DAVIS OGILVIE
ENGINEERS / SURVEYORS / PLANNERS

GEOTECHNICAL REPORT FOR SUBDIVISION

44214 / LOT 3 DP 360520, STATE HIGHWAY 6 (SH6), WESTPORT / JOHN RAYMOND MCLAUGHLIN

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QUALITY ASSURANCE

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DISCLAIMER

This engineering report has been prepared at the specific instruction of John Raymond Mclaughlin. It addresses geotechnical conditions underlying the property at Lot 3 DP 360520, SH6, Westport. This report provides an assessment of the underlying ground conditions at the site, assesses the natural hazards as required by Section 106 of the Resource Management Act (1991) and suitability of the land for residential subdivision.

Davis Ogilvie did not perform a complete assessment of all possible conditions or circumstances that may exist at the site. Variations may occur between investigatory locations and conditions may exist which were undetectable given the limited investigation of the site and have not been taken into account in the report.

Davis Ogilvie's opinions are based upon information that existed at the time of the production of this document. Assessments made in this report are based on the conditions found onsite and published sources detailing the recommended investigation methodologies described. No warranty is included—either expressed or implied—that the actual conditions will conform to the assessments contained in this report.

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1.0 PURPOSE OF REPORT

Davis Ogilvie & Partners Ltd. (Davis Ogilvie) has been commissioned by John McLaughlin to undertake a geotechnical investigation for a proposed sixteen lot residential subdivision of Lot 3 DP 360520, SH6, Westport. The aim of the investigation was to provide an assessment of the underlying ground conditions at the site, assess the natural hazards as required by Section 106 of the Resource Management Act (1991), assess the suitability of the land for residential subdivision, and to provide preliminary geotechnical recommendations for the development.

2.0 SITE DESCRIPTION

The proposed subdivision of Lot 3 DP 360520 containing the fifteen proposed new lots (herein referred to as "the site") is located in the Buller District, West Coast, New Zealand, on the eastern side of State Highway 6 (SH6) approximately 8.0 km south of central Westport. The total area of Lot 3 DP 360520 is 89 ha, but this report concerns the area of the proposed subdivision in the 25 ha area between SH6 in the west and a terrace in the east. What is hereafter referred to as "the site" is just this area. The southwestern corner of the site is adjacent to the intersection of SH6 and Wilsons Lead Road. The site is approximately 3.5 km southwest of the Buller River. An aerial photograph and geotechnical site testing locations are shown Figure 1, and a concept plan for the subdivision showing proposed lot boundaries provided by Davis Ogilvie is shown in Figure 2.

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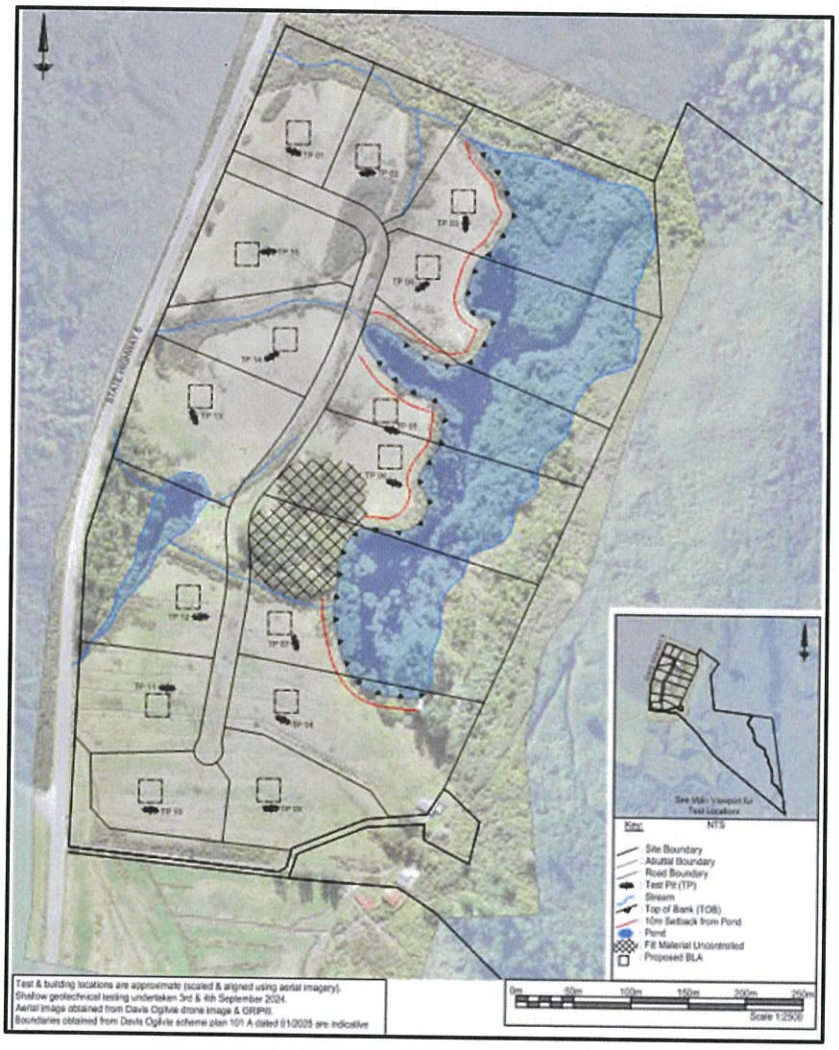

Figure 1: Davis Ogilvie Geotechnical Site Plan showing test pit locations (black rectangles) and water bodies (blue). Proposed lots indicated by black lines. Background image source: Davis Ogilvie - August 2024.

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Figure 2: Excerpt from Davis Ogilvie Concept Plan indicating lot boundaries within the proposed subdivision (yellow). Source: Davis Ogilvie 44214 DWG 101A.

The site topography is generally flat with moderate undulating relief towards the north and a gentle lowering of relief westward across the entire site. Several shallow drainage features directing water away from the site in a westward direction are visible. A large water body is present along the eastern edge of the site, towards the toe of the adjacent slope, cross-cutting proposed Lots 3 – 8. Anecdotal information from the client indicates that the lake is dominantly anthropogenic in origin (related to gravel washing / dredging), with steep cuts along the pond edges.

The site is located at the toe of a terrace immediately east of the site. The terrace face is approximately 80 m high with steep vegetated slopes (approximately 46° slope angle) and the base of which corresponds approximately to the eastern site boundary. Water level in the ponds appears to range from approximately 0.5 – 4 m below EGL compared to the rest of the site, depending on location. A photo of the site is provided below in Figure 3.

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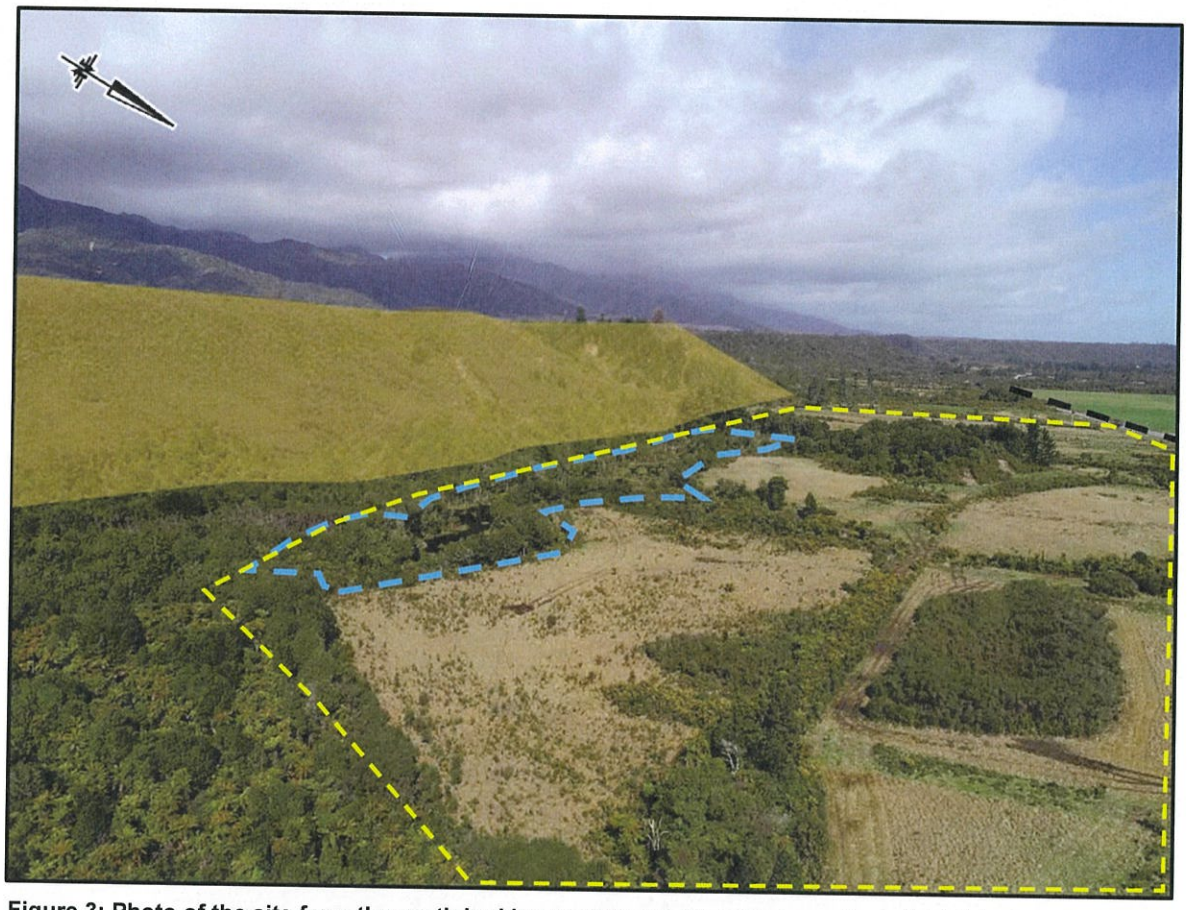

Figure 3: Photo of the site from the north looking southward. State highway 6 on right (black dashed line) and terrace on left (shaded orange). Anthropogenic ponds are visible centrally (blue dashed line). Yellow dashed line indicates approximate area for subdivision. Photo source: Davis Ogilvie, September 2024.

Man-made drainage channels are evident across the southern portion of the site (proposed Lots 7 – 12), oriented west-east to allow for surface water drainage in a westward direction. The site has been predominantly cleared of vegetation and utilised for farming purposes. A currently unsealed access road crosscuts the central part of proposed Lot 13. A large mound of uncontrolled fill material resides across the intersection of proposed Lots 6 and 7, on the western end. This fill pile has become overgrown with large trees and shrubs.

3.0 REVIEW OF PUBLISHED INFORMATION

A review of published information has been undertaken for the site and surrounding area. A summary is presented in the following sections.

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3.1 Site History

Geologically, the Pleistocene age coastal terrace surfaces towards the east represent shorelines formed at the ends of periods of rapid interglacial rise of sea level. Terraces here, paired with tectonic uplift, have formed as the shoreline prograded westward approximately 0.3 million years ago, with coarse river deposits commonly found on top of these terraces due to more recent erosional processes.

The earliest aerial photographs show the site as predominantly farmed / cleared land from at least 1967[1]. The existing driveway and buildings for the property immediately south (along the southern boundary of the site) are evident. The existing entrance on the site, accessed centrally off the eastern side of SH6 and north of the driveway, is also visible since at least 1967.

There appears to be some form of earthworks located centrally on the site, with isolated areas of exposed gravel soils evident between 1967 and 1978. There is also evidence of potential dredge related works (stripped vegetation, exposed soils / fill, and settling ponds) on the terrace within 1 km northeast of the site from at least 1974. Dredging works here appear to be decommissioned by 1985. Ponds on the site itself are also assumed to be anthropogenic in origin, and related to early dredging works, although the exact date of the dredging (and areas of associated fill placement) has not been determined. It is assumed that dredging, and the associated mound of uncontrolled fill, was likely pre-1960s.

The dwelling on the property immediately north of the site was constructed somewhere between 1978 and 1985. Ponds along the toe of the slope of the nearby terrace on site are visible since at least 1985. Some parts of the site towards the south appear to have never been cleared, perhaps due to swampy conditions. The pond on site appears to be anthropogenic in origin, assumed related to historic mining activities.

No major changes to the site are visible between 1985 and present day.

3.2 Published Geology and Hydrogeology

The published geology[2] of the site is identified as primarily Middle Pleistocene ocean beach deposits, described as '[iron oxide] *cemented marine sand and gravel (Q9b)'*. East of the site the terrace geology is identified as Early Pleistocene river deposits, described as *'weathered and locally cemented river gravel and sand (eQa)'*.

A review of the GNS National Water Table Map[3] indicates groundwater level is between 0.0 – 2.5 m below ground level.

[1] Retrolens Historic Image Resource, http://retrolens.nz/
[2] Nathan, S., Rattenbury, M.S., Suggate, R.P. (compilers) 2002. Geology of the Greymouth area. Institute of Geological & Nuclear Sciences 1:250,000 geological map 12. Lower Hutt, New Zealand. GNS Science Limited.
[3] https://rogierwesterhoff.users.earthengine.app/view/nzwatertable

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3.3 Seismicity

There are no active faults on site, however, the site is situated in a region of complex geological structures and active faulting. Structures of note include the Awakiri Syncline, Lower Buller Fault, and Alpine Fault, approximately 1.5 km south, 3.5 km southeast, and 75 km southeast of the site respectively. However, none of these features warrant near-fault factors in excess of 1.0 for structural design as mandated in NZS1170.5:2004[4].

The Alpine Fault has a recurrence interval in the order of 291 ± 23 years (Cochran et al., 2017[5]). The last major fault rupture occurred in 1717, and recent research suggests the probability of a rupture of the central section of the Alpine Fault occurring within the next 50 years to be in the order of 75%[6].

The Buller District Council (BDC) 2006 Alpine Fault Earthquake Scenario & Lifelines Study provides information on the likely impact of an Alpine Fault earthquake on the district[7]. The modelled Modified Mercalli (MM) intensities indicate MM VII intensity shaking in the vicinity of the site for an Alpine Fault rupture, as well as Peak Ground Accelerations of 0.1 – 0.2 g. The Ministry of Business Innovation & Employment (MBIE, November 2021) Peak Ground Accelerations (PGAs) and Earthquake Magnitude (M_w) for Geotechnical Assessment[8], for the region, are presented in Table 1.

Table 1: Design Earthquake PGAs

Seismic Event	Return Period	M_w	PGA (g)
Ultimate Limit State (ULS)	1 in 500	6.0	0.55
Serviceability Limit State (SLS)	1 in 25	6.0	0.14

3.4 Landslide

According the GNS Landslide Database[9] there are no historic large landslides located within close proximity to the site, furthermore, the Te Tai o Poutini Plan (TTPP)[10] indicates the site is not located in a 'Land Instability' hazard and risk area.

[4] https://data.gns.cri.nz/af/.
[5] Cochran et al. 2017. A plate boundary earthquake record from a wetland adjacent to the Alpine fault in New Zealand refines hazard estimates. Earth and Planetary Science Letters. Volume 464, 15 April 2017, Pages 175-188.
[6] Jamie D. Howarth, Nicolas C. Barth, Sean J. Fitzsimons, Keith Richards-Dinger, Kate J. Clark, Glenn P. Biasi, Ursula A. Cochran, Robert M. Langridge, Kelvin R. Berryman, Rupert Sutherland. Spatiotemporal clustering of great earthquakes on a transform fault controlled by geometry. Nature Geoscience, 2021.
[7] Buller District Council (June 2006). Buller District Council Lifelines Plan, Alpine Fault Earthquake Scenario & Lifelines Vulnerability Assessment.
[8] New Zealand Geotechnical Society (NZGS) and Ministry of Business Innovation & Employment (MBIE) Earthquake Geotechnical Engineering Practice in New Zealand Rev 1 Issue Date: November 2021
[9] https://data.gns.cri.nz/landslides/wms.html
[10] https://westcoast.isoplan.co.nz/eplan/property/18501/0/76?_t=property

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3.5 Liquefaction Potential

According to a November 2021 liquefaction assessment report prepared by Beca Limited (BECA)[11], the site is mapped in an area where *"liquefaction unlikely."*

The BDC Lifelines Study indicates the site likely falls within 'Ground Shaking Zone 2'. Ground Shaking Zone 2 is described where a Site Subsoil Class B or C (NZS 1170.5:2004) is inferred depending on the depth of sediment, and where ground settlement is inferred as *"none."*

3.6 Flood Hazard

According to the TTPP[12], the site is not within a 'Flood Hazard Susceptibility' hazard and risk area. The closest inundation / flooding indicated on TTPP, beyond the investigation area, has been mapped within the Buller River, > 3.0 km northeast and approximately 60 m lower in elevation compared to the site.

Several ponds are evident across the site, particularly near the toe of the adjacent terrace. Overland flow from excess stormwater accumulation during periods of heavy rainfall may pose a flooding risk to the site if stormwater is not adequately controlled.

4.0 GEOTECHNICAL INVESTIGATION

A shallow geotechnical investigation and site walkover was undertaken by Davis Ogilvie on 3rd and 4th September 2024 and included fifteen (15) machine-excavated test pits (TPs) to a maximum depth of 4.5 m below Existing Ground Level (EGL), and fifteen (15) Dynamic Cone Penetrometer (DCP) tests to a maximum depth of 2.0 m below EGL. Test pits were terminated either on dense soils, maximum reach of the excavator, or below the groundwater table in saturated soils where further excavation was not possible due to sidewall collapse. Test locations are shown on the geotechnical site plan (DWG G01A) in **Appendix B** and test logs in **Appendix C**.

[11] West Coast Regional Liquefaction Assessment. Prepared for West Coast Regional Council by Beca Limited. Dated 1 November 2021.
[12] https://westcoast.isoplan.co.nz/eplan/property/18501/0/76?_t=property

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4.1 Shallow Geotechnical Investigation

The soil profile indicated by the shallow testing is summarised in Table 2. The surficial soil conditions generally consisted of topsoil and / or organic silt from the ground surface to depths of 0.1 to 0.8 m below EGL. Some variability was encountered below these surficial soils across the site (shown in Figures 4 and 5). Generally, dense granular soils (typically cemented) were encountered at shallow depths ranging from 0.1 to 1.8 m below ground level. These are interpreted to be the cemented marine sands and gravels indicated by the published geology. Some areas encountered softer, more organic units varying in thickness from 0.3 to 1.2 m, generally overlying denser soils. In some places cemented soils were underlain by dense sandy / granular soils to a maximum depth of 4.5 m. There is evidence the upper layers of soil towards the south have been flipped in order to promote drainage, accompanying the incised drainage channels (show on Figure 4).

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Table 2: Indicative Site Soil Profile from Shallow Investigation

Description	DCP (blows / 100 mm)	Relative Density / Consistency	Depth below EGL (m)													
			TP + DCP 01	TP + DCP 02	TP + DCP 03	TP + DCP 04	TP + DCP 05	TP + DCP 06	TP + DCP 07	TP + DCP 08	TP + DCP 09	TP + DCP 10	TP + DCP 11	TP + DCP 12	TP + DCP 13	TP + DCP 14	TP + DCP 15
TOPSOIL	0 – 17+	**	0.0 – 0.1	0.0 – 0.4	0.0 – 0.8	0.0 – 0.1	0.0 – 0.2	0.0 – 0.2	0.0 – 0.5	0.0 – 0.3	0.0 – 0.2	0.0 – 0.3	0.0 – 0.6	0.0 – 0.2	0.0 – 0.1	0.0 – 0.5	0.0 – 0.2
SAND (silty)	1 – 17+	Loose to Very Dense									0.2 – 0.9[F]	0.3 – 0.5[F]		0.2 – 0.6	0.1 – 0.2	0.5 – 1.1	
GRAVEL (silty, sandy, or cemented)	1 – 17+	Loose to Very Dense								0.3 – 1.7[T F]		0.5 – 1.5[F]		0.6 – 1.6[F]			
Organic SILT / SAND	1 – 4	**									0.9 – 1.1[F]	1.5 – 1.9	0.6 – 1.8[F]				
GRAVEL (silty, sandy, or cemented)	1 – 17+	Loose to Very Dense		0.4 – 1.3	0.8 – 0.9[T]				0.5 – 1.1[T F]		1.1 – 2.2[T F]	1.9 – 3.2[T]	1.8 – 2.4[T]			0.5 – 2.1	
SAND	5 – 17+	Medium Dense to Very Dense		1.3 – 3.0[T]		0.1 – 0.5	0.2 – 0.6	0.2 – 0.5						1.6 – 4.5[T]		2.1 – 3.0[T]	
SAND (Q9b, fine, cemented)	10 – 17+	Dense to Very Dense	0.1 – 0.5[T]			0.5 – 3.5[T]	0.6 – 1.8[T]	0.5 – 1.9[T]							0.2 – 1.8[T]		0.2 – 1.7[T]
Water Table/Seepage[S] (m)			-	2.4	0.2[S]	2.9	0.5[S]	0.5[S]	-	0.6[S]	1.0[S]	3.0	0.4[S]	4.2	-	-	-

Notes: Depth rounded to nearest 0.1 m.
[T] = total depth/effective refusal
[F] = potentially flipped and / or disturbed soil
** Relative density not assigned to topsoil, organic soils, or fill.

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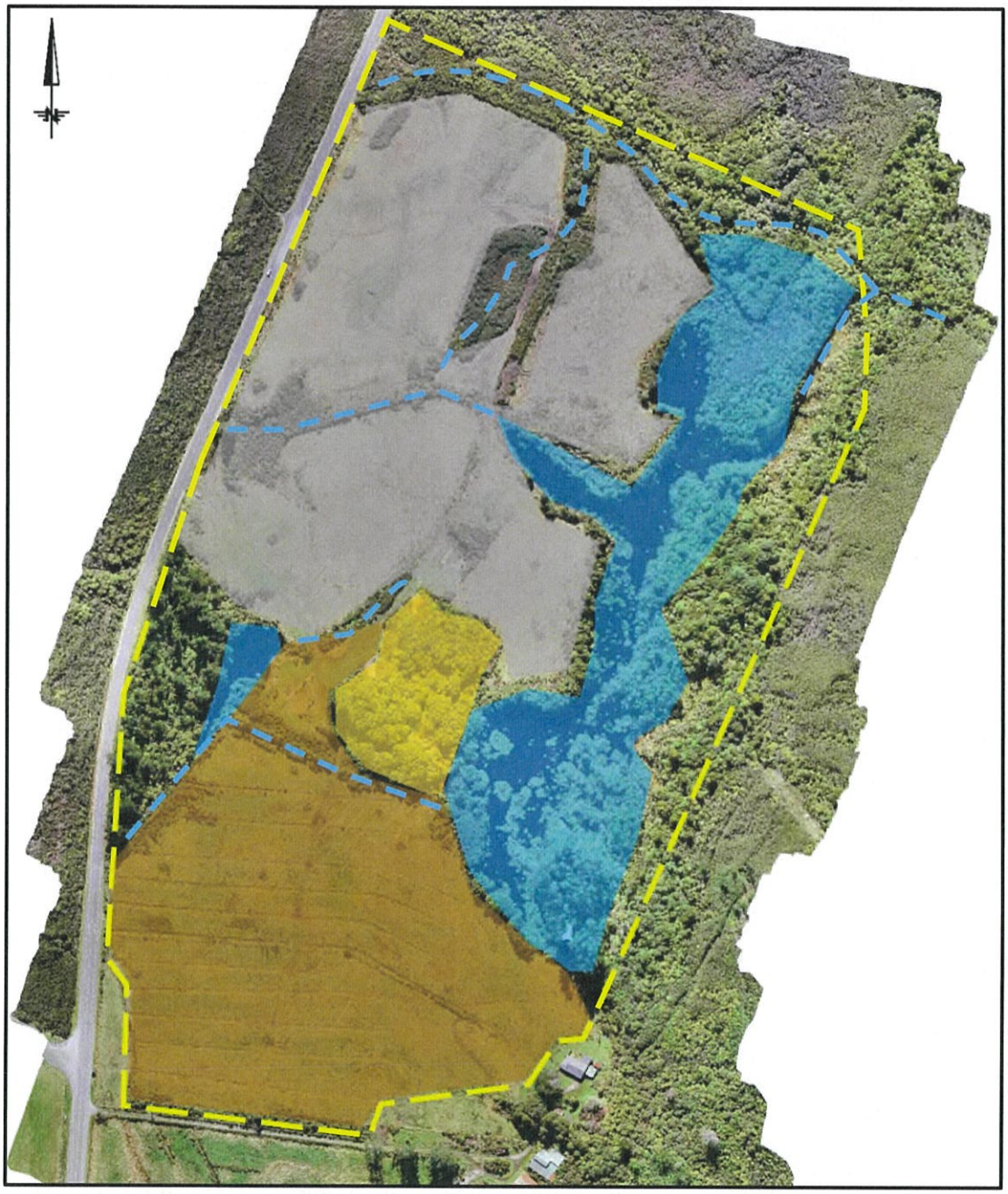

Figure 4: Image showing indicative site boundary (yellow dotted line). Blue shaded areas indicate standing groundwater (ponds), and blue dashed lines indicate natural drainage channels (creeks). Yellow shaded area indicates old gravelly fill stockpile. Areas of shallow cemented sand indicated in grey, and more recent deposits and / or "flipped" soil in brown. Drone imagery provided by Davis Ogilvie September 2024.

Photos of the soil profiles encountered on site are presented in Figure 5.

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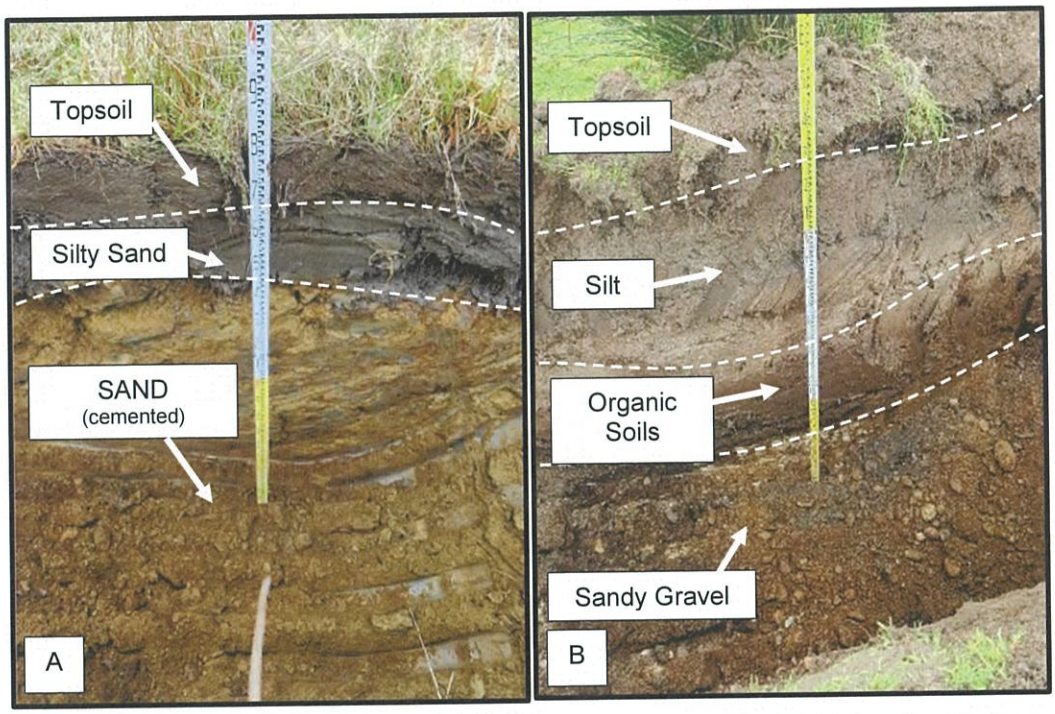

Figure 5: Example of shallow soil profiles encountered on site. Left TP 05 showing shallow granular and cemented soils (A), and right TP 11 showing shallow soft and organic soils with deeper gravelly soils (B). Photos taken September 2024.

Standing groundwater was encountered during the investigation and was generally recorded at depths ranging from 2.4 m to 4.2 m below EGL depending on location, with shallow water seepage from between 0.2 – 0.6 m below EGL. The presence of cemented soils and iron pan layers encountered in several areas across the site has likely led to perched (shallower) groundwater seepage / surface water ponding conditions developing on site following periods of heavy and / or prolonged rainfall, depending on location.

5.0 NATURAL HAZARD ASSESSMENT

Section 106 of the Resource Management Act (RMA) requires an assessment of the potential for material damage to land from natural hazards. These aspects are addressed in the following sections based on published information and our observations on site.

5.1 Seismic

As presented in Section 3.0 of this report, seismic activity presents a significant hazard to the region. There are no mapped active faults within the site. Therefore, the risk of the proposed development being affected by fault rupture is considered low.

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The proposed residential lots are however considered at risk from damage due to earthquake-induced ground shaking. This is expected to be addressed by appropriate foundation design in accordance with NZS 1170.5:2004 and applying an appropriate site Subsoil Class for Importance Level 2 (IL2) structures (refer to Section 7.1).

5.2 Erosion, Slippage, and Falling Debris

The site topography can be described as generally level to gently undulating, with steep slopes surrounding the anthropogenic ponds and along the terrace face. No obvious evidence of significant erosion was seen along the terrace slope; however, some minor erosion features are visible along the terrace face and erosion / slumping has been documented within proximity to the ponds on site. Slope instability surrounding the terrace face is unlikely to affect the proposed build areas due to their distance and the anthropogenic ponds acting as a buffer / catch area for any potential source runout along the terrace face.

The site is located in an area of high rainfall and stormwater and surface runoff is expected to lead to scouring unless diverted away from any un-vegetated slopes and building footprints. A cutoff drain or similar flow path shall be maintained up slope of the Building Location Areas (BLAs) to divert water from them. In addition, hardstand areas formed as part of the development will require appropriate drainage and stormwater control.

Davis Ogilvie considers the present location of BLAs (as indicated in **Appendix B**) are at low risk of erosion, slippage, and falling debris. A preliminary setback of 10 m is recommended from the slopes associated with the ponds on site. Appropriate setbacks should be confirmed on site for each lot containing ponds with steep banks at building consent stage. Vegetation should be also retained around the ponds where possible reduce potential for erosion. Earthworks and drainage must be carefully designed and managed in this regard, both during construction and over the longer term.

5.3 Inundation

Davis Ogilvie has not carried out a detailed assessment of the site's flood potential, however, several localised areas of surface water ponding were observed onsite during the investigation, including ponds, creeks and surface drainage channels. Much of the site also had water collecting at the surface, having likely developed in response to the heavy rainfall that preceded our site visit. The presence of shallow dense / cemented soils and iron pan, and local topographically low areas, can impede surface water infiltration and promote areas of standing water.

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In its current state, and based on published information, Davis Ogilvie are of the opinion that the risk of significant inundation to the proposed BLAs is low, however overland flow from stormwater runoff is likely to occur during periods of heavy rainfall. Stormwater infrastructure and earthworks should be designed and maintained to promote flow towards existing natural drainage channels and future stormwater networks (if applicable).

5.4 Liquefaction Potential and Subsidence

The soil test results indicate that in some areas of the site there is a risk of static settlement of buildings and infrastructure due to the presence of shallow layers of potentially soft and / or organic-rich compressible soils; however, the proposed BLAs are located within areas underlain by deeper, generally dense, granular soils, with groundwater typically recorded at depths greater than 2.4 m.

According to BECA 2021, liquefaction-induced subsidence is unlikely.

Following the site investigation, Davis Ogilvie considers the risk of liquefaction at the site to be low, however, future building location areas must undergo site specific shallow geotechnical investigation at building consent stage to confirm ground conditions and appropriate foundations.

5.5 Section 106 Conclusion

Davis Ogilvie is of the opinion that the site is considered geotechnically suitable for subdivision and residential development under Section 106 of the RMA provided recommendations outlined in this report are followed. Preliminary development recommendations are provided below, and a Statement of Professional Opinion on 'The Suitability of Land for Subdivision' is presented in **Appendix D**.

6.0 PRELIMINARY GEOTECHNICAL RECOMMENDATIONS

The following section provides geotechnical design considerations, parameters for design and general foundation recommendations for future dwellings within the proposed BLAs. Site-specific geotechnical investigation by a suitably qualified Geo-professional is required at building consent stage, following confirmation of building site locations.

6.1 Design Criteria

A site Subsoil Class of C ('shallow soil sites') in accordance with NZS 1170.5 may be assumed for the recommended BLAs.

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6.2 Geotechnical Ultimate Bearing Capacity

'Good Ground', as defined by NZS 3604:2011 was not encountered on the site due to the presence of soft, organic soils, and historic fill material. Davis Ogilvie notes that some areas of shallow cemented soils may be suitable for shallow foundations, however this will be subject to Lot-Specific investigation and confirmation of geotechnical ultimate bearing capacity at building consent stage.

6.3 Preliminary Foundation Recommendations

A timber floor option incorporating piles founded in the dense granular soils from between 0.3 – 1.8 m below EGL depending on location is considered a suitable foundation solution. It is noted that the presence of cemented granular soil/iron pan within the shallow soils on some areas of the site may limit the effectiveness of driven piles, therefore bored or cast-in-situ methods may need to be considered. Sufficient embedment into the dense soils will be required to provide lateral support.

Alternatively, consideration could be given to a reinforced concrete slab foundation system (rib-raft or waffle slab) provided all surface topsoil and vegetation as well as any unsuitable or soft material is removed from under the building footprint and replaced with engineered fill material. A subgrade GUBC of 300 kPa in the underlying granular soils (varying at depth between 0.3 to 1.8 m below EGL depending on location) is recommended for preliminary design. A suitably qualified Geo-professional should be contacted to confirm sufficient removal of unsuitable material, and to supervise fill placement is in accordance with NZS 4431:2022 "*Engineered fill construction for lightweight structures*". Furthermore, an independent accredited contractor must be engaged to undertake nuclear densometer (ND) tests to confirm fill compaction.

During construction, all pile excavations and / or stripped areas below proposed concrete slab foundations or engineered fill should be inspected and approved by a Geo-professional to confirm that the ground conditions and bearing capacity are consistent with those described in this report or are otherwise suitable for residential foundations.

6.4 Test Pits

Test pits have been sited outside of potential building location areas based on the proposed scheme plan available at the time of our investigation. Should test pits be encountered within the development footprint, they should be undercut and backfilled with site concrete or engineered fill in accordance with NZS 4431:2022 *"Engineered fill construction for lightweight structures."*

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7.0 CIVIL CONSTRUCTION CONSIDERATIONS

7.1 Earthworks

The following geotechnical recommendations are provided for earthworks at the site:

- Dewatering and earthworks erosion and sediment controls should comply with NZS 4404:2010 and resource consent condition and must be appropriately designed and implemented prior to any earthworks on the site.
- No permanent cuts and fill shall be made steeper than 27° (2H:1V) except with specific approval by a suitably qualified Geo-professional familiar with the site.
- All engineered fills shall be placed in accordance with NZS 4431:2022 with adequate stripping, benching, and drainage under the direction of the Geo-professional.
- Prior to the use of any material as fill (either site-won or imported), bulk samples should be collected and supplied to an IANZ accredited geotechnical laboratory to assess the particle size distribution, compaction properties and maximum dry density. This information should be reviewed by a Geo-professional to confirm the suitability of this material for use as fill. Field compaction trials and additional laboratory analysis may also be required.
- Test pits for the geotechnical investigation were sited to minimise impact to subsequent development where possible to do so. However, if test pits are located within the zone of influence of proposed hardstanding areas, engineered fills, services or building foundations, they should be undercut and backfilled with site concrete or engineered fill in accordance with NZS 4431.

7.2 Pavement and Roading

Additional geotechnical investigation and detailed design of the roading across the site is recommended, especially at creek crossings. All roading design and construction shall be carried out in accordance with NZS 4404:2010, the New Zealand Building Code (E1 – surface water), and resource consent conditions.

8.0 ONSITE DISPOSAL OF STORMWATER AND HOUSEHOLD EFFLUENT

The Regional Land and Water Plan (RLWP)[13] provides a framework for the integrated and sustainable management of the West Coast's natural and physical resources as they apply in the context of land and water. Onsite disposal of stormwater and household effluent in rural zoned properties is covered by the Plan and contains permitted activity rules for activities that have no more than minor adverse effects on the environment. For other activities, resource consent is required.

[13] The West Coast Regional Council, May 2014. Regional Land and Water Plan.

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Rules in the Plan which must be adhered to for development of the site include:

- Rule 63: Discharge of stormwater from reticulated systems.
- Rule 71: Discharge of any contaminant, or water to water, not complying with Rules 63 to 70.
- Rule 79: Onsite discharge of sewage effluent.
- Rule 81: Discharge of stormwater runoff.
- Rule 91: Discharge to land discretionary activity Rule.

This section is provided for information only from a geotechnical perspective and we suggest these matters are discussed further with a suitably experienced and qualified civil engineer.

8.1 Stormwater Management

Stormwater needs to be adequately controlled on the site to prevent localised erosion and inundation.

It is recommended that a suitably sized and located retention tank and / or drainage channels are incorporated into the design at building consent stage to manage runoff from hardstand areas. The most suitable means of effective disposal of stormwater, from any proposed impervious surfaces, is to discharge runoff into existing onsite surface water channels, however, the presence of property boundaries and SH6 may preclude this method for the site.

The construction of a soakage system will require onsite soakage testing and engineer design. It is noted that static groundwater is generally deeper than 2.4 m and the granular soils encountered will likely have a high permeability and be suitable for stormwater infiltration provided a conduit can be established through any cemented layers.

Any culverts and swales will need to be sized in accordance with the New Zealand Building Code and relevant New Zealand standards, while stormwater swales and pipe networks will require maintenance to ensure the design capacity is maintained.

Appropriate erosion and sediment control must be in place prior to commencement of earthworks to prevent adverse effects on adjacent properties, including sediment transportation and ponding offsite, particularly towards nearby waterways.

Careful consideration and design of site drainage will be required, and drainage systems must be designed by a suitably qualified and experienced engineer, and include regard for the following:

- Maintenance of natural drainage flow paths where possible.
- Direction of stormwater toward existing natural flow paths.
- Installation of cut-off drains above building platforms to redirect overland flow and sub-surface seepage toward natural drainage flow paths.

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9.0 SUITABILITY FOR SUBDIVISION

Provided the recommendations presented in this report are followed, it is determined that the site is considered to be suitable under Section 106 of the RMA for the proposed subdivision. It is considered that the risk of the hazards discussed in this report can be mitigated or managed to an acceptable level.

A statement of professional opinion on the suitability of land for subdivision is attached as **Appendix D**. The granting of subdivision consent is supported, subject to the following conditions:

1. Lot specific Geotechnical investigation shall be required for each lot at Building Consent stage following confirmation of building site locations within the proposed scheme plan.
2. A preliminary offset of 10 m from the pond banks shall be imposed unless SED can demonstrate mitigation of any edge effects.
3. All lot specific earthworks must be undertaken in accordance with NZS 4431:2022 and shall be undertaken under the supervision of a suitably qualified Geo-professional with experience in land development.
4. Backfilled test pits within foundation excavations, should they be encountered, will require remediation by excavation and replacement with site concrete or engineered fill in accordance NZS 4431:2022.
5. Minimum floor levels as per New Zealand Building Code E1 must be confirmed with Buller District Council at building consent stage.

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APPENDIX A

Scheme Plan

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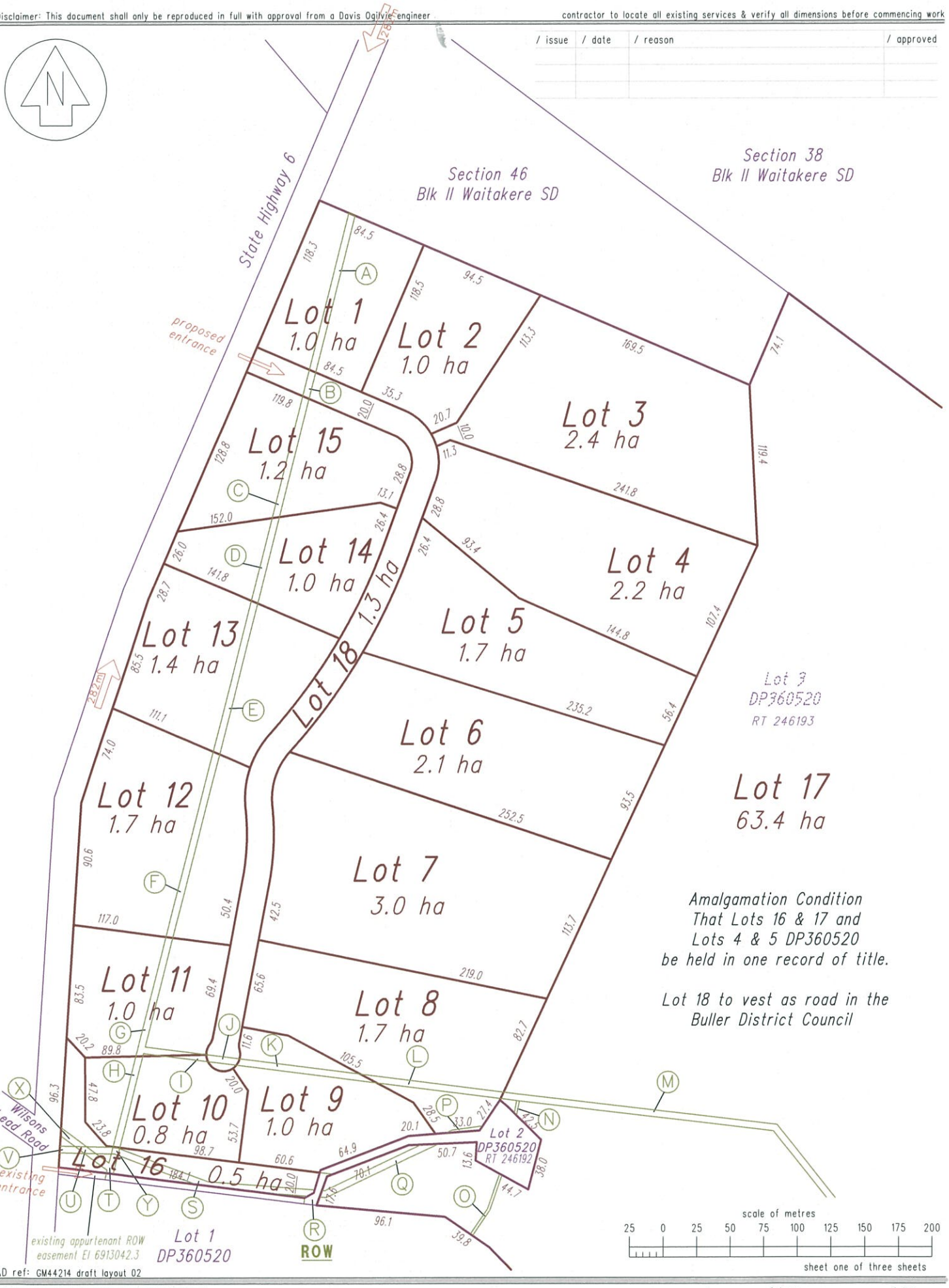
contractor to locate all existing services & verify all dimensions before commencing work

/ issue	/ date	/ reason	/ approved



sheet one of three sheets

CAD ref: GM44214 draft layout 02

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ENGINEERS / SURVEYORS / PLANNERS

Davis Ogilvie & Partners Ltd - Ph. 0800 999 333

Proposed Subdivision of Lot 3 DP360520

/ drawn	/ dwg
Mike Robbins	101
/ scale@ A3 / date / file	/ issue
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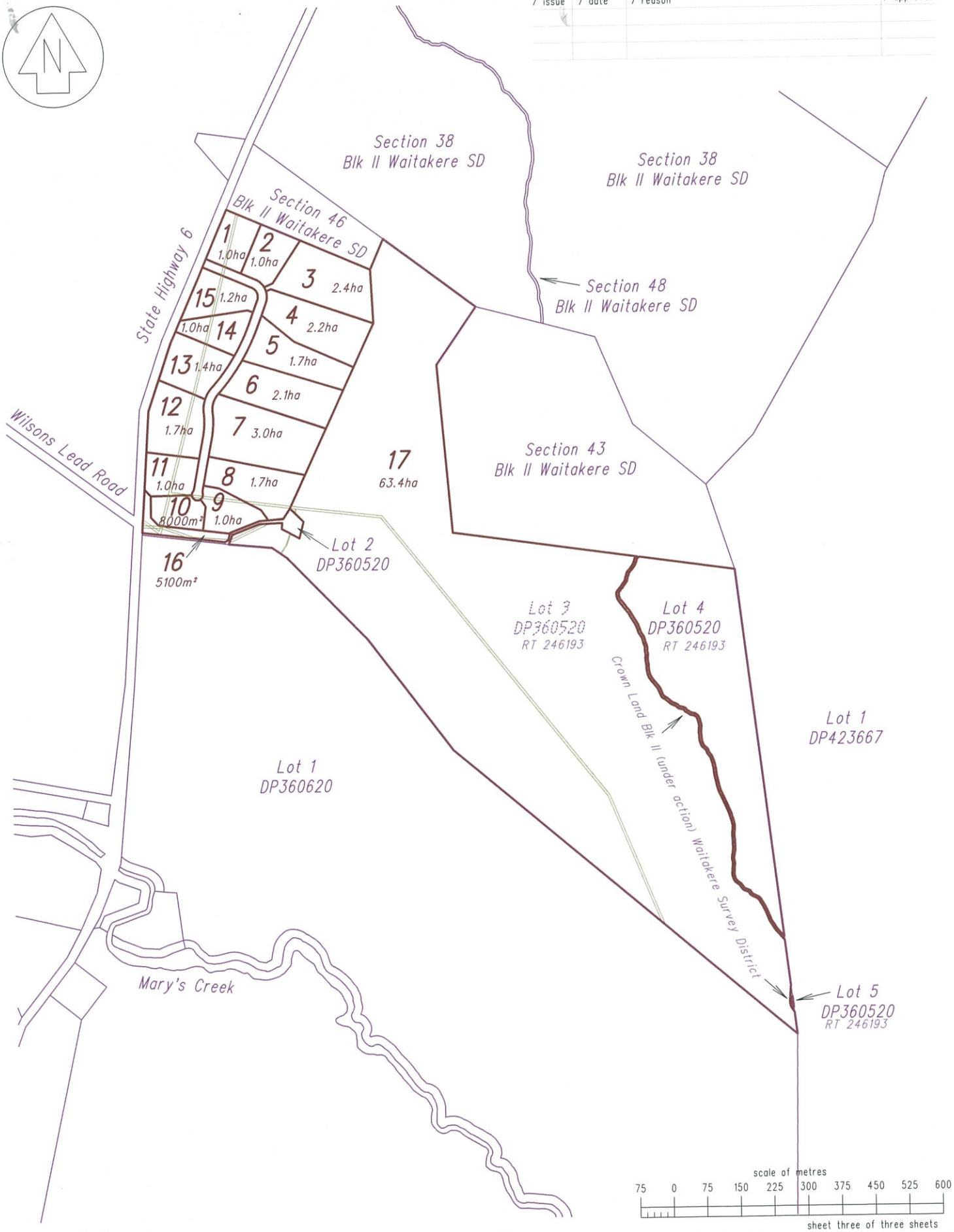

sheet three of three sheets

CAD ref: GM44214 RCA Plan_02 7500

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Proposed subdivision of Lot 3 DP360520

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Mike Robbins	101
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APPENDIX B

Geotechnical Site Plan (G01A)

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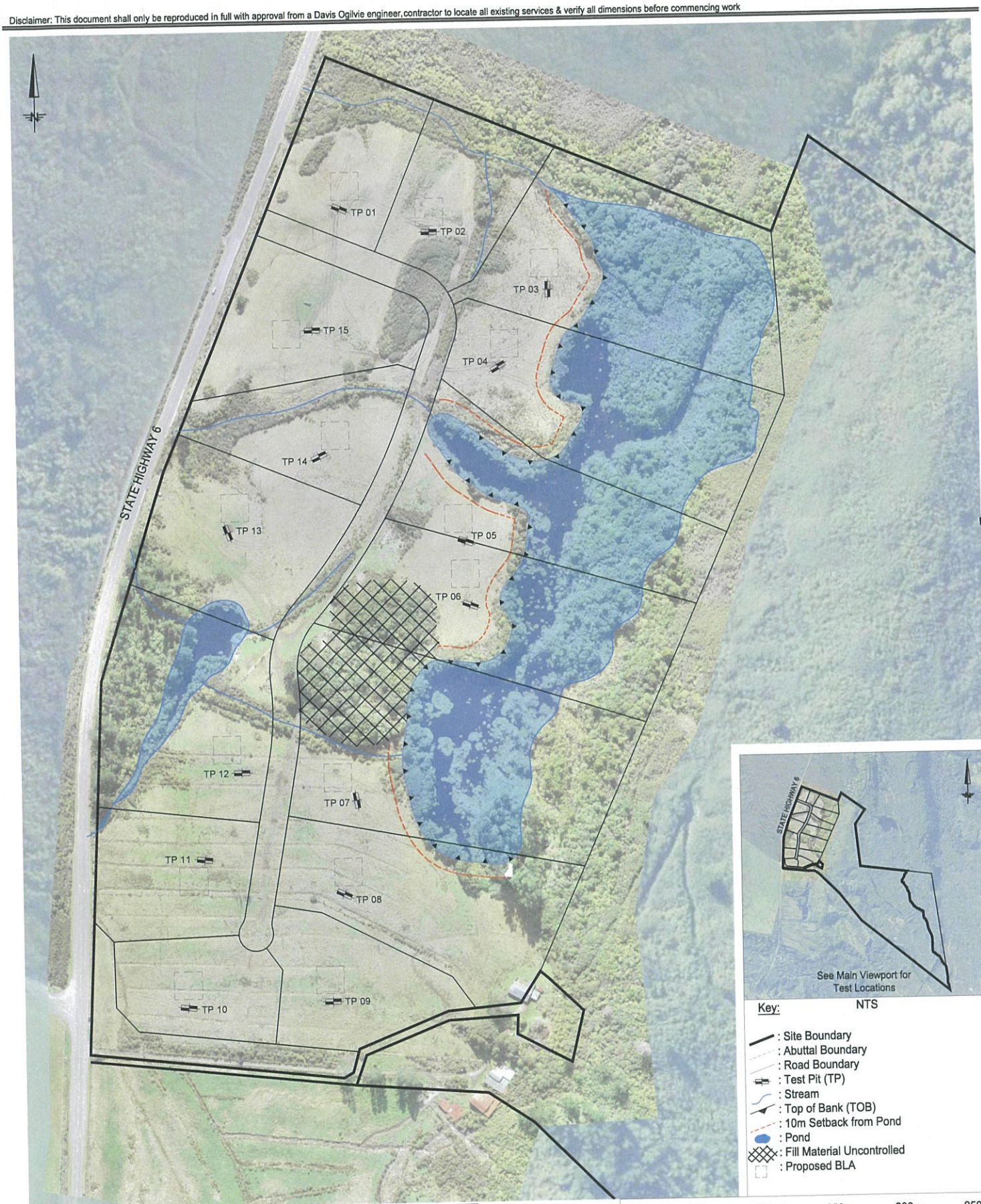

Key: NTS

- : Site Boundary
- : Abuttal Boundary
- : Road Boundary
- : Test Pit (TP)
- : Stream
- : Top of Bank (TOB)
- : 10m Setback from Pond
- : Pond
- : Fill Material Uncontrolled
- : Proposed BLA

0m 50m 100m 150m 200m 250m

Scale 1:2500

Test & building locations are approximate (scaled & aligned using aerial imagery).
Shallow geotechnical testing undertaken 3rd & 4th September 2024.
Aerial image obtained from Davis Ogilvie drone image & GRIP®.
Boundaries obtained from Davis Ogilvie scheme plan 101 A dated 01/2025 are indicative only.

CAD ref: 240911.44214.GeotechnicalSitePlan.dwg

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Geotechnical Site Plan
Lot 3 DP 360520
State Highway 6, Westport

/ design	/ drawn	/ QA check	/ dwg
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/ scale @ A3	/ date	/ file	/ issue
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