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APPENDIX H

Assessment of Geotechnical Effects – Tonkin & Taylor

REPORT

Tonkin+Taylor

Assessment of Geotechnical Effects

Care Retirement Village, 26 Donald Street and 37 Campbell Street, Karori, Wellington

Prepared for

Ryman Healthcare Limited

Prepared by

Tonkin & Taylor Ltd

Date

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Exceptional thinking together

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1 Executive summary

Ryman Healthcare Limited proposes to construct and operate a comprehensive care retirement village ("Proposed Village") at 26 Donald Street and 37 Campbell Street, Karori, in Wellington ("Site").

The Site is the former location of the Victoria University Teachers College Campus. Three Teachers College buildings will be retained and redeveloped and will be supplemented by a series of new buildings including several with base isolated foundations and single level basements. Deconstruction of the remainder of former Teachers College buildings that are not be retained has been completed. Cut and fill earthworks and ground retention structures will be required to establish new building platforms.

Currently the site topography comprises slopes of up to 20° but are more typically around 5° to 10°. A small gully is present in the north eastern corner of the Site. This area is elevated from the surrounding playing fields, tennis courts and south eastern gully, which are located to the south west and south of the previous buildings. These areas are flat to gently sloping, typically less than 10°.

The planning matters considered in this report and the geotechnical assessment findings are summarised in the following table:

Planning matters considered	**Geotechnical assessment findings**
• Natural hazards that may affect the Proposed Village and adjacent land, being earthquake shaking and associated ground liquefaction/lateral spreading and land instability/subsidence • The potential for earthworks, including excavation and retention associated with the Proposed Village, to affect the stability and/or cause ground deformation of land • The potential for works to affect the groundwater regime at adjacent sites and cause ground deformation or	• Seismic effects could affect geotechnical design at return periods of between 100 and 500 years. The effects (softening, settlement and some strength reduction) can be mitigated in the normal design process. The Proposed Village development is not assessed to exacerbate seismic hazard effects at adjacent sites. • The proposed excavations will be assessed and supported with suitably designed and constructed retaining walls. No adverse land stability impacts are expected at or around the Site. • The potential cumulative settlement due to fill and/or structural loads, excavations and groundwater drawdown are expected to result in less than 10 mm of settlement at any boundary of the Site and at the location of any neighbouring structure. The Proposed Village is therefore not expected to cause any consequential adverse ground deformation or settlement effects on adjacent properties.

This assessment concludes that the Site is suitable for the Proposed Village from a geotechnical engineering perspective. The Proposed Village is not expected to cause any consequential adverse effects on seismic liquefaction, land stability, or ground deformation and settlement.

2 Introduction

Ryman Healthcare Limited ("Ryman") engaged Tonkin + Taylor to undertake a geotechnical investigation and assessment for a proposed comprehensive care retirement village ("Proposed Village") at 26 Donald Street and 37 Campbell Street, Karori, Wellington ("Site"). This report provides a summary of identified geotechnical site constraints and associated environmental effects from construction of the Proposed Village.

The geotechnical, structural, civil engineering and contaminated land aspects of design are integrated. This report should be read in conjunction with the Ground Contamination Assessment[1] to inform the groundwater quality in terms of contamination.

3 Proposed Village

A full description of the Proposed Village can be found in the AEE. Broadly, the Proposed Village is to include:

- Refurbishment of the existing Allan Ward VC Hall and Tennant Block buildings.
- New buildings, including:
  - A 3-level apartment building (part B01A) constructed at grade (without significant earth retention near the boundary).
  - A 3-level village centre building (part B01A) with base isolated foundations (seismic damage mitigation), which will require earthworks cut.
  - A 6-level village centre and apartments building (part B01A) including undercroft basement with base isolated foundations (seismic damage mitigation), which will require earthworks cut and earth retention works.
  - Two new 7-level care and apartment buildings with base isolated foundations (seismic damage mitigation) and a combined single level basement below both above ground structures (B01B).
  - Six new 3-level apartment buildings (B02 to B07) which will be founded on shallow foundations on grade.
- Earthworks and ground retention structures, particularly for the B01A / B01B basements and adjacent carparking area, will be required to establish the proposed site levels. Maximum fill heights are expected to be in the order of 3 m. Cut and retention heights are typically in the order of 4 to 6 m with local areas higher (but well offset from the site boundaries).
- New stormwater and wastewater infrastructure including:
  - Various stormwater conveyance pipes ranging from 900 mm to 1500 mm diameter.
  - An 1800 mm diameter scruffy dome inlet adjacent to the boundary with 18 Scapa Terrace.
  - A flood attenuation/storage device including a 45 x 10.5 x 3 m buried tank between Buildings B03 and B04 which is expected to require excavation up to 4 m depth.
  - Various sanitary sewer conveyance pipes including a 300 mm diameter pipe aligned parallel to 29, 33 and 33A Campbell Street with a proposed invert between 2 m and 3.2 m depth below existing ground level.

[1] Ground Contamination Assessment of Environmental Effects. Care Retirement Village, 26 Donald Street and 37 Campbell Street, Karori, Wellington. Tonkin & Taylor Ltd March 2020. 30309.v2

4 Site description and topography

The Site was formerly used by Victoria University of Wellington as the Education/Teacher's College campus (lectures ceased at the Site in 2016). Purpose built buildings that were located on the Site are shown in the architectural development plans (refer Appendix A). At the time of reporting, deconstruction of the buildings not being retained was complete.

Unsupported slopes of up to 20° are present on the Site, but are more typically around 5° to 10° in the north western area. A concrete crib retaining wall supports a 4m high slope north of this area. Areas between the buildings and accessways were previously vegetated. This vegetation has been partly removed during building deconstruction. A small gully is present in the north eastern corner of the Site. This area is elevated from the surrounding playing fields, tennis courts and south eastern gully, which are located to the south west and south of the previous buildings. These areas are flat to gently sloping, typically less than 10°.

5 Existing Environment – subsurface conditions

The geotechnical assessment is based on our experience in the local geological conditions, review of published geology, historical aerial photography and on-site investigations including 6 machine drilled boreholes, 10 Cone Penetrometer Tests (CPT) and 8 hand augered boreholes.

The Site is located on an elevated terrace bounded by the Khandallah and Wellington Faults. Investigations indicate that an outwash fan comprising interbedded silts and sands with minor gravel forms the more elevated slopes in the north east of the Site. Bedded alluvial soils occur on the flatter terrain below. The surface of the basement greywacke appears to dip down towards the north east.

The ground geological profile generally comprises:

- 0 to 0.5 m of fill;
- Interbedded alluvial soils on the lower elevations (up to a depth of 9.5m) and outwash fan deposits on the elevated slopes;
- Greywacke bedrock at depths ranging from 5 m to greater than 29 m.

Groundwater levels relevant to the proposal were monitored at three locations across the Site and ranged from 1.1 to 3.5 m below ground level.

Further details of the ground investigations undertaken at the Site are presented in Appendix B.

6 Planning context

The planning context for the Site and the Proposed Village is addressed in the Assessment of Environmental Effects. In a geotechnical context, we understand that the relevant planning matters to be considered for the application for resource consent include the following:

- Natural hazards that may affect the Proposed Village and adjacent land, being earthquake shaking and associated ground liquefaction/lateral spreading and land instability/subsidence;
- The potential for earthworks, including excavation and retention associated with the Proposed Village, to affect the stability and/or cause ground deformation of land;
- The potential for works to affect the groundwater regime at adjacent sites and cause ground deformation or otherwise affect the utility of that land.

7 Assessment methodology

In order to assess the geotechnical effects of the Proposed Village, the following methodology was adopted.

A geological and geotechnical model for the Site was developed in stages. An initial model was prepared on a review of aerial photographs, geological maps, our internal geotechnical database, and previous investigations available for the Site. Based on this, and considering the proposed land use, a geotechnical investigation was then carried out including boreholes and groundwater monitoring work. Based on the findings of our review and investigations, geotechnical engineering parameters for assessment of stability and potential ground deformation effects on neighbouring properties were developed.

Following the subsurface model development, the Proposed Village has been considered in the context of the subsurface conditions. In carrying out this work, we liaised with other experts (including Structural Engineers) and Ryman to understand the likely foundation and geotechnical requirements for the Proposed Village. The effects of the Proposed Village have then been assessed based on our experience with similar foundation systems and construction, in the context of the subsurface model and proposed geometry and structural form.

The potential for the materials at the Site to liquefy under seismic shaking has been assessed quantitatively based on CPT data using the method set out by Boulanger and Idriss (2014)[2]. This has been supplemented by a qualitative assessment of the site geomorphology, performance of the site during the 2016 Kaikoura Earthquakes, and a review of borehole logs and other investigation data.

The site slope stability has been based on visual appraisal of the Site by a Geotechnical specialist and review of the Proposed Village plans.

The extent of potential groundwater effects have been assessed using established steady state analytical methods[3]. The potential for settlement associated with groundwater drawdown has been assessed using one dimensional consolidation theory and the geotechnical parameters set out in this report.

The results of our assessments are set out below.

8 Assessment of effects

8.1 Seismic and liquefaction

The seismic subsoil class has been assessed in terms of NZS1170.5:2004[4] to be Class C (Shallow Soil) site. This conclusion is based on the results of the investigation data at the southern end of the Site. It is possible that rock may be greater than 40 m depth over a limited area in the northeast corner of the Site. In this case a portion of Site could approach Class D (Deep or soft soil). If this becomes important, it can be confirmed by further investigations during design of the structures in this area, but is not assessed to have a consequential effect on the following liquefaction assessment

The analysis of the CPT data shows that the risk of liquefaction is confined to the upper recent alluvial soils with potentially liquefiable beds occurring from the surface to 6.5m depth. The calculated ‘free field’ liquefaction induced settlements range are estimated to be less than 25mm under Serviceability Limit State, SLS events, and potentially up to 60mm from a 1 in 100 year event, and up to 100mm for Ultimate Limit State, ULS events. This magnitude of settlement due to liquefaction is meets normally accepted SLS and ULS building performance criteria, but must be considered during the detailed design phase.

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[2] Boulanger. R, Idriss. I, (2014); *CPT and SPT based liquefaction triggering procedures.* Center for Geotechnical Modelling Department of Civil and Environmental Engineering University of California Davis, California. Ref: UCD/CGM-14/01, April 2014.

[3] Marinelli, F., & Niccoli, W. L. (2000). Simple analytical equations for estimating ground water inflow to a mine pit. Groundwater, 38(2), 311-314

[4] NZS 1170.5:2004 Structural Design Actions Part 5: Earthquake Actions, New Zealand

Notwithstanding the CPT analysis, there is no evidence from the site geomorphology that liquefaction is a significant hazard. There are slopes at the Site that could not be present if repeated, consequential liquefaction had occurred at the Site. We are not aware of any consequential damage to the land or the structures at the Site as a result of the shaking experienced in the recent Kaikoura earthquakes.

The nature of the soils (silty, and dense) indicates that under seismic loading with greater than 100 year return period, some softening and strength loss may occur. This softening is not as extreme as the liquefaction that has been observed around Christchurch in sandier materials, and little to no liquefaction ejecta is expected. However, the materials may still experience post-liquefaction settlement, and a reduction in bearing capacity, lateral support and stiffness.

In summary, for seismic loads with return periods of less than 100 years, we do not assess the Site to be at risk of liquefaction that would require specific design consideration. The onset of material softening under cyclic loading may occur somewhere between a 100 to 500 return period event. Little to no liquefaction ejecta is expected under these loadings. However, the materials may still experience post-shaking settlement, and some reduction in bearing capacity, lateral support and stiffness.

The Proposed Village can accommodate the seismic hazard risk in normal design methods, and the Proposed Village is not assessed to exacerbate seismic hazard effects at adjacent sites.

8.2 Land stability

Unsupported slopes of up to 20° but typically around 5° to 10° occur in the north east part of the Site that contained the Teachers College buildings. The remainder of the Site is flat to gently sloping (less than 10°). A visual assessment of these slopes does not suggest any signs of slope instability.

Any significant cuts or excavations to be undertaken for the Proposed Village will be assessed and supported with suitably designed and constructed retaining walls. Refer Section 5.3.1 for further discussion regarding retaining walls.

Isolated areas of fill associated with levelling of local depressions within the Site are not expected to be affect land stability. No significant filling is expected on sloping ground.

In summary, the Proposed Village is not expected to adversely impact land stability on or around the Site.

8.3 Ground deformation and settlement

Settlement effects can be caused by in-situ stress condition changes within the soils beneath the Site or neighbouring properties. Effects on neighbouring properties due to the Proposed Village could conceivably be caused by the following mechanisms:

- Fill/structural loads near the boundary causing consolidation settlement;
- Reduced lateral confinement from soil excavation leading to mechanical deformation;
- Groundwater drawdown below historic low levels leading to consolidation settlement.

Each of these items is described in more detail in the following sections.

In summary, the potential cumulative settlement from the assessments described below are assessed to typically result in less than 5 mm of settlement at boundaries, with a worst case of less than 10 mm of settlement at the specific boundary locations identified and at the location of neighbouring structures. On this basis, the Proposed Village is not assessed to have any consequential adverse ground deformation and settlement effects on adjacent properties.

8.3.1 Fill and structural loads

The available final contour and cut/fill plans[5,6] indicate the limited fill earthworks will be required for the Proposed Village. Most fill areas are located remote from the Site boundaries. Filling of about 2.5 m is indicated within a gully close to the boundary with Donald Street. The area of proposed fill grades to existing ground level at the boundary is limited in extent (about 5 m length of boundary). On this basis, consequential settlement at the boundary is assessed as very unlikely.

Based on our review of the proposed building layouts[7], new buildings and associated foundations will be greater than 4 m from the boundaries in all cases. No settlement effects beyond the Site boundaries are expected due to structural loading from the buildings.

8.3.2 Excavations and retaining walls

8.3.2.1 Permanent excavations and basements

The plans indicate several areas of permanent cut will be required to form the proposed Site levels. Possible lines of ground retention works are indicated in Figure 8.1. However, in many areas where cut depths are low or distance from the boundary permits, temporary batters may also be adopted. A description of each area identified in the figure and the potential effects is provided below.

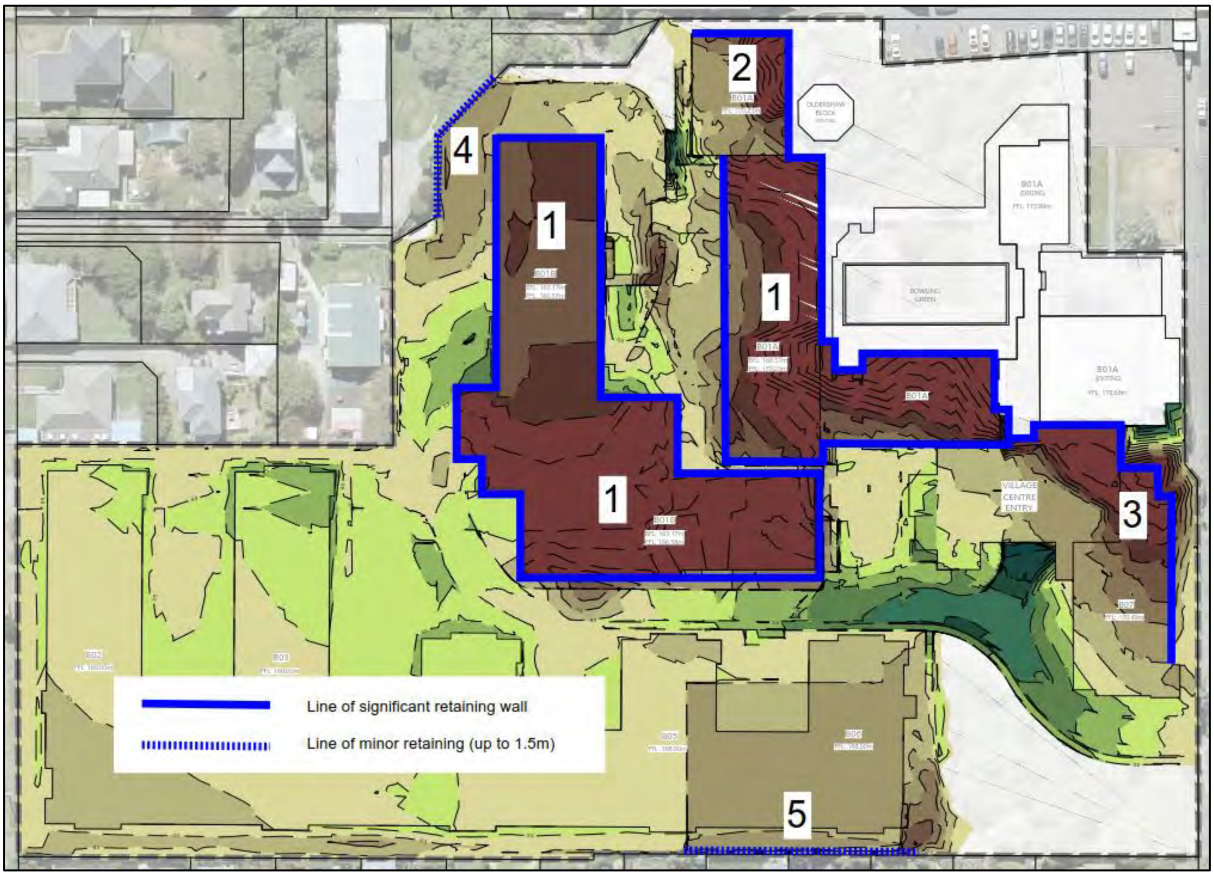


Figure 8.1: Anticipated retaining walls

[5] Woods drawing; Final contour plan. Ref: 042-RCT_401_C0-120, Rev 1, 14/8/20
[6] Woods drawing; Depth (cut/fill) contour plan. Ref: 042-RCT_401_C0-120, Rev 1, 14/8/20
[7] Ryman drawing; Proposed site plan with aerial. Ref .A0-021, RC04, Rev A, July 2020

1 Basement excavations for north wing of Building B01B will be up to 2.5 m and more than 15m from the boundaries. Greater excavation depths are required for the Building B01B southern wing and Building B01A (typically 5 to 6 m, locally up to 7m). These areas are located at least 40 m from the Site boundaries. No mechanical effects related to these excavations are assessed.

2 Excavations (about 5.5m depth) associated with the northern portion of B01A close to the boundary with 22 Donald St (Karori Swimming Pool) are indicated. Temporary retaining walls will be required to support the cut during foundation construction, and the building will provide long term support. The wall alignment will be offset from the boundary by about 3 m. Beyond the boundary is a concrete footpath and stairs. The nearest structure is about 5 m from the proposed wall alignment. Further assessment of potential settlement effects due to retaining wall deformation is provided in Section 8.3.2.2 below and concludes that a specifically designed 'high stiffness' wall can suitably mitigate against adverse effects.

3 Excavations (about 4 m depth) associated with forming the B07 building platform and close to the boundary with Donald St. The final contour plan indicates batter slopes about 2m high and 1(V):2(H) adjacent to the boundary. The architectural plans for this area[8] indicate the finished lower floor of the building below ground level. We therefore assess that any batter in this area will be temporary and will be backfilled against the new building or a temporary retaining wall with permanent support from the building will be adopted to support the cut associated with this building platform. The toe of the batter or wall alignment will be around 5 m from the boundary. The nearest potentially sensitive public service is a 150 mm cast iron water supply main located about 9 m from the wall alignment. Further assessment of potential settlement effects due to retaining wall deformation is provided in Section 8.3.2.2 below and concludes that a suitably designed wall can suitably mitigate against consequential adverse effects.

4 A zone of minor cut (1.0 m to 1.5 m indicated) adjacent to open ground in the Council owned land between the site and 27A Campbell St (Karori RSA). This is a minor regrading, and effects beyond the site boundary are assessed as negligible.

5 A zone of minor cut (1.0 m to 1.5 m indicated) is associated with the removal of the existing sports courts close to the boundary with 8 to 14 Scarpa Tce. Architectural plans[9] indicate retained heights at the boundary of up to 1.6 m. Based on the ground conditions and retained heights, we assess that ground stability and deformation beyond the Site boundaries can be controlled by normally adopted retaining wall solutions in this area and no consequential effects to neighbouring properties is expected.

8.3.2.2 Retaining wall deformation related settlement (mechanical settlement)

A preliminary assessment of the potential retaining deformation and settlement effects at the nearest boundary has been undertaken. The assessed locations are the northern wall of B01A (zone 2 description above) and the possible eastern wall of B07 (zone 3 description above). Ground movement estimates are based on empirical published relationships[10] for embedded retaining walls in stiff cohesive soils. Figure 8.2 shows the adopted relationships for high and low stiffness walls. The results of this assessment are summarised in Table 8.1.

[8] Ryman drawing; Site – cross sections. Ref: A0-52, RCA14, A, July 2020.
[9] Ryman drawing; Site – cross sections. Ref: A0-53, RCA15, A, July 2020.
[10] CIRIA 2017; Guidance on embedded retaining wall design. C760(2017).

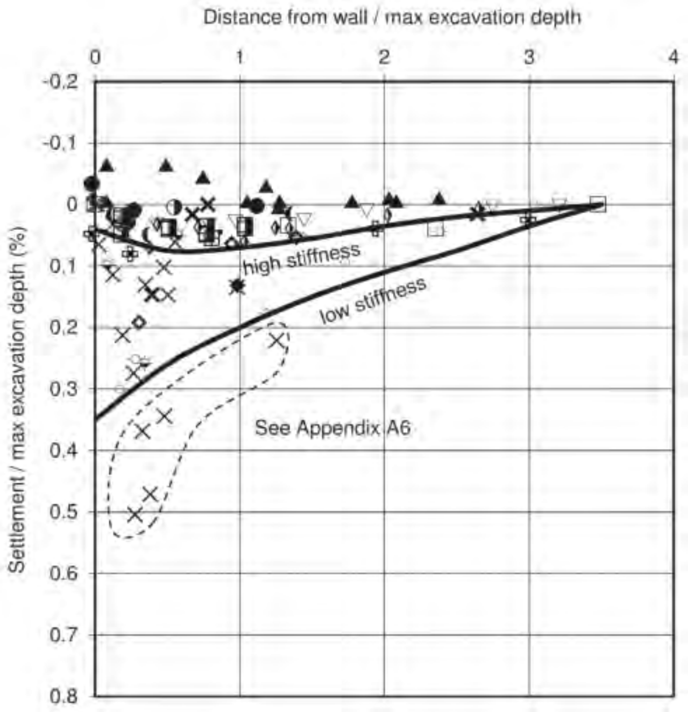


Figure 8.2: Empirical relationships for retaining wall induced vertical ground movement (source: CIRIA C760)

Table 8.1: Retaining wall induced mechanical settlements

Location (refer 8.3.2)	Wall stiffness*	Wall height	Distance to boundary	Distance to nearest building	Estimated ground settlement	
					Boundary	Nearest building/service
Zone 2, B01A	High	5.5 m	3.0 m	5.0	4 mm	4 mm
Zone 3, B07	Low	4.0 m	5.0 m	N/A	7 mm	4 mm

Notes: * Typical examples of low stiffness walls: soldier pile cantilever or with low-stiffness temporary props or props installed at a low level. Typical examples of high stiffness walls: piled walls propped at a high level or top-down construction.

A high stiffness wall (such as a propped or anchored soldier pile wall, or specifically designed retention system) will be required at the northern end of B01A to control deformation at 22 Donald St (Karori Swimming Pool) to within typically accepted criteria (deformations assessed are less than the 10 mm normally tolerated). The possible retaining wall at the eastern perimeter of B07 and adjacent to the Donald St road reserve can be constructed with a low stiffness wall (such as a cantilever soldier pile and deformations will remain within typically acceptable criteria (deformations assessed are less than the 20 to 30 mm normally tolerated for road reserves). The adopted methodology assesses less than 5 mm settlement at the location of a cast iron water supply main and no consequential adverse effects are expected. Furthermore, we consider that this is a conservative estimate and the risk of consequential mechanical ground deformation at the pipe alignment is very low.

8.3.2.3 Temporary excavations and services

Aside from the areas discussed below, the excavations for the stormwater conveyance and detention infrastructure are set back from the boundary greater than the required depth of excavation and do not require specific assessment. We recommend the use of suitable construction techniques (shoring of trenches or retention of cut faces greater than 1.5 m depth). Based on that recommendation, the risk of mechanical settlement at the Site boundaries is assessed as negligible.

Isolated areas which do not fit this description are indicated in Figure 8.3 / Figure 8.4 and further assessed below. The estimated settlements for shored excavation have been estimated by the same methodology described in Section 8.3.2.2 and a low support stiffness.

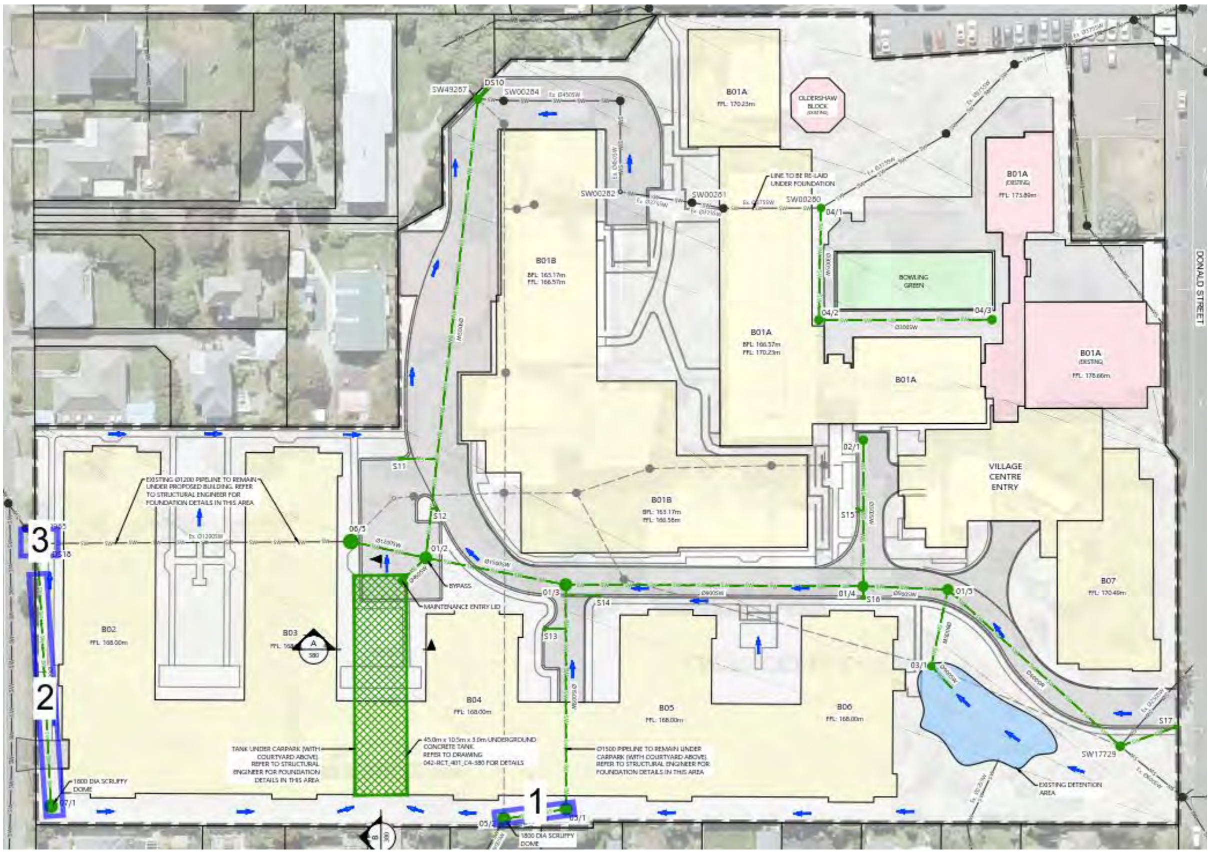


Figure 8.3: Stormwater infrastructure with potential effects on adjoining properties.

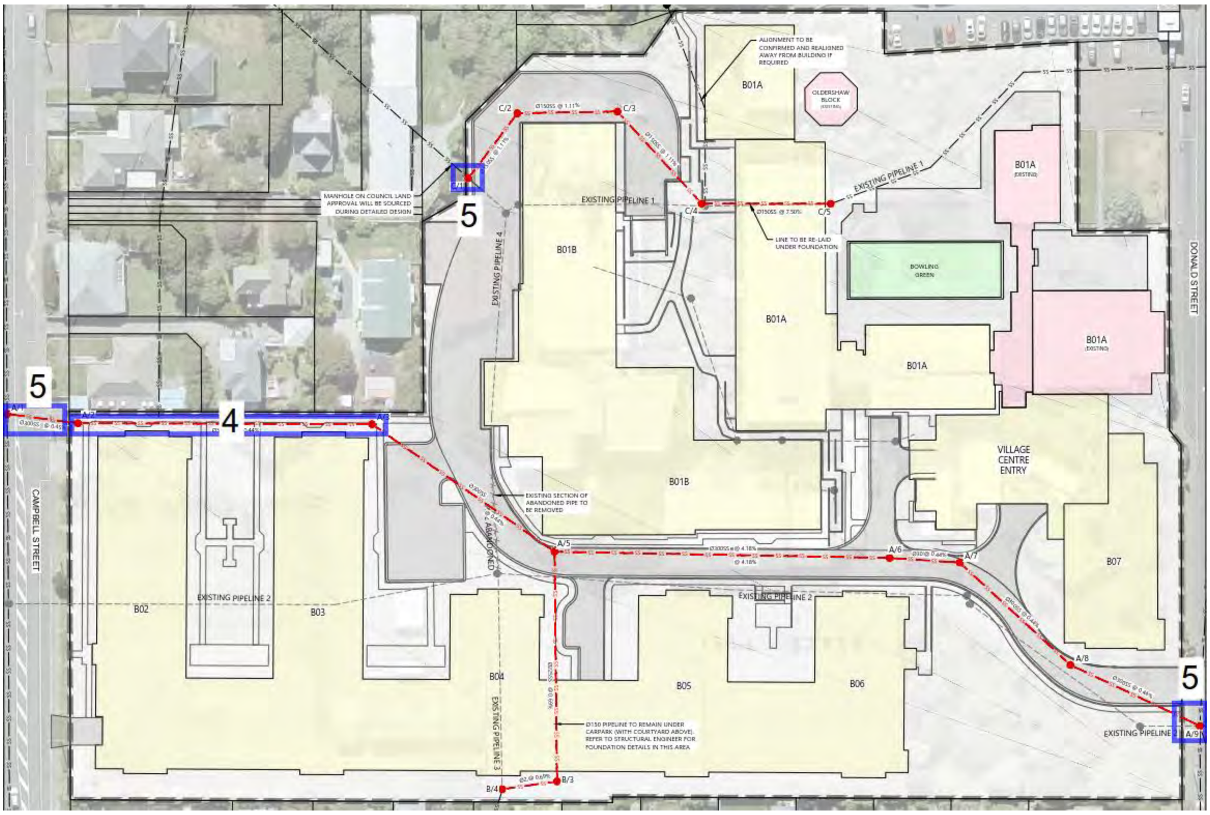


Figure 8.4: Wastewater infrastructure with effects on adjoining properties.

1 An 1800 mm diameter scruffy dome inlet and 1500 mm diameter stormwater pipe adjacent to the boundary with 16 and 18 Scapa Tce. The invert of the scruffy dome chamber is 2.4 m depth and around 0.5 m from the boundary. The neighbouring manhole 13 m to the northeast has an invert of 3.0 m depth at about 3 m from the boundary. Some ground movement at the boundary and within neighbouring landscaping areas could occur, reducing to no deformation expected at the nearest residential and ancillary structures over 7 m away. The magnitude of settlement at the boundary could be in the order of 5 to 10 mm, depending on the retention system selected and proximity of excavation to boundary. These settlements are within typically accepted tolerances and no consequential adverse effects are expected.

2 An 1800 mm diameter scruffy dome inlet adjacent to the boundary with 49 Campbell St and a 600 mm diameter stormwater pipe aligned approximately parallel with Campbell St. The invert is less than 1.5 depth and where they are greater than 1.5 m from the site boundaries no consequential effects are assessed. However, the pipeline comes within close proximity the boundary at the northern end of the alignment where it discharges to the manhole noted in item 2 below. Some minor ground disturbance in the Campbell St road reserve may be required to install the pipe (noting the excavation is no more than 1.5 m depth). These works area routine activity , and the effects can be addressed with Council.

3 A new manhole adjacent to Campbell St with an invert of 5.85 m depth is proposed. Ground disturbance in the Campbell St road reserve will be required in order to install the manhole chamber. Manhole installation is a routine activity, and the effects can be addressed with Council.

4 A 300 mm diameter sanitary sewer conveyance pipe aligned parallel and 2.5 m from the boundaries with 29, 33 and 33A Campbell St. Each is discussed below:

– 29 Campbell St: depth to invert 2.0 to 2.3 m. Open trench excavations will offset from the boundary at or marginally less than the required excavation depth. Some minor settlement (<10mm) may occur at the property boundary and neighbouring landscaped areas reducing to less than 5 mm at the nearest secondary structure within the property and therefore consequential effects are not expected.

– 33 Campbell St: depth to invert 2.8 to 3.2 m. Open trench excavations may incur settlement in the order is 5 to 10 mm at the boundary (depending on the retention system selected and proximity of excavation to boundary). The potentially affected property in this case is owned by Ryman and therefore effects not further assessed.

– 33A Campbell St: Depth to invert 2.3 to 2.8 m Open trench excavations will offset from the boundary at or less than the required excavation depth. Some minor settlement (<10mm) may occur at the property boundary reducing to very low levels (<5 mm) at the residential dwelling located about 5 m from the excavation. Consequential adverse effects are not assessed at this level of deformation.

5 New sanitary sewer connections at Campbell St, Donald St and Council owned land between the site and 27A Campbell St (Karori RSA). In these locations new sewer connections will require ground disturbance within Council land.

8.3.3 Groundwater drawdown

8.3.3.1 Geological profiles and settlement parameters

The adopted geological profiles and consolidation parameters for the groundwater drawdown assessment are as summarised in Table 8.2. The consolidation parameters are generally in accordance with the geotechnical parameters adopted for the Site and detailed in Appendix B.

Table 8.2: Consolidation parameters for drawdown assessment

Location	Depth to groundwater	Thickness of soft to firm alluvial layers below groundwater, mv = 0.25 (m^2/MN)	Thickness of stiff to very stiff alluvial layers below groundwater, mv = 0.05 (m^2/MN)
BH1	1.2 m (measured)	4.5 m	-
BH2	1.2 m (inferred)	0.9 m (at depth)	3.1 m
BH3	3.5 m (measured)	1.2 m	-
BH5	3.5 m (inferred)	-	8.5 m
BH6	3.4 m (inferred)	-	2.3 m

8.3.3.2 Proposed basement excavations

Permanent groundwater drawdown is expected due to the construction of various basement and foundation construction on the site. The critical section through the Site indicating the potential drawdown extents is shown in Figure 8.5 below.

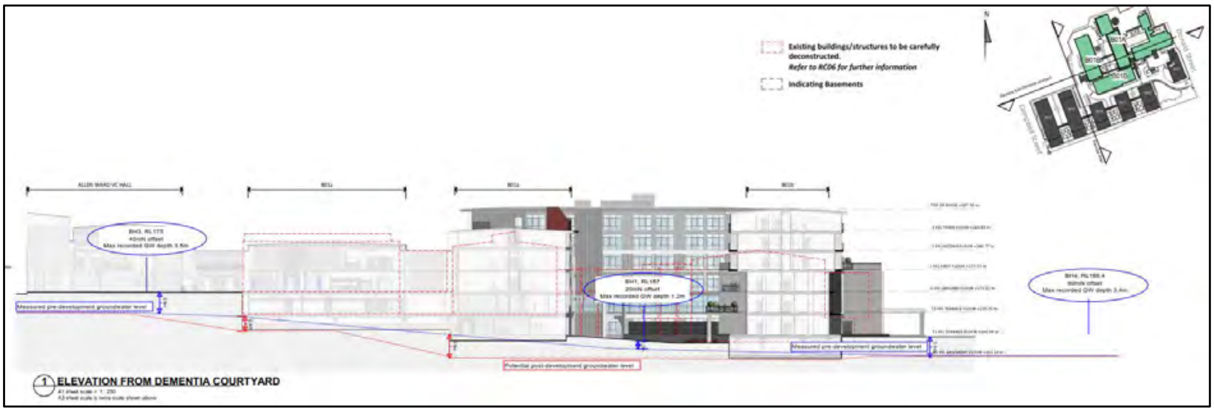


Figure 8.5: Groundwater drawdown critical section

Potential groundwater drawdown has been assessed based on the measured groundwater levels and proposed basement extents. The extent of groundwater drawdown may be expected to be negligible at a horizontal distance from the excavation within 5 times the drawdown height. On this basis the following drawdown below pre-development levels associated with the various basement and foundation construction.

The drawdown at each location has been compared with the geological profile at the nearest borehole location to assess the maximum expected drawdown related settlement and extent.

The assessed settlement values summarised in Table 8.3 are considered likely to be conservative as they do not allow for long term groundwater fluctuation or possible non-hydrostatic groundwater profiles.

Table 8.3: Groundwater related settlement and extents

Building	Maximum expected drawdown at basement	Geological profile adopted	Calculated settlement	Distance to nearest boundary	Settlement at the nearest boundary
B01A (ex Oldershaw Building location)	1.8 m (Note 1)	BH02	6 mm	3 m	<5 mm
B01A (ex Gray Building location)	2.4 m	BH03	<5 mm	30 m	Negligible
B01A (ex Waghorn Building location)	1.1	BH02, BH03	<5 mm	30 m	Negligible
B01B (ex Pankhurst Building location)	1.3	BH01, BH02	<5 to 13 mm (Note 2)	10 m	Negligible
B01B (ex Theatre and Dance Studio)	3.9	BH01, BH06	<5 to 35 mm (Note 2)	25 m	Negligible
B07	0.5	BH05	<5 mm	5 m	< 5 mm

Notes: 1 – Additional drawdown below ex Oldershaw lower floor level

2 – The upper range of settlement is based on greater thickness of weak alluvial soils encountered in BH01. These materials are not present to similar extents in other surrounding boreholes and therefore settlement effects are expected to be limited in extent within the site.

The assessment of groundwater drawdown induced settlement due to the proposed basement / foundation excavations and permanent drainage indicates potential settlement of less than 5mm at the boundaries of the Site. On this basis the potential groundwater drawdown is not assessed to have any consequential adverse effects on adjacent properties. A summary of the combined effects of mechanical and drawdown induced settlements are provided in Section 8.3.4.

8.3.3.3 Stormwater attenuation/storage tank

The flood attenuation/storage device including a 45 x 10.5 x 3 m buried tank between Buildings B03 and B04 which is expected to require excavation up to 4 m depth must consider potential buoyancy effects due to the presence of groundwater. This may be addressed in several ways, including piling or using drainage to control groundwater pressures. The method assessed here (that would cause maximum groundwater drawdown) prevents groundwater pressures rising at the tank location by permanently draining the backfill surrounding the tank. We have assessed effects based on this method and conclude they are negligible. Other methods will have lower effects if they are contemplated during detailed design.

For the assessment of groundwater drawdown effects we have adopted a drawdown level of 161.5 mRL. The lowest recorded groundwater level at location BH4 which is within the area of the proposed flood attenuation/storage device was 3.4 m depth (163 mRL) giving a groundwater drawdown for assessment of 1.5 m within the Lower Alluvium strata. The effective drawdown zone of influence tank is assessed to be no more than 10 m, and more likely around 5 m. The soils within the zone of influence generally comprise up to 6 m of very stiff sandy silt and medium dense silty sand for which a coefficient of volume compressibility of 0.05 m^2/MN been adopted.

Based on the above, the settlement due to 1.5 m of groundwater drawdown is calculated to be less than 2 mm at the maximum drawdown location. This will reduce away from the tank footprint, and we do not therefore assess any consequential drawdown effects at the Site boundary.

8.3.4 Combined deformations and settlement effects

Assessed ground deformation and settlement due to combined effects are of excavation (mechanical) and groundwater drawdown are generally expected to be negligible and no more than 5 mm. Exceptions to this are summarised for each affected property below. In all cases these values are within typically accepted tolerance for the neighbouring conditions, i.e. landscaping areas or typical structures types.

Summary of affected properties:

- 22 Donald St (adjacent proposed B01A): combined deformations based on a high stiffness retaining system are assessed as less than 10mm at the site boundary, reducing again to the nearest structure (Karori Swimming Pool). These are within normally accepted tolerances, and so we assess no consequential adverse effects.
- Donald St road reserve (adjacent proposed B07): combined deformations based on a low stiffness retaining system are assessed as likely to be in the order of 5 to 10 mm, and possibly up to 15mm at the site boundary. This is lower than normally tolerated for road reserves. The risk of ground deformation at the identified cast iron water main within the road reserve (9 m from the wall) is assessed to be very low. We therefore assess the potential for consequential adverse effects as negligible.
- Residential properties.
  The below residential properties are assessed with typically less than 5 mm of settlement potential at any structure. The risk of consequential adverse effects is therefore assessed as negligible.
  - 16 and 18 Scarpa Tce: mechanical deformation at the boundary and within neighbouring landscaped area of 5 to 10 mm is assessed during installation of

stormwater infrastructure. This may be expected to reduce to negligible deformation at the nearest residential and ancillary structures.

  - 29 Campbell St: mechanical settlement of less than 10 mm is assessed at the boundary and within the neighbouring landscaped area reducing to less than 5 mm at the nearest secondary structure within the property.
  - 33 Campbell St: mechanical settlement of 5 to 10 mm is assessed at the boundary and neighbouring residential type structure. We note that the potentially affected property in this case is owned by the developer (Ryman Healthcare).
  - 33A Campbell St: mechanical settlement of less than 10 mm is assessed at the boundary and within the neighbouring landscaped area, reducing to less than 5 mm and the nearest residential structure within the property.
- Various locations within Council administered land will require ground disturbance to install new stormwater and wastewater connections required by the Proposed Village.

9 Regional planning rules, engineering assessment

An assessment against the relevant permitted rules in the Proposed Natural Resource Plan (Rule 140) and Regional Freshwater Plan (Rules 7 and 9B) is provided below:

Rule	Engineering assessment
Proposed Natural Resource Plan Rule 140 Rule R140: Dewatering – permitted activity. The take of water and the associated diversion and discharge of that water for the purpose of dewatering a site, including but not limited to, maintenance, excavation, construction or geotechnical testing, is a permitted activity, provided the following conditions are met: (a) the take continues only for the time required to carry out the work but does not exceed one month, and (b) the take and diversion and discharge is not from, onto or into contaminated land or potentially contaminated land, and (c) the take does not cause ground subsidence, and (d) the take does not deplete water in a water body, and (e) there is no flooding beyond the boundary of the property.	(a) the take of groundwater will be continue for greater than month. Drained basements on the site will collect and discharge groundwater to the stormwater system. (b) the T+T Ground Contamination Assessment (ref: 30309.v3, August 2020) states: “With one minor exception, groundwater concentrations… …comply with the trigger levels for the protection of freshwater species. The concentrations indicate that contaminants are not being discharged to groundwater within the Site. (c) the drawdown related settlement at the boundary has been assessed as negligible. (d) the Proposed Village does not deplete water in a water body (e) the proposed civil infrastructure design is intended to improve the flooding risk in the area. In summary, the proposed dewatering does not meet the permitted activity standards of R140.
Regional Freshwater Plan Rule 7 Rule 7 Minor abstractions The taking or use of less than 20,000 litres per day of fresh water (including fresh water from any aquifer), other than the taking	(1) groundwater flow rates will be (significantly) less than 2.5 l/s.

Rule	Engineering assessment
of water from the Lower Hutt Groundwater Zone, is a Permitted Activity, provided that it complies with the conditions specified below. Conditions (1) The water shall be taken at a rate of no more than 2.5 litres per second. (2) In the case of groundwater, there are no adverse effects on the take from adjacent bores. (3) There shall be no more than one abstraction point serving the land described in a particular certificate of title. (4) Fish, including small fish, are prevented from entering the reticulation system.	(2) the groundwater will be taken from within 7 m of the ground surface. No adverse effects on water takes may be expected. (3) the groundwater will be taken from multiple points for drained basements proposed in the Site. (4) fish, including small fish, are not expected to be present in the intercepted groundwater. In summary, the application does not meet the permitted activity standards for minor abstraction and consent as a Discretionary Activity is required for the dewatering of the site under Rule 16.
Regional Freshwater Plan Rule 9B Rule 9B Diversion of groundwater. The diversion of groundwater is a permitted activity, provided that it complies with the conditions specified below: (1) There shall be no adverse effects on water supply other than for a temporary period during construction of no more than 24 hours. (2) There shall be no flooding of land on any neighbouring property. (3) There shall be no lowering of water levels in any river, lake, or wetland. (4) There shall be no lowering of groundwater levels on any neighbouring property.	(1) the Proposed Village will not affect water supply. (2) the proposed civil infrastructure design improves the flooding risk in the area. (3) no surface water features will be lowered. (4) groundwater levels are expected to be lowered without consequential effects on some neighbouring properties. The drawdown has been assessed and no adverse effects are expected. In summary, the diversion of groundwater meets the permitted activity standard under Rule 9B.

10 Conclusions

This assessment finds the Site is suitable for the Proposed Village from a geotechnical engineering perspective and is not expected to cause any consequential adverse effects on seismic liquefaction land stability, or ground deformation and settlement.

11 Applicability

Recommendations and opinions in this report are based on data from discrete investigation locations. The nature and continuity of subsoil away from these locations are inferred but it must be appreciated that actual conditions could vary from the assumed model.

This report has been prepared for the exclusive use of our client Ryman Healthcare Limited, with respect to the particular brief given to us and it may not be relied upon in other contexts or for any other purpose, or by any person other than our client, without our prior written agreement.

We understand and agree that our client will submit this report as part of applications for resource consent and that Wellington City Council and Greater Wellington Regional Council as the consenting authorities will use this report for the purpose of assessing that application.

Tonkin & Taylor Ltd

Report prepared by:	Authorised for Tonkin & Taylor Ltd by:
....	....
Matthew Kent	Pierre Malan
Geotechnical Engineer	Project Director

MMK
\\ttgroup.local\corporate\auckland\projects\30309\workingmaterial\geotechnical planning report\mmk.200821.geotecheffectsreport.v4.docx

Appendix A: Figures

Notes: 1) 1m countours obtained from Wellington City Council GIS database. (Based on a DTM created from 2006 LiDAR and updated using 2009 photogrammetry masspoints and breaklines where significant changes in the terrain were detected. WCC generalised the contours by up to 0.2m in order to reduce the number of vertices in the dataset. Contours generated from LiDAR and photogrammetry should not be used for detailed engineering design.)

A3 SCALE 1:1,000

0 10 20 30 40 50 Meters

Tonkin+Taylor
105 Carlton Gore Rd, Newmarket, Auckland
www.tonkintaylor.co.nz

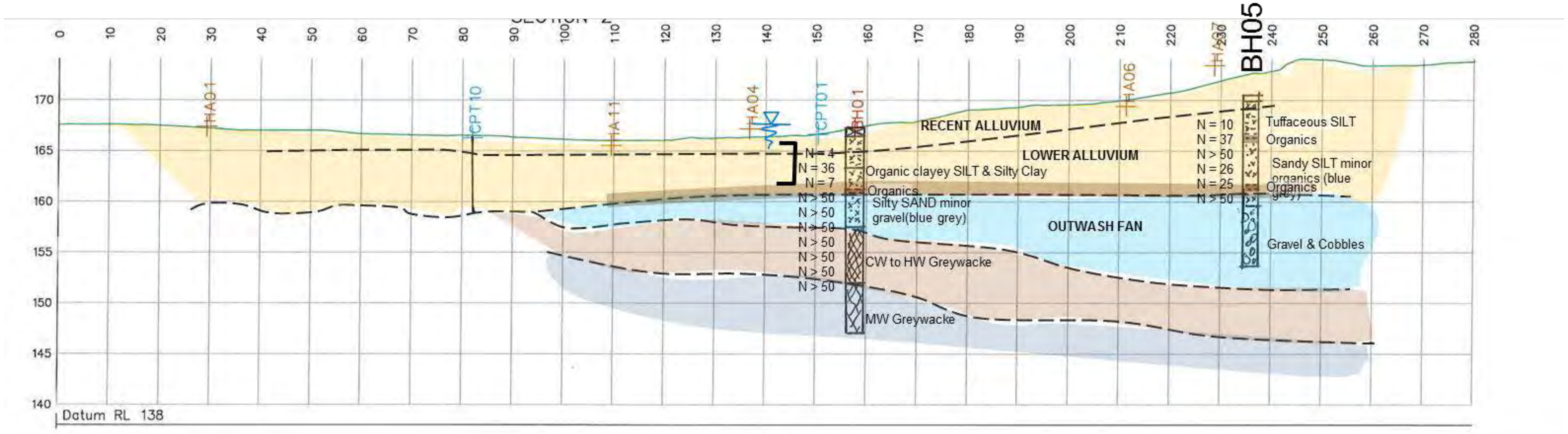
DRAWN	MHU	Oct.17
CHECKED		
APPROVED		
ARCFILE	GroundInvestigation.mxd	
SCALE (AT A3 SIZE)	1:1,000	
PROJECT No.	30309	

Ryman Healthcare Ltd
Karori Campus Redevelopment
Karori, Wellington
Geotechnical Investigation Plan

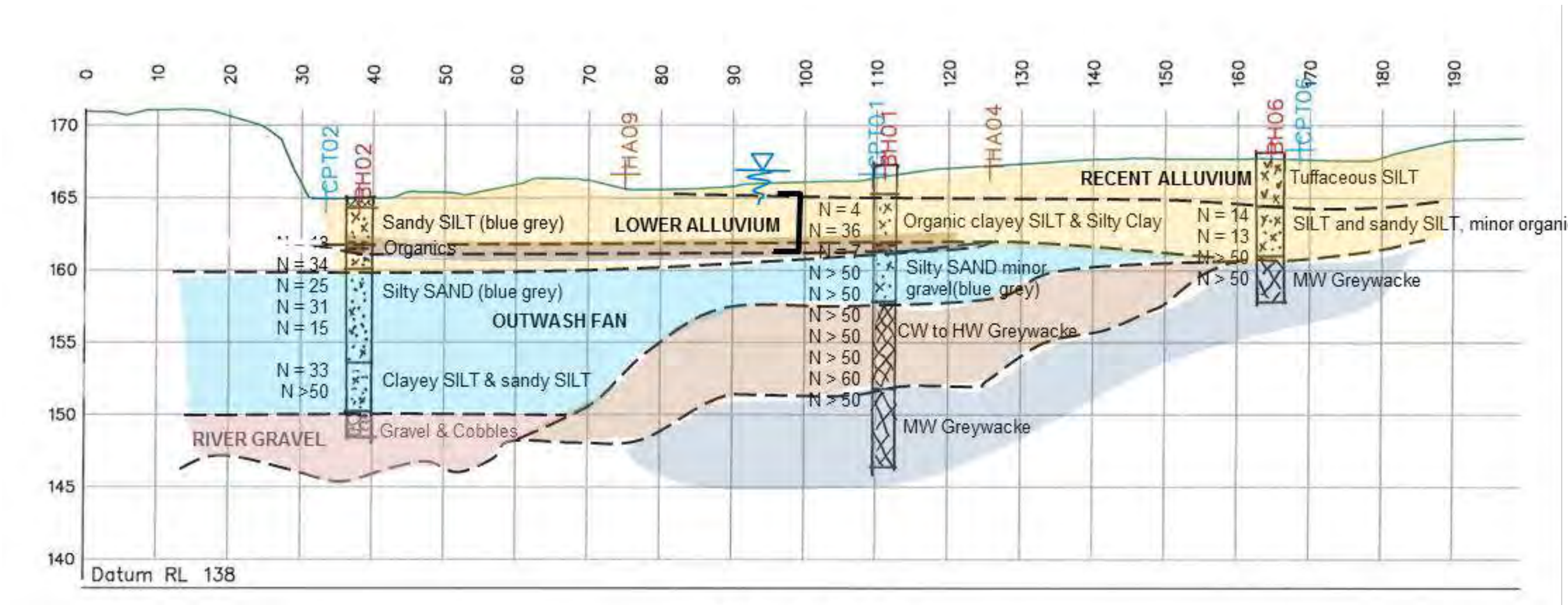
FIGURE No. Figure 1.

Rev. 0

Path: P:\30309\WorkingMaterial\Report Plans\ArcGIS\GroundInvestigation.mxd Date: 27/10/2017 Time: 10:04:57 a.m.



SECTION 1



SECTION 2

LEGEND

- Recent and Lower Alluvium
  - organics
- Outwash fan
- River gravel
- Completely to highly weathered greywacke
- Moderately weathered greywacke
- Piezometer response zone
- Measured groundwater level

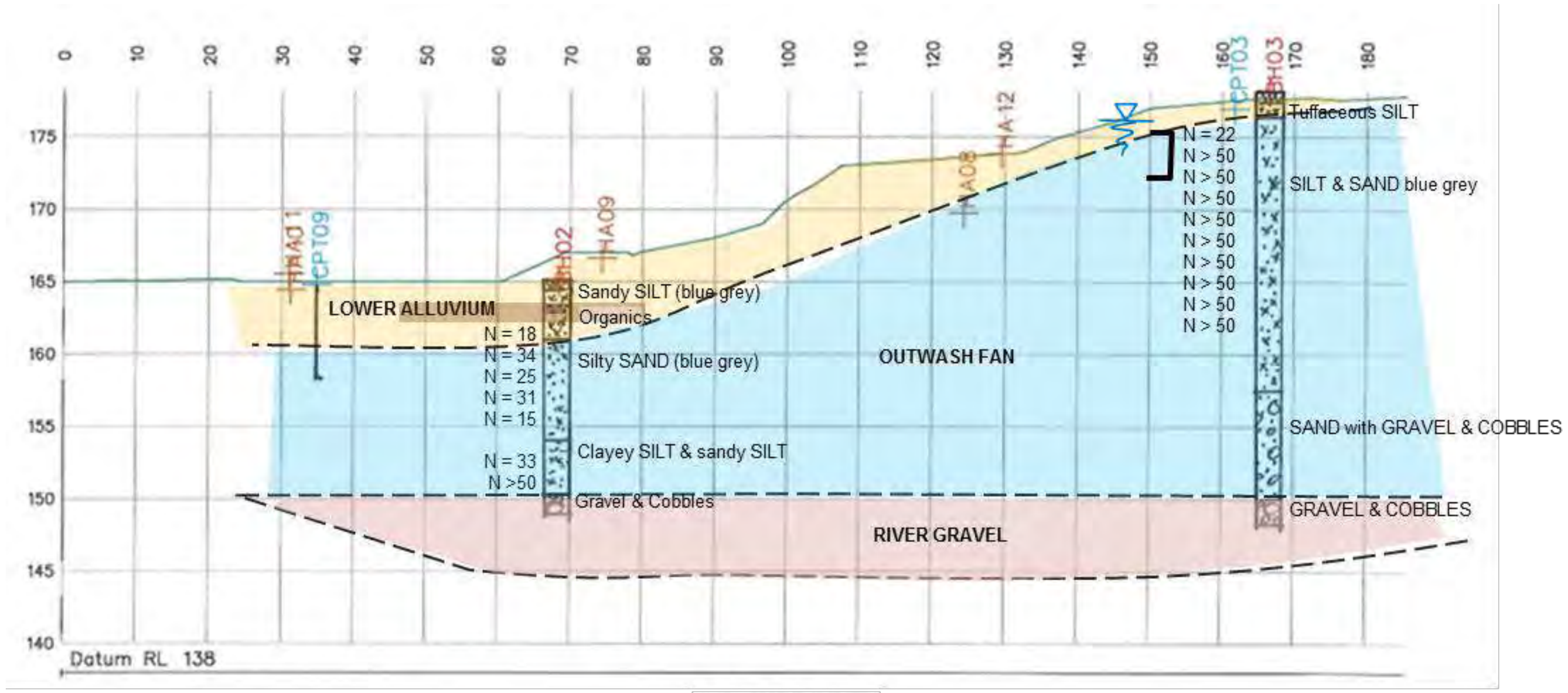
Tonkin+Taylor	www.tonkintaylor.co.nz
DRAWN	RHGR Oct.2017
DRAFTING CHECKED	
APPROVED	
FILE :	FILE
APPROX. SCALE (AT A3 SIZE)	Horizontal = 1:50 : Vertical = 1:100
PROJECT No. 30309	Karori Campus Redevelopment
FIG. No.	Figure 3
REV.	0

Ryman Healthcare Ltd

Karori Campus Redevelopment

Karori, Wellington

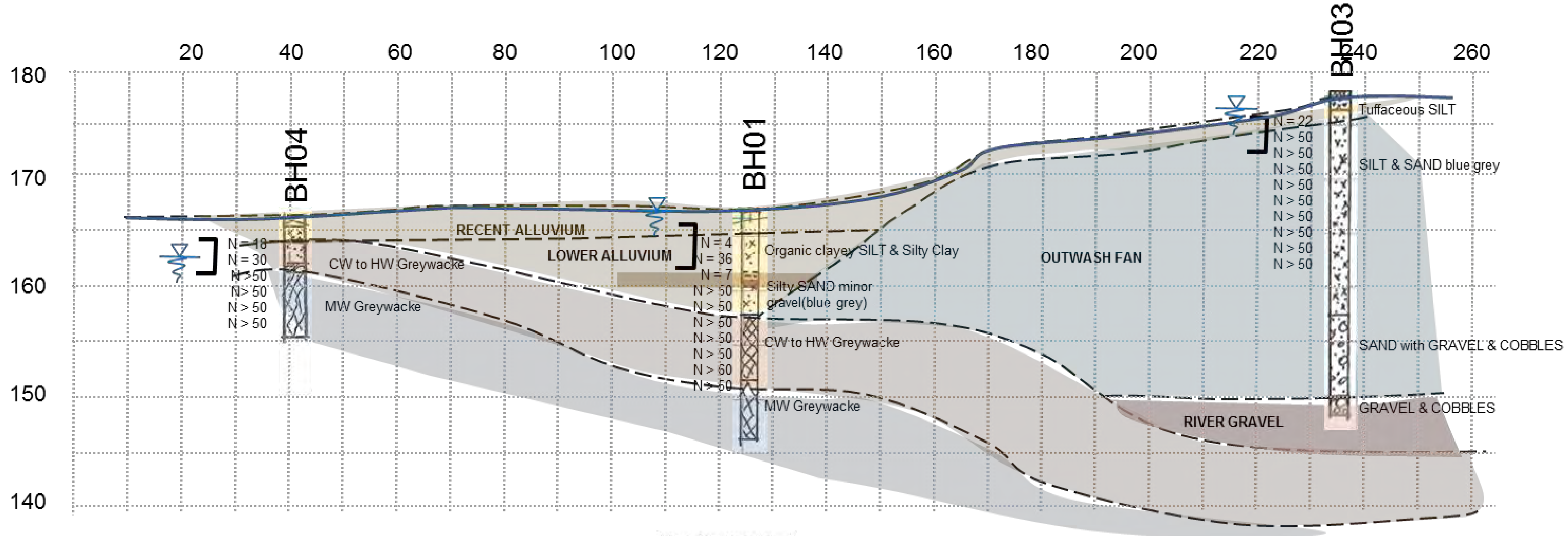
Sections 1 to 2



SECTION 3

LEGEND

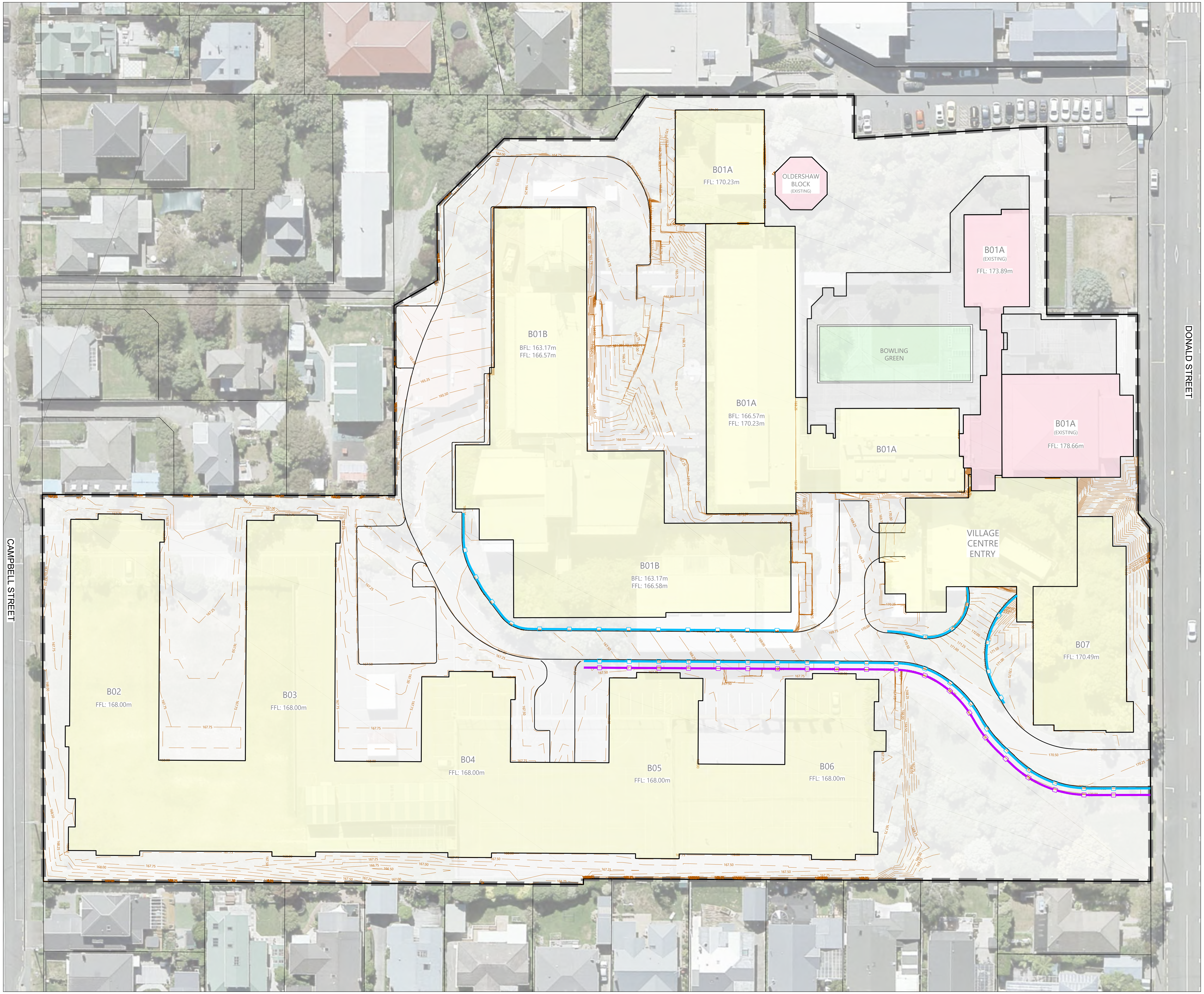
- Recent and Lower Alluvium
  - organics
- Outwash fan
- River gravel
- Completely to highly weathered greywacke
- Moderately weathered greywacke
- Piezometer response zone
- Measured groundwater level



SECTION 4

)

Tonkin+Taylor www.tonkintaylor.co.nz	DRAWN: RHGR Oct.2017 DRAFTING CHECKED APPROVED FILE: FILE APPROX. SCALE (AT A3 SIZE): Horizontal = 1:50 : Vertical = 1:100 PROJECT No. 30309 Karori Campus Redevelopment	Ryman Healthcare Ltd Karori Campus Redevelopment Karori, Wellington Sections 3 to 4 FIG. No. Figure 4	REV. 0



WOODS
EST. 1970

LEGEND

SITE BOUNDARY

PROPOSED BOUNDARY

EXISTING BOUNDARY

PROPOSED CONTOURS MAJOR (0.5m INTERVAL) — 19.0

PROPOSED CONTOURS MINOR (0.1m INTERVAL)

PROPOSED RETAINING WALL

NOTES

1. SURVEY INFORMATION SUPPLIED BY AURECON:
   LEVELS IN TERMS OF WELLINGTON 1953 DATUM (MSL)
   SITE BENCHMARK IR2, RL=166.85
   COORDINATES IN TERMS OF NZGD 2000 WELLINGTON CIRCUIT
   ORIGIN - SS 17K24 SO 30955
   801579.521mN
   396967.337mE
2. ALL WORKS AND MATERIALS TO COMPLY WITH THE WCC STANDARDS, NZBC AND WOODS SPECIFICATIONS. ANY AMBIGUITY BETWEEN DRAWINGS AND STANDARDS SHALL BE REPORTED TO THE ENGINEER FOR CLARIFICATION.
3. IT IS THE RESPONSIBILITY OF THE CONTRACTOR TO CONFIRM THE LOCATION AND PROTECT EXISTING SERVICES DURING WORKS.
4. CONTRACTOR TO CONFIRM ALL INVERT LEVELS OF EXISTING SERVICES BEING CONNECTED INTO PRIOR TO COMMENCEMENT OF WORKS. ANY DISCREPANCY WITH THE LEVELS SHOWN ON THE DRAWINGS SHALL BE BROUGHT TO THE ENGINEERS IMMEDIATELY.
5. ALL UNSUITABLE MATERIAL BE REMOVED AND THE STRIPPED AREAS INSPECTED BY THE ENGINEER BEFORE FILLING COMMENCES.
6. EARTHWORKS ARE NOT TO BE EXTENDED INTO ADJOINING PROPERTIES UNLESS THE ENGINEER HAS ISSUED SPECIFIC INSTRUCTIONS.
7. UNDERFILL DRAINAGE IS TO BE INSTALLED AT THE DIRECTION OF THE ENGINEER. IF THE CONTRACTOR ENCOUNTERS NATURAL SPRINGS OR OTHER SOURCES OF WATER HE/SHE IS TO NOTIFY THE ENGINEER.

REVISION DETAILS		BY	DATE
1	ISSUED FOR CONSENT	WMV	14/08/20

SURVEYED	-	29 DONALD STREET KARORI WELLINGTON 6012
DESIGNED	JLS	
DRAWN	JLS	
CHECKED	MC	
APPROVED	MC	WOODS.CO.NZ

N

RYMAN HEALTHCARE

KARORI
RETIREMENT VILLAGE

FINAL CONTOUR PLAN

STATUS	ISSUED FOR CONSENT	REV
SCALE	1:750 @ A3	1
COUNCIL	WELLINGTON CITY COUNCIL	
DWG NO	042-RCT_401_C0-110	

LEGEND

STAGE BOUNDARY

PROPOSED BOUNDARY

EXISTING BOUNDARY

CUT/FILL DEPTH CONTOUR (0.5m INTERVAL)

CUT HATCH	FILL HATCH
>3.5m	0.0 to 0.5m
3.0 to 3.5m	0.5 to 1.0m
2.5 to 3.0m	1.0 to 1.5m
2.0 to 2.5m	1.5 to 2.0m
1.5 to 2.0m	2.0 to 2.5m
1.0 to 1.5m	2.5 to 3.0m
0.5 to 1.0m	3.0 to 3.5m
0.0 to 0.5m	>4.0m

DEPTHS SHOWN ARE BETWEEN THE EXISTING SURFACE AND THE FINAL SUBGRADE SURFACE

ASSUMPTIONS:

ROAD PAVEMENT	350mm
FOUNDATION DEPTH	1000mm
OTHER AREAS	300mm

NOTES

1. SURVEY INFORMATION SUPPLIED BY AURECON:
   LEVELS IN TERMS OF WELLINGTON 1953 DATUM (MSL)
   SITE BENCHMARK IR2, RL=166.85
   COORDINATES IN TERMS OF NZGD 2000 WELLINGTON CIRCUIT
   ORIGIN - SS 17K24 SO 30955
   801579.521mN
   396967.337mE
2. ALL WORKS AND MATERIALS TO COMPLY WITH THE WCC STANDARDS, NZBC AND WOODS SPECIFICATIONS. ANY AMBIGUITY BETWEEN DRAWINGS AND STANDARDS SHALL BE REPORTED TO THE ENGINEER FOR CLARIFICATION.
3. IT IS THE RESPONSIBILITY OF THE CONTRACTOR TO CONFIRM THE LOCATION AND PROTECT EXISTING SERVICES DURING WORKS.

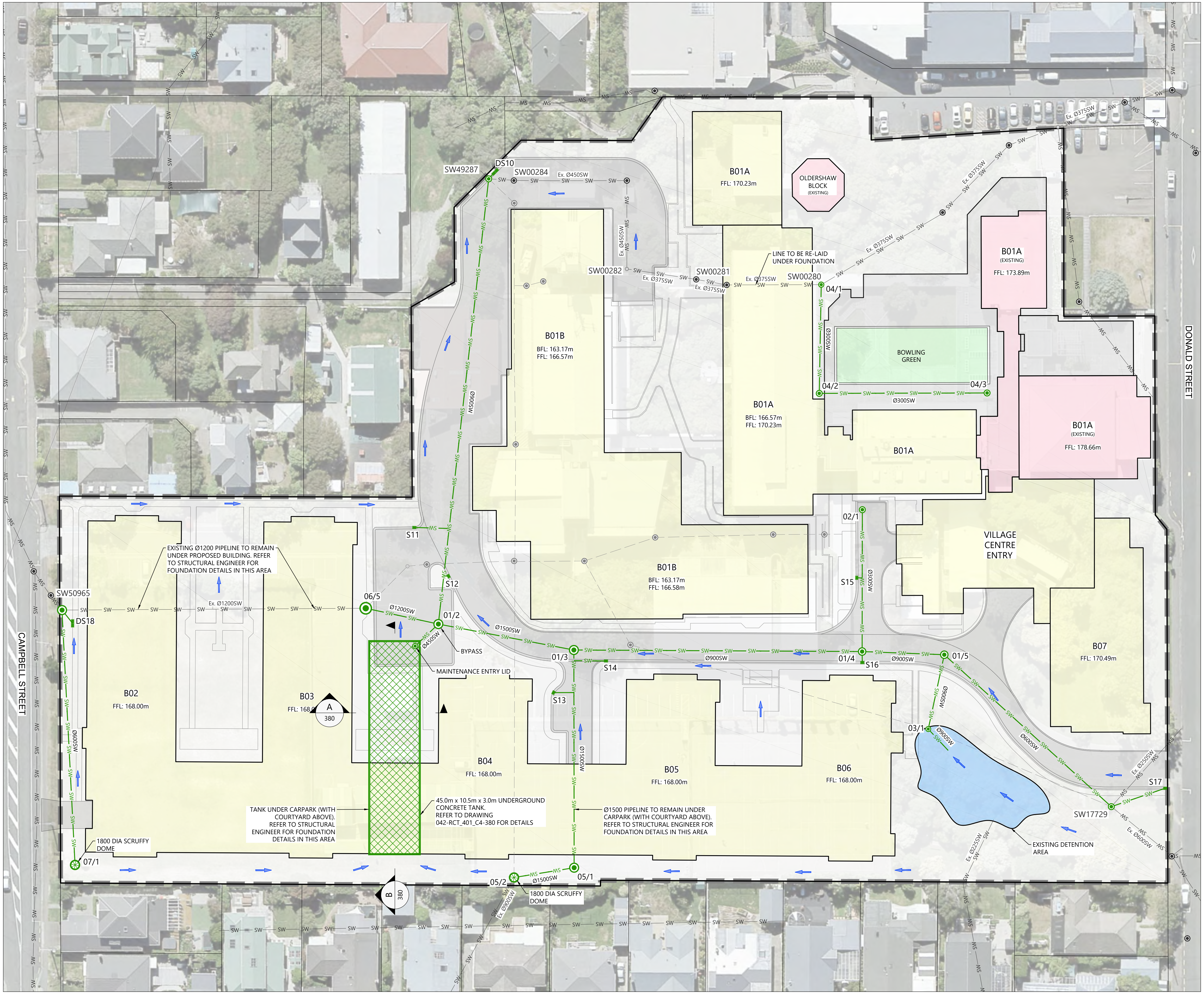
REVISION DETAILS		BY	DATE
1	ISSUED FOR CONSENT	WMV	14/08/20

SURVEYED	-	29 DONALD STREET KARORI WELLINGTON 6012
DESIGNED	JLS	
DRAWN	JLS	
CHECKED	MC	
APPROVED	MC	WOODS.CO.NZ

KARORI
RETIREMENT VILLAGE

DEPTH (CUT/FILL) CONTOUR PLAN

STATUS	ISSUED FOR CONSENT	REV
SCALE	1:750 @ A3	1
COUNCIL	WELLINGTON CITY COUNCIL	
DWG NO	042-RCT_401_C0-120	



WOODS
EST.1970

LEGEND

SITE BOUNDARY	
EXISTING BOUNDARY	
PROPOSED STORMWATER	— SW — SW — SW —
STORMWATER MANHOLE	01/1
STORMWATER SUMP	S1 DS1
SECONDARY FLOWPATH	
EXISTING STORMWATER TO REMAIN	— SW — SW —
EXISTING STORMWATER TO BE REMOVED	

NOTES

1. SURVEY INFORMATION SUPPLIED BY AURECON:
   LEVELS IN TERMS OF WELLINGTON 1953 DATUM (MSL)
   SITE BENCHMARK IR2, RL=166.85
   COORDINATES IN TERMS OF NZGD 2000 WELLINGTON CIRCUIT
   ORIGIN - SS 17K24 SO 30955
   801579.521mN
   396967.337mE
2. ALL WORKS AND MATERIALS TO COMPLY WITH THE WCC STANDARDS, NZBC AND WOODS SPECIFICATIONS. ANY AMBIGUITY BETWEEN DRAWINGS AND STANDARDS SHALL BE REPORTED TO THE ENGINEER FOR CLARIFICATION.
3. IT IS THE RESPONSIBILITY OF THE CONTRACTOR TO CONFIRM THE LOCATION AND PROTECT EXISTING SERVICES DURING WORKS.
4. ALL STORMWATER PIPES TO BE uPVC SN8 OR RCRRJ CLASS 4 TO AS/NZS:1260.

REVISION DETAILS		BY	DATE
1	FOR DISCUSSION	WMV	11/08/20
2	ISSUED FOR CONSENT	WMV	14/08/20

SURVEYED	-	29 DONALD STREET KARORI WELLINGTON 6012
DESIGNED	JLS	
DRAWN	JLS	
CHECKED	MC	
APPROVED	MC	WOODS.CO.NZ

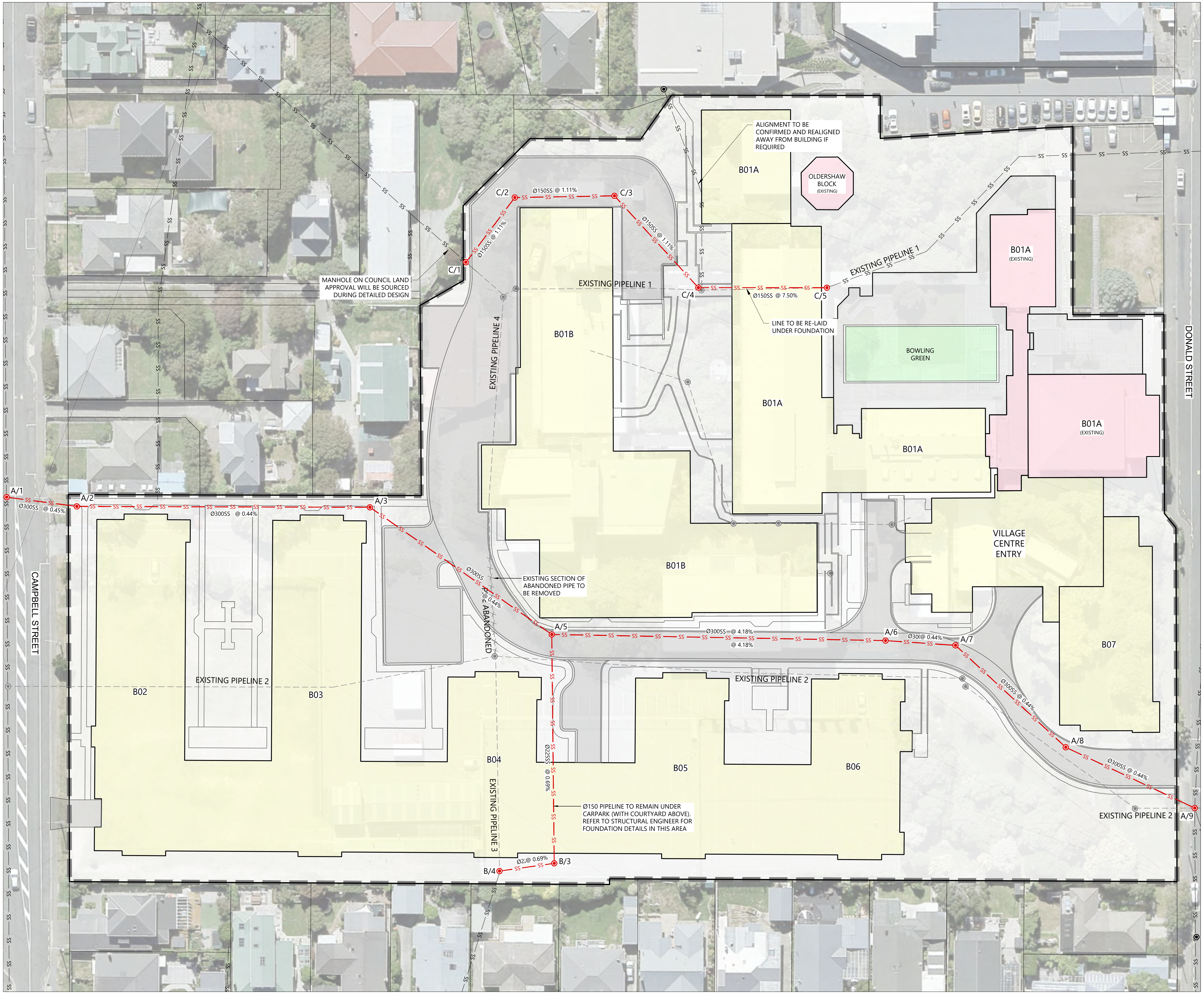
N

RYMAN HEALTHCARE

KARORI
RETIREMENT VILLAGE

STORMWATER DRAINAGE LAYOUT

STATUS	ISSUED FOR CONSENT	REV
SCALE	1:750 @ A3	2
COUNCIL	WELLINGTON CITY COUNCIL	
DWG NO	042-RCT_401_C0-300	



WOODS Est.1970

LEGEND

- STAGE BOUNDARY
- PROPOSED BOUNDARY
- EXISTING BOUNDARY
- EXISTING WASTEWATER TO REMAIN
- PROPOSED WASTEWATER (A/1)
- EXISTING WASTEWATER TO BE REMOVED

NOTES

1. SURVEY INFORMATION SUPPLIED BY AURECON:
   LEVELS IN TERMS OF WELLINGTON 1953 DATUM (MSL)
   SITE BENCHMARK IR2, RL=166.85
   COORDINATES IN TERMS OF NZGD 2000 WELLINGTON CIRCUIT
   ORIGIN - SS 17K24 SO 30955
   801579.521mN
   396967.337mE
2. ALL WORKS AND MATERIALS TO COMPLY WITH THE WCC STANDARDS, NZBC AND WOODS SPECIFICATIONS. ANY AMBIGUITY BETWEEN DRAWINGS AND STANDARDS SHALL BE REPORTED TO THE ENGINEER FOR CLARIFICATION.
3. IT IS THE RESPONSIBILITY OF THE CONTRACTOR TO CONFIRM THE LOCATION AND PROTECT EXISTING SERVICES DURING WORKS.
4. ALL PVC PIPES TO BE uPVC SN8 TO AS/NZS:1260

REVISION DETAILS		BY	DATE
1	FOR DISCUSSION	JLS	30/07/20
2	ISSUED FOR CONSENT	WMV	14/08/20

SURVEYED	-	29 DONALD STREET KARORI WELLINGTON 6012
DESIGNED	JLS	
DRAWN	JLS	
CHECKED	MC	
APPROVED	MC	WOODS.CO.NZ

N

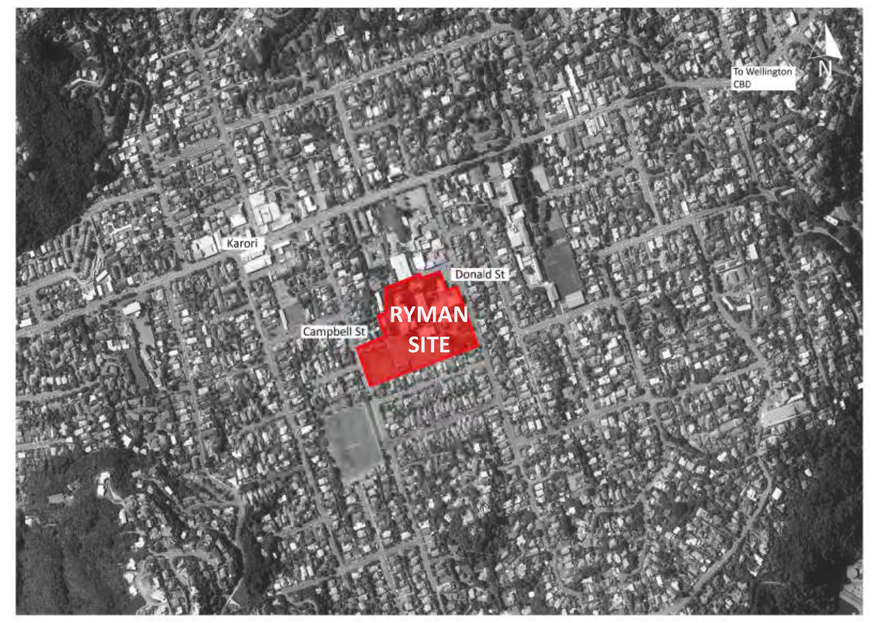
RYMAN HEALTHCARE

KARORI
RETIREMENT VILLAGE

WASTEWATER DRAINAGE LAYOUT

STATUS	ISSUED FOR CONSENT	REV
SCALE	1:750 @ A3	2
COUNCIL	WELLINGTON CITY COUNCIL	
DWG NO	042-RCT_401_C0-400	

NORTH



SITE INFORMATION

SITE AREA:	3.056ha (30,563m²)
SITE ADDRESS:	26 DONALD STREET, KARORI, WELLINGTON & 37 CAMPBELL ST, KARORI
LEGAL DESCRIPTION:	SECTION 1 SURVEY OFFICE PLAN 28414 SECTION 2 SURVEY OFFICE PLAN 515832 COT's 790147, 812554, NA22A/355

LEGAL BOUNDARIES

FIRE EXIT

1 EXISTING SITE PLAN WITH AERIAL

A1 sheet scale = 1 : 500
A3 sheet scale is twice scale shown above

RYMAN HEALTHCARE LTD

92 RUSSLEY ROAD, CHRISTCHURCH, NEW ZEALAND
PH: 64 - 3 - 366 4069

COMPREHENSIVE CARE RETIREMENT VILLAGE - DONALD STREET, KARORI, WELLINGTON

AMENDMENTS:

A JULY 2020 RESOURCE CONSENT ISSUE

LOCATION:
DONALD STREET, KARORI, WELLINGTON

DRAWING TITLE:
EXISTING SITE PLAN WITH AERIAL

BLOCK NO.:	S01	AMENDMENT:	A
PROJECT NO.:	042	STAGE NO.:	RCT
PDF NAME:	042 - RCT _ S01 _ ..A0-11 _ A		
SCALE:	As indicated	DRAWING NO.:	
DRAWING STATUS: RCA04		..A0-11	

200mm
100mm
ORIGINAL SCALE
0mm

_8/07/2020 10:38:05 AM_
BIM 360://042 - Karori/042-RCT_A01_S01_SITE_OPTION_B.rvt

NORTH

SITE INFORMATION

SITE AREA: 3.056ha (30,563m²)

SITE ADDRESS: 26 DONALD STREET, KARORI, WELLINGTON & 37 CAMPBELL ST, KARORI

LEGAL DESCRIPTION: SECTION 1 SURVEY OFFICE PLAN 28414 SECTION 2 SURVEY OFFICE PLAN 515832 COT's 790147, 812554, NA22A/355

BUILDING REFERENCES:

B01A & B01B VC or MAIN VILLAGE CENTRE
ALS = ASSISTED LIVING SUITES
CARE ROOMS

B02-B07 IA - INDEPENDENT APARTMENTS

SITE NOTES:

THIS SITE PLAN IS TO BE READ IN CONJUNCTION WITH THE CIVIL ENGINEERS PLANS & DETAILS

TREES & OTHER LANDSCAPING FEATURES ARE INDICATIVE ONLY, REFER TO THE LANDSCAPE PLANS & SCHEDULES

LEGAL BOUNDARIES

FIRE EXIT

NOTES:

- OVERALL VILLAGE WASTE MANAGEMENT STRATEGY TO BE DEVELOPED IN LATER STAGES BUT GENERALLY OPERATES AS FOLLOWS:
- **VILLAGE CENTRE** - ALL WASTE TO BE TRANSPORTED TO THE WASTE STORE BY STAFF
- **APARTMENTS** - RESIDENTS DISPOSE OF WASTE IN BINS LOCATED IN DEDICATED BIN ROOMS WITHIN UNDERCROFT PARKING AREA. STAFF TO TRANSFER BINS TO WASTE STORE OR TO REFUSE COLLECTION ZONES
- **ALS & CARE** - STAFF TO TRANSFER WASTE TO WASTE STORE

PROPOSED SITE PLAN WITH AERIAL

A1 sheet scale = 1 : 500
A3 sheet scale is twice scale shown above

200mm
100mm
0mm
ORIGINAL SCALE

RYMAN HEALTHCARE LTD

92 RUSSLEY ROAD, CHRISTCHURCH, NEW ZEALAND
PH: 64 - 3 - 366 4069

COMPREHENSIVE CARE RETIREMENT VILLAGE - DONALD STREET, KARORI, WELLINGTON

AMENDMENTS:		
A	JULY 2020	RESOURCE CONSENT ISSUE

LOCATION:
DONALD STREET, KARORI, WELLINGTON

DRAWING TITLE:
PROPOSED SITE PLAN WITH AERIAL

BLOCK NO.:	S01	AMENDMENT:	A
PROJECT NO.:	042	STAGE NO.:	RCT
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SCALE:	As indicated	DRAWING NO.:	
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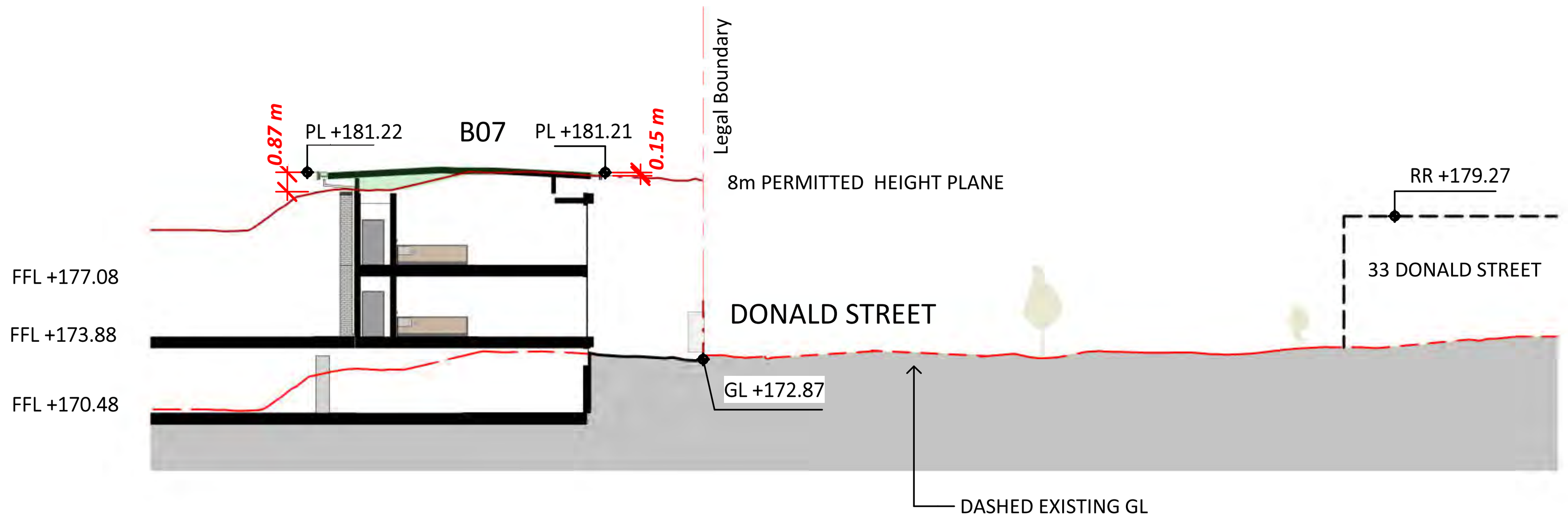

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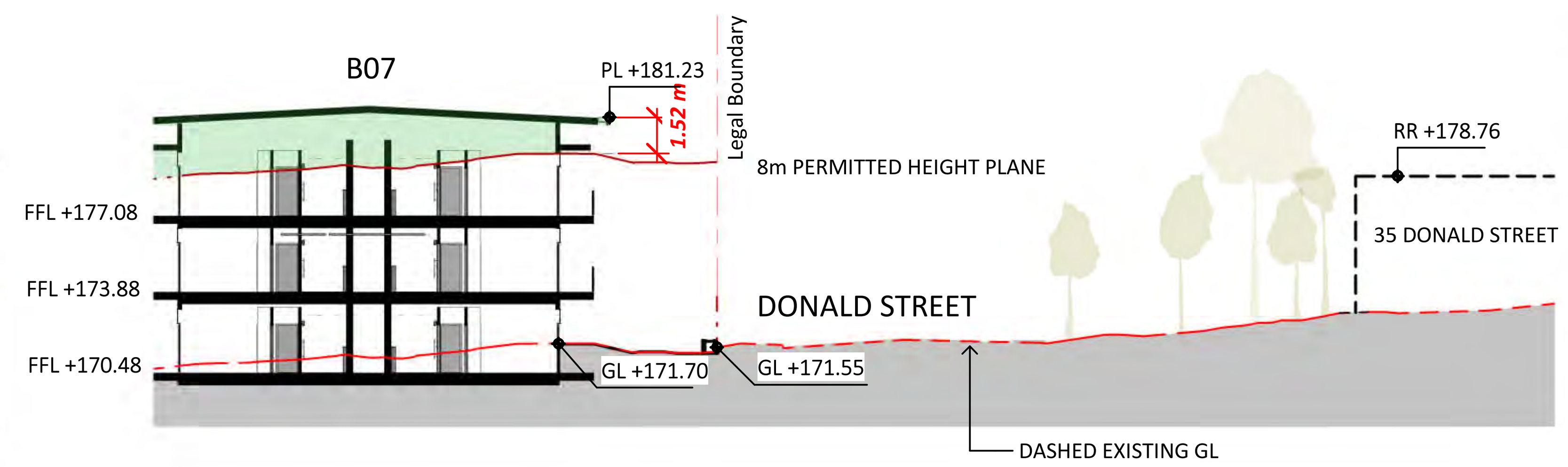

2 **35 Donald Street HIRB SECTION**
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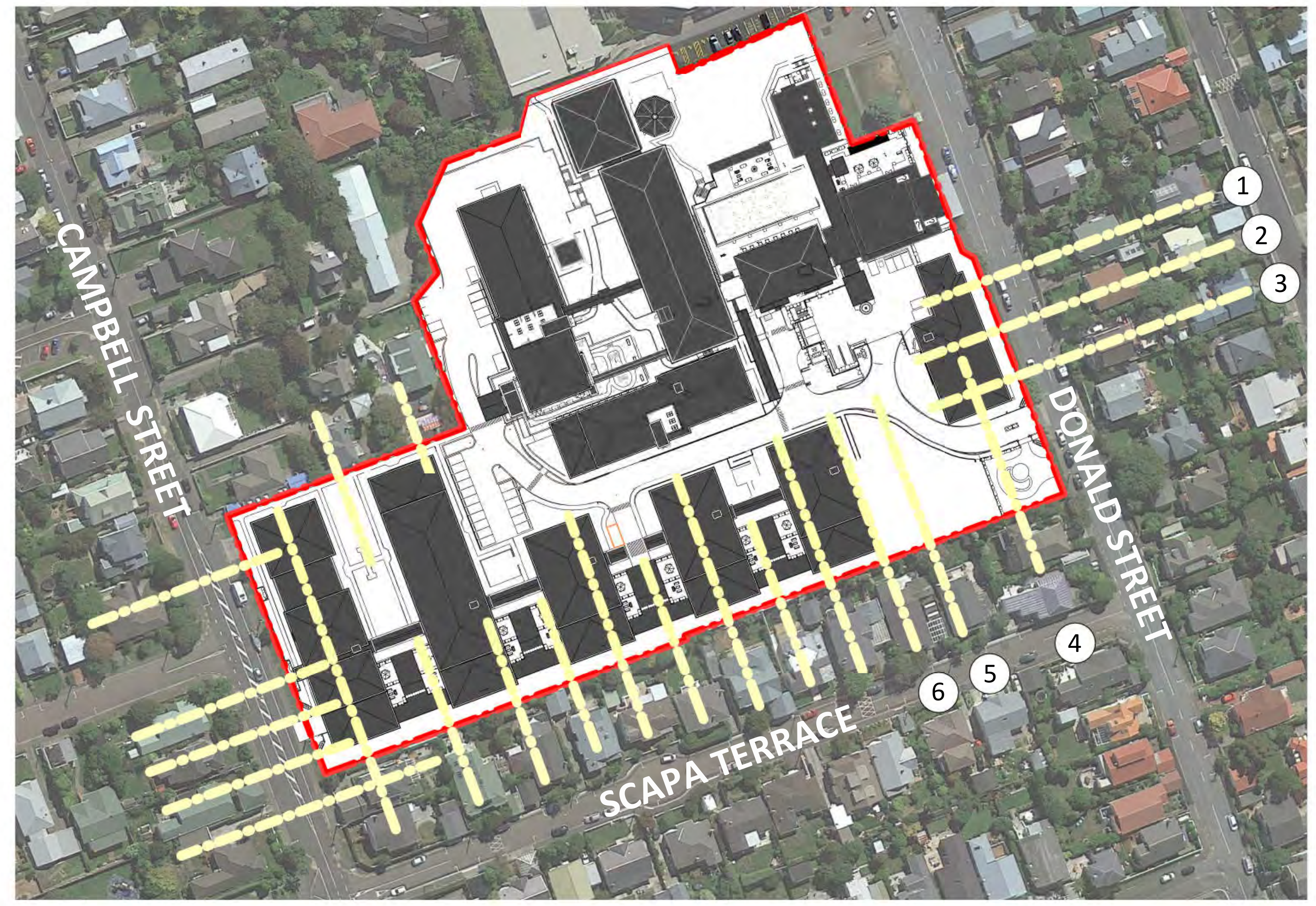


SITE PLAN LEGEND 1:1500

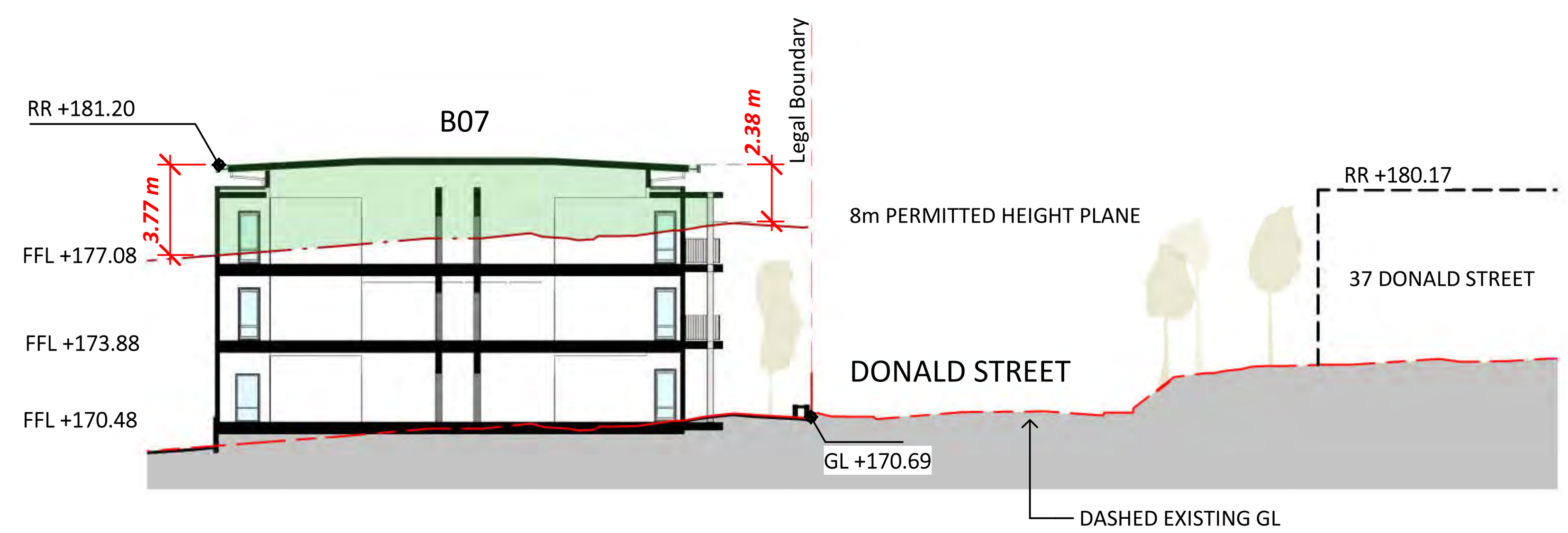
THIS SHEET TO BE READ WITH SEPARATE SHEET **A0-51**

SURVEY DATA:
GROUND LEVELS, TREES & DASHED LINES INDICATING EXISTING NEIGHBOURING PROPERTIES (INCL. ROOF HEIGHTS,) & GROUND LEVELS ARE TAKEN FROM LIDAR DATA SUPPLIED BY AURECON. LEVELS OF ACCURACY ARE AS NOTED ON AURECONS SURVEY DATA, REFER TO ASSESSMENT PACK.
REFERENCES INDICATED:
RR = RIDGE LEVEL (AS PER SURVEY)
PL = PROPOSED LEVEL
FFL = PROPOSED FINISHED FLOOR LEVEL
GL = GROUND LEVEL

1 SECTION REF. NUMBER THIS SHEET

NOTES:

SECTIONS OF PROPOSED BUILDINGS OUTSIDE THE 8m HEIGHT LIMIT



3 **37 Donald Street HIRB SECTION**
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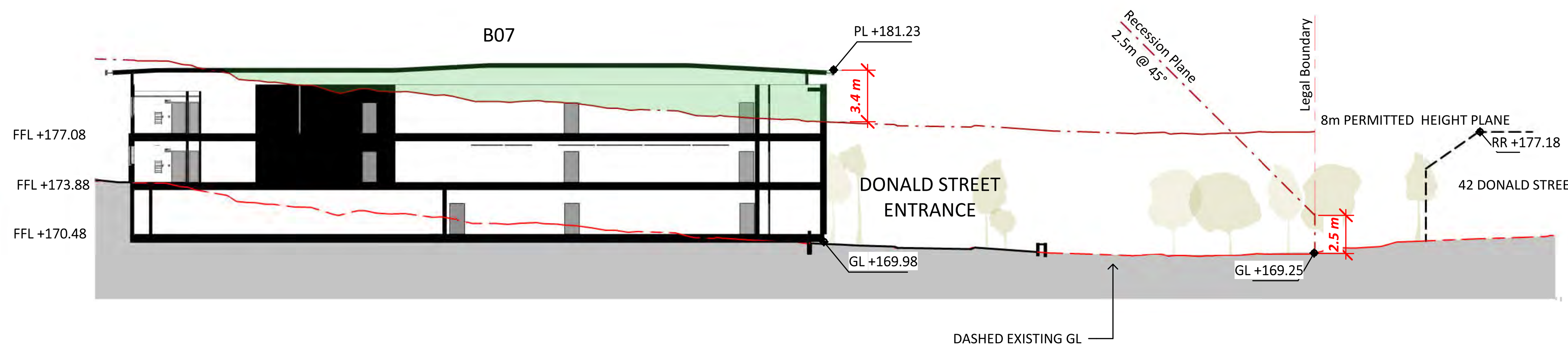
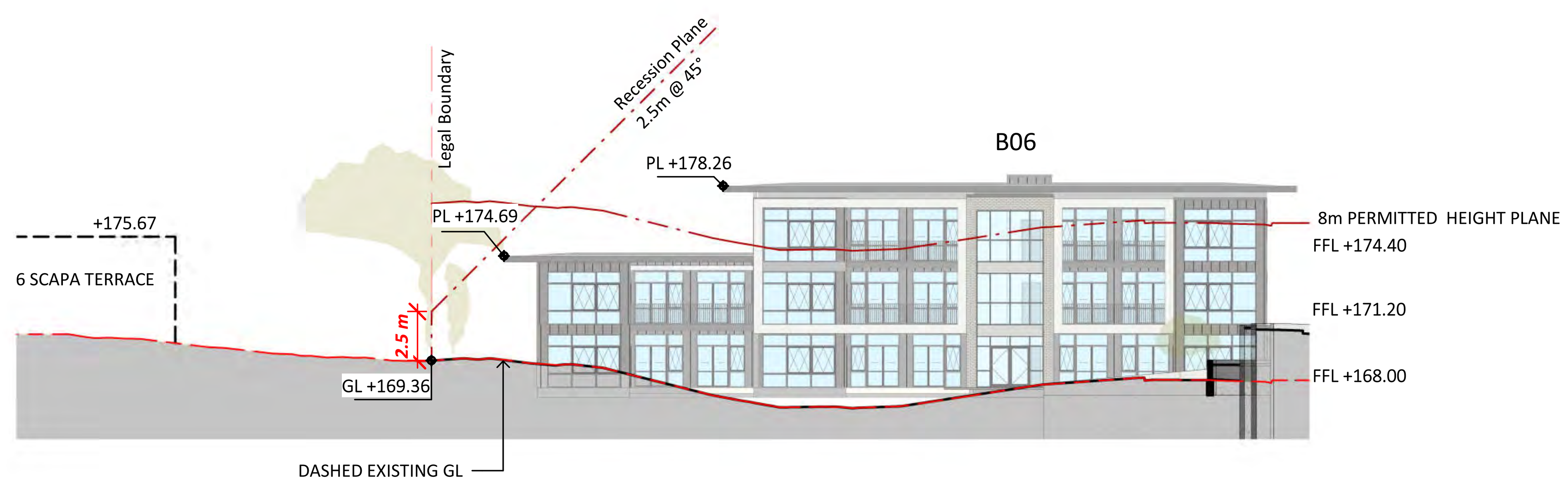


4 **42 Donald Street HIRB SECTION**
A0-51 A1 sheet scale = 1 : 250



5 **6 Scapa Terrace HIRB SECTION**
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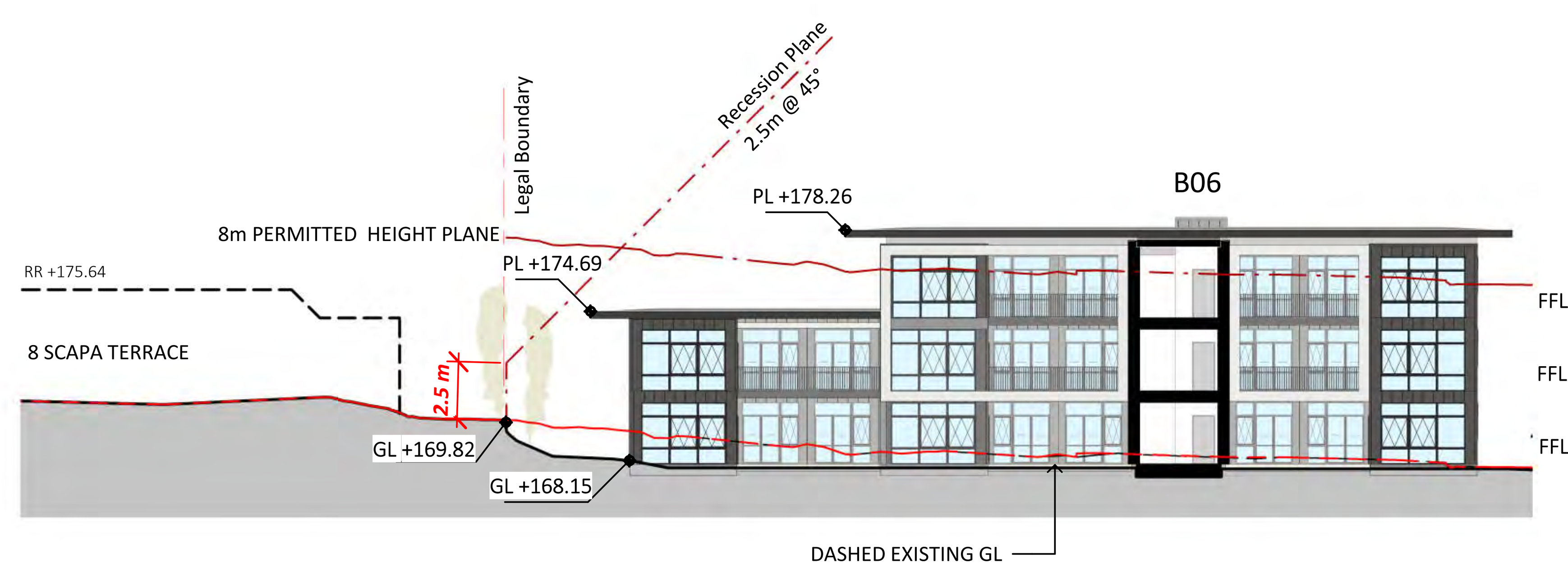

6 **8 Scapa Terrace HIRB SECTION**
A0-51 A1 sheet scale = 1 : 250

200mm
100mm
ORIGINAL SCALE
0mm

RYMAN HEALTHCARE LTD

92 RUSSLEY ROAD, CHRISTCHURCH, NEW ZEALAND
PH: 64 - 3 - 366 4069

COMPREHENSIVE CARE RETIREMENT VILLAGE - DONALD STREET, KARORI, WELLINGTON

AMENDMENTS:
A JULY 2020 RESOURCE CONSENT ISSUE

LOCATION:
DONALD STREET, KARORI, WELLINGTON

DRAWING TITLE:
SITE - CROSS SECTIONS

BLOCK NO.:	S01	AMENDMENT:	A
PROJECT NO.:	042	STAGE NO.:	RCT
PDF NAME:	042 - RCT _ S01 _ ..A0-52 _ A		
SCALE:	As indicated	DRAWING NO.:	
DRAWING STATUS: RCA14			..A0-52

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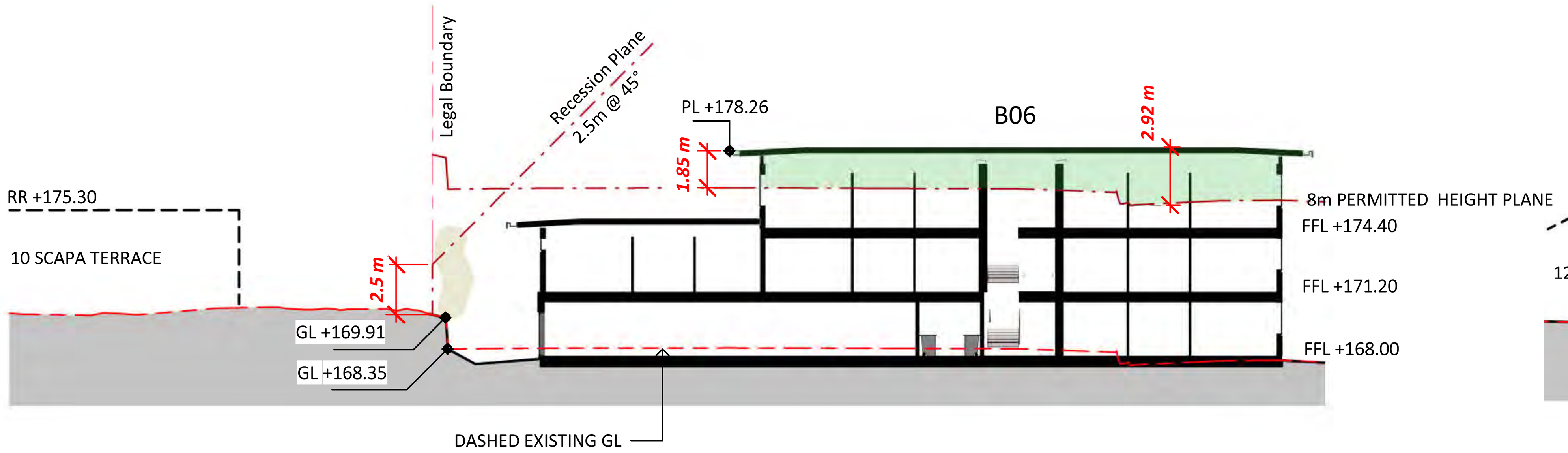
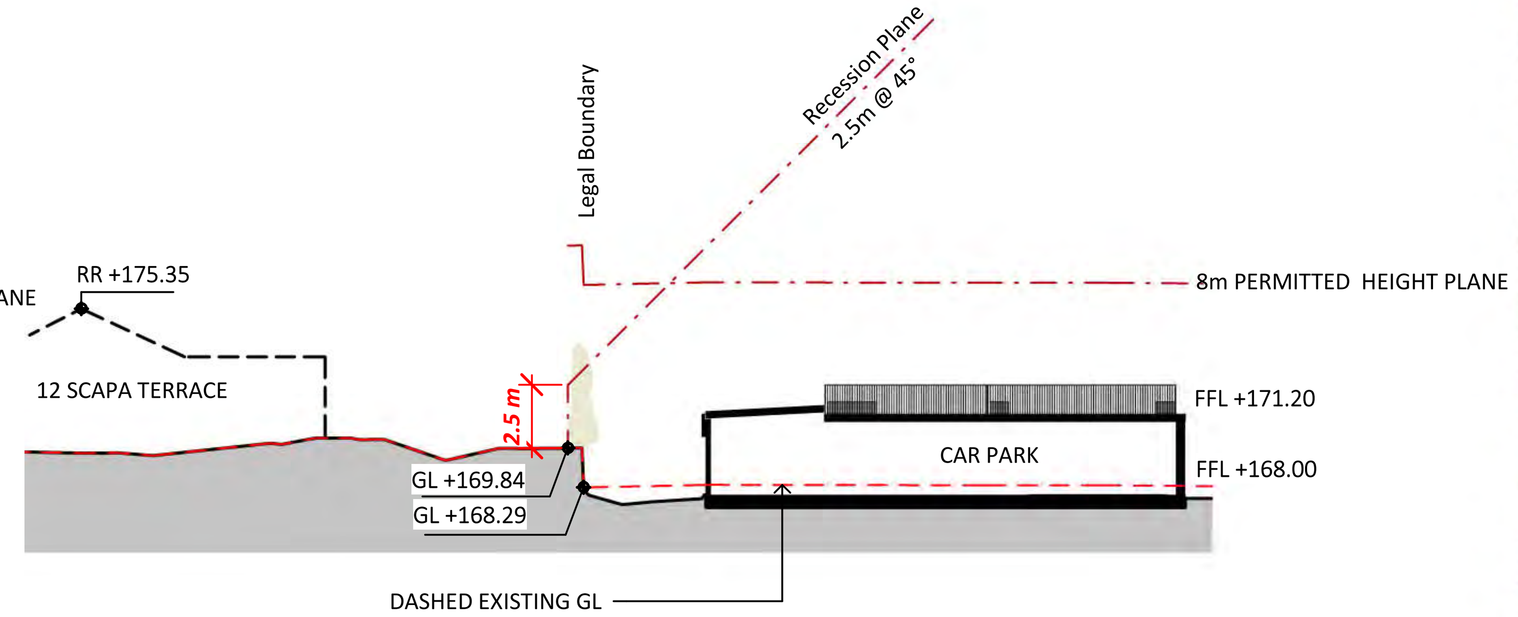


1 10 Scapa Terrace HIRB SECTION
A1 sheet scale = 1 : 250



2 12 Scapa Terrace HIRB SECTION
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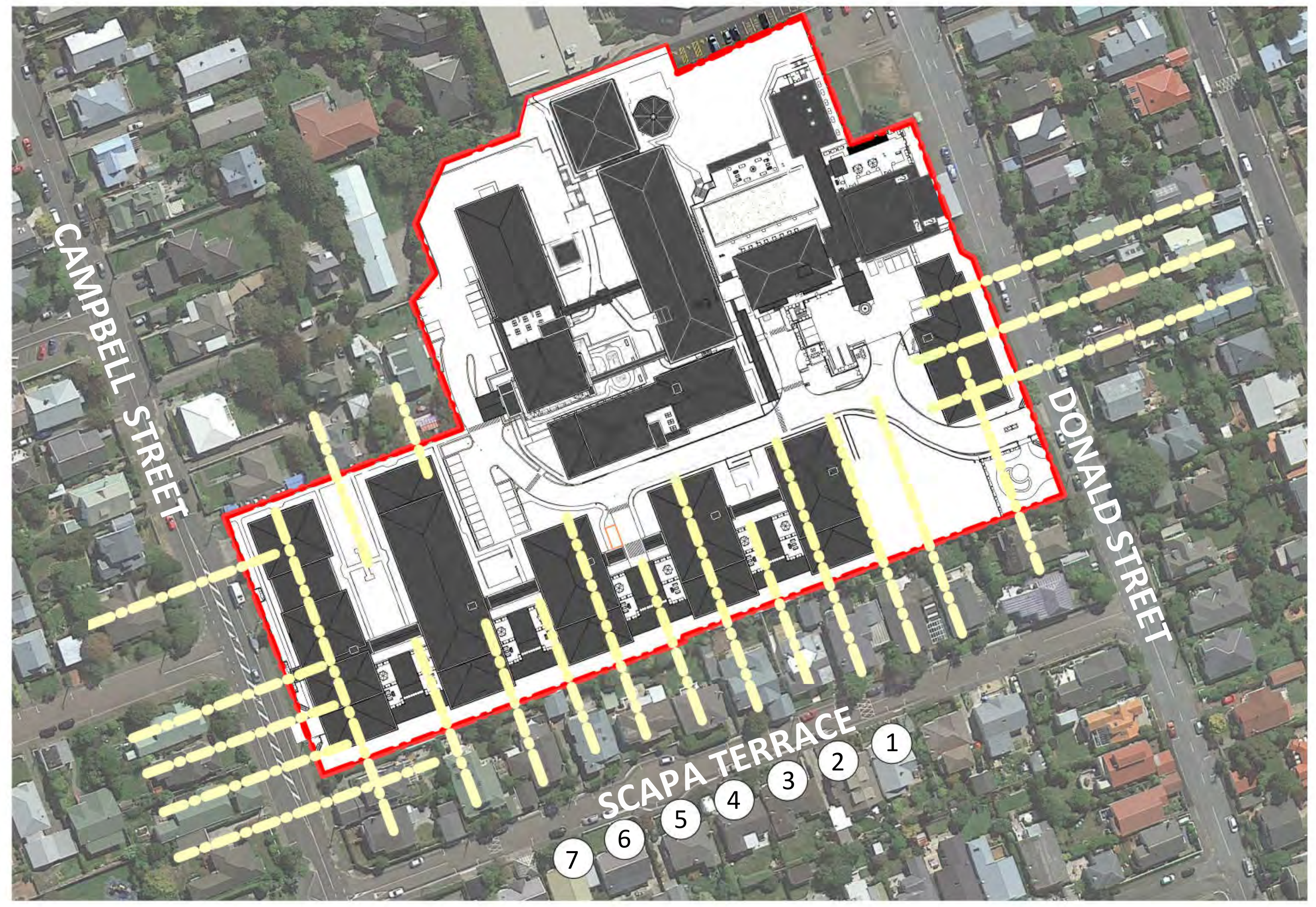


SITE PLAN LEGEND 1:1500

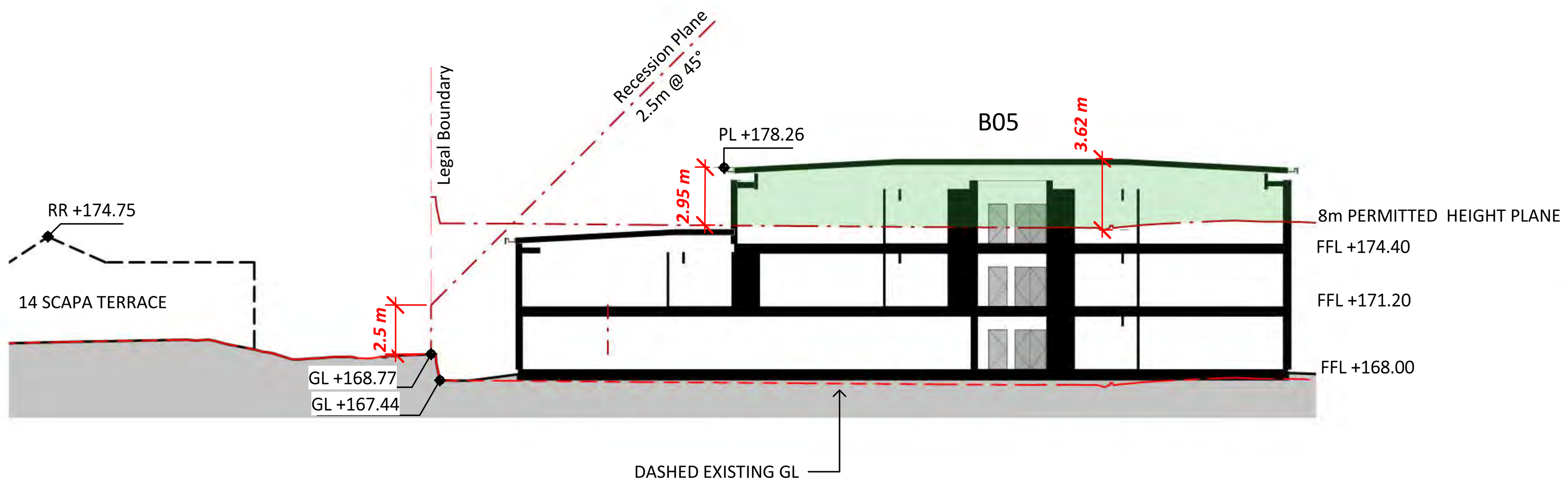
THIS SHEET TO BE READ WITH SEPARATE **SHEET A0-51**

SURVEY DATA:
GROUND LEVELS, TREES & DASHED LINES INDICATING EXISTING NEIGHBOURING PROPERTIES (INCL. ROOF HEIGHTS,) & GROUND LEVELS ARE TAKEN FROM LIDAR DATA SUPPLIED BY AURECON. LEVELS OF ACCURACY ARE AS NOTED ON AURECONS SURVEY DATA, REFER TO ASSESSMENT PACK.
REFERENCES INDICATED:
RR = RIDGE LEVEL (AS PER SURVEY)
PL = PROPOSED LEVEL
FFL = PROPOSED FINISHED FLOOR LEVEL
GL = GROUND LEVEL

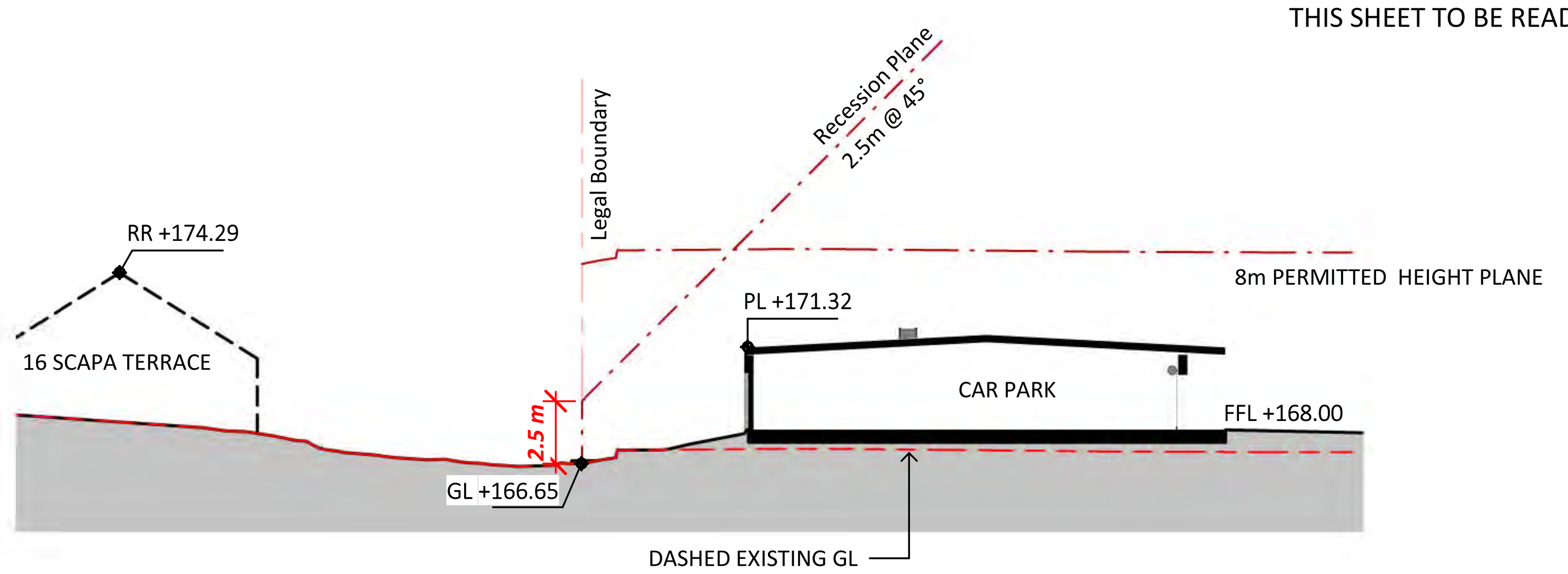
1 SECTION REF. NUMBER THIS SHEET

NOTES:

SECTIONS OF PROPOSED BUILDINGS OUTSIDE THE 8m HEIGHT LIMIT



3 14 Scapa Terrace HIRB SECTION
A1 sheet scale = 1 : 250



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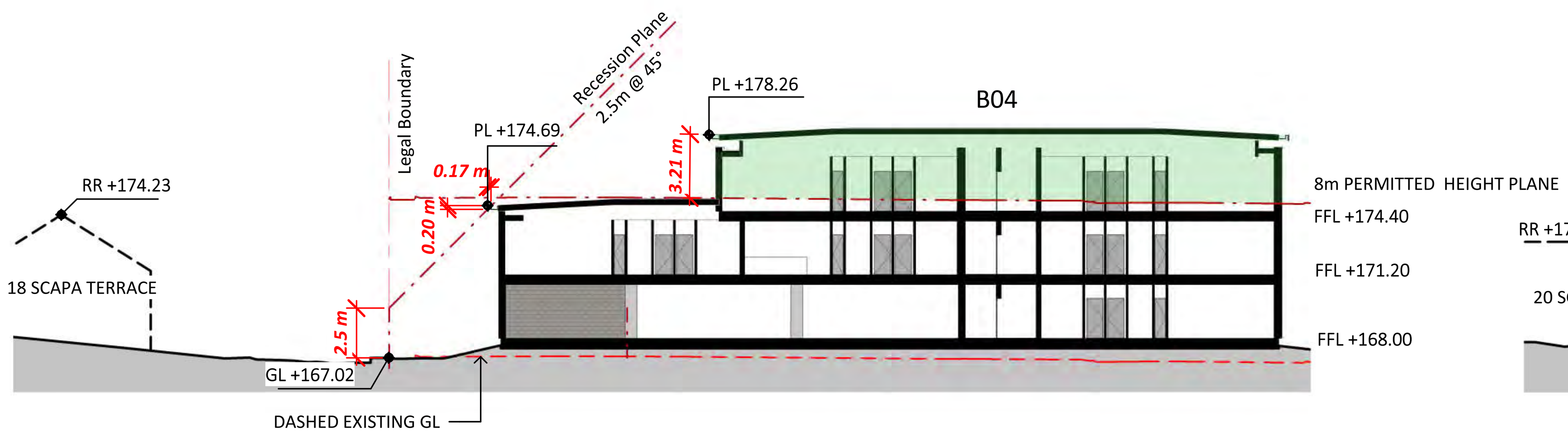

5 18 Scapa Terrace HIRB SECTION
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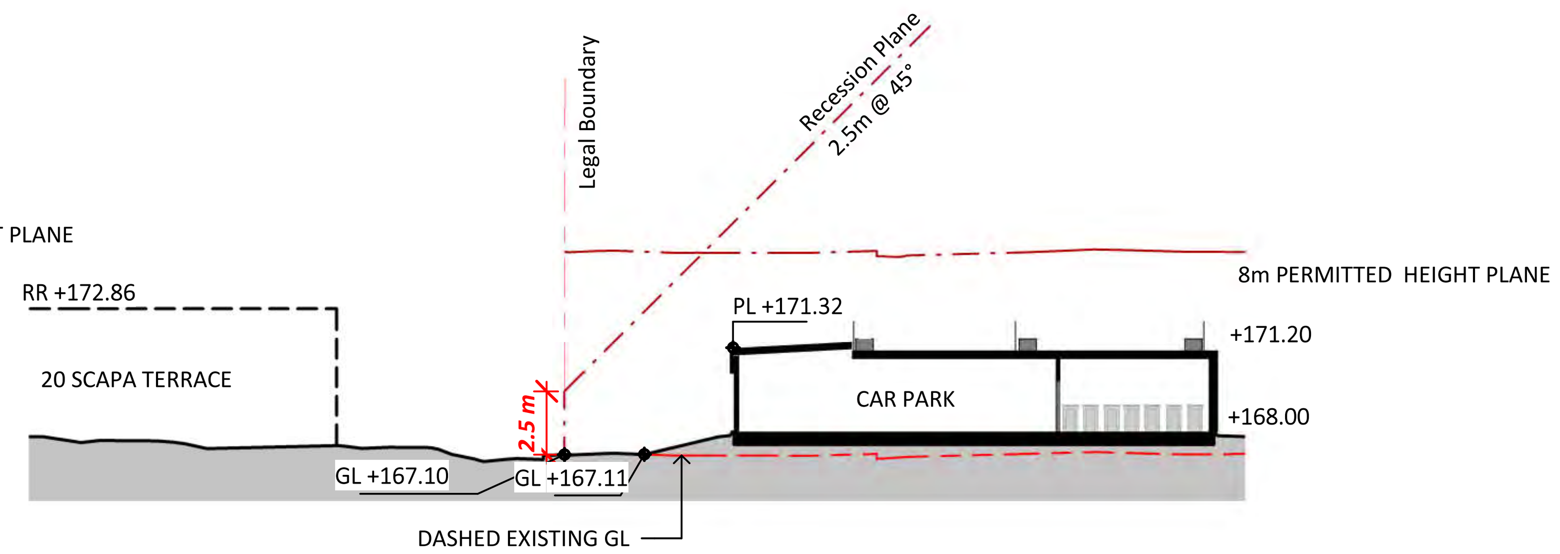
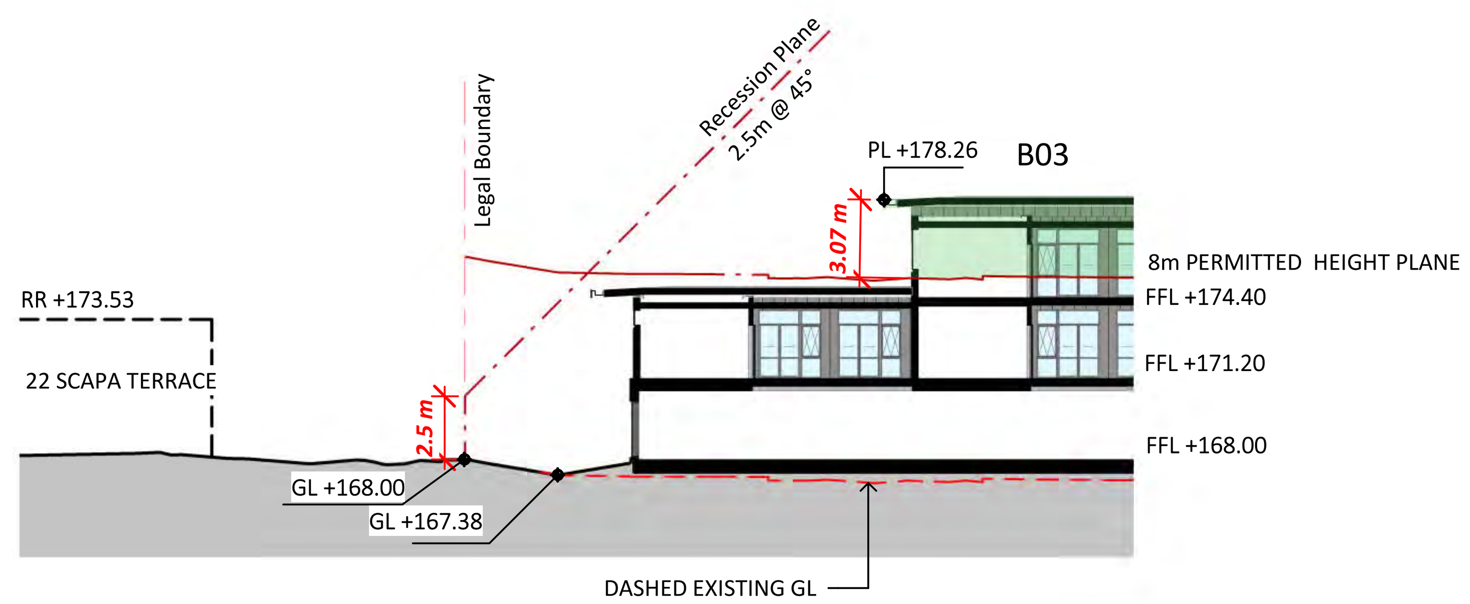


6 20 Scapa Terrace HIRB SECTION
A1 sheet scale = 1 : 250



7 22 Scapa Terrace HIRB SECTION
A1 sheet scale = 1 : 250

200mm
100mm
ORIGINAL SCALE
0mm

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COMPREHENSIVE CARE RETIREMENT VILLAGE - DONALD STREET, KARORI, WELLINGTON

AMENDMENTS:
A JULY 2020 RESOURCE CONSENT ISSUE

LOCATION:
DONALD STREET, KARORI, WELLINGTON

DRAWING TITLE:
SITE - CROSS SECTIONS

BLOCK NO.:	S01	AMENDMENT:	A
PROJECT NO.:	042	STAGE NO.:	RCT
PDF NAME:	042 - RCT _ S01 _ ..A0-53 _ A		
SCALE:	As indicated	DRAWING NO.:	
DRAWING STATUS: RCA15		..A0-53	

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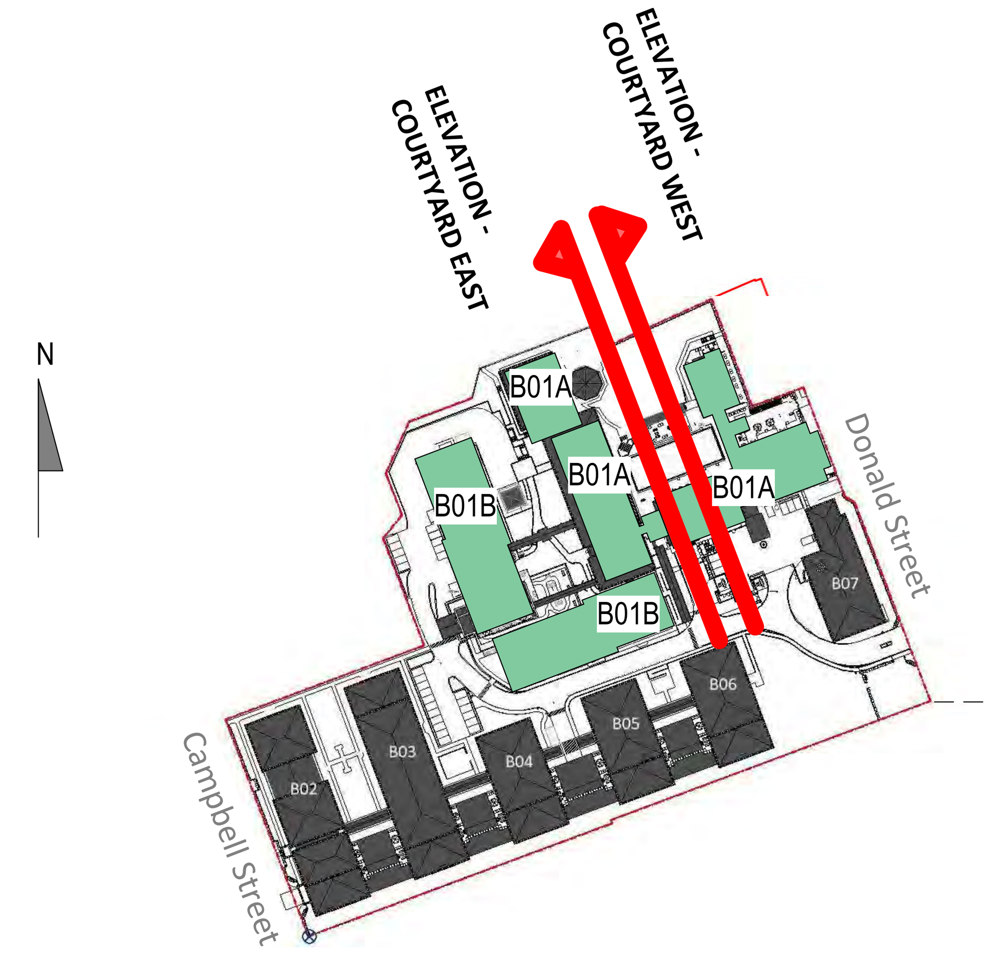
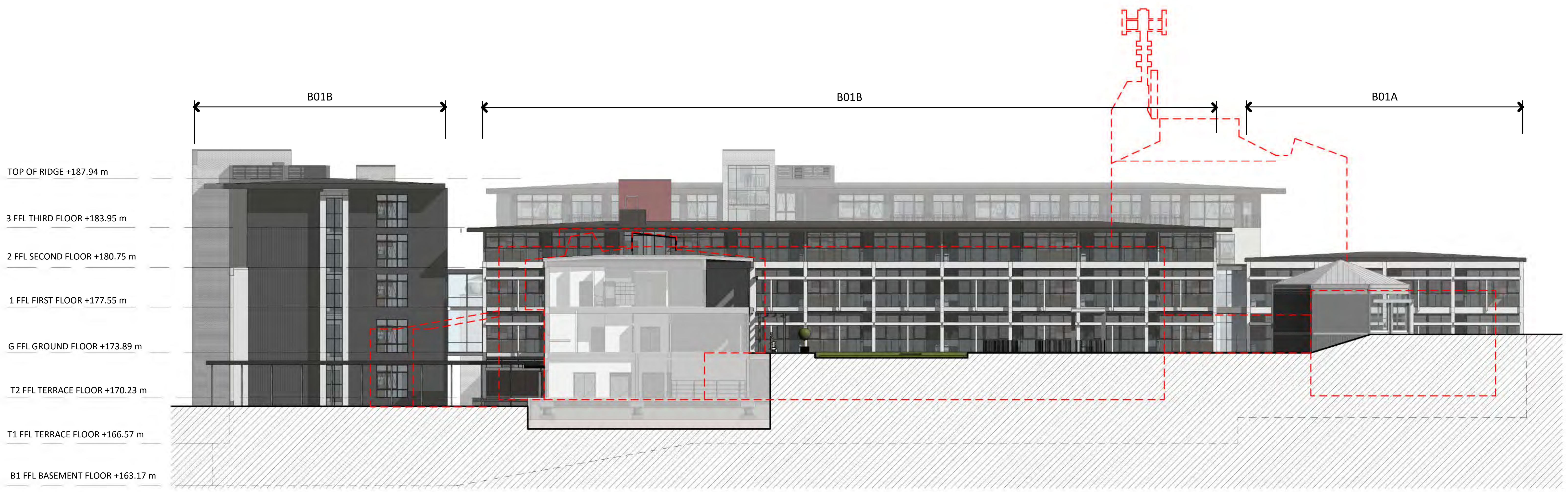
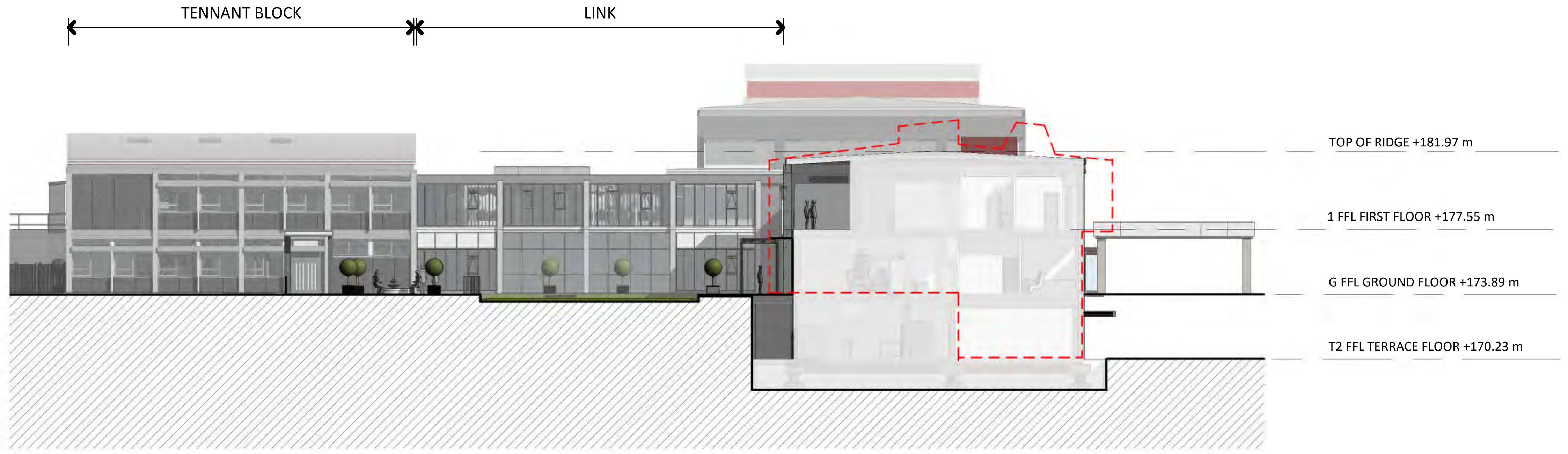




3 ELEVATION FROM COURTYARD EAST.

A1-080

A1 sheet scale = 1 : 250
A3 sheet scale is twice scale shown above



2 ELEVATION FROM COURTYARD WEST

A1-080

A1 sheet scale = 1 : 250
A3 sheet scale is twice scale shown above

200mm

100mm

0mm

ORIGINAL SCALE

SITE PLAN LEGEND NOT TO SCALE

MATERIALS

- Brick : Light Grey
- Vertical metal sheeting : Coloursteel - Ironsand
- Roof Fascia : Coloursteel - Ironsand
- Roof Finish : PTO Membrane - Ironsand
- Textured Plaster Finish : Winter Mushroom
- Concrete : Exposed precast concrete frame, mullions & hoods
- Exposed Panels : Coloured rough textured concrete or similar light weight panel
- External screens: PC Ironsand Frames/PC wood effect louvers
- Window & Door Joinery - PC Ironsand
- Glass to Balconies - Grey tinted
- Fascia : Smooth precast concrete or similar light weight panel
- Horizonal metal sheeting: Coloursteel - colour to match existing

Existing buildings
Refer to RC06 for further information

Indicating Basements

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COMPREHENSIVE CARE RETIREMENT VILLAGE - DONALD STREET, KARORI, WELLINGTON

AMENDMENTS:

A JULY 2020 RESOURCE CONSENT ISSUE

LOCATION:
DONALD STREET, KARORI, WELLINGTON

DRAWING TITLE:
PROPOSED VILLAGE CENTRE ELEVATIONS

BLOCK NO.:	B01	AMENDMENT:	A
PROJECT NO.:	042	STAGE NO.:	RCT
PDF NAME:	042 - RCT _ B01 _ .A2-010 _ A		
SCALE:	As indicated	DRAWING NO.:	
DRAWING STATUS:	RC26		.A2-010

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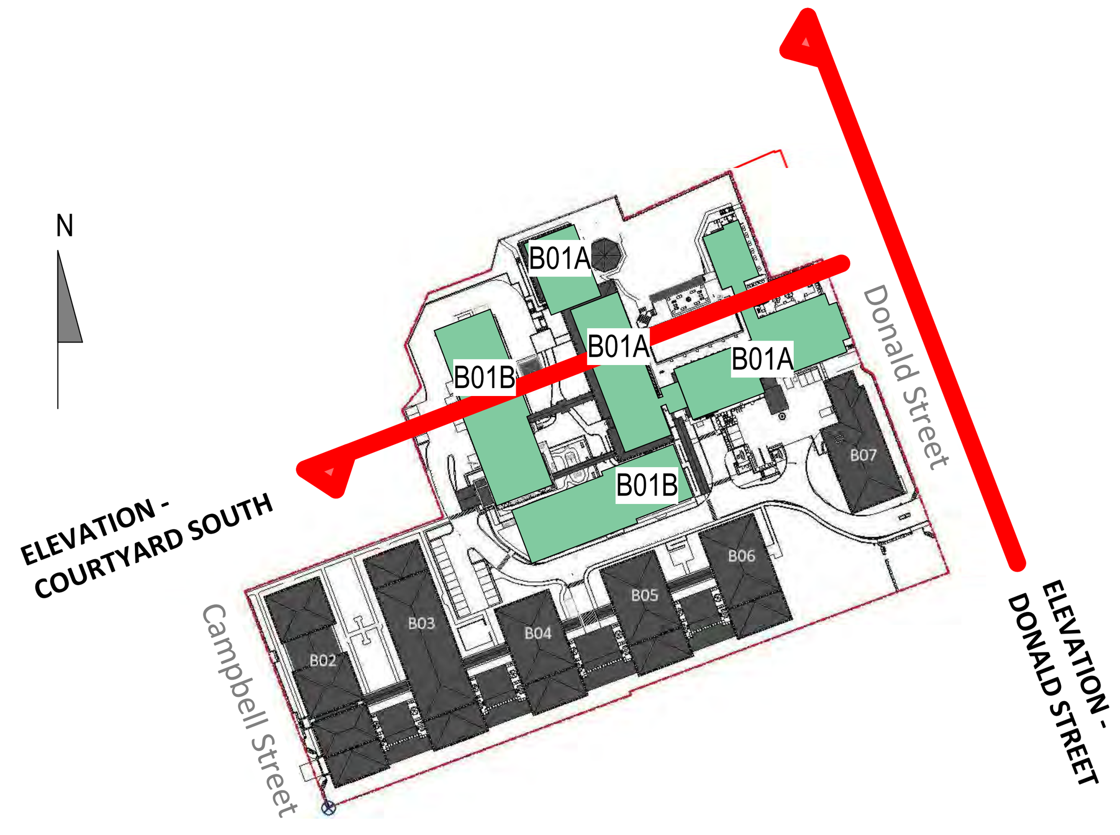
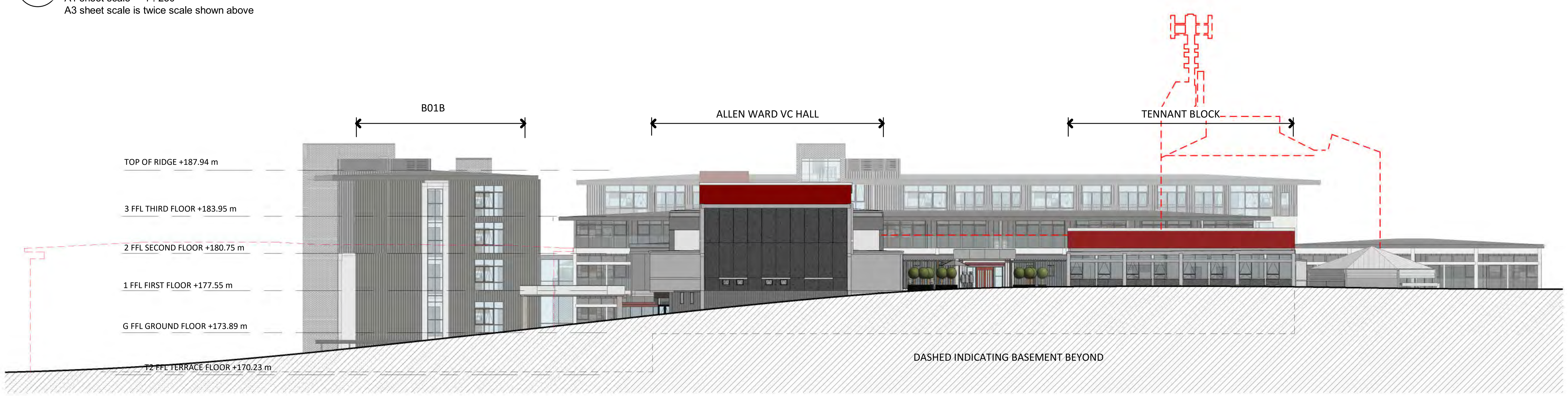


SITE PLAN LEGEND NOT TO SCALE

2 ELEVATION FROM COURTYARD NORTH

A1-080

A1 sheet scale = 1 : 250
A3 sheet scale is twice scale shown above



1 ELEVATION FROM DONALD STREET

A1-080

A1 sheet scale = 1 : 250
A3 sheet scale is twice scale shown above

MATERIALS

- Brick : Light Grey
- Vertical metal sheeting : Coloursteel - Ironsand
- Roof Fascia : Coloursteel - Ironsand
- Roof Finish : PTO Membrane - Ironsand
- Textured Plaster Finish : Winter Mushroom
- Concrete : Exposed precast concrete frame, mullions & hoods
- Exposed Panels : Coloured rough textured concrete or similar light weight panel
- External screens: PC Ironsand Frames/PC wood effect louvers
- Window & Door Joinery - PC Ironsand
- Glass to Balconies - Grey tinted
- Fascia : Smooth precast concrete or similar light weight panel
- Horizonal metal sheeting: Coloursteel - colour to match existing

Existing buildings
Refer to RC06 for further information

Indicating Basements

ORIGINAL SCALE 0mm

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COMPREHENSIVE CARE RETIREMENT VILLAGE - DONALD STREET, KARORI, WELLINGTON

AMENDMENTS:
A JULY 2020 RESOURCE CONSENT ISSUE

LOCATION:
DONALD STREET, KARORI, WELLINGTON

DRAWING TITLE:
PROPOSED VILLAGE CENTRE ELEVATIONS

BLOCK NO.:	B01	AMENDMENT:	A
PROJECT NO.:	042	STAGE NO.:	RCT
PDF NAME:	042 - RCT _ B01 _ .A2-020 _ A		
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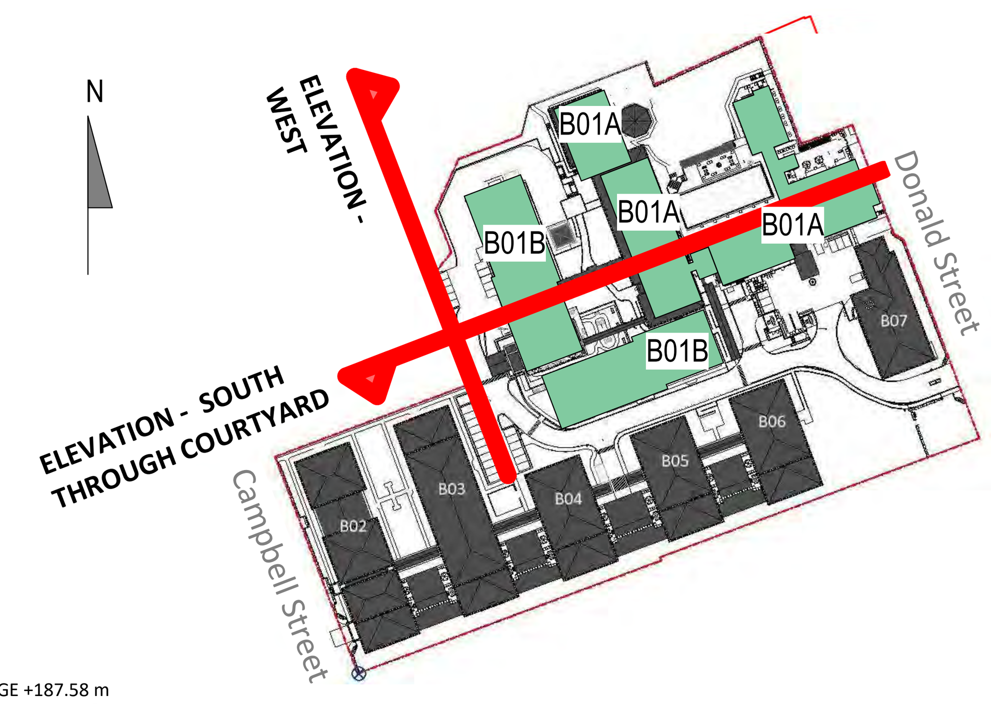
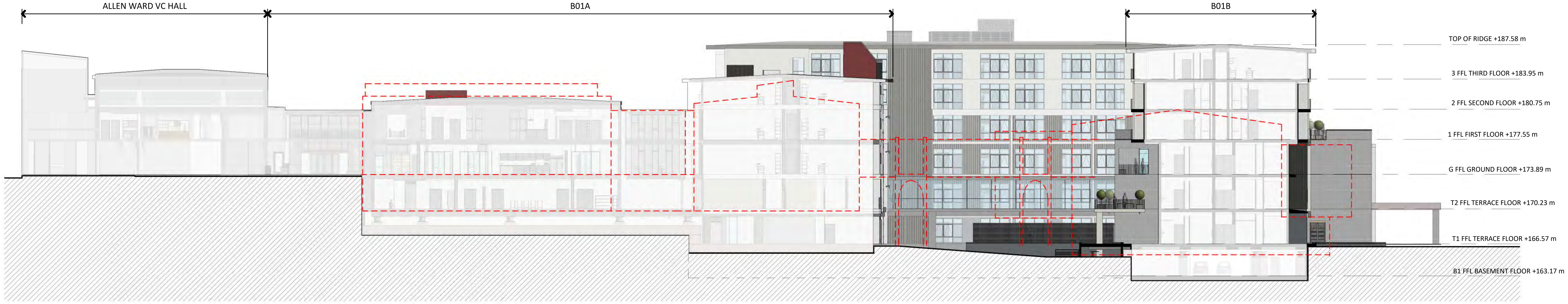
DRAWING STATUS:	RC27		.A2-020





1 ELEVATION - SOUTH THROUGH COURTYARD

A1-080

A1 sheet scale = 1 : 250
A3 sheet scale is twice scale shown above

2 ELEVATION WEST

A1-080

A1 sheet scale = 1 : 250
A3 sheet scale is twice scale shown above

200mm
100mm
0mm
ORIGINAL SCALE

SITE PLAN LEGEND NOT TO SCALE

MATERIALS

- Brick : Light Grey
- Vertical metal sheeting : Coloursteel - Ironsand
- Roof Fascia : Coloursteel - Ironsand
- Roof Finish : PTO Membrane - Ironsand
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- External screens: PC Ironsand Frames/PC wood effect louvers
- Window & Door Joinery - PC Ironsand
- Glass to Balconies - Grey tinted
- Fascia : Smooth precast concrete or similar light weight panel
- Horizonal metal sheeting: Coloursteel - colour to match existing

Existing buildings
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Indicating Basements

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COMPREHENSIVE CARE RETIREMENT VILLAGE - DONALD STREET, KARORI, WELLINGTON

AMENDMENTS:

A JULY 2020 RESOURCE CONSENT ISSUE

LOCATION:
DONALD STREET, KARORI, WELLINGTON

DRAWING TITLE:
PROPOSED VILLAGE CENTRE ELEVATIONS

BLOCK NO.:	B01	AMENDMENT:	A
PROJECT NO.:	042	STAGE NO.:	RCT
PDF NAME:	042 - RCT _ B01 _ .A2-030 _ A		
SCALE:	As indicated	DRAWING NO.:	
DRAWING STATUS:	RC28		.A2-030

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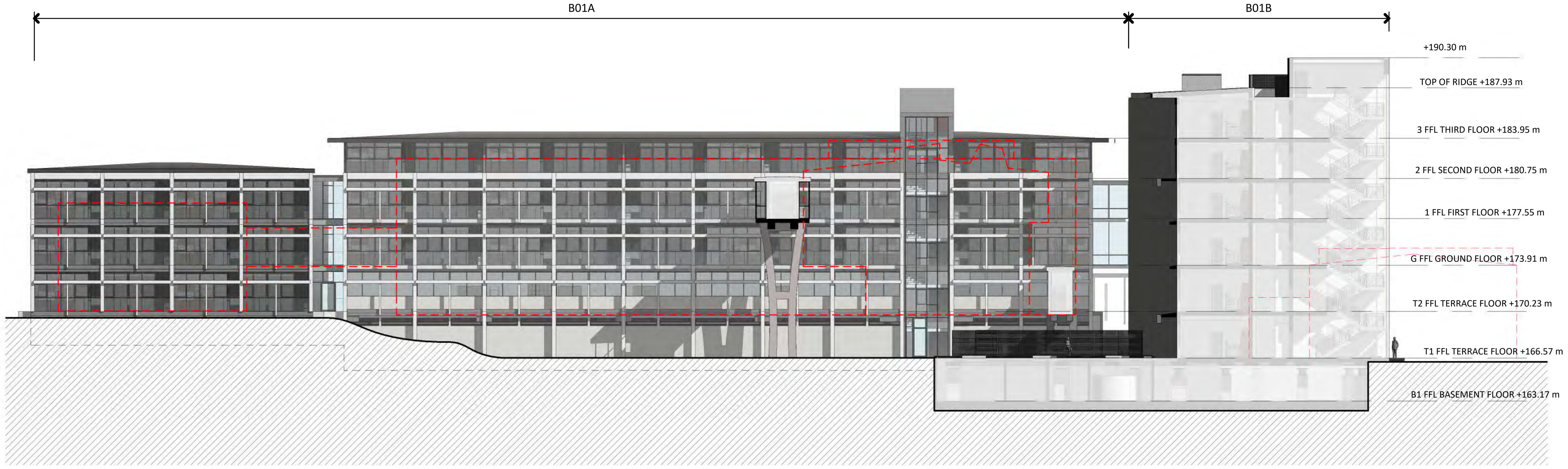
1 COURT YARD ELEVATION - EAST

A1-080

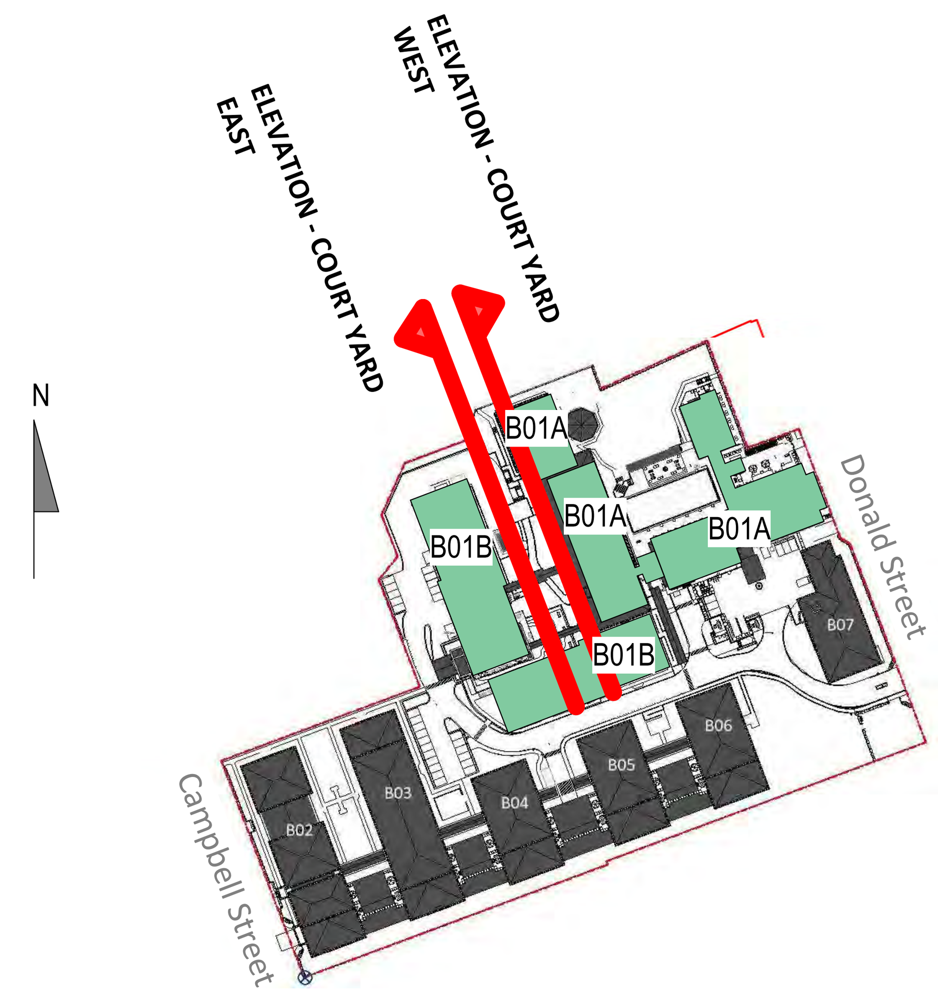
A1 sheet scale = 1 : 250
A3 sheet scale is twice scale shown above



2 ELEVATION FROM COURTYARD WEST.

A1-080

A1 sheet scale = 1 : 250
A3 sheet scale is twice scale shown above



SITE PLAN LEGEND NOT TO SCALE

MATERIALS

- Brick : Light Grey
- Vertical metal sheeting : Coloursteel - Ironsand
- Roof Fascia : Coloursteel - Ironsand
- Roof Finish : PTO Membrane - Ironsand
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- Window & Door Joinery - PC Ironsand
- Glass to Balconies - Grey tinted
- Fascia : Smooth precast concrete or similar light weight panel
- Horizonal metal sheeting: Coloursteel - colour to match existing

Existing buildings
Refer to RC06 for further information

Indicating Basements

200mm
100mm
ORIGINAL SCALE
0mm

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COMPREHENSIVE CARE RETIREMENT VILLAGE - DONALD STREET, KARORI, WELLINGTON

AMENDMENTS:

A JULY 2020 RESOURCE CONSENT ISSUE

LOCATION:

DONALD STREET, KARORI, WELLINGTON

DRAWING TITLE:

PROPOSED VILLAGE CENTRE ELEVATIONS

BLOCK NO.:	B01	AMENDMENT:	A
PROJECT NO.:	042	STAGE NO.:	RCT
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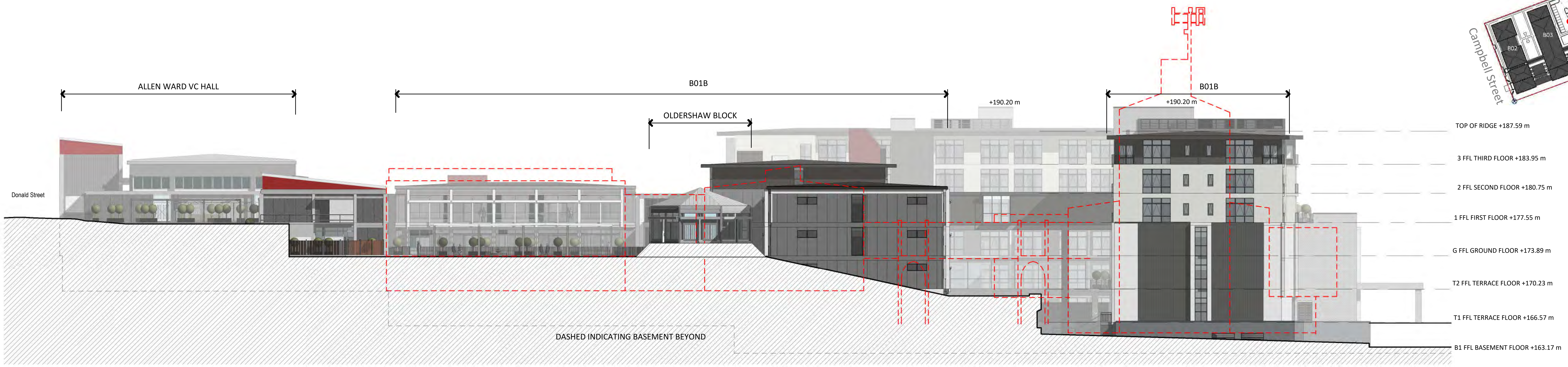
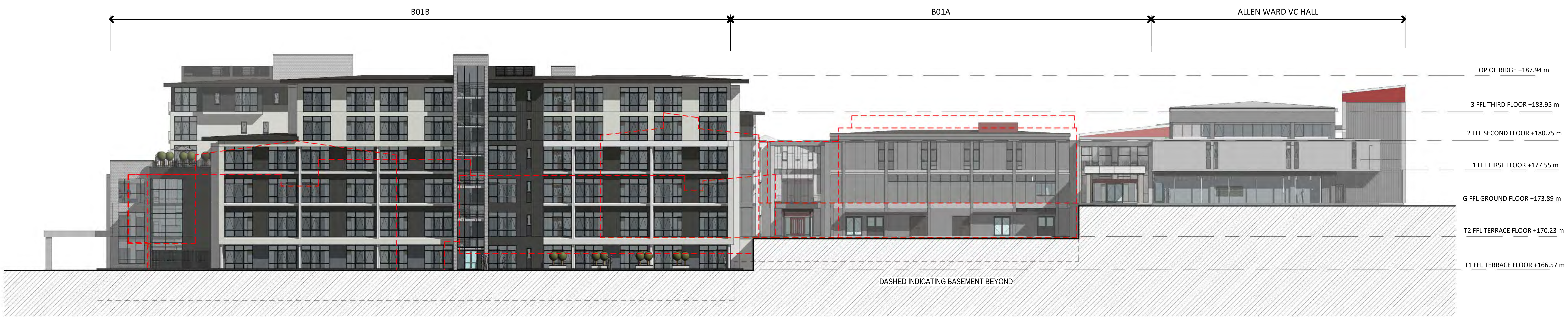


1 COURT YARD ELEVATION -NORTH

A1-080

A1 sheet scale = 1 : 250
A3 sheet scale is twice scale shown above



2 SOUTH ELEVATION ENTRANCE

A1-080

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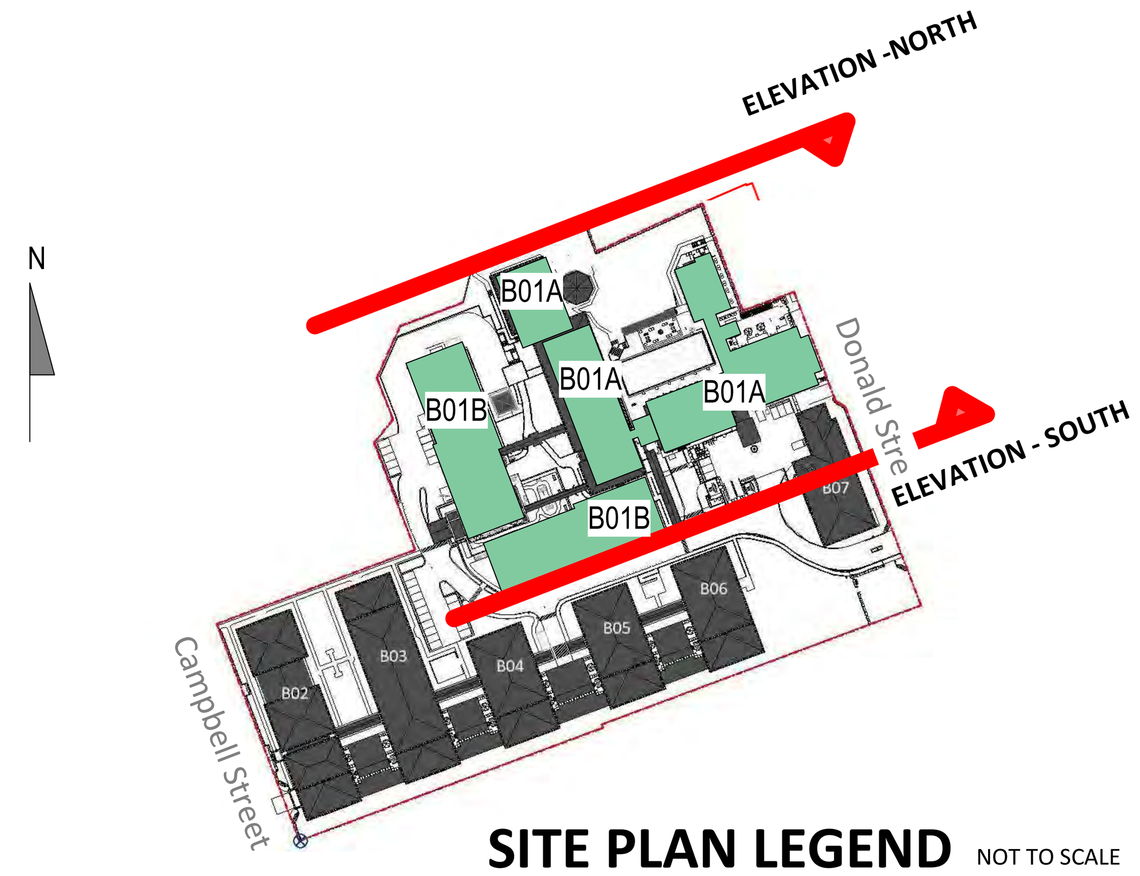

SITE PLAN LEGEND NOT TO SCALE

MATERIALS

- Brick : Light Grey
- Vertical metal sheeting : Coloursteel - Ironsand
- Roof Fascia : Coloursteel - Ironsand
- Roof Finish : PTO Membrane - Ironsand
- Textured Plaster Finish : Winter Mushroom
- Concrete : Exposed precast concrete frame, mullions & hoods
- Exposed Panels : Coloured rough textured concrete or similar light weight panel
- External screens: PC Ironsand Frames/PC wood effect louvers
- Window & Door Joinery - PC Ironsand
- Glass to Balconies - Grey tinted
- Fascia : Smooth precast concrete or similar light weight panel
- Horizontal metal sheeting: Coloursteel - colour to match existing

Existing buildings
Refer to RC06 for further information

Indicating Basements

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COMPREHENSIVE CARE RETIREMENT VILLAGE - DONALD STREET, KARORI, WELLINGTON

AMENDMENTS:
A JULY 2020 RESOURCE CONSENT ISSUE

LOCATION:
DONALD STREET, KARORI, WELLINGTON

DRAWING TITLE:
PROPOSED VILLAGE CENTRE ELEVATIONS

BLOCK NO.:	B01	AMENDMENT:	A
PROJECT NO.:	042	STAGE NO.:	RCT
PDF NAME:	042 - RCT _ B01 _ .A2-050 _ A		
SCALE:	As indicated	DRAWING NO.:	
DRAWING STATUS:	RC30		.A2-050

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Appendix B: Summary of ground investigations and geotechnical parameters

B1 General

Geotechnical design parameters included in this report are based on site investigations which comprised the following:

- Desk study of readily available information,
- 6 No. fully cored machine boreholes,
- 10 No. Cone Penetration Tests (CPTs), and
- Laboratory Atterberg limits testing of selected soil samples
- Measurement of groundwater levels in standpipe piezometers
- Rising and falling head tests to assess permeability

A summary of the site subsurface conditions is provided in the sections below.

B2 Subsurface Conditions

A published geological map for the site[11] shows the site to be is underlain by alluvial gravels underlain by "grey sandstone-mudstone sequence and poorly bedded sandstone" of the Rakaia Terrane, commonly referred to as greywacke (indurated sandstone) and argillite (indurated mudstone).

The project area is located within a fault bound valley (graben), which has been infilled by alluvial sediments. The valley is bounded by the Khandallah Fault around 500 m to the north, and the Wellington Fault around 1km to the south. A series of north – south trending normal faults crosses the graben approximately 600m to the west and east of the site.

B2.1 Results of Investigation

Geotechnical units encountered during the site investigation are summarised in Table B1 below.

[11] Begg, J.G., Mazengarb, C., 1996. Geology of the Wellington area. Scale 1:50,000. Institute of Geological & Nuclear Sciences geological map 22. 1 sheet + 128 p. Institute of Geological & Nuclear Sciences Ltd., Lower Hutt, New Zealand.

Table B1: Summary of encountered geotechnical units

Investigation Location	Fill	Silt and Recent Alluvium	Lower Alluvium	Colluvium (Outwash Gravel)	River Gravel	CW to HW Greywacke	MW Greywacke
BH01	-	1.6 – 3.75	3.75 – 9.0	-		9.0 – 15.5	15.5 – 20.25
BH02	0 – 0.25	-	0.25 – 14.25	-	14.25 – 16.5	-	-
BH03	0 – 0.45	0.45 – 1.6	-	1.6 – 29.0	25.5 – 29.0	-	-
BH04	0 – 0.5	0.5 – 3.8	-	-		3.8 – 5.0	5.0 – 10.5
BH05	-	0.9 – 5.0	5.0 – 9.5	9.5 – 16.5		-	-
BH06	-	0 – 6.8	-	-		-	6.8 - 10.5

B3 Geological Model

Based on the investigation and our experience in the local geological conditions, a geological model has been developed and a section of it is presented on Figure B1.

The site is located on an elevated terrace bound by the Khandallah and Wellington Faults. Investigations indicate that an outwash fan comprising interbedded silts and sands with minor gravel forms the more elevated slopes in the north east of the site. Bedded alluvial soils occur on the flatter terrain below. The surface of the basement greywacke appears to dip down towards the north east.

The ground model generally comprises:

- 0 to 0.5 m of fill
- Interbedded alluvial soils on the lower elevations (up a depth of 9.5m); and outwash fan deposits on the elevated slopes, the base of the outwash fan was not encountered in BH03. Indicating the depth to rock exceeds 29m.
- Greywacke bedrock at depths ranging from 5 to greater than 29m

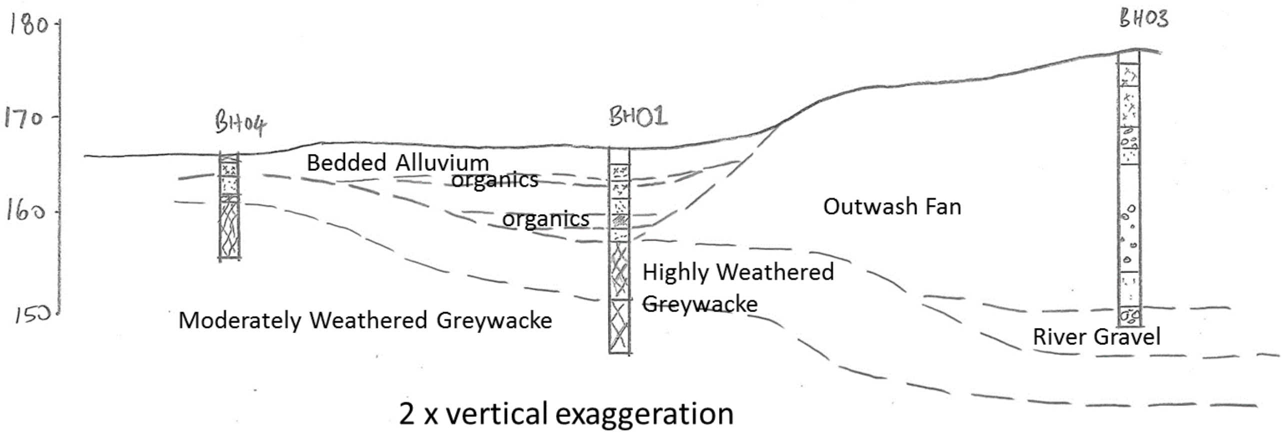

Figure B1: Geological section through site

B4 Groundwater levels

The groundwater regime at the site has been assessed based on daily drilling fluid levels and monitoring of the piezometers installed around the site. These show that the hydrostatic groundwater level is typically at depths ranging from 2.0 to 3.0 m below ground level. Artesian groundwater pressures were measured in BH02 at the contact between the river gravel and underlying weathered greywacke at 16.5 m. Groundwater levels measured in standpipe piezometers indicate groundwater levels from the ground surface to 3.0 m depth. A summary of measured groundwater readings are presented in Table B2. The permeability of soils has recently been assessed by undertaking rising and falling head tests in standpipe piezometers.

Table B2: Summary of groundwater readings

Borehole	Static water depth (m) – measured following drilling	Screened interval (mBGL)		Measured Piezometer depth (m)					
		From	To	20/10/2017	18/1/2019	25/01/2019	27/02/2019	28/11/2019	26/02/2020
BH01	1.90	1.6	5.0	0.00 (at surface)	*	1.16	1.06	1.12	1.07
BH02	Artesian (0.50 while hole at 16.50 m)	Not installed		N/A	N/A	N/A	N/A	N/A	N/A
BH03	2.0, 2.40	3.0	6.0	2.62	2.72	2.48	3.46	1.83	3.33
BH04	1.90	2.0	5.0	3.12	*	3.35	3.4	3.24	3.28
BH05	2.80	Not installed		N/A	N/A	N/A	N/A	N/A	N/A
BH06	2.10	Not installed		N/A	N/A	N/A	N/A	N/A	N/A
*Inaccessible due to fencing for demolition works									

Groundwater levels were also measured in CPTs following completion of probing. CPT dipped water levels can be dynamic and un-representative of actual conditions, so are not relied upon except as indicative background information.

B5 Rising and Falling Head Tests

B5.1 Methodology

Rising and falling tests were undertaken in standpipe piezometers in BH01, BH03 and BH04. The methodology comprised the following:

Falling head test: measurement of the water level in the standpipe piezometer before and after water levels are raised by displacing a “slug” of water with a steel tube (or by the addition of water) the water levels are measured as they return to within 80% of the static water level.

Rising head test: measurement of the water level in the standpipe piezometer before and after water levels are lowered by removing a “slug” of water with a steel tube (or by the addition of water) the water levels are measured as they return to within 80% of the static water level.

The water levels are measured by a data logger at the base of the standpipe piezometer. The Collected data was analysed using Aquifer Test Pro software to give estimates of the hydraulic conductivity of the aquifer in the vicinity of the standpipe piezometer.

B5.2 Results

Results of the rising and falling head tests were analysed using Aquifer Test Pro (Version 2016.1). Permeability results are presented in Table B3.

Table B3: Permeability results from rising and falling head tests

Borehole	Screened interval (mBGL)		Horizontal Permeability K_h (m/s)
	From	To	
BH01	1.6	5.0	1.12×10^{-6}
BH03	3.0	6.0	Inconclusive
BH04	2.0	5.0	5.6×10^{-7}
Inconclusive; piezometer appears to be silted up and is not responding			

B6 Design Parameters

Based on the geotechnical data obtained during the recent geotechnical investigations, the laboratory testing carried out and our experience in similar materials, the preliminary geotechnical design parameters set out in Table B4 can be adopted for preliminary design. The values may be modified subject to a specific assessment for a specific purpose (i.e. these are typical site-wide parameters that can be re-assessed for more specific design purposes).

Table B4: Preliminary geotechnical design parameters

Unit	Unit Weight (kN/m^3)	Friction Angle, ϕ'	Drained cohesion, c' (kPa)	Typical undrained strength range (kPa)	Drained Young's Modulus, E' (MPa)	K_o	K_a
Recent Alluvium	18	30	5	75	20	0.4	0.3
Lower Alluvium	18	32	7	100	30		
Outwash fan	18	34	10	125	50		
Completely to highly weathered greywacke	20	26	20	n/a	50	-	-
Moderately weathered greywacke	20	30	45	n/a	150	-	-

The highly to completely weathered greywacke is logged as having rock strengths from extremely weak to very weak. This indicates an unconfined compressive strength (UCS) of between 1 MPa and 5MPa. The moderately weathered greywacke is described as very weak to weak indicating a UCS of 5 MPa to 20MPa. The rock is expected to perform as a homogenous rock mass and strength parameters have been determined using the Hoek-Brown failure criterion assuming low confining stress (500 kPa). Hoek-Brown parameters for jointed greywacke are provided in Table B5

Table B5 Greywacke rock mass Hoek-Brown failure criterion parameters

Property	Completely to highly weathered greywacke	Moderately weathered greywacke
Unconfined Compressive Strength	2 MPa	10 MPa
Geological Strength Index	20	30
Mi (greywacke)	18	18
Disturbance factor, D	0 (nil disturbance)	0 (nil disturbance)

Permeability values have been assessed from in-situ rising and falling head tests performed in BH01, BH03 and BH04. The outwash fan material is comprised of dense silt and gravel. The silt is tightly packed between the gravel clasts (Photograph B1) and it is not expected to be particularly permeable. The silt material dominates the permeability of the material.

Photograph.B1: Typical dense silt and gravel in the outwash fan

Coefficients of compressibility (mv) for use in consolidation analysis associated with groundwater drawdown are presented in Table B6 below. These values are based on experience of typical values for similar soils in Wellington and on the correlations from CPT readings processed using the CPT interpretation software, CPeT-IT[12].

Table B6: Permeability and settlement design parameters

Unit	Permeability K[3] (m/s)	Coefficient of Compressibility: mv (m^2/MN)
Recent Alluvium - Soft to firm silt and clay[1]	5.6x10-7	0.25
Lower Alluvium – Stiff to very stiff silt and clay[1]		0.05
Outwash fan	1x10-6	0.05
Completely to highly weathered greywacke[2]	1x10-6	0.001
Moderately weathered greywacke[2]	1x10-7	0.001
1 – sand and gravel beds are ignored in compressibility assessment 2 – effectively incompressible 3 – K_v is assumed to equal K_h		

[12] Geologismiki. CPeT-IT version 2.0

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