Geophysical testing for rock assessment and pile design

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ABSTRACT

Geophysical testing has increasingly been used for geotechnical investigations to identify subsurface irregularities such as fill, cavities and variable strata. It can also be used to obtain quantitative information that is useful for foundation assessment and design. This paper describes the use of SUBS© (Site Uniformity Borehole Seismic) geophysical testing for the geotechnical investigation and foundation design of St Falls development in Falls Creek Alpine Resort, VIC, Australia. The site complexities included up to 7m of fill, two buried gullies and two geological faults. Several underground services, including a fibre optic cable, were present within the site, restricting the number of boreholes that could be drilled for the investigation. Previous investigations within and adjacent to the site indicated the weathering of the underlying rock to be extremely variable.

The SUBS©testing was conducted in four deep boreholes located to cover the site. This testing was used for profiling of the site. The seismic velocities obtained from the SUBS© testing could not directly be used for the assessment of pile design parameters as no accepted correlation has yet been developed, but were used in an indirect manner. The seismic velocities were initially used to estimate the SPT 'N' values and Unconfined Compressive Strength characteristics of the subsurface materials. These values were then used to assess the pile design parameters. The ultimate end bearing pressures for piles thus obtained were then compared with the presumptive values used by the Hong Kong Building Department for rocks of similar geological origin. The comparison indicated that there is reasonable similarity between the values.

1 INTRODUCTION

The St Falls development in the Victorian alpine resort of Falls Creek includes construction of two new buildings of five and six levels in the village centre. The site comprised a gently sloping lower platform and a near level upper terrace separated by a steep embankment up to 6m high. The lower platform was sealed and had been used as a car park and the floor level of the new buildings was set approximately at the car park level. In preparation for the building platforms the upper terrace was excavated to depths of up to 7m on the western and southern sides of the west building and depths of up to 3m within the footprint of the east building.

Previous developments of the site involved substantial excavation and placement of fill within the site and surrounds. The site previously featured drainage gullies, which have been filled to the existing level for the development of the car park at the lower level and the ski platform at the upper level (Falls creek resorts Pty Ltd 1985). Up to 7m of fill was placed to create the ski platform.

Earlier geotechnical investigations within and adjacent to the site (Coffey Geosciences 2005a, 2005b) indicated the presence of extremely variable weathered rock in the site ranging from extremely weathered (XW) to slightly weathered (SW) granite. Extremely to highly weathered materials were observed within layers of moderately (MW) to slightly weathered (SW) rock.

This paper describes the use of seismic velocities to estimate the SPT 'N' values and Unconfined Compressive Strength characteristics of the subsurface materials, and the subsequent use of these values to assess the pile design parameters. The ultimate end bearing pressures for piles thus obtained are compared with the presumptive values used by the Hong Kong Building Department for rocks of similar geological origin.

2 GEOLOGY OF THE SITE

The Falls Creek Village and the surrounding alpine areas lie within the Omeo Metamorphic Complex comprising mainly gneiss, schist and phyllite (Natural Resources and Environment 1997). This Complex is intruded by Silurian granite and granodiorite. Tertiary extrusive basalts also overly these rocks on some of the ridge crests of the Village.

There are also several mapped faults within the Falls Creek Village. These are at interfaces between the intrusive granite bands and the gneiss. Natural groundwater discharge is concentrated along these faults that are also marked by erosion gullies and alpine bogs.

Both a major fault and shear zone traverses the project site (GHD, 2005). The fault trends northeast to southwest and appears to pass through the central part of the proposed development. The shear zone passes through the site in a northwest to southeast direction.

The general subsurface conditions at the site, as assessed from six boreholes, are summarised in Table 1.

Table 1: Summary of subsurface conditions

Interpreted Geotechnical Unit	Depth to Top of Unit (m)	Unit Thickness (m)	Description
Fill	0	0.9 to 7.0	SANDY SILT: low plasticity, fine to coarse grained sand, some fine to coarse grained gravel, brown, black.
Colluvium	0.3 to 5.6	0.5 to 2.4	CLAYEY SILT: low and medium plasticity, fine to coarse grained sand, brown, black, stiff.
Weathered rock	1.0 to 8.0	Not penetrated	GRANITE: Extremely to slightly weathered.

3 SUBS© TESTING

Several underground services including a fibre optic cable traversed the site and restricted the number of boreholes that could be drilled for the investigation. As a result of this and the high weathering variability of the rock, Site Uniformity Borehole Seismic (SUBS©) testing (Whiteley 2003) was adopted to investigate the site. While the SUBS© testing required drilling of only four deep boreholes to assess the entire site, a few additional boreholes were also drilled to further assess the subsurface conditions and to compare with the results of the SUBS© test.

Three boreholes (BH7, BH9 and BH12) to 20m depth and a fourth borehole (BH8) to 15m depth were drilled using a combination of wash boring and NMLC method to facilitate the SUBS© testing. A PVC pipe of 50mm diameter was installed within each of the four boreholes and the sides were backfilled with sand. After sealing the base of the PVC pipe using bentonite, the PVC pipe was filled with water.

For SUBS© testing, the process involves an array of closely spaced hydrophone detectors, encased in an oil-filled tube or "eel", being lowered into the PVC-cased borehole. The PVC pipe is then filled with water to achieve seismic coupling of the hydrophone detectors to the earth. Seismic energy is generated by a sledge hammer source on the ground surface at varying distances from the borehole. A horizontal investigation radius of two to three times the depth of the hydrophone array is possible, depending on the subsurface conditions. Seismic waves that travel from each surface-impact point to the detector array are modified by the ground conditions around the borehole within an effective volume of investigation. Should the subsurface conditions or elastic properties of these materials vary laterally around the borehole, or should voids be present, the travel times to each subsurface detector will be different for identical source offsets around the borehole. Any significant condition that weakens the material will scatter the seismic energy and delay its travel time.

Seismic refraction measurements were also made along the seismic scan lines to assist the production of the SUBS© images by determining the travel time of a first-arriving refracted seismic wave as it travels from the surface, through the fill and soil, and is refracted back from underlying rock interfaces to the surface. The travel time of a seismic wave is a function of density and strength of the materials. Seismic refraction data were acquired in accordance with accepted practice (Whiteley 1994) along a total of two lines with a geophone spacing of 3m.

In borehole BH8, the seismic testing was carried out to a depth of 9.1m below ground level due to a block in the borehole.

4 RESULTS OF TESTING

The results of the SUBS© testing are shown in Figures 1 and 2. For each of the two lines, the progressive increase in seismic velocity with depth reflects the transition from fill, through colluvium to granite rock which, in general, experiences a decreasing degree of weathering as the depth increases. The seismic velocities to approximately 5 to 7 m depth are relatively low, indicating mainly fill, colluvium or extremely weathered rock. These velocities increase more rapidly below these depths within less weathered rock.

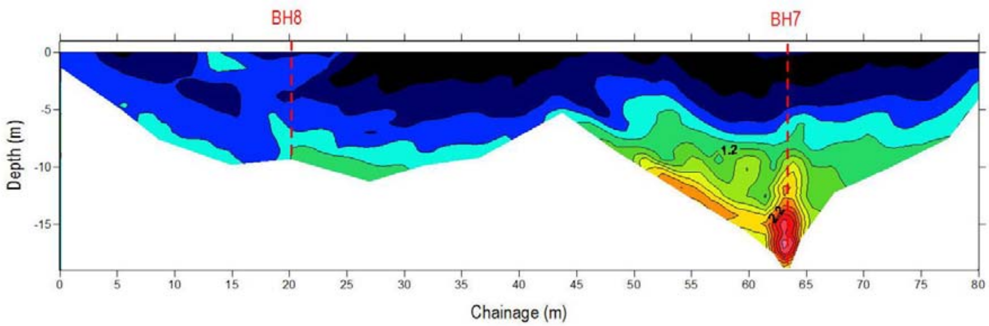


Figure 1. Seismic Images BH7-BH8

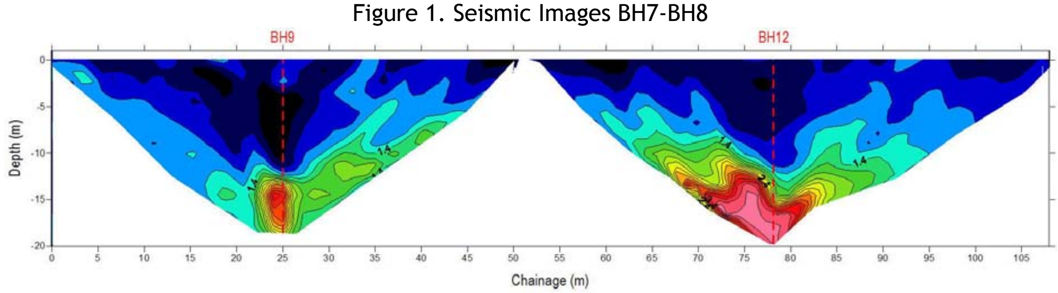
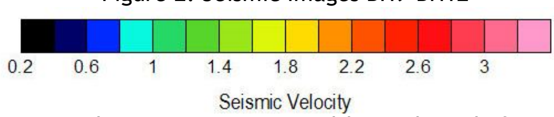


Figure 2. Seismic Images BH9-BH12



On the basis of these tomographic images, upper and lower bounds for average seismic velocities assessed at each borehole along the two seismic lines are summarised in Table 2. It will be seen that the average seismic velocity generally increases with depth, and reaches values consistent with moderately weathered granite when the depth exceeds about 12m to13m below the existing ground level.

4.1 Use of geophysical data to assess foundation design properties

Seismic P-wave velocities can be used directly to calculate the shear modulus or Young's modulus of the soil and rock strata, provided a Poisson's ratio and density is available or by measurement of S-wave velocity, but cannot directly provide information on design parameters for foundation design.

Thus, the following approximate process has been employed to assess foundation design parameters:

- The seismic velocity values have been used with previous correlations to estimate the SPT values for the soil strata;
- The seismic velocity values have been used with previous correlations to estimate the equivalent unconfined compressive strength characteristics of the rock strata;
- For the soil strata, the pile design values such as skin friction and end bearing pressure have been correlated with the estimated SPT values;
- For the rock strata, the pile design values such as skin friction and end bearing pressure have been correlated with the equivalent unconfined compressive strength.

TABLE 2: Summary of inferred ranges of seismic velocity

Borehole	*Depth Range (m)*	*Min. Seismic Velocity (km/s)*	*Max. Seismic Velocity (km/s)*
BH7	0.0-4.4	0.20	0.80
"	4.4-7.4	0.80	1.20
"	7.4-8.8	1.20	1.60
"	8.8-11.2	1.60	2.00
"	>11.2	2.00	3.40
BH8	0.0-6.5	0.20	0.80
"	6.5-9.1	0.80	1.20
BH9	0.0-3.2	0.40	0.80
"	3.2-10.8	0.80	1.20
"	10.8-12.5	1.20	2.80
"	>12.5	2.80	3.20
BH12	0.0-7.9	0.40	0.80
"	7.9-10.8	0.80	1.50
"	10.8-13.2	1.50	2.80
"	>13.2	2.80	4.40

To check that the foundation design parameters so derived are consistent with previous experience, reference has then been made to commonly used design parameters for the granite rocks in Hong Kong.

4.2 Seismic velocity vs SPT & unconfined compressive strength (UCS)

On the basis of experience from previous Coffey projects the correlations shown in Tables 3 and 4 have been used as a guide to the SPT and UCS values.

TABLE 3 Correlations between seismic velocity and SPT

Seismic Velocity (km/s)	*Geotechnical Description*	*SPT 'N' Value*
<0.35-0.43	Very loose	0-4
0.43-0.52	Loose	4-10
0.52-0.73	Medium dense	10-30
0.73-1.68	Dense	30-50
>1.68	Very Dense	>50

4.3 Correlations between SPT & foundation design parameters

Using the correlations developed by Decourt (1995) as a basis, the following correlations have been used to estimate foundation design parameters from SPT 'N' values:

- Ultimate bearing capacity of shallow foundations q_u:

$q_u = K.N$ *kPa,* where K=90 for sands, 80 for intermediate soils and 65 for saturated clays

- Ultimate pile skin friction for f_s:

  $f_s = a(2.8N+10)$ *kPa,* where a = 0.6 for bored piles and 1.0 for driven piles

- Ultimate end bearing for piles f_b:

  $f_b = K_b N$ *kPa,* where K_b = 165 to 205 for displacement piles in clayey silt to sandy silt, and 100 to 115 for non-displacement piles in clayey silt to sandy silt.

TABLE 4: Correlations between seismic velocity and UCS

Seismic Velocity (km/s)	*Geotechnical Description*	*UCS (Mpa)*
<2.0	Low strength rock	<10
2.0-2.5	Medium strength rock	10-20
2.5-3.5	High strength rock; stratified, jointed	20-60
3.5-7.0	Very high strength rock, stressed	>60

4.4 Correlations between foundation design parameters and UCS

The following correlations have been used to assess the foundation parameters for rock:

1. Ultimate bearing capacity for shallow foundations, $q_u = 3\ (UCS)^{0.5}$ *MPa*
2. Ultimate pile skin friction for bored piles, $f_s = 0.3\ (UCS)^{0.5}$ *MPa*
3. Ultimate end bearing for bored piles, $f_b = 4.8\ (UCS)^{0.5}$ *MPa*

4.5 Foundation design parameters

On the basis of the measured seismic velocities, the inferred representative SPT and UCS values for the various strata encountered at the site are summarized in Table 5. Using these representative SPT and UCS values, and the correlations set out above, the suggested design values for the various strata, based on the seismic velocities, are shown in Table 6.

TABLE 5: Summary of representative SPT and UCS values.

Material	*Representative Seismic Velocity (km/s)*	*Inferred Representative SPT 'N' Value*	*Inferred Representative UCS (MPa)*
Fill/colluvium	0.4	4	-
EW Rock	0.8	30	-
HW Rock	1.2	40	-
MW Rock	1.8	-	3.6
SW Rock	2.8	-	32

TABLE 6 Summary of recommended foundation design parameters

Material	*Ultimate Bearing Pressure for Shallow Footing (kPa)*	*Ultimate Pile Skin Friction (kPa)*		*Ultimate End Bearing Pressure (kPa)*	
		Driven Piles	*Bored Piles*	*Driven Piles*	*Bored Piles*
Colluvium	320	21	13	720	440
EW Rock	1800	94	56	5400	3300
HW Rock	3200	120	73	7200	4400
MW Rock	5690	-	560	-	9100
SW Rock	17000	-	1700	-	27000

For comparison purposes, values of the allowable (presumptive) bearing pressures employed in Hong Kong have been used to obtain equivalent ultimate bearing pressures, assuming a factor of safety of 3, and are shown in Table 7. While there is not a direct equivalence between the rock descriptions in the two cases, a comparison between Tables 6 and 7 indicates that there is a reasonable similarity between the values. It is likely that the conservative value in the Hong Kong recommendations is due to the need to restrict settlements, rather than to satisfy ultimate bearing capacity requirements. Ideally, foundation design based on ultimate bearing pressures should also check that the allowable load induced settlements and differential settlements are not exceeded.

TABLE 7 Summary of ultimate bearing pressures derived from presumptive values used by Hong Kong Buildings Department (factor of safety of 3 assumed)

Material & correspondence to Materials at St Falls	*Ultimate Bearing Pressure (kPa)*
Loose sands & gravels (N = 4-10) – worse than the fill	120
Highly-completely decomposed rock (Grade V) -possibly equivalent to EW – HW rock	3000
Moderately decomposed (Grade IV+) – equivalent to MW rock	9000
Slightly to moderately decomposed rock (Grade III+); UCS >25 MPa – equivalent to SW rock	15000

The values in Table 6 are considered appropriate for the design of foundations within and on the rock. The final foundation design will also depend on the allowable settlements specified for the buildings. The Young's modulus of the various strata based on the measured seismic velocities may also be used for the estimation of settlements.

5 CONCLUSIONS

This paper has emphasized the usefulness of geophysical testing for identifying the subsurface conditions for the St Falls Plaza development. It has also indicated how the seismic velocities deduced from geophysical measurements can be used, via indirect correlations, to estimate pile design parameters. It is also demonstrated that the ultimate end bearing pressures for piles thus obtained are comparable with the presumptive values used by the Hong Kong Building Department for rocks of similar geological origin.

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