# Settlement considerations for AMETI Panmure Phase 1

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## ABSTRACT

AMETI Panmure Phase 1 Project involves construction of three bridges and a covered box in Auckland Volcanic Field consisting of a thin basalt cover underlain by tuff and Puketoka Formation alluvium. Kaawa Formation "bedrock" is at approximately 40 m depth. The Serviceability Limit State (SLS) has been found to be the critical state for the design of foundations across the site. This paper presents considerations of settlement effects on the bridge piles and shallow foundations. Settlement analysis using PLAXIS is discussed and is compared with monitoring results.

## 1 INTRODUCTION

AMETI Panmure Phase 1 Project involves the construction of a 2 km Link Road (main alignment) adjacent to the existing NIMT rail, upgrading of Mountain Rd over-bridge and Ellersie-Panmure Highway (EPH) over-bridge, and enhancement of the Panmure Rail Station (see Figure 1). Between the EPH and Mt Road over-bridge, the Link Road is built as a covered box to create a bus and rail interchange. A footbridge and a concourse are built over the existing rail to connect with the covered box. The upgrading of the existing EPH bridge involves the construction of two new bridges built immediately adjacent to each other in two stages.

The site is located in Auckland Volcanic Field consisting of a thin basalt cover underlain by tuff and Puketoka Formation alluvium. Kaawa Formation sandstone is at approximately 40m depth. The Mt Wellington volcano is immediately adjacent to the site, from which the basalt has originated. The basalt varies significantly in its thickness. Excavation up to 5 m deep was required to build the Link Road. The remaining basalt after the excavation is approximately 3 m thick near the southern end of the covered box and is thicker near the northern end.

The presence of a thin basalt cover underlain by a thick layer of compressible soil complicated the design. Both ultimate limit state (ULS) and serviceability limit state (SLS) have been analysed. It is found that the SLS is more critical than ULS given an adopted SLS criterion of 10 mm settlement. The covered box and the footbridge are on shallow foundations founded on the basalt. The Mt Road overbridge is on short piles which are installed into the basalt cover. EPH bridges and the concourse are on deep piles embedded into the underlying Kaawa Formation bedrock.

This paper presents geotechnical analyses with a focus on settlement considerations in the design of shallow foundations and bridge piles for this part of the AMETI project.


**Figure 1: Site aerial photo looking north (February 2013)**

## 2 GEOLOGY AND SUBSURFACE CONDITIONS

The site lies within Holocene aged Auckland Volcanic Field Basalt (GNS, 1992). The basalt has originated from the nearby Mt Wellington volcano. The centre of the crater is approximately 550 meters to the north-west of the site. The Mt Wellington volcano first erupted 9300 (±150) years ago (Searle, 1981). The basaltic lava flowed towards the west and reached as far as Penrose in the south-west and Remuera in the north-west. Some lava also flowed towards the east and south but didn't extend far away from the crater. The existing Panmure rail corridor and Ellerslie-Panmure Highway are located near the edge of this lava flow. Figure 2 shows an indicative Mt Wellington lava field and is inferred from the above published geology.

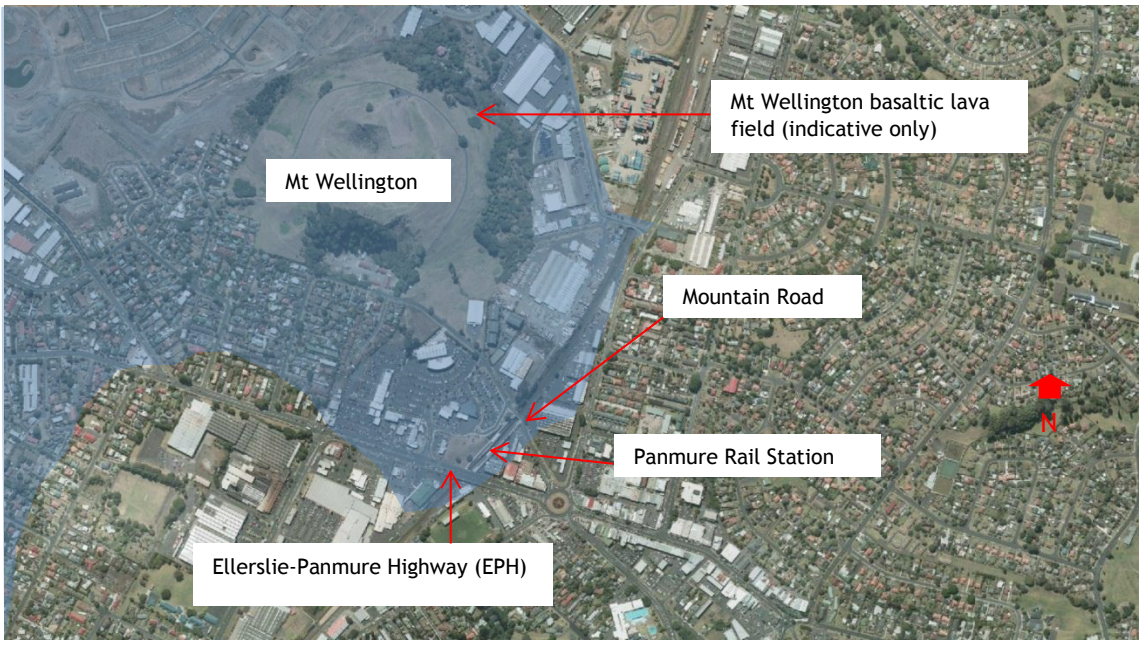


**Figure 2: Indicative Mt Wellington lava field (inferred from published geology)**

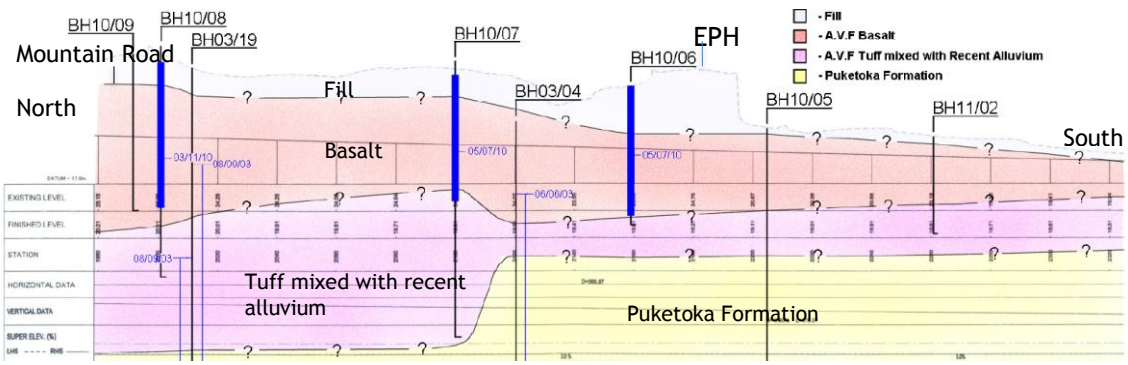


**Figure 3: Geological long section**

The geotechnical investigation results indicate that basalt exists beneath the entire site although its thickness varies significantly. It was found to be thickest to the north and thinning towards the south as shown in Figure 3. Man-made fill of varying thicknesses was encountered overlying the basalt. Tuff mixed with inter-bedded layers of recent alluvium was encountered beneath the basalt. Underlying this sequence is a thick layer of Pleistocene aged Puketoka Formation alluvium which overlies Pliocene aged Kaawa Formation soil and sandstone at approximately 40 m depth. The groundwater table was found to be within 1m of the rail tracks.

## 3 GENERAL ASSUMPTIONS AND PARAMETERS

Auckland volcanic field basalt is highly fractured and contains voids in variable size. The geotechnical investigation indicates that the basalt at the site is slightly to highly vesicular. Major vertical fractures are typically 0.5 m to 1.0 m spaced and are cross cut by horizontal fractures at variable depth. Although there were some lost cores during the investigation, no large voids were encountered. The core recovery was generally greater than 80%. The typical RQD value was around 60% or higher. The basalt was moderately strong to strong with a median UCS of 50 MPa and a 20$^{th}$ percentile UCS of 30 MPa. The GSI was assessed to be in the range between 50 and 75 for the majority of basalt and occasionally in the range between 40 and 50.

Based on the conditions of basalt on the site, a minimum width of 2 m was recommended for shallow foundations in the basalt. No grouting was considered necessary for the basalt beneath foundations. Basalt conditions were further verified by proof bores during the construction, which did not identify any significant voids (>300mm diameter).

Settlement was analysed using Plaxis. For simplicity and consistence, the Mohr-Coulomb failure criterion was utilised for both basalt and soil. However, for the basalt, equivalent Mohr-Coulomb parameters were converted from the Hoek-Brown failure criterion (Hoek E. et al, 2002) taking into account rock mass quality.

The underlying soil has a typical SPT N value of 12 for tuff and 8 for Puketoka Formation alluvium. Thin sand layers are present within the alluvium. Oedometer consolidation tests on samples from 9 m to 12 m depth showed that the volume compressibility coefficient ($m_v$) was in the range between 0.1 and 0.2 $m^2$/MN. The ratio of soil modulus between rebound and compression was assessed to be 6 to 12.

Excavation induced rebound was modelled and was assumed to have occurred prior to the reloading/recompression. Rebound and recompression were assumed to be linear elastic corresponding to higher stiffness. The above oedometer tests of soil were used to develop a ratio of Young's modulus between rebound/recompression and inelastic compression. Parameters adopted for Plaxis settlement analyses are summarised in Table 1.

Table 1: Parameters for Plaxis settlement analyses

| Material type | Bulk density ($kN/m^3$) | Young's Modulus (MPa) | | C' (kPa) | Φ' (°) | Poisson's ratio | Dilation (°) |
|---|---|---|---|---|---|---|---|
| | | Rebound & Recompression | Inelastic Compression | | | | |
| **Basalt** | 27 | 3450 | 3450 | 3540 | 40 | 0.3 | 10 |
| **Tuff** | 18 | 150 | 15 | 2 | 31 | 0.3 | 0 |
| **Puketoka soil** | 18 | 100 | 10 | 2 | 29 | 0.3 | 0 |

The consolidation analysis was also analysed using Plaxis. A coefficient of permeability of $2.2\times10^{-5}$ m/day ($2.5\times10^{-10}$ m/s) was adopted for the Puketoka Formation cohesive soil. A thin sand layer was applied in the middle and at the bottom of the cohesive soil. Based on the analysis, it was considered that a three-month period should be allowed for the settlement to occur. This was used as a general guidance in the construction program, e.g. the timing of installing roof beams of the covered box.

## 4 ANALYSES

### 4.1 EPH/RTN Bridges

The upgrading of the existing EPH rail over-bridge was required for the rail electrification and the construction of the new AMETI Link Road (i.e. main alignment). Two new bridges were built immediately adjacent to each other and were constructed in two stages. The existing EPH was widened on both southern and northern sides, but mainly towards the northern side in order to create the Rapid Transit Network (RTN). To keep the traffic running, the upgrading was undertaken in two stages. During the first stage, the existing EPH was partially demolished to build the RTN bridge on the northern side while the traffic was maintained on the remaining EPH. At the second stage, after the RTN bridge and its approaches were completed, the traffic was diverted to the RTN and the remaining EPH was then demolished for construction of the new EPH bridge.

Each of these two bridges has two spans (25m and 29m). To increase the clearance over the rail for electrification, fill up to 3.5 m high was placed above the existing road for the EPH bridge approach embankment. On the northern and southern sides, fill up to 7 m high was placed in order to widen the road. The bridge abutment is not perpendicular to the deck but is skewed at 67º to the deck. The skewed width of the two bridges is 52 m for the eastern abutment and 61 m for the western abutment. The Plaxis model for the settlement analysis is shown in Figure 4 for the eastern abutment.

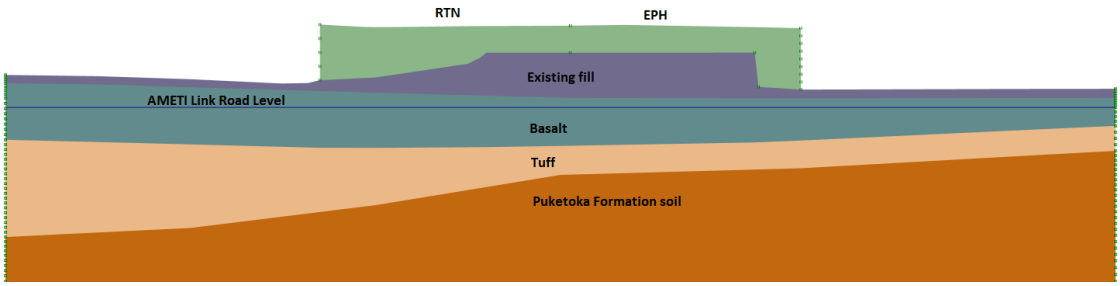


**Figure 4: Plaxis model for settlement analysis (EPH/RTN eastern abutment)**
(Only part of Puketoka Formation soil is shown)

In addition to the construction of the abutments, an area near the southern end of the west abutment is to be landscaped involving filling up to 7m high in the stage 2 after the EPH bridge is built. This has been included in the analysis for the western abutment (not shown). A summary of the calculated settlement is shown in Table 2.

Table 2: RTN/EPH abutment calculated settlement

| Location / Construction Stage | East abutment | | West abutment | |
|---|---|---|---|---|
| | RTN | EPH | RTN | EPH |
| **Stage 1: RTN construction** | 40 mm | 25 mm | 20mm | 15 mm |
| **Stage 2: EPH and landscape construction** | 30 mm | 35 mm | 40 mm | 55 mm |
| **Total settlement** | 70 mm | 60 mm | 60 mm | 70 mm |

The settlements at the abutments are taken to be 60% of the settlements obtained from the Plaxis analyses as the 2D model is for an infinite length while the abutments are at the edges of the approaches which have a limited length only.

The influence of the landscape fill is ignored for the east abutment, but is taken into account for the west abutment. For the west abutment, the future backfilling to the west of the covered box is also considered in the modelling. (Note: the western wall of the covered box abuts the northern end of the west abutment of RTN.)

It should be noted that due to the excavation between the east and west abutments, the ground near the abutments undergoes mainly elastic re-compression while the ground away from the abutments undergoes more inelastic compression. However, this combination of the soil behaviours cannot be modelled in a 2D transverse cross section model (note: excavation area is between the two abutments and is more on the western side). For the results given in Table 2, the ground is assumed to undergo fully inelastic compression. For the extreme case assuming the ground is in fully elastic re-compression, analyses indicate that settlements are approximately 50% lower.

Bridge piles for RTN/EPH were installed into the Kaawa formation. There is a risk that approach embankment settlements may induce negative skin friction to the piles of the east and west abutments. Although the risk is considered to be relatively low, it was considered prudent that this be considered in the design and construction of bridge piles (excluding the piles for the central piers). As it was assessed that the settlement would occur within approximately three month, either a three-month preload time should be allowed for the approach embankment prior to the pile construction or the negative skin friction be considered in the pile design. Due to the tight construction program, the latter option was selected.

Other considerations include settlement effects on the RTN approach embankment due to the staged construction. A surcharging loading was recommended to preload the RTN. Otherwise it was considered to be likely that the RTN pavement would need to be reshaped following the construction of EPH. During the construction, no preload was undertaken in order to speed up the construction.

At the completion of RTN, a maximum settlement of 15 mm was obtained at the RTN eastern abutment. The settlement became stable within 1.5 to 3 months. At the time of preparing this paper, the EPH construction (i.e. stage 2) had not been completed. No further settlement monitoring results were available for discussion in this paper.

## 4.2 Mountain Road Bridge

The Mountain Rd bridge has two spans (23m and 17.5m). Piles were installed in basalt. The thickness of basalt below the piles is at least 3 m. The tuff layer at the Mountain Road is at least 7 m in thickness (thicker than RTN/EPH sites). The top-down construction method was adopted for this bridge. The piles and bridge deck were constructed first and the excavation was undertaken later from underneath the bridge.

The Plaxis model for settlement analysis is shown in Figure 5. The working load per pier is 6050 kN at Pier A, 8000 kN at Pier B and 7420 kN at Pier C. In addition to the working loads, excavation and the new filling road were taken into account in the settlement analysis by modelling the top-down construction sequence. The maximum calculated settlement is 16 mm at Pier A, 8 mm at Pier B and 6 mm at Pier C. The maximum differential settlement is about 1/2000. (Note: this bridge was open for traffic in late 2012. No settlement monitoring information was available.)

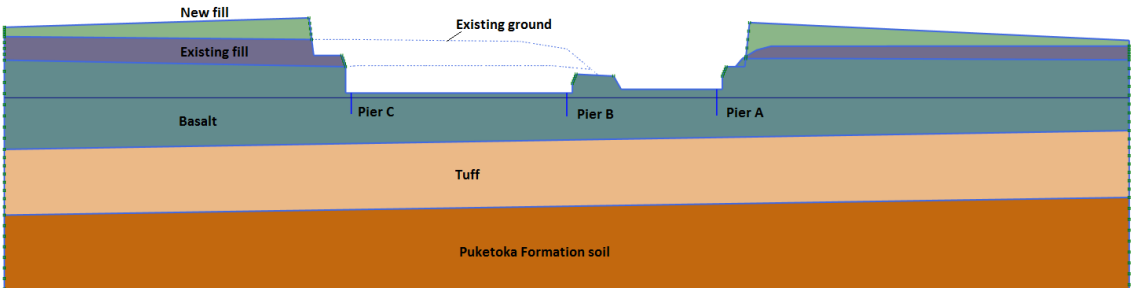


**Figure 5: Plaxis model for settlement analysis (Mountain Rd Bridge)**
(Only part of Puketoka Formation soil is shown)

## 4.3 Panmure Covered Box (PCB)

The construction of PCB involved excavation of basalt/basaltic fill up to 5 m deep. The box is exposed on the eastern side while it is buried on the western side with new fill above the existing ground (see Figure 6). This results in the requirement of the western wall foundation being larger than the eastern wall foundation mainly due to the retention design. Settlement considerations include:

- A settlement criterion (i.e. 10 mm) was adopted for the shallow foundation design;
- A three-month settlement period was recommended for the new fill at the western wall prior to the installation of roof beams.

The analysis allows for an initial linear elastic recompression (apparent elastic only) and subsequent inelastic compression when the loading is over a limiting pressure. It is assumed that this limiting pressure is such that the footing pressure is spread at an angle of 45˚ through the basalt layer and generates a uniform pressure of 120 kPa at the top of the underlying soil. Plaxis analyses were undertaken for 3 m and 7 m wide footings. The settlement was assessed to be less than 10 mm for the following pressures:

- 360 kPa for 3 m wide footing (eastern side)
- 200 kPa for 7 m wide footing (western side)

The existing ground to the west of the box was about 1.5m to 2.5m below the finished roof level of the box. The backfilling would result in a differential settlement of about 10 mm between the eastern and western footings of the covered box. Although the calculated differential settlement was considered to be small for normal structures, the settlement was largely localised to the western side. It was considered it might cause stress concentration in the box structure and in the pavement near the western footing. Therefore, it was recommended that a settlement period of three month be allowed prior to the construction of the roof structure of the box. This recommendation was followed during the construction. The roof structure was installed following the recommended settlement period.

The monitoring results show that the fill-induced settlement became stable within approximately two month corresponding to a maximum settlement of 8 mm.

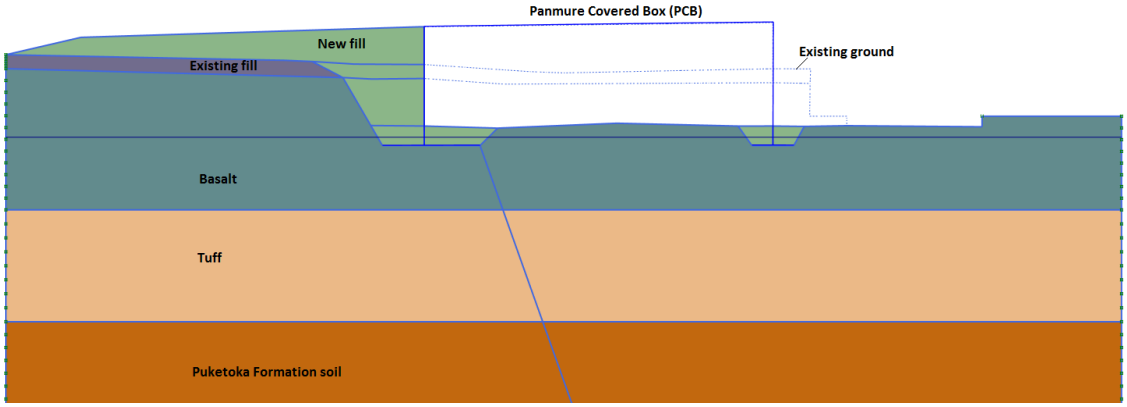


**Figure 6: Plaxis model for settlement analysis (Panmure Covered Box)**
(Only part of Puketoka Formation soil is shown)

## 5 CONCLUSIONS

The following conclusions have been drawn based on the design, proof bores and construction monitoring:

- The basalt left after the excavation is approximately 3 m thick and is underlain by tuff and alluvium up to 40 m deep;
- Basalt across the site is moderately strong to strong. Vertical fractures are typically spaced at 0.5 m to 1.0 m. Proof bores and original investigation didn't identify any voids larger than 300 mm in diameter in basalt; therefore no grouting was required for the shallow foundations of the covered box;
- Rebound/recompression behaviours of soil have significant effects on settlement. The settlement prediction has been found accurate if rebound/recompression behaviours are modelled (e.g. the covered box);
- The construction of RTN/EPH approach embankment has not been completed. The current monitoring results show that the actual settlement is close to the lower bound of the prediction which corresponds to fully elastic re-compression behaviours of soil;
- It has been found that settlements at the site can become stable within three month or less. This is consistent with the Plaxis analysis;
- A settlement criterion of 10 mm was adopted for the foundation design which has been found to govern the design.

## ACKNOWLEDGEMENTS

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