GEOTECHNICAL DESIGN OF THE SOUTHERN MOTORWAY UNDERPASS

Ka-Ching Cheung, Duncan Peters
Connell Wagner Ltd, Auckland
New Zealand

SUMMARY

The Southern Motorway Underpass takes the new South Eastern Highway beneath the Southern Motorway (State Highway 1), one of the busiest roads in Auckland. One of the major requirements for the construction of the Southern Motorway Underpass was to minimise disruption to the motorway traffic. This was done by building the northern half of the bridge and moving traffic onto it before building the southern half of the bridge and also by adopting the top down method of construction. The top down method was achieved by initially constructing the soldier piles for the abutment walls, tying back to deadmen with pre-stressed anchors and initially excavating just sufficient material between the bridge abutments to construct the piers and place the bridge beams. To reduce the depth of temporary sheet piling required between the north and south halves of the bridge the remaining bulk excavations beneath the bridge deck were only carried out once both halves of the deck were opened to traffic. The skewness of the bridge meant that the distance between the bridge abutments and deadman had to be unusually short to keep the deadmen within the motorway embankment. Because of the soft ground conditions, there was potential for the 6.5m deep excavations to induce lateral abutment wall movements which would not have been acceptable to the bridge or the pavement of the motorway. This paper addresses the geotechnical design and construction aspects of the project.

1 INTRODUCTION

The Southern Motorway bisects the industrial area of Penrose in the west and the light commercial and residential area of Mt Wellington in the east. Prior to the completion of the South Eastern Highway, the nearby Mt Wellington Highway was one of the most heavily trafficked urban roads in Auckland and it was usually heavily congested during peak hours. The South Eastern Highway provides a convenient traffic link between the western and eastern areas via the Southern Motorway Underpass beneath the Southern Motorway.

The Southern Motorway (State Highway 1) is one of the busiest roads in Auckland and any significant disruption of the traffic due to the construction of the underpass was unacceptable. The first step in the construction of the underpass was to divert the northbound traffic on the motorway away from the bridge onto what was later to become a part of the South Eastern Highway and moving the southbound traffic onto the northbound carriageway. After the diversion was completed the northern half of the underpass which would carry the future southbound motorway lanes was constructed using the top down construction technique. Soldier piles and the dead men were constructed for the northern half of the bridge. The prestressed anchors between the abutments and the deadmen were placed in directionally drilled holes and stressed. Only enough soil was excavated between the bridge abutments to construct the bridge piers and place the deck beams. After the completion of the northern half of the underpass southbound traffic was moved onto it and construction was then commenced on the southern half of the bridge. Once both bridge decks were completed and opened to motorway traffic the bulk excavations beneath the new bridge were commenced. As excavation progressed the gaps between the soldier piles were closed with fibre reinforced shotcrete arches. Finally precast concrete panels were placed in front of the soldier piles to enhance the appearance of the bridge.

The underpass is a highly skew two span bridge designed to carry for a total of 7 lanes of traffic. Its construction involved a 6.5m deep excavation in soft ground, Figures 1 and 2. The bridge decks carrying the north and southbound carriageways are separated along the centerline of the motorway. Because of the bridge's skewed geometry it was not practical to use the decks as props for the abutments. Instead, the soldier pile abutment walls were tied back to deadman anchors by prestressed anchors. The combined resistance of the piers and abutments was needed to provide sufficient seismic restraint for the decks. This was achieved by connecting the decks to the piers with fixed bearings and gaining further resistance by utilising the shear stiffness of rubber bearings at the abutments. As a result only a limited amount of abutment movement could be accommodated. As the motorway is on an embankment, the very high skew of the bridge meant that the deadmen had to be placed closer to the bridge than normal practice would require. In order to address these two aspects one dimensional and two dimensional elastoplastic soil modelling were employed in the design to obtain an understanding of the interactive behaviour between the bridge abutment, the deadman anchor and the ground movement caused by the excavation.

2 GROUND CONDITIONS

The ground underlying the underpass site varies from one end to the other. Based on information from four boreholes, the ground profile may be generalised by a four layer system. A typical ground profile is shown in Figure 3. The near surface material is a volcanic ash layer varying from 2m to 4m thick. Beneath the volcanic ash is an alluvial silty clay layer varying in thickness from 7m to 14m with standard penetration test SPT N values in the range of 4 to 10 however N values as low as 2 were also recorded in one of the boreholes. The alluvium is underlain by a layer of residually to moderately weathered Waitemata mudstone, siltstone and sandstone. Its thickness varies from 5m to 10m with N values typically between 7 to 30. The underlying slightly weathered Waitemata formation generally has N values of over 50.

3 SOIL PARAMETERS

Drained and undrained analyses were carried out in the design. The following effective stress parameters which are based on a limited number of triaxial test results around the project site, were used in the drained analysis:

Volcanic ash	c'= 3kPa	Ø' = 30°
Alluvial silty clay	c'= 15kPa	Ø' = 18°
Residual to moderately Weathered Waitemata formation	c'= 10kPa	Ø' = 24°
Slightly Weathered Waitemata formation	c'= 15kPa	Ø' = 35°

For the undrained analyses, undrained shear strength Su values for the soils were estimated from the SPT N blow counts measured in the boreholes based on Su = ION kPa. Undrained Young's modulus values were estimated to be related to 200Su. The total densities of the soils are assumed to be 1.85t/m^3.

4 DESIGN PHILOSOPHY

The underpass passes obliquely through the embankment beneath the existing motorway. Because of its skewed geometry, the earth pressure from the two abutment walls will generate a torque and cause the underpass to rotate in plan if the decks are used to support the abutment walls. One option to eliminate the torque is to tie back the soldier pile abutment walls with either rock anchors anchored in the slightly weathered mudstone and sandstone materials at about 20m depth or anchors tied to deadmen located in the upper soil layer.

It was estimated that the cost for the rock anchors was likely to be in the order of 500% higher than the deadman anchors. The deadman anchors were therefore employed on the project. The adoption of the deadman anchors required careful consideration for the following reasons:

- The excavation for the underpass is within a softer soil zone and it could induce both lateral ground movements affecting the completed bridge and settlement of the existing motorway pavement.
- As the deadmen are located just above the softer material the excavation could induce the deadman and the wall to move together.
- As a result of the skewness of the bridge, the deadmen have to be brought so close to the abutments as to render them ineffective.

5 STRUCTURAL LAYOUT

The sizes of the soldier piles and the deadmen and the capacity of the tie-back anchors adopted for the underpass are given below:

- 0.9m diameter reinforced concrete piles at 2.6m centre-to-centre spacing within the abutment wall area founded at ±20m depth within the slightly weathered Waitemata mudstone, siltstone and sandstone formation.
- 2m high deadman located about 12.5 m behind the abutments and buried 1m below ground surface.
- Tie back anchors of 1750kN minimum breaking capacity were installed at 5m spacing.

6 DEADMAN DESIGN

The ultimate bearing capacity of the deadman in the horizontal direction was derived from Prantl's solution or the so-called "punching shear" problem of classical plasticity theory. In the design, the undrained shear strength of the surrounding soil was assumed to be 50kPa and the contribution of the soil confinement behind the back of the deadman to the bearing capacity was ignored. The method assumes that no soil movement occurs outside the shear zones enclosed by the shear surfaces in front of the deadman. Soil excavation in front of the soldier pile wall may however induce small ground movements around the deadman. To mobilise the bearing resistance of the deadman, the actual deadman movement must be larger than the excavation induced ground movement. If the relative movement between the deadman and the ground is small, the ultimate bearing capacity of the deadman cannot be developed. Therefore, the ultimate bearing capacity of the deadman may not be a governing factor.

7 SOLDIER PILE WALL DESIGN METHODS

The geometry and the layout of the underpass's soldier pile abutment walls and the deadmen are shown in Figures 1&2. In the design, sensitivity analyses were undertaken at various cross sections along the abutment walls using a range of selected soil parameters and the methods listed below.

(i) One dimensional limit equilibrium method for soldier pile walls without tie back anchors.
(ii) One dimensional elastoplastic finite element method for an embedded cantilever wall with and without tie back anchors using the computer program Wallap
(iii) Two dimensional elastic finite difference method using the computer program FLAC for the soldier pile walls subjected to stage excavation.
(iv) Two dimensional elastoplastic finite difference method using the computer program FLAC for the soldier pile walls subjected to stage excavation

One Dimensional Limit Equilibrium Method
The method proposed by Burland et al (1981) which is commonly used for embedded cantilever walls was used. To ensure that the abutment walls have sufficient stability at all times, the soldier pile walls were considered as an "unpropped" wall. A minimum factor of safety of 1.5 against overturning failure was selected. Both drained and undrained analyses were carried out to determine the governing condition. When the active earth pressure in undrained cohesive soil is small or negative, a minimum earth pressure calculated from a minimum equivalent fluid density of 5kN/m3 was used in accordance with codes of practice CP2.

One Dimensional Elastoplastic Finite Element Analyses
The analyses were undertaken to evaluate the bending moment envelope for the soldier pile wall and the load at the deadman anchors caused by the excavation. In the method, the wall was considered as an elastic wall while the soil was assumed to remain elastic until its stress state reached the given yield envelope. Analyses for the both free cantilever and deadman tied back conditions were also carried out. Deadman movement of 1-5mm were also modelled to check for the influence of the deadman movement on the bending moment of the wall. The analyses showed that the undrained soil condition generated the most unfavourable bending condition in the soldier pile wall.

Two Dimensional Elastic Finite Difference Analyses

To evaluate the interactive behaviour between the bridge abutment, the deadman anchor and the overall ground movement caused by the excavation, two dimensional finite difference analyses were performed. Both the pile and the soil were modelled as elastic materials. The results of the one dimensional finite element model were used to calibrate the validity and accuracy of the 2D model. The following construction stages were analysed:

(i) Excavate 2m of the upper soil for the installation of the tied back anchors
(ii) Pre-stress the tie back anchors to the design load estimated from the one-dimensional Wallap analysis.
(iii) Excavate the soil to the final excavation depth of 6.5m.

Two Dimensional Elastoplastic Finite Difference Analyses

A Mohr Coulomb elastoplastic soil model was also used in the analyses. In the elastoplastic model, the material response was assumed to be elastic for any stress state of the soil inside the defined yield envelope. Plastic flow would occur when the stress state reached the yield envelope. The results of the 2D elastic model were first used to calibrate the validity and accuracy of the elastoplastic model. Parametric studies using a range of soil parameters and various anchor lengths were then undertaken to evaluate the interactive behaviour between the bridge abutment, the deadman anchor and the overall ground movement caused by the excavation.

8 RESULTS OF ANALYSES

A series of undrained analyses were undertaken based on soil parameters calculated from the SPT results from the boreholes at the four corners of the underpass. The two dimensional finite difference model is shown in Figure 4 and the calculated ground movements after the completion of the excavation at the location of borehole BH11, where softest ground condition is located, are shown in Figure 5. Generally, it was found that the differences between the 2D elastic and elastoplastic models were not significant indicating that the stress system within the ground after the excavation remained elastic. The range of results for the excavation induced movement on the deadman tied back soldier pile wall from both one and two dimensional models are summarised below:

Table I Comparison of Results

	Calculated Movement	
	1-D Wallap Analyses	2-D FLAC Analyses
Top of wall movement after 2m excavation	3-10mm	5-15mm
Top of wall movement after 6.5m excavation	10-40mm	12-45mm
Deadman movement after 6.5m excavation	1-5mm (Assumed input parameter)	3-l0mm

Generally, the 2-D finite difference analyses indicated that minor deadman movement due to the excavation is unavoidable because of the presence of the softer layer below the deadman level. The analyses showed that, after the installation of the anchor, subsequent soil excavation would induce about 5% additional anchor load. It also indicated that moving the deadman further from the wall would result in very little advantage over the adopted scheme because a longer anchor tie would have a greater elongation under the excavation induced load.

The results generally indicated that it was not possible to eliminate the soldier pile wall movements induced by excavation. It was therefore desirable to carry out as much excavation as possible prior to the placing bridge deck beams. Furthermore the results showed that the excavation induced ground movements would increase the anchor tie load above the initial pre-tension load. It was therefore important to allow for the load increase in determining the pre-tension load to avoid overstressing the anchors ties.

9 CONSTRUCTION AND MONITORING RESULTS

From the results of the analyses, it was believed that soldier pile abutment wall movements in the order of l0mm-30mm could to be expected due to the excavation. It was therefore decided that after the installation of the soldier piles and the construction of top capping beam as much excavation as possible should be undertaken in the central part of the underpass before placing the deck beams so that subsequent wall movement could be reduced to a minimum. It was determined that if the monitored abutment wall movement was in excess of 5mm the bridge beams would be lifted to allow the rubber bearings at the abutment to release their accumulated shear deflection. De-stressing of the anchor would also be required if the abutment movement caused the stress in the anchor ties to exceed 60% of their ultimate tensile capacity.

The excavation was first carried out to 2m depth to enable the installation of the tie-back anchors. The anchor ties were stressed to the design value of 30% of their ultimate tensile capacity giving a short term allowance of 30% (20% long term) for any excavation induced increase in the tension load.
Excavation at the centre part of the underpass was then carried out to leave a 3m wide ledge at the anchor level with a 45^0 cut slope to protect the retained soil from squeezing through the gaps between the piles. After placing the deck beams and casting the deck slab the bridge was opened to traffic and excavation between the abutments continued. The soil was cut to a semi-circle between each pair of the soldier piles and sprayed with fibre reinforced shotcrete.

Regular deformation measurements were carried out as the excavation progressed. It was found that the excavation before placing the bridge beams induced less than 3mm of wall movement. After the completion of the bridge deck, the removal of the remainder of the excavation induced only 2mm to 3mm of movement and it was not necessary to re-seat the rubber bearings. A total abutment movement of less than 6mm was measured which was below the predicted lower bound movement of 10mm. The under estimation was probably due to (i) the Young's modulus profiles being slightly underestimated and (ii) the 3-dimensional layout the abutment and the wing walls was stiffer than 1 and 2 dimensional models.

10 CONCLUSIONS

The geotechnical design and construction aspects of the Southern Motorway Underpass have been discussed in this paper. One-dimensional finite element and two dimensional finite difference modelling were employed to evaluate the interactive behaviour between the soldier pile bridge abutment wall, the deadman anchors and the overall ground movement caused by the 6.5m excavation in soft ground. The results of the analyses indicated that the excavation induced abutment wall and deadman movement could not be avoided. It was therefore decided that to carry out as much excavation as possible before the placing of the bridge deck beams so that subsequent wall movement due to the removal of the soil temporary supporting the soldier pile abutment wall could be reduced to a minimum. Construction monitoring showed that 50% of the abutment wall movement occurred before the placing the bridge beams. The wall movement induced by the removal of the remaining soil was within acceptable limits for the rubber bridge bearings and it was not necessary to reseat them or de-stress the anchor ties. Tying back the top of the abutment walls to the deadmen was still effective even though the deadmen were much closer to the abutment than would normally be the case. Numerical modelling was found to be an important tool in developing a suitable design for the tied back soldier pile abutments.

11 REFERENCES

[1] Burland J B and Potts D M and Walsh N M (1981) "The overall stability of free and propped embedded cantilever retaining walls", Ground Engineering, pp 23-38.
[2] CP2 (1951) "Earth Retaining Structures" Civil Engineering Code of Practice No.2, Institution of Structural Engineers.
[3] FLAC (1995) "Fast Lagrangian Analysis of Continua" Version 3.3.
[4] Wallap "Anchored and Cantilevered Retaining Wall Analysis Program" Version 3.4.

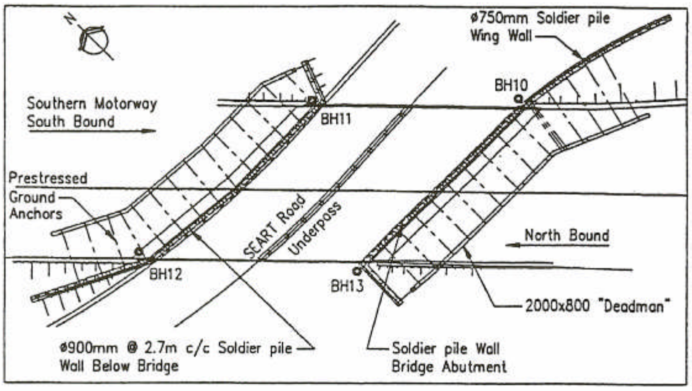

Figure 1 Underpass Layout and Borehole Locations

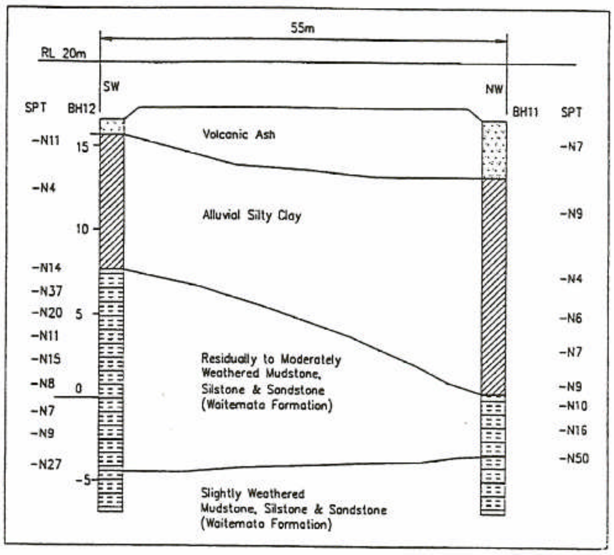

Figure 3 Typical Ground Profile

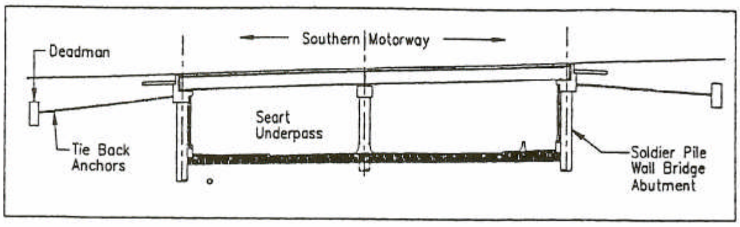

Figure 2 Cross Section of the Underpass

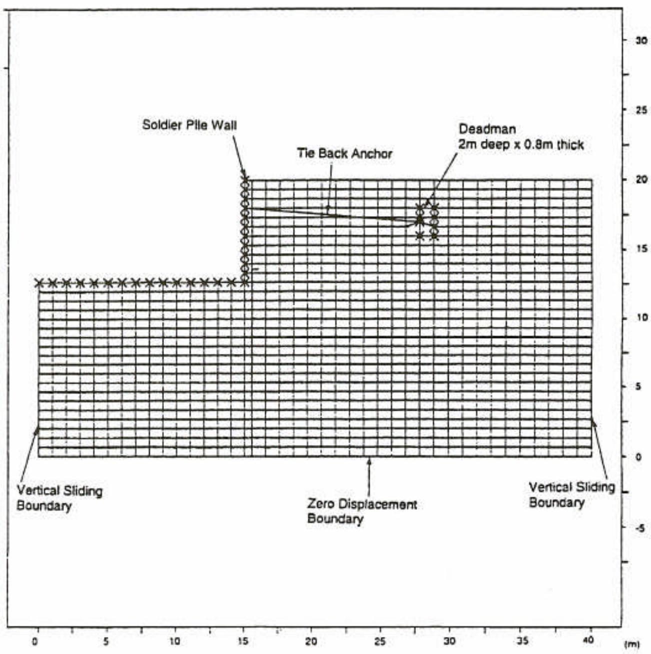

Figure 4 Two Dimensional Finite Difference Model

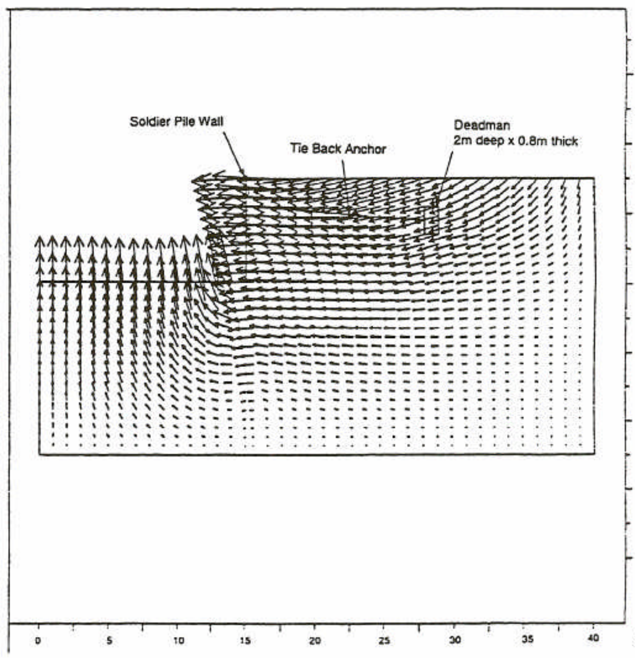

Figure 5 Ground Movement Pattern After Excavatic