ENGINEERING REPORT

PROPOSED DEVELOPMENT

PARKLANDS AVE SUBDIVISION

STAGES 6, 7 & 8

FOR ROBE AND ROCHE INVESTMENTS

Red Jacket Limited, 3 Davidson St, New Plymouth 4310, PO Box 904, New Plymouth 4340, New Zealand
P. 06 759 0999 E. info@redjacket.nz www.redjacket.nz

CLIENT: ROBE AND ROCHE INVESTMENTS

PROJECT TITLE: PROPOSED DEVELOPMENT

DOCUMENT NUMBER: RPT-3917-01 REV-A

Issue	Description	Date	Prepared By	Checked By	Approved
A	CONSENT	Signature	W Adlam	L Bunn	C Miller
		Date	May 2021	May 2021	May 2021
		Signature			
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		Signature			
		Date			

Report Prepared by:

Will Adlam

Civil Structural Engineer MEngNZ

Reviewed by:

Luke Bunn

Senior Civil Engineer, CMEngNZ (Eng.Tech.)

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1. INTRODUCTION

Red Jacket Engineering have been engaged to report on the suitability of the proposed Parklands Avenue residential subdivision, Stages 6, 7, & 8, located at 56 Pohutukawa Place, Bell Block, Lot 2 DP 521660.

We have reported on the suitability of the proposed residential subdivision, specifically covering Stages 6, 7, & 8 for the purpose of subdivision Resource Consent.

The future development to the south shall utilise the proposed utilities installed as part of proposed Stages 6, 7, & 8 therefore, the future development has been evaluated as part of the overall network capacity assessment.

The extents of the proposed development are shown on the McKinlay Surveyors Ltd Scheme plan below, Figure 1-1, and Appendix A.

Figure 1-1 McKinlay Surveyors Scheme Plan

1.1 EXISTING SITE CONDITIONS

The proposed subdivision is located to the southwest of Parklands Avenue. The land is currently grassed and being utilised as a part of an operational farm.

There is an existing farm implement shed along the western boundary of the site which will removed as part of the proposed development.

Typically, the land covered by the proposed subdivision is relatively flat with only minor undulations. All the proposed lots are relatively flat and will require minimal earthworks to provide level building platforms.

The site currently falls in four main directions to four distinct overland flowpaths as indicated in Figure 1-2 below. Typically, all overland stormwater from the existing site discharges into the existing Waipu Lagoons to the north east and north west of the existing site.

The first flowpath includes the northern most portion of the site, discharging in the north western corner of the site to overland flow which eventually reaches the northern end of the Waipu Lagoon.

The second flowparth includes the central and eastern portion of the proposed development and discharges to the southern portion of the Waipu Lagoon.

The third flowparth includes the southern and western portion of the site and discharges via overland flow to the western portion of the Waipu Lagoon.

The fourth flowpath includes the north western portion of the site and discharges to the Waipu Lagoon to the west.

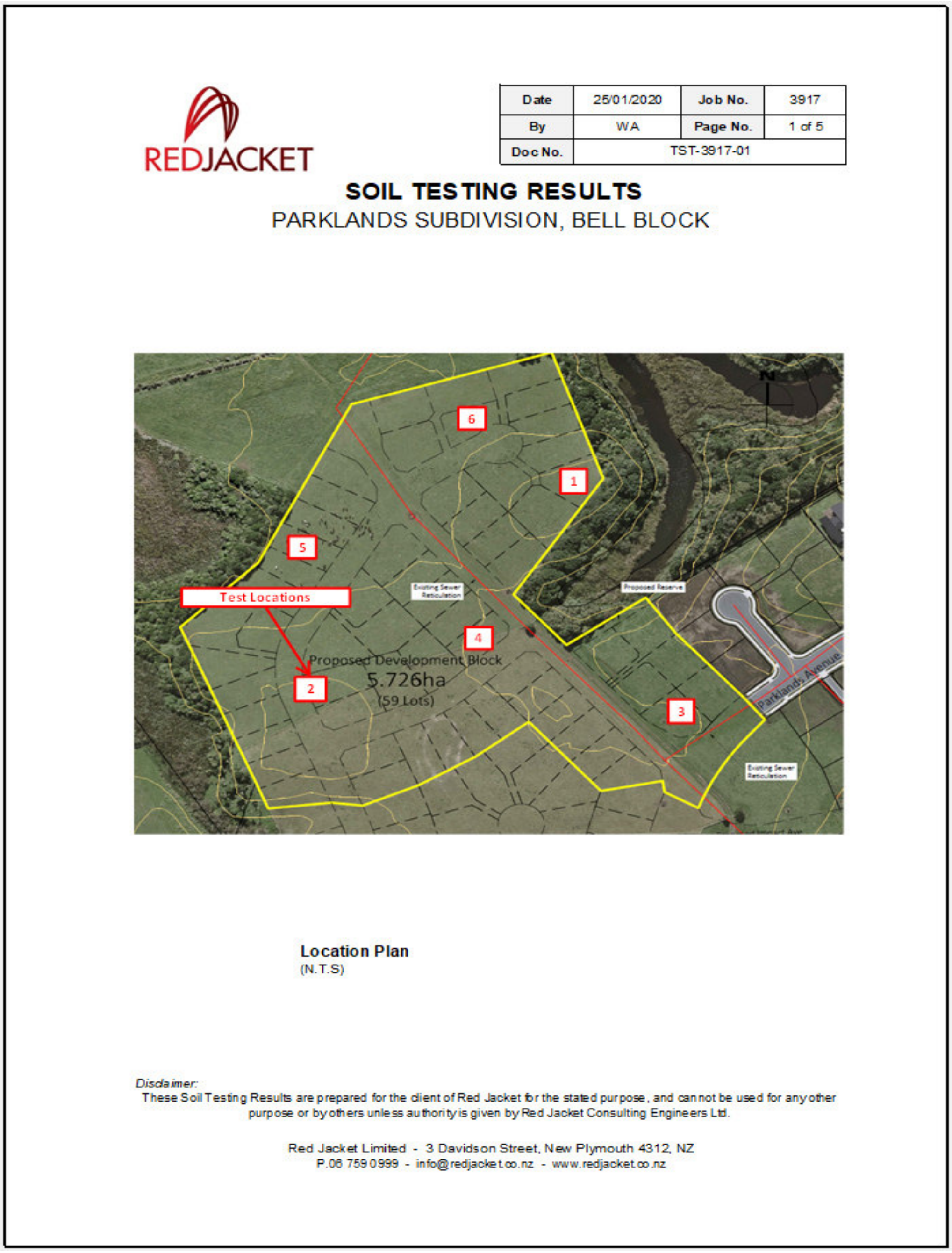
Figure 1-2: Existing overland flowpaths

2. SUBSOIL INVESTIGATION

An initial subsoil investigation for the proposed Lots and carriageway was completed on the 25th of January 2021.

The test locations are shown in Figure 2-1, and the results are provided in Appendix B.

All depths indicated are depths below existing ground level as of 25th January 2021.



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SOIL TESTING RESULTS
PARKLANDS SUBDIVISION, BELL BLOCK

Location Plan
(N.T.S)

Disclaimer:
These Soil Testing Results are prepared for the client of Red Jacket for the stated purpose, and cannot be used for any other purpose or by others unless authority is given by Red Jacket Consulting Engineers Ltd.

Red Jacket Limited - 3 Davidson Street, New Plymouth 4312, NZ
P.06 759 0999 - info@redjacket.co.nz - www.redjacket.co.nz

Figure 2-1: RJL Test Location Plan

2.1 BOREHOLES

Machine augered boreholes were undertaken at all test locations outlined above to depths between 8.0 and 16.0 m below ground level, bgl. Auger results were relatively consistent across the full extent of the site.

A general summary of all bore logs is shown in Tables 2-1 and 2-2 below and full borehole logs are provided in Appendix B.

The general soil summary indicates that the site is overlain by a 0.3 m thick layer of organic topsoil. Beneath the topsoil layer there where 2 distinctly different soil profiles observed in the different test locations.

In Test locations 1 and 2 the topsoil layer was underlain by relatively firm sandy silt (Taranaki Ash) which was in turn underlain by a relatively firm sand layer which extended to approx. 10.0 m bgl. This sand was then underlain by soft Taranaki Ash to 14.0 m BGL which was ultimately underlain by a very firm light grey sand layer.

In Test Locations 3, 4, 5 and 6 the overlying topsoil was followed by a layer of sand silt (Taranaki Ash) layer. This was in turn underlain by a Black Organic Peat layer which extended to approx. 13.0 m bgl where it was ultimately underlain by a very firm grey/ blue sand.

The groundwater table depth varied across the site from 1.95 m in test location 3 to 6.75 m at test location 2. It is thought that the groundwater is on a relatively level plain across the site and the variance in depth below ground level is predominantly a result of the surface contour.

Table 2-1: Typical Borehole Log Summary (Location 1 & 2)

Depth (m)	Soil Description	Shear Capacity	
		(Depth)	(kPa)
0 - 0.3	TOPSOIL, dark brown	-	-
0.3 – 3.0	Dark brown sandy SILT (Taranaki Ash), moist with low plasticity.	0.5 m – 3.0 m	Greater than 100 kPa
3.0 – 5.0	Light brown, sandy SILT (Taranaki Ash), moist and plastic.	-	-
5.0 – 10.0	SAND with some sandy silt (Taranaki Ash). Lenses of grey and dark brown, firm and moist.	-	-
13.0 – 16.0	Very firm SAND, light grey in colour.	Extended beyond borehole	-

Table 2-2: Typical Borehole Log Summary (Locations 3, 4, 5 and 6)

Depth (m)	Soil Description	Shear Capacity	
		(Depth)	(kPa)
0 - 0.3	TOPSOIL, dark brown	-	-
0.3 – 3.0	Dark brown sandy SILT (Taranaki Ash), moist with low plasticity.	0.5 m – 3.0 m	Greater than 100 kPa
3.0 – 5.0	Light brown, sandy SILT (Taranaki Ash), moist and plastic.	-	-
5.0 – 13.0	Black organic peat.	-	-
13.0 – 16.0	Very firm SAND, light grey in colour.	Extended beyond borehole	-

3. BUILDING PLATFORMS

3.1 FOUNDATIONS

Earthworks

The preliminary site inspection and soil investigation has determined that the proposed building platforms within all lots will require minimal earthworks to achieve a suitable building platform for a standard concrete slab construction type dwelling.

The topsoil layer was approximately 0.3 m deep and shall be stripped to the top of the ash material in the position of the proposed building platform prior to construction.

When constructing building platforms, the topsoil shall be replaced with imported hardfill material to a suitable building platform level. The hardfill shall be tested and certified by a suitably qualified engineer, ensuring there is adequate support for shallow foundations in accordance with NZS 3604:2011.

Where areas of fill greater than 600 mm in depth are required to for a level building platform, the fill shall be certified by a suitably qualified engineer.

Subsoil Conditions

We have assessed the existing subsoil conditions for all Lots, being to the underside of topsoil at the test locations. The subsoil conditions meet the strength requirements as outlined in NZS 3604:2011 'Good Ground' meaning 'any soil or rock capable of permanently withstanding an ultimate soil bearing capacity of 300 kPa, being an allowable bearing pressure of 100 kPa using a factor of safety of 3.0'.

The above statement is based on the testing undertaken for the purpose of Resource Consent only.

A detailed soil investigation may be required at the time of Building Consent to confirm subsoil conditions for the proposed foundations, and this report does not limit those results.

4. STORMWATER MANAGEMENT

4.1 PROPOSED DESIGN

The proposed stormwater management system includes the following:

- Developed stormwater from the proposed Lots shall utilise on-site stormwater disposal and overland flowpaths in accordance with NZBC E1.
- Developed stormwater from the proposed carriageway shall utilise a piped stormwater system and overland flowpaths in accordance with NPDC Land Development and Subdivision Infrastructure Standard.

The proposed stormwater management system can be split into four separate catchments, each with its own discharge point into the Waipu Lagoon as Identified in Figure 4-1 below.

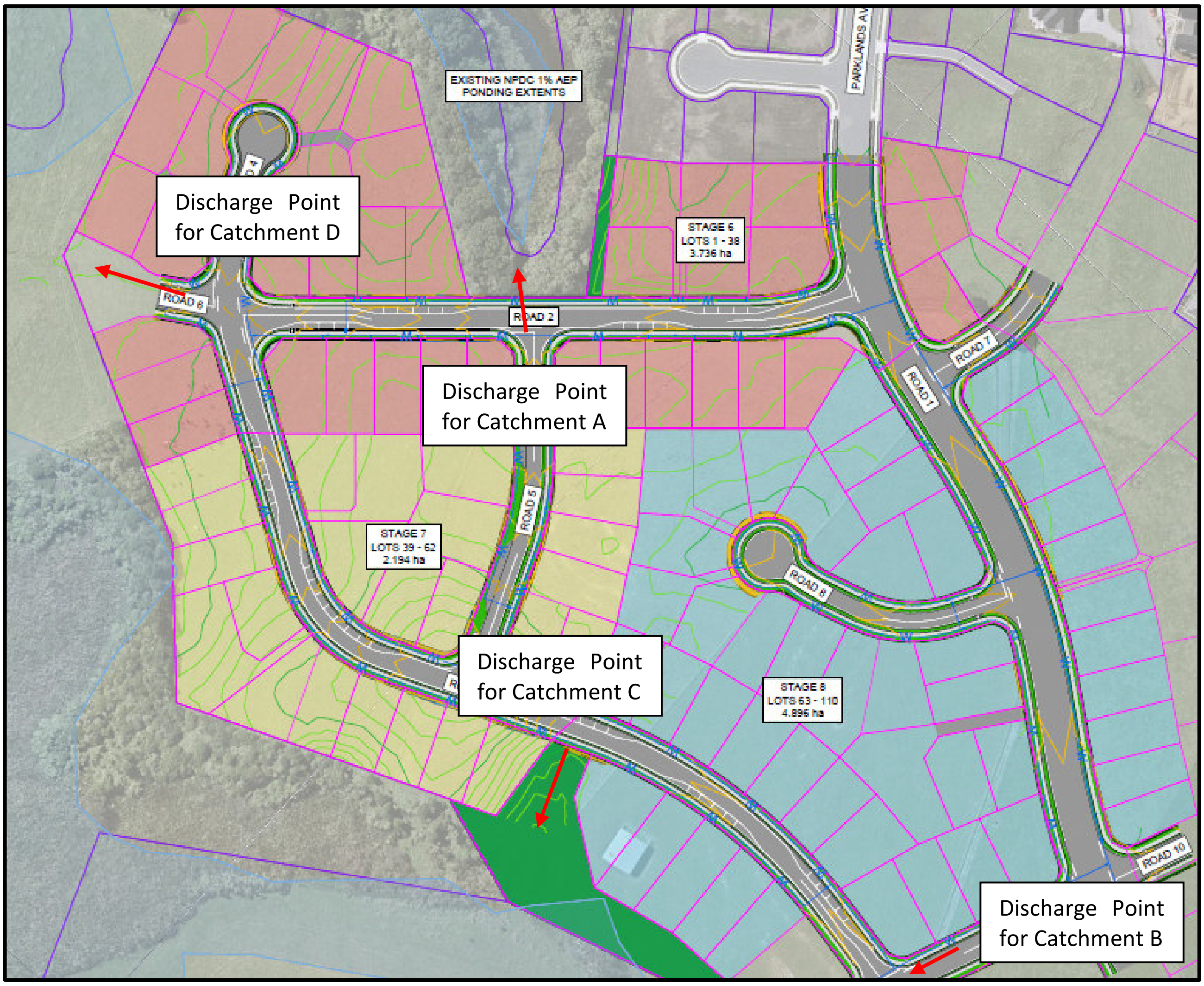

Figure 4-1: Stormwater discharge points

4.1.1 Primary System – Residential Lots

All residential Lots shall utilise onsite soakage systems as a primary means of stormwater disposal. All on-site soakage systems shall be designed and installed in accordance with NZBC E1.

All runoff from the proposed Lots that exceeds the capacity of the on-site soakage systems shall utilise proposed overland flow paths as outlined in Section 4.1.3 below.

4.1.2 Primary System – NPDC Vested Roadway

The primary system shall be a piped network accommodating up to a 10% AEP design storm (level of service) in accordance with the NPDC Land Development and Subdivision Infrastructure Standard. The peak design discharge at each of the four discharge points is outlined in Tables 4-1 to 4-4 below.

The primary system shall utilise a primary treatment system before discharging to the Waipu Lagoon. The primary treatment systems shall be Downstream Defender units as outlined in Section 4.4 below.

4.1.3 Secondary System

The secondary system shall follow the proposed carriageway with capacity to accommodate up to a 1% AEP design storm assuming the primary system is fully blocked.

The secondary system shall discharge at the same four discharge points as the primary system.

4.2 CATCHMENT ANALYSIS

4.2.1 Assumptions

The stormwater design is based on the following assumptions:

- NIWA Rainfall Intensity Data, HIRDS RCP 6.0 for the period 2081 – 2100, as shown in Appendix C. One event has been used across the full catchment.
- The proposed stormwater catchment including paved areas (run-off coefficient C = 0.9) and grassed/vegetated areas (run-off coefficient C = 0.30), in accordance with the New Zealand Build Code E1 (NZBC E1). A modified runoff coefficient of 0.69 has been used for the undeveloped land. The post developed catchments have a modified runoff coefficient of 0.75 for the residential lots and 0.85 for the developed carriageway.
- The piped stormwater system for the Road to vest shall provide a level of service for stormwater runoff up to a 10% AEP design storm with a secondary overland flow path providing a level of protection for dwellings up to a 1% AEP design storm as per NPDC Land Development Standard.
- The piped stormwater system shall discharge to the Waipu Lagoon as indicated in Figure 4-1 above, no stormwater detention shall be provided as part of this development.
- Based on the small nature of the proposed urban catchments the time of concentration shall be no less than 10 min.

4.2.2 Catchment Peak Flows

Table 4-1: Stormwater Catchment A – Catchpit Peak Flows

Catchpit	Catchment (m^2)	10% AEP FLOW (m^3/s)
CP 1A	1006	0.02
CP 2A	956	0.02
CP 3A	1051	0.02
CP 4A	478	0.01
CP 5A	613	0.01
CP 6A	949	0.02
CP 7A	1155	0.03
CP 8A	503	0.01
CP 9A	1155	0.03
CP 10A	2327	0.05
CP 11A	958	0.02
CP 12A	910	0.02
CP 13A	1013	0.02
CP 14A	482	0.01
CP 15A	523	0.01
Total	12682	0.25

Table 4-2: Stormwater Catchment B – Catchpit Peak Flows

Catchpit	Catchment (m^2)	10% AEP FLOW (m^3/s)
CP 1B	770	0.02
CP 2B	624	0.01
CP 3B	630	0.01
CP 4B	884	0.02
CP 5B	1065	0.04
CP 6B	1158	0.03
CP 7B	716	0.02
CP 8B	709	0.02
Total	5605	0.48

Table 4-3: Stormwater Catchment C – Catchpit Peak Flows

Catchpit	Catchment (m^2)	10% AEP FLOW (m^3/s)
CP 1C	952	0.02
CP 2C	1024	0.02
CP 3C	635	0.01
CP 4C	603	0.01
Total	3232	0.06

Table 4-4: Stormwater Catchment D – Catchpit Peak Flows

Catchpit	Catchment (m^2)	10% AEP FLOW (m^3/s)
CP 1D	727	0.02
CP 2D	827	0.02
CP 3D	1382	0.03
CP 4D	1117	0.03
CP 5D	697	0.02
CP 6D	705	0.02
Total	5394	0.34

4.2.3 Future Development

The area to the south of the proposed subdivision covered by this report is also planned for future development. Due to the contour to the existing land, the stormwater from this area will flow into the Stage 6 network when it is developed.

To account for this, the future upstream development has been estimated to have a roading network area of approximately 14,700 m^2 with a 10% AEP flow of 0.34 m^3/s.

For the assessment of pipe sizes, the entirety of the future development flow has been assumed to join the proposed Network B via SWMH 6 and discharged via Outlet B.

4.3 PIPE SIZING

The following pipe sizes have been selected for the proposed Road to vest stormwater system based on a 10% AEP design storm as per NPDC Land Development and Subdivision Infrastructure Standard, Table 4.1.

Pipe sizes are based on a maximum pipe capacity using the Colebrook White Formula and the continuity of flow, refer to Table 4-5.

$$V = Discharge\ veolcity\ (\mathrm{m}/sec) = -2(2gDS)^{0.5} log\left(\frac{k}{3.7D} + \frac{2.5v}{D(2gDS)^{0.5}}\right)$$

$$Q = Culvert\ Peak\ Flow\ (m^3/sec) = V \times A$$

$$k = Colebrook\ White\ Roughness\ Coefficient\ (m),$$

$$k = 0.003\ \mathrm{mm} - \mathrm{Concrete\ Pipe}$$

$$k = 0.015\ \mathrm{mm} - \mathrm{PE\ Pipe}$$

$$D = Pipe\ Diameter\ (m),$$

$$A = Area\ (ha), \quad S = Slope\ (m\ /\ m),$$

$$v = Kinematic\ viscosity\ of\ water\ (m^2\ /\ sec)$$

Table 4-5: Stormwater Pipe Sizes

PIPELINE	TOTAL FLOW (m^3/s)	TOTAL FLOW (l/s)	PIPE GRADE (%)	PIPE SIZE (DN)
SWMH1 - SWMH2	0.04	40	0.80	300
SWMH2 - SWMH3	0.04	40	0.80	300
SWMH3 - SWMH4	0.14	140	0.80	375
SWMH4 - SWMH7	0.17	170	0.80	375
SWMH7 - SWMH8	0.17	170	0.80	375
SWMH8 - SWMH9	0.19	190	0.80	375
SWMH17 - SWMH10	0.02	20	1.00	300
SWMH10 - SWMH9	0.06	60	1.00	300
SWMH9 - OUTLET A	0.25	250	0.80	450
SWMH20 - SWMH6	0.04	40	0.80	300
SWMH5 - SWMH6	0.07	70	1.00	300
SWMH6 - SWMH18	0.48	480	0.80	525
SWMH18 - SWMH19	0.51	510	0.80	600
SWMH19 - OUTLET B	0.51	510	0.80	600
SWMH16 - SWMH15	0.02	20	1.00	300
SWMH15 - SWMH14	0.02	20	1.00	300
SWMH14 - OUTLET C	0.06	60	2.50	300
SWMH13 - SWMH12	0.04	40	1.40	300
SWMH11 - SWMH12	0.06	60	0.80	300
SWMH12 - OUTLET D	0.1	100	0.80	300

4.4 PRIMARY TREATMENT SYSTEM

The primary treatment system will service the NPDC vested Roadway for up to a 10% AEP storm event. It is proposed to use an HYNDS Downstream Defender (DD) stormwater treatment system. It is proposed to install a DD unit at each of the four discharge points into the Waipu Lagoon.

Each of the four outlet points have had an appropriate DD unit sizes for them based on the 10% AEP flows that the outlet will see. A summary of the chosen units and their capacities is included in Table 4-6 below.

The primary treatment system for Outlet B has been sized based on the design flow for the proposed Stage 6 only. The primary treatment system for the future development flow shall be designed and upgraded in accordance with the design requirements.

Table 4-6: Downstream Defender sizing.

Outlet Structure	10% AEP FLOW (m^3/s)	Downstream Defender Model	Downstream Defender Capacity (m^3/s)
Outlet A	0.25	DD3000KIT	0.37
Outlet B	0.17	DD2550KIT	0.20
Outlet C	0.06	DD3.1800KIT	0.10
Outlet D	0.34	DD3000KIT	0.37

The Downstream Defender units have been chosen due to their ability to remove a range of pollutants including fine particles, floatable debris, Liquid and sedimentary hydrocarbons, heavy metals, and nutrients. The system can remove up to 60-90% of total suspended solids with a mean particle size of 150 microns. Refer to Figure 4-2 below for the Downstream Defender General Arrangement.

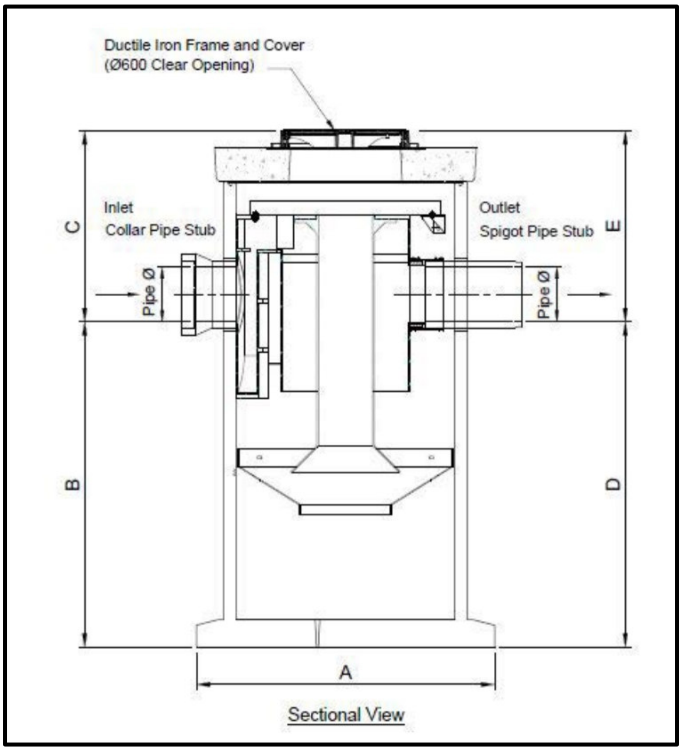

Figure 4-2: Downstream Defender General Arrangement – HYNDS Drawing Excerpt

4.5 STORMWATER NEUTRALITY

The stormwater design for the proposed subdivision is based on the princiapl of stormwater neutrality.

The predicted stormwater flowpaths for the stages covered by this report and likely future devlopment is shown in Figure 4-3 below and Appendix E.

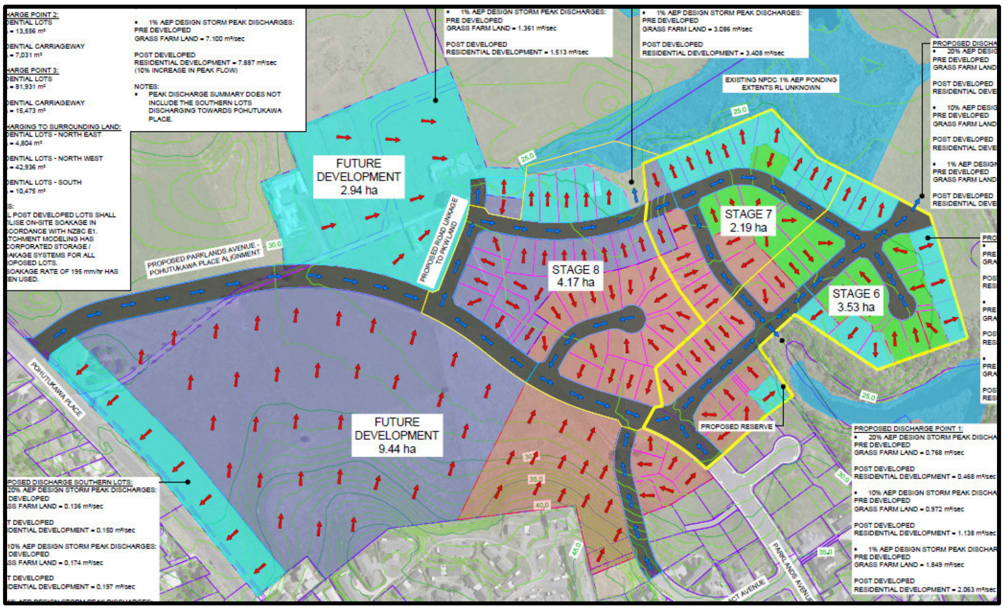

Figure 4-3: Preliminary Stormwater catchment plan

The pre and post development flows for each of the proposed discharge points are outlines in Table 4-7 below. At all locations, the post development 20% AEP flow is less than the predevelopment value. For both the 10% AEP and 1 % AEP flows, the post development value is approx. 10% greater than that of the predevelopment scenario.

Based on the location of the subdivision to the coast it is best engineering practice to allow the stormwater from the proposed development to discharge unrestricted to avoid the likelihood of impacts to the upstream catchment.

Table 4-7: Summary of pre and post development flows from preliminary analysis

Discharge Point	20% AEP		10% AEP		1% AEP	
	Pre Dvlp m^3/s	Post Dvlp m^3/s	Pre Dvlp m^3/s	Post Dvlp m^3/s	Pre Dvlp m^3/s	Post Dvlp m^3/s
A	0.768	0.468	0.972	1.138	1.849	2.063
B	0.558	0.337	0.715	0.808	1.361	1.513
C	1.266	1.007	1.622	1.864	3.086	3.408
D	0.268	0.196	0.343	0.403	0.652	0.737

4.6 OUTLET STRUCTURES

All outlet structures shall be designed at the detailed design stage, in conjunction with NPDC and local Iwi. The outlet structures shall be designed to minimise impacts on the adjacent Waipu Lagoon.

5. WASTEWATER MANAGEMENT

5.1 PROPOSED SYSTEM DESIGN

The wastewater system for the proposed subdivision has been designed to connect to the existing sewer main from south to north along the eastern extent of the subdivision. The proposed sewer layout can be divided into three separate sections, each connecting to their own existing sewer manhole or new as shown in Table 5-1 below, including 17 Lots long proposed Road 2 which connect directly to the existing main.

Table 5-1: Additional Sewer demand from proposed development.

Section	Number of Lots/ connections	Existing sewer connection	Max Flow (l/s)
1	40	40028987	1.6
2	9	NA	0.4
3	32 + 12	400289984	1.8
3A (Rising main)	12	NA	0.45
4 (Connections to existing)	17	40033532	0.64
5 (Proposed Future Lots)	90	NA	3.4
Total	**197**		**7.75**

Design assumptions

(a) Average daily flow shall be 250 L/person/day.

(b) Average dwelling occupants shall be 2.6 people/dwelling.

(c) Peak wet weather factor shall be 5.

Within the network mentioned above, 12 Lots (network 3A) between the intersections of Roads 3 & 5 and Roads 3 & 9 shall be designed as a pumping main with individual pumps for each Lot. The proposed common rising main shall connect to a manhole within the intersection of Roads 3 and 5 where it joins the section 3 gravity system.

In addition to the proposed new sewer system, it is proposed to remove three sections of existing sewer main as indicated in Figure 5-1 below as they will be superseded by the proposed system.

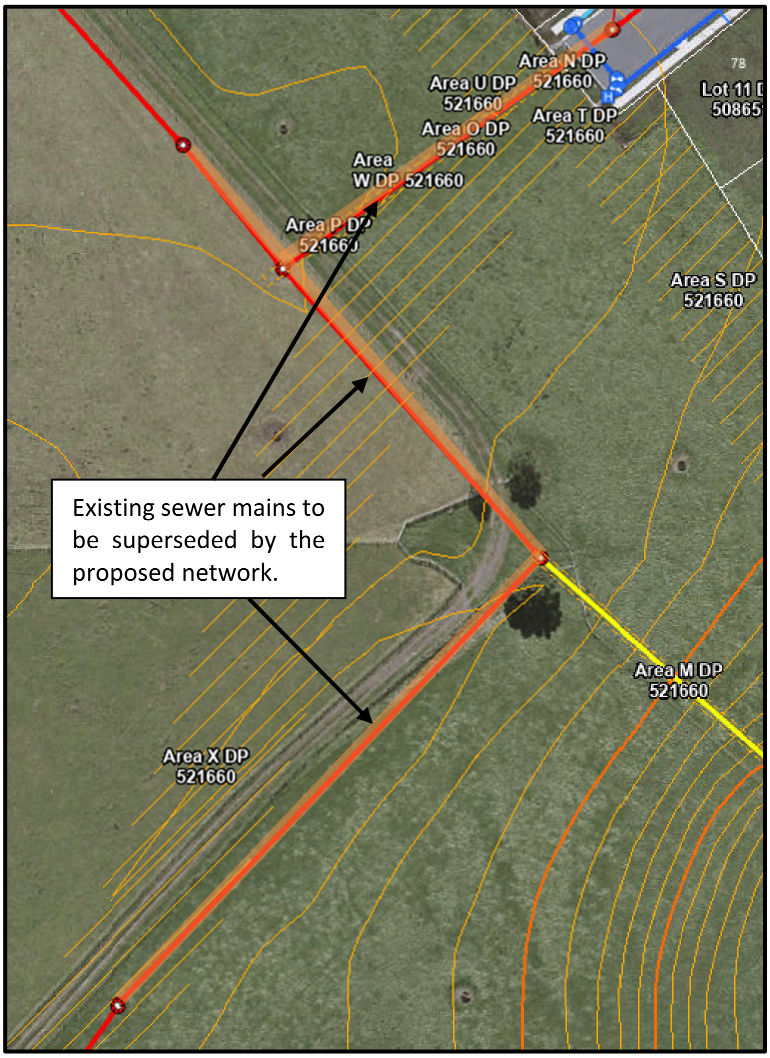

Figure 5-1: Existing sewer mains to be superseded.

5.2 NETWORK CAPACITY

The maximum sewer discharge from the proposed subdivision and proposed future development has been assessed as 648,000 L/day based on the assumptions in Section 5.1 above. NPDC shall confirm that the DN200 sewer main along eastern extent of the subdivision has adequate capacity for additional flow from the proposed development.

5.3 GRAVITY MAIN DESIGN

All gravity mains have been designed as DN150 UPVC pipes at minimum grade (0.67%). These gravity mains shall have a minimum capacity of 250 Lots as per Table 5.3 of NZS 4404. All laterals have been designed as DN 100 UPVC at or above minimum grade and are therefore suitable to service 1 dwelling unit as per Table 5.3 of NZS 4404.

5.4 PUMPING MAIN DESIGN

The rising main system has been designed with each lot having its own private pump station. These individual pump stations will pump into a common rising main beneath Road 3 and discharging into a chamber at the termination of the gravity system near the intersection of Roads 3 and 5.

The pipe sizes of both the rising main and laterals have been sized as DN 50 PN12 PE pipes for this preliminary stage of design. It is proposed that the pumping system including the individual pumps are designed by specialist firm Aquatec Enviro in accordance with WA-07 Pressure Sewer Design Guidelines.

6. RETICULATED WATER SUPPLY

6.1 RESIDENTIAL LOTS

The proposed reticulated water for Stages 6, 7 and 8 of the proposed development shall be an extension of the existing DN150 PE water main within the existing section of Parklands Avenue. As part of future stages of the development it is expected that this DN150 PE supply will be extended through to Pohutakawa Place. The likely watermain link to Pohutukawa Place has not been assessed as part of this report.

The proposed DN150 water mains and DN50 mm rider mains have been evaluated for capacity (Refer Table 6-1 below for evaluation) as per NPDC Land Development Standard, Table 6.2 and 6.3.

For the purpose of this design the DN50 mm rider main has been assumed as Medium pressure (400 - 600 kPa) and is a two-end supply.

Table 6-1: Watermain Capacity

Identity	Capacity of main (Residential Lots)	Design Capacity (Residential Lots)
DN150 Watermain	160	Stage 6 = 38 Stage 7 = 23 Stage 8 = 48
DN50 Rider main (Medium Pressure) Two end supply	30	Road 8 =12 Road 4 = 13 Road 5 = 3 Road 2 = 12 Road 3 = 17

6.2 FIRE FIGHTING

The purpose of this section is to confirm the suitability of the water supply network in regard to the proposed fire hydrants providing the FW2 requirements set out in SNZ PAS 4509:2008 *New Zealand Fire Service Firefighting Water Supplies Code of Practice*.

Design Check Assumptions

The following design assumptions have been made as part of the design check:

Water supply pipe and fire hydrant layout as per RJL water reticulation plan, refer RJL DWG 100-433.

In the absence of flow data provided by New Plymouth District Council, NPDC, the following values have been assumed. NPDC shall confirm that the available pressure is equal or greater than these values and if less the design shall be adjusted accordingly.

- Logged static pressure of 650 kPa
- Logged residual pressure of 550 kPa

The residual pressure was measured with a flow of 12.7 L/s at the NPDC fire hydrant ID 31850903 at the current termination of Parklands Avenue, near the existing intersection with Waipu View Drive while also releasing 12.5 L/s at Hydrant ID 21850998 on the other side of the intersection with Waipu View Drive to give a total residual flow of 25.2L/s.

Hazen-Williams equations for friction loss in pipe: $V = k.C.(D/4)^{0.63}.S^{0.54}$

where $S = h_f/L$ and $Q = V.\pi.D^2/4$

$Flow\ (Q) = 25\,L/s\ \ (required\ flow\ as\ per\ FW2)$

$Veloctiy\ (V) = Variable\ \ m/s$

$Pipe\ Diameter\ (D) = DN100\ water\ supply\ main$

$Hazen - Williams\ (C) = 130\ (for\ PE\ pipes)$

$k = 0.85\ (SI\ conversion\ factor)$

$Energy\ Slope\ (S) = variable\ m/m$

$Length\ (L) = variable\ m$

$Head\ Loss\ \left(h_f\right) = variable\ m$

All minor friction losses caused by pipe fittings have been omitted from the design checks. Only major friction losses have been accounted for including static head loss.

Summary

Table 6-2 below summarises the fire water flow and pressure information to obtain compliance with fire water classification FW2 as per SNZ PAS 4509:2008.

Table 6-2: Fire Water Flow and Pressure Summary

Hydrant	Elevation (m)	Change in head from existing (kPa)	Length from existing	Pressure Loss from friction	Total Pressure Loss (kPa)	Residual Pressure (kPa)	Compliance with FW2
FH1	25.96	-6.5	105	15.1	8.6	541	Y
FH2	27.02	-4.0	213	30.7	26.7	523	Y
FH3	26.17	+4.5	283	40.7	36.2	513	Y
FH4	25.53	-10.7	405	58.3	47.6	502	Y
FH5	24.34	-22.4	361	52	29.6	520	Y
FH6	25.38	-21.0	394	56.7	35.7	514	Y
FH7	24.75	-18.5	346	49.8	31.3	520	Y
FH8	24.91	-16.8	277	39.9	23.1	526	Y
FH9	25.94	-6.7	258	37.1	30.4	519	Y
FH10	24.73	-18.5	142	20.4	1.9	548	Y

7. PAVEMENT DESIGN

The pavement for proposed roading network has been designed by way of mechanistic analysis pavement design using CIRCLY 7.0 and in accordance with Austroads pavement design guide for unbound flexible pavements and the NZTA's NZ supplement to Austroads pavement design.

7.1 SUBSOIL CONDITIONS

Due to the extent of earthworks to be completed to form the carriageway subgrade, specific subgrade soil testing will not be carried out until the road formation has been formed to subgrade level.

The road formation will have varying depths of cut and fill throughout the network up to 2.0 m in depth.

It is proposed to use Taranaki Ash cut material from the site works for all fill under the proposed carriageway.

For the purpose of this design a conservative CBR value of 2% has been selected to represent the overall subgrade strength. This is based on the likely strength of the proposed Taranaki Ash fill beneath the subgrade level in some areas of the carriageway and the observed subsoil strength during preliminary soil testing.

The subsoil conditions shall be confirmed at the time of subgrade inspection and the pavement design updated accordingly.

7.2 DESIGN TRAFFIC

The design traffic for use in the CIRCLY 7.0 analysis is calculated as per the Austroads Pavement Design Guide 2004, AP DG 2004 Section 7.4.1.

Table 7-1: Austroads Table 12.2 – Indicative heavy vehicle axle group volumes for lightly- trafficked urban streets.

Street type	AADT two-way	Heavy vehicles (%)	Initial daily heavy vehicles in design lane (single lane)	Design period (years)	Annual growth rate (%)	Cumulative growth factor[1]	Axle groups per heavy vehicle	Cumulative HVAG over design period	ESA/ HVAG	Indicative design traffic (ESA)
Minor with single lane traffic	30	3	0.9	20	0	20	2.0	13 140	0.2	3×10^3
				40	0	40	2.0	26 280	0.2	5×10^3
Minor with two lane traffic	90	3	1.35	20	0	20	2.0	19 710	0.2	4×10^3
				40	0	40	2.0	39 420	0.2	8×10^3
Local access with no buses	400	4	8	20	1	22.0	2.1	128 480	0.3	4×10^4
				40	1	48.9	2.1	285 576	0.3	9×10^4
Local access with buses	500	6	15	20	1	22.0	2.1	240 900	0.3	8×10^4
				40	1	48.9	2.1	535 455	0.3	1.5×10^5
Local access in industrial area	400	8	16	20	1	22.0	2.3	256 960	0.4	1.5×10^5
				40	1	48.9	2.3	571 152	0.4	3×10^5
Collector with no buses	1200	6	36	20	1.5	23.1	2.2	607 068	0.6	4×10^5
				40	1.5	54.3	2.2	1 427 004	0.6	10^6
Collector with buses	2000	7	70	20	1.5	23.1	2.2	1 180 410	0.6	8×10^5
				40	1.5	54.3	2.2	2 774 730	0.6	2×10^6

The cumulative number of Heavy Vehicle Axle Groups associated with traffic along each of the proposed roads over the design period is obtained from the following equation using the appropriate data from Table 7-1 above:

DESA = ESA/HVAG x AADT x DF x %HCV/100 x N/HVAG x LDF x CGF x 365

Where,

DESA = design number of Equivalent Standard Axles, ESA's

ESA/HVAG = average number of ESA's per heavy vehicle axle group

AADT = annual average daily traffic

DF = direction factor

%HCV = average percentage of all traffic comprising of heavy vehicles

N/HVAG = average number of axle groups per heavy vehicle

LDF = lane distribution factor

CGF = cumulative growth factor

Table 7-2: Design ESA's for each of the proposed roads

Street Type	Roads included	DESA
Minor with single lane traffic	Roads 5 & 8	$5x10^3$
Local access with buses	Roads 2, 3, 4, 7 & 9	$1.5x10^5$
Collector with busses	Road 1	$2x10^6$

7.3 MECHANISTIC ANALYSIS

Using CIRCLY 7.0 pavement analysis software, mechanistic analysis has been completed to determine a minimum pavement depth for the proposed Roads for the determined DESA and subgrade conditions.

The CIRCLY model has been completed allowing for the following assumptions:

Cohesive subgrade material being Taranaki Ash with a minimum CBR of 2%.

Granular pavement made up of imported AP40 and AP65. Characteristics are based on Austroads Pavement Design Guide.

Design Traffic load:

Austroads Standard Axle – Dual Tyres – Axle Group Load – 80 kN.

Contact Stress 0.75 kPa, Contact Radius 92.1 mm - Austroads Pavement Design Guide.

Design traffic (DESA) calculated above.

Minimum 150 mm AP40 thickness, AP65 thickness determined by CIRCLY analysis.

Table 7-2, 7-2, and 7-3 below identifies design cumulative deformation factors, CDF for the proposed pavement, with acceptable pavements having a CDF < 1. As shown, the proposed pavement does achieve CDF values < 1 at both the subgrade and surfacing interfaces.

Table 7-3: CIRCLY Pavement Analysis Results – Minor with Single Lane Traffic

Layer No.	ID	Material	Modulus (MPa)	Poisson's Ratio	Thickness	CDF
1	Surfacing	AC – DG10	2,200	0.40	30 mm	1.00×10^{-00}
2	Pavement	AP40	500	0.35	150 mm	-
3	Pavement	AP65	450	0.35	200 mm	-
4	Subgrade	Cohesive soil, CBR 2%, volcanic ash	20	0.45	-	$1..00 \times 10^{-00}$

Table 7-4: CIRCLY Pavement Analysis Results – Local Access with Buses

Layer No.	ID	Material	Modulus (MPa)	Poisson's Ratio	Thickness	CDF
1	Surfacing	AC – DG10	2,200	0.40	30 mm	8.59×10^{-1}
2	Pavement	AP40	500	0.35	150 mm	-
3	Pavement	AP65	450	0.35	310 mm	-
4	Subgrade	Cohesive soil, CBR 2%, volcanic ash	20	0.45	-	9.04×10^{-1}

Table 7-5: CIRCLY Pavement Analysis Results – Collector with Buses

Layer No.	ID	Material	Modulus (MPa)	Poisson's Ratio	Thickness	CDF
1	Surfacing	AC – DG10	2,200	0.40	30 mm	5.71×10^{-1}
2	Pavement	AP40	500	0.35	150 mm	-
3	Pavement	AP65	450	0.35	400 mm	-
4	Subgrade	Cohesive soil, CBR 2%, volcanic ash	20	0.45	-	8.38×10^{-1}

7.4 PAVEMENT DEFLECTIONS

The proposed road surfacing is to be asphaltic concrete, AC and as such the New Plymouth District Council, NPDC require stringent maximum deflections at the pre-seal surface, that being 1.5 mm average deflections with a maximum of 1.8 mm.

There is a risk that pavement deflections may not achieve the maximum average of 1.5 mm during the pre-seal Benkelman beam testing. In our experience, volcanic ash subgrades in Taranaki tend to recover and improve strength characteristics over time, once pavements are constructed and sealed.

7.5 SUBGRADE REINFORCEMENT

Due to the proposed fill material within the carriageway and typically variable parent Taranaki As, we recommend a layer of geogrid reinforcement placed at the subgrade interface on all roads, overlying a layer of filter fabric to prevent the migration of fines from the subgrade into the pavement.

7.6 PAVEMENT

We recommend the following pavement profile, based on the pavement design undertaken:

30 mm thick DG10 asphaltic concrete – Polymer may be required post Benkelman Beam testing.

Grade 4 hard chip prime coat.

AP40 Basecourse – TNZM4. (As per the Circly design table above)

Subbase – TNZM3 (As per the Circly design table above)

Duragrid 30x30 Geogrid

A29 Filter Fabric

In the event that maximum pavement deflections are exceeded at the pre-seal stage, it is likely that deflections will be achieved prior to the end of a 12-month defects liability period.

It should also be noted that this design for the unbound granular pavement only and does not include the mix design for the proposed AC surfacing. This should be completed and confirmed by the contractor prior to construction.

7.7 PAVEMENT TESTING PRIOR TO SURFACING

Testing shall be undertaken on the finished pavement surface to ensure the base-course is compacted to NZTA B/2 specification density requirements achieving 98% MDD with no single test being less than 95% MDD, as per NPDC & STDC Land Development and subdivision Infrastructure Standard, based on NZS4404:2010, incorporating amendment No.1. Testing shall include Nuclear densometer (NDM) testing by a suitable qualified lab with readings taken in the wheel path of both lanes at a maximum interval of 10m.

Benkelman Beams shall be carried out on the finished pavement surface prior to surfacing (readings shall be taken in the wheel path in both lanes and at a maximum interval of 10 m) achieving a minimum average deflection of 1.5 mm with no single test greater than 1.8 mm deflection, as per NPDC & STDC Land Development and subdivision Infrastructure Standard, based on NZS4404:2010, incorporating amendment No.1.

All test results shall be supplied to Red Jacket Ltd for approval prior to surfacing.

8. LIMITATIONS

This report is prepared for the use of Robe and Roche Investments Limited for the purpose of the proposed development.

This report cannot be used for any other purpose or by others unless authority is given by Red Jacket Ltd.

Appendix A McKinlay Surveyors Scheme Plan

McKinlay SURVEYORS

28A Manadon Street
PO Box 116
New Plymouth 4340
New Plymouth 06 758 5342
Hawera 06 278 4456
www.mckinlaysurveyors.co.nz

TITLE
PROPOSED SUBDIVISION OF LOT 2 DP 521660
56 Pohutukawa Place, Bell Block, New Plymouth
SCHEME PLAN

RECORD OF TITLE	
825757	
TOTAL AREA	JOB No
23.8797ha	R-210409
DATE	DWG No
09/05/21	01
SCALE	SHEET OF
1:2500@A3	1 1

APPLICANT: Robe and Roche InvestmentsLimited

TERRITORIAL AUTHORITY: New Plymouth District Council

This plan is prepared only for the purpose of obtaining a Resource Consent pursuant to the Resource Management Act 1991.
It must not be used for any other purpose. Areas and dimensions are approximate only and are subject to change on final field survey.

Appendix B Soil Testing Results

Date	25/01/2021	Job No.	3917
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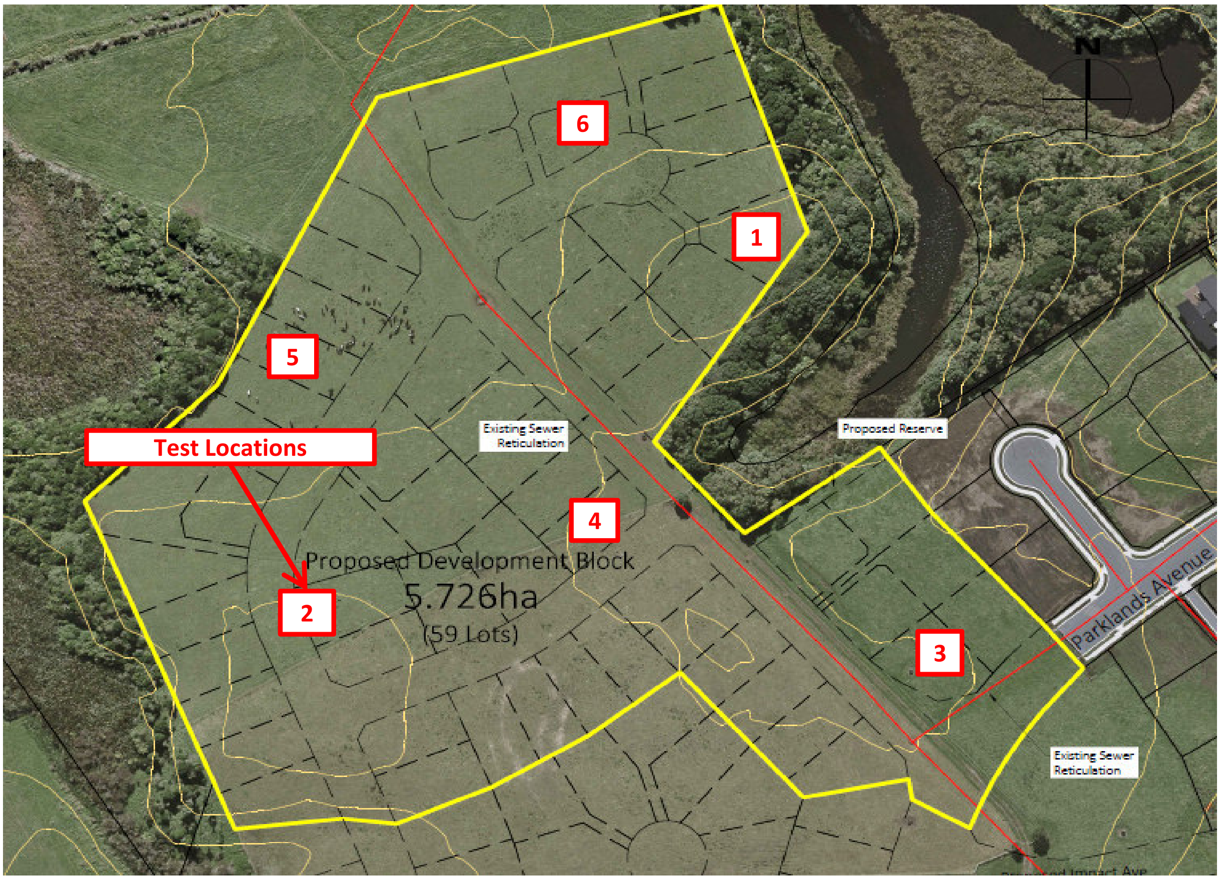
Doc No.	TST-3917-01		

SOIL TESTING RESULTS

PARKLANDS SUBDIVISION, BELL BLOCK



Location Plan
(N.T.S)

Disclaimer:
These Soil Testing Results are prepared for the client of Red Jacket for the stated purpose, and cannot be used for any other purpose or by others unless authority is given by Red Jacket Consulting Engineers Ltd.

Red Jacket Limited - 3 Davidson Street, New Plymouth 4312, NZ
P.06 759 0999 - info@redjacket.co.nz - www.redjacket.co.nz

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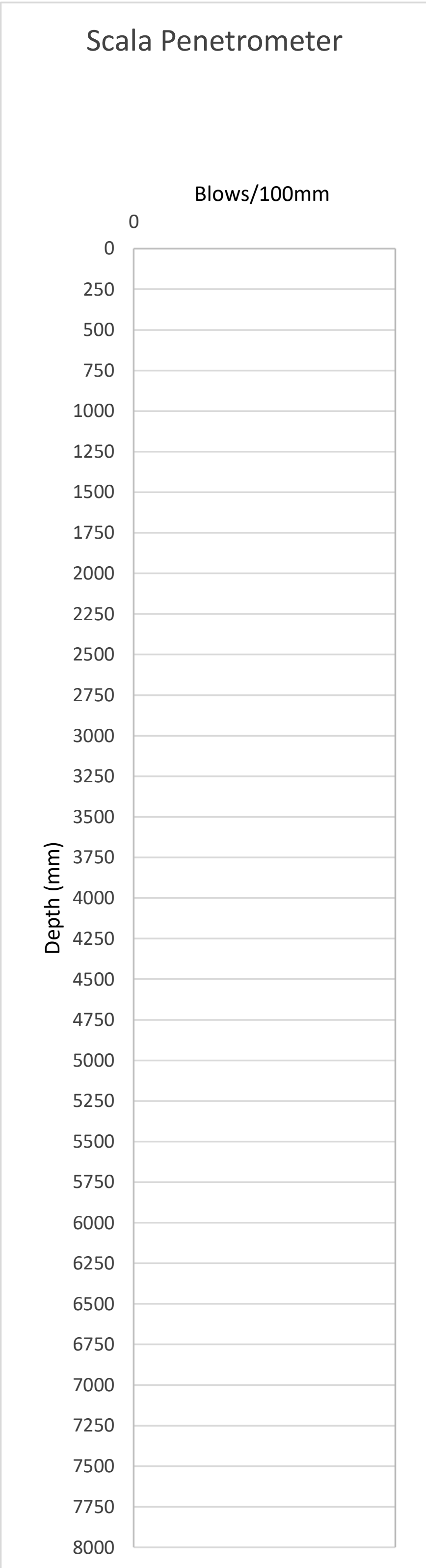
SOIL TESTING RESULTS - Location No. 1

PARKLANDS SUBDIVISION, BELL BLOCK



NOTES: Part 1 - For the bor elog from 8 m to 16 m depth see page 2.
GWT at 5.7 m depth.

Shear Vane Used: Serial Number 2011

Shear Strength (kPa)	Soil Class.	Soil Type
157 / 47	OH	Dark organic topsoil. (0.5 m)
141 / 47 157 / 94	CL	Sandy SILT (Taranaki Ash),dark brown, moist and firm.
180 / 71 157 / 71	CH	Sandy SILT (Taranaki Ash),dark brown, moist with high plasticity.
	MH	Sandy SILT (Taranaki Ash), light grey/brown in colour with come coarse black sand. Moist and plastic.
	SC	SAND with some sandy silt (Taranaki Ash). Lenses of grey and dark brown, firm and moist.

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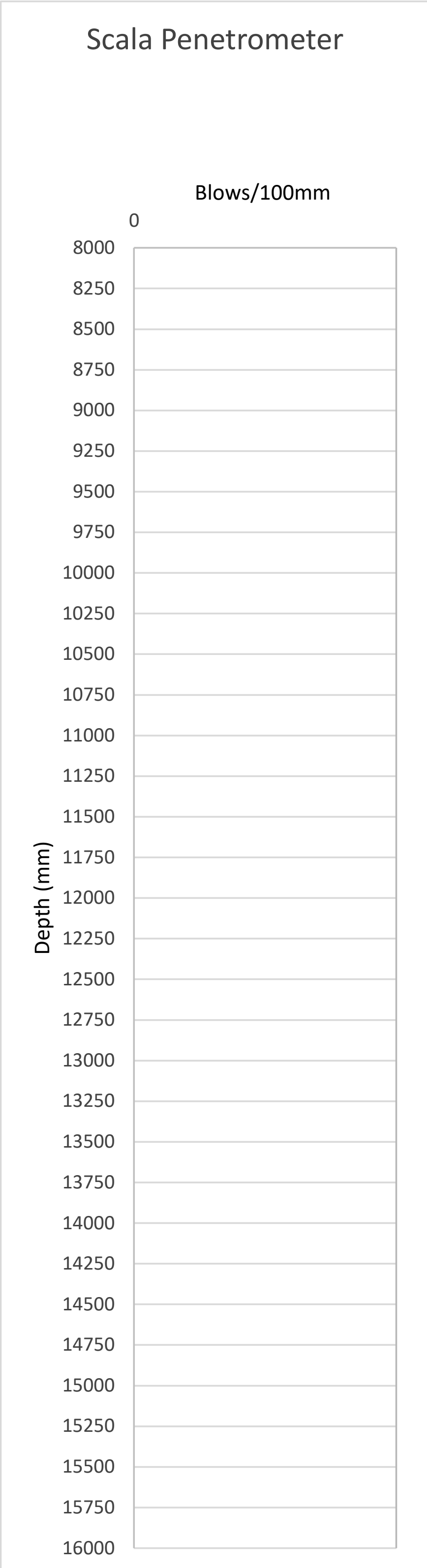
SOIL TESTING RESULTS - Location No. 1

PARKLANDS SUBDIVISION, BELL BLOCK



NOTES: For the bore log from 0 m to 8 m depth see page 2.

Shear Vane Used: Serial Number 2011

Shear Strength (kPa)	Soil Class.	Soil Type
	SC	SAND with some sandy silt (Taranaki Ash). Lenses of grey and dark brown, firm and moist.
	CH	Silty SAND (Taranaki Ash). Soft, moist and very plastic.
	SW	SAND, very firm and wet.

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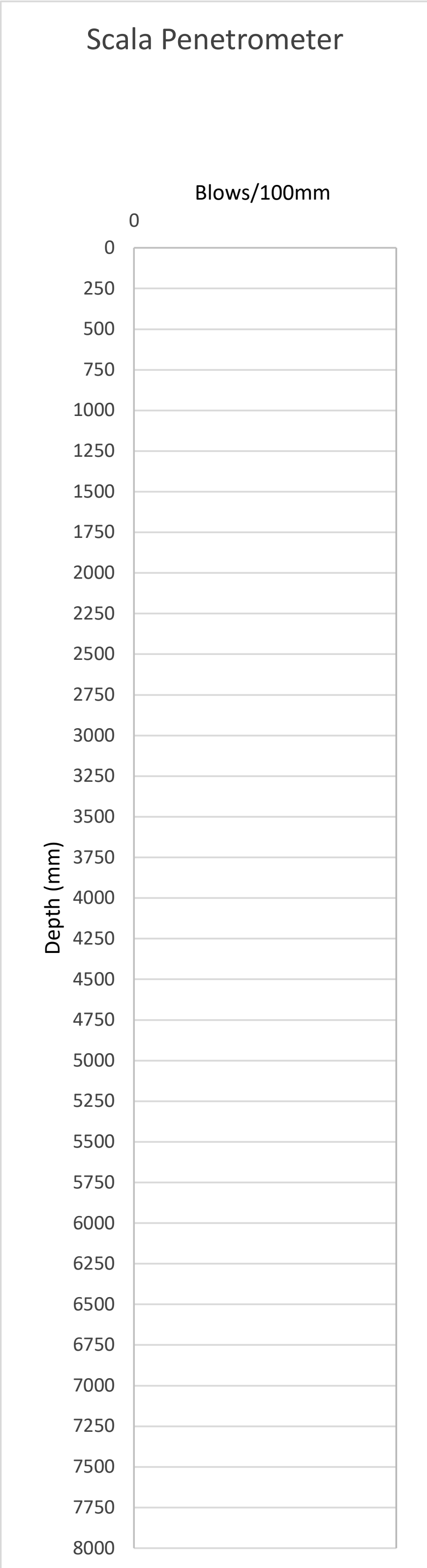
SOIL TESTING RESULTS - Location No. 2

PARKLANDS SUBDIVISION, BELL BLOCK



NOTES: Part 1 - For the bore log from 8 m to 16 m depth see page 5.
GWT at 6.75 m depth.

Shear Vane Used: Serial Number 2011

Shear Strength (kPa)	Soil Class.	Soil Type
	OL	Dark brown organic topsoil, firm and sandy.
188 / 71 188 / 55 133 / 63 172 / 63 141 / 31	CL	Sandy SILT (Taranaki Ash),dark brown, moist and low plasticity.
125 / 39	CH	Sandy SILT (Taranaki Ash), brown, moist, very firm with high plasticity.
	SC	Sandy SILT (Taranaki Ash), orangy brown in colour with traces of sand, moist and plastic.
	SW	SAND, hard, light grey/ brown and moist.

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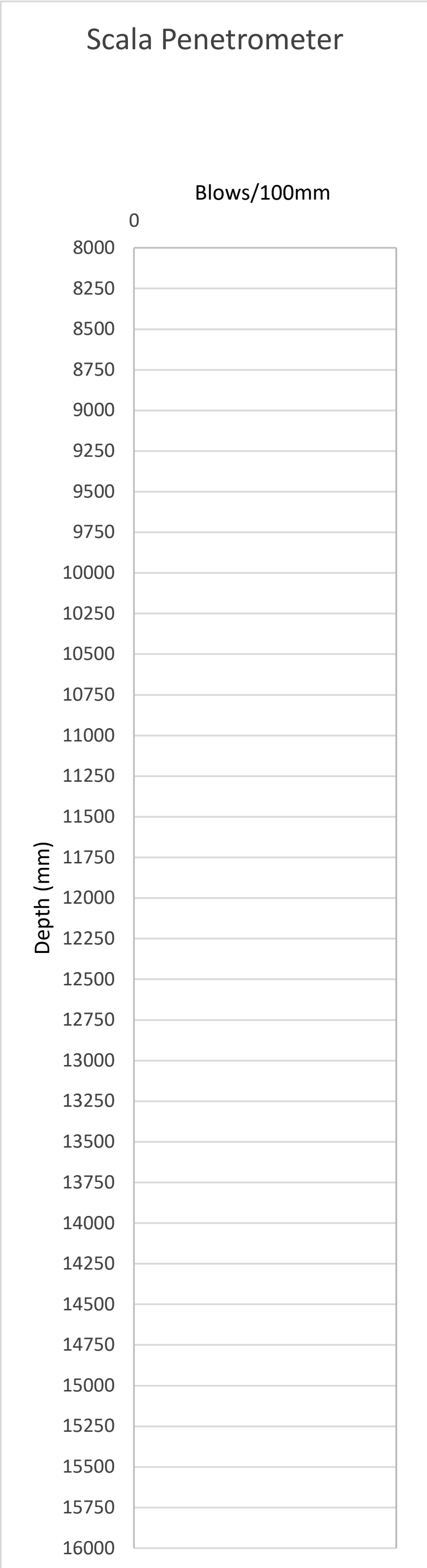
SOIL TESTING RESULTS - Location No. 4

PARKLANDS SUBDIVISION, BELL BLOCK



NOTES: For the bore log from 0 m to 8 m depth see page 4.

Shear Vane Used: Serial Number 2011

Shear Strength (kPa)	Soil Class.		Soil Type
		SW	SAND, hard, light grey/ brown and moist.
		SC	Silty SAND (Taranaki Ash). Soft, moist and very plastic.
		SW	SAND, very firm, light grey in colour.

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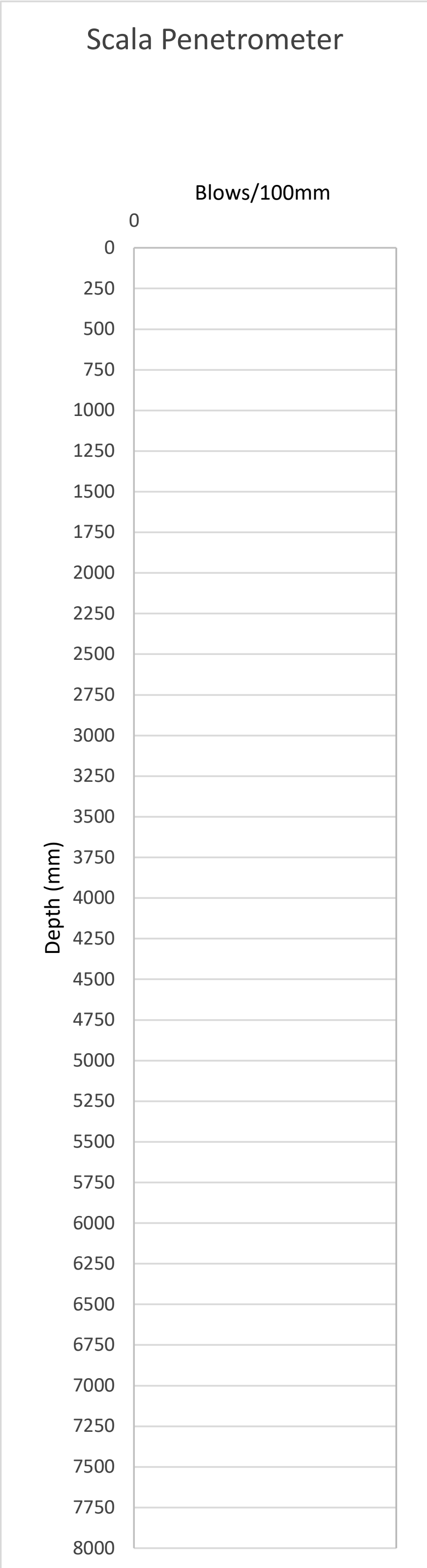
SOIL TESTING RESULTS - Location No. 3

PARKLANDS SUBDIVISION, BELL BLOCK



NOTES: Part 1 - For the bor elog from 8 m to 16 m depth see page 7.
GWT at 1.95 m depth.

Shear Vane Used: Serial Number 2011

Shear Strength (kPa)	Soil Class.	Soil Type
	OL	Dark brown organic topsoil, firm and sandy.
157 / 63 172 / 78 133 / 63	CL	Sandy SILT (Taranaki Ash),dark brown, moist and low plasticity.
157 / 63 204 / 47 219 /	CH	Sandy SILT (Taranaki Ash), brown, moist, very firm with high plasticity.
	PT	Organic PEAT. Wet, black in colour with wood debris and traces of sand.

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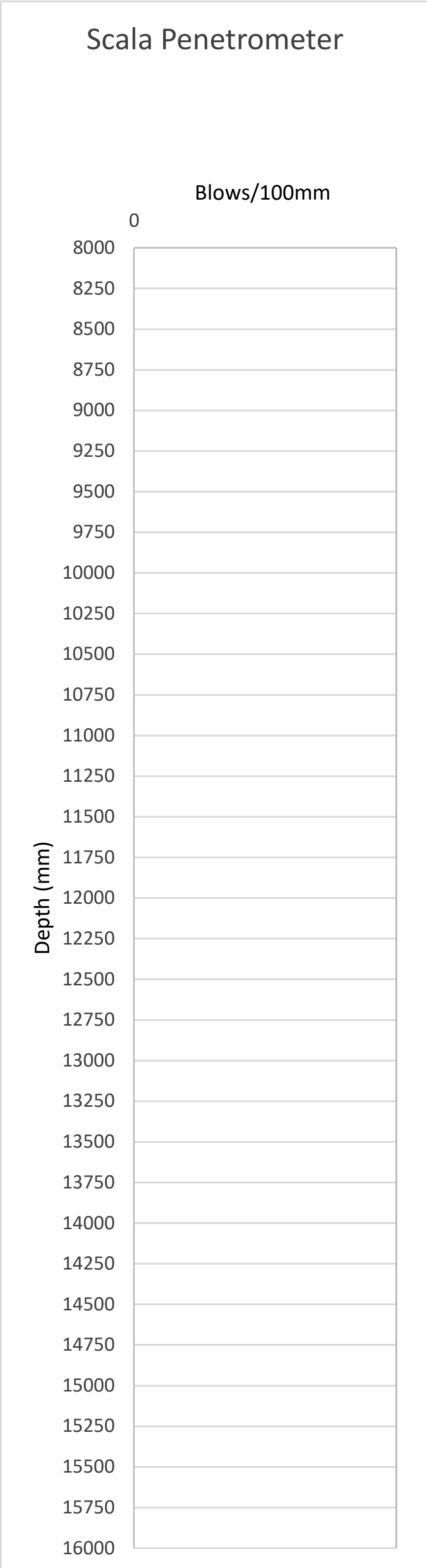
SOIL TESTING RESULTS - Location No. 3

PARKLANDS SUBDIVISION, BELL BLOCK



NOTES: For the bore log from 0 m to 8 m depth see page 6.

Shear Vane Used: Serial Number 2011

Shear Strength (kPa)	Soil Class.		Soil Type
		PT	Organic PEAT. Wet, black in colour with wood debris and traces of sand.
		SW	SAND, very firm, grey/ blue in colour.
			End Of Hole

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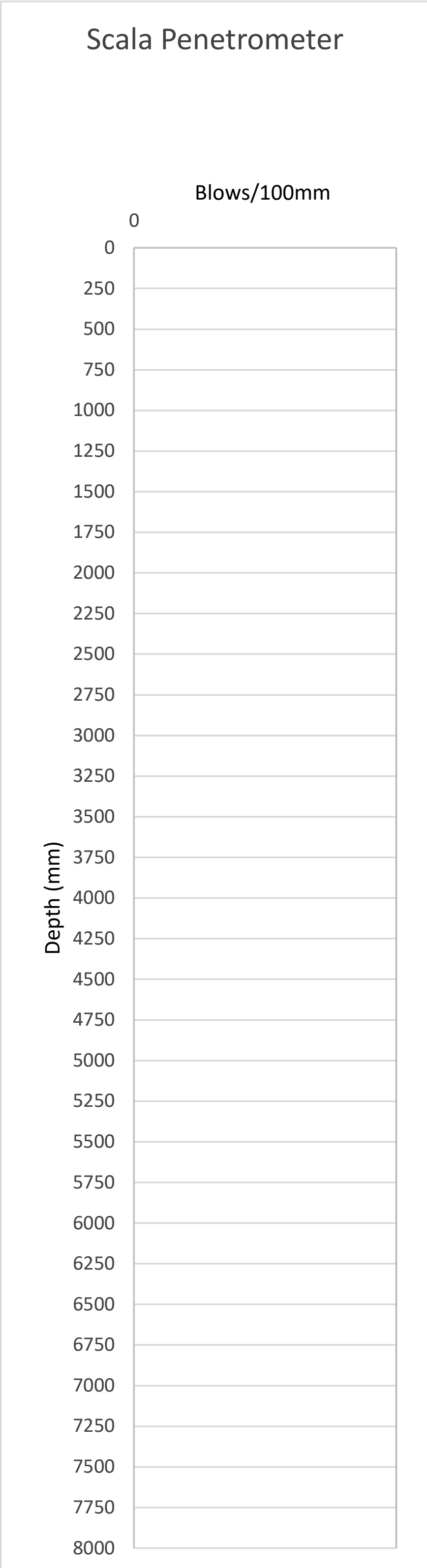
SOIL TESTING RESULTS - Location No. 4

PARKLANDS SUBDIVISION, BELL BLOCK



NOTES: GWT at 2.5 m depth.

Shear Vane Used: Serial Number 2011

Shear Strength (kPa)	Soil Class.	Soil Type
		Dark brown organic topsoil, firm and sandy.
141 / 47 204 / 71 172 / 63	CL	Sandy SILT (Taranaki Ash),dark brown, moist and low plasticity.
219 / 219 / 188 /	CH	Sandy SILT (Taranaki Ash), brown, moist, very firm with high plasticity.
	SC	Sandy SILT, (Taranaki Ash). Pale brown in colour, moist and very plastic.
	ML	SILT with traces of sand, grey in colour, soft and moist.
		End Of Hole

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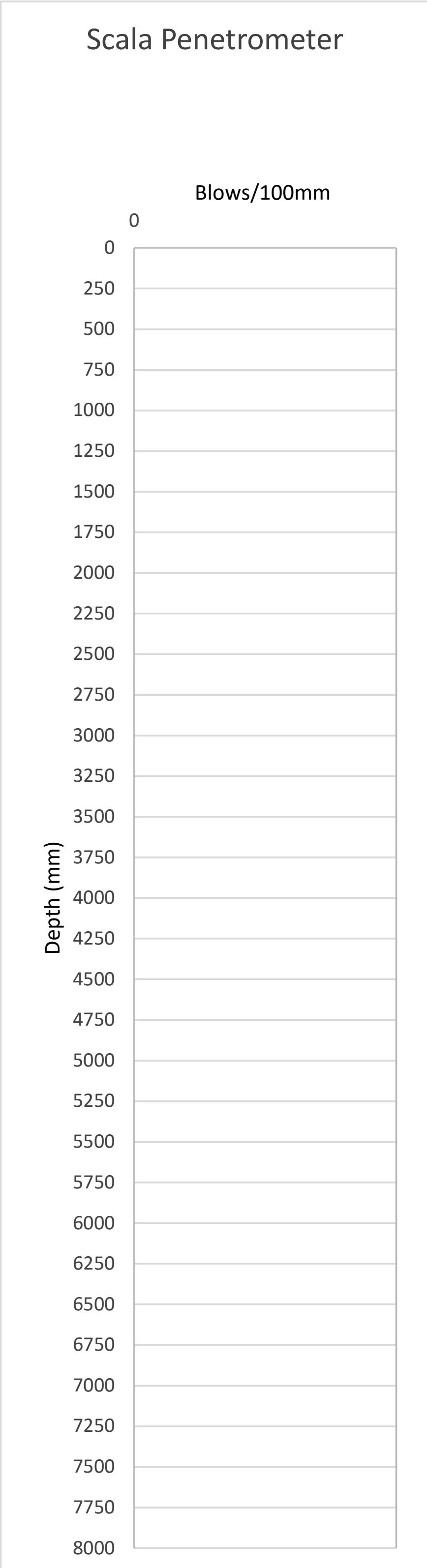
SOIL TESTING RESULTS - Location No. 5

PARKLANDS SUBDIVISION, BELL BLOCK



NOTES: GWT at 2.7 m depth.

Shear Vane Used: Serial Number 2011

Shear Strength (kPa)	Soil Class.	Soil Type
110 / 47	OL	Dark brown organic topsoil, firm and sandy.
172 / 78		
141 / 78		
180 / 63	CL	Sandy SILT (Taranaki Ash),dark brown, moist and low plasticity.
204 / 71		
204 / 55		
	CH	Sandy SILT (Taranaki Ash), brown, moist, very firm with high plasticity.
	PT	Organic PEAT. Wet, black in colour with wood debris and traces of sand.
		End Of Hole

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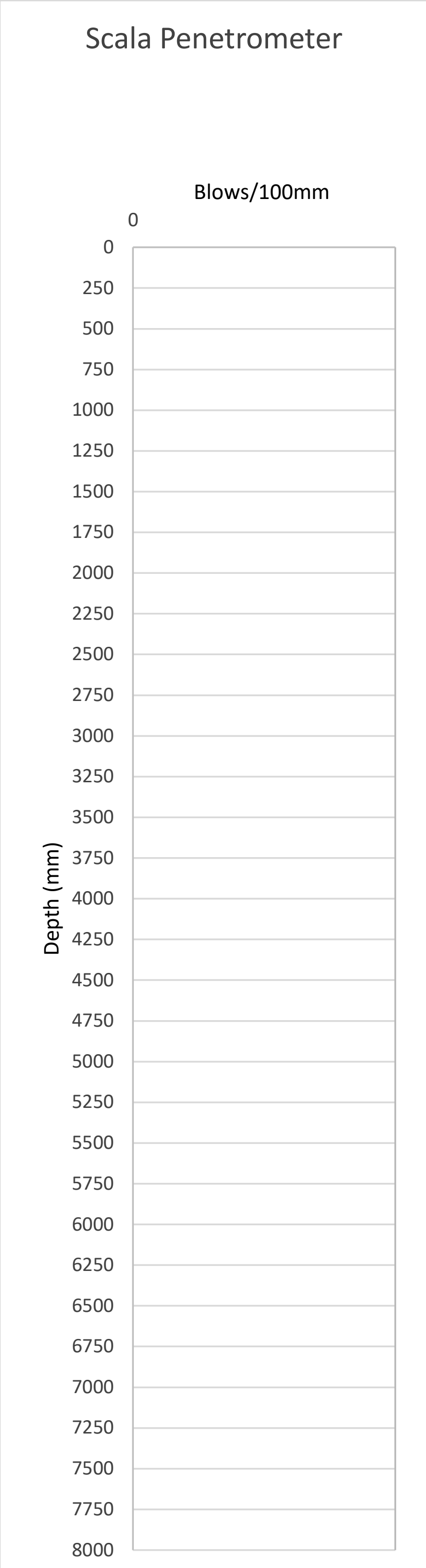
SOIL TESTING RESULTS - Location No. 6

PARKLANDS SUBDIVISION, BELL BLOCK



NOTES: GWT at 2.8 m depth.

Shear Vane Used: Serial Number 2011

Shear Strength (kPa)	Soil Class.	Soil Type
133 / 47	OL	Dark brown organic topsoil, firm and sandy.
125 / 78 133 / 63 204 / 55	CL	Sandy SILT (Taranaki Ash),dark brown, moist and low plasticity.
141 / 39 219 /	CH	Sandy SILT (Taranaki Ash), brown, moist, very firm with high plasticity.
	PT	Organic PEAT. Wet, black in colour with wood debris and traces of sand.
		End Of Hole

Appendix C NIWA HIRDS Rainfall Data

Rainfall intensities (mm/hr) :: RCP6.0 for the period 2081-2100

ARI	AEP	10m	20m	30m	1h	2h	6h	12h	24h
1.58	0.633	68.5	48.5	39.2	26.8	17.7	8.73	5.40	3.28
2	0.500	75.1	53.2	43.0	29.4	19.5	9.58	5.94	3.59
5	0.200	97.9	69.3	56.0	38.3	25.4	12.5	7.75	4.69
10	0.100	115	81.6	66.0	45.1	29.9	14.7	9.13	5.53
20	0.050	134	94.6	76.5	52.3	34.7	17.1	10.6	6.40
30	0.033	145	103	83.1	56.8	37.7	18.6	11.5	6.95
40	0.025	153	109	87.8	60.0	39.8	19.7	12.2	7.36
50	0.020	160	113	91.7	62.6	41.6	20.5	12.7	7.67
60	0.017	166	117	94.8	64.8	43.0	21.2	13.1	7.95
80	0.012	175	124	100	68.3	45.4	22.4	13.9	8.37
100	0.010	182	129	104	71.1	47.2	23.3	14.4	8.72
250	0.004	212	150	121	82.9	55.1	27.2	16.8	10.2

Appendix D Photographs

Photo 1: Typical Taranaki Ash sample.

Photo 2: Typical underlying firm sand sample.

Appendix E Preliminary Stormwater Plan

CATCHMENT CHARACTERISTICS:

RESIDENTIAL LOTS
- CN = 69 (PRE DEVELOPED)
- CN = 75 (POST DEVELOPED)

RESIDENTIAL CARRIAGEWAY
- CN = 69 (PRE DEVELOPED)
- CN = 85 (POST DEVELOPED)

DISCHARGE POINT 1:
RESIDENTIAL LOTS
AREA = 44,843 m²

RESIDENTIAL CARRIAGEWAY
AREA = 13,514 m²

DISCHARGE POINT 2:
RESIDENTIAL LOTS
AREA = 13,556 m²

RESIDENTIAL CARRIAGEWAY
AREA = 7,031 m²

DISCHARGE POINT 3:
RESIDENTIAL LOTS
AREA = 81,931 m²

RESIDENTIAL CARRIAGEWAY
AREA = 15,473 m²

DISCHARGING TO SURROUNDING LAND:
RESIDENTIAL LOTS - NORTH EAST
AREA = 4,804 m²

RESIDENTIAL LOTS - NORTH WEST
AREA = 42,936 m²

RESIDENTIAL LOTS - SOUTH
AREA = 10,475 m²

NOTES:
- ALL POST DEVELOPED LOTS SHALL UTILISE ON-SITE SOAKAGE IN ACCORDANCE WITH NZBC E1.
- CATCHMENT MODELING HAS INCORPORATED STORAGE / SOAKAGE SYSTEMS FOR ALL PROPOSED LOTS.
- A SOAKAGE RATE OF 195 mm/hr HAS BEEN USED.

CATCHMENT PEAK DISCHARGE SUMMARY:

- 20% AEP DESIGN STORM PEAK DISCHARGES:

PRE DEVELOPED
GRASS FARM LAND = 2.992 m³/sec

POST DEVELOPED
RESIDENTIAL DEVELOPMENT = 2.061 m³/sec
(31% REDUCTION IN PEAK FLOW)

- 10% AEP DESIGN STORM PEAK DISCHARGES:

PRE DEVELOPED
GRASS FARM LAND = 3.732 m³/sec

POST DEVELOPED
RESIDENTIAL DEVELOPMENT = 4.303 m³/sec
(13% INCREASE IN PEAK FLOW)

- 1% AEP DESIGN STORM PEAK DISCHARGES:

PRE DEVELOPED
GRASS FARM LAND = 7.100 m³/sec

POST DEVELOPED
RESIDENTIAL DEVELOPMENT = 7.887 m³/sec
(10% INCREASE IN PEAK FLOW)

NOTES:
- PEAK DISCHARGE SUMMARY DOES NOT INCLUDE THE SOUTHERN LOTS DISCHARGING TOWARDS POHUTUKAWA PLACE.

PROPOSED DISCHARGE NORTH WEST LOTS:
- 20% AEP DESIGN STORM PEAK DISCHARGES:

PRE DEVELOPED
GRASS FARM LAND = 0.558 m³/sec

POST DEVELOPED
RESIDENTIAL DEVELOPMENT = 0.337 m³/sec

- 10% AEP DESIGN STORM PEAK DISCHARGES:

PRE DEVELOPED
GRASS FARM LAND = 0.715 m³/sec

POST DEVELOPED
RESIDENTIAL DEVELOPMENT = 0.808 m³/sec

- 1% AEP DESIGN STORM PEAK DISCHARGES:

PRE DEVELOPED
GRASS FARM LAND = 1.361 m³/sec

POST DEVELOPED
RESIDENTIAL DEVELOPMENT = 1.513 m³/sec

PROPOSED DISCHARGE POINT 3:
- 20% AEP DESIGN STORM PEAK DISCHARGES:

PRE DEVELOPED
GRASS FARM LAND = 1.266 m³/sec

POST DEVELOPED
RESIDENTIAL DEVELOPMENT = 1.007 m³/sec

- 10% AEP DESIGN STORM PEAK DISCHARGES:

PRE DEVELOPED
GRASS FARM LAND = 1.622 m³/sec

POST DEVELOPED
RESIDENTIAL DEVELOPMENT = 1.864 m³/sec

- 1% AEP DESIGN STORM PEAK DISCHARGES:

PRE DEVELOPED
GRASS FARM LAND = 3.086 m³/sec

POST DEVELOPED
RESIDENTIAL DEVELOPMENT = 3.408 m³/sec

PROPOSED DISCHARGE POINT 2:
- 20% AEP DESIGN STORM PEAK DISCHARGES:

PRE DEVELOPED
GRASS FARM LAND = 0.268 m³/sec

POST DEVELOPED
RESIDENTIAL DEVELOPMENT = 0.192 m³/sec

- 10% AEP DESIGN STORM PEAK DISCHARGES:

PRE DEVELOPED
GRASS FARM LAND = 0.343 m³/sec

POST DEVELOPED
RESIDENTIAL DEVELOPMENT = 0.403 m³/sec

- 1% AEP DESIGN STORM PEAK DISCHARGES:

PRE DEVELOPED
GRASS FARM LAND = 0.652 m³/sec

POST DEVELOPED
RESIDENTIAL DEVELOPMENT = 0.737 m³/sec

PROPOSED DISCHARGE NORTH EAST LOTS:
- 20% AEP DESIGN STORM PEAK DISCHARGES:

PRE DEVELOPED
GRASS FARM LAND = 0.062 m³/sec

POST DEVELOPED
RESIDENTIAL DEVELOPMENT = 0.057 m³/sec

- 10% AEP DESIGN STORM PEAK DISCHARGES:

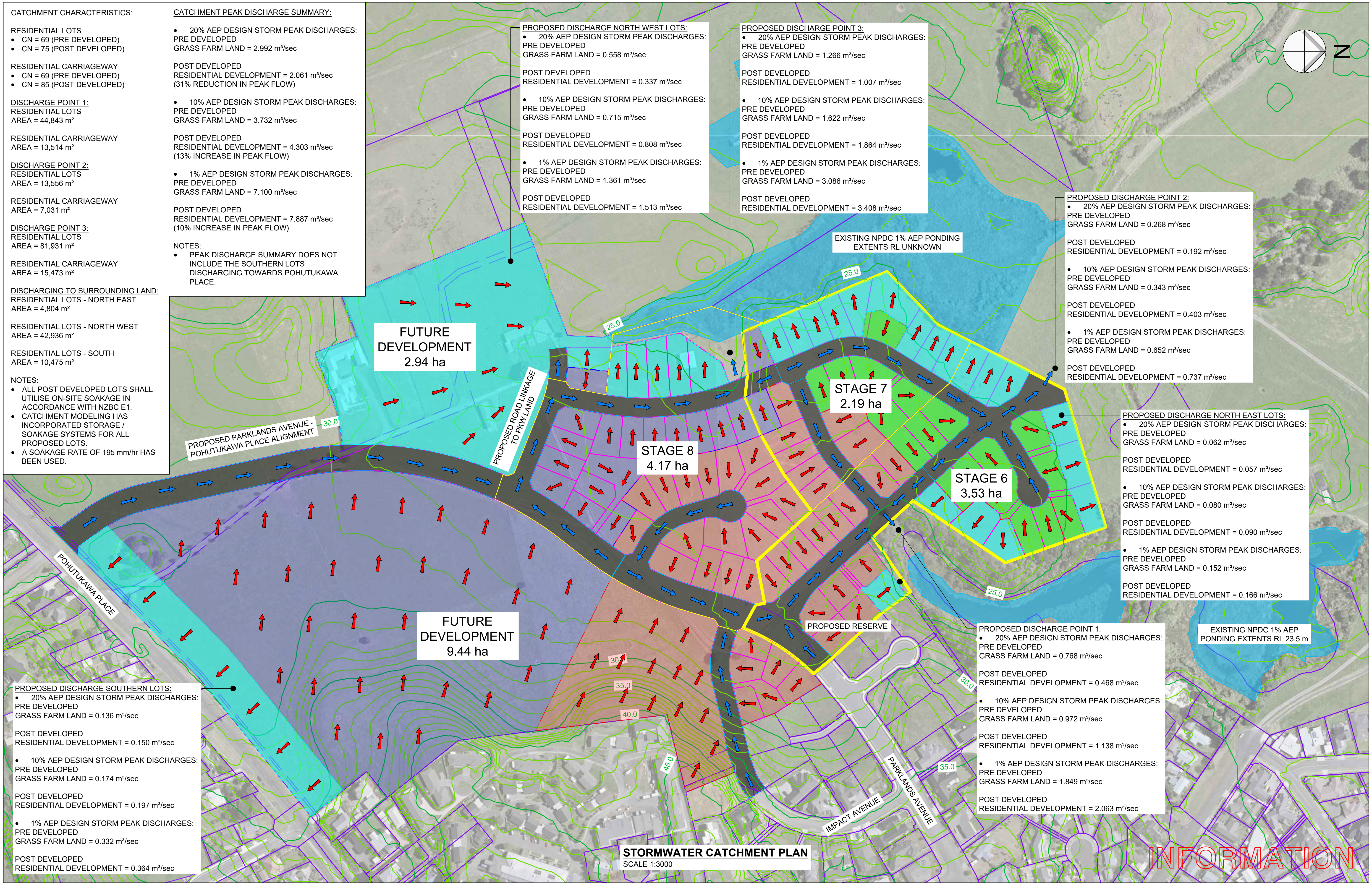
PRE DEVELOPED
GRASS FARM LAND = 0.080 m³/sec

POST DEVELOPED
RESIDENTIAL DEVELOPMENT = 0.090 m³/sec

- 1% AEP DESIGN STORM PEAK DISCHARGES:

PRE DEVELOPED
GRASS FARM LAND = 0.152 m³/sec

POST DEVELOPED
RESIDENTIAL DEVELOPMENT = 0.166 m³/sec



PROPOSED DISCHARGE POINT 1:
- 20% AEP DESIGN STORM PEAK DISCHARGES:

PRE DEVELOPED
GRASS FARM LAND = 0.768 m³/sec

POST DEVELOPED
RESIDENTIAL DEVELOPMENT = 0.468 m³/sec

- 10% AEP DESIGN STORM PEAK DISCHARGES:

PRE DEVELOPED
GRASS FARM LAND = 0.972 m³/sec

POST DEVELOPED
RESIDENTIAL DEVELOPMENT = 1.138 m³/sec

- 1% AEP DESIGN STORM PEAK DISCHARGES:

PRE DEVELOPED
GRASS FARM LAND = 1.849 m³/sec

POST DEVELOPED
RESIDENTIAL DEVELOPMENT = 2.063 m³/sec

PROPOSED DISCHARGE SOUTHERN LOTS:
- 20% AEP DESIGN STORM PEAK DISCHARGES:

PRE DEVELOPED
GRASS FARM LAND = 0.136 m³/sec

POST DEVELOPED
RESIDENTIAL DEVELOPMENT = 0.150 m³/sec

- 10% AEP DESIGN STORM PEAK DISCHARGES:

PRE DEVELOPED
GRASS FARM LAND = 0.174 m³/sec

POST DEVELOPED
RESIDENTIAL DEVELOPMENT = 0.197 m³/sec

- 1% AEP DESIGN STORM PEAK DISCHARGES:

PRE DEVELOPED
GRASS FARM LAND = 0.332 m³/sec

POST DEVELOPED
RESIDENTIAL DEVELOPMENT = 0.364 m³/sec

STORMWATER CATCHMENT PLAN

SCALE 1:3000

INFORMATION

REDJACKET
3 DAVIDSON STREET
NEW PLYMOUTH 4310
Ph. 06 759 0999

DATE	REV	REV RECORD	BY	CHD	VER	APP
31/10/20	B	INFORMATION	LB	CM	-	-

Client: -

Project: SUBDIVISION PARKLANDS AVE BELL BLOCK

Sheet Title: STORMWATER CATCHMENT PLAN

Drawing No.: DWG-100-433

A3

REV.: B

Job No.: 3915

Sheet No.: C1-1

File Name: Z:\JOBS\3501-4000\3901-3950\3917 Parklands Ave Subdivision - Stage 6\Dwg\100-433.dwg - C1-1   Plot Date: 31/10/2020   Plot Time: 10:57