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Geotechnical Investigations and Construction in Active Geothermal Areas: Rotorua, New Zealand.

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SUMMARY

The Whakarewarewa thermal area in Rotorua, New Zealand, is an active geothermal area and major tourist attraction. Current upgrade of the facility includes a 1.8 km paved and suspended timber deck track, three shallow cut and cover tunnels, two new bridges and an upgrade of an existing glulam bridge, all constructed on difficult and dynamic ground conditions well beyond the range of conventional geotechnical engineering design solutions. Construction of three new architecturally designed buildings is also programmed to begin in 1998. Comprehensive field and laboratory investigations over a four year span were undertaken, complimented by ongoing design during the construction phase.

1. INTRODUCTION AND DEVELOPMENT PROPOSALS

Civil construction in active geothermal areas is recognised as a challenging and demanding undertaking requiring detailed, creative and sometimes unique design solutions to overcome difficult and dynamic ground conditions. The Whakarewarewa thermal area in Rotorua, New Zealand, is an active geothermal area and major tourist attraction. Three new architecturally designed buildings are in the process of being constructed on a terrace area south-west of the current New Zealand Maori Arts and Crafts Institute complex, together with a 1.8 km paved and suspended timber deck track carrying tourists in electric powered vehicles looping around the adjacent thermal reserve. The track development also involved three shallow 'cut and cover' tunnels, two new bridges and an upgrade of the existing Pohutu Geyser bridge as well as specialist construction measures to deal with geothermal ground conditions.

Detailed geotechnical and geological investigations were undertaken prior and during track design to establish suitable design parameters, construction methodologies and the best track location.

An "observational method" approach was adopted for the construction phase of the development to account for variations in ground conditions and unforeseen construction difficulties.

2 GEOLOGY

The Whakarewarewa thermal area (Figure 1) is located inside the southern wall of a large circular rhyolitic caldera, about 15 km in diameter, with Lake Rotorua offset towards the northern end. The caldera formed following collapse during and after the eruption of the Mamaku Ignimbrite some 220 ka ago.

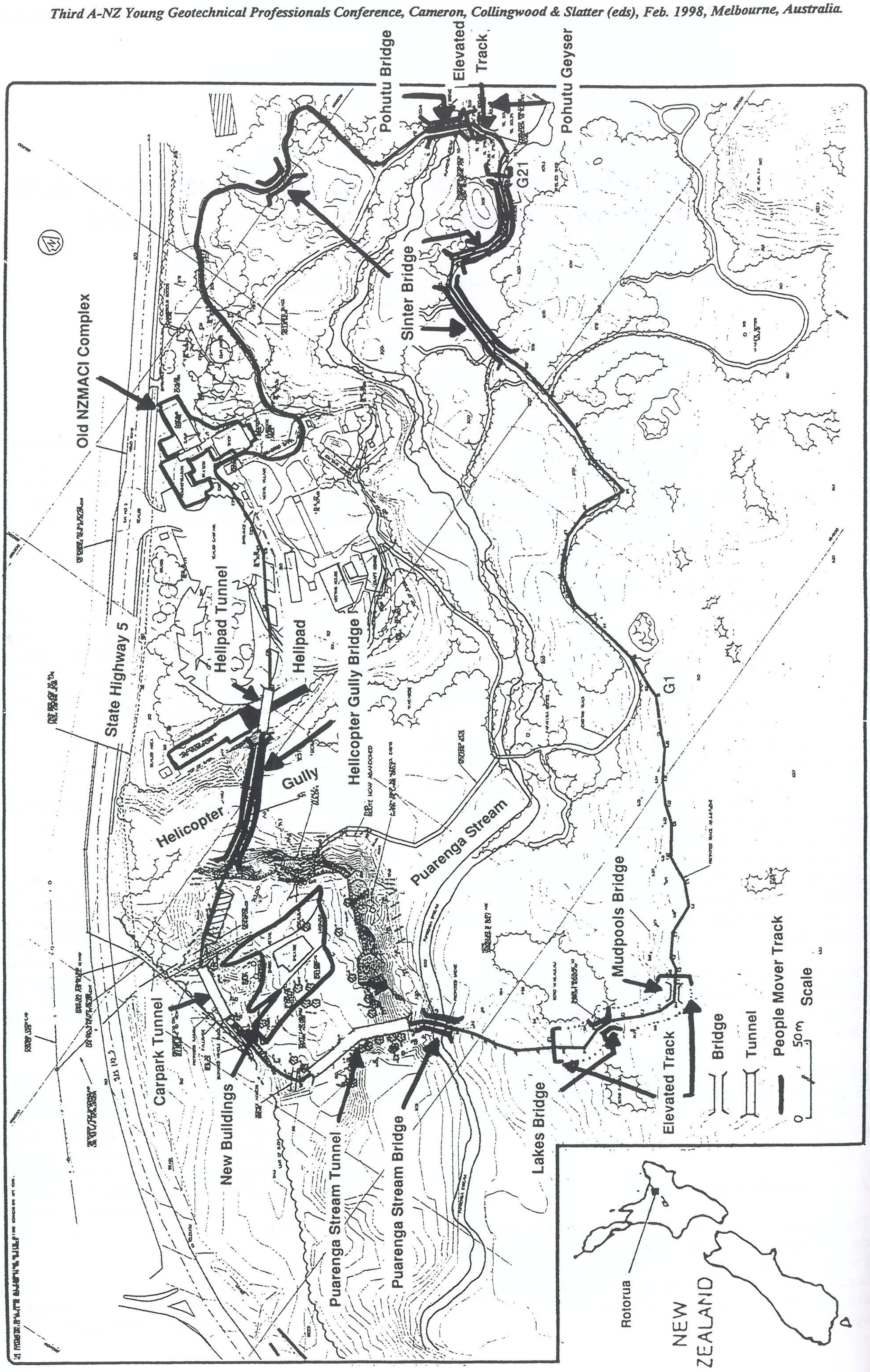
Lake Rotorua has been dammed on may occasions since the Mamaku ignimbrite eruption by deposits from the Okataina Volcanic Centre, forming larger and deeper lakes than that of the present day Lake Rotorua. During 26.5 ka to 20 ka ago, water levels fell progressively to that of present day, exposing much of the Whakarewarewa area above water (1).

In terms of immediate site stratigraphy there are a least two surficial volcanic airfall ashes comprising orange/ brown/yellow, loose, uncompacted silty sands/sandy silts with age ranges between 5 and 10 ka. Beneath these is a 2 to 3 metre thick coarse gravel pumice ash with an age of around 13.5 ka.

Underlying both the above is a sequence of stiff, moderately dense, alluvial (lake terrace) very fine grained silica silts and pumice lapilli, forming much of the higher ground over the development. This unit is highly impermeable, with the upper part of the unit exhibiting thixotrophic properties when disturbed. It acts as a cap to the underlying geothermal activity contained within the basal Huka Group sediments, essentially hydrothermally altered and silicified alluvial sands, silts and coarse pumice/rhyolite gravels.

The Whakarewarewa thermal area has a high vertical conductive heatflow with thermal activity concentrated along the many faults which parallel the Puarenga Stream and criss-cross the area (1). There are two main classes of hot springs: alkali-chloride springs, including geysers, which usually have a high fluid discharge at or near boiling point, and semi-stagnant acid sulphate mudpools, with little or no discharge (2). The physical and chemical differences between them are mainly due to their positions relative to the water table.

Chloride springs emerge close to the water table and at Whakarewarewa they are concentrated where the faults intersect the water table close to the Puarenga Stream. The


Pohutu Bridge
Elevated Track
Pohutu Geyser
G21
Sinter Bridge
Old NZMACI Complex
State Highway 5
Helipad Tunnel
Helipad
Helicopter Gully Bridge
Helicopter Gully
Puarenga Stream
G1
Mudpools Bridge
Lakes Bridge
Elevated Track
Bridge
Tunnel
People Mover Track
Scale
0
50m
Puarenga Stream Bridge
Puarenga Stream Tunnel
New Buildings
Carpark Tunnel
Rotorua
NEW ZEALAND

water and gases are flushed from the system before they are oxidised and precipitate out silica to form sinter deposits. However, on the higher ground where the water table is deeper, little or no water, or only steam and gases can reach the surface. Ground conditions are invariably very acidic (sulphurous acids) due to oxidation of sulphide gas from out of the geothermal system; steam heating makes this acid attack even more aggressive. This attack causes ongoing decomposition of rock and soil minerals, resulting in multi-coloured clays stained with iron minerals, gradual void or cavity formation, subsidence and boiling mudpools.

From an engineering viewpoint, wide spread hydrothermal alteration of constituent soil and rock masses has resulted in areas of fragile ground conditions with poor bearing capacity, having potential for both extensive total and differential settlement and potentially rapid ground collapse.

3 FIELD & LABORATORY INVESTIGATION

Fieldwork for the Whakarewarewa project began in late 1993 as part of a preliminary geotechnical feasibility study for the overall development. More intensive investigations were carried out in November 1996 with additional investigations in early and mid 1997. These were designed to investigate geotechnical conditions and the engineering implications of constructing structures on an active, dynamic geothermal field. Distinct operations were carried out to investigate ground conditions on the building platform, bridge foundations, tunnel cuts, suspended track and on-grade pavement areas (Figure 1). The 1993 and 1996/1997 field work is summarised in Table 1, with laboratory testing summarised in Table 2.

Table 1: Summary of Field Investigation

Date	Investigation Method	Quantity	Maximum Depth (m)
1993	100mm HQ Wireline Machine Borehole	3	14.5m
	Cone Penetrometer Test	10	17.5m
	50mm Ø Hand Auger Borehole with in-situ soil strength test	64	4.2m
	Scala Penetrometer Test	59	5.0m
1996	100mm HQ Wireline Machine Borehole	6	15.2m
	Plate Load Test	11	0.4m
	50mm Ø Hand Auger Borehole with in-situ soil strength test	40	7.0m
	Scala Penetrometer Test	67	5.0m
	California Bearing Ratio (in-situ CBR's)	6	surface
	Ground Penetration Radar Survey	approx. 300m	2.2m
1997	100mm HQ Wireline Machine Borehole	2	13.5m
	Scala Penetrometer Test	Several 100	1.0m
	50mm Ø Hand Auger Borehole with in-situ soil strength test	8	2.4 m

NB: Further hand auger boreholes and scala penetrometer tests were carried out as ground conditions were exposed during the construction phase.

All results were Telarc endorsed where appropriate. Plate load tests were carried out in accordance with ASTM D1194 and in-house methods where appropriate.

Table 2: Summary of Laboratory Testing

Description	Test Number	No.
Atterberg Limits	NZS 4402, 1986: various	1
NZ Standard Compaction Test	NZS 4402, 1986: 4.1.1	1
Effective Stress - consolidated undrained triaxial compression test with pore pressure measurement	FEL in house test 19(b)	2
Solid Density	NZS 4402 1986: 2.7.2	1
Undrained Triaxial Compression Test	NZS 4402:1986 Test 6.2.1	3
Unconfined Compression Test	NZS 3112:1986 Test 9	7
Particle Size Analysis	NZS 4402:1986 Test 2.8.4	1
Nuclear Density	NZS 4407:1991 Test 4.2	20-30

3.1 Machine boreholes and cone penetrometer testing

Machine boreholes and CPT probes were undertaken to establish vertical soil stratigraphy, recover undisturbed samples for laboratory testing, obtain relative soil densities via SPT's, and log geothermal conditions including soil temperatures and hot/steaming ground at depth.

3.2 Hand auger boreholes and scala penetrometers

Extensive use of hand auger boreholes and scalas was undertaken to investigate shallow ground conditions, especially in difficult access areas or on the track alignment itself.

3.3 Plate Load & In-situ CBR Tests

Eleven plate load tests utilising a 500 mm diameter steel plate were undertaken, with seven tests on areas of predominantly geothermally altered soils around the track alignment and three further tests on the airfall ashes over the building platform. Two large tanks on a loading frame were used as reaction mass for the plate load tests, the tanks and frame being flown in by helicopter to each test site. Water was pumped to each site and placed in the tanks to increase reaction mass.

Loads at 20 mm deflection for the track alignment ranged from 55 to 159 kPa (Figure 2), while loads at 15 mm deflection over the building platform ranged from 65 to 110 kPa (Figure 3).

Six in-situ California Bearing Ratio (CBR) tests were also carried out adjacent to the plate load tests. CBR's ranged from 1.0 to 1.5.

3.4 Ground Radar Survey

The ground radar survey was carried out along part of the existing walkway track from G1-G21 (Figure 1). Interpretation of the survey traces suggested there were at least six to seven areas along the existing track of thin sinter crust overlying boiling mud or alkaline springs which would not be able to support the proposed paved track. These were confirmed by additional hand auger boreholes and scala penetrometers.

3.5 Stratigraphy, Soils Distribution & Engineering Properties

Field investigations revealed a 1 to 2 metre thick layer of generally soft to loose, non-plastic, orange/brown/yellow airfall volcanic ashes, typically silty sands/sandy silts ("type 1" soils) overlying pumiceous and rhyolitic sands and gravels to around 4 metres depth ("type 2" soils). Thixotrophic/sensitive hydrothermally altered lacustrine silts and clayey silts ("type 3" soils) underlie the surficial airfall deposits, which are in turn underlain by weak to moderately silica cemented sands and gravel breccia's ("type 4").

Ground conditions beneath the track alignment were somewhat more variable than those outlined for the building platform. Soft, non-plastic, acid leached and geothermally altered silts ("type 5" soils) and loose to weakly cemented silica sinters ("type 6" soils) were also encountered in the intervening lower lying areas. Descriptions, distribution and the engineering properties of each soil type is summarised in Table 3.

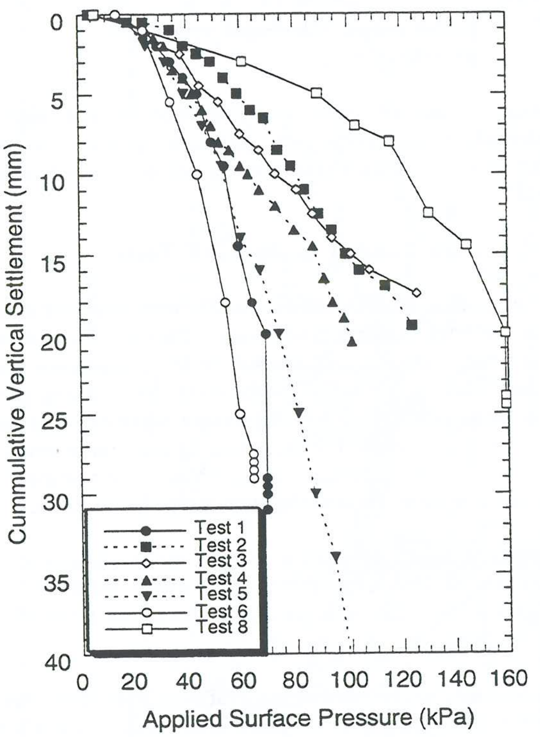


Figure 2: Track alignment plate load tests

3.6 Laboratory Results

Atterberg limit tests and particle size gradings confirmed the sandy/silty nature of the overlying surficial ashes (type 1). Compaction testing of type 1 soils returned a maximum dry density of 1.02 t/m^3 at an optimum water content of 47%. Further in-situ density tests yielded dry densities of 0.50 and 0.52 t/m^3 at water contents of between 103% and 112% for type 1 soils and dry densities of 0.80 to 0.90 t/m^3 at water contents of between 59% and 67% for type 3 soils.

The variability in strength of the type 3 soils with depth was illustrated from in-situ Standard Penetration Tests (SPT's), and undrained triaxial compression tests carried out on recovered block samples from the lower part of the type 3 unit. SPT's in type 3 soils ranged from N=0 (at the top of the unit) to N=21, increasing with depth. Confined compressive strengths in the base of the unit ranged from 0.4 to 2.2 MPa, at axial strains of 0.49 to 0.64%. Within the type 4 materials unconfined compressive strengths measured for pile design ranged from 2.2 to 30 MPa.

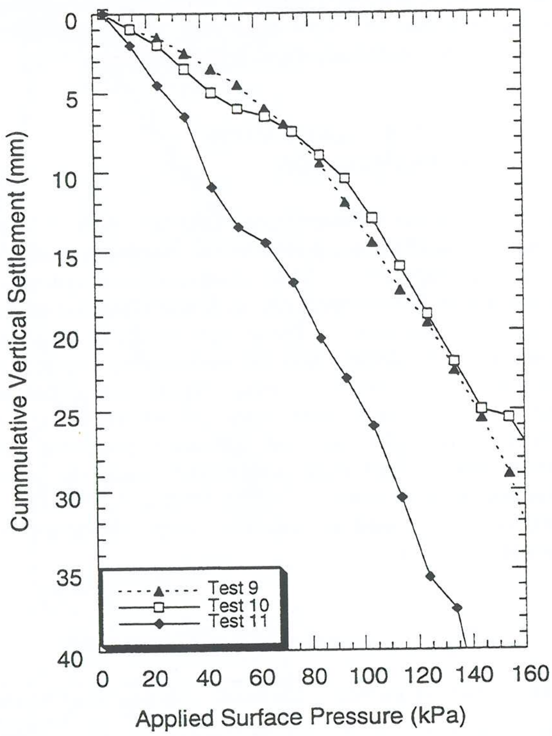


Figure 3: Building platform plate load tests

4 BUILDING PLATFORM CONSTRUCTION

4.1 Proof Rolling

Construction of the reinforced concrete floor slab for the new building began in June 1997, essentially being completed by late September. Actual building construction is programmed to commence in early 1998. The buildings are to be constructed of lightweight glulam timber portal frames and beams, with laminated veneer lumber (LVL) beams, purlins and girts to accommodate the curved nature of the building. The floor slab and foundations were designed for dead plus live loads of 40 kPa allowing for ground improvement via proof rolling to limit differential settlement.

Static proof rolling trials of the building platform began in early July 1997, following partial clearing of bush and scrub.

Table 3: Summary of ground conditions encountered at Whakarewarewa

Unit No.	Geological Description	"Engineering" Description	Distribution	Engineering Properties	Comment
Type 1	Whakatane and Mamuku ashes	Airfall volcanic ashes. Soft, loose, friable, non plastic, orange/brown/yellow fine sandy silts and silty fine sands.	Surface to between 1.6 & 2.2m depth. Average thickness approx 1.8m. Mantles building site as well as other remnant terrace areas across Whakarewarewa thermal area.	Loose, compacts under vehicle tyres. Shear vanes (SV's) range from 10-60 kPa, average 30 kPa. Sensitivities to disturbance 1.8-7.0, typical values 3.5-4.5. Equivalent CBRs almost all <1 for top 0.8-1.2 m. SPT results all N=0.	Seriously unfavourable load bearing characteristics. Hand auger boreholes 3 & 5 have 4+ m of this unit at northern end of building platform - material was hot and moderately altered. Airfall deposit mantles pre-existing topography, expect variable thicknesses across site.
Type 2	Rotoma and Rotorua ashes. Note Rotoma ash locally absent in some areas	Airfall volcanic ashes. Loose, non-plastic, orange/cream/brown, fine to coarse pumice sands & fine-medium pumice gravels with inclusions of fine gravel size rhyolitic lithic fragments. All particles angular to subangular.	Typically between 1.8 & 3.6m depth but ranging from 1.6 & 2.2m to 3.1 & 4.1m depth. Average thickness 1.8m.	Loose pumice sands and gravels. Highly permeable. Equivalent CBRs range <1 to 5, typical values CBR=3. Note values quite variable across building platform. SPT results all N=0 or at best N=1.	Highly permeable unit inferred to rapidly drain rainfall infiltration and hence enhance bank stability. Loose gravelly nature likely to substantially compact where exposed in building platform benches. Airfall deposit - mantles pre-existing topography, expect variable thicknesses across site.
Type 3	Oruanui breccia. Note erosional break between overlying ashes (Type 1 and 2) and this unit/soil type	Lacustrine terrace deposit, range of soil types: Soft to firm, non to moderately plastic, orange/red/ pink/ brown/cream silts and clayey silts. Beds and bands of 5-15mmØ lapilli pumice gravels. Bands of abundant amorphous silica, sensitive, thixotrophic - collapses on disturbance. Areas of hydrothermally altered montmorillite clays, swelling potential when unloaded. Increasing strength with depth, becoming stiff to very stiff and weakly cemented below 10-12 m depth	Top of unit typically 3.6m depth but ranging from 3.1-4.1 m. Base of unit variable range approx 11.1-13.6 m in machine boreholes. Mean thickness 8.5 m ranging from 7.2-10.4 m.	Firm to stiff, becoming weakly silica cemented at depth. SV's range from 10 to >140 kPa, mean approx 50 kPa. SV values higher than in two overlying units. Sensitivities to disturbance range 1.8 to >20 (extremely sensitive - quick). Equivalent CBRs range 3-10+, typical values CBR = 5. SPT results variable, range N=0-N=21, average N=6	Lacustrine terrace deposit , deposited approx 22,000 years BP when lake level was approx 40 metres higher. Some sedimentary structures seen in stream bank exposures. Sorting and grading of particles by size and density seen in borehole core. Highly impermeable unit effectively caps and seals off underlying geothermal aquifer and consequent activity.
Type 4	Huka Group sediments	Pre 65,000 year BP lacustrine sediments. Very weak to weak, weakly to moderately silica cemented fine to coarse grained silica sandstones, cemented gravels and weak, weakly cemented siltstones. Variable cementation.	Exposed along Puarenga Stream banks at bridge and in base of some machine boreholes. Inferred top of unit 11.1-13.6m depth but difficult to define precisely due to altered and weakly cemented nature of overlying Oruanui breccia unit	Very weak to weak, (non) to weakly to moderately cemented, soft rock. Bands of uncemented sands, interbeds of silicified sediments and sinter. SPTs range from N=3 to N>50. UCS range 2.2-29.9 MPa.	Good founding conditions for Puarenga Stream bridge but variable depth to top of competent founding surface. Undercutting and backfilling with site concrete or aggregate likely to be required.
Type 5	Acid geothermal muds and silts, hot barren ground, geothermally altered terrace deposits and airfall ashes	Soft to very soft, non plastic, cream/grey silts, often moist and hot. In places muds, actively bubbling. Thin, 0.2-1.0m thick crust of cool to warm, non plastic (white) silts, typically firm, in places friable, loose, prone to collapse where undermined by active acidic activity. Often mantled with 0.4-0.8m of moderately altered orange/brown/yellow ashes (type 1 soils)	Areas of site typically below RL 310m.	Very difficult founding conditions, poor ground bearing and high settlement characteristics coupled with elevated temperatures and on-going ground decay and settlement. SV's range 10-30 kPa, mean approx 15-20 kPa. Equivalent CBRs <1 (in places considerably <1!). Actual CBRs (done in conjunction with plate loads) 1-1.5. At best CBR = 1 available for design. Some ground modification will be required in some areas.	Ground conditions at absolute limit of conventional geomechanics applications.
Type 6	Boiling alkaline springs, active geyser areas, alkaline sinters	Loose, non-plastic, cream/ grey coarse silty sands and sandy fine gravels. Weak, weakly cemented cream/grey silica sinters interbedded with type 1, 2 and 3 soils above. Actively depositing amorphous silica sinter from boiling geothermal fluid.	Predominantly existing developed area of Whakarewarewa, southern abutment of Pohutu Bridge. Some older sinters adjacent to Puarenga Stream bridge area.	Where present without overlying ashes, silica sinter has relatively better engineering characteristics than much of higher ground. Shear vanes typically 5-80 kPa but obvious problems due to nature of substrata. Sensitivities to disturbance range from 2-8, typically 4. Equivalent CBRs range 1-5, typically CBR=2.	Ground conditions at absolute limit of conventional geomechanics applications. Potential for collapse of sinters, especially around existing developed area

Various types of equipment were trialed including excavators, rollers, graders, front end loaders and log loaders of up to 30 tonnes in weight. The most effective machine was found to be a medium sized log loader.

Approximately 50 to 100 mm induced compaction occurred in the type 1 and type 2 soils, with isolated areas of over 600 mm occurring where underlying pumice gravels were close to the surface and weakened by low grade geothermal activity. Due to the wet winter vehicle traction was a problem and 150 mm of GAP 65 rhyolite was laid down prior to further proof rolling. Areas of excess deflection (i.e. greater than 50 to 100 mm) were further undercut, and backfilled. Finally, a further 100 mm layer of rhyolite was added.

4.2 Building Foundations

Continuous external strip footings for the buildings were excavated to 0.8 metres depth by 1.8 metres wide and proof rolled. Further undercutting was undertaken where excess deflections occurred. Typical scala results at cut invert were 1 to 2 blows per 1500 mm penetration with several tests falling under self weight to 2 metres depth. A layer of Polyweave HR geotextile was placed at the base of the footings followed by 600 mm of compacted GAP 65 rhyolite. Concrete footings 800 mm wide by 600 mm deep were then constructed (Figure 4).

Further excavation occurred along the front (north-west) of the building platform adjacent to an area of hot steaming ground, and was built back up with Polyweave HR and compacted pumice. Pumice is more resistant to geothermal attack and was used in areas of elevated activity. The polyweave HR was used in the area as a construction expedient, and was not expected to survive beyond 6 to 8 weeks.

The south-eastern footprint of the building was also piled on account of being inside an 18 metre building restriction zone recommended from stability analyses on the adjacent steep bank above the Puarenga Stream. Eight, 600 mm diameter bored and concreted piles were constructed to depths of between 5.0 and 5.7 metres, socketing into the stiffer, weakly cemented lower type 3 unit.

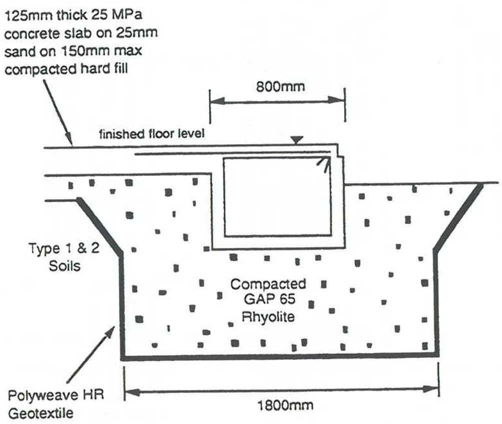


Figure 4: External strip footings of building

5 TRACK CONSTRUCTION

5.1 Main Track Design

The people mover fleet consists of five battery powered electric vehicles, each seating up to 36 people. The articulated trams are 14 metres in length and will operate continuously for a ten hour day throughout the year, charging at the main station during passenger pick up/drop off periods, and overnight. It is estimated the people mover will transport around one million visitors annually, the technology a world first in terms of its innovative engineering design. Typical axle loads are in the order of 2.0 tonnes (20 kN), approximately 25% of standard design axle loads. Considerable effort was expended in pavement design taking into account the lower vehicle induced stresses due to lighter loadings and low CBR's available for design.

Due to the low CBR and bearing capacity of the type 1 and 2 soils over parts of the track alignment (CBR's in places less than 1, and as low as 15 kPa safe bearing available on unimproved ground) and the presence of geothermal mudpools, several options were considered to prevent subgrade failures and provide safe passage for the people mover. These included increased basecourse hardfill thickness, use and selection of appropriate geotextile reinforcing, subgrade cement stabilisation and the use of specialised structures to overcome the very challenging conditions.

Approximately 1.8 km of paved and elevated timber deck track was constructed for the people mover. Much of the track was relatively straightforward to construct apart from a 400 metre stretch of track adjacent to two lakes at the southern end of the site and areas of relatively thin silica sinter on the main existing track, adjacent to and south of the Pohutu geyser.

5.2 Paved Track Alignment

The paved part of the track alignment was constructed on the elevated terrace areas where ground conditions, although soft or loose in places due to the overlying type 1 ashes, were sufficiently cool and competent to accommodate standard track construction of undercutting and backfilling with compacted hardfill. Construction of the paved track section consisted of 60 mm cobble block pavers, laid in a herringbone pattern, overlying CBR dependant depths (typically 150 to 400 mm) of compacted GAP 65 rhyolite hardfill subbase.

Paved track design was based on plate load, in-situ CBR and scala penetrometer test results carried out over the whole track alignment. Basecourse depths were also based on design charts and output from software supplied by the manufacturers of the cobble pavers as well as methods given by Giroud and Noiray (3), AUSTROADS (4) and Transit New Zealand (5). To limit basecourse volumes and costs, geotextiles were used where ground conditions permitted. Where excess deflections occurred from construction traffic, additional undercutting and backfilling with compacted rhyolite was undertaken. Cement stabilisation was not considered a viable option due to expense and accessibility for stabilisation gear.

5.3 Elevated Track Structure

Some 400 metres of glulam timber decking on an elevated timber frame structure was constructed on the southern part of the development and adjacent to the Pohutu geyser, predominantly on hot geothermally affected and altered ground. Bearing capacity on these areas was as low as 15 kPa, with CBR's of less than 1.

The timber framed structure was bolted onto 600 mm by 300 mm fabriform bags filled with lightweight pumped concrete (Figure 5). Where excessively uneven ground was encountered, the underlying ground was benched and shotcrete used to 'lock' the fabriform bags in, although the majority of the of bags were laid on the ground surface. The elevated track structures were designed for dead plus live loads of 12 to 15 kPa.

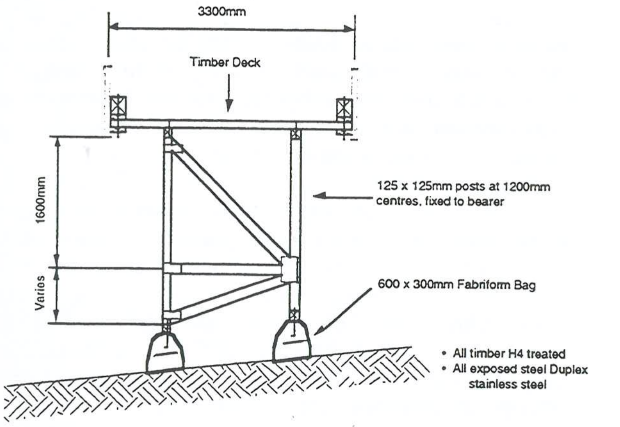


Figure 5: Elevated track structure

Two small glulam timber bridges were also constructed as part of the elevated track. The Lakes Bridge (10 metres long) spans between the narrow isthmus of two lakes, while the Mudpools Bridge (12 metres long) spans a narrow ridge with active mudpools on either side. Both bridges were designed for 15 kPa safe bearing, the Lakes Bridge having additional piles to account for seismic bearing capacity and possible liquefiable ground conditions.

Both the Lakes and Mudpools bridges have grouting ports inserted into the bridge footings to allow for future compaction grouting if required.

5.4 Sinter Bridges

Up to seven areas of inferred shallow crust were revealed by the ground radar survey from G1 to G21, typically on type 5 and 6 soils. To span these areas of thin sinter overlying possible cavities, three on-grade "sinter bridges" of 4 metres width were constructed, consisting of 100 mm thick 35 MPa concrete with fibreglass rod reinforcing. The Sinter bridges were up to 38 metres in length.

6 BRIDGES

6.1 Helicopter Gully Bridge

This 3.4 metre wide bridge spans from the helipad tunnel portal to the building terrace area, a distance of some 78 metres (Figure 1). Apart from two tunnel portal abutment positions, there are two bridge piers in the gully area, located 28 metres apart. Ground conditions at each bridge pier consisted of a veneer of type 1 ashes underlain by hydrothermally altered silts, muds and clays (type 5) to around 6 metres depth, which were in turn underlain by silica cemented sands and gravel breccia's (type 4). Unconfined compressive strengths in the underlying type 4 materials ranged from 2.2 to 30 MPa.

Due to high lateral loads and to enable adequate loading capacity under static and seismic loading conditions (only 20 kPa available on unimproved ground at the surface) four, 600 mm diameter piles at each bridge pier were initially pendulum drilled through the surficial ashes and type 5 soils, then percussion hammer drilled to enable piles to be socketed at least 1.5 metres into the silicified type 4 materials (Figure 6). The bridge entry and exit portal tunnel piles were drilled to approximately 7 metres depth.

Due to the extreme ground conditions encountered at the base of each pile hole (steam venting and groundwater in excess of 90°C, pH approximately 2-3, and 400-600 ppm sulphate), and mudpools adjacent to the eastern pier, duracem cement, a blend of cement and ground granulated blastfurnace slag, was specified for all piles and bridge footing concrete.

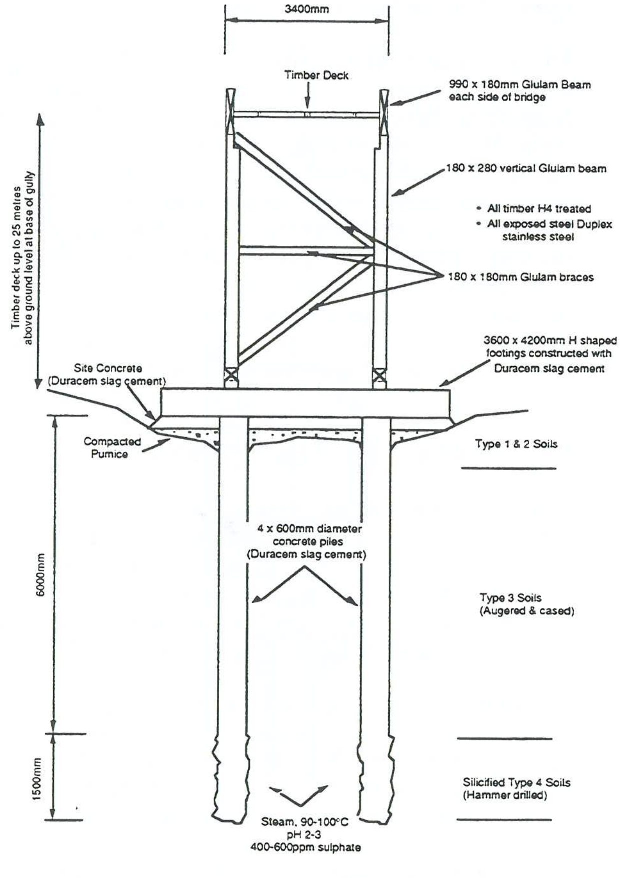


Figure 6: Helicopter Gully bridge foundations

6.2 Puarenga Stream Bridge

The glulam timber Puarenga Stream bridge runs some 33 metres from the southern portal of the Puarenga Stream Bridge tunnel across the stream. Due to low bearing pressures and proximity to the steep bank, piles were drilled to depths of between 1.7 and 5 metres to found on competent basal type 3 materials under individual footings.

6.3 Pohutu Bridge Upgrade

The existing glulam beam timber bridge adjacent to the Pohutu Geyser was widened by 2.2 metres to allow access for both the people mover and pedestrian traffic. Despite the age of the bridge (around 20 years old) and the hot ground conditions (in excess of 80°C beneath surficial sinter and gravel fill) the existing foundations have performed well.

7 TUNNELS

Three shallow 2.5 metre high by 3 metre wide box culvert cut and cover tunnels were constructed, primarily to ease approach and exit gradients from some of the bridges. Tunnel design had to take into account seismic and soil lateral loads, especially in the type 3 soils which are prone to swelling when unloaded. Stability of the temporary batters also required careful monitoring during construction.
Available CBR's along tunnel subgrades ranged from 4 to less than 1. Up to 800 mm of compacted hardfill and Polyweave HR geotextile was placed under all tunnel sections with selective undercutting and further backfilling with hardfill where soft ground was encountered.

All tunnel cuts were backfilled with hardfill following tunnel section placement. Backfill material was specifically designed to prevent internal soil erosion. 110 mm diameter draincoils were placed along the tunnel invert line along either side to control subsurface groundwater. Expanded polystyrene panels were initially specified for back filling where type 3 soils were encountered to account for any swelling pressures developed within this unit. However, due to the length of time tunnel cuts were open (up to 3 months) and size of excavations on either side (up to 2 metres) polystyrene panels were not used. Backfill over the tunnel sections consisted of reworked type 1 and 2 soils.

8 RETAINING STRUCTURES

Three flexible gabion basket retaining structures were constructed mainly to retain filled ground above tunnel entrances and exits. PVC coated galvanised mesh gabions were used, backfilled with 150/80 gap graded rhyolite fill. Gabions were founded on either compacted GAP 65 rhyolite or compacted pumice, depending on ground conditions.

Several areas of the track required cuts of up to 3.5 metres to achieve requisite track gradients of 12% downslope and 10% upslope. Cut slopes were battered back to 60° for slopes up to 2 metres high, and 50° for slopes greater than 2 metres in height. Punga logs were then laid over the batter slopes.

Battering the cut slopes versus fully retaining slopes with crib walls or similar was seen as a risk-cost trade off. The cost savings in leaving steeper batters will be offset by the future risk of having to repair any instabilities that may occur. There were also issues of safety for the public in terms of leaving slopes steeper than ideal, although to some extent these will be offset by regular detailed safety surveys and maintenance/observation procedures put in place to monit track conditions.

9. CONCLUSIONS

The Whakarewarewa project has proven how difficult civ construction in active geothermal areas can be, with grour conditions well beyond the range of conventional geotechnic engineering design solutions. Despite an extensiv geotechnical investigation, there was an expectation th ground conditions would vary from what the project designe had assumed. Thus, design values for geotechnical aspec of the buildings, structures, and the roadways were based c anticipated worst case conditions, imposing minimu practical construction loads. The adopted "observation method" approach allowed the design to be modified durir the construction phase by finding the best way out, implementing design alternatives when unexpected grour conditions were encountered.

A commissioning process is to be undertaken prior to tl development being open to the public. This process wi involve monitoring dynamic loadings on the structures by tl people mover. While failure/collapse of structures is n expected, there may be some excessive live load differenti movements. Contingencies for such problems incluc compaction grouting through grout ports installed in bridg footings and additional retaining structures.

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