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MNCE Ref: C3301

PLANS AND DOCUMENTS
referred to in the PDA
DEVELOPMENT APPROVAL

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Queensland Government

SITE BASED STORMWATER MANAGEMENT PLAN

221 Teviot Road, Greenbank

Commissioned By
N & A Campbell Developments Pty Ltd

BRISBANE OFFICE
5/63 Annerley Road, Woolloongabba QLD 4102
PO Box 8380, Woolloongabba QLD 4102
T 07 3255 1877 F 07 3255 1878
W mnce.com.au ABN 68 678 825 458

IPSWICH OFFICE
18 Canning Street, North Ipswich QLD 4305
PO Box 2118, North Ipswich QLD 4305
T 07 3281 6603 F 07 3281 6602

MILANOVIC NEALE
CONSULTING ENGINEERS

REPORT CONTROL SHEET

MNCE Ref. No.:	C3301
Site:	221 Teviot Road, Greenbank
Report Title:	Site Based Stormwater Management Plan
Report Author:	Jordan Hughes/Richard Dugdell

Revision / Checking

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Rev No.	XP-RAFTS File Name	MUSIC Model File Name	Rational Method File Name
A	C3301-190403-CALCS-RAFTS-REVB	C3301-190405-CALCS-REVB(SPEL)	C3301-190403-CALCS-REVB
B	C3301-190403-CALCS-RAFTS-REVC	C3301-190405-CALCS-REVC(SPEL)	C3301-190403-CALCS-REVC
C/D	C3301-190403-CALCS-RAFTS-REVC	C3301-190405-CALCS-REVC(SPEL)	C3301-190403-CALCS-REVC
E	C3301-190403-CALCS-RAFTS-REVE	C3301-190405-CALCS-REVC(SPEL)	C3301-190403-CALCS-REVE

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1 INTRODUCTION

1.1 Overview and Background

A development application for a material change of use has been lodged over Lot 9 on RP184971 at 221 Teviot Road, Greenbank. The site is located within a priority development area in Logan City Council and has a total area of approximately 2.0ha. Figure 1.1 below provides an aerial locality of the site and adjacent areas.

Figure 1.1: Subject Site bound by Teviot Road to the east (nearmap)

The proposed development involves a material change of use to a childcare facility. Refer Appendix A for proposed development layout.

The site generally falls in a north westerly direction on a grade of approximately 3% away from Teviot Road with no stormwater infrastructure present on the site.

It is to be noted that the site does not appear to be affected by any flood overlay based on Council mapping.

1.2 Objectives and Scope

Milanovic Neale Consulting Engineers have been commissioned by Nick Campbell to undertake an assessment of stormwater quality and quantity impacts associated with the proposed development. Stormwater management strategies are also to be identified for the proposed development as required.

The scope of works undertaken for this project is summarized below:

1. Assessment of the development against State and Local Government legislation to identify water quality management measures to be adopted for the proposed development.

2. Undertake pollutant export modelling of anticipated pre and post development pollutant concentrations and loads and conceptual design of water quality improvement devices.

3. Assessment of the pre and post development stormwater discharge and undertake preliminary design and commentary of any mitigation devices required to control site discharge if required.

This report also aims to respond to Additional Stormwater Items from a Further Issues Email dated 11th August 2020 from the Department of State Development, Tourism, and Innovation. The Stormwater Items are as follows with Milanovic Neale Consulting Engineers' (MNCE's) response in *italics*:

Stormwater

The Applicant is to provide an amended report which demonstrates that the appropriate quantum of stormwater attenuation and a suitable method of stormwater discharge can be provided in order to achieve a no worsening outcome on the downstream property. The amended report needs to consider the following issues;

1. The existing runoff from the existing site appears to have been overstated and calculations need to consider the following;

    - The existing dwelling has several rainwater tanks which have the effect of mitigating stormwater discharge from the roofed areas, and

      Rainwater tanks have been incorporated into the hydrologic analysis, refer Section 5.2.2.

- The existing dam on the site attenuates discharge from the site generally (and it is likely that runoff from the house is directed towards the dam resulting in further mitigation of existing flows) and should be reflected in the calculation of discharge from the existing site

  The dam has been incorporated into the hydrologic analysis, refer Section 5.2.2 and Section 5.4.2.

2. The proposed discharge from developed site may have been understated and calculations need to consider the following

   - Playground areas % impervious used in calculations is assumed to be "vegetation" (table 5.6) with a 0% impervious. Please demonstrate that this accurately represents the design outcome in the playground, and

     The playground catchment imperviousness has been amended to 40% as agreed with John Atkinson, Economic Development Queensland Engineer, refer Section 5.4.1.

   - The deceleration lane in Teviot Road (with K&C) and proposed driveway crossover will direct significant external flows down the internal driveway. Provide details of how this external catchment will be managed in the design.

     A proposed swale drain was requested to direct flow from the proposed deceleration lane in a northerly direction to discharge to existing table drain in Leanne Court by John Atkinson, Economic Development Queensland Engineer.

     A proposed swale however is not envisaged to achieve gravity discharge to the Leanne Court table drain owning to natural contours fronting the neighbouring property 205-219 Teviot Road. As can be seen below figure, there is 35m portion of verge that is flat, hence any proposed swale through this area is envisaged to be too deep to discharge into the existing table drain invert within Leanne Court Teviot Road intersection , especially considering the swale has to be deep enough to accommodate a pipe (with cover) under the immediate upstream rural driveway, flow, freeboard, and fall through this flat section of verge.

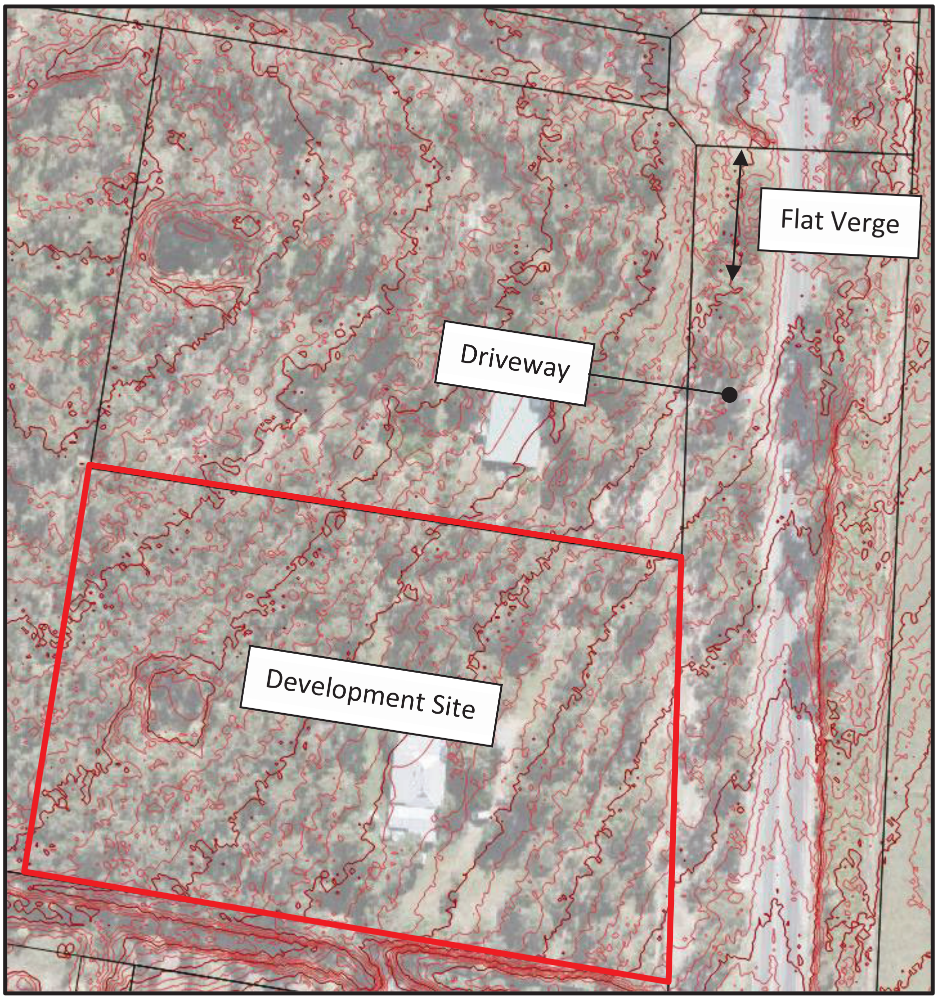

Figure 1.2: Verge Details

Alternately then, runoff from the proposed deceleration lane is to sheet flow through the site per existing conditions. The additional runoff generated from the change in pervious to impervious area of the deceleration lane has been found to be negligible. Based on a Rational Method calculation utilising a catchment of 0.037ha, a time of concentration of 25 mins to the rear neighbouring property, an existing C10 of 0.70 and a proposed C10 of 0.88, runoff was found to increase 1L/s, 2L/s, 2L/s, 3L/s, and 2L/s in the 39%, 10%, 5%, 2%, and 1% storm event respectively, which can be generally offset by the difference in flow rates between existing and mitigated conditions, refer Section 5.7.

3. The Applicant should confirm whether the existing dwelling and other improvements are being removed, or whether they are to remain on site. The Hydraulic assessment prepared by Milanovic Neale Consulting Engineers appears to only consider the stormwater discharge generated from the proposed child care centre in assessing post development flows. The report does

not appear to include the existing stormwater discharge generated from the existing dwelling which is required in order to determine the amount of stormwater attenuation required.

A portion of the existing dwelling is to be retained, and has been incorporated into the hydrologic analysis, refer Section 5.4.2.

4. The Applicant proposes to use a proprietary SPEL system to address stormwater quality, however it appears that the SPEL system is also being relied upon to attenuate stormwater discharge. The applicant is required to provide details of the proposed underground stormwater detention/ stormwater quality tank system to demonstrate that both quality and quantity impacts can be appropriately managed.

   Details of the stormwater detention and quality treatment tank has been provided, refer SK24 in Appendix A.

5. The applicant proposes a "spreader" located upstream of the effluent treatment area diversion bund so as to mimic existing sheet flows. The bunds to be constructed around the effluent treatment area will re concentrate any sheet flow achieved by the spreader back into a concentrated point source on the downstream property boundary and consequently this proposal is not supported.

   The shape of the effluent irrigation area has been amended so that the "spreader" is no longer located upstream of its associated diversion bund. Further, the effluent irrigation area is shaped per existing topography to reduce the requirements for bunds that may discharge concentrated flow onto neighbouring properties. Refer architect plans for irrigation layout.

6. If the Applicant proposes to discharge stormwater upstream of the effluent treatment area, it should be demonstrated that the stormwater discharge will not adversely impact the effluent treatment area through increased stormwater infiltration potentially mobilising pollutants into the groundwater.

   The shape of the effluent irrigation area has been amended so that the "spreader" is no longer located upstream of its associated diversion bund. Further, the effluent irrigation area is shaped per existing topography to reduce the requirements for bunds that may discharge concentrated flow onto neighbouring properties. Refer architect plans for irrigation layout.

2 DATA

2.1 State and Local Government Policies

The *Planning Scheme Policy 5 - Infrastructure* and the *Infrastructure Code* from *Logan City Council's Planning Scheme Version 3.0*, the *State Planning Policy – July 2017* and the *PDA Guideline No. 13 – Engineering Standards* have been used as a guide to establish the required stormwater objectives and requirements for the development.

2.2 Level and Modelling Data

Contour information for the site and adjacent properties was obtained from survey data prepared by Ian Davis Surveys which comprise surface level contours at 0.25m intervals.

Rainfall data for the Logan Region was extracted from AR&R 2016 through the IFD tool on the Bureau of Meteorology website.

The *Queensland Urban Drainage Manual (QUDM)* and the *Australian Rainfall and Runoff, 2016* have been used as a guide to establish the modelling inputs and methodologies applicable for the development.

3 OPPORTUNITIES AND CONSTRAINTS

3.1 Site Opportunities

Currently there are no stormwater quality measures on site, thus there are some opportunities to improve the quality of the stormwater leaving the site before discharge through stormwater treatment devices.

3.2 Site Constraints

The site does not currently possess a lawful point of discharge. The proposed development flows are to therefore discharge to the rear of the site via a level spreader to mimic pre-developed runoff conditions as much as possible. The location of this outlet needs to avoid disturbing any vegetation downhill of the site due to ecological constraints.

4 WATER QUALITY MANAGEMENT

This section of the report will provide an assessment of the development against State and Local Government legislation to identify water quality management measures to be adopted for the proposed development.

Pollutant export modelling will also be undertaken of anticipated pre-and post-development pollutant concentrations and loads and conceptual design of water quality improvement devices.

4.1 Risk Category

The *State Planning Policy – July 2017, PDA Guideline 13 – Engineering Standards* and *Planning Scheme Policy 5 - Infrastructure* from *Logan Planning Scheme 2015* identifies developments as high risk with respect to stormwater quality if any of the following criteria are triggered:

1. Material change of use urban purposes that involves a land area greater than 2,500m^2 that:
    a. Will result in an impervious area greater than 25 percent of the net developable area; or
    b. Will result in six or more dwellings;
    c. Located in the Loganholme local plan area; or
    d. Included in the Highway business precinct of the Specialised centre zone and located between the Pacific Highway and the southern boundary of the Loganholme local plan area; or
2. Reconfiguring a Lot for urban purposes where;
    a. The size of the subject premises is greater than 2,500m2 and results in an increased number of non-residential lots (e.g. industrial, commercial, etc); or
    b. it results in the creation of six or more residential lots;
    c. located in the Loganholme local plan area and results in a total of four or more lots; or
    d. Where included in the Highway business precinct of the Specialised centre zone and located between the Pacific Highway and the southern boundary of the Loganholme local plan area and results in a total of four or more lots.
3. Operational work for urban purposes that involves disturbing more than 2,500m^2 of land;

With respect to the above, the Policy can be applied to the proposed development due to point 3 being triggered. Therefore, in order to demonstrate required stormwater quality objectives are achieved, pollutant export modelling will be incorporated in the design of stormwater quality treatment train for the site.

4.2 Water Quality Objectives

4.2.1 Construction Phase

The *Urban Stormwater Quality Guidelines 2010* identify that eroded soils and litter are major pollutant sources during construction activity. There is also potential for hydro-modification of streams due to increased run-off coefficients when subsoils are exposed, for longer term major developments. Water sensitive urban design principles and reducing erosion during construction are fundamental to achieving water quality objectives in relevant waterways.

It is therefore proposed to prepare an erosion and sediment management plan during the operational works phase of the development which will incorporate a range of control measures to be implemented during the construction phase of the project.

4.2.2 Operational Phase

Stormwater quality management design objectives for the operational phase of the development are identified in the following table in accordance with the *Urban Stormwater Quality Guidelines 2010.* These objectives provide an emphasis on the reduction of mean annual loadings associated with suspended sediments and nutrients.

Table 4.1: Stormwater Quality Objectives

Region	Minimum Reductions in Mean Annual Loads from Unmitigated Development (%)			
	Total Suspended Solids (TSS)	Total Phosphorus (TP)	Total Nitrogen (TN)	Gross Pollutants > 5mm (GP)
South East Queensland	80	60	45	90

4.3 Pollutant Export Modelling

4.3.1 Model Selection

In order to determine on site pollutant generation, discharge concentrations of target pollutants and the effectiveness of Stormwater Quality Improvement Devices the Model for Urban Stormwater Improvement Conceptualisation (MUSIC) has been used to model the development proposal.

4.3.2 MUSIC Model Configuration

The following sections identify the modelling parameters used in the configuration of the MUSIC Model adopted for the development. The following figure provides a screenshot of the MUSIC model schematic for reference.

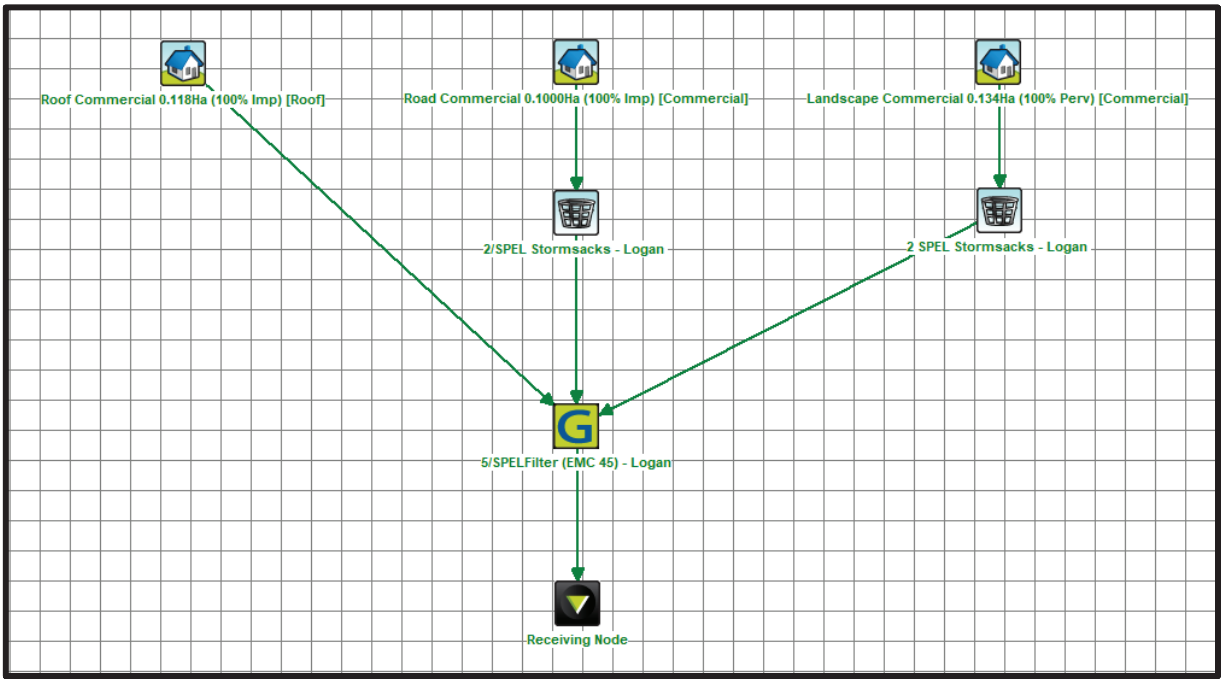

Figure 4.1: MUSIC Model Layout

4.3.2.1 Meteorological and Time Step

Meteorological Data used in the MUSIC model has been identified in accordance with the *MUSIC Modelling Guidelines 2010*, incorporating the following parameters:

Rainfall Period: 01/01/1980 12:00 AM to 31/12/1989 11:54 PM

Rainfall Station: 40659 GREENBANK

A model time step of 6 minutes has also been adopted as recommended in Section 3.2 of the *MUSIC Modelling Guidelines 2010.*

4.3.2.2 Catchment Properties

The MUSIC model for the development adopts a split catchment approach, whereby the site has been broken down into various catchments in accordance with site specific land uses. The following table provides a summary of the catchment data used in the MUSIC analysis.

Table 4.2: MUSIC Catchment Parameters

Catchment Name	Area (ha)	Fraction Impervious (%)	MUSIC Source Node
Roof	0.118	100	Commercial
Driveway	0.100	90	Commercial
Vegetation	0.134	0	Commercial

4.3.2.3 Rainfall Runoff Parameters

The following table provides a summary of the rainfall runoff parameters adopted for the source nodes used in the MUSIC analysis which has been extracted from Table 3.7 of the *MUSIC Modelling Guidelines 2010.*

Table 4.3: MUSIC Catchment Parameters

Parameter	Quantity
Rainfall Threshold (mm)	1
Soil Storage Capacity (mm)	18
Initial Storage (%)	10
Field Capacity (mm)	80
Infiltration Capacity Coefficient a	243
Infiltration Capacity Coefficient b	0.6
Daily Recharge Rate (%)	0
Daily Base-Flow Rate (%)	31
Daily Seepage Rate (%)	0

4.3.2.4 Pollutant Export Parameters

The following table provides a summary of the pollutant export parameters for split catchment surface types which have been adopted for the source nodes used in the MUSIC analysis. This information has been extracted from Table 3.8 of the *MUSIC Modelling Guidelines 2010.*

Table 4.4: MUSIC Source Node Base and Storm Flow Concentration Parameters

Land Use	Parameter	Total Suspended Solids (Log_{10} mg/L)		Total Phosphorus (Log_{10} mg/L)		Total Nitrogen (Log_{10} mg/L)	
Roof	Mean	N/A	1.3	N/A	-0.89	N/A	0.25
	Std Deviation	N/A	0.44	N/A	0.36	N/A	0.32
Road	Mean	0.78	2.43	-1.11	-0.30	0.14	0.25
	Std Deviation	0.45	0.44	0.48	0.36	0.20	0.32
Ground	Mean	0.78	2.16	-0.6	0.5	0.32	0.3
	Std Deviation	0.39	0.38	-0.39	0.34	0.37	0.34

4.3.3 Developed Unmitigated Conditions

MUSIC Modelling has been performed to determine the pollutant export and corresponding concentrations from the development site under the proposed conditions. The mean annual loads that have been estimated for proposed unmitigated conditions are given in Table 4.5 below, also shown are the mean annual load percentage reductions required to meet current State Planning Policy objectives.

Table 4.5: MUSIC Unmitigated Mean Annual Loads (kg/yr)

Pollutant	Unmitigated Mean Annual Load (kg/yr)	Reduction required to meet Council Requirements (%)
Total Suspended Solids (TSS)	379	80
Total Phosphorous (TP)	0.845	60
Total Nitrogen (TN)	6.08	45
Gross Pollutants (GP)	37.2	90

4.3.4 Developed Mitigated Conditions

The scale of the development is large enough to support the implementation of effective water quality design measures. Therefore, a consideration of a variety of best management measures has been given with a focus on water quality treatment measures and reducing polluted runoff volumes from the site.

Key elements of the proposed stormwater management plan are as follows:

- Runoff associated with the proposed access, car parking and playground area is to be treated through 4 source pollutant filters (SPEL Stormsacks or similar) before being piped to proposed underground detention tank within the driveway (see Section 5)
- Runoff associated with proposed roof areas to be piped to the proposed underground detention tank (see Section 5) which is to be fitted with 5. SPELFilter cartridges.
- Treated and detained site flows to be discharged towards the rear of the site via level spreader, sheet draining as per existing conditions.

The table below demonstrates that the required pollutant reductions have been achieved through the implementation of the proposed stormwater treatment train.

Table 4.6: MUSIC Model Removal Efficiencies

Pollutant	Unmitigated Mean Annual Load (kg/yr)	Mitigated Mean Annual Load (kg/yr)	Reduction required to meet Council Requirements (%)	Removal Efficiency Achieved (%)
Total Suspended Solids (TSS)	379	72	80	81
Total Phosphorous (TP)	0.845	0.284	60	66.4
Total Nitrogen (TN)	6.08	3.0	45	50.7
Gross Pollutants (GP)	37.2	1.12	90	97

5 WATER QUANTITY MANAGEMENT

This section of the report will provide an assessment of the pre and post development stormwater discharge and undertake preliminary design and commentary of any mitigation devices required to control site discharge.

5.1 Existing Conditions

This section of the report will analyse and comment on the existing site stormwater discharge conditions. The XP-RAFTS modelling software, utilising the Laurenson Routing Method will be used to generate the hydrographs and peak flows from the site for all storm events up to an including the 1% AEP storm event.

5.2 Existing Hydrologic Model

5.2.1 Catchment Definition

An examination of the existing site land topography and land use was undertaken to quantify the number of sub catchments and sub catchment areas applicable for the site. It was established that the existing site consists of three fundamental sub catchments; the impervious roof areas associated with the existing dwelling, the hardstand areas which make up the existing driveway, and the vegetated/grassed area which makes up the rest of the site.

Refer Table 5.1 below for existing site catchment details. The catchment roughness for impervious and pervious areas has been defined using manning's 'n' values of 0.015 and 0.05 respectively which are considered reasonable. The catchment slope has been determined by reviewing survey data prepared by Ian Davis Surveys.

Table 5.1: Catchment Data Summary Table

Catchment	Area (ha)	Impervious (%)	Slope (%)	Manning's 'n'
Roof	0.049	100	12	0.015
Hardstand	0.034	100	3	0.015
	0.004	0	3	0.030
Vegetation	1.915	0	3	0.050

5.2.2 Mitigation Parameters

The site has an existing dam that will attenuate flows under existing conditions, hence a portion of the vegetation catchment denoted above was directed to a retarding basin included in the proposed model.

The following parameters were used to define a stage storage discharge relationship for the dam:

- Minimum dam depth of 0.80m (depth available at time of survey) as agreed with as agreed with John Atkinson, Economic Development Queensland Engineer.
- Total Storage Volume of 194m^3 (storage available at time of survey)
- 1 x 7m wide weir at basin obvert

There are also tanks on site which will also attenuate roof flows under existing conditions. One tank is assumed to receive flows from a shed on the northern side of the residence (which is to be removed as a result of the development) and two tanks are assumed to receive flow from the residence and a shed on the southern side of the residence (which are to be retained as part of the development). The northern tank has hence been included in the proposed model. The southern tanks have not been considered however as discharge to and from the tanks are envisaged to be the same under both existing and proposed conditions, and the catchment are negligible in relation to the whole site.

The following parameters were used to define a stage storage discharge relationship for the northern tank:

- Minimum Tank depth of 1.80m.
- Total Storage Volume of 2.5m^3 (75% full) as agreed with as agreed with John Atkinson, Economic Development Queensland Engineer.
- 1 x 100mm outlet at basin obvert.

Refer to Appendix B for XP-RAFTS hydrographs and XP-RAFTS stage storage discharge relationship.

5.2.3 Rainfall Parameters and Losses

As indicated in Section 2.2 above, Rainfall data for the Logan Region has been extracted from AR&R 2016 through the IFD tool on the Bureau of Meteorology website.

The uniform loss method was adopted to account for rainfall losses throughout the existing catchment. Based on information provided in the *ARR Data Hub tool*, initial and continuing loss coefficients adopted for the impervious and pervious sub catchments have been tabulated below.

Table 5.2: Initial and Continuing Loss Parameters

Parameter	Value (mm)
Initial Loss (Pervious)	20.0
Continuing Loss (Pervious)	1.6

Temporal patters applicable to East Cost North has been adopted in accordance with the *Australian Rainfall and Runoff, 2016*.

5.2.4 Existing Hydrological Results

The following table provides a summary of the peak discharge from the existing site that was estimated by the XP-RAFTS model for all storm events up to and including the 1% AEP storm event.

Table 5.3: Existing Site Peak Discharges

Storm Event	XP-RAFTS Peak Runoff (m^3/s)
39% AEP	0.192
10% AEP	0.389
5% AEP	0.505
2% AEP	0.584
1% AEP	0.679

5.2.5 Runoff Comparison

QUDM Section 4.1.3 recommends hydrologic models are calibrated with actual flow data rather than to an alternate runoff routing model. In the absence of such data, Section 4.1.3 continues to state that as an alternative, model results may be compared with the results to a Rational Method peak discharge for catchments less than 500ha.

Based on the methods outlined in *QUDM*, the following parameters were used to estimate the peak runoff from the upstream catchment affecting the site. Refer Appendix C for detailed rational method calculation summary.

Table 5.4: Rational Method Parameters

Area (ha)	Runoff Coefficient (C10)	Time of Concentration (mins)
2.002	0.71	25

It is to be noted that the time of concentration has been derived based on the application of Friend's Equation.

The following table provides a comparison of the results of both the Rational Method and XP-RAFTS Model results for peak discharges up to and including the 1% AEP storm event.

Table 5.5: Existing Site Peak Discharge Comparison

Storm Event	XP-RAFTS Peak Runoff (m^3/s)	Rational Method Peak Runoff (m^3/s)	Difference	
			+/-	%
39% AEP	0.192	0.252	0.060	24
10% AEP	0.389	0.439	0.050	11
5% AEP	0.505	0.532	0.027	5
2% AEP	0.584	0.686	0.102	15
1% AEP	0.679	0.798	0.119	15

As can be seen above, the Laurenson Routing Method established from the XP-RAFTS model generates peak flows that are generally within 15% to what is estimated by the Rational Method, with the exception of the minor storm event which may be attributed to the Laurenson Method directly accounting for dam and tank storage where as the Rational Method does not. Hence the scenario that has been modelled and applied to the site is considered suitable.

5.3 Developed Conditions

This section of the report will analyse and comment on the developed site stormwater discharge conditions. The XP-RAFTS modelling software, utilising the Laurenson Routing Method will be used to generate the hydrographs and peak flows from the site for all storm events up to an including the 1% AEP storm event.

5.4 Developed Unmitigated Model

5.4.1 Catchment Definition

An examination of the developed site land topography and land use was undertaken to establish the quantity of sub catchments applicable to the proposed development. Three fundamental sub catchments were considered for the site; the impervious roof areas associated with the proposed childcare centre, the hardstand areas which make up the driveway, parking and access paths, and the vegetated/grassed area which facilitate planting and recreational activities. The remaining undeveloped area was also included in the RAFTS model for comparison purposes to existing site conditions.

Since the Laurenson Routing Method is being adopted to estimate the catchment runoff, the ground surface has been further broken down to pervious and impervious sub catchments and Manning's 'n' numbers were adopted for specific surface areas. Refer Appendix A for proposed stormwater management layout and Table 5.6 below for developed site catchment details.

The catchment slope has been estimated from proposed architectural plans.

Table 5.6: Catchment Data Summary Table

Catchment	Sub Catchment	Area (ha)	Impervious (%)	Slope (%)	Manning's 'n'
Roof	Impervious	0.118	100	12	0.015
Driveway	Impervious	0.090	100	3	0.015
	Pervious	0.010	0	3	0.030
Ground	Impervious	0.054	100	3	0.015
	Pervious	0.080	0	3	0.030
Vegetation	Pervious	1.589	0	3	0.050
Ex Roof	Impervious	0.039	100	12	0.015
Ex Drive	Impervious	0.020	100	3	0.015
	Pervious	0.002	0	3	0.030

5.4.2 Mitigation Parameters

Mitigation parameters have been adopted in accordance with Section 5.2.2 of this report.

5.4.3 Rainfall Parameters and Losses

Rainfall Parameters and Losses have been adopted in accordance with Section 5.2.3 of this report.

5.4.4 Developed Hydrological Results

The following table provides a summary of the peak discharge from the developed site that was estimated by the XP-RAFTS model for all storm events up to and including the 1% AEP storm event.

Table 5.7: Developed Site Peak Discharges

Storm Event	XP-RAFTS Peak Runoff (m^3/s)
39% AEP	0.231
10% AEP	0.457
5% AEP	0.551
2% AEP	0.656
1% AEP	0.776

5.4.5 Runoff Comparison

As noted in Section 5.2.5 of this report, a comparison between the XP-RAFTS discharge and Rational Method peak discharge is recommended to be undertaken by *QUDM*.

Based on the methods outlined in *QUDM*, the following parameters were used to estimate the peak runoff from the upstream catchment affecting the site. Refer Appendix C for detailed rational method calculation summary.

Table 5.8: Rational Method Parameters

Area (ha)	Runoff Coefficient (C10)	Time of Concentration (mins)
2.002	0.74	19

It is to be noted that the time of concentration is based on the combination of Standard Inlet Times and Friends Equation.

The following table provides a comparison of results of the Rational Method and XP-RAFTS Model results for peak discharges up to and including the 1% AEP storm event.

Table 5.9: Developed Site Peak Discharge Comparison

Storm Event	XP-RAFTS Peak Runoff (m^3/s)	Rational Method Peak Runoff (m^3/s)	Difference	
			+/-	%
39% AEP	0.231	0.295	0.064	22
10% AEP	0.457	0.514	0.057	11
5% AEP	0.551	0.624	0.073	12
2% AEP	0.656	0.802	0.146	18
1% AEP	0.776	0.925	0.149	16

As can be seen above, the Laurenson Routing Method established from the XP-RAFTS model generates peak flows that are generally within 18% to what is estimated by the Rational Method, with the exception of the minor storm event which may be attributed to the Laurenson Method directly accounting for dam storage where as the Rational Method does not. Hence the scenario that has been modelled and applied to the site is considered suitable.

5.5 Potential Impacts of Development

The following table provides a summary of the peak runoff from the site under both existing and developed scenarios.

Table 5.10: Runoff Comparison

Storm Event	Existing Peak Runoff (m^3/s)	Developed Peak Runoff (m^3/s)	Difference	
			+/-	%
39% AEP	0.192	0.231	0.039	17
10% AEP	0.389	0.457	0.068	15
5% AEP	0.505	0.551	0.046	8
2% AEP	0.584	0.656	0.072	11
1% AEP	0.679	0.776	0.097	13

The proposed development has increased the proportion of the site that is impervious, consequently the runoff characteristics from the site will be altered as a result of the development. As demonstrated above, the development has increased runoff volumes and peak flow rates downstream in comparison to the existing conditions for all storm events. If left unmitigated, the developed runoff could have an adverse effect on downstream properties and/or stormwater infrastructure. It is therefore proposed that an on-site detention system be constructed within the site to mitigate increased peak discharges to below pre-development levels.

5.6 Proposed Mitigated Conditions

This section of the report will specify the mitigation device(s) required to detain the post development flow rate to match existing site runoff conditions. The XP-RAFTS modelling software, utilising the Laurenson Routing Method will be used to generate the hydrographs and peak flows from the mitigated site in conjunction with the design of on-site detention devices for all storm events up to an including the 1% AEP storm event.

5.7 Developed Mitigated Model

Since the site falls to the north west, development area flows are to be directed to the western end of the site whereby a detention basin is to be constructed beneath the proposed driveway turnaround area. The basin will detain flows prior to discharging as sheet flow to the north west of the site via a level spreader as per existing conditions.

All storm events up to and including the 1% AEP storm event have been modelled in the XP-RAFTS software package. A retarding basin has been included in the proposed model to account for the addition of the detention basin.

The following parameters were used to define a stage storage discharge relationship for the basin:

- Minimum Tank depth of 1.20m.
- Total Storage Volume of 144m^3.
- 1 x 140mm low flow orifice at basin invert.
- 1 x 140mm mid flow outlet at 0.47m from basin invert
- 1 x 210mm high flow outlet at basin obvert.

Refer to Appendix B for XP-RAFTS hydrographs and XP-RAFTS stage storage discharge relationship.

The table below is a summary of the peak discharge flows from the site with the proposed detention measure implemented:

Table 5.11: Mitigated Runoff Comparison

Storm Event	Existing Peak Runoff (m^3/s)	Proposed Mitigated Peak Runoff (m^3/s)	Difference	
			+/-	%
39% AEP	0.192	0.190	-0.002	-1
10% AEP	0.389	0.388	-0.001	0
5% AEP	0.505	0.501	-0.004	-1
2% AEP	0.584	0.568	-0.016	-3
1% AEP	0.679	0.677	-0.002	0

All storm events have been mitigated to below pre-developed levels; as such the proposal is considered to satisfy Council's condition of a non-worsening effect in regards to peak stormwater runoff.

6 INTERPRETATION AND CONCLUSIONS

Since the site falls to the north west, all development area flows are to be directed to the western end of the proposed driveway area where a detention basin is to be constructed in order to detain flows prior to discharge. A discharge location for the development shall be obtained by surface and sheet draining basin outflows across the rear of the site via a level spreader.

The proposed development, if unmitigated, would have a negative effect on runoff water quality from the site. Best practices in water quality design have been implemented to improve the quality of the stormwater runoff generated on the site.

The predicted pollutant export load concentrations for the mitigated conditions as determined by the MUSIC model show that the proposed water quality treatment measures comply with Council's mean annual load reduction requirements for total suspended solids (TSS), total phosphorous (TP), total nitrogen (TN) and gross pollutants (GP).

Key elements of the proposed stormwater treatment train are as follows:

- Runoff associated with the proposed access, car parking and playground area is to be treated through 4 source pollutant filters (SPEL Stormsacks or similar) before being piped to proposed underground detention tank within the driveway (see Section 5)
- Runoff associated with proposed roof areas to be piped to the proposed underground detention tank (see Section 5) which is to be fitted with 5. SPELFilter cartridges.
- Treated and detained site flows to be discharged towards the rear of the site via level spreader, sheet draining as per existing conditions.

An analysis of the pre and post development site discharge has been undertaken which has revealed that the development has increased runoff volumes and peak flow rates downstream in comparison to the existing conditions for all storm events. As a result, if left unmitigated, the developed runoff could potentially have an adverse effect on downstream properties and/or stormwater infrastructure.

It is therefore proposed that a 144m^3 on-site detention system be constructed at the western end of the proposed driveway area to mitigate increased peak discharges to below pre-development levels.

Refer Appendix A for the proposed stormwater management layout.

APPENDIX A: Proposed Development Layout

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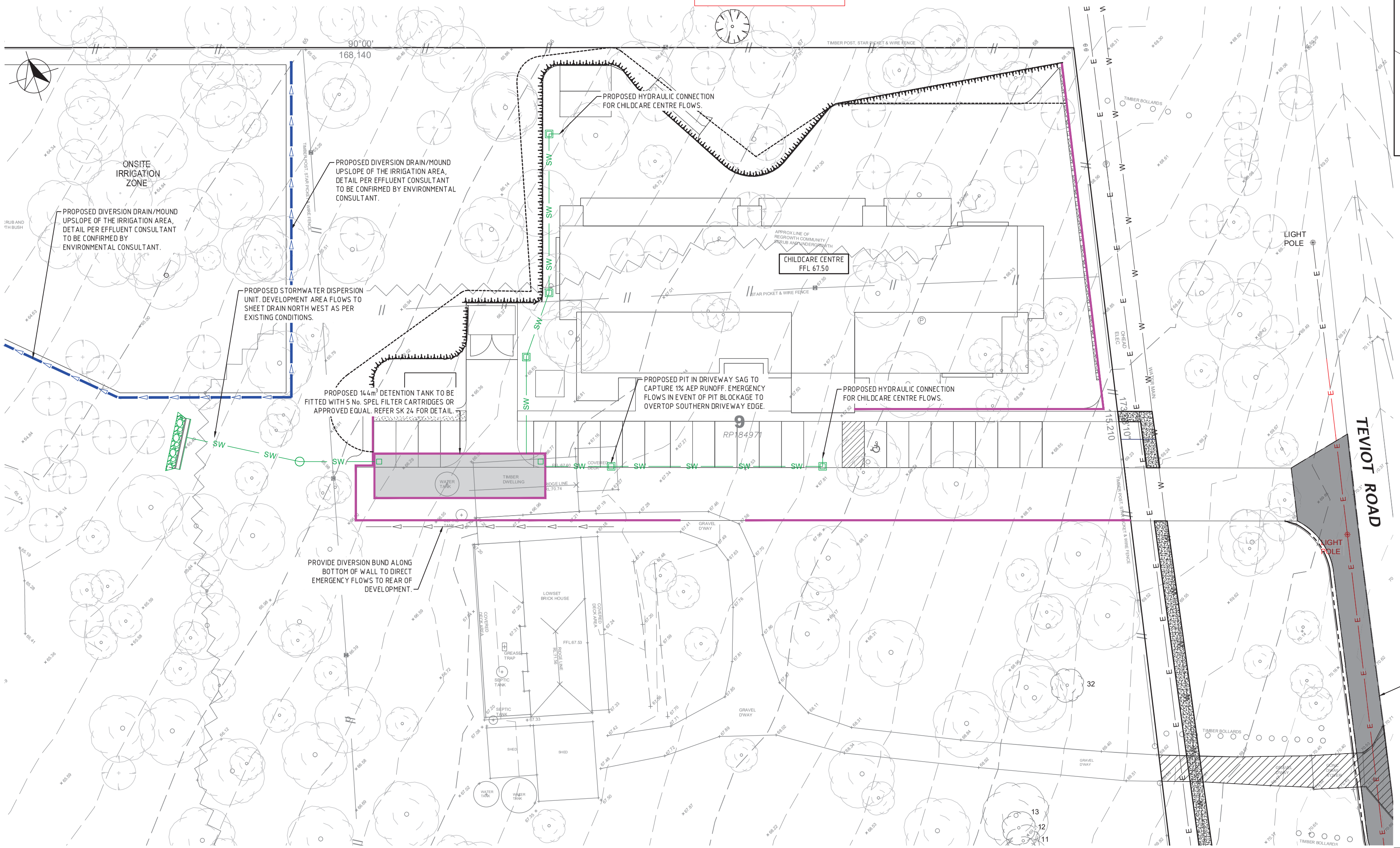

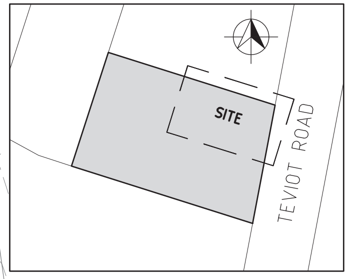

LOCALITY PLAN
SCALE 1:2500

LEGEND

E	EXISTING ELECTRICAL UNDERGROUND
OH	EXISTING ELECTRICAL OVERHEAD
W	EXISTING WATER MAIN
9.999	EXISTING MAJOR CONTOURS
SW	PROPOSED STORMWATER DRAINAGE
	TEMPORARY DIVERSION DRAIN

NOTES:
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PLAN
SCALE 1:250

SCALE 1:250 (A1) 0 5 10 15m (SCALE BEFORE REDUCTION)

SCALE 1:2500 (A1) 0 50 100 150m (SCALE BEFORE REDUCTION)

REAL PROPERTY DESCRIPTION
LOT 9 ON RP184971

REV.	DESCRIPTION	DATE	INIT.
A	PRELIMINARY ISSUE	03.05.19	AW
B	APPROVAL ISSUE	05.07.19	DH
C	RFI ISSUE	03.02.20	CA
D	RFI ISSUE	18.06.20	SG
E	FURTHER ISSUES RESPONSE	02.11.20	CA

MILANOVIC NEALE CONSULTING ENGINEERS

CIVIL
STRUCTURAL
TRAFFIC
PROJECT MANAGEMENT

BRISBANE PH No. (07) 3255 1877
IPSWICH PH No. (07) 3281 6603
SYDNEY PH No. 1300 827 901
GOLD COAST PH No. 1300 827 901

CONTACT DETAILS
E mail@mnce.com.au
W www.mnce.com.au

CLIENT
N & A CAMPBELL DEVELOPMENTS PTY LTD

PROJECT
221 TEVIOT ROAD, GREENBANK

TITLE
STORMWATER MANAGEMENT LAYOUT PLAN

DRAWN	DESIGNED	DATE
AW	CFR	APR 2019
CHECKED	APPROVED	
DH		
DRAWING No.		REV.
C3301 - SK21		E

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PLAN
SCALE 1:500

LEGEND

10.000 — EXISTING MAJOR CONTOURS

ROOF CATCHMENT
A = 0.039 ha
I = 100%
S =12%
n = 0.015

HARDSTAND CATCHMENT
A = 0.038 ha
I = 90%
S =3%
n(I) = 0.015
n(P) = 0.030

VEGETATED CATCHMENT
A = 1.796 ha
I = 0%
S = 3%
n = 0.05

ROOF CATCHMENT
A = 0.010 ha
I = 100%
S =12%
n = 0.015

DAMCATCHMENT
A = 0.119 ha
I = 0%
S =3%
n = 0.05

NOTES:
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SCALE 1:500 (A1) 0 10 20 30m
(SCALE BEFORE REDUCTION)

REAL PROPERTY DESCRIPTION
LOT 9 ON RP184971

REV.	DESCRIPTION	DATE	INIT.
A	PRELIMINARY ISSUE	03.05.19	AW
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BRISBANE PH No. (07) 3255 1877
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E mail@mnce.com.au
W www.mnce.com.au

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PROJECT
221 TEVIOT ROAD, GREENBANK

TITLE
STORMWATER MANAGEMENT EXISTING CATCHMENT LAYOUT PLAN

DRAWN	DESIGNED	DATE
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CHECKED	APPROVED	
DH		

DRAWING No. C3301 – SK22
REV. D

DO NOT SCALE FROM DRAWING
ALL SCALES ARE AS SHOWN (A1)

NOT FOR CONSTRUCTION

PLAN
SCALE 1:500

LEGEND

- SW — PROPOSED STORMWATER DRAINAGE
- 10.000 — EXISTING MAJOR CONTOURS
- ROOF CATCHMENT: A = 0.0118 ha, I = 100%, S = 12%, n = 0.015
- HARDSTAND CATCHMENT: A = 0.100 ha, I = 90%, n(I) = 0.015, n(P) = 0.030
- VEGETATED CATCHMENT: A = 1.650 ha, I = 0%, n = 0.05
- GROUND CATCHMENT: A = 0.134 ha, I = 40%, n(I) = 0.015, n(P) = 0.030
- HARDSTAND CATCHMENT: A = 0.022 ha, I = 90%, S = 3%, n(I) = 0.015, n(P) = 0.030
- ROOF CATCHMENT: A = 0.039 ha, I = 100%, S = 12%, n = 0.015
- DAM CATCHMENT: A = 0.119 ha, I = 0%, S = 3%, n = 0.05

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SCALE 1:500 (A1) 0 10 20 30m
(SCALE BEFORE REDUCTION)

REAL PROPERTY DESCRIPTION
LOT 9 ON RP184971

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BRISBANE PH No. (07) 3255 1877
IPSWICH PH No. (07) 3281 6603
SYDNEY PH No. 1300 827 901
GOLD COAST PH No. 1300 827 901

CONTACT DETAILS
E mail@mnce.com.au
W www.mnce.com.au

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PROJECT
221 TEVIOT ROAD, GREENBANK

TITLE
STORMWATER MANAGEMENT DEVELOPED CATCHMENT LAYOUT PLAN

DRAWN	DESIGNED	DATE
AW	CFR	APR 2019
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DH		

DRAWING No. C3301 – SK23 REV. E

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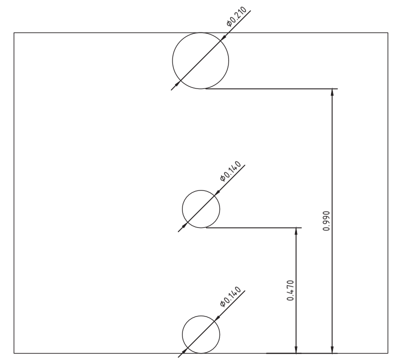
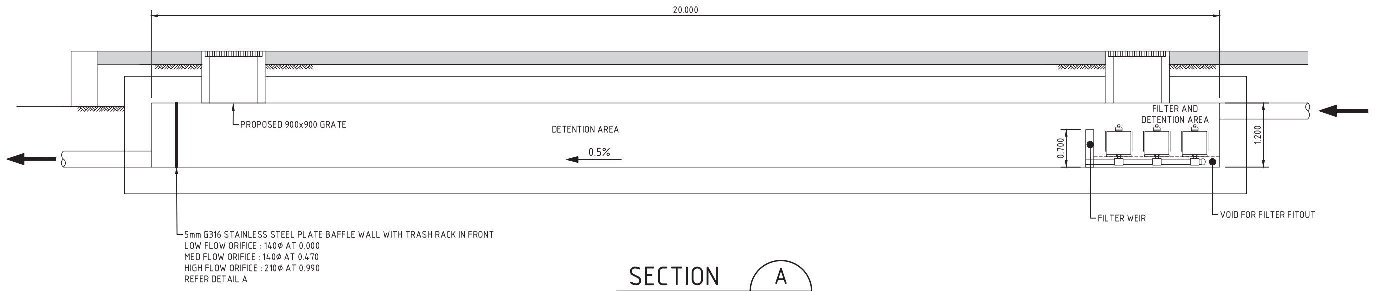
DETENTION TANK DETAIL
SCALE 1:50



DETAIL 'A'
SCALE 1:10



SECTION A -
SCALE 1:50

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SCALE 1:10 (A1) 0 200 400 600mm (SCALE BEFORE REDUCTION)
SCALE 1:50 (A1) 0 1 2 3m (SCALE BEFORE REDUCTION)

REAL PROPERTY DESCRIPTION
LOT 9 ON RP184971

REV.	DESCRIPTION	DATE	INIT.
A	FURTHER ISSUES RESPONSE	02.11.20	CA

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E mail@mnce.com.au
W www.mnce.com.au

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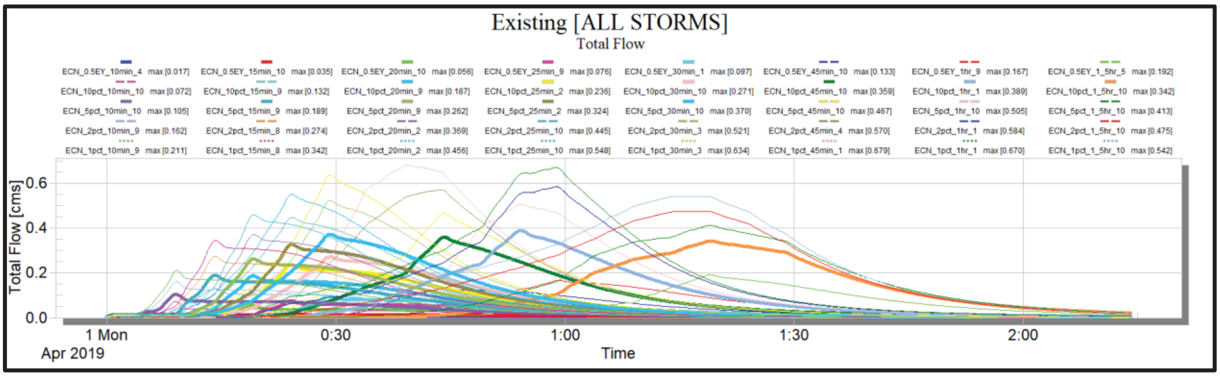
221 TEVIOT ROAD, GREENBANK

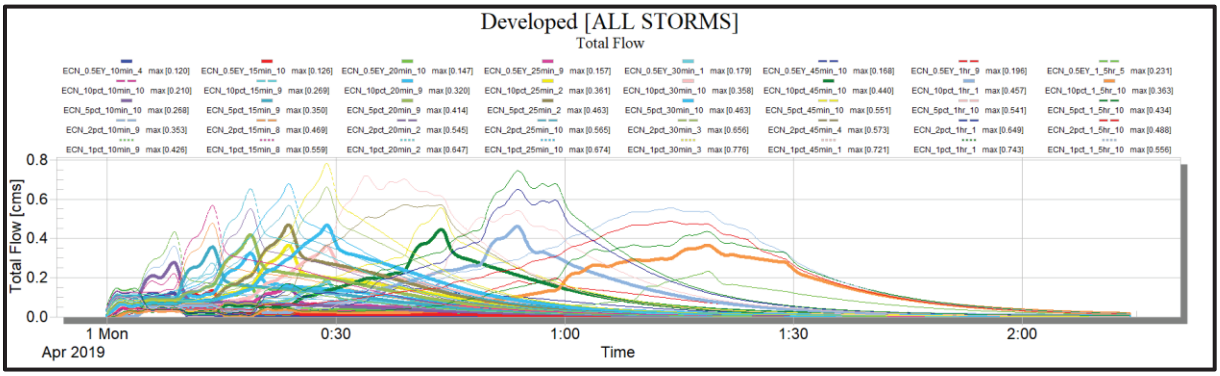
TITLE
STORMWATER DETAILS

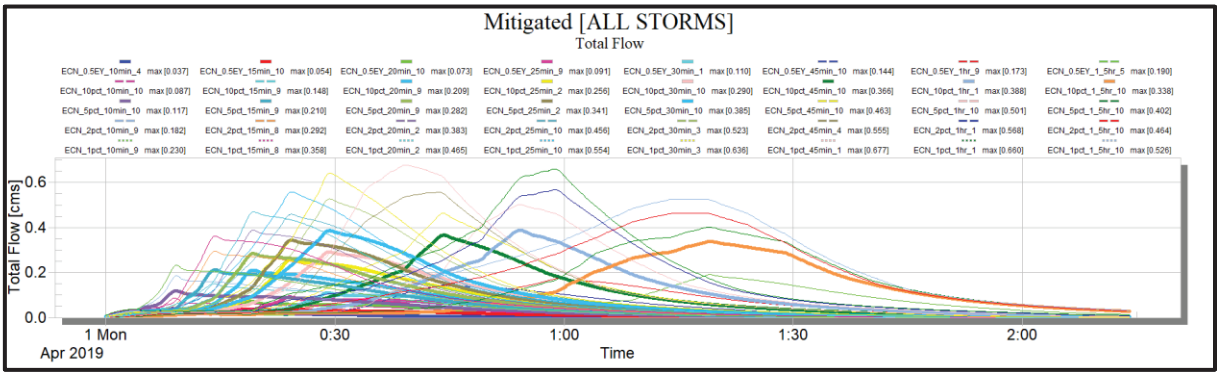
DRAWN	DESIGNED	DATE
CA	RDU	OCT 2020
CHECKED	APPROVED	
DH		

DRAWING No. C3301 - SK24
REV. A

APPENDIX B: XP-RAFTS Hydrographs and Stage Storage Relationship


Existing [ALL STORMS]
Total Flow
Total Flow [cms]
0.0
0.2
0.4
0.6
1 Mon
Apr 2019
0:30
1:00
1:30
2:00
Time


Developed [ALL STORMS]
Total Flow
Total Flow [cms]
0.0
0.2
0.4
0.6
0.8
1 Mon
Apr 2019
0:30
1:00
1:30
2:00
Time


Mitigated [ALL STORMS]
Total Flow
Total Flow [cms]
0.0
0.2
0.4
0.6
1 Mon
Apr 2019
0:30
1:00
1:30
2:00
Time

Existing Dam

Orifice Details								
Low Flow		*Mid Flow*			High Flow Conditions	Weir	Number	1
Size (m)		Size (m)		Number	Length (m)	7.35	Height (m)	0.75
		Height (m)						

						Low Flow Details			Mid Flow Details			High Flow Details			Stage Storage Discharge Table		
Basin Level	S.A.	Ave S.A.	Thickness	Volume	Cumulative Volume	Basin Level	Head	$Q_{Orifice}$	Basin Level	Head	$Q_{Orifice}$	Basin Leve	Head	Qweir	Basin Level	Volume	Outflow
(m)	*(m^2)*	*(m^2)*	*(m)*	*(m^3)*	*(m^3)*	*(m)*	*(m)*	*(m^3/s)*	*(m)*	*(m)*	*(m^3/s)*	*(m)*	*(m)*	*(m^3/s)*	*(m)*	*(m^3)*	*(m^3)*
0.000	67.521	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
0.100	178.338	122.930	0.100	12.293	15.097	0.100	0.000	0.000	0.100	0.000	0.000	0.100	0.000	0.000	0.100	15.097	0.000
0.200	198.790	188.564	0.100	18.856	33.946	0.200	0.100	0.000	0.200	0.100	0.000	0.200	0.000	0.000	0.200	33.946	0.000
0.300	220.211	209.501	0.100	20.950	54.888	0.300	0.200	0.000	0.300	0.200	0.000	0.300	0.000	0.000	0.300	54.888	0.000
0.400	242.601	231.406	0.100	23.141	78.020	0.400	0.300	0.000	0.400	0.300	0.000	0.400	0.000	0.000	0.400	78.020	0.000
0.500	265.960	254.281	0.100	25.428	103.440	0.500	0.400	0.000	0.500	0.400	0.000	0.500	0.000	0.000	0.500	103.440	0.000
0.600	290.288	278.124	0.100	27.812	131.245	0.600	0.500	0.000	0.600	0.500	0.000	0.600	0.000	0.000	0.600	131.245	0.000
0.700	315.585	302.937	0.100	30.294	161.530	0.700	0.600	0.000	0.700	0.600	0.000	0.700	0.000	0.000	0.700	161.530	0.000
0.800	340.725	328.155	0.100	32.816	194.381	0.800	0.700	0.000	0.800	0.700	0.000	0.800	0.100	0.388	0.800	194.381	0.388

Existing Rainwater Tank

Orifice Details								
Low Flow		*Mid Flow*			High Flow Conditions	Orifice	Number	1
Size (m)		Size (m)		Number	Size (m)	0.1	Height (m)	0.37
		Height (m)						

						Low Flow Details			Mid Flow Details			High Flow Details			Stage Storage Discharge Table		
Basin Level	S.A.	Ave S.A.	Thickness	Volume	Cumulative Volume	Basin Level	Head	$Q_{Orifice}$	Basin Level	Head	$Q_{Orifice}$	Basin Leve	Head	Qorifice	Basin Level	Volume	Outflow
(m)	*(m^2)*	*(m^2)*	*(m)*	*(m^3)*	*(m^3)*	*(m)*	*(m)*	*(m^3/s)*	*(m)*	*(m)*	*(m^3/s)*	*(m)*	*(m)*	*(m^3/s)*	*(m)*	*(m^3)*	*(m^3)*
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
0.028	5.308	2.654	0.028	0.074	0.074	0.028	0.028	0.000	0.028	0.028	0.000	0.028	0.000	0.000	0.028	0.074	0.000
0.055	5.308	5.308	0.028	0.147	0.221	0.055	0.055	0.000	0.055	0.055	0.000	0.055	0.000	0.000	0.055	0.221	0.000
0.083	5.308	5.308	0.028	0.147	0.368	0.083	0.083	0.000	0.083	0.083	0.000	0.083	0.000	0.000	0.083	0.368	0.000
0.111	5.308	5.308	0.028	0.147	0.515	0.111	0.111	0.000	0.111	0.111	0.000	0.111	0.000	0.000	0.111	0.515	0.000
0.139	5.308	5.308	0.028	0.147	0.662	0.139	0.139	0.000	0.139	0.139	0.000	0.139	0.000	0.000	0.139	0.662	0.000
0.166	5.308	5.308	0.028	0.147	0.809	0.166	0.166	0.000	0.166	0.166	0.000	0.166	0.000	0.000	0.166	0.809	0.000
0.194	5.308	5.308	0.028	0.147	0.956	0.194	0.194	0.000	0.194	0.194	0.000	0.194	0.000	0.000	0.194	0.956	0.000
0.222	5.308	5.308	0.028	0.147	1.103	0.222	0.222	0.000	0.222	0.222	0.000	0.222	0.000	0.000	0.222	1.103	0.000
0.249	5.308	5.308	0.028	0.147	1.250	0.249	0.249	0.000	0.249	0.249	0.000	0.249	0.000	0.000	0.249	1.250	0.000
0.277	5.308	5.308	0.028	0.147	1.397	0.277	0.277	0.000	0.277	0.277	0.000	0.277	0.000	0.000	0.277	1.397	0.000
0.305	5.308	5.308	0.028	0.147	1.544	0.305	0.305	0.000	0.305	0.305	0.000	0.305	0.000	0.000	0.305	1.544	0.000
0.332	5.308	5.308	0.028	0.147	1.691	0.332	0.332	0.000	0.332	0.332	0.000	0.332	0.000	0.000	0.332	1.691	0.000
0.360	5.308	5.308	0.028	0.147	1.838	0.360	0.360	0.000	0.360	0.360	0.000	0.360	0.000	0.000	0.360	1.838	0.000
0.388	5.308	5.308	0.028	0.147	1.985	0.388	0.388	0.000	0.388	0.388	0.000	0.388	0.000	0.000	0.388	1.985	0.000
0.416	5.308	5.308	0.028	0.147	2.132	0.416	0.416	0.000	0.416	0.416	0.000	0.416	0.000	0.000	0.416	2.132	0.000
0.443	5.308	5.308	0.028	0.147	2.279	0.443	0.443	0.000	0.443	0.443	0.000	0.443	0.023	0.001	0.443	2.279	0.001

Detention Tank Design

Orifice Details									
Low Flow		*Mid Flow*				High Flow Conditions	Orifice	Number	1
Size (m)	0.14	Size (m)	0.14	Number	1	Size (m)	0.21	Height (m)	0.99
		Height (m)	0.47						

						Low Flow Details			Mid Flow Details			High Flow Details			Stage Storage Discharge Table		
Basin Level	S.A.	Ave S.A.	Thickness	Volume	Cumulative Volume	Basin Level	Head	$Q_{Orifice}$	Basin Level	Head	$Q_{Orifice}$	Basin Leve	Head	Qorifice	Basin Level	Volume	Outflow
(m)	*(m^2)*	*(m^2)*	*(m)*	*(m^3)*	*(m^3)*	*(m)*	*(m)*	*(m^3/s)*	*(m)*	*(m)*	*(m^3/s)*	*(m)*	*(m)*	*(m^3/s)*	*(m)*	*(m^3)*	*(m^3)*
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
0.070	120.000	60.000	0.070	4.200	4.200	0.070	0.000	0.000	0.070	0.000	0.000	0.070	0.000	0.000	0.070	4.200	0.000
0.140	120.000	120.000	0.070	8.400	12.600	0.140	0.070	0.012	0.140	0.000	0.000	0.140	0.000	0.000	0.140	12.600	0.012
0.210	120.000	120.000	0.070	8.400	21.000	0.210	0.140	0.017	0.210	0.000	0.000	0.210	0.000	0.000	0.210	21.000	0.017
0.280	120.000	120.000	0.070	8.400	29.400	0.280	0.210	0.021	0.280	0.000	0.000	0.280	0.000	0.000	0.280	29.400	0.021
0.350	120.000	120.000	0.070	8.400	37.800	0.350	0.280	0.024	0.350	0.000	0.000	0.350	0.000	0.000	0.350	37.800	0.024
0.420	120.000	120.000	0.070	8.400	46.200	0.420	0.350	0.027	0.420	0.000	0.000	0.420	0.000	0.000	0.420	46.200	0.027
0.470	120.000	120.000	0.050	6.000	52.200	0.470	0.400	0.029	0.470	0.000	0.000	0.470	0.000	0.000	0.470	52.200	0.029
0.540	120.000	120.000	0.070	8.400	60.600	0.540	0.470	0.031	0.540	0.000	0.000	0.540	0.000	0.000	0.540	60.600	0.031
0.630	120.000	120.000	0.090	10.800	71.400	0.630	0.560	0.034	0.630	0.090	0.014	0.630	0.000	0.000	0.630	71.400	0.048
0.700	120.000	120.000	0.070	8.400	79.800	0.700	0.630	0.036	0.700	0.160	0.018	0.700	0.000	0.000	0.700	79.800	0.055
0.770	120.000	120.000	0.070	8.400	88.200	0.770	0.700	0.038	0.770	0.230	0.022	0.770	0.000	0.000	0.770	88.200	0.060
0.840	120.000	120.000	0.070	8.400	96.600	0.840	0.770	0.040	0.840	0.300	0.025	0.840	0.000	0.000	0.840	96.600	0.065
0.910	120.000	120.000	0.070	8.400	105.000	0.910	0.840	0.042	0.910	0.370	0.028	0.910	0.000	0.000	0.910	105.000	0.070
0.980	120.000	120.000	0.070	8.400	113.400	0.980	0.910	0.044	0.980	0.440	0.030	0.980	0.000	0.000	0.980	113.400	0.074
1.050	120.000	120.000	0.070	8.400	121.800	1.050	0.980	0.045	1.050	0.510	0.033	1.050	0.000	0.000	1.050	121.800	0.078
1.120	120.000	120.000	0.070	8.400	130.200	1.120	1.050	0.047	1.120	0.580	0.035	1.120	0.025	0.001	1.120	130.200	0.083
1.190	120.000	120.000	0.070	8.400	138.600	1.190	1.120	0.048	1.190	0.650	0.037	1.190	0.095	0.010	1.190	138.600	0.095
1.260	120.000	120.000	0.070	8.400	147.000	1.260	1.190	0.050	1.260	0.720	0.039	1.260	0.165	0.024	1.260	147.000	0.112
1.330	120.000	120.000	0.070	8.400	155.400	1.330	1.260	0.051	1.330	0.790	0.041	1.330	0.235	0.040	1.330	155.400	0.132

APPENDIX C: Rational Method Calculation Summary

RATIONAL METHOD CALCULATIONS - EXISTING SITE

Job Reference	C3301
Site Address	221 Teviot Road, Greenbank
Council	Logan City Council

Number of Sub-Catchments	3	
Minor Storm Event	10% AEP	*(As per QUDM Table 7.02.1)*
Major Storm Event	2% AEP	*(As per QUDM Table 7.02.1)*

Subcatchment Summary Table

Number	*Catchment Name*	*Catchment Description*	C_{10}	*tc*
1	Roof	Impervious Roof	0.90	25
2	Driveway	Significant paved areas	0.88	25
3	Vegetation	Open Space (eg parks)	0.70	25

Site C_{10} 0.71

Catchment Calculations (Major and Minor Storm ARI's)

Number	*Area*	*C10*	*I10*	*10% AEP*	*C50*	*I50*	*2% AEP*
	ha		*mm/hr*	*m ^{3}/s*		*mm/hr*	*m^3/s*
1	0.049	0.90	111	0.014	1.00	152	0.021
2	0.038	0.88	111	0.010	1.00	152	0.016
3	1.915	0.70	111	0.415	0.81	152	0.649

Total Runoff	Minor	0.439	*m^3/s*
	Major	0.686	*m^3/s*
Total Area		2.002	*ha*

Overland Flow Calculations

Trunk SW Infrastructure		
Pipe Diameter	N/A	*m*
Number of Pipes		
Grade		*m/m*
mannings		
Pipe Capacity		*m^3/s*
Pipe Velocity		*m/s*
Capacity @ 3m/s		*m^3/s*
Overland Flow		*m^3/s*

Runoff Summary

Freq.	*Peak Discharge*	
4 EY	0.094	*m^3/s*
63% AEP	0.188	*m^3/s*
39% AEP	0.252	*m^3/s*
15% AEP	0.351	*m^3/s*
10% AEP	0.439	*m^3/s*
5% AEP	0.532	*m^3/s*
2% AEP	0.686	*m^3/s*
1% AEP	0.798	*m^3/s*

RATIONAL METHOD CALCULATIONS - DEVELOPED SITE

Job Reference	C3301
Site Address	221 Teviot Road, Greenbank
Council	Logan City Council

Number of Sub-Catchments	6	
Minor Storm Event	10% AEP	*(As per QUDM Table 7.02.1)*
Major Storm Event	2% AEP	*(As per QUDM Table 7.02.1)*

Subcatchment Summary Table

Number	*Catchment Name*	*Catchment Description*	*C_{10}*	*tc*
1	Roof	Impervious Roof	0.90	19
2	Driveway	Significant paved areas	0.88	19
3	Vegetation	Open Space (eg parks)	0.70	19
4	Ground	Low-medium to high density residential areas	0.87	19
5	EX Roof	Impervious Roof	0.90	19
6	EX Drive	Significant paved areas	0.88	19

Site C_{10} 0.74

Catchment Calculations (Major and Minor Storm ARI's)

Number	*Area*	*C10*	*I10*	*10% AEP*	*C50*	*I50*	*2% AEP*
	ha		*mm/hr*	*m^3/s*		*mm/hr*	*m^3/s*
1	0.118	0.90	125	0.037	1.00	171	0.056
2	0.100	0.88	125	0.031	1.00	171	0.047
3	1.589	0.70	125	0.387	0.81	171	0.606
4	0.134	0.87	125	0.041	1.00	171	0.064
5	0.039	0.90	125	0.012	1.00	171	0.018
6	0.022	0.88	125	0.007	1.00	171	0.010

Total Runoff	Minor	0.514	*m^3/s*
	Major	0.802	*m^3/s*
Total Area		2.002	*ha*

Overland Flow Calculations

Trunk SW Infrastructure

Pipe Diameter	N/A	*m*
Number of Pipes		
Grade		*m/m*
mannings		
Pipe Capacity		*m^3/s*
Pipe Velocity		*m/s*
Capacity @ 3m/s		*m^3/s*
Overland Flow		*m^3/s*

Runoff Summary

Freq.	*Peak Discharge*	
4 EY	0.110	*m^3/s*
63% AEP	0.220	*m^3/s*
39% AEP	0.295	*m^3/s*
15% AEP	0.411	*m^3/s*
10% AEP	0.514	*m^3/s*
5% AEP	0.624	*m^3/s*
2% AEP	0.802	*m^3/s*
1% AEP	0.925	*m^3/s*