

PATTLE DELAMORE PARTNERS LTD

Three Kings Renewal Stormwater Management Plan - Option 15H1

Fletcher Residential Limited

Three Kings Renewal Stormwater Management Plan – Option 15H1

✦ Prepared for

Fletcher Residential Limited

✦ September 2014



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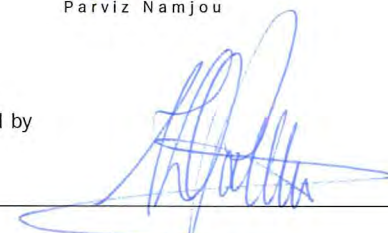
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Limitations:

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Executive Summary

The Three Kings Renewal requires a plan change to the operative Auckland District Plan (Isthmus Section) to enable the comprehensive redevelopment of the former quarry lands at Three Kings. The development includes open space areas, a comprehensive roading and pathway network, apartment blocks of differing heights, and terraced housing. The development is expected to house between 3,500 and 4,275 people.

This report addresses development layout 15H1 which provides a comprehensive development for both the former Three Kings Quarry owned by Fletcher Concrete and Infrastructure Ltd and an area of adjoining land to the south previously operated as a quarry by the former Mt Roskill Borough Council.

Pattle Delamore Partners Ltd (PDP) was engaged to develop the stormwater management concept for the development and demonstrate that the residential development enabled by the plan change can proceed. The development will require raising the existing base of the quarry using imported fill. The depth of fill will vary to a maximum of 28 m below finished ground level. The development will be at a level between 15 m and 17 m below the surrounding land with stormwater to be disposed of by soakage. While the completed fill level on the site will be above the natural groundwater levels (without the need for pumping), the differential ground levels mean the management of stormwater is a key consideration in the overall development design.

This report details the design and modelling of surface and groundwater to assess stormwater effects and using this, sets out the stormwater management plan for the proposed development. The report includes:

- ✧ Catchment hydrology and main flow paths;
- ✧ Determination of required flood storage volumes and levels;
- ✧ Location and nature of flood storage zones;
- ✧ Stormwater quality treatment; and
- ✧ Incorporation of soakage and final discharge of stored water.

A conservative approach has been taken in the design of the stormwater management measures set out in the stormwater management plan, to ensure the plan is feasible and effective in the long term. This approach includes making conservative assumptions concerning catchment hydrology, soakage rates, flood storage, and flood freeboard.

This stormwater management plan concludes that:

- ✧ The groundwater rise due to infiltration of water during the 10-year and 100-year ARI (Average Recurrence Interval) rainfall events is 1.5 m and 2 m, respectively. Therefore, it has been assumed that soil is saturated up to RL 58.0 m and RL 58.5 m for the 10-year and 100-year rainfall events, respectively, and no flood storage is used below these levels.
- ✧ The 10-year and 100-year ARI rainfall events can be appropriately managed with discharges to soakage facilities provided along the eastern wetland, beneath the sports field, and at a number of individual apartment blocks.

- ✧ Flood storage is available to manage the 100-year ARI runoff volume with appropriate freeboard provided to all habitable floors. The flood level is RL 60.0 m at the sports field and RL 61.6 m in the northern cell of the eastern wetland.
- ✧ The freeboard available is such that all of the **rainfall volume** (not runoff volume) from a 100-year ARI rainfall event can be accommodated below all habitable floors.
- ✧ In order to avoid flooding on the sports field in a 10-year ARI rainfall event, the field level has been set at RL 59.0 m. The 10-year maximum flood level will be RL 61.3 m in the northern cell of the eastern wetland channel, with some overland flow paths elsewhere at higher levels.
- ✧ Stormwater quality treatment is provided in several locations: Pond A, the eastern wetland cells, and in individual swales and rain-gardens for some paved areas and roads. High quality roof water will be achieved through the use of non-exposed metal products such as pre-painted steel.

While further work is necessary to refine the design of the stormwater management system, PDP are confident that stormwater from the redevelopment of the site can be appropriately managed. It is expected that the further work will lead to reductions in the flood storage required for the development.

As such, it is considered that the proposed stormwater design concept will comfortably cater for the modelled rainfall events and that the residential development enabled by the Plan Change is supported in terms of stormwater management.

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1.0 Introduction

1.1 Background

The Three Kings Quarry, located south of Mount Eden in Auckland, has been excavated for scoria since the 1840s and began the process of filling in 2012. Fletcher Concrete and Infrastructure Ltd (Fletcher) has owned and operated the 15.2 ha site since 1922 and through its sister company Fletcher Residential Ltd intends to develop it into a residential precinct including apartments, townhouses, and open space including sports fields. The development is expected to house between 3,500 and 4,275 people.

This report addresses development layout 15H1 which provides a comprehensive development for both the former Three Kings Quarry land owned by Fletcher and areas of adjoining land to the south previously operated as a quarry by the former Mt Roskill Borough Council. The site layout plan has been developed by dKO Architecture and Surfacedesign Inc. and is shown in Appendix A.

The Three Kings Renewal development requires a plan change to the operative Auckland District Plan (Isthmus Section) to enable the redevelopment.

Pattle Delamore Partners (PDP) has been involved in the development and operation of the Three Kings Quarry for Winstone Aggregates since 1996. PDP has also been the primary consultant assessing the groundwater resources of the wider Auckland Volcanic field for Auckland Council and former Auckland City Council. Previous work has included:

- ✧ Assessment of the geology and groundwater regime in the Three Kings area for Winstone Aggregates to assess dewatering regimes and groundwater management for quarry operations.
- ✧ Preparing the Assessment of Effects on the Environment related to groundwater for obtaining the groundwater dewatering consent for the continued operation of the quarry.
- ✧ Preparing the Assessment of Effects on the Environment related to groundwater for filling the quarry in 2011.
- ✧ Assessment and modelling of the geology and groundwater within the Auckland Volcanic Field across the Auckland Isthmus in the “Global Aquifer Study” (GAS) for the former Auckland City Council. This informed catchment management with respect to soakage across the isthmus and particularly the Meola catchment (within which Three Kings is located).
- ✧ Assessment of the effects of stormwater soakage discharges and soakage opportunities throughout the Auckland Volcanic Field for Auckland Council and private clients.
- ✧ Preparation of the Soakage Design Manual for the Auckland City Council in 2003 and an update to this for Auckland Council to expand its use to the wider Auckland region in 2013.

- ✧ Assessment of stormwater contaminant loads and development of catchment management approaches for stormwater quality across the Auckland Isthmus for the former Auckland City Council for its network consent applications.

PDP was engaged to assess the potential effects of stormwater discharges, and develop the stormwater management concept for the development to demonstrate that the development enabled by the plan change can proceed. The development will require raising the existing base of the quarry using imported fill. The depth of fill will be a maximum of 28 m below the finished ground level. The development will be between 15 and 17 m below the surrounding land with no surface water outflow. Therefore, the management of stormwater is a key consideration in the design of the layout and the lots.

1.2 Scope of Work

Preliminary design has been carried out for a number of alternative stormwater management options.

Earlier development layouts included two lakes. These were considered for their potential to provide flood storage, water quality improvement and amenity value. The lakes would have required lining to maintain water levels (with low groundwater levels) and were considered to have potential algal problems. This, combined with feedback from consultation with iwi about water quality, led to the development of a stormwater approach which includes a wetland system instead. The wetland system will provide treatment to a wider range of stormwater contaminants, compared to the lake and is the preferred approach in terms of stormwater quality management.

Two key groundwater scenarios have been proposed; maintaining an artificially reduced groundwater level via pumping (referred to as the 'long-term pumping' scenario) to improve soakage and underground storage potential; and allowing groundwater levels to naturally develop (referred to as the 'no long-term pumping' scenario). The 'long-term pumping' scenario is less critical for stormwater design as surplus underground storage is available for soakage due to lower groundwater levels. This report focuses solely on the 'no long-term pumping' scenario to demonstrate that the development is feasible using the more conservative groundwater conditions.

Stormwater modelling was carried out based on the development layout Option 15H1 shown in the dKO Architecture illustrative layout. This layout provides a comprehensive development plan for both the former Three Kings Quarry land owned by Fletcher and an area of adjoining land to the south previously operated as a quarry by the former Mt Roskill Borough Council.

Due to the low elevation of the Three Kings Quarry land, stormwater management planning had to account for the effects of both surface water and groundwater flows. Stormwater and groundwater models were constructed (in HEC-HMS and Visual Modflow respectively) to develop a comprehensive understanding of the behaviour of water in the former quarry. Climate change was accounted for in the models.

This report details the design and modelling of surface and groundwater and, using this, sets out the stormwater management plan for the proposed development including:

- ✧ Catchment hydrology and main flow paths;
- ✧ Determination of required flood storage volumes and levels;
- ✧ Location and nature of flood storage zones;
- ✧ Stormwater quality treatment; and
- ✧ Incorporation of soakage and final discharge of stored water.

The stormwater management plan will inform the development of detailed site levels and piped infrastructure design.

A conservative approach has consistently been taken to develop the stormwater management plan, to ensure that the plan is feasible and effective in the long term.

Key conservative assumptions include:

- ✧ Rainfall values used are 10 to 25% higher than required by Auckland Council's "Guidelines for Stormwater Runoff Modelling in the Auckland Region" Technical Publication 108 (TP108).
- ✧ Short term groundwater levels have been accounted for and assume that water from the surrounding groundwater catchment instantaneously contributes to an increased groundwater level (at the same time as surface water arrives at the flood storage areas). In reality groundwater levels will rise later due to the time water takes to infiltrate through the ground.
- ✧ No pumping of groundwater is allowed for with all flood storage provided above the pre-quarry pumping groundwater levels (although this is an opportunity that could be used to provide extremely significant additional water storage capacity).
- ✧ Zero soakage has been allowed for from the main flood storage areas during the rainfall event – actual storage volumes are expected to be reduced following on site testing and flow routing.
- ✧ Freeboard has been increased to ensure no flooding of habitable floors for a hypothetical case of zero surface infiltration and zero soakage. That is, floor levels are above the water level of the 100-year ARI rainfall event even if there were no infiltration or soakage in the development.
- ✧ Sediment treatment and filtering is provided to maintain soakage capacity in the long term.

While no pumping of groundwater has been conservatively assumed in developing the stormwater management plan for the quarry, there is potential for additional soakage and ongoing pumping would provide a significant opportunity for flood storage in the unsaturated zone. In addition, as much of the existing public drainage network surrounding the quarry is a combined sewer system, there is significant potential to divert stormwater away from the combined sewer to soakage and, as a result, improve the capacity of the sewer. The Auckland Council Stormwater Unit and Watercare are currently considering these opportunities.

2.0 Existing State

2.1 Catchment History

Three Kings is situated at the top of the Meola catchment. The catchment has complex geology with the groundwater catchment underlying a wide volcanic area across the isthmus.

The top of the Meola surface water catchment follows Mt Eden Road, directly adjacent to the quarry, and the Mt Eden area to the north. The surface water catchment includes the Three Kings area and generally heads westwards, taking in parts of Sandringham, Balmoral, Mt Albert, St Lukes and Chamberlain Park before discharging at Meola Reef.

The groundwater catchment is wider and covers parts of the Meola, Motions, Newmarket, Epsom and Central City surface water catchments. The extent of the volcanic aquifer across central Auckland is approximately 5500 ha, or approximately 38% of the isthmus area.

Over time, soakage systems have generally developed to make use of those parts of the aquifer with good soakage rates although the extent of the “soakage system” remains significantly smaller (approximately 2800 ha) than the extent of the aquifer. Within the Meola catchment the basalt aquifer, which provides the primary soakage capability, covers approximately 52% of the surface catchment area. Of the remaining area, combined sewers service approximately 40% of the catchment and separated stormwater systems 10%.

Parts of Meola have been developing since the early 1900s. The Meola catchment is dominated by residential landuse, which accounts for 80% of the catchment area, with open space (10%) and industrial (6%) being the other significant landuses.

The Three Kings site originally consisted of a number of volcanic cones. The quarry has been extensively excavated for basalt and scoria since the 1840s, with only one peak remaining today. In the current state the quarry floor level is between RL 34 m and RL 55 m, approximately 28 m below the surrounding roads. The walls of the quarry consist partly of scoria and fractured basalt, as shown in Figure 1 (below), and the geology map, Figure 6 (refer Appendix A).



Figure 1: Three Kings Quarry Image from 2014

2.2 Geology

The main features of the pre-volcanic topography in the Auckland area were the Waitemata Valley (subsequently drowned to form the current Waitemata Harbour), the Manukau Valley (current Manukau Harbour) and the main dividing ridge between the two valley systems. The Auckland Volcanic Group erupted basalt lavas, scoria, tuff and ash through, and accumulated on, the pre-existing ridge and valley system.

The Three Kings Quarry sits approximately centrally within the Three Kings volcanic crater. The crater is a hole within the older surrounding Waitemata and Tauranga Group rocks, which at the surface is approximately 1000 m in diameter and takes the approximate form of an ice-cream cone. This crater was filled with volcanic material - primarily basaltic lava and scoria - during the eruptive phases of the Three Kings volcano.

During the initial explosive eruptive phases of the Three Kings volcano the pre-existing Waitemata and Tauranga Group rocks were shattered and blown out of the crater to be deposited as tuff in a raised ring around the edge of the crater. The height of this ring varies around the crater and in the north, around Landscape Road to the west of Mt Eden Road, there is a low point. Basalt lava, which ponded within the crater, eventually spilled over this low point. From there it joined with basalt flows from One Tree Hill, Mt Eden and Mt Albert volcanoes to infill the Meola Valley and form the Meola Reef in the Waitemata Harbour.

A number of scoria cones also formed within the crater. Most of these have been removed or heavily modified by quarrying at, and in the vicinity of, the site, although the cone known as Big King remains adjacent to the western boundary of the quarry.

2.3 Groundwater

Groundwater flow in the basalt aquifers typically follows the drainage direction of the pre-volcanic topography. The groundwater flows within the base of the basalt down the paleovalley system towards the harbours. The main dividing ridge separates groundwater that flows towards the Waitemata Harbour from groundwater that flows towards the Manukau Harbour. The GAS model (PDP, 2005) predicts aquifer groundwater levels and the ultimate discharge locations of groundwater and can be used to infer soakage potential across the Greater Western Springs aquifer area.

Around the quarry, due to the high permeability of the volcanic materials and the low permeability of the surrounding tuff and Waitemata and Tauranga Group rocks, rainfall over the crater soaks into the ground to recharge the groundwater which collects within the crater. This soakage occurs either directly through exposed ground or indirectly through stormwater runoff directed to soakage devices. Groundwater levels in the crater stabilise when the inflows to the crater (in the form of rainfall recharge) are balanced by outflows from the crater. Currently outflows are in the form of the groundwater abstraction for quarry dewatering and are currently balanced by inflows. Prior to the advent of quarry dewatering, groundwater levels within the crater were at approximately RL 56.5 m, (23 m higher than present) and outflows occurred (within the high permeability lava flows) by

spilling over the low point in the tuff ring (at approximately RL 48m) in the vicinity of Landscape Road. This groundwater would then have migrated through the basalt flows infilling the former Meola Valley and eventually discharged to the Waitemata Harbour close to the Meola reef.

Dewatering of the quarry takes place from a borehole located along the southern boundary of the quarry ("The Three Kings Well"). The abstracted groundwater is then discharged into the reticulated stormwater network, to the south-east, which ultimately discharges to the Manukau Harbour. Dewatering began in March 1999 and, since October 2002, has been maintained to hold groundwater levels at the bore, and consequently within the crater, steady at an elevation between RL 34 m and 35 m. The average pumping rate required to do this is approximately 2500 m³/day.

Approximately 1.13 km² (80%) of the urban groundwater catchment to the quarry is drained via dedicated stormwater reticulation and combined sewers (which do not discharge to the quarry). The remainder of the urban catchment is assumed to be treated via soakage which discharges to groundwater. Direct infiltration of stormwater into the quarry floor also contributes to groundwater.

2.4 Surface Water Catchment

The high point of the Meola surface water catchment is approximately bounded by Mt Eden Road around the quarry, St Andrews Rd and then Mt Eden Rd again to the north. The catchment includes the Three Kings area and generally heads westwards, taking in parts of Sandringham, Balmoral, Mt Albert, St Lukes, Chamberlain Park and discharges at Meola Reef.

The main piped drainage system for the Meola catchment consists of combined sewers for stormwater and wastewater. In the upper half of the catchment, drainage is provided by the Edendale Branch Sewer and the Mt Albert Branch Sewer. The Edendale Branch Sewer services the parts north of Balmoral Rd and, in the top of the catchment, north of Landscape Rd (approximately). The Mt Albert Branch Sewer caters for the southern part of the upper catchment including the Three Kings area.

Soakage is a significant component of the drainage for the catchment, servicing approximately 50% of the surface area. Soakage and combined sewer areas overlap. Properties may discharge their stormwater to either the combined sewer or soakage depending upon their historical drainage, geological setting, physical constraints and council re-development requirements.

In the areas around Three Kings quarry, the combined sewer system services development to the west (around McCullough and Smallfield Avenues) and Mt Eden Rd itself. To the south of the site, a 300 mm dia combined sewer runs westwards and increases in size incrementally to a maximum 1950 mm dia pipe at Haverstock Road on the southern branch of the Meola Creek. Here, a constructed overflow throttles storm flows and they overflow to the Meola Creek. The continuation pipe, which services dry weather flow, is a 450 mm dia pipe. Combined sewer overflows are a significant issue within the catchment.

The two main constructed overflows within the catchment are at Haverstock Road and Lyons Ave.

In the longer term two significant projects are proposed by Auckland Council to change the drainage in the Meola catchment. A flood relief pipe system is being investigated to provide high flow capacity, and, a new wastewater tunnel, the Central Interceptor tunnel, is proposed by Watercare to provide additional downstream trunk sewer capacity.

The area to the east (between Mt Eden Rd and St Andrews Rd) is drained by a separated stormwater system to a 1200 mm dia tunnel connected to the stormwater system draining towards Royal Oak and Onehunga.

Soakage is not used to a great extent in the local area around the quarry. This may be due to tuff (volcanic ash) covering the basalt and scoria deposits thereby limiting the near surface soakage capacity. PDP has investigated the soakage capacity of sites adjacent to Mt Eden Rd and found good soakage rates (40 to 60 L/s per borehole). Auckland Council is currently investigating potential soakage in the McCullough, Smallfield Avenues area.

Surface water drains to the quarry low point from within the quarry footprint and from external areas outside quarry footprint. These external areas are; part of the Three Kings Reserve to the west, the upper corner of the site which is currently used as the entrance to the quarry and for commercial buildings, part of the residential development around Smallfield Avenue and playing field to the west of the quarry. Stormwater flows from the residential areas to the south-west of the development may occur once the capacity of the combined sewer system in that area was exceeded.

Flows from the Three Kings shopping centre carpark to the south are piped directly to the land previously operated as a quarry by the Mt Roskill Borough Council. Water flowing on to this land would pond and drain by informal soakage into the ground.

3.0 Water Modelling

3.1 Surface Water Model

Surface water modelling was carried out in accordance with Auckland Council's TP 108 and the associated recommended HEC HMS modelling software. A catchment map is shown in Figure 8 (refer Appendix A). Key model parameters are described below.

3.1.1 Catchment Layout

The stormwater management plan maintains the current flow patterns and caters for existing offsite flows. In broad terms there are three catchments; Catchment A for the southern external areas, Grahame Breed Drive and apartments to the south-west, Catchment B for apartments around the sports field and Catchment C for northern development areas.

As shown in Figure 8, three primary storage areas are proposed in the southern part of the development: a south-western pond/wetland (Pond A), the central sports field (Sports Field B), and a south-western park area (Area C). Pond A provides permanent surface

storage while the sports field and Area C flood only during events greater than the 10 year ARI rainfall event. There is no physical boundary proposed between the flood storage provided by the sports field and Area C.

A number of catchments outside the quarry also discharge stormwater to the development. Overland flow from the catchment south of the quarry (Smallfield Avenue) drains to Pond A. Runoff from the carpark at Three Kings Plaza will be piped underground directly to Sports Field B. Runoff from a portion of Big King Reserve, to the northwest of the site, is conveyed to Area C as well as smaller portions to Pond A and Sports Field B.

Northern areas (draining to Area C) are conveyed through an eastern channel/wetland on the eastern edge of the eastern road. This wetland is divided into four cells identified as wetland cells 1, 2, 3, 4. The modelling is also used to determine:

- an appropriate size and conveyance capacity through this eastern wetland;
- if these flood storage areas A, B and C are sufficient, and
- the need for, and size of, any further flood storage areas that may be required.

3.1.2 Sub-catchment Characteristics

The quarry floor will be raised to its finished level with compacted engineered fill. Pervious areas overlying this fill have been characterised as having a SCS curve number (CN) of 74, which reflects the relatively low infiltration potential of compacted engineered fill and a conservative representation of future topsoils. A CN of 74 is typical for Waitemata Group soils, which share similar properties to the material used for filling. As engineered fill is to be used, there is also some control over the properties ensuring that some infiltration capacity will be provided. Paved and road areas were assigned a CN of 98.

External sub-catchments that overlie basalt or scoria were assigned a CN of 39 to reflect their high infiltration potential. Areas that are already above engineered fill were assigned a CN of 74.

Sub-catchments were assigned a composite CN based on an assumed impervious percentage, estimated from the layouts provided in option 15H1. Residential areas were typically assumed to be 50% impervious, and road reserves were assumed to be 90% impervious. Apartments and immediately adjacent areas may be up to 95% impervious. Where possible, areas outside of the quarry were measured accurately from aerial photographs.

Initial abstractions represent the depth of water that an area can infiltrate at the beginning of a storm. Impervious areas were assumed to have zero initial abstraction, and pervious areas were assumed to have 5 mm. A composite initial abstraction was determined based on the proportion of impervious cover.

Lag times represent the travelling time for the water and determine how much the peak of the runoff is distributed. Lag times depend on factors such as travelling distance, slope and nature of the conveyance and affect only peak flow rates, not runoff volumes.

Table 1 summarises the inputs for the HEC-HMS model.

Table 1: Summary of inputs to HEC-HMS Model				
Sub-catchment	Area (m²)	Composite Curve Number	Initial abstraction (mm)	Lag time (min)
A	3,429	98	0	6.67
A1	20,652	82.4	3.25	6.67
A2	6,796	78.8	4	6.67
A3	7,851	83.6	3	6.67
A.External	64,858	86	2.5	6.67
A.roads	9,106	93.2	1	8.14
B	17,483	98	0	6.67
B1	15,163	93.2	1	6.67
B2	17,949	95.6	0.5	6.67
B.roads	11,588	93.2	1	6.67
B.External	2,601	39	5	6.67
B.Carpark	9,597	98	0	6.67
C	4,247	98	0	6.67
C1	10,244	91.3	1.40	6.67
C2	16,682	83.0	3.13	6.67
C3	25,420	90.5	1.57	6.67
C.External	13,597	50.8	4	8.25
C.roads	13,962	93.2	1	8.44

3.1.3 Rainfall

Rainfall depth data for Hec HMS modelling is typically taken from TP108. The depth data for the Three Kings Quarry was checked using HIRDS V3 for the 2-year, 10-year and 100-year ARI 24 hour rainfall events. These values were 10 to 25% higher than those required by TP108 and have been used as they are conservative. The water quality storm was taken as one third of the 2-year rainfall event, as recommended by Auckland Council's "Stormwater Management Devices: Design Guideline Manual", Technical Publication 10 (TP10).

Climate change was accounted for by applying a factor to future rainfall events based on the average temperature increase. Ministry for the Environment (2008) recommends increases of 4.3%, 6.3% and 8% per degree for the 2-year, 10-year and 100-year rainfall

events respectively. MfE (2008) estimates an average temperature increase in Auckland of 0.9°C by 2040 and 2.1°C by 2090. The adopted events were taken from HIRDS rainfall events and increased to allow for climate change to 2040 (which are greater than TP108 values increased for climate change to 2090).

Rainfall depths were used to produce 24 hour Chicago storm events as per Auckland Council's TP108. The adopted design events are shown in Table 2.

Table 2: Design 24 hour rainfall events		
Event	Depth	Purpose
Water Quality Volume adjusted for climate change	29 mm	Determine volumes to be treated by future treatment devices
2-year ARI	83 mm	Assumed capacity of existing soakage devices
2-year ARI adjusted for climate change	87 mm	Design capacity of future soakage devices when assessing flooding
10-year ARI adjusted for climate change	136 mm	Pipe sizing and new soakage areas Meet condition of no flooding on field during 10-year event
100-year ARI adjusted for climate change	240 mm	Meet condition of habitable floors freeboard above 100-year event

3.1.4 Integration with Soakage and Network Discharge

Areas within the site overlying or adjacent to scoria will discharge to soakage through connections to the scoria rock bed. Soakage devices within these areas are designed to dispose of the 10-year ARI rainfall event. However to generate a conservatively high flood storage volume; runoff from apartments around the perimeter of the site and the eastern wetland channel catchment is assumed to go to soakage only up to the 2 year ARI event, and all runoff from other areas goes directly to the flood storage volume.

External areas draining to the site convey water both to soakage devices and to a combined sewer network. Soakage devices were assumed to only convey the 2-year ARI runoff (to allow for clogging and any insufficient initial design). This 2-year ARI flow rate was an assumption used for the "Central Area Stormwater Initiative" project carried out for Auckland Council in 2012- it was considered that existing soakage systems servicing road catchpits across the central Auckland Isthmus were on average operating at this rate. Combined sewer networks were assumed to convey up to the 3-month ARI runoff. Overflows from these devices drain according to the existing contours, with the areas previously outlined flowing to the quarry.

The 2-year soakage rate was also used within the development site to generate conservative runoff volumes for identifying flood storage volumes within the site. However, sediment loads to the site are expected to be lower than typical urban roads and stormwater treatment will also be provided. As such, the actual discharge rate through soakage devices on site is expected to be higher than predicted.

3.1.5 Model Results

The net runoff volume to flood storage (after allowing for the 2-year event to soakage in selected areas) is approximately 31,000 m³ in the 100-year ARI rainfall event and 15,000 m³ in the 10-year ARI rainfall event.

Model results for individual sub-catchments are set out in Appendix B.

3.2 Groundwater Model

A simplified three-dimensional numerical groundwater model was constructed to estimate the rise in the natural groundwater level (above RL 56.5 m) for non-pumping conditions as a result of the 10- and 100-year ARI rainfall events. The modelling code selected for this assessment was Visual Modflow version 4.6.0.162. The conceptual model and model development are discussed in the following sections.

3.2.1 Conceptual Model

The conceptual model for the groundwater within and outside the cone is discussed previously (PDP 2002 and 2008). The main elements of the conceptual model are summarised below.

Geology

The geology in and around the Three Kings quarry can be divided into four main groups: basalt, scoria, tuff, and non-extrusive sediments (Waitemata Group). Basalt, scoria and some tuff dominate the quarry area and form a roughly circular outcrop defined by the volcanic crater. The basalt can be found intercalated with, or intruded into, the scoria and tuff forming a high elevation lava moat around the periphery of the crater. The main bulk of tuff and non-extrusive sediments (principally Waitemata Group and younger Tauranga Group) occur outside this area. The geological map is shown in Figure 6 (refer Appendix A).

Groundwater Level

Pre-pumping water levels within the cone are around RL 56.5 m and are similar for all bores within the cone. This is based on the average groundwater levels in the cone (basalt and scoria) for the pre-dewatering conditions (1993-1999) as shown in Table 3.

Table 3: Average Pre-dewatering Groundwater Levels in Scoria/Basalt within the Three Kings Crater	
Bore ID	Groundwater Level (RL, m)
BH1B	56.77
BH2B	56.55
BH5B	56.12
BH17	56.59
BH3	56.51
BH14	56.60
Average	56.52

The groundwater dewatering at the cone started on 6 April 1999 and currently is held through pumping at approximately RL 34 m.

The seasonal fluctuation of groundwater levels in the centre of cone (BH2B) before the quarry dewatering (1996 to 1999) is shown in Figure 2 (below). The seasonal fluctuation in this bore (based on monthly data) is approximately 1 m.

No short-term groundwater level data is available for any of the bores in the cone. However, as an indication of the potential groundwater level variation in basalt, the Global Aquifer Study Leslie Avenue Bore (in Sandringham) showed a 1.5 m rise in the groundwater level in response to an event (84 mm/24 h) on Feb 2004 (PDP 2005). This is because the maximum variations in groundwater level in response to rainfall are controlled by the local aquifer conditions (aquifer storage parameters) rather than effects transmitted from up gradient parts of the aquifer. Due to the open nature of the basalt aquifer and the rapid transmission of rainfall recharge to the water table through soakage devices, the local recharge and the local aquifer conditions determine the water table rise. For the quarry, where scoria and tuff are also present, aquifer storage will be greater and the variation in groundwater level in response to rainfall is expected to be less than that at the Leslie Avenue bore.

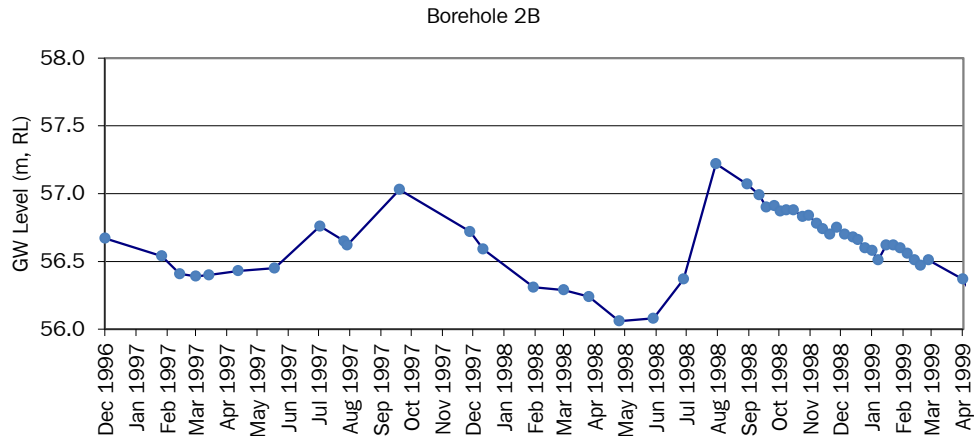


Figure 2: Seasonal Groundwater Fluctuation in the Cone (BH2B, before dewatering)

Water levels in the cone both in the basalt and scoria have behaved similarly suggesting that there are no significant hydraulic barriers within the crater.

The main groundwater level in the Waitemata Group adjoining the cone ranges from RL 60m to RL 35m with perched groundwater in various zones at higher elevation. Following, the cessation of the groundwater abstraction within the Three Kings volcanic cone, the groundwater level within the crater will recover to its pre-dewatering level (RL 56.5 m). This will result in a resumption of the groundwater flow reaching the Western Springs Aquifer through the tuff ring breach and cause the groundwater levels in downstream areas such as Meola and Western Springs to return to the pre-pumping levels. The results of the above study indicate that the tuff ring breach is approximately 8.5m below the pre-dewatering groundwater level of RL 56.5 m. This means if the groundwater level in the cones was kept at approximately RL 48 m (or lower) there would be no increase in groundwater flow from the crater towards Meola and Western Springs.

Hydraulic Properties

Previous work (PDP 2002 and 2008) has shown high permeability volcanics within the volcanic cone (2×10^{-4} m/s) and lower permeability for the disturbed Waitemata Group sediments in a 200 m wide collar outside the cone (9.4×10^{-7} m/s). Based on the GAS study (PDP 2005), the specific yield in the volcanic aquifer (basalt and scoria) ranges from 0.01 to 0.08 (1 to 8%).

Recharge

Before dewatering began, rainfall recharge to the crater would have been balanced predominantly by groundwater outflow in the form of overspill to the basalt flow to the north. Since the onset of steady-state dewatering the rainfall recharge has been balanced by outflow through the dewatering abstraction. The total long-term groundwater inflow to the quarry under the current dewatering conditions is approximately 2,500 m³/d. This includes some groundwater contribution from the surrounding Waitemata Group (disturbed zone) which is diverted to the pit.

Recharge has been estimated previously to range from 120 mm/year (or approximately 10% of the annual rainfall) for the disturbed Waitemata Group to approximately 88% of the rainfall for the volcanic cone.

3.2.2 Model Set-up

System Geometry

The model domain contains the volcanic cone and about a 200 m zone of the disturbed Waitemata Group with a uniform grid spacing of about 10 m and nine layers. The basalt and scoria within the cone are simulated with a thickness of about 400 m. The layout is shown in Figure 3 below.

Model Input Parameters

The model input parameters are based on the conceptual model discussed above. The aquifer hydraulic properties represent average values over the whole saturated thickness of the layer. The areas of different rates of infiltration during the rainfall event are shown on Figure 7 (refer Appendix A). Area D has the highest infiltration rate, being the remaining scoria cone. Areas E and F have low rates of infiltration as they are predominately impervious. Areas A, B and C have moderate rates of infiltration as they represent residential development with moderate amounts of imperviousness.

Boundary Conditions

“No flow boundaries” represent the boundary between active and inactive model cells (across which there is no flow). This boundary was assigned outside the disturbed Waitemata Group as any throughflow from the undisturbed Waitemata Group to the cone is minor. The no flow boundary was also assigned to the north of the tuff ring breach.

The drain boundary was assigned to nodes located along the tuff ring breach between RL 56.5 and RL 48 m. The head along the drain cells was set at the pre-dewatering groundwater level (RL 56.5 m). Drain cells remove water from the aquifer at a rate proportional to the difference in heads between the aquifer and the water in the drain. The drain cells are active only if the groundwater level in the cone is higher than the drain elevation.

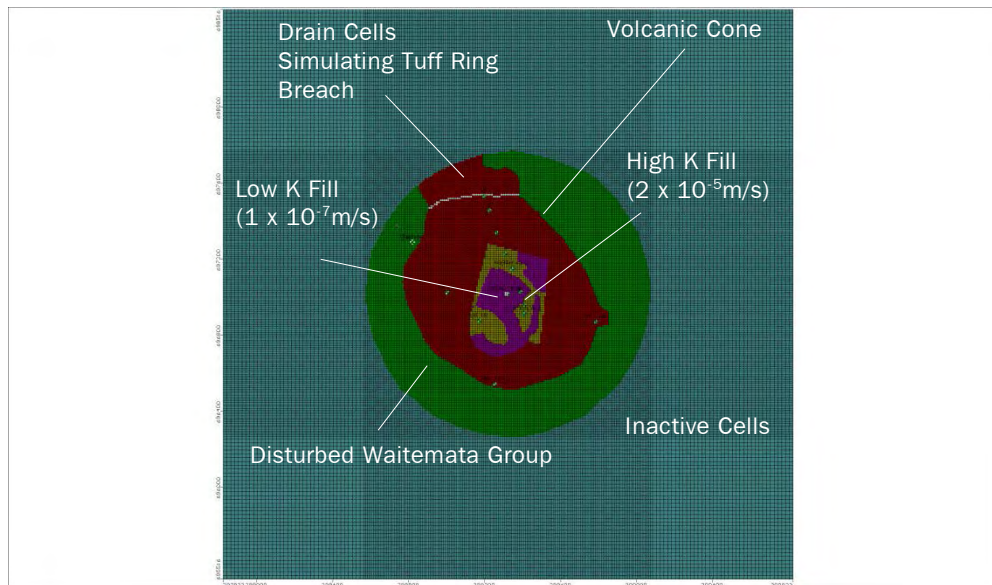


Figure 3: Model Grid and Boundary Conditions

3.2.3 Model Calibration

Steady State Calibration

The steady state calibration was carried out based on groundwater levels for pre-dewatering conditions within the cone and stabilised average groundwater pump out. The aim of the calibration was to define hydraulic conductivity and long-term recharge over the cone area. The average pre-dewatering groundwater levels (refer Table 3) in the cone were used for this calibration.

For this model, calibration was focussed on matching the historical no pumping groundwater level responses within the crater with the expected recharge of the scoria and basalt zones.

The Root Mean Squared calibration coefficient of about 0.5 m was considered appropriate as the calibration target. The parameters used in the calibrated model are in agreement with the results from the previous work (PDP 2002 and 2008) and are shown in Table 4.

Table 4: Steady State Calibrated Parameter		
Parameters	Basalt/Scoria	Disturbed Waitemata Group
Hydraulic Conductivity (m/s)	2×10^{-4}	9.4×10^{-7} ⁽¹⁾
Recharge (mm/year)	1,000 (83% of rainfall)	120 (10% of rainfall)
1) Based on PDP (2008)		

The calibrated water balance result is set out in Table 5. A less than 0.5% difference was achieved between the average simulated and the observed groundwater discharge.

Table 5: Groundwater Mass Balance	
Calculated groundwater output (through drain cells) (m ³ /d)	Measured groundwater pump- out (m ³ /d)
2,509	2,500

Transient Calibration

The transient calibration was undertaken to estimate the basalt storage parameters. The observed groundwater level response to the pumping in the cone (BH2B) and average pumping rates for about 13 years (from 1999) were used for this calibration. The average pumping rate during this period was reduced from approximately 5000 to 2500 m³/d. The storage coefficients were adjusted until the modelled groundwater fluctuations generally matched the measured levels. The calibration result is given in Figure 4 (below).

The best calibration was achieved using the specific yield of 0.08. This is in agreement with the upper range of the storage identified as part of the GAS study (0.01 to 0.08).

This calibration is currently being checked further through the direct measurement of water levels in the quarry bore 2B in response to rainfall.

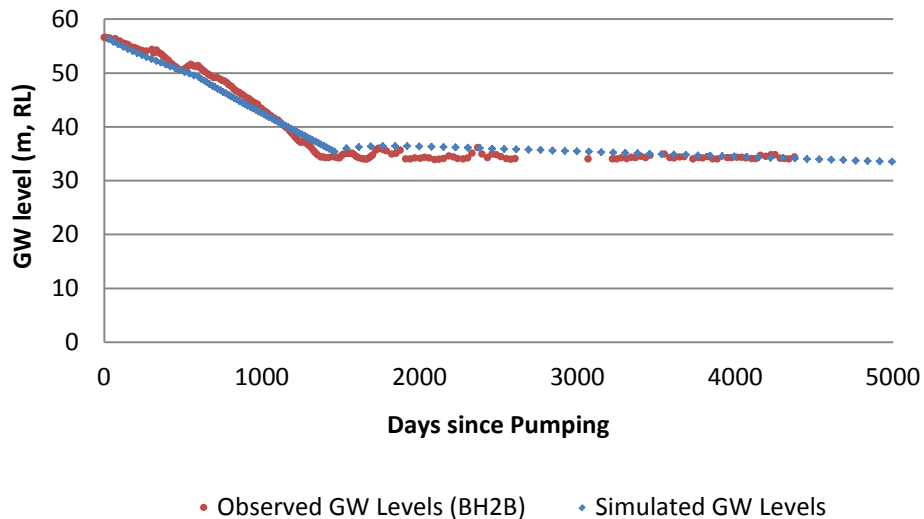


Figure 4: Calibration of Transient Groundwater Model

3.2.4 Model Results

The calibrated model was setup to simulate effects of rainfall induced infiltration from peak flows on groundwater levels in the cone as a result of 100-year ARI 24 hour duration rainfall event.

For the simulations the backfill to the quarry was added to the model to match the conditions that will exist when the development is complete. The fill permeability was set between 2×10^{-5} and 1×10^{-7} m/s. However, as the runoff over the fill will be diverted to soakage in scoria/basalt, the fill permeability does not affect the predicted water level rise in the cone.

The modelling results show that a 100 year ARI rainfall event causes approximately a 1.2 m rise in the groundwater level in the centre of the cone. Considering the observed seasonal variations in groundwater levels for bores in the cone (refer Figure 2 above), such a rise is not unexpected. Based on the historical data a larger response to such a rainfall event is expected in bores drilled in basalt outside the cone. For example, during the GAS study, it was identified that a 1.5 m rise occurred in the Leslie Avenue bore (Sandringham) in response to an event on the 2 Feb 2004 (PDP 2005). A smaller predicted groundwater level rise in the cone is not unexpected considering the occurrence of more scoria in the cone (giving higher storage).

The results show that the groundwater level recovers gradually (within 1.5 months) after the event (refer Figure 5 below).

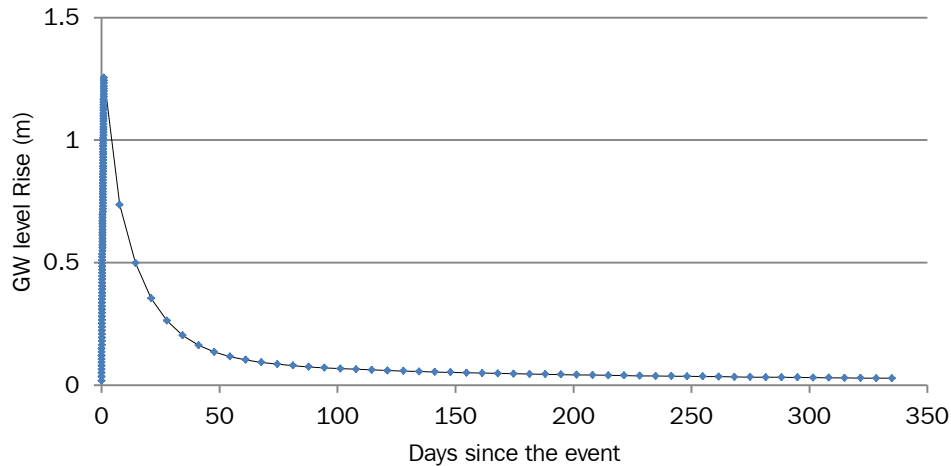


Figure 5: Groundwater Level Rise after 100 year ARI Rainfall Event

Maximum Groundwater Table Design Levels

Maximum water table design levels were derived from the various considerations as follows:

100-Year ARI rainfall event

Normal crater water level (no pumping)	RL 56.5 m
Normal seasonal maximum water table level variation	0.5 m
Rainfall event water table response	1.2 m
Allowance for future soakage development in surrounding area	0.3 m
Minimum design water level for surface water storage	RL 58.5 m

For the 10-year ARI rainfall event a response of 0.8 m was used giving a water table design level of RL 58.0 m. The development requires the quarry to be filled to above these levels for all residential areas and roads.

The 0.3 m groundwater level rise for future soakage development in the surrounding area represents an increase in the average infiltration rate by 24 mm/day in zones C and G shown on Figure 7 in Appendix A. This area includes Fyvie, Smallfield and Henshaw Aves where the primary stormwater discharge is to the combined sewer system. The 0.3 m rise allows for some stormwater to be diverted from the combined sewer to soakage. The exact amount diverted depends upon the current amount captured by the combined sewer and the extent of existing impervious area in these zones. Assuming pervious parts of these zones already drain to soakage, the 24 mm/day increase in average infiltration across the zone represents a higher rate of new infiltration/soakage from existing impervious areas.

4.0 Design Considerations

4.1 General

A number of design considerations and constraints directed the stormwater management plan. The design parameters are closely related and result in an iterative process of determining an appropriate stormwater management plan. These included:

- ✧ Building floor levels and freeboard to flood levels;
- ✧ Grades and water levels for primary and secondary drainage systems;
- ✧ Flood levels;
- ✧ Likely depths for other services;
- ✧ Road levels and geometry;
- ✧ Location and volume of flood storage zones;
- ✧ Stormwater quality treatment devices locations and geometry;
- ✧ Soakage discharge points.

General Requirements

The Proposed Auckland Unitary Plan (PAUP), in Section 4.12 3.1, identifies the following matters of discretion when processing consents related to flooding:

- a. design of the structure, works or infrastructure
- b. extent of any earthworks proposed
- c. construction methodology
- d. potential impacts on overland flow paths including:
 - i. obstruction of flows
 - ii. any change to location and capacity
 - iii. any change to overland flow on other properties.
- e. provision of secondary flow paths
- f. effects on existing infrastructure
- g. potential changes in flood depth and frequency upstream and downstream of the site and potential flooding of habitable floors
- h. provision of site access and potential effects of chosen access route
- i. ongoing access, maintenance and reporting requirements
- j. methods of providing for long term maintenance and protection such as easements

4.2 Design Standards

Hydrology

Surface water hydrology has been assessed generally in accordance with Auckland Council TP 108 as described in Section 3.1.

The Auckland Council Code of Practice for Land Development and Subdivision, Oct 2013, Part 4, Stormwater, Section 4.3.5.1 requires estimation of surface water run-off shall be carried out in accordance with TP108.

Groundwater hydrology has been assessed as described in Section 3.2.

Consideration of Climate Change

The potential effects of climate change have been factored into the stormwater designs.

The assessment was carried out in accordance with MfE document "Preparing for Climate Change" (2008).

Reticulating the 10 year ARI Rainfall Event

A combination of soakage and stormwater reticulation on the site will dispose of the 10-year ARI rainfall event to groundwater or storage without generation of overland flows.

The Building Code, Performance Standard E1.3.1 requires the effects of stormwater on other property to be managed up to a 10% probability event.

The Auckland Council Code of Practice for Land Development and Subdivision, Oct 2013, Part 4, Stormwater, Section 4.3.5.2 requires primary drainage systems be designed for the 10% AEP event. Note that the 10% AEP event and the 10-year ARI event are considered equivalent.

Providing Overland Flow Paths for the 100-Year Event

Up to the 100-year ARI rainfall event, overland flow paths should be provided which convey stormwater with a minimum of nuisance.

The Auckland Regional Plan: Air, Land, Water controlled activity Rule 5.5.2 includes a requirement for overland flow paths to be provided and discharge with a minimum of nuisance and damage up to the 100-year ARI rainfall event.

The Auckland Council Code of Practice for Land Development and Subdivision, Oct 2013, Part 4, Stormwater, Sections 4.3.4.2 and 4.3.5.2 requires a secondary drainage system, consisting of ponding areas and overland flow paths, be designed for the 1% AEP event.

Providing Underground Flood Storage for the 10-Year Event

The 10 year ARI runoff volume should be stored sufficiently so that there is no surface flooding on the sports field during the 10-year event. Fletcher Residential Ltd requires this to ensure only very occasional interruption to the use of the sports field. Auckland

Council have advised that they support in principle the use of public spaces such as parks and carparks for flood attenuation for events greater than 10 years.

Providing Flood Freeboard for the 100 Year Event

Storage should be provided to ensure that appropriate freeboard is provided to habitable floors during the 100-year ARI event.

The PAUP gives permitted activity status to “vulnerable activities in flood sensitive areas” that have 500 mm freeboard above the 1% AEP flood level (Part 3: Chapter H, Section 4.12 (2.1.1) (1)). The Auckland Council Code of Practice has the same requirement. The Auckland Regional Plan: Air, Land, Water controlled activity Rule 5.5.2 includes a requirement for 500 mm freeboard between the 100 year ARI flood level and habitable building floor levels.

Notwithstanding this, the design has been developed with a higher standard of freeboard – namely floor levels are set above a hypothetical flood level assuming there is no short term soakage occurring and allowing for the entire 100-year ARI rainfall depth (i.e. not just runoff) to be stored. This means floor levels are at least 750 mm above the 100-year ARI flood levels identified for flood storage areas A, B, C.

Providing for Ultimate Disposal of Stored Stormwater

Stormwater which is stored during and after rainfall events should be eventually discharged via soakage or another means.

The fractured basalt and scoria which makes up part of the quarry walls and floor provides capacity to discharge surface water to ground via soakage. This is an effective and desirable means of stormwater discharge as it reduces storage requirements and peak flows within the development. It also contributes to recharge of the local aquifer. Soakage should be utilised to the fullest extent practical.

Face mapping of the quarry walls (refer Tonkin & Taylor Figure 2 in Appendix A) has identified scoria areas where it is proposed to discharge via soakage. These are: generally around the perimeter road in between apartment blocks, the scoria pit in the south western corner of the quarry (pond A is to be placed above this) and a “peninsula” of scoria extending beneath the sports field from the south.

Tonkin & Taylor Ltd are monitoring the placement of fill material within the quarry and they have provided soil void ratios to be used for calculating underground flood storage. They have advised that no underground flood storage should be assumed within one metre depth of the surface to allow for road construction and compacted fill/landscaping over the sports field and residential areas. Within one to two metres depth, they have advised that a graded filter material be placed with a void ratio of 15 to 20% and below this the bulk fill can be assumed to have a void ratio of 25%.

Water Quality Treatment

All contaminant generating surfaces such as roads and roofs should discharge to a water quality treatment train prior to ultimate disposal of stormwater. These treatment trains should be integrated into public amenities such as gardens and wetlands to maximise use of available space.

The Auckland Regional Plan: Air, Land, Water controlled activity Rule 5.5.2 includes a requirement for removal of 75% total suspended solids from any new impervious surface greater than 1000 m². The PAUP has a range of stormwater quality provisions under Part 3, Chapter H, Section 4.14 (3).

Sizing the treatment devices will be consistent with the former Auckland Council's TP10 as required by the Auckland Council Code of Practice for Land Development and Subdivision, Oct 2013, Part 4, Stormwater, Section 4.3.6.1.

We understand Fletcher Residential Ltd has worked closely with iwi to ensure that this element of the proposal will deliver a sustainable long term solution.

4.3 Options Analysis

A qualitative assessment of options for the stormwater management approach is provided in Appendix D.

The matrix of the options in Appendix D shows that all options have common stormwater infrastructure requirements such as an internal stormwater pipe system and stormwater quality treatment. All options also require some form of flood storage on site – either because the proposed development levels are below the surrounding area’s ground levels or because flow detention is required to avoid effects on downstream pipe systems and flood prone areas. Therefore these are assumed to be generally similar in terms of cost.

Options where the quarry is filled to the surrounding ground levels incur additional cost and time to obtain fill and reduced revenue from a reduced development yield. These have therefore been discounted.

Of the options where the development is constructed at levels of approximately RL 59m to 65m, the tunnel option incurs significantly greater cost in terms of tunnel construction. Furthermore significant flow detention and possibly also downstream pipe upgrades are expected indicating additional cost and practicality issues. This has therefore been discounted. The two soakage options are considered broadly similar in terms of stormwater capital cost - however the pumping option introduces ongoing operational cost. A soakage system can be designed without pumping being required and this is considered preferable in terms of long term robustness and certainty. Soakage systems receiving high sediment loads are susceptible to blockage. However in this case, long term sediment loads are expected to be relatively low as they are from residential development and stormwater treatment will manage the loads generated.

Iwi have identified that they prefer the discharge to be to the Meola catchment. This means that they prefer a discharge to soakage without on-going pumping.

The “soakage without ongoing pumping” option avoids potential effects on existing flooding areas off-site and is considered to a practical solution for stormwater management on site. This option is therefore used as the basis for the development.

5.0 Proposed Stormwater Management

5.1 Design Summary

Stormwater runoff within the Three Kings Renewal project will be managed through a combination of soakage, reticulated networks, stormwater treatment and overland flow. Treatment will include sedimentation ponds, a wetland channel with four cells on the eastern side of the site, swales and rain-gardens. Surface water which does not infiltrate and is not directly discharged to soakage will be conveyed to soakage areas adjacent to the wetland and three main storage areas toward the south of the site. Pond A utilises both below and above ground storage. Sports field B and Area C uses below ground stormwater storage under the adjacent roads and will provide additional above ground storage during greater events. Additional sites for underground storage will be provided by large soakage chambers adjacent / under the eastern wetland channel and at soakage areas for individual apartment blocks.

The two most limiting design features for storage are the upper and lower elevation limits available for water. The lower limit for the 10- and 100-year ARI rainfall events are the respective groundwater levels. The upper limit for storing the 10-year ARI rainfall event is the surface of the field (accounting for freeboard) and the upper limit for the 100-year ARI rainfall event is determined from the freeboard for habitable floor levels.

It is desirable to have the floor levels as low as feasible so that the maximum space is available for apartment blocks; however these must have adequate freeboard above the 100-year ARI flood level. It is also desirable to have the sports field as low as possible (while maintaining field serviceability up to the 10 year event), as surface flooding provides a significant amount of storage for the 100-year ARI rainfall event. Finally, development areas need to be suitably drained to provide competent foundations under roads and buildings – underground storage is therefore not contemplated within 1m of the ground surface and only limited storage within a graded filter material is provided between 1 and 2m depth below ground level. Soakage areas, underground storage, ponds, wetlands and the field level were iteratively adjusted to optimise these parameters.

The 10-year groundwater level was calculated as RL 58.0 m. The field level was originally specified at RL 58 m in Option 15D, however this leaves no storage depth or room for the field foundations. For this reason, the field was raised to RL 59 m, allowing for the foundation layer and up to 1m of storage in the adjacent areas under the perimeter road.

The short term groundwater level in response to the 100-year ARI rainfall event was calculated as RL 58.5 m. As such, roads around the field were set to a minimum of RL 60 m, allowing for up to 1.5 m of storage in Pond A, and up to 1 m of storage on the surface of the sports field and Area C. Habitable floor zones must be above the 100-year flood level (as well as any overland flow paths). As stormwater pipes are only designed up to the 10-year event, the excess will be conveyed through overland flow on the road. For this reason, habitable floors must also have freeboard above the water level in adjacent overland major flow paths (down the roads).

Soakage capacity is available at a number of locations within the development, primarily where a connection to the scoria cliffs or underlying scoria rock are available. Soakage adjacent/below the eastern wetland cells and flood storage areas A, B, C will provide drainage for the majority of the site. Soakage pits will be separately provided for individual apartment blocks in the west.

The overall Stormwater Management Concept is shown on Figure 9 (refer Appendix A).

5.2 Main Development System

5.2.1 Flow Paths and Reticulated Network

Flow enters the quarry from a number of off-site catchments identified on Figure 8 (refer Appendix A). These contributions will be collected via reticulation or overland flow paths.

A reticulated network will be provided to convey stormwater from the central part of the development towards the east into a wetland channel and flood storage areas A, B and C in the south.

Areas around the sports field will be either reticulated to Pond A and cell 4 of the eastern wetland channel and then into flood storage, or, reticulated to rain-gardens around the perimeter of the sports field and then into flood storage.

5.2.2 Flooding

Flood Storage

The total runoff from the 100-year ARI rainfall event is approximately 31,000 m³, to be stored within the site. Above and underground storage volumes at Pond A, Sports field B and Area C are approximately 3,500 m³, 18,000 m³ and 4,000 m³ respectively, for a total of 25,500 m³. Additional storage chambers have therefore been included, and storage of the 10-year and 100-year runoff volumes therefore comprises of:

- ✧ Surface storage on Pond A, Sports field B and Area C;
- ✧ Underground storage around Pond A and under roads adjacent to Sports field B and Area C;
- ✧ Underground storage in the soakage chambers adjacent to and under the wetland cells; and
- ✧ Underground storage in the western soakage chambers (for the individual apartment blocks – refer section 5.3).

All other storage areas (such as surface storage in the wetland and conveyance devices) are assumed negligible.

Surface storage in Pond A, the Sports field and Area C is provided over the storage areas indicated in Figure 11 (refer Appendix A). Pond A is assumed to hold some permanent water; it is assumed it fills up due to short term groundwater rise. Above RL 58.0 m and 58.5 for the 10-year and 100-year rainfall events respectively, the pond can provide

surface storage. The sports field and Area C holds surface water above its surface, RL 59 m. It holds no surface water in the 10-year ARI rainfall event. Note there is no physical barrier between the sports field and Area C flood storages.

Underground storage is provided in the void spaces in the construction fill adjacent to Pond A, below the roads adjacent to the sports field and Area C as shown in Figure 11. This is necessary to ensure that no flooding occurs on the sports field in the 10 year ARI rainfall event. The areas are hydraulically connected to ensure a constant level across the fill media. Tonkin and Taylor advised that a void ratio of 25% can be reliably achieved for this storage material. A 1m layer (with no storage) is included below all surfaces for foundation purposes with a further 1m graded fill layer required under residential areas and roads. Tonkin and Taylor specify that this graded fill will have a storage capacity of approximately 15% to 20%. These underground fill areas are hydraulically connected to the surface storages in Pond A, Sports field B and Area C, and therefore maintain a constant level while filling.

Underground storage is also provided in the soakage chambers adjacent/under the eastern wetland cells (also shown on Figure 11). Soakage is provided below and adjacent to the wetland channel for events up to the 10-year runoff volume, with the chambers also functioning as storage devices. Similarly, underground storage and soakage is provided in two south-western soakage areas for apartments off Grahame Breed Drive and four soakage areas servicing western apartments. These provide storage and soakage for runoff from residential areas and roads draining to them, reducing the storage requirements downstream. These soakage areas are not hydraulically connected to the sports fields or other soakage devices, and therefore are able to fill above RL 59m and RL 60m for the 10-year and 100-year rainfall events where available.

The storage available for 10-year and 100-year runoff volumes could be determined from a storage-elevation curve. The total storage available to RL 59 m and RL 60 m for the 10-year and 100-year volumes is shown in Table 6. The following points are noted:

- ∴ Underground storage at areas A, B and C is less for the 100-year runoff volumes than the 10-year due to the increased groundwater level in the 100-year event.
- ∴ Approximately 60% of the 100-year event is managed by surface storage at areas A, B, and C; approximately 75% of this storage is from flooding the sports field.
- ∴ Soakage chambers elsewhere in the development are designed to hold up to the 10-year runoff volume only. It is conservatively assumed that they provide no additional storage/soakage in the 100-year event. In practice, the chambers would be full for a greater duration of the storm, and therefore provide increased soakage.
- ∴ Storage values provided in the soakage chambers conservatively do not account for losses during the storm to soakage. However, runoff values from apartments managed by soakage have already accounted for the soakage reduction.

- ✧ It is considered that both the area draining to soakage and the effectiveness of soakage is currently understated. This is to ensure that the calculated flood storage volumes over cater for incoming water. This will be refined during the site investigation and detailed design phases.

Table 6: Storage provided for 10 and 100 year ARI Runoff Volumes		
	10-year	100-year
Total runoff volume (after soakage)	15,265	31,385
Above ground at A, B and C	664	19,482
Underground storage at A, B and C	5,678	5,900
Wetland storage	4,913	3,688
Western Apartments Soakage/ Storage	4,335	4,335
Excess Storage Available, m³	325	2,020

Water which is stored above and below ground is discharged over time to the surrounding aquifer via connections to the scoria rock bed. Connections to the scoria bed exist below the sports field, in the cliffs adjacent to Pond A and adjacent to the wetland channel.

Floor Levels

Once flood storage requirements and flood levels were determined, site levels and minimum floor levels were identified and iterated. This process was carried out in conjunction with Harrison Grierson who are designing the detailed site levels and infrastructure services.

Floor levels adjacent to the sports field were set based on either:

- ✧ keeping building floor levels above the 100-year flood level with required freeboard;
- ✧ keeping road low points above 100-year ARI flood levels and minimum crossfall and height requirements for roading and landscaping; or
- ✧ setting the outlets from local drainage pipes from the apartment blocks above the 10 year ARI flood level, hydraulically following this back through treatment and collection devices and allowing for required cover over the pipes.

Floor levels of apartment blocks adjacent to the eastern wetland channel were set based on the 100-year ARI flood level in flood storage areas with backwater allowances made for flood flows in the wetland and then required freeboard. Note that the freeboard requirements have been increased around Area A, the sports fields and Area C to allow for storage of the full volume of the 100 year ARI **rainfall** event.

This resulted in different minimum building floor levels being set for various development areas. Minimum floor levels due to flood freeboard around the sports field are RL 60.75 m and at the critical location on the eastern wetland channel, Block A-8 must have a minimum floor level of RL 61.0 m.

Table 7: Flood Levels in the 100 year ARI Rainfall Event		
Location	Water Level.	Minimum floor level
Around Sports Field	RL 60.0 m	RL 60.75 m
Wetland Cell 4	RL 60.1 m	RL 60.75 m
Wetland Cell 3	RL60.45 m	RL 61.0 m
Wetland Cell 2	RL61.3 m	RL 61.8 m
Wetland Cell 1	RL61.6 m	RL 62.1 m

Floor levels for the apartment blocks have been determined and are all set above the relevant value and are shown on Figure 11 (refer Appendix A).

Floor levels for other buildings, such as other housing and sports field club rooms, will also be set above the relevant minimum floor level.

5.2.3 Soakage

A soakage chamber is proposed below each of the four cells of the eastern wetland channel. These chambers will provide storage and soakage of all water entering the wetland up to the 10-year ARI rainfall event. These are therefore expected to be significantly larger than the chambers addressing an individual lot. These significantly reduce the volume required to be stored at the Pond A, Sports fields B and Area C.

In addition, Pond A and Sports field B will be hydraulically connected to the aquifer through connections to the underlying scoria, and discharge stored runoff over the 48 to 72 hours following a storm. Area C discharges to the aquifer through a connection to Sports field B.

5.2.4 Stormwater Treatment

Source Control

Roof water quality is assumed to be clean and not require stormwater quality treatment provided no exposed metal roofing products are used. The use of pre-painted steel roofing is suggested. Notwithstanding this, roof flows and road flows from most of the main development area will be mixed and pass through the eastern wetland cells.

Swales and Tree pits

Within catchment B, roads will mostly be treated either by swales or tree pits, following which they will drain to soakage at the sports field. A rain-garden will be provided for the

service lane within Super Lot C. There are opportunities for swales or tree-pits for roads in catchments A and C also (which are shown on Figure 9H1 in Appendix A), however these are not required in addition to treatment provided by the eastern wetland and Pond A.

Tree pits will be formed by a local depression below footpath or ground level with a diversion of roadside water via a kerb cut into the depression. Soakage will be free to occur through the base of the tree pit media – no box chamber is proposed to contain and collect flow.

Sedimentation Basins

For flows draining to Pond A or the wetland channel within catchment C, primary treatment will be provided by sedimentation basins designed to remove coarse sediments. These will be sized to operate as sediment forebays and will therefore be sized at approximately 15% of the water quality volume as set out in TP10.

Flows from the upper sections of the entry road from Mt Eden Road will be treated by a sedimentation chamber, following which they will discharge to soakage. The lower section of this road will drain to the wetland channel.

Pond A

Pond A will be sized to provide treatment for Grahame Breed Drive.

Sediment forebays will provide initial treatment for flow entering the pond. Wetland vegetation will be established within and around the pond. Water will discharge to a soakage area under the pond.

Underground flood storage is provided in the areas adjacent to the pond. This will be fed from water which has firstly passed through the pond and a filter media to remove sediment.

Wetland Channel

The eastern wetland channel will have a permanent water level which provides storage for the water quality volume for the contributing catchment. The water quality volume for the wetland catchment is 1032 m³ and the available permanent volume within the wetland is at least 1640 m³.

4,875 m³ of subsurface storage will be provided adjacent/below the wetland channel to make up the total flood storage volume required. Flow to the flood storage and soakage areas will be fed from water which has firstly passed through the wetland and a filter media to remove sediment. Flow from the wetland cells will preferentially be discharged to soakage areas along the eastern quarry face. Flows in excess of soakage capacity at these points will flow south to flood storage areas A, B, C.

In dry periods open water will be present in the main channel sections and any margin sections that do not also operate as soakage areas. To maintain permanent water levels within the wetland cells it may be necessary to line the wetland cells. A small constant

feed of water inflow is also proposed at the head of the wetland in the north-west corner of the site to top up water levels and discourage stagnant pools forming.

In larger events, the wetland will act as a channel, conveying surface flows from sub-catchments in the north of the development to the sports field. A culvert will convey flows from wetland cell 4 under the road into Area C.

The interaction of the storage, water levels and diversion of flows from the wetland channel to soakage discharge points is shown on Figure 10 (refer Appendix A). A planting programme will be implemented for the wetland to provide treatment for water, and to allow it to be integrated into the landscape as a visual amenity.

5.3 Individual Apartment Blocks

Soakage devices are proposed for some individual apartment blocks. At this stage, six are proposed (as shown on the western edge of Figure 9 – refer Appendix A). These provide treatment, soakage and storage for up to the 10-year runoff volumes.

Roof water quality is assumed to be clean and not require stormwater quality treatment provided no exposed metal roofing products are used. The use of pre-painted roofing is suggested.

Soakage devices have been designed based on the Auckland Council Soakage Design Manual (2003).

A conceptual design was developed for a soakage device and treatment train for a typical 5,000 m² apartment block in the north-western residential area. This assumed a soakage rate of 0.4 L/min/m², though this requires confirmation testing and may be much higher than this. The design consists of a sedimentation chamber followed by a combined rain-garden/soakage chamber. The dimensions of each device are shown in Table 8.

Table 8: Individual Apartment Blocks Treatment Train to Soakage		
Device	Size	Runoff managed
Sedimentation chamber	1.8 m diameter (2.5 m deep)	Coarse sediment treatment for vehicle and landscaping areas.
Collection chamber	1.5 m diameter (1 m deep)	Roof water and pre-treated runoff from sedimentation chamber (i.e. all areas)
Combined rain-garden and soakage pit	4.5 x 25 m (5 m deep)	Pre- treated runoff from collection chamber chamber. Further treatment and storage provided in rain-garden and then discharges to aquifer

This design is based on a conservative estimate of soakage rates, and demonstrates feasibility to service up to the 10-year rainfall event while providing water quality treatment. It is noted that the volume of the soakage chamber could be reduced if the soakage capacity is found to be greater than assumed.

It will be necessary to test and confirm soakage rates at the proposed discharge location prior to detailed design being carried out.

5.4 Multiple Flood Events

Surface water runoff stored on site in the flood storage areas will drain via soakage into the groundwater table. This is in addition to the rise in groundwater level due to infiltration (calculated in the section on groundwater modelling). Drainage of the flood storage volume will be governed by a combination of; the rate of the receding short term rise in groundwater level, lateral dissipation of water to fill up rock void space in the underground basin, and (depending upon the rate of the first two mechanisms) additional surface water storage.

The groundwater model shows that the short term ground water level rise will fall by 0.75 m in the first 7 days (refer Figure 5). The model is to be re-calibrated and the rate of recession recalculated following the current monitoring of the short term groundwater level response to rainfall events.

The volume of underground storage within the 1000 m diameter Three Kings volcanic crater is approximately 62,000 m³ per vertical m assuming an average 8% voids in the volcanic materials. This volume is conservative as while 8% is appropriate for basalt the void ratio of the scoria layers present will be 30 to 50%.

The runoff volume in the 100-year event stored in above ground storage is 25,500 m³. Connections from the above ground stored runoff volume to soakage will need to be engineered to allow this stored runoff volume to soak away quickly enough to make space for a potential subsequent rainfall event.

The potential rise in groundwater level following the rainfall event is assessed as:

- ✧ Design short term groundwater level in 100-year rainfall event = RL 58.5 m
- ✧ Groundwater level rise due to soakage of stored runoff = 0.41 m (25,500 m³ / 62,000 m³)
- ✧ Fall in short term groundwater level after 24 hours = 0.10 m (0.75 m / 7 days)
- ✧ Potential ground water level 24 hours after the rainfall event: RL 58.81 m

If soakage into the surrounding rock was not quick enough, there would still be water in the flood storage areas A, B, C during a second extreme rainfall event. In this case, a conservative check on potential flooding is to add a second 100 year ARI runoff volume onto the existing 100 year ARI flood level of RL 60.0 m – i.e. the flood level for the second 100 year ARI event = RL 60.0 m + 31,000 m³ / available above ground area. The area above Pond A, Sports fields B and Area C is approximately 3 ha. The potential flood level from a second extreme event may therefore be up to RL 61.0 m. The lowest building floor levels are set at RL 61.5 m.

It must be emphasised that two sequential 100-year ARI rainfall events and nil soakage is not considered a realistic risk. This check is done simply to demonstrate that the building floor levels have been conservatively set.

These checks show that the above ground storage volume is available for a subsequent large rainfall event. More detailed levels will be developed following the current groundwater monitoring in response to rainfall and further modelling.

5.5 Operational Issues

5.5.1 Soakage Testing

Face mapping of the quarry wall geology (refer Tonkin & Taylor Figure 2 in Appendix A) has identified areas of scoria. It is expected that these areas will be suitable for soakage discharges. In addition soakage testing of the rock adjacent to Mt Eden Road has identified very good soakage rates of more than 50 L/s down a single 100 mm bore in several locations (refer Appendix C for the soakage test report). Soakage tests at all the proposed discharge points in the quarry will be carried out prior to the detailed design stage.

5.5.2 Long Term Ownership

It is envisaged that the majority of the stormwater system would be vested with Auckland Council. The details of this ownership will be discussed with Council during further design iterations.

5.5.3 Blockage of Inlets

Inlets to the stormwater system are distributed throughout the development. On roads adjacent to the wetland cells inlets are via catch-pits. In other areas, inlets may be direct from roofs, via swales or rain-gardens. The number of inlets provided means that total blockage of all inlets simultaneously would not occur.

However, the potential risk of all inlets being blocked during an extreme rainfall event has been considered and mitigated by setting building floor levels above the 100-year ARI water level with no soakage occurring.

5.5.4 Blockage of Soakage Areas

Sedimentation and clogging of soakage pits can be a significant issue where there are high sediment loads. Over time discharges containing sediment can fill interstitial cracks and reduce the capacity of soakage bores and soakpits. This is particularly an issue where sediment loads are high such as from industrial sites, heavily trafficked roads and eroding landscaping areas.

The Three Kings development will not generate high volumes of sediment once the construction phase is complete.

During construction a high degree of erosion and sediment control will be provided and final soakage systems will be constructed late in the construction programme to minimise the risk of construction related sediment blockage.

Following construction the residential development and low traffic volumes on the internal roads will not generate large volumes of sediment. Significant treatment (through sedimentation basins and wetlands) will be provided to reduce the sediment that is generated. Prior to the outlet diversions to soakage, water will be flowing slowly through the wetlands allowing sediment to both settle and be filtered by wetland vegetation. At the diversions, flows will drain from the surface (where the cleanest water will be) and graded sand filters will be provided at the entrance to soakage pits to further catch any floatables and sediment. Finally, monitoring wells will be installed within underground storage areas and soakage pits to allow soakage rates to be monitored, identify any reductions in soakage capacity and trigger maintenance if required. These details are shown on Figure 10 (refer Appendix A).

5.5.5 Emergency Vehicle Access during Floods

Figure 11 (refer Appendix A) shows the 100-year flood levels and building floor levels. Floodwater in the wetland cells is below adjacent road levels. The only roads potentially affected by ponded flooding are adjacent to the sports fields. Final levels on this road will be set to ensure that the road is passable in the 100-year flood.

Overland flow paths are provided down roadways. The depth of this flow will be confirmed at detailed design and will be kept to acceptable levels for vehicle traffic.

Vehicle access to all dwellings is therefore available for emergency services under the 100-year flood conditions.

6.0 Resource Management Approach

6.1 Plan Change Provisions

6.1.1 Existing Provisions

Fletcher Residential Ltd is seeking a District Plan change for the development area to provide an integrated outcome for the development and surrounding area. Existing and proposed provisions relating to stormwater are discussed below.

The Operative Auckland City District Plan: Isthmus Section sets out development criteria in Section 7.8.2 and assessment criteria and information requirements for the Planned Unit Development provisions (pages B17-18 and B43-B45). These provisions do not specifically address stormwater management matters, however the Stormwater Management Plan is considered to be consistent (or rather not inconsistent) with them.

6.1.2 Additional Matters

Key matters to be addressed are set out below.

Table 9: Proposed Plan Change Provisions		
Key matter	Issue	Addressed by Plan provision
Erosion and Sediment control will be required during construction, with particular emphasis on avoiding sediment discharges to long term soakage fields.	Placement of fill material for filling the quarry and construction phase work has potential for mobilising sediment. Where sediment is directed toward long term soakage discharge points their capacity may be reduced.	A criterion for the Erosion and Sediment Control Plan to specifically address.
Habitable floor levels must avoid the potential for flooding during the 100-year ARI rainfall events.	Failure of stormwater systems due to blockage or inaccuracy in flood level predictions is allowed for by setting habitable floor levels above predicted flood levels. Given the development is set below surrounding ground levels, additional freeboard is proposed to provide a higher degree of protection.	Add additional criteria to normal freeboard requirements: Floor levels have been set above 100-year ARI flood levels with minimum freeboard required by either; 750 mm above the flood level in the main storage areas, or, 500 mm above flood levels with respect to flood levels in the eastern wetland.
Soakage fields and flood storage areas must be robust and operate effectively in the long term. To achieve this, sediment treatment removal must be effective and ongoing monitoring of the system is required.	While expected sediment loads post construction are expected to be low, maintaining the soakage capacity of soakage fields in the long term is necessary to ensure the long term viability of the Stormwater Management Plan.	Add specific testing and maintenance requirements to give certainty to the long term operation of the soakage system.
Groundwater levels need to be maintained within the predictions of the groundwater model to ensure freeboard and flood storage remains available in the long term.	Significant changes to the amount of stormwater discharged to groundwater in the surrounding local areas should be checked for its effect on groundwater levels within the quarry.	Add assessment criteria for significant additional discharges to ground in the surrounding area.
Stormwater treatment is provided for higher contaminant load generating activities.	Roads and paved areas are expected to generate higher contaminant loads compared to residential roofs constructed of pre-painted materials.	Add assessment criteria

7.0 Opportunities Provided by this Proposal

7.1 Soakage for Surrounding Areas

Fletcher has tested four new soakholes adjacent to Mt Eden Rd. Soakage rates were a minimum of 40 L/s and some were taken as greater than 50 L/s as no constant water head could be achieved during the test. These holes could be used to divert stormwater from the existing catchpits on Mt Eden Rd which drain to the combined sewer.

Similarly, the combined sewer area around McCullough and Smallfield Avenues could be diverted to soakage. Auckland Council is proposing to undertake soakage tests in this area to identify specific opportunities.

A 0.3 m increase in groundwater level from additional soakage in the surrounding area has been allowed for in setting the short term groundwater level.

7.2 Flood Storage

This assessment has conservatively assumed that there will be no long term pumping of groundwater from the quarry to ensure that there is no risk to the development from groundwater and stormwater up to the 100-year ARI rainfall event.

However, if pumping were to continue, groundwater levels could be maintained at RL 34 m and there would be an extremely large volume of potential soakage and flood storage volume between this and the highest groundwater level of RL 58.5 m. If stormwater from off site was conveyed to this underground storage volume, the load on the combined sewer could be reduced and existing local flooding and sewage overflow problems could be relieved.

8.0 Conclusions

The groundwater rise due to infiltration of water during the 10-year and 100-year ARI rainfall events is 1.5 and 2 m, respectively. It has therefore been assumed that soil is saturated up to RL 58.0 m and RL 58.5 m for the 10-year and 100-year ARI rainfall events, respectively, allowing for no surface water storage below these levels.

Stormwater runoff from the 10-year and 100-year ARI rainfall events can be appropriately managed through soakage and flood storage on site.

Flood storage can be provided so to manage 100-year ARI runoff volumes while making conservative assumptions about the amount of soakage available.

The amount of freeboard to all habitable floors is such that all of the rainfall volume from a 100-year ARI rainfall event can be accommodated on site.

In order to avoid flooding on the field in a 10-year ARI rainfall event, the ground level for the field was set at RL 59.0 m. With this level, the 10-year ARI rainfall event can be stored below RL 59 m in Area B and raising to RL 60.0 m in the northern cell of the eastern wetland channel, though some overland flow paths may be higher than this.

Stormwater quality treatment is provided in Pond A, swales the eastern wetland and in individual rain-gardens for some paved areas and roads. Roof water quality will be managed through the use of non-exposed metal products such as pre-painted steel.

Further work is necessary to validate and refine the assumptions, though these have been consistently conservative throughout the process.

Overall it is considered that this Stormwater Management Plan provides a high level of robustness for stormwater management at the Three Kings Renewal development. Therefore, the Plan Change is supported in terms of stormwater management.

9.0 References

Auckland City Council, 1999 – Operative District Plan, Isthmus Section

Auckland City Council, 2003 - Soakage Design Manual, prepared by PDP

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Three Kings

July 2014

Plan Option 1



THREE KINGS RENEWAL SCALE 1:1000 JULY 29, 2014

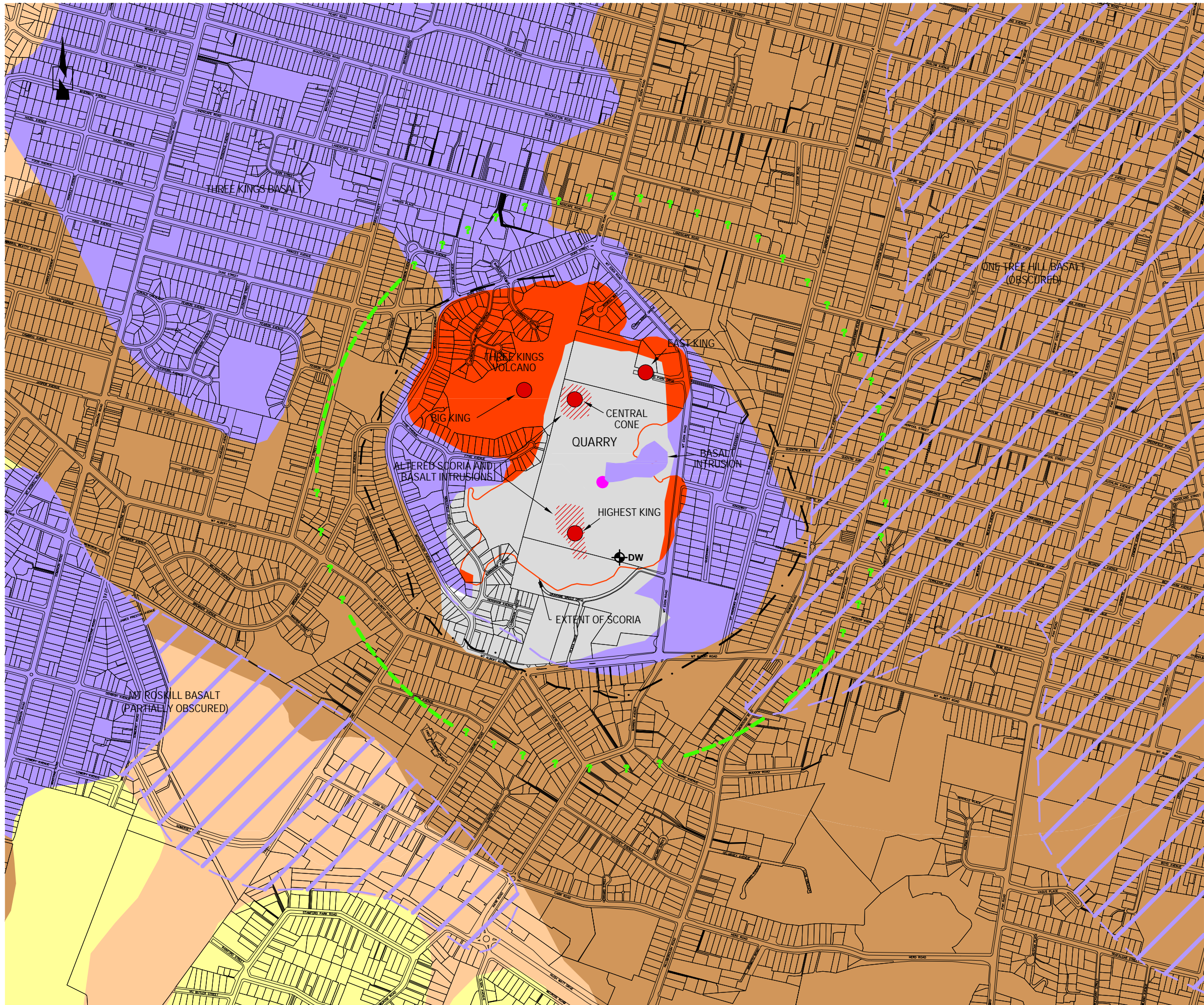
These plans outline the vision of Fletcher Residential Ltd and are not endorsed by Auckland Council.



SURFACEDSIGN INC



FLETCHER RESIDENTIAL



KEY :

- CENTRE OF CRATER
- DEWATERING WELL
- ESTIMATED EXTENT OF HIGH PERMEABILITY ZONE (WAIEMATAS)
- MODIFIED AREAS
- RECENT DEPOSITS
- BASALT
- OBSURED BASALT
- SCORIA
- TUFF DEPOSITS
- ECBF GROUP
- INFERRED EXTENT OF OBSURED BASALT
- THREE KINGS CRATER

NOTES :
1. GEOLOGY BASED PARTLY ON IGNS GEOLOGICAL MAP, SHEET R11. BASE MAP SUPPLIED BY GRANT FISHER INDUSTRIAL GEOLOGY, DATED DEC 01, AND SUBSEQUENTLY AMENDED BY PDP LTD (JAN 08) TO CLARIFY AREAS OF OBSURED BASALT.

B	MINOR AMENDMENT	JUL 14	
A	ISSUED FOR APPLICATION	APR 14	
NO.	REVISION	DATE	APP.

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PROJECT :

**THREE KINGS RENEWAL
STORMWATER
MANAGEMENT PLAN**

TITLE :

GEOLOGY

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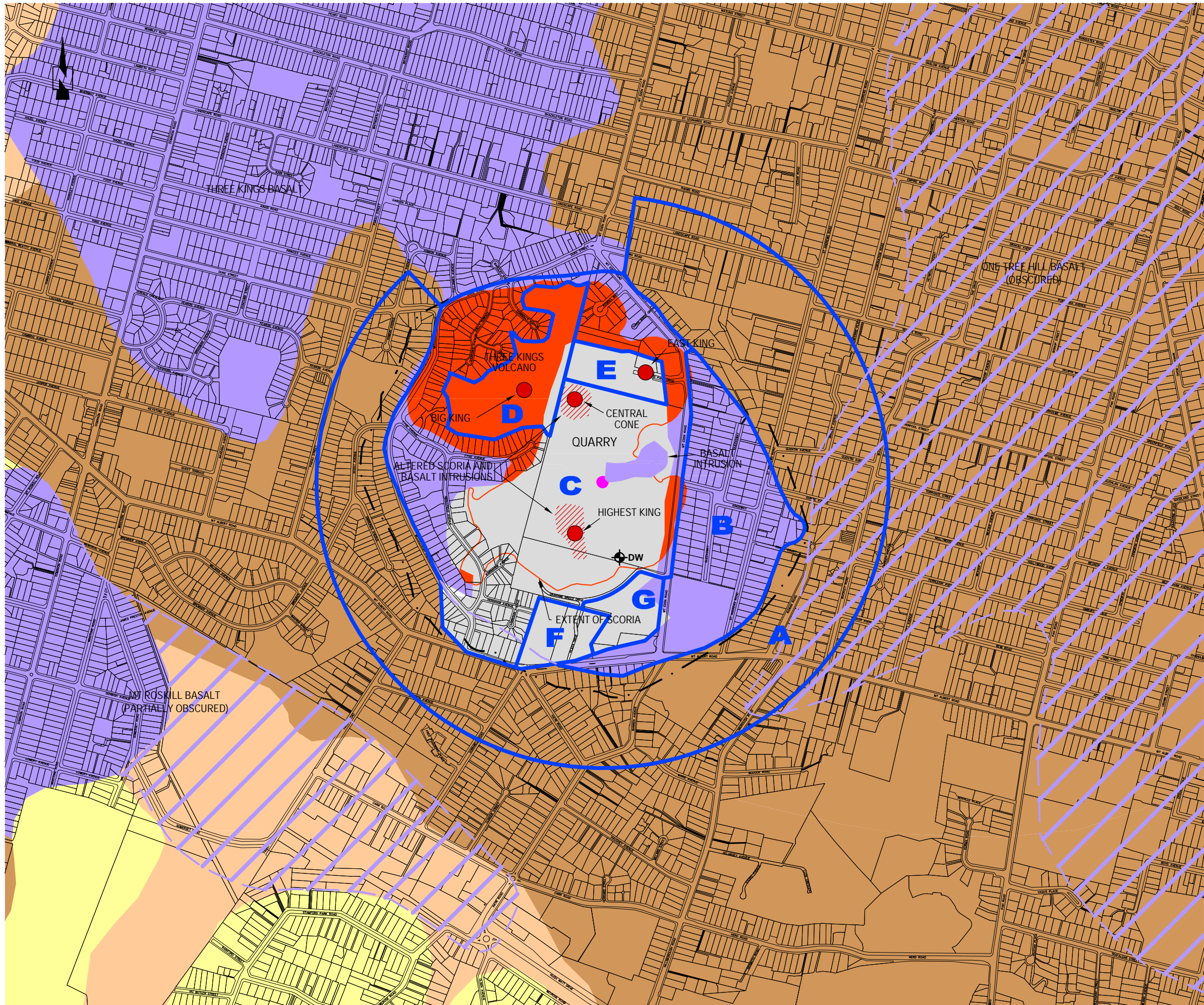
PATTLE DELAMORE PARTNERS LTD
Auckland Tauranga Wellington Christchurch

SCALE 1:10,000 (A3)

0 100 200 300 400 500m

PROJECT NO. :	FIGURE NO. :	REVISION :
AJ456300	6	B

FILED : AJ456300D004.dwg



KEY :

- CENTRE OF CRATER
- DEWATERING WELL
- GROUNDWATER CATCHMENT

- MODIFIED AREAS
- RECENT DEPOSITS
- BASALT
- OBSURED BASALT
- SCORIA
- TUFF DEPOSITS
- ECBF GROUP
- INFERRED EXTENT OF OBSURED BASALT
- THREE KINGS CRATER

NOTES :

1. GEOLOGY BASED PARTLY ON IGNS GEOLOGICAL MAP, SHEET R11. BASE MAP SUPPLIED BY GRANT FISHER INDUSTRIAL GEOLOGY, DATED DEC 01, AND SUBSEQUENTLY AMENDED BY PDP LTD (JAN 08) TO CLARIFY AREAS OF OBSURED BASALT.

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PROJECT :

**THREE KINGS RENEWAL
STORMWATER
MANAGEMENT PLAN**

TITLE :

**CATCHMENT MAP FOR
GROUNDWATER
CONTRIBUTIONS**

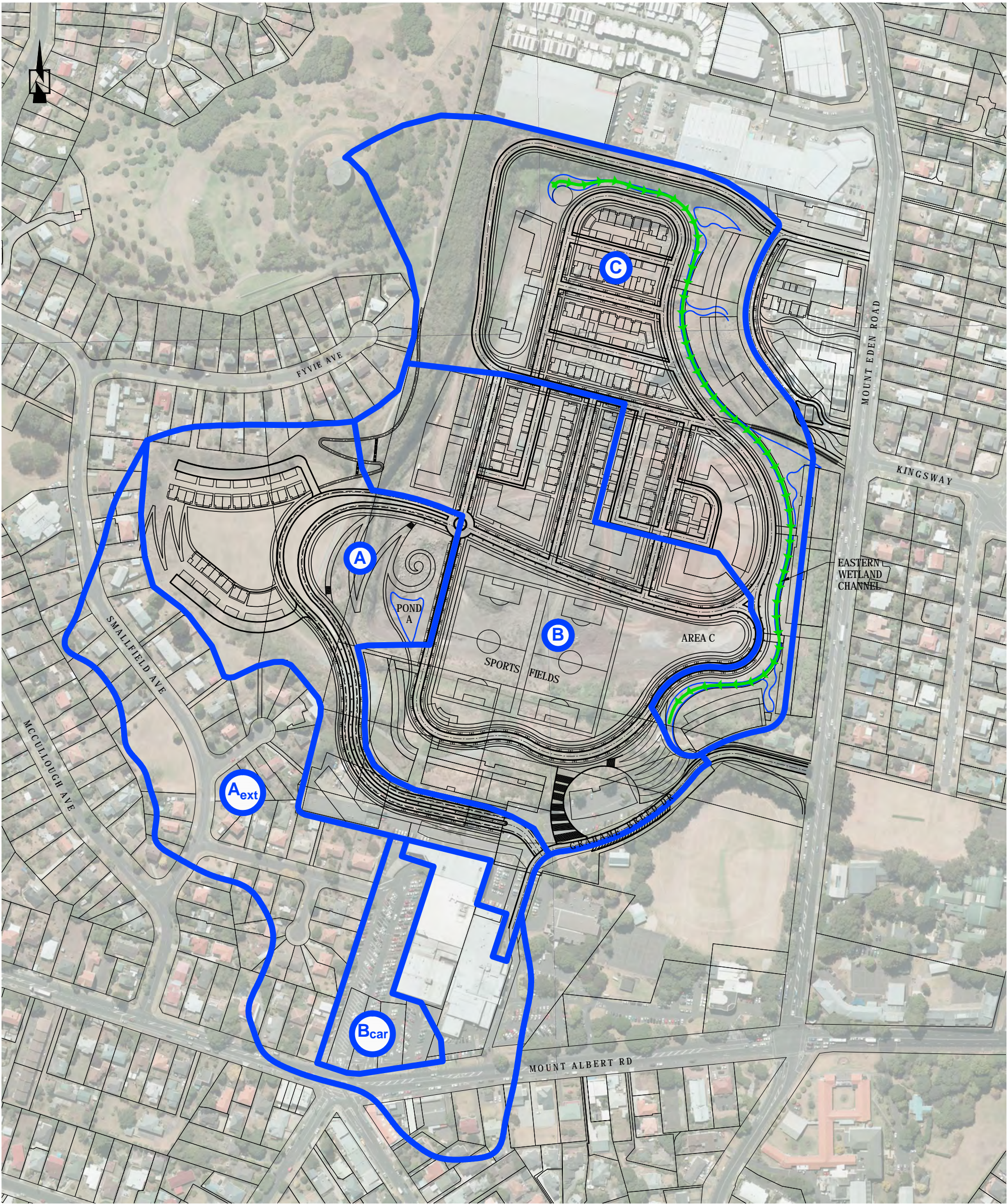
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SCALE 1:1,000 (A3)


PROJECT NO. :	FIGURE NO. :	REVISION :
AJ456300	7	B

FILED : AJ456300D004.dwg



KEY :


 WETLAND CHANNEL

 CATCHMENT BOUNDARY

- NOTES :
1. AERIAL IMAGERY (FLOWN 2010) PROVIDED UNDER LICENCE FROM AUCKLAND COUNCIL WHO MAKES NO CLAIMS AS TO ITS RELIABILITY, ACCURACY OR ADEQUACY FOR ANY PARTICULAR PURPOSE.
 2. CADASTRAL INFORMATION DERIVED FROM LINZ DATA.
 3. SOME BASE DATA DERIVED FROM 122314-03-500.dwg, REV -. SUPPLIED BY HARRISON GRIERSON, RECEIVED 29/04/2014.
 4. OPTION 15H1 LAYOUT DERIVED FROM XL-BASE_H1.dwg SUPPLIED BY dKO ARCHITECTURE (NSW) PTY LTD., RECEIVED 18/05/2014.
 5. THIS PLAN OUTLINES THE VISION OF FLETCHER RESIDENTIAL LTD. AND IS NOT ENDORSED BY AUCKLAND COUNCIL.

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A	ISSUED FOR APPLICATION	MAY 14	
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PROJECT :

**THREE KINGS RENEWAL
STORMWATER
MANAGEMENT PLAN**


TITLE :

**OPTION 15H1
CATCHMENT MAP:
SURFACE WATER**

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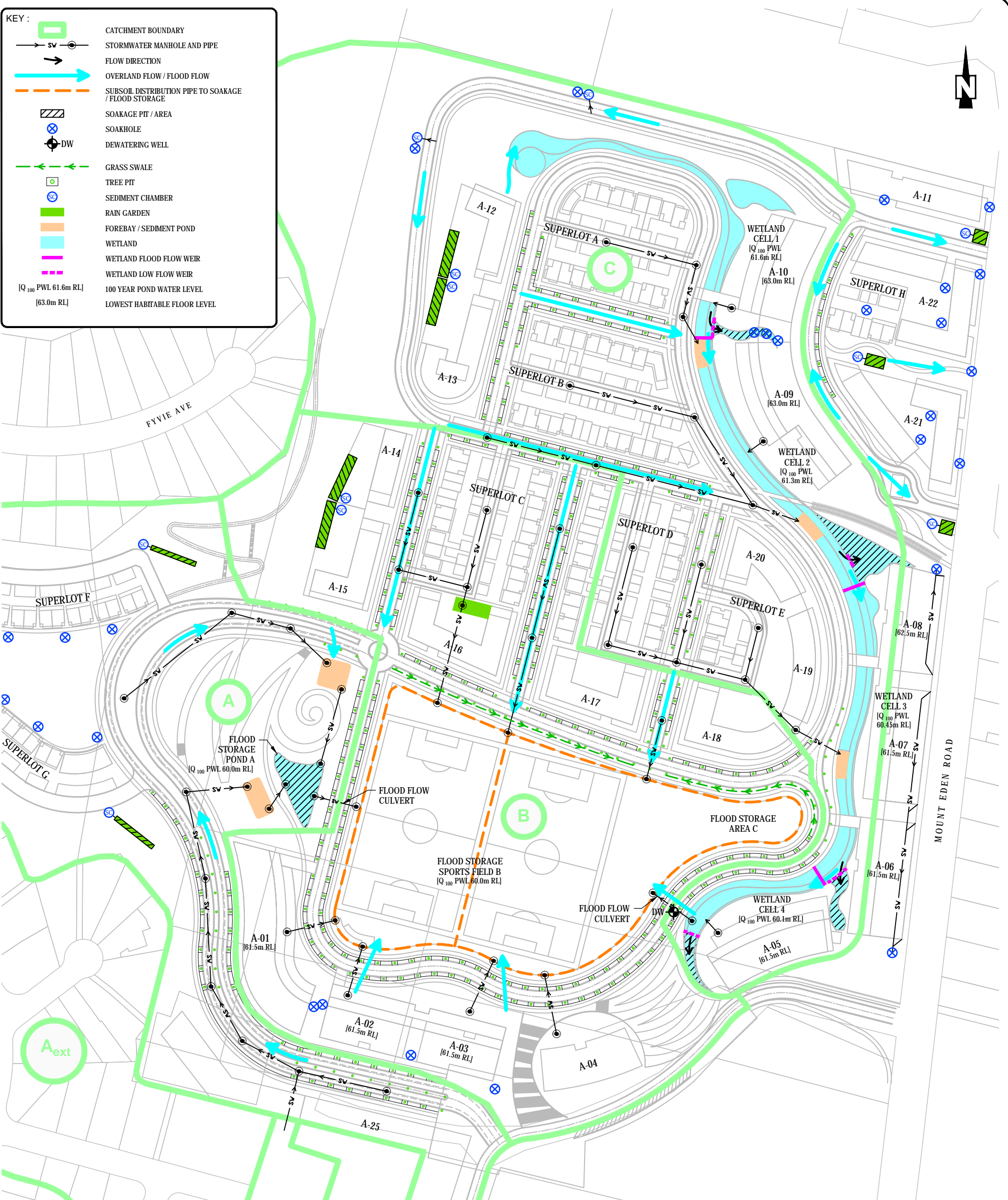
SCALE 1:3,000 (A3)



PROJECT NO. :	FIGURE NO. :	REVISION :
AJ456300	8 H1	B

FILED : AJ456300D001.dwg

- KEY :
- CATCHMENT BOUNDARY
 - STORMWATER MANHOLE AND PIPE
 - FLOW DIRECTION
 - OVERLAND FLOW / FLOOD FLOW
 - SUBSOIL DISTRIBUTION PIPE TO SOAKAGE / FLOOD STORAGE
 - SOAKAGE PIT / AREA
 - SOAKHOLE
 - DEWATERING WELL
 - GRASS SWALE
 - TREE PIT
 - SEDIMENT CHAMBER
 - RAIN GARDEN
 - FOREBAY / SEDIMENT POND
 - WETLAND
 - WETLAND FLOOD FLOW WEIR
 - WETLAND LOW FLOW WEIR
 - [Q₁₀₀ PWL 61.6m RL]
 - [Q₁₀₀ PWL 61.3m RL]
 - 100 YEAR POND WATER LEVEL
 - LOWEST HABITABLE FLOOR LEVEL



- NOTES :
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 - CADASTRAL INFORMATION DERIVED FROM LINZ DATA.
 - SOME BASE DATA DERIVED FROM 122314-03-500.dwg, REV. RECEIVED 29/04/2014 AND H1 Stormwater Piped Network Concept Scheme REV.B.dwg, RECEIVED 23/05/2013, SUPPLIED BY HARRISON GRIERSON.
 - OPTION 15H1 LAYOUT DERIVED FROM XL-BASE_H1.dwg SUPPLIED BY dKO ARCHITECTURE (NSW) PTY LTD., RECEIVED 18/05/2014.
 - UPDATED APARTMENT BUILDING LAYOUT 140523_3K H1 Building Outlines.dwg SUPPLIED BY dKO ARCHITECTURE (NSW) PTY LTD., RECEIVED 23/05/2014.
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PROJECT :

THREE KINGS RENEWAL
STORMWATER
MANAGEMENT PLAN

TITLE :

OPTION 15H1
STORMWATER
MANAGEMENT
CONCEPT



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SCALE 1:2,000 (A3)



PROJECT NO. :

AJ456300

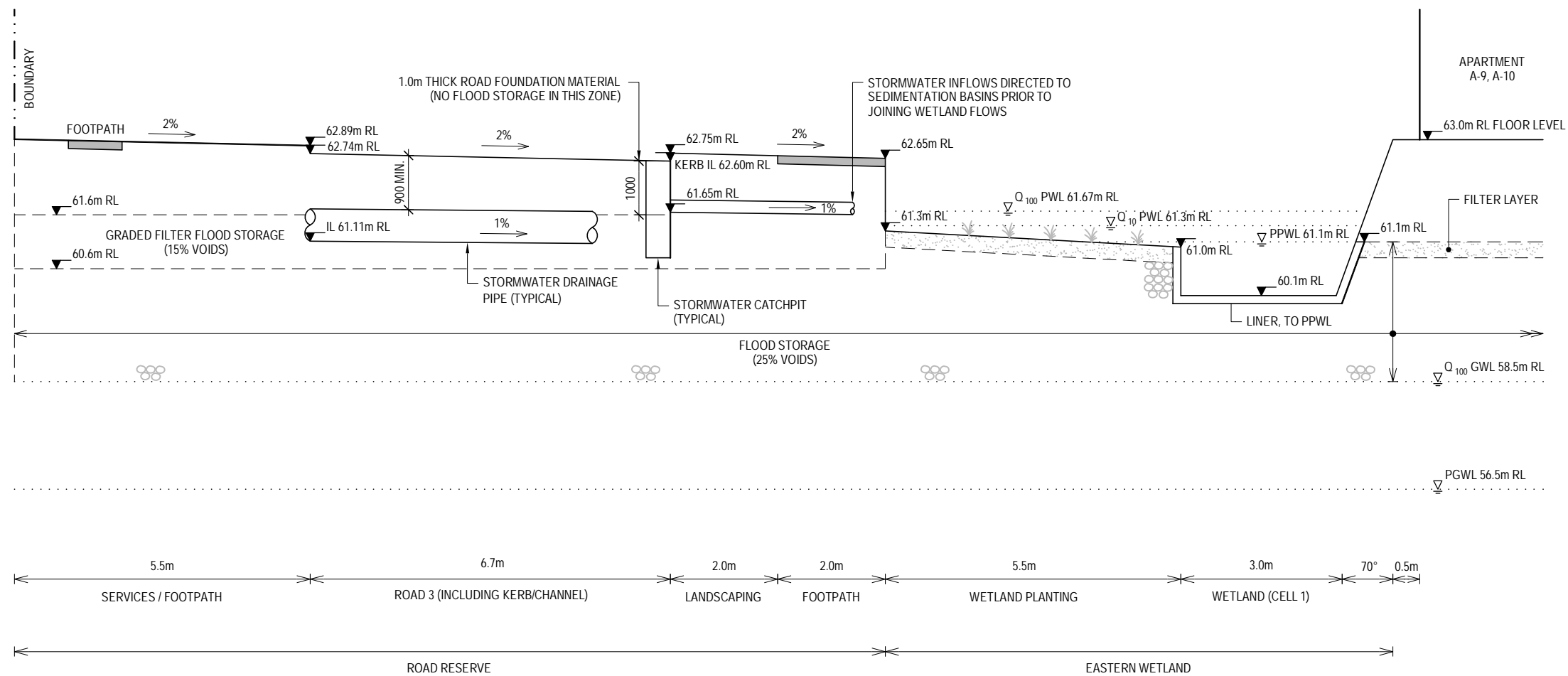
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9 H1

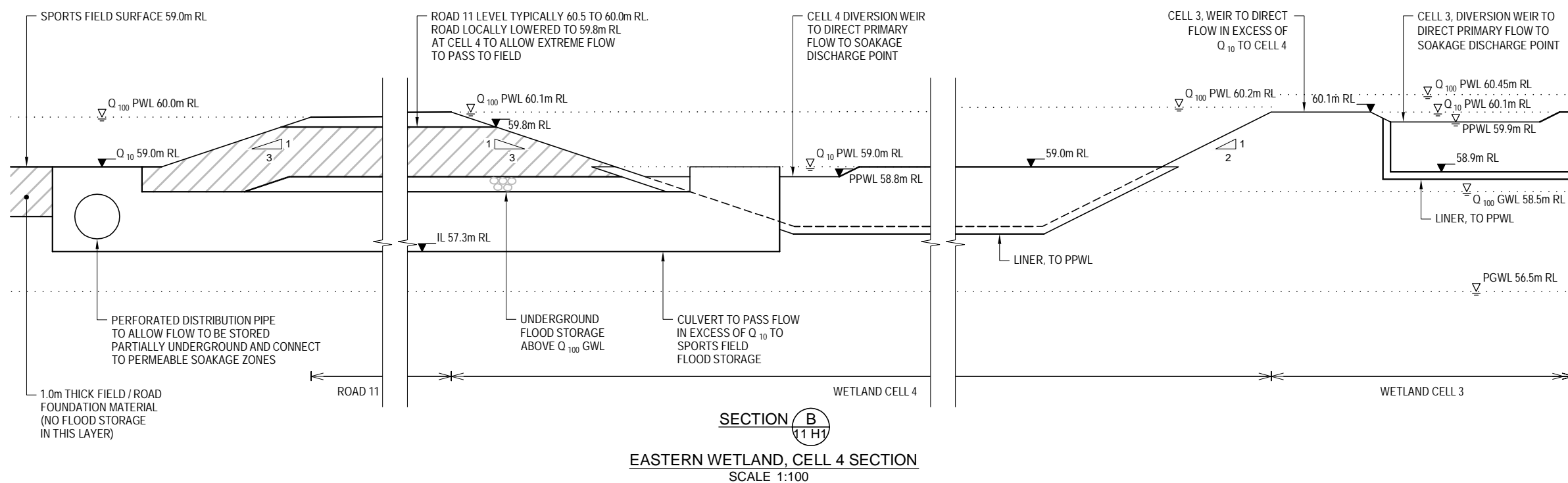
REVISION :

B

FILED : AJ456300D001.dwg



SECTION **A**
11 H1
ROAD 3 / EASTERN WETLAND CELL 1 SECTION
SCALE 1:100



SECTION **B**
11 H1
EASTERN WETLAND, CELL 4 SECTION
SCALE 1:100

KEY :

GWL	GROUNDWATER LEVEL
PGWL	PERMANENT GROUNDWATER LEVEL
PWL	POND WATER LEVEL
PPWL	PERMANENT POND WATER LEVEL
Q ₁₀₀ PWL	100 YEAR POND WATER LEVEL
Q ₁₀ PWL	10 YEAR POND WATER LEVEL
Q ₁₀₀ GWL	100 YEAR SHORT TERM GROUNDWATER LEVEL

NOTES:
1. THIS ARRANGEMENT RELATES TO THE FOUR CELL WETLAND FOR OPTION 15H1. A SIMILAR ARRANGEMENT WITH THREE CELL WETLAND APPLIES TO OPTION 15H2.
2. THIS PLAN OUTLINES THE VISION OF FLETCHER RESIDENTIAL LTD. AND IS NOT ENDORSED BY AUCKLAND COUNCIL.

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A	ISSUED FOR APPLICATION	MAY 14	
NO.	REVISION	DATE	APP.

CLIENT :



PROJECT :
**THREE KINGS RENEWAL
STORMWATER
MANAGEMENT PLAN**

TITLE :
**EASTERN WETLAND
CROSS SECTIONS**



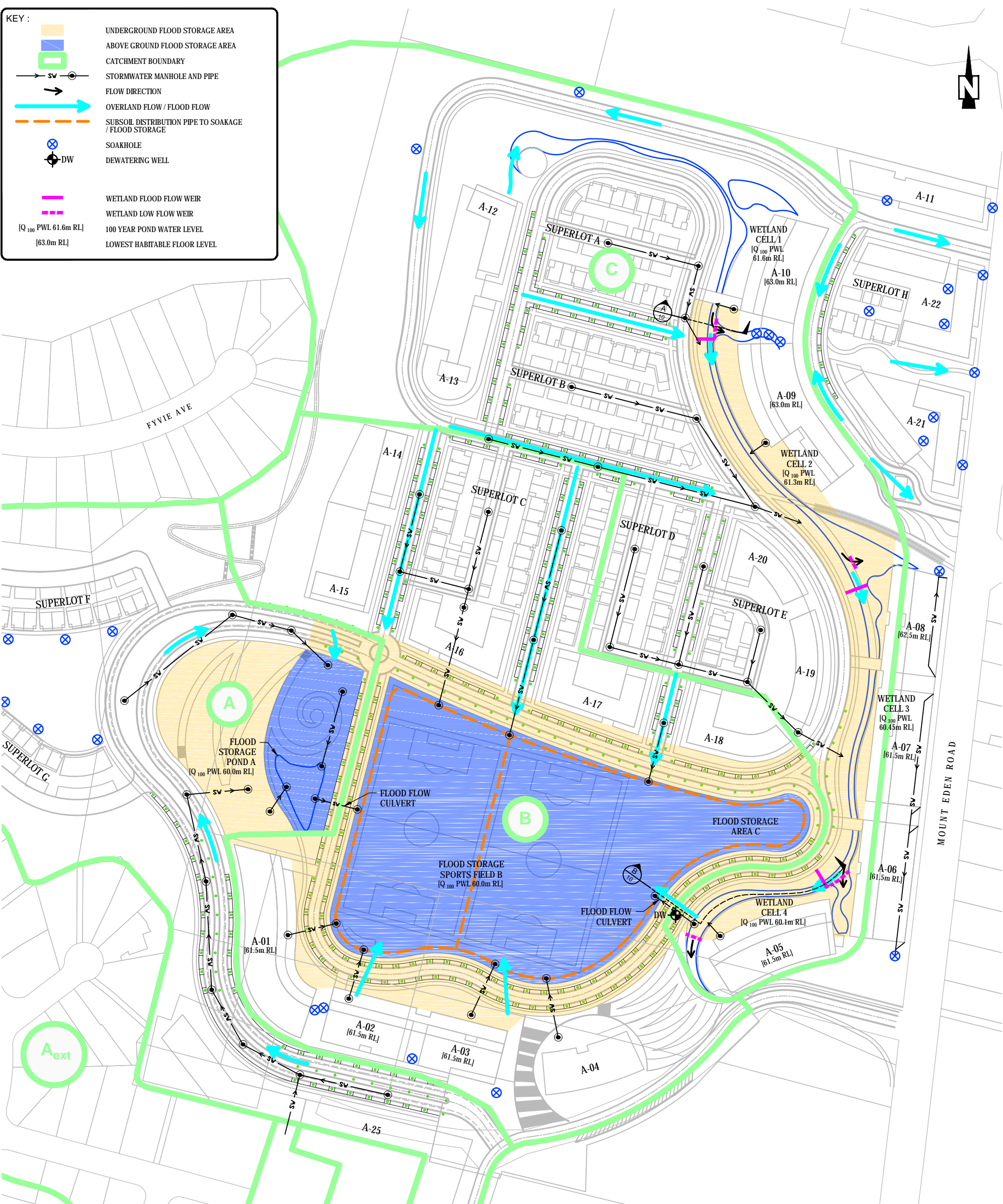
PATTLE DELAMORE PARTNERS LTD
Auckland Tauranga Wellington Christchurch

SCALE AS SHOWN (A3)

PROJECT NO. :	FIGURE NO. :	REVISION :
AJ456300	10	B

FILED : AJ456300D003.dwg

- KEY :
- UNDERGROUND FLOOD STORAGE AREA
 - ABOVE GROUND FLOOD STORAGE AREA
 - CATCHMENT BOUNDARY
 - STORMWATER MANHOLE AND PIPE
 - FLOW DIRECTION
 - OVERLAND FLOW / FLOOD FLOW
 - SUBSOIL DISTRIBUTION PIPE TO SOAKAGE / FLOOD STORAGE
 - SOAKHOLE
 - DEWATERING WELL
 - WETLAND FLOOD FLOW WEIR
 - WETLAND LOW FLOW WEIR
 - [Q₁₀₀ PWL 61.6m RL]
 - [Q₁₀₀ PWL 61.3m RL]
 - 100 YEAR POND WATER LEVEL
 - LOWEST HABITABLE FLOOR LEVEL



- NOTES :
- AERIAL IMAGERY (FLOWN 2010) PROVIDED UNDER LICENCE FROM AUCKLAND COUNCIL WHO MAKES NO CLAIMS AS TO ITS RELIABILITY, ACCURACY OR ADEQUACY FOR ANY PARTICULAR PURPOSE.
 - CADASTRAL INFORMATION DERIVED FROM LINZ DATA.
 - SOME BASE DATA DERIVED FROM 122314-03-500.dwg, REV. RECEIVED 29/04/2014 AND H1 Stormwater Piped Network Concept Scheme REV.B.dwg, RECEIVED 23/05/2013, SUPPLIED BY HARRISON GRIERSON.
 - OPTION 15H1 LAYOUT DERIVED FROM XL-BASE_H1.dwg SUPPLIED BY dKO ARCHITECTURE (NSW) PTY LTD., RECEIVED 18/05/2014.
 - UPDATED APARTMENT BUILDING LAYOUT 140523_3K H1 Building Outlines.dwg SUPPLIED BY dKO ARCHITECTURE (NSW) PTY LTD., RECEIVED 23/05/2014.
 - THIS PLAN OUTLINES THE VISION OF FLETCHER RESIDENTIAL LTD. AND IS NOT ENDORSED BY AUCKLAND COUNCIL.

B	MINOR AMENDMENT	JUL 14	
A	ISSUED FOR APPLICATION	MAY 14	
NO.	REVISION	DATE	APP.

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CLIENT :



FLETCHER RESIDENTIAL

PROJECT :

THREE KINGS RENEWAL
STORMWATER
MANAGEMENT PLAN

TITLE :

OPTION 15H1
FLOOD STORAGE AREAS



PATTLE DELAMORE PARTNERS LTD

Auckland Tauranga Wellington Christchurch

SCALE 1:2,000 (A3)

0 20 40 60 80 100m

PROJECT NO. :

AJ456300

FIGURE NO. :

11 H1

REVISION :

B

FILED : AJ456300D001.dwg

Appendix B

Surface Water Model Results

Three Kings Renewal, Option 15H1, Sub-catchment surface runoff characteristics																		
Subcatchment	2-year						10-year						100-year					
	Gross Runoff		To Soakage		Net Runoff		Gross Runoff		To Soakage		Net Runoff		Gross Runoff		To Soakage		Net Runoff	
	Peak Flow (m³/s)	Volume (m³)	Peak Flow (m³/s)	Volume (m³)	Peak Flow (m³/s)	Volume (m³)	Peak Flow (m³/s)	Volume (m³)	Peak Flow (m³/s)	Volume (m³)	Peak Flow (m³/s)	Volume (m³)	Peak Flow (m³/s)	Volume (m³)	Peak Flow (m³/s)	Volume (m³)	Peak Flow (m³/s)	Volume (m³)
A	0.05	300	n.a.	n.a.	0.05	300	0.08	500	n.a.	n.a.	0.08	500	0.14	800	n.a.	n.a.	0.14	800
A1	0.20	1100	0.20	1100	0	0	0.37	2000	0.2	1900	0.17	100	0.74	4000	0.20	3400	0.54	600
A2	0.06	300	n.a.	n.a.	0.06	300	0.11	600	n.a.	n.a.	0.11	600	0.28	1200	n.a.	n.a.	0.28	1200
A3	0.08	400	0.08	400	0	0	0.15	800	0.08	700	0.07	40	0.29	1500	0.08	1300	0.21	200
A.EXTERNAL	0.70	3700	0.32	1800	0.38	1900	1.26	6700	0.32	3100	0.93	3600	2.42	13100	0.32	5600	2.09	7600
A.ROADS	0.11	600	0.09	500	0.03	100	0.19	1100	0.09	800	0.10	300	0.34	2000	0.09	1400	0.26	600
B	0.25	1400	n.a.	n.a.	0.25	1400	0.40	2300	n.a.	n.a.	0.40	2300	0.70	4100	n.a.	n.a.	0.70	4100
B1	0.20	1100	n.a.	n.a.	0.20	1100	0.33	1800	n.a.	n.a.	0.33	1800	0.60	3400	n.a.	n.a.	0.60	3400
B2	0.25	1400	0.05	300	0.20	1100	0.40	2300	0.05	400	0.35	1800	0.71	4100	0.05	700	0.66	3400
B.ROADS	0.15	800	0.02	100	0.13	700	0.25	1400	0.02	200	0.23	1200	0.46	2600	0.02	300	0.43	2200
B.EXTERNAL	0.01	40	n.a.	n.a.	0.01	40	0.02	100	n.a.	n.a.	0.02	100	0.04	200	n.a.	n.a.	0.04	200
B.CARPARK	0.14	800	n.a.	n.a.	0.14	800	0.22	1300	n.a.	n.a.	0.22	1300	0.38	2300	n.a.	n.a.	0.38	2300
C	0.06	300	n.a.	n.a.	0.06	300	0.10	600	n.a.	n.a.	0.10	600	0.17	1000	n.a.	n.a.	0.17	1000
C1	0.13	700	0.13	700	0	0	0.22	1200	0.13	1100	0.09	100	0.40	2200	0.13	2000	0.27	300
C2	0.16	900	0.16	900	0	0	0.31	1600	0.16	1500	0.14	100	0.60	3200	0.16	2800	0.44	500
C3	0.31	1700	0.31	1700	0	0	0.53	2900	0.31	2800	0.22	100	0.98	5500	0.31	4800	0.67	700
C.EXTERNAL	0.05	300	n.a.	n.a.	0.05	300	0.12	600	n.a.	n.a.	0.12	600	0.29	1600	n.a.	n.a.	0.29	1600
C.ROADS	0.17	1000	0.15	900	0.03	100	0.29	1700	0.15	1400	0.14	200	0.52	3100	0.15	2400	0.37	700

Appendix C

Soakage Test Results: Mt Eden Road /
Upper Development Area



5 June 2014

Bernie Chote
Fletcher Construction Company Ltd
Private Bag 92114
AUCKLAND 1142

Dear Bernie

THREE KINGS RENEWAL: SOAKHOLE DRILLING AND TESTING ADJACENT MT EDEN ROAD

1.0 Introduction

The Three Kings Quarry, located south of Mount Eden in Auckland, has been excavated for scoria since the 1840s and began the process of filling in 2012. Fletcher Construction Company Ltd has owned and operated the 15.2 ha site since 1922 (through its subsidiary Winstone Aggregates) and intends to develop it into a residential area including apartments, terrace housing, and open space including sports fields (through its subsidiary Fletcher Developments). The development is expected to house between 3420 to 4275 people.

The Three Kings Renewal development requires a plan change to the operative Auckland District Plan (Isthmus Section) to enable the comprehensive redevelopment of the former quarry lands at Three Kings.

Pattle Delamore Partners (PDP) were engaged by Fletcher Developments to develop the stormwater management concept for the development and therefore demonstrate that it is appropriate to proceed. This report relates to soakage capacity in the north-eastern corner of the site which is on a similar level to the adjacent Mt Eden Rd. The soakage in this area is to be used to divert existing water from Mt Eden Rd which currently drains to the combined sewer and drainage of the development on this "upper development area". The upper development area is the same irrespective of whether Option 15H1 or 15H2 is enacted for the wider project.

1.1 Site Description

The site is currently a commercial property located in Three Kings, Mt Eden. It consists of a Quarry operation, offices and several retail stores. The Quarry takes up the majority of the area of the site. The north-eastern end of the property has been developed for commercial purposes and contains the Quarry offices and several retail stores along Mt Eden road.

The Auckland Geological Map indicates the property overlies construction fill, basalt and basanite scoria (Auckland Volcanic Field). The soil varies from fill with coarse gravel and cobbles to volcanic scoria. The drilling occurred approximately 1 m from the kerb of Mt Eden Road in the grass berms between the retail stores and the footpath (see Figure 1). Two original soakholes were also tested - one in the Animates car park and the other in Eiffel En' French Café car park (see Figure 1).

Existing information on soakage in the general area is provided by two investigations undertaken to the north of the quarry. Soil and Rock Consultants undertook testing for the Hunters Park Drive subdivision immediately to the north of the quarry and found soakage rates of 61.9 L/s and approximately 60 L/s (Soil and Rock letter of 6 July 2001, ref

01124). Geotek Services Ltd undertook soakage testing for a development at 955 Mt Eden Rd (Geotek Services Ltd letter of 14 December 2000, ref 1324). They encountered a mix of scoria and ash with some fractured rock in a series of eight boreholes (4.5 to 8.5 m depth) with soakage rates of between nil and 22 L/s. Bores 1 and 8 (with soakage rates of 22 and 7.6 L/s) are immediately next to Mt Eden Rd and are about 10m from the end of a 375 dia pipe crossing Mt Eden Rd which is being considered for diversion away from the combined sewer to soakage.

1.2 Purpose of Work

The work was to test the availability of soakage adjacent to Mt Eden Rd for:

- a) diversion of stormwater from Mt Eden Road to soakage for the purpose of freeing up flow capacity in the combined sewer.
- b) the disposal of storm water from the upper development area (Superlot H).

2.0 Methodology

The methodology for drilling the test bores consisted of drilling to a depth where significant fractured basalt or scoria was reached. A pneumatic rig was used so the underlying geology was inferred from drillers' comments and the fragments collected at the surface of the test hole. Three of the test holes were drilled at 100mm and one at 150mm. An environmental geologist was onsite supervising all drilling and soakage testing operations. A senior hydro-geologist was present for two soakage tests.

Borehole positions have generally been set to match the existing or future catch-pit locations and therefore facilitate the future diversion of stormwater from the combined sewer.

The test methodology followed for soakage testing was from the Auckland Council Soakage Design Manual: 3.6 Constant-Head Percolation Tests. The holes were pre-soaked for at least 10 minutes before the constant head test was attempted. A hydrant and a 13,500 L water tanker were used when the hydrant was available otherwise two tankers were used to get to acceptable flow rates over the test duration. The flow rate was increased until a constant head was observed or until the maximum flow rate of the equipment was reached with no constant head. The flow rates were then recorded at various intervals during the test to ensure the flow rate was not changing. When the flow rates could not be sustained for the full 10 minutes, two 5 minute tests were run successively.

2.1 Tests Undertaken

The soakage testing undertaken is summarised in Table 1 below for each borehole. Borehole locations have been set in available space adjacent to Mt Eden Rd within the Fletcher road frontage.

THREE KINGS RENEWAL: SOAKHOLE DRILLING AND TESTING ADJACENT MT EDEN ROAD

Table 1: Soakage Tests				
Borehole	Location	Borehole dia	Soakage testing	Comment
01	Northern most borehole to the east of Winstone Aggregates offices.	150 mm	Soakage testing conducted using two 13,500 Litre Tankers (Hydrant hoses couldn't be run across Winstones accessway)	
02	In front of Animates store approximately 1.2 m in from the kerb.	100 mm	Soakage testing conducted using 1 tanker and nearby hydrant.	
03	In front of the Pet doctors approximately 1.5 m from building.	100 mm	Soakage testing conducted using 2 tankers and the hydrant.	
04	In front of Rest bedroom store 1.2 m in from the kerb.	100 mm	Soakage testing conducted using a Tanker and the hydrant.	
Existing Soakhole 1 (Café)	Manhole in the French Café parking lot.	Four PVC sleeves visible	Soakage testing conducted with just the hydrant.	Sediment and vegetation in manhole
Existing Soakhole 2 (Animates)	Manhole in Animates parking lot.	No sleeves visible	Soakage testing was conducted using just the hydrant.	Sediment and vegetation in manhole

3.0 Results

3.1 Material Encountered

The materials encountered drilling were varied which reflects the complex volcanic geology of the area. When attempting to hand clear to 1.5m for services, it became apparent that it would be impossible to do this with a hand auger. This was due to scoria and basalt cobbles and boulders within the top 1.5m. Several items of debris such as a bottle dated 1914 and a horseshoe supports the surface lithology as being construction fill. It appears that the basalt underlying the site is highly fractured in places and also contains areas of scoria as seen in BH01.

The materials encountered are summarised below.

THREE KINGS RENEWAL: SOAKHOLE DRILLING AND TESTING ADJACENT MT EDEN ROAD

BH01		BH02	
Depth (m)	Unit	Depth (m)	Unit
0-0.5	Topsoil/Fill	0-2	Topsoil/Fill
0.5-1.5	Scoria	2.0-3.0	Basalt
1.5-2.7	Scoria boulders and ash	3.0-5.0	Fractured Basalt
2.7-3.5	Scoria boulders	5.0-5.5	Basalt
3.5-5.0	Scoria	5.5-7.0	Fractured Basalt
5.0-6.5	Scoria	7.0-7.8	Basalt
		7.8-8.2	Fractured Basalt
BH03		BH04	
Depth (m)	Unit	Depth(m)	Unit
0-1.5	Topsoil/Fill	0-1.5	Topsoil/Fill
1.5-3.0	Basalt	1.5-2.6	Basalt
3.0-7.0	Basalt	2.6-4.0	Fractured Basalt
7.0-9.2	Fractured Basalt	4.0-5.0	Basalt
9.2-12.5	Basalt	5.0-7.5	Fractured Basalt
10-12.5	Basalt	7.5-7.8	Basalt

3.2 Test Results

The test results for each borehole are described below. The soakage rates are also reported in terms of Worksheet 2 from the Soakage Design Manual – this rate is lower than the test rate as a factor of safety of 1.3 is applied in the method.

BH01 - Borehole 1 soakage test ran for 9 minutes and 30 seconds and had an average flow rate of 51 Litres/second (L/s). No constant head was observed. BH01 had a flow of 63 L/s for the first 3 minutes but this was reduced to ensure the test lasted 10 minutes. Worksheet 2 from the soakage design manual was used to calculate the capacity of the rock bore from the test. The average capacity was found to be 35 L/s but this figure is a minimum as no constant head was observed at any point.

BH02 - Borehole 2 soakage test ran for 11 minutes at an average flow rate of 40 L/s. A constant head was observed at this flow rate and was maintained at the top of the bore casing for the duration of the test. Worksheet 2 calculated the capacity of BH02 to be 31 L/s.

BH03 - Borehole 3 soakage test ran for 10 minutes and 16 seconds and had an average flow rate of 44 L/s maintaining a constant head at the top of the casing. Worksheet 2 calculated the capacity of BH03 to be 34 L/s.

BH04 - Borehole 4 soakage test consisted of two 5 minute tests as a compromise as we could not obtain high enough flow rates for 10 minutes with 1 tanker and a hydrant. The two tests ran for 5 minutes 55 seconds and 5 minutes and 45 seconds and had average flow rates of 50 L/s and 51 L/s respectively. No constant head was observed at any point. Worksheet 2 calculated the capacity of BH04 to be 39 L/s but this is a minimum value as no constant head was observed.

Soakhole 1(Café) - Soakhole 1 soakage test ran for 10 minutes with a constant head observed with a flow rate of 16.1 L/s. Worksheet 2 calculated the capacity of this soakhole to be 12 L/s.

Soakhole 2 (Animates) - Soakhole 2 outside animates was unable to be tested accurately due to blockages in the siphons. Even during pre-soaking at flow rates of less than 10 L/s the pit would fill, then empty sporadically. We

THREE KINGS RENEWAL: SOAKHOLE DRILLING AND TESTING ADJACENT MT EDEN ROAD

attempted to clear the blockage but were unable to do this sufficiently to conduct the test. A worksheet was not completed for this soakhole as a sustained rate was not obtained.

Table 2: Soakage results		
Location	Test capacity	Design capacity
BH 1	51 L/s minimum	35 L/s minimum
BH 2	40 L/s	31 L/s
BH 3	44 L/s	34 L/s
BH 4	50 L/s minimum	39 L/s minimum
SH 1 (Café)	16 L/s	12 L/s
SH 2 (Animates)	Not tested	Not tested
1. Where a flow rate is denoted "minimum", no constant head could be achieved with the available flow from the hydrant /water cart test set-up.		

4.0 Required Flow Rates

As noted above, the soakage assessment has two purposes:

- To allow for diversions of stormwater away from Mt Eden Road catchpits and the combined sewer to which they are connected;
- Provide for stormwater disposal for the upper development area.

Harrison Grierson Consultants Ltd have inspected road catchpits along Mt Eden Rd and identified catchment areas that could be diverted. These are shown on the attached plan 122314-SW01, identified as Stormwater areas C and D. The new soakholes are located generally adjacent to catchment C and part of D. Irrespective of the existing actual flow from the Mt Eden Rd catchpits to the combined sewer, it is assumed that the new soakage diversions will need to meet the required flow capacity for new primary drainage – the 10 year ARI rainfall event. A further potential area of diversion is a 375 dia pipe crossing Mt Eden Rd – the catchment for this pipe is currently being confirmed.

dKO Architects have developed a layout of the upper development area with development of up to 1.24ha (imperviousness is 78% for this layout). Flows have been estimated for the 10 and 100 year ARI rainfall events using the Auckland Council "Guidelines for Stormwater Runoff Modelling in the Auckland Region" (TP108) methodology. Twenty four hour rainfall depths are 140mm and 240mm (for the 10 and 100 year rainfall events) and include allowance for climate change. Curve numbers were taken as 74 for pervious areas (representing a silt clay material) and 98 for impervious areas.

Stormwater runoff flows are as set out in Table 3 below:

Table 3: Stormwater runoff			
Location	Area (m ²)	10 year ARI flow (L/s)	100 year ARI flow (L/s)
Mt Eden Road, C	1,750	40	NA
Mt Eden Road, D	510	12	NA
Upper Development Area	12,400	284	493

5.0 Discussion

There is very good soakage in the north-eastern end of the site. This is due to the highly fractured basalt and permeable scoria. The range of calculated soakage rates from the drilled test boreholes was 30.9 L/s to 39.4 L/s (including the factor of safety required by the Soakage Design Manual).

The Mt Eden road diversion of catchpits from the combined sewer catchments C and D to soakage is feasible with a required 10 year flow rate of 52 L/s compared to a total available soakage capacity of 151 L/s in BHs 1 to 4 and existing SH1. In addition BH01 and BH04 may handle extra flow as no constant head was observed during testing.

Stormwater runoff for the upper development area is 284 L/s for the 10 year ARI rainfall event – this is less than the total available soakage of 151 L/s in the holes tested to date. The new soakholes drilled to date provide good soakage and achieved soakage rates of 31 L/s to 39 L/s minimum and it is expected soakage rates similar to the new holes could be achieved due to the permeable underlying geology. Based on an average soakage capacity of 35 L/s per new hole identified so far, approximately eight new soakholes will need to be used for the 10 year ARI (or 14 holes for the 100 year ARI).

It appears sediment has limited the capacity of the existing soakholes. Little or no sediment control appears to be present and the area has been previously used as an entry area for the quarry and other industrial uses. The proposed residential usage will have a significantly lower sediment load. It is suggested that sedimentation basins are however used for all future soakholes in the area and this will appropriately reduce the risk of blockage.


6.0 Limitations

The information contained within this report applies to the date of the site inspection and soakage testing (14-22 April 2014). With time, the site conditions could change so that the reported assessment and conclusions are no longer valid. Thus, in the future, this report should not be used without confirming the validity of the report's information at that time.

This report has been prepared by PDP on the specific instructions of Fletcher Developments for the limited purposes described in the report. PDP accepts no liability if it is used for any other purpose without PDP's prior written consent. In addition, unless PDP agrees in writing to a request from a third party to rely on or use this report, PDP also accepts no liability to any other person for their use of or reliance on this report, and any such use or reliance will be solely at their own risk.

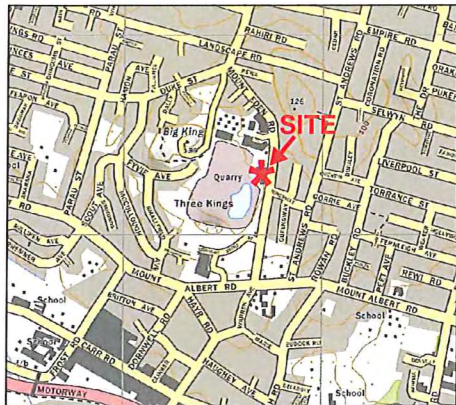
Yours faithfully

PATTLE DELAMORE PARTNERS LIMITED

A handwritten signature in black ink, appearing to read 'Roger Seyb', written over a horizontal line.

Roger Seyb

SOAKAGE HOLE DRILLING AND TESTING – THREE KINGS RENEWAL, UPPER HOUSING DEVELOPMENT AREA
– FLETCHER CONSTRUCTION COMPANY LTD



SITE LOCATION

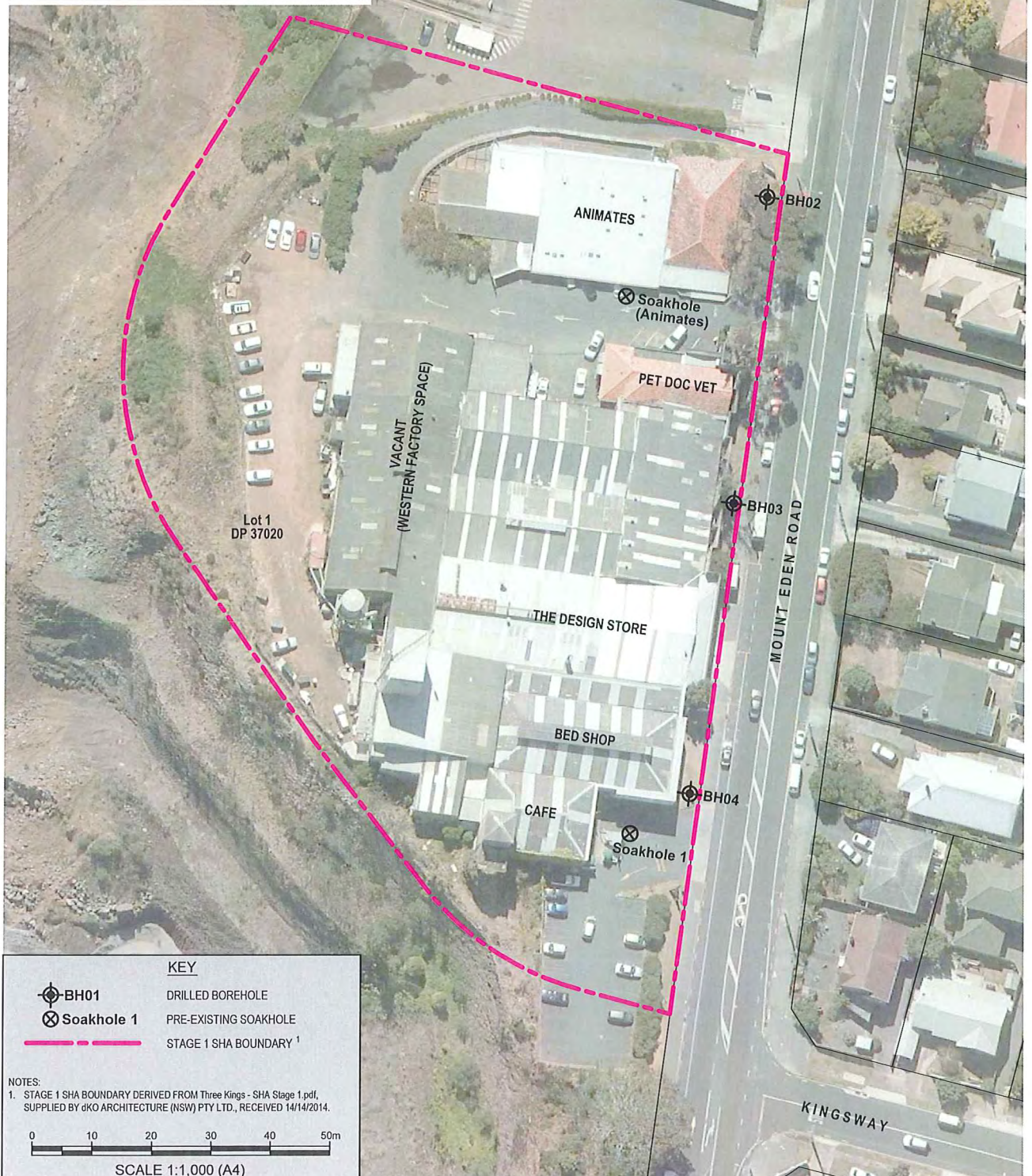
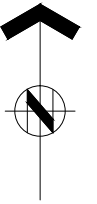


FIGURE 1: SITE PLAN



DRAFT ONLY

		ORIGINATOR:	DATE:	SIGNED:	PLOT BY:
		SLM	10.03.2014		SLM
		DRAWN:	DATE:	SIGNED:	PLOT DATE:
		SLM	10.03.2014		11.03.14
		CHECKED:	DATE:	SIGNED:	SURVEY BY:
		MZB	10.03.2014		
A	FOR DISCUSSION	SLM	11.03.2014	APPROVED:	DATE:
REF	REVISIONS	BY	DATE	MZB	10.03.2014
				SIGNED:	SURVEY DATE:



ASSOCIATION OF
CONSULTING ENGINEERS
NEW ZEALAND



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PROJECT:
MOUNT EDEN ROAD

TITLE:
STORMWATER AREAS C & D

ISSUE STATUS: FOR DISCUSSION PURPOSES		
PROJECT No: 1021-122314	SCALES: 1:500 (A1) 1:1,000 (A3)	A1
DRAWING No: 122314-SW01		REV A

Appendix D

Matrix of Stormwater Management
Options

Appendix D: Matrix of stormwater management options

Stormwater Management Option	Description	Flooding	Stormwater Quality	Practicality	Works/ Costs	Conclusions
Soakage without pumping	Construct a gravity pipe system within the site with stormwater fully contained within the site. Connect to new soakage system.	Construct development above flood levels and short and long term groundwater levels without pumping. Flood storage incorporated within sports fields.	The Greater Western Springs aquifer would be the receiving environment. TP10 stormwater treatment would be required.	System may require maintenance if high sediment loads are allowed to discharge to soakage.	Gravity pipe system within site. Stormwater quality/wetland. Flood storage within sports fields. Soakage discharge system.	Preferred methodology. Iwi support. Future proofs upgrades to Three Kings Network. Robust and certain.
Soakage with ongoing pumping	Construct a gravity pipe system within the site with stormwater fully contained within the site. Discharge to soakage and then pump out to existing stormwater system.	Construct development above short and long term groundwater levels without pumping. Use underground flood storage to attenuate stormwater flows.	The Greater Western Springs aquifer would be the primary receiving environment. TP10 stormwater treatment would be required.	Pump reliability and back-up issues.	Gravity pipe system within site. Stormwater quality/wetland. Flood storage underground. Soakage discharge system. Ongoing pumping.	Less preferred to other soakage option due to lack of preference for long term diversion of stormwater from its natural discharge.

Stormwater Management Option	Description	Flooding	Stormwater Quality	Practicality	Works/ Costs	Conclusions
Gravity piped stormwater system and tunnel	Construct a gravity pipe system within the site fully contained within the site. The closest gravity drainage point would be to tunnel 650m south-east to an AC MH near 595 Mt Albert Rd. More likely the connection would be further away where the receiving pipe system was larger.	The area drained by the existing pipe system which would receive the flow is identified as a flood plain and flood prone area at multiple points. Detention would be required on site to not exacerbate this. The downstream pipe system capacity is unknown, but it is suspected upgrades would be required.	The Manukau Harbour would be the ultimate receiving environment. TP10 treatment would be required.	Tunnelling would be through volcanic rock, sedimentary rock and require management of groundwater - requiring multiple tunnelling methods. It is not certain that sufficient detention could be provided to avoid exacerbating existing downstream flooding.	Gravity pipe system within site. Flood detention. Stormwater quality. Tunnelling. Potential downstream pipe upgrades.	Impractical. Very costly. No net downstream benefit.

Stormwater Management Option	Description	Flooding	Stormwater Quality	Practicality	Works/ Costs	Conclusions
Fill quarry, gravity piped stormwater system then discharge to existing public system	Fill the quarry to surrounding ground level. Construct a gravity pipe system within the site. Connect to existing public drainage to the adjacent catchment to the east and drain via the existing 1200 dia tunnel.	The area between Kingsway and St Andrews Rd to the east is identified as a flood plain and flood prone area. Very large detention would be required on site to not exacerbate this.	The Manukau Harbour would be the ultimate receiving environment. TP10 stormwater treatment would be required.	Filling operations would take many years and the source of fill is unknown. It is not certain that sufficient detention could be provided to avoid exacerbating existing downstream flooding.	Gravity pipe system within site. Very large flood detention. Stormwater quality treatment. Reduced development yield.	Filling timeline prohibitive. Downstream capacity issues.
Fill quarry, gravity piped stormwater system and then discharge to soakage	Fill the quarry to surrounding ground level. Construct a gravity pipe system within the site. Connect to new soakage system.	Stormwater would need to be fully contained within the site so that existing flooding to east was not exacerbated. High flow rate soakage or detention on site would be required to achieve this.	The Greater Western Sprigs aquifer would be the receiving environment. TP10 stormwater treatment would be required.	Filling operations would take many years and the source of fill is unknown. Greater difficulty in connecting to the aquifer.	Gravity pipe system within site. Stormwater quality treatment. Potential on site or underground flood storage prior to soakage. Soakage discharge system. Reduced development yield.	Filling timeline prohibitive. Far more constrained access to aquifer.