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Deep Compaction of Sand Causing Horizontal Stress Change

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ABSTRACT: Deep soil compaction is usually required for the control of total and differential settlement, and mitigation of liquefaction. An important, often neglected aspect, is the increase in horizontal stress which occurs due to deep compaction. The increase in horizontal stress means that also the preconsolidation stress and thus the overconsolidation ratio have increased. Re-analysis of calibration chamber tests employing CPT and DMT soundings show that both the CPT and DMT can measure changes in horizontal stress and, thus, be used to show a simple relationship between the increase in horizontal stress index from DMT and the overconsolidation ratio. The application of the tangent modulus method is illustrated using information by CPT and DMT. A hypothesis is proposed that explains aging effects in compacted soil by the redistribution of horizontal stresses after treatment. The significance of the overconsolidation ratio for the liquefaction resistance of loose, water-saturated soils is illustrated. The increase in effective stress due to compaction is of significance for analysis of compacted fill, and in particular for the assessment of settlement. Stress changes due to compaction are also important for other types of advanced geotechnical analyses.

KEYWORDS: Sand, Compaction, Settlement, Liquefaction, CPT, DMT, *OCR*

1. INTRODUCTION

The objective of this paper is to demonstrate that compaction causes a permanent change of horizontal stress in the treated soil, affecting the behavior of granular soils during static as well as dynamic/cyclic loading. This effect is usually not appreciated, which has sometimes resulted in unreasonably conservative design solutions. Currently available in-situ investigation methods, such as the cone penetration test (CPT) and the flat dilatometer (DMT) can be used to reveal changes in horizontal stress caused by soil compaction, as will be exemplified in the following.

The most common application of deep compaction is to reduce total and differential settlement or to mitigate liquefaction. Land reclamation is extensively used in connection with large engineering projects, such as the construction of airports, harbors, railways, or highways, and is frequently needed for housing projects or the development of industrial areas in coastal areas. Such projects are typically large-scale and need to be planned and designed with competence and care.

In the context of this paper, soil compaction is defined as methods where granular soil is improved by dynamic and cyclic loading. Deep soil compaction can be achieved by a variety of methods, such as dropping heavy weights on the ground surface (also called "Dynamic Consolidation"), by inserting different types of vertically oscillating probes excited by an impact hammer, by a heavy vibrator mounted on the top of the probe, or using horizontally vibrating depth vibrators (also called Vibroflotation). The compaction process involves the application of repeated cycles of dynamic force, acting in the vertical and horizontal directions. The dynamic force induces strain and small movement to the soil structure, resulting in a reconfiguration of soil particles, usually to a denser state. Deep compaction is carried out in a grid pattern, in one or several passes.

The density of the soil is usually highest close to the treatment point and decreases with increasing distance. An important aspect is that the compacted soil can become more heterogeneous than before treatment due to stress changes in the horizontal direction. Gradually with time, there will be a tendency of stress adjustment, which can at least partially explain the frequently observed "time effect" after compaction.

The objective of this paper is to describe how compaction projects can be designed, taking into consideration the effect of horizontal stress change and preconsolidation. This aspect is of significance for settlement control and mitigation of liquefaction hazard. However, consideration of stress changes in compacted soils is of importance also for other types of advanced geotechnical analyses.

2. COMPACTION DESIGN

Deep soil compaction requires the involvement of competent geotechnical engineers during all phases of the project as they need to address the following key issues: a) is compaction required; b) if so, then to what degree; c) can the fill material or natural soil be improved by compaction; d) evaluation of alternative compaction methods (equipment and process); e) formulation of compaction specifications taking into account the project-specific requirements; and f) monitoring and verification that desired compaction has been achieved.

With respect to compaction design, the formulation of compaction criteria, addressing quantitatively the actual requirements with respect to settlement or liquefaction, is a challenging task for the cost-effective implementation of the project. Alas, even on large projects, compaction specifications are frequently based on empirically developed rules of thumb or more or less well-fitting compaction requirements lifted from specifications used on previous projects. This approach can have serious adverse consequences as it usually leads to overcompaction without taking into consideration the project-specific requirements with respect to settlement or liquefaction. In the opinion of the authors, designers frequently lack an understanding of the compaction process and its effect on the treated ground. This has often resulted in unnecessarily stringent design specifications (overcompaction).

The designer is faced with two separate design considerations: 1. process design (execution of compaction work) and 2. functional design (performance of the structure on improved ground). The objective of the process design is to identify, based on geotechnical and other site-specific conditions, the optimal compaction solution. This requires the identification of alternative compaction methods (selection of equipment and execution process), an aspect which requires close cooperation between the designer and specialist foundation contractor(s).

During the past decades, new powerful equipment and machinery as well as sophisticated compaction processes have become available. Guidance documents and standards have been developed with the aim of increasing the efficiency and quality of soil treatment systems. European standard EN 14713 (CEN 2005) covers the execution of deep vibration achieved by depth vibrators and compaction probes.

The standard deals with planning, execution, testing, and monitoring of vibratory compaction. The intention for the development of these standards and guidelines has been to make soil compaction more competitive and to expand their application. However, the challenge for the designer is to specify compaction requirements that meet the requirements, but do not result in unjustifiable (often demanding too high) compaction, causing excessive costs that may even eliminate compaction as a foundation alternative.

The objective of the second consideration, the functional design, is to specify requirements that reflect changes in soil stiffness and soil stresses due to compaction. Compaction design shall assure the required performance of the foundation, usually with respect to settlement or potential liquefaction. In many cases, the purpose of a compaction process is to reduce total and differential settlement of a foundation. Obviously, the designer must have knowledge and experience how to calculate short-term and long-term, total, and differential settlements. The most important difference between calculated and actual settlement is usually not caused by the choice of analytical methods, but by the selection of relevant geotechnical input parameters, notably compression modulus and preconsolidation stress.

Another important application of deep compaction is mitigation of the liquefaction hazard due to cyclic loading of loose, water-saturated granular soils and improving bearing capacity as expressed by increase of the shear strength (friction angle) of the treated soil. Similar to settlement problems, liquefaction design is strongly affected by the selection of relevant input parameters. Although overconsolidation is known to have a strong beneficial influence on liquefaction, this aspect is usually not included in a rational design approach on soil compaction projects.

3. HORIZONTAL STRESS CHANGE

Deep vibratory compaction of granular soils changes the horizontal stress conditions. This effect has been documented by in-situ tests (CPT and DMT) on several deep compaction projects, for example, Brown (1989), Massarsch (1994), van Impe et al. (1994), Howie et al. (2000), Massarsch and Fellenius (2002), Asalemi (2006) or Massarsch and Fellenius (2017). The change in horizontal stress is of great practical significance, as it increases the pre-consolidation stress and, thus, the overconsolidation ratio, *OCR*. Another important aspect is that horizontal stresses after compaction vary significantly laterally. The highest horizontal stresses can be expected close to the center of a compacted soil column, but decrease with increasing distance from the center. However, at the center of the compaction point, horizontal stresses could be lower following extraction of the compaction tool. Large variations in horizontal stress tend to equalize during compaction in adjacent locations and with time, attaining stress equilibrium. This aspect will be discussed in more detail below.

3.1 Horizontal Stress Increase

Early studies of the change in stress conditions due to soil compaction was in connection with the use of surface vibratory rollers. Broms (1971) stated that when granular soils are compacted adjacent to a rigid wall, e.g., a basement wall, high permanent horizontal stresses are induced against the wall. Duncan and Seed (1986) and Symons and Clayton (1992) developed semi-empirical concepts for the prediction of horizontal stresses due to vibratory compaction. Duncan and Seed (1986) offered considerations regarding the effect of surface compaction of granular soils, as quoted:

1. *The compaction of soil represents a process of load application and removal which can result in significant increases in residual lateral earth pressure. These earth pressures may be many times greater than the theoretical at-rest values, and may approach passive earth pressure magnitudes.*
2. *The depth to which compaction increases lateral earth pressures appears to be a function of the dimensions and vertical thrust of the compaction roller, varying from on the order of 2 to 3 m for small hand-operated vibratory rollers to as much as 15 m for very heavy compaction equipment.*
3. *At depth where available overburden pressures are sufficient that possible passive failure does not limit residual lateral earth pressures, a high percentage (40 - 90%) of the peak lateral earth pressure increases induced during compaction may remain as residual pressures.*
4. *The compaction of soil against deflecting structures can significantly increase structural deflections, generally increases near-surface residual lateral pressures to greater than at-rest values, and generally decreases lateral pressures at depth, apparently as a result of increased structural deflections. The mode of structural deflections can, however, significantly influence this pattern.*
5. *In previously compacted soils (soils with previously "locked-in" compaction stresses), additional compaction loading can result in much smaller increases in peak earth pressures during compaction than in uncompacted soils, and a negligible fraction of these peak increases may be retained as residual earth pressure increases upon the completion of compaction.*

Several theories and analytical methods have been developed to explain and/or analyze the residual horizontal earth stress induced by soil compaction. Common to all of these is the concept that compaction represents a form of overconsolidation wherein stresses resulting from a temporary or transient loading condition are retained to some extent following removal of this peak load. Early on, Rowe (1954) proposed that compaction could be interpreted as the repeated application and removal of a static surcharge and suggested that virtually all peak soil stresses induced by the surcharge loading would be retained after surcharge removal. These considerations also apply to deep compaction of granular soils, which fact needs to be recognized, as stated by Massarsch and Fellenius (2002).

In an important paper on this subject, Schmertmann (1985) discussed the significance of horizontal stress, and listed over 60 references to demonstrate the importance of horizontal stress for geotechnical design. He pointed out that horizontal stress represents key site condition which engineers should consider in their investigations and analyses and that failure to measure and use the in-situ horizontal stress in design can result in uneconomical and excessively conservative foundation solutions.

3.2 Estimation of OCR

Massarsch and Fellenius (2002) described the results of comprehensive CPT investigations in connection with the compaction of a hydraulic fill, where sleeve resistance increased about by the same ratio as cone stress. They presented a simplified concept how the increase of horizontal stress (based on the ratio of sleeve resistance after to that before compaction) could be used to estimate *OCR*. Although some uncertainty exists regarding the accuracy of the measured sleeve resistance, the relative increase in horizontal stress (determining the ratio of sleeve resistance values after compaction with values before compaction) can be considered a useful indicator of increase in horizontal stress (Robertson 2016). Based on results of triaxial tests, several investigators have proposed an overconsolidation ratio, *OCR*, as an empirical relationship between the earth stress coefficient of normally sand, K_0, and that of overconsolidated sand, K_1, as expressed in Eqs. 1 and 2.

$$\frac{K_1}{K_0} = OCR^{\beta} \qquad (1)$$

from which follows

$$OCR = \left[\frac{K_1}{K_0}\right]^{\frac{1}{\beta}} \quad (2)$$

Where K_0 = coefficient of earth stress at rest for normally consolidated soil, K_1 = coefficient of earth stress at rest for overconsolidated sand, $ß$ = empirically determined exponent.

Based on laboratory tests, Schmertmann (1975) recommended a value of 0.42 for $ß$, Lunne and Christophersen (1983) suggested 0.45 and Jamiolkowski et al. (1988) indicated a range from 0.38 to 0.44 for medium dense sand. For $ß$ = 0.42, Eq. 2 implies that a relatively small increase in the earth stress ratio, K_1/K_0, say by a factor of 2, results in a significant increase of *OCR* (from unity to larger than 5).

Because an earth stress coefficient larger than about 0.5 in sand indicates that the soil is likely to be overconsolidated, it is of great practical significance to include this aspect in geotechnical design, particularly, when assessing settlement and liquefaction.

3.3 Time Effect

The effect of time on the geotechnical properties of compacted sand has been subject to extensive discussions in the geotechnical literature. Mitchell (2008) came to the following conclusions:

"Although chemical precipitation-cementation reactions had initially been considered a primary cause, the evidence clearly favors a secondary compression-like process during which particle rearrangements and internal interparticle stress changes and redistributions among groups of particles occur, accompanied by only small volumetric compressions. It is seen that there is considerable variability, dependent on the sand type, its initial state, applied stress conditions, and the specific property being measured. Thus, while the case history information may provide useful guidance about how much property change there will be due to aging and how fast it may occur, each case should be evaluated separately by means of field measurements. Further improvement in the understanding and quantification of sand aging may be possible using rate process and discrete element analysis methods."

The authors concur with the observations by Mitchell that an aging effect can—but not necessarily, will—take place following compaction. However, in addition to the "*aging*" phenomenon described by Mitchell (2008) and others, also the equalization in horizontal stress after compaction can cause changes in soil strength and stiffness. There is little doubt that compaction of granular soils causes an increase in horizontal earth stress. However, a frequently asked question is whether this increase is permanent or if it changes (increases or decreases) with time. The authors suggest that with time after treatment, there is a tendency of a gradual equalization of horizontal stresses across the compacted soil volume, toward stress equilibrium. Thus, in zones with high densification, horizontal stresses will decrease, while in zones with initially lower stresses (at the center of - or further away from the compaction point) horizontal stresses can increase.

This stress equalization process can be one of the reasons that in-situ tests show a change of soil resistance and/or stiffness with time after compaction. Such change in penetration resistance has been observed on several compaction projects (often called "aging"), but cases are also known where with time after compaction the penetration resistance has remained unchanged or decreased, which might be due to the uncertain position of the particular in-situ test compared to the location of the nearest compaction point, Massarsch and Westerberg (1995) and Mitchell (2008).

4. INTERPRETATION OF CONE PENETRATION TEST

The CPT—and variations thereof, such as the CPTU (CPT with pore water pressure measurement) or the SCPT (CPT with seismic downhole test)—is today the most widely used field investigation method on deep compaction projects. The CPT is standardized, thereby reducing the risk that equipment and operation affect the measured parameters (ISSMGE 1999, ISO 22476-1:2012). An important advantage of the CPTU is that it measures three independent parameters as function of depth (at a depth interval of 10 or 20 mm): cone stress (q_c), sleeve resistance (f_s), and pore water pressure (u). (The term "cone stress" is preferred to the, perhaps, more widely used term "cone resistance". In contrast, as sleeve resistance in many soils is due to a combination of cohesion and friction, the term "sleeve resistance" is an appropriate term). On many—even major—compaction projects, often only the cone stress is specified and thus reported, while the sleeve resistance, although being measured, is not reported, as the design requirements usually—and regrettably—are based on cone stress, only. Not reporting the sleeve resistance, limits the ability to analyze the CPT-records.

In the case of the CPTU, the cone stress, q_c, is adjusted to the measured pore water pressure, u_2, acting on the cone shoulder. The symbol for the so-adjusted stress is q_t. However, in granular soils—and in particular on soil compaction projects—the measured pore water pressure is small relative to the cone stress and the pore pressure adjustment can therefore be neglected. Moreover, representative pore water pressure measurements can be difficult to obtain on soil compaction projects, in particular in the dry and partially saturated zone above the groundwater table. Similar difficulties can be encountered when the CPTU penetrates through very warm shallow layers (with surface temperatures of around 100 degrees C) frequently found in hot climate zones. These effects can distort the accuracy of pore water pressure measurements. Nevertheless, in the case of deep compaction of stratified natural or man-made soils, sleeve resistance and pore water pressure measurements can alert to the presence of impermeable soil layers. In such cases it is recommended to use pre-boring through the hot layers and then to use of the CPTU rather than the CPT.

4.1 Adjustment of CPT measurements

Robertson (1990) proposed using normalized (dimensionless) CPT parameters. The pore-pressure adjusted cone stress, q_t, is reduced by subtracting the vertical total stress, σ_{v0}, and then dividing this value by the vertical effective stress, σ'_{v0}, which results in the "normalized cone stress", $Q_t = (q_t - \sigma_{v0})/\sigma'_{v0}$. Also, the sleeve resistance, f_s, is divided by the adjusted cone stress ($q_t - \sigma_{v0}$); the so-obtained normalized friction ratio is denoted R_f. (N.B., the pore pressure parameter, B_q, is less relevant in free-draining soils and is not widely used for compaction projects). In the opinion of the authors, for compaction projects, these rather complex adjustments of CPT parameters are not warranted, as will be shown below.

Repeatable and accurate measurements of q_c and f_s are the key to successful design and execution of sand compaction projects, in particular in loose soils, such as hydraulic fills below the groundwater table. Of the two, the cone stress is the most reliable parameter. The sleeve resistance is more difficult to measure as the accuracy of f_s can be affected by quality of the equipment, such as slight variations of the diameter and wear of the sleeve (Cabal and Robertson 2014). In some cases, the sleeve resistance of hydraulic fill can be very low or close to zero, as has been addressed by Massarsch (1994), Debats and Sims (1977), Massarsch and Broms (2001), van Impe et al. (2015), Massarsch and Fellenius (2017). Consequently, the interpreted friction ratio can be very low or close to zero, making results of data interpretation using F_R unreliable.

All measurements have limits of precisions or errors and, the ratio of q_t to f_s, i.e. the R_f has, therefore, a larger error than either of its components, which further indicates the need for caution in using R_f.

4.2 Soil Profiling

Begemann (1963) pioneered soil profiling using the CPT. He showed that soil type is a function of the relationship between cone stress and sleeve resistance, as indicated by the slope of a plot of cone stress vs. sleeve resistance. Sanglerat et al. (1974) were the first to introduce the concept of "friction ratio", R_f and changed from plotting the results according to the manner used by Begemann–cone stress vs. sleeve resistance–to plotting cone stress vs. friction ratio, which manner the practice then followed. The change is unfortunate as plotting cone stress vs. friction ratio hides important information regarding the actual values of two independent variables, cone stress and sleeve resistance.

Robertson (1990) introduced the widely used "*simplified soil behavior type classification chart*", the SBT chart. It identifies twelve different soil categories, ranging from sensitive fine grained and organic material to very stiff sand and gravel. This concept of soil type identification can be useful on conventional projects. However, R_f does not clearly reflect soil type, because f_s, as will be shown below, is likely to change due to changes in horizontal stress caused by compaction. For this reason, assessing soil type from R_f essentially applies only to normally consolidated soils.

It is important to observe that changes in horizontal stress will result in changes in sleeve resistance and, thus, the friction ratio. This limitation of the soil behavior index, $\underline{I}_c$, has been noted by Nguyen et al. (2014) and can result in a reduction of the measured $\underline{I}_c$ value, and a corresponding decrease of apparent fines content. However, it is impossible for the vibratory compaction process to produce a change in fines content. Nguyen et al. (2014) proposed a correction method to compensate for the shift in I_c and to maintain the same fines content in the pre- and the post-treatment CPT-based liquefaction analyses.

Thus, in the case of soil compaction (usually granular soils), an increase in friction ratio can be misinterpreted as the soil becoming more fine-grained, while, in reality, the soil type remained unchanged. This problem has been observed on several compaction projects, e.g., Brown (1989) and Massarsch and Fellenius (2014). It is therefore preferable to plot cone stress against sleeve resistance, as suggested by Fellenius and Eslami (2000), and Massarsch and Fellenius (2002), so as to avoid the distortion introduced by using the normalization process and friction ratio. (Unless the range of data to plot exceeds two orders of magnitude, a linear scale diagram is preferable to plotting in logarithmic scale).

4.3 CPT Calibration Chamber Tests

CPT investigations can be performed under controlled conditions in a calibration chamber (CC). Relationships between CPT measurements and fundamental geotechnical properties have been proposed by for example Baldi et al. (1986), Houlsby and Hitchman (1988), Salgado (1993), and Jamiolkowski et al. (2003). The present paper builds on the findings reported by Baldi et al. (1986), who performed careful tests on two types of sands: Ticino and Hokksund sands. Tests were carried out at normally and overconsolidated stress conditions. They concluded that "*q_c is almost completely controlled by the initial effective horizontal stress. This statement is strongly supported by the result of an interpolation of the CC test data available for both NC and OC (Ticino sand)*." These findings have been confirmed by other researchers, leading to the conclusion that horizontal effective stress has a profound influence on cone stress, e.g., Houlsby and Hitchman (1988), Salgado (1993), Jamiolkowski et al. (2003), and Ahmadi et al. (2005). Yet, on many compaction projects, correlations between cone stress and density index (relative density) are frequently based on vertical effective stress, the main argument being that it is not possible to assess horizontal effective stress. Yet, modern in-situ testing methods, such as the CPT and DMT, can demonstrably be used to determine changes in horizontal earth stress, and in particular in the case of soil compaction projects.

Although a large number of CC tests have been reported, these have focused almost exclusively on cone stress, while sleeve resistance measurements have been rarely presented. Usually, the tests are assumed performed at OCR = 1. However, when a soil sample is constructed and compacted in the CC to a dense state, high horizontal stresses do exist already before the start of the test. Therefore, the validity of assuming normally consolidated stress condition (OCR = 1) at high initial densities is questionable.

Baldi et al. (1986) reported CPT CC tests that did include measurements of sleeve resistance and, additionally advantageous, also measurements of horizontal effective stress. The tests were performed at different values of both overconsolidation ratio and density index (I_D)—the OCR ranged between 1 and 10 and I_D ranged between 30 and 96 %. Figure 1 shows the measured sleeve resistance, f_s, as a function of density index, I_D (also called 'relative density') at OCR = 1.

Inspection of Figure 1 leads to the following conclusions. Sleeve resistance is sensitive to variations in horizontal stresses. The increasingly higher—but variable—resistance reflects similar variations in the horizontal stress, which in the above shown CC test results are due primarily to sample preparation. At low I_D, in spite of some scatter, there is a reasonably consistent correlation between sleeve resistance and density index. However, when the I_D exceeds about 60 %, the scatter of sleeve resistance values increases, which reduces the consistency. The high sleeve resistance values beyond about 60 % I_D confirms that high horizontal stresses were created during to placement of the sand. That means that, in compacting the sample to a dense state beyond I_D = 60 %, high horizontal stresses are built up and the soil became overconsolidated already prior to the start of the tests. Obviously, assuming OCR = 1 would be incorrect.

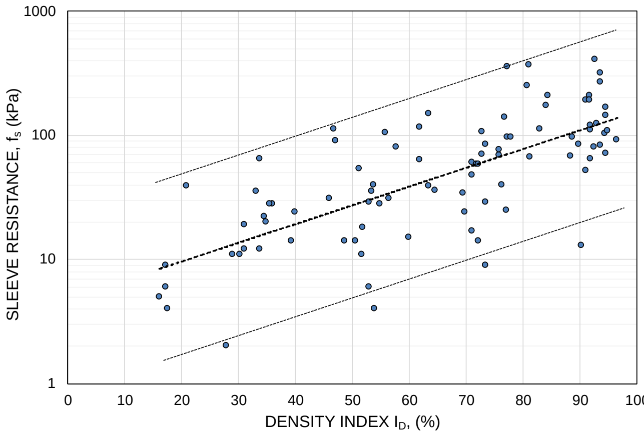

Figure 1 Relationship between sleeve resistance and density index (relative density) for normally consolidated Ticino sand, based on data by Baldi et al. (1986)

5. INTERPRETATION OF DILATOMETER TESTS

The flat dilatometer test (DMT) was developed by Marchetti (1980) and is a relatively recent field testing addition. Guidelines for the DMT equipment and application techniques have been presented by ISSMGE Technical Committee 16 (Marchetti et al. 2001). For a detailed description of the DMT, recent developments in data interpretation, and practical application of results, refer to the geotechnical literature, e.g., proceedings of the 3rd DMT Conference (Marchetti 2015). In the context of this paper, only aspects of using the DMT for compaction monitoring of granular soils are addressed.

5.1 Evaluation of DMT measurements

A key characteristic, which distinguishes the DMT (and to some extent the Menard pressuremeter, PMT) from other in-situ methods, is its ability to measure soil parameters that reflect the stress conditions in the horizontal direction. From the derived values of p_0 and p_1, the following DMT index parameters are calculated:

$$I_D = \frac{p_1 - p_0}{p_0 - u_0} \tag{3}$$

$$K_D = \frac{p_0 - u_0}{\sigma'_{v0}} \tag{4}$$

$$E_D = 34.7(p_1 - p_0) \tag{5}$$

where: I_D = material index, K_D = horizontal stress index, E_D = dilatometer modulus, u_0 = hydrostatic pore water pressure, σ'_{v0} = vertical effective stress.

The dilatometer modulus, K_D, is a key parameter for the evaluation of the DMT records. The factor 34.7 (the factor was 38.2 in the initial paper by Marchetti (1980)) depends on Poisson's ratio and, thus, on horizontal stress and changes thereof. This fact is of particular importance in the case of soil compaction projects, where changes in horizontal stress occur. The ISSMGE reference procedure (ISSMGE 1999) states: "E_D in general should not be used as such, especially because it lacks information on stress history. E_D should be used only in combination with K_D and I_D." This comment needs to be considered when applying the DMT on soil compaction projects.

Marchetti et al. (2001) suggested transferring the dilatometer modulus, E_D, to a vertical, drained, constrained modulus, M, as expressed in Eq. 6 - 12.

$$M = R_M\, E_D \tag{6}$$

$I_D < 0.6$ $\quad R_M = 0.14 + 2.36 \log K_D$ (7)

$I_D > 3$: $\quad R_M = 0.5 + 2 \log K_D$ (8)

$0.6 < I_{DM} < 3$ $\quad R_M = R_{M,0} + (2.5 - R_{M,0}) \log K_D$ (9)

with:

$$R_{M,0} = 0.14 + 0.15\,(I_D - 0.6) \tag{10}$$

if $K_D > 10$: $\quad R_M = 0.32 + 2.18 \log K_D$ (11)

if $R_M < 0.85$ $\quad$ assume $R_M = 0.85$. (12)

where M = vertical, drained, constrained modulus, R_M = correction factor based on empirical data (Marchetti 1980), I_D = material index, K_D = horizontal stress index, E_D = dilatometer modulus, u_0 = equal to p_2 = hydrostatic pore water pressure.

Important advantages of DMT measurements on compaction projects are its ability of measuring horizontal stresses (K_D) according to Eq. 4, and thus the change in horizontal effective stress (from the change in the horizontal stress index, see below) and estimating the drained, constrained modulus, M, from Eq. 6.

As pointed out by Marchetti (2015) the increase of M with stress history is essentially due to the increase of E_D and K_D, which parameters must first be combined with stress history. This aspect is of particular importance in the application of the DMT on soil compaction projects, where significant changes in K_D occur. Therefore, the validity of Eqs. 7 – 12 needs to be verified on compaction projects, and especially when significant changes in horizontal stress (as reflected by K_D) are found.

5.2 DMT Calibration Chamber Tests

Calibration chamber (CC) tests are an efficient means to determine geotechnical parameters under controlled conditions. Unfortunately, too little attention has been given in the past to studying the relationship between horizontal stress and overconsolidation ratio.

Jamiolkowski et al. (2003) reported results of DMT investigations from CC tests. Three different types of silica sand were investigated: Ticino, Hokksund, and Tumour, the geotechnical properties of which have been described in the geotechnical literature, e.g., Jamiolkowski et al. (2003). The mean particle size, D_{50}, varied between 0.60 mm (Ticino), 0.45 mm (Hokksund) and 0.22 mm (Tumour). The uniformity coefficients, C_u (= D_{60}/D_{10}), were 1.30 (Ticino), 1.91 (Hokksund), and 1.31 (Tumour), respectively.

Lee et al. (2011) reported CC tests on Bussan sand which is a natural silica sand. The D_{50} was 0.32 mm and the roundness was angular to sub-angular. The maximum and minimum void ratios, e_{xam} and e_{mir}, were = 1.063 and 0.658, respectively. The C_u was 2.35.

The K_D vs. I_D data from a series of DMT CC test by Jamiolkowski et al. (2003) and Lee et al. (2011) are combined in Figure 2, showing that the horizontal stress index, K_D, increases with the density index, I_D. The significant scatter, is because K_D is not just affected by I_D, but also by the overconsolidation ratio, *OCR*. Indeed, when placing sand to a density larger than very loose (i.e., I_D > 60 %), *OCR* will tend to become larger than unity and, also, to become rather variable for higher values of I_D. A similar effect can be seen from CPT tests shown in Figure 1. However, Lee et al. (2011) were able to separate the dependence of horizontal stress index, K_D, between the density index, I_D (previously termed "relative density", D_R) and the *OCR*, as shown in Figure 3. The test data now display a clear trend that I_D and K_D both increase as a function of *OCR*. However, as indicated in Figure 2 by the high K_D-values at I_D exceeding about 60 %, the sample preparation, even when prepared at *OCR* = 1, has induced high horizontal stresses.

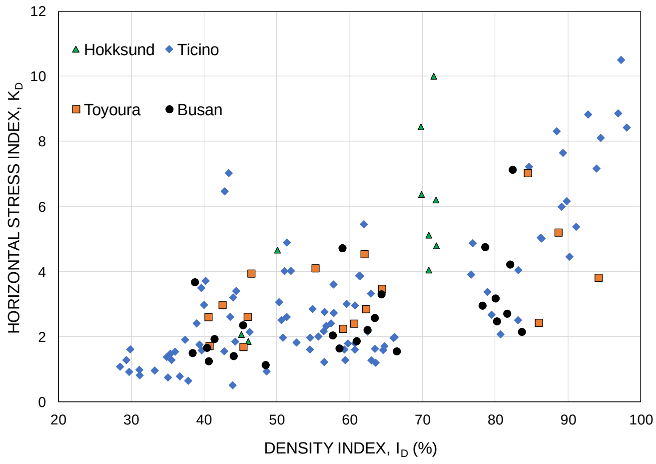

Figure 2 Horizontal stress index, K_D, as function of density index from DMT CC tests. Data are based on tests reported by Jamiolkowski et al. (2003) and Lee et al. (2011)

The data published by Lee et al. (2011) are shown in Figure 3. The data were extracted and are replotted in Figure 4 to show the interdependency of the *OCR* and K_D. (N.B. the I_D-values for 50 and 70 % are interpolated). Moreover, as compiled in Table 1, the diagram indicates that for normally consolidated sand (i.e., *OCR* = 1), I_D, ranges from 40 through 80 % and K_D, ranges from 1.31 through 2.58.

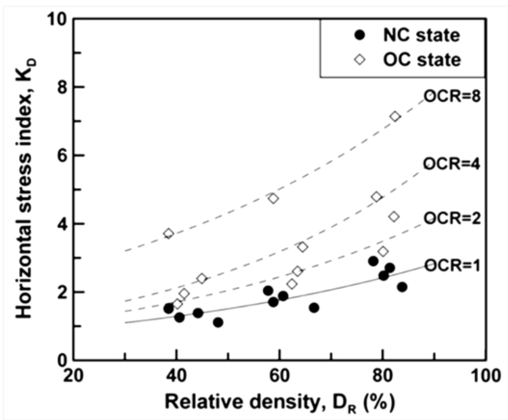

Figure 3 Effect of stress history on the horizontal stress index. Lee et al. (2011). Reproduced with permission 4275801110968 from Elsevier. Note that the present paper uses the term Density Index, I_D, in lieu of Relative Density, D_R, used in the original

Table 1 Average values of K_D as function of I_D for *OCR* = 1

I_D	K_D average
40	1.32
60	1.78
80	2.58

Figure 4 can be used to approximately estimate the increase in *OCR*, when compacting normally consolidated, uncemented sand, typical of hydraulic fills (but not applicable for calcareous material), for K_D-values measured prior to and after compaction.

5.3 Overconsolidation due to Compaction

As has been mentioned above, empirical correlations were obtained from triaxial tests between *OCR* as the ratio of K_1/K_0, where K_0 is the horizontal earth stress coefficient before compaction (normally consolidated condition) and K_1 the equivalent value after compaction, cf. Eqs. 1 and 2.

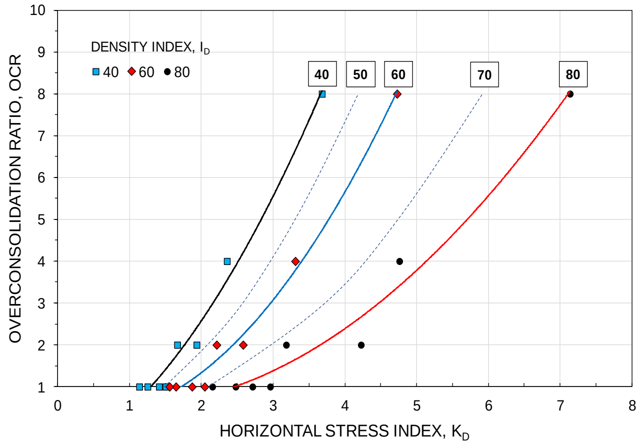

Figure 4 Dependence of horizontal stress index, K_D, cf. Eq. (1) on overconsolidation ratio, *OCR*, for Busan sand as function of density index, I_D. Also shown are the interpolated values of density index for I_D = 50 and I_D = 70 %

Reorganizing the DMT CC test data shown in Figure 4 makes it possible to appreciate that *OCR* depends on the normalized horizontal stress index, $K_D/K_{,ref}$, where $K_{D\ \mathrm{ref}}$ is the horizontal stress index of the uncompacted soil. The following analytical procedure was used: All data were grouped in three categories (I_D: 40, 60, and 80%) and the average K_D-ratio at *OCR* = 1 for each group was calculated, cf. Table 1. A K_D-ratio = 1.0 represents *OCR* = 1. The K_D-ratio for each group was then normalized in respect to the average for each I_D-group. Thus, the ratio $K_D/K_{D\ \mathrm{ref}}$ corresponds to the stress ratio in Eqs. 1 and 2 with K_0 taken as the reference stress, $K_{D\ ref}$. The curve resulting from this normalization process is shown in Figure 5 and it is a function of the *ß*-exponent of Eqs. 1 and 2.

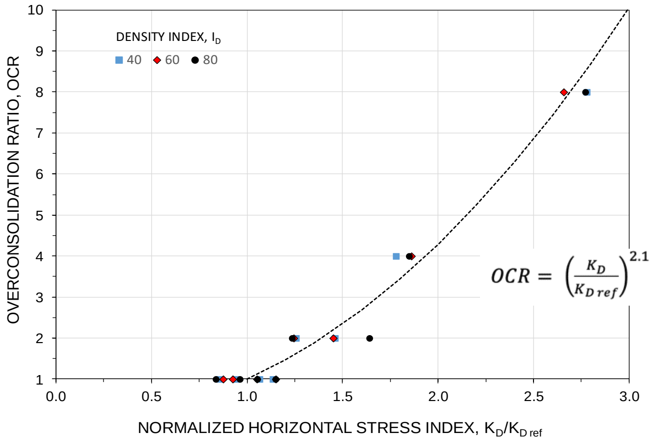

Figure 5 Overconsolidation ratio as function of normalized horizontal stress, also shown by Eq. 13. Data interpreted from Figure 4

It is interesting to note that in spite of the wide range of I_D-values, there exists a good correlation between the normalized horizontal stress index, $K_D/K_{D\ ref}$ and *OCR* as given in the following relationship, cf. Figure 5.

$$OCR = \left(\frac{K_D}{K_{D\,ref}}\right)^{2.1} \tag{13}$$

where: *OCR* = overconsolidation ratio, K_D = horizontal stress index, $K_{D\ ref}$ = horizontal stress index reference. The exponent 2.1 corresponds to *ß* by: 1/ß = 0.48. The values of the *ß*-exponent determined by laboratory tests are in good agreement with Eq. 12 obtained from CPT data. Thus, it can be concluded that *OCR* due to a change in horizontal stress as a result of compaction can be approximately estimated from in-situ tests based on Eq. 2, using an exponent *ß* = 0.48.

6. SETTLEMENT ANALYSIS

An important consideration of settlement design is that an in-situ test reflects the soil conditions at the time of testing. This obvious fact is not always recognized when evaluating in-situ tests before and after compaction. When the test is carried out prior to compaction, a sand fill, particularly a hydraulic fill, can be assumed to be in a normally consolidated state, or close to. However, after the compaction, the soil will be overconsolidated. Thus, the in-situ test after compaction will reflect soil behavior at the overconsolidated state, a fact which needs to be taken into consideration in the estimation of the preconsolidation stress.

On many soil compaction projects, the critical design consideration is to limit total and differential settlement. Unfortunately, settlement analyses are often based on over-simplified concepts, such as the frequently used empirical approach to indirectly relate settlement to density index (or relative density), despite the fact that more reliable analytical methods are available. For settlement estimates on compaction projects, the preferred concept by the authors is the tangent modulus method (Ohde 1951, independently developed by Janbu 1963; 1985), also discussed by Massarsch (1994), CFEM (1992), Massarsch and Fellenius (2014).

An important design consideration with respect to settlement is that the same settlement criterion (limit of total and/or differential settlement) can be achieved by a multitude of compaction requirements. However, frequently, a constant cone stress with respect to depth is specified, without considering that the mean effective stress affects the measured cone stress. It is preferable to express compaction specifications in terms of acceptable settlement rather than by a single minimum compaction criterion in terms of penetration resistance (cone stress), as the latter restricts optimizing the compaction procedure. Specifying a minimum density index as compaction requirement is counter-productive, because the density index cannot be correlated to settlement, in particular considering the uncertainty in expressing the density index based on CPT.

Estimating settlement is a challenging task and it would be naïve to expect in granular soils an accuracy better than ±30 % between estimated and actual settlement by any method. Uncertainties associated with settlement analyses are usually not caused primarily by the choice of numerical method. Rather, the selection of appropriate soil parameters (deformation modulus values) and assumed stress conditions (preconsolidation stress) to be used in the analysis are critical to the relevance of the results. The fundamental aspect of settlement analysis is that settlement depends significantly on the preconsolidation stress (increase in horizontal stress) and that the compression modulus in most cases is non-linear, as will be addressed in the following paragraphs.

6.1 Preconsolidation Stress

As has been shown by Massarsch and Fellenius (2002), vibratory compaction has two effects on the compacted sand: I) due to densification, soil stiffness (modulus) will increase and II) the treated soil will become overconsolidated. The effect of compaction on a normally consolidated soil deposit is illustrated in a schematic diagram (Figure 6), which shows the stress-compression curve for normally consolidated (uncompacted) sand (A) and for the same sand after compaction (B). The range C indicates the preconsolidation margin, $\Delta\sigma'$, of the compacted soil, an increase of the post-compaction preconsolidation stress from $\sigma'_{v,\,nc}$ to σ'_{pc}.

The average constrained moduli are defined as follows: M_A: normally consolidated soil prior to compaction; M_B: normally consolidated soil after compaction; and M_C: overconsolidated soil after compaction.

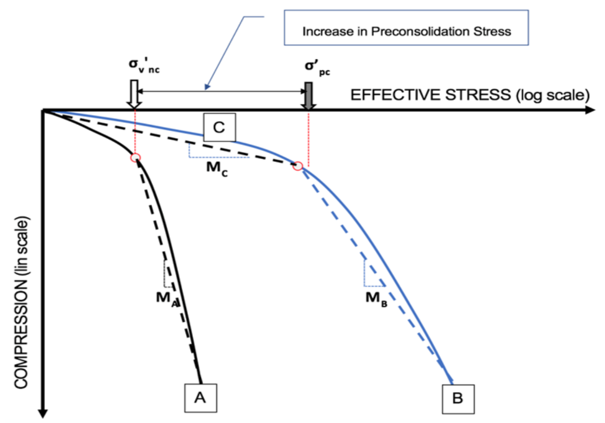

Figure 6 Schematic diagram showing effect of compaction on compression curve

Before compaction, the settlement due to an applied stress larger than $\sigma'_{v,nc}$ would follow the line marked M_A. After the compaction, the settlement for an applied stress depends on the magnitude of stress increase. If the stress increase remains below the compaction-induced preconsolidation stress, σ'_{pc}, the modulus M_C must be used in the analysis. However, if part of the applied load exceeds the post-compaction preconsolidation stress, σ'_{pc} the modulus M_B must be used for the load exceeding the preconsolidation stress. It is not possible to measure M_B by in-situ tests. However, in the event that the applied stress after compaction exceeds the post-compaction pre-consolidation stress, it can be conservatively assumed that M_B is equal to, or higher than M_A.

Static or dynamic preloading affects the horizontal stress, as has been discussed in previous paragraphs. The increase of soil stiffness can be directly related to the increase in horizontal stress. Estimation of the preconsolidation stress due to compaction is an important part of settlement analysis and must not be neglected. By assessing the preconsolidation stress, compaction requirements can be optimized and usually made less severe, potentially resulting in significant cost savings. Neglecting the increase of the preconsolidation stress violates basic geotechnical principles. When this negligence—which most likely would result in failing an undergraduate exam—occurs in practice, unjustified costs and excessive compaction requirements result.

6.2 Compression Modulus

The compression modulus of sand (i.e., sand compressibility) is difficult to determine in the laboratory due to the problems associated with soil sampling. In the case of the tangent modulus method, the modulus can be estimated based on empirical information. However, it is preferable to determine the compressibility of soil before and after compaction based on in-situ tests, such as CPT (or DMT) data, for example, according to the CPT-approach proposed by Massarsch (1994) and Massarsch and Fellenius (2002). Comparison of measured settlement and settlement calculated using compressibility (i.e., modulus numbers) determined from CPT-tests have shown good agreement.

The DMT in-situ test contributes an important advantage: the increase in horizontal stress ($K_D/K_{D\,ref}$) and the constrained modulus, M, can be estimated by the same method. However, as has been pointed out above, in the case of soil compaction projects, the increase in horizontal stress is significant and its influence on the relationship between E_D (which is affected by the increase in horizontal stress) and M (which defines soil stiffness in the vertical direction) needs to be verified, cf. Eq. 6.

7. LIQUEFACTION

Deep compaction is widely used to mitigate the risk of liquefaction or associated phenomena (cyclic mobility, lateral spreading, and total and differential settlement or instability of dams, embankments or natural slopes). Usually, the objective of deep compaction is to increase soil density, reducing the risk of partial or full loss of soil strength due to cyclic loading. Compaction requirements are normally expressed in terms of penetration resistance (most frequently based on CPT or SPT). Or, alternatively, they are expressed by a density index (relative density), as based on CPT or SPT. The results of laboratory tests have shown that liquefaction resistance is strongly correlated to the overconsolidation ratio, *OCR*, which aspect needs to be taken into consideration in the design of compaction projects.

Liquefaction has been studied extensively in the literature and a detailed discussion exceeds the scope of this paper. However, the importance of *OCR* on the resistance to cyclic loading will be illustrated in few examples. The effect of cyclic loading is commonly expressed by the cyclic stress ratio (*CSR*) versus the number of cycles to liquefaction initiation. Different definitions of *CSR* are used in the literature, but a common concept is to express *CSR* as the ratio of the shear stress, τ, divided by the initial effective confining stress, σ'_c.

$$CSR = \frac{\tau}{\sigma'_c} \tag{14}$$

where CSR = cyclic stress ratio, τ = ratio of the shear stress, σ'_c = initial effective confining stress.

Liam Finn (1981) reported cyclic simple shear test on specimens of Ottawa sand. Tests were performed at a density index, I_D, of 45 to 47 %. The vertical effective confining stress was 200 kPa. The overconsolidation ratio, *OCR*, ranged between 1 and 4. Figure 7, which reproduces the test data by Liam Finn (1981), clearly shows that *OCR* has a strong effect on liquefaction resistance in terms of *CSR*. The number of stress cycles required to cause liquefaction increases clearly with increasing *OCR*.

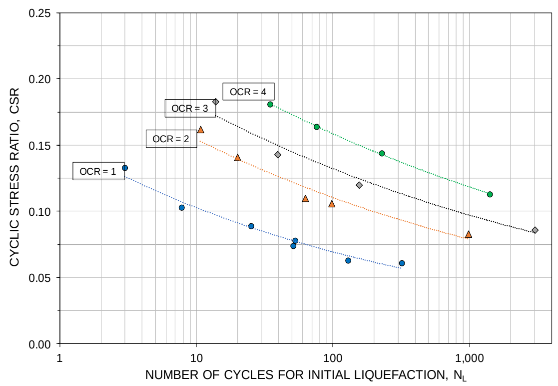

Figure 7 Effect of overconsolidation on liquefaction resistance, based on data by Liam Finn (1981). Ottawa sand at density index I_D = 45 to 47 %; vertical effective stress: 200 kPa. *OCR* parameter interpolated between data points to be used in Figure 8

Liam Finn (1981) concluded that the stress ratios required to cause initial liquefaction depend significantly on the *OCR* and the value of the lateral stress coefficient at rest, K_0. He attributed the increase in liquefaction resistance with increasing *OCR* to the increase in the earth stress coefficient, which reflects the increase in mean effective confining stress. Although, according to Liam Finn (1981) and other experts, the effect of overconsolidation cannot be entirely explained in terms of changes in K_0 or the mean confining stress, they are clearly responsible for a major part of the effect. Overconsolidation is apparently due also to other changes in the soil and, probably, related to grain structure or grain contacts, which have a further beneficial effect on liquefaction.

The data (from Liam Finn 1981) presented in Figure 7 are replotted in Figure 8 to show *OCR* as a function of number of cycles required to cause liquefaction and demonstrating that an increase of *OCR* has a significant effect on the number of loading cycles required to cause liquefaction.

Nagase et al. (1996) reported similar results for cyclic undrained triaxial tests on reconstituted sample of Toyoura sand. They showed that the liquefaction resistance increased significantly as *OCR* increased, similar to the test results reported by Liam Finn (1981).

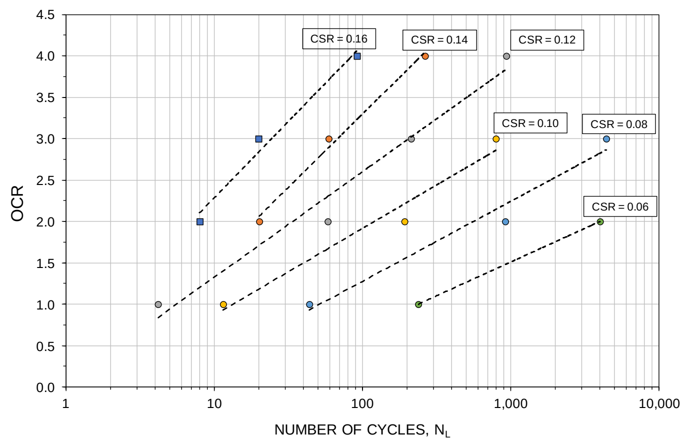

Figure 8 Presentation of data from Figure 7, showing the effect of *OCR* and *CRS* as a function of number of cycles

The ratio of increase in liquefaction strength increased as the number of cycles for repeated overconsolidation increased, although the ratio of increase did not considerably increase after the 4th cycle. An important conclusion was that the increase of liquefaction resistance due to overconsolidation was not correlated to the increase in density of the specimen, but to a slight change in the arrangement of sand particles in the specimen.

8. CASE HISTORY

In the following, a case by van Impe et al. (1993) has been re-evaluated, with results of CPT and DMT investigations prior to and after treatment. Although vibratory compaction was used to improve a sand fill, the results of field measurements can be considered relevant also for other types of compaction methods (e.g. vibroflotation or dynamic compaction).

8.1 Evaluation Concept

The objective of this study was in relation to the change in horizontal stress, measured by CPT and DMT as described above. In order to reduce the influence of soil layering and minor variations in measurement results, the geometric average of the measured values (q_c and f_s) over a depth interval of approximately 0.4 m were determined.

From the CPT investigations, the cone resistance, q_c, and the sleeve resistance, f_s were determined. The friction ratio was used to show the variation of soil type before and after treatment. The ratio of cone resistance and sleeve resistance after treatment can then be compared with the values prior to compaction. From the sleeve resistance ratio (increase in horizontal stress), the overconsolidation ratio, *OCR*, can be calculated according to Eq. 2.

In the case of DMT measurements, at first the material index, I_D and the measured horizontal stress index, K_D are shown. Thereafter, the horizontal stress index after treatment can be compared with the values prior to compaction. From the horizontal stress ratio, *OCR* is determined using Eq. 13.

8.2 Resonance Compaction, Antwerp, Belgium

The resonance compaction method was applied in connection with the construction of a container harbor in Antwerp. The soil to be improved consisted of a 6 to 11 m thick sand fill, placed behind a concrete retaining wall. The project has been described by van Impe et al. (1993). The objective of compaction was to reduce the risk of settlement from static loading (60 kPa) and from cyclic loading due to heavy vehicle traffic. In order to meet the requirements with regard to total and differential settlement, compaction was required to increase the cone resistance to at least 6 MPa. At the beginning of the project, compaction trials were performed using different compaction grid spacings, ranging from 2 to 5 m. The duration of compaction varied between 10 and 15 minutes, depending on compaction depth. In the trial area, the compaction depth was 7 m.

Compaction was carried out using a hydraulic vibrator of type MS 50HFV with an eccentric moment of 50 kgm. The vibrator frequency was varied between 10 and 30 Hz. The maximum movement amplitude (of the suspended vibrator) was 26 mm. At the maximum frequency, the vibrator generated a 1,500-kN centrifugal force.

The resonance compaction system takes advantage of the vibration amplification effect which occurs when the vibrator operates at the resonance frequency of the vibrator-probe-soil system. The operating frequency of the vibrator can be adjusted gradually with the aid of a monitoring and process control system. The vibration frequency is varied during the compaction process, using high frequencies during the penetration and extraction phase. The vibration response of the ground is measured by geophones placed in the vicinity of the compaction probe. The optimal compaction process is determined during field trials. The effect of compaction is verified by in-situ tests (CPT and DMT) before and after compaction. The application of resonance compaction has been described by Massarsch and Fellenius (2017).

8.3 Results of CPT Investigations

The results of the CPT investigation are shown in Figures 9a (cone resistance) and 9b (sleeve resistance). The cone resistance prior to compaction shows a stiff surface layer down to about 2 m depth. Below follows loose sand to a depth of 8 m. After compaction, the cone resistance increased to between 10 and 20 MPa, with the exception of a fine-grained layer, which can be detected from the higher friction ratio, cf. Figure 10.

The sleeve resistance displays a similar soil profile, with a dense surface layer and low sleeve resistance below 2 m depth. After compaction, the sleeve resistance between 6.5 and 7.5 m depth, increased from about 20 kPa to 80 kPa, with the exception of the fine-grained layer between 6.5 and 7.5 m depth.

It is interesting to note that due to the stronger increase of sleeve resistance, compared to the cone resistance, the friction ratio increased markedly after compaction, suggesting a change in material from silty to sandy soil. However, it could be verified that the soil particle size did not change due to compaction. Thus, the change in friction ratio is merely a consequence of changes in horizontal stress.

Figures 11 shows the friction resistance ratio after compaction. The stiff surface layer has been disregarded in the evaluation. Below 2 m depth, the sleeve resistance ratio varies between 2 and 3, with the exception of the layer below 7 m depth, which was already preconsolidated, having a relatively high sleeve resistance prior to compaction. The overconsolidation ratio was derived from the sleeve resistance ratio and clearly shows the preconsolidation effect resulting from resonance compaction, cf. Figure 12. The low *OCR* values below 7 m depth are purely the result of the conservative assumption that this layer was normally consolidated. This is clearly not the case as can be seen from the relatively high cone resistance and sleeve resistance values prior to compaction.

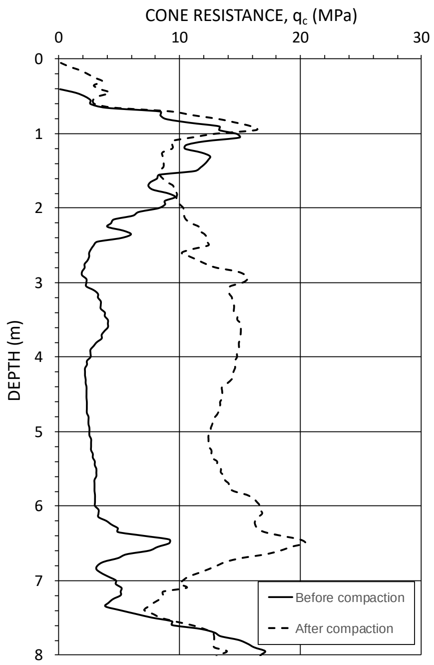

Figure 9a Cone resistance prior to, and after resonance compaction, Antwerp

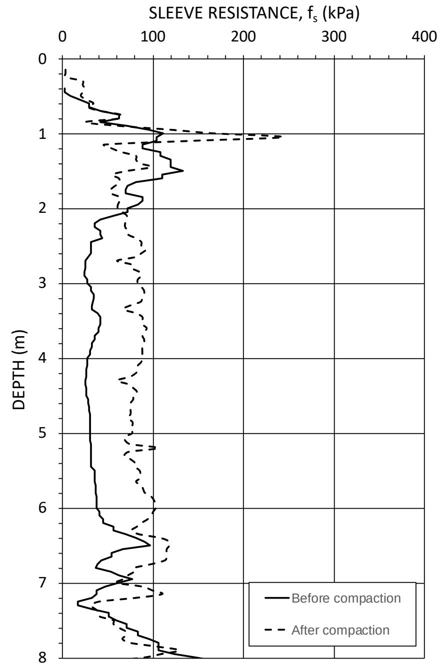

Figure 9b Sleeve resistance prior to, and after resonance compaction, Antwerp

8.4 Results of DMT Investigations

The results of the DMT investigations are shown in Figures 13a (I_D) and 13b (K_D). The material index before compaction is relatively low, suggesting some silt content, which is in agreement with the friction ratio diagram, cf. Figure 10.

After compaction, the material index changed, indicating more sandy soil and an apparent reduction in fines. As has been stated above, this effect is due to the change in horizontal stress, which is included in the determination of the material index.

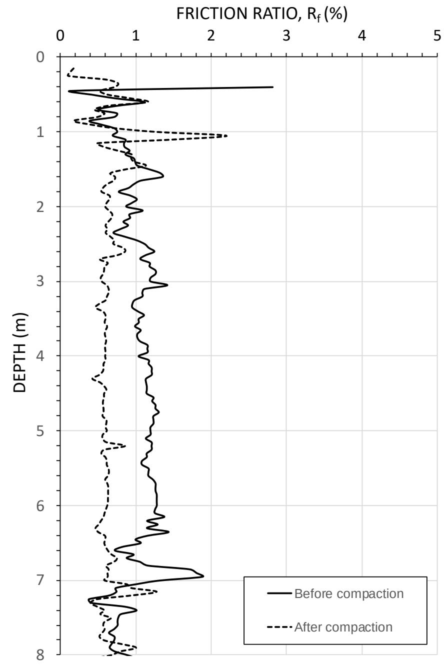

Figure 10 Friction ratio before and after resonance compaction, Antwerp

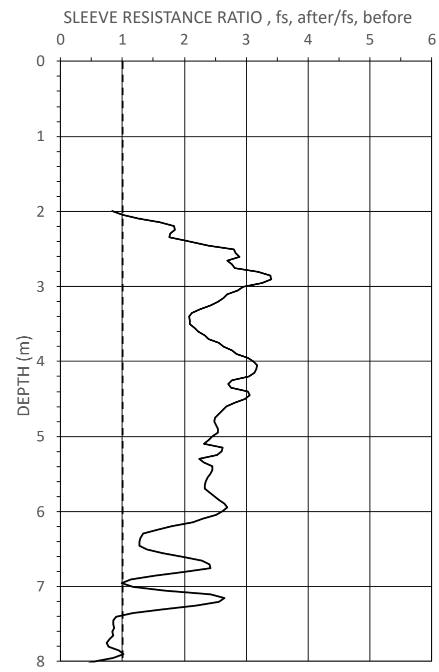

Figure 11 Change in sleeve resistance after compaction

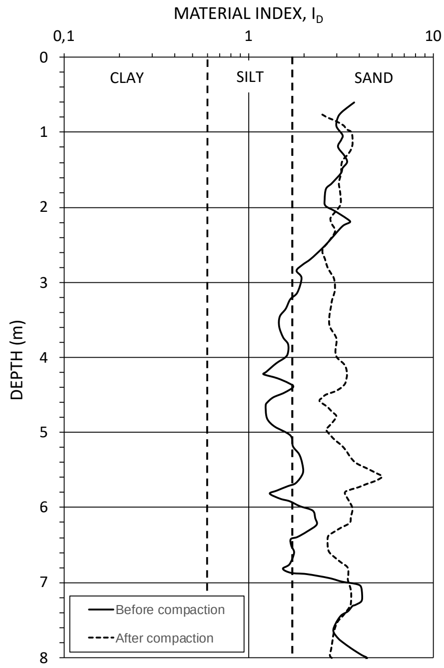

Figure 13a Material index measured prior to, and after resonance compaction, Antwerp

The horizontal stress index in Figure 13b is high in the upper layer down to about 2 m depth and is rather low in the loose soil down to approximately 7 m. In the dense bottom layer, the horizontal stress index increases again. After resonance compaction, the horizontal stress index increased throughout the compacted soil deposit, with very high values in the layer down to about 3 m depth.

The high horizontal stress values in the top layer are likely due to compaction caused by the movement of heavy construction equipment. The change in horizontal stress is shown in Figures 13b.

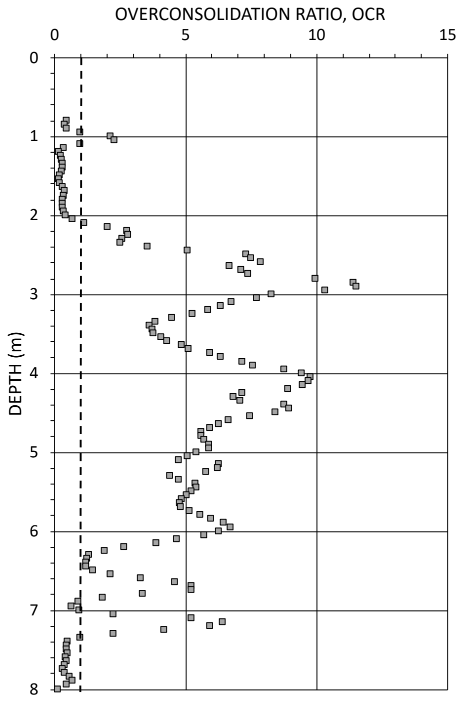

Figure 12 Increase in overconsolidation ratio due to compaction

The horizontal stress ratio, shown in Figure 14, increased on average between 2 and 5, and is thus somewhat higher than the increase determined from the sleeve resistance measurements.

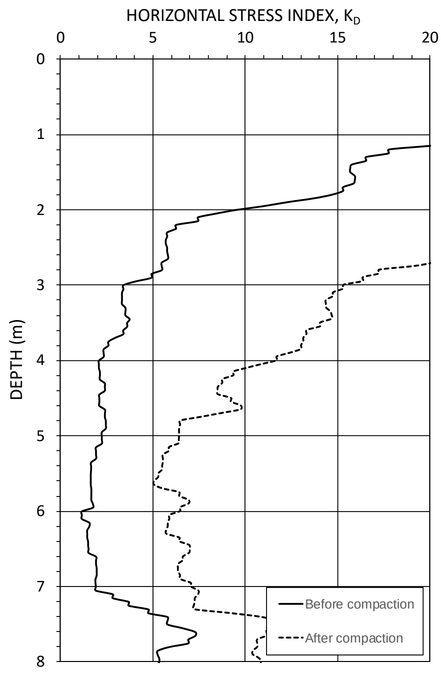

Figure 13b Horizontal stress index measured prior to, and after resonance compaction, Antwerp

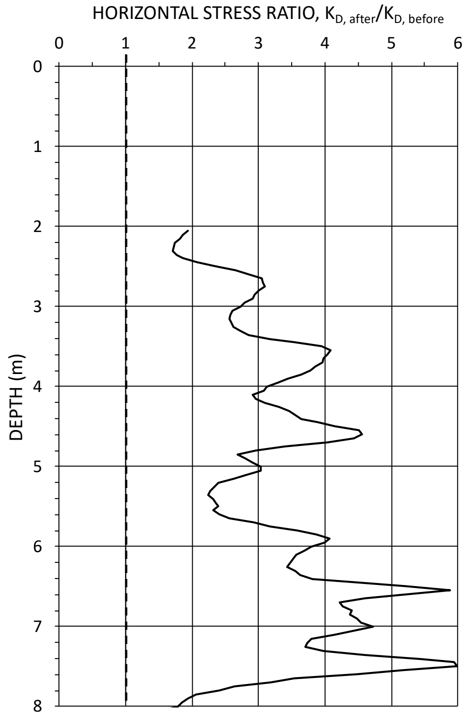

Figure 14 Change in horizontal stress due to compaction

As a result of the high horizontal stress ratio, very high values of the overconsolidation ratio, OCR are derived, ranging from 5 to 25, cf., Figure 15.

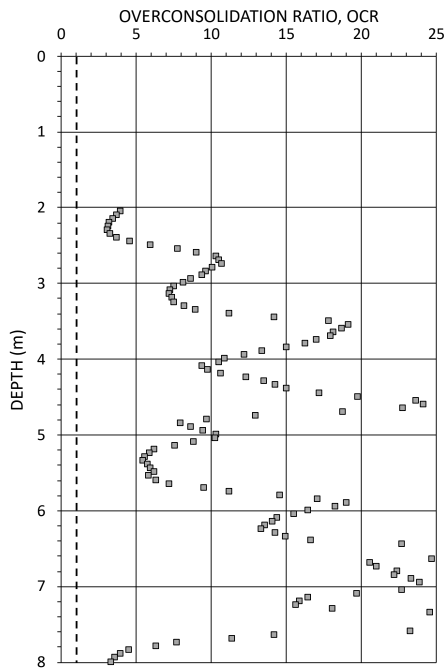

Figure 15 Increase in overconsolidation ratio due to compaction

9. CONCLUSIONS

Design of compaction projects is frequently done without taking into consideration the consequences of changes in horizontal stress. Neglecting this important aspect usually results in unnecessarily conservative compaction requirements, at the same time limiting the practical application of soil compaction.

The two most important applications of deep compaction are reduction in total and differential settlement and mitigation of liquefaction hazard. A major detrimental factor is that compaction design almost exclusively is based on density index (relative density) or cone stress, neglecting the important information provided by sleeve resistance measurement (CPT) or horizontal stress index (DMT).

Soil compaction has two main effects on granular material: a) compression of soil structure, resulting in higher stiffness (modulus) and b) increase in horizontal stress (preconsolidation stress).

It is difficult to predict the in-situ stress conditions of natural soil deposits. However, field testing methods, such as the CPT and the DMT, provide valuable insight into the change of soil properties and stress conditions, following compaction.

CPT investigations reported by Baldi et al. (1986) in compression chamber tests, (CC), were re-analyzed with respect to sleeve resistance. The results demonstrate that CPT sleeve resistance is strongly affected by horizontal stress. However, CC-based data do not allow a reliable determination of the sleeve resistance ratio as function of *OCR*. An important factor is that preparation of sand specimen to a high density index can induce high, uncontrolled horizontal stresses before the start of the test, which affect the interpretation of test data.

Lee et al. (2011) presented carefully performed DMT CC-results, at different values of density index, I_D, and overconsolidation ratio, *OCR*. Figure 4 shows a consistent relationship between horizontal stress index, K_D, and overconsolidation ratio, *OCR* as a function of density index, I_D.

Moreover, when normalizing the horizontal stress index by a reference value, $K_{D\ ref}$ (at $OCR = 1$), a distinct relationship was obtained (Figure 5) that showed agreement of the stress exponent, (β = 0.48) with published correlations from CPT investigations and laboratory tests. It can be assumed that the DMT relationship gives reliable results for uncemented, un-aged silica sands, such as hydraulic sand fill (excluding calcareous material). Comparison of the CPT CC and DMT CC tests suggested that the DMT provides more reliable information on the horizontal stress changes in a soil deposit.

Settlement analyses require the determination of two important in-put parameters, the vertical, constrained soil modulus, *M*, and the preconsolidation stress, σ'_{pc}. It is not generally appreciated that due to compaction, the soil deposit becomes permanently overconsolidated. This effect is reflected by the increase in horizontal stress after compaction.

When in-situ tests (CPT or DMT) are performed in uncompacted sand, the then derived soil properties correspond to the normally consolidated state. This assumption is conservative as on occasions, the soil may be overconsolidated prior to treatment, for instance close to the ground surface. When in-situ tests are performed after compaction, CPT and DMT provide information of the soil in the overconsolidated stress range.

It is possible to estimate the overconsolidation ratio (*OCR*) based on the increase in horizontal stress, cf. Eq. 13 and Figure 5. Based on the *OCR* it is possible to estimate the stress range within which the compacted soil behaves as overconsolidated material.

The soil modulus beyond the preconsolidation stress cannot be determined directly from in-situ tests, but can be estimated based on engineering judgment. It is conservative to assume that beyond the preconsolidation stress, the soil modulus is equal to somewhat higher or, at least, equal to that prior to compaction.

Another important application of deep compaction is mitigation of liquefaction hazard. Extensive laboratory tests on different sands clearly demonstrate that a change in *OCR* determined by in-situ tests performed before and after compaction has a strong beneficial effect on soil resistance to cyclic loading. It is essential in the compaction design that the increase in horizontal stress (and thus of *OCR*) is taken into consideration.

A case history of vibratory compaction has been re-analyzed to investigate whether horizontal stresses were increased after compaction. Changes in horizontal stress were measured by CPT sleeve resistance and DMT horizontal stress index. Both methods show a significant increase in horizontal stress, resulting in an increase in overconsolidation ratio.

Finally, it must be emphasized that the change in horizontal stress due to compaction is an important aspect not only for the analysis of settlement or assessment of liquefaction susceptibility. With the increasing use of advance analytical methods, such the FEM, the selection of relevant input parameters becomes even more important.

10. ACKNOWLEDEMENTS

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