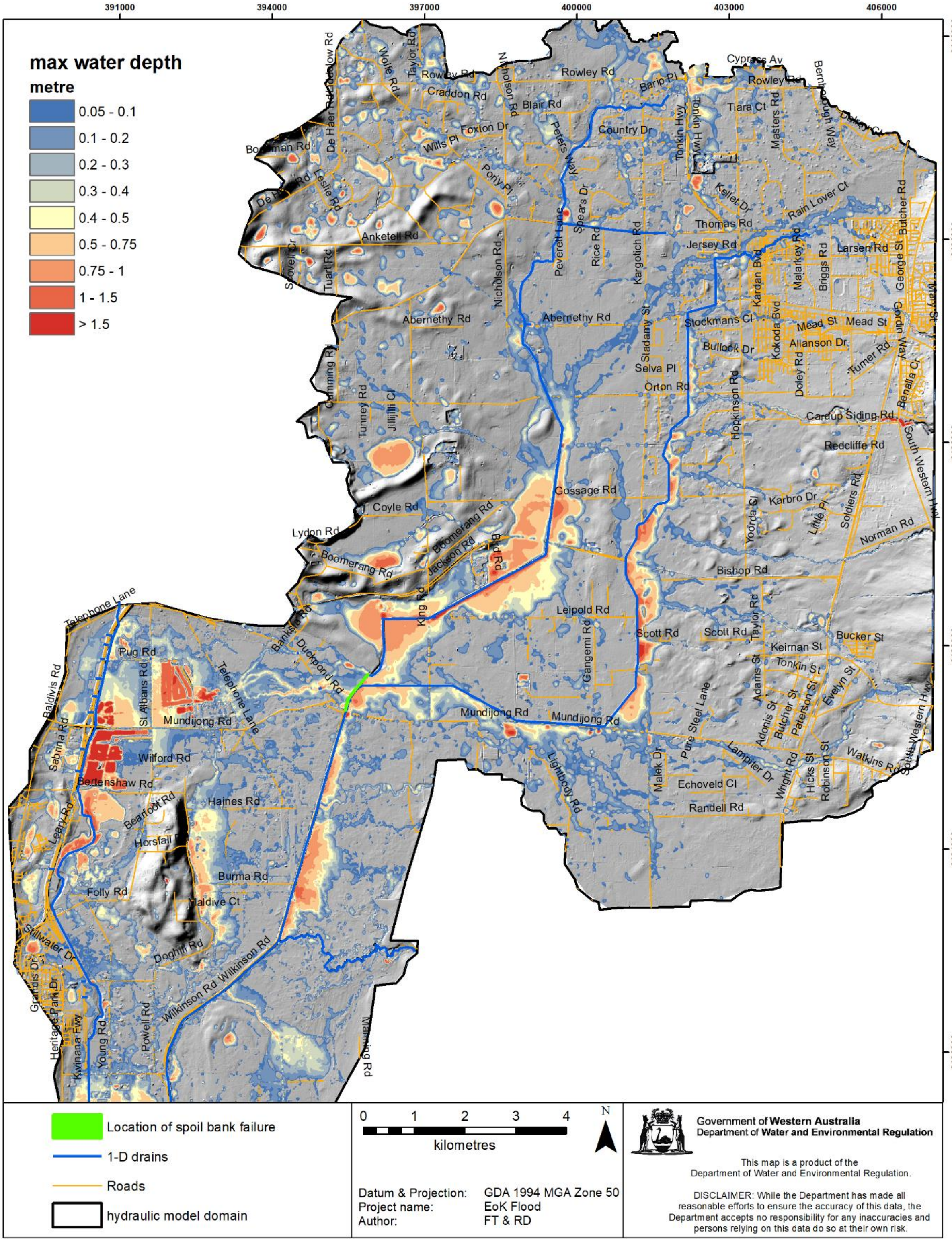
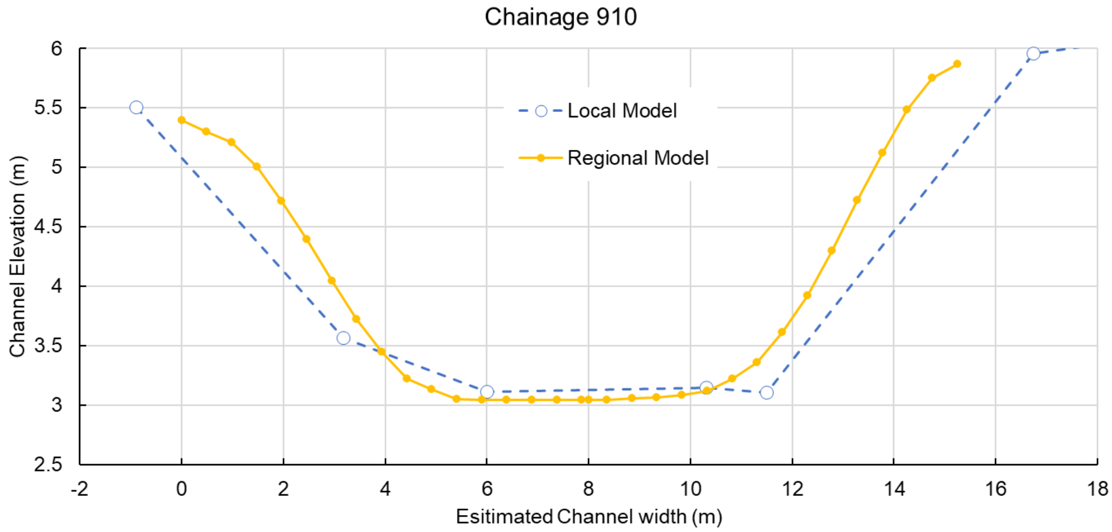
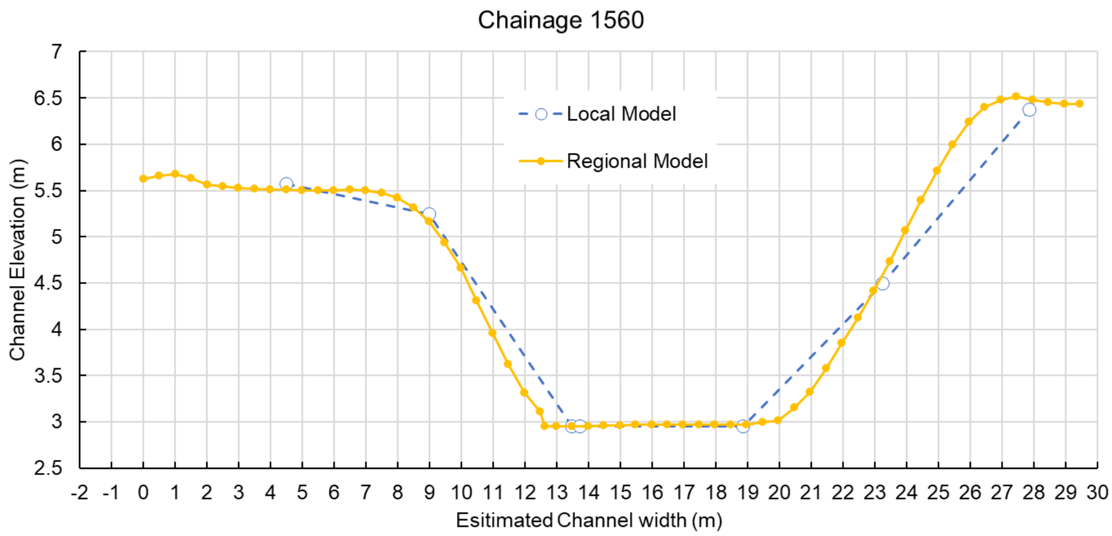


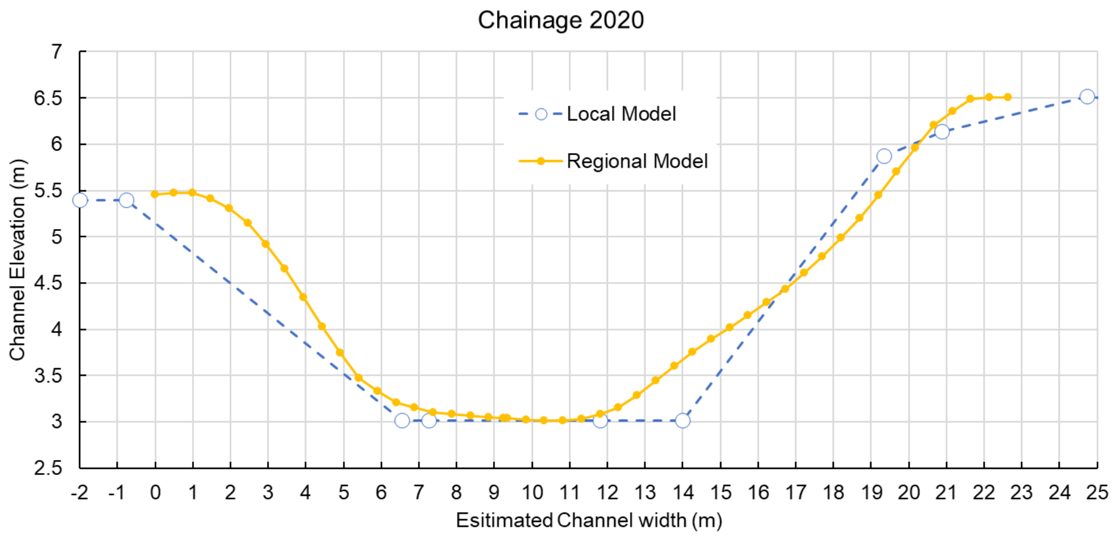
Figure F-13 Maximum flooding water depth with the build levee to 10.5 m AHD scenario (at 1% AEP 18-hour Tp1 rainfall)

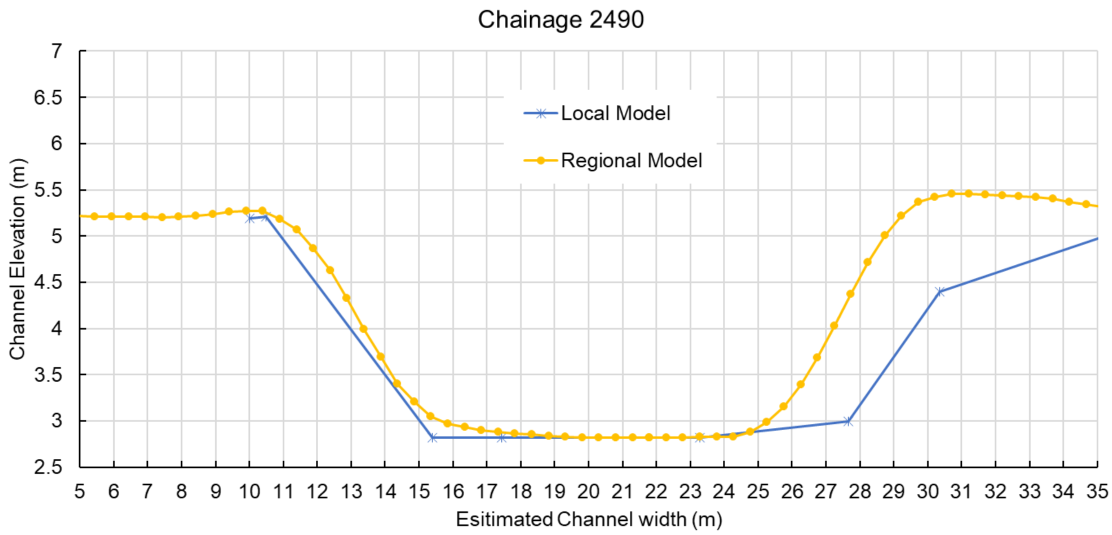
Appendix G Peel Main Drain Channel

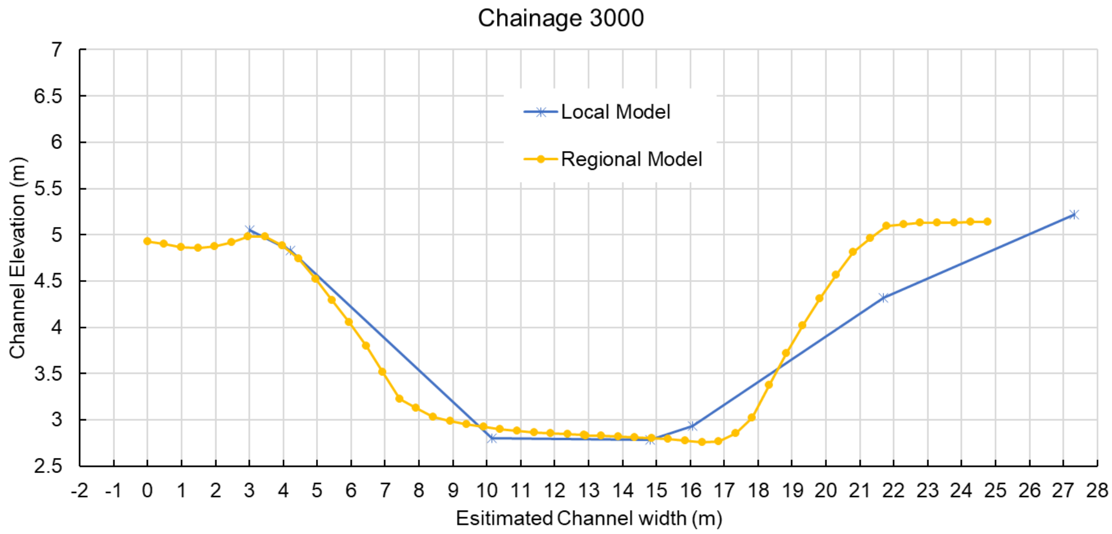
Selected channel bathymetry in the mesh of the local model (pre-development modelling), demonstrating the fine mesh used in the local model can resolve the channel without the MIKE 11 element.






Chainage 2020
Local Model
Regional Model
Channel Elevation (m)
Esitimated Channel width (m)


Chainage 2490
Local Model
Regional Model
Channel Elevation (m)
Esitimated Channel width (m)


Chainage 3000
Local Model
Regional Model
Channel Elevation (m)
Esitimated Channel width (m)

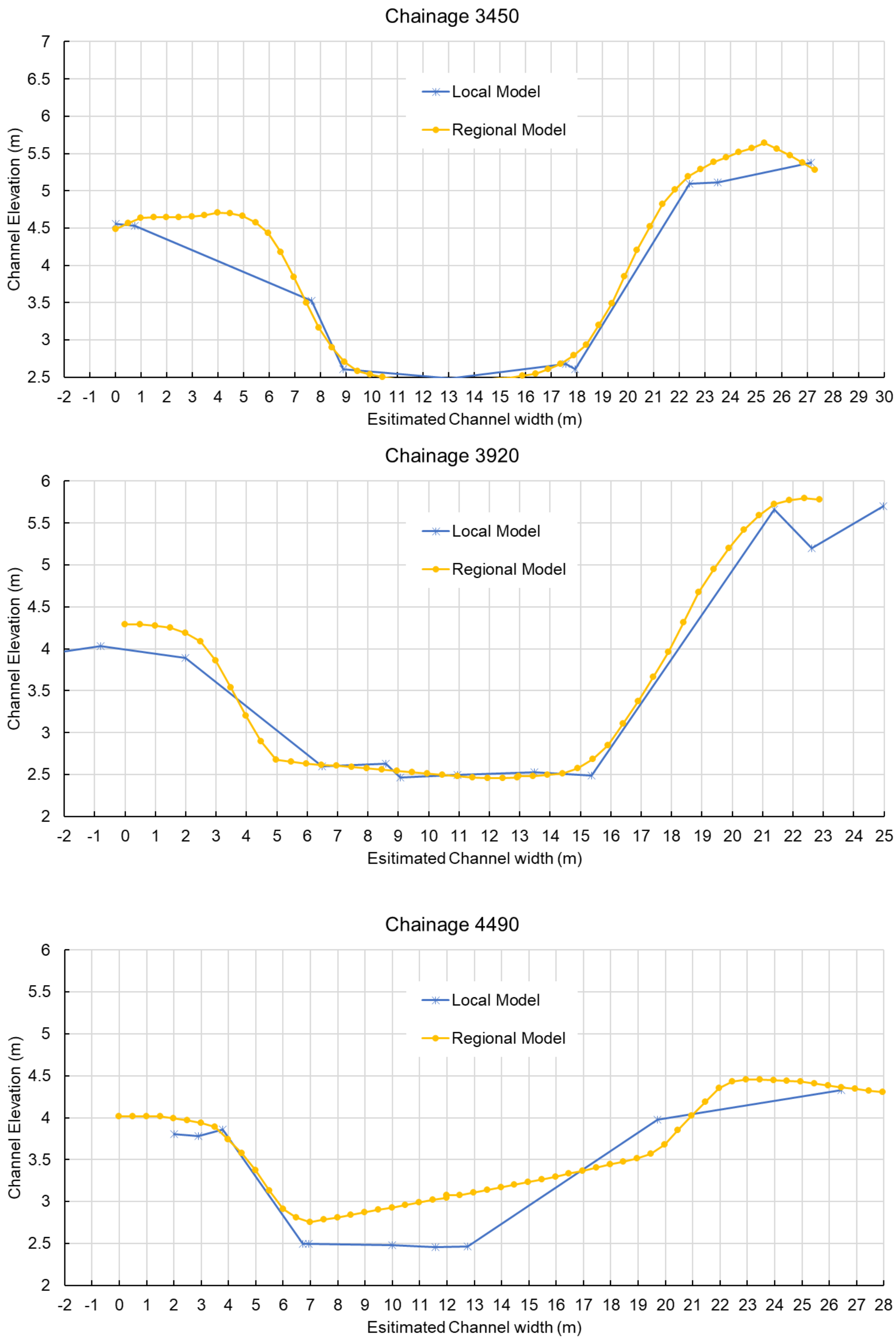


Figure G-14 Comparison of the PMD's bathymetry between the local (MIKE 21) and the regional model (MIKE 11)

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