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A comparison of observed dewatering induced settlement in alluvial soils against industry standard models

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ABSTRACT

This paper compares the predicted and observed construction-induced settlement for a project in Hamilton, N.Z. The project involved the construction of four shafts and associated pipelines in high-permeability alluvial (Hinuera) soils in an area with a high water-table. The city is embarking on significant development and expansion with a number of similar projects in development. The predictive model used was an industry-standard technique widely accepted by local regulators and which allows for rapid assessment, this was adopted over more sophisticated methods in order to minimise consenting delays. The settlement effects from dewatering of four sheet-piled shafts over a 6-14 week period were assessed against those predicted. Surface monitoring was undertaken by surveying points located in a number of surface types (grass, pavement and structures) and included deep points unaffected by seasonal variation. Surveys identified no discernible settlement in the zone of measurement between 9m and 68m from the shaft edges, groundwater monitoring showed a similar subdued response. The results are presented in a distance versus settlement chart and a discussion for the refinement of future projects is provided. Given the widespread occurrence of Hinuera soils over the Hamilton basin, this study may be applicable when consenting future developments in the region.

1 INTRODUCTION

Pipelines installed by open trench methods often require dewatering of the trenches and shafts to permit safe installation and provide a dry base to the excavation. This process is known to cause two principal settlement effects on the surroundings, the first is settlement related the change in effective stress as a result of the drawdown, the extent and amount of settlement being a function of the soil permeability, groundwater level and trench depth. The second is settlement related to the lateral movement of the trench sides upon excavation due to changes in stress field and the resultant vertical movement that occurs. The former effect is more dominant in coarse grained soils, whilst the latter tends to be the more dominant effect in fine grained soils.

As part of the consenting process these effects are routinely modelled so that sensitive receptors can be identified and effects mitigated and/or monitored. A long-standing method for modelling the dewatering-related effects is that of Somerville (2005) in CIRIA Report R113 whilst methods for modelling the lateral wall movement component are described in CIRIA Report C760.

These modelling methods were adopted for the assessment of effects of the Western Wastewater Interceptor Duplication project in Hamilton, NZ. The project was a 2.4km long pipeline with a 1050mm internal diameter concrete pipe installed up to 6m deep. The project duration was over nine months and was consented in two portions. This staged approach provided the opportunity to compare the modelled versus observed settlement in the first portion, and refine the model for the consenting of the second portion in order to reduce excessive conservatism and refine the monitoring regime. The benefit of adopting the Somerville method over other methods was its ability to rapidly assess the entire pipeline and the wide acceptance of the method by consenting authorities both of which minimised consenting delays.

1.1 Engineering Geological Setting

The study site lies entirely on the alluvial plains of Hamilton, within soils of the Hinuera Formation. The Hinuera Formation is derived from weathering of pumiceous ejecta from the volcanoes of the central plateau laid down in the braided river systems of the ancestral Waikato River on a large, low angle fan surface, which passed northward into an extensive braided river plain. The latest depositional episode for the formation is dated at between 22,000 and 17,000 years ago (Lowe 2010). Cross bedded sands, silts and gravels dominate this highly variable unit.

Localised bogs have developed across the alluvial plain and whilst the study area lies close to the boundary of one such bog (Rukuhia Bog) boreholes undertaken for the project identified only minor organic horizons within the trench envelope. For the purposes of the study, the soils in the trench and shaft envelope can be considered to be interbedded fine to coarse sands and silt-sand mixtures such as are indicated in the core photos from the northern reception shaft MH1 in Figure 1 below.

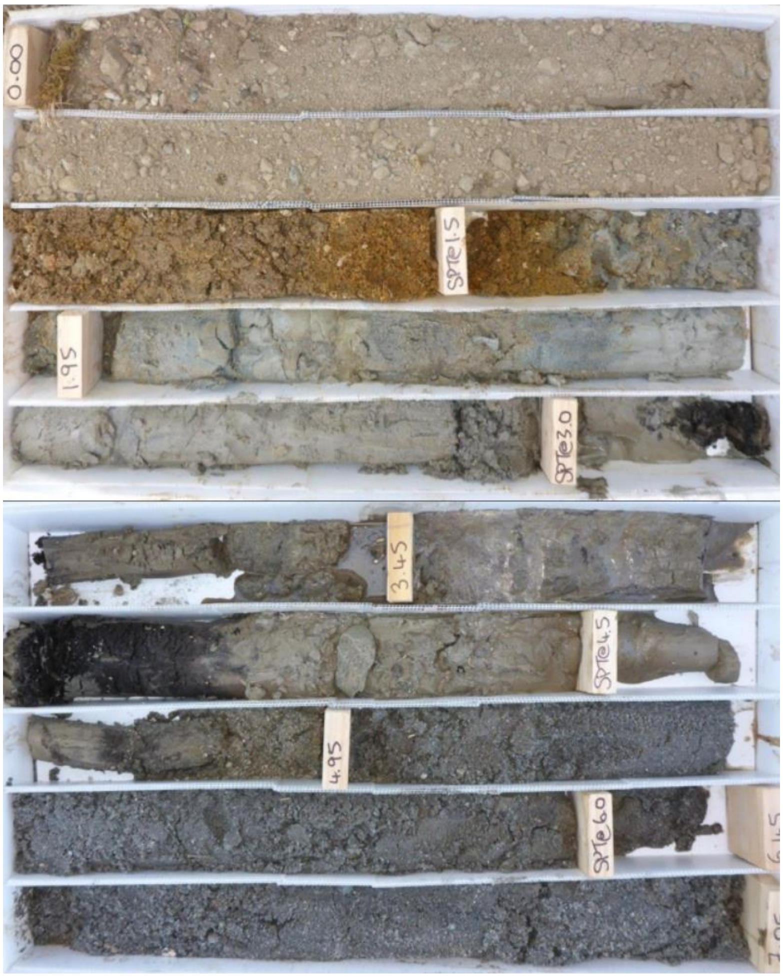

Figure 1: borehole core photographs for the northern reception shaft

Long-term groundwater monitoring was undertaken as part of the project and identified an unconfined aquifer with a seasonal high generally between 1-2m depth below ground level and a seasonal low typically 1-1.5m deeper. This is typical for the alluvial plains in the vicinity of Hamilton City.

2 DEWATERING INDUCED SETTLEMENT MODEL

The component of vertical movement due to dewatering and the component due to lateral displacement were modelled separately using GIS techniques and the separate settlement models (grids) were combined in order to assess the overall effects.

The process for the dewatering component, described in Sommerville 2005, is as follows:

First the radius of influence is established. The radius of influence (R_0) is a function of the drawdown (h) and the permeability (K) described by:

$$R_0 \cong Ch\sqrt{K} \tag{1}$$

Where R_0 and h are in metres, k is in m/sec and C is a factor equal to 3000 for radial flow to pumped wells and between 1500 and 2000 for line flow to trenches or to a line of wellpoints.

The drawdown effects and therefore the amount settlement, decrease in a log spiral manner with increasing distance from the trench. The Somerville model assumes the drawdown curve in Figure 2.

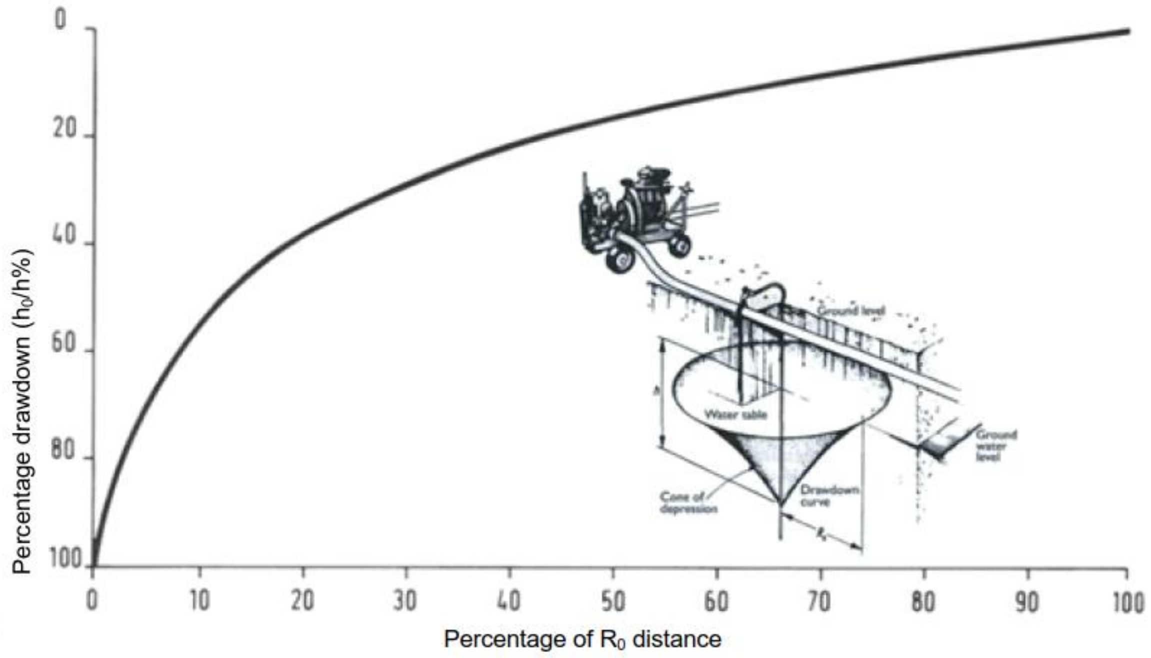

Figure 2: Relation of Drawdown to Distance from Centre of Cone of Depression

It can be seen from Figure 2 that the amount of drawdown (and therefore settlement) decreases rapidly from the trench edge as distance increases such that at 50% of the R_0 distance (50% of the radius of influence), drawdown may be only 20% of that experienced at the trench crest.

The calculations predicted a wide range in the extent of effects which reflects the wide range of permeabilities in the drawdown zone and the variability in drawdown height (h) along the pipeline route. Soil permeabilities were derived through the use of standard correlations with CPT data over the drawdown height.

The amount of settlement (mm) was estimated using the following method set out in Sommerville 2005:

$$\frac{gh_0 \times 50}{q_a} \qquad (2)$$

Where $g = 9.8$, h_0 is the drawdown experienced at the point of interest and q_a is the allowable bearing capacity at the point of interest, taken to be 100kPa for the model in order to equate to approximately 300kPa ultimate bearing capacity as is common for these granular Hinuera materials.

Through the use of GIS techniques, the alignment was broken into sections of similar soil permeability (K) and drawdown height (h). Drawdown (h) itself being a function of the relationship between pre-construction seasonal high water level and the invert level of the pipe at that location, plus an additional 1m depth to account for the depth of wellpoints below the invert level that is required in order to keep the point mid-way between the two rows of wellpoints dry. Currently only a full observation data set is available for the four Tunnel Boring Machine (TBM) shafts and therefore this paper focusses only on those four shafts.

Through GIS is was possible to derive settlement contours for each pipe section and shaft in turn and then blend them into a combined settlement model grid which could be later contoured.

Figure 3: Example of settlement contours derived for the southern shafts MH6 and MH7

3 LATERAL WALL MOVEMENT MODEL

The approach for the modelling of the effects of lateral wall movement was undertaken using the empirical methods described in CIRIA 760 and assuming the maximum of the reported surface settlements in the case studies provided for sand as indicated by the bold black line in Figure 4 below. From this it is apparent that the extent of settlement effects due to lateral movement in sand are typically limited to within twice the trench height.

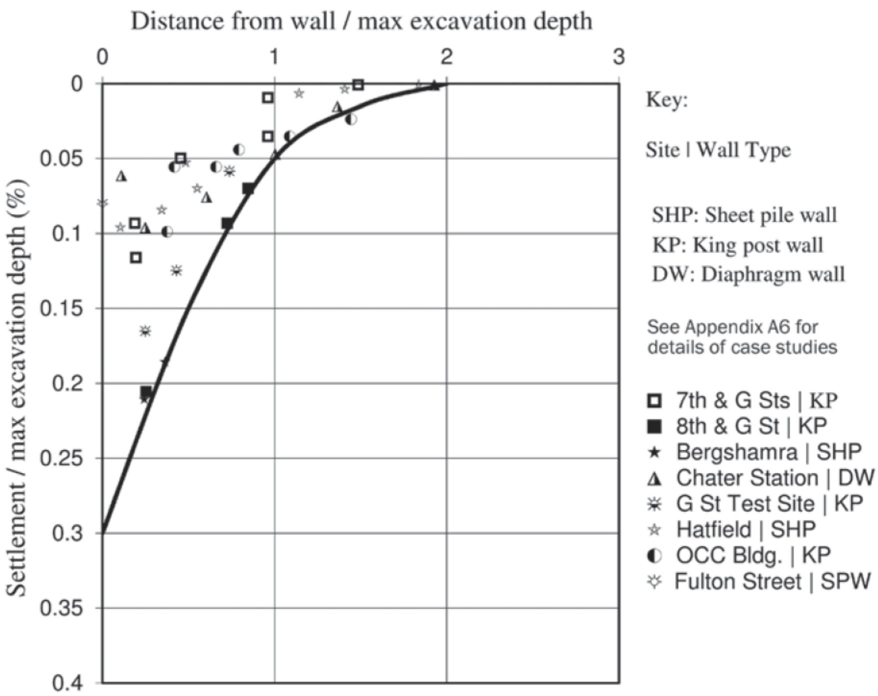

Figure 4: Ground Surface Settlement due to excavation in front of a wall in sand

An example for a 4m deep trench is as follows:

Table 1: An example for a 4m deep trench

Distance from wall (m)	**0.0**	**0.8**	**1.6**	**2.4**	**3.2**	**4.0**	**4.8**	**5.6**	**6.4**	**7.2**
Settlement (mm)	12	10.4	7.2	5.2	3.2	2	1.2	0.8	0.4	0.2

From which is can be seen that, in sand and for the range of trench depths commonly required in pipeline projects, the effects of lateral wall movement can be expected to approach the accuracy of traditional topographic survey methods (+/- 2mm) at around 1 x trench height from the trench edge and become undetectable thereafter.

Once again, using GIS techniques the alignment was split into sections of similar excavation depth and a settlement grid formed.

The two settlement grids (the one from dewatering and the second from lateral wall movement) were combined and contours derived for the 0mm, 2mm and 5mm contour etc.. The 2mm contour representing the limit of the accuracy of the surveying method and the limit beyond which any effect could be considered negligible (e.g. with reference to Rankin 1988).

Because GIS techniques had been adopted, it was possible to derive settlement slopes across the project area and compare those slopes to tolerable differential settlement criteria such as the building damage classification criteria of Burland (e.g. Burland and Wroth 1974 and Mair, Taylor and Burland 1996) and the equivalent for buried services (e.g. Bracegirdle et al. 1996).

4 MONITORING METHODS

Monitoring was undertaken by traditional topographic surveying techniques by a registered professional surveyor to assess the vertical settlement. Four types of survey point were installed: A shallow monument mounted just below the surface in grassed areas, survey pins in the pavement, a bolt located on the slab adjacent to foundations of structures and finally a series of deep points, consisting of a 0.5m rebar rod set with a top at a depth of 0.5m below ground level inside a sleeved hole, these latter points being designed to be highest-quality survey points unaffected by seasonal surface effects.

The survey points were supplemented by datums/benchmarks located well outside of the zone of influence of any dewatering.

Groundwater was measured at two cross sections each comprising of a pair of boreholes and each hole containing two vibrating wire piezometers (VWP) grouted-in and data logged with telemetry. In each hole, one VWP was located just below the level of the well points and another at around the mid-point of the trench depth.

5 DEWATERING REGIME

Around the shaft perimeter, wellpoints were typically 6m deep (the shafts being 5m depth) and spaced 1-1.5m apart, the shafts were supported by sheet piles with a whaler beam. Wellpoint dewatering was undertaken at the following rates and durations for the various shafts.

- MH1 northern reception shaft – dewatering at a rate of approximately 5-6l/sec for a period of 14 weeks
- MH2 northern launch shaft – dewatering at a rate of approximately 5-6l/sec for a period of 14 weeks
- MH6 southern reception shaft – dewatering at a rate of approximately 5-6l/sec for a period of 6 weeks
- MH7 southern launch shaft – dewatering at a rate of approximately 5-6l/sec for a period of 7 weeks

Given the permeabilities of the ground, it would be expected that a steady state of drawdown would be achieved in a matter of a few days to three weeks, therefore for all four shafts, it is assumed a steady state of drawdown was achieved during the monitoring period.

6 SURVEY REGIME

Dewatering and construction for the shafts started in late August 2019 and some 16 weeks of survey data was collected as construction proceeded. Surveying frequency was initially daily, reducing to twice weekly, weekly and then fortnightly in agreement with the regulator as confidence in the effects (or rather, the lack thereof) increased as the works progressed.

7 COMPARISON OF THE MODELLED SETTLEMENT AGAINST THAT OBSERVED.

The settlement model for the shafts has been compared to the observed settlement in Figure 5 below. Much of the observed data indicates simple scatter associated with the surveying method, generally +/-2mm accuracy, occasionally +/-3mm e.g. due to road-borne vibration. The data presented are results from the final survey round, undertaken once dewatering had stopped, this represents up to 14 weeks of dewatering for the MH1 and MH2 shafts and approximately 6-7 weeks at the MH6 and MH7 shafts. The predicted movement for each shaft has been shown individually. For clarity, only those points located in a zone modelled to experience >2mm settlement has been shown in Figure 5. The data for the areas modelled to experience <2mm settlement show the same trend however.

The figure shows only the dewatering-induced settlement component of the model because all of the observation points for the shafts were sufficiently far from the shafts for the effects of lateral wall movement to be zero.

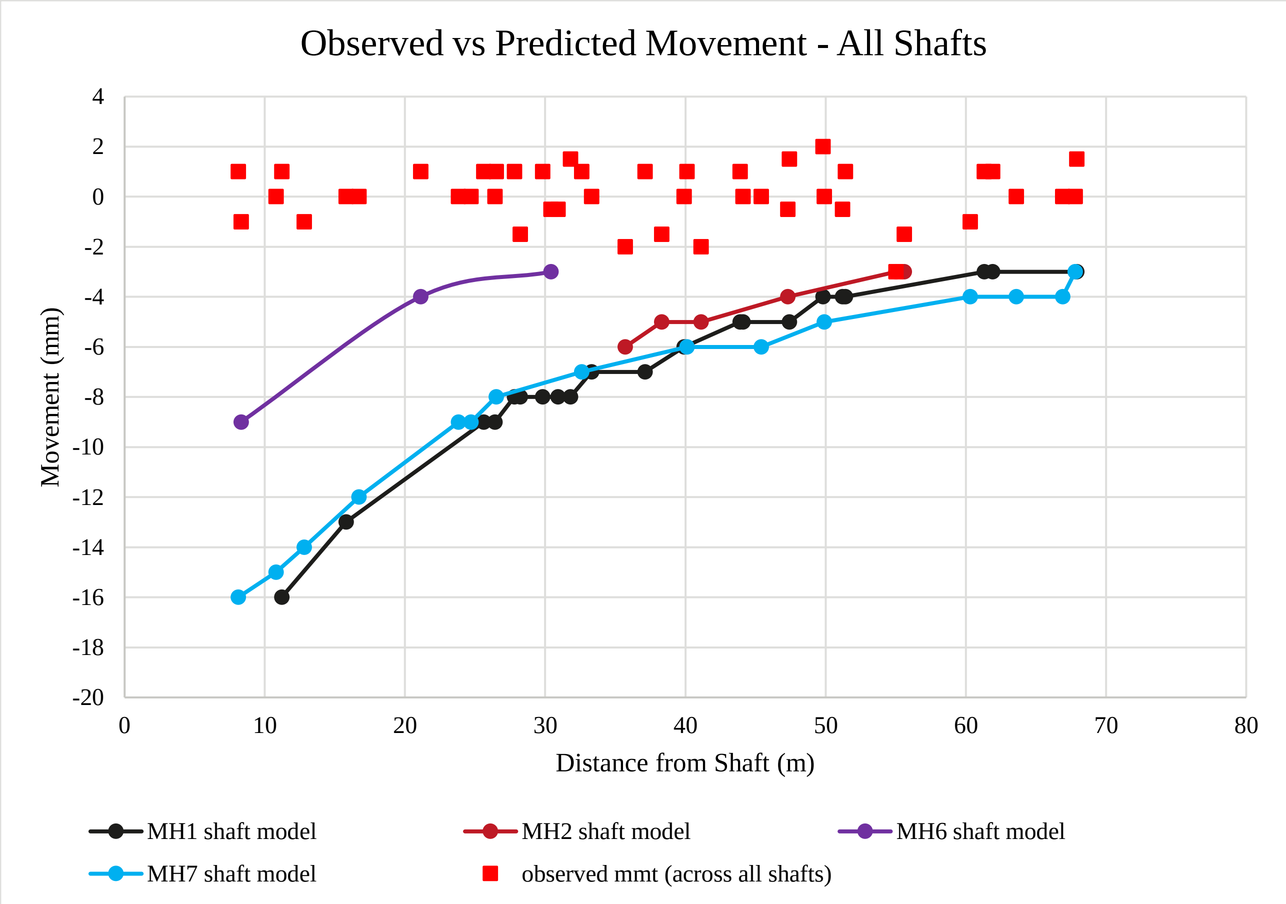

Figure 5: Observed vs Predicted Movement for all shafts

It can be seen from the above figure that no observed movement is discernible beyond the scatter associated with the survey technique, regardless of the type of monitoring point employed. We have already noted that the +/- 2mm scatter associated with the survey technique is too small to be of concern to structures and infrastructure.

In the model, settlements up to 38mm are predicted immediately next to the shaft walls and settlements over 20mm within 7m of the shaft walls are commonly predicted. In this this study it was not practical to install survey points in such close proximity to the shafts due to the scale of construction activities (e.g trafficking and sheet pile driving), additionally, there were no sensitive receptors in such close proximity and therefore surveying in close proximity to the shafts was not warranted.

Piezometers showed a similar subdued response, with the maximum groundwater drawdown recorded on the project being 0.8m (from 2m below ground level (bgl) to 2.8m bgl) in piezometers set back 40m from the active wellpoints and more a typical drawdown being <0.5m at 20m from the active wellpoints.

8 DISCUSSION AND CONCLUSION

This study found that beyond the zone of construction traffic, no surface effects from dewatering could be discerned. Therefore, whilst the methodology in Somerville 2005 might be appropriate as a first-pass for assessing effects, if those effects are found to be less than modest, the approach may be lead a project to undertake excessive and unnecessary monitoring and mitigation.

Reasons for the disparity between the settlement predicted by this method and that observed might be due to assumptions in groundwater level, permeability, allowable bearing capacity factor or simply the semi-empirical nature of the method.

The model conservatively assumed groundwater level to be the seasonal high, but for the purposes of settlement, this may be unrealistic in granular soils because negligible rebound may occur after the seasonal low and therefore a lower groundwater level may be more appropriate.

The study derived soil permeability from industry-standard correlations with CPT data, averaged over the range depths being dewatered using two averaging techniques and selecting the higher permeability of the two. CPTs are known to underestimate the fines content in mixed granular soils (Robertson and Cabal 2015) and therefore, may overestimate the permeability of such soils, resulting in a greater predicted effect. The study soils are complex and variable over short vertical and lateral distances, in such soils it may be beneficial to expend more effort in establishing mass vertical and horizontal permeability if the first pass assessment identifies more than modest effects.

On the basis of this study, it is recommended that should the Sommerville 2005 method identify anything other than modest effects, the cost-benefit of undertaking additional studies be considered. Additional studies may include reference to other published real-world studies in similar soil types in order to justify the use of reduced factors in the model, full scale dewatering trials in similar soils, and/or detailed mathematical modelling of the effective stress changes in specific problem areas.

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