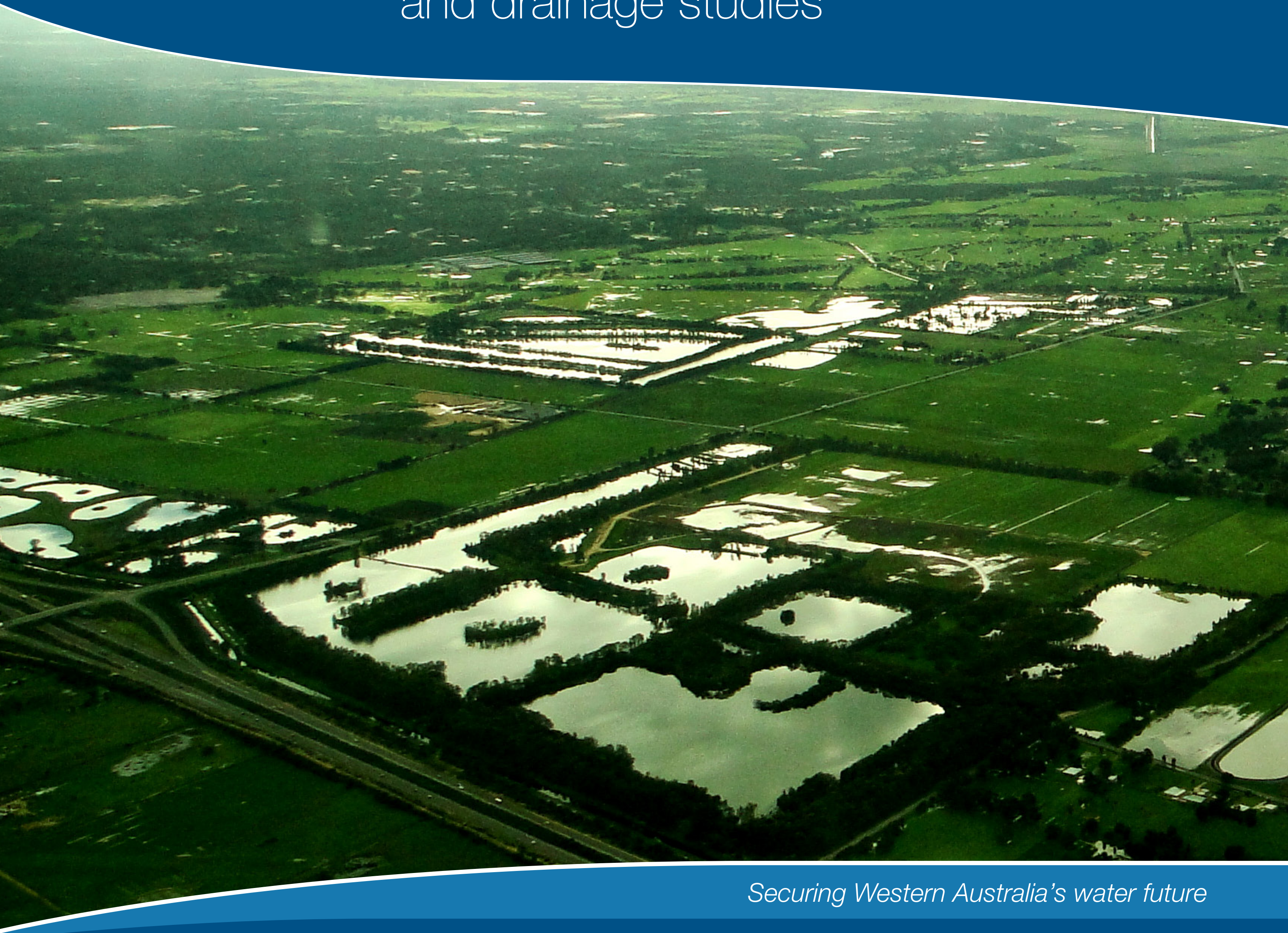
Government of **Western Australia**
Department of **Water**

North-east Baldivis flood modelling and drainage studies



Securing Western Australia's water future

Water*Science*
technical series

Report no. WST 73
August 2015

Government of **Western Australia**
Department of **Water**

North-east Baldivis flood modelling and drainage study

Supporting the North-east Baldivis drainage and water management plan

Department of Water

Water Science Technical series

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Department of Water
168 St Georges Terrace
Perth Western Australia 6000
Telephone +61 8 6364 7600
Facsimile +61 8 6364 7601
National relay service 133 677
www.water.wa.gov.au

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For more information about this report, contact Ben Marillier, Water Science Branch, the Department of Water, Western Australia.

Cover photograph: Aerial view of north-east Baldivis area, looking east, Ben Marillier, 2011

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Summary

The north-east Baldivis area was identified by the Western Australian Planning Commission as a potential site for developing a non-heavy industrial estate in the *Economic and Employment Land Strategy* (EELS; WAPC 2012). The Department of Planning requested that the Department of Water prioritise flood studies covering the potential industrial site and surrounds to identify constraints associated with flooding in the area, and provide technical information to support development of the site if required. In response the Department of Water completed the *North-east Baldivis flood modelling and drainage study*, which included development of a hydraulic flood model using MIKE FLOOD, covering a 78 km^2 area of land surrounding the area identified in the EELS. The area includes sections of the Peel, Serpentine and Birrega main drains, and a section of the Serpentine River.

This flood study assessed the pre-development site conditions within the hydraulic modelling area. Design rainfall events were simulated using the model for flows for the 5, 10, 20, 100, and 500 yr average recurrence interval (ARI) events, for durations of 6, 12, 24, 36, 48 and 72 hours. A levee failure scenario was simulated for the 100 yr design events, assuming that the western levee banks on the Birrega and Serpentine main drains were removed. The potential for overtopping the Birrega Main Drain at Duck Pool was also considered. The output of the study includes detailed 100 yr floodplain mapping, long-sections of the Peel, Serpentine and Birrega main drains, and estimates of design flood levels and discharge at multiple locations within the study area.

Several characteristics contribute to flooding risk within the north-east Baldivis area. The area is prone to regular winter inundation from shallow groundwater, the low landscape position and flat topography limit drainage potential within the site, and the area is located between the Birrega and Peel main drains. Therefore, the site can potentially be flooded through four mechanisms: groundwater inundation, direct rainfall, flooding and/or backwater from the Peel Main Drain, and levee overtopping or failure on the Birrega Main Drain.

A comprehensive review of catchment hydrology was completed to develop design rainfall and flow for the study area, and a MIKE FLOOD model was used to simulate flooding mechanisms and assess flooding potential in the north-east Baldivis area and surrounds. The channels of the Birrega, Serpentine and Peel main drains were simulated using a 1D hydraulic model, MIKE 11, which was coupled to a 2D hydraulic model, MIKE 21, which simulated the floodplain and overland flows. The model was calibrated to two flood events in 1987 and 2005 at the Dog Hill and Karnup gauging stations.

Modelling indicated that widespread shallow inundation would occur over much of the study area in a 100 yr ARI event. The most extensive inundation was located on the eastern side of St Albans road, where culverts limit discharge from east to west towards the Peel Main Drain. Areas around the water-ski park and aquaculture farm between Telephone Lane and St Albans Road are also prone to flooding, as is the eastern side of Dog Hill. Flow velocities were generally very low within the study area, outside of the main drains. Recent studies identified that in a 100 yr event it is likely that the Birrega Main Drain would overtop and/or fail in the area to the east of Duckpond Road. This scenario was simulated, and indicated that a peak discharge of 55 m^3/s would be directed through the potential industrial estate in

this event, resulting in a substantial increase in flooding throughout the site and along Peel Main Drain.

This study indicates the potential for large areas of land to be inundated in the event of a 100 yr ARI event. However, the depth and extent of flooding are less relative to the previous study completed by SKM (2010). The review of catchment hydrology undertaken for this current study identified that previous work had substantially overestimated peak flow and event volume from the Serpentine River and Peel Main Drain. The additional water resulted in an overestimate of flooding along the Peel Main Drain. In some locations, the current study has revised peak flood stage downward by up to 1 m.

Several important considerations may affect the suitability of the north-east Baldivis area for industrial development. These do not rule out development of the site but are likely to influence the feasibility of development from a technical and financial perspective. It is recommended that future drainage design at the site considers the following:

- the potential for levee overtopping or failure on the Birrega and Serpentine Main Drains
- the availability of free-draining outlets where winter groundwater levels are high and back-water effects are present
- the low hydraulic grade and capacity of existing drains within the study area
- the capacity of the Peel Main Drain to convey drainage water without influencing downstream landholders where there are breaks in the levee.

The results presented in this study should be used to inform future development and drainage design within the study area.

1 Introduction

The north-east Baldivis area was identified by the Western Australian Planning Commission as a potential site for development of a non-heavy industrial estate in the *Economic and Employment Land Strategy* (EELS; WAPC 2012). The Department of Planning requested that the Department of Water prioritise flood studies covering the potential industrial site and surrounds to identify constraints associated with flooding in the area, and provide technical information to support development of the site if required. In response the Department of Water completed the *North-east Baldivis flood modelling and drainage study*.

The study aims to address the requirements of the Department of Planning, and provide supporting information for the drainage and water management plan (DWMP) which is being developed for the Serpentine region (approximately 20 km south of Perth). The land is predominantly zoned rural, and coming under increasing pressure for urban development as WA's population increases. The study area experiences periodic flooding and one of the major components of the DWMP is the completion of a floodplain development strategy for the major waterways in the area.

This floodplain development study comprises two major components:

1. **Floodplain study:** The floodplain study involves the development and calibration of a floodplain model and subsequent floodplain mapping based on simulation of a range of design storm events.
2. **Floodplain management strategy:** The floodplain management strategy is based on the detailed hydrologic and hydraulic modelling of the investigation area. The strategy was derived using risk-based floodplain management principles, with a hazard assessment of the existing conditions flooding, and principles of design based on flood modelling scenarios.

This report outlines the floodplain study component of the project. The study covers the north-east Baldivis area, and the land surrounding Dog Hill, between Millar Road in the north and Karnup Road in the south (Figure 1-1). The floodplain management strategy will be incorporated within the DWMP.

Figure 1-1: Location of the flood study area and north-east Baldivis

1.1 Scope

The north-east Baldivis flood study includes the following components:

1. **Catchment hydrology** to determine design flows for the 5, 10, 20, 50, 100, 200 and 500 yr average recurrence interval (ARI) design flows for the upper Peel Main Drain and Serpentine River. Design flows were developed using the two techniques:

    a. Flood frequency analysis

    b. Hydrological catchment modelling

    Outflow hydrographs from the upper Peel Main Drain and the Serpentine River at the hydraulic model boundary were calculated using the rainfall-runoff model RORB (Laurenson et al. 2010). A combination of regional parameters and calibrated parameters were used in the RORB model. The RORB models were used to simulate design flows for the upper Peel Main Drain and Serpentine catchments using the Australian Rainfall and Runoff (AR&R1987 – Pilgrim 2001) rainfall intensity frequency duration (IFD) data. The Birrega and Oaklands hydraulic model (Hall 2014) was used to provide boundary conditions for the Birrega Drain and surrounds.

2. **Floodplain mapping within the study area** through development of a hydraulic model based on digital terrain data. Hydraulic model development involved the following:

    a. Development of a MIKE FLOOD hydraulic flood model within the study area

    b. Collection of data required for hydraulic modelling including stage height, and bridge, culvert and channel dimensions

    c. Collection of historic and anecdotal river stage information

    d. Use of historic flood information for the calibration of the hydraulic model

    e. Producing 5, 10, 20, 100 and 500 yr ARI floodplain mapping within the study area, including plan and long section drawings for specified channels

    f. Incorporation of hydraulic boundary conditions and levee break scenarios from the Birrega and Oaklands MIKE FLOOD model (Hall 2014)

    g. Producing flood animations for the different ARI flood events.

3. **Recommendations for drainage planning:**
    This component of the study involved identification of the major natural flow paths within the development areas; calculation of peak discharge rates and event volumes for all major flow paths within the study area; and identification of important considerations for drainage planning in the area.

A floodplain development strategy which delineates the floodplain mapping will be produced. The strategy will make recommendations for the location of ‘safe’ areas within existing and future developments for resident evacuation plans for flood events in excess of 100 yr ARI. A hydraulic assessment of the flood hazard will be conducted to understand the variation in flood hazard over the investigation area.

1.2 Catchment and drainage

The study area for the hydraulic model covers 78 km^2 of the Swan Coastal Plain, adjacent to the Kwinana Freeway, south of Bollard Bulrush Swamp. The broader study area includes the catchments of the upper Peel Main Drain, the Birrega Main Drain, and the Serpentine River/Drain. Figure 1-2 shows the hydrological features of the study area.

Peel Main Drain

The upper section of the Peel Main Drain is external to the hydraulic study area, with a catchment area of 58 km^2. The entire catchment of the Peel Main Drain is located on the Swan Coastal Plain, and includes a variety of land uses, with the lower section of the catchment comprised of mostly rural residential and grazing land. The upper section of the 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 area of the hydraulic model includes the lower section of the Peel Main Drain and consists of sections of trapezoidal drain, divided by the Folly and Maramanup pools. The Peel Main Drain eventually joins the Serpentine River around 3 km south of Karnup Road.

In its upper reaches, the Peel Main Drain intersects a series of wetlands including Mandogalup Swamp and the Spectacles Wetlands. These wetlands intersect superficial groundwater in winter, receive a baseflow contribution in most winter events, and also provide large storage areas along the drain, effectively reducing peak flows and increasing rainfall response times. Within the hydraulic model area, the Peel Main Drain has a very low hydraulic grade, dropping only 2 m over 11 km. There are several large storage areas within and adjacent to the drain. Similar to the upper reaches of the drain, these two factors lower peak flow velocities, and slow the response time of the drain to rainfall. As such, rainfall events which produce large peak flows in waterways which have their headwaters in the scarp may produce only a small response in the Peel Main Drain.

The trapezoidal sections of the Peel Main Drain are generally between 20 and 25 m wide, including levee banks, most pronounced below Karnup Road, which are 1–2 m higher than the surrounding land. The drain receives inflows from several smaller Water Corporation drains which extend through Baldivis, around Dog Hill, and the western side of the freeway. These drains convey flow from the surrounding farmland, which has a fine network of privately owned drains. The Water Corporation drains collect flow from the agricultural drains and discharge into the Peel Main Drain through culverts or breaks in the levee bank. Figure 1-3 shows lateral drainage to the Peel Main Drain, rock reinforcement at a lateral discharge point, Maramanup Pool, and culverts at Karnup Road.

Rural subdrains

Within the Peel Main Drain catchment several rural subdrains (shown as Water Corporation drains in Figure 1-2) traverse rural properties and convey water to the main drain. These typically have low capacity (in some cases < 6 months ARI) and are likely to overtop in flood events.

Several subdrains within the study area direct flow to the Peel Main Drain:

- The drain running east-west through the north-east Baldivis area, parallel to Pug Road. This drain conveys water from the eastern side of Telephone Lane towards the Peel Main Drain.
- The drain running east-west through the north-east Baldivis area, parallel to and 500 m north of Mundijong Road. This drain conveys water from the eastern side of Telephone Lane towards the Peel Main Drain.
- The long drain running from the eastern side of Dog Hill through Haines and Burma Roads, south along Powell Road, and the west along Serpentine Road to Peel Main Drain.

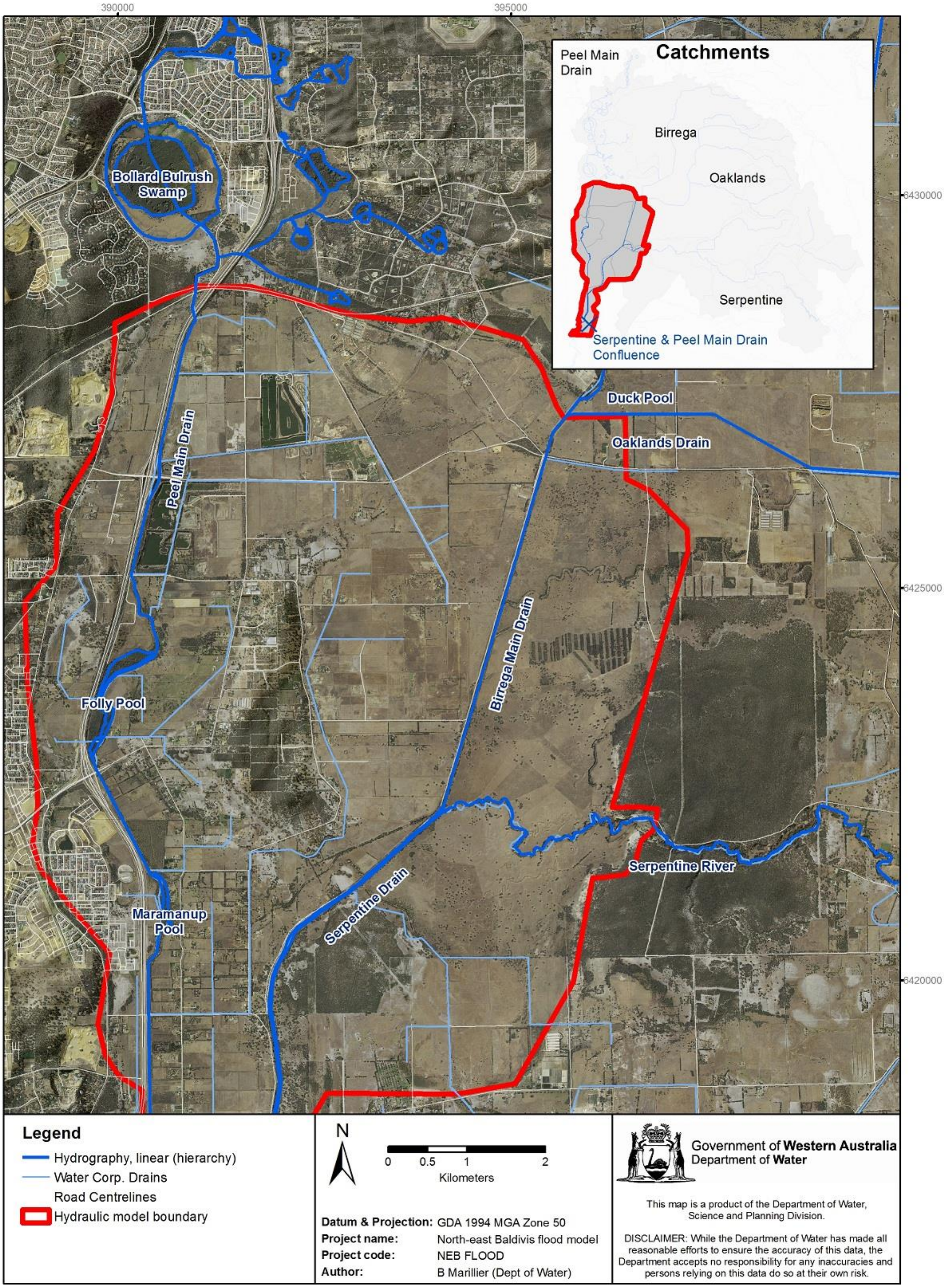


Figure 1-2: Hydrological features of the floodplain study area

Figure 1-3: Clockwise from top left, lateral drainage from Baldivis, rock reinforcement at a lateral discharge point, Maramanup Pool, culverts at Karnup Road

Birrega Main Drain, Serpentine River/Drain

Under low flow conditions, runoff from the Darling Scarp in the east is directed through the Serpentine Drain and the Birrega Main Drain. Both drains are considerably larger than the Peel Main Drain and have much higher peak capacities. In the event of levee bank failure or drain overtopping, overland water would flow from east to west through the Baldivis area and around Dog Hill, and so it is necessary to include these drains and associated off-channel storage areas within the hydraulic model.

Within the study area, the Birrega Main Drain, running from north to south, is a large trapezoidal drain 40–50 m wide within the model area. Levee banks, generally 1.5–2 m high, on both sides of the drain are continuous on the western side and have small breaks for lateral drainage on the eastern side. The condition of the levee banks on the western side varies, with several distinct low points where the drain would preferentially spill in sufficiently large events. Previous studies (SKM 2010; Hall 2014) have identified that the large storage areas along the Birrega Drain and the low-lying areas on the eastern side also have large storage potential, so it is important that they are included within the hydraulic modelling domain to correctly simulate flood behaviour. The Birrega Drain joins the Serpentine River, and becomes the Serpentine Drain which eventually connects to the Peel Main Drain to the south of Karnup Road.

The Birrega catchment, which is external to the flood study area, totals 241 km^2. Most of the catchment area is located on the coastal plain, with a smaller portion located within the Darling Scarp. The Birrega Drain receives inflows from Oaklands Drain just outside the study area, near Mundijong Road, and the drain widens at his point. Figure 1-4 shows the Birrega Main Drain at Mundijong Road.

Figure 1-4: Birrega Main Drain facing north (left) and south (right)

Downstream of the confluence of Birrega Main Drain and the Serpentine River, the Serpentine Drain continues south, with a slight widening of the channel to 60 m in the same trapezoidal shape. Similar to the Birrega Drain, the channel has levee banks generally 2 m high along the length of the drain, with breaks in the eastern bank to allow lateral inflows. The western levee bank prevents the Serpentine Drain from spilling to the west and directing overland flow towards the Peel Main Drain.

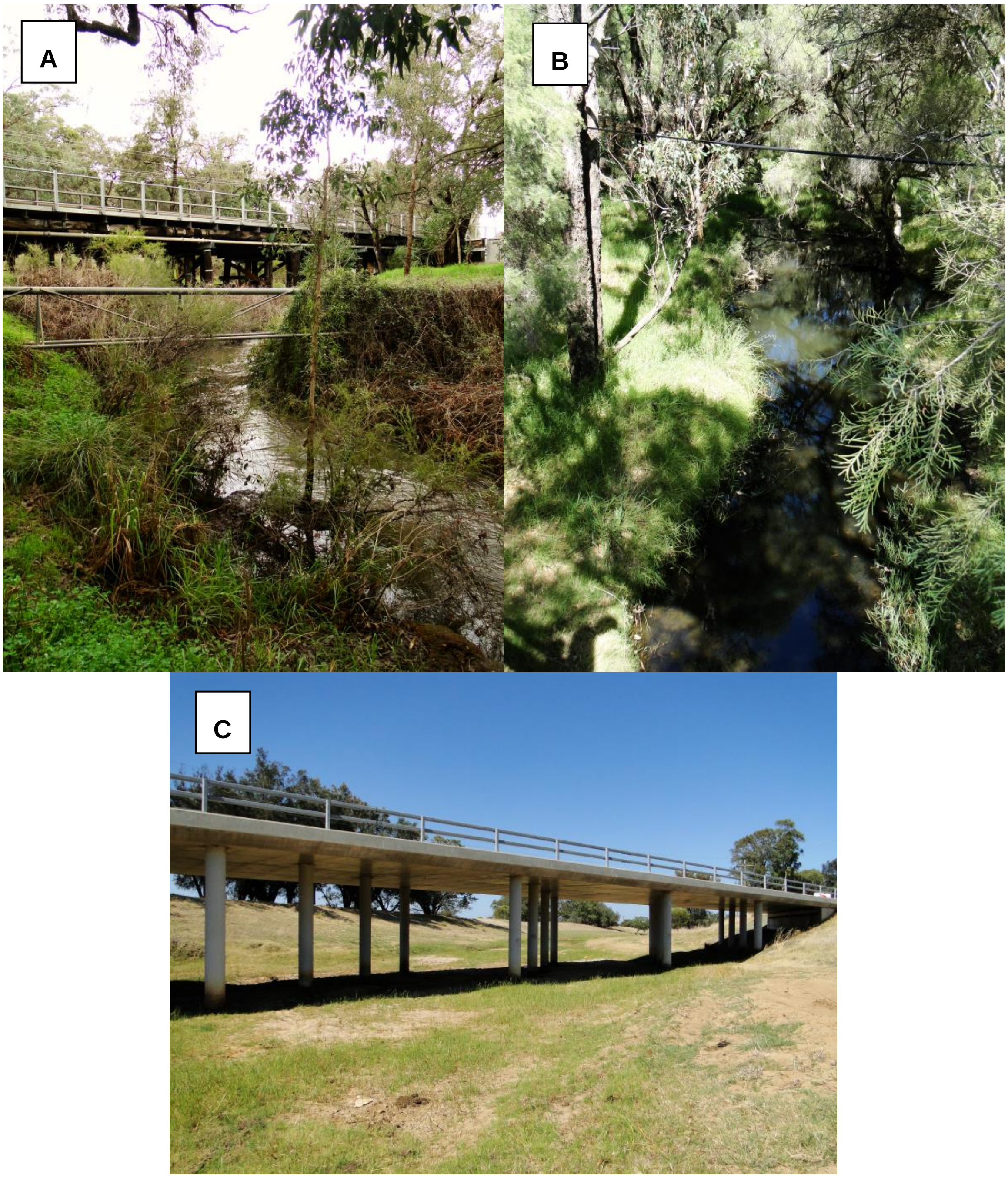


Figure 1-5: Serpentine River at South Western Highway (A) and Rapids Road (B), and Serpentine Drain at Karnup Road (C)

The Serpentine River enters the study area from the east in the form of a natural channel with an external catchment area of 219 km^2. This section of the river, which meanders through the coastal plain, is thickly vegetated until it reaches the Serpentine Drain, just after the Lowlands gauging station. Figure 1-5 shows the marked difference in channel morphology between the Serpentine River and Drain, with the latter channel designed for flow conveyance.

Development constraints to rural drainage districts

Most major rural drains in the study area are managed by the Water Corporation. When development is planned within a rural drainage area, it is important that drainage management is undertaken according to guidelines set out by the Water Corporation. This is a major constraint that requires assessment during the flood study and consideration in the development of the DWMP.

A summary of the constraints on urban development can be found in Water Corporation's *Development Services Information Sheet No. 59* (Water Corporation 2008). Major points of the summary include:

- It is important that developers ensure, to the satisfaction of the Water Corporation, that the level of service to the rural drainage district is not compromised by the outflow from the development. Development projects need to minimise discharge to rural drains, and take into account their limited capacity. The design of compensating basins and drainage discharge must demonstrate that the functionality of the Water Corporation's drains will not change.
- Flows to any Water Corporation rural drainage system, from a storm event of an average recurrence interval level of protection determined by local government, are not increased as a direct or indirect result of the development.
- In addition, any naturally occurring storage capacity of the floodplain of existing drains is retained.
- Where additional drainage infrastructure has been provided by the Water Corporation for flood protection purposes to urban areas the design of the internal drainage system for any development must recognise the impacts of a major storm event on the flood protection works. Urban areas affected by flood protection works must be protected from a major flood event either by upgrading that infrastructure to incorporate that event or by providing protection within the development for the effects of the event. The design of the internal drainage system shall identify and incorporate upgrades to existing food protection as required. This will extend to the integrity of levee systems to meet the change in risk from rural to urban land. These requirements are in addition to Australian Rainfall and Runoff level of protection requirements for urban developments.

1.3 Flooding mechanisms

There are several potential flooding mechanisms for the north-east Baldivis area and surrounds. The mechanisms can be broadly categorised as follows.

Groundwater inundation

Groundwater inundation is responsible for extensive flooding during the winter months over much of the study area (see Marillier et al. 2012b) and contributes to the existing regional floodplain storage volume. In agricultural areas such inundation is generally discharged via shallow drains over several days. Urban development typically manages such inundation with fill and subsurface drainage infrastructure. Groundwater inundation is not always considered in flood studies; however, in this section of the Swan Coastal Plain, it is likely to contribute to flooding during winter as it effectively increases impervious surface and reduces infiltration capacity. Therefore, seasonally inundated areas will produce more runoff than areas with several metres clearance from groundwater. Figure 2-4 shows inundation from the maximum groundwater level (1981–2010) throughout the study area, sourced from the Lower Serpentine regional model (Marillier et al. 2012b).

Riverine flooding from the Peel Main Drain

The upper catchment of the Peel Main Drain is small relative to the catchments of the other rivers and drains within the study area. The catchment is sandy and interspersed with a series of large wetlands. Hence, flooding from upstream flows around the Peel Main Drain poses a relatively low risk. However, within the main study area the Peel Main Drain receives lateral inflows from agricultural drains along its entire reach, and in sufficiently large rainfall events this additional flow may result in flooding adjacent to the Peel Main Drain where gaps in the levee bank or lateral culverts allow discharge. A secondary factor is the influence of flows from the Serpentine Drain which will affect water levels in the Peel Main Drain. It is noted that the Karnup Road gauging station is influenced by backwater effects from the Serpentine River.

Localised flooding from catchment rainfall

The main study area experiences localised flooding after large or intense rainfall events. This flooding occurs as a result of infiltration excess runoff or saturation excess runoff, the latter being heavily influenced by landscape position and groundwater levels.

Soil within the study area can be classified into two broad categories: Bassendean Sand, and Pinjarra Clay (mixed sand and clay) with some heavy clays present adjacent to the Peel Main Drain. The presence of clays throughout the area increases the risk of localised flooding from infiltration excess runoff.

Low-lying areas within the study area frequently experience surface ponding, either from groundwater inundation or individual storms. In August 2005 the area around Cobby Lane experienced flooding after the passing of a frontal system (Figure 1-6).

Figure 1-6: Flooding around Cobby Lane, August 2005

Overtopping of the Birrega Main Drain levee bank at Duck Pool

In the location of Duck Pool (Figure 1-2) on the Birrega Main Drain (immediately upstream of Mundijong Road), the western levee bank is at 11 mAHD, and in larger flow events (> 50 yr ARI) it is likely that the levee will be overtopped and possibly fail, with both scenarios resulting in a redirection of flow through the north-east Baldivis area. Advice from the Water Corporation and the Shire of Serpentine Jarrahdale indicated that the Birrega levee overtopped during the 1987 event at this location. Modelling of the Birrega drain by Hall (2014) indicated that the levee would overtop in this location for the 100 yr ARI event.

Levee failure or breakout on the Birrega Main Drain, Serpentine Drain or Serpentine River

The western levee banks along the Birrega Main Drain and the Serpentine Drain introduce a potential flood hazard to the Baldivis and Dog Hill area in the event of failure. For this reason, it is important that these two drains are included within the hydraulic model, with the associated storage areas on the eastern side of the drains included also.

SKM (2010) and Hall (2014) demonstrated that levee failure in the Birrega Main Drain and Serpentine Drain would result in extensive overland flooding to the west of the drains. The Birrega and Oaklands MIKE FLOOD model indicates that in a 100 yr 24 hr event an extensive storage area would hold a large volume of water with a level sufficient to overtop the levee bank on the Birrega Main Drain at Duck Pool. It is likely that the levee would fail in such a scenario. Therefore, levee failure should be considered as a realistic possibility for events close to and above the 100 yr event in size. The Duck Pool location will overtop in a 100 yr event, and is therefore the most likely location of levee failure. But it is possible that the levee bank may fail in other locations on the main drains.

In the event of levee failure, a portion of flow from the Birrega Main Drain will be redirected to the west, towards the Peel Main Drain.

1.4 Literature review

Birrega and Oaklands floodplain and drainage study
Hall (2014)

A MIKE FLOOD model was developed for a 176 km^2 section of the Birrega and Oaklands drain catchment located on the Swan Coastal Plain. The study consisted of a hydrological component including flood frequency analysis and development of a calibrated RORB model for external catchments; and the development of a coupled 1D-2D hydraulic model in MIKE FLOOD. 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 for an event from the winter of 2011, equivalent to approximately a 10 yr ARI design event. Calibrated parameters for the RORB model and the MIKE FLOOD model are tabulated within the report.

The hydraulic model was used to simulate the 5, 10, 20, 100 and 500 yr ARI events for durations of 6, 12, 24, 36, 48 and 72 hr. A levee fail scenario, in which the left and right levee banks on the Birrega and Oaklands drains were removed was also simulated. These design events are suitable for providing boundary conditions for the Birrega Drain for the model being developed for the North-east Baldivis study area, which is located immediately to the south of the Birrega and Oaklands model.

The 100 yr and 500 yr 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.

Murray floodplain development strategy
GHD (2010)

Seasonal flood frequency analysis was performed for the Murray and Serpentine rivers. 100 yr ARI peak flows were estimated at 830 m^3/s on the Murray River at Pinjarra, and 120 m^3/s on the Serpentine River at Lakes Road.

Hydrodynamic modelling of the Peel Harvey Estuary was used to establish the boundary conditions used in flood modelling. The 1 in 100 ARI peak water level was 1.2 mAHD. A further 0.9 m allowance for sea-level-rise and 0.6 m freeboard were added to this level.

Murray River hydrology

A RORB model was calibrated for the Murray River to several summer and winter events. Calibrated values of *Kc*, *RoC* and initial loss for different events are tabulated in the report. Baseflow was excluded from events during calibration using the constant slope method, as baseflow made up 14–59% of flow for winter events. For design events baseflow was added to RORB flows using a time-varying method. RORB models were configured using regional parameterisation for 18 small 'hills' catchments.

Serpentine River hydrology

Inflows to the study area were sourced from the SKM (2010) modelling of the Serpentine River. Design hydrographs for the 25, 100 and 500 yr ARI events were available, and the 5 and 50 yr ARI events were calculated from these events.

Hydrodynamic estuarine modelling

A MIKE 21 HD-FM model was configured and calibrated for the Peel Harvey Estuary to provide boundary conditions for the flood model. The model achieved an RMSE for tidal levels of 0.05 m for the Peel Inlet and 0.06 m for the Harvey Estuary. Eleven scenarios were simulated to provide design estimates for use in flood modelling.

Hydraulic modelling

A MIKE 21 model was used for calibration, and a MIKE FLOOD model was used for design events. Model grid size was 20 m, and model topography was varied for calibration events in different years. Direct rainfall was simulated within the model domain, with boundary inflows derived from the RORB catchments and Serpentine model (SKM 2010). A proportional loss was applied to the rainfall based on the parameters used for the foothills RORB catchments. A selection of bridges and culverts located along the new Perth to Bunbury Highway was included in the simulations for design events.

Gridded flood depths are available for the 5, 10, 25, 50, 100 and 500 yr ARI events within the model domain, with gridded hazard categories and flood fringe/flood plain mapping available for the 100 yr ARI event. Peak discharge and water levels were provided for the 5, 10 and 50 yr ARI events for a number of specific locations within the catchment.

Small dams flood study – regional analysis, Water Corporation (Pearce 2006)

This study developed methods for flood estimation in the range of the 50 yr ARI to the probable maximum precipitation design flood (PMP-DF) for small dam catchments (up to 100 km^2) in the south-west of Western Australia. Previously, flood estimates were based on backward extrapolation from larger catchments. This study derived flood estimation curves and catchment modelling parameters for use in small catchments.

RORB modelling of eight small dam catchments was undertaken, including calibration to observed historical events (baseflow removed). Extreme design rainfall events were derived using the Bureau of Meteorology PMP estimates. CRC-FORGE would typically be used to determine design rainfall for large flood events. However, it only includes storm durations of 24 hr or longer and critical events in the dam catchments were less than 24 hr. Therefore large events were estimated by interpolation between the 1 in 100 yr determined by flood frequency analysis, and the PMP event.

Regional parameters are provided for RORB hydrological modelling. An equation and parameters are defined for regional large and extreme flood estimation for the 50 yr and 100 yr ARI events.

Serpentine hydrological studies, Department of Water
Marillier et al. (2012a & b; 2014)

The Serpentine hydrological studies series of reports describes the construction and calibration of a regional surface water and groundwater interaction model using MIKE SHE. The model provides estimates of superficial groundwater levels, surface inundation and river flows over a 728 km^2 section of the Swan Coastal Plain, south of Perth. Several major rivers were modelled within MIKE 11, including sections of the Peel Main Drain, Birrega and Oaklands Drains, the lower Serpentine River, Punrack Drain and Dirk Brook. The model achieved an average absolute error in head of 0.46 m for the Superficial Aquifer, and an average Nash-Sutcliffe coefficient of efficiency of 0.77 for calibrated flow gauges. Estimates of seasonal groundwater inundation sourced from the model are appropriate for use in determining antecedent conditions for calibration and design events for flood modelling.

Serpentine River floodplain management study – flood modelling report
SKM (2010)

SKM completed flood modelling for the 10, 25, 100 and 500 yr ARI events for sections of the lower Serpentine River (downstream of South West Highway), the Birrega Main Drain and the Peel Main Drain. The Upper Serpentine catchment was excluded from the RORB hydrologic model downstream of Serpentine Dam as it was deemed unlikely that the Dam would overtop in a 100 yr or 500 yr ARI event. The RORB model was calibrated to an event in 1987 at the Dog Hill (614028) and Dirk Brook (614030) gauging stations. Issues with the rating curve of Dog Hill for the 1987 event, and the storage associated with the Birrega Main Drain were identified as problematic for calibration of the RORB model. The design hydrologic parameters selected for use in the RORB model were:

Coefficient *Kc*: 400
Exponent *m*: 0.80
Initial loss for area 1 (forested steep catchments): 0 mm
Initial loss for area 2 (flat cleared catchments): 0 mm
Proportional loss for area 1: 75–80%
Proportional loss for area 2: 60–75%

Variable initial and proportional losses were provided for the two areas for each design ARI event. CRC-FORGE was used to derive rainfall depths and areal reduction factors for design events, with temporal patterns sourced from Australian Rainfall and Runoff. A baseflow of 20% of discharge was applied to flows from the Upper Serpentine catchment.

Flood Frequency Analysis was applied to the Dog Hill gauging station using observed annual maxima, with the 100 yr ARI showing a peak discharge of 154 m^3/s. The statistical model used and fitting method were not reported.

A hydraulic model of the area was developed using MIKE 21 with a grid resolution of 12 m. Structures and culverts were not modelled. The RORB hydrologic models were used to provide internal model inflows at locations through the drainage network. The model achieved a reasonable fit for flood levels at several recorded locations.

Floodplain mapping based on the 100 yr ARI event shows extensive inundation in the north-east Baldivis area, and low-lying areas to the west of the Birrega Main Drain. A levee break scenario, in which the western levees for the Serpentine and Birrega drains were removed, showed a significant increase in the extent of inundation in these two areas.

Jandakot drainage and water management plan: Peel Main Drain catchment
Department of Water (2009)

Appendix A of the Jandakot drainage and water management plan describes the development of a 1D hydraulic model of the Peel Main Drain using InfoWorks CS. Design rainfall events for 1, 3, 6, 12, 24, 48 and 72 hr durations were simulated using for the 2, 10 and 100 yr ARI events. Results were used in the design of the stormwater drainage system.

The model was calibrated to the Hope Valley (614013) gauging station. The modelled peak flow for the critical 100 yr ARI event was 1.59 m^3/s at the Hope Valley station. The peak flow reported for June 2000 was 1.34 m^3/s and was estimated as an 80 yr ARI event. Flood frequency analysis was not completed for this study. Model calibration hydrographs and statistics were not shown, and the calibration event was also not specified.

At the model outlet at Millar Road Bridge, peak discharge was calculated at 4.76 m^3/s for the 100 yr ARI. The average winter baseflow was reported as 90 L/s, and was incorporated in the model by inclusion of a constant groundwater level within Mandogalup Swamp.

Revised Lower Serpentine Flood Study
Water Authority of Western Australia (1990)

This study revised flood estimates for the lower Serpentine River on the coastal plain, downstream of Serpentine Dam. Design flood estimates were determined for the 1, 4 and 10% AEP events at several locations along the Serpentine River (approximately the 10, 25 and 100 yr ARI events), including estimated spilling from the Serpentine Dam. Peak flow at the South Western Highway was estimated to be 118 m^3/s for the 100 yr ARI event. Flood frequency analysis using flows from Dog Hill, located further downstream, estimated the 100 yr ARI event peak flow at that point to be 215 m^3/s.

A RORB model was developed for the study area, and calibrated to the Dog Hill gauging station. Design rainfall excesses were applied to the RORB model and runoff coefficients were adjusted for the design events to meet the FFA peak flows at Dog Hill gauging station.

Byford town-site drainage and water management plan
Department of Water (2009)

The Byford drainage and water management plan was prepared by GHD on behalf of the Department of Water. It addresses management of the total water cycle within the Byford townsite and surrounds, including flood mitigation measures.

Infoworks CS was used with results from a previous MIKE FLOOD model to simulate flood events and determine critical flows and levels within Cardup Brook, Beenyup Brook, Oaklands Drain and several tributaries. Design rainfall events for the 1, 5 and 100 yr ARIs were run for critical durations. The model was used to calculate the detention volume required to manage the post-development peak flows for the 5 yr and 100 yr ARI events for the subcatchments defined within the study area.

Estimation of rare design rainfalls for Western Australia: Application of the CRC-FORGE method
(Durrant & Bowman 2004)

This study applied the CRC-FORGE method of design rainfall estimation to a large dataset in Western Australia. Design rainfall estimates and aerial reduction factors were developed for each of four regions in WA, for event durations greater than 24 hr. Where applicable, analysis of rainfall data was performed on a seasonal (winter and summer) basis. The design rainfall estimates and areal reduction factors calculate in this study are appropriate for use in south-west WA and can be combined with design estimates for more frequent and shorter duration events calculated using the methods of Australian Rainfall and Runoff 1987.

2 Data collection

2.1 Rainfall data

A number of pluviometers measure rainfall within the region but their lengths of record vary. For some events assessed within this report, only one or two pluviometers have data available for the full duration of the event. The Dog Hill (9295) site is located within the hydraulic model area, and several other stations are located with the RORB hydrological catchments. Table 2-1 lists the pluviographs used in this study, and the site locations are shown in Figure 2-1.

Table 2-1: Pluviographs used in floodplain development study

BOM reference	BOM context	BOM name	Commence	Cease
9039	SERPENTINE	SERPENTINE	31/12/1905	-
9135	DIRK BROOK	MYARA ROAD	22/07/1971	25/05/1999
9245	DIRK BROOK	KENTISH FARM	1/03/1974	28/05/2001
9269	SELDOM SEEN CREEK	GARDENS	1/06/1974	-
9270	MORE SELDOM SEEN CREEK	CERIANI FARM	1/06/1974	-
9331	PEEL DRAIN	MANDOGALUP	15/06/1976	24/05/1983
9387	DIRK BROOK	HOPELANDS ROAD	4/04/1979	25/05/1999
9232	39 MILE BROOK	JACK ROCKS	14/04/1981	-
9194	MEDINA RESEARCH CENTRE	MEDINA RESEARCH CENTRE	31/03/1983	-
9295	SERPENTINE DRAIN	DOG HILL	9/06/1983	-
9023	JARRAHDALE	JARRAHDALE	31/01/1882	-

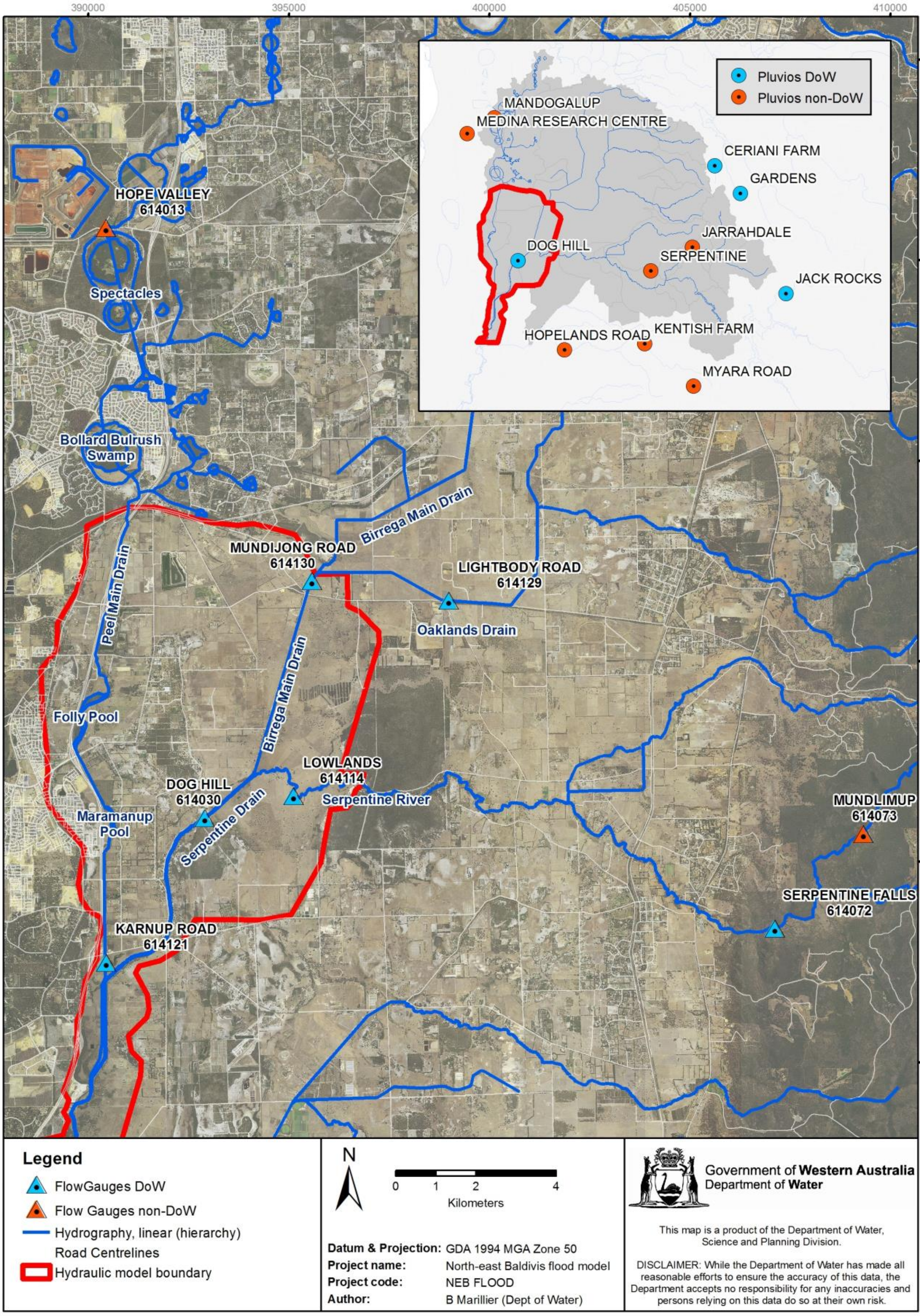


Figure 2-1: Locations of pluviographs and flow gauges used in the floodplain study

2.2 Streamflow data

There is moderate availability of streamflow data throughout the study area though gauges are concentrated in the hills catchments and along the Serpentine River.

The gauges on the Serpentine River used in this study include: Serpentine Falls (614072) which was used for flood frequency analysis (FFA) and regional parameter validation; Lowlands (614114) which was used in RORB parameter estimation; and Dog Hill (614030) which was used for FFA, RORB calibration and hydraulic model calibration.

For the Birrega and Oaklands catchments, discharge information for calibration and design events is available from the Birrega flood model (Hall 2014). The model was calibrated to the Mundijong Road (614130) and Lightbody Road (614129) gauges, both of which have only short flow records.

On the Peel Main Drain, the quality and availability of data are limited. The Hope Valley gauging station (614013) in the upper catchment has 22 full years of record but the Karnup Road gauging station (614121) has been operational only since 2005. The regional hydrographer assessed the Hope Valley station as having rating curve issues associated with variable tailwater and siltation (Kevin Firth pers. comm.) which introduce an estimate error of +-30% to peak flows. Acoustic Doppler velocimeters were installed at the Karnup site in 2009 giving reliable discharge measurements; however, the earlier data (2005–09) is affected by backwater. The Hope Valley station was used in FFA and to calibrate the upper Peel Main Drain RORB model. The short period of record at the Karnup Road station limits its applicability in this study for larger events though flood level information was used for calibrating the 2005 flooding at Cobby Lane and surrounds.

The locations of gauging stations used for this study are shown in Figure 2-1 and their periods of record are listed in Table 2-2.

Table 2-2: Streamflow gauging stations analysed in the study

AWRC reference	AWRC context	AWRC name	Commence	Cease
614130	BIRREGA DRAIN	MUNDIJONG ROAD	2011	-
614073	GOORALONG BROOK	MUNDLIMUP	1951	1999
614129	OAKLANDS DRAIN	LIGHTBODY ROAD	2008	-
614013	PEEL DRAIN	HOPE VALLEY	1976	2001
614121	PEEL MAIN DRAIN	KARNUP ROAD	2005	-
614030	SERPENTINE DRAIN	DOG HILL	1979	-
614072	SERPENTINE RIVER	SERPENTINE FALLS	1911	2001
614114	SERPENTINE RIVER	LOWLANDS	1998	-

2.3 Terrain data

LiDAR (Light Detection And Ranging) data is available for the proportion of the catchment that is located on the Swan Coastal Plain. A representation of the extent of the LiDAR coverage is shown in Figure 2-2. These data were captured on 25 February 2008 by Fugro Spatial Solutions Pty Ltd, and have a point density of 1 point per square metre and an accuracy of 0.15 m at 67% confidence. LiDAR was used to develop the bathymetric layer for the 2D overland flow hydraulic model, to develop the cross-sections of waterways used in the 1D channel flow hydraulic model.

Sections of the LiDAR were corrected where errors were identified. Other modifications were required for conversion of the LiDAR 1m resolution grid to the 10 m grid used within the numerical model. These modifications are discussed in more detail in Section 5.

2.4 Land use data

Land use data was developed by Department of Water for the region (Kelsey et al. 2011), and is based on Landgate cadastre (2008) with aerial imagery (2008), and LiDAR non-ground returns to determine vegetation extent. Land use data is important for resistance categories in hydraulic modelling and for regional parameterisation of hydrologic models. The current land-use data is shown in Figure 2-3.

2.5 Groundwater and surface water interactions

Parts of the Baldivis area are prone to very shallow groundwater resulting in regular winter inundation. The Lower Serpentine MIKE SHE model (Marillier et al. 2012b) was developed by the Department of Water to identify areas of groundwater inundation, and provide reliable groundwater level information, and estimates of drainage within the study area. The north-east Baldivis area experiences partial inundation from groundwater almost every winter, even in dry seasons, and therefore the interaction between groundwater and surface water must be considered as a component of this flood study. Groundwater levels from the MIKE SHE model can be used to define antecedent conditions for both calibration and design events, effectively determining areas which should be considered impervious as a result of inundation.

2.6 Roads and structures

Culverts, bridges and road elevations were considered in the construction of the hydraulic model, as described in Section 5. More than 90 structures were assessed in the field for inclusion in the hydraulic model. Many of these were too small or overgrown to justify inclusion in the model but larger culverts under the freeway and along Peel Main Drain were explicitly included. In some locations the road and drain elevations were modified in the model bathymetry to better represent actual levels where appropriate.

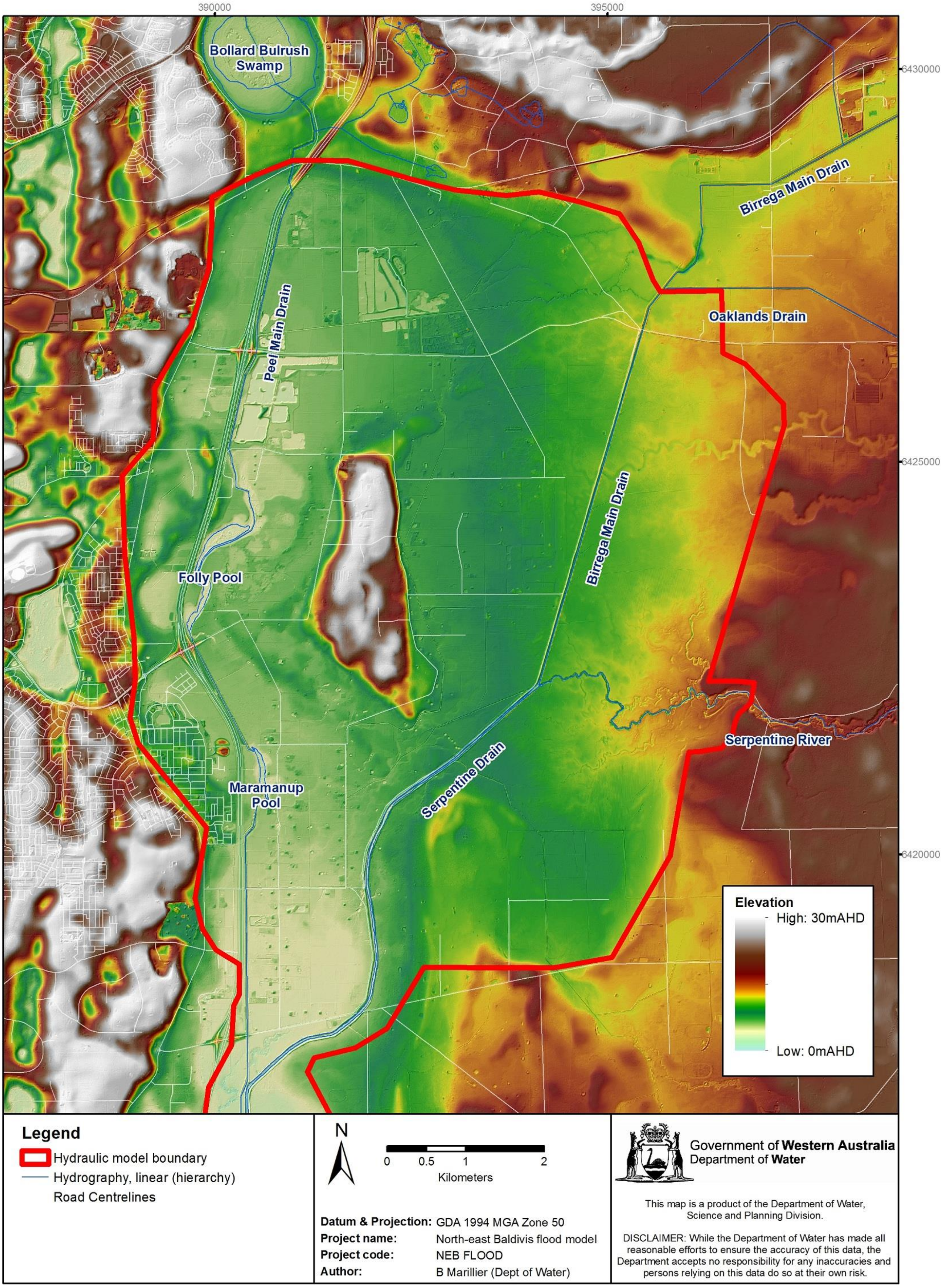


Figure 2-2: Digital elevation model (DEM) derived from LiDAR data

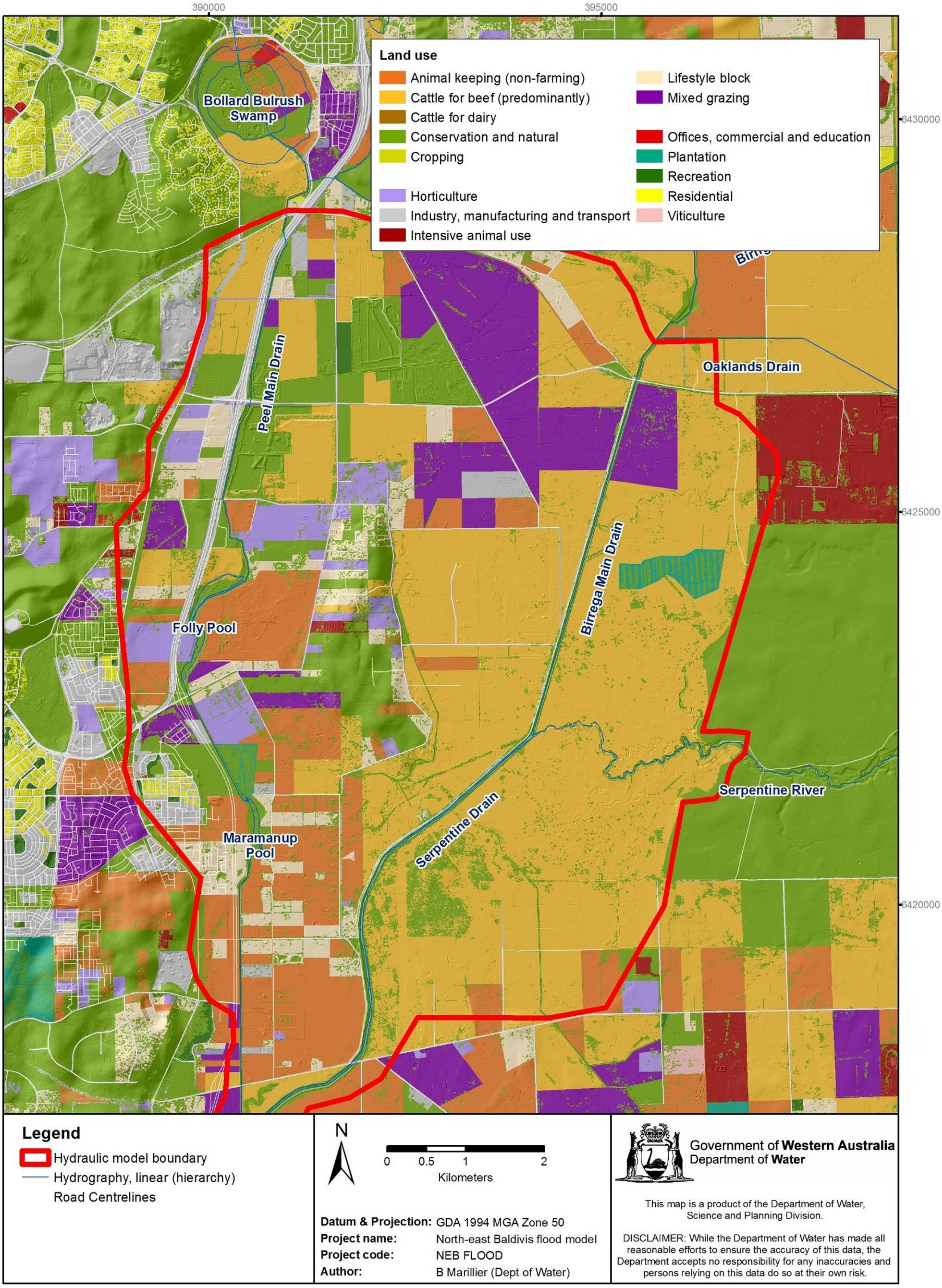


Figure 2-3: Land use categories

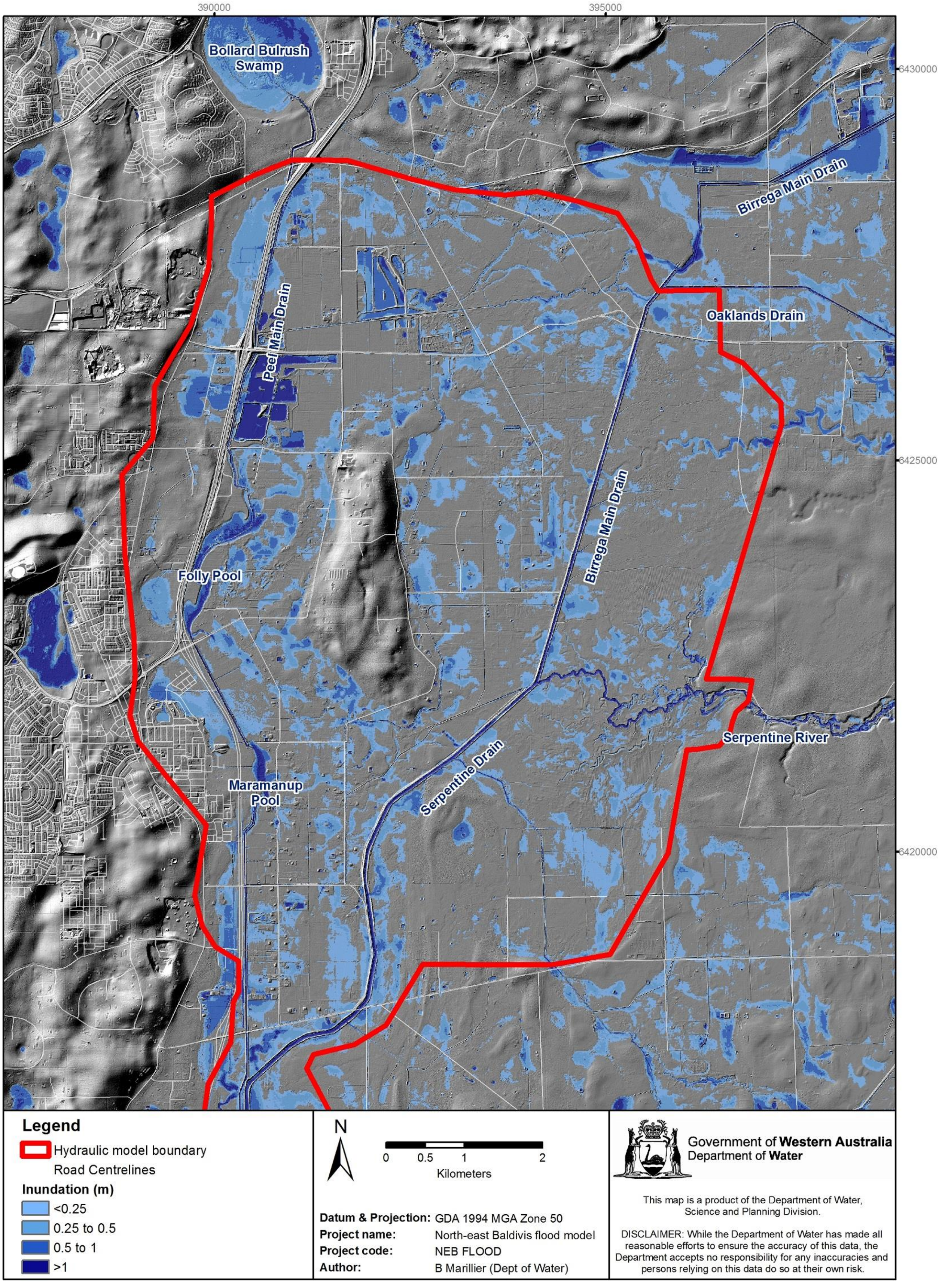


Figure 2-4: Likely areas of inundation based on maximum groundwater levels for the period 1981-2010 (Marillier et al. 2012b)

3 Flood frequency analysis

Flood frequency analysis (FFA) was undertaken for three flow gauges located within or near the study area. FFA is important for assessing historical flows against RORB modelled flows developed using AR&R1987 design rainfall, and gives an indication of the relative contributions of flow from the various source catchments to the study area. The flow gauges analysed included Hope Valley (614013) on the upper Peel Main Drain, Dog Hill (614030) on the Serpentine Drain, and Serpentine Falls (614072) located on the Serpentine River within the Darling Scarp.

3.1 Methodology

The peak annual flow data series was used for FFA for the three gauges. Although the peak series included several summer events, the record length was insufficient to justify partial FFA. The flow series for each gauge was assessed for completeness and accuracy, and the quality of the gauging station and flow record were discussed with the regional hydrographer before analysis.

FFA was recently completed by Hall (2014) for the Serpentine Falls gauge, and the results were used for this study without modification. Only the flow record post-1975 was used for the Serpentine Falls FFA to exclude the period prior to the Serpentine Dam construction and any major dam releases. The Dog Hill gauge has a period of record beginning in 1979 and therefore is not influenced by pre-dam conditions.

The program Flike V4.50 (Kuczera 2001) was used to fit Log Normal, Log Pearson type III (LPIII) and Generalised Extreme Value distributions to the annual data series. LPIII distribution was the closest fit to the data in all cases, and was used for subsequent analysis.

3.2 Results

A summary of the results of FFA for the three gauges is shown in Table 3-1, and plots of the fitted LPIII distributions are shown in Appendix A. Due to the differences in catchment size, Dog Hill has much higher flood flows than Serpentine Falls, which are in turn much larger than those measured at Hope Valley.

Table 3-1: FFA for design flows at Hope Valley, Dog Hill and Serpentine Falls

Annual recurrence interval (ARI) (1 in y)	Peel Main Drain Hope Valley (614013)* (m^3/s)	Peel Main Drain Hope Valley (614013)** (m^3/s)	Serpentine Serpentine Falls (614028) (m^3/s)	Serpentine Dog Hill (614030) (m^3/s)
5	1.1	1.0	9.8	89.9
10	1.5	1.3	12.9	106.7
20	2.0	1.5	16.6	120.8
50	2.7	1.9	22.5	136.7
100	3.3	2.1	27.8	147.0
200	4.0	2.4	34.1	156.2
500	5.1	2.7	44.2	166.9

**Including 1992 event in annual peak flow series*

***Excluding 1992 event in annual peak flow series*

The larger recorded events at the three gauges are shown in Table 3-2. The largest event in the winter of 1987 was calculated to be a 44 yr ARI event at Serpentine Falls and a 54 yr event at Dog Hill. The ARIs calculated for these two sites are fairly consistent for the larger events; however, the fitted LPIII distribution for Dog Hill flattens faster than Serpentine Falls as ARI increases (see Appendix A). The difference in the fitted distributions is probably related to the availability of storage in the Birrega Main Drain catchment.

Table 3-2: ARI values from FFA for selected gauges and events

Event	Peel Main Drain Hope Valley (614013)* ARI (1 in y)	Peel Main Drain Hope Valley (614013)** ARI (1 in y)	Serpentine Serpentine Falls (614028) ARI (1 in y)	Serpentine Dog Hill (614030) ARI (1 in y)
1987	14	35	44	54
1988	9	13	16	20
2000	-	-	3	12
1984	-	-	4	9
1994	4	5	1	7
1985	2	2	3	6
1983	-	-	6	5
1996	2	1	2	4
1992	37	-	13	3

**Including 1992 event in annual peak flow series*

***Excluding 1992 event in annual peak flow series*

Flows at Hope Valley are two orders of magnitude smaller than those at Dog Hill, and the largest flow (2.5 m^3/s) recorded was in 1992 during a summer event associated with 139 mm of rainfall in 20 hr (Dog Hill pluviometer). FFA for the Hope Valley gauge was completed twice, first for a peak annual flow series which includes the 1992 event, and then for a series excluding the event. The 24 hr rainfall depth for this event is in excess of a 100 yr ARI event, and as there is only a single event of this magnitude recorded the LPIII distribution is very sensitive to its inclusion. When the 1992 event is included in FFA, the 1987 and 1988 events show as only 14 and 9 yr ARIs, and the magnitude of the 100 yr ARI event is 3.3 m^3/s, compared to 2.1 m^3/s when the 1992 event is excluded. The estimated ARIs for observed events are much more consistent between the three gauges when the 1992 event is excluded. Rainfall from this event was associated with a west coast trough, and the various pluviometers in the area indicate very patchy rainfall within the region, which could explain this difference.

Table 3-3 compares the FFA results for this study to previous studies of the Serpentine River and Peel Main Drain.

Table 3-3: 100 yr ARI flow at locations on the Upper Serpentine River and Peel Main Drain

Location	Study	100yr ARI peak discharge estimate (m^3/s)
Hope Valley	DoW, 2009 (modelled)	1.6
Hope Valley	current*	3.3
Hope Valley	current**	2.1
South Western Highway	SKM, 2010	166.0
South Western Highway	WAWA, 1990	118.0
Serpentine Falls	current	28.0
Dog Hill	WAWA, 1990	215.0
Dog Hill	SKM, 2010	154.0
Dog Hill	current	147.0

**Including 1992 event in annual peak flow series*

***Excluding 1992 event in annual peak flow series*

DoW (2009) estimated the 100 yr ARI peak discharge at Hope Valley as 1.57 m^3/s using InfoWorks CS model and design rainfall. The current FFA estimates a peak discharge of 3.3 m^3/s when 1992 is included, and 2.1 m^3/s when it is excluded. While DoW (2009) mentions that the model was calibrated to the Hope Valley gauging station, it is unclear which events were calibrated, and calibration results were not provided. It is also mentioned that the June 2000 peak flow of 1.34 m^3/s was estimated to be an 80 yr ARI event, which is inconsistent with the observed peak flow series, which shows that 6 of the 22 peak annual flows exceed 1.0 m^3/s. Given that FFA was not completed by DoW (2009), and that it is unclear how the InfoWorks CS model was calibrated, the 100 yr ARI of 2.1 m^3/s (FFA excluding the 1992 event) was considered the best estimate. The primary flood risk for the study area is internal rainfall and levee bank failure on the Birrega and Serpentine drains, so the inconsistencies in peak discharge are of minor importance within the hydraulic modelling domain.

Peak discharges estimated for the 100 yr ARI event at South Western Highway by SKM (2010) and WAWA (1990) are an order of magnitude larger than the current FFA estimate from Serpentine Falls. Hall (2014) provides a discussion of the differences, and concludes that the previous studies substantially overestimated the peak discharge at South Western Highway. A summary of the reasoning is as follows:

- The difference in catchment size between the SW highway site and Serpentine Falls site is only 6%, which is insufficient to explain the difference in peak discharge.
- The flows in the previous studies were calibrated against stage data at South Western Highway for the 1987 event with no record of the data source. SKM (2010) estimated the peak flow as 80 m^3/s but reliable data from the Serpentine Falls gauge indicates a peak flow of 27 m^3/s. This site has a good gauging structure and is unlikely to be erroneous to that magnitude.
- The previous studies did not perform a FFA on the Serpentine Falls gauge as a check for these peak flow estimates.

The WAWA (1990) study used FFA to calculate the Dog Hill 100 yr ARI event by extending the Dog Hill annual peak discharge series using data from the Gooralong Brook (614073) gauge. The estimate of 215 m^3/s was revised by SKM (2010) to 154 m^3/s as a result of the extended record available for the analysis, which is consistent with the value of 147 m^3/s calculated for this study. Peak annual flows at Dog Hill after 1990 have generally been lower than those available from the 1980s, and for this reason the more recent FFA has resulted in lower estimates of peak discharge at Dog Hill.

4 Hydrology studies

The purpose of the hydrological studies was to establish the inflow hydrographs at the boundary of the hydraulic model (Figure 1-2). Inflow hydrographs were required for the upper Peel Main Drain at the southern edge of Bollard Bulrush Swamp; from the Serpentine River along the edge of the Lowlands bushland; and from the Birrega Main Drain in the north-eastern corner of the hydraulic model. The Birrega/Oaklands hydraulic model (Hall 2014) provided inflow hydrographs for design and calibration events from the north-east. For the Serpentine River and upper Peel Main Drain, rainfall-runoff routing techniques were used to generate the required calibration and design hydrographs. The software package RORB (version 6.15) was used for this purpose. RORB is runoff and stream flow routing program, and is described in Laurenson et al. (2010). A RORB model divides a catchment into sub-areas and routing reaches, which generate and route flow through the catchment based on an input rainfall time-series.

RORB models were developed for the upper Peel Main Drain and Serpentine River respectively. Different techniques were used to parameterise and calibrate the two RORB models.

For the upper Peel Main Drain, the Hope Valley (614013) flow gauge was available for calibration, which has 22 years of reliable record from which to select calibration events.

For the Serpentine River, regional methods for estimation of RORB parameters (*kc* and *RoC*) were used to select parameters for the cleared catchments along the Darling Scarp. The regional methods were developed by the Water Corporation for south-west Western Australia (Pearce 2006). Hall (2014) validated these parameters against the Serpentine Falls (614072) and Mundlimup (614073) gauges, and deemed them acceptable but a much better fit was achieved via calibration; therefore the calibrated parameters were adopted for this study upstream of the Serpentine Falls. Regional parameters are not appropriate for the lower section of the Serpentine River located on the Swan Coastal Plain between the Lowlands (614114) gauging station and the scarp due to differing slope and soil type. Therefore, several smaller events at the Lowlands gauge were used to calibrate parameters for this section of the catchment.

Baseflow separation

Before calibration of the RORB models, baseflow was removed from gauged data using the Eckhardt two parameter digital filter (Eckhardt 2005) shown below. RORB is suitable for routing direct runoff only so it is necessary to remove baseflow from the input hydrographs prior to calibration. Similarly, baseflow must be added to RORB outflows for design events to ensure peak flows and event volumes are accurately represented.

$$B_{k+1} = \frac{(1 - BFI_{max}).\alpha.B_k + (1 - \alpha).BFI_{max}.Q_{k+1}}{1 - \alpha.BFI_{max}}$$

where:

α = baseflow filter parameter
BFI_{max} = maximum value of ratio between baseflow and total flow
B_k = baseflow at timestep k
Q_k = total flow at timestep k

The parameters of α and BFI_{max} were adjusted to remove baseflow from each hydrograph and match the recession curve at the tail of the event. The parameter α varied between 0.990 and 0.995, and BFI_{max} between 0.5 and 0.7. Note that these parameters were determined for use on a discharge series at 15 min timestep. An example of the baseflow separation is shown in Figure 4-1. Baseflow separation was performed for all calibration events in the Serpentine and upper Peel Main Drain RORB models, and baseflow volumes and under-peak estimates for each event are shown in Table 4-2 and Table 4-4.

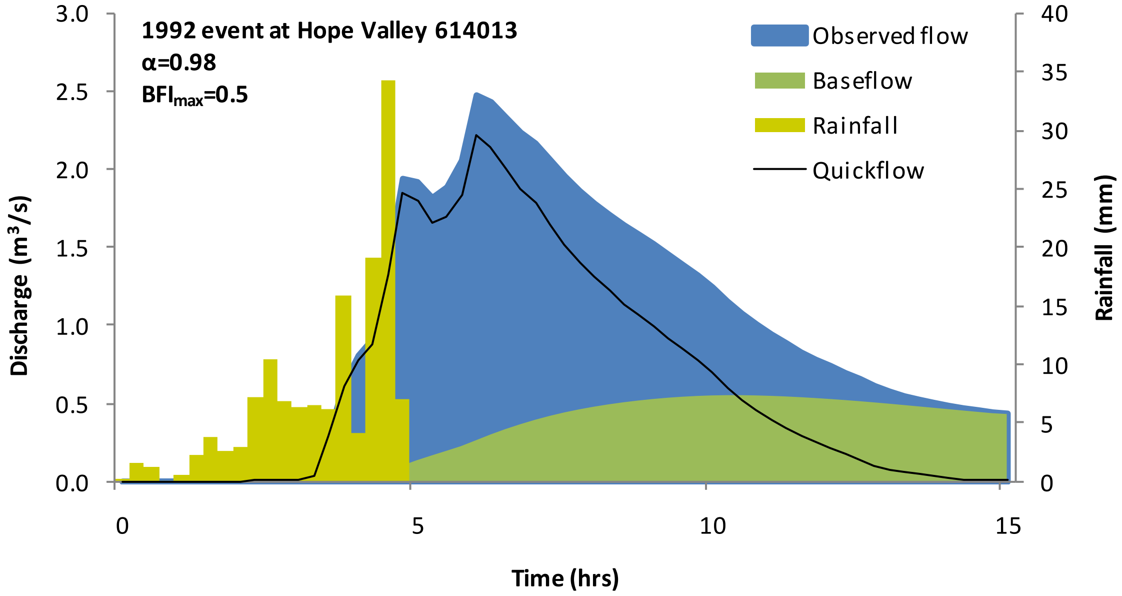


Figure 4-1: Example baseflow separation for the 1992 calibration event at the Peel Main Drain

Baseflow addition for design events

Baseflow is not simulated by RORB and therefore design events generated using RORB must have the baseflow component of the hydrograph added. The baseflow addition method was adapted from work undertaken by GHD (2010) as part of the Murray floodplain development strategy. Equation 4-1 shows how baseflow is derived from direct runoff based on a RORB modelled hydrograph.

$$B_k = (Br.B_{k-1}) + (Bc.Qr_k)^{Bm}$$

where:

B_k = baseflow at timestep k
B_k = baseflow at timestep k-1
Qr_k = direct runoff at timestep k
Br, Bc, Bm = calibrated parameters

Equation 4-1: Baseflow addition

The calibrated parameters were determined by first performing a baseflow separation on an observed hydrograph, calculating the direct runoff component (quickflow), and then deriving the appropriated parameter values to correctly add the baseflow component and reconstitute the hydrograph. Using the baseflow separation parameters defined in the previous section, the calibrated baseflow addition parameters were *Br* = 0.995, *Bc* = 0.003 and *Bm* = 1.00. These parameters were used to calculate design baseflow for all events. Figure 4-2 shows RORB modelled flow with added baseflow for a design event in the Serpentine River with an initial baseflow of 1 m^3/s. At the Lowlands gauge these parameters result in a baseflow that is 33% of the event volume, and 10% of under-peak flow.

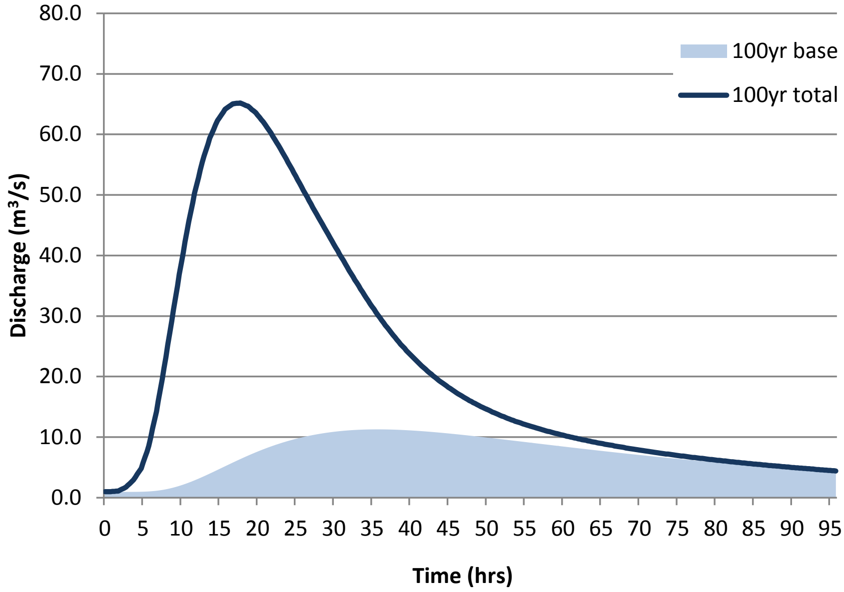


Figure 4-2: Example of baseflow addition for the 100 yr 24 hr event at Lowlands gauge

4.1 Upper Peel Main Drain RORB model

Catchment delineation

The upper Peel Main Drain subareas were delineated using ArcGIS based on terrain data and the existing drainage network. The catchment area is 58 km^2, and 20km^2 of this is located upstream of the Hope Valley gauge (614013). The subareas were delineated to ensure that at least four were present upstream of the calibration gauge. The RORB catchment and drainage network is shown in Figure 4-3.

Rainfall and flow data

The Hope Valley gauging station was used for calibrating the RORB model. The station has a period of record 1976–2001. The gauging station was assessed by the regional hydrographer as having only a fair quality of record, with peak flows estimated to have +-30% error due to the quality of the control structure and potential tailwater effects. The highest peak flow recorded was 2.49 m^3/s on the 9 February 1992, and a discharge measurement was taken at the site on this date.

Several pluviometers were located within 10 km of the RORB catchment and operational during the period of the gauging. Where possible the closest pluviometer was used for the calibration event but in some cases missing data or inconsistent temporal patterns resulted in poor calibration and an alternate pluviometer was used.

Table 4-1: Pluviometers used for Peel Main Drain RORB calibration events

Event	Pluviometer
1978	Mandogalup (9331)
1982	Mandogalup (9331)
1987	Dog Hill (9295)
1988	Dog Hill (9295)
1992	Dog Hill (9295)
1994	Ceriani Farm (9270)

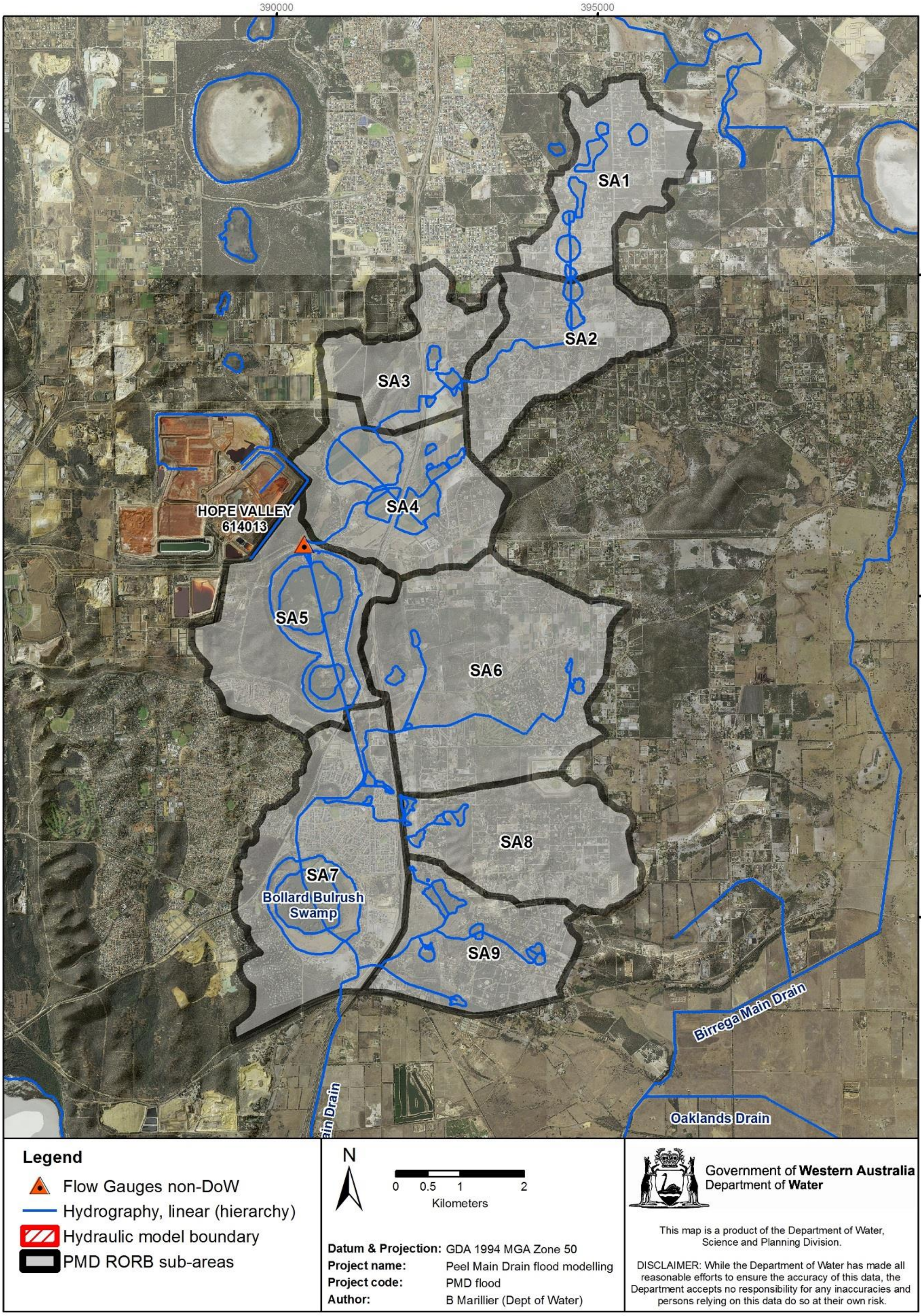


Figure 4-3: Upper Peel Main Drain RORB catchment

Calibration events and parameters

Six flow events were selected for calibration based on availability of data. Peak flows for the events range between 1.1 and 2.5 m^3/s. The calibration involved adjusting the RORB parameters *Kc*, *RoC* and IL for each event individually, though parameters were kept consistent between events where possible. A summary of the different events and calibrated parameters is shown in Table 4-2, and calibration of each event is discussed individually in Appendix B.

Table 4-2: Calibrated RORB parameters for Peel Main Drain

Event details							Parameters			
Event	Month	Peak flow* (m^3/s)	Baseflow volume %	Baseflow underpeak %	Event rainfall (mm)	Approx. event duration (hrs)	Kc	m	IL (mm)	RoC
1978	September	1.2	57%	19%	63.3	20	14	0.85	5	0.08
1982	January	1.1	41%	15%	124.7	51	14	0.85	70	0.09
1987	July	1.4	44%	27%	77.7	18	14	0.85	5	0.07
1988	July	1.3	54%	25%	65.3	21	14	0.85	5	0.08
1992	February	2.5	32%	11%	139	20	14	0.85	70	0.10
1994	August	1.1	56%	19%	62	14	14	0.85	5	0.07

**Peak recorded flow before baseflow separation*

All events calibrated adequately with a *Kc* of 14, and *RoC* of between 0.07 and 0.10 as shown in Appendix B. For the two summer events, the initial loss was set to 70 mm to account for infiltration. There is some indication of a positive correlation between *RoC* and rainfall depth and consequently, *RoC* was varied with event duration for design events to match FFA at Hope Valley.

The parameters are physically plausible for a catchment with a low slope containing wetland systems, and on predominantly sandy soils. The slope and wetlands result in a higher *Kc* than would typically be assigned to a catchment of this size in the hills, effectively extending and flattening the hydrograph. The *RoC* is low as a result of infiltration in sandy soils, and wetland storage.

4.2 Serpentine River RORB model

Catchment delineation

The Serpentine River RORB catchment was delineated into subareas using ArcGIS, based on terrain and drainage datasets. The model consists of subareas located on the Darling Scarp, on the Swan Coastal Plain and in the foothills. The subareas were defined so that regional parameters developed by Pearce (2006) and validated by Hall (2014) could be used for the cleared scarp catchments, with a separate parameter set on the Swan Coastal Plain, and in the vegetated catchments within the scarp. The RORB catchment and drainage network is shown in Figure 4-4.

As reported by SKM (2010) the Serpentine Dam is not likely to overtop in a 100 or 500 yr ARI based on the additional storage available above the annual median storage. Therefore, the catchment area upstream of the dam was not included in the RORB model.

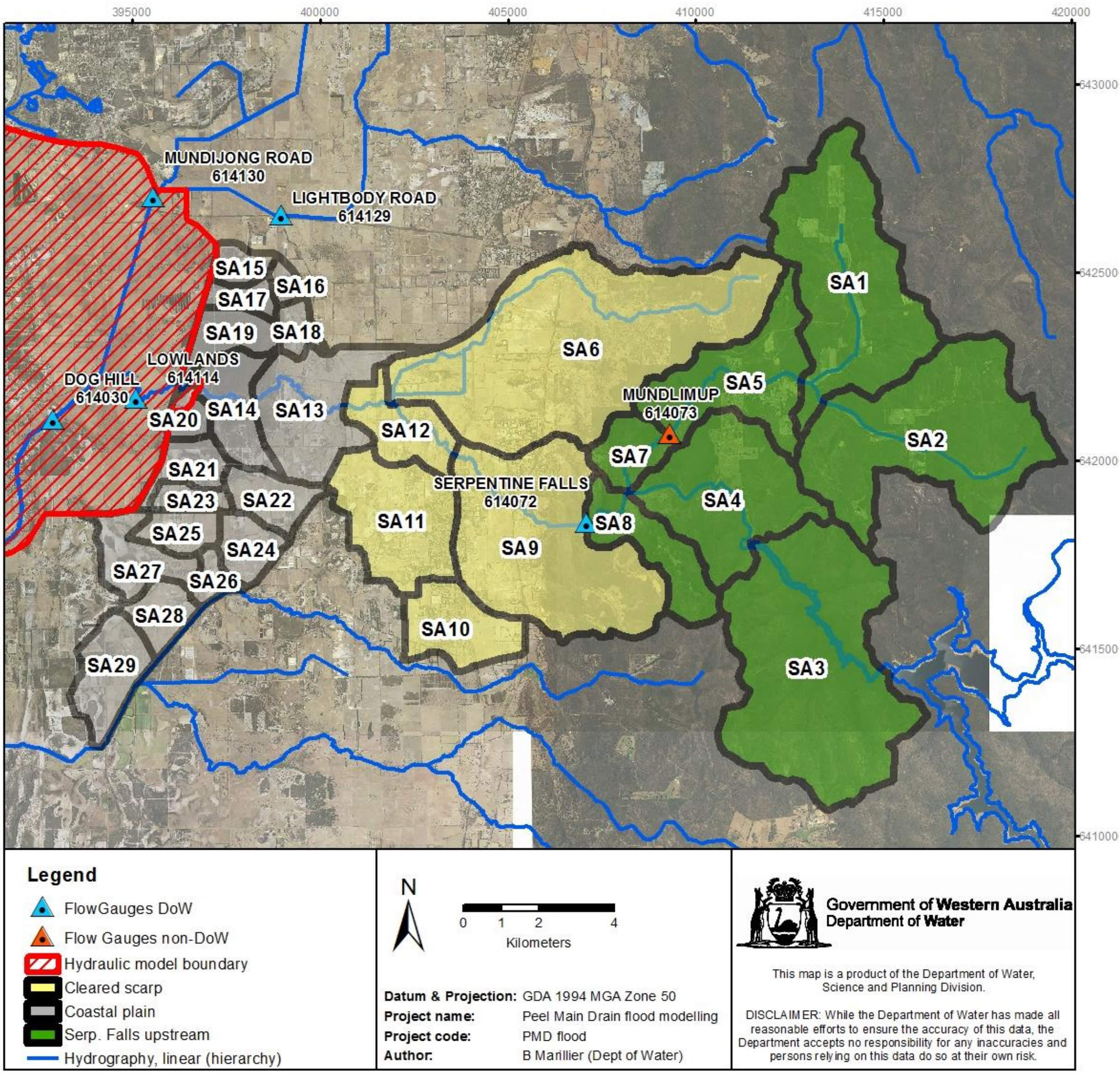


Figure 4-4: Serpentine RORB catchment

Rainfall and flow data

The Lowlands gauge (614114) was used to calibrate several winter flow events on the Serpentine River between 2000 and 2007. These events were used to validate parameters used on the Darling Scarp, and to select appropriate parameters for the flatter, sandier part of the catchment.

The large 1987 and 1988 rainfall events were used to compare the outflows from the Serpentine RORB model with observed flows at Dog Hill (614030), and to calibrate the RORB model upstream of the Serpentine Falls and Mundlimup gauges. The Birrega catchment was not modelled with RORB so the flows from the Serpentine RORB model make up only a component of the flow at Dog Hill. Nevertheless, it is useful to assess the shape of the flood hydrograph against observed data.

Rainfall pluviometer data was available across several stations within, or near, the Serpentine RORB catchment. Where possible, two pluviometers were used to take into account spatial and temporal variability in rainfall, as these are significant in the Serpentine catchment due to orographic effects associated with the scarp. The stations were selected for use based on availability of data and rainfall timing.

Table 4-3: Pluviometers used for Serpentine RORB calibration events

Event	Pluviometer	
1987	Dog Hill (9295)	Kentish Farm (9245)
1988	Hopelands Road (9387)	Kentish Farm (9245)
2000	Dog Hill (9295)	Kentish Farm (9245)
2002	Dog Hill (9295)	Serpentine (9039)
2005	Dog Hill (9295)	Gardens (9269)
2007	Dog Hill (9295)	Jack Rocks (9232)

Calibration events and parameters

Parameters for the Serpentine RORB model were selected based on a combination of regional parameter validation and calibration to recorded events. The model was configured with three separate groups of subareas, each with different values for *Kc* and *RoC*. The groups include:

- Serpentine Falls upstream: 10% cleared, calibrated *Kc* and *RoC*
- Cleared scarp: 73% cleared, parameters defined based on regional parameter sets with 60–80% clearing
- Coastal plain: calibrated *Kc* and *RoC*.

Six events were used for parameter calibration and validation. The 1987 and 1988 events were used to assess the parameter set against the Dog Hill gauge, and to ensure that the timing and volume of the event were appropriate for the Serpentine River’s contribution to flow at Dog Hill. The four events from the 2000s were used to select appropriate *Kc* and *RoC* for the Lowlands subareas, which are located on the relatively flat and sandy coastal plain where regional parameters are inappropriate. The parameters used for each event are shown in Table 4-4 below, and calibration results are shown in Appendix B for each event.

Table 4-4: Calibrated RORB parameters for the lower Serpentine River

Event details															
				Serpentine Falls (614072)			Mundlimup (614073)			Lowlands (614114)			Dog Hill (614030)		
Event	Month	Event rainfall (mm)	Approx. event duration (hrs)	Peak flow* (m^3/s)	Baseflow volume %	Baseflow underpeak %	Peak flow* (m^3/s)	Baseflow volume %	Baseflow underpeak %	Peak flow* (m^3/s)	Baseflow volume %	Baseflow underpeak %	Peak flow* (m^3/s)	Baseflow volume %	Baseflow underpeak %
1987	July	100	28	26.6	33%	3%	12.3	33%	4%	na	na	na	137.8	28%	23%
1988	July	85	39	na	na	na	7.3	54%	29%	na	na	na	118.0	40%	24%
2000	July	48	11	na	na	na	na	na	na	26.1	47%	17%	na	na	na
2002	August	60	6	na	na	na	na	na	na	29.0	42%	11%	na	na	na
2005	August	58	40	na	na	na	na	na	na	29.9	58%	39%	na	na	na
2007	July	84	68	na	na	na	na	na	na	37.1	55%	32%	na	na	na

Parameters												
	Cleared scarp				Coastal plain				Serpentine Falls upstream			
Event	Kc	m	IL (mm)	RoC	Kc	m	IL (mm)	RoC	Kc	m	IL (mm)	RoC
1987	6.2	0.85	5	0.38	12.0	0.85	5	0.25	2.0	0.85	5	0.06
1988	6.2	0.85	5	0.37	12.0	0.85	5	0.25	2.0	0.85	5	0.06
2000	6.2	0.85	5	0.35	12.0	0.85	5	0.25	2.0	0.85	5	0.06
2002	6.2	0.85	5	0.35	12.0	0.85	5	0.25	2.0	0.85	5	0.06
2005	6.2	0.85	5	0.35	12.0	0.85	5	0.25	2.0	0.85	5	0.06
2007	6.2	0.85	5	0.36	12.0	0.85	5	0.25	2.0	0.85	5	0.06

**Peak recorded flow before baseflow separation*

Results from RORB modelling of the 1987 event produced peak flows and hydrographs that were consistent with observed data at Serpentine Falls and Mundlimup. For the 1988 event, only Mundlimup data was available, and the modelled peak flows were close to observed data but the timing of the event and shape of the hydrograph were quite different, probably due to the rainfall data available.

The 1987 and 1988 events produced volumes and peak flows which are consistent with the Serpentine River's contribution to flows at the Dog Hill gauge. RORB modelling indicated that the Serpentine River produced 29% and 33% of the event volume observed at the Dog Hill gauge for the 1987 and 1988 events. The Serpentine River catchment area above Lowlands is 39% of the Dog Hill gauge catchment area which includes Birrega Main Drain. The event volume, magnitude and hydrograph shape are all consistent with observations at the Dog Hill gauge.

For the smaller flow events in the 2000s, event volume was variously over- and under-estimated at the Lowlands gauge. This is probably due to the patchy nature of rainfall in these smaller events, and may also be a result of non-varying *RoC* and initial loss.

The *RoC* and *Kc* for the coastal plain portion of the RORB model were calculated through calibration of the events from the 2000s. The hydrograph shape is particularly sensitive to changes in *Kc* for the lower subareas just before the model outlet at Lowlands gauge. A *Kc* value of 12 produced the best results on average across all of the events. The *RoC* was set to 0.25 to account for the sandy soils of the coastal plain. This value is a mid-point between the calibrated value of 0.08 used for Peel Main Drain, and the recommended 0.40 regional parameter value for 'foot hills' for a 50 mm event. It is also consistent with the coefficient of runoff used for design and calibration events in the Birrega and Oaklands hydraulic model (J Hall, pers. comm., July 2013).

4.3 Design rainfall

Design rainfall depth

Hall (2014) recently developed design rainfall at Byford for the Birrega flood model, and this data is appropriate for use in the Peel Main Drain model. Design intensity frequency duration (IFD) information was calculated using the methods outlined in Australian Rainfall and Runoff (Pilgrim 2001) and CRC Forge (DoE 2004).

AUS-IFD was used to generate IFD data for all events of ARI from 5–50 yr, with duration of 24 hours or less. AUS-IFD is a program which calculates the design average rainfall intensities and temporal patterns for any location in Australia, and is located on the Bureau of Meteorology website (BOM 2011; http://www.bom.gov.au). The procedure for the calculation of rainfall intensity is described in Chapter 2 of Australian Rainfall and Runoff (AR&R1987; Pilgrim 2001).

WA-CRC Forge was used to derive the rainfall depth for the 50–500 yr ARI events for durations > 24 hours (only events of 24–72 hr duration are available from the CRC Forge database). The WA CRC-Forge '*EXTRACT*' computer program has been produced to facilitate the extraction of large rainfalls from the Western Australian database (Department of Environment 2004).

For events shorter than 24 hr duration and greater than a 50 yr ARI, an interpolation of values based on the relative magnitude of the design event to the 50 yr 24 hr event was used to derive the rainfall depth for the 100–500 yr ARI events. Events of duration > 24 hr and < 50 yr ARI were interpolated using the same technique.

Design rainfall depths and intensities are shown in Table 4-5. A plot of event rainfall versus event duration is shown in Figure 4-5, and a probabilistic plot of annual exceedance versus event rainfall (on a log scale) is shown in Figure 4-6. As both plots show a consistent smooth response for all event durations and ARI categories, the IFD information is considered suitable to use in the design hydrology.

Table 4-5: Rainfall IFD data used in modelling

Duration (hr)	ARI (1 in y) Event rainfall (mm)							ARI (1 in y) Rainfall intensity (mm/hr)						
	5yr	10yr	20yr	50yr	100yr	200yr	500yr	5yr	10yr	20yr	50yr	100yr	200yr	500yr
6	50	56	64	77	88	101	122	8.4	9.3	10.7	12.8	14.7	16.8	20.3
12	65	72	82	99	113	130	157	5.4	6.0	6.8	8.2	9.4	10.8	13.0
24	84	94	106	124	142	164	197	3.5	3.9	4.4	5.2	5.9	6.8	8.2
36	93	104	118	137	156	177	212	2.6	2.9	3.3	3.8	4.3	4.9	5.9
48	100	112	126	147	166	188	223	2.1	2.3	2.6	3.1	3.5	3.9	4.6
72	111	124	140	164	185	207	242	1.5	1.7	1.9	2.3	2.6	2.9	3.4

values taken from CRC Forge (DoE 2004) (green)

values taken from ARR (Pilgrim 2001) (orange)

interpolated values (grey)

Sourced from Hall (2013) using AustIFD and CRC forge

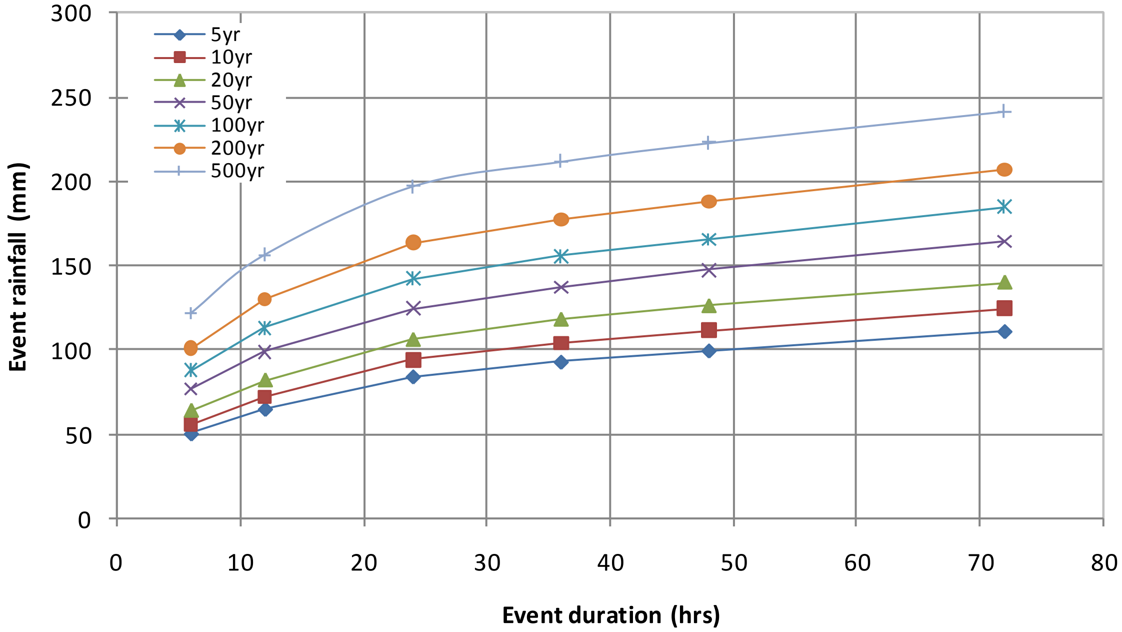


Figure 4-5: Rainfall IFD data plotted with event duration versus event rainfall

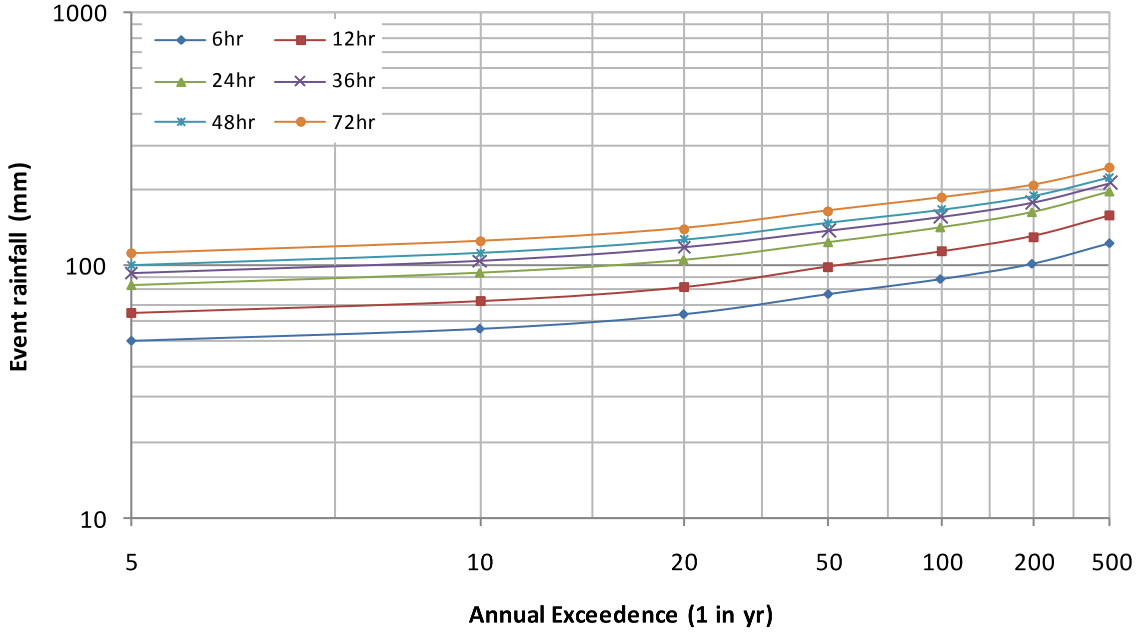


Figure 4-6: Rainfall IFD probabilistic plot for annual exceedance versus log event rainfall

Temporal patterns

Temporal patterns were extracted from the AusIFD software and applied to the rainfall depth data. The methodology used to calculate design temporal patterns is described in Book II Section 2 of AR&R1987. The method provides a separate temporal pattern for events with ARI < 30yrs and events with ARI > 30 yr. The temporal patterns for the design rainfall events are shown in Table 4-6.

Table 4-6: Design rainfall temporal pattern

	Temporal pattern for events <30 yr ARI						Temporal pattern for events >30 yr ARI					
Duration	6hr	12hr	24hr	36hr	48hr	72hr	6hr	12hr	24hr	36hr	48hr	72hr
Timestep	0.5hr	0.5hr	1hr	2hr	2hr	4hr	0.5hr	0.5hr	1hr	2hr	2hr	4hr
	Percent of design rainfall											
1	9.1	13.8	13.9	15.2	14.1	33.1	9.1	11.4	11.6	12.9	11.7	27.0
2	18.3	27.0	26.4	10.5	28.4	16.7	16.1	21.6	21.2	9.3	22.8	14.4
3	4.2	8.5	8.7	28.9	8.5	10.5	5.4	7.3	7.4	23.4	7.3	9.5
4	30.6	4.3	7.1	7.4	6.0	7.8	25.3	4.4	6.4	7.0	5.7	7.5
5	12.9	6.7	4.6	6.1	6.4	5.4	12.0	6.0	4.8	6.1	5.7	5.9
6	6.4	5.5	7.0	5.5	5.3	6.9	6.9	5.2	6.7	5.8	5.3	7.1
7	4.3	4.2	3.7	3.9	4.8	2.7	5.3	4.6	4.0	4.6	5.0	3.5
8	5.3	4.9	5.6	4.8	3.4	4.3	6.1	4.8	5.6	5.3	3.9	5.0
9	3.3	3.7	3.1	1.7	1.0	1.3	4.6	4.2	3.7	2.6	1.6	2.1
10	2.3	1.6	3.6	1.0	1.1	1.7	3.5	2.3	4.1	1.7	1.7	2.5
11	1.9	1.8	2.6	2.0	2.1	2.1	3.1	2.5	3.3	2.9	2.7	2.9
12	1.4	1.4	2.2	2.5	1.3	0.3	2.6	2.1	2.9	3.4	1.9	0.9
13	0.0	3.1	1.9	3.6	0.8	0.7	0.0	3.6	2.6	4.4	1.4	1.4
14	0.0	2.7	1.6	2.9	4.1	0.5	0.0	3.3	2.3	3.8	4.5	1.2
15	0.0	2.3	1.4	1.1	1.5	0.6	0.0	2.9	2.1	1.8	2.1	1.4
16	0.0	2.0	1.2	0.7	2.4	3.4	0.0	2.7	1.9	1.3	3.0	4.2
17	0.0	1.2	1.0	1.4	0.9	1.1	0.0	1.9	1.6	2.3	1.5	1.9
18	0.0	0.9	0.6	0.8	1.8	0.9	0.0	1.6	1.1	1.4	2.5	1.6
19	0.0	0.4	0.9	0.0	0.7	0.0	0.0	0.7	1.5	0.0	1.2	0.0
20	0.0	1.1	0.3	0.0	2.8	0.0	0.0	1.8	0.5	0.0	3.3	0.0
21	0.0	1.0	0.5	0.0	0.6	0.0	0.0	1.7	0.9	0.0	1.2	0.0
22	0.0	0.7	0.9	0.0	0.6	0.0	0.0	1.2	1.6	0.0	1.3	0.0
23	0.0	0.7	0.5	0.0	0.7	0.0	0.0	1.3	1.0	0.0	1.3	0.0
24	0.0	0.5	0.7	0.0	0.7	0.0	0.0	0.9	1.2	0.0	1.4	0.0

Areal reduction factors

The design rainfall discussed in the previous section was derived from analysis of rainfall occurring in a single point in space. Over large areas, the average rainfall depth across an entire catchment must be considered. The design rainfall across an entire catchment is related to design rainfall through a point using an areal reduction factor (ARF).

The ARFs described in AR&R1987 were developed in the United States and, in many regions of Australia, revised ARFs appropriate for use locally have been developed. For parts of Western Australia, the CRC Forge technical manual (Durrant & Bowman 2004) provides methods to estimate ARFs for catchments between 1 and 10 000 km^2. Chapter 6 of the manual describes the appropriate method for calculation of ARFs for catchments in the south-west. This method was used to calculate the ARFs for the two external RORB models, and the internal hydraulic model which includes the north-east Baldivis area. For annual series in the state's south-west, the equation provided by CRC forge does not vary ARF with event magnitude (Table 4-4).

Table 5-4: Areal reduction factors (ARFs) for the hydraulic model area, the Peel Main Drain RORB catchment, and the Serpentine RORB catchment

Catchment name	Catchment area (km^2)	ARFs for duration: 6	12	24	36	48	72	96	120
Peel Main Drain (RORB)	58.6	0.89	0.93	0.95	0.96	0.97	0.98	0.98	0.98
North-east Baldivis hydraulic	68.1	0.88	0.92	0.95	0.96	0.97	0.97	0.98	0.98
Serpentine (RORB)	219.4	0.85	0.89	0.93	0.94	0.95	0.96	0.97	0.97

Based on equation 6.2 in CRC forge technical manual (Durrant and Bowman 2004)

4.4 Design hydrology

The areally reduced IFD design rainfall datasets were imported into a RORB design template for the Serpentine and Peel Main Drain catchments. Design hydrology was calculated in a four step process.

1. The RORB model was run for the design events using the calibrated and regional parameters of *Kc* described in sections 4.1 and 4.2, and by varying *RoC* with event duration or rainfall depth.
2. A baseflow component was added to the resulting design flows to produce a total flow for each event.
3. The critical duration storm was calculated for each average recurrence interval.
4. Design flows were compared to FFA for Hope Valley and Serpentine Falls to validate the *RoC* selected in step 1.

Runoff coefficient for design events

As event size increases, it is likely that catchment wetness and saturation excess will increase and therefore, result in a larger *RoC*. To account for this affect, *RoC* must be varied with event size, as indicated by Pearce (2006) in development of regional runoff coefficients.

In the case of the Peel Main Drain RORB model, *RoC* was varied with ARI using the parameters shown in Table 4-7. For the Serpentine RORB model, *RoC* was varied with ARI for the area upstream of Serpentine Falls, and with rainfall depth and ARI for the cleared scarp and coastal plain areas. The variable values of *RoC* are shown in Table 4-7.

The design runoff coefficients are slightly smaller than those used in the RORB model calibration for equivalent event sizes, in order to match the FFA curves derived earlier. This is because the rainfall temporal patterns of the calibration events do not have in individual time-steps rainfall intensities that are as high as those of the design rainfall events.

Table 4-7: RoC used for design events for the Peel Main Drain RORB model

ARI	Peel Main Drain	ARI	Serpentine Falls upstream	Areally reduced rainfall	Cleared scarp	Coastal plain	Areally reduced rainfall	Cleared scarp	Coastal plain
(1 in yr)	RoC	(1 in yr)	RoC	(mm)	RoC	RoC	(mm)	RoC	RoC
5	0.050	5	0.035	50	0.35	0.25	140	0.40	0.30
10	0.060	10	0.040	60	0.36	0.26	150	0.41	0.31
20	0.065	20	0.045	70	0.36	0.26	160	0.42	0.31
50	0.075	50	0.055	80	0.37	0.27	170	0.42	0.32
100	0.080	100	0.060	90	0.37	0.27	180	0.43	0.33
200	0.085	200	0.065	100	0.38	0.28	190	0.43	0.33
500	0.090	500	0.065	110	0.39	0.29	200	0.44	0.34
				120	0.39	0.29	210	0.44	0.34
				130	0.40	0.30	220	0.45	0.35

Design peak flows and FFA

Table 4-8 shows peak discharge for critical duration storms at Hope Valley, Lowlands and Serpentine Falls gauging stations, in comparison with FFA results.

Table 4-8: Comparison of flood frequency analysis and design events

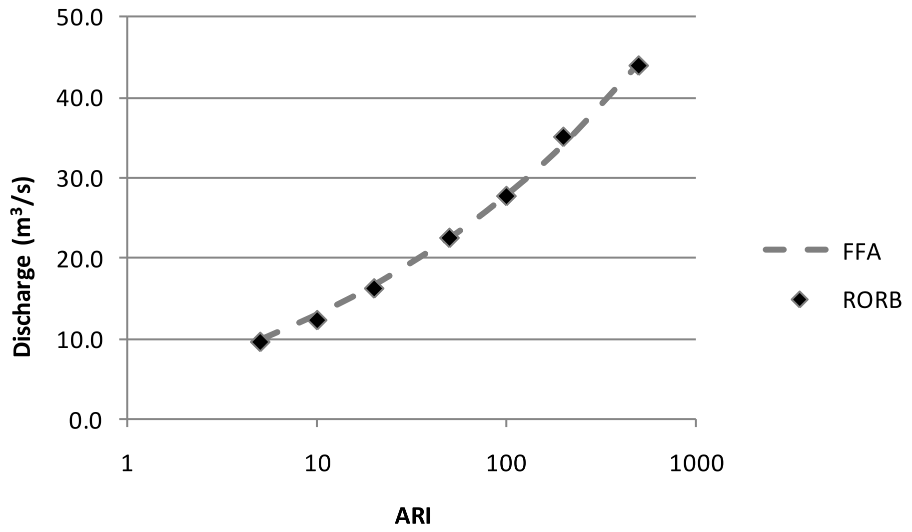
FFA and RORB CD with non-variable RoC									
Average recurrance interval	Hope Valley Peel Main Drain (614013)				Lowlands Serpentine River (614114)		Serpentine Falls Serpentine River (614072)		
(1 in yr)	FFA*	FFA**	RORB	CD	RORB	CD	FFA	RORB	CD
5	1.1	1.0	1.0	24hr	34.6	72hr	9.8	9.7	24hr
10	1.5	1.3	1.4	24hr	40.5	72hr	12.9	12.4	24hr
20	2.0	1.5	1.7	24hr	47.9	72hr	16.6	16.3	24hr
50	2.7	1.9	2.2	24hr	53.2	24hr	22.5	22.6	6hr
100	3.3	2.1	2.8	24hr	65.2	24hr	27.8	27.7	6hr
200	4.0	2.4	3.5	24hr	78.7	24hr	34.1	35.1	6hr
500	5.1	2.7	4.6	24hr	101.4	24hr	44.2	43.9	6hr

**Including 1992 event in annual peak flow series*

***Excluding 1992 event in annual peak flow series*

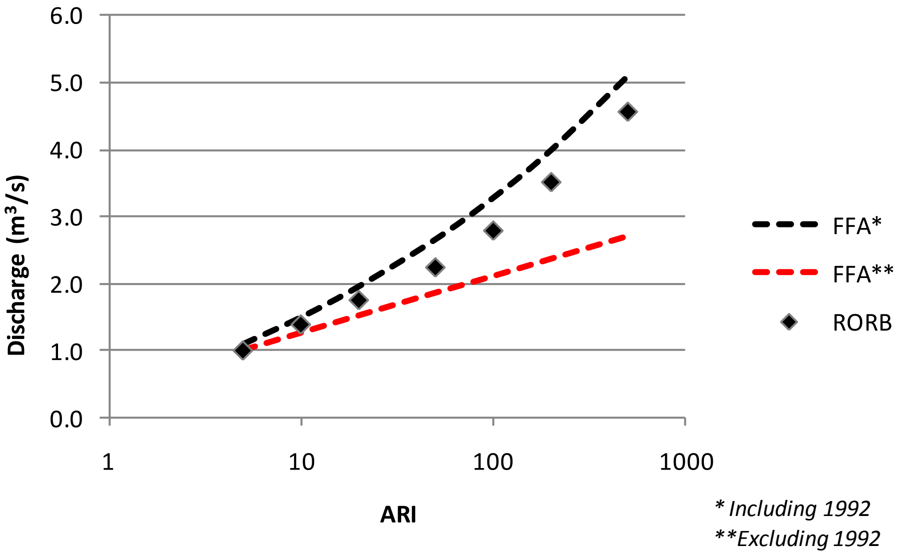
The critical duration storm for the Serpentine RORB model at the Lowlands gauge is 72 hr for ARIs of 5, 10 and 20 yr, and 24 hr for the other ARIs. FFA was not undertaken for the Lowlands gauge due to the short length of record. However, the 5 yr and 10 yr ARI events result in a 35 m^3/s and 40.5 m^3/s peak discharge, which is consistent with the maximum flow recorded in the last 13 years at the site, of 37.0 m^3/s. Higher in the catchment at Serpentine Falls, critical durations were 24 hr for the smaller events, and 6 hr for the larger events. Figure 4-7 shows that the results from the RORB modelling are consistent with the FFA for Serpentine Falls.

Figure 4-7:Comparison of FFA and RORB critical duration peak flow at Serpentine Falls



For the upper Peel Main Drain RORB model at Hope Valley gauge, the critical duration is 24 hr for all events. Figure 4-8 shows that the RORB modelled flows are within the bounds of the two FFA curves (with and without the 1992 event). Note that the 1992 event rainfall at Hope Valley was equivalent to a 100 yr ARI 24 hr storm, and resulted in an measured peak flow of 2.5 m^3/s. This is consistent with the RORB modelled peak flows of 2.8 m^3/s for a 100 yr event. Due to the short and dry record available for the FFA at Hope Valley, it is likely that the LPIII distribution which excludes the 1992 event underestimates peak flows while including the event overestimates peak flows. Therefore the design *RoC* parameters were selected to fall between the two FFA curves.

Figure 4-8:Comparison of FFA and RORB critical duration peak flow at Hope Valley



Comparison with previous studies

Design flows estimate by RORB modelling were compared to several previous studies in the area. On the Peel Main Drain, DoW (2009) provide modelled design flows for a number of locations in the upper portion of the catchment, including at the Hope Valley gauge, and at the catchment outlet at Millar Road. WAWA (1990) and SKM (2010) report peak discharges along the Serpentine River. Table 2-1 shows the estimated peak discharge at these locations for the current and previous studies.

Results from the Jandakot DWMP (DoW 2009) are similar to the RORB modelled design flows from the current flood study for the 10 yr ARI at both Hope Valley and Millar Rd. For the 100 yr event the current study estimates relatively larger flows (7.5 m^3/s compared to 4.8 m^3/s). It seems strange that the difference in peak flow between the 10 and 100 yr ARIs reported in the Jandakot DWMP is so little but this could be accounted for by the storage components of the infoworks model which were not included in the RORB model of the current study. The design peak discharges are fairly comparable between the Jandakot DWMP, the RORB model and the FFA for Hope Valley.

The results at Serpentine Falls and the South Western Highway between the current and previous studies are very different. These differences have been discussed in Section 3 and can be attributed to the differences in methods used for calibration of the RORB models.

Both WAWA (1990) and SKM (2010) calibrated to a single recorded flood level at the South Western Highway. To meet the reported stage at this location very high discharge is required in the Serpentine River, which in turn influences the RORB calibration. For this study (and for Hall 2014) calibration of the RORB model was based on the gauged data at the Serpentine Falls and Mundlimup gauges, with the Lowlands and Dog Hill gauges used as verification further downstream. The RORB modelled flows were consistent with the FFA for Serpentine Falls, and the calibrated parameters produced acceptable hydrographs for all events at all gauges (see Appendix B). Given that the 100 yr ARI estimated by SKM (2010) at Dog Hill using FFA was 154 m^3/s, it seems unlikely that the peak discharge for the same magnitude event could be 166 m^3/s at the South Western Highway, considering that the South Western Highway catchment is around 1/3 the size of the Dog Hill catchment, and is mostly forested. This discrepancy in discharge seems too large to be accounted for by storage effects along the Birrega Main Drain. So, the revised peak discharge estimates presented in this study will be used.

Table 4-9: Comparison of design flows with previous studies

River	Location	Study	10yr ARI peak discharge (m^3/s)	100yr ARI peak discharge (m^3/s)
Peel Main Drain	Hope Valley	Current	1.4	2.8
Peel Main Drain	Hope Valley	DoW, 2009	1.2	1.6
Peel Main Drain	Millar Rd	Current	3.8	7.5
Peel Main Drain	Millar Rd	DoW, 2009	4.3	4.8
Serpentine River	Serpentine Falls	Current	13	28
Serpentine River	South Western Hwy	SKM, 2010	91	166
Serpentine River	South Western Hwy	WAWA, 1990	80	118
Serpentine River	Dog Hill	Current	107	147
Serpentine River	Dog Hill	SKM, 2010	107	154
Serpentine River	Dog Hill	WAWA, 1990	141	215

Design hydrology for external catchments

Design hydrology was simulated for the upper Peel Main Drain and Serpentine RORB models using the parameters described above, for 5, 10, 20, 50, 100, 200 and 500 yr events for 6, 12, 24, 36, 48 and 72 hr storm durations. Figure 4-9 and Figure 4-10 show the design flows at the RORB catchment outlets for the Peel Main Drain and Serpentine River, including the baseflow component. Note that the Peel Main Drain model has only one outlet whereas the Serpentine model includes one main outlet at the Lowlands flow gauge, and several small outlets located on the coastal plain (Figure 4-4).

For the upper section of the Birrega main drain catchment, design discharge was extracted from the Birrega and Oaklands MIKE FLOOD model (Hall 2014). Design events were available for the 5, 10, 20, 100 and 500 yr events for 6, 12, 24, 36, 48 and 72 hr storm durations. The advantage of using the pre-existing MIKE FLOOD model is that the storage effects of the coastal plain around Birrega Main Drain are accounted for in the hydraulic model. For the larger (> 100 yr ARI) events, flow from the Birrega MIKE FLOOD model enters the northern boundary of the north-east Baldivis model in the form of both channelised flow, and overland flow. As such, flow from design events for this boundary is input in the form of isolated flow sources, and as a hydrograph in the Birrega MIKE 11 channel.

For the 100 yr 24 hr event, overtopping of the Birrega Main Drain at Duck Pool was simulated; this directs up to 20 m^3/s from the drain into the north-east Baldivis area. For the 100 yr 24 hr complete levee failure scenario, discharge from the Birrega Main Drain into north-east Baldivis is up to 55 m^3/s and is a significant flood hazard for the area.

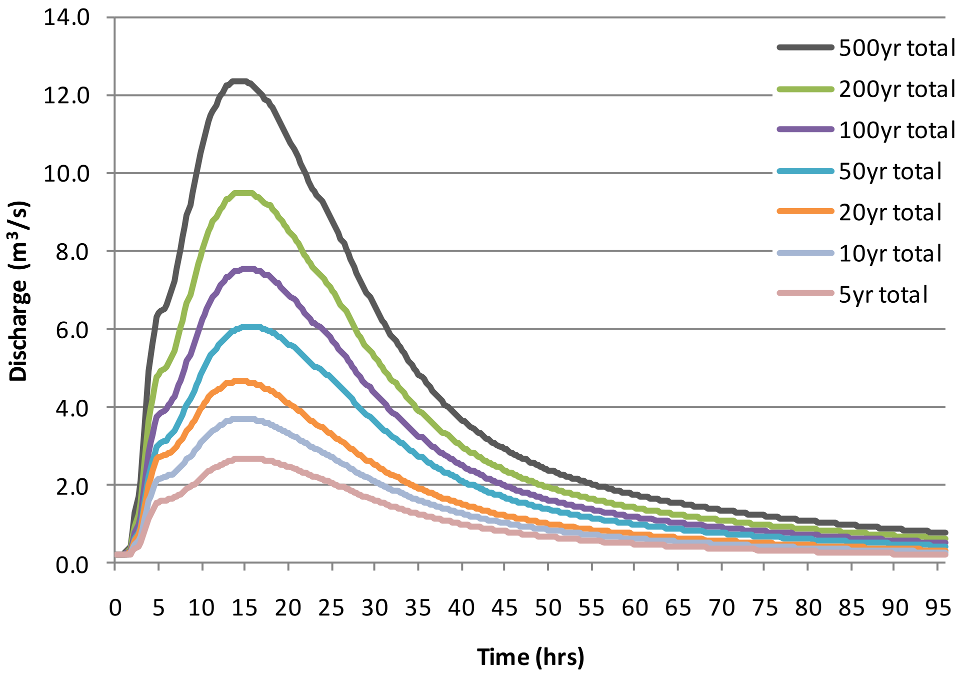


Figure 4-9: Upper Peel Main Drain RORB design flows for the 24 hr duration for various ARIs

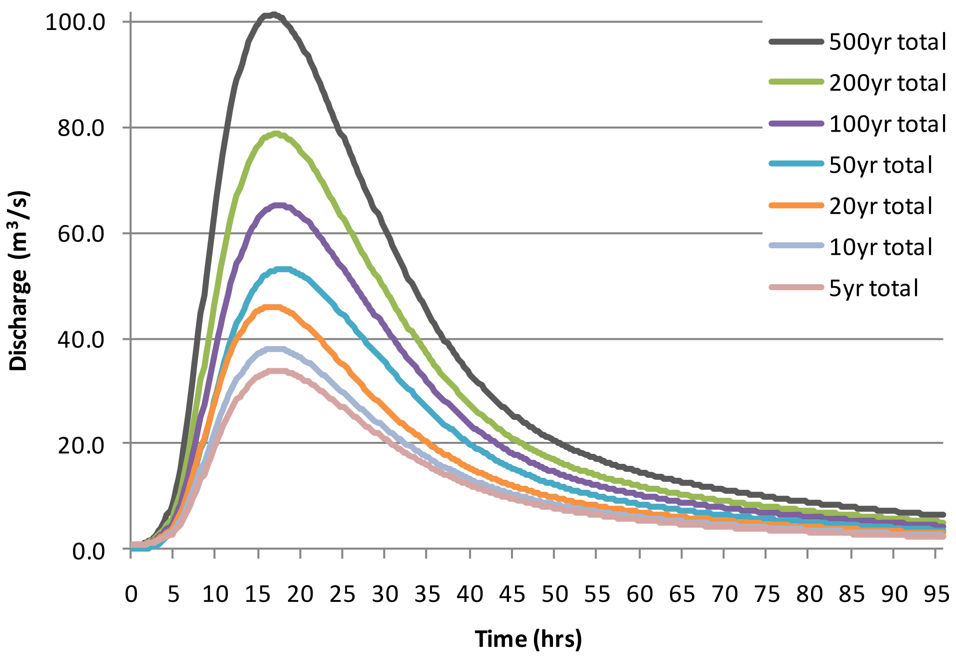


Figure 4-10: Serpentine RORB design flows at Lowlands for the 24 hr duration for various ARIs

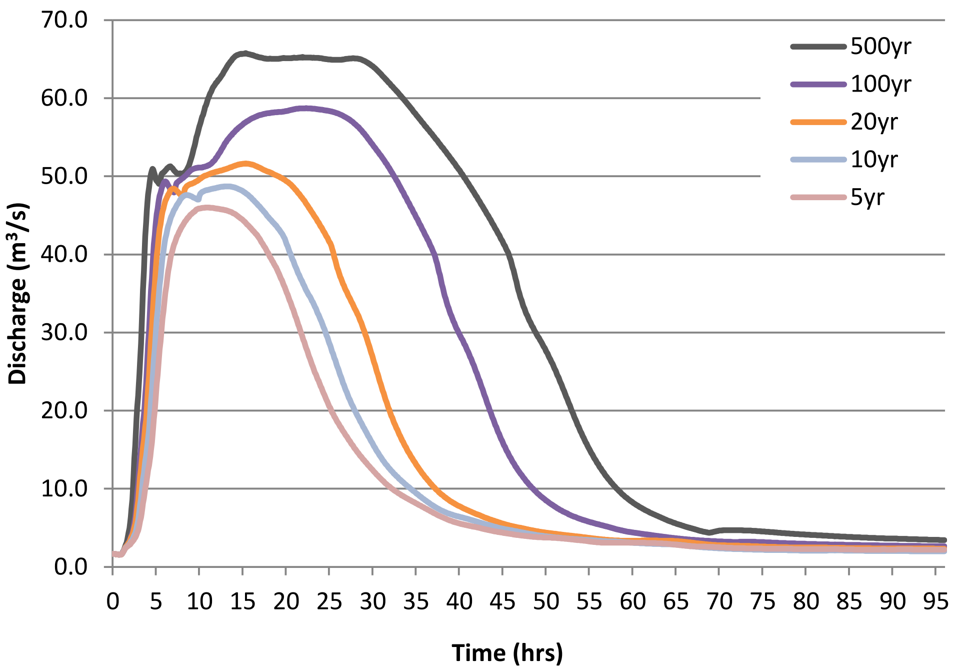


Figure 4-11: Design flows (channelised flow only) from the Birrega Main Drain, sourced from the Birrega and Oaklands hydraulic model

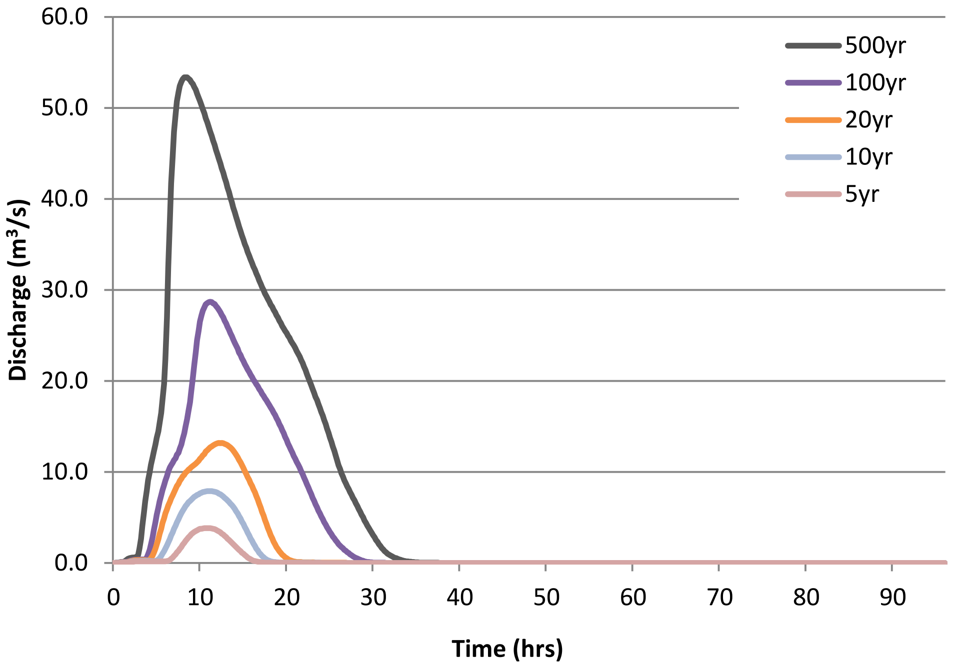


Figure 4-12: Overland flow adjacent to Oaklands Drain, sourced from the Birrega and Oaklands hydraulic model

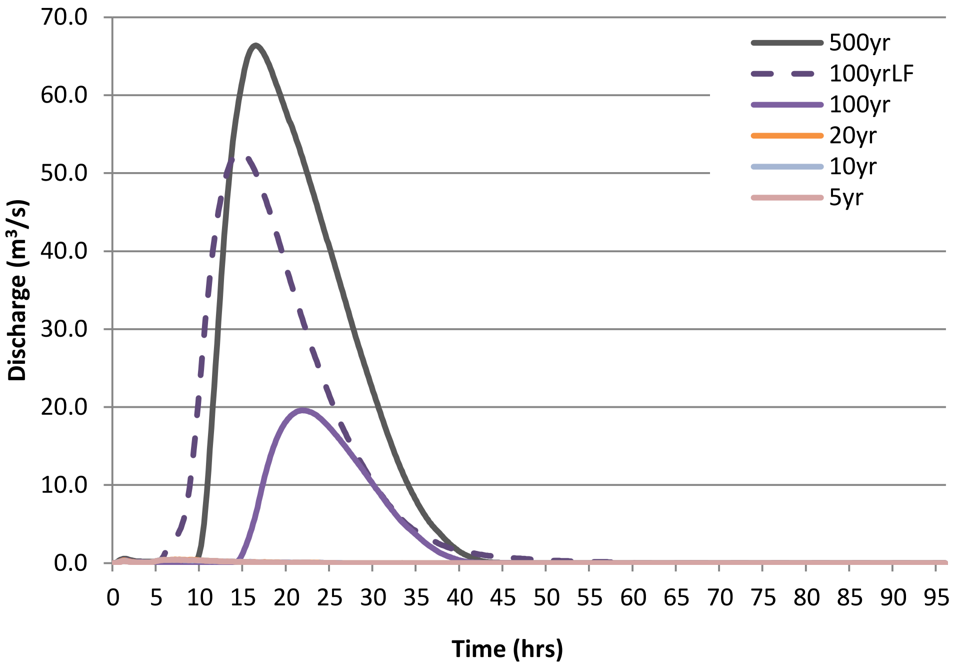


Figure 4-13: Overland flow to the west of Birrega Drain, sourced from the Birrega and Oaklands hydraulic model, 100 yr LF indicates 100 yr flows in the event of levee failure

Climate change

The influence of climate change on design rainfall was not considered as part of this study. There is a high level of uncertainty associated with the output of global climate models (GCMs) and climate projections are not available at the subdaily timestep for many of the datasets available in the Coupled Model Intercomparison Project archives. In this respect, it is difficult to modify rainfall IFDs and temporal patterns based on the results of GCMs.

The Bureau of Meteorology and Engineers Australia are currently reviewing AR&R1987, and future editions may address the influence of climate change. Adjustment to design rainfall on the basis of climate change should not be undertaken until the AR&R review is completed, and any revised changes are agreed by the engineering profession across WA.

5 Hydraulic modelling

The floodplain mapping component of the project involved development of a two-dimensional (2D) hydraulic model of the study area, as defined by the hydraulic model boundary shown in Figure 1-1. The hydraulic model was used to simulate flooding extent across the study area, and determine water level and discharge at various locations.

The hydraulic model was built using the MIKE FLOOD modelling package (2012 release), which enables coupling of a 2D hydraulic model (MIKE 21) with a one-dimensional (1D) hydraulic model (MIKE 11). The 2D component was used to model overland flow, rainfall-runoff and infiltration; and the 1D component was used to model the main drains and culverts within the study area.

The hydraulic model was developed in three phases:

- The **model design and construction** involved developing the model topography, boundary conditions, coupling 1D and 2D models, insertion of structures, development of resistance and infiltration grids, and numerical stabilisation.
- The **model calibration** involved calibrating to two events, one in 1987 and one in 2005. This enabled calibration to observed stage at Dog Hill gauging station for the 1987 event, and at Karnup Road gauging station for the 2005 event, with photographs also available for this event.
- **Model design runs** were completed after the model was satisfactorily stabilised and calibrated. This phase used the design rainfall and discharge data discussed in Section 5 for boundary conditions.

Sensitivity analysis was completed for important parameters including resistance values in the MIKE 21 and MIKE 11 models, infiltration parameters, and downstream boundary conditions on the Serpentine River.

5.1 MIKE 21 model construction

Model domain and boundary

The MIKE 21 model domain has a total area of 78 km^2, which is much larger than the area of interest around north-east Baldivis. The hydraulic model was extended to include the area east of the Serpentine and Birrega Drains to accurately model lateral storage in these drains, and the potential for levee failure. The model was extended around 2 km south of the confluence of the Peel Main Drain and Serpentine Drain to correctly simulate the storage and backwater effects at the confluence. The model boundary was designed to account for all fluxes into and out of the model domain.

MIKE 21 was configured using a 10 m resolution grid using a simulation timestep of 1.5 s. Flooding and drying depths were set to 5 mm and 2.5 mm. A global eddy viscosity of 3 m^2/s was used with a velocity based formulation.

Topography

The model topography was developed from the 2008 Swan Coastal Plain LiDAR dataset, and the 2010 LiDAR dataset which includes the Perth to Bunbury Highway. The topography was developed using the following steps:

1. The 2008 LiDAR was updated with the 2010 dataset along the highway.
2. Any holes within the LiDAR were filled using a zonal mean filter.
3. Any errors in the LiDAR were patched by reprocessing the original LAS files for the problem area.
4. The 1 m LiDAR DEM was converted to a 10 m resolution DEM using ArcGIS.
5. Land cells were introduced into the DEM within MIKE ZERO to define the model boundary, and to mask any cells which would be modelled by MIKE 11 – preventing dual conveyance in these areas.
6. Elevation was modified for individual cells as follows:
   a. For lateral links between MIKE 11 and MIKE 21, left and right levee bank elevations were derived based on MIKE 11, and MIKE 21 was updated to match these elevations to ensure the most realistic levee bank level.
   b. For standard links on road culverts, MIKE 21 coupled cells were modified to match the level of the culvert.
   c. For drains which were not modelled within MIKE 11 using either a channel or standard link, single cells were removed to allow flow across roadways.
   d. Several mid-sized drains not modelled in MIKE 11 were burnt into the DEM using minimum bed levels as one cell wide drains with cell-to-cell connectivity.
   e. Elevation was corrected for roads in some locations.

The resulting MIKE 21 topography is shown in Figure 5-1.

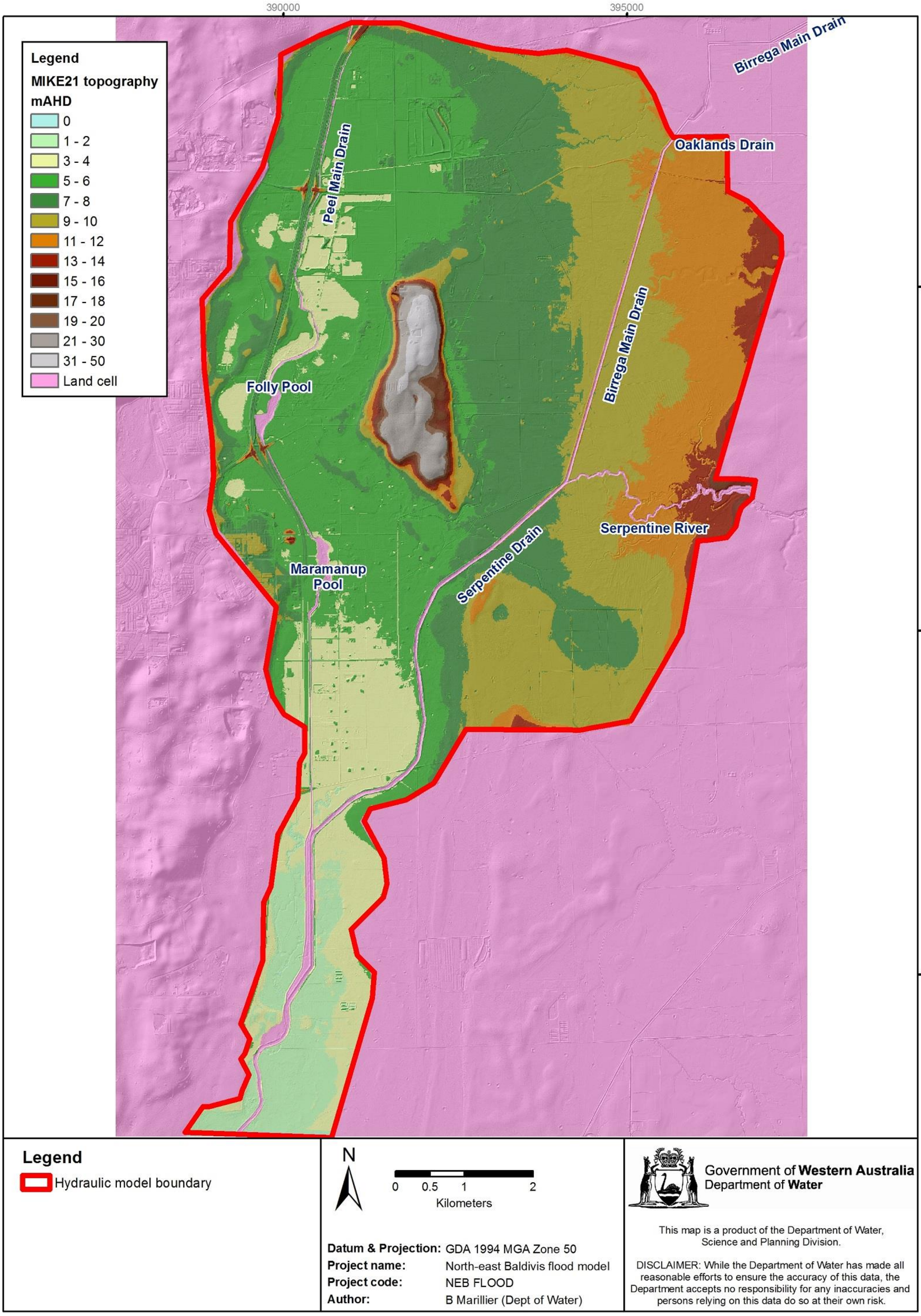


Figure 5-1: MIKE 21 model topography

Rainfall

The design rainfall developed in Section 5 was implemented within the hydraulic model using a direct-rainfall method. This method is appropriate for use within the study area given the very flat topography and the potential for cross-catchment flows. Design and calibration rainfalls were converted to a 2D time-series of rainfall within the hydraulic model domain. Rainfall was assumed to be spatially homogenous within the domain, however, a no-rainfall mask was applied at cells within MIKE 21 which were coupled to MIKE 11, which prevented numerical instabilities.

For the calibration events, the model rainfall was sourced from the Dog Hill pluviometer.

Resistance

Distributed resistance values were used in the MIKE 21 model based on three broad land-use classes; urban residential, roads, and open, vegetated, and cleared pasture. The model was only sensitive to changes in resistance values for the cleared pasture land use, which covered most of the hydraulic model area. Resistance values are reported as Manning's M, which is the inverse of Manning's n.

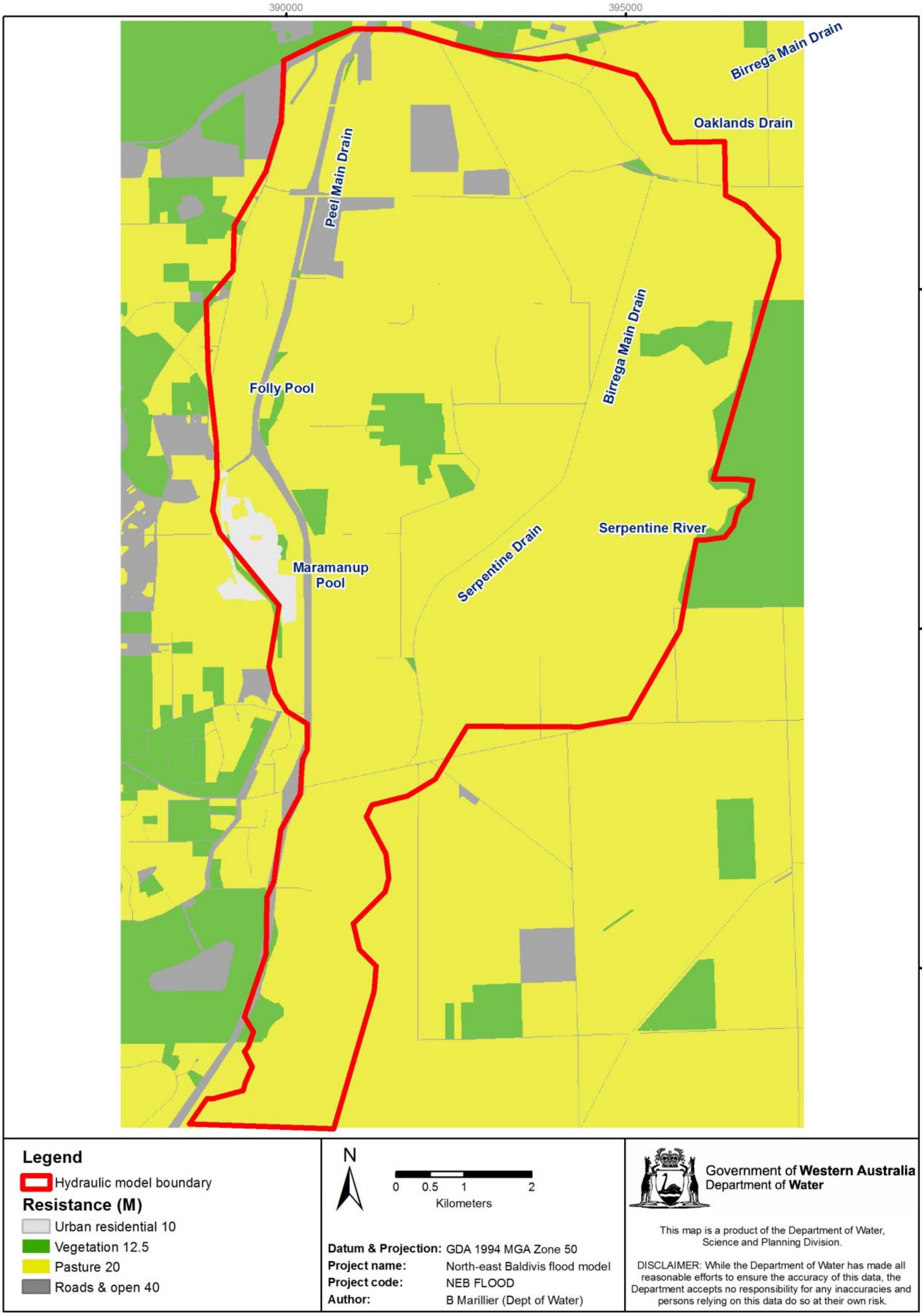


Figure 5-2: Resistance values used for design and calibration runs

Infiltration and inundation

Infiltration was modelled within the hydraulic model using distributed infiltration rates. The study area was divided into three categories, each with different infiltration parameters.

Groundwater inundation

The first category was defined by areas which experience waterlogging from groundwater. These areas were identified using the Lower Serpentine regional groundwater model (Marillier et al. 2012b), which provides better than 50 cm accuracy in modelled superficial groundwater head within the study area. For the calibration runs, the groundwater level was extracted for the first day of the modelled flood event. This level was then intersected with the 10 m model topography to determine inundated areas, and these areas were then mapped laterally to provide a mask where no infiltration would occur due to groundwater inundation. Thus realistic antecedent conditions were available for the 1987 and 2005 calibration event.

For the design runs, it was necessary to choose a realistic groundwater level to use as an antecedent condition. The average July maximum groundwater level (1981–2010) was used to define the extent of inundation for design runs, as July is the month of peak storm activity within the study area. Figure 5-3 shows the extent of groundwater inundation for design events.

Infiltration soil types

Soils within the study area were broadly classified into two categories based on the Department of Agriculture and Foods soil map unit database. The sandy phases of the Pinjarra, Spearwood and Bassendean soil groups were combined into a single category which would allow for some infiltration of direct rainfall. The second category included the poorly drained phases of the Pinjarra soil group, which have higher clay and organic content, and are generally located in depressions. Figure 5-3 shows the location of the three infiltration categories, including inundated areas.

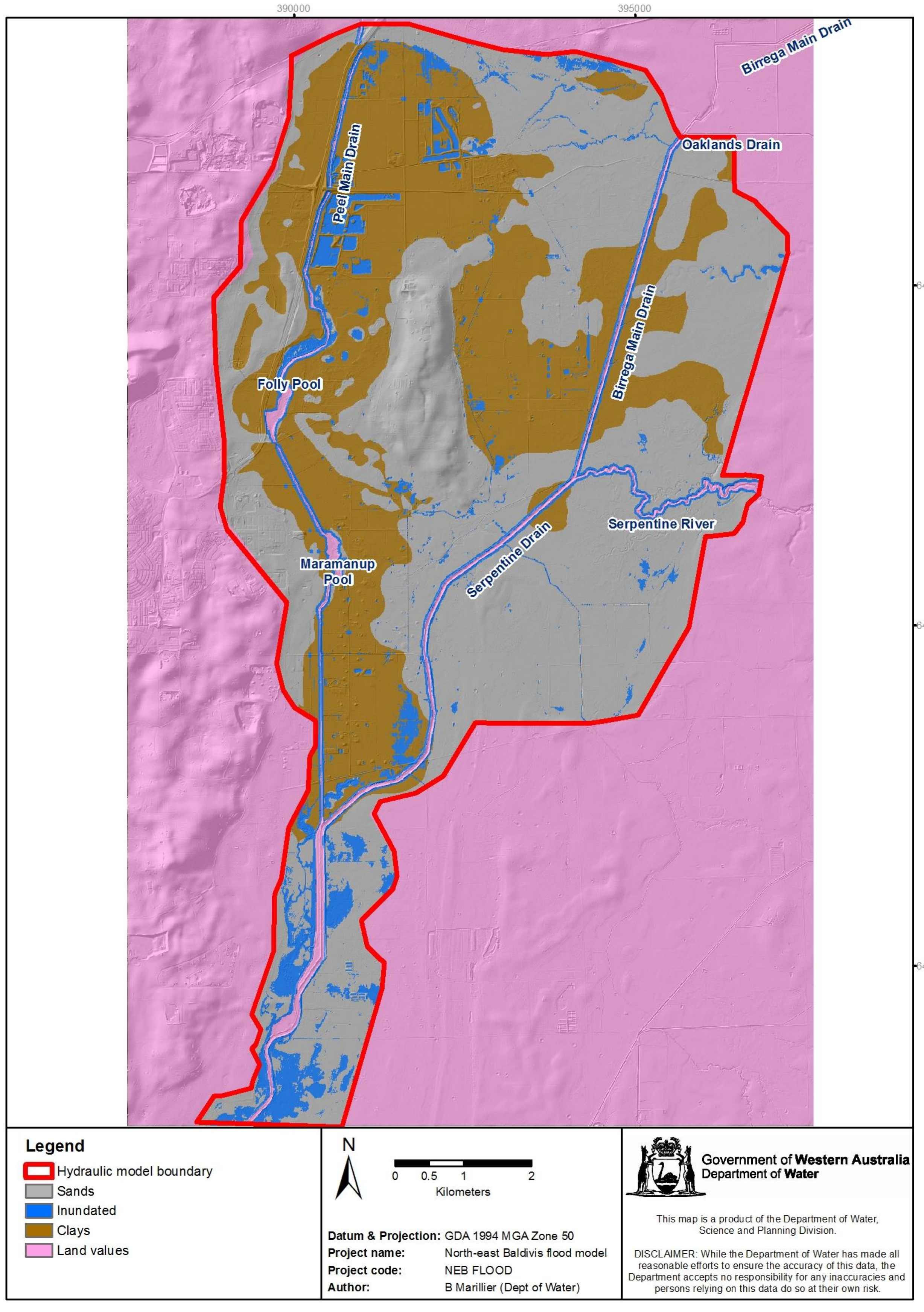


Figure 5-3: Infiltration classes

Infiltration rates

Infiltration rates are listed in Table 5-1. Infiltration rates are physically plausible for the respective soil groups, and resulted in satisfactory calibration for both the 2005 and 1987 events.

Table 5-1: Infiltration rates for inundated areas and soil groups

Group	Rate (mm/day)	Rate (m/day)	Comments
Inundated	0	0.00	
Clays	20	0.02	Poorly drained soil phases of the Pinjarra soil group
Sands	240	0.24	Sandy soil phases (Bassendean, Pinjarra, Spearwood)

Inflows from the RORB and Birrega MIKE FLOOD models

Discharge from the Serpentine RORB model was introduced within MIKE 21 using source points for six catchment RORB outflows. Two source points adjacent to the Birrega Drain were introduced to transfer overland flow from the Birrega MIKE FLOOD model. Inflows to the Peel Main Drain, Birrega Drain and Serpentine River main channels were included as boundary conditions within MIKE 11. The locations of the source points are shown in Figure 5-4, relative to the RORB and Birrega model boundaries.

Boundary conditions for overland flow

All inflow boundaries were included either as MIKE 21 source points, or as hydraulic discharge boundaries within MIKE 11. An overland flow open boundary was defined at the southern end of the Serpentine River, south of the Peel Main Drain confluence. This boundary was implemented as a 0 m AHD water level boundary. In sensitivity analysis increasing this boundary condition to 0.9 m did not influence water levels in the major drains and floodplain north of Karnup Road.

Antecedent conditions

An initialisation storm was simulated to develop initial conditions for the MIKE21 model. A rainfall depth of 20 mm was applied over a duration of 12 hr with a further 12 hr simulated after the rainfall event. The depth of overland water from the final timestep of this simulation was used for initialisation of all design simulations in MIKE21.

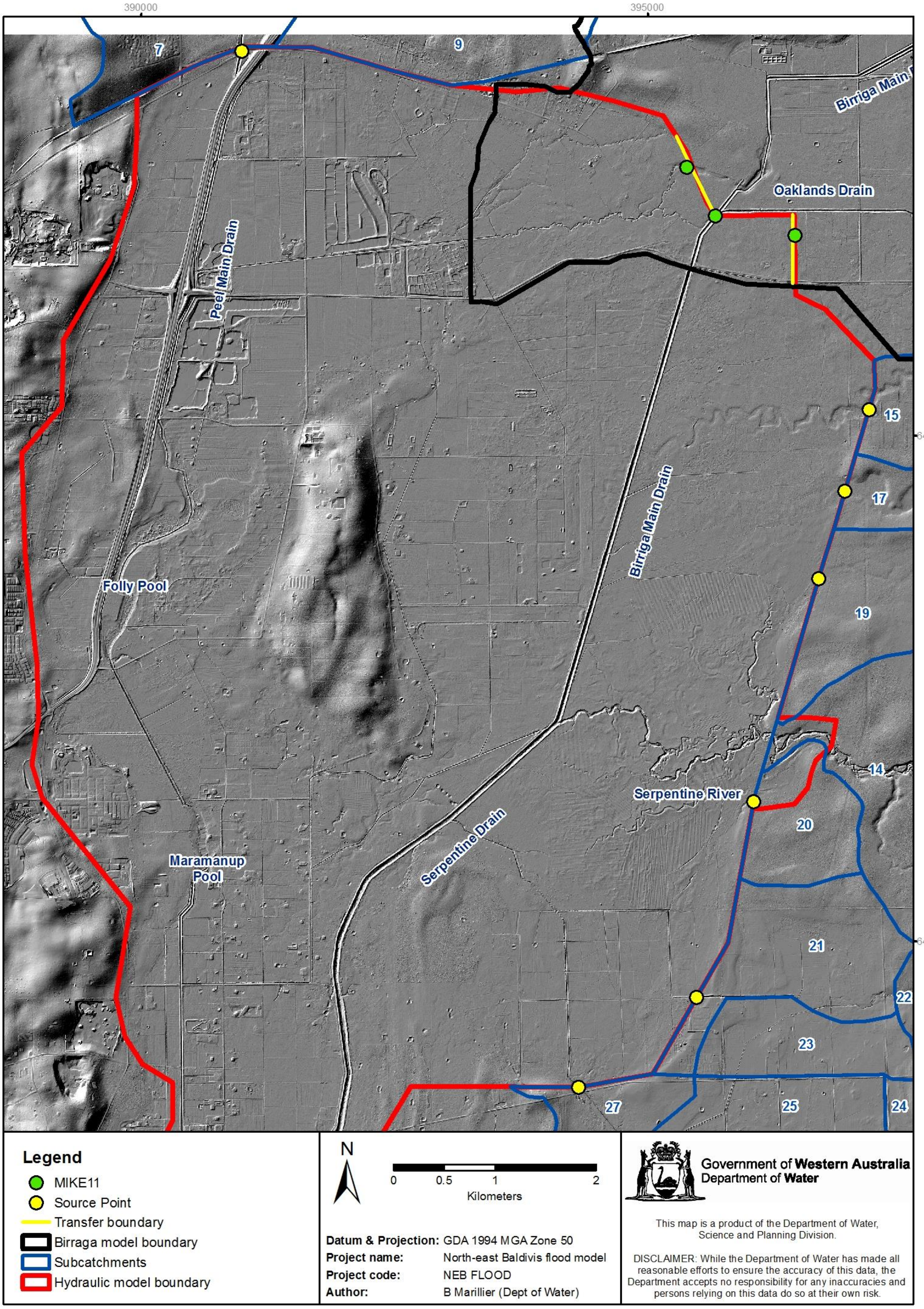


Figure 5-4: Source points within the hydraulic model

5.2 MIKE 11 model construction

The MIKE FLOOD modelling package allows for the coupling of MIKE 21 and MIKE 11. This enables MIKE 21 to simulate overland flow and rainfall runoff processes, while MIKE 11 simulates channel hydraulics for sub-grid-scale features. Within the study area, the Serpentine River, Birrega, Serpentine and Peel Main Drains are between 10 and 30 m wide, which means they cannot be accurately simulated using MIKE 21 at a 10 m resolution. So these channels were simulated using MIKE 11, with coupling to the MIKE 21 model to allow exchange of water between the channels and model topography.

Several culverts were introduced to simulate flow from the western side of the freeway, under the road, to the Peel Main Drain. Culverts were also introduced within the north-east Baldivis area, and along the Peel Main Drain MIKE 11 channel.

The MIKE 11 model was developed using two primary sources: the Department of Water 1 m resolution LiDAR dataset which was used to determine cross-sectional and network dimensions; and field surveys using an RTK GPS system which was used to measure culvert dimensions and invert levels.

Network

The DHI software MIKE 11GIS was used to define the drain centreline for the four major waterways within the study area (Figure 5-5).

Cross sections

Using the MIKE 11 network, cross-sectional information was captured within MIKE 11GIS for all channels within the study area. Cross-section locations were defined manually approximately every 50 m, using 1 m LiDAR dataset to ensure that all major changes in channel form and levee structure were captured. MIKE 11GIS was then used to extract cross-sectional levels directly from the LiDAR for use in the 1D model. Figure 5-5 shows the MIKE 11 network with the cross-section extraction locations.

The cross-section and network file 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 every 10 m, which ensures spatial equivalence between the MIKE 11 cross-sections and the grid-spacing in MIKE 21.

For the 100 yr ARI levee fail scenario, the right levee bank of the Birrega and Serpentine drains was removed for the first 7 km of 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. The MIKE 21 topography was modified to meet the modified cross-section levels.

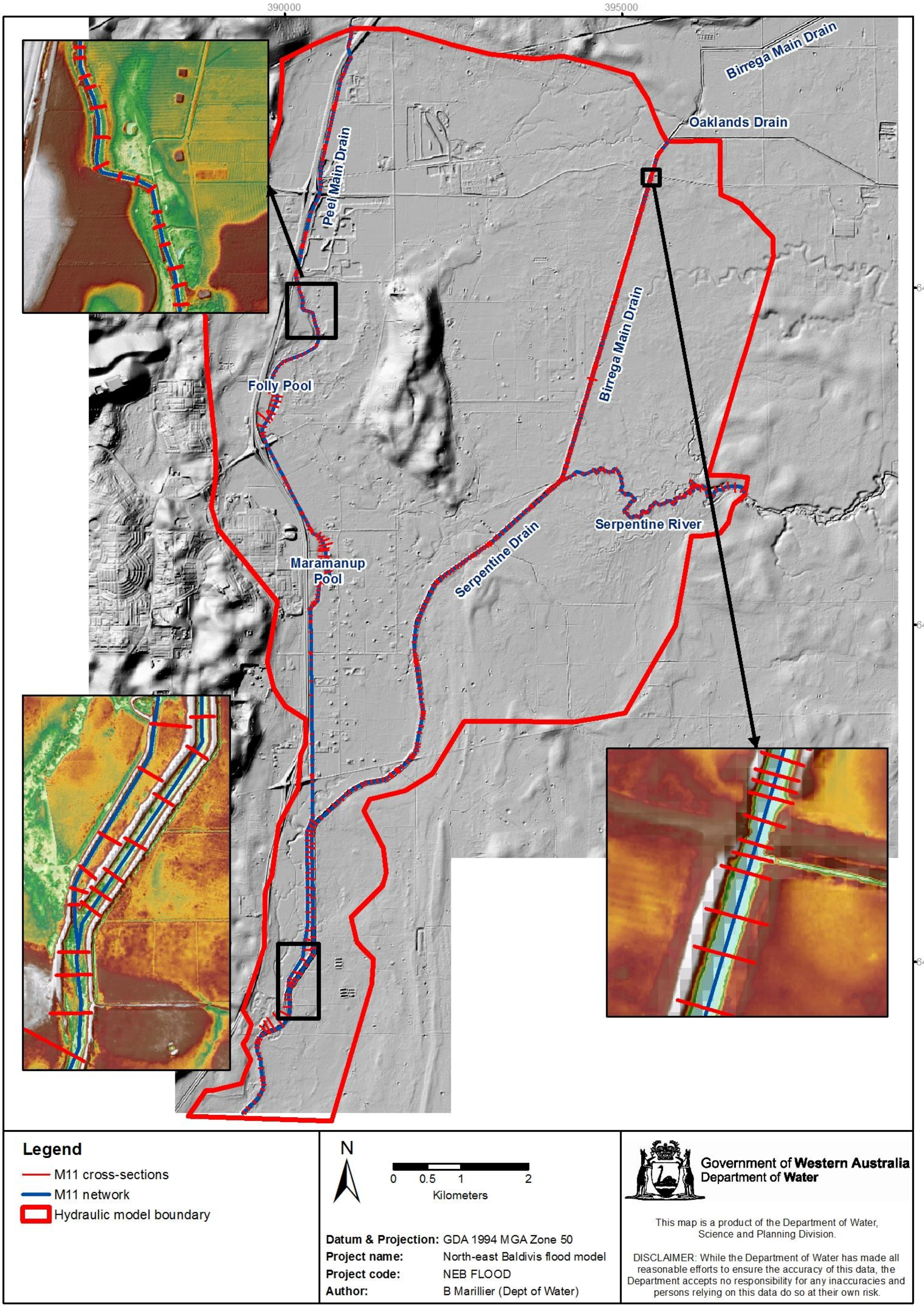


Figure 5-5: MIKE 11 channel network, cross-sections and structure locations – insets show gaps in levees, and the confluence of Serpentine and Peel Main Drains

Structures

More than 90 culverts and bridges were identified within the hydraulic model area for investigation. All of these locations were visited in the field to be assessed for inclusion within the model. Many of the locations had only very small culverts, or culverts which were overgrown or buried to the point that they would be ineffective in most flood events. Many of the small road bridges were modelled by modifying the MIKE 21 topography to remove the road section and decreasing the Manning's M locally to a value of 5.

Of the 90 structures, 22 culverts were identified for inclusion within the model. Five culverts were surveyed for inclusion along the Peel Main Drain; at the northern end of the study area beneath the freeway; and at Mundijong Road, Bertenshaw Road, Folly Road and Karnup Road. A further 13 culverts underneath the freeway running west to east were included, and four road culverts were added within the main study area between the freeway and the Birrega Main Drain.

Figure 5-6 shows which structures were modelled within MIKE 11, along with those that were excluded due to size or condition, and those which were incorporated within the MIKE 21 topography. The dimensions for all structures included in MIKE 11 are shown in Appendix C. Note that in some instances invert levels or slope were slightly modified in the numerical model to better fit the channel geometry, or reduce numerical instabilities.

All structures were modelled using the 'H-H conduit' method, which is numerically stable and appropriate for use in culverts that have a length greater than the diameter of the culvert. Using this method, the dimensions of the culvert were defined using closed cross-sections within the MIKE 11 model. Culverts were incorporated within the main MIKE 11 channel along the Peel Main Drain. The road culverts were incorporated within MIKE 11 using short branches within the network, with small cross-sections defined on either side of the road. These cross-sections were coupled to MIKE 21 as standard links to exchange flow across the roadway using the MIKE 11 structure.

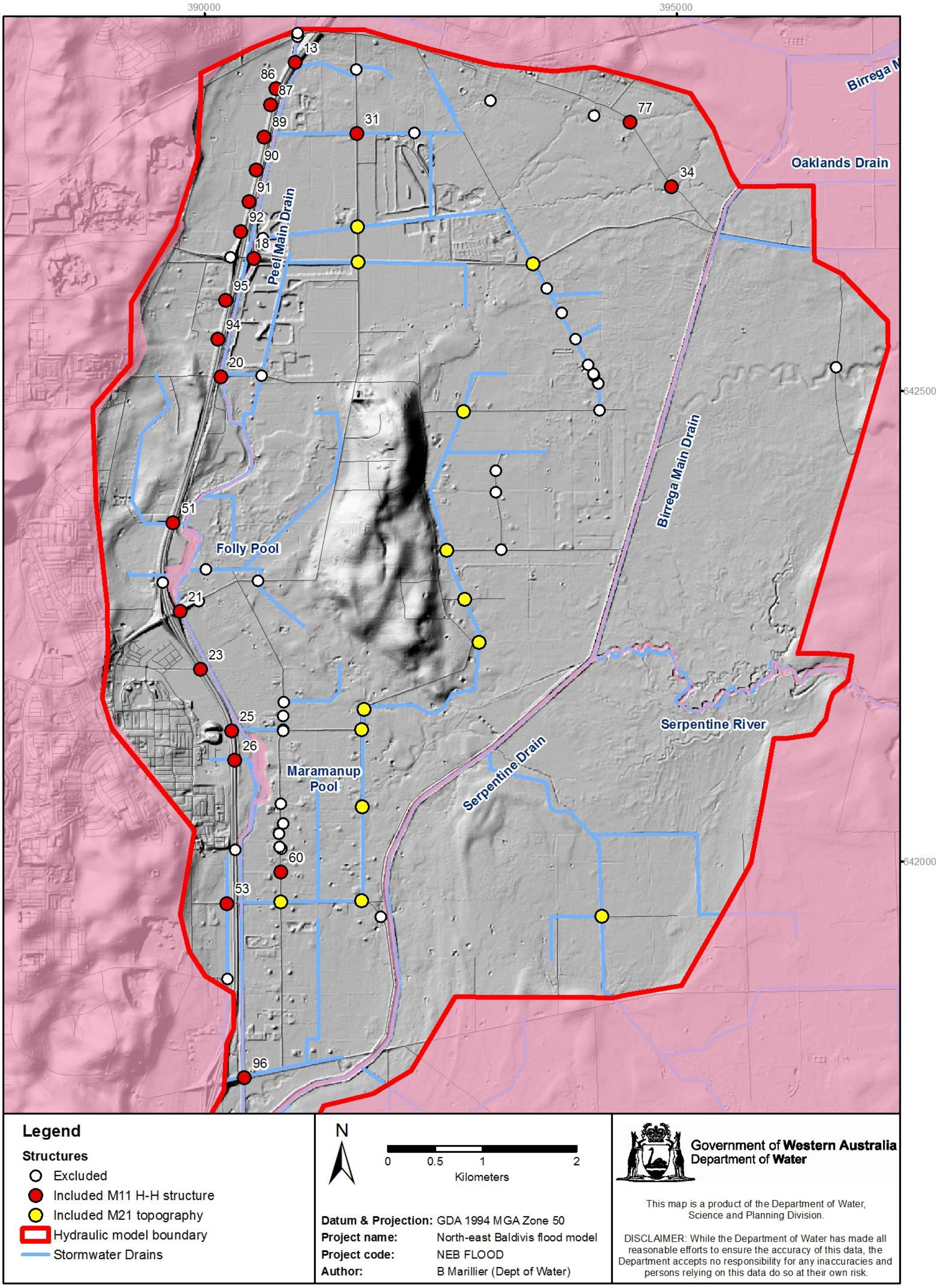


Figure 5-6: Structures included and excluded from the hydraulic model

Boundary conditions

The RORB models provided discharge time-series at inflow boundaries on the Serpentine River and Peel Main Drain. Discharge from the Birrega MIKE FLOOD model was used as the discharge boundary for the Birrega Main Drain. A stage-discharge relationship was calculated within MIKE 11 for the outflow of the Serpentine River at the southern end of the model.

Antecedent conditions

For design and calibration runs, the MIKE 11 model used a hotstart file containing antecedent flow and stage within the MIKE 11 channels. The hotstart file was generated by running a 24 hr simulation with discharge boundaries set at a flow rate of 0.5 m^3/s, which represents antecedent baseflow in the network.

Resistance

Channel geometry and vegetation growth is similar along most of the Peel Main Drain, Serpentine Drain, and Birrega Drains, with channels being straight, trapezoidal, and free of vegetation. The exception is the Serpentine River, and the Maramanup and Folly pools on the Peel Main Drain, which have more vegetation, and winding sections. A Manning's *n* of 0.03 is recommended for clean, straight channels with no pools, and this value was used for the main channels. An *n* of 0.04 was used for the winding reaches with pools (Chow 1959).

Table 5-2: Mannings M values used in the MIKE 11 network

Reach	Start chainage	Start chainage	Manning's M	Manning's n
Peel Main Drain	0	5900	33	0.03
Peel Main Drain (Folly Pool)	5900	6900	25	0.04
Peel Main Drain	6900	8500	33	0.03
Peel Main Drain (Maramanup Pool)	8500	9800	25	0.04
Peel Main Drain	9800	15500	33	0.03
Peel Main Drain	15500	0	33	0.03
Serpentine Drain	0	16955	33	0.03
Serpentine River	0	4415	25	0.04

5.3 MIKE FLOOD: coupling MIKE 11 and MIKE 21

A MIKE '.couple' file was developed to link the MIKE 21 and MIKE 11 models. Each component model was built and stabilised before combination, ensuring any instabilities or issues were identified prior to running the more complex MIKE FLOOD model.

MIKE FLOOD has several options for schematisation of links between MIKE 21 and MIKE 11. Each 'link' represents a cell within MIKE 21 which is configured to exchange water with the MIKE 11 network. The two types of links which were included for the north-east Baldivis model were lateral links, which link MIKE 11 channels to MIKE 21 using left and right banks, and a weir formula; and standard links, which link a MIKE 21 cell with a MIKE 11 boundary condition based on water surface elevation. The techniques that were used to ensure stable coupling are described for each link type below.

Lateral links

Lateral links enable MIKE 21 cells to be linked laterally to a reach within MIKE 11. The link can be used to model exchange between a river and floodplain. For each lateral link, water is exchanged using a weir formula, and the water surface elevation within MIKE 11 and MIKE 21. Water can be exchanged in both directions across the weir, allowing discharge from the channel to the floodplain; and overland flooding discharging to the channel. Figure 5-7 shows an example of left and right lateral links on the Peel Main Drain.

To ensure equivalent geometry which assists with model stability, additional processing of the model topography was performed in ArcGIS. The basic steps for implementing the lateral links within MIKE FLOOD were as follows:

1. The MIKE 11 channel geometry was extracted from the 1 m LiDAR and processed as described in section 5.2.

2. Within the .couple file, lateral links were defined for the left and right bank of all rivers and drains.

3. A list of linked cells was exported from the .couple file, and imported into ArcGIS, to remove erroneous and overlapping cells, and ensure spatial equivalency between MIKE 21 and MIKE 11. The final list of linked cells was then imported back into the .couple file in the links dialog window.

4. The MIKE FLOOD model was run to generate pre-processing files, including ‘MFLateral.xns11’ which contains the geometry of the weir which is used to exchange water between MIKE 21 and MIKE 11. This file can be used to examine the difference in elevation of the MIKE 21 topography and MIKE 11 left and right bank markers for linked cells.

5. For each of the linked cells, the elevation of the bank marker from the linked MIKE 11 cross section was used to updated the model topography at that cell, such that the MIKE 21 cell was always 0.2 m lower than the MIKE 11 bank marker. This processing was completed using Spatial Analyst within ArcGIS.

6. The .couple file was configured to use the highest elevation of MIKE 21 and MIKE 11 to define the weir geometry for each lateral link, and the processing completed in step 5 ensures that MIKE 11 is always used to define the weir height. As the MIKE 11 geometry was extracted from the 1 m resolution DEM, it is superior in accuracy to the topography of MIKE 21 which is sourced from a 10 m resampled DEM.

7. The MIKE FLOOD model was run using very high flows at all MIKE 11 inflow boundaries, and the results were checked for stability. All breaks in the levee were examined to ensure that water was spilling from the MIKE 11 model into the MIKE 21 model at the appropriate locations.

When using lateral links within MIKE FLOOD, it is important to remove all river channel cells within MIKE 21 which are already represented by MIKE 11, to avoid duel conveyance in the model. In this instance, all topography cells located between lateral link cells were converted to land cells, so that they were excluded from computation within the model. No lateral links were implemented where culverts existed within the MIKE 11 model (see Figure 5-7). Rainfall and infiltration were disabled on all couple cells to improve model stability.

Within the .couple file it is possible to apply an exponential smoothing factor to dampen oscillations within linked cells. For all lateral links, a factor of 0.2 was applied to improved model stability. Other weir structure parameters were left at default values, including the depth tolerance (0.1 m), weir coefficient (1.838) and friction term (Manning's n = 0.05).

Standard links

Standard links are used to link one or more MIKE 21 cells to the end of a MIKE 11 branch. These links are typically used to combine the end of a MIKE 11 reach to a MIKE 21 grid, or to add a detailed MIKE 11 structure to a MIKE 21 grid. In this instance, standard links were used to model culverts underneath the freeway and for several roads within the MIKE 21 model. All culverts were modelled using the H-H conduit method, where fluxes in the culvert are controlled by water level at both ends of the MIKE 11 reach.

The culverts were configured by defining a MIKE 11 model, which consisted of a short reach, with two cross-sections, representing the open channel at either end of the road culverts. Using the surveyed culvert data, two additional closed cross-sections were inserted representing the upstream and downstream invert levels, and the culvert geometry. Within MIKE 11, cross-section chainages were used to define the length of the culvert. At either end of the MIKE 11 reach, a couple cell was added in the .couple file to link the reaches, and a water level boundary was configured in MIKE 11. Where necessary, the MIKE 21 topography was modified at the couple cell to ensure that the channel geometry was consistent.

To improve stability at the standard links, these cells were configured to allow flow only in the horizontal X or Y direction using 'zero flow links', depending on the orientation of the culvert and drain. An exponential smoothing factor of 0.2 was used for all standard link cells.

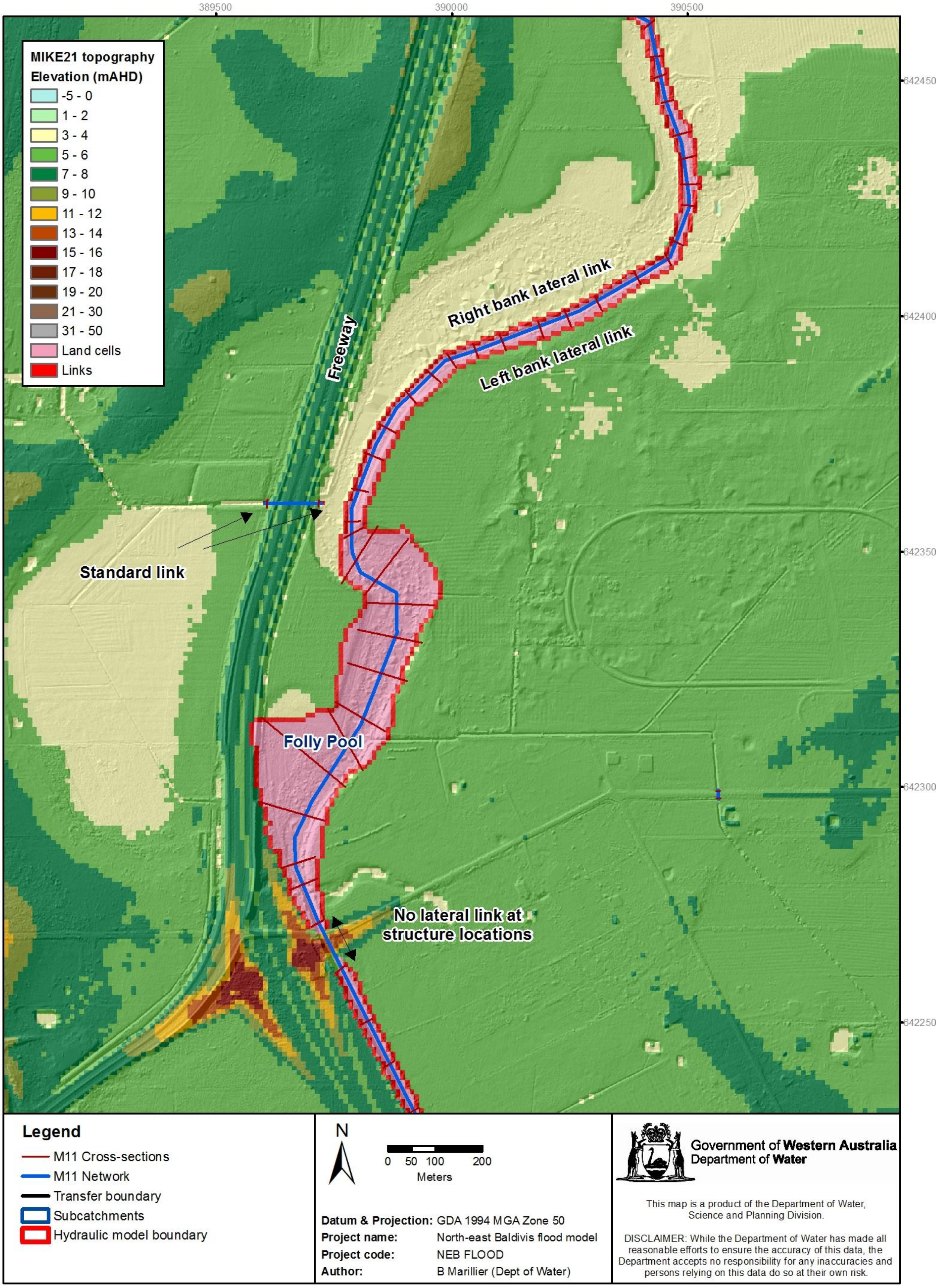


Figure 5-7: Example of lateral and standard links for a section of Peel Main Drain near Folly Pool

5.4 Model stabilisation and mass balance verification

The MIKE FLOOD water balance tool was used to assess the 100 yr 36 hr model for errors in mass balance and stability issues. The mass balance error was 1% of the total model outflow, which was considered acceptable for the purposes of this modelling study.

The MIKE 11 model was assessed for instabilities in discharge and water level. Some minor oscillations were found for some structure locations at low flows, but did not impact substantially on peak flows or event volumes.

The 100 yr 36 hr design model was assessed independently by DHI and found to be of suitable construction and stability for simulation of design events.

5.5 Model calibration

Calibration is an important stage in model development that demonstrates that the hydraulic model is capable of reproducing realistic flood behaviour within the study area (Engineers Australia 2012). The north-east Baldivis flood model includes 1D and 2D elements, several boundary conditions, direct rainfall, and infiltration. As such, it is more complex than more traditional 1D or 2D flood modelling, and therefore the importance of calibration cannot be overestimated.

Reliable flood stage data is available at the Dog Hill gauging station on the Serpentine Drain for the 1987 event and at the Karnup Road station on the Peel Main Drain for a smaller event in 2005. Both of these events were used for model calibration, which ensures that the catchment response to rainfall is realistic at the scale of the modelling considered. Photographs of localised flooding around Cobby Lane were used as supplementary information for calibration of the 2005 event.

Unfortunately, there is a high degree of uncertainty around the rating curve at Dog Hill, and as SKM (2010) found in earlier flood studies on the Serpentine River, the differences in peak flow estimates at the gauge for the 1987 vary between 100 m^3/s and 140 m^3/s depending on the rating curve used. So, calibration at Dog Hill was focused on using the gauge’s measured stage data which is a more reliable data source. The Karnup Road gauging station now has Acoustic Doppler Velocimeters installed; however, in 2005 a rating curve was used and the backwater effects of the Serpentine River are known to have influenced the rating at low stage. Therefore the calibration at Karnup Road was completed using only the more reliable stage data.

The parameter values used in calibration included the gridded Manning’s *M* values in MIKE 21, *M* values in the MIKE 11 network, and gridded infiltration rates within MIKE 21. No modification to the RORB hydrological model was required for calibration.

Calibration to the 1987 event at Dog Hill

Figure 5-8 shows that the modelled stage for Dog Hill closely resembled the observed stage for all components of the flood hydrograph. Note that around half of the event volume was sourced from the Birrega MIKE FLOOD model (Hall 2014), which was calibrated to an approximate 5–10 yr event in 2011. The calibration at Dog Hill indicates that inflows from the Birrega model are reliable for the larger 1987 event. No substantial changes were required to calibrate stage at Dog Hill though the Manning's M was increased slightly (from 30 to 33) to increase conveyance and reduce peak stage. The modelled time to peak stage was 2 hours later than the observed stage. The maximum recorded peak was 6.84 mAHD, and this was closely matched in the model, which peaked at 6.87 mAHD.

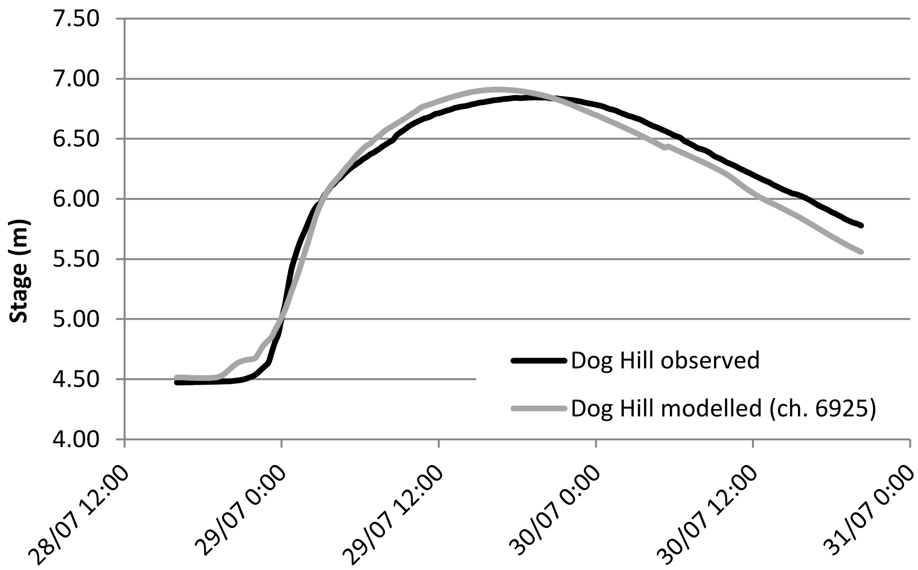


Figure 5-8: Modelled and observed stage at Dog Hill for the 1987 calibration event

Discharge data from Dog Hill was not used in calibration due to the uncertainty associated with the site's rating curve. However, for comparisons sake, modelled discharge is shown against estimated discharge at Dog Hill using both the 1987 and the 2010 rating curves. The modelled peak flow is 121.6 m^3/s compared with 137.8 m^3/s for the 1987 rating, and 93.2 m^3/s for the 2010 rating.

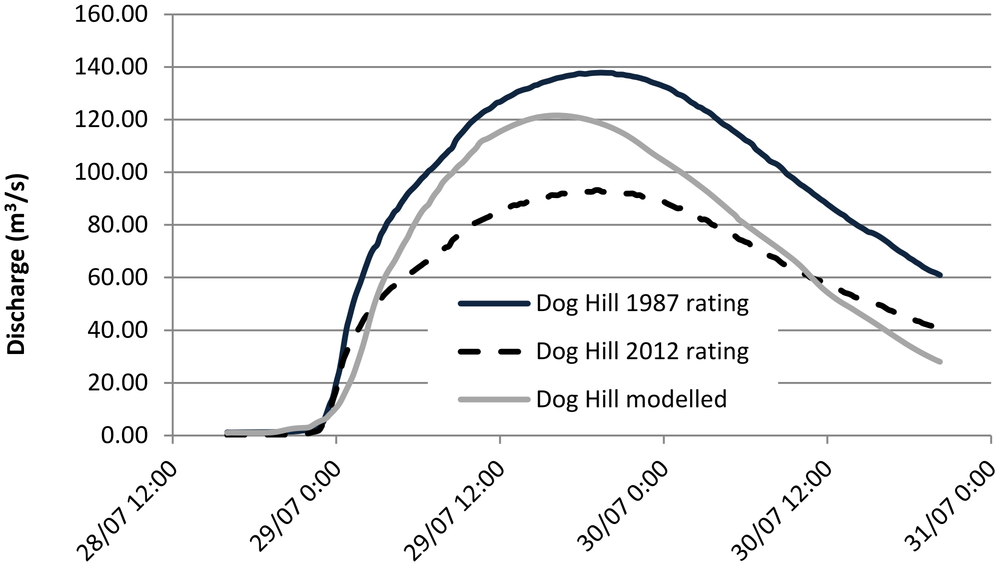


Figure 5-9: Modelled discharge and discharge estimated from the 1987 and 2010 rating curves at Dog Hill for the 1987 calibration event

In the SKM (2010) study, the hydraulic model was calibrated to the discharge hydrograph estimated using the more recent rating curve rather than to the stage data which was used in this study. The flood stage time-series did not appear to be used in calibration, though SKM report that their model overestimated flood peak stage at Dog Hill by 30 cm. This may indicate that conveyance in the channel is underestimated either due to channel geometry or resistance parameters. Using only discharge to calibrate the model probably resulted in an overestimate of peak discharge and stage which would have ramifications for the design flood estimates provided by SKM (2010).

Calibration to the 2005 event at Karnup Road

The 2005 event was much smaller than the 1987 event, with only 52 mm of rainfall received over 36 hr. However, antecedent groundwater levels were higher than for the 1987 event, resulting in a smaller area for infiltration in some sections of the hydraulic model. The event was calibrated using the observed stage at the Karnup Road gauging station and street photography on Cobby Lane, near the Peel Main Drain.

The calibration at Karnup Road was successful using unchanged parameters from the 1987 calibration run. The flood peak was overestimated by 0.06 m, and the peak stage was 1.25 hr earlier than the observed stage. The hydrograph shape is influenced by backwater from the Serpentine River and discharge down the Peel Main Drain. The calibration indicates that simulation of the confluence of the Serpentine River and Peel Main Drain is reasonable, and reproduced realistic backwater effects in the Peel Main Drain at Karnup Road.

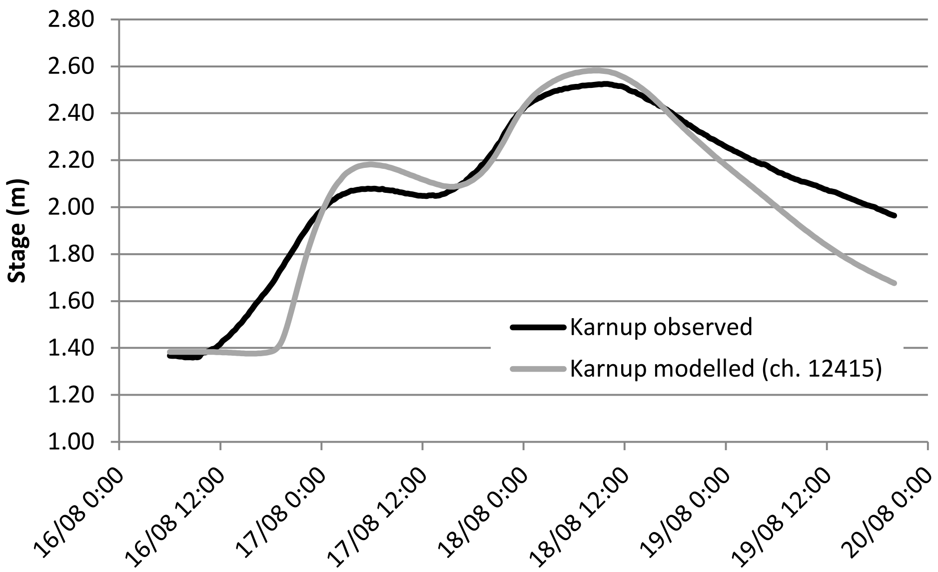


Figure 5-10: Modelled and observed stage at Karnup Road for the 2005 calibration event

Figure 5-11 shows the modelled flood depth at 12 pm on 18 August 2005, which is approximately the peak stage recorded at Karnup Road. The photographs available on this day indicate extensive shallow flooding around the three houses on Cobby Lane, with only the house pads above the flood waters. The model predicts a similar extent of flooding on the same day, with flooding on the western side of Cobby Lane caused by a combination of overflow from the Peel Main Drain upstream of Folly Pool and localised rainfall, and the flooding on the eastern side of the lane caused by overland flow received from the surrounding paddocks.

Calibration summary

The calibration demonstrates that the MIKE FLOOD is realistically simulating flood behaviour on the Peel Main Drain and the Serpentine Drain for the events considered. The 1987 calibration shows that the Serpentine RORB hydrological model is providing reasonable estimates of discharge from the catchments on the scarp, and also that the MIKE 11 model is exchanging with MIKE 21 to provide realistic simulation of lateral storage effects adjacent to the main channel of the Serpentine Drain. The 2005 event indicates that the backwater effect from the Serpentine River is modelled appropriately on the southern reaches of the Peel Main Drain. It also shows that the direct rainfall processes, including infiltration, are realistic, and comparison between model results and photographs supports this. Note that the largest event calibrated is the 1987 event, which is approximately a 50 yr ARI event. Therefore, results of design events above this size should be considered less reliable than smaller events.

Based on the calibration results the model is appropriate for simulation of design events.

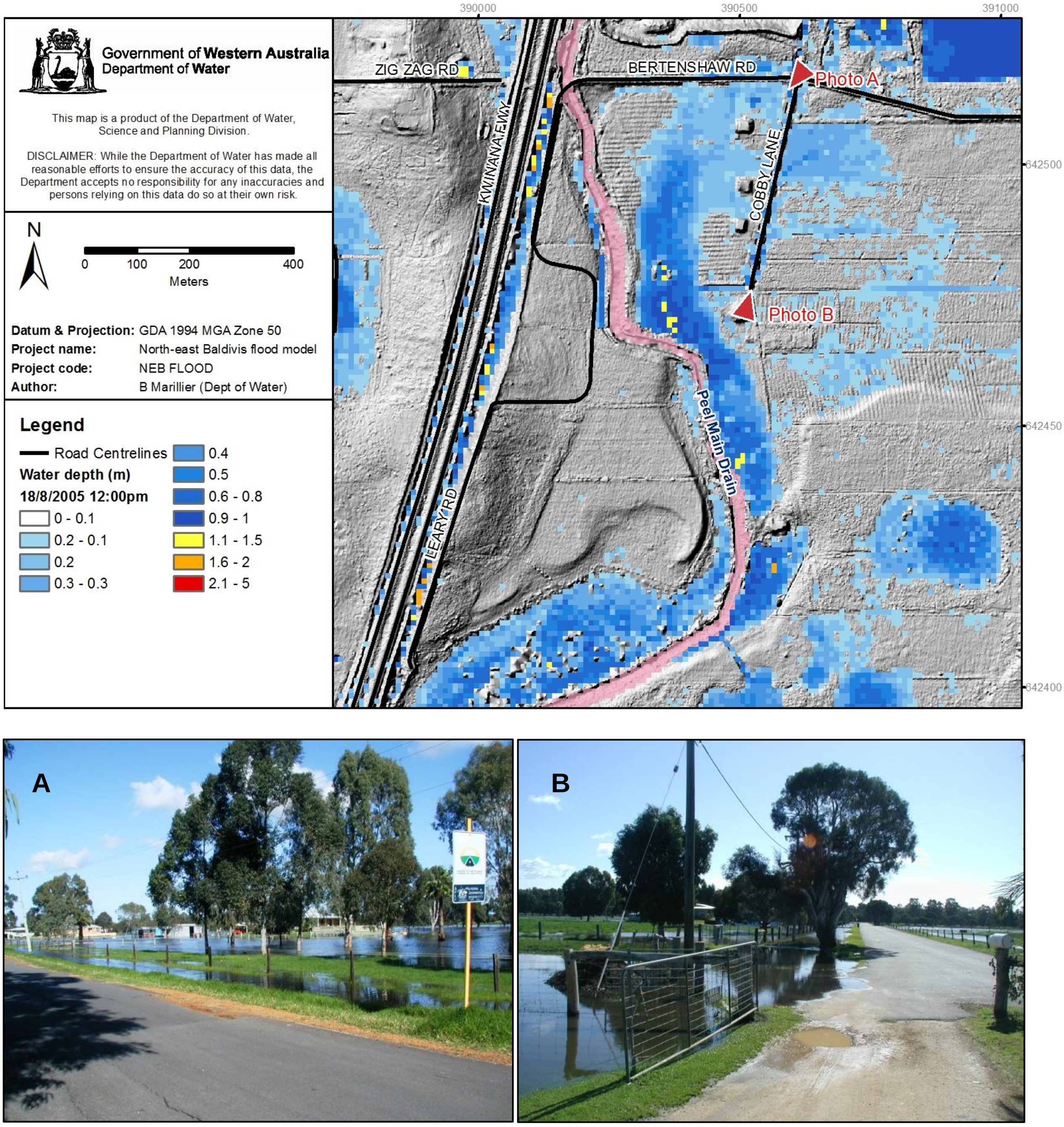


Figure 5-11: Modelled flooding around Cobby Lane in 2005 (comparison with photographs)

6 Hydraulic model results

Design floods were simulated using the hydraulic model for the 5, 10, 20, 100 and 500 yr ARI events, for durations of 6, 12, 24, 36, 48 and 72 hr. An additional 100 yr ARI levee fail (LF) scenario was simulated for all durations, assuming that the western bank along the Birrega and Serpentine main drains was absent. The 100 yr 24 hr simulation includes overtopping from the Birrega Main Drain at Duck Pool.

Model results are presented in this section in several forms, which include:

- **Floodplain mapping:** Simulated maximum levels and flood extent are based on the combined maximum of the 100 yr and 100 yr LF scenarios. Gridded results for the other design events are presented in Appendix D.
- **Main drain long-sections:** These illustrate peak flood levels and discharge for the Peel Main Drain and sections of the Birrega and Serpentine Main Drains.
- **Peak discharge and event volumes:** These are calculated at multiple locations within the hydraulic model, where channelised flow is present.
- **Discharge hydrographs at Karnup Road:** Hydrographs for the calibration events and multiple design events are reported at Karnup Road for the Peel Main Drain and the Serpentine River. These hydrographs may be used as inflow boundaries for modelling to the south of the current study area.

Results are reported for most of the hydraulic model domain, excluding the area south of Karnup Rd. Note that some locations on the western side of the freeway have been developed since the model's topographic LiDAR dataset was flown, and so any flooding reported in this area should be disregarded.

6.1 Flooding mechanisms

Four flooding mechanisms were simulated using the hydraulic model. These included flooding resulting from direct rainfall, simulated using the direct-rainfall technique; riverine flooding from Peel Main Drain; overtopping of the levee bank at Duck Pool on Birrega Main Drain; and failure of the levee bank along the Birrega and Serpentine Main Drains.

Direct rainfall

As a result of the extremely flat terrain in this area, many small depressions are prone to shallow flooding and ponding of water as a result of direct rainfall. In many cases these depressions are disconnected from any significant flow paths or drains, and so flooding occurs locally, and the area will remain inundated until excess water infiltrates or evaporates. Figure 6-1 shows an overview of the 100 yr floodplain extent and flood depth across the study area, without levee failure. Direct rainfall generates overland sheet flow which contributes to flows in the main drains, where there is sufficient gradient to concentrate flow.

While the entire study area has sections which are prone to flooding as a result of direct rainfall, floodwaters tend to concentrate at the following locations:

- On the eastern side of St Albans road, where culverts limit discharge from east to west towards the Peel Main Drain. Areas around the water-ski park and aquaculture farm between Telephone Lane and St Albans Road are also prone to flooding
- The area bounded by Millar Road, the Kwinana Freeway, Bertenshaw Rd and St Albans Rd
- The area to the east of Dog Hill Rd accumulates floodwater, and discharge from this location is slow due to the extremely low grade of the subdrain running through this area.
- Low points on the western side of the freeway between Safety Bay Rd and Millar Rd
- The strip of land between the Peel Main Drain, Serpentine Main Drain, in the southern section of the hydraulic model
- Areas adjacent to the left (eastern) levee bank along Birrega Main Drain and Serpentine Main Drain
- The low-lying area in the south-eastern corner of the model domain on the eastern side of the Serpentine Main Drain.

Typically, the flooding from direct rainfall results in only shallow areas of inundation, generally less than 50 cm deep, but up to 1 m in relatively low areas and drains. Most of these areas have low flow velocities, although local drains and areas of channelised flow may have higher flow velocities.

Riverine flooding

Riverine flooding occurs from the Birrega, Serpentine and Peel Main Drains in locations where the left levee-bank is lower than the peak flood levels within the drains. Along all of these drains, the left bank has a series of culverts or lateral inflow channels at an elevation level with, or lower than, the surrounding plain. These breaks are not present on the right bank. Where there are breaks in the levee, the drains receive flow from the plain and discharge water to the plain, depending on the relative levels of the floodwaters. The long-sections shown in Appendix E indicate the locations of these low points along the drains, and flooded areas are visible in Figure 6-1.

Most of the flooding adjacent to the Birrega and Serpentine drains on the eastern side is a result of discharge from the main channel with a smaller contribution from direct rainfall, and inflows from the catchments outside the model domain to the east. These areas are significant floodplain storages and limit peak stage and flow within the main drains, thus reducing the risk of water spilling over the right levee bank and flowing westwards.

In the 100 yr event the Peel Main Drain receives net inflows from the east between Millar Road and Mundijong Road. However, the drain loses water between Mundijong Road and Safety Bay Road. Much of the inundation around the Folly Pool area is a result of discharge from the Peel Main Drain, although runoff from direct rainfall also contributes to downstream flooding at St Albans Rd and along Peel Main Drain.

Overtopping of the Birrega Main Drain levee at Duck Pool

The 100 yr 24 hr event simulated overtopping the Birrega Main Drain levee at Duck Pool, redirecting flow to the west through the north-east Baldivis area. This results in flooding between Duckpond Road and Telephone Lane, and contributes to flooding at St Albans Road and the Peel Main Drain further downstream. The flood hazard resulting from overtopping is a lower risk scenario than a complete levee failure at this location, as flood water must reach 11 mAHD in the Birrega Main Drain before overtopping can occur.

Levee failure

The levee failure design runs involved removing the right-hand (western) bank from the Birrega and Serpentine drains. Although this scenario has been called a levee failure, it is important to note that these banks are not flood levees, and may be geotechnically unstable in high flows. Therefore, it is probable that this bank would fail in events approaching the 100 yr ARI in size. The 100 yr ARI design modelling completed in the Birrega catchment upstream of the study area (Hall 2014) indicated that a peak flood stage would overtop at Duck Pool, making a complete levee failure likely in this area.

As it is not possible to indicate the exact location of levee failure, the entire right levee bank was adjusted to the level of the surrounding plain along the length of Birrega Main Drain, and on the Serpentine Drain downstream to around 1 km past the Dog Hill gauging station.

Design inflows for the levee failure runs were sourced from an equivalent levee fail scenario in the Birrega model (Hall 2014). A peak flow of 55 m^3/s (24 hr critical duration event) was modelled to discharge from the Birrega Drain onto the plain in the north-east of the study area, at the source point identified in Figure 5-4.

The levee fail scenario results in a substantial increase in the extent and depth of flooding to the area north of Mundijong Road (Figure 6-2). In the absence of the modified drainage network in the study area, this is the natural flow path for floodwaters in this area, directing flow around the base of sand dunes to the north, towards the Peel Main Drain. The southern portion of the study area around Serpentine Road and Young Road also experiences more extensive inundation, as does low-lying land around Cobby Land and Folly Pool. The area to the east of Dog Hill receives some additional flow from the Birrega Drain.

In the 100 yr LF event, floodwaters would overtop Mundijong Road just to the east of the freeway, with floodwaters flowing across the road towards the Wellard Wetlands to the south.

St Albans Road would be overtopped near the intersection with Mundijong Road. Duck Pond Road would be overtopped in multiple locations in the event of levee failure, due to the small capacity of the culverts in this location.

In the absence of the right levee bank, the water which was stored adjacent to the Birrega and Serpentine drains on the east is redirected westwards, causing much of the additional flooding.

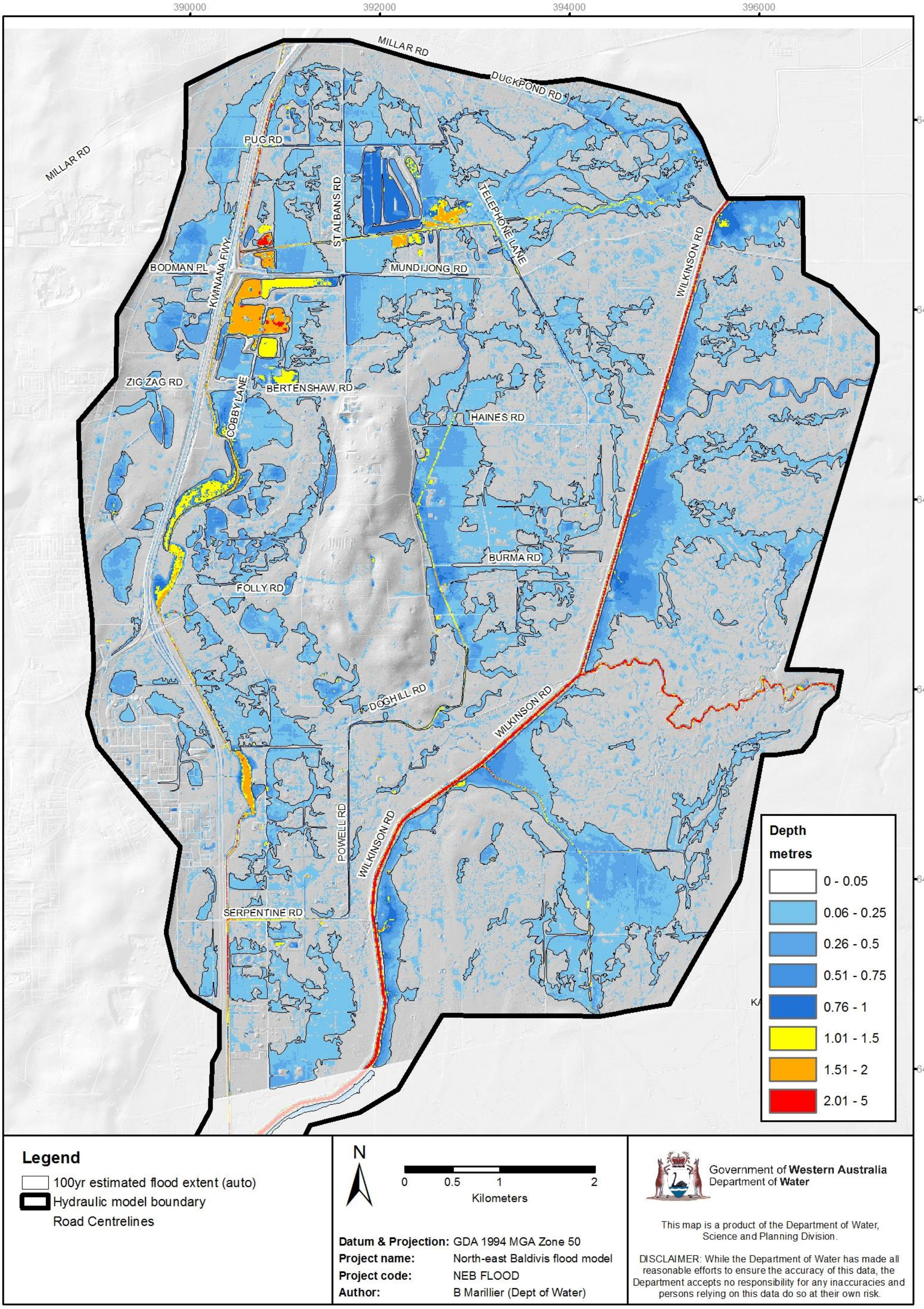


Figure 6-1: 100 yr maximum flood levels without levee failure

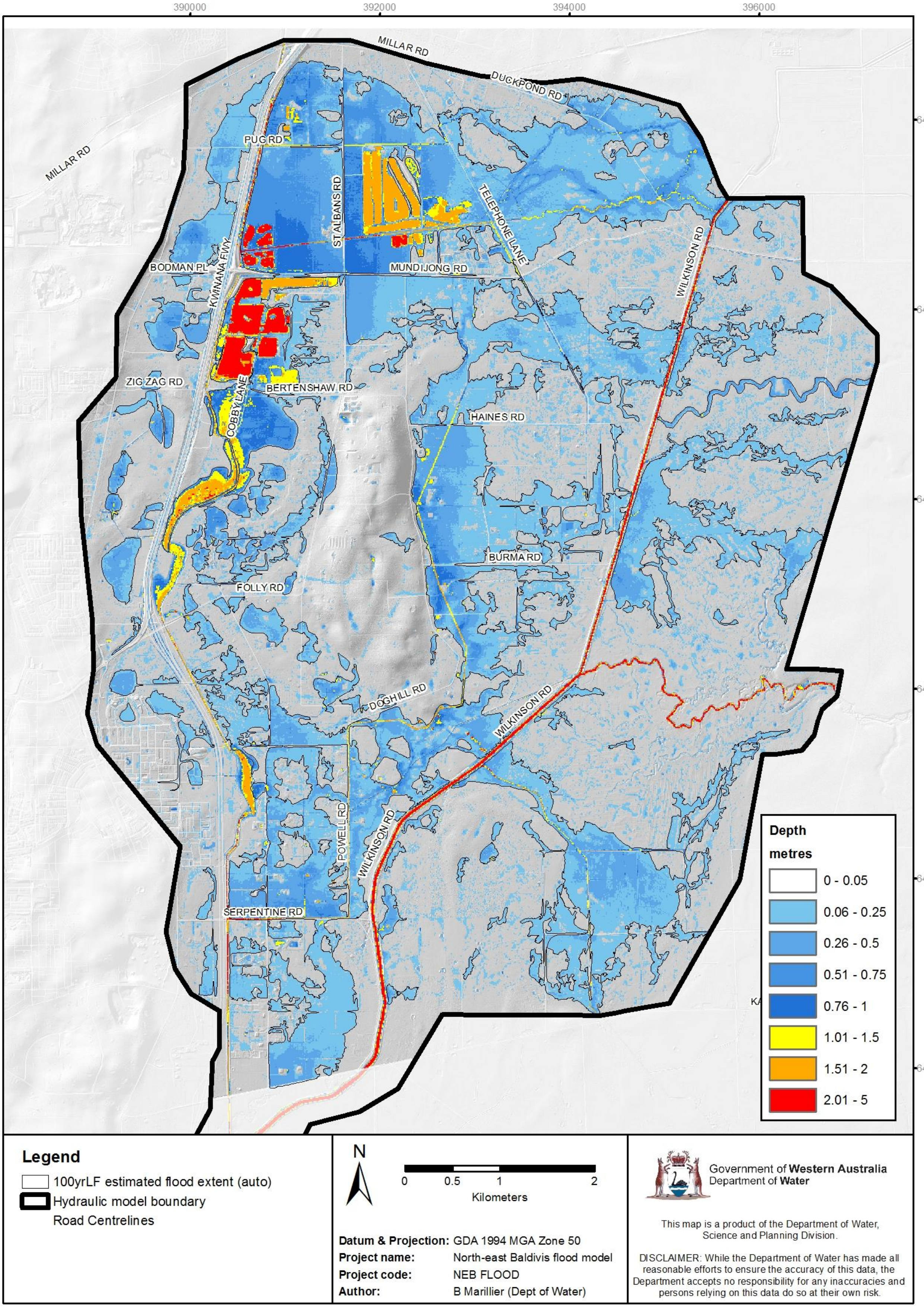


Figure 6-2: 100 yr maximum flood levels with levee failure

6.2 Floodplain mapping

Detailed floodplain mapping was prepared for the 100 yr ARI event, using a combination of the maximum modelled flood levels for all event durations, including levee failure scenarios. Detailed mapping was not completed for the 5, 10, 20 and 500 yr events though gridded maximum flood extents and depths for these events are shown in Appendix D.

Methodology

Floodplain mapping involved post-processing of MIKE 21 and MIKE 11 results and combination of the results into a single spatial dataset. A total of 12 designs runs were used to develop a composite maximum flood level, based on a combination of six event durations, the levee fail scenario, and the standard 100 yr event. The maximum flood level modelled for each grid cell was calculated from the gridded MIKE 21 results files, and the maximum of these grids was calculated to determine an overall maximum flood depth. Note that this does not give a flood level for a given point in time, but rather the maximum potential flood level for a given location for the 100 yr event.

The maximum flood level at each of the cross-sections defined in the MIKE 11 results files was calculated, and converted into a gridded format for combination with the MIKE 21 results files. MIKE 11 results were gridded by developing flood contours across each cross-section, and interpolating the flood level within the gridded channel cells which were original removed from the MIKE 21 grid to prevent dual conveyance. In this fashion, the MIKE 11 results were added into the MIKE 21 results for the main channels.

The floodplain extent was mapped manually at a scale of 1:2500 by tracing inundated areas. A maximum depth threshold of 0.05 m was used to eliminate very shallow areas of flooding. Small disconnected areas of ponding (< 1000 m^2) were not included in this mapping. However, larger disconnected areas were included as a separate category ‘ponded areas’. While these areas do not convey flow through the study area, in some cases they accumulate large volumes of water.

Flood level contours were derived using the same gridded results datasets. Flood contours were generated in ArcGIS, and were manually edited for consistency and readability. Final floodplain mapping was formatted and quality controlled by GIS technicians. The datasets were used to produce a series of A1 maps at 1:5000 scale covering the study area.

An overview of the floodplain mapping for the 100 yr event is shown in Figure 6-3.

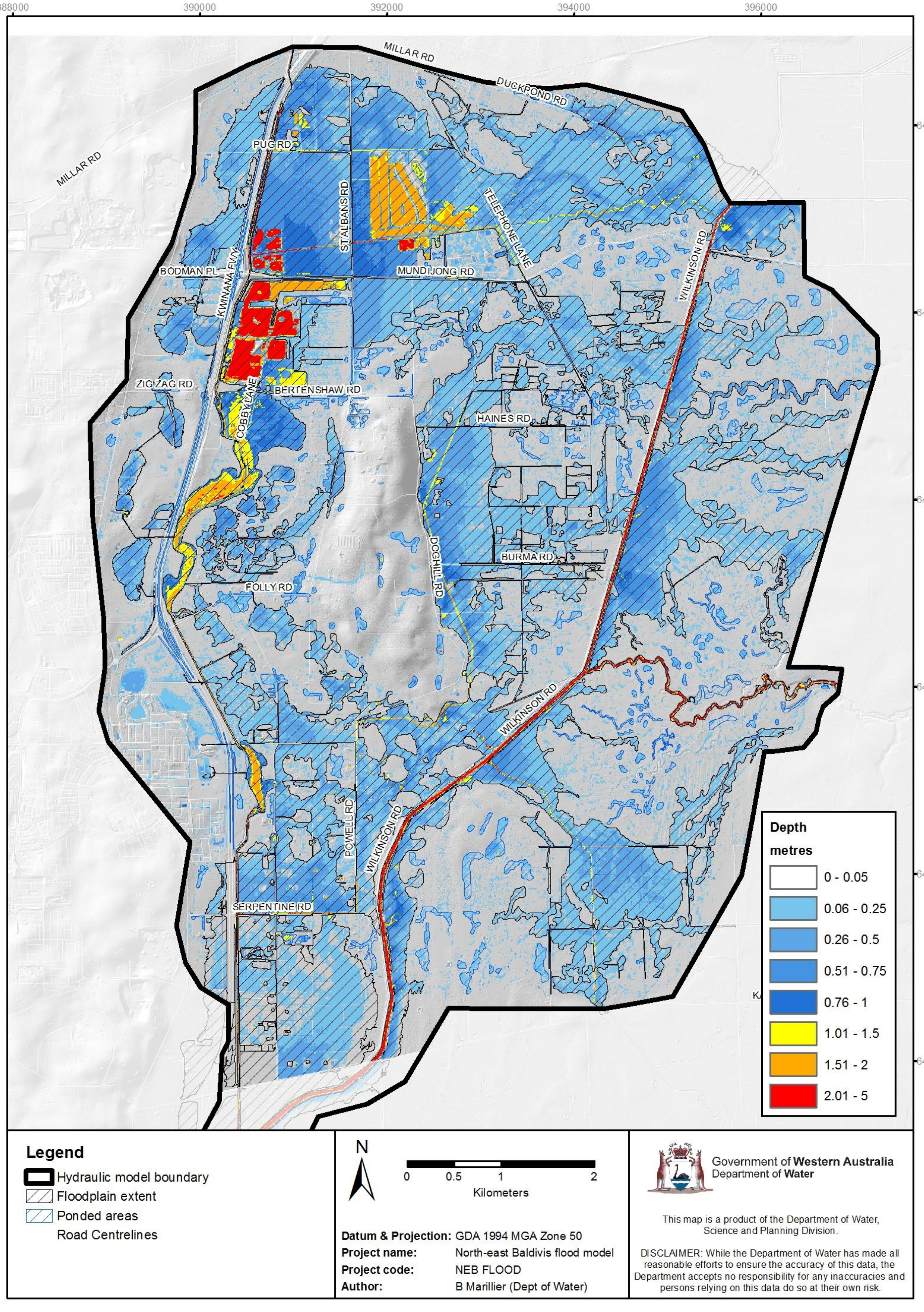


Figure 6-3: Floodplain mapping for the 100 yr event based on maximum flood levels

Comparison with previous studies

There are considerable differences in the extent and depth of flooding for the current study in comparison to the most recent flood study within the area completed by SKM (2010). The current study predicts less extensive flooding, lower event volumes and consequently shallower flood depths.

Figure 6-4 shows the differences in peak flood level between the SKM (2010) 100 yr 24 hr maximum flood elevation and results from the current study. Along sections of the Peel Main Drain and the Serpentine River, modelled flood levels are up to 1 m higher. The differences in the two studies can be attributed to three factors: the initial treatment of catchment hydrology; the treatment of losses within the model domain; and model geometry.

Section 4 contains a discussion of the differences in treatment of catchment hydrology. In the current study, considerable effort was made to match RORB catchment modelling to calibration events and the results of FFA. This resulted in a significant reduction in the peak flows estimated to discharge from the Serpentine River.

The area of greatest difference in flood extent is along the Peel Main Drain. To better understand the cause of this, results at the inflow boundary of the Peel Main Drain were extracted from the MIKE 21 model used by SKM (2010). The peak discharge to the Peel Main Drain used by SKM for the 100 yr event was found to be 176 m^3/s, which is physically implausible. By way of comparison, the Jandakot DWMP (DoW 2009) estimated 4.8 m^3/s for the same event, and the RORB modelling used in the current study estimated 7.5 m^3/s. It is unclear how the SKM modelling resulted in such a large peak flow from such a small, flat and sandy catchment; however, inappropriate choice of RORB parameters is the most likely reason. Point inflows were extracted from the SKM model at all source points (shown in Figure 3-2 SKM, 2010). In all cases inflows were found to be unrealistically high. It is likely that over most of the SKM study area the depth and extent of inundation was substantially overestimated.

Another major difference between the two studies is the treatment of losses. The SKM study incorporated proportional losses within the RORB models for the internal catchments. In the current study, losses were incorporated using infiltration parameters on the floodplain, which is equivalent to a spatially varying continuing loss, based on soil properties. The influence of this is evident in the critical duration mapping for the two studies. In SKM's results, over much of Peel Main Drain the critical duration is for the 72 hr event. This is because the 72 hr event always contains the greatest volume, and there are no internal infiltration losses in the model. In the current study, shorter durations are critical in many of the sandier areas, as the longer events are generally associated with less intense rainfall, allowing for additional infiltration time, resulting in greater overall losses in comparison to shorter events.

The model schematisation for the current study better represents the channel geometry through the use of MIKE 11 for the main channels. The SKM's chosen grid scale was 12 m. This is insufficient resolution to accurately capture the channel shape along the main drains which are generally less than four cells wide. The cross-sectional area in the main channels is approximately 30% larger in MIKE 11 compared to the 12 m SKM grid. As such, these channels can convey substantially more flow before overtopping in the current model.

These multiple factors account for the substantial differences in the final floodplain mapping for the two studies.

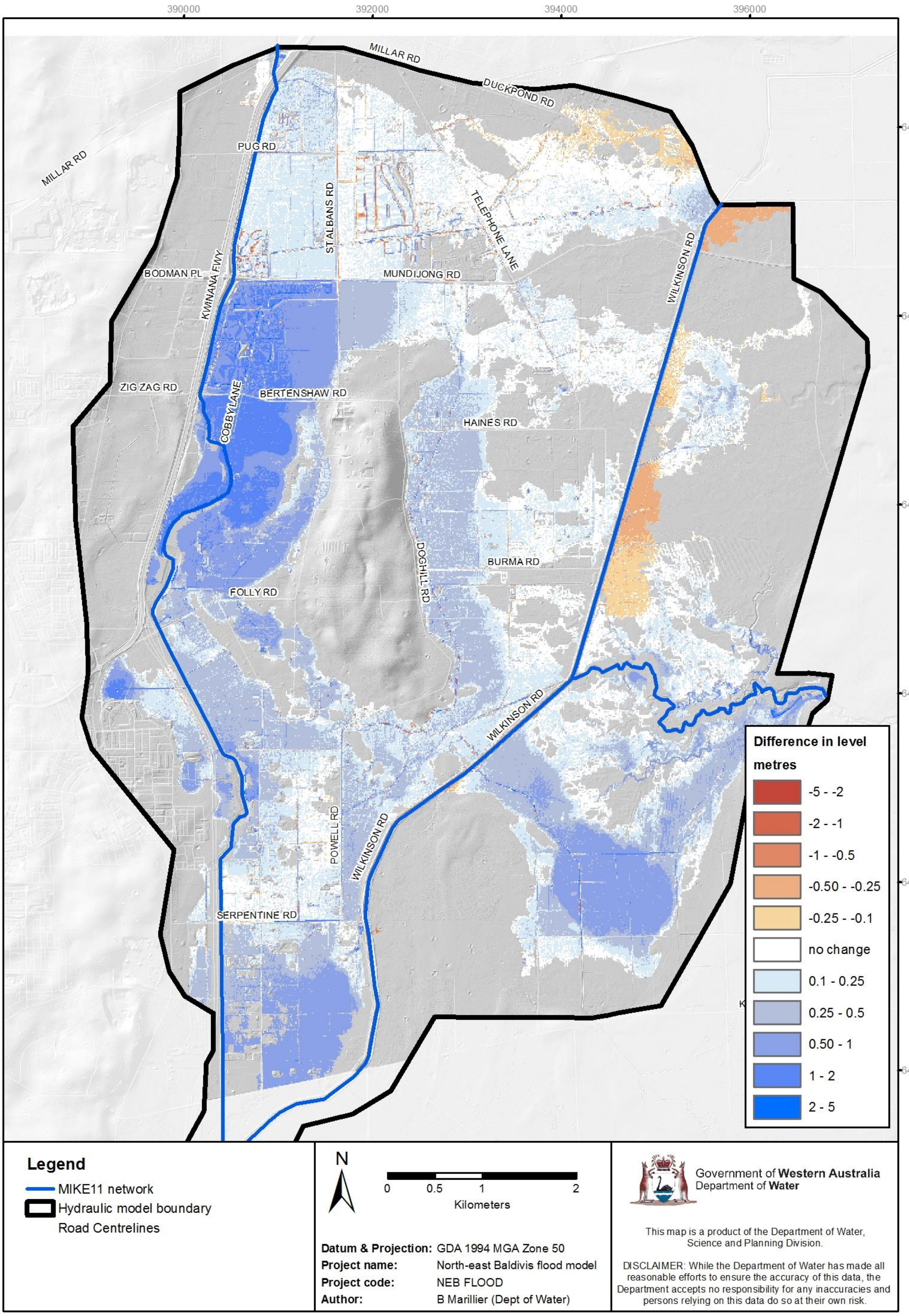


Figure 6-4: Differences in peak flood levels between SKM (2010) and current study

6.3 Long-sections

Long-sections of the Peel Main Drain and the Birrega/Serpentine Main Drain were developed using results from MIKE 11 (Appendix E). Results were extracted from all design runs, and the maximum discharge and stage were calculated for each Q and H point within the channel network. Levels and discharges reported for the 100 yr event were calculated from the 100 yr levee failure and non-levee failure scenarios. Water levels and peak flows reported in the long-section at the upper end of the Birrega Main Drain were integrated with the results from the Birrega and Oaklands flood model (Hall 2014) upstream of Mundijong Road.

The long-sections give results at key locations along the main drains and show channel geometry. The left and right bank geometries are shown in more detail to illustrate the discharge/inflow points along the main channels. Where the design peak stage is higher than the height of one of the banks, the water may discharge from the main channel to the surrounding floodplain, depending on the relative flood levels. The main channels also receive inflows from the floodplain where there are low points in the levee bank.

The long-section for the **Peel Main Drain** shows the following:

- Levee overtopping and levee failure of the Birrega Main Drain increase peak water levels and flows in the Peel Main Drain for the 100 yr event.
- The three culverts located on Mundijong Road in the main channel of the Peel Main Drain are close to or over capacity for the 100 yr levee fail event. There is a drop in peak stage from the north of Mundijong Road to the south for these events.
- At peak stage, between Millar Road and Mundijong Road, the Peel Main Drain is a net receiver of water. Between Mundijong Road and Folly Road, the Peel Main Drain loses water. Most of this loss is to the Wellard Wetlands through a large lateral culvert but also to the low-lying areas around Cobby Lane and Folly Pool. Note that water discharged to the floodplain during periods of high water levels later returns to the drain as water levels drop.
- At Serpentine Road the Peel Main Drain receives inflows from the Water Corporation subdrain running from the eastern side of Dog Hill, reflected in a higher peak discharge downstream of this drain from 14.3 to 20.0 m^3/s.
- Peak levels in the Peel Main Drain do not go above the bank level in most locations, however, where there are lateral culverts and breaks in the left levee bank, surrounding areas may be prone to flooding.
- Peak discharge from the lower end of the Peel Main Drain was modelled as 20.0 m^3/s for the 100 yr event, which is a substantial increase in comparison to the peak inflows of 7.5 m^3/s at the upper end of the drain. This reflects the additional inflow received as a result of levee failure along the Birrega Main Drain, and water received from the subdrains in the area.

The long-section for the **Birrega and Serpentine Main Drains** shows the following:

- At many locations along the left levee bank, peak stage exceeds the minimum levee bank height, resulting in flooded areas to the east of the drains.
- Peak levels for the 100 yr and the 500 yr events are sufficient to breach the right levee bank at Duckpond Road north of the model domain, and this is reflected in the levee fail scenario. Within the model domain, the 100 yr and 500 yr events overtop only the left levee bank of the Birrega and Serpentine Drains.
- Peak discharge from the lower end of the Serpentine Main Drain was modelled as 147.3 m^3/s for the 100 yr event.

6.4 MIKE 21 discharge calculations

A total of 32 cross-sections were defined within MIKE 21, and these were used to extract peak discharge and event volume from the MIKE 21 results. The 24 hr event was the most common critical duration for channelised flow within the MIKE 21 model, so this duration was used to report discharge and volume at each location.

Figure 6-5 shows the locations of all cross-sections included in the discharge calculations. At each cross-section location, results from all design runs were extracted. The peak discharge and associated event volume are reported for the 24 hr event for ARIs of 5, 10, 20, and 100 yr, and for the 100 yr levee fail scenario in Table 6-1.

The cross-sections were located where channelised flow paths were present, and only at locations where flows were realistic for the scale of modelling. Caution should be used when interpreting results for smaller drains and shallow overland flow paths, where the scale of modelling may be too coarse to provide realistic results. In the 100 yr events, drain capacities are exceeded, and flow is no longer constrained within the cross-section extent.

In the area to the north of Mundijong Road, the two east-west Water Corporation drains are the main flow paths for flood waters moving towards Peel Main Drain. The modelled 100 yr 24 hr peak flow at ID18 is 1.66 m^3/s, and at ID8 is 3.04 m^3/s. At cross-sections at ID17 and ID7 the 100 yr 24 hr peak flows across St Alban's Road are 1.23 m^3/s and 3.45 m^3/s respectively.

The cross-sections at ID1 and ID9 give an indication of the flows coming from the Birrega Main Drain in the event of levee failure in a 100 yr event. Under this scenario, peak flows would reach 43.3 m^3/s and 14.5 m^3/s through these locations respectively. The levee failure scenario substantially increases the discharge and event volume flowing through the area to the north of Mundijong Road.

Cross-section ID22 shows peak discharge of 2.39 m^3/s flowing towards the Wellard Wetlands on the western side of St Albans Road and southern side of Mundijong Road.

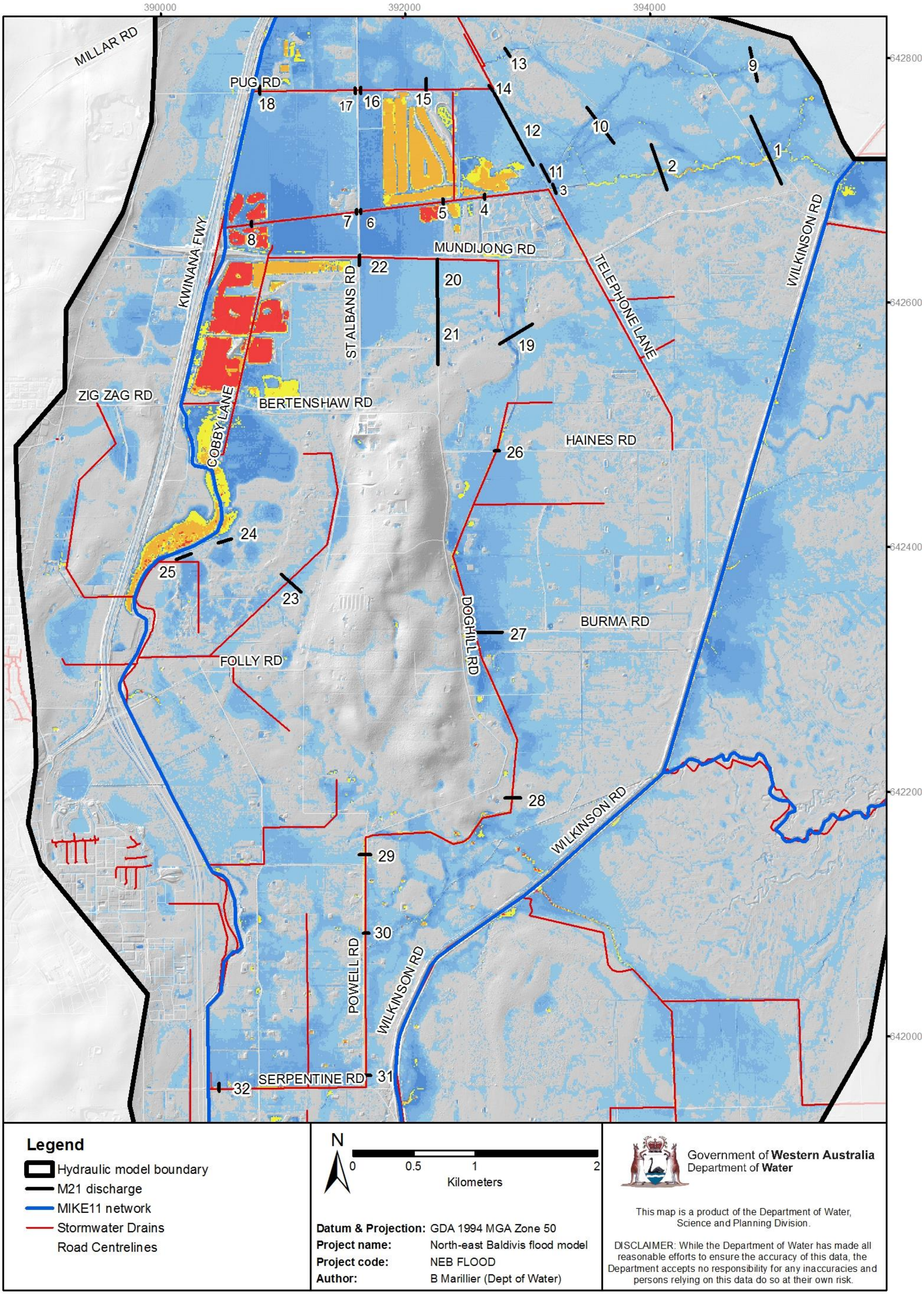


Figure 6-5: Cross-section locations used for MIKE 21 discharge calculations

Table 6-1: Peak discharge and event volume from MIKE 21 at cross-section locations for the 24 hr duration, 5, 10, 20, 100 yr and 100 yr levee fail scenarios

ID	5yr peak discharge (m^3/s)	5yr volume (ML)	10yr peak discharge (m^3/s)	10yr volume (ML)	20yr peak discharge (m^3/s)	20yr volume (ML)	100yr peak discharge (m^3/s)	100yr volume (ML)	100yrLF peak discharge (m^3/s)	100yrLF volume (ML)
1	0.27	9	0.30	11	0.37	14	10.89	539	43.3	2179
2	0.18	8	0.23	10	0.28	13	7.64	408	26.0	1414
3	-	-	-	-	-	-	1.44	61	7.9	361
4	0.43	14	0.53	17	0.67	22	4.21	248	7.0	519
5	0.46	18	0.64	23	0.78	29	3.64	298	4.9	314
6	0.84	33	1.09	41	1.32	51	3.39	338	21.1	1742
7	0.85	33	1.10	42	1.35	52	3.45	341	22.6	1822
8	1.51	73	1.70	86	1.91	100	3.04	355	4.1	313.3+
9	0.29	4	0.34	5	0.42	6	5.38	221	14.5	714
10	0.51	15	0.75	20	1.08	28	8.51	373	31.4	1499
11	0.16	4	0.25	7	0.41	12	5.03	237	17.3	884
12	0.17	5	0.27	9	0.41	15	5.53	245	25.7	1149
13	-	-	0.20	3	0.35	5	0.52	16	2.9	102
14	-	-	0.15	3	0.38	5	0.60	17	1.3	63
15	0.29	11	0.42	15	0.67	19	3.02	133	9.2	448
16	0.82	36	0.95	44	1.08	53	1.41	169	1.9	233
17	0.85	38	0.96	45	1.05	55	1.23	82	2.2	260
18	1.24	57	1.41	69	1.55	83	1.66	198	1.7	223
19	0.64	26	0.78	33	0.94	41	1.15	70	4.8	258
20	0.94	43	1.13	52	1.35	62	1.67	98	4.2	265
21	0.54	11	0.62	14	0.74	18	0.81	30	1.3	79
22	1.19	75	1.45	90	1.76	110	2.39	172	5.6	411
23	0.29	10	0.39	13	0.53	16	0.63	26	0.7	26
24	-	-	-	-	-	-	-	-	-	-
25	-	-	0.14	7	0.35	17	0.88	44	0.9	46
26	0.50	16	0.60	19	0.71	23	0.83	31	0.9	24
27	0.66	88	0.74	106	0.79	128	1.33	194	2.4	300
28	0.92	127	1.07	155	1.19	190	1.90	286	2.4	358
29	0.91	123	1.04	152	1.23	187	1.83	282	3.7	529
30	0.93	122	1.05	151	1.23	187	1.82	282	6.2	649
31	0.91	117	1.04	146	1.21	182	1.79	279	7.3	817
32	1.16	150	1.39	186	1.63	232	2.05	352	7.5	723

**empty cells indicate flows of less than 0.1 m^3/s*

+ Note flood bypasses cross-section 8 in levee failure scenario

IDs 26 to 32 show the peak discharges at various locations around the long subdrain which runs from the base of the eastern side of Dog Hill to the south, and towards the Peel Main Drain. Note that this drain has relatively low discharge rates, due to the drain's low grade. At ID32, the drain has a 100 yr 24 hr peak flow rate of 2.05 m^3/s, although a fairly large volume of water (352 ML) is conveyed to the Peel Main Drain in this event.

6.5 Discharge hydrographs at Karnup Road

Appendix F shows discharge hydrographs for all design events at Karnup Road, on the Peel Main Drain, and the Serpentine Main Drain. The critical durations for both drains are typically 24 or 36 hours. Peak flow for the 100 yr event on the Peel Main Drain is 12.4 m^3/s, and 147.4 m^3/s on the Serpentine Main Drain.

6.6 Sensitivity analysis

Sensitivity analysis was completed for the hydraulic model for resistance parameters, infiltration and boundary conditions. The analysis involved varying the parameters by plus and minus 30% or, in the case of the boundary conditions, increasing the elevation of the tailwater condition. Table 6-2 shows the parameter values used for the calibrated model, and for the sensitivity runs. The 100 yr 24 hr design event was used for these runs assuming no levee overtopping or failure on the Birrega Main Drain. Model sensitivity was assessed by comparing the water level within the model domain, and the difference in discharge and stage on the Peel Main Drain and the Serpentine Main Drain at Karnup Road.

Table 6-2: Parameter values applied for sensitivity analysis

Reach	Units	Calibrated value	Sensitivity low	Sensitivity high
Manning's M - MIKE11 straight channels	-	33	23	43
Manning's M - MIKE11 pools	-	25	18	33
Manning's M - MIKE21 - Urban residential	-	10	7	13
Manning's M - MIKE21 - Vegetation	-	13	9	16
Manning's M - MIKE21 - Pasture	-	20	14	26
Manning's M - MIKE21 - Roads & open	-	40	28	52
Infiltration - Clay	mm/day	20	14	26
Infiltration - Sand	mm/day	240	168	312
Tail water level	m	0.25	0.5	0.9

Sensitivity: Manning's M in MIKE 21

The hydraulic model is relatively insensitive to reasonable changes in the Manning's M values in the MIKE 21 model. Figure 6-6 shows that peak discharge in the Peel Main Drain is slightly increased with increasing values of M (smoother). In the Serpentine Main Drain, discharge is not significantly influenced by the M values, as most of the flow is sourced from outside the hydraulic model. Flood depth was not sensitive to Manning's M within the MIKE 21 model domain.

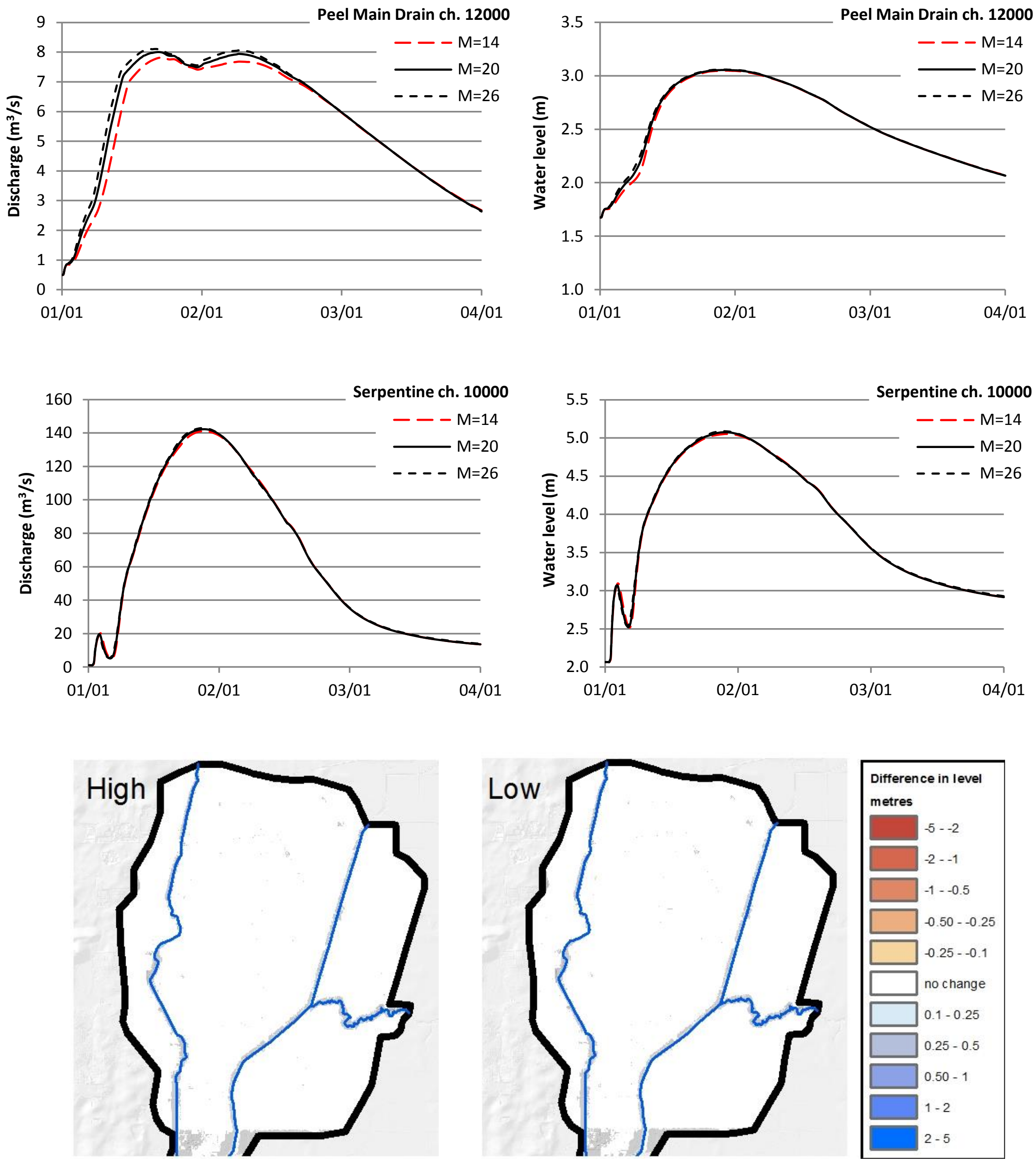


Figure 6-6: Sensitivity to Manning's M parameter in MIKE 21

Sensitivity: Manning's M in MIKE 11

The MIKE 11 model is moderately sensitive to changes in the Manning's M within the main channel network. For all of the main drains, peak stage, peak discharge and event volume are all influenced by changes to Manning's M. Flood levels in the lateral storage areas adjacent to the drains are influenced though over most of the MIKE 21 domain water levels within the main channels are not substantially influenced by changes in M. Although the model is sensitive to Manning's M, this parameter is well constrained by calibration.

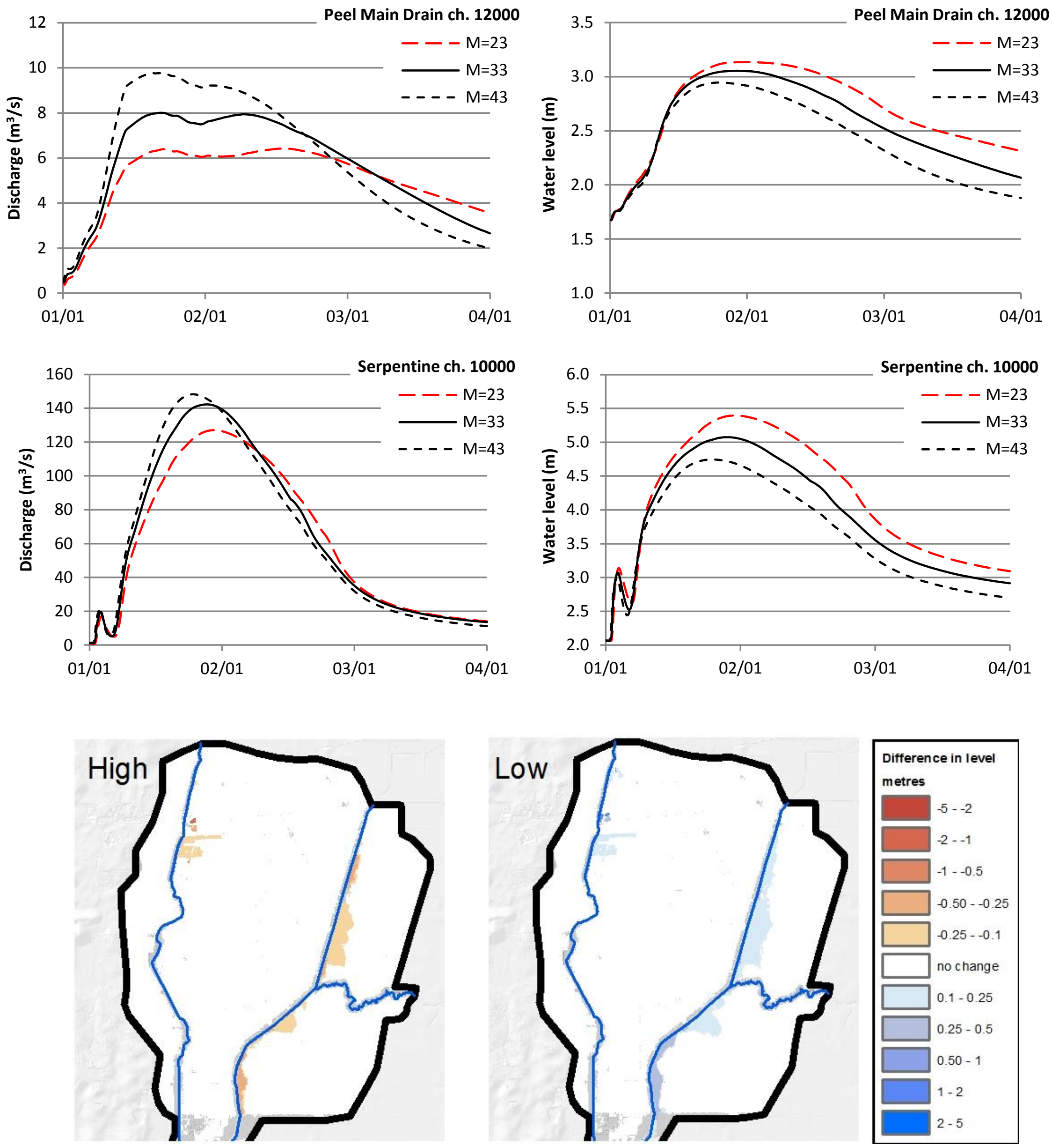


Figure 6-7: Sensitivity to Manning's M parameter in MIKE 11

Sensitivity: infiltration

Discharge rates and volumes are influenced by the infiltration parameter, which is the most important parameter controlling the runoff-coefficient within the model domain. This is illustrated by the discharge graph for Peel Main Drain (Figure 6-8). Flood depths are generally very shallow in the model domain so changes to the infiltration parameter do not have a substantial influence in flood level over most of the catchment. There is uncertainty associated with the infiltration parameter, as the direct-rainfall and infiltration technique is relatively new, with limited references available for appropriate parameter choice on the Swan Coastal Plain. However, the parameter ranges used in calibration were selected based on sensible infiltration rates for the soils.

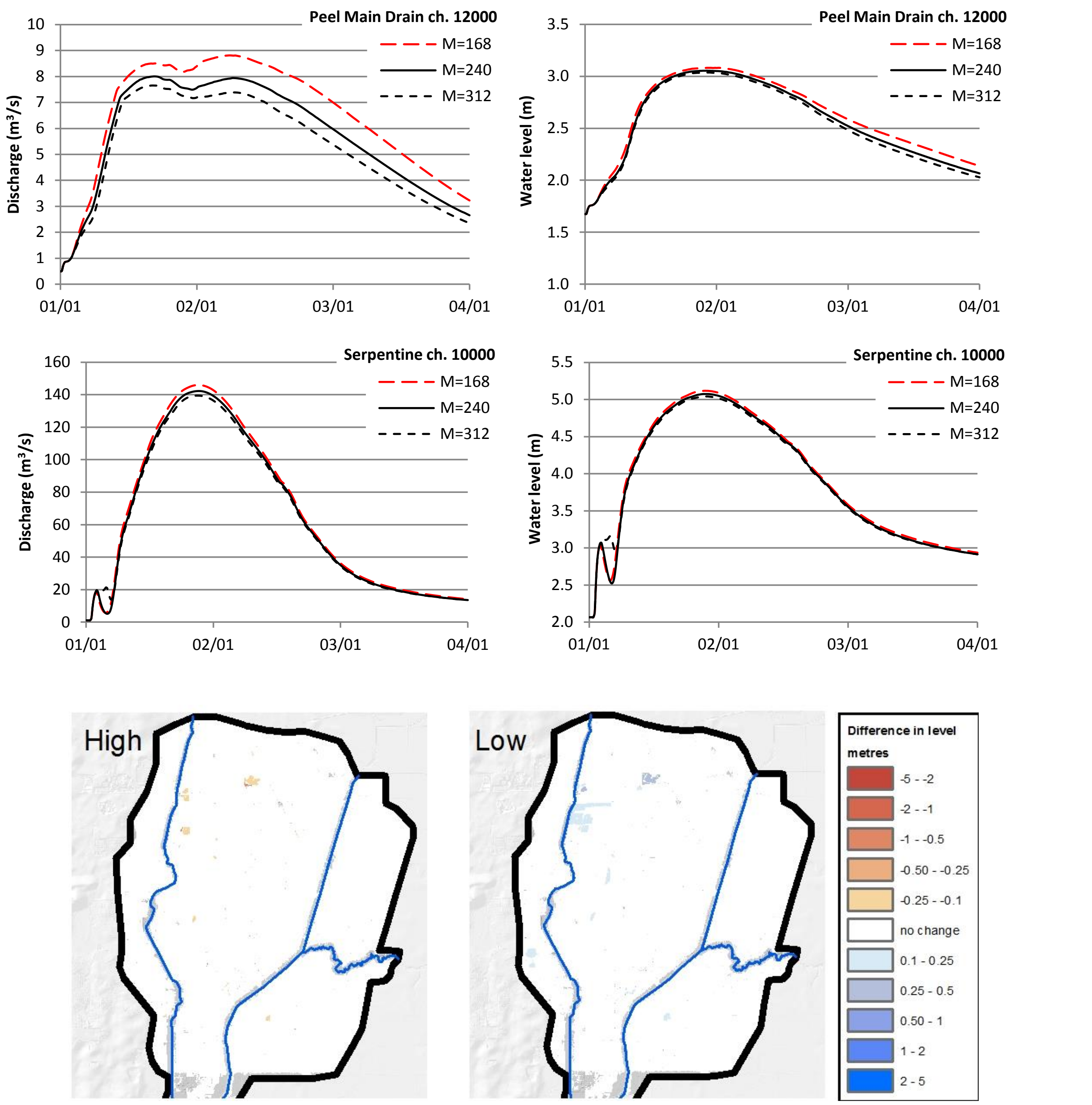


Figure 6-8: Sensitivity to MIKE 21 infiltration parameters

Sensitivity: Tail water level

Figure 6-9 shows that north of Karnup Road, the model is insensitive to the tail water condition up to 0.9 mAHD. However, note that the Peel Main Drain is influenced by raised water levels in the Serpentine Main Drain. This sensitivity analysis indicates that flooding in the Peel and Serpentine main drains is unlikely to be influenced by a 0.9 m sea-level-rise to the north of Karnup Road.

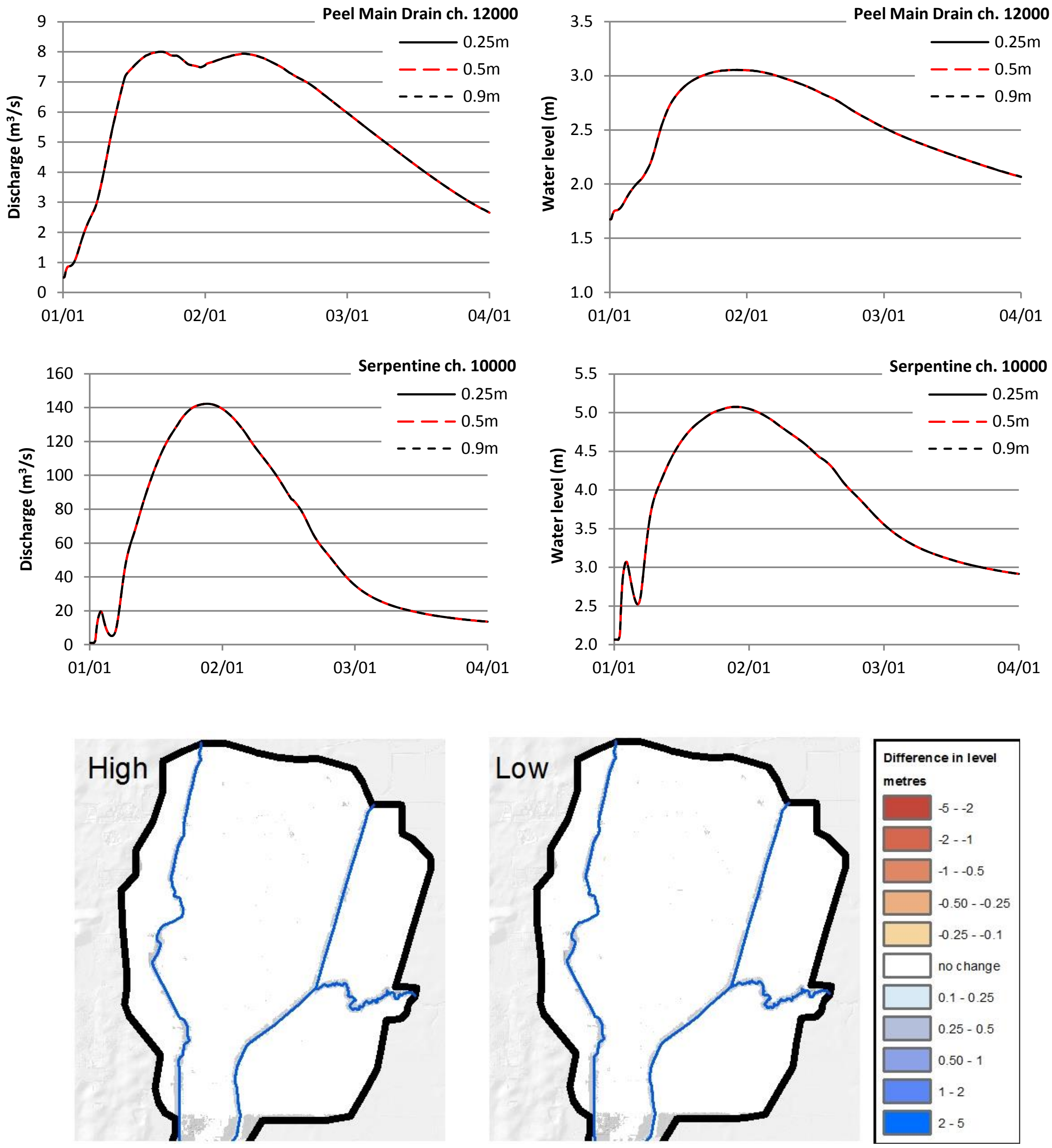


Figure 6-9: Sensitivity to tail water conditions

6.7 Considerations for drainage design within the study area

This flood modelling study highlights several important considerations related to urban development and drainage design for the north-east Baldivis area. Although this modelling does not attempt to make prescriptive statements regarding design, it aims to identify major flood hazards and potential issues associated with urban development. It is recommended that any development or drainage design on the western side of the Birrega and Serpentine Main Drain within the study area considers the following:

- The contribution of flood water from water overtopping in the Duck Pool section of Birrega Main Drain. In a 100 yr event, the western levee bank of the Birrega Main Drain would overtop, directing up to 20 m^3/s into the north-east Baldivis area.

- The potential for failure of the levee banks on the Birrega and Serpentine Main Drains. This study indicates that around 55 m^3/s would discharge to the north-east Baldivis area in a 100 yr event with complete levee failure.

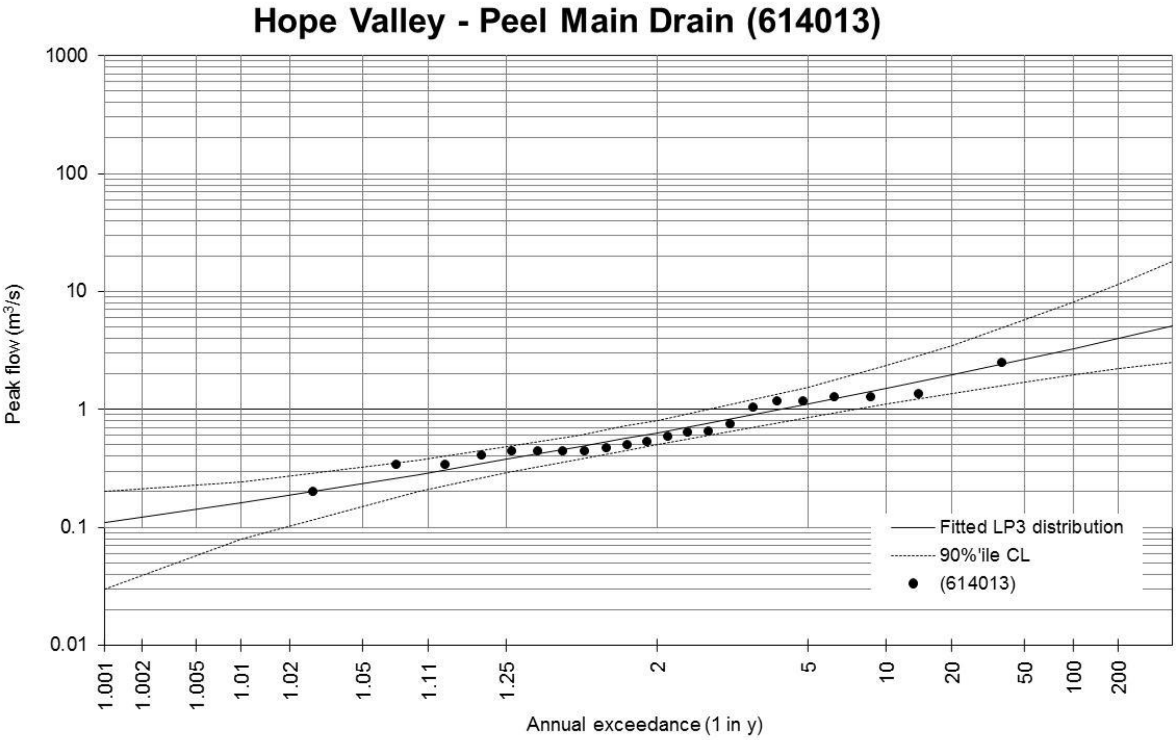
- The availability of free-draining outlets where groundwater levels are high and back-water effects are present. The low hydraulic grade will also influence drainage capacity.

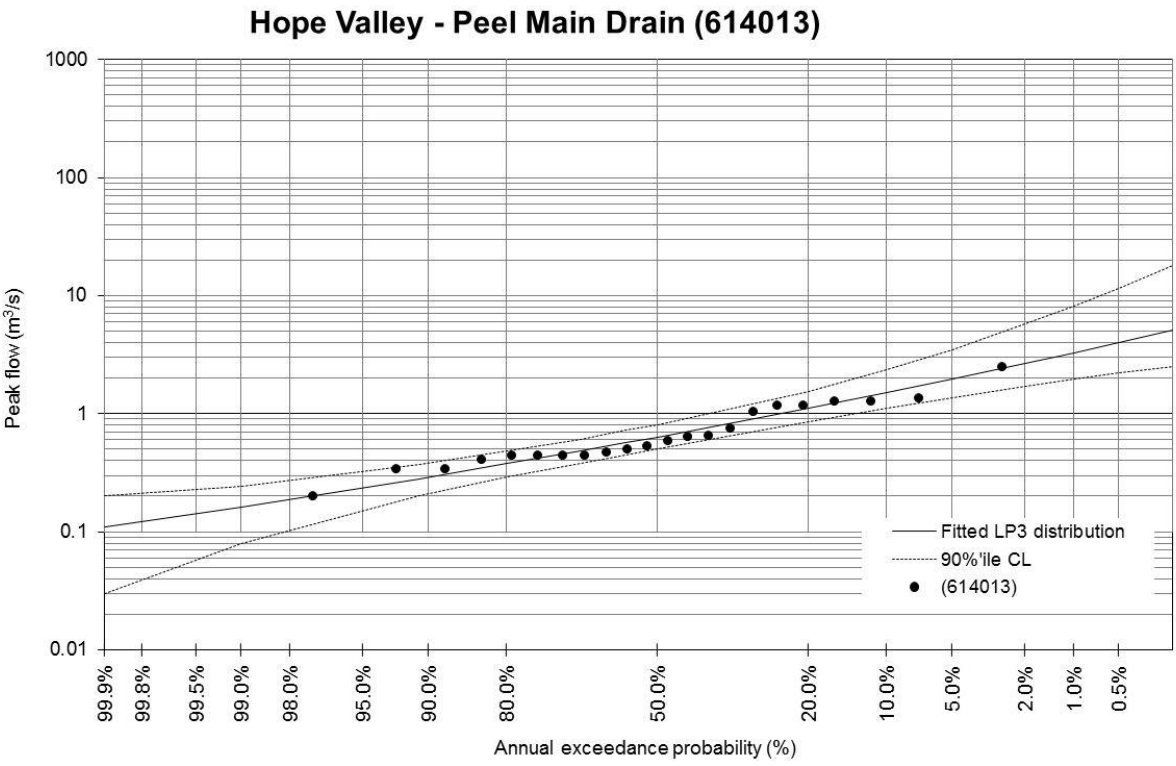
- The capacity of the Peel Main Drain to convey drainage water without influencing downstream land holders. The regular breaks and lateral culverts in the eastern bank of the Peel Main Drain mean that additional discharge to the drain upstream could result in increased flooding downstream.

Although none of these considerations rule out development within the study area, they may require that additional land be set aside for conveyance, storage and retention of flood water in comparison with other areas with a greater capacity for infiltration or a steeper hydraulic grade.

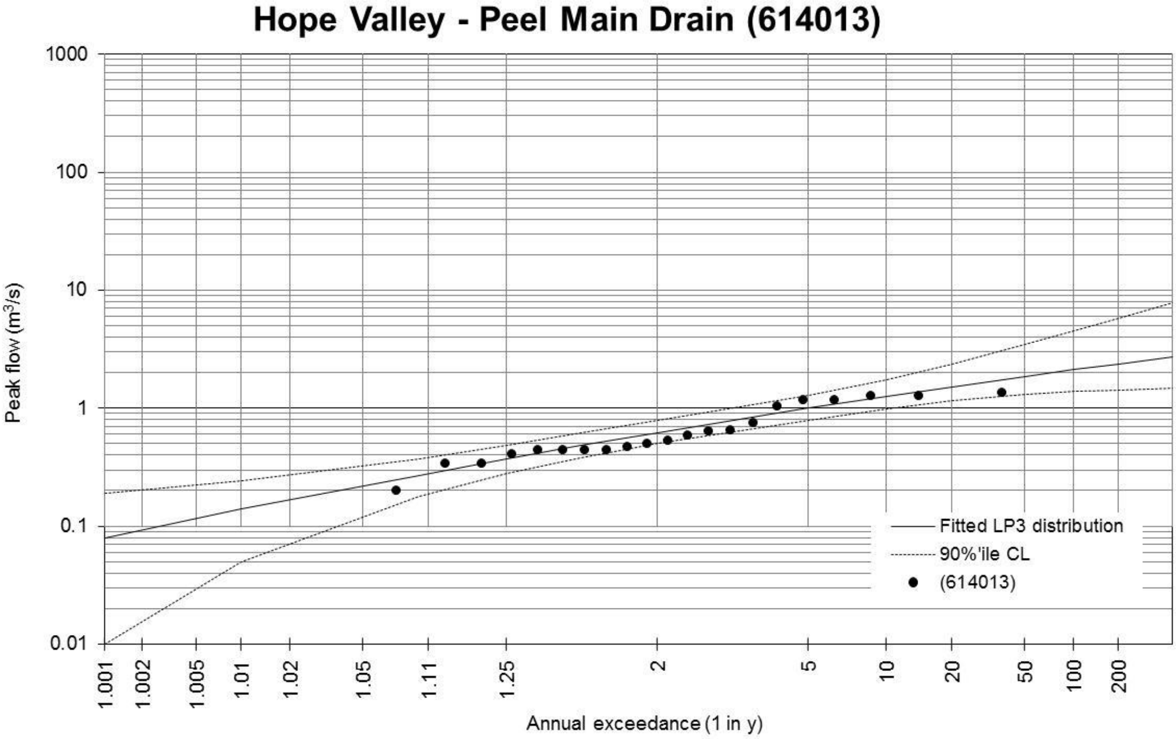
Appendix A – Flood frequency analysis plots

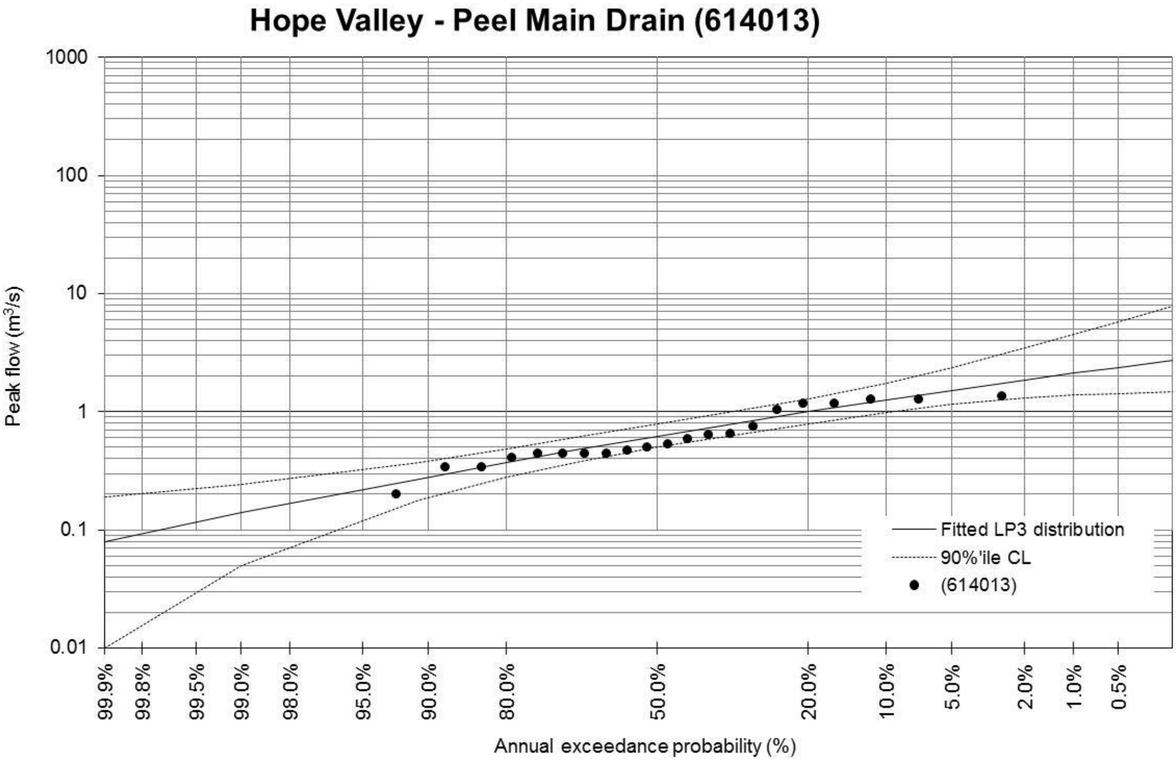
Hope Valley (including 1992 event)



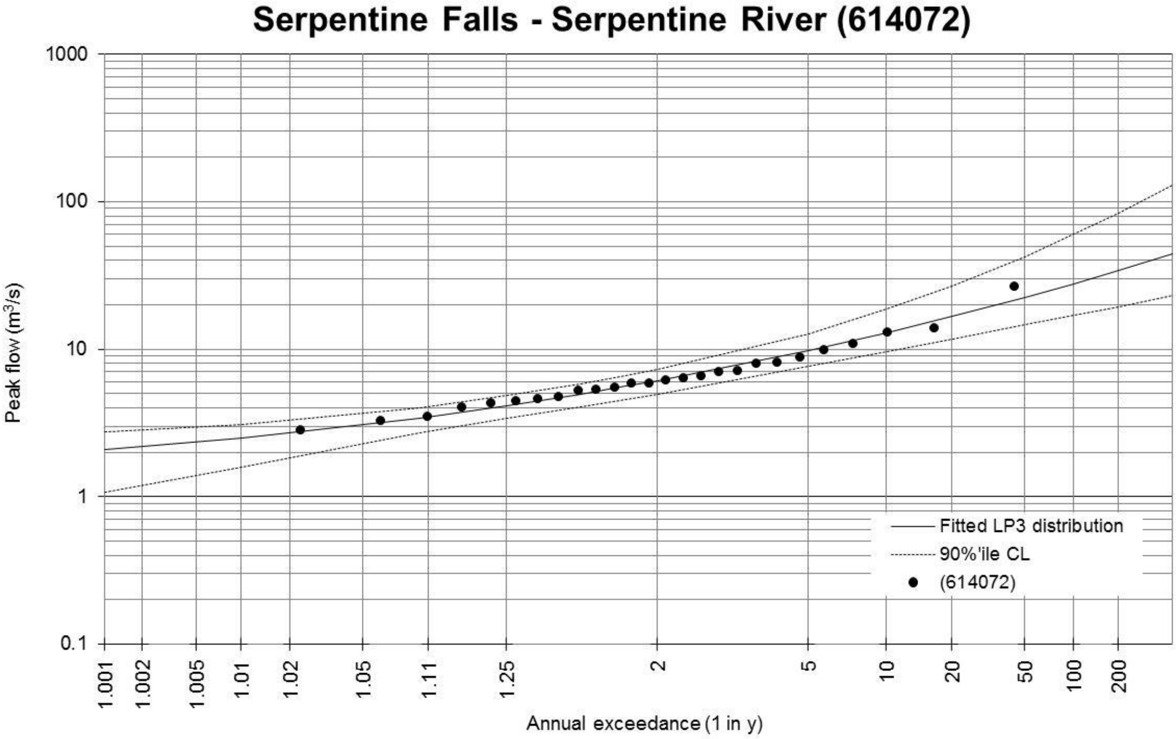


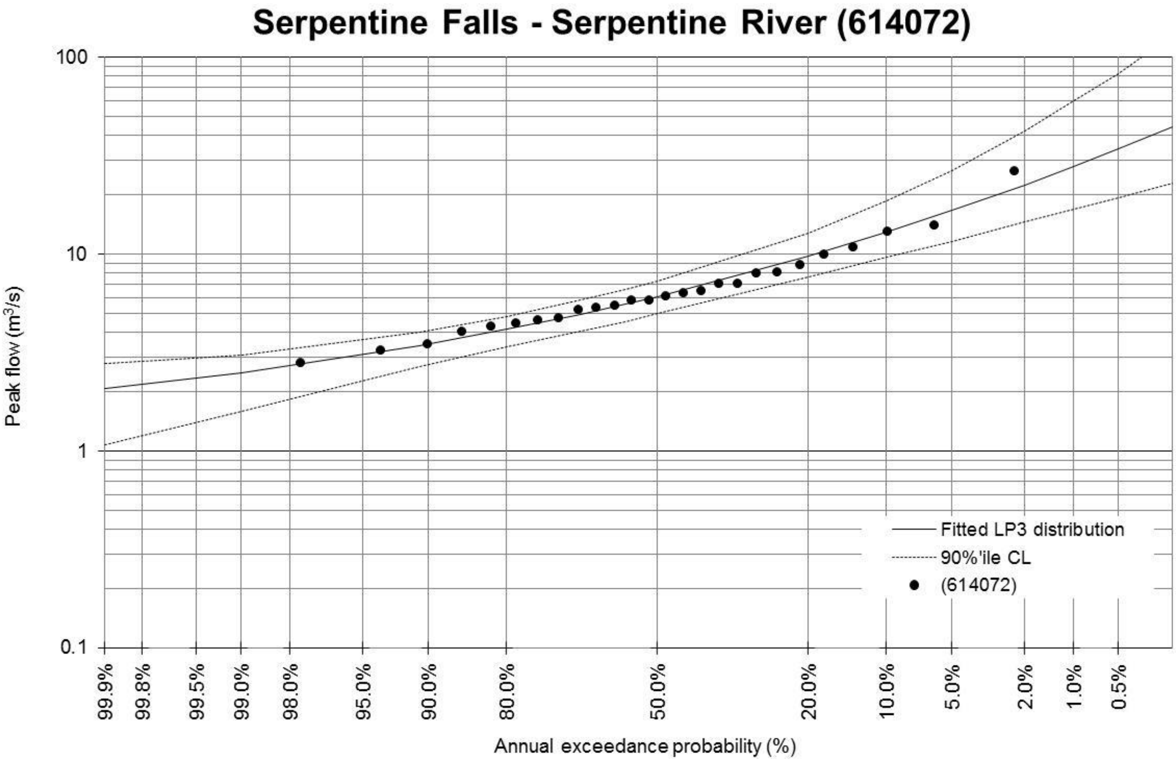
Hope Valley (excluding 1992 event)



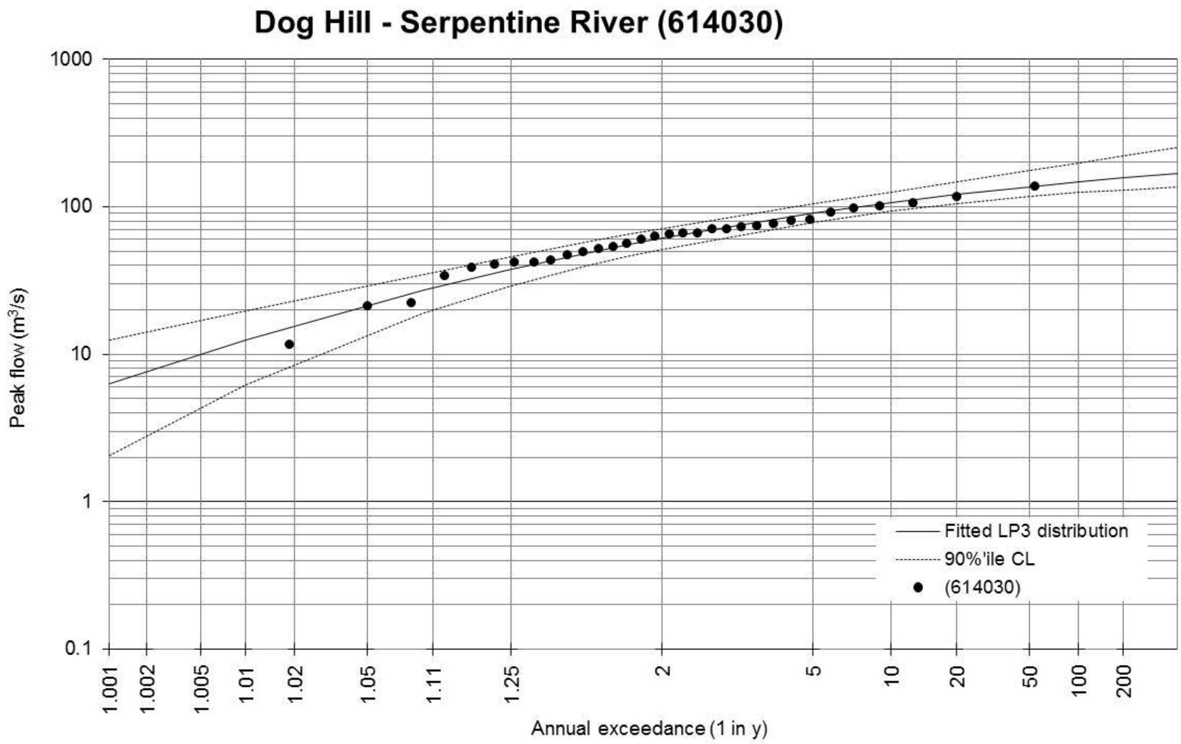


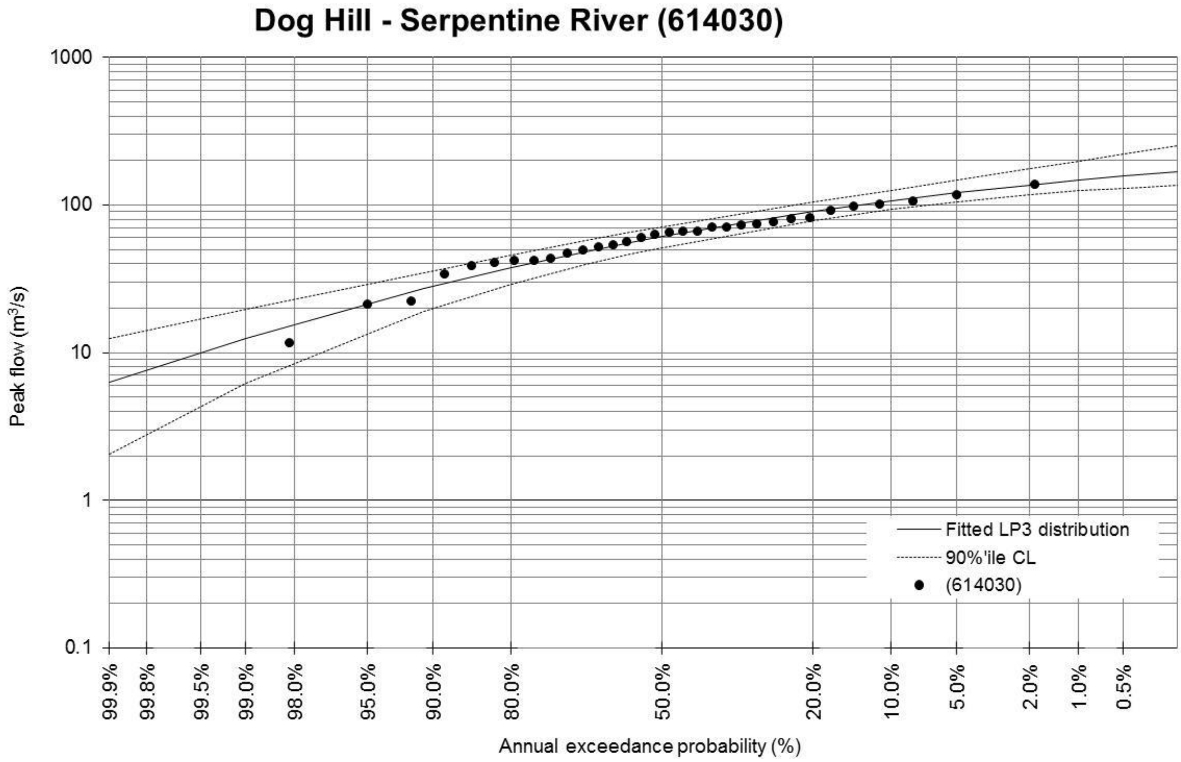
Serpentine Falls





Dog Hill


Dog Hill - Serpentine River (614030)
Peak flow (m3/s)
1000
100
10
1
0.1
Fitted LP3 distribution
90%'ile CL
(614030)
1.001
1.002
1.005
1.01
1.02
1.05
1.11
1.25
2
5
10
20
50
100
200
Annual exceedance (1 in y)


Dog Hill - Serpentine River (614030)
Peak flow (m3/s)
1000
100
10
1
0.1
Fitted LP3 distribution
90%'ile CL
(614030)
99.9%
99.8%
99.5%
99.0%
98.0%
95.0%
90.0%
80.0%
50.0%
20.0%
10.0%
5.0%
2.0%
1.0%
0.5%
Annual exceedance probability (%)

Appendix B – RORB calibration plots

Upper Peel Main Drain

September 1978 winter event

The RORB model matched the peak flow of the September 1978 event but the hydrograph volume was overestimated, mostly on the rising limb. The time to peak was accurately simulated by RORB.

Table B-1: RORB calibration parameters for the Peel Main Drain (September 1978)

Flow gauge	Pluviometer/s	Rainfall (mm)	Rainfall duration (hrs)	Event start date & time
Hope Valley (614013)	Mandogalup (509311)	63.5	20	30/09/1978 12:15

		Hydrograph		Error	
	Units	Calc.	Obs.	Abs.	%
Peak flow	m^3/s	1.03	0.97	0.06	5.7
Time to peak	h	20	21	-1.2	-5.9
Volume	ML	93.0	68.7	24.3	35.4

Calibrated parameters	
IL (mm)	5
RoC	0.08
Kc	14
m	0.85

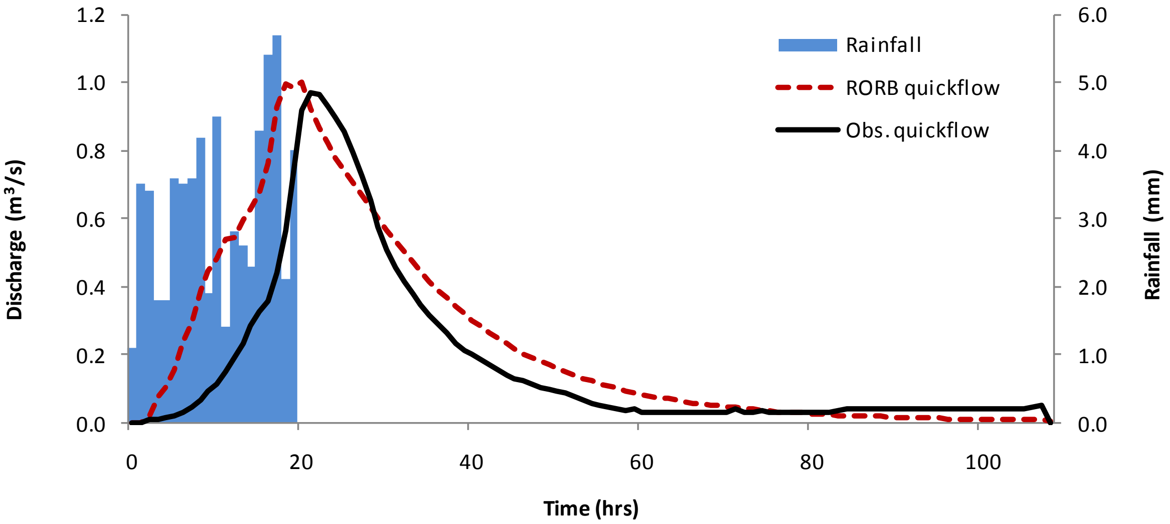


Figure B-1: RORB modelled and observed flows for the Peel Main Drain (September 1978)

January 1982 summer event

The 1982 event occurred in summer and therefore an initial loss of 70 mm was required for calibration. The modelled time to peak was around 4 hr slower than the observed hydrograph; however, peak flows and flood volume are closely matched, and the modelled recession limb is similar to the observed.

Table B-2: RORB calibration parameters for the Peel Main Drain (January 1982)

Flow gauge	Pluviometer/s	Rainfall (mm)	Rainfall duration (hrs)	Event start date & time
Hope Valley (614013)	Mandogalup (509311)	124.7	51	20/01/1982 0:15

	Units	Hydrograph Calc.	Hydrograph Obs.	Error Abs.	Error %
Peak flow	m³/s	0.96	0.92	0.04	4.5
Time to peak	h	49	44	4.5	10.2
Volume	ML	96.9	92.9	3.9	4.3

Calibrated parameters	
IL (mm)	70
RoC	0.09
Kc	14
m	0.85

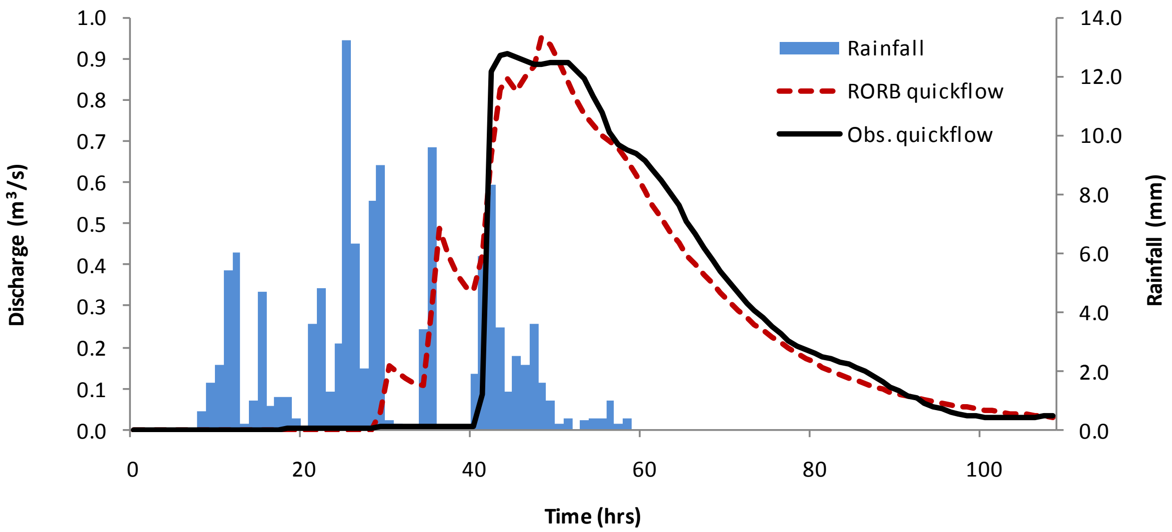


Figure B-2: RORB modelled and observed flows for the Peel Main Drain (January 1982)

July 1987 winter event

Peak flows and volumes calibrated well for the July 1987 event. The flow volume is over-estimated for the recession limb of the hydrograph. Although the absolute timing of the peak reported by RORB shows an error of around 8 hr, in fact, the timing of the first and second peaks in the model results are within 2 hr of the respective observed peaks.

Table B-3: RORB calibration parameters for the Peel Main Drain (July 1987)

Flow gauge	Pluviometer/s	Rainfall (mm)	Rainfall duration (hrs)
Hope Valley (614013)	Dog Hill (509295)	77.7	18

Event start date & time
28/07/1987 9:00

	Units	Hydrograph Calc.	Hydrograph Obs.	Error Abs.	Error %
Peak flow	m³/s	1.08	1.02	0.06	5.5
Time to peak	h	15	24	-8.2	-35.1
Volume	ML	95.4	83.4	1.2	14.4

Calibrated parameters	
IL (mm)	5
RoC	0.07
Kc	14
m	0.85

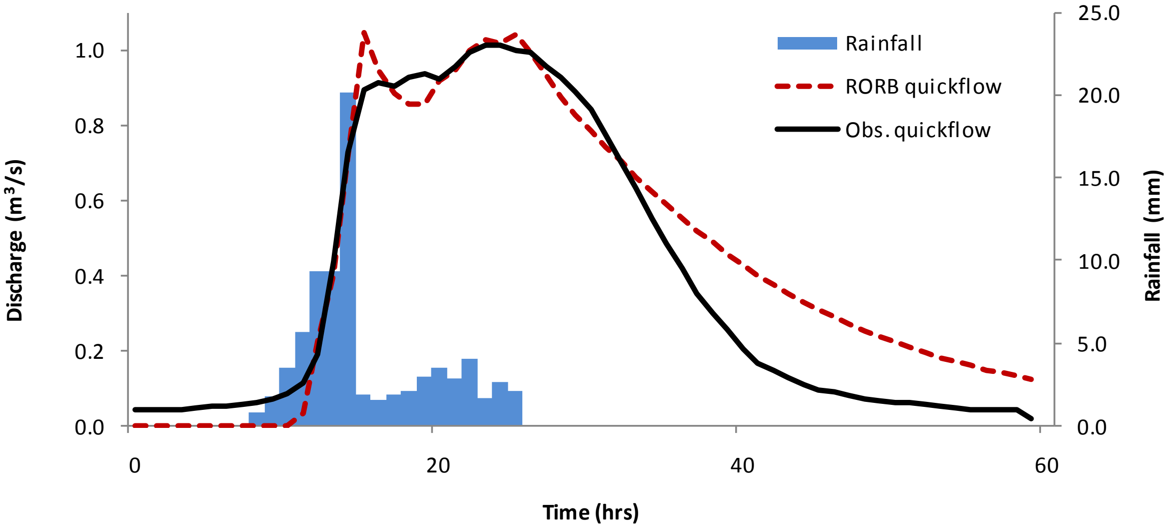


Figure B-3: RORB modelled and observed flows for the Peel Main Drain (July 1987)

July 1988 winter event

The RORB model calibrated to the peak and volume of the July 1998 event. Volume is slightly overestimated under the recession limb of the RORB modelled quickflow.

Table B- 4: RORB calibration parameters for the Peel Main Drain (July 1988)

Flow gauge	Pluviometer/s	Rainfall (mm)	Rainfall duration (hrs)	Event start date & time
Hope Valley (614013)	Dog Hill (509295)	65.3	21	22/07/1988 1:00

	Units	Hydrograph Calc.	Hydrograph Obs.	Error Abs.	Error %
Peak flow	m³/s	1.01	0.97	0.04	4.0
Time to peak	h	47	47	-0.8	-1.6
Volume	ML	95.8	88.5	7.3	8.2

Calibrated parameters	
IL (mm)	5
RoC	0.08
Kc	14
m	0.85

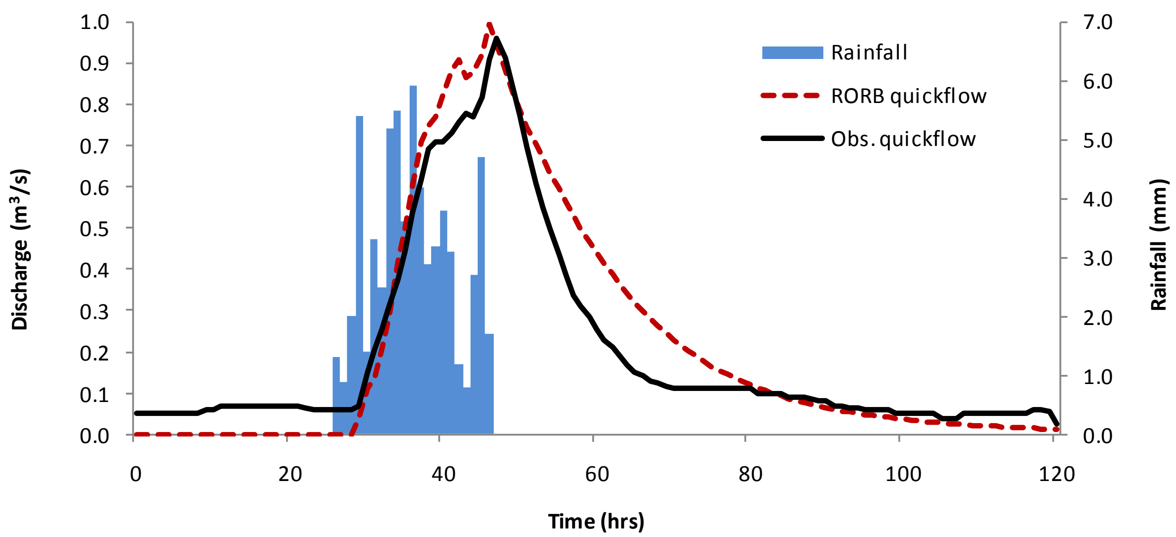


Figure B-4: RORB modelled and observed flows for the Peel Main Drain (July 1988)

February 1992 summer event

An initial loss of 70 mm was required for this February event and the RORB model calibrated well with parameters similar to the 1982 event. It was not possible to replicate the two peaks observed in the gauged hydrograph despite the apparent two bursts of rainfall. Most of the first burst is absorbed in the initial loss. The peak, timing and volume of the event are similar for the modelled and observed hydrograph. It is worth noting that the shorter duration of the 1992 summer event relative to the 1982 summer event (20 hr versus 50 hr) results in a significantly higher peak discharge, despite similar rainfall depths, indicating a shorter critical duration for this catchment.

Table B-5: RORB calibration parameters for the Peel Main Drain (February 1992)

Flow gauge	Pluviometer/s	Rainfall (mm)	Rainfall duration (hrs)	Event start date & time
Hope Valley (614013)	Dog Hill (509295)	139	20	8/02/1992 8:00

	Units	Hydrograph Calc.	Hydrograph Obs.	Error Abs.	Error %
Peak flow	m³/s	2.20	2.23	-0.03	-1.3
Time to peak	h	22	24	-2.2	-9.3
Volume	ML	129.0	144.0	-15.3	-10.6

Calibrated parameters	
IL (mm)	70
RoC	0.10
Kc	14
m	0.85

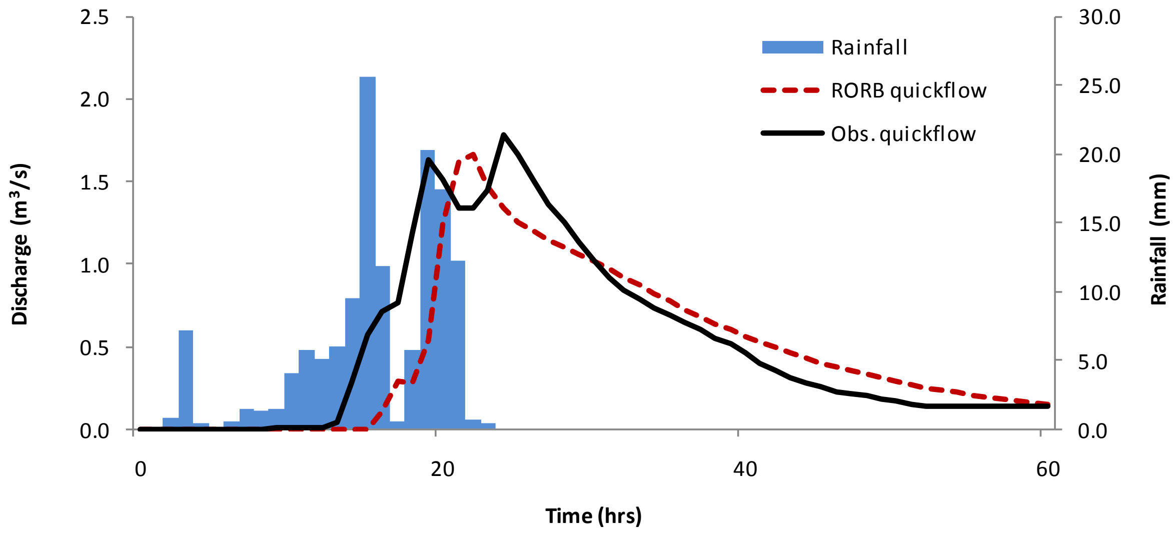


Figure B-5: RORB modelled and observed flows for the Peel Main Drain (February 1992)

August 1994 winter event

The observed hydrograph associated with the August 1994 has a more distinct peak relative to the other calibration events, the modelled recession limb is longer, and contains more volume than that of the observed hydrograph. Time to peak and peak flow are both accurately simulated by the RORB model.

Table B-6: RORB calibration parameters for Peel Main Drain August 1994

Flow gauge	Pluviometer/s	Rainfall (mm)	Rainfall duration (hrs)	Event start date & time
Hope Valley (614013)	Ceriani Farm (509270)	62	14	3/08/1994 0:00

	Units	Hydrograph Calc.	Hydrograph Obs.	Error Abs.	Error %
Peak flow	m^3/s	0.93	0.93	0.03	0.3
Time to peak	h	25	26	-1	-3.8
Volume	ML	79.3	62.1	17.2	27.6

Calibrated parameters	
IL (mm)	5
RoC	0.07
Kc	14
m	0.85

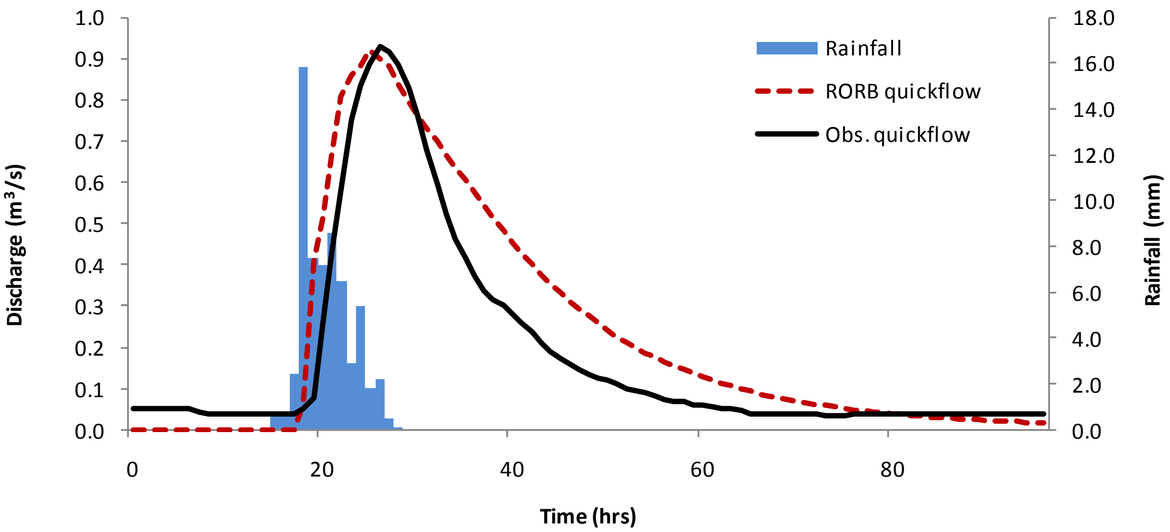


Figure B-6: RORB modelled and observed flows for Peel Main Drain (August 1994)

Serpentine River

July 1987 winter event

The July 1987 event was used to assess the shape of the hydrograph produced by the RORB model at Lowlands against the area-weighted observed hydrograph at Dog Hill for the same event. The Lowlands catchment area is 39% of the Dog Hill catchment so the observed quick flow was scaled by 0.39 to produce the area-weighted flow. Figure B-7 shows that the timing of the peak is appropriate.

At the Serpentine Falls gauge, the hydrograph peak is underpredicted by 11%, and is 1 hour later than the observed peak time. The timing of the second peak in the event is accurate but discharge is underpredicted, which results in the large error in volume. A good calibration was achieved at the Mundlimup gauge, with a slight overprediction in peak flow. The difference in the timing of the peak is due to the rainfall data only, as the observed peak occurs before the rainfall event occurs. As with the Serpentine Falls gauge, event volume is underpredicted at Mundlimup.

Table B-7: RORB calibration parameters for the Serpentine River, Lowlands (July 1987)

Flow gauge	Pluviometer/s	Rainfall (mm)	Rainfall duration (hrs)	Event start date & time
Dog Hill (614030) observed	Dog Hill (509295)	77.7	18	28/07/1987 9:00
Lowlands (614114) modelled	Kentish Farm (509245)	99.7	28	

**Calibration statistics are not applicable as hydrographs represent modelled flows at Lowlands gauge, and observed flows at Dog Hill*

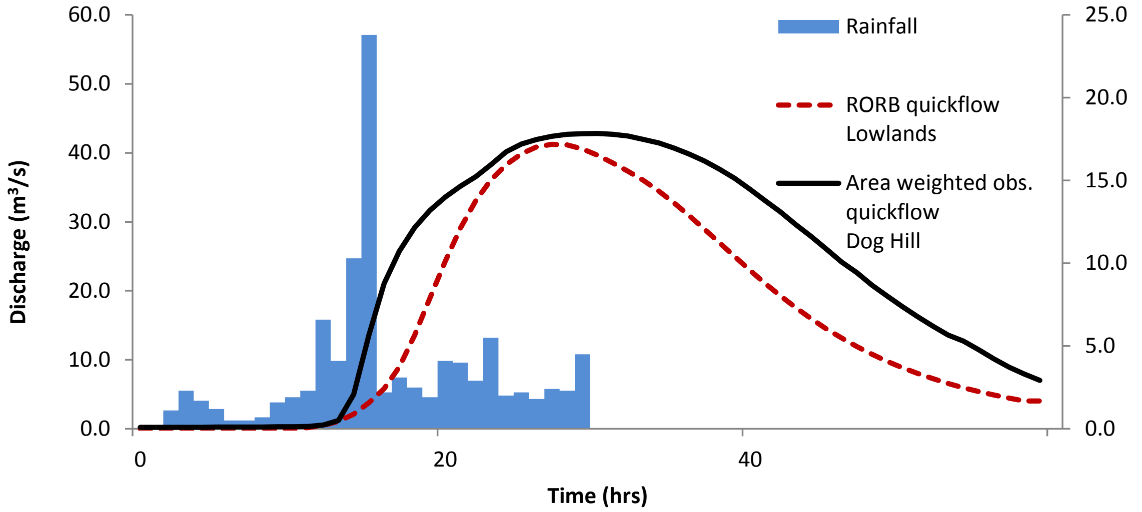


Figure B-7: RORB modelled and area-weighted observed flows for Serpentine River, Lowlands (July 1987)

Table B-8: RORB calibration parameters for the Serpentine River, Serpentine Falls (July 1987)

Flow gauge	Pluviometer/s	Rainfall (mm)	Rainfall duration (hrs)	Event start date & time
Serpentine Falls (614072)	Dog Hill (509295)	43.6	10	3/07/2000 18:00
	Kentish Farm (509245)	48.1	11	

	Units	Hydrograph Calc.	Hydrograph Obs.	Error Abs.	Error %
Peak flow	m³/s	23.0	25.9	-2.90	-11.2
Time to peak	h	16	15	0.8	4.9
Volume	ML	577.0	1410.0	833.0	-59.1

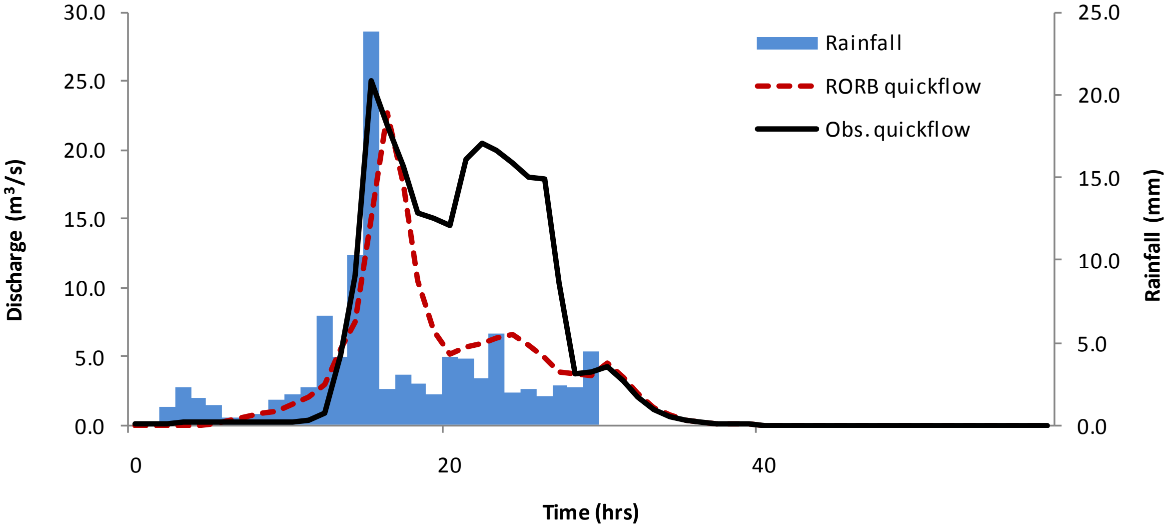


Figure B-8: RORB modelled and observed flows for Serpentine River, Serpentine Falls (July 1987)

Table B-9: RORB calibration parameters for the Serpentine River, Mundlimup (July 1987)

Flow gauge	Pluviometer/s	Rainfall (mm)	Rainfall duration (hrs)	Event start date & time
Serpentine Falls (614072)	Dog Hill (509295)	43.6	10	3/07/2000 18:00
	Kentish Farm (509245)	48.1	11	

	Units	Hydrograph Calc.	Hydrograph Obs.	Error Abs.	Error %
Peak flow	m³/s	13.2	11.8	1.39	11.8
Time to peak	h	16	13	3	22.6
Volume	ML	294.0	506.0	212.0	-41.8

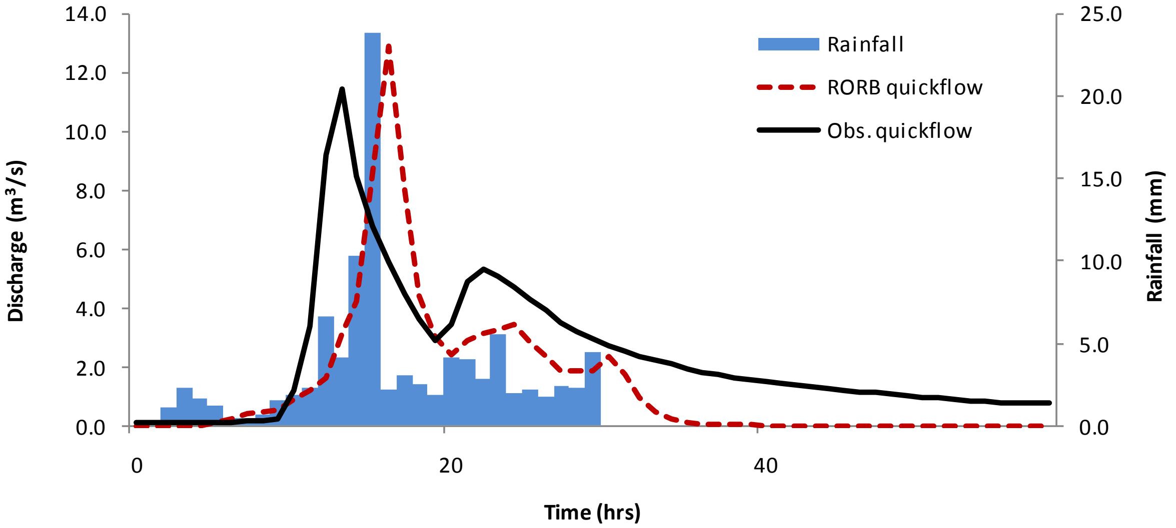


Figure B-9: RORB modelled and observed flows for Serpentine River, Mundlimup (July 19870

July 1988 winter event

The July 1988 event was used to assess the shape of the hydrograph produced by the RORB model at Lowlands against the area-weighted observed hydrograph at Dog Hill for the same event. The timing and shape of the hydrograph are similar at Dog Hill and Lowlands. The event volume from Lowlands is 33% of the total volume recorded at Dog Hill.

The Mundlimup gauge calibrated poorly for this event, peak flow was overestimated by 14% and event volume was underpredicted. It is possible to achieve a better calibration with higher *Kc* and *RoC* but the parameters are inconsistent with the 1987 calibration event which included calibration to Serpentine Falls. It is possible that the poor calibration is due to rainfall data, and the difference in volume is due to remaining baseflow in the observed beyond the 60 hr mark. Excluding the recession limb, volume is comparable over the event.

Table B-10: RORB calibration parameters for the Serpentine River, Lowlands (July 1988)

Flow gauge	Pluviometer/s	Rainfall (mm)	Rainfall duration (hrs)	Event start date & time
Dog Hill (614030) observed	Hopelands (509387)	71.2	18	22/07/1988 1:00
Lowlands (614114) modelled	Kentish Farm (509245)	84.8	40	

**Calibration statistics are not applicable as hydrographs represent modelled flows at Lowlands gauge, and observed flows at Dog Hill*

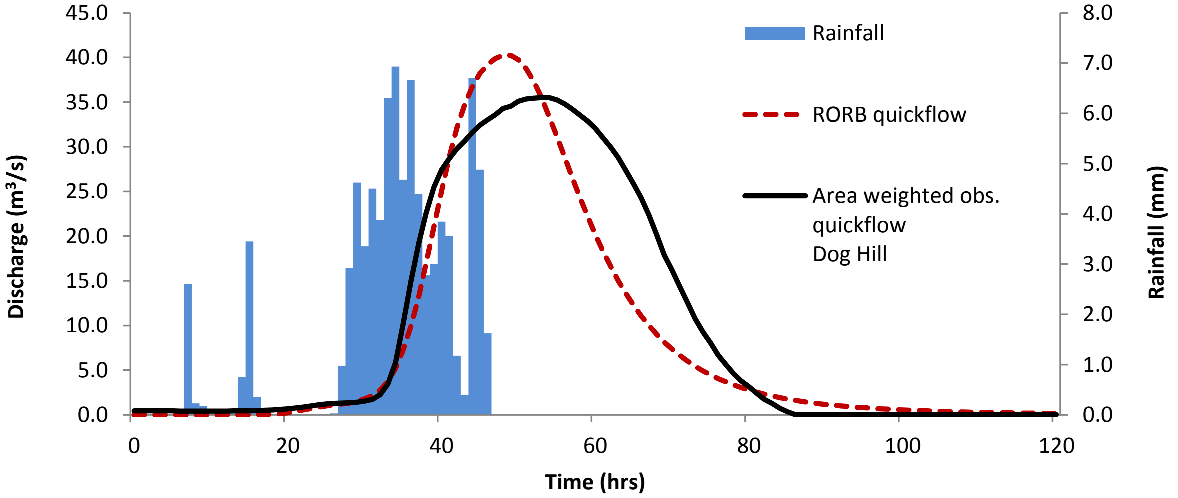


Figure B-10: RORB modelled and area-weighted observed flows for Serpentine River, Lowlands (July 1988)

Table B-11: RORB calibration parameters for the Serpentine River, Mundlimup, July 1988

Flow gauge	Pluviometer/s	Rainfall (mm)	Rainfall duration (hrs)	Event start date & time
Dog Hill (614030) observed	Hopelands (509387)	71.2	18	22/07/1988 1:00
Lowlands (614114) modelled	Kentish Farm (509245)	84.8	40	

	Units	Hydrograph Calc.	Hydrograph Obs.	Error Abs.	Error %
Peak flow	m³/s	6.1	5.3	0.75	14.0
Time to peak	h	35	45	-9.8	-21.8
Volume	ML	294.0	539.0	245.0	-45.5

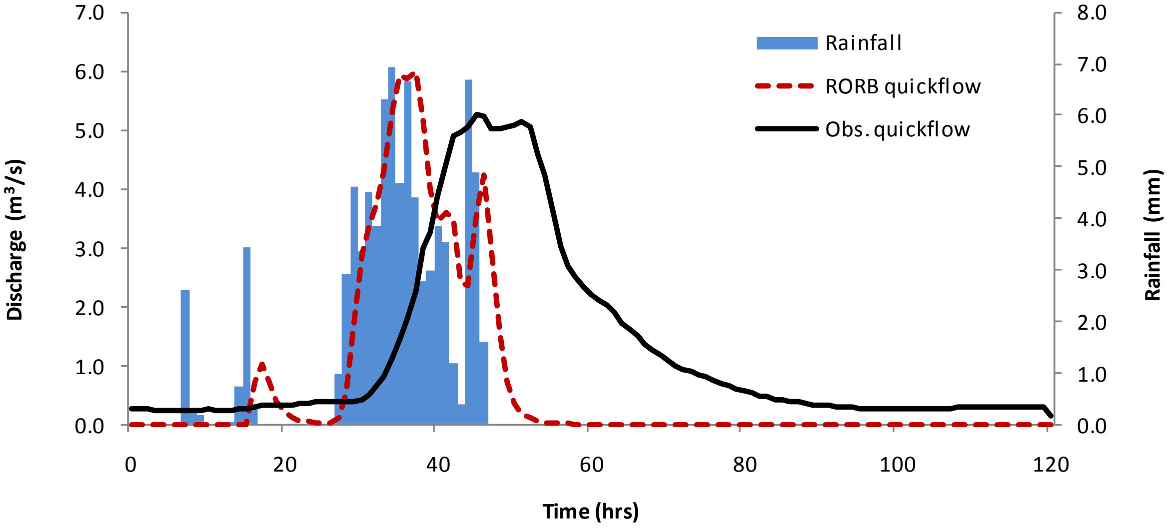


Figure B-11: RORB modelled and observed flows for Serpentine River, Mundlimup (July 1988)

July 2000 winter event

The timing, volume and peak discharge of the July 2000 event are well calibrated at Lowlands.

Table B-12: RORB calibration parameters for the Serpentine River (July 2000)

Flow gauge	Pluviometer/s	Rainfall (mm)	Rainfall duration (hrs)	Event start date & time
Lowlands (614114)	Dog Hill (509295)	43.6	10	3/07/2000 18:00
	Kentish Farm (509245)	48.1	11	

	Units	Hydrograph		Error	
		Calc.	Obs.	Abs.	%
Peak flow	m^3/s	22.28	21.66	0.62	2.9
Time to peak	h	21	24	-2.5	-10.6
Volume	ML	1630.0	1550.0	80.0	5.1

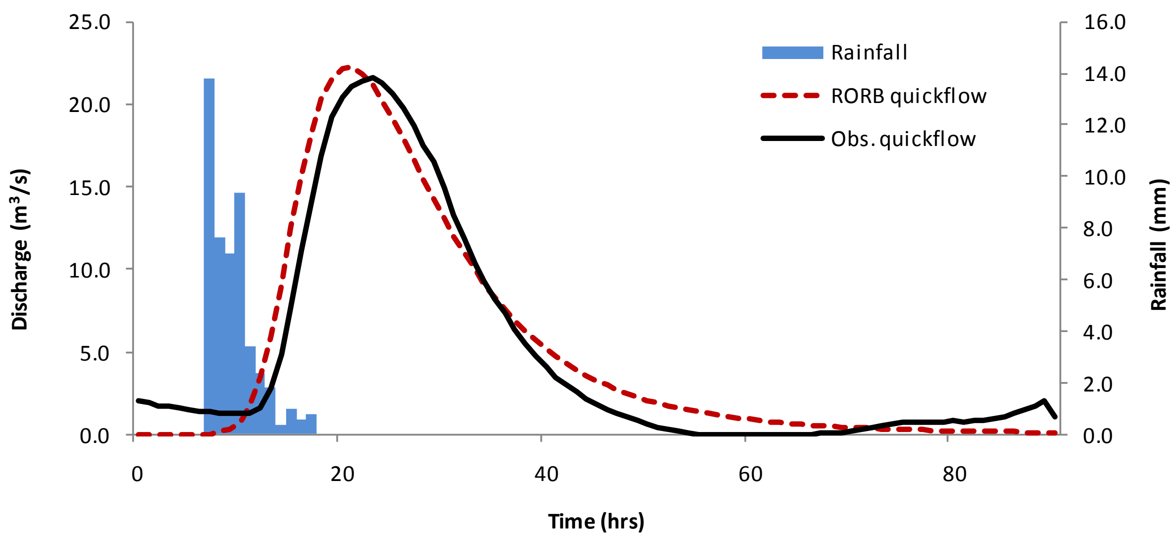


Figure B-12: RORB modelled and observed flows for Serpentine River (July 2000)

August 2002 winter event

The August 2002 event volume and peak are both overestimated by the RORB model.

Table B-13: RORB calibration parameters for the Serpentine River (August 2002)

Flow gauge	Pluviometer/s	Rainfall (mm)	Rainfall duration (hrs)	Event start date & time
Lowlands (614114)	Dog Hill (509295)	47.7	6	8/08/2002 1:00
	Serpentine (009039)	60.2	6	

	Units	Hydrograph Calc.	Hydrograph Obs.	Error Abs.	Error %
Peak flow	m³/s	30.97	25.97	5.00	19.3
Time to peak	h	33	36	-2.2	-6.3
Volume	ML	2050.0	1520.0	529.0	34.8

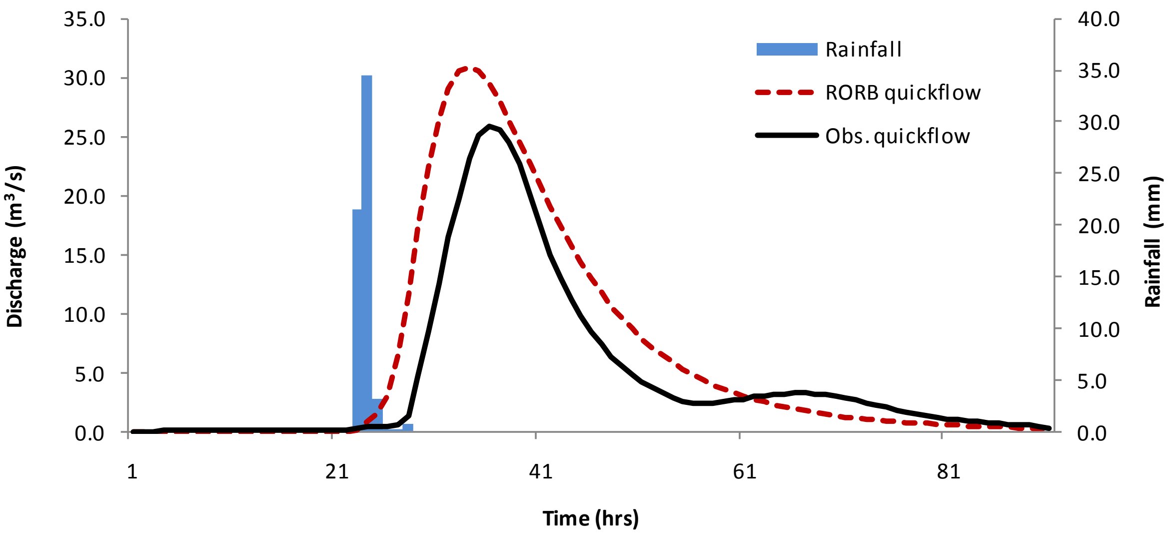


Figure B-13: RORB modelled and observed flows for the Serpentine River (August 2002)

August 2005 winter event

The August event results in a two-peaked hydrograph from two rainfall bursts. The timing of both peaks is replicated well by the RORB model, and the magnitude of the second, larger peak is underpredicted by 17%. The first peak is overestimated, as is the overall volume of the event.

Table B-14: RORB calibration parameters for the Serpentine River August 2005

Flow gauge	Pluviometer/s	Rainfall (mm)	Rainfall duration (hrs)	Event start date & time
Lowlands (614114)	Dog Hill (509295)	52.5	38	16/08/2005 0:00
	Kentish Farm (509245)	58.3	38	

	Units	Hydrograph		Error	
		Calc.	Obs.	Abs.	%
Peak flow	m³/s	15.75	18.99	-3.24	-17.1
Time to peak	h	49	52	-3	-5.8
Volume	ML	2040.0	1620.0	419.0	25.9

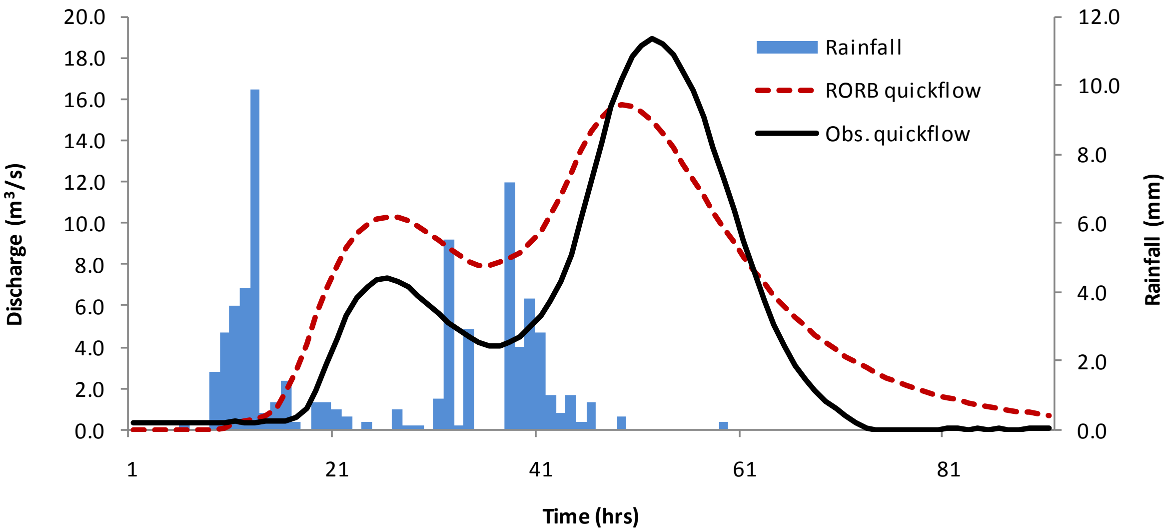


Figure B-14: RORB modelled and observed flows for Serpentine River (August 2005)

July 2007 winter event

The pluviometer data available indicates a two-burst rainfall event in July 2007 though the observed hydrograph at Lowlands recorded only a single peak. The RORB model accurately simulates the timing and magnitude of the first peak but overestimates the recession limb of the hydrograph and produces a second peak in response to the rainfall burst. This results in an event volume overestimate of 19.5%.

Table B-15: RORB calibration parameters for the Serpentine River (July 2007)

Flow gauge	Pluviometer/s	Rainfall (mm)	Rainfall duration (hrs)	Event start date & time
Lowlands (614114)	Dog Hill (509295)	42.4	69	29/07/2007 13:30
	Jack Rocks (509232)	83.5	72	

	Units	Hydrograph Calc.	Hydrograph Obs.	Error Abs.	Error %
Peak flow	m^3/s	21.33	26.22	-4.89	-18.6
Time to peak	h	32.2	31.0	1.2	4.0
Volume	ML	2830.0	2370.0	462.0	19.5

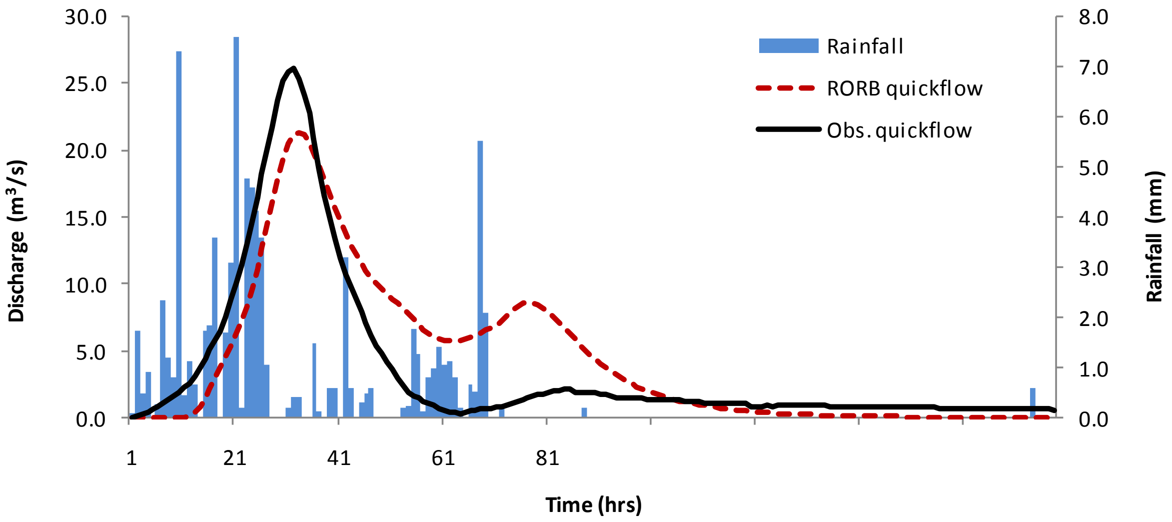


Figure B-15: RORB modelled and observed flows for the Serpentine River (July 2007)

Appendix C – Structures and dimensions

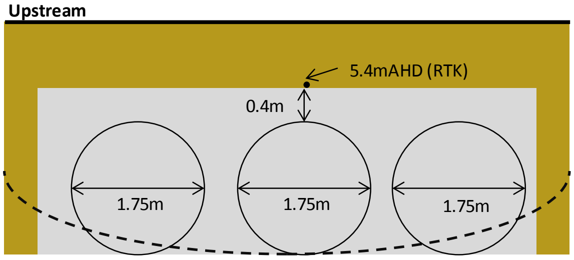
Peel Main Drain: in-line culverts

Culvert ID 13 - Long culvert under freeway on Peel Main Drain

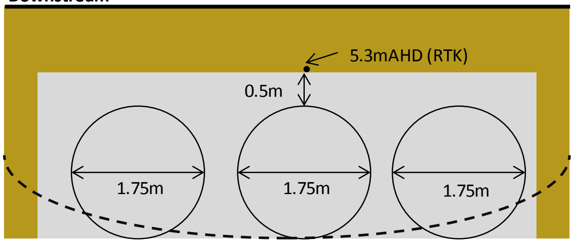
Not to scale

Culvert length: 137m

Upstream



Downstream

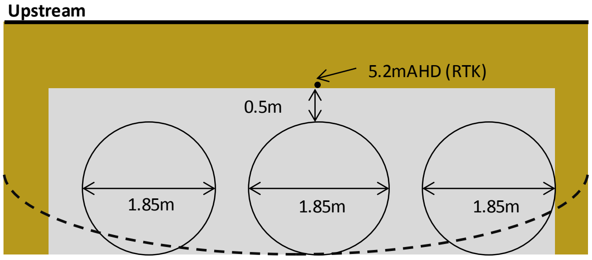


Culvert ID 18 - Peel Main Drain Mundijong Rd

Not to scale

Culvert length: 80m

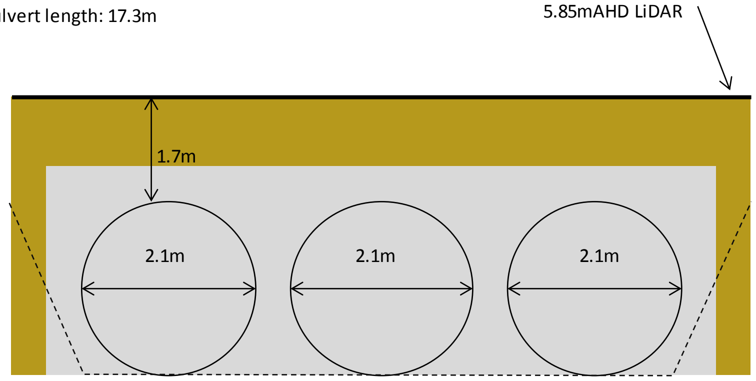
Upstream



Culvert ID 20 - Bertenshaw Road - Peel Main Drain

Not to scale

Culvert length: 17.3m

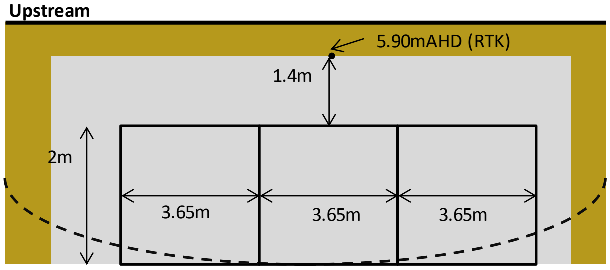


Culvert ID 21 - Safety Bay Rd Peel Main Drain

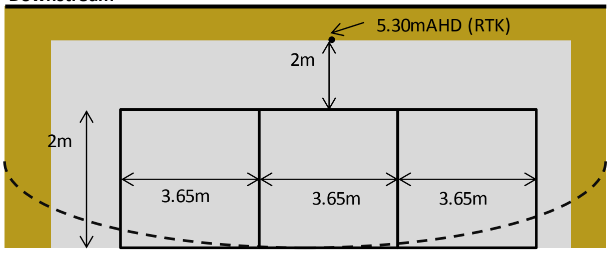
Not to scale

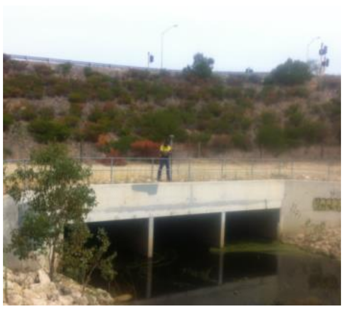
Culvert length: 74m

Upstream



Downstream

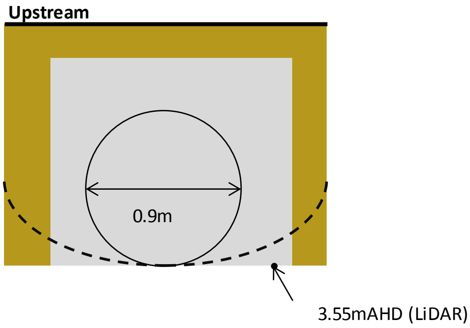




Culverts under the Kwinana freeway

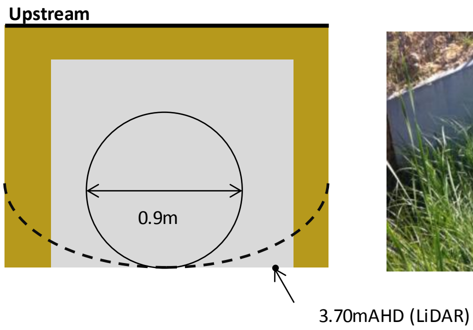
Culvert ID 23 - Under fwy	Not to scale

Culvert length: 110m (To PMD)



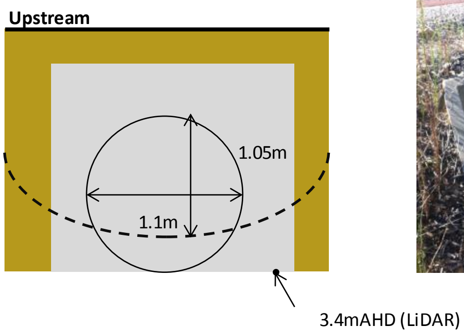
Culvert ID 25 - Under fwy	Not to scale

Culvert length: 100m



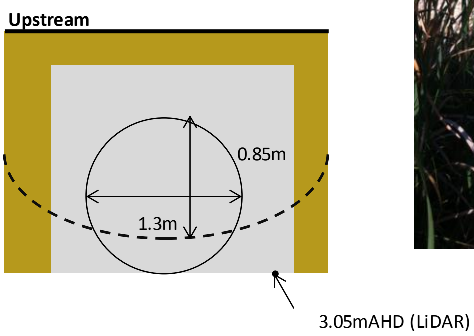
Culvert ID 26 - Under fwy	Not to scale

Culvert length: 100m



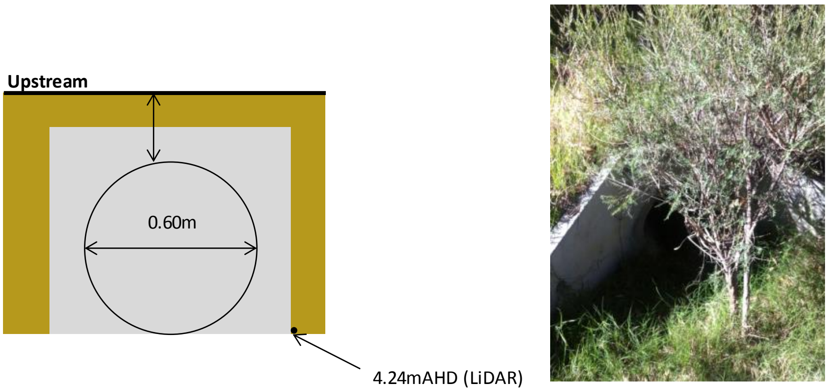
Culvert ID 51 - Under fwy	Not to scale

Culvert length: 104m



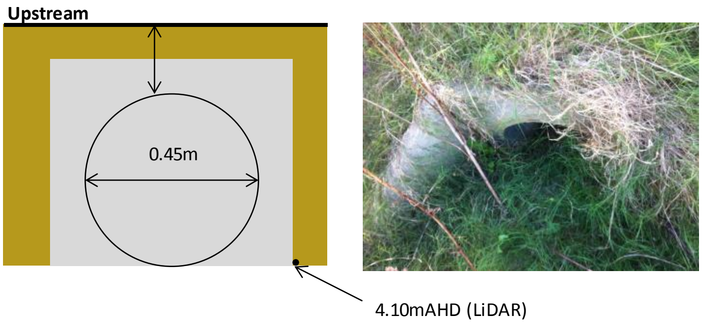
Culvert ID 86 - Fwy

Culvert length: 90m



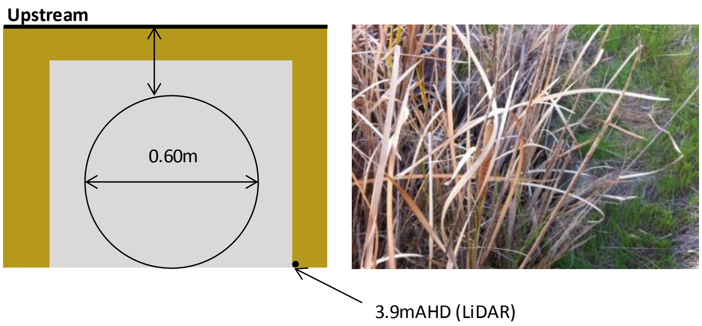
Culvert ID 87 - Fwy

Culvert length: 90m



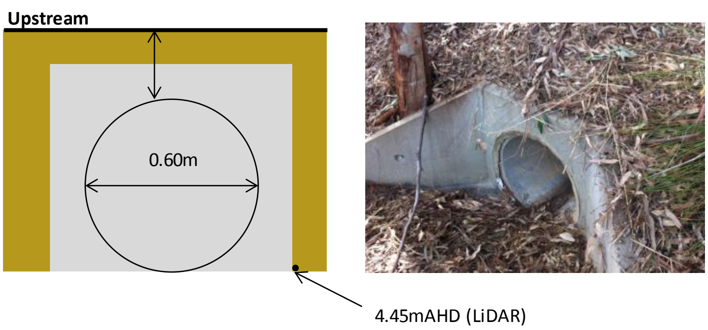
Culvert ID 89 - Fwy

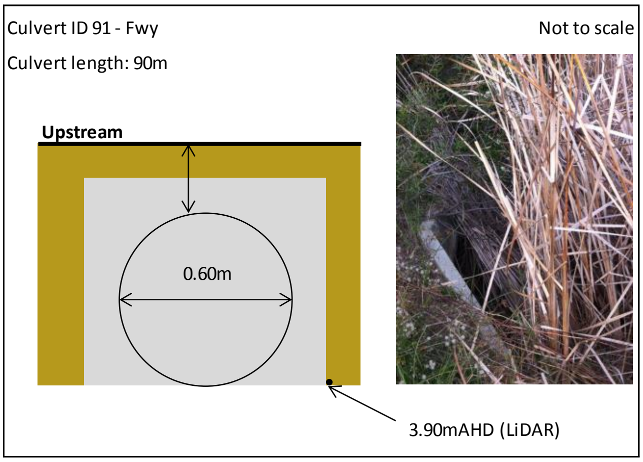
Culvert length: 90m

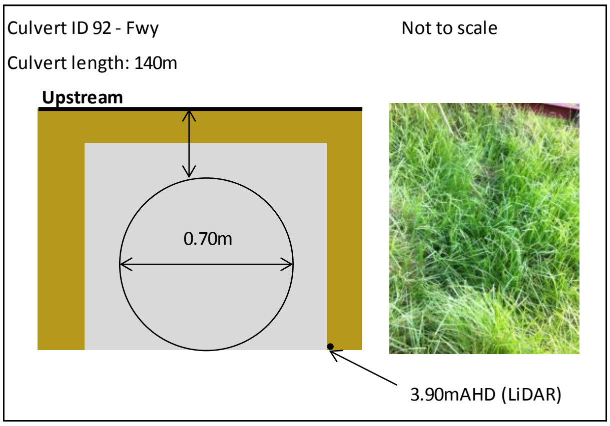


Culvert ID 90 - Fwy

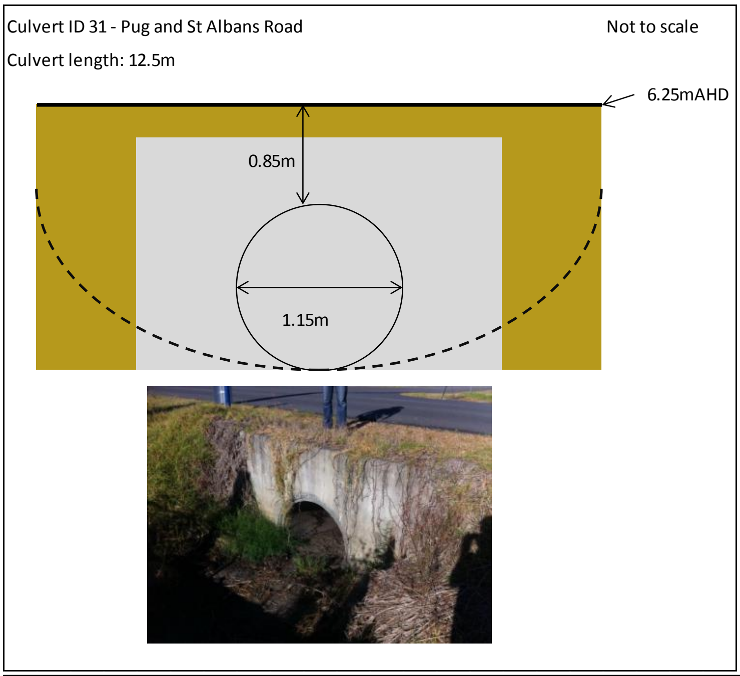
Culvert length: 90m

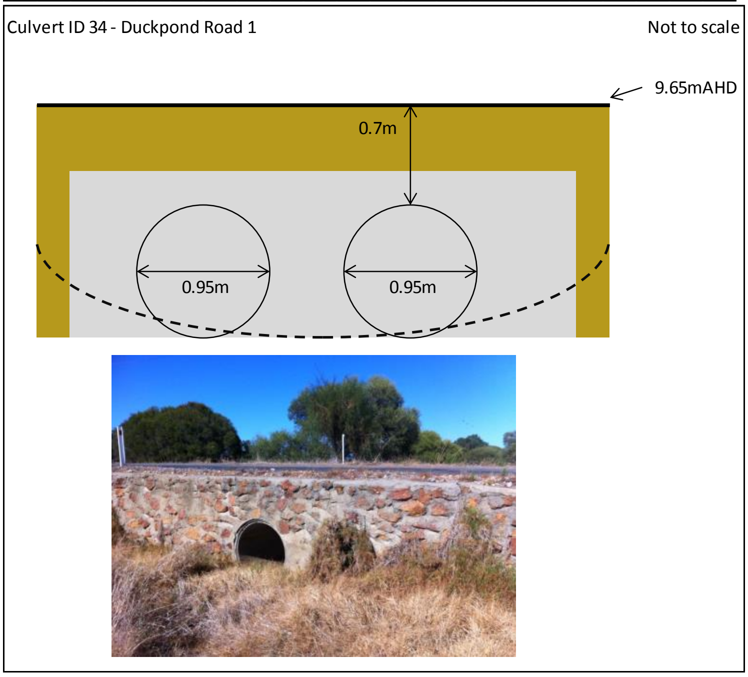






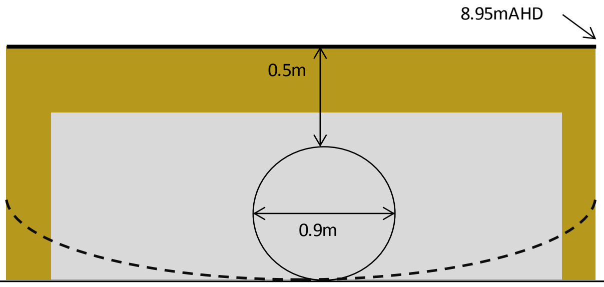
Culverts in the north-east Baldivis area





Culvert ID 77 - Duckpond Road 2 Not to scale

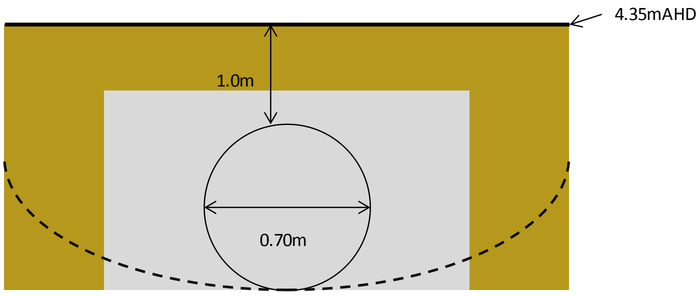
Culvert length: 9.5m



Culverts at Young and Serpentine West Roads

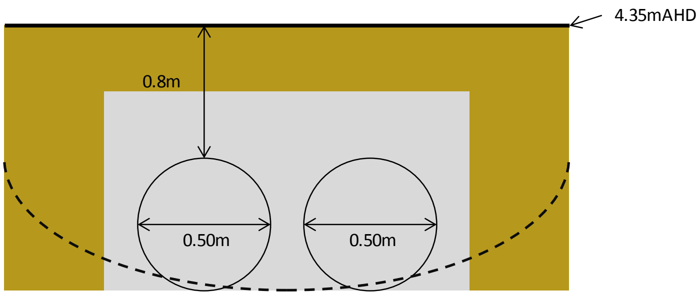
Culvert ID 53 - Serpentine Road West Not to scale

Culvert length: 14m



Culvert ID 60 - Young Road Not to scale

Culvert length: 7.4m



Appendix D – Maximum flood extent and depth for 5, 10, 20, 100 and 500 yr events

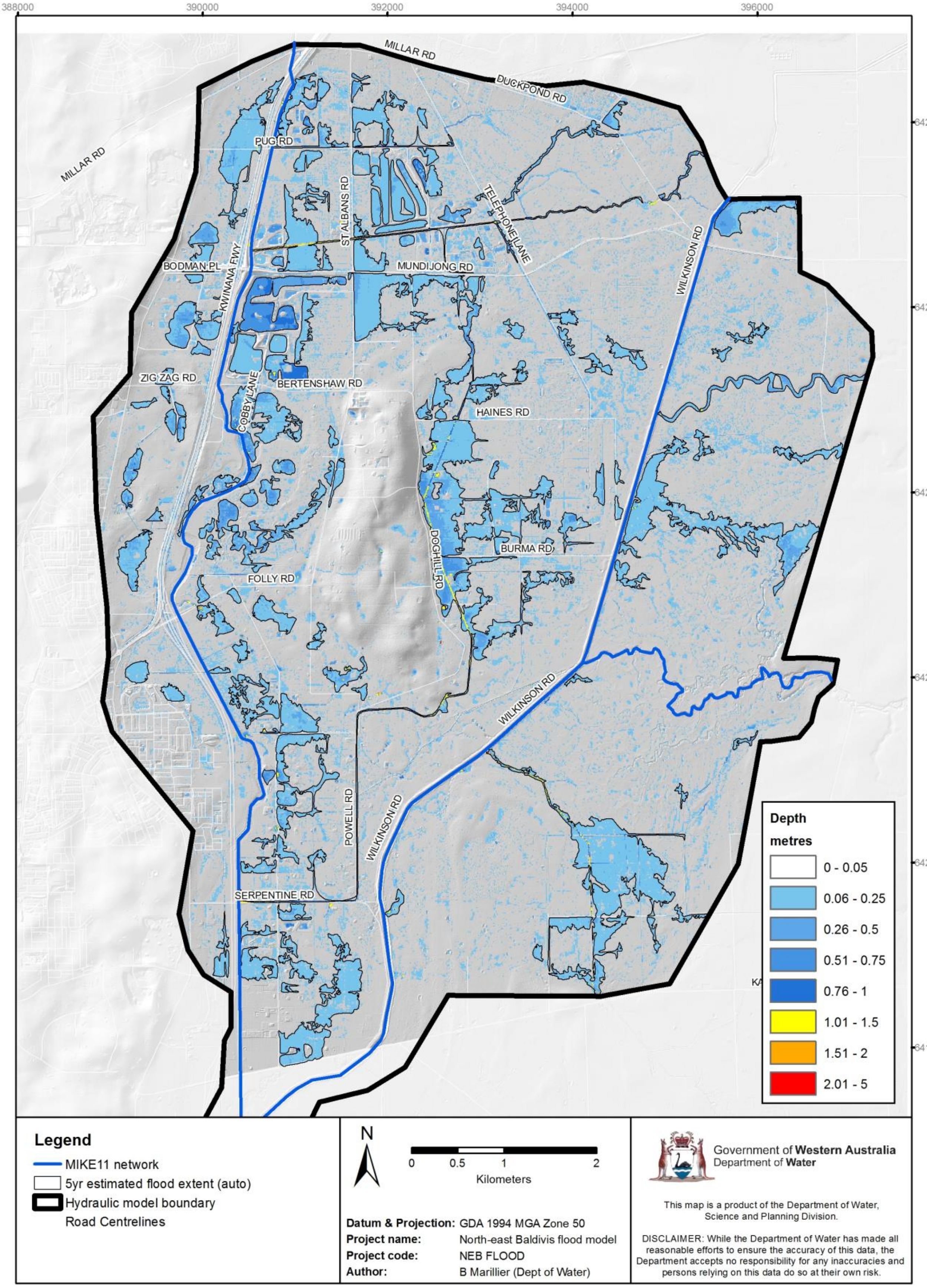


Figure D-1: 5 yr maximum flood levels, refer to long-sections for levels in drains

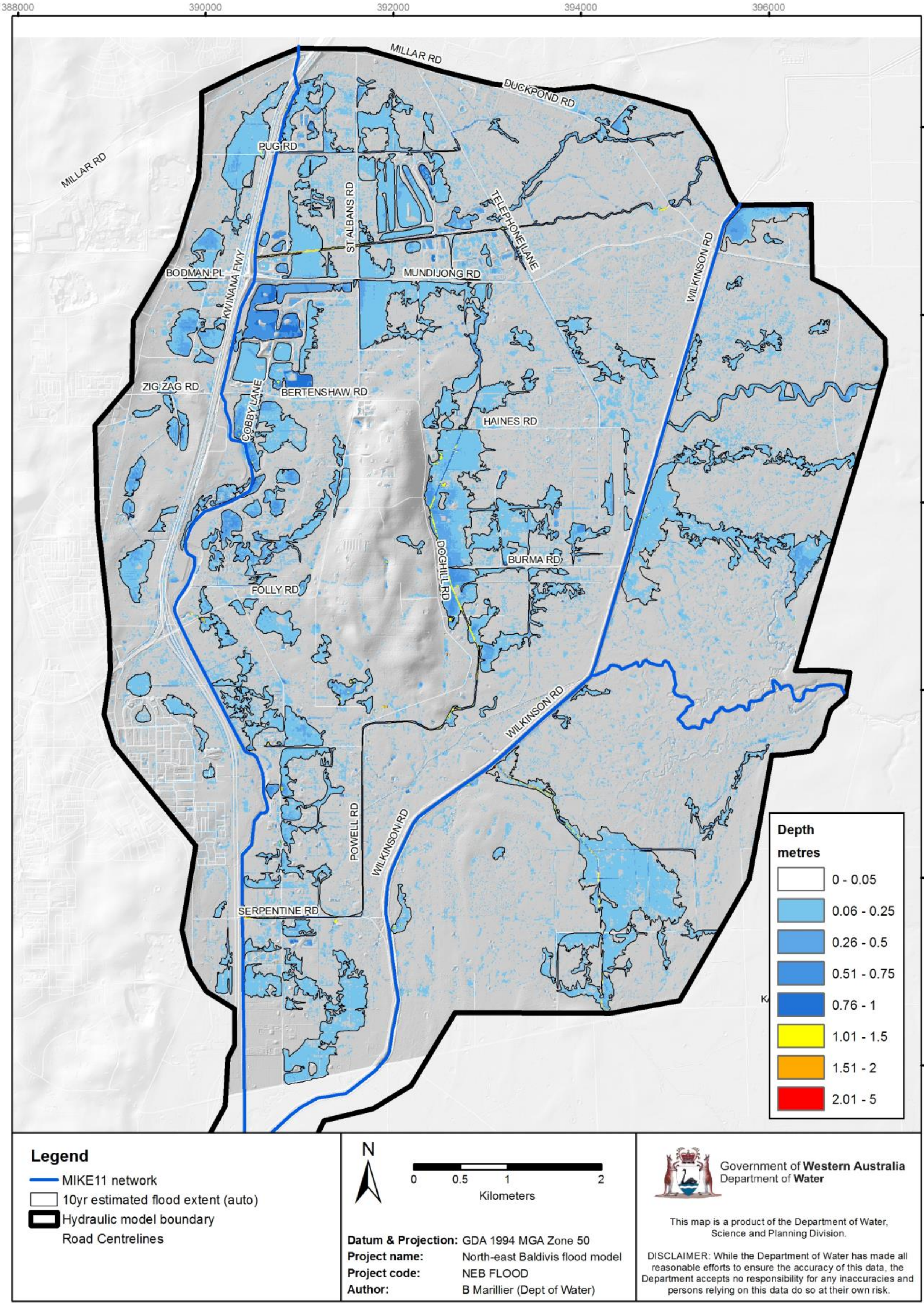


Figure D-2: 10 yr maximum flood levels, refer to long-sections for levels in drains

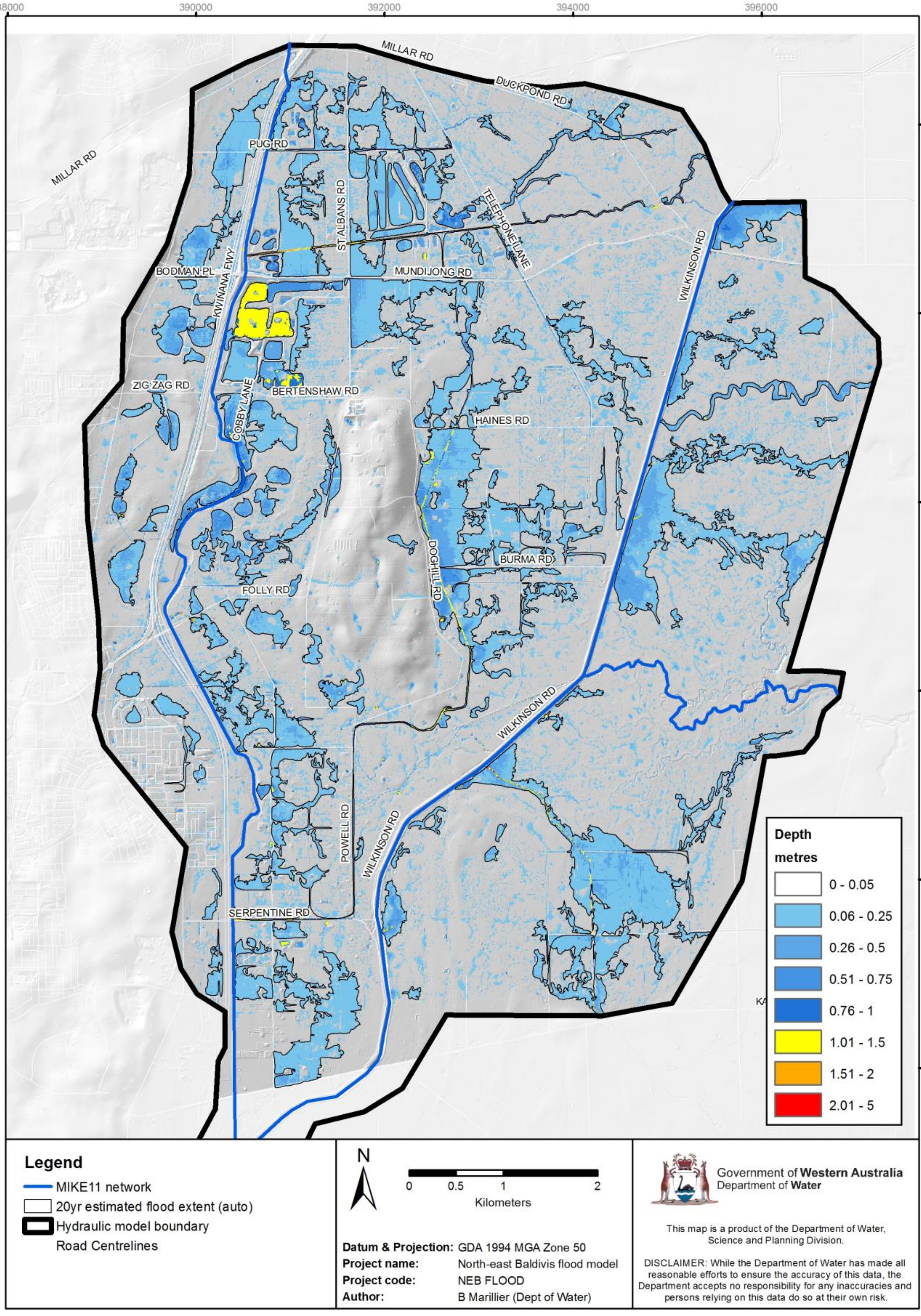


Figure D-3: 20 yr maximum flood levels, refer to long-sections for levels in drains

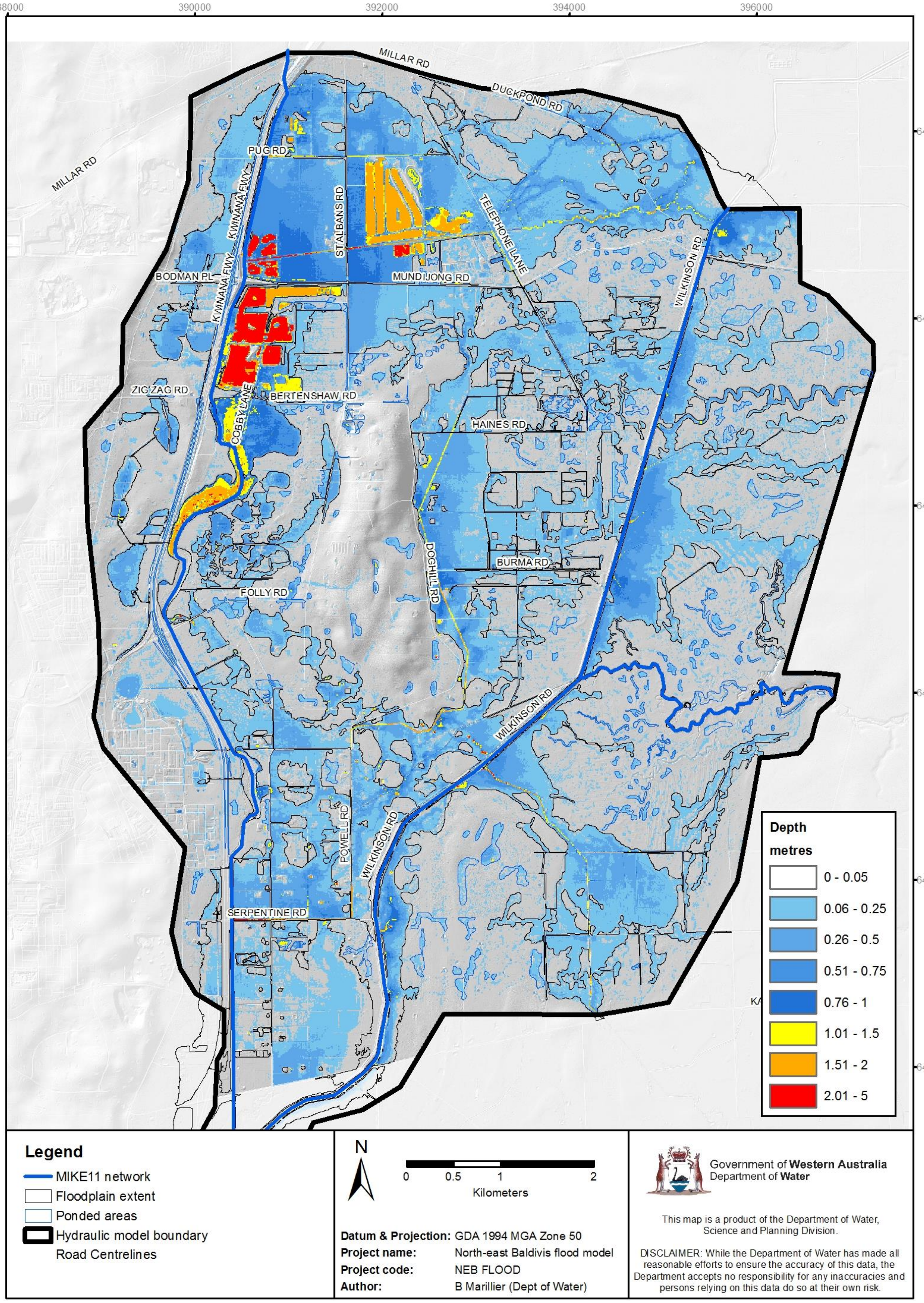


Figure D-4: 100 yr maximum flood levels, refer to long-sections for levels in drains

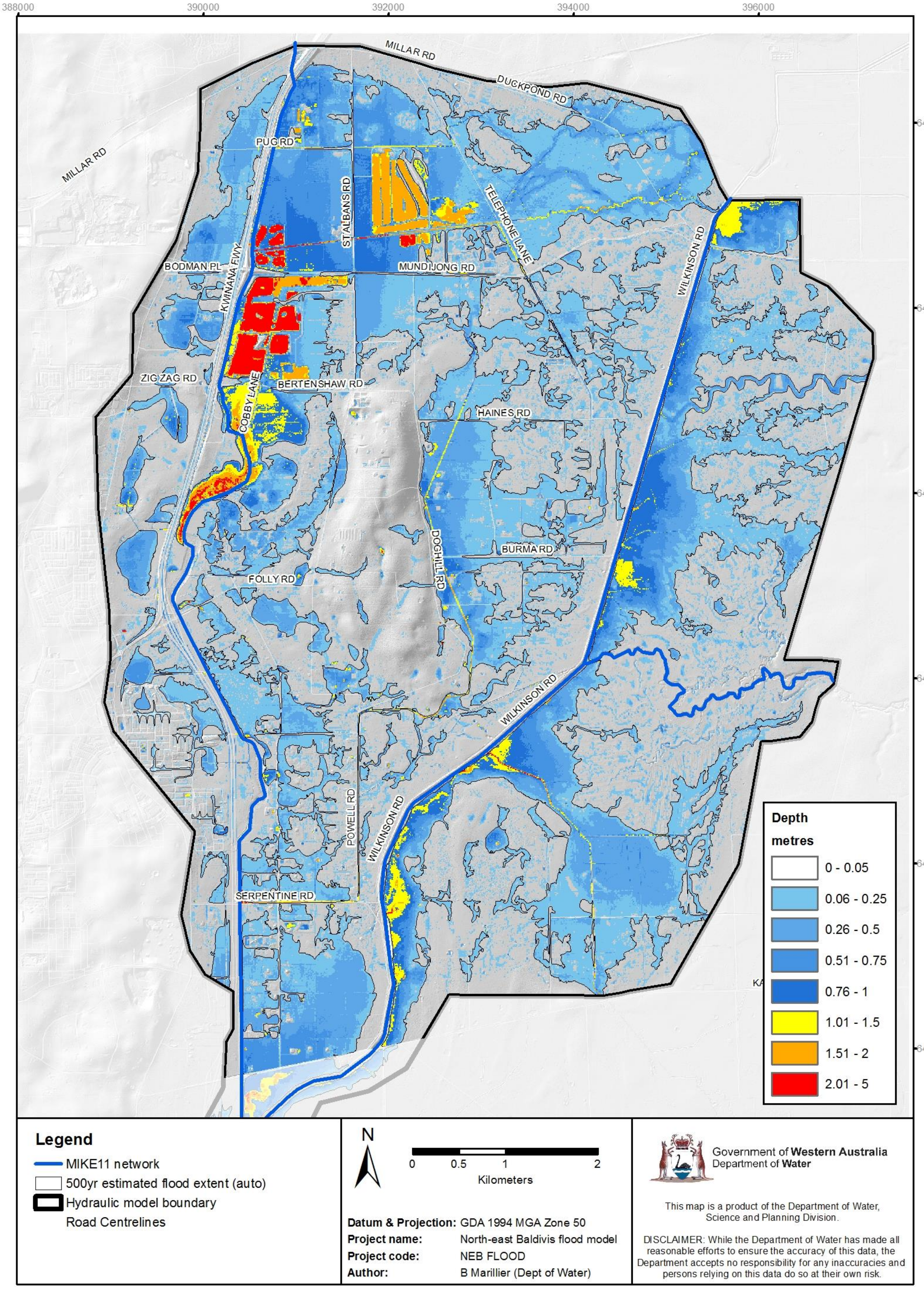
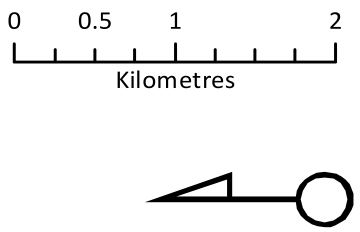
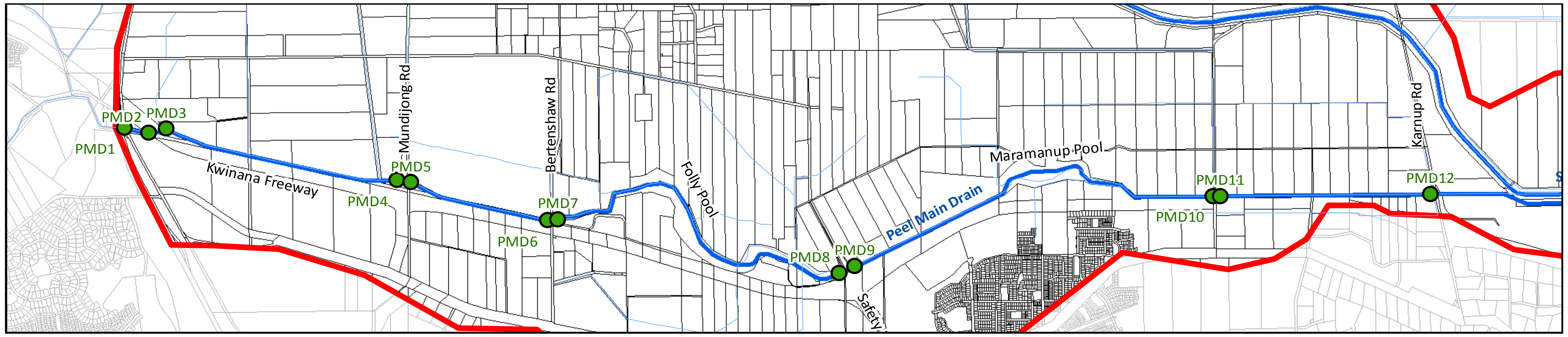


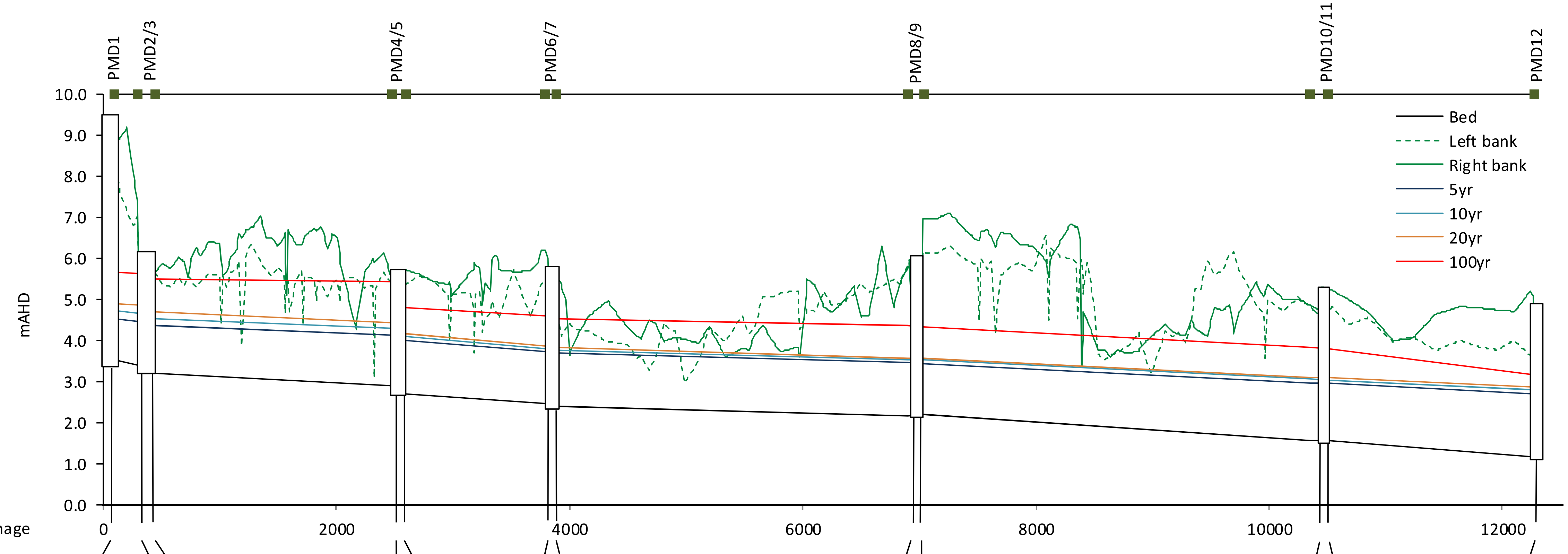
Figure D-5: 500 yr maximum flood levels

Appendix E – Long-sections

Vertical Datum: AHD
Horizontal Datum: GDA 94
Projection System: MGA Zone 50







Size (mm)												
Chainage (m)	100	300	450	2480	2600	3800	3890	6900	7040	10350	10500	12280
Left bank level (mAHD)	8.54	7.02	5.63	5.37	5.36	5.52	4.80	5.83	6.15	4.81	4.65	3.41
Right bank level (mAHD)	9.14	7.40	5.67	5.45	5.71	6.19	5.56	5.77	6.97	4.86	5.25	3.12
5 year ARI HGL (mAHD)	4.52	4.47	4.37	4.14	3.98	3.73	3.71	3.46	3.44	2.98	2.96	2.70
10 year ARI HGL (mAHD)	4.72	4.68	4.54	4.29	4.09	3.81	3.78	3.53	3.51	3.06	3.03	2.78
20 year ARI HGL (mAHD)	4.90	4.86	4.69	4.43	4.18	3.87	3.84	3.58	3.55	3.11	3.09	2.87
100 year ARI HGL (mAHD)	5.65	5.64	5.49	5.44	4.79	4.59	4.54	4.37	4.32	3.84	3.79	3.17
Invert level (mAHD)	3.53	3.40	3.20	2.88	2.70	2.48	2.40	2.15	2.20	1.55	1.55	1.15
5 year ARI flow (m^3/s)	2.7	2.7	2.7	4.7	4.7	4.3	4.3	3.5	3.5	5.0	5.9	5.9
10 year ARI flow (m^3/s)	3.7	3.7	3.7	6.1	6.1	4.9	4.9	4.2	4.2	5.4	6.6	6.6
20 year ARI flow (m^3/s)	4.7	4.7	4.7	7.4	7.4	5.5	5.5	4.4	4.4	5.5	6.9	6.7
100 year ARI flow (m^3/s)	7.5	7.5	7.5	12.9	14.7	14.3	14.3	13.3	13.3	14.3	20.0	20.0

Peel Main Drain - Telephone Lane to Karnup Road

LONGITUDINAL SECTION

Pre-development HGLs & flows

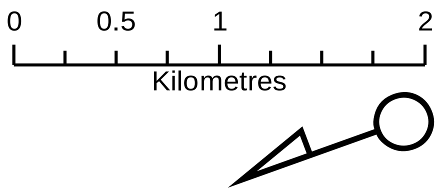
Figure E-1

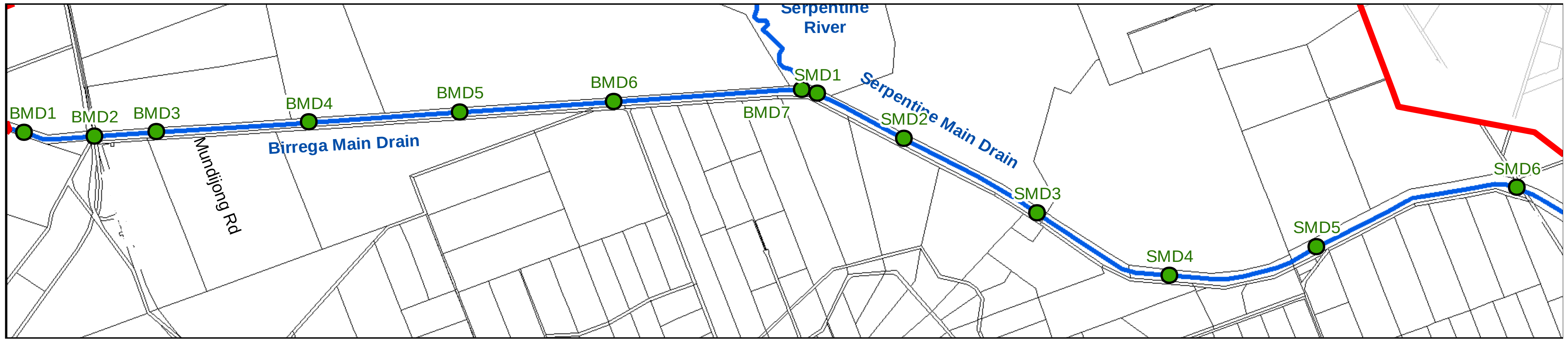
Author: B. Marillier
Date: 24/2/2014

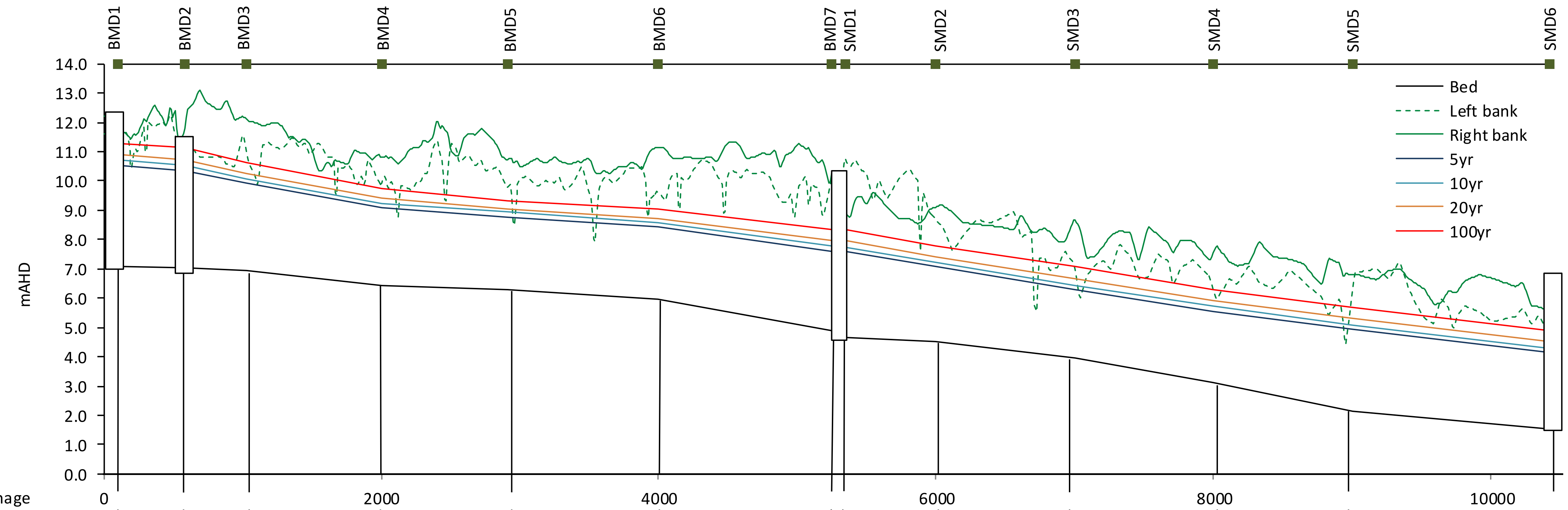
While the Department of Water has made all reasonable efforts to ensure the accuracy of this data, it accepts no responsibility for any inaccuracies and persons relying on this data do so at their own risk.

The Department of Water acknowledges the following datasets and their custodians in the analysis of data and production of the maps: Water Corporation Pipes, Water Corporation, 2009; Cadastre, Landgate, 2013.

Vertical Datum: AHD
Horizontal Datum: GDA 94
Projection System: MGA Zone 50







Size (mm)													
Chainage (m)	100	590	1030	2000	2910	4000	5250	5350	6000	7000	8000	9000	10420
Left bank level (mAHD)	11.78	9.93	11.00	9.92	9.77	9.63	10.10	10.77	8.69	7.00	6.31	6.30	6.20
Right bank level (mAHD)	11.73	11.80	12.11	10.81	10.73	11.16	10.27	8.97	9.11	8.67	7.54	6.81	5.96
5 year ARI HGL (mAHD)	10.56	10.37	9.92	9.09	8.79	8.43	7.62	7.61	7.09	6.31	5.57	4.93	4.16
10 year ARI HGL (mAHD)	10.73	10.54	10.08	9.25	8.93	8.57	7.77	7.76	7.23	6.45	5.72	5.09	4.31
20 year ARI HGL (mAHD)	10.89	10.71	10.24	9.39	9.06	8.72	7.96	7.95	7.42	6.65	5.92	5.29	4.50
100 year ARI HGL (mAHD)	11.30	11.13	10.63	9.73	9.33	9.04	8.36	8.35	7.79	7.07	6.28	5.68	4.90
Invert level (mAHD)	7.09	7.04	6.92	6.42	6.29	5.97	4.90	4.68	4.52	3.96	3.11	2.13	1.56
5 year ARI flow (m^3/s)	41.6	48.8	48.9	49.7	49.8	50.0	49.8	81.6	81.6	81.4	81.4	81.2	81.1
10 year ARI flow (m^3/s)	41.4	48.8	48.8	55.9	56.8	56.0	56.2	92.9	92.9	92.7	92.7	92.3	92.2
20 year ARI flow (m^3/s)	47.9	62.4	62.5	62.8	64.2	60.3	62.8	108.7	108.6	108.5	109.4	108.6	108.5
100 year ARI flow (m^3/s)	54.0	81.4	81.9	84.3	86.2	69.1	81.8	149.5	144.9	137.3	149.7	139.9	147.3

Birrega & Serpentine Main Drains - Mundijong Rd to Karnup Rd

LONGITUDINAL SECTION

Pre-development HGLs & flows

Figure E-2

Author: B. Marillier
Date: 24/2/2014

While the Department of Water has made all reasonable efforts to ensure the accuracy of this data, it accepts no responsibility for any inaccuracies and persons relying on this data do so at their own risk.
The Department of Water acknowledges the following datasets and their custodians in the analysis of data and production of the maps: Water Corporation Pipes, Water Corporation, 2009; Cadastre, Landgate, 2013.

Appendix F – Discharge hydrographs

Discharge reporting locations

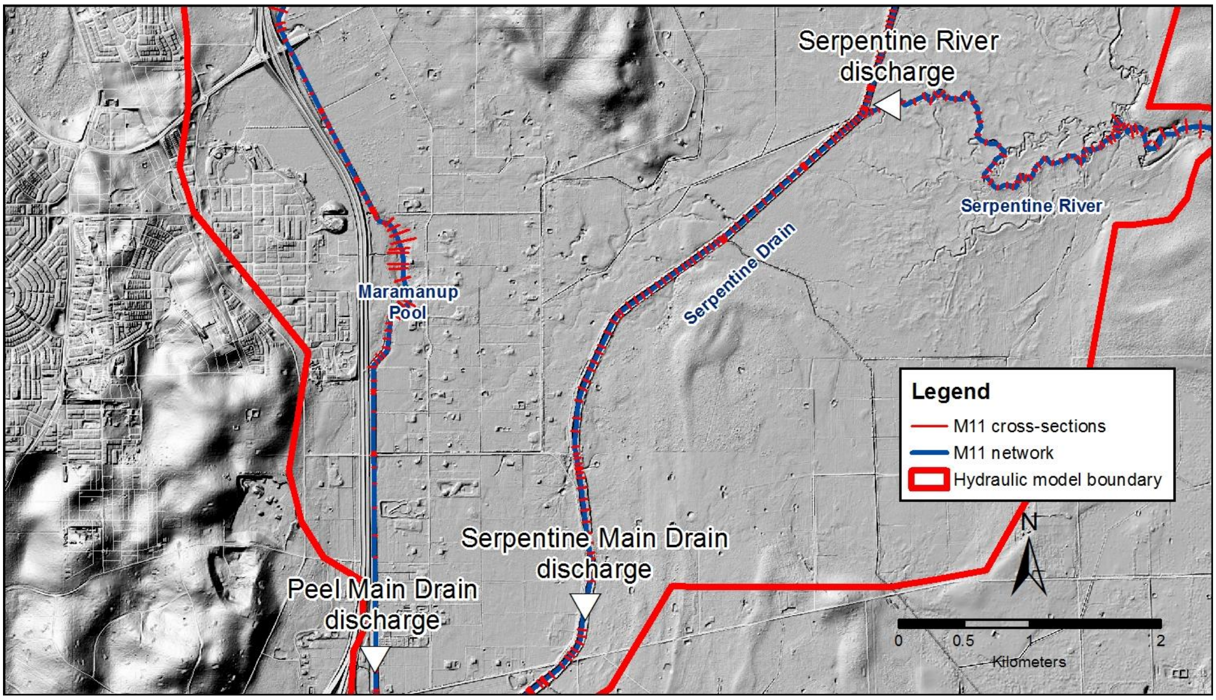
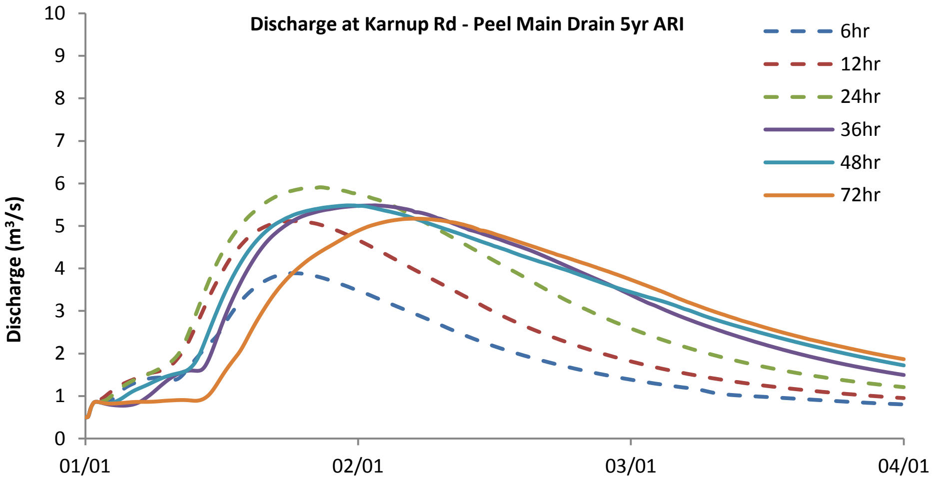
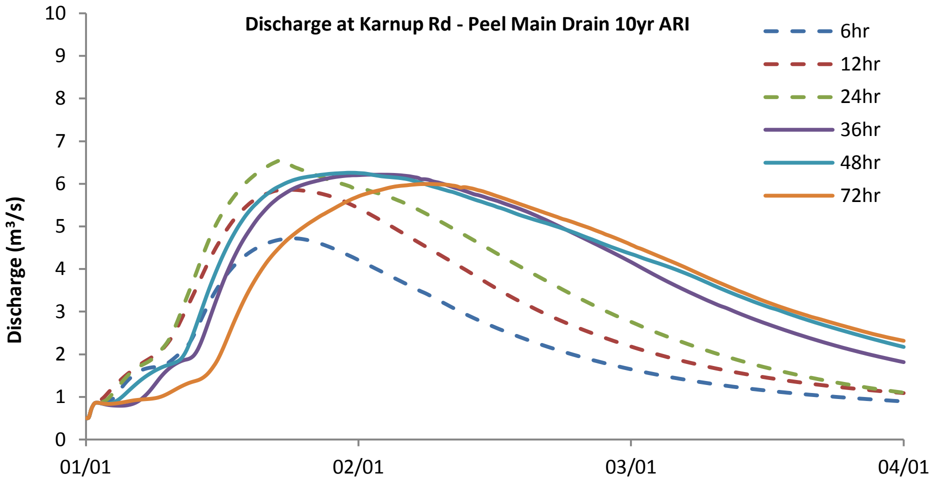
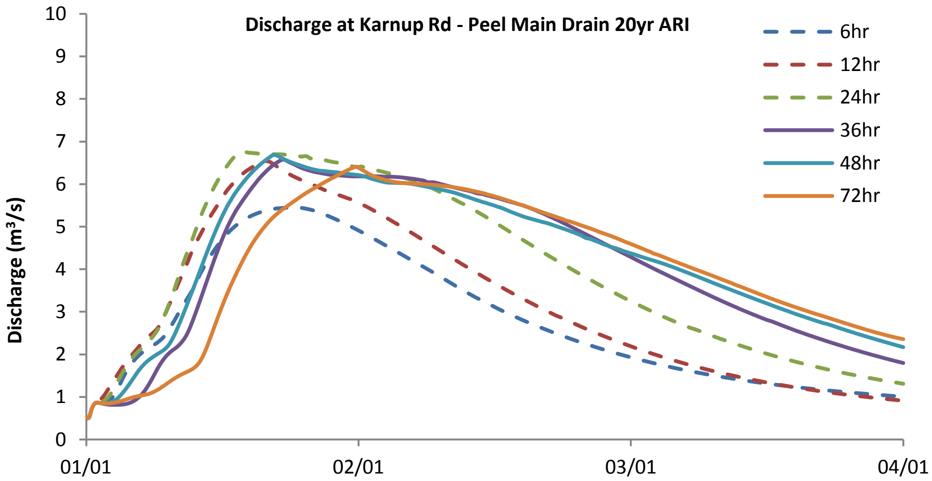


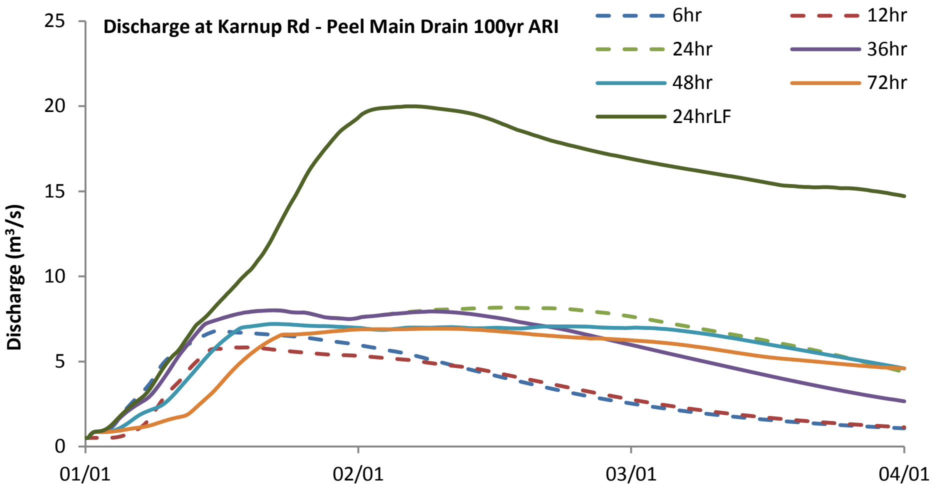
Figure F-1: Locations of discharge hydrograph extractions

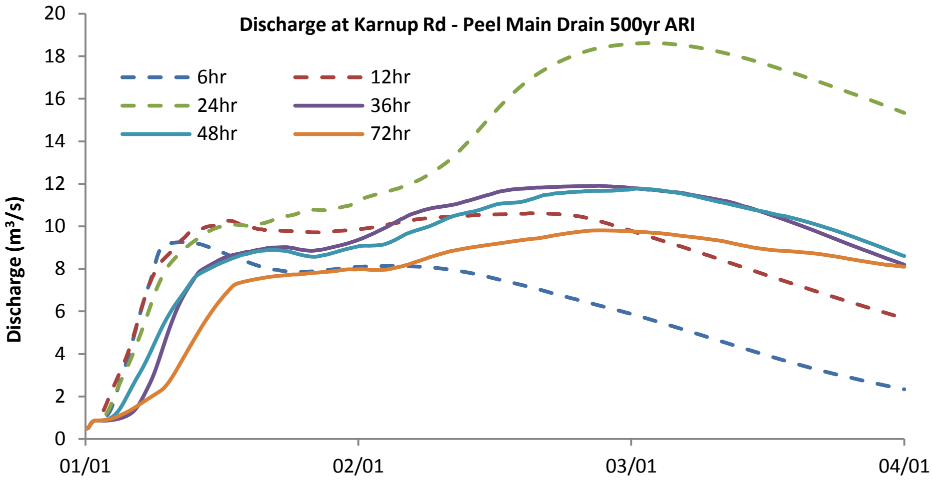
Peel Main Drain



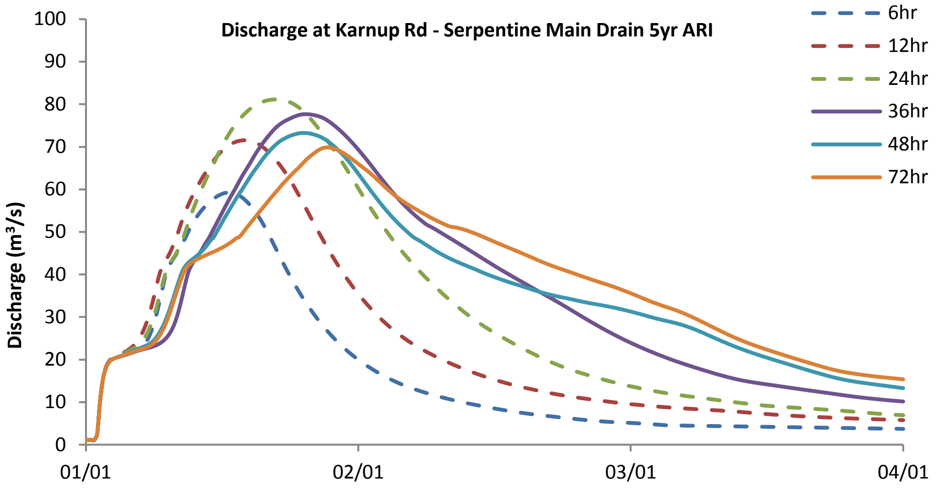


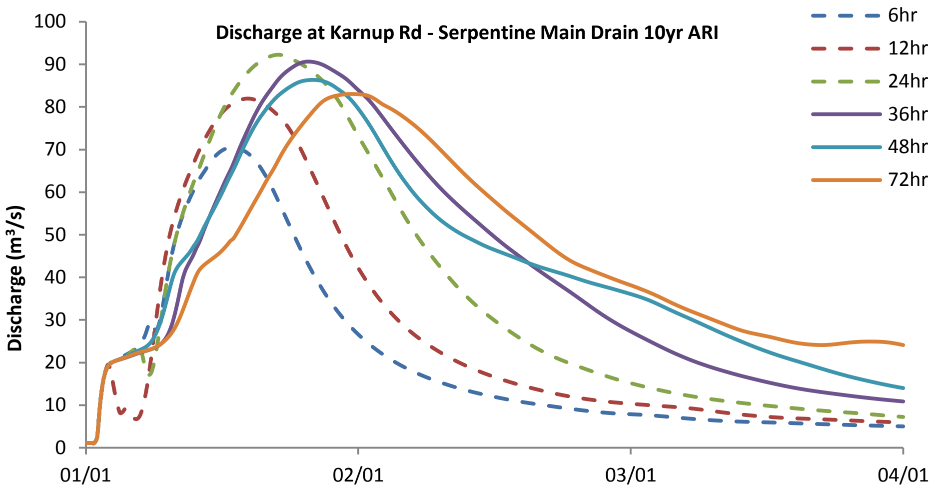


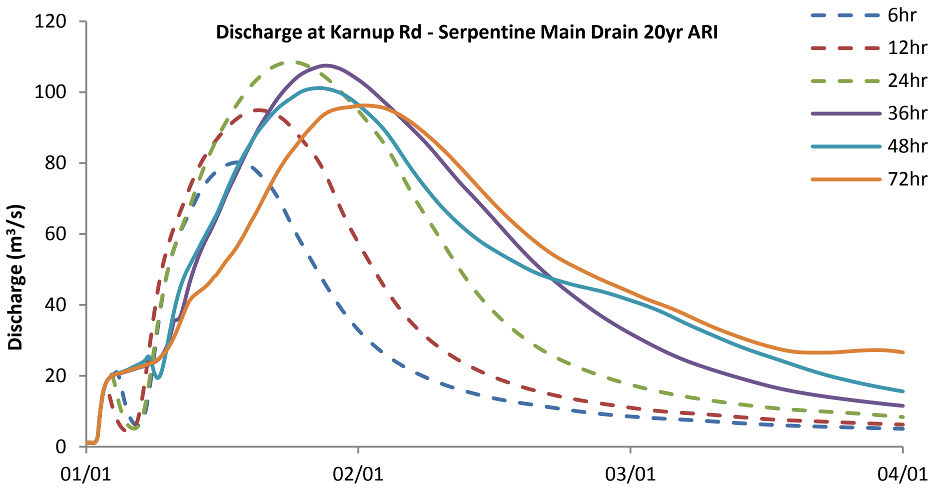

Discharge at Karnup Rd - Peel Main Drain 100yr ARI
6hr
12hr
24hr
36hr
48hr
72hr
24hrLF
Discharge (m³/s)
0
5
10
15
20
25
01/01
02/01
03/01
04/01

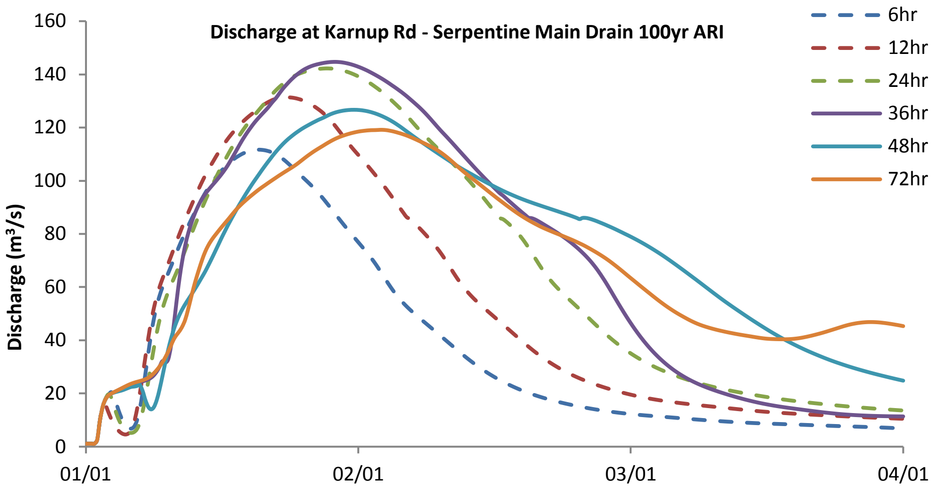

Discharge at Karnup Rd - Peel Main Drain 500yr ARI
6hr
12hr
24hr
36hr
48hr
72hr
Discharge (m³/s)
0
2
4
6
8
10
12
14
16
18
20
01/01
02/01
03/01
04/01

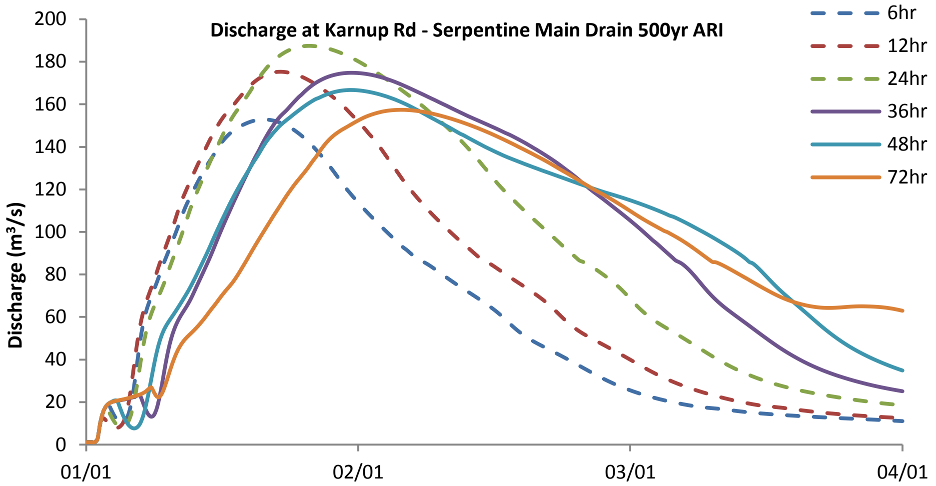
Serpentine Main Drain



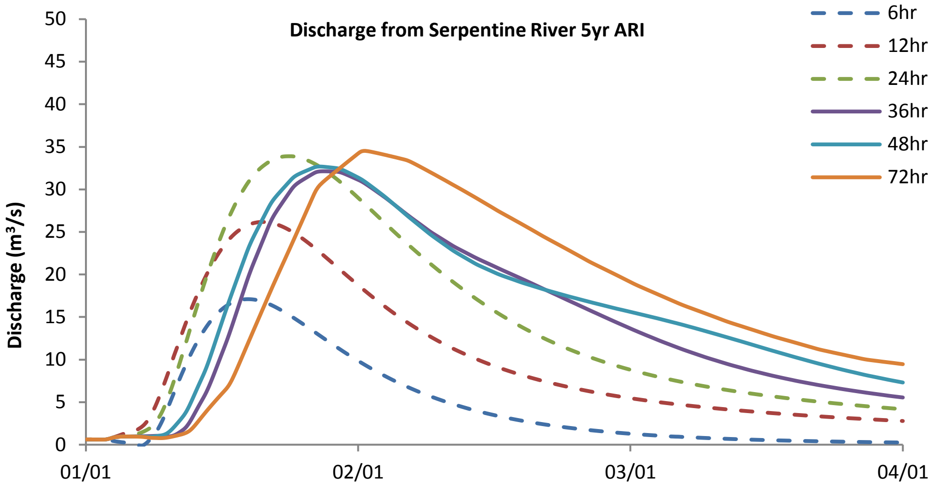


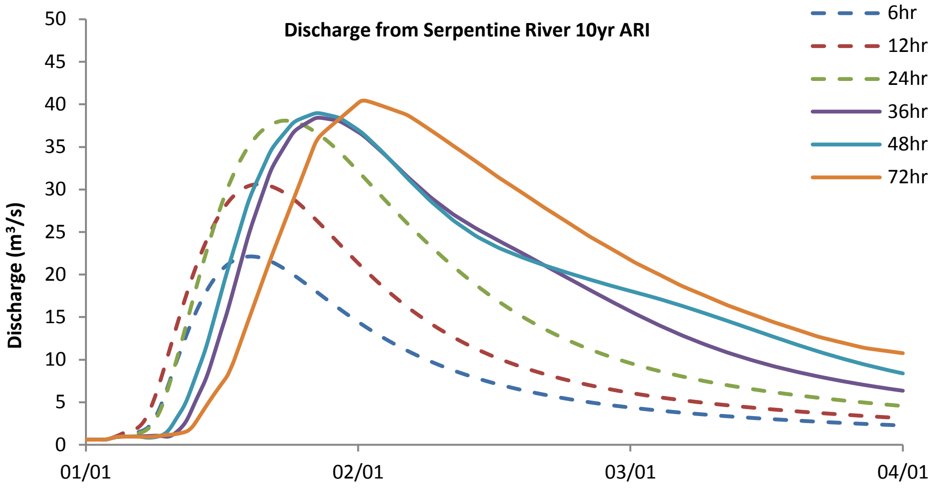


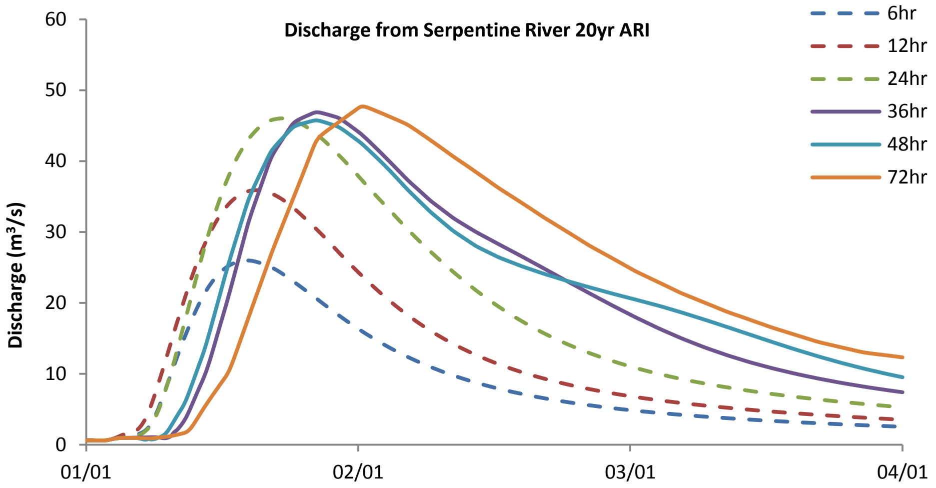

Discharge at Karnup Rd - Serpentine Main Drain 100yr ARI
Discharge (m³/s)
0
20
40
60
80
100
120
140
160
01/01
02/01
03/01
04/01
6hr
12hr
24hr
36hr
48hr
72hr

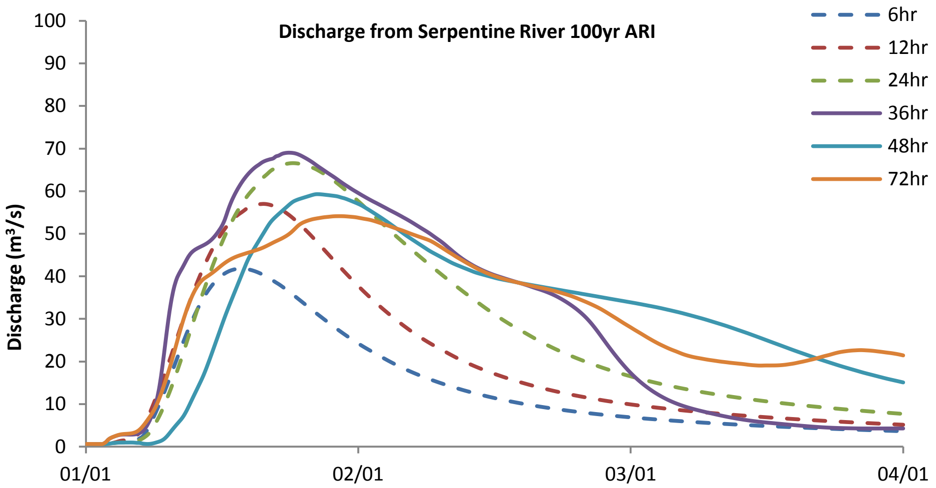

Discharge at Karnup Rd - Serpentine Main Drain 500yr ARI
Discharge (m³/s)
0
20
40
60
80
100
120
140
160
180
200
01/01
02/01
03/01
04/01
6hr
12hr
24hr
36hr
48hr
72hr

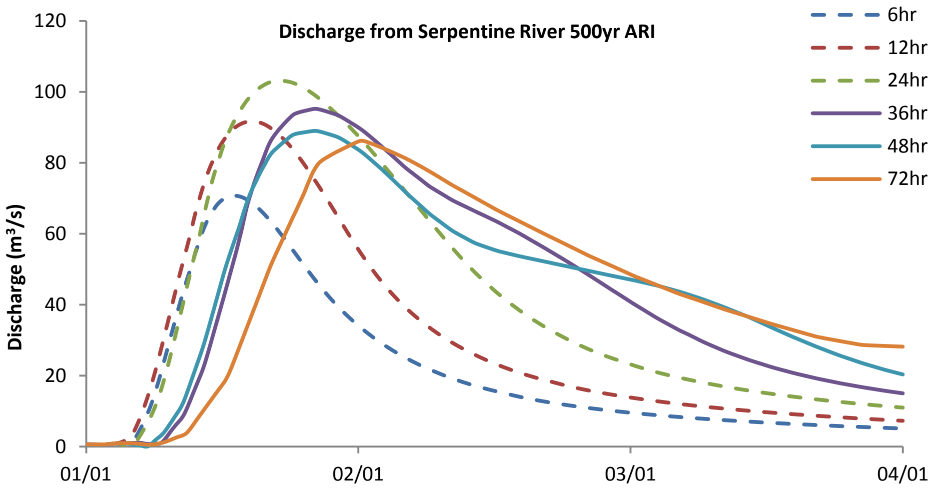
Serpentine River








Discharge from Serpentine River 100yr ARI
Discharge (m³/s)
100
90
80
70
60
50
40
30
20
10
0
01/01
02/01
03/01
04/01
6hr
12hr
24hr
36hr
48hr
72hr


Discharge from Serpentine River 500yr ARI
Discharge (m³/s)
120
100
80
60
40
20
0
01/01
02/01
03/01
04/01
6hr
12hr
24hr
36hr
48hr
72hr

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Bureau of Meteorology (BoM) 2011, Bureau of Meteorology, ACT <http://www.bom.gov.au/>

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Department of Water
168 St Georges Terrace, Perth, Western Australia
PO Box K822 Perth Western Australia 6842
Phone: (08) 6364 7600
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