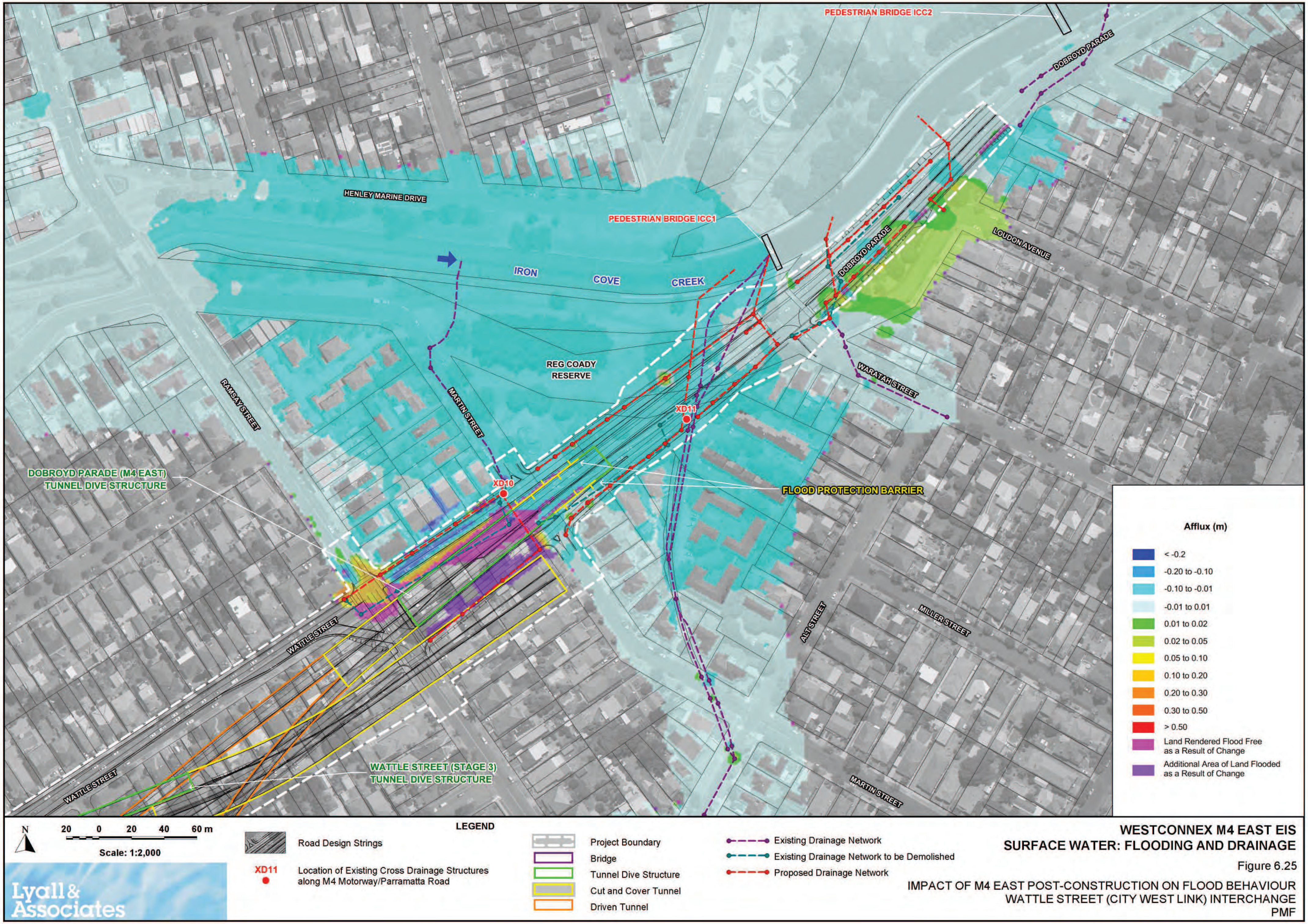



WESTCONNEX M4 EAST EIS
SURFACE WATER: FLOODING AND DRAINAGE

Figure 6.25

IMPACT OF M4 EAST POST-CONSTRUCTION ON FLOOD BEHAVIOUR
WATTLE STREET (CITY WEST LINK) INTERCHANGE
PMF

# APPENDIX A

Overview of previous studies

# Appendix A Overview of previous studies

Previous studies that have been reviewed as part of the flood assessment for the EIS are outlined below in chronological order.

The "*Powells Creek and Saleyards Creek Flood Study*" (Webb, McKeown and Associates (WMA), 1998) was requested from Strathfield Council, but was not available at the date of this report for use in the project.

### Lower Parramatta River Flood Study (PWD, 1986)

NSW Public Works Department (PWD), 1986 provided Parramatta, Auburn, Ryde and Concord councils with information on the flood hazard along the lower reaches of Parramatta River and its tributaries. The study covered the Parramatta River from Charles Street Weir to Ryde Bridge and the lower reaches of Duck River, Haslams Creek and Powells Creek to Mona Street, M4 and Pomeroy Street respectively.

Hydrologic modelling was carried out using the Regional Stormwater Model (RSWM) to determine discharge hydrographs from design storms. Design rainfall data was based on the 1977 version of Australian Rainfall and Runoff. The study predated the release of the current version of AR&R (IEAust, 1998). Hydraulic modelling was undertaken using a combination of the USTFLO one-dimensional unsteady state software for the Lower Parramatta River, and the FLOWBD one-dimensional steady state software for Duck River, Haslams Creek and Powells Creek.

Survey data collected for PWD, 1986 along the lower reach of Powells Creek has been used in the present investigation to define creek bed levels within the tidal zone.

### Fort Denison Sea Level Rise Vulnerability Study (DECC, 2008)

DECC, 2008 was carried out to assess the impact on Fort Denison of sea level rise projections under future climate changes conditions. The assessment was based on a comparison of current and future design still water and wave run-up levels with the existing level of infrastructure and assets on Fort Denison.

Design still water levels at Fort Denison were derived for recurrence intervals ranging from 0.02 to 200 years based on extreme value analysis using the Gumbel probability distribution function of tide gauge records at Fort Denison over the period 1914 to 2006.

The design still water levels derived in DECC, 2008 have been adopted in the present investigation (refer **Appendix C** for further details).

### Parramatta River Estuary Data Compilation and Review Study (CLT, 2008)

Cardno Lawson Treloar (CLT), 2008 was prepared for Parramatta City Council, DECC (now OEH) and the Sydney Metropolitan Catchment Management Authority as part of the development of an Estuary Management Plan for the Parramatta River Estuary. The study covered the whole of the Parramatta River Estuary, which comprises the waterways, bays, foreshores and adjacent lands of the Parramatta River and its tidal tributary creeks, extending from Charles Street Weir, Parramatta and Clarkes Point, Woolwich in the north, to Yurilbin Point, Birchgrove in the south.

Relevant data comprised information on: catchment characteristics (such as climate and land use); urban stormwater, hydrology and flood behaviour; bathymetry and estuary sediments; hydrodynamics and water quality.

CLT, 2008 included contour mapping of bathymetry obtained from DECC. This mapping has been used in the present investigation to define bed levels in Homebush Bay, Canada Bay and Iron Cove.

### Leichhardt Flood Study (CLT, 2010)

Cardno Lawson Treloar (CLT), 2010 was prepared for Leichhardt Council to define flood behaviour across the Leichhardt local government area. The study included the catchments of Whites Creek, Johnstons Creek and Hawthorne Canal.

Hydraulic modelling was carried out using the SOBEK two-dimensional modelling approach. Inflows to the hydraulic model were based on a combination of direct rainfall within the study area and an XP-RAFTS hydrologic model developed for catchments external to the study area.

The hydraulic model was calibrated to historical records of flooding that occurred in 1993 and validated against floods that occurred in 1991 and 1998. The 1993, 1991 and 1998 storms were estimated to be approximately 50, 20 and 10 year ARI rainfall events, respectively.

Historical flood records for the 1993 event included two observed flood levels in the Hawthorne Canal catchment, located south of Parramatta Road on George Street and Upward Street. However, CLT, 2010 found that the observed flood levels did not match well with modelled ground levels or flood depths. On this basis these observed flood levels were not considered suitable for use in the present investigation.

The use of a direct rainfall approach in CLT, 2010 (involving the application of rainfall to the surface of the hydraulic model) in lieu of the more traditional rainfall runoff hydrologic modelling of the catchments and the absence of quoted flows means that it was not possible to provide a direct comparison of design flows from the CLT, 2010 study with the present investigation. There are also no design flood levels in CLT, 2010 that could be compared with the results of the present investigation.

### North Strathfield Rail Underpass Concept Design Report DP11.2 – Flood Impact Assessment (SKM, 2012)

Sinclair Knight Merz (SKM), 2012 was prepared for the Transport Construction Authority to assess the flood impacts of the proposed North Strathfield Rail Underpass (NSRU) on flows in Powell Creek. NSRU is located on the Northern Rail Line between the M4 and Pomeroy Street.

A hydraulic model was developed of the Powells Creek floodplain to assess the impact of the proposed works on existing flood behaviour. Inflows were based on hydrology developed for the *Powells Creek and Saleyards Creek Flood Study* ((WMA, 1998). The hydraulic model was developed using the TUFLOW two-dimensional modelling approach and extended from Homebush Bay to Parramatta Road.

No design flow estimates or flood levels are provided in SKM, 2012 for comparison with results of the present investigation. SKM, 2012 does include a long section profile of the peak 100 year ARI flood level along the main arm of Powells Creek. However, the scale of the figure precludes the extraction of flood levels at a level of accuracy sufficient to provide comparison with results from the present investigation.

### Dobroyd Canal Flood Study (WMAwater, 2014)

WMAwater, 2014 was prepared for Sydney Water, Ashfield Council and Burwood Council to *"identify local overland flow as well as mainstream flow and define existing flood liability"* within the Dobroyd Canal catchment (referred to in this report as Iron Cove Creek).

Hydraulic modelling was carried out using the TUFLOW two-dimensional modelling approach. Inflows to the hydraulic model were based on a DRAINS hydrologic model developed for define the conversion of rainfall to runoff within subcatchments within the study area.

WMAwater, 2014 contains peak flood levels at key locations within the Dobroyd Canal (Iron Cove Creek) catchment that have been used for comparison purposes with the results of the flood assessment for the EIS. The findings of this comparison are presented in **Appendix C**.

# APPENDIX B

Background to hydrologic model development

# Appendix B Background to hydrologic model development

## B1. General

The assessment of runoff characteristics from the catchments which contribute flows to the drainage systems along the project corridor was based on a hydrologic model developed using the DRAINS software.

DRAINS is a simulation program which converts rainfall patterns to stormwater runoff and generates discharge hydrographs. These hydrographs are then routed through networks of piped drainage systems, culverts, storages and open channels to calculate hydraulic grade lines and analyse the magnitude of overflows. Alternatively, discharge hydrographs generated by DRAINS can be used as inflows to alternative hydraulic models (such as the TUFLOW two-dimensional hydraulic modelling software) to calculate water surface levels and flooding patterns. The latter approach is particularly appropriate for modelling complex flood behaviour in urban areas involving multiple flow paths and has therefore been adopted in the present investigation. Refer Appendix C for further discussion on the development of the TUFLOW hydraulic models which have been used to define flood behaviour in the vicinity of the tunnel portals.

The extents of the various catchments that contribute flow to the existing drainage systems crossing the proposed motorway corridor are shown on **Figures 4.2** to **4.4**. The following sections of the report contain a brief description of the adopted modelling approach and present derived peak flows.

## B2. DRAINS model development

### B2.1. General

A number of hydrologic sub-models are available within DRAINS to simulate the conversion of rainfall to runoff. For the purpose of this present investigation, the ILSAX sub-model was selected as it is well suited to the urbanised nature of the study area.

**Figures B1** to **B3** show the layout of the various sub-catchments which comprise the DRAINS models developed for the study area. Sub-catchment boundaries were digitised based on available contour information, which comprised ALS and two metre contour data. Sub-catchment slopes used for input to the DRAINS model were derived using the average sub-catchment slope, which were computed using available contour data. Aerial photography and site observations were used to assess the degree of urbanisation which is present in the study catchments.

### B2.2. Design storms

Rainfall intensities for the 100 year ARI event were derived using procedures outlined in Australian Rainfall and Runoff (ARR) (IEAust, 1998) for storm durations ranging between 25 minutes and six hours. Separate design rainfall intensities were generated for each catchment to account for the variability in design rainfall values across the extent of the project corridor. The design rainfall depths were then converted into rainfall hyetographs using the temporal patterns presented in ARR.

No Aerial Reduction Factor (ARF) was applied to the design rainfall intensities obtained from ARR due to the size of the catchments within the study area (the largest of which is Dobroyd Canal (Iron Cove Creek) with an area of 6.4 square kilometres at the proposed motorway corridor).

Estimates of probable maximum precipitation were derived using the Generalised Short Duration Method (GSDM) as described in the BoM's update of Bulletin 53 (BoM, 2003). This method is appropriate for estimating extreme rainfall depths for catchments up to 1000 square kilometres in area and storm durations up to six hours.

## B2.3. Model parameters

Adopted DRAINS model parameters comprised initial losses of one and five millimetres for paved and grassed areas respectively. The soil type was set equal to three, which corresponds with a soil of comparatively high runoff potential. An antecedent moisture condition (AMC) of three was adopted, reflecting rather wet conditions prior to the onset of runoff producing rainfall.

Lagging was adopted to describe the translation of the hydrograph generated at each sub-catchment outlet along the various links to the next downstream sub-catchment. This approach required specifying a velocity of the flow along the link. The sensitivity of the results to assumed velocities ranging between one and three metres per second was tested for the 100 year ARI critical storm. After consideration a velocity of two metres per second was adopted for design.

In the absence of gauged streamflow data that could otherwise be used to calibrate the DRAINS model, peak 100 year ARI flows arriving at the project road corridor were compared to peak flow estimates derived using the Rational Method for urban catchments presented in ARR.

# B3. Peak flow estimates for present day conditions

**Table B1** over gives peak flow rates generated by DRAINS for each of the catchments that contribute runoff to existing cross drainage structures along the route of the project. Peak flow estimates derived by the Rational Method (RM) for the 100 year ARI event are also given for comparative purposes.

Peak 100 year ARI flows derived by DRAINS compared closely with those derived using the RM approach for all of the modelled catchments with the exception of the main arm of Dobroyd Canal (Iron Cove Creek) (i.e. the catchment contributing runoff to XD08). The reason for the higher flow in DRAINS is attributed to the shape of the catchment, where a large number of lateral branches in the drainage system combine a short distance upstream of the cross drainage structure. (The Rational Method does not account for the layout of the stormwater drainage system.)

The peak flows derived for the PMF are generally between four to five times greater than those for the corresponding 100 year ARI event, a finding which is consistent with those of similar flooding investigations undertaken in highly urbanised catchments.

**Table B1 Peak flows at locations of existing cross drainage along project corridor**

| Catchment | I.D. | Catchment Area (hectre) | Peak Flows (cubic metres per second)(1,2) 100 year ARI RM | DRAINS | PMF |
|---|---|---|---|---|---|
| Powells Creek | XD01a | 261 | 66.5 | 68.8[60] | 362[30] |
| | XD01 | 3.8 | 2.1 | 1.8[25] | 8.4[15] |
| | XD02 | 112 | 30.3 | 30.6[25] | 154[30] |
| | XD03 | 300 | 71.4 | 74.9[60] | 379[45] |
| Exile Bay | XD04 | 3.7 | 2.1 | 2.3[25] | 9.1[15] |
| | XD05 | 21.3 | 8.8 | 7.7[25] | 40.0[15] |
| St Lukes Park Canal | XD06 | 121 | 38.8 | 46.3[20] | 224[15] |
| Barnwell Park Catchment | XD07 | 36.4 | 15.6 | 12.7[20] | 67.4[15] |
| Dobroyd Canal (Iron Cove Creek) | XD08 | 636 | 135 | 192.1[60] | 773[45] |
| | XD09(3) | 9.4 | 5.9 | 4.0[25] | 17.8[15] |
| | XD10 | 7.8 | 4.9 | 3.4[25] | 14.9[15] |
| | XD11 | 49.0 | 18.3 | 18.4[25] | 80.4[15] |
| Hawthorne Canal | XD12 | 9.7 | 6.3 | 5.6[25] | 23.2[15] |
| | XD13 | 17.9 | 9.4 | 7.5[25] | 32.9[15] |
| | XD14 | 295 | 87.4 | 98.1[60] | 383[30] |
| | XD15 | 59.7 | 27.7 | 26.2[25] | 115[15] |

(1) Peak flows represent local catchment flows only and do not include bypass flows from nearby cross drainage systems.

(2) Values in [ ] represent critical storm duration in minutes.

(3) XD09 incorporates the catchments of the three cross drainage structures XD09a, XD09b and XD09c shown on **Figure 4.12**.


N

200 0 200 400 600 m

Scale: 1:20,000

Lyall & Associates

**LEGEND**

Sub-Catchment Boundary - Powells Creek

Sub-Catchment Boundary - Concord Road

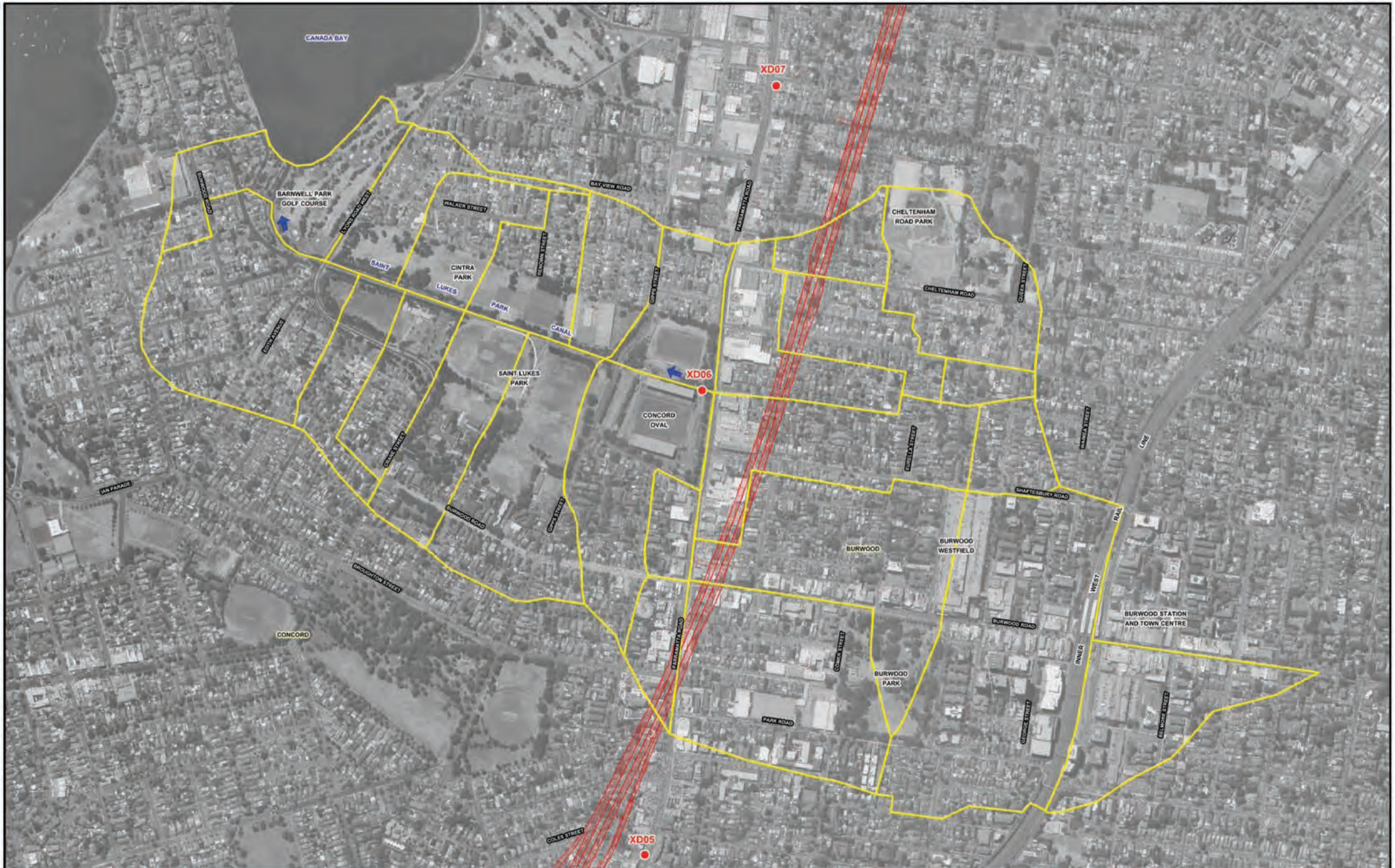
Road Design Strings

XD01 Location of Existing Cross Drainage Structures along M4 Motorway/Parramatta Road

**WESTCONNEX M4 EAST EIS**
**SURFACE WATER: FLOODING AND DRAINAGE**

Figure B1

POWELLS CREEK AND CONCORD ROAD SUB-CATCHMENT LAYOUT




80 0 80 160 240 m

Scale: 1:8,000

Lyall & Associates

**LEGEND**

Sub-Catchment Boundary

Road Design Strings

XD06 Location of Existing Cross Drainage Structures along M4 Motorway/Parramatta Road

**WESTCONNEX M4 EAST EIS**
**SURFACE WATER: FLOODING AND DRAINAGE**

Figure B2

SAINT LUKES PARK CANAL SUB-CATCHMENT LAYOUT


N

160 0 160 320 480 m

Scale: 1:16,000

Lyall & Associates

**LEGEND**

Sub-Catchment Boundary

Road Design Strings

XD05 Location of Existing Cross Drainage Structures along M4 Motorway/Parramatta Road

**WESTCONNEX M4 EAST EIS**
**SURFACE WATER: FLOODING AND DRAINAGE**

Figure B3

DOBROYD CANAL (IRON COVE CREEK) SUB-CATCHMENT LAYOUT

# APPENDIX C

Background to hydraulic model development

# Appendix C Background to hydraulic model development

## C1. General

Detailed two-dimensional hydraulic modelling was undertaken using the TUFLOW software to define flooding behaviour along the main drainage lines which cross the proposed motorway corridor in the vicinity of the tunnel portals.

Three TUFLOW models covering the catchments of Powells Creek (denoted the ‘Powells Creek TUFLOW Model’), St Lukes Park Canal (‘St Lukes Canal TUFLOW Model’) and Dobroyd Canal (Iron Cove Creek) (‘Iron Cove Creek TUFLOW Model’) have been developed as part of the present investigation.

An additional TUFLOW model was established to undertake a detailed assessment of overland flow behaviour in the vicinity of drainage line XD04 (Exile Bay Catchment) where it crosses the project west of Concord Road (‘Concord Road TUFLOW Model’.

The TUFLOW models were initially developed to define flood behaviour at the M4 Motorway Bridge at Saleyards Creek, Powells Creek off-ramp and the four interchange under present day conditions.'[9] **Chapters 5** and **6** describe how these models have subsequently been used to assess the impacts of the proposed works on flooding and drainage patterns and to evaluate potential mitigation measures.

## C2. The TUFLOW modelling approach

TUFLOW is a two-dimensional hydraulic model which does not rely on a prior knowledge of the pattern of flood flows in order to set up the various fluvial and weir type linkages which describe the passage of a flood wave through the system.

The basic equations of TUFLOW involve all of the terms of the St Venant equations of unsteady flow. Consequently the model is "fully dynamic" and once tuned will provide an accurate representation of the passage of the floodwave through the drainage system in terms of extent, depth, velocity and distribution of flow.

TUFLOW solves the equations of flow at each point of a rectangular grid system which represent overland flow on the floodplain and along streets. The grid system may also be used to describe the waterway area available in the channel system. Channel systems can also be modelled as one-dimensional elements embedded in the larger two-dimensional domain which typically represents the wider floodplain. Flows are able to move between the one and two-dimensional elements of the model depending on the capacity characteristics of the drainage system being modelled.

The approach adopted in the present analysis was to model open channels, culverts and pit and pipe networks as one-dimensional elements embedded in the larger two-dimensional domain representing the floodplain. The choice of grid point spacing depends on the need to accurately represent features on the floodplain which influence hydraulic behaviour and flow patterns (e.g. buildings, streets, changes in floodplain dimensions, hydraulic structures which influence flow patterns, etc.).

[9] *The TUFLOW models have been developed to define flow behaviour along mainstream and major overland flow paths. Protection of tunnels and ancillary facilities against local drainage and minor overland flow would be carried out through the sizing of the drainage infrastructure and design of surface grading, rather than setting minimum design levels based on existing flood behaviour. This is discussed further in **Chapter 6**.*

## C3. Model layout

The layouts of the TUFLOW models are shown on **Figures C1** to **C4**.

An important consideration of two-dimensional modelling in an urbanised area is to ensure adequate representation of the roads, fences, buildings and other features which influence the passage of flow over the natural surface. A grid spacing of two metres was adopted to provide an appropriate level of definition of those features whilst maintaining a reasonable simulation run time.

Grid elevations were based on ALS survey data flown in 2013, with the exception of the Powells Creek TUFLOW Model (refer below for details). Ridge and gully lines were added to the model where the grid spacing was considered too coarse to accurately represent important topographic features which influence the passage of overland flow, such as road centrelines and bridge approaches.

Harbour bed levels in Canada Bay and Iron Cove were defined using bathymetric contour data provided in the *Parramatta River Estuary Data Compilation and Review Study* (CLT, 2008).

Open channels, culverts and pit and pipe networks were defined using GIS based data obtained from SW and the local councils. This information included dimensions of channels, culverts and pipes and locations of pits, headwalls and channel junctions. At the date of this report, Strathfield Council had not provided any data pertaining to its drainage assets.

An assumed cover of 700 millimetres was adopted for those drainage elements where invert levels were not available (which applied to most of the system). This assumed cover was adjusted to ensure that the drainage system had positive fall in the downstream direction.

The footprints of a large number of individual buildings located in the two-dimensional model domain were digitised and either:

- assigned a high hydraulic roughness value which accounted for their blocking effect on flow while maintaining storage in the model; or
- assigned a grid elevation above the level of the PMF event which removed flood storage from the model and also accounted for their blocking effect on flow (this alternative approach was applied to large buildings where visual inspection identified locations where floodwater would preferentially flow around the perimeter of the structure rather than through it).

Bridge crossings over the main arms were typically defined using a combination of ALS survey data (to set bridge deck levels), SW's GIS data (to define the clear opening width) and visual inspection (to estimate the bridge deck thickness). Bridge crossings within the immediate vicinity of the project corridor, which are described in the summary below, were measured during a field inspection.

Model features and assumptions specific to each of the four models are summarised below.

**Powells Creek TUFLOW Model (Figure C1):**

- Dimensions of the Powells Creek (XD03) and Saleyards Creek (XD01a) channels were defined using GIS based data obtained from SW. These data were also used to define SW owned channel and pipe networks within the hydraulic model extent, including the two piped tributaries that cross the proposed motorway corridor (XD01 and XD02). Council pit and pipe data are not yet available in the Strathfield LGA.
- Invert levels in the main arm downstream of Pomeroy Street were defined using the long section profile of Powells Creek provided in the *Lower Parramatta River Flood Study* (PWD, 1986). Invert levels upstream of Pomeroy Street were based on ALS data.
- Bridge crossings over the Powells Creek (XD03) channel at Conway Avenue, Pomeroy Street, Allen Street and Parramatta Road were defined using a combination of ALS survey data (to set bridge deck levels) and measurements taken during a field inspection (to measure the thickness of the bridge deck and depth of the channel below bridge deck level).
- Bridge and culvert arrangements over the Saleyards Creek (XD01a) channel at Underwood Road, the M4, Parramatta Road, Western Rail Line and The Crescent were also defined based on the ALS survey data and field measurements described above.

- A separate model was established for the PMF to better represent flow behaviour in this extreme event. Local inflow boundaries that were causing surcharging in the channel upstream of the Western Rail Line were redistributed over a wider area. Hydraulic loss coefficients were adjusted for the bridge across Saleyards Creek at the M4 to reflect the change in flow conditions in the waterway from “unsubmerged” (in the 100 year ARI flood) to “submerged” (in the PMF).
- The reach of Saleyards Creek that runs under the Sydney Markets buildings between Parramatta Road and the Western Rail Line could not be accessed during the field inspection. The overbank either side of the main arm was therefore defined based on a uniform section. The dimensions of the overbank sections were based on the profile immediately upstream of Parramatta Road. It was also assumed that flow would be unrestricted by the buildings that lie over the channel.

**Concord Road TUFLOW Model (Figure C2):**

- Drainage pits and pipes were defined based on GIS based data obtained from City of Canada Bay Council.

**St Lukes Park Canal TUFLOW Model (Figure C3):**

- Dimensions of the main arm (XD06) were defined using GIS based data obtained from SW. Channel invert levels were defined based on ALS survey data. No ALS survey data were available within the channel in the lower reach of the canal below Gipps Street due to tidal inundation at the time the level data was captured. As a result, invert levels were defined based on the top of bank level (defined by the ALS) less the depth of channel provided in the SW data.

**Iron Cove Creek TUFLOW Model (Figure C4):**

- Grid elevations were defined using a combination of ALS survey data and detailed field survey in the vicinity of Dobroyd Parade.
- Dimensions of the main arm at XD08 and the two piped tributaries that cross the project corridor (XD10 and XD11) were defined using GIS based data obtained from SW.
- Invert levels in the main arm were defined using ALS survey data. These levels were then checked and adjusted between Timbrell Drive and Waratah Street based on detailed ground survey collected by WDA (to define bridge deck levels) and measurements taken during a field inspection (to measure the depth of the channel below bridge deck level).
- The bridge arrangements over the main arm at Timbrell Drive and the two pedestrian bridges opposite Crane Avenue and Waratah Street were also defined based on the detailed survey and the field measurements described above.
- Drainage pits and pipes were defined based on a combination of GIS based data obtained from Ashfield Council and SW, supplemented with detailed field survey along Dobroyd Parade.
- A separate model was established for the PMF event to remove inlet pits at Waratah Street that were causing upwelling.

# C4. Model boundary conditions

## C4.1. Upstream boundaries

Discharge hydrographs generated by DRAINS were applied at the inflow boundaries of the six TUFLOW models. These comprised both inflows applied at the external TUFLOW model boundary and internal point source and region[10] inflows as shown on **Figures 5.1** to **5.6**.

## C4.2. Downstream boundary

The downstream boundary of the TUFLOW models comprised a tailwater representing the tidal conditions in Homebush Bay (Powells Creek TUFLOW Model), Canada Bay (St Lukes Park Canal TUFLOW Model) and Iron Cove (Iron Cove Creek TUFLOW Models). Due to the relatively short duration of catchment storm events affecting the study area, harbour levels were applied to the TUFLOW model as a static water level.

For the Concord Road TUFLOW model, the downstream boundary comprised a tailwater level based on normal depth flow conditions. The model extent was selected to ensure the downstream boundary was located a sufficient distance downstream of the project corridor to prevent any influence on flow behaviour within the vicinity of the proposed works.

### Tidal harbour water levels

For the purpose of the present investigation, a static harbour level of RL 1.0 metre AHD was adopted for simulation of local catchment flood events in the absence of a storm tide. A water level of RL 1.0 metre AHD approximately corresponds to the peak water level reached on average once or twice per year during a HHWSS tide.

### Storm tide harbour water levels

Office of Environment and Heritage's (OEH) guideline entitled *Flood Risk Management Guide: Incorporating Sea Level Rise Benchmarks in Flood Risk Assessments*, (DECCW, 2010) was prepared to assist councils, the development industry and consultants to incorporate the sea level rise planning benchmarks in floodplain risk management planning for new development. The guideline contains an appendix on modelling the interaction of catchment and coastal flooding for different classes of tidal waterway. The appendix may be used to derive scenarios for coincident flooding from those two sources for both present day conditions and conditions associated with future climate change.

For a catchment draining directly to the ocean via trained or otherwise stable entrances such as is the case for the catchments within the study area, the guideline offers the following alternative approaches for selecting storm tidal conditions under present day conditions. In order of increasing sophistication they are:

- A default tidal hydrograph which has a peak of 2.6 metres AHD for the 1 in 100 year event; or 2.3 metres AHD for the 1 in 20 year event. This default option is acknowledged (in DECCW, 2010) as providing a conservatively high estimate of tides for these types of entrances. Results achieved with these levels have been determined in the present investigation, but are only presented as a sensitivity study.
- A site-specific analysis of elevated water levels at the ocean boundary. The analysis should include contributions to the water levels such as tides, storm surge, wind and wave set up. The analysis should examine the duration of high tidal levels, as well as their potential coincidence with catchment flooding. This approach requires a more detailed consideration of historic tides and the entrance characteristics, but provides information which is more directly relevant to a particular entrance.

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[10] *In parts of the model area, inflow hydrographs were applied over individual regions called "Rain Boundaries". The Rain Boundaries act to "inject" flow into the one and two-dimensional domains of the TUFLOW model, firstly at a point which has the lowest elevation, and then progressively over the extent of the Rain Boundary as the grid in the two-dimensional model domain becomes wet as a result of overland flow.*

The latter approach has been adopted for design purposes in the present investigation. Design still water levels applicable to Sydney Harbour were obtained from the *Fort Denison Sea Level Rise Vulnerability Study* (DECC, 2008) (refer **Table C1**). An estimate of the Extreme Tide design still water level was obtained by extrapolating the design still water level probability curve provided in Appendix C of DECC, 2008 and assuming a recurrence interval of 1 in 100,000 years.

An allowance of 0.3 metres to account for local storm effects such as wind setup and wave conditions was added to the design still water levels to yield the design peak 'storm tide' levels (also shown in **Table C1**) that were adopted for assessment of storm tide flooding in the study area.

**Table C1 Design harbour water levels**

| Event | Design Still Water Level (1) (metres AHD) | Design Peak Storm Tide Level (metres AHD) |
|---|---|---|
| 1 in 20 year | 1.375 | 1.675 |
| 1 in 100 year | 1.435 | 1.735 |
| Extreme Tide | 1.6 (2) | 1.9 |

Source: DECC, 2008

(1) The design still water level for the Extreme Tide has been estimated based a return period of 1 in 100,000 years and extrapolation of design still water levels provided in DECC, 2008 for events up to the 1 in 200 year.

## Derivation of design flood envelopes

A flood envelope approach was adopted for defining water surface elevations and flooding patterns throughout the study area. The process was as follows:

- Step 1 – Run the hydraulic model for local catchment storms of various return periods and durations in combination with the HHWSS tide level. [Note that a static water level of RL 1.0 metre AHD was adopted as the downstream boundary of the hydraulic model for these runs].
- Step 2 – Combine the results of Step 1 to create an envelope of maximum local catchment flood levels for each return period (i.e. the results of running storms of the same return period but different duration were combined to create a single envelope).
- Step 3 – Run the hydraulic model for local catchment storms in combination with peak design storm tide levels of various return periods. [Note that the static water levels shown in **Table C1** were adopted as the downstream boundary of the hydraulic model for these runs].
- Step 4 – Prepare a final set of flood envelopes for each return period using a combination of the envelopes derived from Step 2, and a corresponding storm tide condition from Step 3. **Table C2** sets out the combination of local catchment and storm tide conditions which were used to compile the design flood envelopes for the study area.

**Table C2 Derivation of design flood envelopes**

| Design Flood Envelope(1) | Local Catchment Flood | Harbour Boundary Condition |
|---|---|---|
| 100 year ARI | 100 year ARI (2) | HHWSS peak tide level |
| | 20 year ARI (3) | 1 in 100 year peak storm tide level |
| PMF | PMF (4) | HHWSS peak tide level |
| | 100 year ARI (3) | Extreme Tide peak storm tide level |

(1) Indicates use of local catchment floods for durations ranging between 25 and 90 minutes.

(2) Indicates use of local catchment flood for duration of 60 minutes only.

(3) Indicates use of local catchment floods for durations ranging between 15 and 90 minutes.

# C5. Model parameters

The main physical parameter represented in TUFLOW is the hydraulic roughness, which is required for each of the various types of surfaces comprising the overland flow paths in the two-dimensional domain, as well as for the streams incorporated as one-dimensional elements. In addition to the energy lost by bed friction, obstructions to flow also dissipate energy by forcing water to change direction and velocity, and by forming eddies. Hydraulic modelling traditionally represents all of these effects via the surface roughness parameter known as "Manning's n".

Hydraulic roughness values adopted for design purposes were selected based on site inspection, past experience and values contained in the engineering literature (refer **Table C3**).

**Table C3 'Best estimate' of hydraulic roughness values adopted for TUFLOW modelling**

| Surface Treatment | Manning's n Value |
|---|---|
| Reinforced concrete pipes and box culverts | 0.015 |
| Open channels – concrete lined | 0.015 - 0.02 |
| Roads/railways | 0.02 |
| Open channel – heavily vegetated | 0.12 |
| Grassed reserves | 0.035 - 0.045 |
| Treed areas | 0.08 |
| Buildings | 10 |

# C6. Sensitivity analyses

## C6.1. Increase in hydraulic roughness

A sensitivity analysis was undertaken to assess the impact of a 20 per cent increase in the 'best estimate' values of hydraulic roughness (refer **Table C3**) on flooding patterns in the vicinity of the proposed tunnel entries and ancillary facilities during a PMF event. The findings of the sensitivity analysis were as follows:

- Homebush Bay Drive interchange – Peak flood levels on the northern side of the existing M4 corridor increased by up to 0.23 metres adjacent to the Underwood Road underpass, but are typically 0.1 metres or less in the vicinity of the proposed tunnel ventilation building.
- Wattle Street (City West Link) interchange – Peak flood levels adjacent to the proposed tunnel dive structure increased by up to 0.08 metres.

- Cintra Park fresh air supply and water treatment facility - Peak flood levels adjacent to the proposed facility increased by up to 0.01 metres.

Consideration of these sensitivity analyses in setting minimum design levels is discussed further in **section 6.4**.

Flood levels at the Concord Road and Parramatta Road interchanges would be dependent on the design the local surface grading, flood protection barriers and drainage infrastructure.

## C6.2. Partial blockage of hydraulic structures

The impact a partial blockage of the safety fences associated with two pedestrian bridges that are located on the main arm of Dobroyd Canal (Iron Cove Creek) downstream of Ramsay Street (denoted respectively as Pedestrian Bridge ICC1 and ICC2 on **Figures 4.8** and **4.12**) has on flooding patterns in the vicinity of the Dobroyd Parade tunnel dive structure during a PMF event was assessed.

Due to the relatively close spacing of the vertical bars in the safety fence associated with Pedestrian Bridge ICC2 when compared to those on Pedestrian Bridge ICC1, blockage factors of 50 per cent and 25 per cent were respectively applied to each.

The analysis showed that peak flood levels at the location of the Dobroyd Parade tunnel dive structure would increase by 0.04 metres, from 4.12 metres AHD to 4.16 metres AHD should the safety fences along both pedestrian bridges experience a partial blockage during a PMF event.

## C6.3 Increase in tailwater level

A sensitivity analysis was undertaken to assess the impact an increase in tailwater level would have on flooding patterns in the vicinity of the Dobroyd Parade tunnel dive structure and Cintra Park fresh air supply facility during a PMF event. A tailwater level of 2.6 metres AHD was modelled based on the 1 in 100 year ARI storm tide level of 1.7 metres AHD plus a 0.9 metres sea level rise[11].

The analysis showed that were a PMF event to occur following a rise in sea levels of 0.9 metres and in combination with a 1 in 100 year storm tide, then peak flood levels at the location of the Dobroyd Parade tunnel dive structure would only increase by 0.03 metres when compared to present day conditions (i.e. from RL 4.12 metres AHD to RL 4.15 metres AHD). The corresponding increase in peak flood level at the Cintra Park fresh air supply facility would be 0.04 metres.

# C7. Comparison of results with previous studies

Results of hydraulic modelling were compared to peak flood levels presented in WMAwater, 2014 at five locations in the Dobroyd Canal (Iron Cove Creek) catchment in the vicinity of the project for the 100 year ARI and PMF events. The locations selected for the comparison are shown on **Figure C4** while a summary of results is presented in **Table C4**.

**Table C4** shows that peak 100 year ARI flood levels from the present study are typically 0.2 to 0.4 metres higher than those in WMAwater, 2014, while PMF levels from the present study are typically 0.2 metres higher than corresponding values in WMAwater, 2014. The main exception to these trends was at Frederick Street (P5) where the result of the present study were within 0.02 metres of the WMAwater, 2014 results for both the 100 year ARI and PMF events.

The differences in peak flood levels can be attributed to the inclusion of catchment storage in the WMAwater, 2014 hydraulic model. For the present study the extent and detail in the hydraulic model has been tailored specifically to the assessment of flood behaviour in the vicinity of the project. In comparison, the WMAwater, 2014 study is based on a broad scale, catchment wide hydraulic model that includes catchment storage within areas upstream of the project. Upstream catchment storage has less of an influence on peak PMF flood levels due to the volume of runoff in this event. There is

[11] *A rise in sea level of 0.9 metres is based on the 2100 projection from the OEH guideline "Floodplain Risk Management Guide: Incorporating Sea Level Rise Benchmarks in Flood Risk Assessments" (DECCW, 2010). Refer to **Section 6.5** for an assessment of the potential impacts of climate change on sea level rise and increased rainfall intensities across the project.*

limited catchment storage upstream of Frederick Street, which would explain the lower differences in results at this location.

With due consideration of the above comparison, the hydraulic model developed for the EIS is considered appropriate in the assessment of the concept design and in particular confirming PMF protection to the tunnel entries and ancillary facilities and relative changes in flood behaviour.

**Table C4 Comparison of peak flood levels (m AHD)**

| Location | | 100 year ARI | | PMF | |
|---|---|---|---|---|---|
| ID | Description | EIS | WMAwater(1) | EIS | WMAwater(1) |
| P1 | Dobroyd Canal (Iron Cove Creek) upstream of Timbrell Drive | 2.15 | 1.77 | 3.10 | 2.89 |
| P2 | Timbrell Drive | 1.97 | Not flooded | 2.70 | 2.72 |
| P3 | Dobroyd Parade | 2.37 | 2.23 | 3.16 | 2.99 |
| P4 | Dobroyd Canal (Iron Cove Creek) downstream of Parramatta Road | 3.40 | 3.19 | 5.74 | 5.50 |
| P5 | Frederick Street | 9.42 | 9.41 | 9.94 | 9.92 |

(1) Based on results presented in Table 24 of WMAwater, 2014.

# C8. Model results – present day conditions

Results of the hydraulic modelling are presented in the following figures and described in **section 4.3** of this report:

- **Figures 4.5**, **4.6**, **4.7** and **4.8** show design 100 year ARI and PMF water surface profiles along the main arms of Saleyards Creek, Powells Creek, St Lukes Park Canal and Dobroyd Canal (Iron Cove Creek), respectively.
- **Figures 4.9**, **4.11** and **4.12** show 100 year ARI flooding patterns along the modelled reaches of Powells Creek, St Lukes Park Canal and Dobroyd Canal (Iron Cove Creek), respectively. **Figure 4.10** also shows detailed 100 year ARI flooding patterns in the vicinity of the Concord Road interchange.
- **Figures 4.13**, **4.15** and **4.16** show flooding patterns in a PMF event along the modelled reaches of Powells Creek, St Lukes Park Canal and Dobroyd Canal (Iron Cove Creek), respectively. Detailed PMF flooding patterns in the vicinity of the Concord Road interchange are show on **Figure 4.14**.

Figures showing flooding patterns from local catchment flooding in a 5, 20 and 200 year ARI event are provided at the end of this Appendix.

# C9. Assessment of post-construction conditions

This section describes the changes that were made to the structure of the TUFLOW models that were originally developed to define flooding behaviour under present day conditions to incorporate details of the project under post-construction construction conditions.

Changes made to the TUFLOW models were based on concept road and drainage design drawings and models provided by WDA in May and June 2015. Assumptions and limitations of the hydraulic modelling based on available details of the concept design provided by WDA are identified that would need to be confirmed during detailed design.

**M4 Motorway - Homebush Bay Drive to Pomeroy Street (Figure 6.1):**

- The Powells Creek TUFLOW model representing present day conditions was modified to reflect the proposed concept design arrangement.
- Proposed bridge arrangements and design surface elevations were obtained from the TUFLOW model developed by the Leighton Samsung John Holland joint venture (LSJH) as part of the concept design (LSJH Powells Creek TUFLOW model).
- No details have been provided of the dimensions of the new bridge downstream of the existing M4 to accommodate the M4 eastbound cycleway overpass. For the purpose of the flood impact assessment it was assumed that this bridge would be elevated above the floodplain. Details of this bridge and its impact on flooding would need to be confirmed during detailed design.

**Homebush Bay Drive interchange (Figure 6.5):**

- The Powells Creek TUFLOW model representing present day conditions was modified to reflect the proposed concept design arrangement.
- Modifications to transverse drainage structures XD01 and XD02 and design surface elevations were obtained from the LSJH Powells Creek TUFLOW model.
- Existing buildings over the alignment of the cut and cover tunnel were removed from the TUFLOW model representing post-construction conditions as shown on **Figure 6.5**.
- For the purpose of assessing a 'worst-case' scenario, no changes were made to existing topographic features, including building footprints, in areas of the construction ancillary facilities north of the M4 (refer construction site C3a in **section 5.3** of this report). During details design it would be necessary to design the reinstatement of cut and cover and construction ancillary facilities to minimise changes to existing topographic features that would otherwise lead to an obstruction and/or redistribution of overland flow and adverse flood impacts in areas outside the project corridor.

**Powells Creek off-ramp (Figure 6.5):**

- The Powells Creek TUFLOW model representing present day conditions was modified to reflect the proposed concept design arrangement.
- Details of the Powells Creek off-ramp were not included in the LSJH Powells Creek TUFLOW model. The location of the bridge abutments were therefore obtained from concept road design.
- No details were provided of the proposed location and dimensions of piers to support the bridge structure. For the purpose of the flood impact assessment no piers have been included in the TUFLOW model representing post-construction conditions. Details of the pier layout and their impact on flooding would need to be confirmed during detailed design.

**Concord Road interchange (Figure 6.9):**

- The Concord Road TUFLOW model representing present day conditions was modified to reflect the proposed concept design arrangement.
- Modifications to the TUFLOW model to reflect the concept design was based on the concept road and drainage design drawings.
- Design surface elevations, including concrete barriers, were obtained from concept road design model.
- The proposed pit and pipe layout and alignment of the grass lined channel between Sydney Street and Alexandra Street were obtained from the concept drainage design drawings.
- Inflow boundaries in the TUFLOW model were adjusted to reflect the changes in catchment runoff attributable to the surface road works.

**Cintra Park fresh air supply and water treatment facility (Figure 6.13):**

- The St Lukes Park Canal TUFLOW model representing present day conditions was modified to reflect the proposed concept design arrangement.
- The footprint of the Cintra Park water treatment facility and emergency smoke extraction ventilation outlet were raised above the PMF level to represent a complete blockage to flow.
- A nominal three metre wide overland flow path was provided along the western side of the facility between Parramatta Road and Gipps Street based on details provided in the TUFLOW model developed by LSJH as part of the concept design (LSJH St Lukes Park Canal TUFLOW model).
- The car park area located in the north-west corner of the site was assumed to be constructed at-grade.
- No details have been provided on the peak combined discharge from the water treatment plant and water quality basin. For the purpose of the flood impact assessment no change has been made to the model inflow conditions. During detailed design it would be necessary to design the discharge of treated water from the water treatment facility to control peak discharges that would otherwise lead to adverse impacts on flood behaviour in St Lukes Park canal.

**Parramatta Road interchange (Figure 6.17):**

- The Iron Cove Creek TUFLOW model representing present day conditions was modified to reflect the proposed concept design arrangement.
- The following modifications were made based on details provided in the TUFLOW model developed by LSJH as part of the concept design (LSJH Iron Cove Creek TUFLOW model):
  - Ground levels within the project footprint were adjusted to suit the design elevations of the surface road works.
  - A flood protection barrier was inserted along the eastern side of the tunnel dive structure as shown in **Figure 6.17**.
  - Stormwater drainage line XD09c was diverted along Parramatta Road to connect into the Sydney Water trunk drainage line in Bland Street. The location of the connection point was adjusted from the southern side to the northern side of Bland Street to connect into the 1600 millimetre RCP instead of the 1350 millimetre RCP and thus reduce hydraulic losses that would otherwise cause surcharge of the drainage system in Bland Street.
  - The stormwater detention tank was modelled as an oversized culvert based on the dimensions provided in the LSJH Iron Cove Creek TUFLOW model. The location of the stormwater detention tank, which had been included in the LSJH Iron Cove Creek TUFLOW on the western side of Parramatta Road, was relocated to suit the latest concept design drawings (Revision W).
- In addition, inflow boundaries in the TUFLOW model were also adjusted to reflect the changes in catchment runoff attributable to the surface road works.

**Wattle Street (City West Link) interchange (Figure 6.21):**

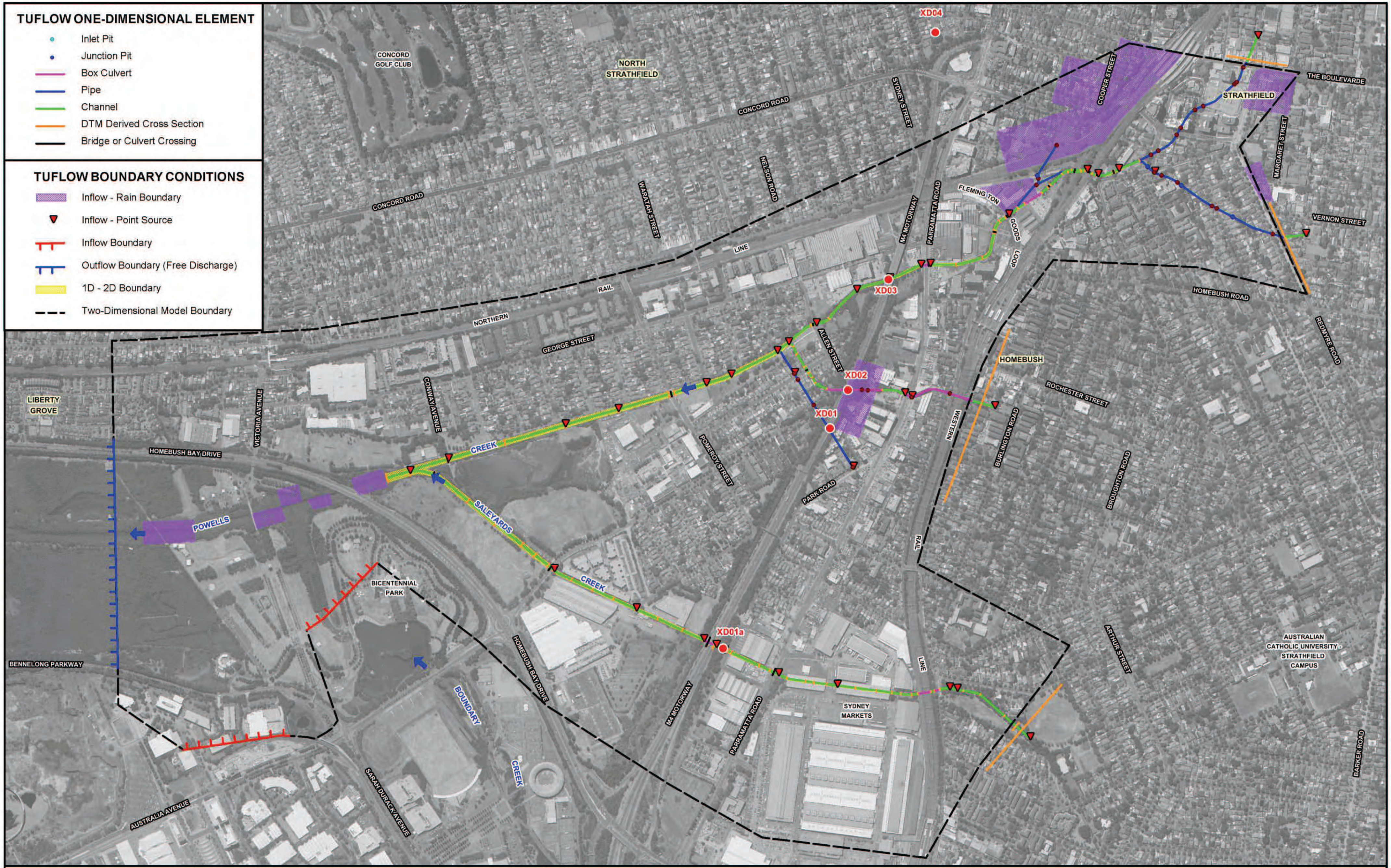
- The Iron Cove Creek TUFLOW model representing present day conditions was modified to reflect the proposed concept design arrangement.
- The following modifications were made based on details provided in the TUFLOW model developed by LSJH as part of the concept design of the Wattle Street interchange (LSJH Iron Cove Creek TUFLOW model):
  - Ground elevations within the project footprint were adjusted to suit the design elevations of the surface road works. This included a flood protection barrier at the tunnel portals.
  - Adjustments were made to the pit and pipe drainage systems along Dobroyd Parade to accommodate the proposed road works.
  - In addition, inflow boundaries in the model were adjusted to reflect changes in catchment runoff attributable to the proposed surface road works.

# C10. Model results – post-construction conditions

Results of the hydraulic modelling of post-construction conditions are presented in the following figures and described in **section 6.2** of this report:

- **Figures 6.1** to **6.21** and **6.23** to **6.25** show flooding patterns and impacts under post-construction conditions in a 100 year ARI and PMF event.
- **Figure 6.24** shows a comparison of flooding patterns at Dobroyd Parade under present day and post-construction conditions in a five year ARI event.

Figures showing flooding patterns and impacts under post-construction conditions during a 5, 20 and 200 year ARI event are provided in **Appendix D**.




100 0 100 200 300 m

Scale: 1:10,000

LEGEND

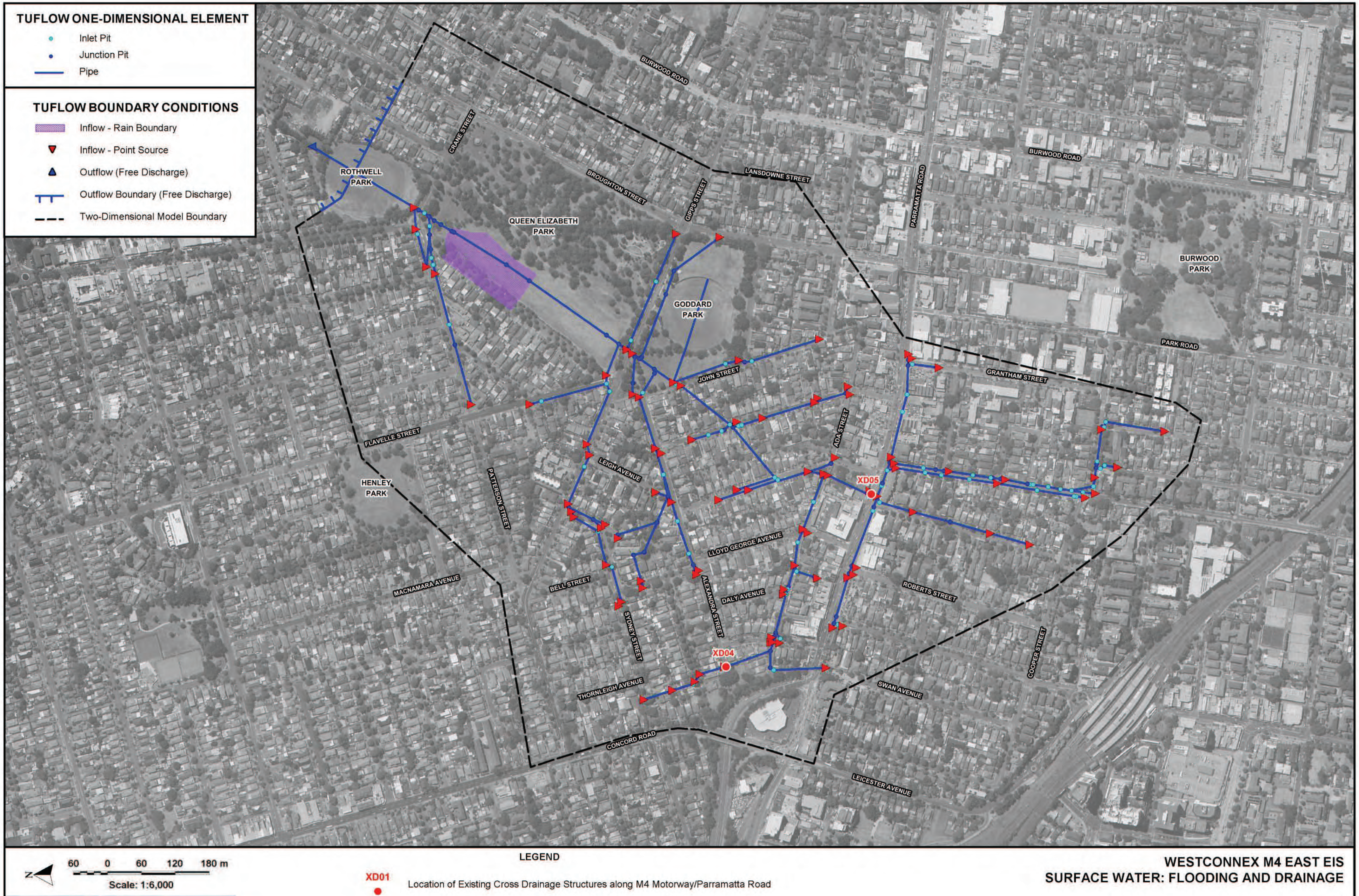
XD01 Location of Existing Cross Drainage Structures along M4 Motorway/Parramatta Road

Lyall & Associates

**WESTCONNEX M4 EAST EIS**
**SURFACE WATER: FLOODING AND DRAINAGE**

Figure C1

POWELLS CREEK TUFLOW MODEL LAYOUT




LEGEND

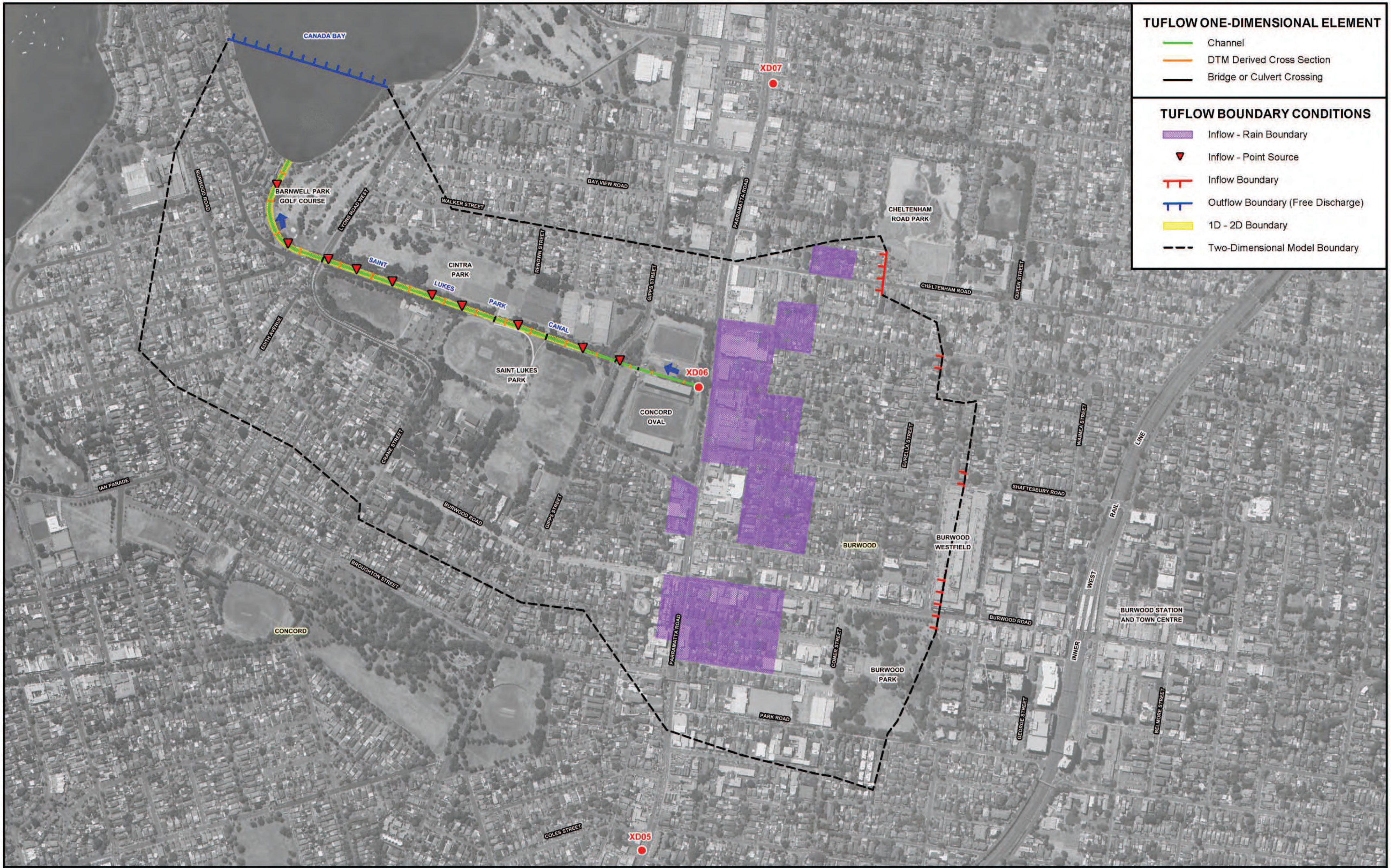
XD01 Location of Existing Cross Drainage Structures along M4 Motorway/Parramatta Road

Lyall & Associates

**WESTCONNEX M4 EAST EIS**
**SURFACE WATER: FLOODING AND DRAINAGE**

Figure C2

CONCORD ROAD TUFLOW MODEL LAYOUT




N

80 0 80 160 240 m

Scale: 1:8,000

Lyall & Associates

LEGEND

XD06 ● Location of Existing Cross Drainage Structures along M4 Motorway/Parramatta Road

**WESTCONNEX M4 EAST EIS**
**SURFACE WATER: FLOODING AND DRAINAGE**

Figure C3

SAINT LUKES PARK CANAL TUFLOW MODEL LAYOUT