13th Australia New Zealand Conference on Geomechanics – Acosta-Martínez & Lehane (Eds.)
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Application of the pressuremeter test in ground improvement

B. Hamidi
Menard Oceania, Sydney, Australia

ABSTRACT: The pressuremeter test (PMT) can be carried out in almost any kind of ground, and unlike most field tests that relate to bearing capacity or ground deformations through correlations, the PMT provides shear and deformation parameters that can be directly utilised for bearing and settlement calculations. These advantages make the PMT the ideal tool for verifying ground improvement works and designing both shallow and deep foundations. This paper will briefly describe the pressuremeter test and the equations that are used for determining the bearing capacities and settlements of foundations. A case study for ground treatment of a warehouse project will also be presented. The ground consisted of up to 10 m of highly heterogenous loose fill, and dynamic compaction was used to improve the soil parameters and to satisfy the project requirements. PMTs, standard penetration tests, plate load tests and dynamic cone penetration tests were used to verify the treatment.

1 INTRODUCTION

1.1 *Geotechnical testing and ground improvement*

The intent of ground improvement is to enhance the performance of the ground to an extent that will enable it to satisfy the project's geotechnical requirements. This means that firstly there must be a proper comprehension of the ground conditions and secondly there must be a criterion or criteria that are initially not satisfied but will be met after treatment.

Ground improvement has a very profound relationship with geotechnical testing. Every proper ground improvement project begins and ends with geotechnical testing. It is through geotechnical testing that the need to perform ground improvement can be realised, the type and magnitude of treatment can be worked out and accomplishment of the desired results could be verified.

Whilst there are many geotechnical testing methods that are applicable for ground improvement means, not every test is suited for all instances. The author has come across many projects that had specified testing methods that were, if not impossible, extremely difficult to perform, provided irrelevant or low-value information or were highly dependent on correlations.

There are numerous in-situ geotechnical testing methods that can be carried out and, if relevant, should yield results that demonstrate that ground improvement works have been carried out to the satisfaction of the design requirements. Testing should be carried out to depths that enable the geotechnical engineer to input sufficient data into the calculation and analysis processes to capture ground behaviour with sufficient accuracy. The frequency of testing should be to the extent that ground behaviour could be explained with sufficient accuracy throughout the treatment zone. However, the testing regime should not turn into a critical path activity that governs the project's schedule. Testing is not a purpose, it is a quality control device to verify that ground improvement works satisfy acceptance criteria.

Popular geotechnical field tests that are globally utilized are the Standard Penetration Test (SPT), the Cone Penetration Test (CPT) and the Pressuremeter Test (PMT), but other types of tests such as the Dilatometer Test (DMT) are also commonly used in certain regions and countries (Varaksin and Hamidi, 2018). It is the author's experience that among these testing methods, the PMT has proven to be a widely applicable, very reliable and a practical tool for verification of ground improvement works.

2 THE PRESSUREMETER TEST

Louis Menard developed the pressuremeter as the dissertation of his bachelor's degree in civil engineering and filed for its patent in 1954. He later improved his invention and carried out the first tests with the new probe while studying for a master's degree under the supervision of Peck and filed for a second patent in 1959 (Hamidi, 2014).

The PMT is an in-situ geotechnical test. It is performed on the borehole wall using a laterally expanding cylindrical probe that produces a stress-strain curve, which allows the evaluation of both the deformation and failure properties of the ground in a single test by measuring the pressuremeter modulus, creep pressure and limit pressure. The test is well described in detail in previously published literature; e.g. Baguelin et al (1978) and Clarke (1994). The general layout of the PMT is schematically shown in Figure 1.

Contrary to SPT, CPT and DMT that are advanced into the ground by hammering or pushing, the PMT is performed in a borehole. Hence, the test can be done in almost any kind of ground, from soft soils to rock.

Also, unlike most field tests that relate to bearing capacity or ground deformations through correlations that are occasionally over-extended to non-similar ground conditions, the PMT yields shear failure and ground deformation parameters that directly result from the test.

Bearing capacity is classically calculated based on Mohr-Coulomb failure criterion and Prandtl type factors (1920), but the PMT relates the ground's limit pressure, P_{LM}, to its bearing capacity without introducing cohesion and friction parameters. The Menard modulus, E_M, can be calculated as a function of a cylindrical cavity's volumetric change with pressure. These significant advantages, which make the PMT a more reliable tool for estimating the ground behaviour not only allow this device to be the ideal tool for designing shallow and deep foundations but have also resulted in a highly appreciated apparatus for implementation in ground improvement works.

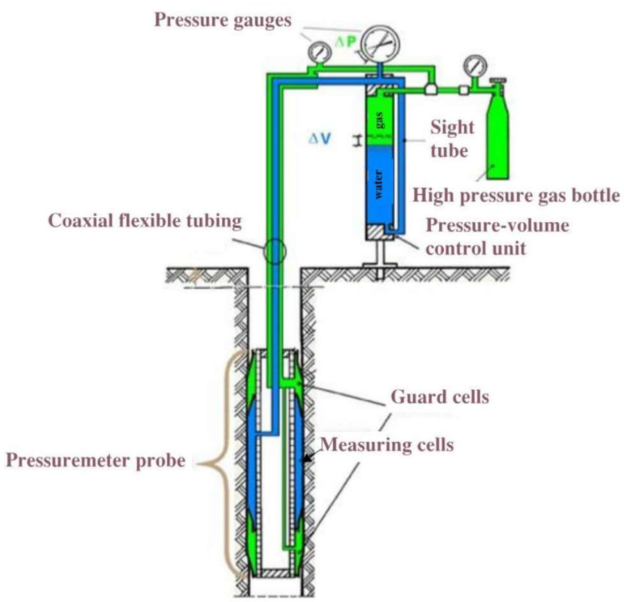

Figure 1. The general layout of the Menard pressuremter test (Varaksin and Hamidi, 2018)

Ultimate bearing capacity, q_u, of a foundation can be calculated using Equation 1 (Menard, 1975):

$$q_u = q_o + k(P_{LM} - P_o) \tag{1}$$

q_o= total overburden pressure at the periphery of the foundation level after construction.

k= a bearing factor varying from 0.8 to 9 according to the embedment and the shape of the foundation level after construction. This factor can be obtained from charts that have been prepared by Menard (1975).

P_o= total at rest horizontal earth pressure at the test level (at the time of the test).

The difference between P_{LM} and P_o is defined as net limit pressure, P^*_{LM}. When the footing is on ground with variable strengths at different depths, the equivalent limit pressure, P^*_{LMe}, is defined as the geometric mean of P^*_{LM} values obtained near the level of the foundation:

$$P^*_{LMe} = \sqrt[3]{P^*_{LM1} P^*_{LM2} P^*_{LM3}} \tag{2}$$

P^*_{LM1}, P^*_{LM2}, and P^*_{LM3}= geometric mean of values measured respectively in levels +1.5B to +0.5B above the founding level, +0.5B to -0.5B above and below founding level, and -0.5B to -1.5B below founding level, where B= width of footing.

Baguelin et al (1978) have further detailed the various scenarios for the calculation of bearing capacity.

According to Menard and Rousseau (1962) and Menard (1975), foundation settlement due to external loads is the result of two completely different phenomena; i.e. volumetric compression and shear deformation. The first phenomenon is caused by the spherical component of the stress tensor. The increase of bulk pressure (volumetric compression) causes a reduction in volume of the material in relation to the modulus of volumetric compression. The second phenomenon is caused by the deviatoric components of the stress tensor, and displacements occur without change in volume of material.

The spherical and deviatoric components of the stress tensor are very different at depth; i.e. the spherical component has a maximum value under the base of the footing; however, the deviatoric component has a maximum value at a depth that is equal to half of the width of the footing. Shear deformation is dominant under footings, shafts and piles, but volumetric compression predominates under rafts or embankments.

Total (final) settlement, s, can be estimated for a footing with width and length respectively equal to B and L, that is embedded (strictly speaking, the equivalent embedment depth) by at least the footing width. For the case where B > 0.6 m (Menard and Rousseau, 1962):

$$s = (q - \sigma_{vo})\left[\frac{2}{9}\frac{B_0}{E_d}\left(\lambda_d \frac{B}{B_o}\right)^{\alpha} + \frac{\alpha\lambda_c}{9E_c}B\right] \quad (3)$$

q= design normal pressure applied on the footing; σ_{vo}= total (initial) vertical stress at the level of the footing base; B_o= reference footing width, equal to 0.6 m; B= footing width; α= rheological factor (Menard, 1975); λ_d and λ_c= shape factors, (Menard, 1975); E_c= E_1, which is the weighted value of E_M from under the footing to a depth equal to one half of the footing width (Menard, 1975); E_d= harmonic mean of E_M in all layers down to the depth of $8B$ (or 16 times half footing width, R) below the footing.

As shown in Figure 2, E_d is calculated by dividing the ground beneath the footing base into 16 layers, each with a thickness equal to half the footing width. The subscript of each modulus designates the layer number and location. The harmonic mean of the moduli is calculated for each layer or group of layers using Equation 4:

$$E_i = \frac{1}{n}\sum_{j=1}^{n}\frac{1}{E_j} \quad (4)$$

E_i= harmonic mean of moduli of layer I; n= number of moduli layer I; E_j= moduli values in layer I.

E_d is calculated from Equation 5 (Menard, 1975):

$$E_d = \frac{4}{\frac{1}{E_1} + \frac{1}{0.85E_2} + \frac{1}{E_{3,4,5}} + \frac{1}{2.5E_{6,7,8}} + \frac{1}{2.5E_{9\ to\ 16}}} \quad (5)$$

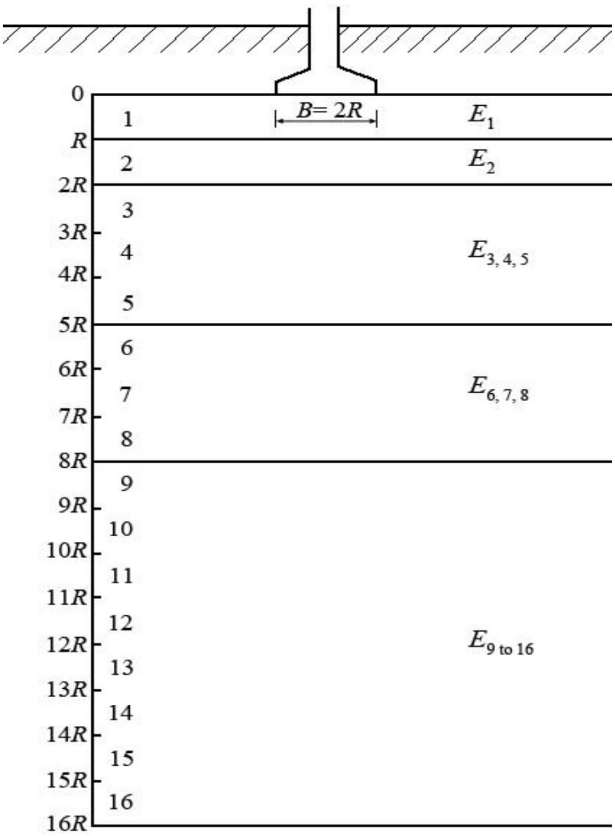

Figure 2. Layers of ground under the footing that are taken into consideration for the calculation of E_d (Menard, 1975).

The PMT can also be used for the design of piles; however, that discussion is beyond the scope of this paper and the readers are referred to Menard et al (1971), Baguelin et al (1978), Briaud (1986), Bustamante et al (2009) and Gambin and Frank (2009).

3 GROUND IMPROVEMENT CASE STUDY: ERSKINE PARK WAREHOUSE PROJECT

3.1 *Project description, ground conditions and project requirements*

The project was the site preparation works for the construction of two industrial buildings and open warehouse areas covering an area of approximately 50,000 m^2 that was located in Sydney's industrial western suburb of Erskine Park. One building was intended for a pallet factory and the latter was to support a heavy-duty crane yard.

The site was previously part of a quarry and was used for stockpiling approximately 1.5 million m^3 of soil and waste rock from about 1965 to 1994. Materials contained in the stockpiles were spatially highly heterogeneous, both in the vertical and lateral dimensions, and contained a mixture of various sizes of boulders, cobbles and gravels extracted during quarrying operations in a fine soil matrix. Boulder sizes were reported to be up to approximately 1.5 m in size.

Groundwater was not observed in any of the test pits.

Test pit logs noted that 30% to 85% of the ground was generally composed of fine soils. Sieve analysis of soil samples extracted from test pits suggested that fines content varied from 9% to 57% although higher fines content was also observed.

Liquid limit of the soil samples extracted from test pits ranged from 30% to 53%. Plastic limit was in the range of 18% to 23%, and plasticity index was from 7 to 35%.

With consideration that the stockpile was placed on site in such a way that the ground levels varied greatly, it was decided to cut the stockpile in some areas and fill the lower ground in other areas to derive a balanced flat site level. Performing this earthworks operation would have resulted in a site with 2 to 10 m of highly heterogeneous loose fill.

The project required that the below criteria be satisfied:

- bearing capacity: 150kPa with a floor load of 20 kPa
- Settlement: 15 mm between warehouse columns at 10 m centres
- Differential settlement: 15mm for floor slabs
- Subgrade CBR: 4%
- Young's Modulus: 15MPa

It was assessed that the above would not be satisfied if the platform was constructed without implementation of special measures. Hence, a solution was accordingly developed.

3.2 *Ground treatment: dynamic compaction*

Conventional roller compaction was considered as a solution with multiple drawbacks, including:

- Scope: In its simplest form, roller compaction was applicable to the layers that would be cut and used as backfill. Large volumes of the stockpile that were below the balanced platform level would still remain loose and compressible. Excavation to base of stockpile would have greatly complicated the work process.
- Time: placing and compacting 0.3 m thick layers of fill was assessed to be very time consuming.
- Cost: Due to the significant compaction time and number of required plant and workforce, it was assessed that execution of the works would be very costly.
- Material grading: The stockpile was highly heterogeneous both vertically and laterally. Hence, the placement of fill would have resulted in a complex combination of layers with high fines or coarse material, which would have required a complex supervisory programme and the allocation of vibratory smooth and sheep foot rollers (Australian Standard, 2007) for each layer type.
- Oversize material: whilst the dominant matrix of the stockpile material was fine soils, the presence of large size cobbles and boulders would have required an extensive screening process to remove oversize stones. Australian Standard (2007) limits the maximum particle dimensions of any rock or other lumps within a layer after compaction generally to two-thirds of the compacted layer thickness.

With consideration that piling could have only been a solution that was applicable to the structures, ground improvement was considered as a feasible and practical solution. Initially dynamic compaction and impact rolling compaction were considered.

In dynamic compaction (DC) a heavy pounder is dropped from a significant height. The impact creates high energy waves that improve the soil's mechanical properties, which results in higher bearing capacity and reduced settlements under loading. The pounder weight is most often in the range of 80 to 250 kN although lighter or heavier pounders are occasionally used. Drop heights are usually in the range of 10 to 20 m, but higher or lower drop heights are also sometimes used. (Hamidi et al, 2009).

Impact rolling compaction is performed by using a three or five-sided roller that generally weighs 8 to 16 tons. These rollers do not have a motorised form of energy such as the vibratory rollers and derive their energy by turning on their corner and falling onto the flat side. Fall heights are generally from 0.15 to 0.23 m (Hamidi et al, 2009).

Depth of influence is the depth where there is no more observable improvement in the soil. Menard and Broise (1975) developed an empirical equation in which the depth of influence was less than the square root of the impact energy; i.e. the product of the pounder weight, W in tons, by the drop height, H in metres. Others, e.g. Varaksin (1981), have added a reducing factor or a combination of factors to the equation; thus, as shown in Equation (6):

$$D = c\sqrt{WH} \tag{6}$$

c= depth of improvement coefficient.

With consideration of the significant difference in the amount of energy per impact, it can be readily assessed that there should be a notable difference between the depth of improvement of the two techniques. Whilst dynamic compaction is usually applied for the soil treatment with depths of typically up to 10 m, Hamidi (2014) has reported several projects with greater depths of treatment. According to a study by Berry et al (2004) the improvement depth using impact roller compaction is 2 to 3 times the compactor's width (900 mm) and the maximum improvement is achieved at about 0.67 to 1 times the compactor's width. This will yield an improvement depth of approximately 1.8 to 2.7 m.

As treatment depth exceeded the achievable depth of improvement by impact roller compaction, this method was not pursued, and the project was awarded to a geotechnical specialist contractor with expertise in the design and execution of dynamic compaction.

Whilst dynamic compaction is theoretically applicable to all soils, the degree of improvement for the same amount of energy can be related to the soil type and amount of fines (Hamidi et al, 2010). Lukas (1986) has categorised the effect of dynamic compaction based on soil grading (permeability); i.e. pervious soils, semi pervious soils and impervious soils. Pervious deposits were considered as soils with grain size of up to boulders and up to 35% silt. The application of dynamic compaction would achieve excellent to good results in these soils. Semi-pervious soil deposits were assumed to be generally silty soils containing some sand but less than 25% clay and plasticity index of less than 8. Treatment of these soils would yield fair to good results. Impervious soil deposits were envisaged as soils with plasticity index of greater than 8. Dynamic compaction was not recommended when saturation was high and minor to fair improvement was expected when water content was less than plasticity index.

Using a pounder with a nominal weight of 19 tons, two HPT (heave and penetration tests) (Lukas, 1986, Hamidi, 2014) were performed. The purpose of this test is to assess the suitability of the dynamic compaction parameters; i.e. grid spacing, drop height, number of drops in areas of the site with the fine soil matrix. Unless there is a special reason or justification, these parameters will generally remain unchanged when ground conditions and design requirements remain the same. Hence, unlike other field tests that are performed at much higher frequencies to verify ground performance, only one or two HPTs will suffice on normal-sized projects. The author has previously encountered projects that specified HPT in frequencies that resulted in overlapping of the test zones. Such a test plan not only imposes great costs to the project and causes delay, but also does not bring any value to the project.

Whilst the author has observed that HPT is sometimes performed in its simplified form of only measuring the volumetric changes of the DC crater with the presumption that the volume of ground around the crater is either not changing or is also compressing. This may be true in granular soils but may be incorrect in soils containing fines. In such soils, the ground around the pounder may heave, and it is the balance between the ground compression and heave that determines the outcome of the HPT. Not measuring the volumetric changes around the crater can result in misinterpretation of ground behaviour.

The results of an HPT that was performed at a location with approximately 10 m of fill is shown in Figure 3. It can be observed that application of DC blows to the ground caused compression within the crater and heave around it. The volume of compression is greater than the heave, which indicates that dynamic compaction is successful in compacting the soil with high fines on this site. Consistent with a typical HPT result, Figure 3 also indicates that the magnitude of ground compaction reduces with the increase of number of pounder blows until the application of additional energy practically yields negligible improvement.

The project specifications required that peak particle velocity (PPV) be limited to 10 mm/s at the closest building to site, which was approximately 25 m away. According to Hamidi et al. (2011) PPV generated by dynamic compaction can be estimated by Equation 7:

$$PPV \leq 25\left(\frac{\sqrt{WH}}{d}\right)^{1.1} \quad (7)$$

d= distance between pounder and point of measurement in metres.

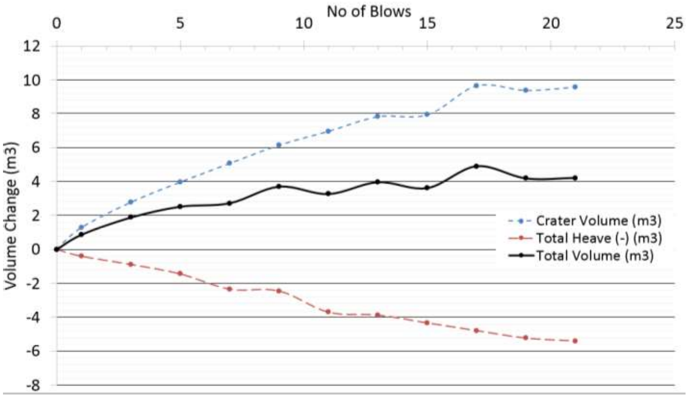

Figure 3. Result of an HPT test

Hamidi et al. (2012) have shown that vibration isolation open-trenches can reduce DC vibrations on average to half; hence, an isolation trench was also excavated to an approximate depth of 2 to 3 m along the boundary of the site.

Vibration triggers were installed next to the nearest building, but the triggers did not activate throughout the duration of the works. Figure 4 shows the execution of DC works near the closest building.

For quality assurance and verification of works, in total 16 PMTs, 5 Standard Penetration Tests (SPT), 23 Dynamic Cone Penetration Tests (DCP), and 4 Plate Load Tests (PLT) were performed. Five PMTs were carried out before ground improvement works.

The interest of this paper is the results of the PMT, and for demonstrative purposes two PMT results have been chosen. These tests were performed in an area with approximately 10 m of fill. One test was performed before treatment and the other was performed after application of dynamic compaction. The limit pressures of the two tests are shown in Figure 5.

Whilst the distance between these two points was approximately 20 m, the data is still able to represent the ground conditions before and after treatment. It can be observed that the ground was initially loose and compressible to the depth of 4 m, then gained strength before becoming loose once again at 8 m depth. Probably due to a large boulder, the ground was locally very hard at 7 m, and would have resulted in refusal of SPT, DCP and CPT (Cone Penetration Test).

It can be observed that the ground gained strength with a sickle shaped profile, which is classical of dynamic compaction treatment (Hamidi, 2014). It can also be observed that the ground became quite compacted to the end of testing depth at 10 m.

With a safety factor of 3, *k*= 0.85 (Menard, 1975), and conservatively assuming the footing to be on the ground, it can be calculated from Equation 1 that a footing of 3 m width will have a safe bearing capacity of 350 kPa.

Figure 4. Application of dynamic compaction approximately 25 m away from an existing building

4 CONCLUSION

The pressuremeter test is a very reliable tool that can assist the geotechnical engineering in assessing the ground strength and deformation properties using two parameters, namely the limit pressure and the Menard modulus. Bearing capacity and settlements can be calculated using Menard's formulas that have the least reliance on correlations and empirical input. This test can be performed in almost all grounds, from soft soils to rock, to almost any depth that can be practically drilled. These advantages make the PMT an ideal tool for verification of ground improvement works.

5 ACKNOWLEDGEMENT

The author would like to express his appreciation and gratitude to Menard Oceania for their support and providing the project information.

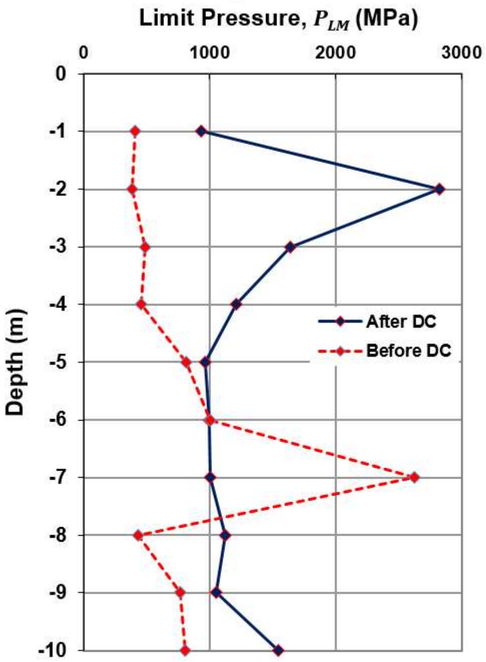

Figure 5. Before and after DC pressurmeter results

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