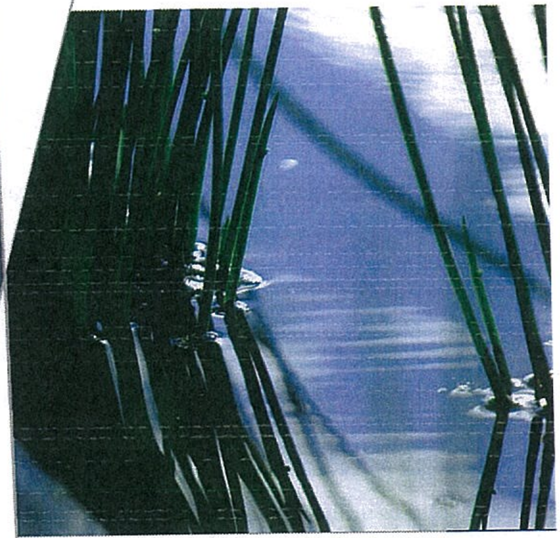
Project Waters

Flood Report

J11078



Prepared for
Metro Property Developments Pty Ltd

21 December 2012

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1 Introduction

It is proposed to develop a mixed use development at a lot located at Water Street, Fortitude Valley. The development site is described as Lot 1 on RP42507, parish of North Brisbane, County of Stanley. The location of the site is shown in Figure 1.

A Flood Study has been completed to determine the measures need to ameliorate the impact of the proposed development.

2 Site Description

The proposed development site has an approximate area of 0.74 hectares. The site location is shown in Figure 1.

The site is bounded to east by Water Street and to the north, south and west by commercial developments. The ground levels across the site range from 10.5 mAHD to 16.5 mAHD. The site is currently used as a carpark as shown in Figure 2.

For any rainfall causing a flow greater than the capacity of the underground drainage system, the remaining flow is conveyed overland. In the case of the Water Street catchment, the overland flow occurs along Water Street, with a consequent flooding of properties located on either side of the street.

3 Hydrology

3.1 Rational Method Calculations

The flows used for the TUFLOW model were derived using a RAFTS rainfall runoff model of the catchment. The model parameters were adjusted until a good agreement was obtained between predicted peak flow rates and those calculated using the Rational Method calculations as outlined in the Brisbane City Council (BCC) *Subdivision & Development Guidelines* and the *Queensland Urban Drainage Manual* (QUDM). This approach was considered to be acceptable due to the uniformity of the catchment and the relatively small subcatchment areas used for the comparison.

3.1.1 Rainfall Intensities

The rainfall intensities provided in Table BA2.7.1 of the BCC *Subdivision & Development Guidelines* were used to determine the peak flows.

3.1.2 Catchment Areas

The catchment area was broken up into 9 smaller sub catchments in both the existing and developed cases to allow for a good representation of the input of flows into the model. The areas of each of the sub catchments are shown below in Table 3-1. Due to the existing level of development within the development site, the same inflows were adopted for both the existing and developed cases. The catchment areas are shown in Figure 3.

Table 3-1 Catchment Areas – Existing Case

Name	Area (ha)	Name	Area (ha)
A0	34.83	A5	2.210
A1	6.180	A6	1.860
A2	5.550	A9	1.670
A3	4.540	A10	1.790
A4	7.940		

3.1.3 Runoff Coefficient Values

The ultimate level of development for each sub catchment external to the site, determined from the Brisbane City Council Planning Scheme has been used to identify the corresponding runoff coefficients. The runoff coefficients provided in Table B2.2 of the *Brisbane City Council Subdivision and Development Guidelines* were used for the corresponding land uses. The three main land uses with the Water Street catchment and the corresponding runoff coefficients used are listed below in Table 3-2.

Table 3-2 Runoff Coefficients

Developed Category	C_{10}
High Density Residential	0.87
Low/Medium Density Residential	0.85
Commercial	0.88

3.1.4 Times of Concentration

The times of concentration were determined using the standard inlet times based on the characteristics of the catchment and assuming a pipe velocity of 2 m/s. A summary of the calculations shown below in Table 3-3.

Table 3-3 Time of Concentration Calculations – Existing Case

Catchment	Standard Inlet time (min)	Pipe Flow			Channel Flow			Total Tc (min)
		Length (m)	Velocity (m/s)	Time (min)	Length (m)	Velocity (m/s)	Time (min)	
A0	5	890	2	7.42	-	-	-	12.42
A1	5	240	2	2.00	-	-	-	7.00
A2	5	-	-	-	300	1.5	3.33	8.33
A3	5	-	-	-	200	1.5	2.22	7.22
A4	5	-	-	-	290	1.5	3.22	8.22
A5	5	135	2	1.13	-	-	-	6.13
A6	5	130	2	1.08	-	-	-	6.08
A9	5	65	2	0.54	-	-	-	5.54
A10	5	-	-	-	-	-	-	5.00

3.1.5 Peak Flows

The peak flows were calculated for the 10, 50 and 100 year events. The results of the Rational Method are shown below in Table 3-4.

Table 3-4 Rational Method Peak Flow

Catchment	Contributing Area (ha)	Coefficients of Runoff C10	100 Year Event	
			Rainfall Intensity (mm/h)	Peak Flow (m^3/s)
A0	34.83	0.86	233.67	22.61
A1	6.180	0.85	288.00	4.94
A2	5.550	0.85	270.67	4.17
A3	4.540	0.85	284.67	3.59
A4	7.940	0.85	271.78	5.99
A5	2.210	0.88	301.50	1.85
A6	1.860	0.88	302.33	1.56
A9	1.670	0.88	313.17	1.45
A10	1.790	0.88	325.00	1.62

3.2 Hydrologic Modelling

A RAFTS hydrologic model of the catchment was setup to determine the discharge hydrographs from each of the sub catchments at their outlet points. RAFTS is an urban and rural rainfall runoff routing program that can be used to determine the peak stormwater flows for a catchment, based on parameters such as area, fraction impervious, slope and catchment storage.

A RAFTS models were setup to represent the catchment. The model was compared to the results obtained with the Rational Method for the 100 year ARI event. RAFTS model parameters such as Manning's n, slope and Bx were varied within reasonable limits until an acceptable agreement was obtained between the RAFTS and Rational Method flow estimates for the 100 year ARI event. This approach was considered to be acceptable given the relatively small size of the catchments considered and their uniform nature. The results and comparisons to the Rational Method are presented below in Table 3-5.

A Bx value of 1.0 was adopted for the RAFTS model for the existing catchment conditions.

The RAFTS model was run for a range of storm durations from 15 minutes to 6 hours to determine the peak flow rate for a given ARI event. Rainfall losses of zero initial and continuing loss were adopted for impervious areas and a zero initial loss and continuing loss rate of 2.5 mm/h were adopted for pervious areas.

Given the results presented in Table 3-5 it was considered acceptable to use the RAFTS model to calculate the discharge hydrographs at the outlet points of the catchment.

Table 3-5 RAFTS Model Calibration – 100 Year ARI Event

Catchment	Rational Method	RAFTS	Difference (%)
A0	22.61	22.62	0.0
A1	4.94	4.87	-1.4
A2	4.17	4.21	1.0
A3	3.59	3.56	-0.8
A4	5.99	5.98	-0.2
A5	1.85	1.84	-0.5
A6	1.56	1.57	0.6
A9	1.45	1.43	-1.4
A10	1.62	1.55	-4.5

The calibrated model was used to derive runoff hydrographs for the 100 year event. The hydrographs were input to the hydraulic model to enable peak flows and flood levels within the study area to be calculated.

4 Hydraulic Analysis

4.1 Data Sources

The sources of the data used as part of the flood assessment of the subject site are listed below:

- Survey – external to the site, aerial laser survey data (collected in 2002) ; and within the site, detailed survey completed by Jensen Bowers
- Aerial Photography – Aerial photography of the site was obtained from the Brisbane City Council's (BCC) eBimap (2009).

4.2 Drainage Network

At present the current flooding situation is exacerbated by the inadequate existing pipe and overland drainage network. If designed today, the underground system would be sized to convey the 10 year ARI event flow (i.e. the flow that can be expected to occur on average every 10 years). The current system can convey slightly less than the 2 year ARI event flow (i.e. the flow that can be expected to occur on average every 2 years). Therefore flooding currently occurs more frequently and to a greater extent than would now be considered acceptable.

Runoff produced by small floods in the Water Street catchment is piped firstly to Alexandria Street, and then piped in a north-westerly direction to Gregory Terrace. The flow is then piped beneath Gregory Terrace and beneath the No.1 Show Ring before crossing the railway, skirting the No.2 Show Ring and reaching the northern boundary of the site at O'Connell Terrace. The piped flow ultimately discharges to Breakfast Creek. No overland or surface flow occurs between Gregory Terrace and O'Connell Terrace due to the presence of high ground levels at certain locations.

For any rainfall causing a flow greater than the capacity of the underground drainage system, the remaining flow is conveyed overland. In the case of the Water Street catchment, the overland flow occurs along Water Street, with a consequent flooding of properties located on either side of the street.

The existing stormwater drainage system is shown in Figure 4. The pipe details such as size, length and invert levels for the existing drainage network in the catchment were taken from the BCC's eBimap and stormwater drainage drawings obtained from the BCC Plan Custodian. This data was verified against information listed in a previous study completed for Council (Tod Group, circa 1997) and detailed survey completed by Jensen Bowers. As sections of the existing drainage network are quite old, some of the required information was not available. In these instances the best estimates were taken, e.g by assuming slope of the pipes matched the surface slope. Appropriate manhole losses were adopted based on the recommendations of the *Queensland Urban Drainage Manual (QUDM 2007)*.

A key consideration in modelling is the interaction of the surface and underground drainage networks. Allowing water to freely transfer between the two networks (a common modelling assumption) can lead to erroneous results as the quantum of water transferred (for instance in an area where the capacity of the piped drainage system is reduced) can be unrealistic.

To overcome this issue, particular care was taken to model the gully pits and small pipes that connect the pits to the trunk drainage system. All gully inlets for the entire catchment area were surveyed during the site visit.

4.3 Existing Case TUFLOW Model Setup

4.3.1 Model Data

The stormwater drainage through the subject site was modelled using the linked one-dimensional/two-dimensional hydraulic model TUFLOW (Build 2009-07-AB). TUFLOW was considered to be suitable for use in this case due to its ability to model the underground drainage network one-dimensionally while allowing a detailed representation of the overland flow via a two-dimensional grid.

A digital terrain model (DTM) of the study site was setup based on ground level survey obtained from BCC. The extent of the TUFLOW study area is shown in Figure 4. Due to the urban nature of the study area, a grid with a spacing of 3 metres (i.e. ground levels being represented every 3 metres) was adopted.

Stormwater pipes and gully inlets were modelled as one dimensional links, connected to the two dimensional domain.

4.3.2 Roughness Values

The Manning's n roughness values for the study area were derived from aerial photographs and site inspection. The values adopted for the model are listed below in Table 4-1. Based on site inspection certain brick buildings and brick fences within the site which were deemed to block the flow were modelled as blockages to provide an accurate representation of the flow patterns.

Table 4-1 Roughness Values

Land Use	Manning's n
Residential/ Commercial Areas	0.20
Roads and Carparks	0.02
Open Space and Parks	0.04
Fences and Gates	0.08

4.3.3 Inflows

The discharge hydrographs calculated for the catchments using the RAFTS model were used in the TUFLOW model (refer to Section 3). The location of the inflow points in the TUFLOW model are shown in Figure 4.

4.3.4 Tail water Conditions

A normal depth corresponding to a slope of one percent was assumed as the tail water condition occurring at the northern boundary of the model (corresponding to the slope of the tributary along Water St downstream of Anderson St). A water level of 8.04 mAHD, equal to obvert of the pipe at the downstream end of the model along the eastern boundary, has been adopted for the one dimensional drainage network.

4.3.5 Time Step

The time step used for the one dimensional/ two dimensional model was one second. This relatively short time step was required to increase the model stability and reduce the continuity error within the model.

4.4 Developed Case TUFLOW Model Setup

The developed case used the same data and setup as for the existing case model. The only difference to the model that additional drainage culverts and storage was added to the model to mitigate the impacts of the development.

It is proposed to provide a number of vertical and horizontal grates along the boundary of the property to capture the flow and pipe it to a storage area to offset the impacts as a result of filling the development site. Modelling has shown that a minimum of 325m³ of storage along with an additional 105m³ of storage provided within the drainage pipes is required to offset the impacts of the development.

Drawings showing the proposed drainage storage and drainage network are shown in Appendix A.

5 TUFLOW Results

The TUFLOW model described above was used to determine the 100 year flood levels in the vicinity of the subject site.

Points around the site were selected (Refer to Figure 4) so that the peak water levels and flows for the existing and developed cases could be easily compared. The resultant water levels at each point are shown in Table 5-1.

Maps of the peak flood levels and impacts for the existing and developed cases are in in Appendix B for the 100 Year ARI event.

Table 5-1 Peak Water Levels – 100 Year Event

Point of Interest	100 Year ARI Event – Peak Flood Levels (mAHD)		
	Existing Case	Developed Case	Afflux (mm)
WaterSt_01	12.328	12.327	-1
WaterSt_02	12.053	12.029	-24
WaterSt_03	11.902	11.902	0
WaterSt_04	11.568	11.565	-3
WaterSt_05	11.280	11.291	+11
WaterSt_06	10.807	10.815	+8
WaterSt_07	10.672	10.661	-11
WaterSt_08	10.191	10.177	-14
WaterSt_09	10.462	10.471	+9
WaterSt_10	10.469	10.469	0

With the use of additional storage provided within the site in the form of pipes and underground basins, the proposed development results in negligible change in peak water levels and therefore will not cause significant impacts on neighbouring properties.

6 Conclusion

The proposed development involves the construction of a mixed use development at Lot 1 on RP 42507 in Water Street, Fortitude Valley. A flood study of the catchment draining to the site has been carried out to determine the existing flood levels within the site, and the impact development on flood levels.

The flood study was carried out using a RAFTS hydrologic model and a TUFLOW hydraulic model.

It is proposed to provide additional stormwater drainage and additional storage within the site to compensate for the loss in flood plain storage as a result of the development and mitigate impacts.

The results of the TUFLOW model are shown in Table 5-1 and Appendix B. The results indicate that the proposed development does not adversely impact on properties within the vicinity of the subject site.

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FIGURES

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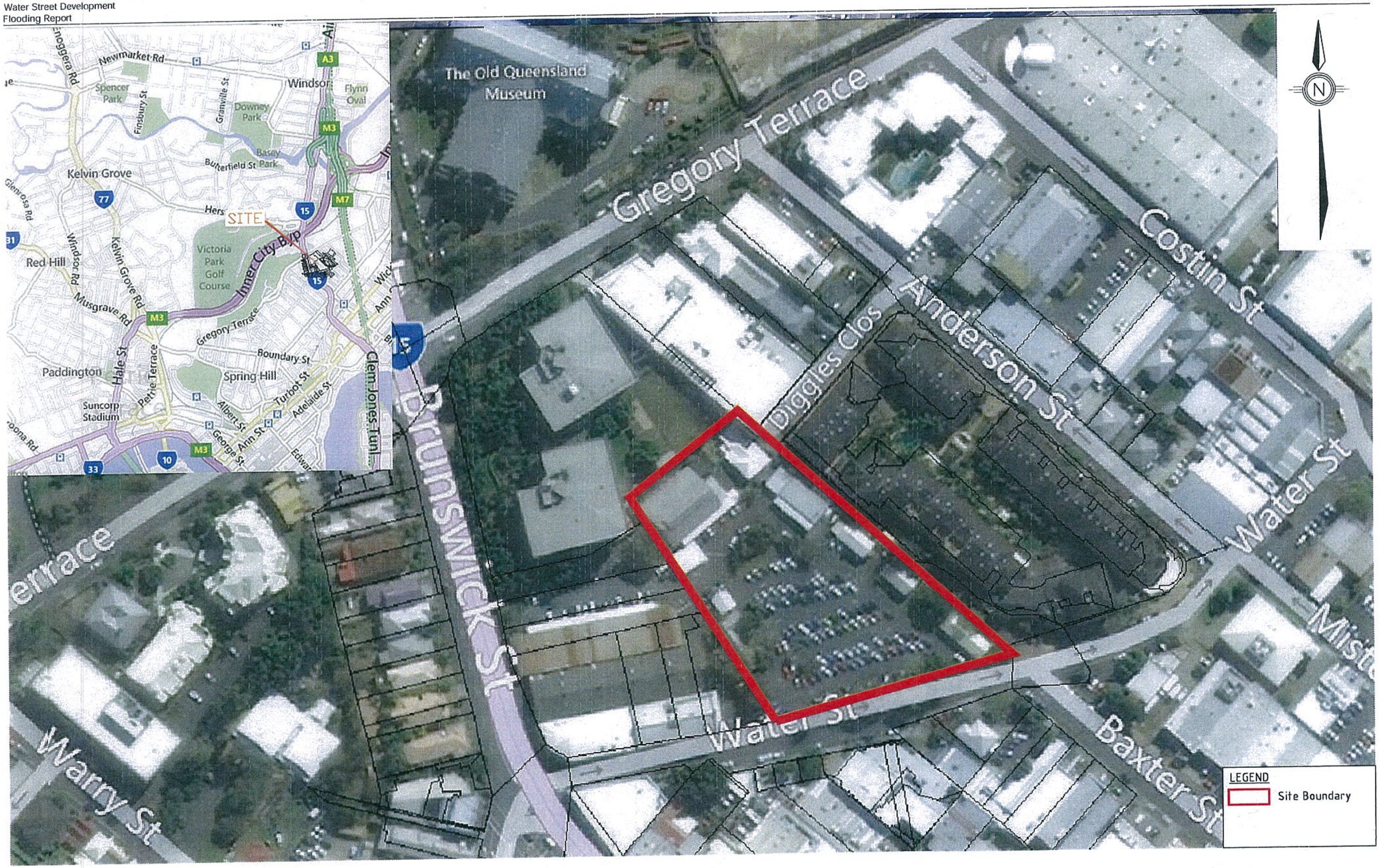

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FIGURE 1
LOCALITY PLAN

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FIGURE 2
AERIAL PHOTO

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FIGURE 3
CATCHMENT AREAS

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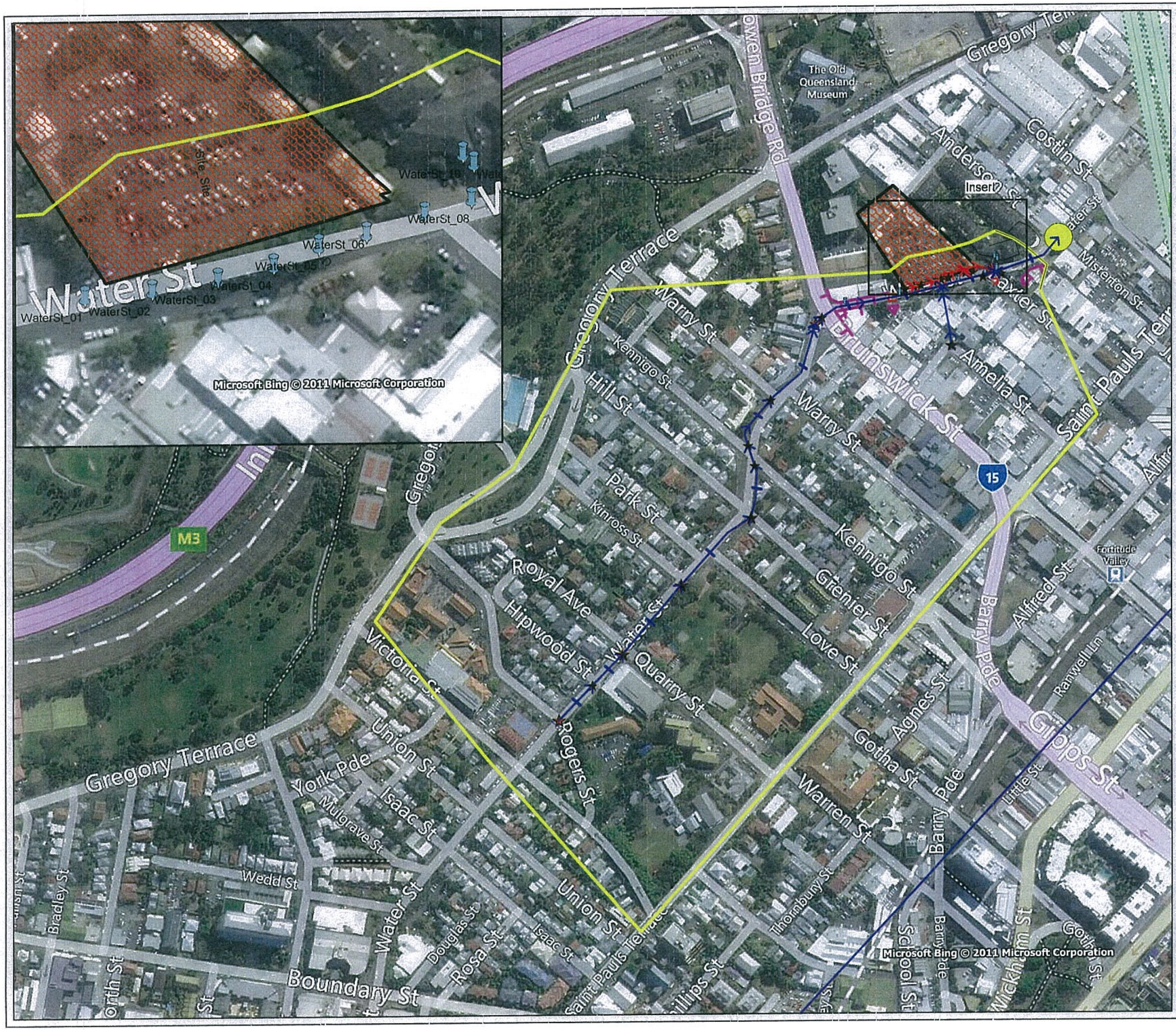

Figure 4

TUFLOW Model

WATER ST DEVELOPMENT

LEGEND

- Existing Pipe Drainage
- Proposed Pipe Drainage
- TUFLOW Model Extents
- Development Fill Extents

40 0 40 80 120
Metres
Scale: 1:4,000

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