CREATING OPEN WATERCOURSES IN PEAT FIELDS, THE CHALLENGES

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ABSTRACT

Installing open channels through peat land has been successfully employed by farmers for hundreds of years. The implementation of open channels in an urban planning environment in similar geotechnical conditions is quite different. The design and consenting of 1.7 km of open channel in Takanini to accommodate proposed future urban development has identified a number of challenges, both as part of the design, and in the greater planning context.

The Takanini east urban intensification area is currently unserviced by stormwater infrastructure, located in peat fields and is predominantly flat. The urban area is currently within a predicted 1% AEP (annual exceedance probability) floodplain and is unable to be comprehensively developed until a stormwater solution is implemented.

Traditional piped systems, open channels and hybrid systems including attenuation were considered and evaluated. An open watercourse was selected as the most resilient solution and one that would also provide amenity and ecological benefits. However, the design of such is not without its challenges. This paper outlines the key issues that have been identified as part of the design process and how these have been resolved during the planning and design.

KEYWORDS

Conveyance channel, open channel, watercourse, stormwater, peat, settlement analysis

PRESENTER PROFILE

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1 INTRODUCTION

Land supply for new development in the Auckland Region is a major issue. It is a key factor that influences house prices in the region and the National Economy due to the relative size of the Auckland Market. The low-lying peat country in Takanini is close to existing transport services and also the Manukau and Papakura urban Centres. The demand for more land supply for urban development in Auckland is the driver for development in to this area despite the risks associated with flooding and ground settlement.

Facilitating urban development on the Takanini Peat, whilst managing the risks, is a significant engineering challenge. This paper outlines the investigations, analysis and solutions that have been developed to enable development in this area. The Takanini Cascades stormwater conveyance channel will be the main stormwater system for the Takanini Structure Plan Areas 2a, 4 and 2b. Currently these areas have no formalised drainage, and are predicted to flood in the 1% AEP.

The proposed conveyance channel will extend a distance of 2.1 km, with the channel width varying between 25 and 45 m. The channel consists of a main branch, between Old Wairoa Road and Grove Road, and a northern branch that extends between 91 and 131 Grove Road. At maximum probable development the channel will convey peak flows of 24 m3/s.

The channel design allows for a low flow channel, planted wetland bench, 10% AEP channel, with planted riparian zone and conveyance of the 1% AEP. Provision has been made for footpaths, cycleways and parkside roads (in part) within the designation extents.

Ground conditions comprise recent alluvium and Puketoka Formation alluvium, comprising soft peats, organic silts and sands. Groundwater levels vary seasonally, varying between 1 and 3 m, although in some lower lying areas is known to be at or above existing ground levels. Lowering of groundwater tables in these conditions is known to induce both primary and secondary consolidation.

This paper outlines the overall conveyance scheme and in particular provides comment on the issues and opportunities with the development of the proposed open channel in the Takanini peat fields, particularly in relation to settlement. Note that the information presented in this paper is based on the current scheme design which will be further advanced at the detailed design stage.

2 TAKANINI STORMWATER SCHEME

The Takanini stormwater conveyance channel forms part of a larger scheme to provide drainage, flood minimisation and water quality treatment for the Takanini structure plan areas 2a, 4 and 2b as displayed in Figure 2 below. Phase 1 of the scheme will involve the construction of a 2.5 m diameter tunnel over approximately 1.1 km between the Pahurehure Inlet and McLennan wetland (shown in Figure 1 below). The tunnel will convey overflows from the wetland of up to 24 m3/s via a high level outflow.

The McLennan wetland was constructed in 2002 and is designed to attenuate flows from the surrounding and upper catchment as well as provide water quality treatment. A large dam was created to provide for the attenuation at the Mclennan Wetland.

In addition to the McLennan wetland, source treatment of stormwater is also required in the catchment for both contaminant removal and ground water recharge. Once complete this scheme aims to provide a resilient drainage network that maximises the use of open channel systems to enhance biodiversity and convey flood flows, whilst optimising the land available for development and minimising risks associated with ground settlement.

Figure 1: McLennan Wetland

The Grove Road culvert is currently proposed to be a 4 m x 2 m box culvert, of approximately 400 m in length that will connect the proposed conveyance channel from Grove Road to the McLennan wetland. Housing New Zealand is developing the area between the wetland and Grove Road, and will incorporate the culvert into their development.

The proposed main branch of the Takanini stormwater conveyance channel extends 1500 m between Grove Road and Old Wairoa Road. A northern branch of about 600 m will provide servicing for properties up to and including Walters Road. The total channel width is between 25 m and 50 m. The full stormwater scheme is shown in Figure 2 below.

The total scheme will take a minimum of 5 years to construct, and as dratrinage paths do not exist, can only be built sequentially starting from the downstream section at the Pahurehure Inlet and finishing at the uppermost section of the channel at Old Wairoa Road.

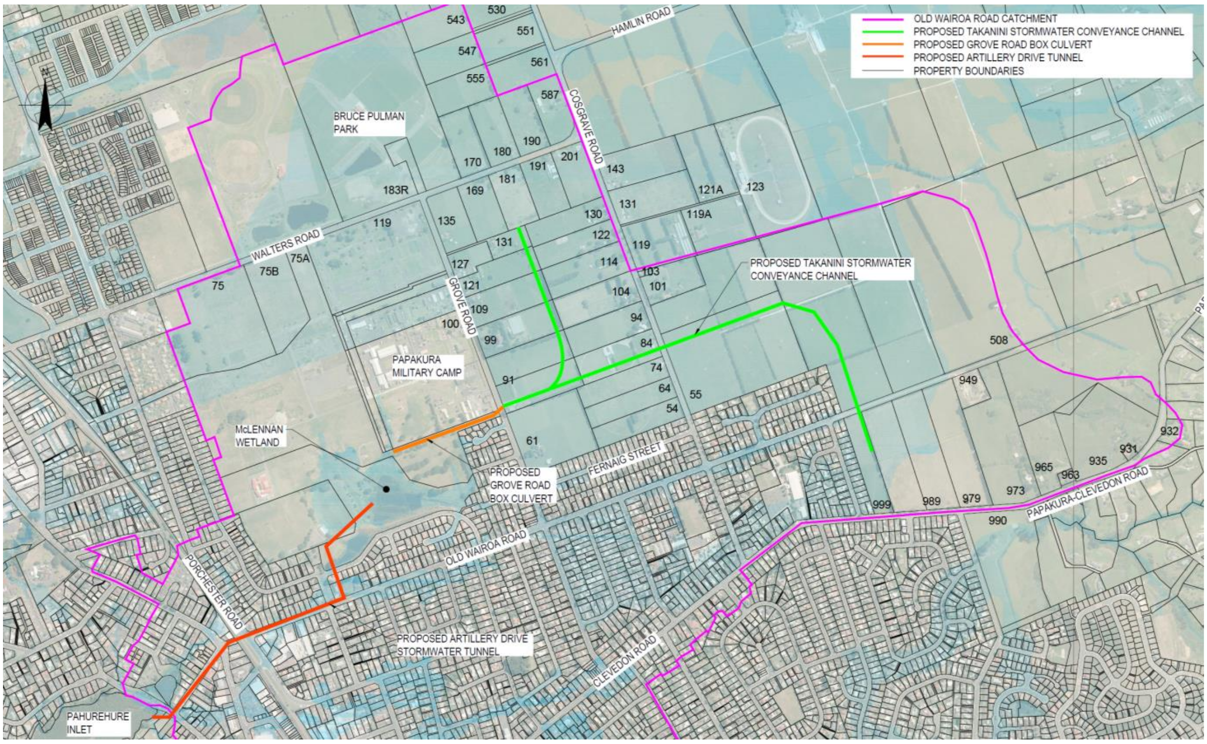


Figure 2: Proposed Takanini stormwater scheme

3 THE TAKANINI STORMWATER CONVEYANCE CHANNEL

3.1. DRAINAGE OPTIONS

Options for servicing the Takanini Structure Plan Areas 2a, 2b and 4 and reducing the 1% AEP floodplain have been considered as early as 2000. Generally the outcome was that an open channel was the preferred option.

At the outset of this project options were again evaluated to determine the best long-term solution for servicing these areas. All options aim to achieve removal of the 1% AEP floodplain from proposed future development areas.

3.1.1. PIPES VERSUS OPEN CHANNELS

Pipelines are perceived to be a cheaper alternative to open channels because of reduced operational costs, however a pipeline has a definite end of life that is often unaccounted for, even in apparent whole of life costing. As well as the obvious ecological and amenity benefits that an open channel affords, they also offer resilience to increased flows and have a total life span that can almost be considered infinite provided the correct maintenance and relative frequency are employed. The additional cross sectional area over a wide channel, particularly for larger events (provided adequate freeboard is included in the design) means that additional catchment flows can be accommodated whereas a pipeline will be sized for the 10% AEP often with no or little contingency.

The disadvantage is that in a normal development scenario stormwater pipelines are constructed within the road reserve by the developer (typically this is a 20 m width). The stormwater reticulation is then vested as a Council asset. The result is that Council transfer responsibility for capital works to the developer and little or no land is required by Council. In real terms overland flowpaths for the 1% AEP need to be accounted for.

Piped versus open channel options, as well as hybrid options were assessed in the early phases on the concept design. For a piped system, a pipe with a diameter of up to 3 m would be required with an allowance for an overland flowpath that would have a total width of approximately 60 m (due to the requirements of the Auckland Council Stormwater Code of Practice). No buildings can be built within the overland flowpath but non-critical services such as roads, footpaths and cycleways as well as road reserves can be.

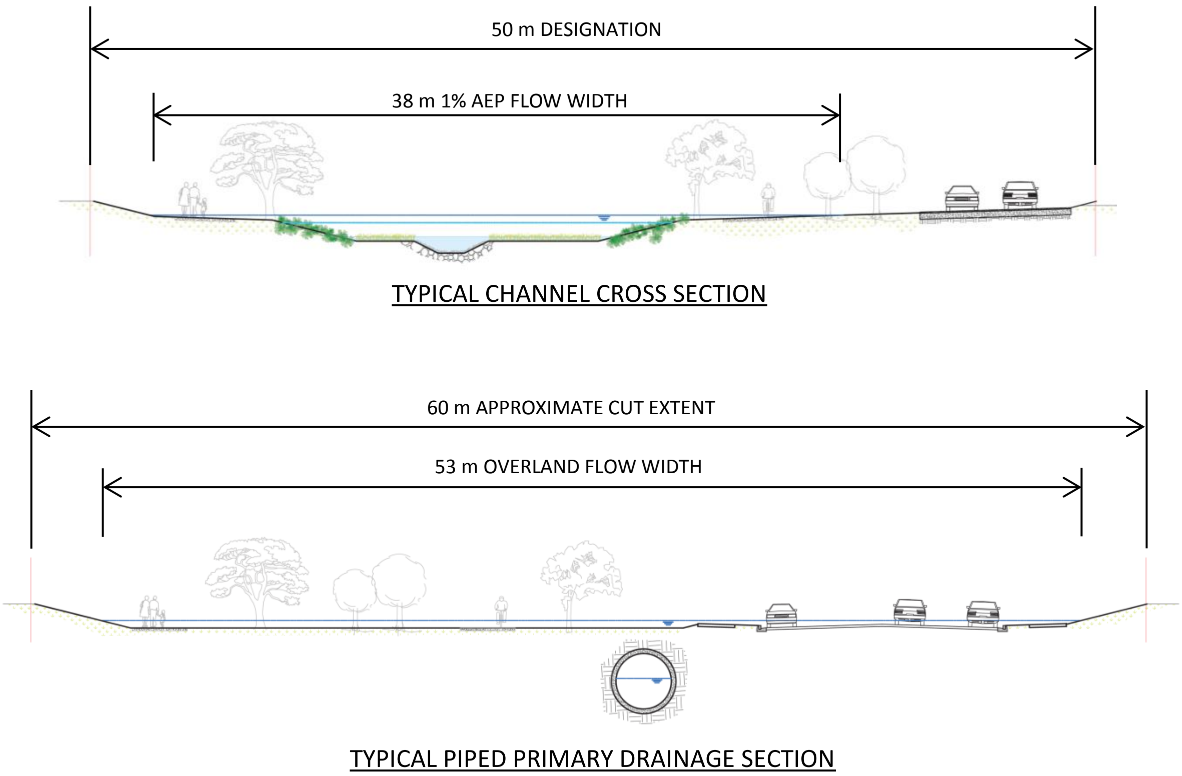


Figure 3: Comparison of open channel versus piped system for Takanini

With an open channel a similar model can be adopted and this is what has been considered in the Takanini stormwater conveyance channel. The total 1% AEP channel has provision for a cycleway, footpath and parkside road, at least part thereof. The intention in doing so is to actively promote frontage onto the channel by offering available land within the designation, thereby producing a significant cost saving for developers.

The exception here is that the project will be constructed by Auckland Council and because of settlement effects, a wider designation than would normally be required has been allowed for to account for the worst long term estimated primary settlement effects of up to 260 mm, and short term temporary works settlement effect estimated to be up to 466 mm. Total settlement is estimated to be considerably greater, but it is estimated that secondary “creep” settlement will occur at a rate of less than 3 mm per month for a period of 50 years over a regional area, which is tolerable for modern lightweight structures, and is currently being experienced and accommodated by existing buildings and infrastructure in the area.

Due to the flat topography the piped scenario would also require a significant cut, 6.5m depth to invert to allow for subgrade improvements and for the piped network to drain. In the peat material this will induce significant short-term primary settlement without significant, and expensive, mitigation options. The reality of trying to inlet the 1% AEP would also require substantial inlet structures to be built and even then, the actuality of getting flows into the pipe is debatable.

Additionally, unless extensive shoring is used, low slope angles will be required for excavation resulting in a significant excavation similar to that required for channel construction. Shoring in this ground is problematic due to the presence of large stumps. Buoyancy and settlement of the pipe also need to be addressed.

It is likely that any attempt to seal pipelines will over time be lost due to pipe or manhole displacement even with subgrade improvement, or general deterioration. Displacement or deterioration will result in groundwater ingress and will effectively result in long-term dewatering and potential localised settlement. Indeed site observations and groundwater level monitoring indicate that the area of existing piped networks (potentially stormwater and wastewater) are currently inducing localised groundwater drawdown.

3.1.2. ALTERNATIVE OPTIONS

Alternative options included a piped system for the 10% AEP with 1% AEP flow attenuation, and a combination of partially piped and open systems to convey the 1% AEP flow.

A 10% AEP piped system, or even partially piped system does remove many of the issues identified in preceding sections such as temporary drawdown, width of excavation for construction, and possible long-term drawdown through infiltration. It also increases the overall width required for the overland flowpath if the full 1% AEP event is considered. This is considerably wider than the proposed channel designation.

Flow attenuation for the 1% AEP would be significant. A total of almost 12,000m3 of storage (minimum area of 25,600 m2) would be required and these values to not consider the need for near surface ponds (and therefore greater areas) because of high groundwater tables.

3.1.3. COST VERSUS BENEFIT

The benefits and relatively low cost for the channel option were found to outweigh those associated with a piped system or variations of that. In general the open channel was the least expensive ($20M) versus a 1% AEP piped network ($40M), 10% AEP piped network with 1% AEP flow attenuation ($33M) or hybrid open channel and piped system for the 1% AEP ($27M) (rough order estimates only).

3.2. ALIGNMENT

The stormwater conveyance channel was identified in the Cosgrave Structure Plan that forms part of the Takanini Structure Plan. The route conveyance channel has changed over time for a variety of reasons, however in general follows the structure plan alignment.

The general alignment is linear to minimise the impact on future development and land acquisition. Within the channel extents however, the low flow channel has the ability to meander and it is expected that this will naturalise further over time. Meanders will be incorporated into the detailed design.

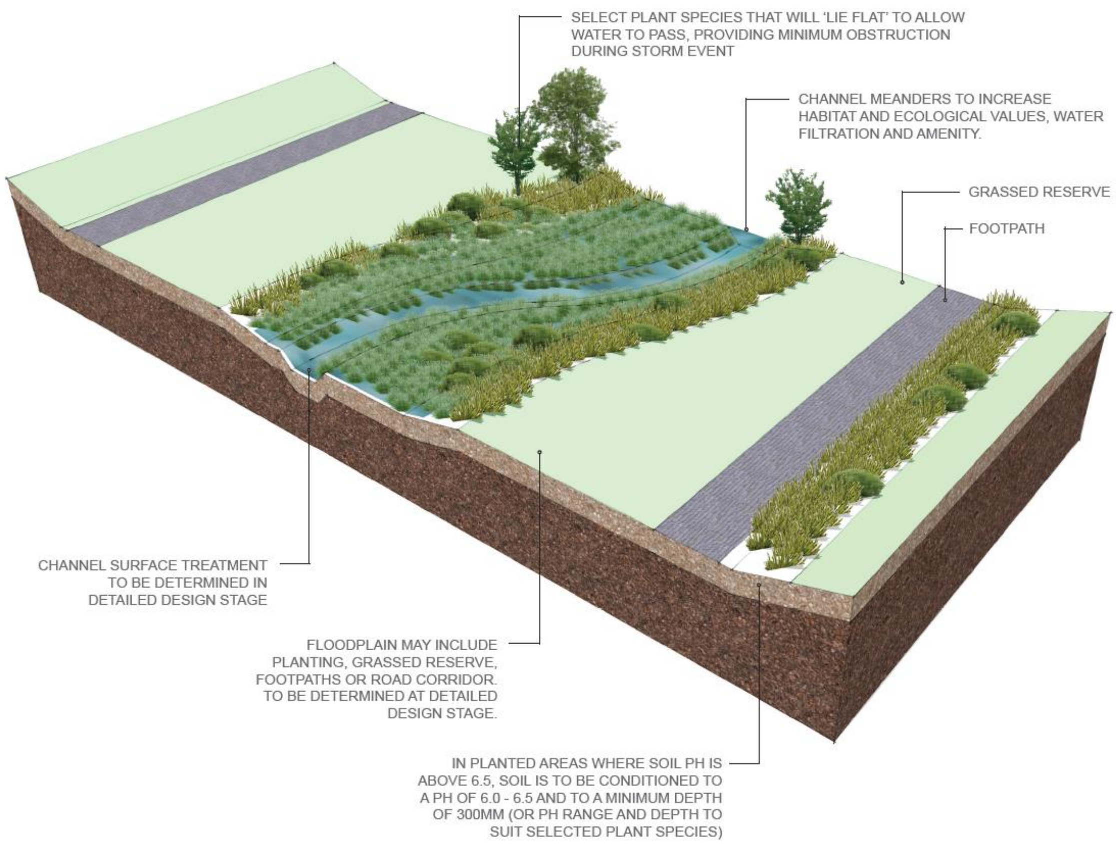


Figure 4: Potential channel meander within the channel designation

3.3. OVERALL CHANNEL DIMENSIONS

The main stormwater conveyance channel between Grove Road and Old Wairoa Road will consist of an open channel varying in width from some 25 m to 45 m at its widest. The deepest section is approximately 4.0 m below existing ground level.

The northern branch that extends between 91 and 131 Grove Road is nominally 30 to 40 m in width with the deepest channel section of about 3.5 m where it approaches the main channel (the channel reduces gradually in depth to about 2.5 m to channel invert at the upstream section).

The proposed channel is almost wholly within private property. A proposed designation ranging from 25 to 50 m has been selected to allow for the widest channel extent and most significant settlement. The Notice of Requirement was officially notified in September 2014.

Figure 5: Proposed Takanini stormwater conveyance channel

3.4. DESIGN FLOWS

The total catchment area that will be served by the conveyance channel is 168 ha. The catchment extends from Grove Road in the west, to Walters Road in the north, Old Wairoa Road in the south and greenfield sites north of this.

Design flows have been calculated using TP108 and increase sequentially up to 24.8 m3/s at the point of discharge to the Grove Road box culvert. Design flows are summarised in Table 1*: Proposed Takanini stormwater conveyance channel design flows*. All flows consider the 2090 climate change scenario.

Table 1: Proposed Takanini stormwater conveyance channel design flows

Channel section	Chainage	10% AEP flows (m^3/s)	1% AEP flows (m^3/s)
Main channel	>1300 m	1.9	3.4
Main channel	600 m	8.6	14.9
Main channel	400 m	8.6	15.5
Main channel	0 m	14.2	24.8
Northern channel	550 m	3.2	5.5
Northern channel	0 m	5.6	9.8

3.5. CREATING DEFINED ZONES

In designing the channel, ecological habitat and hydraulic conveyance were considered simultaneously from the initial conceptualisation. Without doing so neither outcomes would have been achieved. The philosophy of the channel is to:

1. Provide a low flow channel with a permanent water level to provide habitat and reduce drawdown and subsequent settlement effects. This low flow channel incorporates a wetland planted bench to improve water quality.
2. Allow for the conveyance of 10% AEP flows
3. Deliver the safe passage of the 1% AEP within a defined channel.

3.5.1. LOW FLOW CHANNEL

The low flow channel is typically 1.3 m wide with slope batters 2H: 1V. The wetland bench above the low flow channel varies in width from 1.5m to 4.15 m and is predominantly flat. The low flow channel will incorporate rip-rap placed to promote a naturalised channel. The wetland bench will be planted with wetland species.

3.5.2. 10% AEP WATER LEVEL

The channel banks are battered at 4H: 1V to a height between 0.65 m and 1.05 m to allow for conveyance of the 10% AEP. The batters will incorporate riparian planting by way of stabilised socks (or similar) that will be planted with riparian species.

3.5.3. 1% AEP WATER LEVEL

The channel above the 10% AEP water level continues at a gradient of 33H:1V to allow for conveyance of the 1% AEP. Provision for a footpath, cycleway and access road, or part thereof has been allowed for in this zone. This portion of the channel will be grassed with amenity provided by specimen trees.

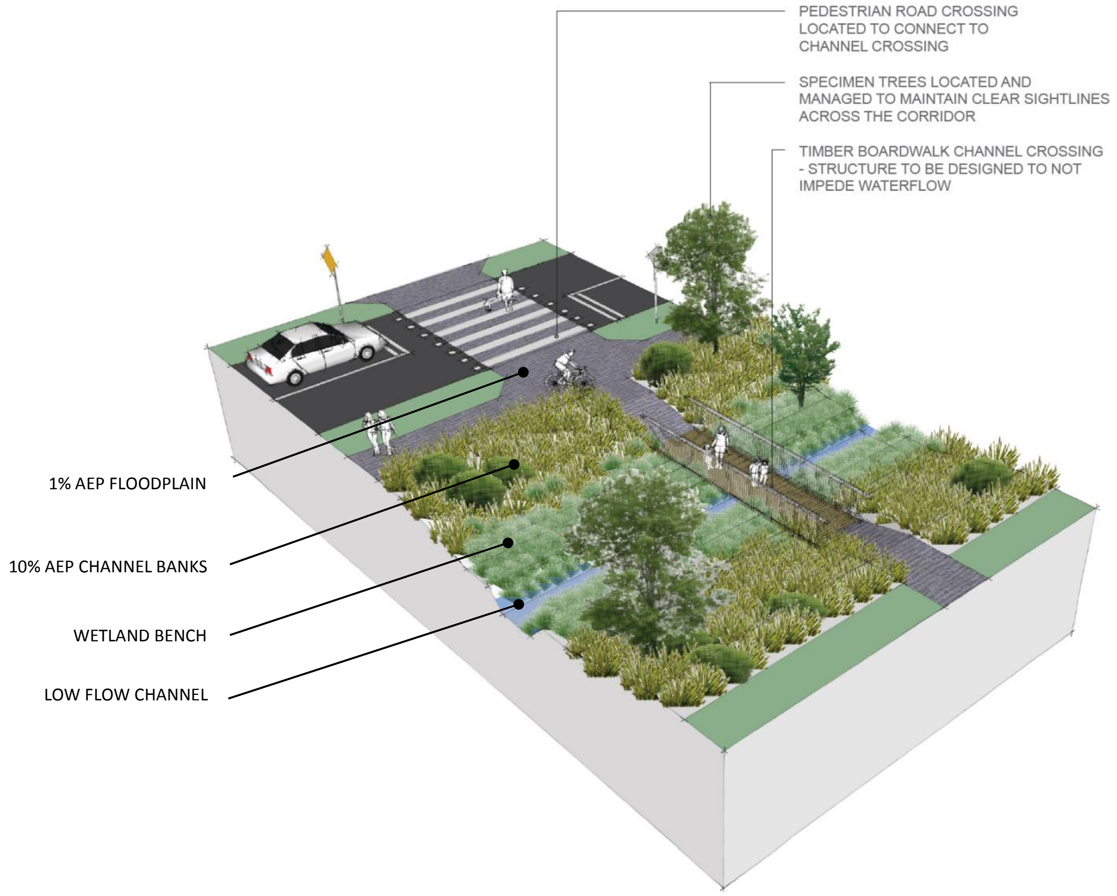


Figure 6: Artist depiction showing different channel zones

4 ADDRESSING CHALLENGING GROUND CONDITIONS

Unlike most other urban open channels, this project will be constructed in peat in close proximity to existing and planned residential development. The result is that dewatering and subsequent settlement is predicted to occur as a result. Although the benefit would then be to provide improved ground conditions over a long-term basis (the reason most farmers will construct open channels in peat), in a built environment and a future development setting this requires special consideration and significant mitigation measures to be employed.

4.1. GEOLOGICAL SETTING

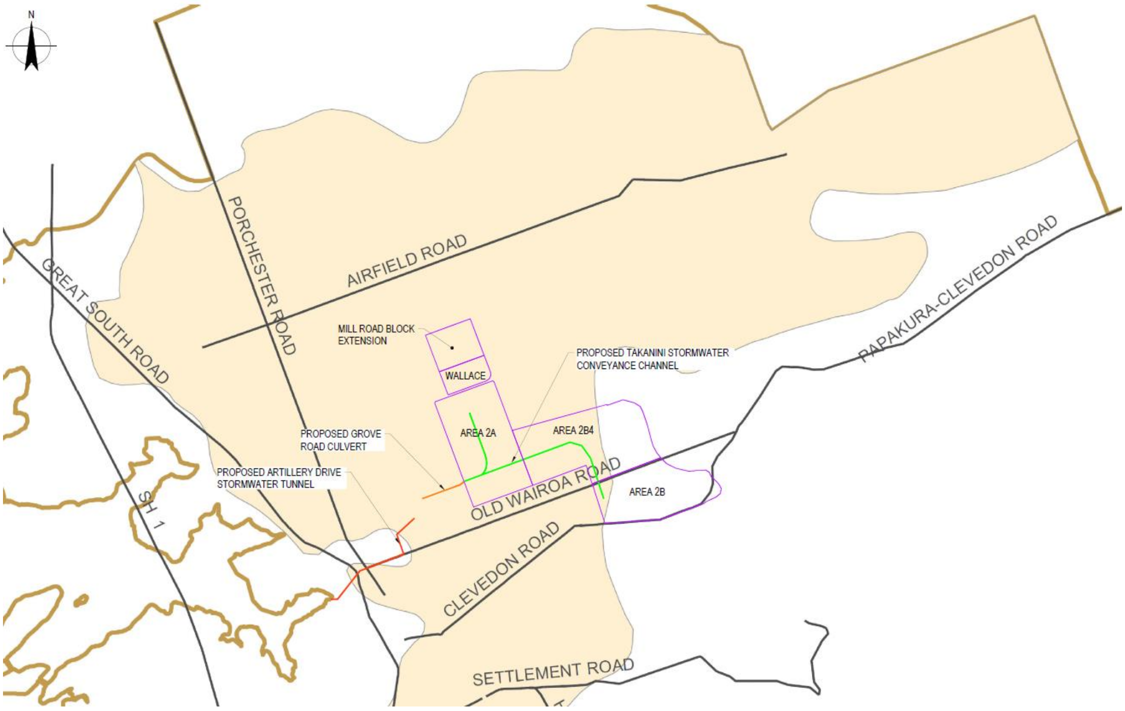
The geology in the area of the proposed conveyance structure as presented in the Geology of the Auckland Area, 1:250 000 (Edbrooke, 2001) as Recent Alluvium and Puketoka Formation Alluvium comprising soft peats, organic silts and sands. In the eastern portion of the alignment residually weathered East Coast Bays Formation are mapped, comprising stiff clays and silts and clays of the Waitemata Group, which itself comprises weathered volcaniclastic turbidite sandstones and mudstones. Soils and rock of the Auckland Volcanic Group are also present near the site.

Various investigations for developments in the Papakura area have been undertaken and reported. Previous investigations in Puketoka Formation Alluvium have encountered alluvial and reworked ignimbrite sediments, interbedded with peats to greater than 10 m depth. These sediments typically comprise soft, sensitive clays, silts and pumiceous sands with soft organic (peaty) silt and peat strata with typical Scala penetrometer test (SPT) 'N' Values of <1 to 5.

4.2. EXTENT OF PEAT

The Papakura District Peat Area Stormwater Discharge Review (PDP, 2006) reviewed soil characteristics in the Papakura District in relation to disposal of stormwater and mapped the extent of peat as part of their review ((PDP, **2006)**

Figure 7). The peat is shown to extend throughout Areas 2a and 4 but does not extend significantly into Area 2a. The majority of the conveyance channel is within the inferred peat zone.



(PDP, 2006)

Figure 7: Extent of peat

4.3. GROUNDWATER

In general terms the groundwater level is expected to be near the surface in the alluvium deposits and seasonally fluctuating. For the most part the entire thickness of peat is saturated with the water table lying close to the upper boundary (PDP, 2006). Generally groundwater in the area of the proposed conveyance channel varies from around 1 to 3 metres below ground surface, fluctuating around 1 metre seasonally.

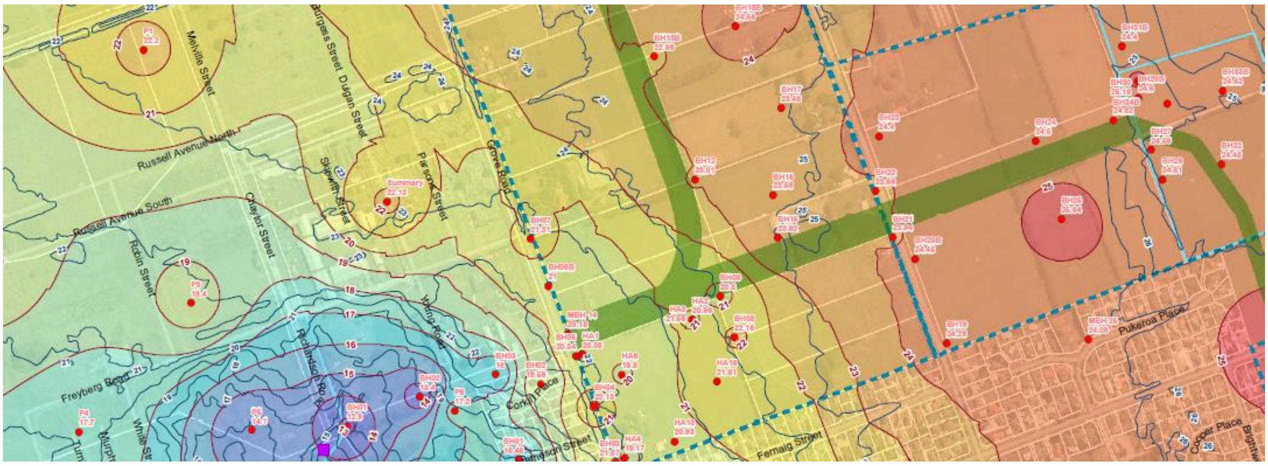


Figure 8: Average Inferred Ground Water Levels (RL, red contours) and Surface Contours (RL, blue contours).

Generally groundwater has on average been encountered at 1m below ground level (BGL) to the east of the alignment, and 2m below ground level in the west. There is a topographical low around the bend to the north east of the alignment, where groundwater has been encountered 0.5 m below the ground level. Groundwater levels surrounding the McLenan Wetland average 3-4 m BGL on average, and show a rising trend towards Grove Road some 420 metres away. The main drawdown effect of the wetland appears from the preliminary contouring to be around 200 to 250 m from the wetland.

Generally the seasonal variance in the area has been shown to be 1-2 m on average, however more extreme variance from 5 m BGL to 0.5 m BGL has been observed. This more extreme variance, is likely due to local drainage from factors such as trees, or existing drainage pathways, such as table drains and existing pipe networks.

A network of groundwater monitoring points has been installed across the site to determine the site specific minimum and maximum groundwater levels.

Figure 9: Cosgrave Road Table Drain

4.4. DISCHARGE TO GROUND

The Papakura District Peat Area Stormwater Discharge Review (PDP, 2006) found that the majority of stormwater in the undeveloped areas of the Takanini 2a2b and surrounding rural areas enters the ground via direct infiltration. Impervious surfaces in areas designated as rural discharge to ground soakage or open channels. Soakage test results indicate some of the highest soakage rates were found within peat areas. However, sample testing indicated the peat also had low permeability.

Geological units described generally as peat in this area consist of a material that ranges from humic, fibrous peat to amorphous organic clay and are generally horizontally stratified, somewhat explaining the variance in permeability.

5 DEWATERING AND SETTLEMENT PREDICTIONS

5.1. HYDROGEOLOGICAL MODEL

A Plan Pseudo 3D hydrogeological model (part of the SEEP/W groundwater modelling software) was developed by GWS to determine the extent of drawdown. Both short and long-term drawdown effects were considered. Short term drawdown is largely affected by temporary drawdown as a result of temporary works construction, particularly at the proposed culvert locations, where the initial cut exceeds the depth of the proposed channel to enable ground improvement to take place.

Long term groundwater drawdown was determined to develop over 30 years.

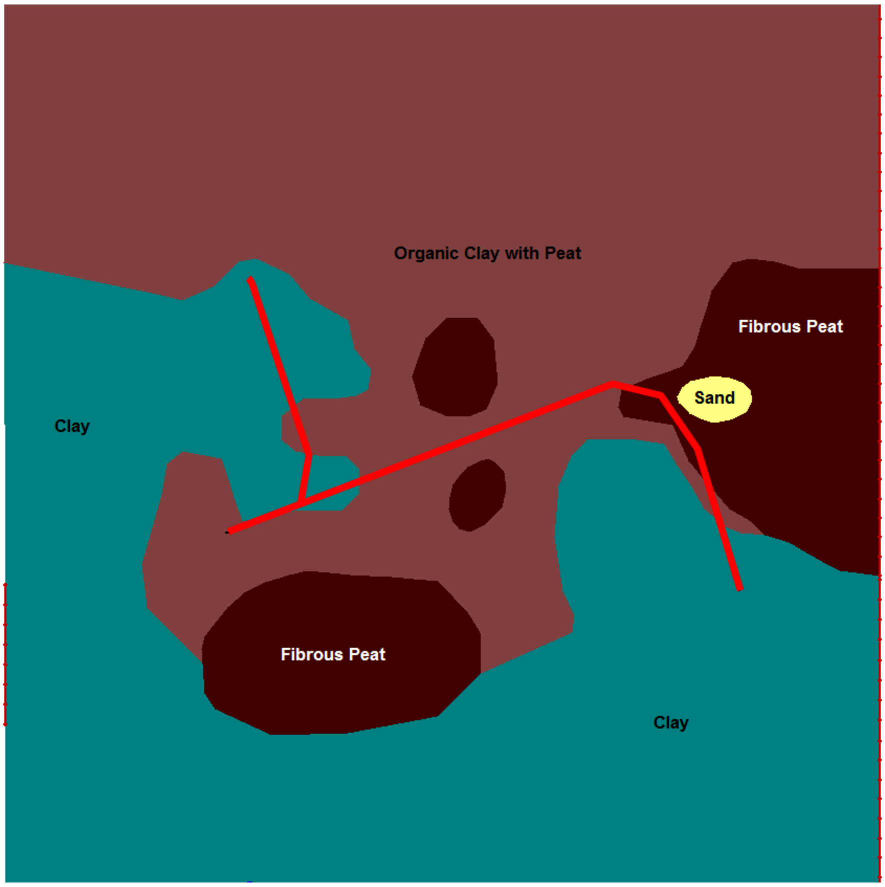


Figure 10: Plan Pseudo 3D model (2 km wide)

5.2. MATERIAL PROPERTIES

The adopted model permeability values were assumed on the basis of the derived summary values from the in-situ testing on the piezometers.

Water content values were adopted from the laboratory testing or generic assumed values.

Table 2: Assigned material properties

Material	Kh *(m/d)*	Ratio	Kv *(m/d)*	Water content *(%)*
Sand	8.64	0.5	4.32	0.38
Fibreous peat	4.32	0.2	0.864	0.44
Sandy silt	0.1728	0.1	0.01728	0.43
Organic clay	0.0432	1	0.0432	0.43
Clay	0.000864	1	0.000864	0.39

5.3. GROUNDWATER INFLOWS

Based on a unitised flow of 0.2 m3/d/m of channel length and assuming 2100 m length, the predicted long term groundwater seepage into the channel would be in the order of 420 m3/d. During construction short term inflows could be up to 1000 m3/d assuming the entire channel length was contributing. In reality the channel will be constructed in stages and these values would be considerably less. Excavation length and staging is to be the subject of further assessment in the detailed design stage.

5.4. DRAWDOWN EXTENT

The drawdown effects associated with the dewatering by the proposed channel can be summarised by stating:

- Within a 75 m zone each side of the channel the drawdown profile maximum depth is driven by construction levels.
- Beyond a 75 m zone drawdown profile maximum depth is driven by the permanent drawdown level or the height of the weirs.
- Maximum drawdown distance is driven by the permanent drawdown level and extends up to 300m from the channel.

The drawdown profiles presented in the figures below show the range of results derived and provides an envelope of drawdown defined by the above criteria (i.e. encompasses the short term maximum drawdown effects closest to the channel, and long term drawdown effects furthest from the channel).

5.5. WATER LEVELS

Plan Pseudo 3D model RHS and LHS constant heads were adjusted to attain a reasonable overall calibration to the observed groundwater level distribution of the unconfined aquifer as represented by the measured groundwater levels obtained from the installed piezometers.

At the time of this paper preparation, limited long term monitoring data is available to demonstrate the extent of seasonal fluctuation for the different geological units, nor is any data available that can be used to assess the sensitivity of changes in groundwater levels relative to recharge events. Further monitoring and assessment will enable this evaluation of transient conditions to be undertaken prior to detailed design.

5.6. MODELLED SCENARIOS

Model scenarios were analysed based on short term construction invert levels and long term weir controlled invert levels. In addition, a range of scenarios were tested to assess the drawdown profile sensitivity to K ratio (vertical permeability) and recharge.

5.7. RESULTING DRAWDOWN

Drawdown across the channel is variable but the general drawdown trends as follows:

- Drawdown extents are as much as 250 to 300 m from the channel.
- Up to 1- 1.5 m of drawdown is expected within 50 m of the channel.
- The worst drawdown (i.e. > 0.25 m approximately) is within 0 to 150 m of the channel.

5.8. SENSITIVITY

Following initial calibration of the model boundary conditions to replicate observed groundwater level conditions a range of model scenarios were run that assessed the sensitivity of the models to various conditions.

The Plan model is shown to be sensitive to the anisotropy ratio (i.e. Kv value) and to the amount of rainfall recharge applied. At this time neither of these parameters are well constrained and, as mentioned previously, further work is planned that will enable a better assessment of the relationship between groundwater levels over time; the amount of recharge entering the system; vertical permeability values and changes in storativity (i.e. water retention curves). Longer term monitoring of groundwater levels, rainfall and soil moisture, combined with a trial excavation and pump test, will provide a better assessment of the sensitive model parameters and provide data to which transient groundwater conditions can be calibrated.

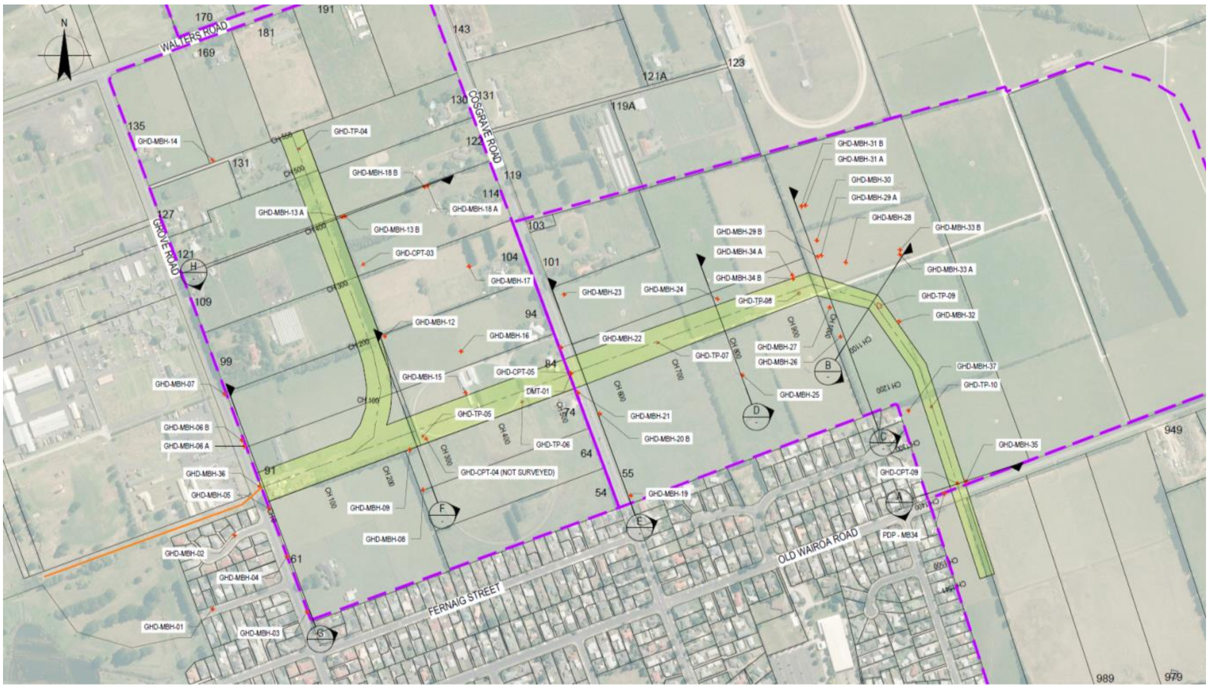


Figure 11: Model and Settlement Section Locations

5.9. PRIMARY CONSOLIDATION

Peat deposits comprise of organic content primarily derived from dead vegetation therefore undergoes continuous biological decomposition that causes progressive decomposition of the peat fabric, reduction in fibre and organic content and bio gas generation.

Depending on the degree of decomposition, the organic solids can exist as fresh fibres, slightly decomposed or completely decomposed (amorphous). The degree of decomposition has significant impact on the engineering properties.

More decomposed peat generally undergoes lower primary and secondary consolidation compared to lesser decomposed peat. Therefore, the compressibility varies with the decomposition and has significant impact on total and differential settlements. Settlements can vary within a short distance. The thickness of the peat layers also has impact on the settlements. The settlement due to decomposition cannot be easily estimated. The drawdown consolidation settlement resulting from reduction in moisture content can be estimated via soil mechanics consolidation theory.

5.9.1. METHODOLOGY

Primary consolidation settlements have been assessed using coefficient of volume compressibility (Mv) method. The value of Mv varies with effective overburden pressure thereby incorporates the effects of consolidation of the soils. Effective overburden pressure increases as the groundwater drops, hence removing the buoyancy effect.

$$S_C = M_v \times dp \times H$$

Where: M_v = average coefficient of volume compressibility over depth (m^2/MN)

dp = average increase in vertical effective stress over depth (kPa)

H = compressible layer thickness (m).

The ground surface settlements were estimated at 20 m intervals to a lateral distance of 300 m from the edge of the channel for a range of sections A-A to H-H (refer Figure 11). The average depth to peat was used for each section when calculating the primary settlement; settlement was assumed to occur in the upper of the peat when estimating the primary settlement.

Estimated settlements have been prepared to assess the effect of settlement without any mitigation measures applied. Short term settlements can be mitigated as part of the temporary works construction design. Long term settlements will be required to be mitigated or managed.

The following figure represents a sample of the groundwater drawdown estimates generated at selected chainages intervals along the alignment. Brown lines represent the surface RL, with the blue dashed line representing the assumed average groundwater level in RL. Corresponding tabulated data of the calculated short and long term settlement estimates at that chainage is also presented below. A map showing the predicted settlement contour is also being developed based upon this information for use during the detailed design and consenting process.

5.9.2. CHAINAGE 1000

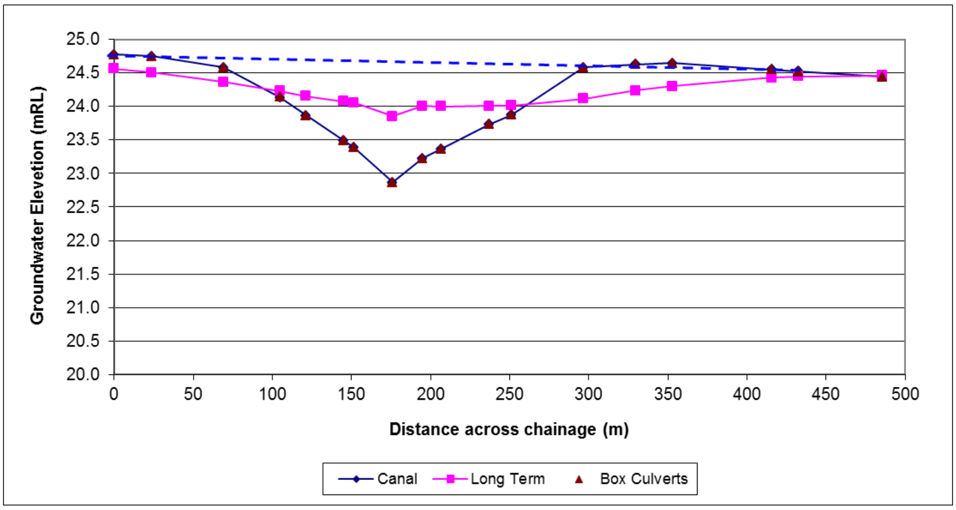


Figure 12: Estimated groundwater drawdown at section B-B' (chainage 1000)

Table 3: Estimated short term groundwater drawdown at section B-B' (chainage 1000)

Distance from Channel Edge (m)	Short term Drawdown (m)	Estimated Primary Settlement (mm) $M_v = 2.07\ (m^2/MN)$
0	Unloading	Unloading
20	1.45	137
40	1.20	113
60	0.92	86
80	0.67	63
100	0.40	38
120	0.13	12
140	0.06	5
160	0.05	5

5.10. SECONDARY CONSOLIDATION

5.10.1. METHODOLOGY

The secondary consolidation causes part of the settlement which takes place after hydrostatic excess pore pressures has fully dissipated, commencing after primary consolidation is complete.

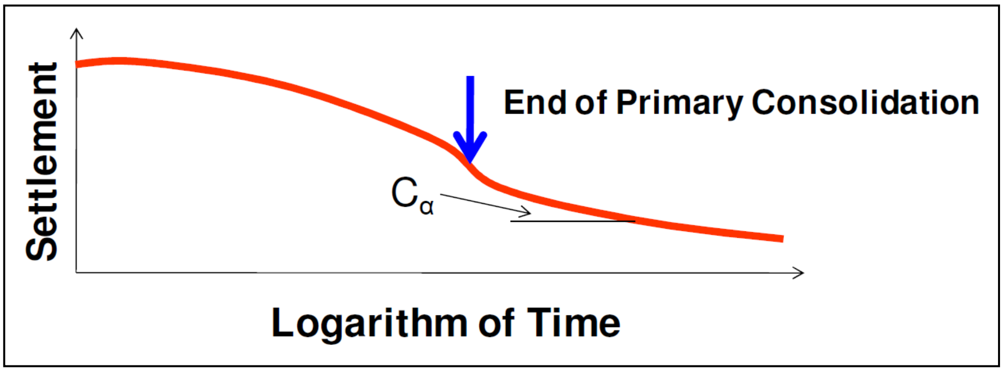


Figure 13: Time versus settlement curve representing coefficient of secondary settlement.

$$Secondary\ Consolidation\ Settlement\ (S_C) = C_\alpha.H.log_{10}\left(\frac{t_{sec} - t_{prim}}{t_{prim}}\right)$$

Where C_α = Coefficient of secondary consolidation

H = Layer thickness (m)

T_{sec} = Time taken for secondary consolidation (years)

T_{prim} = Time taken to complete primary consolidation

The lab C_α was used as a lower bound value, and the C_α calculated from the moisture content was used as an upper bound value. The average of the two was used to estimate the secondary settlement.

$$C_\propto = 0.00018 \times M_C(\%)$$

Where:

C_α = Coefficient of secondary consolidation

M_C = moisture content (%)

It is estimated that secondary consolidation settlement is to occur at a rate of less than 3 mm per month for a period of 50 years.

5.11. SETTLEMENT PREDICTION RESULTS

It is important to consider that settlement due to decomposition is impossible to estimate accurately, other than to say it will occur, and will occur to a greater magnitude generally in areas of less decomposed peat. This settlement must not be ignored, especially in the more fibrous peat areas, and is additional to the predicted consolidation settlement.

It should also be noted that general settlement predictions have a large margin of error for accuracy.

The extent of predicted primary settlement due to long term drawdown is currently being developed. This will be used to establish the long term effect that must be assessed and controlled. Secondary consolidation is already occurring in the area, but will be further triggered as a result of the dewatering. The secondary consolidation is of large magnitude, but occurs very slowly, i.e. less than 3 mm per month. Currently modern lightweight buildings have been consented on this basis in the area; however the effect of this settlement needs to be considered in the context of the tolerance of existing structures.

Differential movements are often the most adverse settlement effect for existing foundations. AS2870 -2011 provides tolerable limits for differential movements, which depend on the form of construction, surface finish and the actual detailing of the superstructure. This information is provided in Table 4.

Table 4: Maximum Design Differential Footing Deflection (Δ) for Design of Footings and Rafts

Type of Construction	Maximum Differential Deflection , as a function of span (mm)	Maximum Differential Deflection (mm)
Clade Frame	L/300	40
Articulated masonry veneer	L/400	30
Masonry veneer	L/600	20
Articulated full Masonry	L/800	15
Full masonry	L/2000	10

Most of the existing buildings and infrastructure in the area will have experienced some form of settlement induced stress in the past, and are likely still experiencing this ongoing effect. At detailed design the existing infrastructure within the area of effect needs to be specifically assessed for tolerance, and then mitigation measures examined to reduce the settlement within these tolerable limits.

The settlement prediction is dependent on the groundwater drawdown. Currently the groundwater drawdown has been estimated from average groundwater levels. These levels vary considerably seasonally and as such at least for a short period expose the soils to a higher stress regime than has been assumed. More work is required at detailed design to further understand the existing groundwater drawdown in the area, and to further refine the ground and hydrogeological models.

Light weight buildings should be adopted within the area of settlement effect. The light weight buildings are expected to reduce the surcharge loading under

12 kPa. Further analysis including a sensitivity analysis with reduced surcharge loading will be conducted during the detailed design stage, however to prevent excessive total and differential settlements one or combination of the following methods could be used for new development:

- Provision of a thick raft foundation with a thick slab or with deep beams in two or three directions on improved ground.
- Raft slab on a geogrid raft.
- Provision of jacking pockets, or brackets in columns to relevel.
- Provision of additional loading on lightly loaded areas in the form of kentledge or embankments.
- Piles founded into rock

It is recommended that consideration should be given to adopt light weight buildings with raft foundations founded on approximately 0.5 m thick geogrid raft layer as a standard design, as has been historically under the Papakura District Council development codes.

6 MITIGATING SETTLEMENT EFFECTS

Numerous possible mitigation measures for groundwater drawdown reduction, and hence settlement reduction exist. These need to be considered during detailed design in a time, quality, and cost framework.

6.1. CONTROLLING DRAWDOWN

To minimise groundwater drawdown a series of weirs have been incorporated along the channel length at 100 m centres. The weirs retain a total water depth of 0.8 m with a notional low flow channel 0.3 m below this. The bed slope of the channel is then reduced from about 0.20% to 0.18% along the length of the channel. This allows permanent pools to form which extend from weir to weir, even without the inclusion of surface water runoff.

6.2. CUT OFF

Due to the proximity of existing buildings, current development in progress, future consented development and the extent of predicted settlement, it is currently recommended that in certain areas of the channel an effective cut-off is provided. This cut off will need to be refined and designed at detailed design stage. Cutoffs will need to be considered for the effect on Cosgrave and Grove Roads, and the associated in ground infrastructure.

Possible options for a cutoff include double steel or Vinyl sheet piles, or a clay column to a depth of at least 6m below ground level.

6.3. PRELOADING

Preloading or pre-compression increases the bearing capacity and reduces the compressibility of weak ground. It is achieved by placing a temporary surcharge on the ground prior to the construction of the planned structure and the total and differential settlements of foundations on soft compressible soil can be reduced by preloading the area of the structure. The surcharge generally consists of earth fill. The preloading material is kept in place until level measurements that the settlement has flattened or that settlement has decreased to a very slow rate.

When preloading soft clays the rate of settlement may be rather slow, requiring the load to be in place for many months. In such cases consideration can be given to accelerating the rate of consolidation by introduction of vertical drains and vacuumed consolidation.

The preloading area must exceed the limit of the final structure in such a way that the stress induced at any depth in the foundation soils by the preload under the edge of the proposed structure is uniform. This should be at least equal to, or preferably greater than, the final stress at the location. In addition, it is desirable to extend the preload area to allow for possible future extension of the proposed structure.

It is currently recommended that generally, and where possible, the area effected by settlement should be preloaded with 1.5m high fill (approximately, density18 kN/m3) for at least 12 months. General preloading pre development, will significantly accelerate the adverse effects that result from the works. All areas within the designation, where the maximum settlement effect will occur, will require preload, and consideration to temporary designation outside this area will be undertaken as part of detailed design. The advantage is for preloading to occur prior to development of the land, to allow consolidation and strengthening of the ground. Undertaking preloading at the time of channel construction would provide a temporary disturbance, but long term benefit.

6.4. EXCAVATION FOR CULVERTS

The project will involve the construction of installation of culverts in saturated peat under Grove and Old Wairoa Roads. The drawdown and the resulting settlements will be higher than those of long-term channel draw down.

Temporary works mitigation measures such as ground freezing or an equivalent control mechanism to control ground water seepage and drawdown during construction will be required.

For permanent works design, consideration to the use of lightweight polystyrene blocks (EPS/Poly Rock) will be given at detailed design. Further work on the impact of dewatering beyond the culvert structure, and on the effect on the existing infrastructure will be required.

7 CONCLUSIONS

- Growth in Auckland demands additional land to be made available for development. This demand causes pressure to utilise areas that previously have not been considered for development, often for geotechnical or flooding reasons.
- Open watercourses can be used as viable, and potentially less expensive alternatives to piped systems. They offer considerable additional benefits to that of a piped network, most significantly ecological and amenity value. However the model for ownership and delivery of any constructed watercourses will need to be further considered.
- The development of open watercourses in peat near existing and planned future development needs to be carefully considered due to drawdown effects and the resulting primary and secondary settlement that may occur as a result. Mitigation measures need to be included in the design of any such channels. In greenfield sites the resulting effects are probably considered to be negligible.
- Although the potential effects of drawdown relating to the Takanini stormwater conveyance channel are considerable, they can be successfully managed and are no less than what may be observed through the implementation of a piped system. In addition, an open watercourse is more resilient to settlement effects and increased flows.
- It is likely that primary and secondary consolidation is already occurring within the Takanini area as a result of open watercourses and groundwater ingress through piped networks. Regional drawdown appears to have been induced by the construction of the McLennan wetland.
- For the Takanini Conveyance Channel, a wider designation than would normally be required has been allowed for to account for the worst case long term estimated primary settlement effects, and short term temporary works settlement effects.
- It is estimated that secondary "creep" settlement will occur at a rate of less than 3 mm per month for a period of 50 years over a regional area, which is tolerable for modern lightweight structures, and is currently being experienced and accommodated by built buildings and infrastructure in the area.
- Most existing and future structures could tolerate 20mm of settlement between adjacent columns. Raft foundations could tolerate settlement of 40-50mm
- Preloading prior to development of the land, will allow consolidation and strengthening of the ground. Undertaking preloading at the time of channel construction would provide a temporary disturbance, but long term benefit.

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