SEISMIC TECHNOLOGY FOR SLOPE STABILITY INVESTIGATION IN YEPPOON NORTH QUEENSLAND

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ABSTRACT

Slope instability has become one of the most significant geotechnical risks in recent times. Unfavourable geologic and climatic conditions, combined with anthropogenic activities, result in thousands of slope failures each year. Standard methods of geotechnical investigation such as boreholes and SPT-tests are not consistent in providing the geotechnical data for the site as a whole. Leaving engineers to assume the soil properties for most of the site, a disadvantage that may significantly increase the cost of remediation measures. Geophysics, in particular seismic surveying can provide alternative methods to solve this geotechnical problem. Although seismic surveys are widely used throughout the mining industry, it has not been applied to slope stability problems in Australia. This is mostly due to a lack of awareness, availability and acceptance/application of modern science. This paper seeks to establish the advantages of seismic methods in resolving slope instability issues.

This paper illustrates the advantages of using seismic technology by presenting a case study, seismic technology for slope stability investigation in Yeppoon North Queensland. After significant rain in February 2008 a medium sized landslide occurred at the top of an embankment in the road reserve at Statue Bay, Yeppoon. The landslide regressed into the rear of the private properties above and therefore remediation measures were required. This is a reasonably standard geotechnical task, but when combined with steep vegetated slopes and very limited access, standard methods were not adequate. Seismic refraction and Multichannel Analysis of Surface Waves (MASW) surveying methods were used to generate 2D & 3D sub-surface images. These images distinctively illustrated the sub-surface material profiles, including the rock line and areas of slip prone material.

1 INTRODUCTION

Slope instability has become one of the most significant geotechnical risks in recent times. In Australia the Thredbo Landslide in 1997 (Tayler, 2010) in which 18 people lost their life, is probably the most well known. This landslide in particular, highlighted to councils and engineers throughout Australia the importance of undertaking an appropriate slope instability investigation.

Landsides occur when the downward forces, including the self weight exceeds the shear strength holding the soil on the slope. Shear strength loss can be attributed to excess pore water pressure, weathering of the soil profile and anthropogenic activities. The most important information required when conducting a slope stability investigation is soil depth, soil profile, soil strength and groundwater conditions (Varnes, 1978).

Most geotechnical engineers at some point have been left inadequately equipped to undertake slope stability investigation when supplied with a common AS2870 & AS1726 compliant geotechnical reports. These are based on standard test methods. These methods give results at the test point. However, even with numerous test points, the engineer is left to assume the soil profiles in-between these points. These assumptions regularly leave the engineer at risk of missing vital sub-surface data, such as the presence of colluvium, soft soils, a spike and or a depression in the rock line.

Seismic technology has been applied to many geotechnical problems, such as bearing capacity evaluation, earthquake site classification, compaction certification, subsidence investigation and now slope stability investigation. It gives the geotechnical engineer another tool to compile the geotechnical model for the site. Combined with standard methods the seismic results can join the test points, reducing assumptions and the giving the engineer confidence in the sub-surface profile. This paper will show how seismic technology can be used to aid in slope stability investigations.

2 CASE STUDY

Figure 1: A view of the landslide from the top of the bluff.

This site is located on the top of a bluff in Statue Bay, Yeppoon North Queensland. It consists of two residential sized blocks with a 27m high steep vegetated embankment at the rear (Figure 1). The embankment was cut by the military in the 1930s to construct a road around the base of the bluff. The construction drawings show that the natural slope was about 30-40° before the cut and that the cut batters vary from about 60-65° in the sedimentary rock, to vertical in the tuff. The road below is considered a main road that connects outlying communities to the city centre of Yeppoon.

The residents advised that the landslide was activated during heavy rain in February 2008. This was confirmed by the rain records where 385mm of rain fell in 3 days, over 30% of the yearly total (Bureau of Meteorology). The landslide occurred in the road reserve below and extended about 1.5m into the rear of the private properties above (Figure 1&3). The toe of the landslide also blocked the road below and it remained closed for a number of days during the cleanup operation. The complex debris slide (Varnes, 1978) was about 16 x 9 x 3m deep and had a volume of approximately 285m^3 (Figure 2&3). To ensure the safety of the road users and the residents remediation measures were required.

This is a reasonably standard geotechnical task, but when combined with steep vegetated slopes and very limited access, standard methods were not considered adequate. Therefore seismic MASW and refraction surveying methods were utilised to give additional information on the sub-surface material.

2.1 FIELD INVESTIGATION

The site was tested by augering four boreholes to provide information on the soil profile in accordance with AS1726-1996, Geotechnical Site Investigations. Due to the site access limitations this was undertaken using a small power auger to about 1.5m then hand augers to greater depths. These results were augmented by the use of four seismic surveying lines, five dynamic cone penetrometer tests and shear vane testing. Disturbed and undisturbed samples were also collected for laboratory testing. Tests selected were Atterberg limits and Shear box testing. A brief site level survey was also carried out to enable computer aided slope analysis.
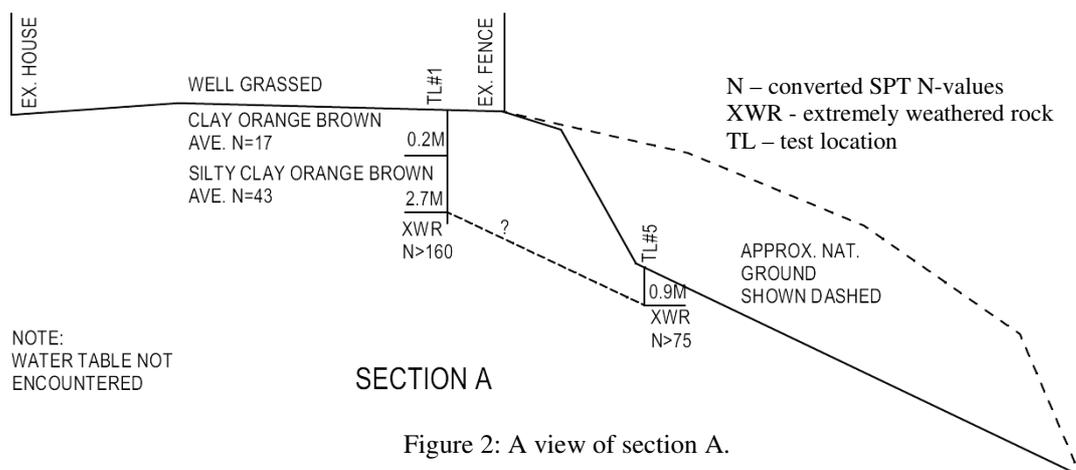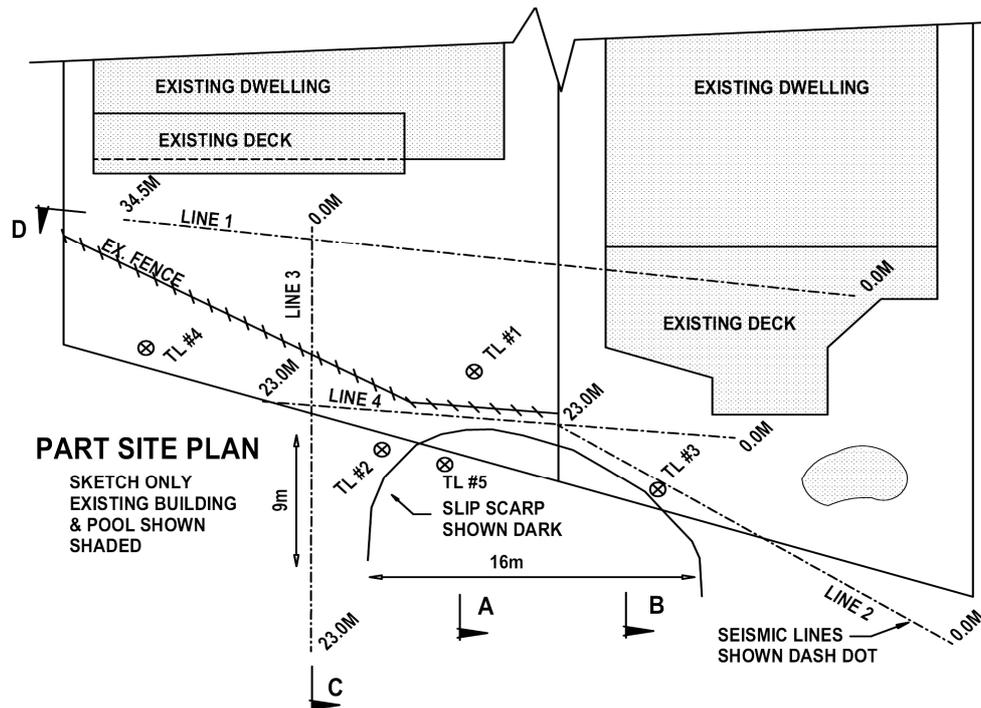

Figure 2: A view of section A.

Figure 3: A view of the site plan (Note: TL indicates test location).

2.2 INVESTIGSTION RESULTS

2.2.1 BORELOGS

The boreholes indicated a variable soil profile. Generally the profile consisted of a clay layer overlying extremely weathered rock. The clay layer varied in thickness from 1m at test location (TL) number 4 to 3m at TL 2. Two examples of the borelogs are included in Tables 1 & 2.

TL#1

Geologic Profile	Depth (m)	Soil or Rock Structure	Consistency	Shear vane (kPa)
Fill	0 to 0.2	SILTY CLAY, dark brown topsoil, cobbles, moist layer, CH	firm	20, 40,50
Residual?	0.2 to 1.2	SILTY CLAY, orange brown, moist layer, CH	stiff	N/A
	1.2 to 2.7	SILTY CLAY, with gravel and sands, orange brown, dry layer, CL	very stiff	N/A
	2.7	UTP, XWR?		

Table 1: Is the borelog from test location number 1 (Note: TL - test location, XWR - extremely weathered rock, UTP – unable to penetrate).

TL#3

Geologic Profile	Depth (m)	Soil or Rock Structure	Consistency	Shear vane (kPa)
Residual?	0 to 1.6	SILTY CLAY, light brown with gravels, dry layer, CH	stiff	N/A
	1.6	UTP, XWR?		

Table 2: Is the borelog from test location number 3 (Note: TL - test location, XWR - extremely weathered rock, UTP – unable to penetrate).

2.2.2 DCP RESULTS

The standard DCP results indicate variable strength material througout the soil profile. As prevsiously stated, these results are only relevant at the test point. In contrast, the seismic velocities are averaged over about four meters. If these averaged seismic velocities are converted to DCP results, using the empical formula by Karai (1966), the profile is more consistent and generally increasing in strength with depth. Figures 4&5 show how the standard DCP results compare to the converted DCP results, for test locations 1&3. These figures show a remarkable correlation between the standard DCP results and the converted DCP results.
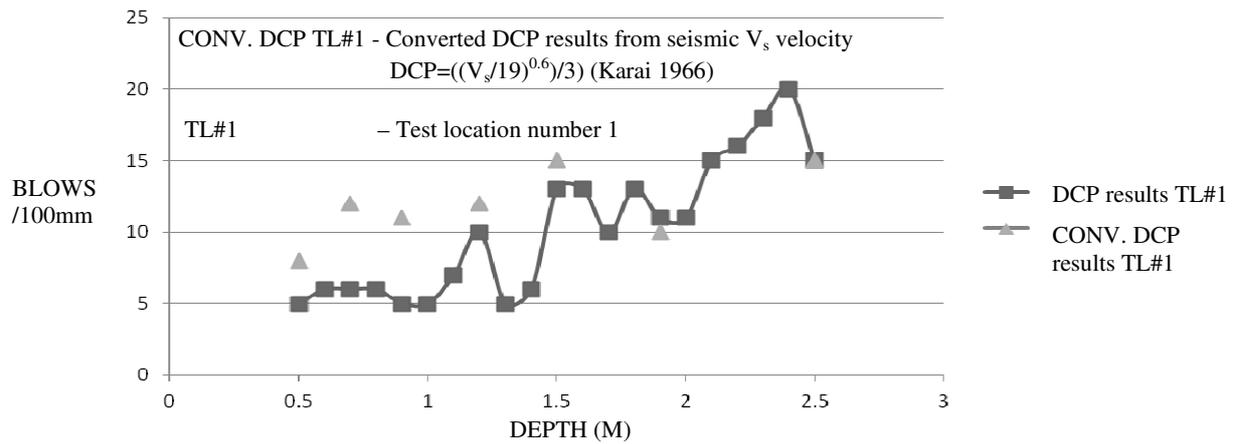
Figure 4: Is an example of the correlation between standard DCP results and Converted DCP results from seismic velocity for test location number 1.
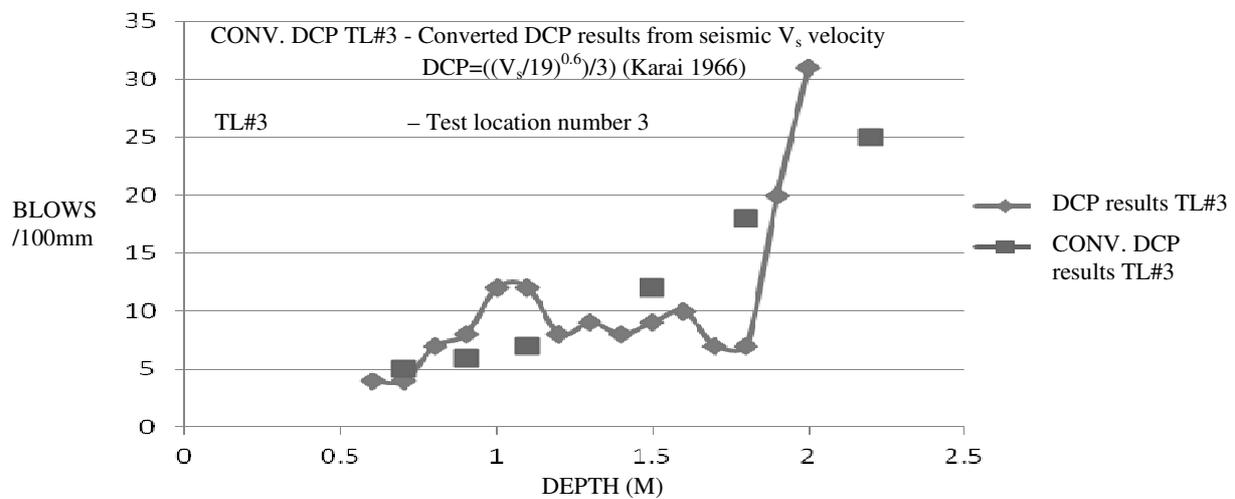

Figure 5: Is an example of the correlation between standard DCP results and Converted DCP results from seismic velocity for test location number 3.

2.2.3 LABORATORY TESTS SUMMARY

Material/location/ depth	Initial moisture content %	Liquid limit %	Plastic limit %	Plasticity index %	CLASSIFICA -TION
SILTY CLAY / TL#1-4 / 1.0-2.0m	23- 28	60-75	27-34	36-40	CH

Table 3: Is a summary of the Atterberg limits results for numerous samples throughout the profile (Note: TL - test location, CH – high plasticity clay).

Material/location/depth	Range of Internal Friction Angles (phi)	Cohesion (C´) Range
SILTY CLAY / TL#1-3 / 1.0-2.0m	34-39°	30-40 kPa

Table 4: Is a summary of the shear box results for three undisturbed samples (Note: TL - test location).

The laboratory results indicate that the clay across the site has similar properties and parameters. The values of cohesion appear higher than expected. This was considered to be due to the strain rate being too fast to allow the pore water to dissipate resulting in higher cohesion values.

It is worth noting that, that the internal friction angle is generally the maximum slope that the soil can remain stable at in the long term. In this case, the natural slope was between 30-40°, which is similar to the measured internal friction angle. After the road was cut, the embankment was steepened to about 60-65°, well above the internal friction angle. This significantly increased the risk of instability of the embankment. This combined with unfavorable climatic conditions and high plasticity clays were some of the main causes of the landslide.

2.2.4 SEISMIC RESULTS
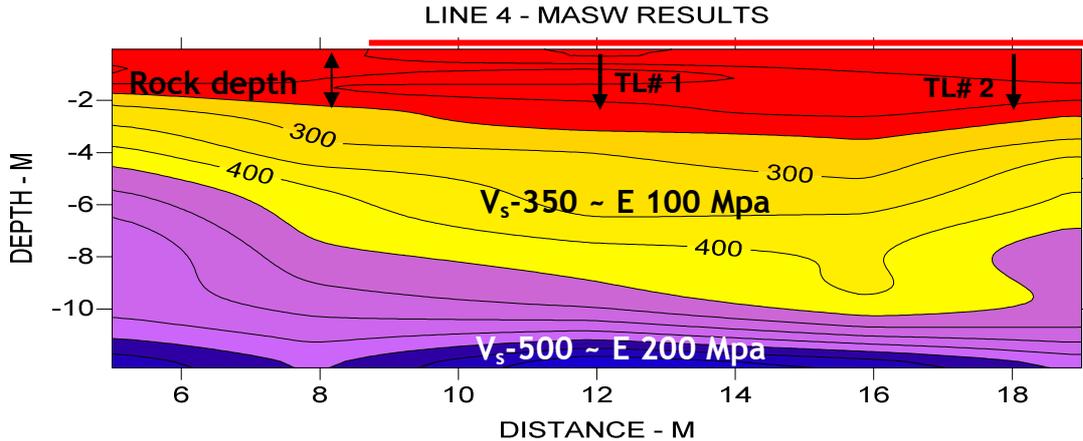
LINE 4 - MASW RESULTS

Figure 6: Results of an example of the 2D V_s profile (Note: red - soft clays, yellow extremely weathered rock (XWR), purple & blue fresh rock, E- Young's modulus, red line - location of landslide).

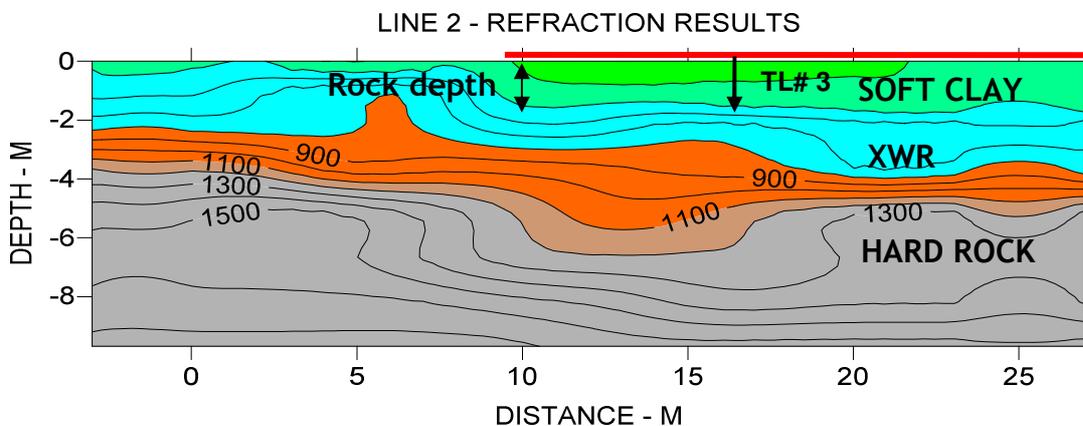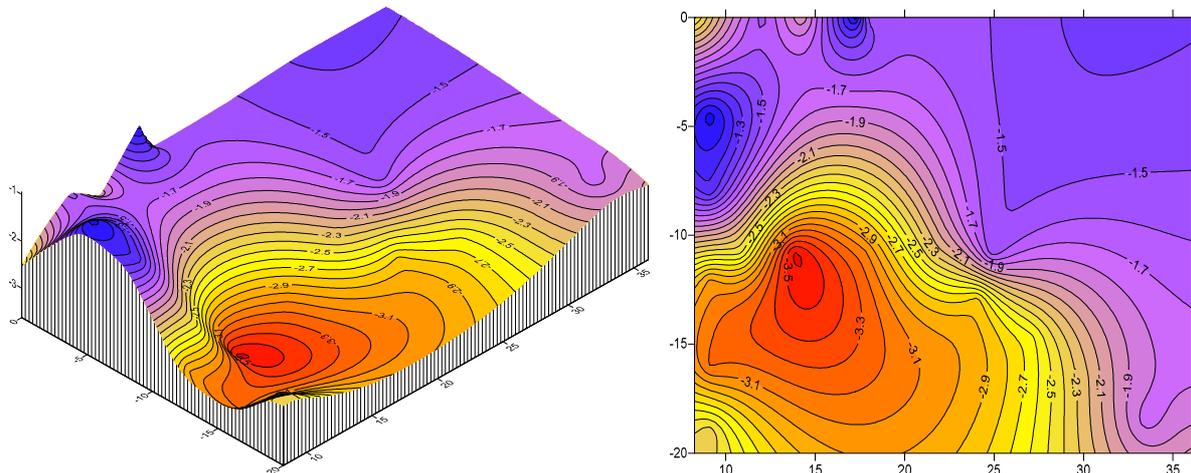### LINE 2 - REFRACTION RESULTS

Figure 7: Results of an example of the 2D V_p profile (Note: green -soft clays , aqua & orange extremely weathered rock (XWR) and grey harder rock, red line indicates location of landslide).

Figure 8: Results of the MASW 3D and contour plan of V_s ~250 (rock line) profile (Note: slip occurred in the depressed zone).

There are many correlations from seismic velocities to geotechnical properties. The two most notable are the empirical relationship to SPT–N value and the mathematical relationship to Young's Modulus. Surface wave velocity (V_s) has been correlated by numerous authors to the SPT N values. The correlation used in this paper is $V_s = 19(N_{60})^{0.6}$ by Karai (1966). The mathematical relationship to Young's Modulus is relevant at small stain. If the strain is higher a correlation factor is required to a factor down the calculated Young's Modulus. In this case it was used to assess the strength of the underlying rock. This was considered to have low stain and the correlation factor was not required. The formula used in this case study is as follows:

$$V_s = [E/\{2(1+\upsilon)\rho\}]^{1/2} \quad (1)$$

V_s=Seismic surface wave speed, as obtained on site by testing (m/sec)
E= Young's modulus at small strain (kPa)
υ= Poisson's ratio usually as tabled in numerous text books (eg Look 2007)
ρ= in-situ weight of material, usually as tabled in numerous text books (g/cm^2)
Note: To assess E, υ and ρ text book values are usually adequate.

3 DISCUSSION

The main cause that contributed to the landslide was that the embankment was significantly steeper than the internal friction angle of the soil. The trigger of this landslide, as with many, was the significant rain event that occurred in February 2008. This rain saturated the high plasticity clays causing an increase in pore water pressure and a decrease in effective stress, resulting in instability.

The seismic output gave an overall view of the sub-surface profiles and strengths, this made it easy for the engineer to construct the geotechnical model for the site and assess the geotechnical issues. In this case, the seismic survey gave a clear indication of how the rock and soil profiles varied over the site. It clearly shows that the rock profile is deeper within the slip zone (Figures 6-8), overlaid by weaker clays. The standard soil testing methods confirmed this result.

Even though the seismic results are averaged both vertically and horizontally the converted DCP results correlated well to the standard DCP results (Figures 4&5). The results did differ, however the differences can be associated to the averaging effect of the seismic survey.

The seismic survey indicated sub-surface strengths to about 12m. Due to the poor site access, this wasn't possible using standard methods. The depth of penetration of the seismic survey enabled the geotechnical engineer to access the rock strengths to a greater depth as required.

The seismic survey provided the engineer confidence to design an economical solution. If this survey had not been carried out the design engineer would not have the required information. He would have been forced to make assumptions which can lead to overly conservative or even worse an overly optimistic design.

4 CONCLUSION

In conclusion, the seismic survey was an efficient and economical way to test the site as a whole. When combined with standard test methods, it enabled the geotechnical engineer to gain a deeper understanding of the sub-surface profile and strength. This ultimately led to an appropriate design solution.

Soil investigations within Australia would greatly benefit by the application of seismic technology. Seismic surveying techniques offer the geotechnical engineer another tool to build the geotechnical model for the site. The depth of penetration and the relationships to sub surface material properties gives the engineer an overall view of the site. The seismic results reduce assumptions and give the engineer confidence to construct a geotechnical model for any site.

5 REFERENCES

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