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Site investigation and numerical modelling for a high bay distribution centre in Melbourne

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ABSTRACT: This paper presents an overview of the site investigations, interpretation of the testing data and the derivation and implementation of the small-strain soil stiffness model for a large warehouse distribution centre in Melbourne. The warehouse comprises a high bay facility up to 50 m high with significantly larger loads than typical warehouses. The fully automated high bay facility imposes stringent differential settlement (tilt) requirement of up to 1V:1250H. A combination of CPTs, cored boreholes and the relatively uncommon Seismic Dilatometer Test (SDMT) were performed. Top of bedrock is approximately 50 m below the existing ground surface. Numerical analysis were undertaken to assess various solutions ranging from piling to bed-rock, shallower CFA piles to raft slabs, each with vastly different financial implications and risk profiles. The tight tilt limit means that the anticipated ground deformation is very small. A small strain ground model was subsequently developed for the CFA piles solutions.

1 INTRODUCTION

This paper presents the foundation improvement design for the high bay storage facility for a regional distribution center of a major domestic retailer constructed in Melbourne South. It is located in a challenging geological environment which features deep subsurface soil profile. This in combination with high storage loads, strict floor tilt requirement and lack of precedence in terms of the scale of the warehouse creates a unique engineering challenge. The design tilt requirement for the floor slab is at least 1V:1,250H.

2 REGIONAL GEOLOGY

The geological map indicates the site is underlain by alluvial deposits of Quaternary age that is described as peaty clay, clay (mainly swamp deposits) and Tertiary unit of Baxter Sandstone formation that is described as ferruginous sandstone, sand, sandy clay, occasional gravel.

3 SITE INVESTIGATION

A series of investigation campaigns were conducted to investigate the subsurface conditions, especially in the proposed high bay area. They included drilling boreholes with standard penetration testing (SPTs) and push tubes conducted at regular intervals in soil units in selected boreholes, cone penetration testing (CPTs), and seismic dilatometer testing (SDMTs). The first four CPTs out of ten tests achieved refusal depth at about 6 m to 7 m depth, i.e., q_c of at least 40 MPa. It was discovered that the refusal was in very dense sand (up to two meters thick). The fifth CPT was able to punch through this dense layer and intercepted weaker material below. At depth, the Young's modulus derived from CPT correlations shows as low as 25 MPa. The remaining CPTs were undertaken using a dummy cone within this very dense layer to allow for further testing and assessment on the material underneath.

Six boreholes were drilled to assess the extent of the weak material and to locate the bedrock. Bedrock was encountered at a depth of up to 50 m below existing surface and was cored to assess its strength. Seismic dilatometer testing (SDMTs) were undertaken in two boreholes within the soil units. Pushing the dilatometer blade to test the ground was difficult due to lenses of very dense material. Predrilling was undertaken to allow testing of weaker materials. Based on the results of the investigation, a geotechnical model was developed for the site. Table 1 presents the summary of the inferred geotechnical units.

Table 1. Summary of subsurface units encountered

Inferred unit	Depth of each unit m	Description
1	0 – 2.3	SILT to CLAY; grey and dark brown, medium plasticity with inferred consistency between firm to very stiff.
2	2.3 – 3.3	Clayey SAND to SAND; brown, medium to coarse grained, inferred to be medium dense.
3	3.3 – 6.8	SAND; brown to light grey, medium grained, inferred to be very dense to hard. At some locations, CPTs refuse at the top of this unit.
4	6.8 – 32.3	Interbedded SAND and CLAY. Proportion of the Sand bed is up to approx. 30 %. Medium dense to very dense Sand and CLAY of inferred consistency between stiff to hard.
5	32.3 – 42.3	CLAY; inferred consistency between stiff to hard.
6	42.3 – 49.3	Clayey SAND to Silty SAND.
7	49.3	BEDROCK (GRANODIORITE); moderately weathered to fresh with inferred strength medium to high.

4 DESIGN PARAMETERS

4.1 *Engineering strain*

Using material stiffness parameters derived based on "typical" engineering strain assessed from the testing and with bedrock at depth of up to 50 m below the existing surface, it was difficult to design the high bay to achieve the required performance, i.e. floor tilt of 1V:1,250H. A range of solutions including cement-treated surficial layer, jet grouting, pile to rock and floating piles were considered.

4.2 *Small strain model*

Understanding the issue and limitation with the "typical" engineering strain model, the soil stiffness design parameters were assessed from the in-situ tests considering the soil strain levels relevant to the design solution.

Figure 1 shows the typical stiffness variation and strain ranges for different structures. For foundation applications, the shear strain is around 0.1%.

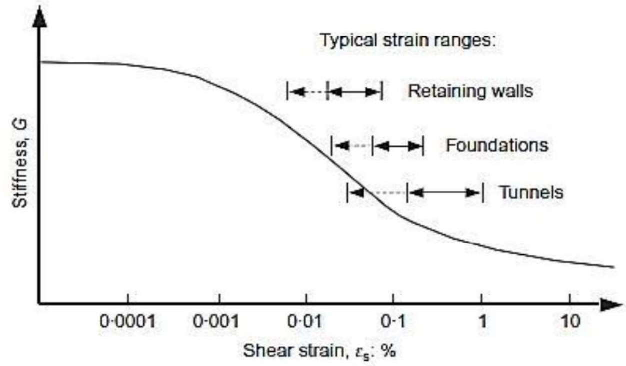

Figure 1. Typical stiffness variation and strain ranges for different structures (Clayton, 2011)

4.2.1 *Simplified small strain model*

Figure 2 shows the range of shear strains associated with DMTs in silts and clays and in sands, together with typical normalised stiffness decay curves for different soil types found in the literature. The stiffness values measured by the DMTs performed in the silts and clays (e.g., Unit 4 and 5) were considered to be representative of shear strains of approximately 0.75%.

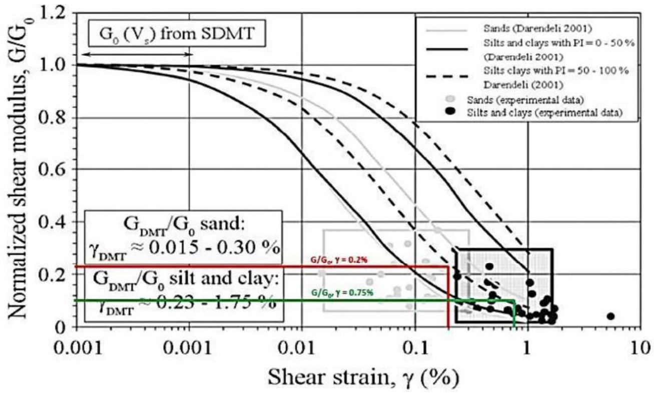

Figure 2. Typical strain levels for DMTs and normalised stiffness decay curves for various soil types (base plot from Amoroso et al., 2014)

A floating CFA pile solution with pile length of 17 m supporting cement-stabilized fine crushed rock (CSFCR) and concrete slab was assessed. The strain level in the soils under the applied pile working loads as determined from FLAC3D analyses is about 0.2%. Table 2 presents the stiffness (elastic moduli) of the inferred units corresponding to 0.2% strain and comparison with the "typical" engineering strain moduli.

Figure 3 presents the contours of the induced shear strain in the inferred soil units from one of the FLAC3D analysis based on the stiffness parameters at 0.2% strain shown in Table 2. The shear strains for the vast majority of the material are less than 0.02% in Unit 1, 2 and 3, with maximum values of approximately 0.1% to 0.2% around the pile toe level in Unit 4. Note that even the maximum shear strains are relatively small for usual engineering practice, and this is a function of the very tight specification on tilt which necessarily results in low strains. The strains reduce below the pile toe to approximately 0.05% to 0.1%.

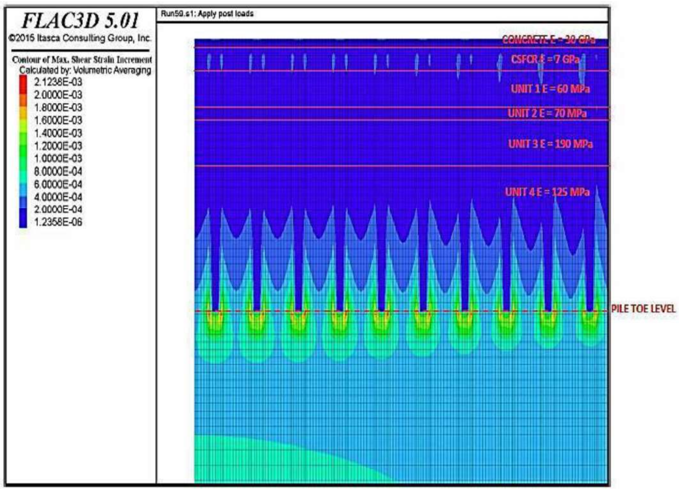

Figure 3. Contours of shear strain based on engineering strain parameters

Table 2. Engineering parameters of inferred geotechnical units

Inferred unit	Typical elastic modulus – engineering strain	Inferred elastic modulus from SDMT*
	MPa	MPa
1	20	60
2	20	70
3	80 -140	190
4	20 - 40	125
5	25	70
6	25	70
7	Fixed boundary condition	

Note: *Elastic Modulus at 0.2% shear strain

4.2.2 *Advanced small strain model*

The small strain stiffness follows a decay curve which defines the shear modulus based on the shear strain. If strain-dependent stiffness can be incorporated into the design process, higher stiffness at lower strain levels could be used. The seismic dilatometer tests provide stiffness values at very small strains (G_0), and at 'working' strain levels of approximately 0.05% to 0.1%. The stiffness values (G) measured by the DMTs performed in the silts and clays (Unit 4 and 5) were considered to be representative of shear strains of approximately 0.75%. These values help define stiffness decay curves for non-linear stiffness models. G_0 and G with the relevant testing data are presented in Figure 4 and 5, respectively. In addition to the shear modulus, curve fitting parameters are required to match the general shape of the decay curve.

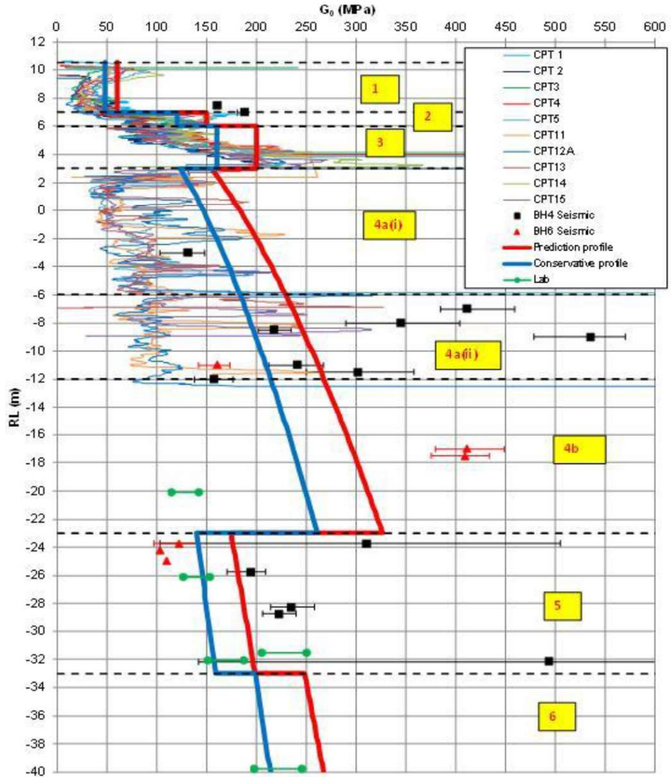

Figure 4. G_0 vs. RL for soil units encountered

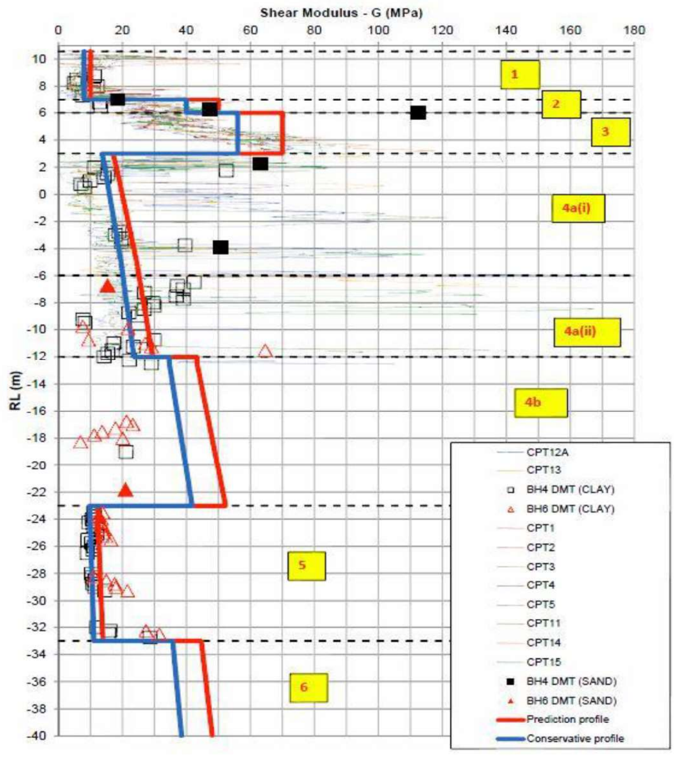

Figure 5. G vs. RL for soil units encountered

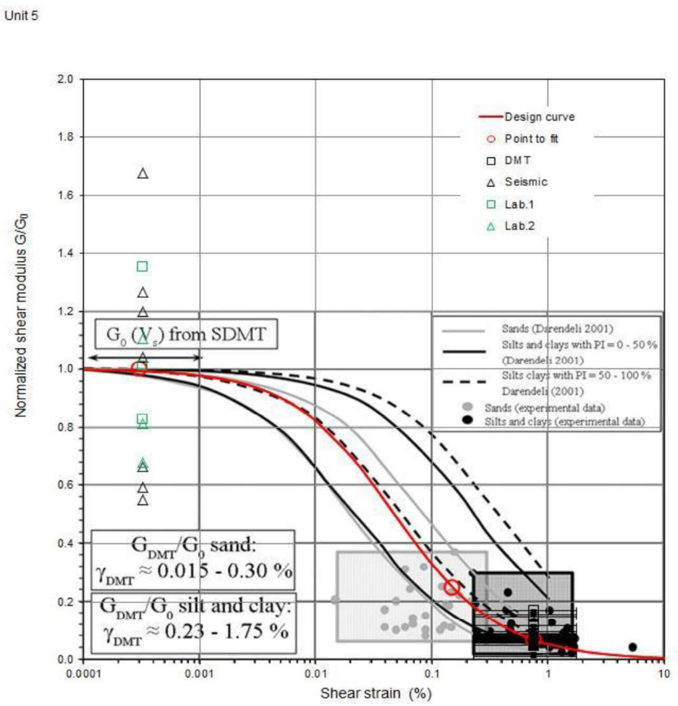

Figure 6. Small strain shear stiffness decay curve for Unit 5 (base plot from Amoroso et al., 2014)

Figure 6 shows the derived normalised shear stiffness decay curve for soil Unit 5 as an example.

Table 3 summarises the small strain stiffness design parameters for all soil units. As aforementioned, bias in the testing data towards weaker material is considered in deriving these parameters. It is noted that another set of more conservative parameters, defined as 80% of the best estimate parameters in Table 3 were also analysed as sensitivity checks.

The stiffness decay curves are set up for each soil unit and they are incorporated into the numerical modelling as described in Section 5.

Table 3. Summary of small strain stiffness parameters

Unit	Thick-ness	G_0	G*	Stiffness curve fitting parameters		
	m	MPa	MPa	a	r	t_y
1	3.55	60	10	6.162	-0.599	0.003
2	1	150	50	7.864	-0.789	0.002
3	3	200	70	6.713	-0.845	0.003
4a(i)	9	155 - 229	17 - 25	6.223	-0.793	0.003
4a(ii)	6	229 - 267	25 - 29	6.223	-0.793	0.003
4b	11	267 - 326	43 - 52	6.734	-0.696	0.002
5	10	174 - 198	12 - 14	5.490	-0.904	0.002
6	7	248 - 267	45 - 48	5.890	-0.677	0.001

Note: *0.1% for sand; 0.75% for clay

5 NUMERICAL MODELLING AND DESIGN OUTCOMES

5.1 *General*

Figure 7 shows the extent of the high bay area in the entire warehouse. It spans over 97 m east-west and 118 m north-south.

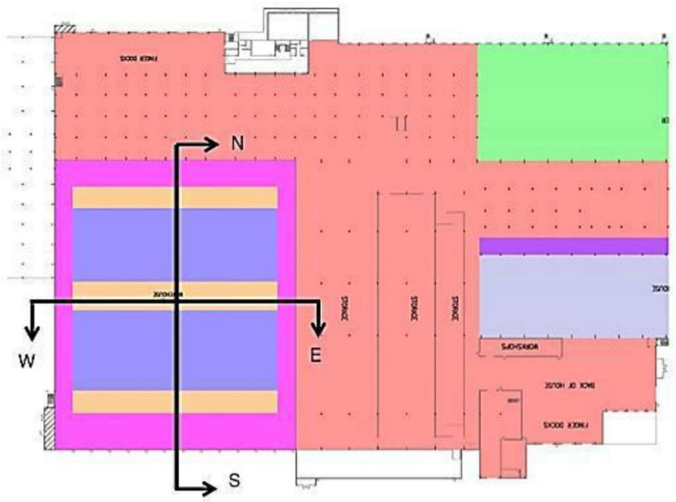

Figure 7. locality plan of the high bay storage area

A 3D strip representing half of the total slab dimension in the north-south direction and one aisle width (7.44m) was modelled using the computer software package FLAC3D by Itasca. Individual concrete columns spaced at 3.02m were included in the model. Figure 8 and 9 present the elevation view and 3D view of the model, respectively. Across the aisle direction and the corners of the high bay area was modelled and examined separately.

A range of vertical dead loads and live loads from 60 kN up to 385 kN were applied as post loads on the slab rather than uniformly distributed load, and pattern loading where different extent of the pallets are loaded were also assessed.

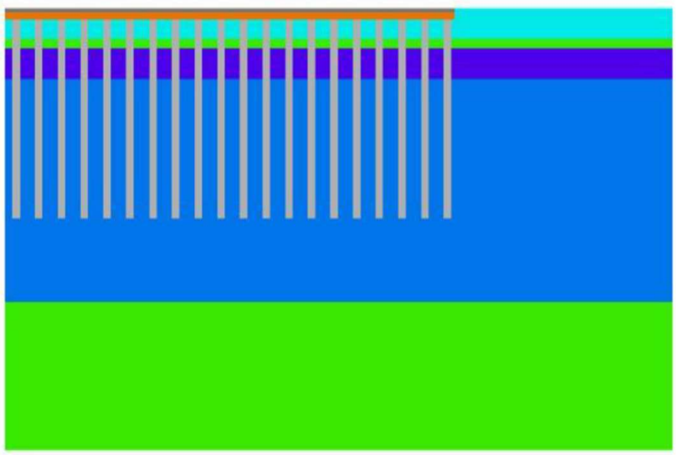

Figure 8 Elevation view of the FLAC3D model

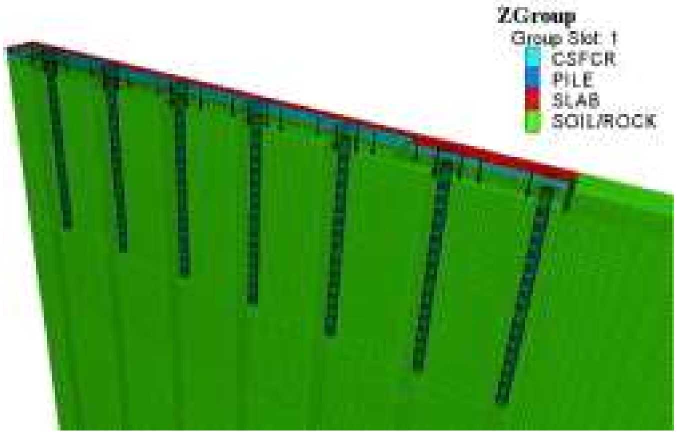

Figure 9 3D view of the FLAC3D model

5.2 *Foundation design details*

As previously described, the foundation design comprises a concrete slab (E=30GPa) between 500 mm thick to 800 mm thick overlying a layer of CSFCR that is between 500 mm to 800 mm thick supported by 750 mm diameter continuous flight auger (CFA) piles. A Young's modulus of 7 GPa is assumed for the CSFCR. A minimum interface shear strength between layers of CSFCR and tensile strength are to be no less than 0.15 MPa. These values relate to properties at 28 days after placement. The piles comprise 750 mm diameter concrete installed using continuous fight auger (CFA) techniques. An overall pile length of 17 m is adopted. The CFA piles are assumed to a Young's modulus of 30 GPa and a tensile strength of at least 2 MPa.

5.3 *Implementation of advanced small strain stiffness model*

FLAC3D does not have built-in function to cater for small strain stiffness model. However it does provide the tool of FISH code for advanced users to incorporate their own coding.

In this project, a module for small strain stiffness which includes decay curves for each geotechnical unit was developed. This is based on the tangent shear stiffness rather than the secant stiffness. The general procedure is that the elements are grouped into units as per Table 2. After stress initialization, each element is assigned an initial stiffness value of G_0, the maximum shear stiffness at small strain, which is a function of the initial stress (Fig. 4). Then post loads are applied in accordance with the layout of the posts. At every iteration, the shear modulus for each element is re-calculated based on the strain results and its decay curve fitting parameters. These moduli are then fed back into the calculation for stress and strain to be re-calculated. This process continues until the difference between calculated strains from two consecutive iterations is below a specified tolerance level. At this point, the results are adopted as final for this particular load step. The calculation then enters the next load step.

5.4 *Results*

Analyses with both "simplified" small strain and "advanced" small strain design parameters were completed. Figure 10 and 11 present the settlement and tilt results for soil design parameters based on the two sets of small strain design parameters for the same loading pattern. The former can achieve a maximum tilt of less than 1:1250 while the latter could achieve a maximum tilt of 1:2000. The effect of the post loads can be seen from the fluctuation in the local floor tilt, i.e., waviness in the tilt curve.

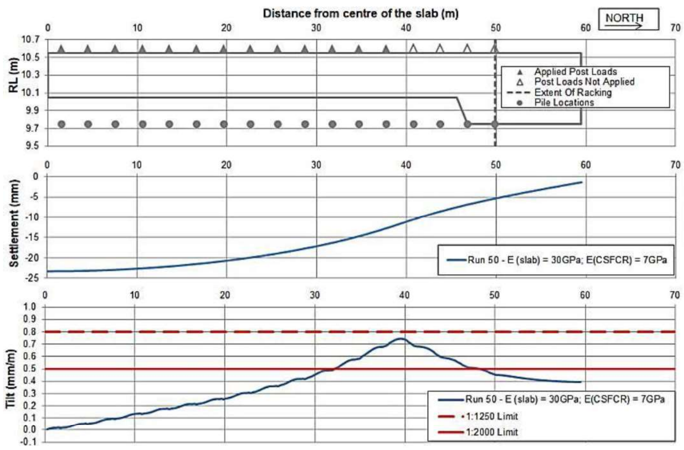

Figure 10. Settlement and tilt results based on simplified small strain parameters

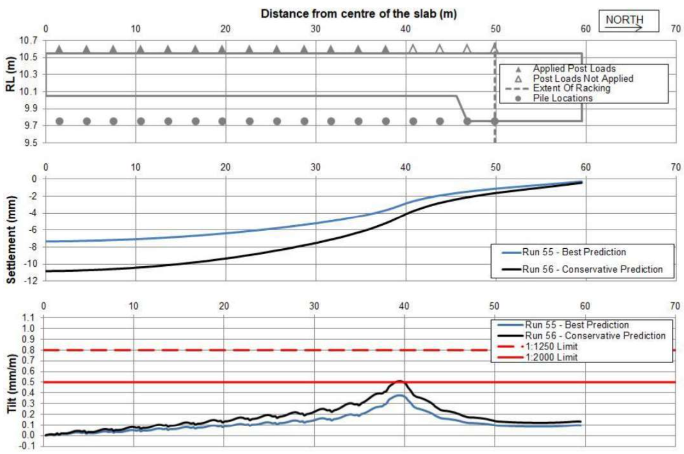

Figure 11. Settlement and tilt results based on advanced small strain parameters

In addition to full storage loading case, a variety of loading patterns were also examined. It was found that the maximum tilt is always associated with large change in the loads. This effect is more pronounced near the edge of the slab, i.e., outer racks.

5 CONCLUSION

The design solution for the warehouse development involves construction of concrete slab overlying cement stabilised fine crushed rock and a grid of CFA concrete piles.

We have completed a sophisticated site investigation involving seismic dilatometer testing (SDMT) which unlike other more common field testing, i.e. standard penetration tests (SPT) or cone penetration tests (CPT), directly measured the stiffness of the soil. We have used the test results to develop engineering strain and small strain stiffness of the soil units. Given that small strain stiffness model is not available in most geotechnical software package, script was written in FISH code to implement it into FLAC3D. The analyses indicate a maximum tilt of 1V in 1,250H could be achieved with modulus assessed at a small shear strain of 0.2%. With the implementation of small strain stiffness decay curves, a maximum tilt of 1V in 2,000H could be achieved.

7 ACKNOWLEDGEMENT

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REFERENCES

Amoroso, S., Monaco, P., Lehane, B.M. ad Marchetti, D., Examination of the potential of the seismic dilatometer (SDMT) to estimate in situ stiffness decay curves in various soil types. *Soils and Rocks*, Sao Paulo, 37(3): 177-194, September-December, 2014.

Clayton, C.R.I., 2011, Stiffness at small strain: re-search and practice, *Geotechnique* 61, No. 1, 5-37.