SOAKAGE DISPOSAL IN HIGH GROUNDWATER

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ABSTRACT

The Christchurch Southern Motorway Stage 1 (CSM) is a $140 million, 10.5km motorway, currently being constructed between Barrington and Halswell. The stormwater design provides treatment and attenuation of stormwater prior to disposal to soakage or surface water, with the soakage disposal being a consent requirement in areas where soils allow, to enhance groundwater recharge.

This paper describes the design challenges with stormwater disposal to soakage for the CSM, and how these were resolved. Due to high groundwater levels, there is a risk of groundwater levels rising close to the base of the soakage disposal areas in extreme conditions. In these circumstances, conventional assumptions about unconstrained vertical discharge to ground no longer apply, and an understanding of horizontal groundwater movement and groundwater mounding was required. The combined probability of an extreme high groundwater level and a large design storm also needed to be understood, as well as the consequences of such events for design. This led to risk management decisions around the implications of low probability but high consequence events, and design of contingency measures to address these. This paper will examine the issues, the design approach, and the solutions adopted.

KEYWORDS

Soakage, groundwater, high groundwater, groundwater mounding

PRESENTER PROFILE

Kate Purton is a Senior Civil Engineer with 12 years' experience in water, wastewater and stormwater. Kate was the lead stormwater designer for the CSM design.

1 INTRODUCTION

The 10.5km Christchurch Southern Motorway Stage 1 (CSM) is currently under construction in south-west Christchurch, approximately 5km from the central business district. The stormwater management for the motorway includes conveyance, treatment and attenuation, prior to discharge to ground or surface water.

This paper describes the design process, challenges and solutions for the basins discharging to ground via soakage.

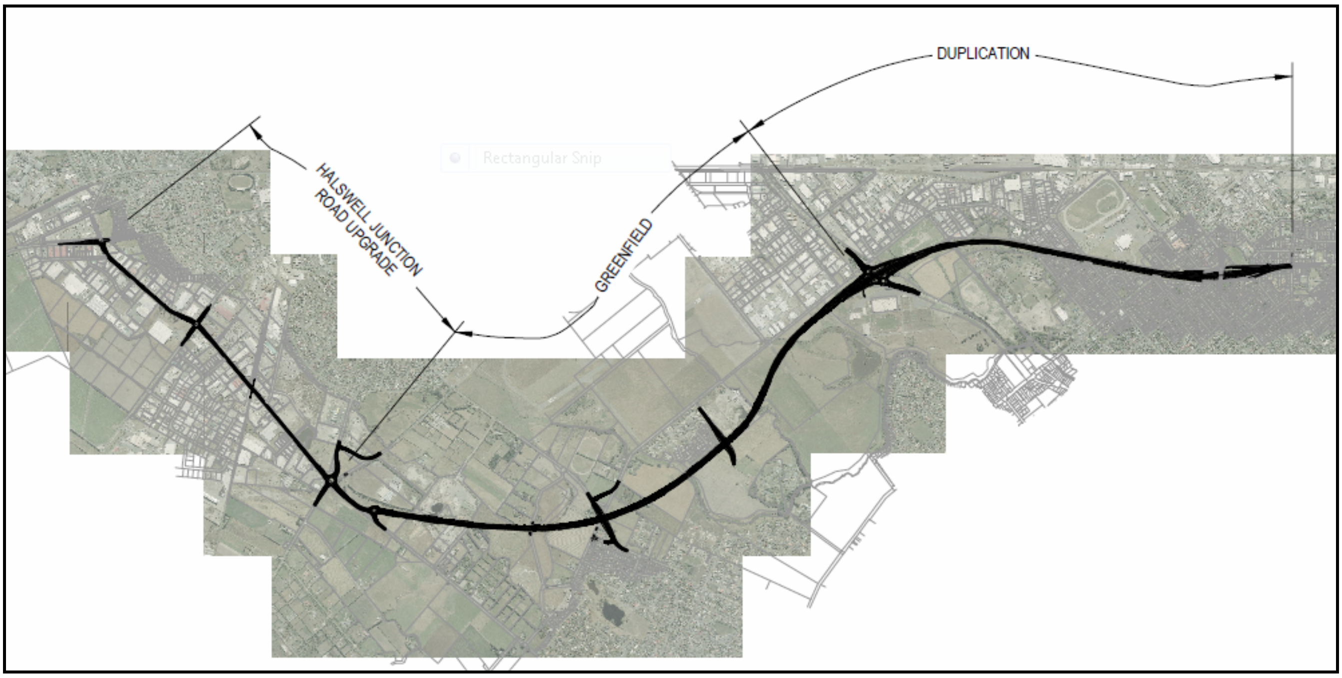
Soakage disposal retains water within the natural hydrological cycle, providing shallow groundwater recharge, which in turn contributes to base flows in waterways. Where ground conditions are suitable, and suitable treatment is provided, soakage disposal is Environment Canterbury's preferred disposal option.

2 CSM PROJECT BACKGROUND

2.1 CSM OVERVIEW

When completed the CSM will duplicate the existing 3km long Southern Motorway, extend the motorway 5km over greenfields, and upgrade 2.5km of Halswell Junction Road to connect to State Highway 1 (SH1). An overview plan of the CSM is included in Figure 1.

Figure 1: CSM overview plan



This project is being procured by the New Zealand Transport Agency (NZTA) through a design and construct contract in which Fulton Hogan are the Contractor and Beca Infrastructure Ltd (Beca) the Contractor's designers. Opus International Consultants Ltd (Opus) is the Principal's Agent. Pattle Delamore Partners Ltd (PDP) is the independent peer reviewer for stormwater, while URS New Zealand Ltd is the peer reviewer for the other disciplines (with Tonkin & Taylor its sub-consultant for geotechnical peer review).

2.2 PROCUREMENT & DESIGN PROCESS

A Specimen Design for the CSM was carried out for NZTA by Opus in 2007/08. This was used to set the designation boundary and obtain consents from Environment Canterbury (ECan).

In 2009 Fulton Hogan, working with Beca, prepared a Concept Design, which formed the basis of its design-build Tender. Fulton Hogan's tender was successful, and in early 2010 work began on the Detailed Design. Construction of the CSM started in late 2010 and is due to be completed in early 2013.

3 CSM STORMWATER DESIGN

3.1 CSM STORMWATER SYSTEM

3.1.1 OVERVIEW

The CSM stormwater management system can be considered by motorway section, with a different approach for each section: the Halswell Junction Road Upgrade section, the Greenfields section, and the Duplication section.

In the Halswell Junction Road Upgrade section, stormwater is conveyed by a conventional piped system to the upgraded Halswell Junction Road wet pond where it is treated and attenuated before discharge to surface waters.

In the Greenfields section stormwater is conveyed, treated and attenuated either by conveyance swales and dry basins disposing to ground or surface water, or by attenuation swales discharging to surface water.

In the Duplication section stormwater is conveyed, treated and attenuated by attenuation swales discharging to surface water or the CCC piped stormwater system.

This paper focuses on the dry basins disposing to groundwater (soakage disposal), which are located in the Greenfield section.

3.1.2 BASINS DISCHARGING TO SOAKAGE

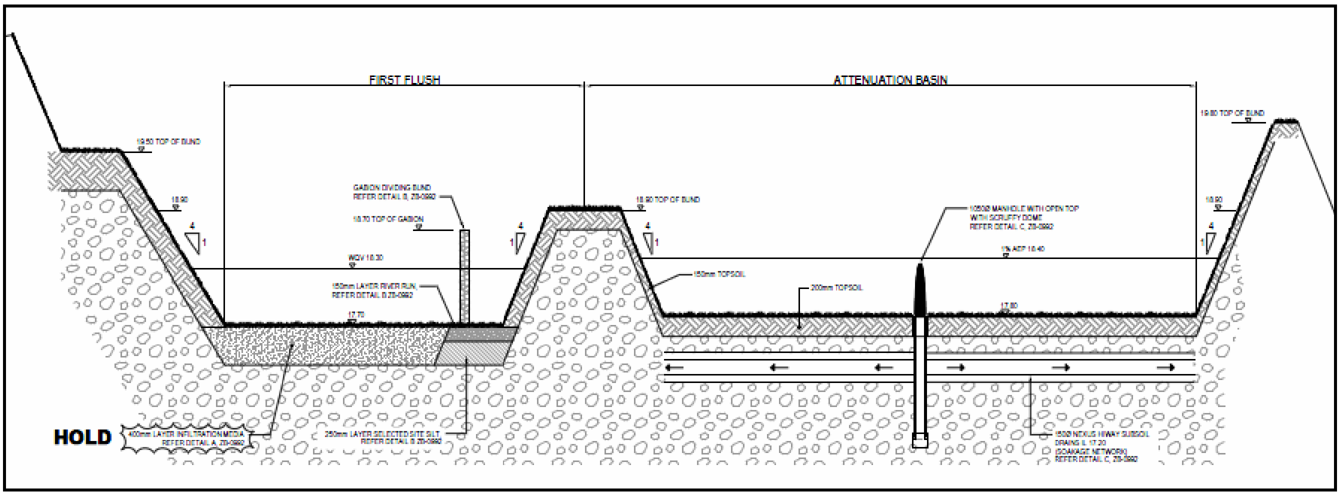
There are four dry basin systems discharging to soakage, each named after adjacent roads or landowners: Mushroom Basin, Lee Basin, Carrs Basin, and Musgroves Basin. The term "dry basin" refers to grassed basins which are dry between events.

Stormwater up to the critical duration 2% Annual Exceedance Probability (AEP) event is conveyed to these basins by swales and pipe reticulation. Secondary flow from the 2% AEP up to the 1% AEP critical duration is conveyed within the designation, and either disposed of to surface water or retained. With the exception of the Lee Basin, the basins have overland flow paths to surface waterways for over-design events.

Each soakage basin system consists of two basins, an infiltration basin which provides treatment of the first flush volume via infiltration through sand infiltration media, and an attenuation basin which provides storage prior to discharge to ground. The exception to this is the Musgroves Basin, where only the first flush basin discharges to ground, while the attenuation basin discharges to the adjacent waterway, Dry Stream.

The discharge to ground occurs via constructed soakage fields or, in areas of high permeability, direct to the underlying gravels as shown in Figure 2.

Figure 2: Cross-section of soakage basin system with attenuation basin discharging direct to ground.



The paper focuses on the overall performance of the soakage basins and their ability to discharge to groundwater, rather than the performance of the sand infiltration media in the first flush basins.

3.2 COMPLIANCE & CONSTRAINTS

The consent conditions for the CSM project required that the basins discharging to ground:

- Contain the critical duration 2% AEP event without spilling
- Drain down within 48 hours after a storm event
- Are planted with grass or other vegetation

The design was also constrained in terms of the area of land available, as the NZTA designation for the motorway had already been set.

4 STANDARD DESIGN ASSUMPTIONS FOR SOAKAGE DISPOSAL

4.1 SOAKAGE TESTING & DESIGN SOAKAGE RATES

In order to determine the soakage rate for a given location, site soakage testing needs to be carried out. A common test procedure involves a test pit or borehole excavated at the proposed soakage pit location and depth. In advance of the soakage test, water is added to the hole for a minimum of four hours to try to saturate the soil. The hole is then filled with water and the drop in water level is measured over time, and plotted (drop in water level versus time) on a graph. The minimum observed soakage rate, or minimum slope of the graph, is the measured soakage rate in mm/hour.

The measured soakage rate is divided by a factor of safety to arrive at a design soakage rate. This is due to the effects of clogging of the soils over time, and the limitations of the test in terms scale and partially unsaturated conditions. The recommended factor of safety varies between guidelines from 1 to 25. The New Zealand Building Code Compliance Document Clause E1 Surface Water recommends a factor of safety of 1, i.e. that this test rate is adopted as the design soakage rate. Auckland Regional Council's TP10 (ARC TP10) recommends a factor of safety of 2. For treated wastewater soakage, the US EPA recommends a factor of 10 to 25. Christchurch City Council (CCC) generally adopts a factor of safety of 3 for soakage systems.

4.2 GRADIENT TO GROUNDWATER

In adopting a design soakage rate based on a small scale soakage test, and using that directly as the disposal rate below the soakage field, there is an underlying assumption that the groundwater level is sufficiently low that the water discharged can be absorbed by the directly underlying unsaturated zone (i.e. the effect of groundwater movement within the saturated zone, which is not measured by the soakage test, is neglected). In other words, a hydraulic gradient of 1 is commonly used. If the water table is high, and volume to be discharged is large, this may not be the case, and the hydraulic gradient can become significantly lower as soakage occurs laterally, reducing the drain-down rate.

To achieve a soakage rate that is not affected by groundwater, the groundwater depth below the basin needs to be sufficient. The depth required depends on the rate and duration of discharge and hydraulic parameters (porosity and hydraulic conductivity). For a porosity of 0.4, neglecting groundwater movement within the unsaturated zone, the groundwater depth below the basin needs to be 2.5 times the depth of the basin. This means that a 1m deep basin needs more than 2.5m of unsaturated soil above the groundwater table.

5 DISCOVERIES DURING DETAILED DESIGN

5.1 INVESTIGATIONS

During the Specimen Design phase, ground investigations were carried out by Opus, including test pits and shallow soakage tests. This information was used to determine soakage rates for the Concept Design. The water levels measured in these test pits, and local groundwater level data, were used to determine groundwater levels at each site.

Early in the detailed design process, a more detailed search of long term local groundwater level data was carried out. Test pits and soakage tests were also carried out at each of the soakage sites, and soil samples taken from each test pit for laboratory grading tests, and two piezometers were installed at each site. These piezometers were then monitored fortnightly to provide site specific data. The results of these investigations are summarised below.

5.2 LONG TERM GROUNDWATER RECORDS

In searching for available groundwater level data during detailed design, it became apparent that directly relevant long term site specific groundwater monitoring data was not available. In Canterbury, records for deeper groundwater, that is the first to fourth aquifers, are readily available from ECan, however shallow groundwater records are much less common. Shallow groundwater records also tend to be project (site) specific and therefore recorded over shorter periods. Shallow groundwater monitoring from the initial investigations for the CSM covered a period of approximately two years. A number of local historical records were identified from CCC shallow groundwater monitoring wells, however these records were of various lengths and ended in approximately 1995, so were not able to be correlated to the more recent piezometer records.

A margin was added to the measured groundwater level, based on the variation within the historical records available, and a maximum groundwater level was assumed at each basin. The design soakage invert levels at the basin were then set above this level, and it appeared that simple vertical soakage would be achievable.

5.3 SOAKAGE TEST RESULTS

Generally two soakage tests were carried out per site. The soakage test results, with the exception of Musgroves Basin, provided acceptable soakage rates. A factor of safety of 3 was generally applied to the results to determine design soakage rates, except where very high soakage rates were measured a maximum design soakage rate of 300 mm/hr was adopted. The soakage test results and design soakage rates chosen for sizing the basins are summarised in Table 1.

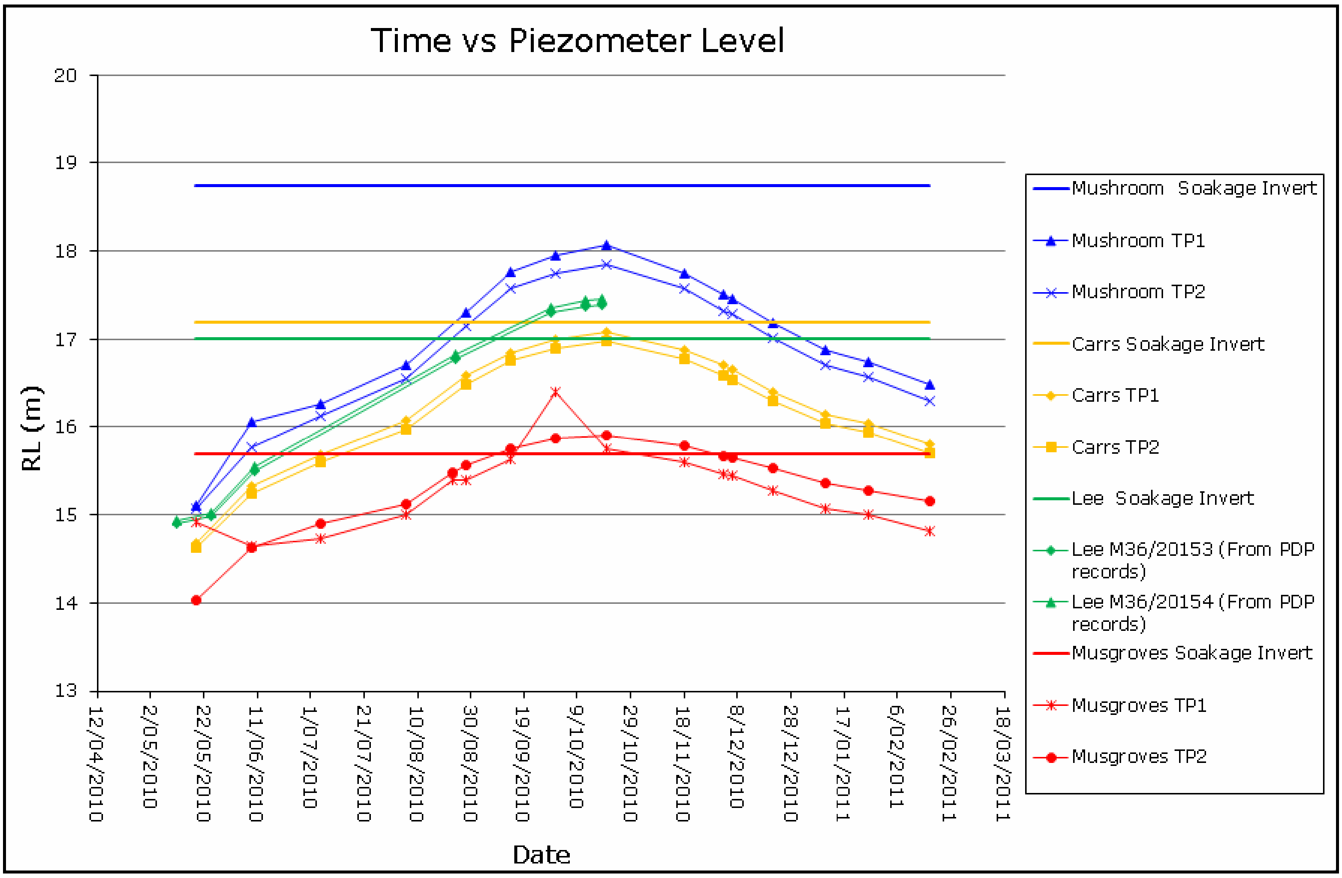
Table 1: Soakage test results and design soakage rates

Basin	Soakage test	Measured minimum soakage rate (mm/hr)	Design soakage rate adopted (mm/hr)
Mushroom Basin	Mushroom SP1	900	300
	Mushroom SP2	1800	
Lee Basin	Lee Basin SP1	3600	300
Carrs Basin	Carrs SP1	450	150
	Carrs SP2	Soakage rate too high to measure	
Musgroves Basin	Musgroves SP1	96	20
	Musgroves SP2	40	Site not used

5.4 GROUNDWATER MONITORING RESULTS

As described, piezometers were installed at each of the soakage basin sites, close to the proposed locations of the soakage fields. The initial piezometer monitoring results were in the expected range, however the measured levels rose over time and revealed groundwater levels much higher than expected. The peak level occurred in October 2010. The piezometer monitoring results, and design invert levels of the soakage fields are shown in Figure 3. Where soakage is direct to the underlying gravels, the design invert level shown in Figure 3 is the invert of the subsoil drains (which discharge the water to ground underneath the basin).

Figure 3: CSM piezometer monitoring results



It can been seen from Figure 3 that as the design was progressing with assumed levels, the measured groundwater levels progressively approached, and in some cases exceeded, the designed soakage invert levels.

With such high groundwater levels, the assumption that the discharged water would be able to be absorbed by the directly underlying unsaturated zone (i.e. a hydraulic gradient of 1) would no longer hold. The standard design approach, assuming simple vertical soakage, would not be suitable.

5.5 BASIN LEVEL CHANGES

The simplest solution to this problem would appear to be to raise the levels of the soakage inverts at the basins. However, the basin levels and soakage invert levels had been arrived at by assessing full hydraulic design gradients from the motorway. Any increase in soakage invert level would require a corresponding increase in motorway level. The motorway levels had been set at the start of detailed design to provide freeboard above 1% AEP flood levels, and geometric design of the motorway was nearly complete. An increase in motorway level would result in the need for additional fill, at a high cost to the project, as well as redesign which would cause delays. Changing the basin levels and motorway levels was therefore a last resort, and a more detailed analysis of the soakage performance was required.

5.6 POTENTIAL EFFECTS OF HIGH GROUNDWATER

The higher than expected groundwater levels raised a number of significant issues.

With high groundwater levels, the base assumptions of the design regarding soakage rates and ability to discharge into an unsaturated zone beneath the basin would not be valid. This could mean that:

- The basins may not contain the 2% AEP event, spilling to their secondary flow paths more often.

- Basin drain-down would occur more slowly. Depending on the time taken to drain, this might cause performance issues with consecutive storms (as the available storage would be reduced by water still in the basins from the previous event). The drain-down could take longer than the 48 hours required by the consents. If prolonged ponding occurred, this might cause issues with grass die-off.

This raised potential issues with consent compliance, with respect to the basins containing the critical duration 2% AEP event, meeting the 48 hour drain-down requirement, and maintaining grass cover.

6 ALTERNATIVE APPROACH

6.1 RISK & EFFECTS ON PERFORMANCE

The return period of the measured groundwater levels needed to be assessed, to determine the likelihood of such high levels occurring again in the future.

In addition to this, the analysis to date had been based on a peak groundwater level coinciding with a 2% AEP storm. The combined probability of a 2% AEP rain storm event and a 2% AEP groundwater level occurring together would be less than a 2% AEP. It was agreed with NZTA that the design case should be an event with a combined 2% AEP, in other words the combination of groundwater and rain storm event which together had a 2% AEP. A combined 2% AEP event could consist of a more common groundwater level with a large storm, or a high groundwater level with a smaller storm.

The effects of this overall 2% AEP event (or events) on performance of the basins needed to be determined. Two-dimensional groundwater modelling was identified as the most appropriate method for assessing the effects of the two principal design scenarios.

6.2 ANALYSIS OF RISK

A shallow groundwater well with a long term continuous record was identified at Weedons Ross Road, West Melton (M35/0931), some 20 km west of the Christchurch airport. With a data set from 1976 to present, this was the closest unconfined monitoring well, with a long term record. Comparison of the records showed that the recent water level variation in this well was very similar to that measured in the CSM piezometers.

An extreme value analysis was carried out on this groundwater level data to determine the return period of the October 2010 groundwater peak. This indicated that the October 2010 peak groundwater level at Weedons Ross Road had a return period of approximately 25 years (a 4% AEP event). Further, from the record, the additional rise to a 2% AEP event was determined.

The likelihood of a high groundwater event coinciding with a storm event was then analysed, in order to determine an overall 2% AEP event or events. This analysis was carried out both from a peak annual 24 hour rainfall perspective to determine a likely groundwater level that might occur at the same time, and from a peak annual groundwater level to determine a likely 24 hour rainfall depth that might occur at the same time. For the analysis, three NIWA Lincoln rainfall gauges were used (4881, 4882 and 17603) as these were the closest long term records to the well site.

In Figure 4, the plot shows the groundwater level coinciding with each annual maximum 24 hour rainfall event analysed, with the recommended design groundwater values marked in red squares for each of the 2% and 1% AEP design rainfall events.

Figure 4: Rainfall ARI vs groundwater level, analysed from an annual maximum storm basis

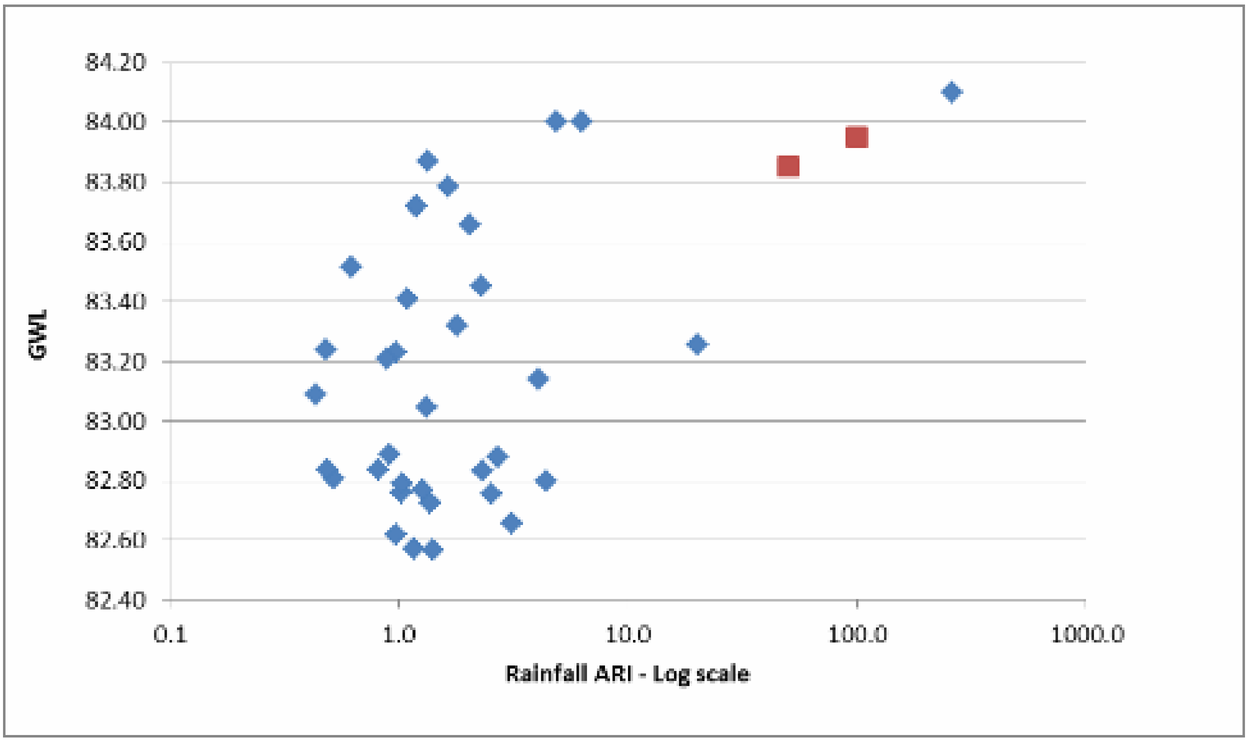


Figure 5 shows the converse plot, with the nearest appropriate 24 hour large rainfall event coinciding with the annual maximum groundwater events. There was less certainty around the most appropriate storm event case to use with the high groundwater level case. Not shown in Figure 5 (due to the altered horizontal scale to give resolution for frequent events) is that there was also one solitary larger rainfall event (60 year ARI) coinciding with a high groundwater level of 84.6m. While a 3 month storm appeared to be appropriate in general, it could be argued that a 3 year ARI storm might be needed as a sensitivity test to reflect the skew imparted by the single larger rainfall event. As a result, it was concluded that two "high groundwater" scenarios were necessary.

Figure 5: Rainfall ARI vs groundwater level, analysed from an annual maximum groundwater level basis

The groundwater and rainfall analysis therefore concluded that three scenarios needed to be modelled:

- 2% AEP storm with "typical" groundwater level (large storm base case)
- 3 month ARI storm with 2% AEP groundwater (high groundwater base case)
- 3 year ARI (33% AEP) storm with 2% AEP groundwater level (high groundwater sensitivity case)

The three cases were modelled for each basin.

6.3 TWO-DIMENSIONAL GROUNDWATER MODELLING

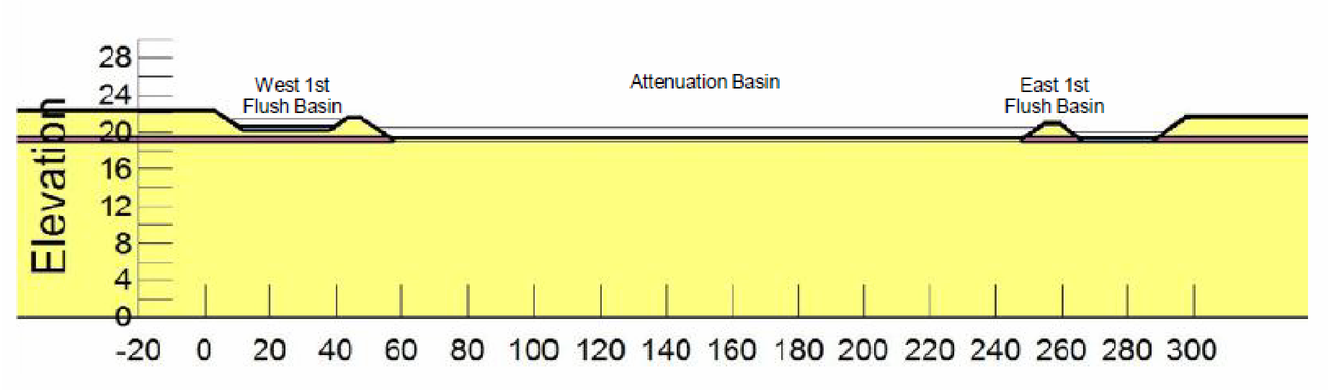
Two-dimensional groundwater modelling was carried out for each basin, for the critical duration storm, for each of the three cases outlined above. This modelling was carried out using GEO-STUDIO SEEP/W.

Each basin was modelled as a two-dimensional cross-section, typically 1000m long, with constant head boundary conditions applied at the far ends of the section to achieve the assumed underlying groundwater level. The inflows into the model (i.e. runoff into the basins) were calculated in a separate spreadsheet analysis using a Rational Method approach, with a peak inflow of twice the average inflow occurring at time 0.7D. This is consistent with the method to calculate runoff described in CCC (2003). These inflow hydrographs were applied to the model as a time variable unit flux over the basin areas ($m^3/d/m^2$).

The surface profile of the basin was included in the model, so that where the applied runoff cannot infiltrate (due to the soil being fully saturated) the volume would pond in the basins, with subsequent infiltration as a function of head.

An example model set-up cross-section for the Mushroom Basin is shown in Figure 6. The yellow colour represents Springston Formation gravel, while the thin pink layer (just visible in the figure) represents Springston Formation sand/silt.

Figure 6: Mushroom Basin model set-up cross-section



The hydraulic conductivities used in the groundwater modelling were much lower than the soakage rates used in the initial simplified spreadsheet design approach, which had been assumed to apply to vertical permeability above the water table, as discussed in section 4.2. The groundwater modelling takes account of the fact that when the groundwater level is high relative to the level of the soakage field, most of the soakage occurs horizontally into saturated soils, at a much lower hydraulic gradient.

The measured soakage rates were reduced by a factor of 15 to allow for reduction in permeability over time (a factor of 15 had been back-calculated from testing of similar soils in north Canterbury), and then in some cases a further factor of 10 to allow for vertical permeability relative to horizontal (reflecting the interbedded fine soils observed in the test pits, i.e. the anisotropy of the soils). This may be a conservative approach. A high permeability sensitivity case was also modelled with the 2% AEP and typical groundwater level to understand the effect of the permeability factors on the basin performance.

Examples of the groundwater modelling results are shown in Figures 7, 8 and 9.

Figure 7: Mushroom Basin modelled groundwater levels at end of 3 year ARI (33% AEP) storm event with 2% AEP groundwater level (high groundwater sensitivity case)

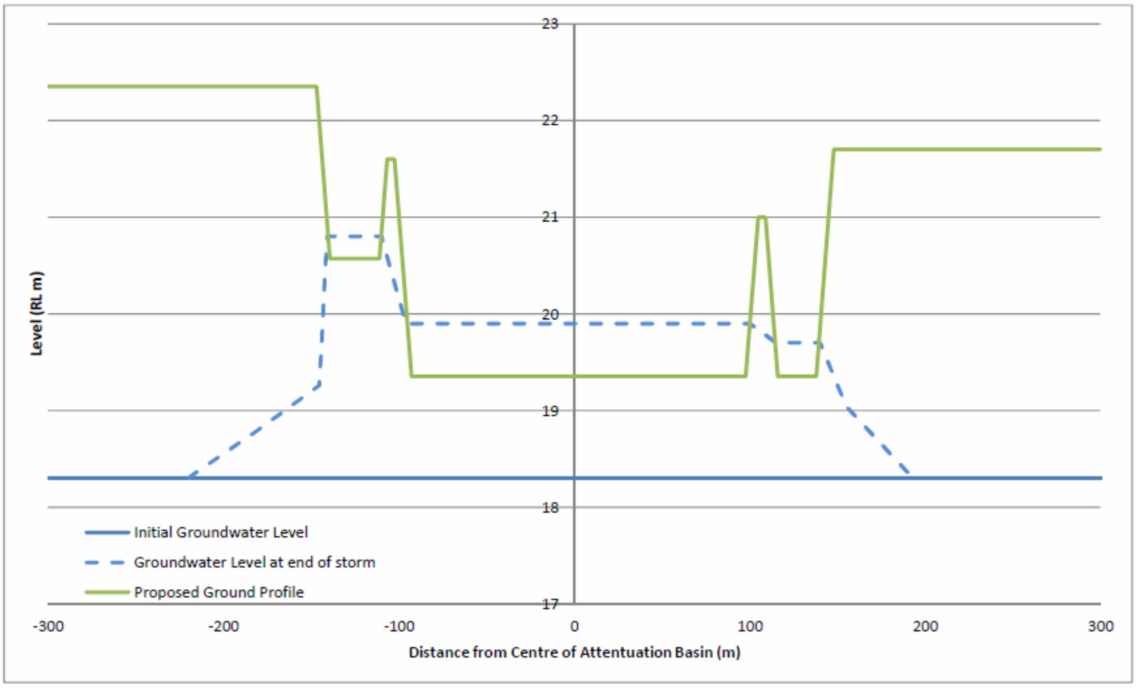
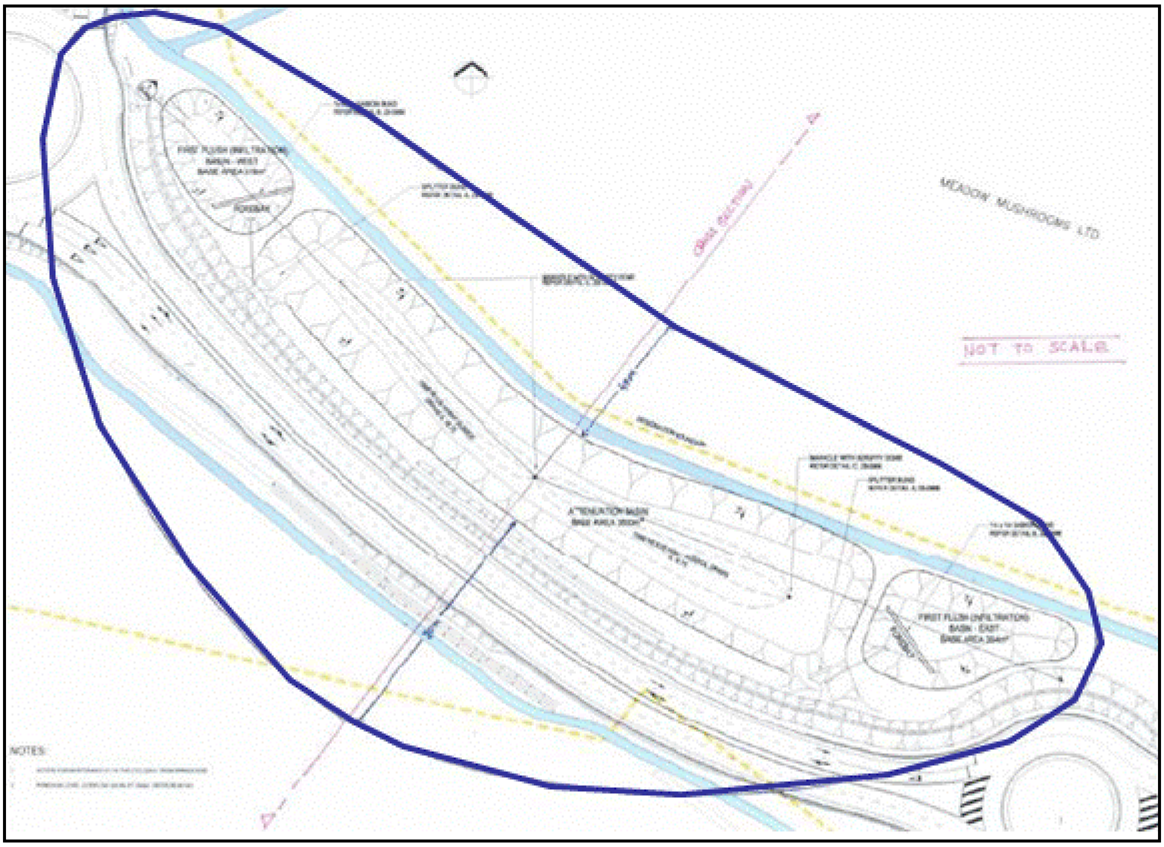


Figure 8: Mushroom Basin modelled groundwater mounding at end of 3 year ARI (33% AEP) storm event with 2% AEP groundwater level (high groundwater sensitivity case)



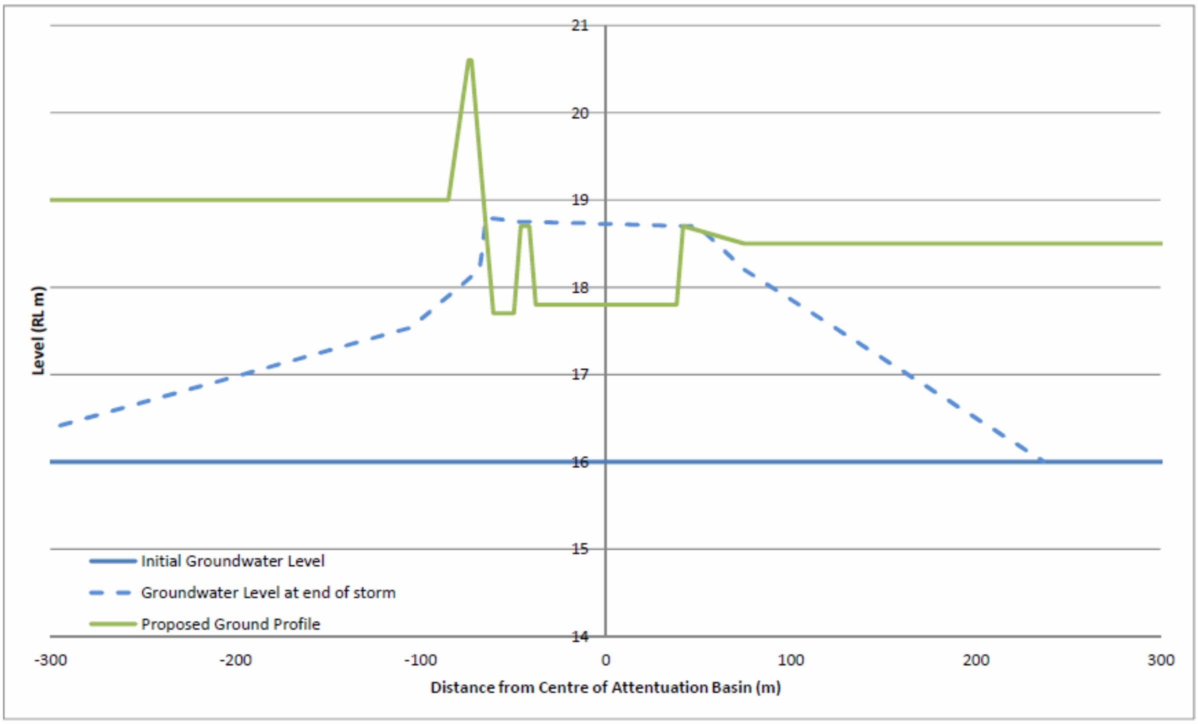
The groundwater modelling showed that the principal issue was not containment, but rather the time for basins to drain down:

- The 2% AEP events modelled could be contained within the basins (with minor modifications at Carrs Basin). The modelled groundwater levels for the Carrs Basin for the large storm base case are shown in Figure 9.

- The drain-down time following a storm event could be much longer than the 48 hours required by consents. Drain-down times for the cases modelled varied from less than one day, to up to two months. The shorter drain-down times were generally for the high groundwater base case and the high permeability sensitivity case. The longer drain-down times were generally for the large storm base case and the high groundwater sensitivity case.

As noted above, the parameters used in the model may be conservative. While there are a large number of infiltration and soakage basins used for stormwater management in the Canterbury area, few or any, are already constructed sufficiently close to the proposed CSM basins to confirm whether the model represents actual conditions or is conservative (i.e. results in higher mounding and longer drain down times). The adoption of a conservative approach provides for operation of the basins in all likely conditions. This provides NZTA with information that will not result in any surprises in the future operation and maintenance of the basins.

Figure 9: Carrs Basin modelled groundwater levels at end of 2% AEP storm event with typical groundwater level (large storm base case)



6.4 DESIGN MODIFICATIONS

As a result of the groundwater modelling the following modifications were made to the design:

- Basin bund levels were increased to provide containment of the design storms. The lowest top of bund level at Carrs Basin was increased by 200mm to contain the 2% AEP event. The 2% AEP event was already contained at the other basins, and therefore no bund modifications were required.

- A soakage field was moved towards a higher permeability subsurface stratum connecting to an adjacent waterway (at a lower level). The Musgroves Basin soakage field was moved towards Dry Stream to improve connectivity between the soakage field and Dry Stream. The location of high permeability material between the soakage field and the stream was confirmed by test pits on site. (If the existing material between the soakage field and stream was found not to have a high permeability it would have been excavated and backfilled with a high permeability material.) This response did not compromise the overall ECan objective of achieving groundwater recharge, as it was only the first flush basin that was disposed to ground, and it would continue to do so unless the groundwater was very high, at which time recharge would not be a requirement.

- Provision to pump out standing water was added. The Mushroom, Carrs and Musgoves Basins all have secondary flow paths to waterways. In the event of prolonged ponding becoming an issue, temporary surface pumps could be set up at the sites to pump water from the basins. The Lee Basin does not have an overland flow path. A rising main from the Lee Basin to the nearest waterway, the future Owaka waterway, was therefore added to the design. In the event of extended ponding in the Lee Basin a temporary surface pump could be set up to pump water from the basin, through the rising main, to the Owaka waterway.

It was noted that if prolonged inundation occurs the grass may die-off. If this became a recurring issue the grass could be replaced with gravel base, with a revised approach to vegetation maintenance.

6.5 COMPLIANCE

This information was then provided to Environment Canterbury and the consent was varied to remove the 48 hour drain-down requirement, recognising that there was already a condition requiring maintenance of a good grass sward in the basins.

7 CONCLUSIONS

The conventional soakage design approach involves adopting soakage rates based on field soakage test results, reduced by a factor of safety. This is based on the assumption that the groundwater table is sufficiently below the basin that soakage can occur near-vertically, into the unsaturated zone, i.e. with a hydraulic gradient of 1.

The CSM design experience has shown that sufficient investigation needs to be carried out early in the design process to confirm that this assumption is correct. Where groundwater levels may approach close to the basin or soakage field invert such that the hydraulic gradient is less than 1, then a different design approach is required, which can account for the horizontal soakage and reduced hydraulic gradient. This may require groundwater modelling.

In carrying out groundwater modelling a conservative approach should be taken in assessing the groundwater mounding that occurs. A conservative approach means that the long term owner of the basin has a system that should operate successfully in all likely scenarios.

In the event that such modelling shows increased containment is needed, or long drawdown times might eventuate on an infrequent basis as a result of extreme events (particularly unusually high groundwater levels), then it is appropriate to identify contingency plans to address these, and to ensure that these measures are noted in any operation and maintenance plans for the eventual owner of the facilities.

ACKNOWLEDGEMENTS

Geoff Griffiths, NZTA Project Manager

Justin McDowell and Peter Savage, Fulton Hogan

Andrew Brough, Mark Pennington and Hilary Lough, PDP

Sian France, Angela Pratt and Brent Hamilton, Beca

Paul Dickson, CCC

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