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THE PRACTICAL APPLICATION OF SEISMIC TESTING IN GEOTECHNICAL ENGINEERING

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Abstract: In the paper, a concept of assessing deformation based on seismic tests is presented. The effect of strain influence on shear modulus has been emphasized and shown in soils of different types. The assessment of deformation properties on the basis of seismic tests has been illustrated by three case histories.

1. INTRODUCTION

Seismic testing methods were originally developed with the aim to characterize soil and rock formations and to locate the ground water level. The objective was therefore primarily to establish the boundaries between materials of different wave propagation velocities. The increasing need to analyze earthquake and off-shore engineering projects, which require information about dynamic soil properties, resulted in a rapid development of different types of seismic field and laboratory investigation techniques. Many innovative testing and analytic methods were developed, which were made possible by new, powerful electronic measuring systems and analytical tools (STOKOE and SANTAMARINA [9]). Recently, seismic testing methods have been used for the definition of static geotechnical problems, such as determining the properties of soil deformation (MASSARSCH [8]). Direct measurement of soil or rock stiffness in the field has the advantage of minimal material disturbance. The modulus is measured where the soil exists. Furthermore, the measurements are not affected by the size of a sample.

The objective of the paper is to present a concept of assessing static deformation properties (moduli) of soils based on seismic tests. The application of seismic measuring techniques to the solution of practical foundation problems in fine-grained and coarse-grained soils is illustrated by three case histories.

2. SEISMIC MEASURING TECHNIQUES

The techniques for measuring wave velocity in situ fall into three categories: cross-hole, down-hole, and uphole. All require that borings be made in the soil. In the cross-hole method, sensors are placed at one elevation in one or more borings and a source of energy is triggered in another boring at the same elevation. The waves travel horizontally from the source to the receiving holes. In the down-hole method, the sensors are placed at various depths in the boring and the source of energy is above the sensors – usually at the surface.

This technique does not require as many borings as the cross-hole method, but the waves travel through several layers from the source to the sensors. The seismic cone downhole method (SCPT) has become popular for determining the shear wave velocity in soils and guidelines have been prepared by ISSMGE TC 10 – Geophysical Testing in Geotechnical Engineering, (BUTCHER et al. [1]). In the up-hole method, the source of the energy is deep in the boring and the sensors are above it – usually at the surface. A recently developed technique that does not require borings is the spectral analysis of surface waves (SASW). This technique uses sensors that are spread out along a line at the surface, and the source of energy is a hammer or tamper also at the surface. The surface excitation generates surface waves, in particular the Rayleigh waves. These are waves that occur because of the difference in the stiffness between the soil and the overlying air. The test results are interpreted by recording the signals at each of the receiving stations and using a computer program to perform a spectral analysis of the data.

Two types of body waves can be used for seismic tests, compression waves (P-waves) and shear waves (S-waves). The compression wave travels faster and thus arrives first at the observation point. The S-wave is slower but has an important advantage that its propagation velocity is not affected by ground water. Also, due to the lower propagation speed, the S-wave velocity can be measured with greater accuracy, as the arrival time interval is longer than in the case of P-waves.

3. DYNAMIC SOIL PROPERTIES

The primary result of a seismic investigation is the wave propagation velocity of either P-waves or S-waves. From the P- and S-wave velocities the shear modulus at small strains $G_{\max}$ and the oedometer (constrained) modulus $M_{\max}$, can be calculated from the following relationships

$$G_{\max} = \rho C_S^2, \tag{1}$$

$$M_{\max} = \rho C_P^2, \tag{2}$$

where ρ is the bulk density of the soil and C_P and C_S are the P-wave velocity and S-wave velocity, respectively. Shear waves propagate at very low strains, and the shear strain level γ can be estimated from the following relationship

$$\gamma = \frac{x}{C_S}, \tag{3}$$

where x is the vibration velocity amplitude. If, for instance, the vibration amplitude is 0.1 mm/s and the shear wave velocity is 100 m/s, then the shear strain level is 10^{-4}%. At such a low strain level, the soil is in the elastic range.

It is often assumed that the rate of deformation during a dynamic test is high. This is not correct, as the strain amplitude is very low. Consequently, the strain rate of a seismic test is comparable to that of a static test, MASSARSCH [8]. Rather, the reason for the difference between the "seismic modulus" and the "static modulus" is that usually the seismic modulus is determined at a much lower strain level than the static modulus.

The magnitude of the shear modulus depends on the strain level as illustrated by figure 1 which shows a typical shear stress–shear strain relationship at undrained loading. Three commonly used definitions of the shear modulus are indicated. At very low stress levels (very low strains), the shear modulus is called the maximum shear modulus $G_{\max}$. With increasing stress level the shear modulus decreases. At a stress level corresponding to 50% of the failure stress the term G_{50} is frequently used, which corresponds to a factor of safety typical of normal operating conditions. At failure the shear modulus is defined as G_f.

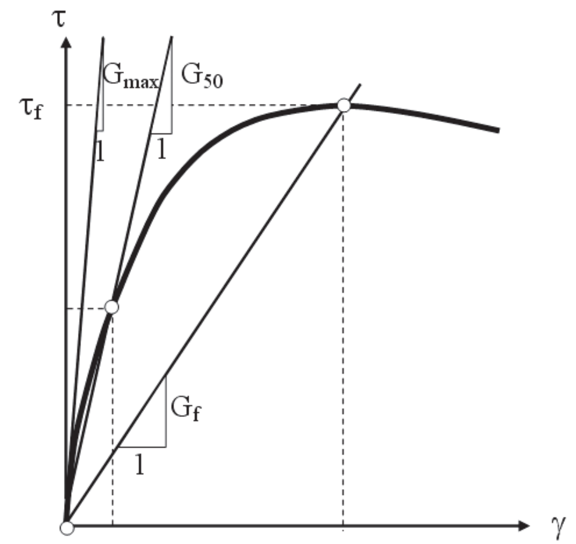

Fig. 1. Shear stress–shear strain relationship
for fine-grained soil at undrained loading

The stress–strain relationship for the case of repeated loading is shown in figure 3. The initial loading curve $G_{\max}$ and the unloading–reloading curves are shown. It is a common practice to define the stress–strain relationship of soils by the secant modulus G_s. Note that at unloading and reloading the modulus is often assumed to correspond to the modulus at initial loading $G_{\max}$.

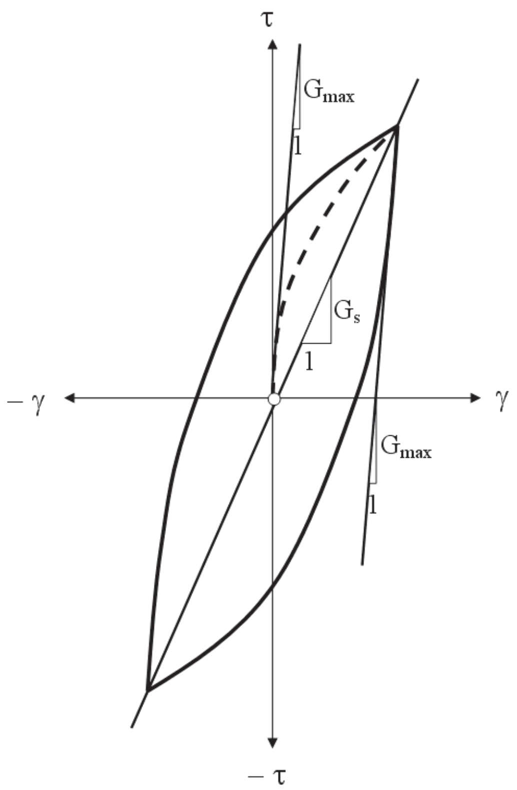

Fig. 2. Stress–strain relationship during shear for soils at repeated loading

4. EFFECT OF STRAIN ON SHEAR MODULUS

The shear modulus is affected by stress level and thus by strain level. The measuring accuracy of conventional laboratory testing devices is limited and these cannot usually measure G_{max}. On the other hand, the resonant column (RC) test can measure shear strain levels down to 10^{-4}% or lower with high precision. Figure 3 shows the results of a RC test on a reconstituted sample of silty clay, DRNEVICH and MASSARSCH [2]. The test was performed at a vibration frequency of approximately 30 Hz. At shear strains lower than 10^{-3}% the shear modulus is almost constant (G_{max} = 77 MPa). However, with increasing shear strains, the modulus decreases markedly and at 0.1% shear strain is equal to 24 MPa, i.e., only 30% of the maximum value. In conventional laboratory tests, the first data readings would usually be taken at this strain level.

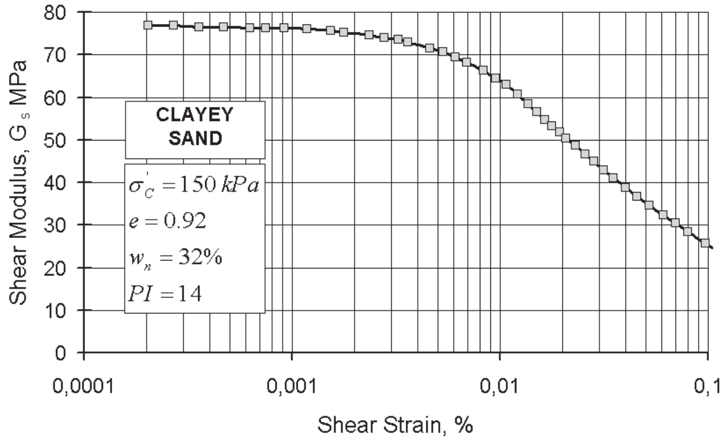

Fig. 3. Change of shear modulus with shear strain determined from resonant column test, after DRNEVICH and MASSARSCH [2]

It is thus not surprising that conventional laboratory tests grossly underestimate soil stiffness. MASSARSCH [5] reported the results from resonant column tests on a variety of fine-grained soils. Figure 4 shows these results with the normalized shear modulus $G_s/G_{\max}$ as a function of shear strain in linear scale. It can be seen that *PI* has a strong influence on the degradation of the shear modulus. The shear modulus decreases more rapidly in low-plastic soils.

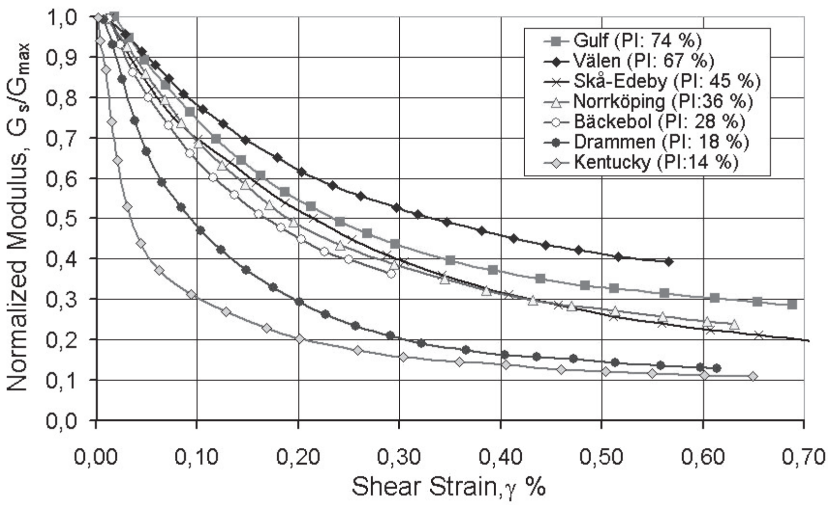

Fig. 4. Normalized stress–strain relationship of silts and clays, determined from RC test (MASSARSCH [5])

From figure 4 a shear modulus reduction factor R_m of fine-grained soils can be estimated:

$$G_{\text{stat}} = R_m G_{\max} \tag{4}$$

which is strongly dependent on the plasticity index *PI*. At 0.5% shear strain the modulus reduction factor varies between approximately 0.50 and 0.10. For clays with a *PI* of about 35 to 45% R_m is about 0.3. Approximate values of R_m of granular soils at approximately 0.5% shear strain can be obtained from table 1.

Table 1

Modulus reduction factor R_m of granular soils at approximately 0.5% shear strain, MASSARSCH [4]

Soil type	Reduction factor R_m
Gravel	0.20
Sandy gravel	0.19
Loose sand	0.18
Medium dense sand	0.15
Dense sand	0.12

It is interesting to note that the static modulus in sand is only about 10 to 20% of the dynamic modulus. Young's modulus E_{stat} and the constrained (oedometer) modulus M_{stat} can be readily calculated from the shear modulus G_{stat} (Poisson's ratio of $\upsilon = 0.3$)

$$E_{\text{stat}} = 2.6\ G_{\text{stat}}, \tag{5}$$

$$M_{\text{stat}} = 3.5\ G_{\text{stat}}. \tag{6}$$

From the above relationships it is obvious that Young's modulus and the constrained modulus are significantly larger than the shear modulus. This aspect must be taken into account when evaluating the results of dynamic soil tests.

5. DEFORMATION PROPERTIES OF LIME-CEMENT COLUMNS

In connection with the expansion of the railway link north of Uppsala, Sweden, an up to 7 m high embankment had to be constructed on very soft, compressible soil. The main objective of the project was to increase the train speed from 130 to 160 km/h. The most common ground improvement method for such problems in the Nordic countries is the installation of lime-cement (LC) columns, using the dry mixing method. While different methods can be used to determine the geotechnical parameters of the columns, only limited data is available on their static and dynamic deformation behaviours. Therefore, extensive field and laboratory investigations were carried out.

The test area consisted of soft, organic clay to a depth of 10 m. Below a 1 m thick dry crust we deal with 2 m of organic clay (gyttja), 2 m of sulfide clay (locally known as "svartmocka") and 4 m of silty clay. Moraine (till) was encountered at 9 m depth. The ground water level was located less than 1 m below the ground surface.

Seismic down-hole tests were carried out in the undisturbed clay, adjacent to the test area where the LC columns were installed. The shear wave velocity was measured at 2.5 and 5.5 m depth. The average shear wave velocity of several measurements was 40 m/s. The predominant frequency of the signal was 20 Hz, corresponding to a wavelength of 2 m. The shear wave velocity gave a small strain modulus G_{max} = 2.3 MPa.

Seismic down-hole tests were also performed at different time intervals after installation of the LC columns. Figure 5 shows the shear wave signal in a column with a mixing quantity of 44 kg/m (156 kg/m^3), 41 days after installation. The measuring points are located at 2.5 and 5.5 m depth. Two tests with polarized signals in the opposite direction are superimposed.

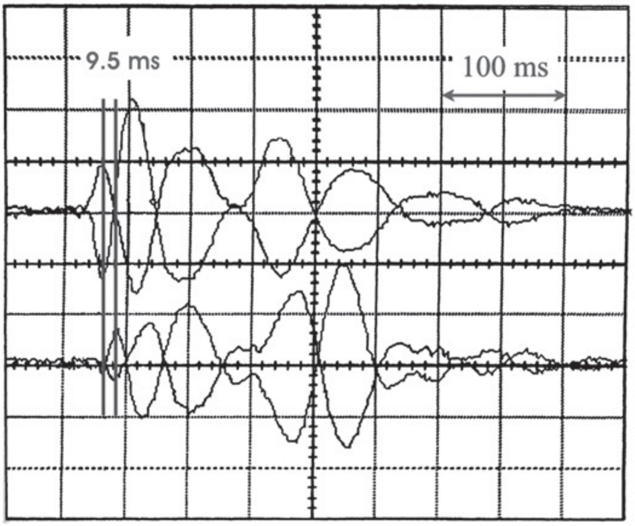

Fig. 5. Signal from reverse impact test (depth interval from 2.5 to 5.5 m) in column with a mixing quantity of 44 kg/m (156 kg/m^3), 41 days after installation

It is relatively easy to identify the arrival of the first (or second) peak of the shear wave. However, it is not equally easy to determine the first arrival of the shear wave. The time interval between the first peak of sensor 1 and the first peak of sensor 2 is 9.5 ms, resulting in a shear wave velocity of 316 m/s. A more accurate and consistent method of determining the travel time is by cross-correlation of the two shear wave signals (figure 6).

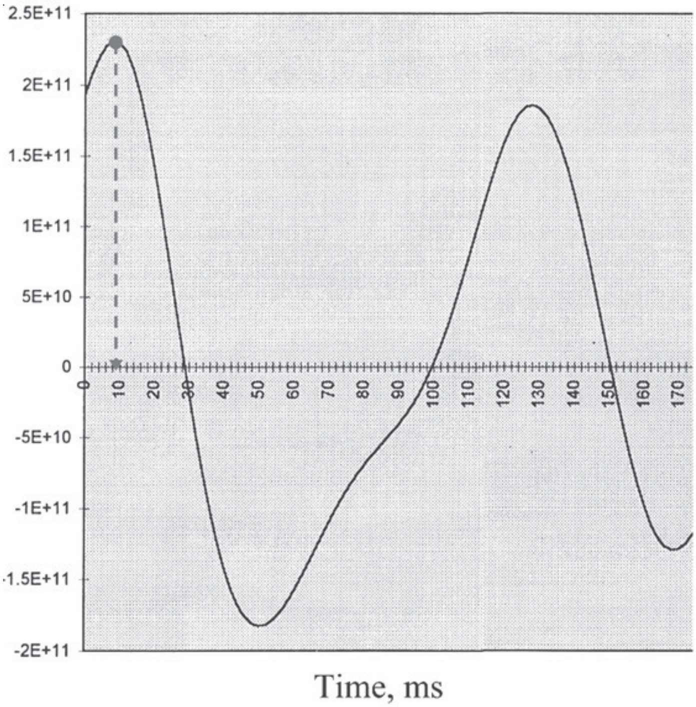

Fig. 6. Determination of the shear wave velocity by cross-correlation of the signals shown in figure 15

There is a good agreement between interpretation methods, both of which give a travel time of 9.5 ms. The predominant period of the shear wave is 72 ms, which corresponds to a frequency of 14 Hz. Assuming a shear wave velocity of 316 m/s, the wavelength is 23 m, which is significantly longer than in the clay. Based on the shear wave velocity and assuming the density of 2 t/m^3, the small-strain modulus, 41 days after construction, is $G_{\max}$ equal to 199 MPa.

The values of the deformation modulus obtained from static and seismic tests in the field and in the laboratory can be compared. Based on the seismic down-hole tests in the field, the increase of the shear modulus with time after installation of LC columns can also be studied. In order to compare the axial strains ε_a from triaxial compression tests with the shear strains γ from seismic tests, the following relationship is used

$$\Delta\gamma = (1+\nu)\Delta\varepsilon_a \,, \tag{7}$$

where ν is Poisson's ratio. At large strains (> 0.1%) and undrained loading it can be assumed that $\nu = 0.5$. Thus, $\gamma = 1.5\varepsilon_a$. As has been shown above, the main difference between the moduli determined from static or seismic tests is strain level, while the rate of loading is practically the same for both test types.

The shear wave velocity was determined in-situ by down-hole tests at different time intervals after installation of two LC columns with different mixing quantities. In figure 7, the increase of shear wave velocity with time is shown. The shear wave velocity of the clay before improvement was 40 m/s. Within 41 days (approx. 1000 hrs), the shear wave velocity increased to 310 m/s. There are also indicated the values of shear wave velocity from the bender element tests (measured after 116 days). These tests suggest that the shear wave velocity continued to increase to about 360 m/s. The measurements give average values in the columns between 2.5 and 5.5 m depth.

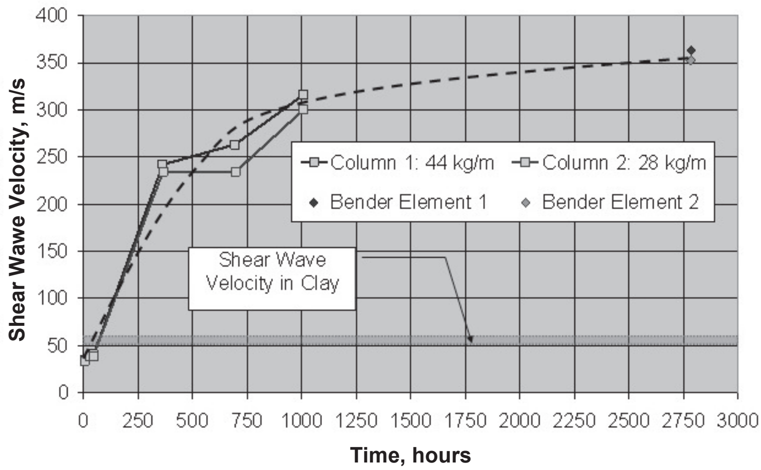

Fig. 7. Variation of shear wave velocity with time, determined in-situ by down-hole tests and in the laboratory by bender element tests

The maximum shear modulus G_{max} can be readily calculated from the shear wave velocity. It was assumed that the density of the LC columns was 2.0 t/m^3, and that of the organic clay – 1.4 t/m^3. The maximum shear modulus in the undisturbed clay was 2.3 MPa and increased after dry mixing within 41 days to about 190 MPa. The maximum shear modulus from bender element tests, performed after 116 days, reached an average value of 255 MPa. It can be assumed that the shear modulus will increase further with time. No significant difference in shear modulus could be observed between the two mixing quantities.

The seismic tests clearly demonstrated that the shear modulus increases with time after installation of LC columns. Approximately 100 days after installation, the shear wave velocity increased in the columns to about 355 m/s. Based on limited data it can be assumed that the curing period is at least 100 days, but probably longer. The maximum shear modulus of the LC columns was G_{max} = 255 MPa at 0.001% shear strain. The shear modulus of LC columns appears not to be influenced significantly by confining stress.

6. EXCESS PORE PRESSURE DUE TO PILE DRIVING IN CLAY

The excess pore water pressure during driving of displacement piles in clay was analysed theoretically based on cavity expansion theory. The concept presented in this paper was used to predict excess pore water pressure more reliably, MASSARSCH and BROMS [3]. Figure 8 shows the variation of excess pore water at increasing distance from the pile. An important parameter is the soil stiffness ratio G/τ_f. In order to obtain a realistic prediction of the excess pore water pressure, the shear modulus and the undrained shear strength need to be estimated. The range of stiffness ratios can vary between 10 and 50 and it is important to select realistic values for the pore pressure prediction.

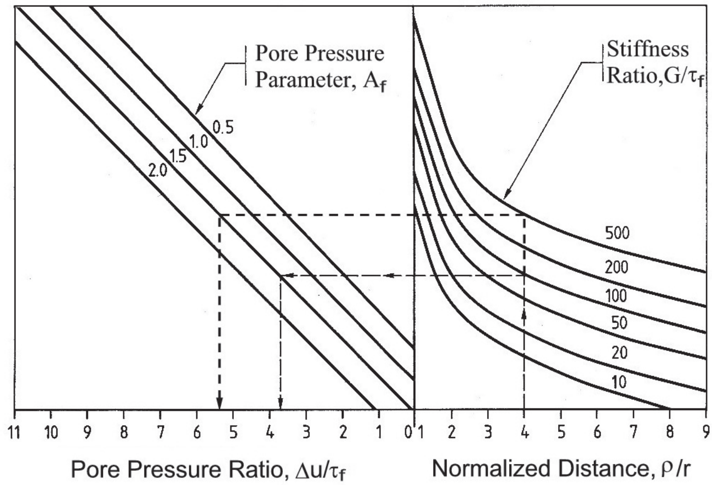

Fig. 8. Relationship between the excess pore water pressure in the vicinity of an expanding cavity for different stiffness ratios, MASSARSCH and BROMS [3]

The stress–strain relationship shown in figure 4 suggests that at failure (i.e., close to the pile) the shear modulus can be as low as 15% of the initial modulus value. At the test site, the shear modulus at the shear strain level of 0.0001% was determined by cross-hole measurements G_{max} = 7600 MPa. The undrained shear strength was 15 kPa, resulting in a soil stiffness ratio of G/τ_f = 506. At high shear strain level, the shear modulus decreased to about 15% of the maximum value (1140 kPa), resulting in a reduction of the soil stiffness G/τ_f = 76. This effect of strain-softening is significant for the outcome of the prediction but is many times neglected. A comparison between measured excess pore water pressure and the values predicted based on figure 8 and taking into account the strain softening effect gave good agreement, (MASSARSCH and BROMS [3]).

7. MONITORING OF SOIL COMPACTION

Seismic testing methods are useful for the estimation of ground settlements prior to compaction, but also for the verification of the compaction effect. They measure the wave propagation velocity between two or several sensors, located in the ground. One of the advantages of seismic testing for compaction control is that the properties of a large soil volume can be determined, MASSARSCH and BROMS [3]. A dynamic test provides thus often more representative results of soil compaction than penetration tests, which are performed in single points.

Special dynamic displacement sensors have been developed for soil compaction monitoring, which are inexpensive compared to accelerometers and geophones, and thus can be left in the ground after the tests (MASSARSCH and WESTERBERG [7]). They are sensitive to horizontal ground excitation and can be installed at several depths and locations in the area to be monitored. If an array of sensors is installed at known distances, both cross-hole and down-hole tests can be carried out simultaneously. The test is fast and can be repeated without affecting the soil. The seismic tests are carried out using a plate which is placed on the ground surface above the sensors, cf. figure 9.

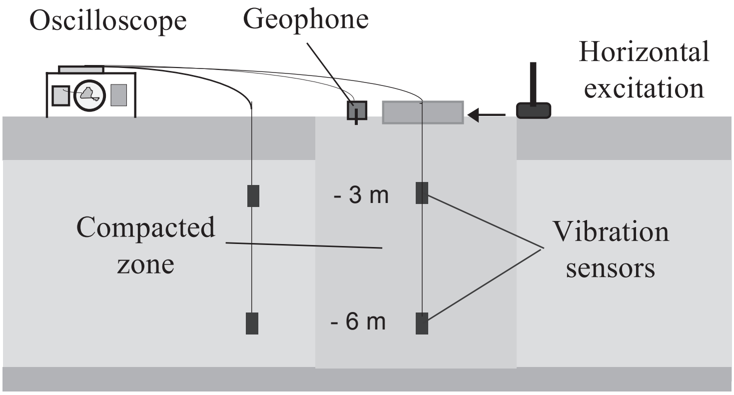

Fig. 9. Principle sketch of dynamic cross/down-hole test

A horizontally polarised shear wave is generated by a hammer blow to the side of a plate anchored to the ground. Alternatively, vibrations generated by the compaction probe can be used as source for the vibration signal. The arrival time of the shear wave at the different sensor locations is measured by a digital oscilloscope. From the known distances between the vibration sensors the shear wave velocity can be calculated. At the ground surface the departure of the shear wave is also measured by a geophone which is placed in the vicinity of the vibration source. The recorded wave velocities are stored in a personal computer and can be evaluated by specially developed software. Tests can be repeated over a long time period, thereby making it possible to investigate a possible change in soil stiffness (modulus) with time.

The resonance compaction system (MRC) was used at the Map Ta Phut site in Thailand (MASSARSCH [6]). The objective of soil compaction was to improve a hydraulic fill where tanks of varying sizes and heights were initially planned to be founded on piles. However, a cost comparison showed that soil improvement of the entire site by MRC compaction (including the reduced costs for foundations of the individual tanks) was cheaper than pile foundations for the tanks only. The requirements regarding differential settlements were stringent as well as the considerations of settlements caused by traffic and construction activities between the various installations at the tank farm. The client required a high degree of quality control and took active part in supervising the project during the initial phase of compaction.

The area to be treated was reclaimed from the sea and consisted of hydraulic sand fill of about 10 m thickness. The ground water table was located approximately 4 m below the ground surface. The geotechnical properties of the soil deposit were investigated by extensive field and laboratory tests. Typical test results are summarised in table 2. The test results from 10 SPT investigations within the area were averaged.

Figure 10 shows a typical cone penetration test with sleeve friction measurements and friction ratio before compaction. Above the ground water table the hydraulic fill is loose to compact (q_c: 10–16, f_s: 35–45 kPa). Below the ground water table the soil becomes loose to very loose down to a depth of 9 m (q_c: 3–8, f_s: 2–10 kPa). Some layers of silty sand and clay occurred in the hydraulic fill and below, which is exemplified by an increase of the friction ratio *FR*.

Table 2

Typical geotechnical properties of major typical soil layers

Depth (m)	Layer thickness (m)	Layer (#)	Soil description	SPT *N*-value
3	0–3	1	dense fine sand	10–15
9	6	2	hydraulic fill loose fine sand	3–10
13	4	3	loose clayey sand	5–15
19.5	6.5	4	stiff silty clay and clayey sand	35–45
>19.5	–	5	decomposed granite	>50

At the start of the project, compaction trials were carried out at different grid spacings, compaction sequences and varying vibration duration. In order to determine the optimal compaction procedure, the resonance frequency was determined before, during and after soil compaction. Trial compaction was carried out in a rectangular grid with gradually decreased compaction point spacing. The final compaction spacing was 4.0 m × 4.4 m in a rectangular grid. The duration of compaction in each compaction point was varied, depending on the initial soil conditions and the required degree of compaction. In some areas with layers of silty and clayey sand, a third compaction pass was required.

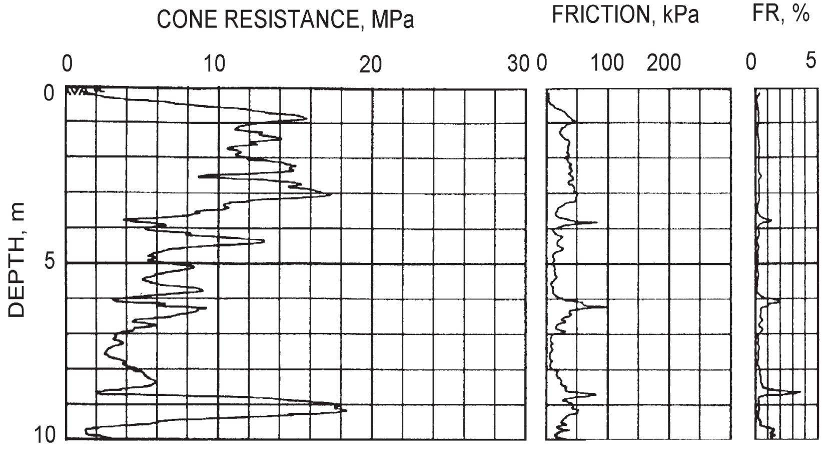

Fig. 10. Cone penetration test before compaction of the hydraulic fill

Extensive dynamic field tests were performed to investigate the soil modulus before, during and after compaction, cf. figure 9. Shear wave velocity (down-hole) measurements were made before start of soil compaction in 6 test locations. Thereafter, compaction was carried out in (up to) five passes with decreasing compaction point spacing. Seismic tests were performed several times during compaction passes as well as at regular time intervals after compaction.

The resonance frequency (the first overtone) was during the initial compaction phase around 11 Hz. After the second compaction pass, the resonance frequency had increased, on average, to 19 Hz, indicating an increase of the ground propagation velocity by 70%.

Evaluation of wave velocities showed that immediately after compaction the shear wave velocity increased (between 3 and 6 m depth) from around 120 m/s to more than 200 m/s. These values are in good agreement with the results from dynamic downhole tests reported by MASSARSCH and BROMS [3]. The shear modulus increased by a factor of 2.8. The small-strain shear modulus increased from 18 MPa to approximately 72 MPa. Using a modulus reduction factor of 0.15 the shear modulus after compaction was 11 MPa. Assuming that Poisson's ratio ν assumes the value of 0.3 yields E_{stat} = 28 MPa and M_{stat} = 37.5 MPa, respectively. These values of the modulus are in good agreement with the settlement calculations based on cone penetration tests, MASSARSCH [6]. The dynamic soil tests indicate that the shear modulus increased by a factor of about 4. This, however, does not include the increase of lateral stress changes as discussed in a previous section.

8. CONCLUSIONS

The assessment of deformation properties is an important part of geotechnical design. Major progress has been made in earthquake engineering and soil dynamics, and reliable methods exist for the determination of deformation properties of soils even at very low strain levels (down to 0.0001% shear strain).

As is well-known, shear strain affects soil stiffness. The reduction of the shear modulus for a wide range of fine-grained soils was determined at three strain levels (0.1, 0.25 and 0.5% shear strain). The modulus reduction factor is strongly affected by the plasticity index. The shear modulus decreases more rapidly in silts and silty clays, while the effect of shear strain is less pronounced in soils with high plasticity. A numerical relationship is proposed, using a modulus reduction factor R_m.

The application of the proposed concept was exemplified by three case histories. The stiffness and its variation with time of LC-columns were determined in the field by seismic down-hole tests. The excess pore water pressure during driving of displacement piles in clay could be predicted more reliably using the small-strain modulus from seismic tests, taking into account the strain-softening effect. Finally, soil compaction was monitored using seismic down-hole and cross-hole tests. The static deformation moduli could be predicted using the modulus reduction factor.

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