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MYALL RIVER DOWNS PTY LTD

Myall River Downs

Water Management Report

301015-01753 – EN-TEN-0001

21-Jul-11

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MYALL RIVER DOWNS

WATER MANAGEMENT REPORTWATER MANAGEMENT REPORT

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PROJECT 301015-01753 - MYALL RIVER DOWNS

REV	DESCRIPTION	ORIG	REVIEW	WORLEY-PARSONS APPROVAL	DATE	CLIENT APPROVAL	DATE
D	Revised for client review	FC	MT	MT	7-Sep-10	N/A	
E	Revised	FC	MT	MT	9-Sep-10		N/A
F	Revised	FC	PT	PT	7-Apr-11		N/A
0	Final	FC	PT	PT	21-Jul-11		26-7-11

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Appendices

EXECUTIVE SUMMARY

E1 Introduction

WorleyParsons, assisted by Martens Consulting Engineers, were engaged by Myall River Downs Pty Ltd to prepare a water management report as part of a Local Environmental Study (LES) for a proposed residential and industrial zoning of the site.

This report has been reviewed extensively by WBM BMT who are the consultants for Great Lakes Shire Council. The Council is preparing the LES.

Advice was sought from the NSW Office of Water regarding management of groundwater on the site. Their key requirement was that the development should not have a significant adverse impact on groundwater quality or flows discharging to the wetlands on the site boundary. The quality of runoff from the development entering wetlands or channels intersecting with groundwater, should be equal to or better than the quality of the ambient groundwater. The flows discharging across the site boundary both above ground (surface water) and below ground (groundwater) should not be significantly altered to avoid an impact on wetland ecology. Council and their consultant WBM BMT are supportive of these criteria.

In August 2010, the Department of Planning issued the guideline “Adapting to Sea Level Rise” which recommends adoption of a 0.4 m and 0.9 m rise in sea level to 2050 and 2100 respectively and a planning approach which considers the merits of locating development in risk zones identified between 2050 and 2100.

E2 Proposed Development

The site has an area of approximately 460 ha and is generally flat with existing ground levels varying from RL 0 m to 3 m AHD. It drains partly to the west (Kore Kore Creek) and to the south (Pindimar Bay). Wetlands and mangroves fringe the areas in which stormwater from the site discharges to the above receiving waters.

The sandy soils over the site result in the groundwater responding relatively quickly to rainfall. The groundwater has a naturally high nutrient concentration and the flows generally mimic the fall of the site and direction of stormwater flows.

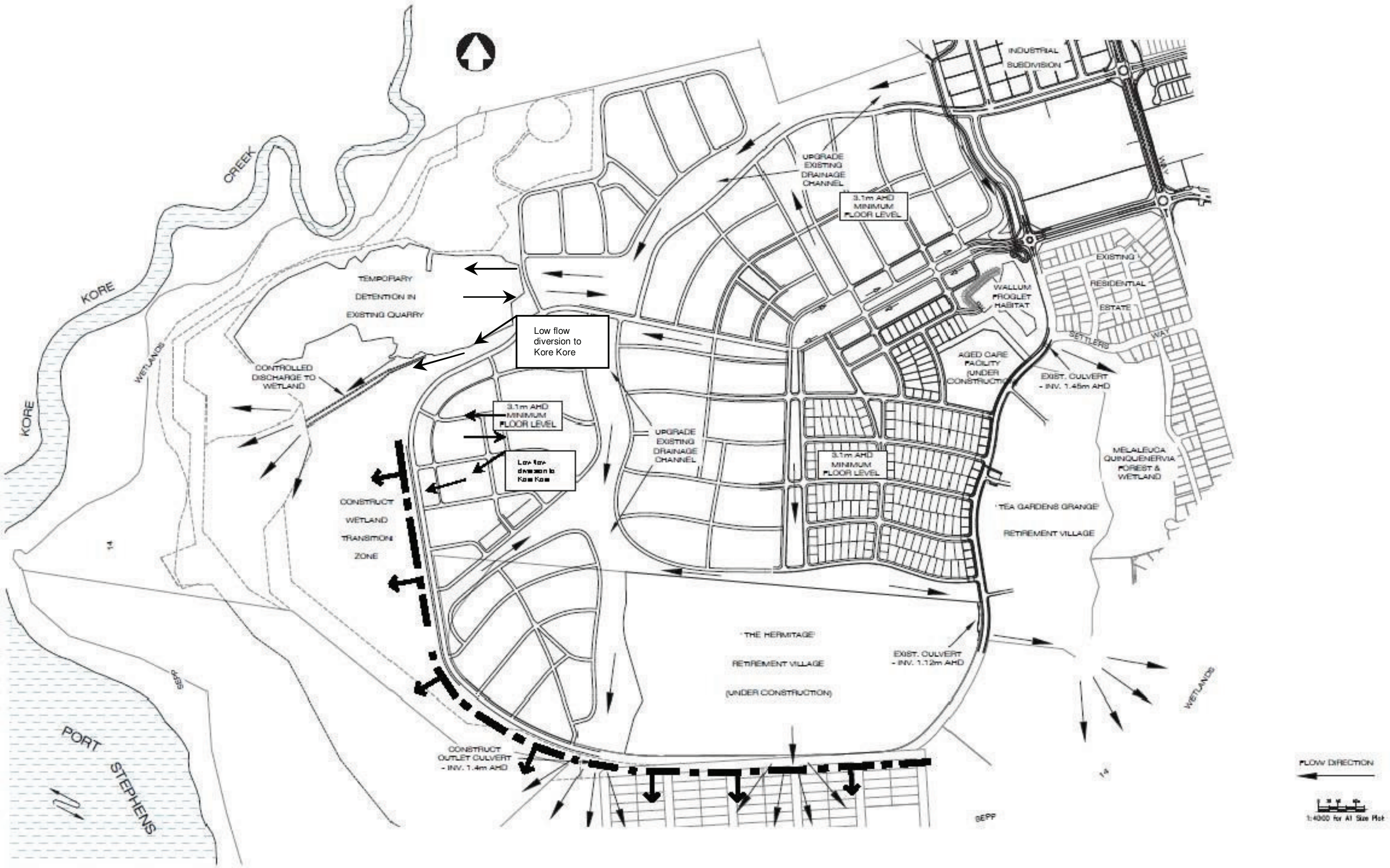
The proposed development and stormwater drainage concept plan is presented in **Figure E1**. The proposed stormwater management system has been formulated to:

- Minimise the quantity of fill required over the site;
- Provide adequate grade to drain runoff;
- Accommodate flows from the external catchment;

- Accommodate 100yr ARI flows in the proposed drainage channels;
- Accommodate a 0.9 m rise in sea level by 2100;
- Maintain the distribution of stormwater flows in accordance with existing conditions;
- Not significantly change to groundwater flow or quality; and
- Have a minimum invert level of the drainage channels at RL 1.4 m AHD.

The minimum channel invert level has been adopted to restrict the incursion of seawater to above the mean high tide level after a 0.9 m sea level rise.

Figure E1 – Stormwater Drainage Concept Plan



A system of primary and secondary drainage channels have been incorporated into the development (see **Figure E2**, Water Quality Management Concept). The secondary drainage channels will drain to the primary drainage channel running generally north-south (see **Figure E2**). These secondary channels reduce the travel distance for drainage pipes and hence reduce the ground surface level required throughout the development.

Parts of both the primary and secondary channels will intersect with the groundwater. In the higher parts of these channels (both on the banks and in the invert where possible), bioretention areas will be incorporated to treat runoff sufficiently to have a water quality better than the groundwater. For instance, Martens Consulting Engineers (**Appendix 5**) has identified that groundwater has typical nutrient concentrations around 8.5 mg/L TN and 2.4 mg/L TP.

A variety of landscape treatments including wetlands have been proposed in the lower sections of these channels in order to achieve a runoff pollutant load reduction of 80% for suspended solids, and 45% for TP and TN at the discharge point from the site.

Linear swales and/or wetlands located outside the perimeter road are proposed to disperse flows. This infrastructure will direct flows into stormwater dissipation zones with the purpose of reducing runoff volumes. Some areas will be revegetated to aid in the evapotranspiration of runoff from the development.

E3 Surface Water Management

The proposed drainage concept is presented on **Figure E2** and as discussed in **Section E2**, it will consist of a variety of WSUD elements. These elements will consist of bioretention swales and basins to treat runoff prior to discharge into waterbodies connected to the groundwater. These will be located as required along some roads, and in the secondary and primary drainage channels. The actual layout of these facilities would be finalised in the DA phase based on achieving the target of matching, if not better, water quality than the groundwater.

Wetlands would be incorporated within the drainage channels to achieve further treatment to achieve the adopted reductions in pollutant load – 80% for suspended solids and 45% for nutrients.

It was found from MUSIC modelling that inclusion of 280 m^2 of bioretention swale/basin and 140 m^2 of wetland for every hectare of development would achieve compliance with both the above water quality targets. This represents an area of WSUD facilities of approximately 4.2% of the development area which is considered reasonable and practical.

Achievement of these outcomes with the proposed drainage concept would ensure no significant impacts on the quality of groundwater and the receiving waters. From previous work and experience in the Tea Gardens area, it is likely that the use of recycled water for outside use on the lots would require an enlargement of the WSUD facilities by approximately 10% which is readily achievable and practical.

E4 Groundwater

The groundwater monitoring and modelling undertaken by Martens Consulting Engineers (refer **Appendices 5, 6 and 7**) for the proposed landform, WSUD facilities and drainage concept indicates that:

- The infiltration areas work to mitigate the introduction of impermeable areas introduced by the development;
- Groundwater is intercepted by the linear wetlands;
- An increase in flows discharging the site results due to a reduction in the potential evapotranspiration within the development, but can be mitigated using the linear swales and/or wetlands outside the perimeter road and associated dissipation areas.
- There will be sufficient freeboard between the proposed finished ground levels and the groundwater levels;
- The area and distribution of infiltration of runoff into the groundwater proposed in the development will maintain the existing flow pattern and volumes;
- The groundwater naturally has high nutrient concentrations of typically around 8.5 mg/L TN and 2.4 mg/L TP;
- The development will not significantly change the nutrient fluxes delivered to the wetlands in groundwater and hence will not adversely impact on the fringing wetlands/mangroves;
- The developments will not significantly change the pH of the groundwater and hence will not adversely impact on the fringing wetlands/mangroves; and
- Use of recycled water for irrigation of lawns/gardens on lots would not adversely impact on the groundwater quality as the volume applied would represent substantially less than the typical nutrient demand and the nutrient quality of the recycled water will be typically better than for the groundwater.

E5 Flooding/Climate Change

The final landform and channel dimensions have been designed to accommodate the 100yr ARI flood in the channel with a 0.9 m sea level rise.

With a minimum stormwater pipe outlet invert level of RL 1.4 m AHD (minimum base level of primary and secondary channels), the predicted 100yr ARI flood level at the downstream end of the development would be RL 2.6 m AHD. This is the absolute minimum level adopted for the main road (evacuation route) and for the ground surface in the lots. The minimum habitable floor level would be RL 3.1 m AHD with a pier foundation and suspended floor level at RL 3.1 m AHD including a 500 mm freeboard (see **Figures E2 and E3**).

The potential for a minimum lot ground level equivalent to the 20yr ARI (approximately RL 2.3 m AHD) with an alternative house construction with pier foundation and suspended floor at RL 3.1 m AHD could be considered and may be appropriate for some of the western residential areas as identified in **Figure E3**.

The 100 year ARI flood level of RL 2.6 m AHD is consistent over the site as it is driven mainly by the sea level. Only in the higher north eastern corner of the site do the flood levels raise above this level.

Sensitivity testing for climate change induced increased rainfall intensity (up to 30% increase) only caused less than a 100 mm increase in the 100yr ARI flood levels. This could be readily accommodated in the freeboard allowance.

The mean high water level around the perimeter of the development would rise to approximately RL 1.4 m AHD if the sea level rises by 0.9 m. An absolute minimum level of the primary drainage channels has been set at RL 1.4 m AHD (refer **Figure E3)** to restrict the incursion of sea water into the drainage channels and pipework. The primary drainage channel will have an invert level of RL 1.4 m AHD along its length up to the vicinity of the quarry. Secondary drainage channels will connect to the primary channel at a minimum of RL 1.4 m AHD and rise in level in an upstream direction.

The groundwater response to a 0.9 m sea level rise was modelled by Martens Consulting Engineers (refer **Appendix 5**). It was found that other than for a small area around the perimeter of the site there was little change in the groundwater level. This was expected as the groundwater levels react primarily to the inflows from the large catchment and any tailwater effects are limited to an area close to the foreshore. The combination of proposed ground surface levels and limited rise of groundwater levels will provide sufficient freeboard for the effective operation of the bioretention WSUD facilities proposed in the development both now and into the future (allowing for climate change).

The fill quantity required for the proposed development has been reduced markedly from the estimated 900,000 m^3 in the previous LES prepared by Parsons Brinkerhoff to an estimated 230,000 m^3 (see **Figure E4**). This has been achieved even with a 0.9 m sea level rise by careful design incorporating secondary drainage channels with smart design of levels and WSUD facilities. This fill volume could be further reduced with smart design of other than slab on ground residential construction. As such, the fill quantity estimate is a worst case scenario. This will be considered at the next stage of the project formulation. This approach to reduce the fill quantity would have other benefits by allowing the area of the fill source (western part of site) to be sculptured more extensively to provide a range of vegetation communities rather than a monoculture and significantly improve habitats for a more comprehensive environmental outcome.

Figure E2 – Water Quality Management Concept

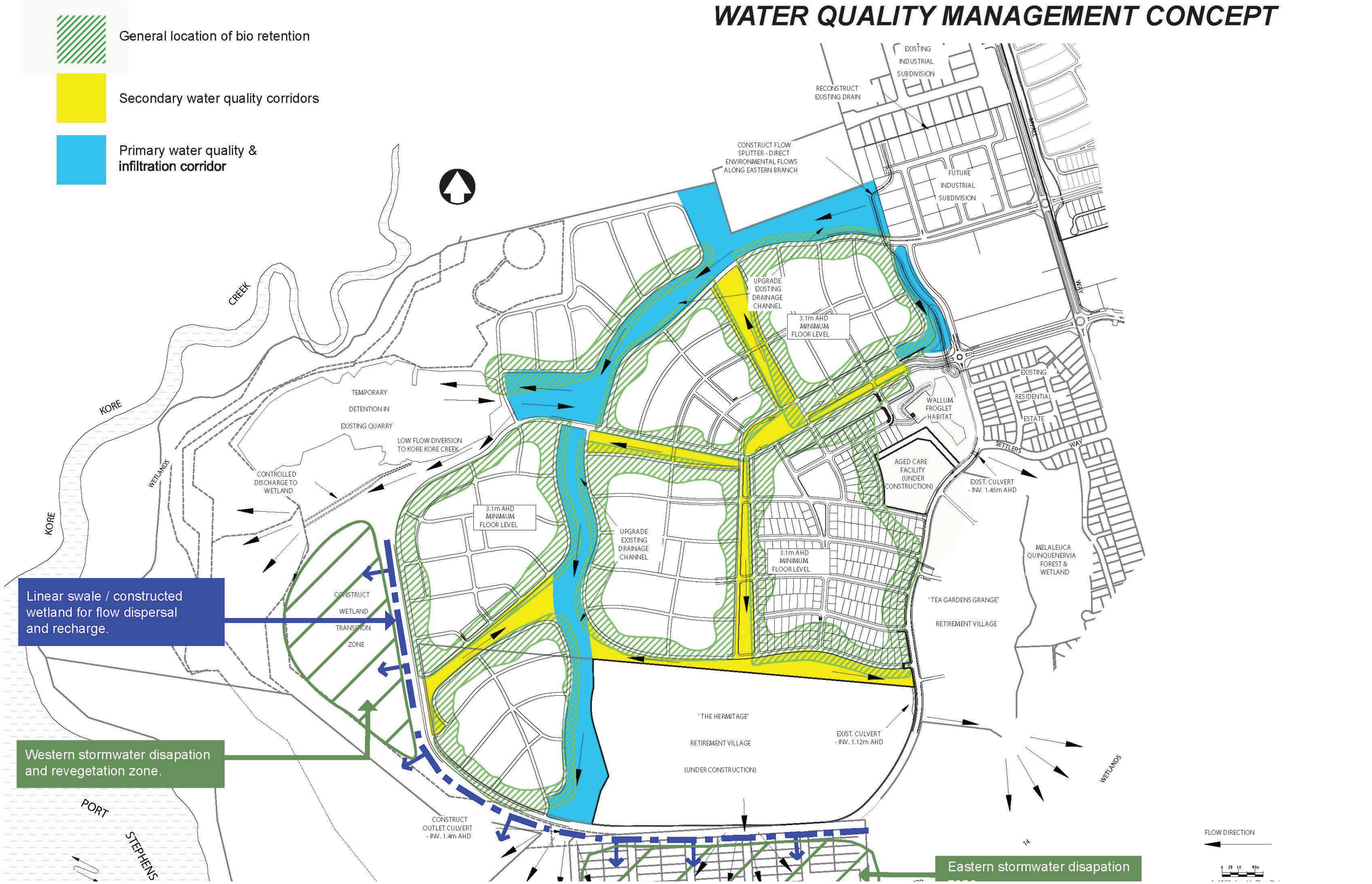


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Figure E3 – Flood Management Concept

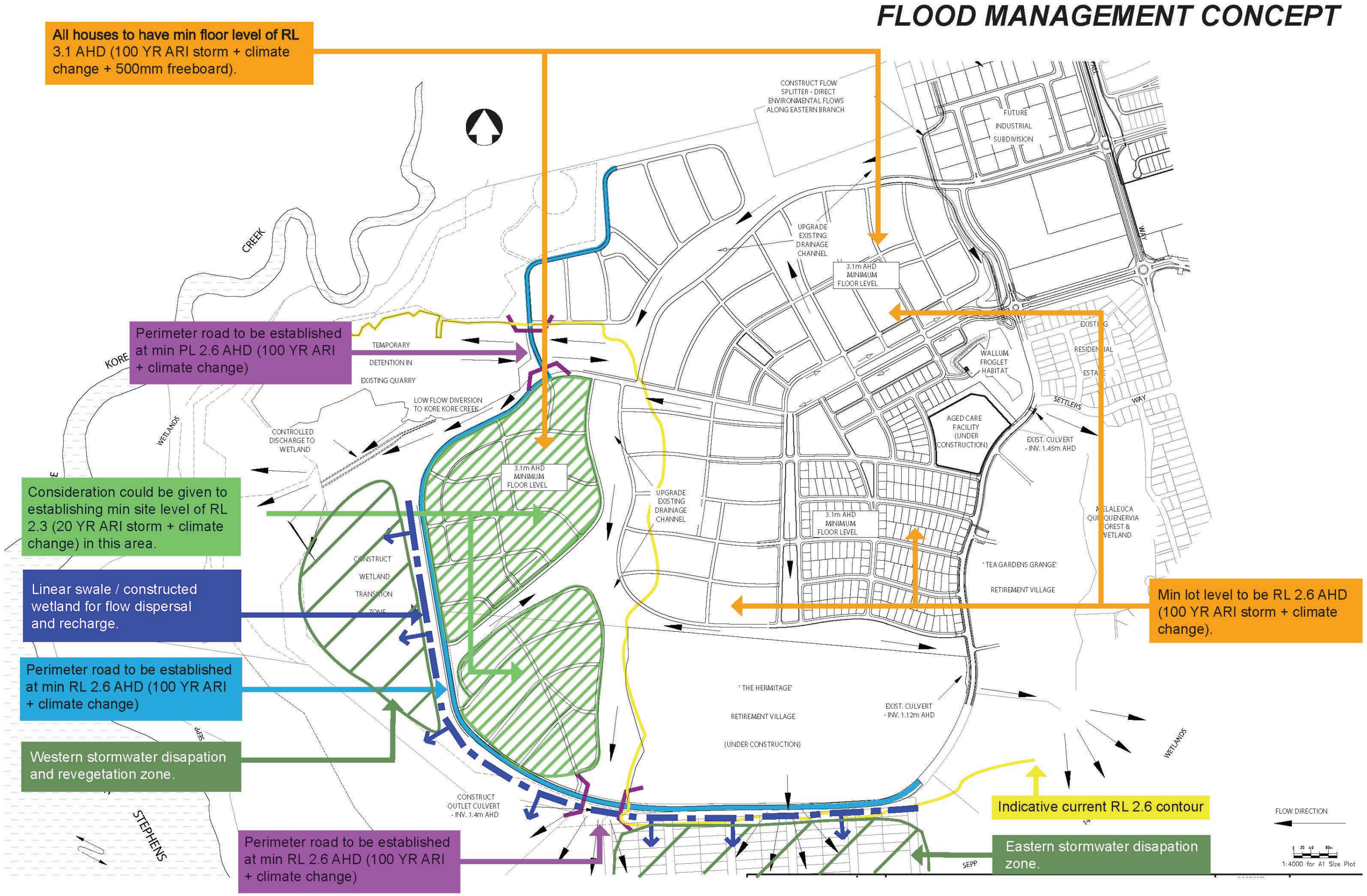
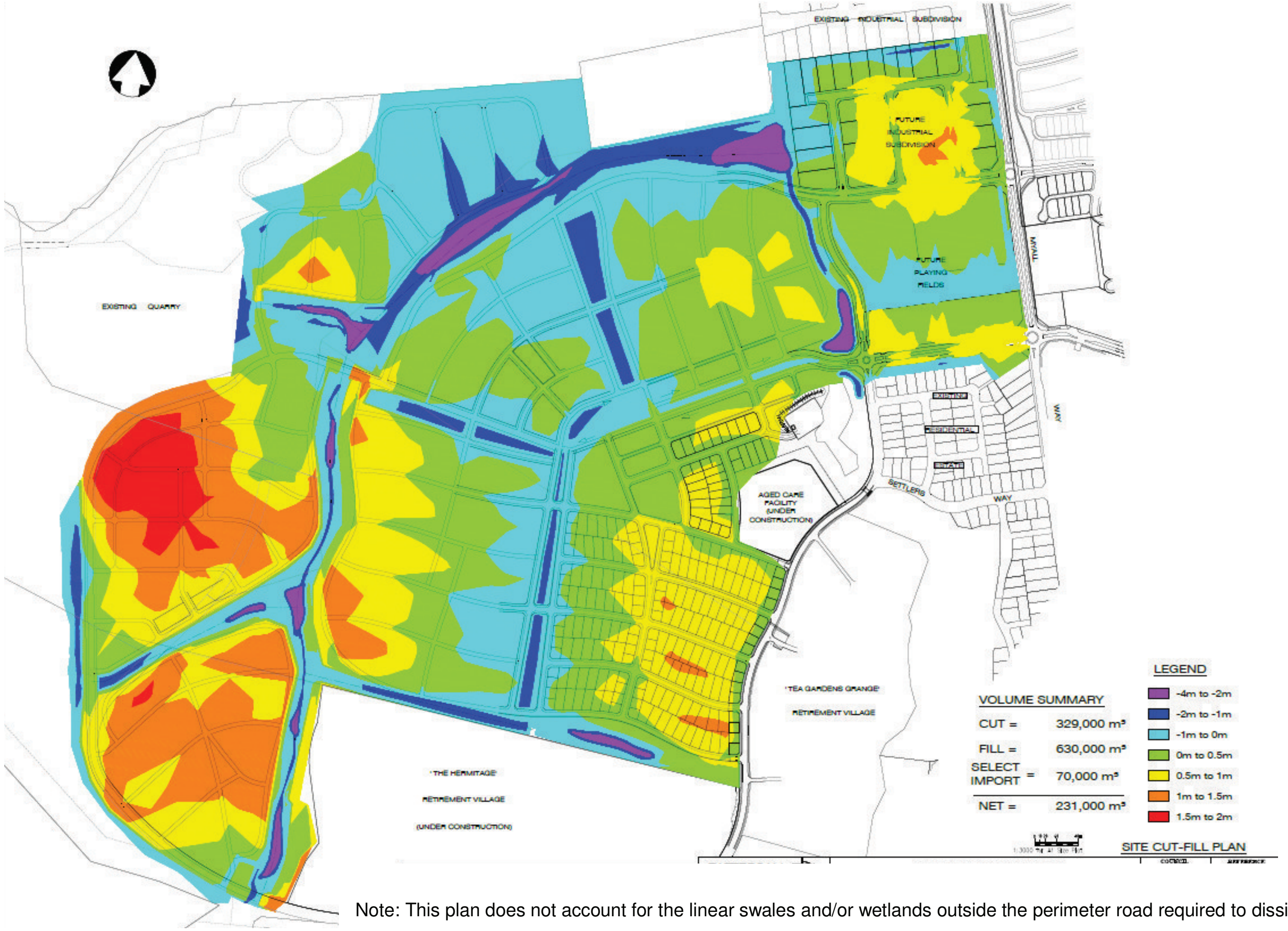


Figure E4 – Site Cut-Fill Plan



Note: This plan does not account for the linear swales and/or wetlands outside the perimeter road required to dissipate site runoff.

E6 Flood Extents

The predicted 100yr ARI flood extents for the site have been plotted to show the comparisons for the flooding scenarios:

- Undeveloped site, present sea level – **Figure E5** – This diagram shows that under current conditions, there would be some minor ponding of water in the south eastern part of the developable area during a 100yr ARI event. This is primarily due to the existing drainage channels and the small depression in the existing topography in this area;
- Developed site, present sea level – **Figure E6 –** This diagram illustrates the effectiveness of the proposed soft engineering works which preclude inundation from the developable area during a peak 100yr ARI flood event;
- Developed site, 0.9 m sea level rise – **Figure E7** – This diagram illustrates the effectiveness of the proposed soft engineering works which preclude inundation from the developable area during a peak 100yr ARI flood event not only under current conditions, but also allowing for a sea level rise of 0.9 m.

The flood extent in **Figure E7** indicates that the 100yr ARI flood can be accommodated within the proposed drainage channels.

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Figure E5 – 100yr ARI Flood Extents: Undeveloped Site, Present Sea Level

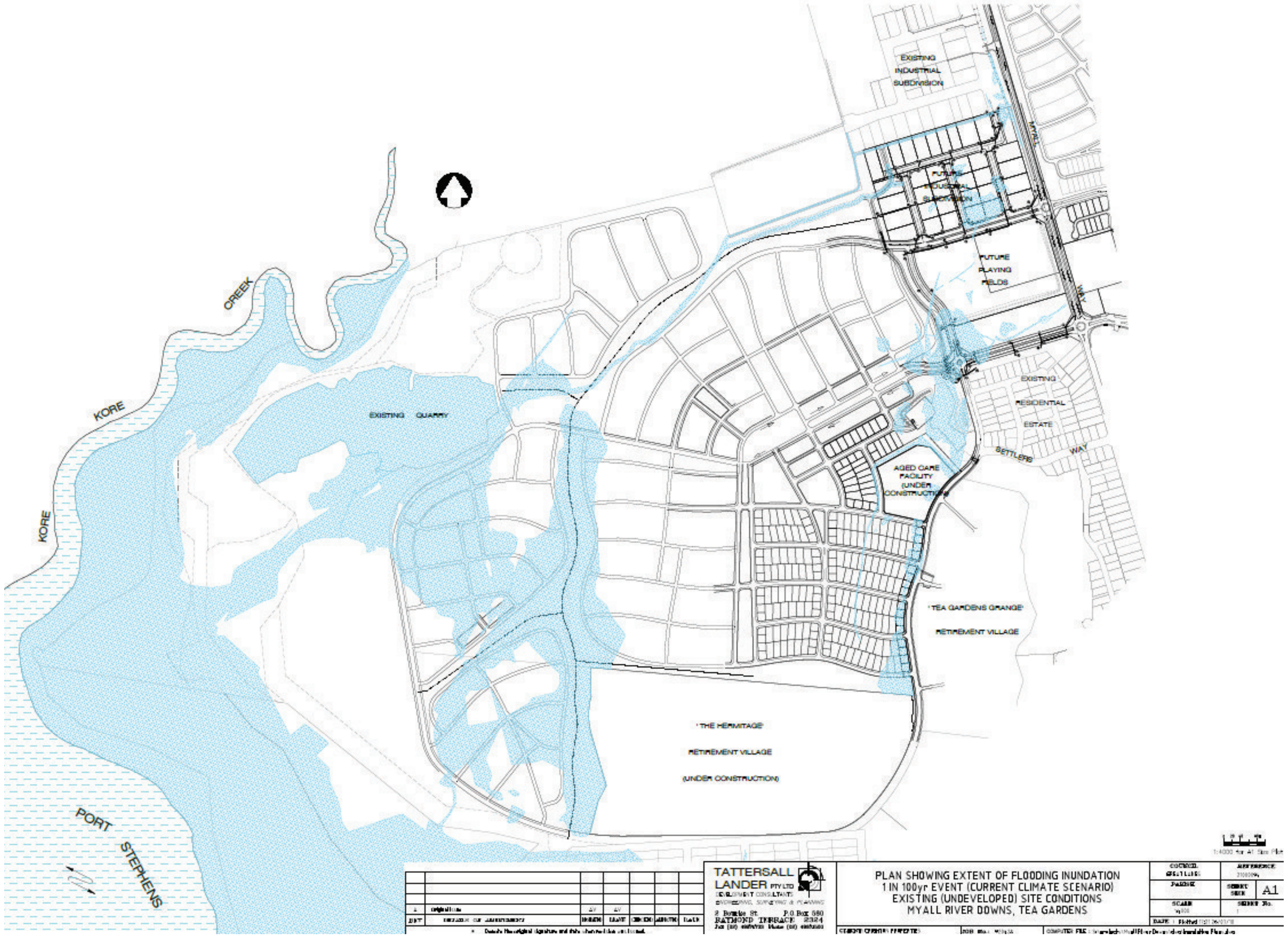

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Figure E6 – 100yr ARI Flood Extents: Developed Site, Present Sea Level

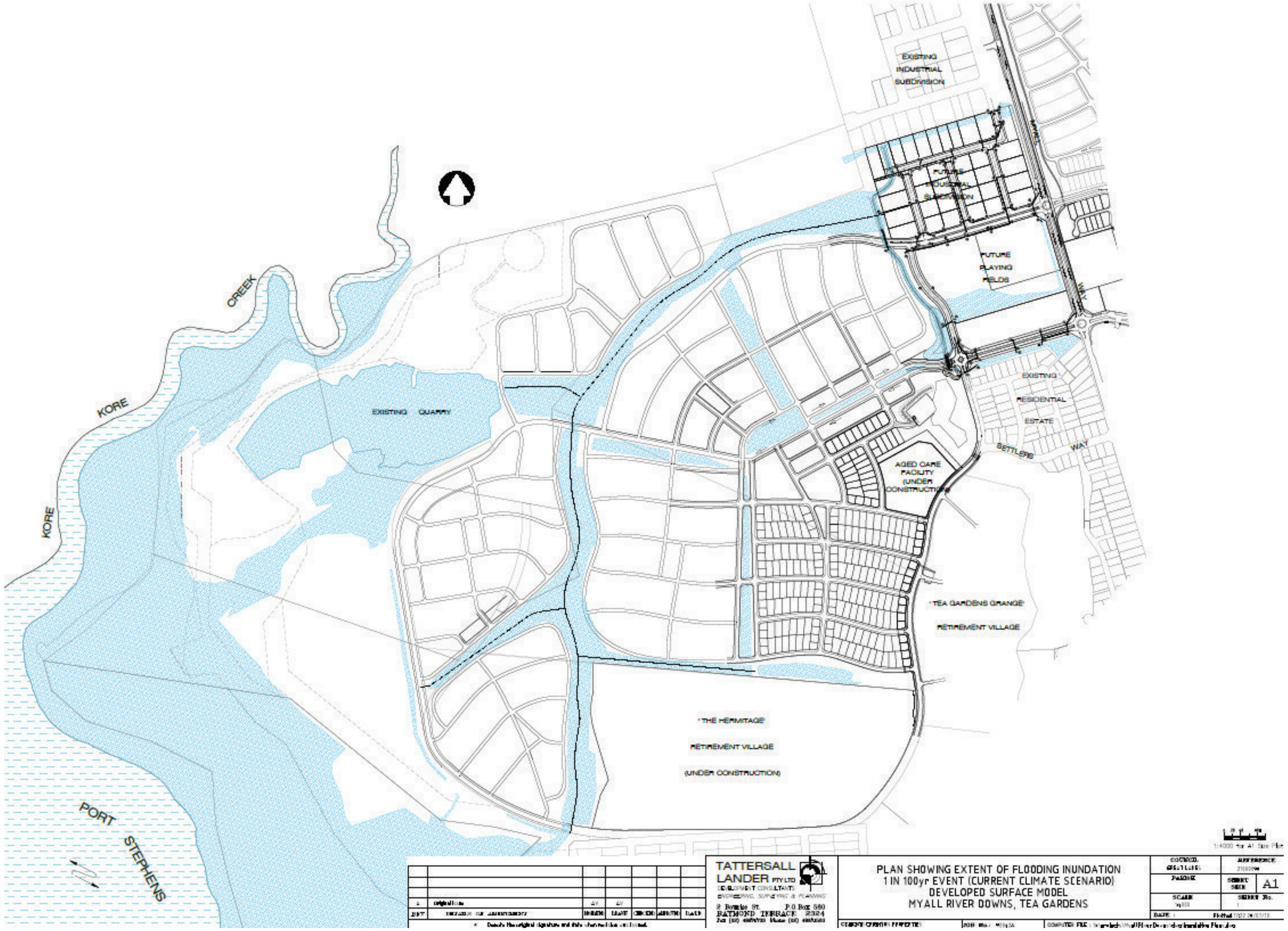
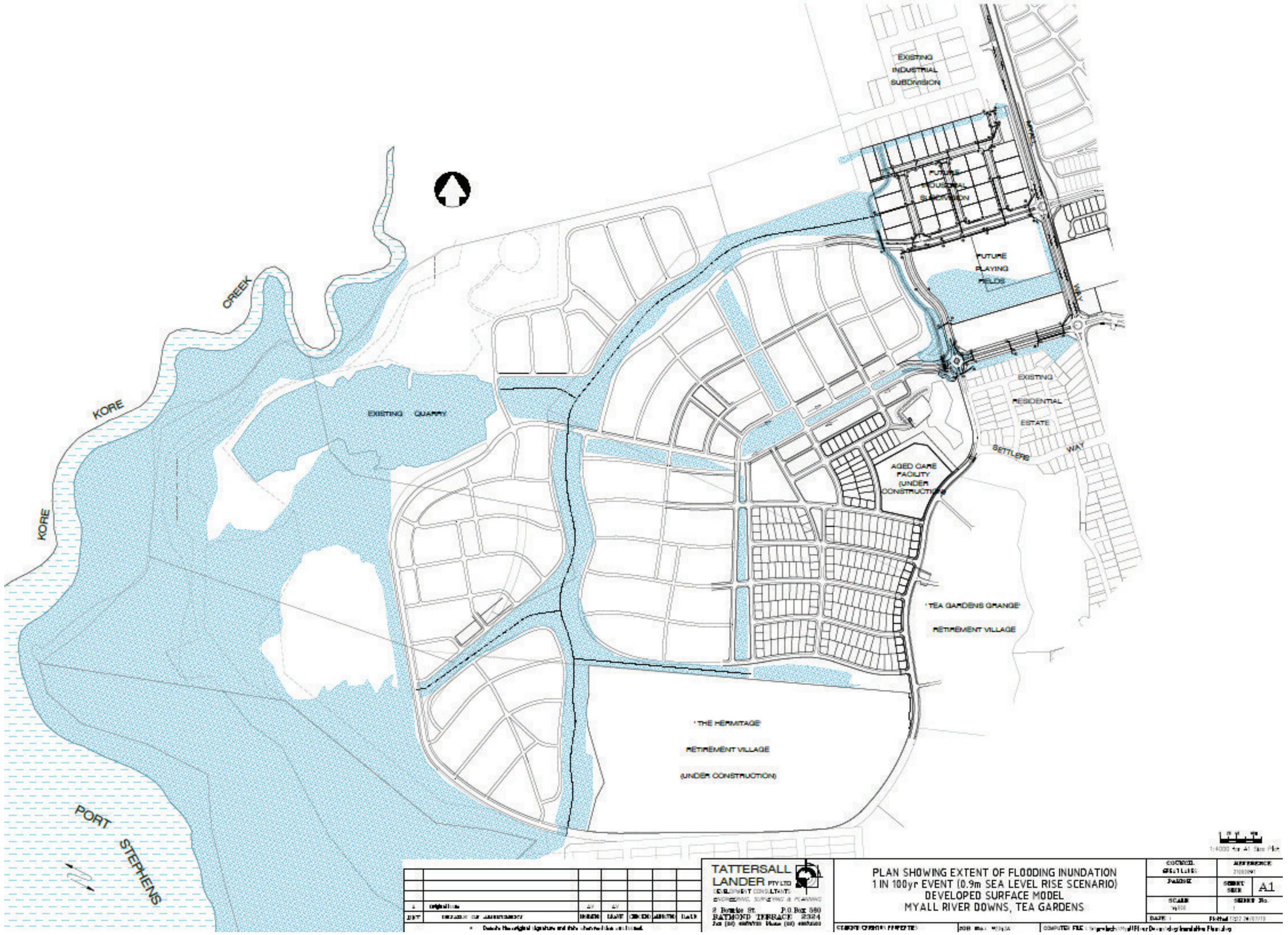


Figure E7 – 100yr ARI Flood Extents: Developed Site, 0.9 m Sea Level Rise



1. INTRODUCTION

The Myall River Downs site is located in Tea Gardens on the NSW Central Coast, within the Great Lakes Council Area.

A Local Environment Study (LES) for the site was released in 2003 by Parsons Brinkerhoff (PB).

WorleyParsons (formerly Patterson Britton and Partners (PBP)) was engaged to investigate water management for the site, including flooding and water quality management. The results of the investigation are detailed in this report.

The development is classified as a state significant development under revised planning legislation, allowing it to be approved by the Minister for Planning.

1.1 Site Description

The site is approximately 460 hectares in area and is bordered by existing residential development, a small industrial development and undeveloped land. Pindimar Bay lies to the south of the site and Kore Kore Creek runs in a southerly direction along the western boundary before discharging to the bay. Myall Street and existing residential development border the site to the east.

The site is flat and low-lying, with levels generally ranging from RL 0 to 3 m AHD. The site lies on sandy soils with high permeability, underlain by coffee rock, peaty layers and sandstone at some depth. Wetlands are present to the south and west of the site, these are protected under the State Environmental Planning Policy (SEPP) 14. The site also contains habitat for the Wallum Froglet, which is a threatened species.

The LES indicated "generally a low risk" for the exposure of acid sulphate soils through the development of the site, with no or a low probability of acid sulphate soils across the majority of the site.

In the past, large areas of the site were cleared for pine plantation and sand extraction, these activities have resulted in substantial alterations from the natural state across the majority of the site.

1.2 Current Development

Within the Myall River Downs site, two retirement villages have been approved:

- The Grange Retirement Village is constructed and occupied; and
- The Hermitage Retirement Village has been approved, but not yet completed.

1.3 This Report

This report relates to the development of the remaining developable site area, not including development that has already been approved. This area is approximately 200 hectares.

This report should be read in conjunction with the groundwater reporting undertaken by Martens Consulting Engineers, which can be found at **Appendices 5, 6 and 7**.

This report and associated work by Martens Consulting Engineers has been reviewed by BMT WBM on behalf of Great Lakes Council. The draft version of this report (issued by Patterson Britton and Partners) was reviewed in 2008. The response prepared by WorleyParsons, including the review comments, has been provided at **Appendix 3.** A further review dated 16 December 2010 is found at **Appendix 4**, and the last review of 29 June 2011 is found at **Appendix 8**. Our responses to the review have, where necessary, been incorporated within the body of this report, such that it is a standalone document.

1.4 Proposed Development

The proposed development consists primarily of residential areas, with an area of industrial development adjacent to the existing industrial area in the north eastern corner of the site.

The proposed development layout is provided on **Figure 10**.

2. FLOODING

2.1 Background

Due to the low-lying and flat nature of the site, flooding is a key issue for the development. In order to accommodate these natural constraints, the site design has employed the following features:

- Overland conveyance of flows in swales and channels;
- Wide trunk drainage channels to minimise flood levels;
- Minimise length of stormwater pipes from the development to the channels;
- Minimise lengths of major overland flowpaths within roads;
- Offline flood storage at the location of the former sand mine; and
- Minimal longitudinal grades for swales and channels (main trunk drainage lines to incorporate wetlands).

This design approach allows design flood levels to be minimised and hence minimises the volumes of fill required to develop the site.

The objectives adopted for management of the runoff quantity in the proposed development were:

- Convey 100-year Average Recurrence Interval (ARI) flood flows through the site in channels/swales;
- No adverse impact on flood levels in the industrial area north of the site;
- Habitable floor levels to be at least 500 mm above the 100-year ARI flood level in the adjacent channel;
- Final ground surface level at or above the 100-year ARI flood level; and
- Quantity of stormwater flow at site boundaries maintained at existing conditions for storms up to the 3-month ARI event.

2.2 Flooding Scenarios

Due to the proximity to the coast, flooding on the Myall River Downs site is caused by both catchment runoff and elevated water levels in Pindimar Bay. The combination of rainfall events with bay water levels leads to various scenarios, which can be modelled to predict flood levels. However, the critical scenarios which affect the final ground surface and habitable floor levels in the proposed development required for flood modelling are those which will generate the 100 year average recurrence interval (ARI) flood levels, which is the flood standard adopted by Great Lakes Council.

This report has adopted similar flooding scenarios to the LES, which drew on the Port Stephens Flood Study (Manly Hydraulics Laboratory, 1997), as follows:

- Scenario 1 – 100 year ARI rainfall combined with a 1 year ARI bay water level (RL 1.26 m AHD); and
- Scenario 2 – 20 year ARI rainfall combined with a 100 year ARI bay water level (RL 1.69 m AHD).

The higher water level of these two scenarios was the 100 year ARI flood level adopted at any point across the site.

2.3 Climate Change

In 2007, the NSW Department of Environment & Climate Change published a guideline entitled *Floodplain Risk Management Guideline: Practical Consideration of Climate Change,* which discusses the potential impact of climate change on flood levels. The report recommends that flood studies incorporate a sensitivity analyses for an increase in rainfall intensity (of 10, 20, and 30%) and also an increase in sea level (where relevant of 0.18, 0.55 and 0.91 m). A climate change sensitivity analysis was undertaken for the site in 2008, this is discussed further in **Section 2.7.** For a full copy of the letter detailing the study, refer to **Appendix 2.**

In 2009, the NSW Department of Environment, Climate Change and Water (DECCW) published the *NSW Sea Level Rise Policy Statement,* which lists the sea level rise planning benchmarks as an increase above 1990 mean sea levels of 40 cm by 2050 and 90 cm by 2100. The document also states the government's commitments as:

1. *Promoting an adaptive risk-based approach to managing the impacts of sea level rise;*
2. *Providing guidance to local councils to support their sea level rise adaptation planning;*
3. *Encouraging appropriate development on land projected to be at risk from sea level rise;*
4. *Continuing to provide emergency management support to coastal communities during times of floods and storms; and*
5. *Continuing to provide up-to-date information to the public about sea level rise and its impacts.*

The DECCW policy statement also states that "Department of Planning guidelines will describe how sea level rise should be considered in land use planning and development assessment".

The NSW Department of Planning released the *Draft NSW Coastal Planning Guideline: Adapting to Sea Level Rise* in 2009, which is centred on the following coastal planning principles:

1. *Assess and evaluate coastal risks taking into account the NSW sea level rise planning benchmarks;*
2. *Advise the public of coastal risks and to ensure that informed land use planning and development decision-making can occur;*

3. *Avoid intensifying land use in coastal risk areas through appropriate strategic and land use planning;*
4. *Consider options to reduce land use intensity in coastal risk areas where feasible;*
5. *Minimise the exposure to costal risks from proposed development in coastal areas; and*
6. *Implement appropriate management responses and adaptation strategies, with consideration for the environmental, social and economic impacts of each option.*

The draft guideline aims to provide direction to consent authorities regarding development assessment in coastal areas, which are affected by coastal erosion or coastal flooding. It recommends assessment against the following Planning Criteria for Proposed Development in Coastal Risk Areas:

1. *Development avoids or minimises exposure to immediate coastal risks (seaward of the immediate hazard line);*
2. *Development provides for the safety of residents, workers or other occupants on-site from risks associated with coastal processes;*
3. *Development does not adversely affect the safety of the public off-site from a change in coastal risks as a result of the development;*
4. *Development does not increase coastal risks to properties adjoining or within the locality of the site;*
5. *Infrastructure, services and utilities on-site maintain their function and achieve their intended design performance;*
6. *Development accommodates natural coastal processes;*
7. *Coastal ecosystems are protected from development impacts; and*
8. *Existing public beach, foreshore or waterfront access and amenity is maintained.*

Hence, for each of the two flooding scenarios listed in **Section 2.2**, the following climate change scenarios are also considered in this flooding investigation:

- Sea level rise of 0.9 m;
- Sea level rise of 0.9 m combined with an increased rainfall intensity of 30%.

It should be noted that an increased rainfall intensity of 30% represents the upper bound of sensitivity analyses to be undertaken per the *Floodplain Risk Management Guideline: Practical Consideration of Climate Change*. Hence, the scenario of 0.9 m sea level rise with a 30% increase in rainfall intensity is provided for comparative purposes only and is not intended for use in flood planning level determination.

For development that is flood affected by either of these scenarios, a merit based assessment addressing the above planning criteria is recommended by the guidelines. It is proposed for the Myall River Downs development that the following strategy for climate change adaptation is adopted:

- All roads are to be to be constructed with minimum 300 mm freeboard to the 100 year ARI flood level without climate change;
- Evacuation routes are to be constructed to have a flood depth no greater than 300 mm in the 100 year ARI flood level event with 0.9 m sea level rise;
- Lots are to be created no lower than the 100 year ARI flood level plus 0.9 m sea level rise (further consideration should be given to some areas with levels of 20yr ARI flood level and 0.9 m sea level rise);
- Habitable floors are to be constructed with 500 mm freeboard to the 100 year ARI flood level with 0.9 m sea level rise; and
- Drainage outlet conditions from developable portion of site to be a minimum of mean high water and 0.9 m sea level rise.

Any flood mitigation measures are not to:

- Create upstream flood impacts, or to impact on downstream exosystems; and
- Water quality management devices are to remain function showing for 0.9 m sea level rise and existing impacts on groundwater heights.

2.4 Model Selection

The MOUSE modelling software was selected for use in this project as it has the following attributes:

- Ability to model complex hydrology and hydraulics;
- Ability to model surface runoff, open channel flow and pipe flow;
- Allows flow splitting, loops and flow reversal;
- Hydrodynamic, allowing for storage and timing of flood flows;
- Ability to model time-varying downstream tailwater conditions eg tides;
- Output can be imported into the waterRIDE software for spatial interpretation of results; and
- Internationally recognized.

MOUSE allows accurate modelling of the interaction between surface runoff and elevated bay water level conditions.

MOUSE was developed by DHI Water & Environment which is an independent research and consultancy organisation, formed by merging Danish Hydraulic Institute (DHI), VKI - Institute for the Water Environment and Danish Toxicology Centre.

The MOUSE model for the Myall River Downs areas was developed with MOUSE version 2003.

The MOUSE system is organized in several modules. Out of these the MOUSE Runoff (surface runoff model) and the MOUSE HD (hydrodynamic network model) were selected for modelling the hydrology and hydraulics associated with the Myall River Downs area.

2.4.1 MOUSE Surface Runoff Module

The MOUSE Surface Runoff Module includes four types of surface runoff computation models and three hydrological levels for the description of the urban catchment surfaces. The computed hydrographs are used as input to the MOUSE network model.

Out of the four different surface runoff models available Model B – Kinematic wave (non-linear reservoir) was used for the hydrology analysis.

2.4.2 MOUSE Hydrodynamic Network Module (HD)

The MOUSE Hydrodynamic Network Module (HD) solves the complete St. Venant (dynamic flow) equations throughout the drainage network (loop and dendritic), which allows for modelling of:

- Backwater effects;
- Flow reversal;
- Surcharging;
- Free-surface and pressure flow;
- Tidal outfalls; and
- Storage basins.

The program can model pipe network systems with alternating free surface and pressurized flows as well as open channel network. The computational scheme uses an implicit, finite-difference numerical solution of the St. Venant flow equations. The numerical algorithm uses a self-adapting time-step, which can provide solutions in multiple connected branched and looped networks. In addition, flow phenomena such as backwater effects and surcharges can be simulated.

2.4.3 Boundary Conditions

Various external loading (hydrological and non-hydrological), meteorological information, hydraulic conditions at the interaction points with the receiving waters, etc are specified through a set of data, termed as boundary conditions according to the usual practice in numerical modelling. For each scenario modelled the time varying bay water levels were specified as boundary conditions.

2.5 Model Setup

2.5.1 Model Network for Proposed Development

The model network in the proposed development consists of the main trunk drainage lines and basin nodes.

The network extends to Pindimar Bay and considers the interaction of bay water levels with the available storage in the low-lying wetland area to the south and west of the site.

Limekilns Road to the south of the site forms a 2.5 km long weir-like barrier between the low-lying wetland area and the bay in the south. This was modelled as a combination of multilevel multiple weirs, with corresponding culverts, where applicable. The hydraulic connection to the bay in the south-west is less controlled, as weir crest levels, where present, are much lower and a number of open channel connections are present.

The layout of the modelled network is illustrated in **Figure 1**. It contains 59 nodes and 70 links. The network contains 8 culvert crossings within the developable area. These locations and dimensions are marked on the figure.

Ten contributing external subcatchments having a combined area of approximately 100 ha feed into the nodes located at the upstream end of the network.

At the uppermost (and northernmost) link, the flowpath bifurcates into the two main branches of the network (eastern and western).

The former sand mine acts as an offline storage basin.

2.5.2 Assessment of Flooding in Existing Scenario

An existing conditions model was not created as all impacts are contained on the site. A qualitative assessment of the likely upstream impacts was undertaken.

The Parsons Brinckerhoff LES (Volume 2, October 2003) shows the flood level at the northern boundary of the industrial estate under existing conditions as RL 3.35 m AHD (Figure 3-6, *100-year ARI Flood Envelope)*. It also shows that under existing conditions, the 100-year ARI flood level does not extend from Myall River Downs into the industrial estate as the general land level in the industrial area is approximately RL 3.75 m AHD.

In the proposed development (without incorporation of the effect of climate change) the 100-year ARI flood level for at the northern boundary of the property is RL 2.93 m AHD. In the scenario where Myall River Downs is subjected to a high level of climate change (Refer to **Figure 23** in **Appendix 2**), the 100-year ARI flood level for this location would increase to 3.01 m AHD. As such, the proposed development is not predicted to adversely impact on the 100-year ARI flood levels in the existing industrial area, upstream.

2.5.3 Hydrology

The model consists of 57 subcatchments. The majority of subcatchments are flat, except for a few subcatchments in the north. Refer to **Figure 2** for the subcatchment layout.

The MOUSE Model B was used for hydrological modelling. The surface runoff computation of the MOUSE Runoff Model B is based on the kinematic wave computation. This means that the surface runoff is computed as flow in an open channel, taking the gravitational and friction forces only. The runoff amount is controlled by the various hydrological losses and the size of the contributing catchment area.

The shape of the runoff hydrograph is controlled by the catchment parameters length, slope and roughness of the catchment surface. These parameters form the base of the kinematic wave computation.

Model B allows input of 5 different types of catchment surfaces, as follows:

- impervious steep;
- impervious flat;
- pervious – small infiltration;
- pervious – medium infiltration and
- pervious – large infiltration

The model applies different hydrological parameters for each of the surface types. Of the above surface types, the impervious steep, impervious flat and pervious – large infiltration types were used in this model with the default hydrological parameters, as defined below. **Table 2-1** indicates the default hydrological parameters.

Table 2–1 – Default Hydrological Parameters for Surface Runoff Model B

Parameter	Impervious		Pervious
	Steep	Flat	Large Infiltration
Wetting (m)	5.00E-5	5.00E-5	5.00E-5
Storage (m)	-	6.00E-4	2.00E-3
Start infiltration (m/s)	-	-	2.00E-5
End infiltration (m/s)	-	-	5.00E-6
Horton's Exponent (s^{-1})	-	-	1.50E-3
Inverse Horton's Exponent (s^{-1})	-	-	5.00E-5

Parameter Definitions:

Wetting loss [m]: One-off loss, accounts for wetting of the catchment surface. Storage loss [m]: One-off loss, defines the precipitation depth required for filling the depressions on the catchment surface prior to occurrence of runoff.

Start infiltration [m/s]: Defines the maximum rate of infiltration (Horton) for the specific surface type.

End infiltration [m/s]: Defines the minimum rate of infiltration (Horton) for the specific surface type.
Horton's Exponent Time Factor "characteristic soil parameter" [s^{-1}]. Determines the dynamics of the infiltration capacity rate reduction over time during rainfall. The actual infiltration capacity is made dependent on the time since the rainfall start only.
Inverse Horton's Exponent [s^{-1}]: Time factor used in the "inverse Horton's equation", defining the rate of the soil infiltration capacity recovery after a rainfall, ie in a drying period.

Design rainfall IFD input parameters were based on the IFD data for the Tea Gardens area.

The adopted subcatchment parameters and peak flow rates from the MOUSE Model B hydrology analysis are summarised in **Appendix 1**.

RATIONAL METHOD VERIFICATION

Peak flows predicted for subcatchments from the MOUSE Model B hydrology analysis were verified using the Rational Method. The Rational Method flow estimation is based on the following formula:

Q (m^3/s) = C * I (mm/hr) * A (ha) / 360

Where:

Q = Flow (m^3/s),

C = Runoff coefficient,

I = Rainfall intensity (mm/hr) and

A = Catchment area (ha)

Rural catchments and urban catchments were analysed separately following the methods recommended in *Australian Rainfall and Runoff*. Out of the 57 subcatchments in the model, 6 rural subcatchments and 6 urban catchments were analysed for the 100 year ARI flows.

Table 2-2 and **Table 2-3** summarise the relevant subcatchment parameters. **Diagram 2.1** and **Diagram 2.2** graphically compare the Rational Method estimate against model results. As there is good agreement between Rational Method estimation and model results it can be concluded that the parameter set used for the MOUSE Model B was appropriate.

A comparison was not undertaken at the catchment outlet as the storage effect of the proposed measures is not accounted for in the Rational Method. Comparisons of MOUSE and the rational method were only undertaken for a selection of individual subcatchments. It is considered that, due to the large storage volumes proposed, comparison of flows at the outlet, generated by the MOUSE model and the Rational Method, would be irrelevant.

Table 2–2 Rational Method Verification of Selected Rural Subcatchments

Subcatchment ID	Slope	Pervious %	Length (m)	Area (ha)	C_{100} (Note 1)	T_c (min) (Note 2)	I_{100} (mm/hr)	Peak Flow Rate (m^3/s)	
								Rational Method	Model Results
13/3_1	10%	100	540	15.21	0.52	22	134	**2.9**	**3.1**
13/2_52	10%	100	400	7.42	0.52	17	151	**1.6**	**1.8**
1/1_3	10%	100	200	2.07	0.52	10	189	**0.6**	**0.7**
1/1_48	10%	100	220	4.07	0.52	14	164	**1.0**	**1.3**
1/1_4	10%	100	250	5.16	0.52	15	159	**1.2**	**1.6**
2/1_26	14%	100	470	9.74	0.52	19	143	**2.0**	**2.4**

Notes:

1. F_{100} based on formula given in Table 5.1 of AR&R for zone B. Values used for $I_{12,50}$ = 14.9, $I_{12,2}$ = 7.3 and C_{10} = 0.38

2. Based on equation 5.4 in AR&R.

Diagram 2-1 Rational Method Verification – Rural Catchments (100yr ARI)

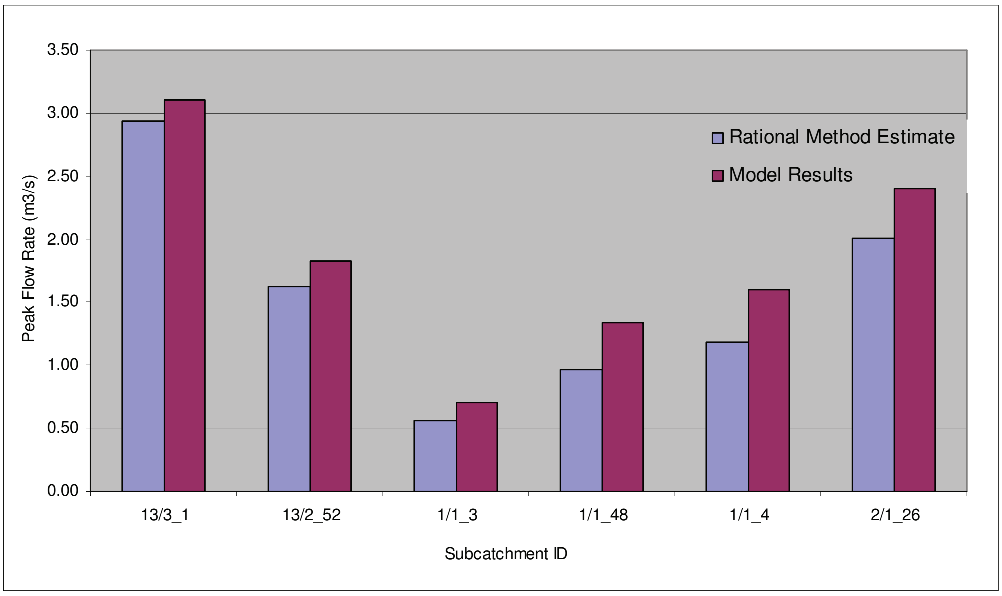
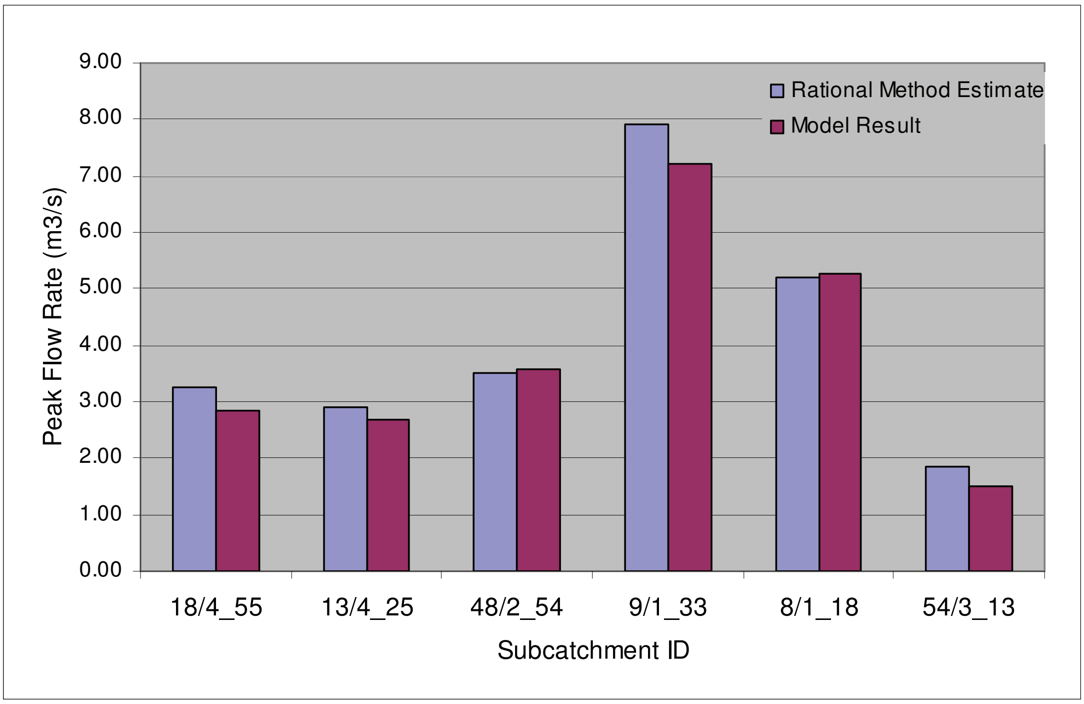


Table 2–3 Rational Method Verification of Selected Urban Subcatchments

Subcatchment ID	Area (ha)	Slope	Impervious %	Length (m)	Surface Roughness n*	Peak Flow Rate (m^3/s)	
						Rational Method	Model Results
18/4_55	7.08	1%	60	275	0.06	**3.3**	**2.8**
13/4_25	7.10	1%	60	340	0.06	**2.9**	**2.7**
48/2_54	13.18	1%	40	460	0.08	**3.5**	**3.6**
9/1_33	21.68	1%	60	420	0.06	**7.9**	**7.2**
8/1_18	9.38	2%	60	33	0.06	**5.2**	**5.3**
54/3_13	3.51	1%	60	200	0.06	**1.8**	**1.5**

Diagram 2-2 Rational Method Verification – Urban Catchments (100yr ARI)



2.5.4 Hydraulics

The hydraulic model has undergone a number of revisions through the process of design development. The model revisions are summarised as follows:

- 2006 – Original Model

- 2008 – Extended detention as required for water quality treatment added
- 2010 – Geometry of proposed channels revised per landform model supplied by Tattersall Lander, extended detention removed from model.

The current landform model cuts flood conveyance channels on average 1 metre below existing surface levels, with a maximum cut in the order of 2.5 m. Filling above existing levels would be on average less than 1 m, but up to in the order of 2 metres. Approximate minimum fill levels are around RL 2.8 m AHD.

A Manning's roughness value of 0.05 was used for all open channels and 0.013 for box and pipe culverts.

Node invert levels of the network are tabulated in **Appendix 1** and are graphically represented in **Figure 3**. Channel cross-sections are also presented in **Appendix 1**.

2.6 Model Results

The flow bifurcates into the two main branches of the network at the uppermost (*north-eastern*) corner of the site. Environmental flows would continue along both branches, while high flows would travel along the western branch. The network has been designed to limit flows along the eastern branch to a maximum of 2.4 m^3/s at node 48/1, this is to ensure no changes to the flooding conditions along this reach. Model results at this location indicate a peak flow rate of 2.4 m^3/s for Scenario 1 (100-year ARI rainfall and 1-year ARI bay water level) and 1.6 m^3/s for Scenario 2 (20-year ARI rainfall and 100-year bay water level). This flow limitation was achieved by limiting the capacity of the uppermost culvert (38/1 → 41/1) on the eastern branch and diverting excess flows along the western branch.

From the junction where flows bifurcate into the eastern and western channels, the western branch then travels downstream for approximately 900 m until reaching the junction at node 13/6. Here, depending on the hydrodynamics, low flows (up to the 1-year ARI storm) will continue along the channel and high flows will be conveyed into the offline storage basin, located to the west (*the former sand mine*).

The existing drainage through the site does not discharge to the sand mine pit other than in severe events. As such, the channel in the proposed development between the western branch and the sand mine pit would be designed with an appropriate structure to maintain environmental flows to the wetlands, whilst attenuating major flows by directing them to the former sand mine pit.

The eastern branch slopes down to about RL 1.4 at the Spinifex Avenue culvert crossing, drops to a level of RL 0.6 and continues to slope down to RL 0.2. Therefore, given the peak bay water levels of RL 1.26 for the 1 year ARI and RL 1.69 for the 100 year ARI, flooding of these low lying sections is significantly dependant on the bay water levels.

The predicted 100 year ARI flood levels throughout the proposed development are indicated on **Figure 4**. The water level profiles for Scenarios 1 and 2 are shown on **Diagram 2-3** and **Diagram 2-4**.

The extended detention through the wetlands has the effect of making the profile appear to have a series of weirs.

The permanent storage in the constructed wetlands has not been modelled and would be provided below what was modelled as the invert of the channel (i.e. what is shown as the ground level in the hydraulic model would be the still water level of the wetlands), which therefore has yielded conservative results. No extended detention is proposed for the wetlands.

The highest of the 100 year ARI flood level from Scenarios 1 and 2 are presented in **Table 2–4**.

Table 2–4 100-year ARI Flood Levels for Scenarios 1 and 2 (m AHD)

Node	100-yr ARI Flood Level
1/1	2.92
13/1	2.68
13/2	2.64
13/3	2.60
13/4	2.51
13/5	2.45
13/6	2.22
5/1	2.20
15/1	2.22
18/1	2.22
18/2	2.22
18/3	2.21
18/4	2.20
6/1	2.20
18/5	2.19
7/1	2.42
18/6	2.16
19/1	2.14
22/1	1.70
38/1	2.68
41/1	2.40
45/1	2.40
48/1	2.28

Diagram 2-3 Western Branch, Scenario 1, 100-year ARI Flood Levels (m AHD)

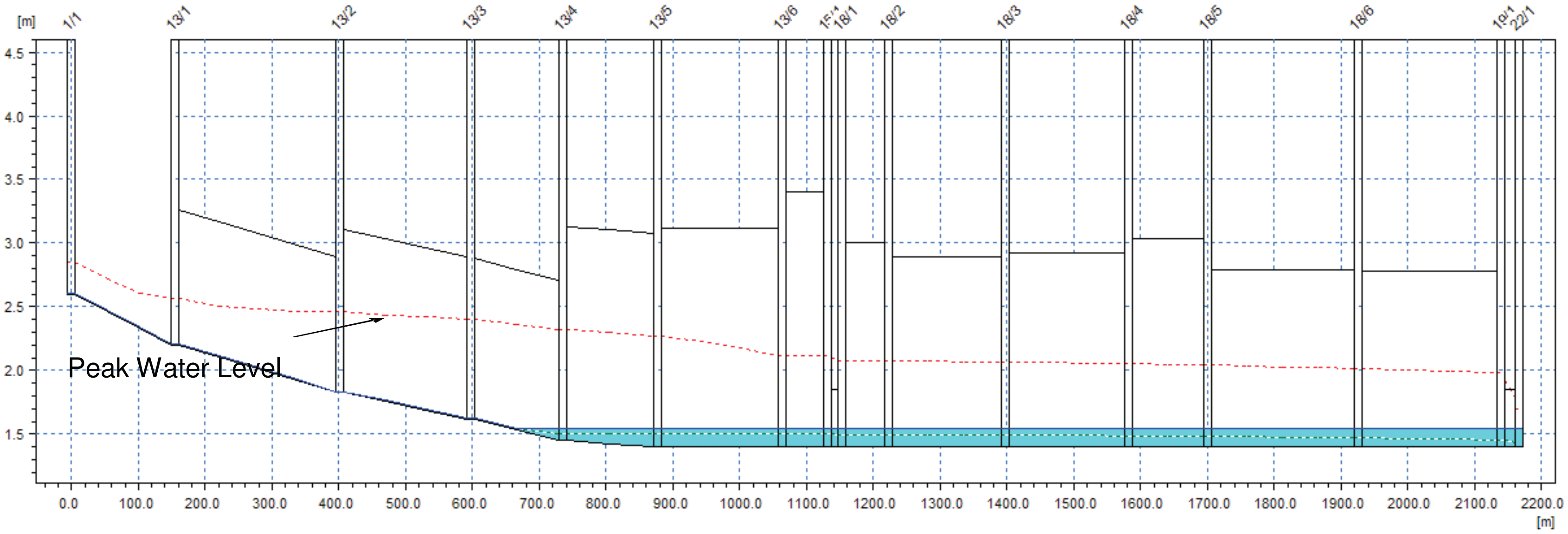
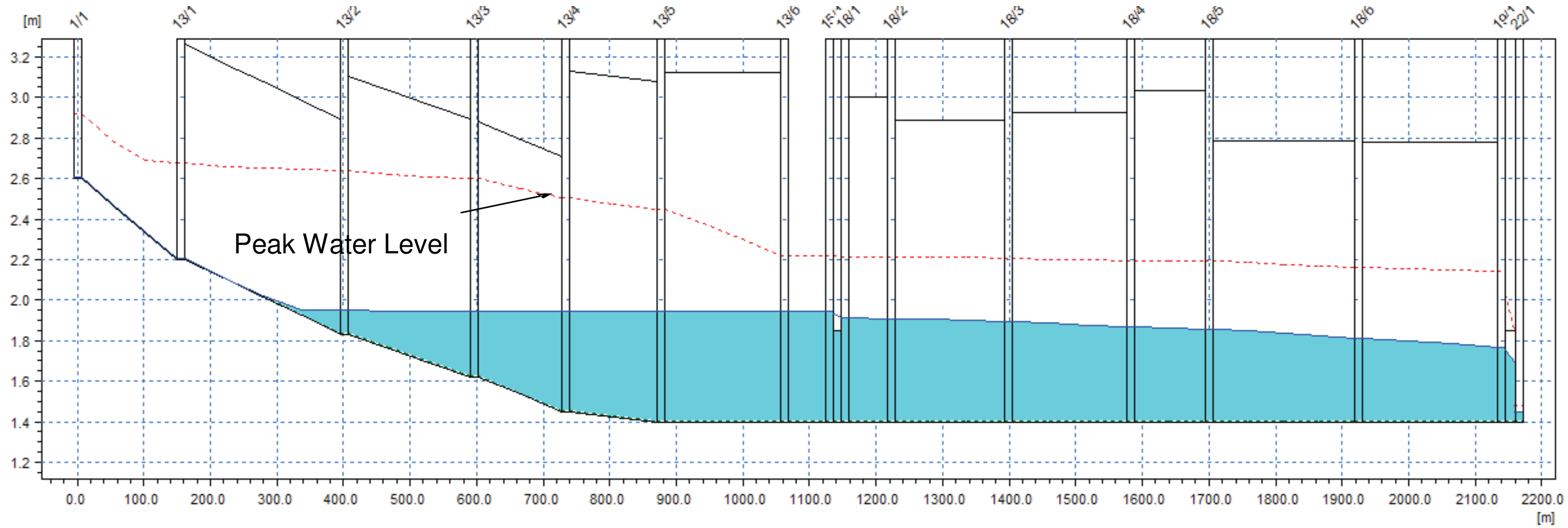


Diagram 2-4 Western Branch, Scenario 2, 100-year ARI Flood Levels (m AHD)



Cumulative minimum and maximum peak flows at critical locations are provided in **Table 2–5** and also in **Figure 5**.

Table 2–5 Cumulative Peak Flows for Scenarios 1 and 2 (m^3/s)

Node References	Description	Scenario 1 100-year ARI Rainfall, 1-year ARI Tailwater		Scenario 2 20-year ARI Rainfall, 100-year ARI Tailwater	
		Min	Max	Min	Max
9/1 to 12/1	3x1500Wx450H RCBCs discharge to wetland from approved development	0.00	4.53	-0.01	3.98
Western Branch					
2/1 to 5/1	3x1500Wx450H RCBCs from wetland to swale	-5.85	0.99	-5.15	0.97
13/3	Western branch	0.00	12.53	0.00	8.62
15/1 to 18/1	3x1500Wx450H RCBCs at road crossing	-1.92	2.22	-1.63	1.97
19/1 to 22/1	3x1500Wx450H RCBCs at southern most road crossing, discharge to wetland	0.00	4.14	-0.33	3.42
Eastern Branch					
38/1 to 41/1	1x1800Wx600H RCBC at northern most road crossing	0.00	2.82	0.00	2.01
45/1 to 48/1	2x1800Wx600H RCBCs at road crossing	-0.22	2.42	-0.22	1.60
51/1 to 54/1	3x1800Wx600H RCBCs at road crossing	0.00	4.26	-0.02	3.08
23/1 to 26/1	3x1200Wx450H RCBCs at road crossing, discharge to wetland	0.00	4.47	-0.03	3.81
54/5 to 54/6	Discharge to wetland	-0.71	6.66	-5.38	5.72

2.7 Climate Change Scenarios

Climate change was considered as discussed in **Section 2.3** and the following climate change scenarios were also modelled:

- Sea level rise of 0.9 m;
- Sea level rise of 0.9 m combined with an increased rainfall intensity of 30%.

The results of the climate change scenarios are presented in **Table 2–6** and on **Figures 6 and 7**.

Table 2–6 100-year ARI Flood Levels compared with Climate Change Scenarios (m AHD)

Node	Without Climate Change	Sea Level Rise of 0.9 m	Sea Level Rise of 0.9 m and 30% larger storm
1/1	2.92	2.92	3.01
13/1	2.68	2.69	2.89
13/2	2.64	2.65	2.86
13/3	2.60	2.61	2.82
13/4	2.51	2.60	2.73
13/5	2.45	2.60	2.68
13/6	2.22	2.60	2.67
5/1	2.20	2.60	2.67
15/1	2.22	2.60	2.67
18/1	2.22	2.59	2.66
18/2	2.22	2.59	2.66
18/3	2.21	2.59	2.65
18/4	2.20	2.59	2.63
6/1	2.20	2.59	2.63
18/5	2.19	2.59	2.63
7/1	2.42	2.59	2.63
18/6	2.16	2.59	2.60
19/1	2.14	2.59	2.59
22/1	1.70	2.59	2.59
38/1	2.68	2.69	2.89
41/1	2.40	2.63	2.71
45/1	2.40	2.63	2.71
48/1	2.28	2.62	2.66

The results shown in **Table 2–6** illustrate that a sea level rise of 0.9 m would increase flood levels by up to 400 mm within the development. The flood level along the lower reaches increases from RL 2.2 to 2.6 m AHD. Therefore, sea level rise can be adequately accommodated by the current development strategies and landform, which indicates a minimum fill level around RL 2.8 for urban areas.

The results also demonstrate that the highest likely increased rainfall of 30% would not have any significant impact for most of the development, as flood levels are increased by less than 100 mm from the sea level rise scenario. A small area towards the north is predicted to have flood levels up to approximately 200 mm higher than the sea level rise scenario. Again, this scenario was provided for comparison purposes only, to demonstrate that the impact of increased rainfall is not significant compared to increased sea level. If an increased rainfall scenario were to be adopted for planning purposes, it is recommended that a future study adopt a lower increase, such as 10% or 20%.

2.8 Conclusions

The model results indicate that the 100 year ARI flood levels across the site can be adequately accommodated through the strategy set out in **Section 2.1**. The results indicate that the site can be developed by:

- Creating the proposed primary and secondary drainage channels; and
- Ensuring the surrounding development area is above the relevant level, by filling. The relevant levels are:
    - Final ground surface level: at or above the 100-year ARI flood level with climate change (consideration to be given to some areas with lot ground levels at 20yr ARI flood level plus 0.9 m sea level rise);
    - Roads: level to be minimum of 300 mm freeboard to 100yr ARI flood level without climate change. Preferable minimum level is 300 mm below 100yr ARI flood level plus 0.9 m sea level rise;
    - Evacuation routes: to have a flood depth no greater than 300 mm in the 100 year ARI flood level event with 0.9 m sea level rise; and
    - Habitable floor levels: at least 500 mm above the 100-year ARI flood level with 0.9 m sea level rise.

The proposed development would maintain environmental flows through both the eastern and western branches of the drainage network, the high flows would travel along the western branch. Limitation of flows along the eastern branch could be achieved by limiting the capacity of the uppermost culvert on the eastern branch and diverting excess flows along the western branch.

High flows along the western branch could be attenuated by utilising the offline storage basin (former sand mine). Design of the interaction of the channels, proposed sand mine water body and low/high runoff rates would ensure that the existing environmental flows were maintained. This would be achieved with appropriate flow diversion structures within the channels.

Based on the flood modelling results, a substantial reduction in flood levels and fill volumes is possible, compared to the estimates made in the LES, due to the strategy proposed in **Section 2.1**.

3. WATER QUALITY

3.1 Background

The quality of stormwater runoff generated by the site is important to maintain the existing wetlands to the south and west (*downstream*) of the developable area. The stormwater quality management strategy is to maintain the existing quality of water discharged to downstream ecosystems. This includes the recharge of groundwater, for the downstream groundwater dependent ecosystems.

The water quality strategy includes the following Water Sensitive Urban Design (WSUD) elements:

- Primary water quality treatment by bioretention/infiltration systems on selected lots, within open space and alongside roads where necessary;
- Secondary water quality treatment and infiltration within wetlands within channels; and
- Vegetated swale along the north-western boundary of the proposed retirement village.

3.2 Objectives

3.2.1 Maintain Existing Conditions

The first objective is to implement sufficient measures on site to maintain the existing annual pollutant loads discharging the site.

3.2.2 Stormwater Quality Treatment

In addition, the minimum treatment requirements of the Department of Environment, Climate Change and Water (DECCW) are to be met, refer **Table 3–1**.

Table 3–1 – DECCW Treatment Requirements

Pollutant	Reduction (%)
Total Suspended Solids	80
Total Phosphorus	45
Total Nitrogen	45

3.2.3 Groundwater

The final water quality objective relates to the interaction of stormwater or surface water with groundwater.

The principle specified by DWE is to treat stormwater runoff prior to discharge (primary treatment) into groundwater-connected wetlands such that the water quality matches or is better than the groundwater quality. The predicted average concentration of pollutants in runoff following treatment would be 12 mg/L for SS, 0.06 mg/L for TP and 0.56 mg/L for TN. Typical nutrient concentrations in the groundwater is around 2.4 mg/L TP and 8.5 mg/L TN (Martens, 2010). As such, the proposed level of treatment readily complies with the DWE requirement for groundwater protection.

3.3 Music Water Quality Model

MUSIC is a continual-run conceptual water quality assessment model developed by the Cooperative Research Centre for Catchment Hydrology (*CRCCH*). MUSIC can be used to estimate the long-term annual average stormwater volume generated by a catchment as well as the expected pollutant loads. MUSIC is able to conceptually simulate the performance of a group of stormwater treatment measures (*treatment train*) to assess whether a proposed water quality strategy is able to meet specified water quality objectives.

To undertake the water quality assessment component of the Stormwater Management Plan, a long-term MUSIC model was established for the proposed subdivision site. The model was used to estimate the annual pollutant load generated under existing state and developed conditions over a 5 year period of historically average rainfall.

MUSIC was chosen for this investigation because it has the following attributes:

- It can account for the temporal variation in storm rainfall throughout the year;
- Modelling steps can be as low as 6 minutes to allow accurate modelling of treatment devices;
- It can model a range of treatment devices;
- It can be used to estimate pollutant loads at any location within the catchment; and
- It is based on logical and accepted algorithms.

3.4 Input Data and Calibration

3.4.1 Rainfall

The nearest rainfall station to the site is located at Hawks Nest. Daily rainfall data only was available for the Hawks Nest station and monthly average rainfalls were obtained from the Bureau of Meteorology for all available years. The mean annual rainfall at Hawks Nest was found to be of the order of 1,356 mm.

In order to develop a model that could comprehensively assess the performance of water quality treatment devices such as bio-retention swales and wetlands, the use of pluviograph rainfall data (*captured at six minute intervals*) was considered necessary. The nearest station to Myall River Downs with similar elevation for which pluvial data was available was the Williamtown station.

Six-minute pluviograph data was used for the five year period of rainfall data from 1/7/1959 to 31/6/1964 from the Williamtown station. The average rainfall for this period is 1,326 mm/yr, the dataset contains the highest recorded rainfall over a five-year period for the Williamtown station. As such, this is the best available dataset to represent a 5-year period at Tea Gardens.

3.4.2 Evaporation

Monthly areal potential evapotranspiration values were obtained for Tea Gardens from the Bureau of Meteorology data and are shown in **Table 3–2**.

Table 3–2 – Monthly Areal Potential Evapotranspiration

Month	Areal Potential Evapotranspiration (*mm*)
January	185
February	140
March	140
April	95
May	65
June	50
July	50
August	68
September	95
October	140
November	155
December	175
Total	**1358**

3.4.3 Soil Data

A rainfall-runoff calibration was undertaken for existing site conditions. The following parameters within the model were calibrated, based on the typical deep sandy soil conditions that are encountered on the site:

- Rain threshold (impervious area);
- Soil storage capacity (pervious area);
- Initial storage (pervious area);