

POST-AUDIT OF A NUMERICAL GROUNDWATER FLOW MODEL DEVELOPED FOR BRITOMART TRANSPORT CENTRE, AUCKLAND, NEW ZEALAND

Parviz NAMJOU, Alan PATTLE
Pattle Delamore Partners, Auckland, New Zealand

ABSTRACT

Groundwater modelling is one of the main tools used for the prediction of future environmental effects of major underground excavations below groundwater levels. The large size of such structures and the construction method give rise to potential effects on groundwater pressures in adjacent areas. Although groundwater modelling is widely used by engineers in the design phase or for obtaining resource consents for such developments, the accuracy of model predictions is not commonly verified by field monitoring and post-audit programmes during and following constructions.

The Britomart Transport Centre was constructed near the waterfront area of the Auckland Central Business District (CBD). The watertight underground structure with its floor 9.5m below the water table covers an area of 13,000m² was excavated into variable geological formations. The lowering of the groundwater pressures during the excavation had potential to result in the consolidation of softer, near surface sediments within the upper Tauranga Group. Potential adverse effects included settlement of nearby buildings and services or groundwater flooding of building basements.

A three-dimensional numerical groundwater flow model with 27 layers was developed for the excavation. The model was used to simulate the magnitude and areal extent of the groundwater level changes, inflow during the 6 construction phases and after the construction as well as the quantity of saline water drawn into the excavation.

A post-audit programme was carried out to assess the performance of the model for the period subsequent to the calibration period. A comprehensive groundwater level monitoring programme in thirteen boreholes (39 multi-level piezometers) was undertaken to assist with the post-audit programme. The monitoring results showed that during construction of the excavation piles, the drawdown limits set for the Waitemata Group were exceeded by more than 3m. However, the post-audit simulation results showed that by incorporating the effects of pile dewatering into the model there is a satisfactory match between the predicted and measured groundwater levels during construction. As predicted by the model, following completion of the watertight structure, groundwater control and stormwater trenches prevented the damming effect or flooding.

KEYWORDS

Groundwater flow model, Underground excavation, Dewatering, Drawdown effects, Post-audit programme, Britomart Transport Centre

1 INTRODUCTION

The Britomart Transport Centre, one of the largest infrastructure project ever undertaken by a local authority in New Zealand, was constructed near the waterfront area of the Auckland CBD (Figure 1). The watertight underground structure with its floor 9.5m below the water table covering an area of 13,000m² within variable geological formations consisting of reclamation fill, silt and sand layers (lower and upper Tauranga Group) and fractured sandstones and mudstones (Waitemata Group).

The lowering of the groundwater pressures during the excavation had potential to result in the consolidation of softer, near surface sediments within the upper Tauranga Group. Potential adverse effects included settlement to nearby buildings and services or groundwater flooding of building basements.

This paper describes a three-dimensional numerical groundwater flow model which was developed for the excavation and its post audit evaluation. The model was used to simulate the magnitude and areal extent of the groundwater level changes, inflow during the 6 construction phases and after the construction as well as the quantity of saline water drawn into the excavation. A post-audit programme was carried out to assess the performance of the model for the period subsequent to the calibration period.

The methodology used in this study to achieve the above objective includes:

- Develop a hydrogeological conceptual model,
- Construct a 3D numerical model,
- Assess model predictions using a post-audit programme.

2 CONCEPTUAL HYDROGEOLOGICAL MODEL

The groundwater in the area of the Britomart Transport Centre lies within the CBD catchment, which is regarded as coincident with the surface water catchment (Figure 1). Within the catchment the groundwater system is complex due to the highly stratified aquifer sequence and the range of geological units present. Data from about 400 boreholes throughout the CBD catchment was used to develop the conceptual model for the groundwater system. A summary of drilling and site investigation programme is given by Pattle Delamore Partners (2000) and Tonkin & Taylor (1997). The geological units within the catchment include the Waitemata Group (both weathered and unweathered), Tauranga Group (lower and upper), Volcanic Rocks (associated with the Albert Park volcano) and Fill.

Figure 2 shows a generalized conceptual model in a plan view and along a typical section perpendicular to Queen Street. The Waitemata Group underlies the entire CBD catchment and drainage patterns from the Pleistocene period are evident on its surface. The CBD catchment represents a steep sided gully, occupied by the northerly flowing Ligar Stream, which follows the Queen Street gully (though it is now routed through the stormwater drainage system).

The hydrology of the catchment has been significantly modified during the development of the city and the increased building and paved coverage (impermeable surfaces). The natural surface water drainage pattern and recharge/discharge mechanisms to the groundwater system have been significantly disrupted. In this regard, there is no longer much significant natural recharge from rainfall infiltration directly into the ground. Most recharge now derives from leakage of the underground services into the adjacent ground.

Separate shallow and deep groundwater flow systems are indicated by the borehole data and by the general attributes of the hydrogeological units (PDP 2000). Recharge sourced in the upper catchment areas flows at depth within the Waitemata Group. This deeper system discharges to the Waitemata Harbour or up through the surficial sediments near the harbour edge.

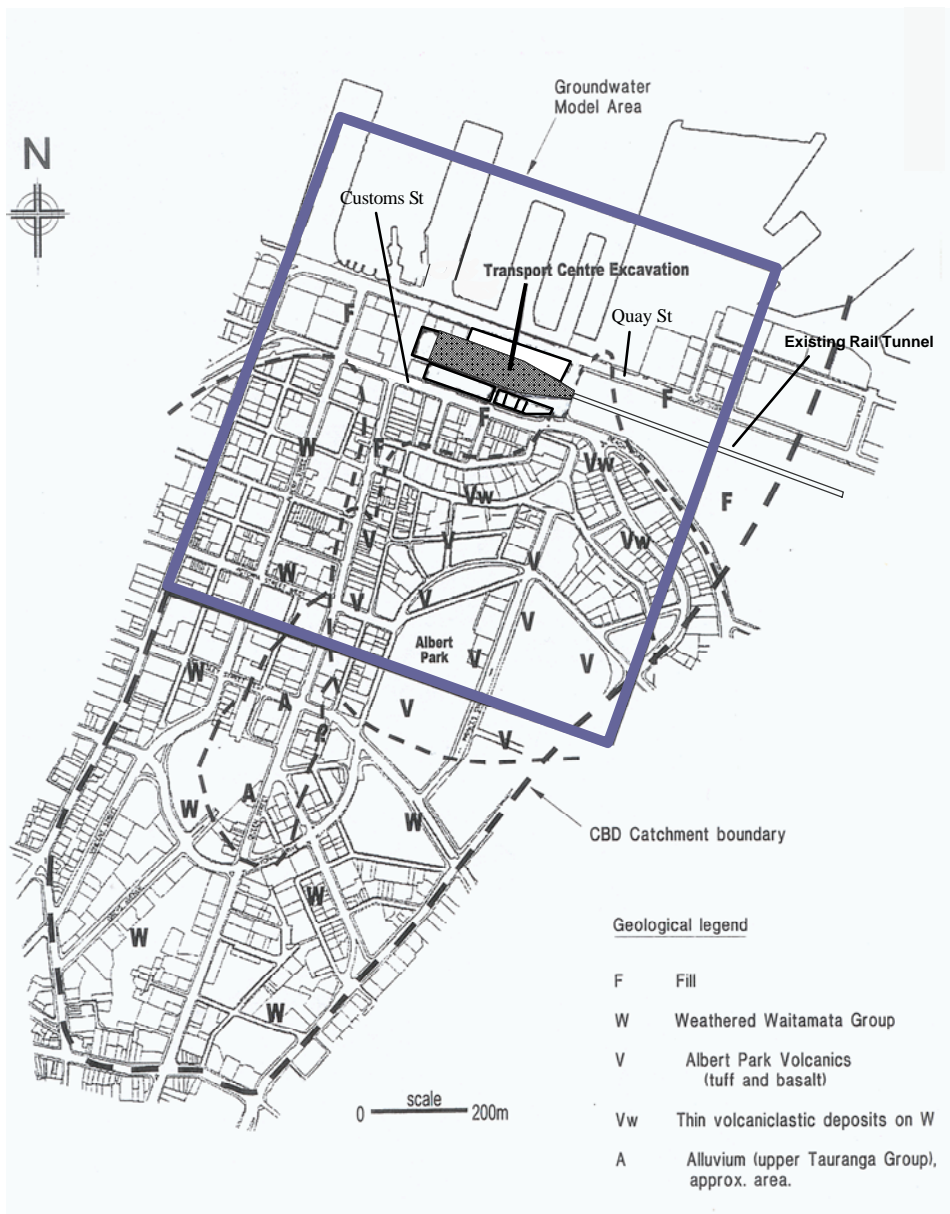


Figure 1: Central Business District Catchment and General Geology

Shallow groundwater occurs in the Fill and upper Tauranga Group. This groundwater is mainly sourced from drainage through the Albert Park volcanics and some upwards leakage from the Waitemata Group aquifer. The groundwater level in the volcanics is similar to the level in the Waitemata Group aquifer to the north (PDP 2000). A major pathway for groundwater flow occurs through a tongue of the lava that extends down Queen Street (Figure 2).

The groundwater system in the Fill materials is complex due to the mixture of materials present. The system is a highly dynamic, controlled principally by tidal effects and the drainage-recharge influences of the stormwater drainage system. Together with other service lines (many over a century old and potentially 'leaky'), the backfill in these service trenches is considered to form rapid flow conduits for groundwater movement. Therefore, water levels within the Fill are highly influenced by tidal effects which are more pronounced immediately adjacent to the coast and around major service conduits up to 200m inland. Some tidal recharge along service conduits is evident with partly saline water detected in some boreholes near the coast (PDP 2000).

Groundwater levels within the shallow and deep systems near the excavation are close to sea level. On average groundwater levels rise gently from the mean sea level, north of Quay Street to about RL 1.0m (RL refers to

elevation above mean sea level) at Customs Street. Many boreholes in this area exhibit strong water level changes due to tidal fluctuations, some in excess of 1.0m (PDP 2000).

The shallow and deep groundwater systems are separated by a low hydraulic conductivity aquitard comprised of the lower Tauranga Group and/or the weathered Waitemata Group. The aquitard varies in thickness from less than 1m to greater than 5m.

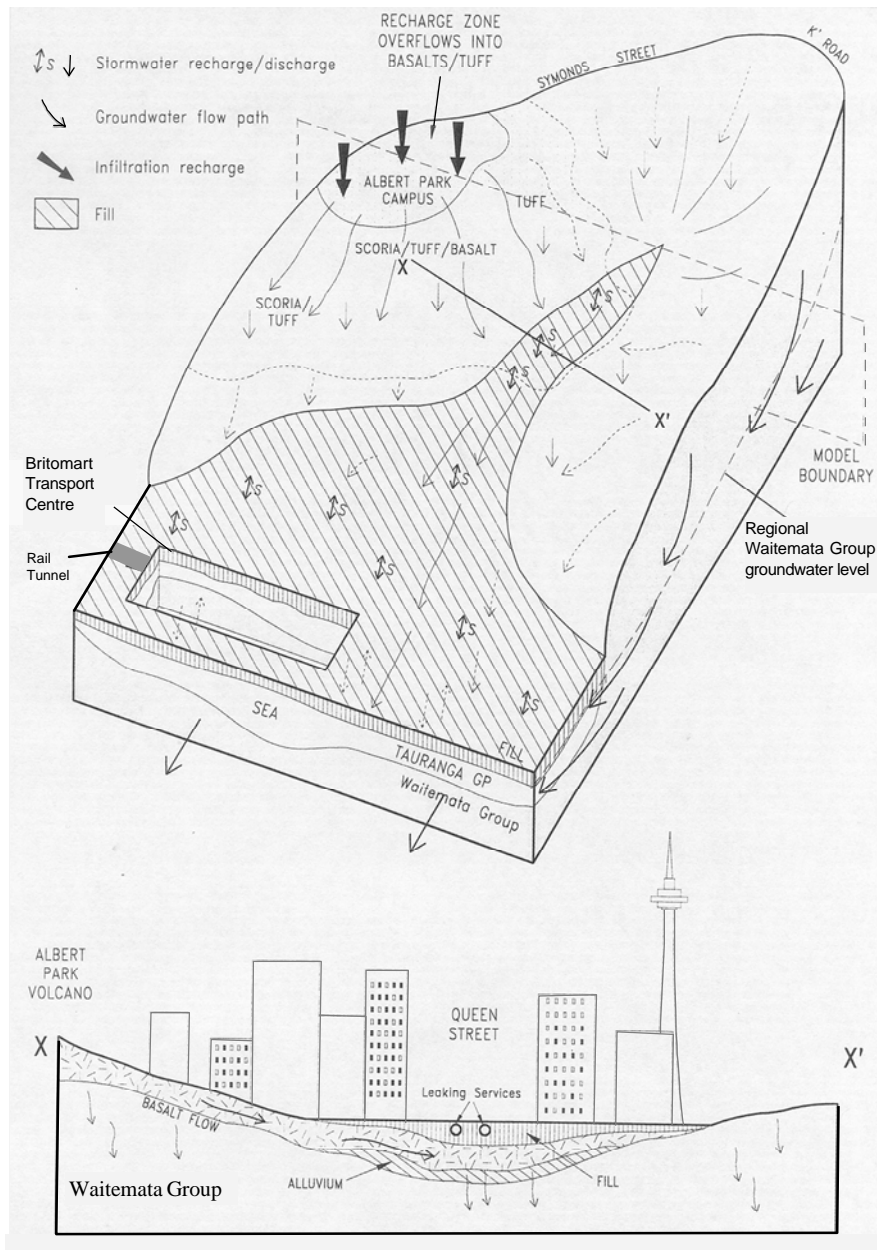


Figure 2: CBD Groundwater Catchment

2.1 HYDROGEOLOGICAL UNITS

The above geological units are classified into four hydrogeological units according to their hydraulic properties:

- Unit 1 incorporates both the Fill and upper Tauranga Group. Both comprise unconsolidated sediments with high silty sand content and share similar hydraulic conductivity properties. The cone penetrometer and pore pressure profiles (Tonkin & Taylor 1997) for these units show that they share similar geotechnical properties.

- Unit 2 combines the lower Tauranga Group with the weathered Waitemata Group. The justification for this is provided by the predominantly clay and stiff silt composition of both these units, whereas the upper Tauranga Group contains a higher sand fraction. The pore pressure profiles (Tonkin & Taylor 1997) in particular show a distinct change near to the boundary between the upper and lower Tauranga Groups indicating that the latter possesses a lower permeability and is more consolidated.
- Unit 3 represents the unweathered Waitemata Group with its distinctive and well-characterised lithological and hydraulic conductivity properties.
- Unit 4 represents Albert Park volcanic rocks (basalt and tuff).

2.2 HYDRAULIC PROPERTIES

An accurate estimation of the permeability of the highly compressible foundation soils and the sandstones/siltstones bedrock (Waitemata Group) was critical. In addition to 40 field permeability tests, three large-scale pump tests with the pumping rates between 15% to 50% of the maximum expected inflow to the excavation site, were carried out in the Waitemata Group for 3 months prior to the excavation (Tonkin & Taylor 1997) and the results were simulated using a numerical method (PDP 1997). The long term pump tests allowed the assessment of the effects of anisotropy on permeability within the Waitemata Group and hydraulic connectivity between the bedrock and the overlying unconsolidated materials.

A study of distribution of the hydraulic conductivity data reveals that there is no clearly defined areal variation in hydraulic conductivity in any of the units. Localised anomalies are evident and are characteristic of the Waitemata Group in particular, but distinct hydraulic conductivity domains could not be justified on the basis of the available data. The hydraulic conductivity results are summarised in Table 1.

Hydrogeological unit	Horizontal K (K_h) m/sec	Vertical K (K_v) m/sec
Unit 1 Fill/upper Tauranga Group	range: $> 1.0 \times 10^{-1}$ to 1.7×10^{-7}	no data
Unit 2 Lower Tauranga Group Weathered Waitemata Group	range: 7.2×10^{-6} to 1.3×10^{-8} quartile range 1.1×10^{-7} to 4.6×10^{-7} mean: 2.6×10^{-7} median: 2.5×10^{-7}	Laboratory tests: 5.5×10^{-9} to 2.5×10^{-10} pump test - numerical analysis: 4.0×10^{-9}
Unit 3 Waitemata Group	all data (piezometer test + pump test (no. analyses: 35): range: 1.0×10^{-7} to 8.3×10^{-7} quartile range: 2.3×10^{-7} to 6.1×10^{-7} mean: 3.8×10^{-7} median: 4.7×10^{-7} pump test data (analytical interpretation (no. analyses: 10): range: 1.4×10^{-7} to 8.0×10^{-7} quartile range: 1.8×10^{-7} to 6.6×10^{-7} mean: 3.8×10^{-7} median: 4.1×10^{-7}	Laboratory tests: 2.6×10^{-11} to 4.0×10^{-11} mudstone 2.8×10^{-6} to 2.0×10^{-7} sstn/siltst pump test - numerical analyses: 5.0×10^{-8}

Unit 4 Albert Park volcanics	pump test data (analytical interpretation): 6.0×10^{-5}	
---------------------------------	---	--

Of the above geological units, the Fill and upper Tauranga Group exhibit the highest average hydraulic conductivity, some two orders of magnitude above the two underlying units. However, due to the different sources of the original Fill materials, there is a high degree of variability in the hydraulic properties of the unit, which leads to irregular groundwater flow paths. Specific storage value of 1×10^{-5} was adopted for the study which was derived from pump test analyses (PDP 1997). Specific yield was estimated to range from 0.1 (Units 2 and 3) to 0.2 (Unit 1).

2.3 RECHARGE

Urbanisation within the CBD catchment has caused marked changes to the natural groundwater and surface water hydrological system. These changes are the result of the introduction of a dominantly impermeable land area which has caused a reduction in direct infiltration from rainfall, and increased surface runoff via a stormwater drainage system. In addition, large quantities of water are imported into the catchment, disposed of into a sewer system and then largely exported from the catchment. Thus, there no longer exists a natural surface water runoff regime and groundwater recharge processes are highly modified. Sources of recharge to both the shallow and regional groundwater systems are as follows:

- Direct infiltration to the shallow groundwater system in parkland and other open areas
- Leakage from stormwater drains
- Leakage from foul water sewers
- Tidal recharge along service conduits
- Leakage from water mains
- Deep infiltration to the regional Waitemata Group aquifer via leakage from overlying layers.

A distinction is made between recharge to the perched Waitemata Group aquifers and the deep regional Waitemata Group aquifer. Recharge to the former largely remains in the shallow perched system, and flows laterally and ultimately enters the municipal drainage system. Some of this water, however, leaks downwards and recharges the regional Waitemata Group aquifer. Recharge to perched Waitemata Group aquifers largely remains in the shallow perched system. Groundwater within these perched layers flow laterally and ultimately enter the municipal drainage system. Some of this water, however, leaks downwards and recharges the regional Waitemata Group aquifer.

Groundwater throughflow down the Queen Street gully enters the Fill, mainly sourced through leakage from the Albert Park volcanics. A short lava flow from the Albert Park centre flowed down the line of Queen Street. Recharge from the Albert Park area into the scoria cone complex is considered to overspill into this basalt and provide recharge into the Fill. Much of this inflow will probably be drained into the stormwater conduits during periods of low flow. Lesser recharge via the associated scoria deposits may occur directly into the Tauranga Group along Fort Street.

The recharge quantification assessment for the CBD catchment is discussed in detailed by Pattle Delamore Partners (2000) and is summarised in Table 2:

Recharge through Albert Park	12,000 m ³ /year or 0.08m/year (7% rainfall)
Infiltration to perched Waitemata Group aquifers	40,000 m ³ /year
Infiltration to regional Waitemata Group aquifer	15,400 m ³ /year or 0.014m/year
Leakage to Fill from services	in equilibrium with drainage to services

In the reclaimed areas, tidal fluctuations will cause seawater to backflow into the stormwater system and locally recharge the Fill during high tides; reversed flow and drainage from the Fill back into the conduits will occur during low tide. The complex local flow dynamics of the Fill is dominated by the service reticulation and in an annual basis is believed to be in equilibrium.

3 NUMERICAL MODEL

3.1 MODEL DESCRIPTION

The modular three-dimensional finite-difference groundwater flow code MODFLOW (McDonald and Harbaugh 1988) was used to simulate the groundwater conditions around the transport centre. The presence of a complex groundwater system and the scale and geometry of the excavations provide justification for the generation of a three-dimensional groundwater flow model.

The model was used to predict the groundwater level behaviour both before and after construction of the transport centre. This was carried out by introducing the proposed excavation and assessing the impact of various excavation phases on the groundwater levels.

3.2 MODEL LAYERS AND GRID SPACING

The whole model domain is approximately 1km², and the grid is aligned NNE to coincide with the principal flow vector within the Waitemata Group aquifer. The model grid and unit boundaries are shown in Figure 3. The grid resolution ranges between a cell size of 8m to a maximum cell size of 25m. Vertically the groundwater model is divided into 27 layers in order to simulate different hydrogeological units and various excavation levels. Some layers are added within the hydrogeological units to accommodate vertical head gradients and therefore to increase the accuracy of the model outputs.

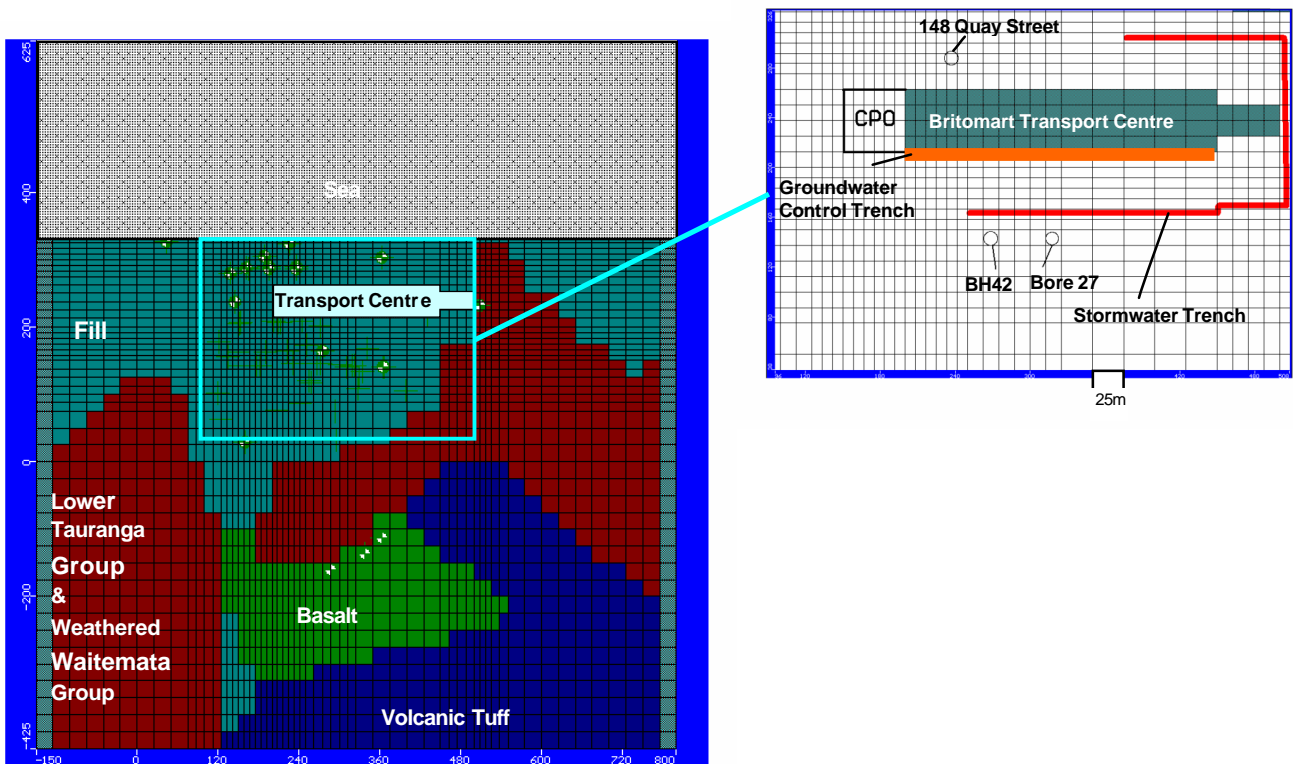


Figure 3: Model Grid and Unit Boundaries for model Layer 4 (left), Grid Structure close to Britomart Transport Centre

3.3 BRITOMART TRANSPORT CENTRE BUILDING DESIGN

The following construction assumptions for the Britomart Transport Centre have been incorporated into the groundwater construction model:

- The secant piles extended about 20m below the ground level and about 4m into the Waitemata Group in the western half of the centre and on each side of the structure. In total, about 400 secant piles were used in order to seal the structure below the water table.
- Sheet piles to be driven 1m into the Waitemata Group. Below that the excavated rock face will be free to drain into the excavation in the eastern half of the centre. The sheet piles will be removed after completion of the construction.
- The completed Britomart structure is sealed from water inflow.
- During construction, the excavation is assumed to be open to seepage inflow in sections. The construction phases and dewatering sections during each phase is given in Table 3.
- A new CBD stormwater trench in Downtown Auckland is incorporated into the model using drain cells (Figure 3). A groundwater control trench has also been incorporated into the model to control the water level rise south of the centre after construction (Figure 3). The proposed trench will be gravel filled to a depth of 2m below the current watertable. The outflow from the trench to the existing manholes occurs at approximately RL 1m, consistent with existing groundwater levels.

Modelled Construction Phase	Excavation Progress	Period (days)	Cumulative Time (days)
1	Rail link tunnel (eastern end of the centre) is completed and sealed Western half of the sheet pile wall is completed	60	60
2	Sheet piles and about 90% of secant walls are completed Eastern section of the excavation (with sheet piles) is completed (about 7,500m ²)	60	120
3	Sheet piles and secant walls are completed The excavation is completed 1,250m ² of the excavation is sealed	60	180
4	6,250m ² of the excavation is sealed	60	240
5	10,600m ² of the excavation is sealed	60	300
6	Excavation is sealed	60	360

3.3.1 MODEL BOUNDARY CONDITIONS

Model boundaries were refined successively on the basis of boundary sensitivity analysis studies and calculations relating to the area of influence of the excavation within the Waitemata Group. Analysis showed that the approximate area of influence of the excavation is within 500m south of the structure in the Waitemata Group. The condition assigned to it was a fixed head of RL 6m (above sea level) which was derived from the extrapolation of the observed Waitemata Group regional groundwater gradient further to the north of 0.007.

Borehole data derived from the Sky Tower geotechnical investigation shows that the deepest (presumed regional) water level lies at RL 6 - 7m (PDP 2000).

Parallel east and west no-flow model boundaries represent groundwater flow lines along the margins of the conceptual CBD groundwater catchment.

The northern boundary for deeper layers is set at a sufficient distance off-shore so that its position and the conditions assigned to it do not affect modelled predictions. Fixed heads were placed along the seabed of RL 0m from the shoreline to the northern boundary.

The base of the model was set as a no-flow boundary at a depth of 100m below the ground surface. Sensitivity analyses showed that the depth of this boundary is safely outside the steady state drawdown effects of the excavation.

3.3.2 MODEL CALIBRATION

The groundwater flow model for the transport centre has been verified initially by comparing the numerical hydraulic head predictions in a steady state condition (i.e. in the absence of the excavations) with the measured water level data (PDP 2000). Pump test drawdown data have provided additional information against which the model was calibrated under transient conditions.

Calibration efforts have only attempted to reproduce observed water levels to less than ± 50 cm (The root mean squared error of the calibrated model is about 30cm). The calibration hydraulic parameters used for the model provide the best calibration results. The Hydraulic conductivity values used in the calibrated model are shown in Table 4. The calibrated model water balance result is given in Table 5.

Table 4: Summary Calibrated Hydraulic Conductivity Values

Unit	K_h (m/s)	K_v (m/s)
1. Fill/Upper Tauranga Group	3×10^{-5}	1×10^{-6}
2. Lower Tauranga Group/ Weathered Waitemata Group	2×10^{-7}	4×10^{-9}
3. Waitemata Group	5×10^{-7}	5×10^{-8}
4. Albert Park volcanics:		
Tuff	8×10^{-7}	4×10^{-8}
Basalt	7×10^{-5}	1×10^{-5}

Table 5: Water Balance for Steady State Model

	Model input (m ³ /d)	Model output (m ³ /d)
Southern boundary	18.8	6.2
Coastal boundary	0	57.9
Surface recharge	45.3	0
Total	64.1	64.1

As part of the calibration process, sensitivity analyses were undertaken to determine the influence of hydraulic parameters and boundary conditions on the calibrated model. The most sensitive model parameter is the horizontal hydraulic conductivity of Unit 3. Reducing the horizontal hydraulic conductivity of the model by one order of magnitude causes an increase of over 1.5m in hydraulic heads in the unweathered Waitemata Group (Unit 3). By increasing the K_h of Unit 3, there is a much smaller change in groundwater heads; they increase by up to 0.5m in the excavation area. Horizontal hydraulic conductivity for the unweathered

Waitemata Group is the most thoroughly tested parameter through extensive site testing. Overall, the model proves relatively insensitive to plausible recharge variations. In addition, fixed heads on the southern boundary of the model have little impact on the unweathered Waitemata Group aquifer heads around the excavation site.

3.4 CONSTRUCTION PHASE MODELS

The construction phase models were set up to identify the effect of the excavation on groundwater in both construction and post construction scenarios. The construction sequence was split into 6 phases (Table 3). The impermeable retaining walls and the stormwater drainage system are incorporated in the models before the simulations. In each phase a different section of the excavation were assumed to be open to seepage inflow. The inflow into the excavation occurs for a total period of 10 months. The stormwater trench (simulated with drain cells) was in place before the simulations.

3.4.1 DRAWDOWN EFFECTS

The simulation results indicated that temporary lowering of the groundwater pressures (drawdowns) in the locality of the site will occur in the Waitemata Group aquifer while the excavation is progressing. These drawdowns, due to groundwater inflows through the base of the excavation, will be present for the period the excavation base remains open. However, in the overlying units, which have a higher potential for ground settlement, there will be less drawdown effects. These units are sufficiently separated from the inflow zone so that drawdowns are attenuated before reaching them.

The drawdown contours around the transport centre after 180 days, i.e., construction Phase 3 (for the top of units) are shown in Figure 4. In this phase, dewatering occurs over 90% of the final base excavation area and the construction programme reaches the point of maximum stress on the groundwater system. Closing of the base in later construction stages progressively reduces the dewatering stress from this point on. The maximum drawdown in the top of Unit 2 occurs to the north of the structure (east of Commerce Street) and is about 0.6m (Figure 4).

In the deep groundwater system, in the top of Unit 3 next to the south side of the structure, drawdown is about 4.5m. The zone of influence of dewatering in the Waitemata Group is about 24 ha in area, extending about 350m to the south of the transport centre. The model predicted that following sealing of the excavation base the groundwater levels will generally return to normal. The stormwater and groundwater control trenches prevent any flooding effects of the sealed structure.

As an example, the predicted hydrograph for BH42 showing the typical measurement of the groundwater pressures over time are shown in Figure 5. BH42 is about 60m south of the excavation and shows about 1.0m drawdown during construction. No significant drawdown is predicted within the overlying upper Tauranga Group and Fill (Unit 1) at this location.

The maximum groundwater inflow is predicted to be in the order of $60\text{m}^3/\text{s}$ after 180 days of construction (Phase 3) when the excavation is 90% complete.

3.4.2 ASSESSING POTENTIAL FOR SALINE WATER INTRUSION

The groundwater quality data from the bores in the vicinity of the site indicated that some saline water currently extends from the harbour into Unit 1, north of Quay Street. Potential existed for this water to be drawn toward the excavation during construction.

The groundwater model predicted that during construction, the natural flow directions north of the structure would be reversed allowing seawater to enter the excavation site. According to the model about 22% of the inflow to the base of the excavation would be sourced from seawater.

After construction the normal groundwater flow direction is restored. However, due to the damming effect of the structure the throughflow toward the sea immediately to the north of the structure is weakened, causing the saline interface to curve toward the structure. However as saline water is already exists north of the structure the movement of the saline water toward the structure during and after construction should have little net effect on the quality of the groundwater close to the site as existing building foundations close to site are already subject to saline water effects.

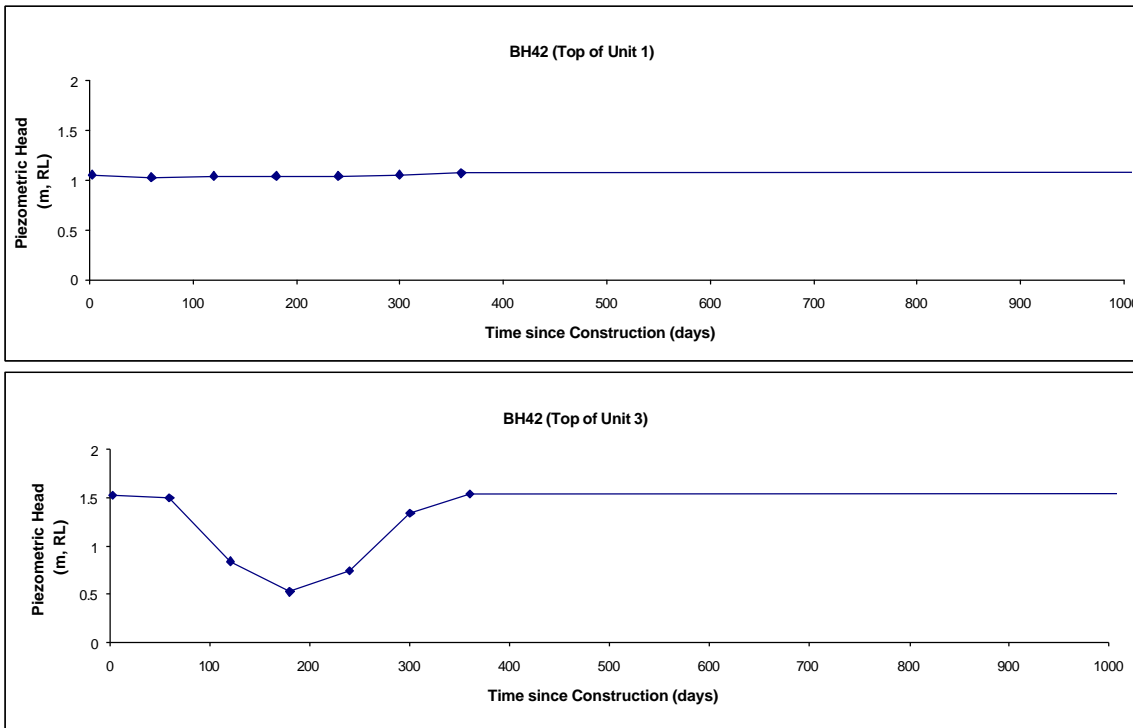


Figure 5: Modelled Piezometric Head versus Time for BH42

3.4.3 CONSTRUCTION MODEL SENSITIVITY ANALYSIS

The Britomart construction model was tested for its sensitivity to changes in the key parameters, i.e., vertical and horizontal hydraulic conductivity values for the respective geological units. The values were each changed in turn by a factor of 10 in order to expose any model sensitivity. It is very unlikely that any model parameters would be out by this order of magnitude and therefore the procedure provided a good test of the model behaviour. The effect of above variations in hydraulic conductivity on the predicted drawdown is less than $\pm 0.20\text{m}$. The Unit 1 horizontal hydraulic conductivity, if reduced by a factor of 10 without alteration to any other parameters, produces apparent additional modelled drawdowns at the base of Unit 1 of about 0.15m. The model has a low sensitivity to higher permeabilities than the preferred model values and therefore the overall sensitivity of the model to the Unit 1 hydraulic conductivity is low.

The vertical hydraulic conductivity of the Waitemata Group influences drawdowns in the fill because groundwater flows must move vertically down through this unit to pass beneath the perimeter retaining wall. The model sensitivity analysis indicates that modelled drawdowns in Unit 1 could increase by about 0.20m in local areas if the Waitemata Group vertical hydraulic conductivity is increased by a factor of 10.

4 POST-AUDIT MONITORING PROGRAMME

A post-audit programme was carried out to assess the performance of the model for the period subsequent to the calibration period. The programme consisted of comprehensive groundwater level monitoring in thirteen boreholes in Units 1, 2 and 3 (total of 39 multilevel piezometers) prior and during the construction phases. Automatic water level recorders with 15min intervals installed in piezometers close to the excavation (QS3, QS2, BH43, BH42, BH35 and BH32) in Unit 3, the other piezometers were monitored daily. The locations of the monitoring piezometers are shown in Figure 6.

The recommended monitoring programme was incorporated into the Auckland Regional Council (ARC) consent conditions. The conditions stated that if unexpected variations in the predictions occurred then the model needed to be recalibrated, the groundwater effects should be reassessed and remedial works should be undertaken.

In addition to the groundwater level monitoring a network of ground and building movement monitoring points on the existing buildings next to the site was established by City Design Ltd to detect vertical and in some cases horizontal movements.

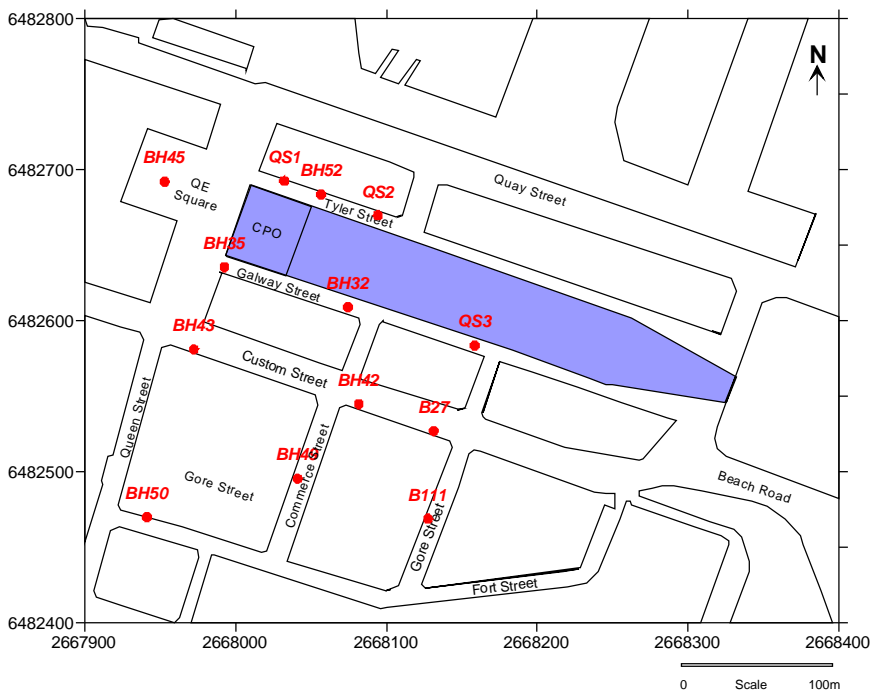


Figure 6: Post Audit Groundwater Level Monitoring Boreholes

4.1 ABSOLUTE MINIMUM GROUNDWATER LEVELS

The predicted maximum drawdown during the construction phases was used as the absolute minimum groundwater level which can be tolerated (the trigger levels set based on 75% of the maximum predicted drawdown levels). If groundwater level exceeds the absolute minimum groundwater level in Unit 3, the settlement risk for surrounding buildings had to be reassessed. The drawdown limits were set for Unit 3 as the base of excavation is within Unit 3 and any reduction in piezometric pressure in this unit controls the subsequent groundwater level drops in overlying units.

The base groundwater levels for the monitoring piezometers were identified following 6 months (April to October 2001) groundwater level monitoring programme before the start of construction. The base groundwater levels and absolute minimum groundwater levels (model predicted values) for 9 piezometers in Unit 3 (close to the excavation) are shown in Figure 7.

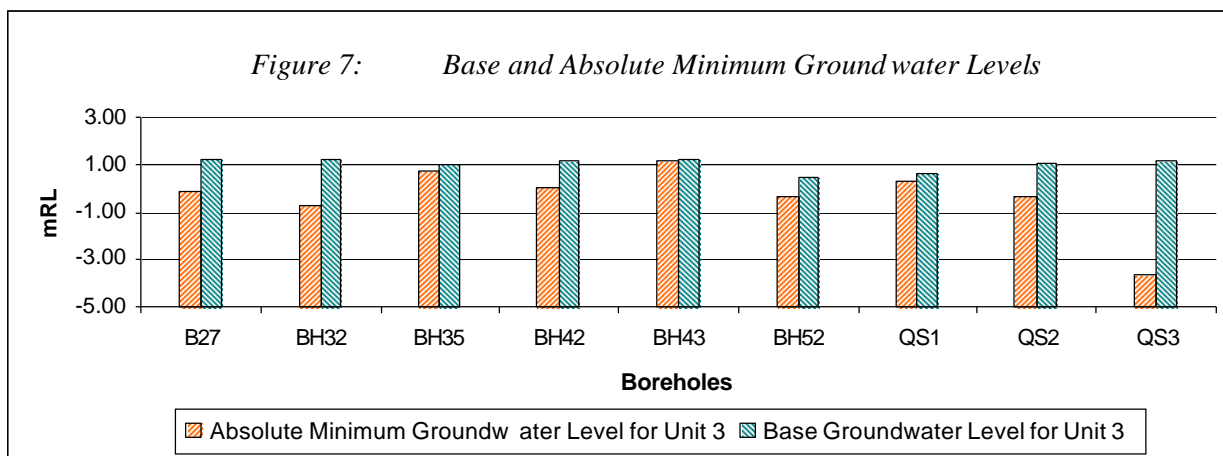


Figure 7: Base and Absolute Minimum Ground water Levels

4.2 RESULTS OF POST AUDIT PROGRAMME

The groundwater level monitoring results for the construction period (November 2001 to October 2002) showed that during the construction of structure piles, the absolute minimum groundwater levels for Unit 3 were exceeded by more than 3m (BH 32 and QS2). However, ground water levels in Unit 1 were less affected by the construction (due to low vertical permeability of Unit 2). Only in one piezometer in Unit 1 (QS1), a maximum drawdown of about 1m was detected during construction. Figure 8 shows the difference between the predicted and measured maximum drawdown contours for Unit 3 during the excavation. The predicted maximum drawdowns for 9 piezometers (close to the excavation) in Unit 3 are shown in Figure 9.

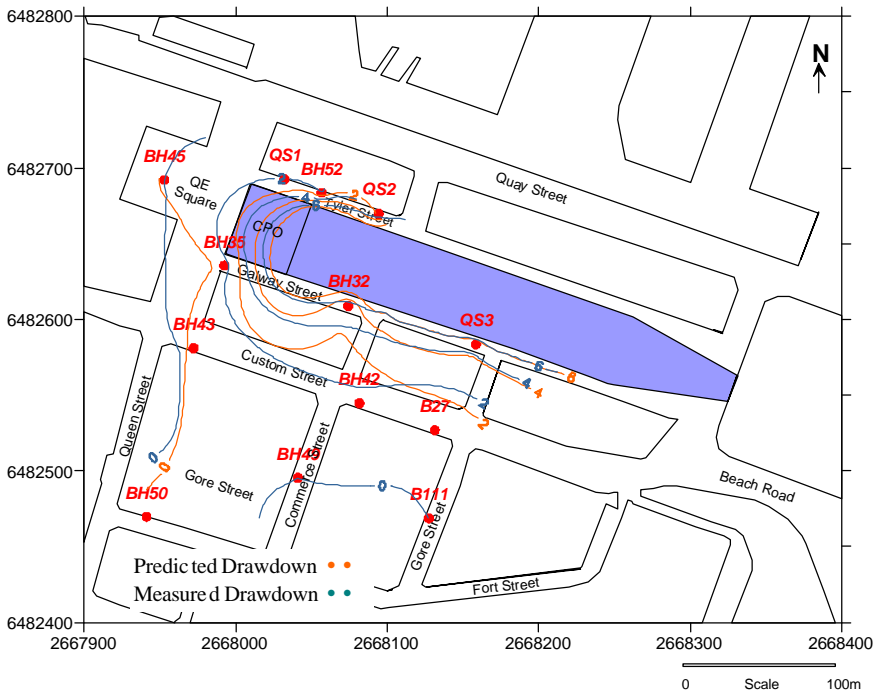


Figure 8: Predicted versus Measured Drawdowns in Unit 3 (Phase 3)

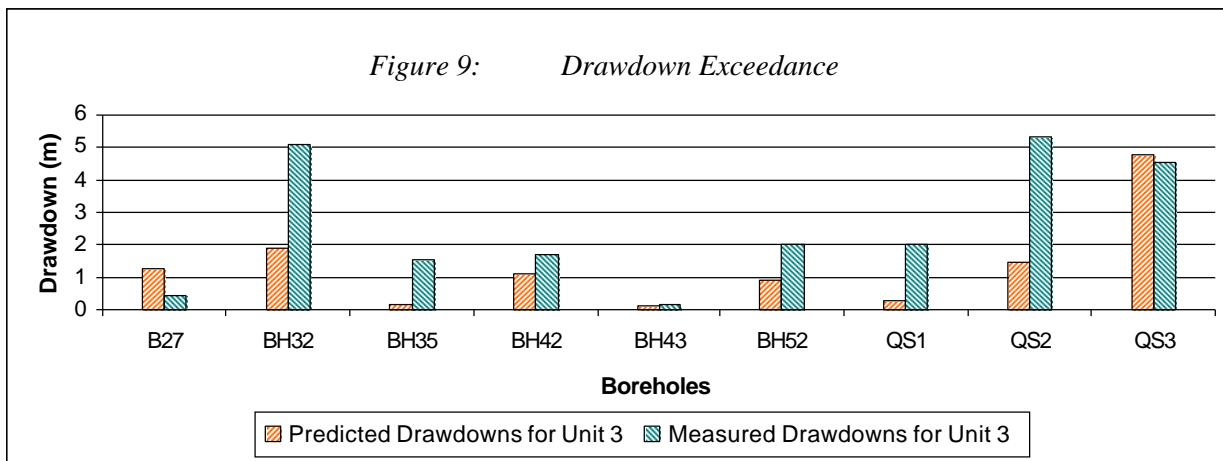


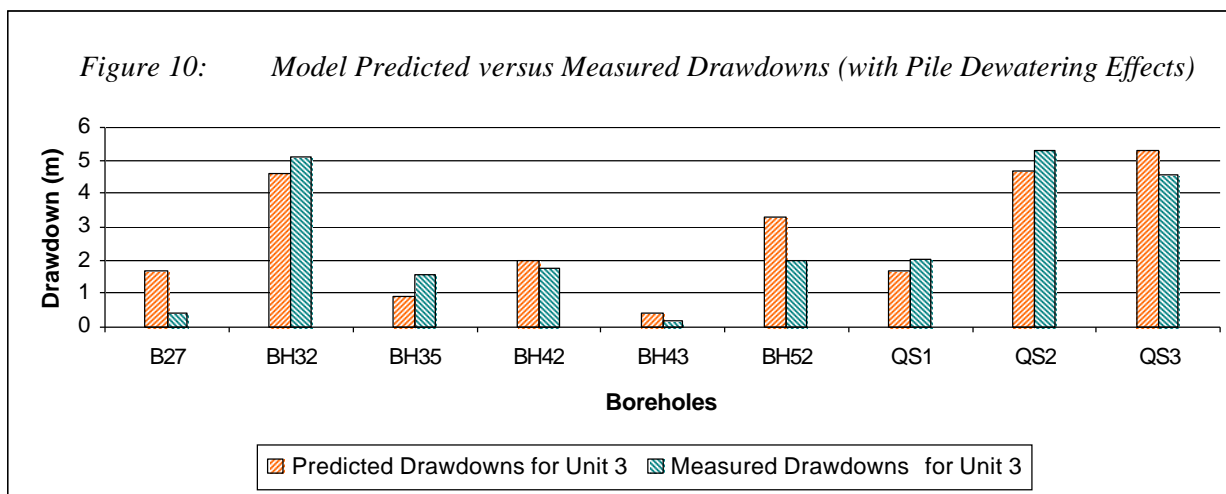
Figure 9: Drawdown Exceedance

By comparing the groundwater level responses to construction activities, the following causes were identified for the above drawdown exceedance:

- Dewatering during the construction of piles (e.g. 1.2 m diameter compression piles),
- Minor leakage from the secant wall,
- Minor leakage from the excavation floor,
- Construction of a trench excavated for a sewer pipe close to the northern boundary of the transport centre.

The most significant exceedance occurs during pumping from the construction piles drilled 4 to 16m into the Waitemata Group. During the design phase of the project the construction team envisaged that the dewatering effects due to the pile construction methodology would not be significant and therefore no pile dewatering effects were incorporated into the model. However considering the low storativity of the Waitemata Group even minor dewatering within Unit 3 or seepages from the secant wall can increase the predicted drawdown as became evident during the post-audit monitoring programme.

Dewatering for construction of each deep pile was taking place generally at RL-26 to RL-30m for up to 8 hours. There were large numbers of piles constructed progressively during 150 day period with various dewatering stages. Simulations were run using the model to identify the likely drawdown effects of such pile dewatering on the western section of the excavation. Pile dewatering was simulated for 5 days (time needed to reach more than 90% of drawdowns) to allow for accumulative drawdown effects of neighboring piles in the same section of the construction. The results show that the dewatering can increase the predicted drawdowns by up to 3m. Figure 10 shows the effects of dewatering on the model predictions. The results show that by incorporating the effects of the pile dewatering into the model, there is a satisfactory match between the predicted and measured groundwater levels during construction. Small variations between the measured and calculated heads (generally <0.5m) were expected considering minor local heterogeneity and variations in fracturing within the Waitemata Group. In addition field observation showed that some minor leakages were taking place through the secant wall which is believed to result in a slight increase in measured drawdowns.



Despite the drawdown exceedance (Figure 9), the settlement monitoring carried out by City Design Ltd in August 2002 showed that the settlement was generally less than the trigger level of 5mm set for buildings close to the excavation. Therefore no further modelling was considered to be necessary to assess the effects of leakage or local heterogeneity. However to reduce the drawdown effects, pumping from all construction piles was ceased so a mitigation measure could be implemented. The new method for piling was prepared by Downer Engineering Ltd and applied from February 2002. The mitigation measure was consisted of using an alternative method for the pile construction which required less groundwater pumping from Unit 3. In the original method, up to 8m of the Waitemata Group could be exposed between the casing shoe and the top of the concrete plug. Pumping from the exposed rock could result in excess drawdowns, however, with the alternative method, the required pumping was minimised by reducing the length of the open hole (some minor pumping still could occur within Unit 2). In addition local groundwater seepages between the secant piles and from the excavation floor were gradually sealed which resulted in further recovery of groundwater levels. By the end of monitoring programme in June 2004 complete recovery was achieved in the majority of piezometers. Lack of complete recovery in a few piezometers (such as QS2 and BH32) was considered to be due to some minor leaks from walls in local areas or due to construction of other drainage systems outside the centre such as a 3m deep trench which was excavated for a sewer pipe close to the northern boundary of the excavation (along Tyler Street).

Following the sealing of the transport centre, damming effects along the southern boundary of the centre (i.e. perpendicular to the groundwater flow in the CBD catchment) was not observed. As predicted by the model, groundwater control and stormwater trenches prevented the damming effect or flooding.

5 CONCLUSION

The groundwater model developed for predicting drawdown effects due to construction of the Britomart Transport Centre had been a critical element in its gaining resource consent. The predicted risks to surrounding buildings from groundwater drawdowns were considered to be high.

The groundwater monitoring results collected as part of the post-audit programme showed that during the construction of excavation piles, the drawdown limits for the Waitemata Group to the west of the excavation were exceeded by more than 3m. This was contributed to by the dewatering during the construction of piles and minor seepages from the secant piles and the excavation floor. However, the post-audit simulation results showed that by incorporating the effects of pile dewatering into the model there is a satisfactory match between the predicted and measured groundwater levels during construction. Small variations between the measured and calculated heads were expected considering minor seepages from the secant wall, local heterogeneity and variations in fracturing within the Waitemata Group. The results show that design tools such as groundwater models should continue to be utilized during construction to ensure successful management of environmental effects through the full project cycle.

ACKNOWLEDGEMENTS

This project was funded by Auckland City Council. We thank staff of and Auckland City Council and Beca for their support throughout the project. We are grateful to Prof. Mike O'Sullivan and Dr Naresh Singhal for their review of the draft paper.

REFERENCES

- McDonald, M. G. and Harbaugh, A. W. (1988) 'Techniques of Water Resources Investigations of the United States Geological Survey: A Modular Three-Dimensional Finite Difference Groundwater Flow Model' Chapter A1, United States Geological Survey Open File Report 83-875.
- Pattle Delamore Partners Ltd (2000) Groundwater Effects Assessments of Queen Street Station, Prepared for Auckland City Council, May 2000.
- Pattle Delamore Partners Ltd (1997) Numerical Groundwater Flow Modelling of the Britomart Transport Centre, Prepared for Auckland City Council, September 1997.
- Tonkin & Taylor Ltd (1997) Britomart Transport Centre, Groundwater Investigations Reports, Prepared for Auckland City Council, August 1997.