REPORT

Tonkin+Taylor

Huia Water Treatment Plant Replacement Project

Addendum to the Groundwater and Settlement Report

Prepared for

Watercare Services Limited

Prepared by

Tonkin & Taylor Ltd

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1 Introduction

Watercare Services Limited (Watercare) is proposing to construct a new water treatment plant (WTP) near Titirangi to replace the aging Huia WTP. The replacement WTP is to be constructed on the southern side of Woodlands Park Road, between Manuka Road and Scenic Drive. Watercare is also proposing to construct two 25 ML treated water reservoirs as part of the overall scheme.

A single 25 ML reservoir (Reservoir No. 1) is to be constructed on the northern side of the Woodlands Park Road, directly opposite the proposed replacement WTP. Associated with Reservoir No. 1 is a tunnel shaft and valve chamber required for the North Harbour No. 2 (NH2) pipeline. A second 25 ML reservoir (Reservoir No. 2) will be constructed within the existing Huia WTP once the latter is decommissioned.

Tonkin & Taylor Ltd (T+T) prepared a groundwater drawdown and settlement assessment[1] as part of the Assessment of Environmental Effects (AEE) documentation submitted to Auckland Council in June 2019 in support of a resource consent for the project. This addendum presents the results of additional assessments undertaken since completion of the AEE.

2 Geological Conditions

Further geological and geotechnical assessments of the Reservoir No. 2 site and the escarpment referred to in this addendum are presented in the following document:

> Tonkin & Taylor, 2019. *Addendum to the Preliminary Land Stability Assessment*. Report to Watercare Services Limited dated July 2019.

3 Surface Water Conditions

Two surface drainage catchments have been identified within the project site and are shown on the Auckland Council GIS, as shown in Figure 3.1. The western catchment is called Armstrong Gully and drains the tunnel shaft, Reservoir No. 1 and Reservoir No. 2 areas. The York Gully is the eastern catchment which drains the replacement WTP site.

Both the Armstrong and York gullies provide pathways for surface water to flow south towards the Manukau Harbour. There are no surface water bodies nor are there permanent flows within the drainage channels. Typically the drainage channels are dry or with a thin cover of water even during periods of rainfall (Figures 3.2 and 3.3). There is no indication of shallow groundwater (e.g. seeps or springs).

[1] Tonkin & Taylor, 2019. *Huia Water Treatment Plant Replacement Project. Groundwater and settlement effects*. Report prepared for Watercare Services Limited dated May 2019.

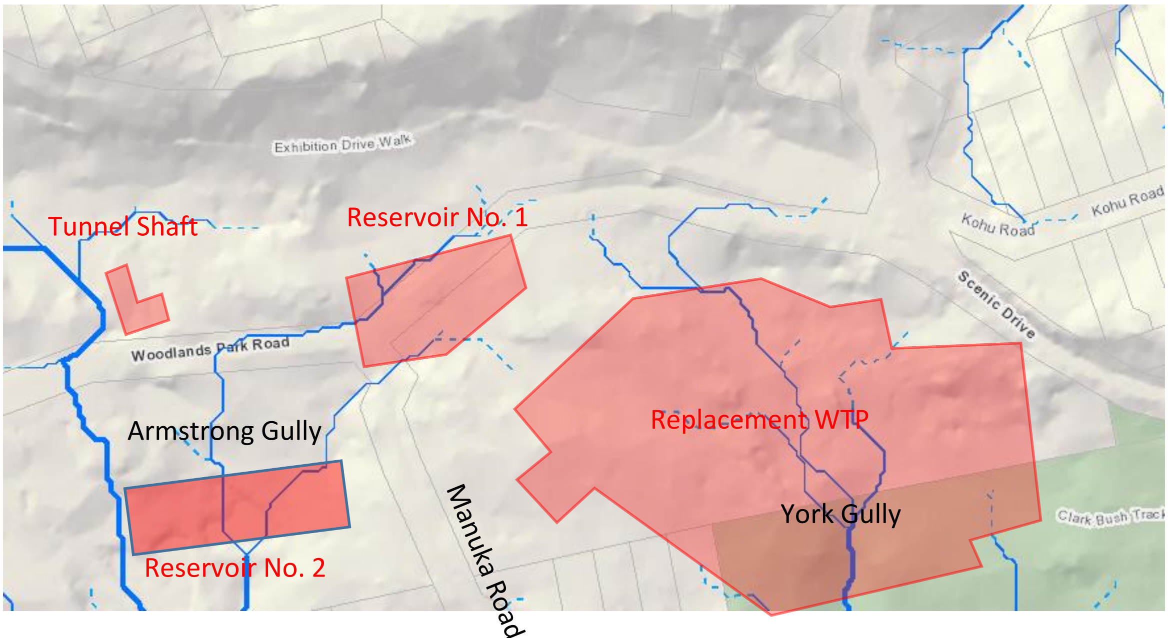

Figure 3.1: Surface hydrology catchments and drainage paths. Manuka Road separated the Armstrong Gully catchment (west) from the York Gully catchment (east).

Figure 3.2: Armstrong Gully next to tunnel shaft showing a muddy drainage path.

Figure 3.3: Main drainage channel of the York Gully within the Replacement WTP site.

4 Groundwater Drawdown Assessment

4.1 Design Groundwater Levels and Sensitivity

A static groundwater level of 5 m below ground level (mbgl) was adopted for the AEE groundwater drawdown analyses. Drawdown was assumed to be limited to Reservoir No. 1 and the tunnel shaft as:

1) The deepest excavations at the replacement WTP would not extend down to the assumed groundwater level; and
2) No excavation was expected to take place at the Reservoir No. 2 site.

The basis for the design level of 5 mbgl was that 75% of the available groundwater data was at 5 mbgl or deeper (Figure 4.1). If groundwater data from the existing WTP site are included, a larger number of readings occur shallower than 5 mbgl, although the proportion of readings at or below 5 mbgl remains nearly the same at 73% (Figure 4.2).

4.1.1 Reservoir No. 1 and Tunnel Shaft

The groundwater drawdown analyses presented in the AEE have been rerun assuming a static groundwater level of 3 mbgl as a means of assessing the sensitivity of the result. This water level accounts for 30 of the 31 available groundwater depth readings (Figure 4.2).

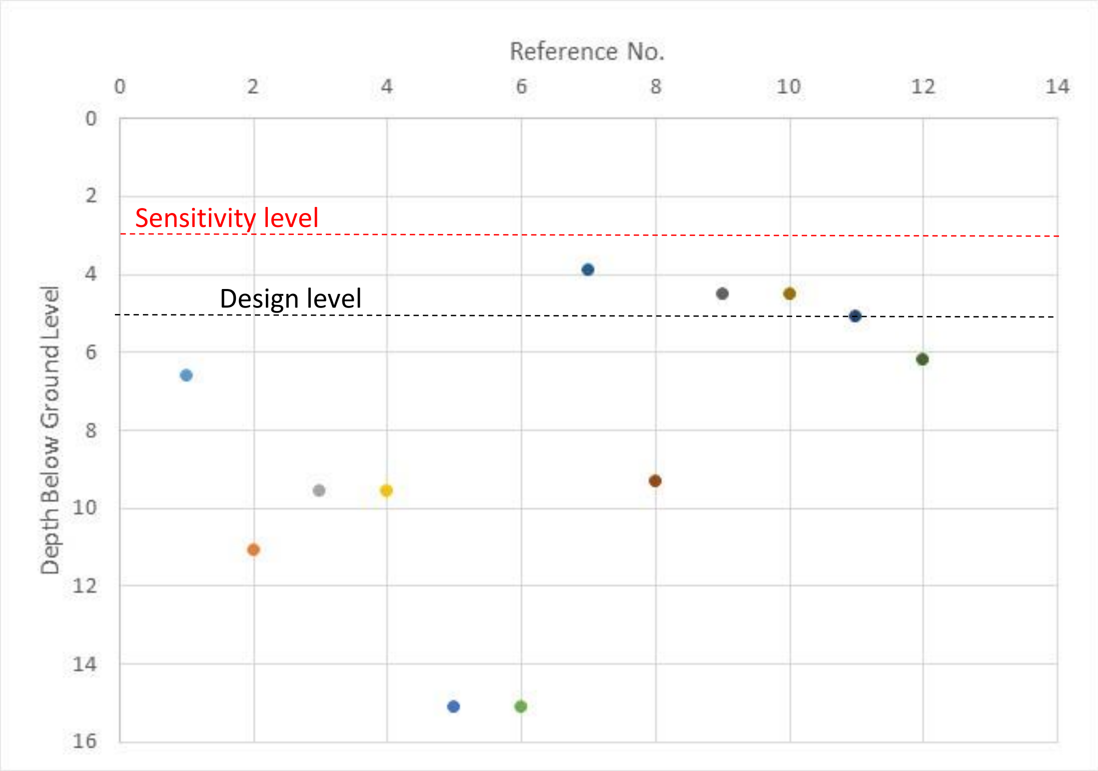

Figure 4.1: Groundwater level data for the area north of Woodlands Park Road.

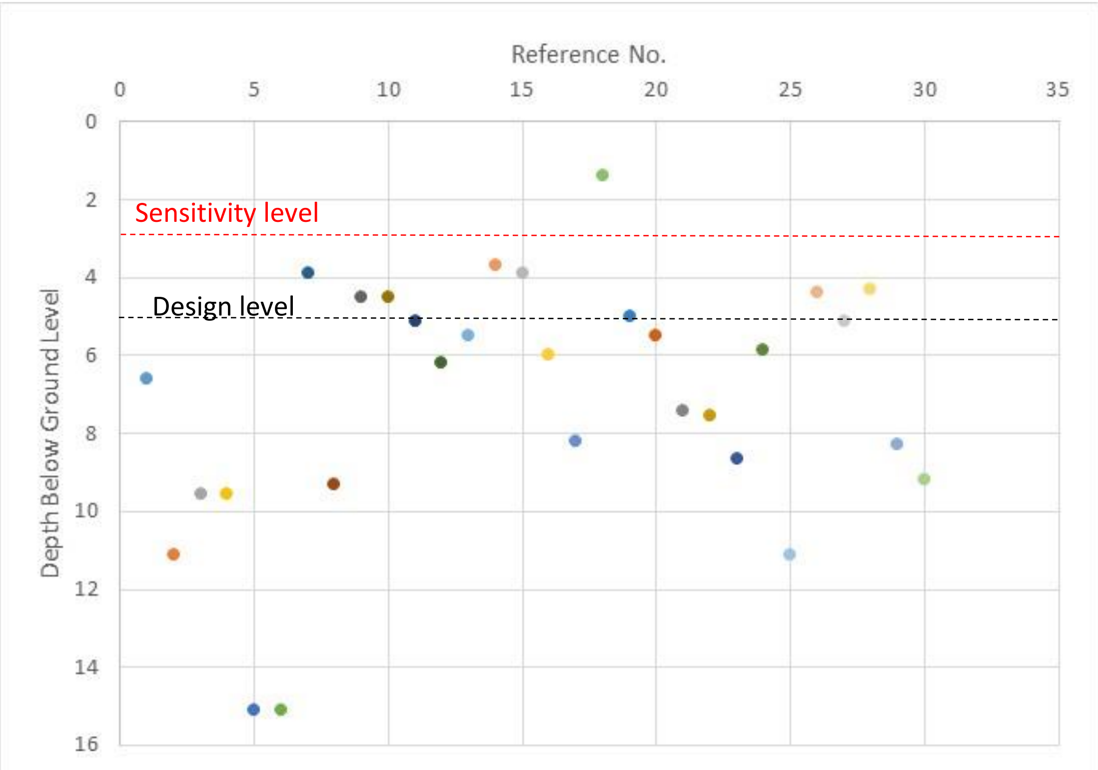

Figure 4.2 *Groundwater level data for both the area north of Woodlands Park Road and the existing WTP.*

The lateral extent of the drawdown curve was estimated using the empirical Sichardt formula, the limitations of which are discussed in Section 4.3. A comparison of the two analyses (Table 4.1) indicates that raising the assumed initial groundwater level from 5 mbgl to 3 mbgl extends the radius of the drawdown by approximately 15 m. This is shown in Figure 4.3.

Table 4.1: Increase in estimated radius of influence due to higher initial ground water level

Location	Initial GL: 5 mbgl	Initial GWL: 3mbgl
Reservoir No. 1		
North face	67	81
South face	54	74
West face	47	60
East face	67	81
Tunnel shaft	55	67

Notes: k = 5 x 10^{-6} m/s.

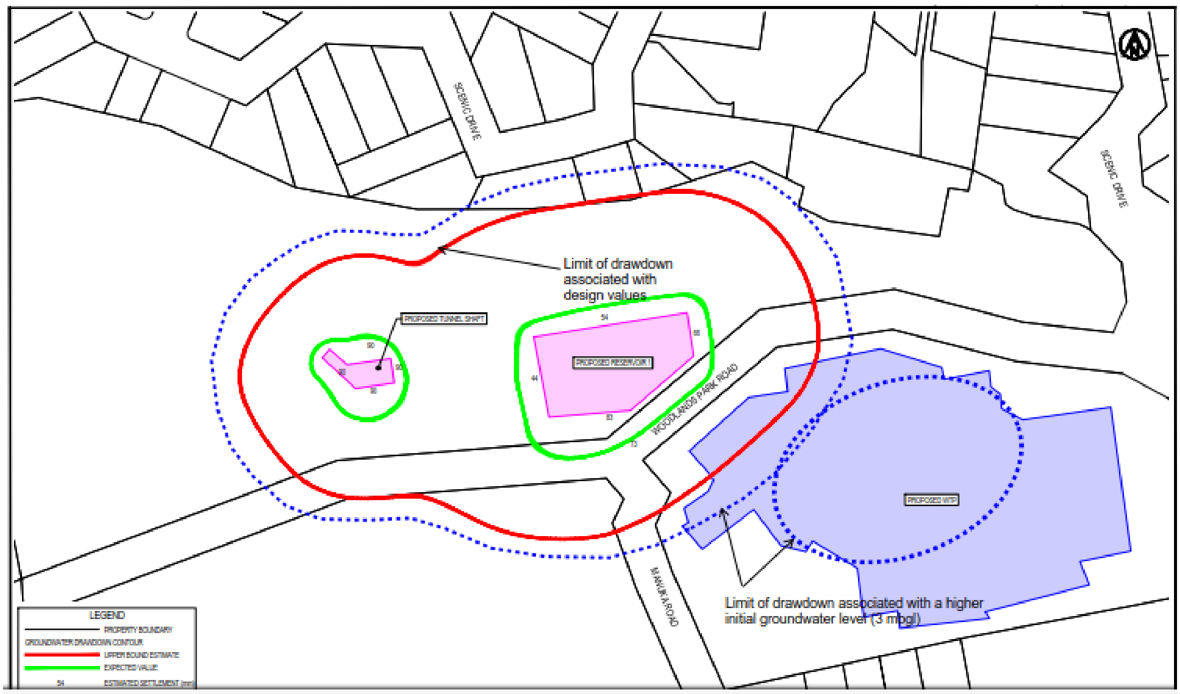

Figure 4.3: Increase in lateral extent of groundwater drawdown that results from raising the design groundwater level from 5 mbgl (red line) to 3 mbgl (blue dashed line).

The increase in radial extent that results from a higher initial groundwater level has negligible impact on the results of the analyses. The drawdown limits are in effect not greatly different to those previous determined and remain outside of private property except for a small number of properties on Scenic Drive. This drawdown beneath the escarpment should not be considered realistic given that the steep topography of the escapement has not been able to be taken into account by the simplified analytical method.

However given that these properties are located at the top of a rock escarpment, it is reasonable to infer that even if the higher initial groundwater level is valid there is negligible effect on those properties located at the top of the escarpment. Lowering of the groundwater beneath the escarpment, should it in fact happen, will result in an increase in stability.

4.1.2 Replacement WTP

As noted above, the drawdown of groundwater at the replacement WTP site was not addressed in the AEE as none of the proposed excavations penetrated the design groundwater level of 5 mbgl. This is not the case, however, if an initial groundwater level of 3 mbgl is adopted.

Figure 4.4 presents the geological section through the proposed WTP that was presented in the AEE. The original groundwater level has been supplement with a higher one located some 3 mbgl. This now places the base of the north face of the BAC structure below the groundwater level, however, the southern face and all other partially buried structures are located either at or above the groundwater.

Using the Sichardt methodology, drawdown with the 8 m deep BAC excavation will extend some 33 m to the north of the BAC structure, but because of topographic effects, not extend beyond the structure to the south. Intermediate values will occur along the side of the structure. An approximate 33 m lateral extent places the limit of drawdown at the northern edge of the DAF structure. Based on geometric considerations alone (Figure 4.5), this would appear to be a reasonable estimate of drawdown extent. These effects are therefore contained entirely within the WTP site.

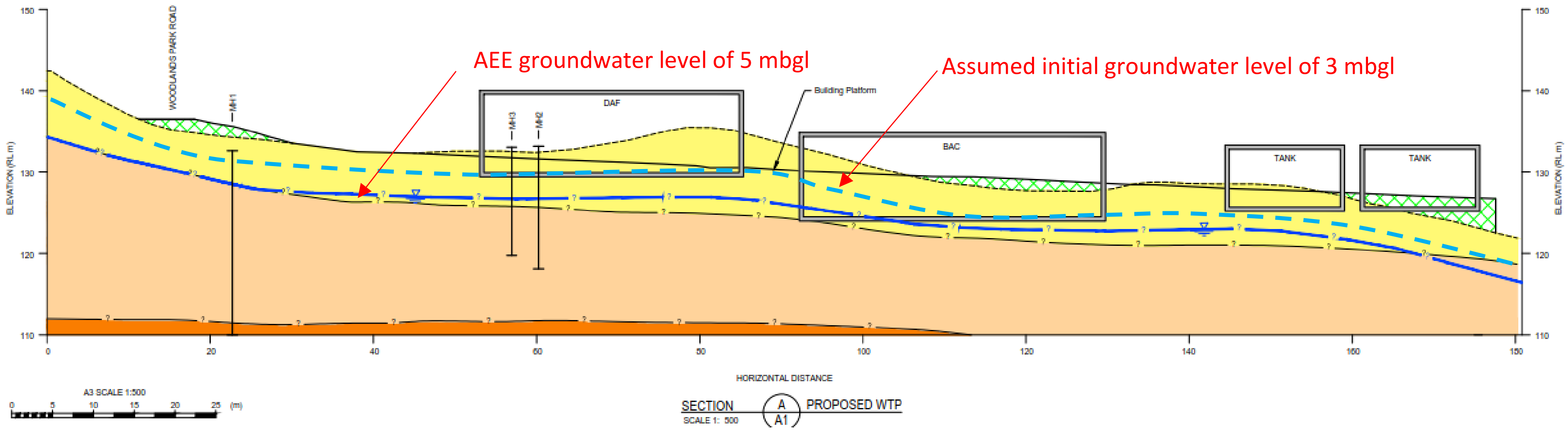

Figure 4.4: Geological section through the replacement WTP showing an inferred 3 mbgl elevation of the initial groundwater

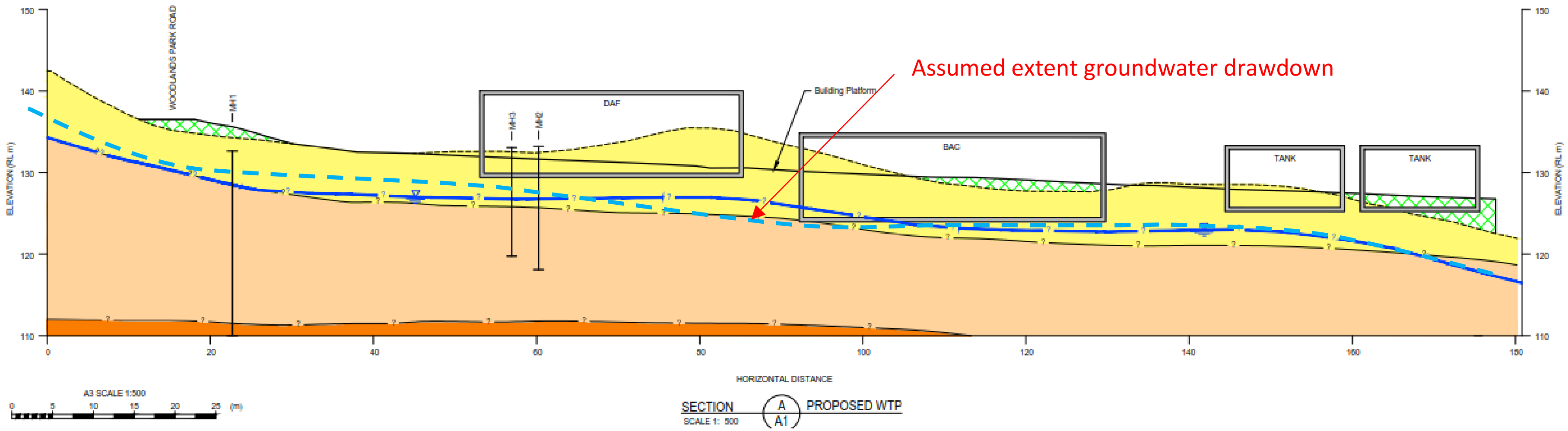

Figure 4.5: Geological section through the replacement WTP showing an inferred 3 mbgl elevation of the initial groundwater

4.1.3 Reservoir No. 2

Figure 4.6 presents a geological section through the Reservoir No. 2 location (see *Addendum to the Preliminary Land Stabilisation Assessment Report*) as originally presented in T+T (2010). Piezometers indicate a groundwater level of 5 mbgl or more beneath the reservoir, deepening to 12 mbgl closer to Woodlands Park Road. Assuming an initial groundwater level of 3 mbgl, the proposed cutting behind the reservoir might reach, but not extend below the groundwater. We expect therefore that there will be no meaningful drawdown of the groundwater at Reservoir No. 2.

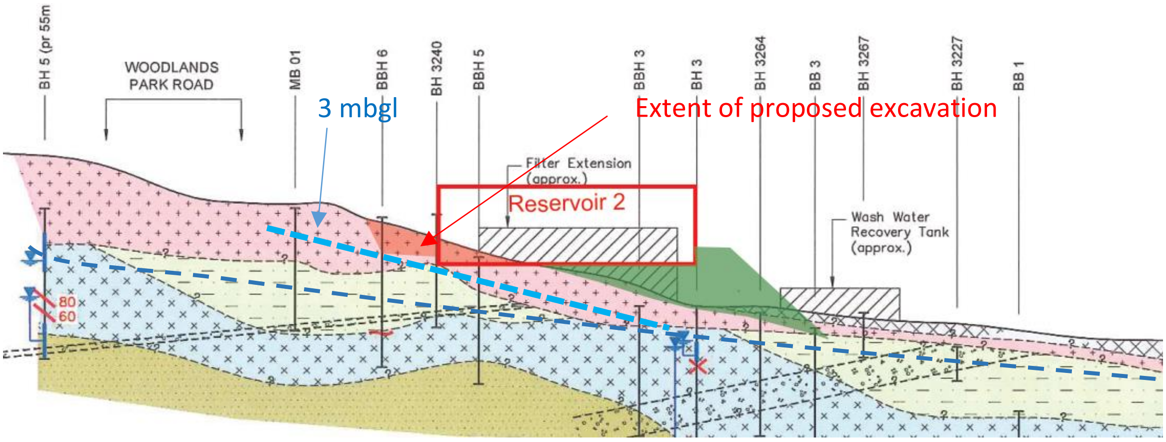

Figure 4.6: Geological section through the Reservoir No. 2 location (from T+T, 2010). Dashed blue line represents groundwater based on piezometers. Extent of proposed excavation is shown in red.

4.2 Design Permeability and Sensitivity

The groundwater drawdown assessment presented in the AEE used a design horizontal permeability value of 5×10^{-6} m/s based on the assumption that the colluvium material would be broadly similar to some of the Tauranga Group soils encountered in many projects across Auckland and for which permeability has been measured.

T+T has considered all of the available historic data when developing the preliminary ground model used to undertake the groundwater drawdown assessment. The Tauranga Group is essentially a mapping unit used to encapsulate a wide variety of surficial soil-like materials of Pliocene to Holocene in age. These range from high plasticity clays through to clean sands, from very soft to very stiff soils and from alluvium through to distal volcanic airfall.

The colluvium, which is the subject of the groundwater drawdown assessment, is typically a sand-silt-clay mixture with a variable rock fragment component. In general the colluvium tends to be siltier and sander than much of the Tauranga Group elsewhere in Auckland and so we would expect that the mass permeability of this material will be higher that what is typically observed in the Tauranga Group.

The rock fragments contained within the alluvium are the most significant departure from typical Tauranga Group material, however, as these are encapsulated within a finer-grained matrix we do not believe that their occasional presence will unduly influence the overall groundwater behaviour of the colluvium.

The Groundwater Drawdown and Settlement Report (AEE Appendix H) presented permeability design values for the Tauranga Derived from a number of prominent Auckland projects. Table 4.2 below presents a more comprehensive list. This data has been plotted on Figure 4.7 below. This shows that the permeability value of 5 x 10^{-6} m/s adopted to represent the colluvium is at the high end of the known values, although it does not represent an upper bound.

Table 4.2: Permeability values for Tauranga Group from previous Auckland projects.

No.	Project	Horizontal Permeability (m/s)
1	Northern Interceptor	1.5x10^{-7} to 2.9 x 10^{-9} (Measured) 2 x 10^{-7} (Design)
2	Wairau Road Rising Main	3.8x10^{-6} to 4x10^{-7} (Measured)
3	Rosedale WWTP	2.5 x 10^{-7} to 1 x 10^{-7} (Measured)
4	Huia No. 2 Pipeline	6x10^{-6} to 6 x 10^{-7} (measured)
5	Britomart (Upper Tga Gp)	3.9 x 10^{-7} (measured) 3 x 10^{-5} (Design)
6	Britomart (Lower Tga Gp)	7 x 10^{-5} – 1.3 x 10^{-8} (measured) 2 x 10^{-7} (Design)
7	City Rail Link	7.2x10^{-6} to 1.8x10^{-8} (Measured) 2 x 10^{-7} (Design)
8	Kohimarama Tank	1 x 10^{-6} to 6 x 10^{-8} (Design)
9	Rosedale Outfall Tunnel	2 x 10^{-7} – 4 x 10^{-9} (Design)
10	Central Interceptor	9.7 x 10^{-5} to 4.6 x 10^{-9} (measured)
11	Central Interceptor - Recent	10^{-5} to 10^{-7} (Design)
12	Central Interceptor - Puketoka	10^{-7} to 10^{-8} (Design)
13	St Marys Bay Tunnel	5 x 10^{-5} to 1x10^{-7} (Measured)
14	Waterview Tunnel	1x10^{-7} to 2.3x10^{-7} (Design)
15	New Lynn Rail	3x10^{-7} (Design)
16	Hobson Tunnel	5 x 10^{-8} (Design)
17	Three Kings Quarry	10^{-8} (Design)

As described above, only the Reservoir No.1 and tunnel shaft excavations are expected to result in groundwater drawdown at the design groundwater level. The extent of this drawdown will depend upon both the magnitude of drawdown and the assumed permeability.

Table 4.3 presents the results of analyses in which the design permeability of the colluvium has been increased from the design value of 5 x 10^{-6} m/s to 10^{-5} m/s, a five-fold increase. This shows that in most cases, the increase in assumed permeability extends the lateral extent of drawdown by approximately 20 m, depending upon the depth of excavation. Table 4.3 also shows that this effect is greater for the assumed change in permeability than it was for the assumed increase in groundwater level.

These lateral extent of drawdown for all three cases are shown graphically on Figure 4.8.

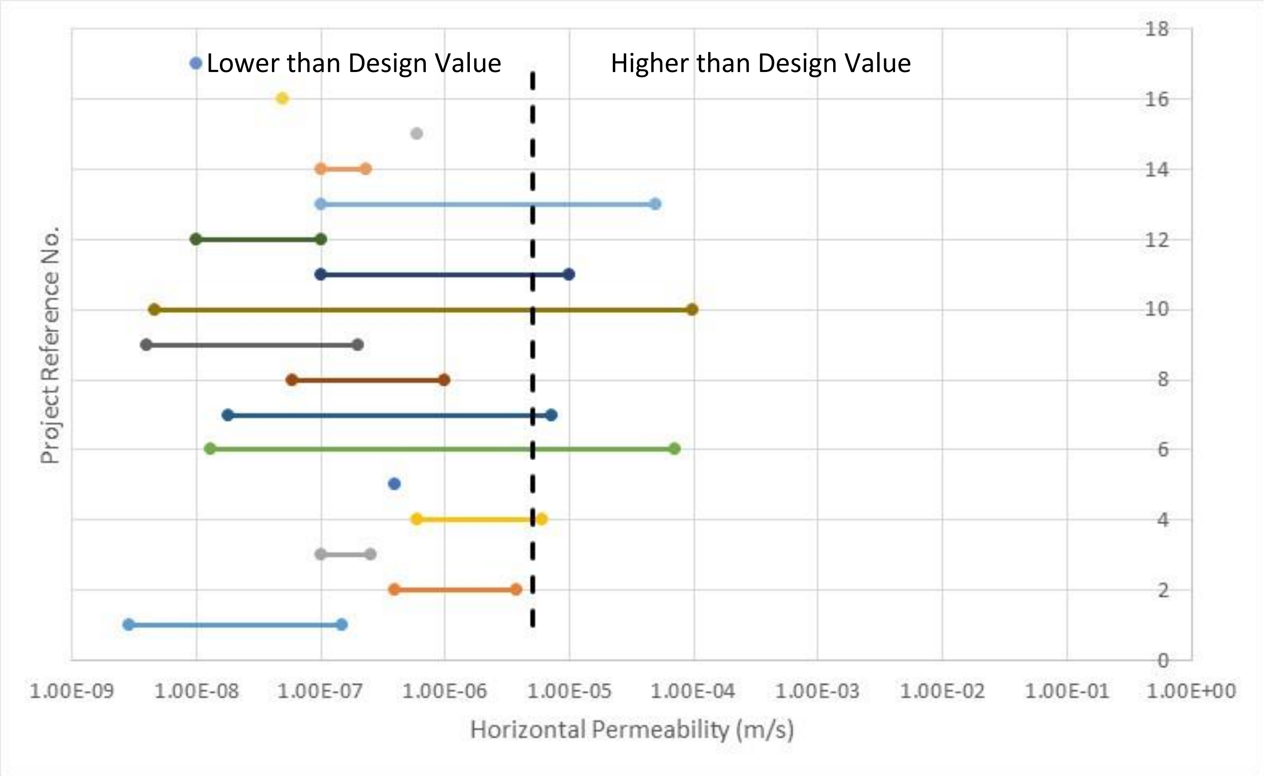

Figure 4.7: Permeability values (measured and design) from Table 4.2. The adopted design value ($5x10^{-6}$ m/s) is shown as a dashed vertical line

Table 4.3: Increase in estimated radius of influence due to higher groundwater or permeability

Location	<u>Design Case</u> Initial GWL = 5 mbgl K = 5 x 10^{-6} m/s	<u>GW Sensitivity</u> Initial GWL = 5 mbgl K = 5 x 10^{-6} m/s	<u>k Sensitivity</u> Initial GWL = 5 mbgl k = 10^{-5} m/s
Reservoir No. 1			
North face	67	81	95
South face	54	74	76
West face	47	60	66
East face	67	81	95
Tunnel shaft	54	67	76

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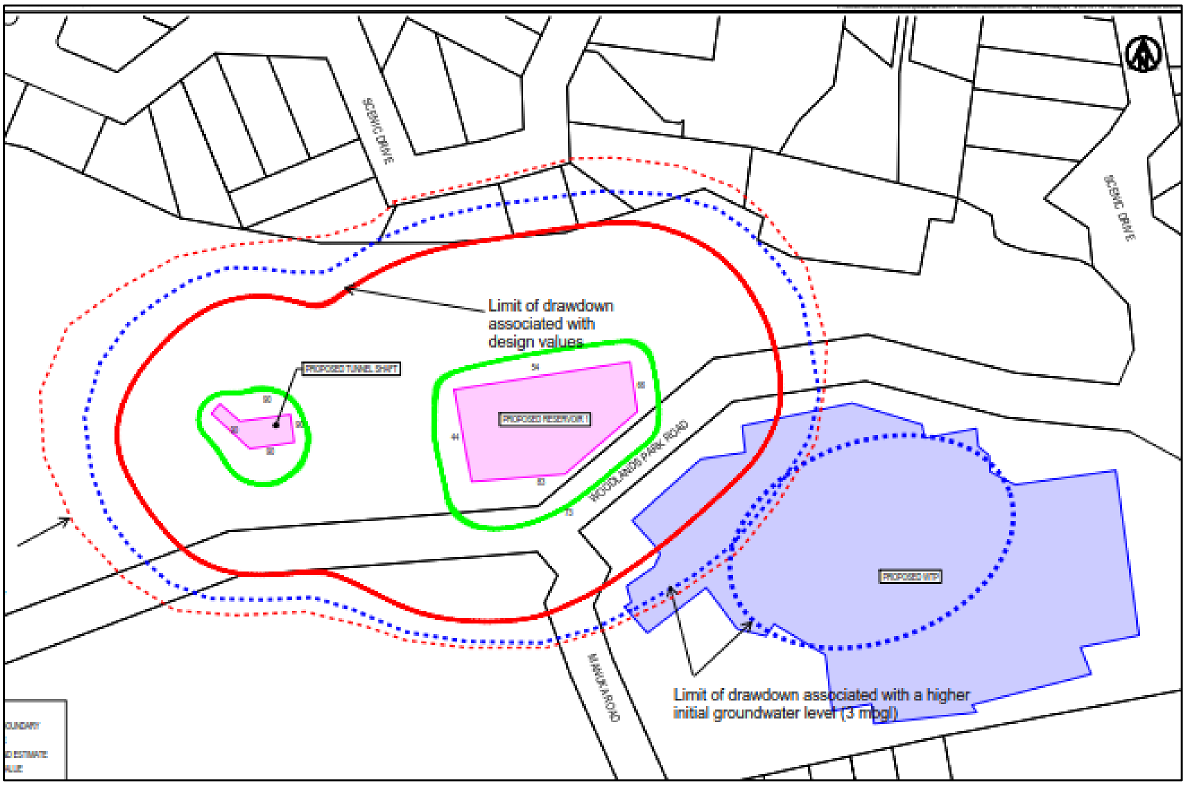

Figure 4.8: Lateral extent of groundwater drawdown in 1) the AEE design case (solid red line); 2) elevated groundwater level (dashed blue line) and; 3) increased permeability (dashed red line)

4.3 Methodology

The drawdown assessment presented in the AEE used the empirical Sichardt formula to estimate the radial extent of groundwater drawdown. This is a very commonly used method for estimating the radius of influence (R_0) under steady state conditions and assuming radial flow and is presented in both CIRIA (2000)[2] and EA (2007)[3]

It was recognised at the time of undertaking this work that the Sichardt methodology had its limitations. This is implicit in its simplicity, which includes:

- A very simple hydrogeological model adopted (i.e. single material);
- An assumed horizontal ground surface;
- No allowance for recharge; and
- Simple empirical controlling parameters.

The origins of the Sichardt formula are somewhat obscure, although it appears to have been derived from a series of pumping tests carried out in an unconfined, granular aquifer in the 1930s. It has been reported that it can under-estimate the radius of influence and over-estimate the inflow rate except in the case of a very permeable gravel aquifer.

[2] CIRIA (2000) Groundwater control – design and practice
[3] EA (2007) Hydrogeological impact appraisal for dewatering abstractions. Environment Agency (EA) science report SC040020/SR1

The reason that it is so commonly used is because of its simplicity and also because dewatering investigations often focus on the rate of inflow and the Sichardt equation generates a conservative value. The focus of the drawdown assessment is however not on inflow rates but the radius of influence hence there is a potential for the method to underestimate the lateral extent of drawdown. The alternative approach is to develop a numerical model. Numerical models can more closely simulate variations in the lateral and vertical extent, and hydraulic properties of the different aquifer units, as well as recharge and discharges. However, numerical models are only as good as the data and level of detail behind them.

Given the simplicity of the data available at this time and the fact that only one geological unit is likely to be involved in dewatering of the major excavations, the Sichardt methodology is considered appropriate for this level of assessment provided that conservatism is included in the assessment by: 1) assuming full and free groundwater inflows into the excavations; and 2) adopting conservative permeability values.

5 Applicability

This report has been prepared for the exclusive use of our client Watercare Services 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.

Tonkin & Taylor Ltd

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