Geotechnical and Geophysical Site Characterisation 5 – Lehane, Acosta-Martínez & Kelly (Eds)

CPT, DMT and MASW allowing economic design of a large residential project over soft soils

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ABSTRACT: A three hectare medium density residential development in Melbourne located within the Yarra Delta overlies a complex sequence of compressible Quaternary sediments covered with 1-3 m of uncontrolled contaminated fill with a near surface water table. The development consisting of nearly three hundred triple storey townhouses required site levels to be raised by up to 1.5 m. Deep piled footings were an option but, due to significant cost savings, a high level footing option was pursued. A detailed geotechnical investigation, with Cone Penetration Tests (CPT) and Marchetti Dilatometer Tests (DMT), was conducted to characterise the sediments, which allowed a detailed finite element settlement analysis to be conducted. Ground improvement with impact compaction and preloading was conducted to reduce modelled differential settlements to acceptable levels for high level footing construction. The ground improvement verification was conducted with a combination of CPT and Multi-channel Analysis of Surface Waves (MASW). Design of preloading also required the use of dissipation tests to estimate the coefficient of consolidation, which was verified with a test preload pad and later confirmed with a preload monitoring program. This case history demonstrates how a combination of high quality in-situ tests (CPT, DMT and MASW) can provide high quality data at a sufficient frequency and economic cost for assessment of high level footings on a challenging soft soil site. The savings by adopting high level footings over deep footings is estimated to be in the order of $10M.

1 INTRODUCTION

1.1 *The development*

This case study details geotechnical investigation and monitoring during construction conducted for a 3.5 hectare medium density residential development in Melbourne located within the Yarra Delta Quaternary sediments. The site, previously a single large industrial premises, was developed with nearly three hundred triple storey townhouses with roof top terraces. The local authority required site levels to be raised by 1-1.5 m to protect the development from flooding.

1.2 *Geotechnical challenges*

Based on local experience and information from the client, the site presented significant geotechnical challenges. The water table was within 0.5 m of the ground surface. During demolition the existing fill was disturbed, up to a depth of 1.5 m, by the removal of footings, which introduced difficulties in recompaction of the fill. Below the fill, thick compressible Yarra Delta sediments were expected to lead to large load-induced settlements. Deep footings for industrial structures that had been left in the ground after building demolition were expected to exacerbate differential settlements.

2 PRELIMINARY GEOTECHNICAL ASSESSMENT

A preliminary geotechnical assessment of the site with nine Cone Penetration Tests with pore pressure measurement at u2 (CPTu) was conducted. The CPTs encountered a sequence of fill, Unnamed Recent Alluvium (URA), Port Melbourne Sand (PMS), Coode Island Silt (CIS) and Fishermens Bend Silt (FBS). Approximate thicknesses of the units encountered are as follows.

Table 1. Sub-surface conditions encountered

Unit	Description	Thickness (metres)
Fill	Loose sand and gravel, no clay	2.4 – 3.0
URA	Soft to firm silty clay	0.0 – 4.0
PMS	Loose to dense sand	0.0 – 5.5
CIS	Soft to firm silty clay	8.0 – 13.0
FBS	Stiff to very stiff clay	1.5 – 9.0

Below the FBS, around 10 m, or so, of Moray Street Gravel was expected based on local geological maps. This unit was not fully penetrated by the CPT due to refusal of the cone.

Deep piled footings were considered, with piles founded at a depth of 25-30 m. Differential settlements between the rigid structure and the surrounding ground would be controlled by ground improvement by impact compaction of the fill and preloading to reduce settlements in the deep compressible sediments.

High level footings were also considered, with stiffened slab footings for the buildings, after impact compaction of the fill and preloading of the deep compressible sediments over the building footprints, where necessary, subject to a detailed assessment of differential settlements.

The high level footing option presented significant cost savings when compared to the deep footing option, and was the preferred option. However, pursuing this option necessitated a much more detailed geotechnical assessment to ensure the project could be constructed and maintain the required limit of post-construction differential settlement gradients of 1/500 over a 50-year period.

3 DETAILED GEOTECHNICAL ASSESSMENT

3.1 *Geotechnical investigation design*

The aim of the detailed geotechnical investigation was to measure the sub-surface variation under each building block sufficiently to allow prediction of post-construction differential settlements. It was known from the preliminary geotechnical assessment that the depth and consistency of the compressible sediments varied significantly so an intensive grid of testing would be required. CPTu was the obvious choice for the site as it provides continuous measurement of the sub-surface profile, is quick, economic, and provides more accurate information than boreholes with standard penetration tests. On average, the proposed testing frequency was one CPTu every 400 m^2 and one Marchetti Dilatometer Test (DMT) every 2000 m^2, targeted within the building footprints.

Recognising that preloading may be required, the coefficient of consolidation (c_v) needed to be estimated in order to determine the likely time required for preloading. To obtain this, a series of rapid-dissipation profiles (RDP) were performed, where CPT dissipations were run for 10 minutes each, every 0.5 m depth within the weak alluvium. Most dissipations reached t_{50} within 10 minutes and for those that did not, t_{50} was estimated by extrapolation.

The presence of old deep footings was investigated via a desktop study of historical aerial photographs, discussions with the demolition contractor, and a search through old construction drawings found on site. A preload test pad was required to verify calculated settlements and rate of consolidation.

3.2 *Geotechnical interpretation*

The CPT results provided excellent information on the stratigraphy of the site, which is complex. They showed the site spans a geological boundary of the PMS (Figure 1) that is present over two thirds of the site and overlies the CIS (Figure 2) that is present over the entire site. Above these units is a varying thickness of URA (Figure 3). The presence of the URA within the Yarra Delta is discussed by Neilson (Neilson 1996).

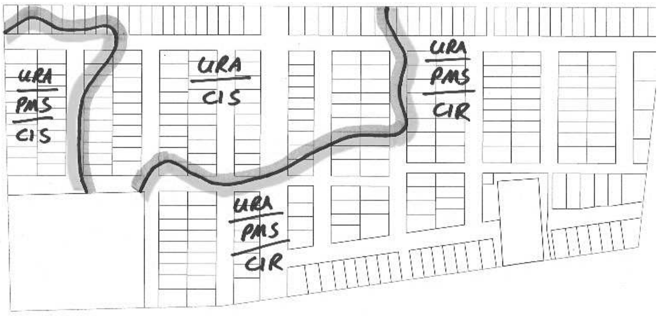

Figure 1. Geological boundary of Port Melbourne Sand. Note the absence of this unit towards the top the figure.

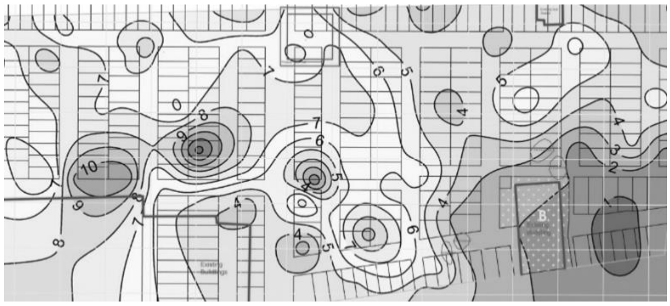

Figure 2. Thickness (m) of CIS with CPT qt < 1 MPa

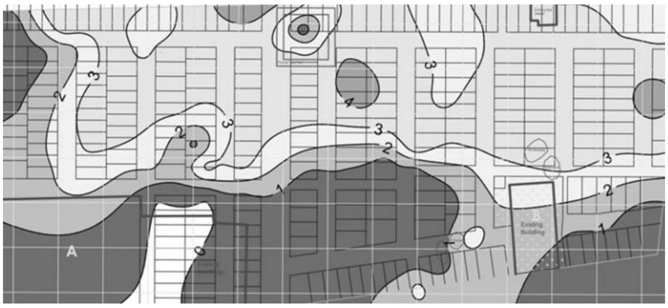

Figure 3. Thickness (m) of Unnamed Recent Alluvium

The CIS and URA are slightly overconsolidated, compressible, silty clays. The PMS is relatively incompressible with the modulus highly dependent on the thickness of this unit.

The most important parameter to consider for the project was the stress-strain moduli of the soil. CPTs allow the estimation of constrained modulus in fine-grained soil by the empirical relationship M = $\alpha_m$qt, where M = constrained modulus; qt = corrected cone tip resistance; and α_m is a factor between 1 and 8 which depends on soil type and qc, the cone tip resistance (after Lunne, et al. 1997). Given the wide range of α_m, the estimation of constrained modulus is not well defined and to provide a better measurement Marchetti Dilatometer Tests (DMTs) were conducted, at a lower frequency of one every 2200 m^2. DMTs provide a more direct measurement of modulus by the relationships $M_{DMT} = R_M E_D$, where E_D = dilatometer modulus. R_M is a factor based on K_D and I_D where I_D = material index (soil type) and K_D = Horizontal stress index, which can be considered as K_0 amplified by the penetration of the DMT blade (Marchetti, 2001). The vertical drained constrained modulus obtained from DMT is considered to be superior to laboratory testing for the following reasons. As it is an in-situ test it inherently accounts for effective stress. Sample disturbance for oedometer or triaxial testing leads to the underestimation of soil moduli (Bowles, 1997, Marchetti, 2001). For use in a 2D Finite Element Analysis, M is converted to drained Young's modulus, E', via drained Poisson's ratio, ν', in Equation 1, below.

$$E' = \frac{(1+\nu)(1-2\nu)}{(1-\nu)} M \qquad (1)$$

Wroth (1975, cited in Kulhawy and Mayne, 1990) present data for ν' versus plasticity index for "several lightly overconsolidated soils", which results in an ν' of about 0.3 for the CIS and URA. Subsequent analysis of the preload test pad (see Section 3.4) indicated that using the constrained modulus directly in the 2D FEA was conservative. Simplified 1D settlement calculations using M gave very similar results to the 2D FEA using the same moduli and a ν' of 0.3. For this reason, the use of constrained modulus without conversion to Young's modulus was adopted for settlement analysis. Using a relatively small number of DMTs (15) allowed the determination of appropriate α_m values and therefore M from the CPT data across the different soil types at the site without sacrificing the efficiency of the testing program.

3.3 *Settlement analyses*

2-D finite element analyses (FEA) were performed on critical sections of the site to determine the likely total and differential settlements. Using moduli obtained from the DMT and from CPT correlated to the DMT results, the maximum post-construction primary settlement (immediate and consolidation) was about 110 mm.

The CIS is reported to undergo significant secondary consolidation, and in relatively thick deposits with high applied stresses this consolidation can become linear with time, at 5 mm to 10 mm per year for sites with 15 m to 20 m of CIS (Neilson, 1996, Srithar, 2010, and Ervin, 1992). It appears the behavior of the secondary consolidation trend is dependent on the applied stress. Donald (1976, cited in Ervin, 1992) suggests at stress levels below the preconsolidation pressure, secondary consolidation decreases linearly with log time, and presents data for the coefficient of secondary compression C_α, versus the log of the ratio of applied stress to effective stress, which shows a general trend of C_α increasing with an increase in the applied stress ratio. At log stress ratios greater than 1.0, the data becomes very scattered, but below this the relationship appears reasonable. The mechanisms behind a constant rate of secondary consolidation are not clear, and Srithar (2010) points to previous construction activity and/or groundwater drawdowns as a potential contributor to the observed settlements. For this development the stress ratio was between 0.3 and 0.5, and a stress-dependent approach was adopted. A C_α of 0.005 and 0.001 was adopted for the URA and CIS, respectively, which lead to a predicted total secondary consolidation of about 30 mm over 50 years.

Without preloading the site, the maximum total settlement including immediate, consolidation, and secondary consolidation computed in the FEA was 140 mm. Although large in total, the differential settlement gradient remained within the limit of 1/500.

The modelling of buried deep footings increased the differential settlements greatly and led to the conclusion that high level footings would not be appropriate without preloading of the building footprints, where deep footings were present. The required surcharge, determined by the FEA, was 7.5 kPa. This included an allowance for recompression after preload removal of 8% of the predicted primary consolidation.

3.4 *Preload test pad*

A preload test pad was constructed in part of the site where there was no PMS and where there was thought to be no deep footings. The preload test pad was about 3 m high over a 20 m by 20 m square area with 1H:1V batters and imposed a 50 kPa surcharge on pre demolition levels.

Four settlement plates and four Vibrating Wire Piezometers (VWP) were installed before the test pad was constructed. A CPTu, DMT and RDP were also conducted at the centre of the test pad location.

Over a period of one month the settlement plates recorded about 30 mm of settlement. However, the full settlement was not measured as the preload test pad was constructed over three days and the first survey base reading was only measured on the fifth day.

Based on the piezometer/dissipation data, it is estimated that a total primary settlement of the test pad was about 50-60 mm, which is about one third of that predicted in the FEA. Although not identified initially, a subsequent desktop study identified buried deep piles along two sides of the pad. The unexpectly low settlement is likely due to the piles taking a large proportion of the applied stress. Another possibility is historical preloading of the area. Not knowing the exact historical footing and loading details it was not possible to verify the predicted settlements from the geotechnical investigation for the site. However, by comparing the response of the VWPs to the preload test pad, it was possible to verify the predicted time of consolidation, discussed in the following section.

3.5 *Consolidation time analysis*

From the preliminary geotechnical assessment, time estimates to reach 90% consolidation (t_{90}), varied widely from 6 months to 20 years. The greatest uncertainty was the length of drainage paths and the RDP test was designed to help provide this information and coefficients of consolidation. The preload test pad was also conducted to provide a field measurement of the consolidation time.

The RDP test results are shown in Figure 4 with horizontal coefficient of consolidation (c_h) estimated using the method recommended by Lunne et al. (1997). The figure shows four RDP profiles that were conducted within the area of the site where there is URA over CIS and no PMS.

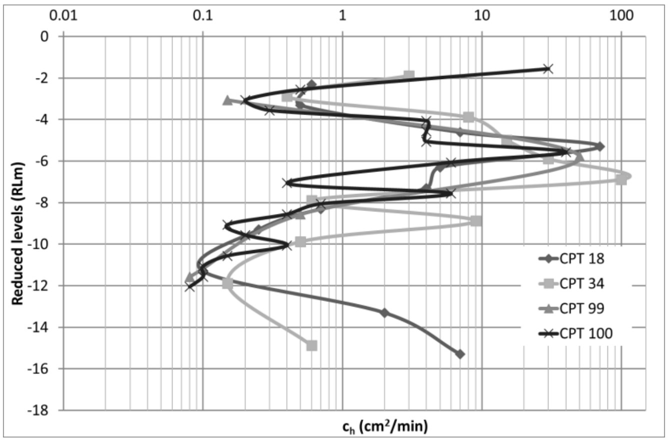

Figure 4. c_h estimates from RDP tests

Based on the lower bound values of c_h estimated from the RDP results a t_{90} of 2 years was adopted, prior to the results of the preload test pad becoming available. The VWP data from the preload test pad initially indicated t_{90} would occur after about 3 months, however, given the presence of buried piles t_{90} is likely to be somewhat longer, but still shorter than the estimated 2 years. The interpretation of the RDP was re-analysed in light of the preload test pad results. A large uncertainty was the ratio of c_h to c_v. As c_v is required for calculation of the consolidation time, a conversion from the RDP interpreted c_h to c_v is required. To convert the c_h to c_v, an estimate of the ratio of permeabilities (k_h/k_v) is required. Considering the formation of the URA/CIS and the CPT/RDP results, there are many sand, clayey sand and sandy clay layers that may be interconnected. If these layers are horizontal and are not connected then the k_h/k_v ratio could be high. However, if the layers are interconnected vertically the k_h/k_v ratio may be close to unity. Day and Woods (2007) suggest k_h/k_v ratio of CIS of 3-5. The initial interpretation, prior to the preload test pad results, used a c_h/c_v ratio of 4 being the mean ratio used by Day and Woods. In light of the preload test pad results and the above mentioned knowledge of the regular high permeability layers, a c_h/c_v ratio of 2 was used, which provided a t_{90} of 9 months for the lower-bound values of c_v. Even considering the presence of some deep footings under the preload test pad, the new consolidation time estimate was considered reasonable and was used for design of the preloading.

4 GEOTECHNICAL RECOMMENDATIONS

4.1 *Proposed ground improvement and monitoring*

The results of the geotechnical investigation showed that the high level footing option is feasible subject to the following recommendations. The demolition fill and loose PMS should be improved by High Energy Impact Compaction (HEIC). Preloading is required on building footprints over about three quarters of the site that may have deep footings present.

4.2 *Ground improvement by High Energy Impact Compaction (HEIC)*

Due to varying site surface levels, the groundwater (at about RL 0.3 m) was within 0.5 m to 1.5 m of the site surface at the time of ground improvement.

Trials were conducted in the lowest part of the site (groundwater within 0.5 m) to check the effectiveness of the impact compaction on saturated loose demolition fill and sand. Initial coverages of the HEIC caused water to flow rapidly from the surface at two locations, producing several sand boils and a large ponded area. The impact response of the ground was very low with significant heaving and mattressing. After resting the ground for 24 hours, the water had disappeared and further impact compaction showed a high response initially before water began to flow again and the impact response became low. This sequence of HEIC and resting was continued for several days and noticeable improvement of the impact response was evident, both visually and from subsequent test results.

The trials showed HEIC adequately improved the demolition fill and PMS to a depth of 3-4 m. The remainder of the site was successfully HEIC and the increase in stiffness over the site was verified with before and after CPTs and Multi-channel Analysis of Surface Waves (MASW).

The before and after CPTs showed a significant increase in the qc of the demolition fill and the PMS, and, as expected, no increase in the stiffness of the soft clay (URA). Two example before and after CPT comparisons are shown in Figure 5.

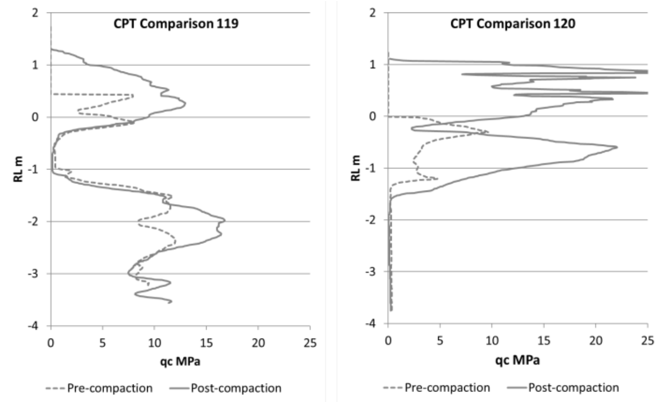

Figure 5. Before and after CPT cone tip resistances

The before and after MASW showed an increase in the shear wave velocity over the top 3-4 m of 20-75 m/s. A target shear wave velocity of 125 m/s was reached and confirmed with the MASW survey. An example before and after MASW comparison, with shear wave velocity contours in m/s, is shown in Figure 6.

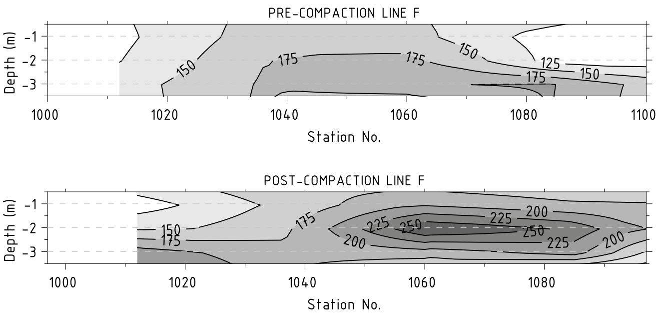

Figure 6. Before and after MASW shear wave velocities

In addition to the CPT and MASW testing, the Continuous Impact Response (CIR) and Continuous Induced Settlement (CIS) data recorded by the impact compactor was analysed. The CIS surveys use differential GPS measurements taken at each drop of the impact roller to produce a detailed level survey of the site. CIS surveys during and after HEIC give an indication of reducing site levels vs HEIC coverages, which helped determine when HEIC was complete. The results of the CIS showed an average settlement over the site of about 90 mm.

The CIR system measures the deceleration of the drums at each drop of the impact roller. The results of the CIR were used to assess potential soft spots and to locate post compaction CPTs in low impact response areas. All of the CIR surveys showed a significant improvement of the soil dynamic response, which indicates the ground improvement was very effective.

Two wet clay soft spots were identified and remediated by removal and replacement with drier site won fill subsequently improved by HEIC.

HEIC was conducted successfully at the site using a combination of CPT, MASW, CIS and CIR results to confirm the required stiffness had been achieved in the demolition fill and PMS over the proposed development area.

4.3 *Preloading building footprints*

A preload height of 1-1.3 m (about 20 kPa) over three months was recommended. The earthworks contractor had enough material on site to increase the surcharge further, typically about 2.5 m high, which increased the surcharge to about 45 kPa, reducing predicted times to reach required settlements to less than one month.

The preloading was conducted in stages to allow placement of engineered fill up to design levels to proceed at the same time. The different stress distributions caused by partial preloading compared to the full preloading were assessed in the FEA model and were found to have negligible effect provided whole blocks of building footprints were preloaded.

The monitoring was conducted with multiple settlement plates within each preloaded building footprint and four sets of VWPs installed at the centre of four preload pads across the site. The VWPs were installed at various depths, targeting low permeability compressible clays, mostly over the top 12 m and one installed at 22 m in the FBS. The VWPs were installed using a CPT rig with sacrificial tips, which was very quick and efficient, allowing eight VWPs to be installed per day. The alternative of installation by boreholes would have required casing and would have been much more time consuming and costly.

At the time of writing this paper three of the four VWPs sets had experienced the preloading.

The response of the VWPs was generally good with a measured increase in pore water pressure (PWP) of 50-100% of the modelled preload induced stress at a depth <5 m and 20-50% at a depth >5 m. The missing PWP response is thought to be due to the limit of the VWP sensitivity to rapid changes in PWP conditions during the construction of the preload.

The VWP measured PWP dissipations show there is, at least, a two stage rate of consolidation (an initial fast stage and following slow stage). This was observed when the expected logarithmic response was compared to the PWP dissipation measurements. The two stage response was also observed in the preload test pad measurements. It is thought the two stage rate of consolidation response is caused by the compressible clays being slightly overconsolidated, with the initial fast stage consolidating along the recompression curve, and the second slow stage, in the normally consolidated range stress range, consolidating along the virgin compression curve. Provided the preload induced stress covers the overconsolidated and normally consolidated range of stress, this would explain the observed measurements.

The Over Consolidation Ratio (OCR) of the compressible clay was estimated from CPT data using the following equation recommended by Lunne, et al. (1997).

$$OCR = k \frac{qt - \sigma_{v0}}{\sigma'_{v0}} \quad (2)$$

where k = 0.2 – 0.6, qt = corrected cone tip resistance. In this instance a k of 0.3 was chosen, and the resulting OCR was between 1.5 and 2.5 (generally closer to 2.0), which agreed generally with the DMT derived OCRs.

The interpreted OCR indicates that the stress range of the preload covers the overconsolidated and normally consolidated range.

The detailed results of the VWPs vary as expected with different c_v and drainage path lengths. The four preload pads with VWP measurements show that after one month on average 50% consolidation was reached on two preload pads and 80% consolidation was reached on the other two preload pads. Considering the two rates of consolidation, this extrapolates to 14 months and 4 months for t_{90}. The variation in the rate of consolidation times may be explained by the presence of deep footings under the preload pads with a lower predicted t_{90}. One of these pads is the preload test pad, which is known to have some deep footing present. Deep footings under a preload pad will attract stress reducing the imposed stress on the soil. Less stress will result in less consolidation and if the effect of the piles is not considered the percentage consolidation, based on remaining excess pore pressures, will be overestimated. One of the pads with 50% consolidation after one month was in an area known to be without deep footings. Therefore, the rate of consolidation of the compressible clay is more likely to be 50% over one month and 90% over 14 months. It should be noted that this assessment of t_{90} is based on a limited number of point tests (VWPs) which may not represent the average degree of pore pressure dissipation within a particular layer. This shows the initial predicted t_{90} of 9 months was shorter but close to that measured. It should also be noted that with larger preload surcharge, a greater proportion of the applied stress will be within the normally consolidated range which will increase the apparent t_{90}.

The measured settlements were typically about 50% of the predicted settlements. A lot of the settlement is likely to have occurred during and just after construction, which was missed by the timing of the first survey reading. Tracing logarithmic curves back from the measured data suggests settlements to within 20 mm of the predicted settlements.

It any case, the recorded settlement was in excess of that required for the proposed building loads and so the questionable extrapolated data did not have to be relied upon.

The majority of the preload pads required a one month period to achieve the required settlement.

5 CONCLUSIONS

The site characterisation was successfully conducted by high quality in-situ testing techniques (CPT, DMT and RDP), without the use of more conventional boreholes and laboratory testing, and they provided more relevant data for significantly lower fees.

Interpretation and analysis of the in-situ test data allowed the use of economic stiffened slab footings instead of the more expensive option of deep footings. The stiffened slab option is estimated to have reduced projects costs by about $10M.

Ground improvement monitoring with various in-situ tests (CPT, MASW, VWP and settlement plates) allowed the confirmation of geotechnical design and the efficient progress of ground improvement alongside construction earthworks.

In summary, there are significant project cost savings by using the above mentioned in-situ testing and monitoring techniques, making development of inner city areas on difficult ground more viable. This type of development is likely to become more common as inner city industrial sites are rezoned as residential.

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