Design of a Pile Reinforced Embankment, PJK Expressways Project, Tauranga

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Abstract: The Route J alignment of the $90M PJK Expressways Project provides a new arterial road for State Highway 2 into Tauranga City from the north. Built over the last three construction seasons, the PJK Project has required significant geotechnical inputs, with observational techniques used to manage risk and allow optimisation of the alternative design-and-construct areas of the project.

Geotechnical aspects of the alternative design for one of the main fill areas, the 14 m high sidling fill at J2 Fill, are discussed. The complex foundation included highly weathered volcanic soils, landslide debris and recent sediments, with artesian groundwater pressure within the toe zone.

The alternative design for the J2 Fill successfully used a combination of revised road geometrics, shear pile reinforcement, lightweight filling, and an extensive underdrainage system to achieve the required design criteria.

INTRODUCTION

The PJK Expressways Project is jointly funded by Transit NZ, Tauranga District Council and Western Bays District Council. As shown on Figure 1, the project links expressways to the Tauranga City peninsula (Route P), to the north (Route J) and the west (Route K).

The J2 Fill section of Route J comprises a sidling fill embankment up to 14 m high constructed on the side of steeply sloping ground just to the east of Cambridge Road, extending partially into the Valley of Te Auetu (Figures 1 and 2). The fill provides for the five lane expressway as well as the on- and off- ramps for Cambridge Road. The conforming design for this area involved approximately 13,000 m^3 of polystyrene fill and geogrid reinforcement.

The J2 Fill area was one of several identified for value engineering initiatives by the Contractor, the Fulton Hogan Ltd – Smithbridge (NZ) Ltd Joint Venture and their design consultant, Tonkin & Taylor Ltd.

Upon award of the contract in late 1999, further geotechnical investigation was undertaken at the J2 Fill to enable development of an alternative design. The extra geotechnical information and other associated factors highlighted slope stability issues which affected the design of the route.

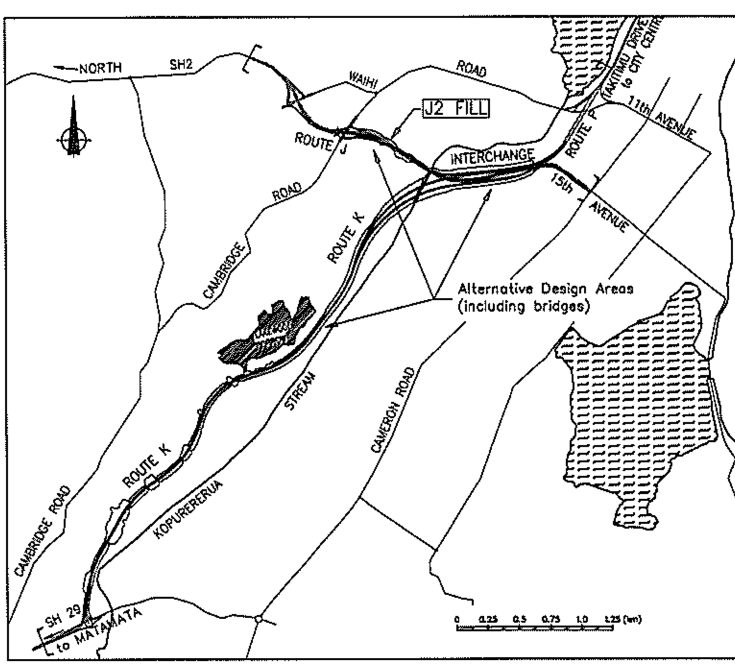

Figure 1 : PJK Project Location Plan

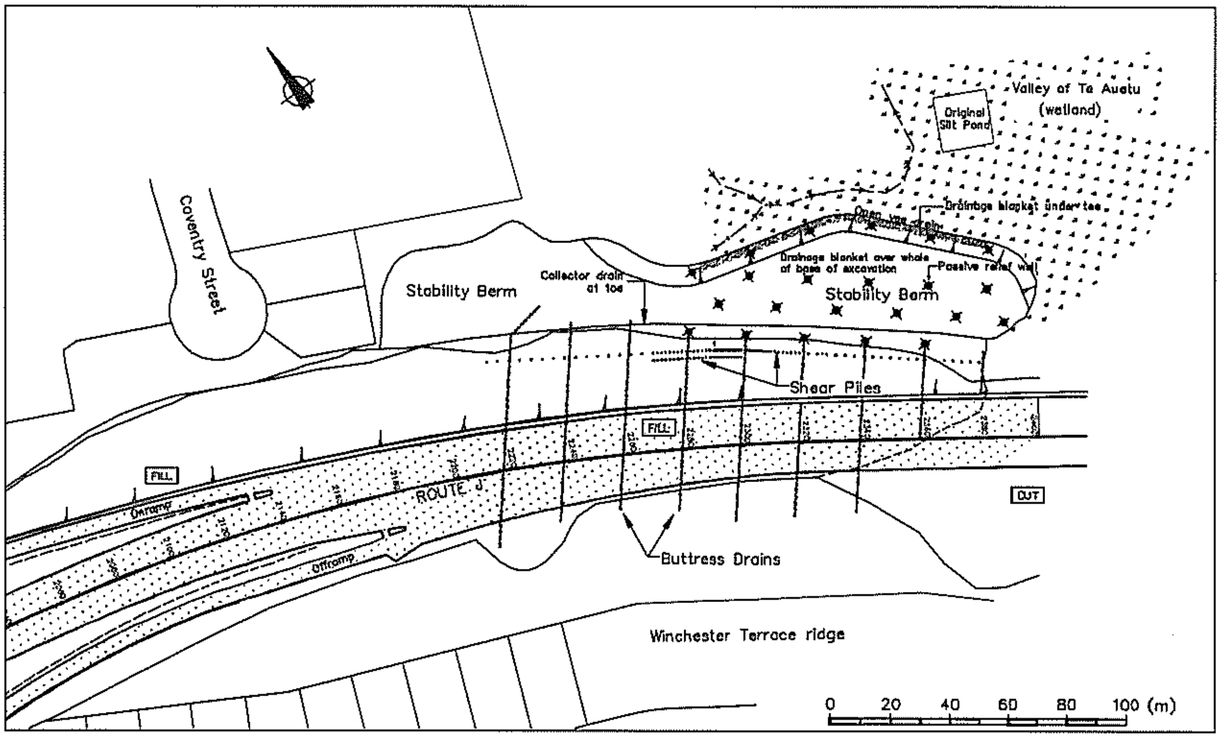

Figure 2 : J2 Fill Area

An alternative concept design was developed for the critical sections of the embankment to be constructed using lightweight pumice fill, a stability berm, foundation reinforcement piles, and revised road geometrics.

The alternative design also included the lowering of Cambridge Road, which allowed the overall height of the embankment to be reduced by up to 1.5 m and flatter batter slopes at the western end of the fill, thereby improving slope stability.

Design criteria had been set as part of the conforming design, and there was little or no change made to these during the alternative design process. Protocols were in place to limit the amount of disturbance of the natural ground and archaeological sites within the area, including an urupa (burial ground) on a small ridge directly in front of the embankment.

The J2 Fill was completed in early 2002 and the Route J Expressway was commissioned by April 2002. The total quantity of filling was approximately 130,000 m^3, comprising weathered brown volcanic ash, imported rockfill and pumice.

STRATIGRAPHY

The J2 Fill area comprises a complex series of primary volcanic deposits, lacustrine sediments and landslide materials, with several buried topographic surfaces. A brief summary of the main subsurface units is as follows, while a summary cross section is given in Figure 2.

Pumice Breccia
The local "basement" unit for the local area comprises a moderately to highly weathered dark yellow to light yellow pumice breccia (poorly welded ignimbrite). Borehole core showed it to be typically a dense fine to coarse sand with fine to medium sized pumice gravel, while the CPT cone resistance regularly exceeded 30 MPa.

Ash-Lapilli Deposits

Overlying the pumice breccia is a 3.5 - 7.5 m thick layer of moderately weathered light greyish brown loose to very loose ash-lapilli. This was generally recovered as very silty fine to very coarse sand. This is in turn overlain by a thick (up to 16m) unit of ash –lapilli. In the boreholes, this unit was recovered as very thin to thin (1 to 50mm) layers of fine to very coarse sand with minor fine gravel.

In the upper northern side of the valley the ash-lapilli materials are finer grained and were recovered as predominantly greyish white, clean, very fine to medium sand, directly overlying the pumice breccia. The materials are medium dense to dense, with CPT cone resistance in excess of 10 MPa.

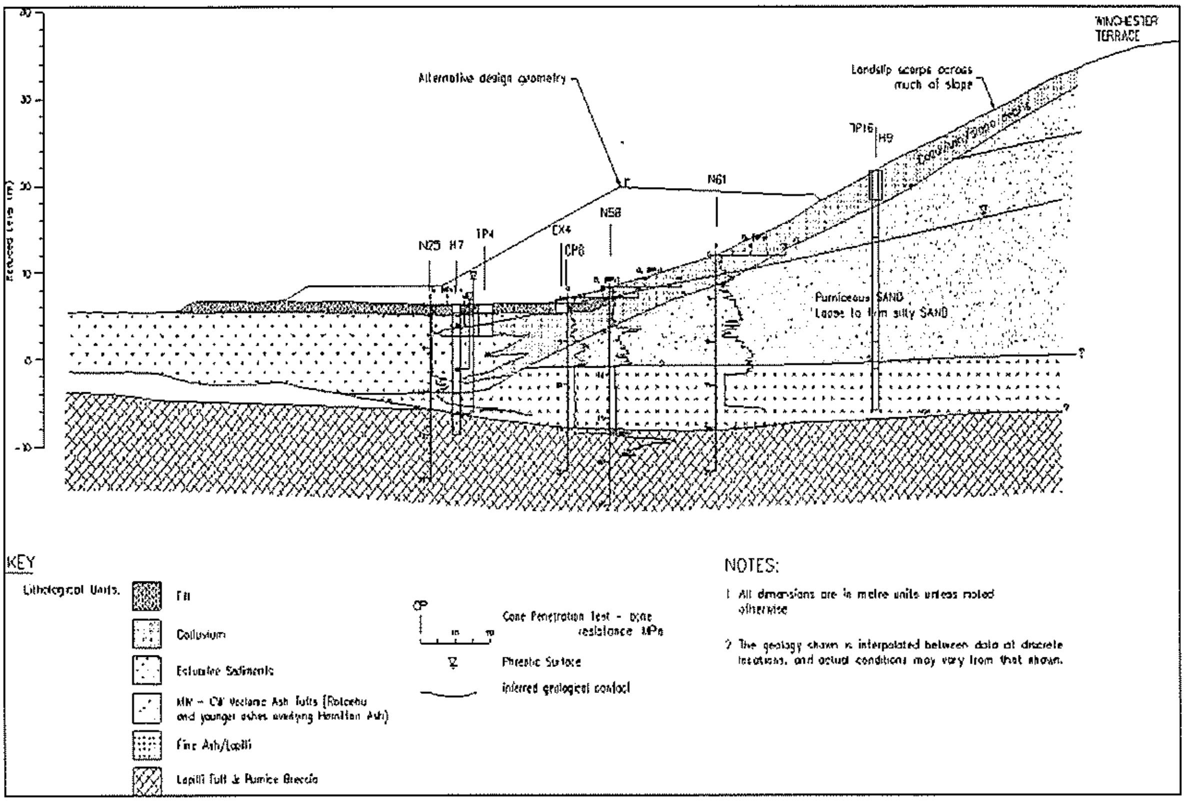

Figure 2(a) : Geological Cross Section, approx. Ch 2300 m

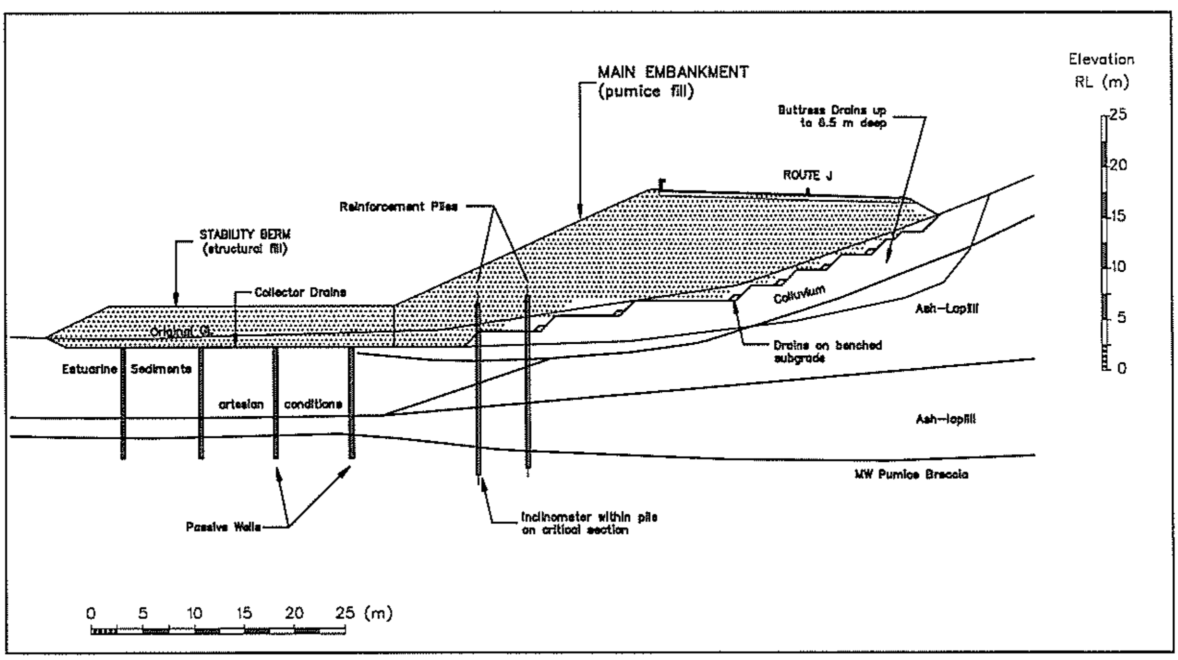

Figure 2(b) : Embankment Design Section, approx. Sta 2300 m

The upper layers comprise highly weathered fine grained volcanic ash layers, some of which may correlate to the Pahoia Tephra. These soils are generally stiff to very stiff, light brownish white clayey silt to clay. Measured water contents were typically in the range 45 – 100%, with a maximum of 143%. The material is typically very to extremely sensitive, deteriorating rapidly upon disturbance by excavation and/or trafficking to an extremely soft (<10 kPa) mud.

Estuarine Sediments
The estuarine sediments typically comprise very soft saturated grey clayey silt with some sandy silt beds containing shell fragments (up to 40mm size). Laboratory testing showed it to be normally consolidated, consistent with its young age (<10,000 years old).

Landslide Debris (Colluvium)
The lower slopes of the J2 Fill area contain a complex series of landslide debris lobes which interfinger with the estuarine sediments. The debris is typically a variable mixture of grey to light grey, very loose to soft, saturated silty sand and sandy silts. These materials also contain organic pockets, and wood (logs up to 0.5m thick were recovered in boreholes).

The colluvium is interpreted to be lobes of slope failure debris extending into the swamp/estuary, which were partially reworked with the estuarine sediments.

The main slope face is underlain by approximately 3 to 4m of stiff, cohesive, landslip debris. These materials typically comprise a variable mixture of stiff yellow brown silty clays and clayey silts with some local sandy zones. The colluvium appears to have been generally deposited in moderately thin to moderately thick (200 to 600mm) layers. It includes landslip debris from two large, mobile, landslips which occurred within the upper parts of the J2 Fill area in 1979, following heavy rainfall.

Ash mantle
The "typical" surficial ash sequence of Hamilton Ash, 60,000 yr BP Rotoehu Ash and post-Rotoehu ashes which mantles much of the topography in Tauranga is generally only present within the elevated areas of the Route J alignment. In the main part of the J2 Fill area, the ash sequence has generally been removed by past landslippage.

GROUNDWATER

Initial groundwater monitoring indicated a general groundwater table rising from the floor of the valley at approximately 1V:20H within the upper part of the valley, increasing to 1V:11H within the slope directly adjacent to the main fill zone (Stn. 2250 to 2380). The presence of thin (<50mm) clay layers within the thinly bedded ash-lapilli unit and instances of small seepage areas on the main slope indicated the presence of perched groundwater flows.

Additional site investigations showed artesian groundwater pressures within the foundation of the main embankment (that is, estuarine deposits, colluvium, and basal ignimbrite). For example, borehole H7 at the toe of the main slope showed a piezometric level +2m above ground level within the estuarine sediments, increasing to +4m above ground level within the underlying ignimbrite.

DESIGN APPROACH

The most significant risk to the embankment was shear failure through the estuarine sediments and colluvium in the valley floor, as shown on the typical cross section in Figure 2. Considerable effort was therefore spent in characterising strength, stiffness and extent of the soils. A summary of the undrained shear strength results is given in Figure 3, with information coming from:

- CPT tests carried out in the area of the proposed stability buttress. These indicated very low cone resistance (typically $q_c < 0.9$ MPa) and high strength friction ratio's, consistent with the very sensitive nature of the soils;

- Results from Geonor vane testing also indicated low strengths (Su <40 kPa) and high sensitivity (typically >20), although the peak strengths were somewhat higher than those based on the CPT results, averaging just over 30 kPa;

- During preparatory earthworks, there was a failure of the large silt pond within the Valley of Te Auetu. The mechanism of failure was clearly deep-seated rotational movement (Figure 4), a mechanism similar to one of the potential modes of failure being considered at the time for the main fill embankment. Back-analysis of the failure indicated the sediments had an undrained shear strength of 14 kPa. This was considered to be slightly conservative given some of the modelling assumptions.

Triaxial testing (CUP) carried out on samples of the estuarine deposits by Uniservices Ltd indicated a drained friction angle of 25° with no apparent cohesion.

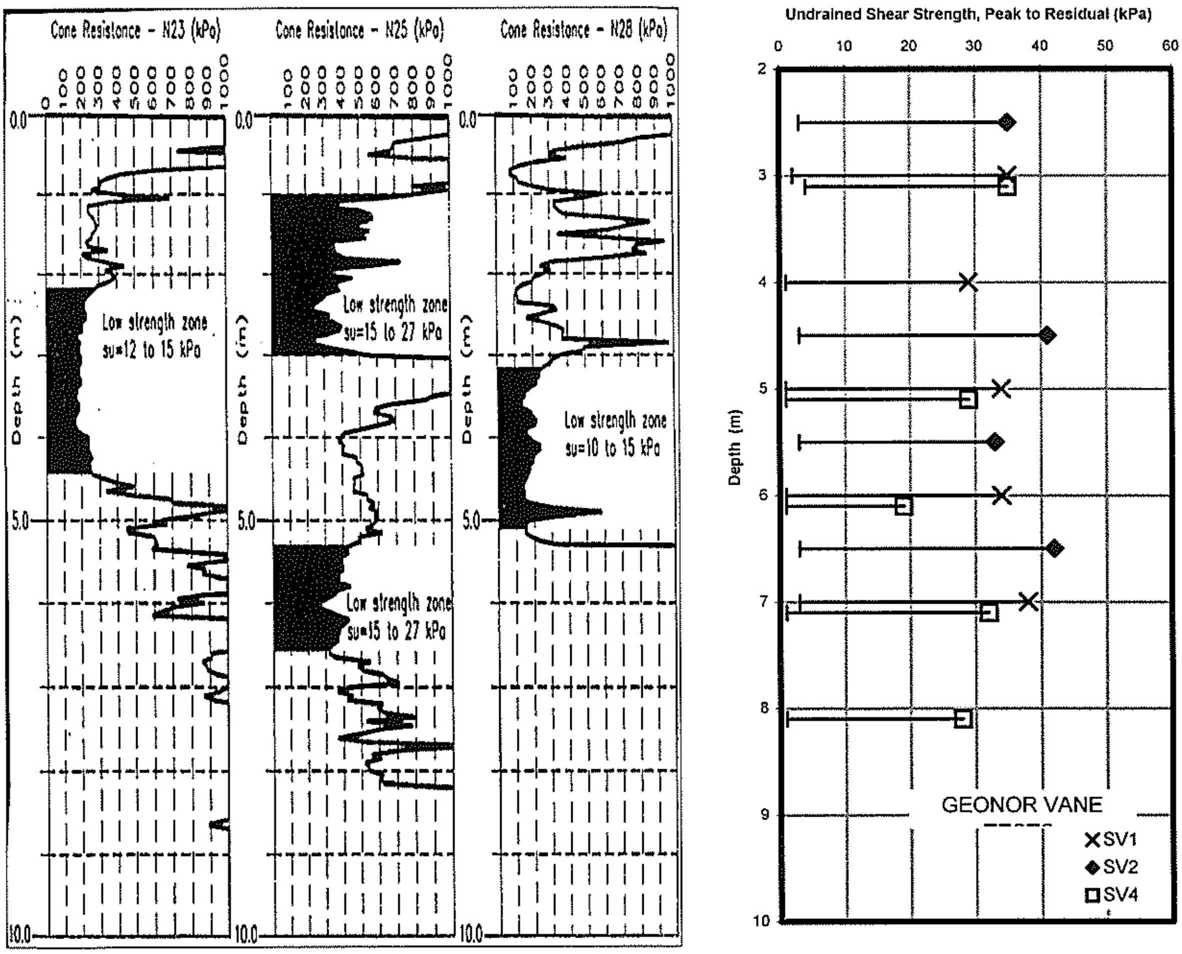

Figure 3 : Estimation of Undrained Shear Strength

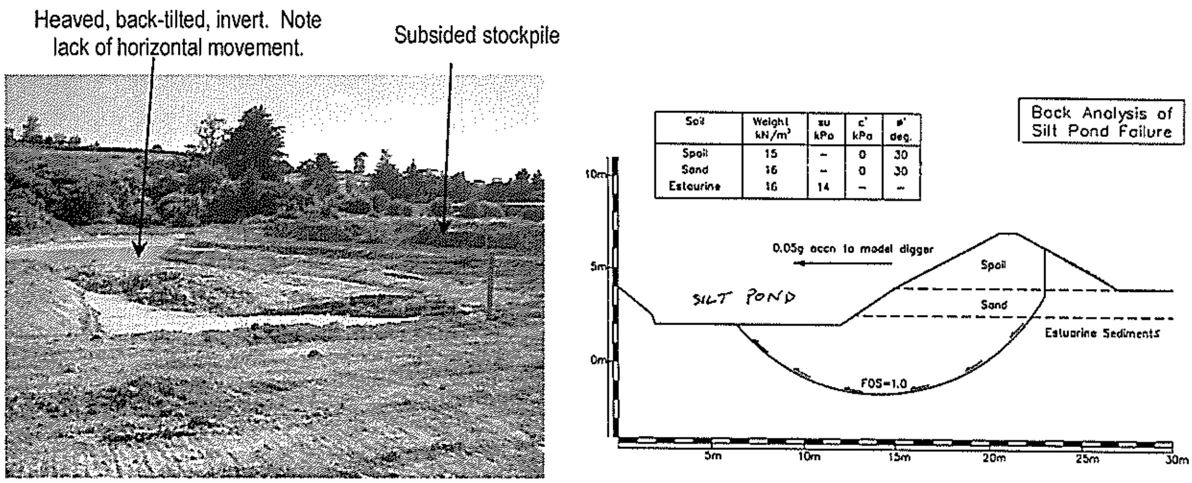

Figure 4 : Silt Pond Failure, Valley of Te Auetu

The parameters for other materials used for design of the embankment were determined from the recent site investigation results, previous design work by Opus International Consultants, historical data on similar materials, and full scale field trial embankments within the Interchange (another alternative design area of PJK). Extensive analyses were carried out to investigate the sensitivity to critical material parameters. Inferred values for key units are given in Table 1.

It was evident that the presence of very low strength estuarine deposits near the toe of the proposed embankment posed a significant risk for deep-seated shear failure. The design solution adopted was a combination of a large stability berm, shear key piles, and lightweight pumice filling.

Material	**Unit Weight (kN/m^3)**	**Su (kPa)**	**Cohesion (kPa)**	**Friction (°)**	**E (MPa)**	**ν**
Structural embankment fill	13 – 19	60	2	30	15	0.3
Stability berm	16	60	2	30	15	0.3
Estuarine deposits	17	14	0 - 2	25	1	0.3
Colluvium	16	50	2 - 5	30	5	0.3
Tephra	16	50 – 70	0 - 5	30	5	0.3

Table 1. Inferred Material Properties for Embankment Design

ANALYSIS

Analysis was largely undertaken as limit equilibrium stability analysis using SLOPE/W software. Some finite difference analysis was also undertaken using FLAC to consider deformation and strain softening characteristics. A Mohr-Coulomb (MC) model was adopted for the embankment, stability berm, colluvium and weathered ignimbrite, while a strain softening (SS) model was adopted for the estuarine deposits and tephra, with properties estimated on the basis of sensitivity and strain levels.

Seismic displacements were calculated using Makdisi & Seed (1979), with predicted displacements varying from 40 mm for the 150 year AEP event, up to 400 mm for the 1,000 year AEP event (0.26g and 0.48g respectively).

Settlement of the main embankment by up to 400mm was predicted using CPT correlation techniques similar to those used elsewhere on PJK and at other projects (Pender et al 2002, Pender et al 1999) and by FLAC analysis. The amount of settlement occurring during construction was assessed semi-qualitatively and then verified by settlement monitoring (actual range was 25% to >60%, depending on the construction programme). Allowance had been made in the design for the possibility of the stability berm settling over a longer time period.

SHEAR PILE REINFORCEMENT

The structural action of the piles is a complex relationship between pile stiffness and soil stiffness/strength. The problem was solved by considering it as two separate piles, one above and one below the failure surface, then solving each pile using finite difference methods (computer program EP6A), with a horizontal shear load applied equal to the required reinforcement loading. Rotation and moment compatibility at the head of the 'two' piles was achieved by varying the pile head restraint, thus providing a full deflection, bending moment and shear force profile for the pile. The variable shear force requirement could then be achieved by varying the pile spacing. P-y curves were calculated for the soils using the method developed by Reese and Matlock (1956).

The pile reinforcement capacity required to achieve a FOS of at least 1.5 for embankment stability was calculated. An economic analysis of steel and concrete piles at variable spacing indicated that 310-UC piles in one or two rows and variable spacing. were the best approach.

SUBSOIL DRAINAGE MEASURES

Monitoring of piezometric levels in the initial stages of the PJK Project showed artesian pore pressures within the estuarine sediments. This was interpreted to be the result of upward groundwater flow from the Cambridge Road ridge towards the valley area.

Calculations showed that adequate stability could not be achieved with the artesian pressures. A series of pressure relief wells were therefore installed within the toe zone (Figure 2) to achieve hydrostatic piezometric levels consistent with a groundwater table at swamp level. The wells consisted of bored shafts through the sediments, backfilled with a two-stage, filter compatible, drainage system. Filter compatibility with the surrounding ground was checked. Lateral collector drains (with some redundancy) were included.

Buttress drains up to 6.5m deep were constructed at 20m centres throughout the length of the fill. These were cut as deep as possible to maximise the interception of perched seepage horizons. Instability tended to occur when the trench encountered either saturated landslip debris or the top of the bedded tephra unit, which had sub-horizontal layers of particularly low strength silt. Backfill drainage material was selected to provide filter compatibility with the in-situ soil, removing the need for filter fabric, leading to simplified construction and cost savings.

Collector drains were included within the toe buttress to provide some redundancy of drainage outlet, and to avoid excavation through the urupa (burial ground) in the base of the valley. The collector drains were constructed in a similar manner to the buttress drains, with filter-compatible drainage material and a 100 mm dia slotted MDPE collector pipe.

Bench drains were included at the back of each bench cut into the in-situ ground. The drains consisted of a 0.5x0.5m gravel drain wrapped in filter fabric with a slotted MDPE pipe. The bench collector drains were 0.5m wide and of variable depth, and typically at 20m centres.

DISCUSSION

Other design changes to those described above included:

- Lowering of the vertical alignment of the main alignment (by optimisation of the geometrics);
- stormwater flowpaths were relocated away from the toe of the structural fill to allow the stability berm to be widened;
- the footprint of the stability berm at the toe of the critical embankment section was undercut by 0.75 m and backfilled with engineered fill.

At least three significant design iterations were undertaken during the construction phase of the embankment, to allow for adverse weather at critical times, as well as for refinement of the design of reinforcement piles and the pumice filling.

Construction monitoring included:

- Inclinometers, including one installed within a shear pile at the critical embankment section;
- Standpipe peizometers;
- Pneumatic peizometers;
- Settlement cells;
- Surface survey, including precise measurement of lateral displacements.

The measured deformations and piezometric levels remained within design tolerances, including lateral deformation of the shear key piles (Figure 5).

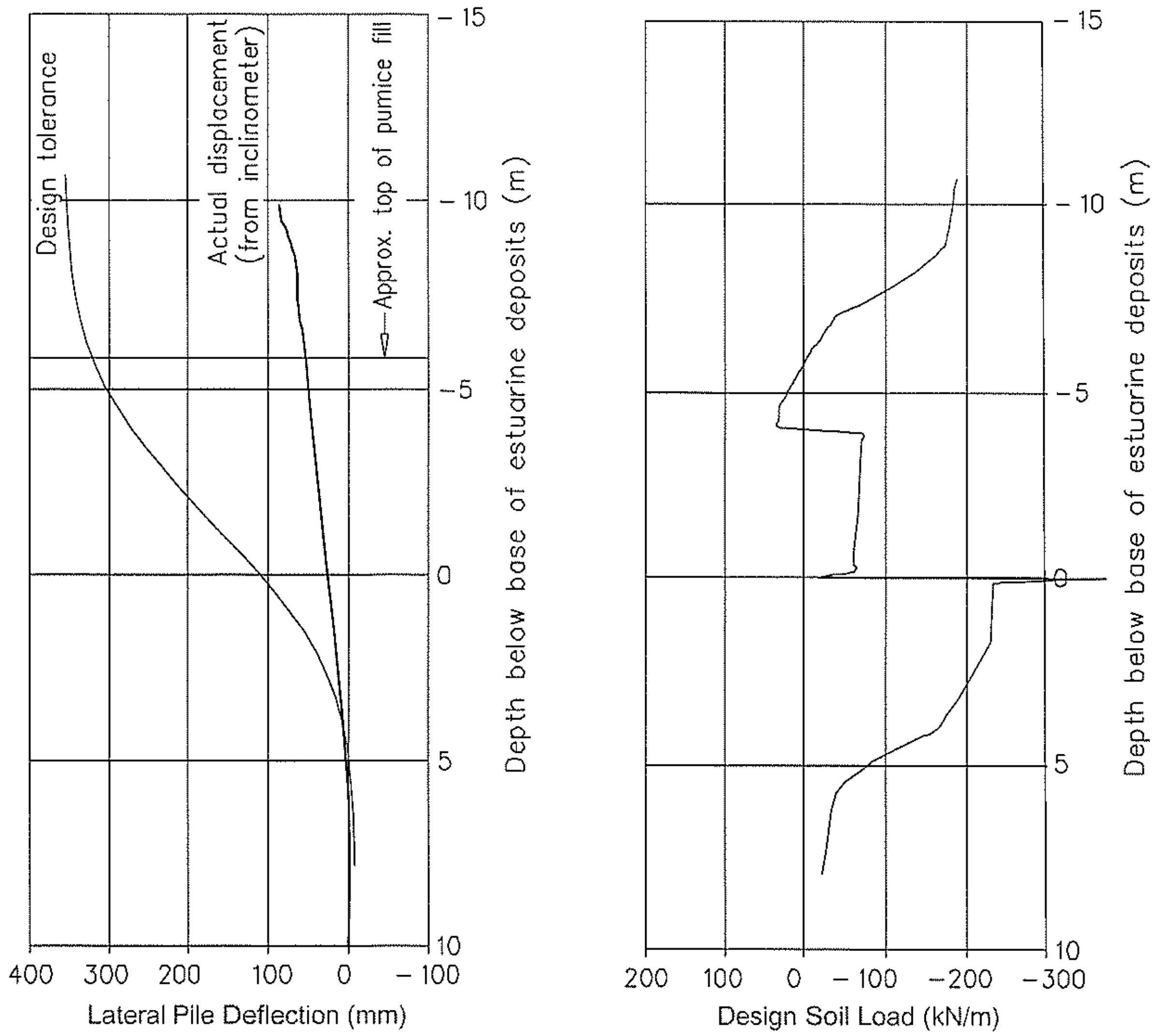

Figure 5. Pile Deflection Monitoring

CONCLUSIONS

The J2 Fill at the PJK Expressways Project in Tauranga involved construction of a 14 m high sidling embankment on a complex foundation of intercalated volcanic and sedimentary deposits.

Developed within the tight construction programme timeframe, the alternative design solution included reinforcement piles, lightweight filling, substantial underdrainage, a stability berm and revised geometrics. The unusual nature of the volcanic ash soils required special design provisions, with allowance made for low strengths, very high sensitivity, and the potential for internal erosion. Observational techniques were successfully used during construction to confirm design assumptions and to optimise the overall embankment design.

ACKNOWLEDGEMENTS

The permission of the project owners, Transit New Zealand and Tauranga District Council, and the Contractor, the Fulton Hogan Ltd – Smithbridge (NZ) Ltd Joint Venture, to use the monitoring data is gratefully acknowledged. The inputs of staff from the JV, subcontractors and T&T were vital to a successful outcome. Robin Dawson of T&T undertook the finite difference modelling. The inputs of the peer reviewer for the alternative design areas of the PJK Project, Professor M Pender, were much appreciated.

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