Interpretation of in situ test results in cohesionless fills before and after ground improvement

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Abstract: The initial density, stress state and fabric of fills govern their engineering behaviour and are dependent upon the mode of deposition of the fill. These parameters also govern the response to in situ testing in these materials. The engineering properties of fills are commonly assessed using in situ testing, with in situ test parameters such as penetration resistance or shear wave velocity used as indices of engineering behaviour. For fills displaying values of penetration resistance or shear wave velocity below target values, ground improvement may be specified. Ground improvement alters density, stress state and fabric. Most correlations between in situ test parameters and engineering behaviour were developed for normally consolidated unaged sands and so are strictly inapplicable for assessment of materials with different initial conditions. This paper discusses the effect of initial conditions on the interpretation of in situ test results obtained with emphasis on the influence of lateral stress and age of the deposit. We show that such correlations should be used with caution or misleading results may be obtained.

Introduction

Land reclamation commonly involves placement of fill or improvement of existing soils. Waterfront developments obtain additional land by placing fill materials through water to construct building platforms. Recent examples of major reclamation projects include Chek Lap Kok Airport in Hong Kong and Osaka Airport, Japan, both of which were constructed on massive man-made islands. After fill placement, engineers must decide whether ground improvement is required to enhance stability or to increase soil strength and stiffness for support of foundations or retaining structures.

The engineering properties of fills are dependent on their initial density, stress state and fabric after placement, each of which is affected by the placement technique used. Ground improvement alters all three of these initial conditions by an amount that depends on the specific ground treatment process. Initial density, stress state and fabric also affect the response measured by in situ test techniques. In this paper, we will discuss some of the factors affecting the interpretation of in situ tests in cohesionless fills and improved ground. In particular, we will focus on the effect of lateral stress and age on the results of penetration testing and the measurement of shear wave velocity. It is suggested that cone penetration resistance, q_t, is affected by changes in small strain stiffness, despite the general belief that q_t represents the large strain properties of soils. We show that correlations between penetration resistance and shear wave velocity and soil properties can give misleading results if the origin of the correlation is not carefully considered. Penetration resistance and relative density tend to be used interchangeably, with other major factors affecting in situ test results such as lateral stress and increases in stiffness with age being commonly neglected. We suggest that consideration of age and lateral stress effects in fills and improved ground can explain apparent inconsistencies in the results of in situ testing.

Methods of fill placement

The optimum conditions for fill placement are achieved where the quality of the fill material, the moisture conditions and the degree of compaction can be tightly controlled. Where clean, granular material is not available, adequate performance can still be achieved from fills constructed of poorer quality materials such as those containing significant fractions of clay or silt fines, provided careful construction control of placement moisture conditions and degree of compaction are exercised.

For placement of fills below water, it is not possible to exercise such precise control on fill placement. From shore, it is possible to gradually move a fill out into the water by end-dumping from trucks and pushing with a bulldozer. In deeper water, hydraulic placement can be achieved by bottom dumping or pipeline discharge. In bottom dumping, soil is discharged from large valves from the bottom of a hopper or dump barge. In pipeline discharge, the material is discharged from pipelines, which may be under water, floating or placed on top of the fill. The discharge point may be above or below water level.

The range of apparent density of hydraulically placed fill is large and the exact factors affecting it are not well understood (Sladen and Hewitt 1989). The initial density and stress state of the fill are affected by the mode of deposition. Past experience has shown that the bottom dumping method yields higher penetration resistances than the pipeline discharge method. (Sladen and Hewitt 1989, Jefferies et al. 1988, Lee et al. 1999). It has been suggested that the difference in penetration resistance is related to differences in the energy of placement, fabric and degree of segregation of the different particle sizes in the fill. The more energy the depositing soil has on impact, the more compact it becomes provided that the mass remains coherent. The impact also causes compaction of the previously placed fill. The mechanism of deposition of pipeline slurry in deep water is closer to sedimentary deposition of individual soil particles and would be expected to result in a deposit similar in density and stress state to an alluvial deposit. A further consequence of placement in water is the tendency towards segregation of different particle sizes as finer grained material will settle more slowly than coarse. The material close to surface of the fill is compacted by construction traffic but the deeper material cannot be compacted during placement.

The post-depositional density, stress state and fabric of a fill is thus very dependent on the material type and gradation and on the method of placement. The initial state and fabric are the principal determinants of the stress-strain behaviour of soil. The response to in situ tests carried out to characterize these materials is also affected by these initial conditions and not just by density.

Compaction control

Terzaghi (1955) observed that the mechanical properties of cohesionless sediments depended almost entirely on their relative density. The engineering properties of dense materials are generally better than loose materials. As a result, field control of fill placement has concentrated on attaining a specified minimum density or relative density, D_r.

For hydraulic fills, assessment of density is commonly based on penetration testing, the penetration resistance being used as an index of soil behaviour. If the penetration resistances indicate that the fill does not possess acceptable engineering properties, then ground improvement procedures may be implemented. For sites where liquefaction is a design consideration, penetration resistance is used as an indicator of liquefaction susceptibility (National Center for Earthquake Engineering Research 1997). In recent years, shear wave velocity has gained increased acceptance as an index of initial stress state and density (Jamiolkowski et al. 1998).

It is unusual to develop new correlations between penetration resistance and engineering properties for a specific fill material on a given project and so index testing is based on previously published correlations. Satisfactory application of published correlations requires that the sands used to develop them are similar in properties to the fill material. Review of the literature indicates that most correlations between in situ test parameters and soil properties for cohesionless soils are based on the determination of D_r, which is then used as the basis for determination of other properties such as friction angle and stiffness.

In situ testing – influence of lateral stress

Penetration resistance

Early work on correlations between penetration resistance and soil properties in cohesionless materials was based on the Standard Penetration Test or SPT N-value (Terzaghi and Peck, 1948). Since the 1970's, the electric cone penetration test (CPT) has gained increasing acceptance for site characterization. Correlations between cone tip resistance, q_t, and soil properties have been developed and much research has been carried out on the factors influencing such correlations (Lunne et al., 1997). Most of the early work used relative density, D_r as an intermediate parameter to the determination of soil properties and there is now a tendency to use D_r and penetration resistance interchangeably. This is not necessarily valid as many factors affect penetration resistance. The other main factors affecting penetration resistance are: lateral stress, vertical stress, ageing, fabric, mineralogy, grain size and gradation, fines content, and drainage conditions during penetration. All these factors are interdependent.

Most correlations have been developed using chamber testing. Sand is placed at a known D_r, usually by air pluviation, and is consolidated to a known stress level prior to insertion of a CPT. The properties of the sand are measured either during consolidation in the chamber or by triaxial testing carried out on samples prepared in a similar fashion to the same D_r. The properties from laboratory testing are correlated to the measured tip resistance. Most of the correlations for the assessment of density and engineering properties of cohesionless material have been developed for clean, unaged sands.

A correlation commonly used for derivation of D_r from CPTU data in moderately compressible, normally consolidated young sands such as those of the Fraser Delta, is the relationship between D_r, effective lateral stress σ_h' and q_t for chamber tests on unaged Ticino Sand. It was given by Baldi et al. (1986) to be:

[1] $q_t = 248\, \sigma_h'^{0.55} \exp[2.38 D_r]$.

q_t is thus dependent on $(K_o)^{0.55}$, where $K_o = (\sigma_h'/\sigma_v')$. Based on equation [1], the main factor causing variation in penetration resistance for an unaged sand, other than density, is lateral stress (Houlsby and Hitchman, 1988; Jamiolkowski et al., 1985). The lateral stress is difficult to measure.

Shear wave velocity

Attempts have also been made to use shear wave velocity, V_s, to determine the initial state of cohesionless soils. The shear wave velocity of unaged sand is a function of soil density (or void ratio), grain characteristics, and horizontal and vertical stresses. Bellotti et al. (1996) found that the relationship between V_s, void ratio and effective stress was given by an expression:

[2] $V_s = C_s\, [F(e)]^{0.5} (\sigma_a')^{na} (\sigma_b')^{nb}$

where C_s is a function of grain characteristics, F(e) is a function of void ratio, (σ_a') is the effective stress in the direction of particle motion, (σ_b') is the effective stress in the direction of propagation, and na and nb are integers.

For Ticino sand, Bellotti et al. found C_s to be around 85 and na = nb = 0.122. For V_s measurements using the seismic cone (SCPT), where waves propagate in an approximately vertical direction, (σ_a') is approximately equivalent to (σ_h') and (σ_b') to (σ_v'). If equation [2] is written in terms of K_o, the following expression is obtained:

[3] $V_s = 85\, [F(e)]^{0.5} \sigma_v'^{0.244} K_o^{0.122}$

Robertson and Fear (1995) attempted to relate V_s to the state parameter, an index of initial state suggested by Been and Jefferies (1985). Again, assessment of field density was based on relationships between V_s and state parameter developed in laboratory tests on unaged samples and requires an estimate of σ_h'. The effect of ageing is not included.

V_s can be related to the stiffness of the soil based on the theory of linear elasticity, using the expression:

[4] $G_o = \rho V_s^2$

where ρ is the soil mass density. Conventionally, this refers to the small strain stiffness of the soil as the deformations created during V_s measurement are small. From equation [3], G_o would be dependent on approximately $(\sigma_v')^{0.5}$ and $K_o^{0.25}$.

Combinations of V_s and q_t

The above discussion shows that q_t and V_s in unaged soil are both functions of vertical stress, lateral stress or K_o and density. The expression for V_s includes $K_o^{0.122}$ which varies from 0.89 to 1.14 for $0.4 < K_o < 3.0$. Thus, K_o has a small effect on V_s and a much greater effect on q_t, as $(K_o)^{0.55}$ ranges from 0.6 to 1.82 for the same range of K_o. As V_s and q_t are different functions of the same variables, attempts have been made to use combinations of the two to derive soil properties.

Previous investigators have investigated the use of the ratio of G_o/q_t in the interpretation of SCPTU data. Baldi et al. (1986) note that G_o/q_t should decrease with increasing D_r, as q_t increases much faster with increasing D_r than does G_o. Fig. 1 shows data from a site in the Fraser Delta at which ground improvement by vibro-replacement had been carried out. The ground improvement caused a slight decrease in G_o/q_t, which would be consistent with an increase in D_r.

Howie et al. (2000) noted that results in Fig. 1 could be explained by an increase in D_r using conventional correlations (i.e. ignoring K_o) but could also be explained by an increase in lateral stress and no change in D_r. It is more likely that both lateral stress and D_r have been changed by ground improvement and that the increase in tip resistance is due to a combination of the two. The relative effects are difficult to quantify.

Eslaamizaad and Robertson (1996) combined expressions for q_t and G_o and used calibration test data to derive by statistical correlation a relationship for the evaluation of K_o in which the effect of D_r has been eliminated. Their relationship can be rearranged to give an expression for K_o as follows:

[5] $K_o = 3.4 \times 10^{-6} (p_a/\sigma_v')^{0.718} [(G_o/p_a)/(q_t/p_a)^{0.25}]^{2.165}$

For a given depth or σ_v', an increase in K_o should result in an increase in the parameter, $[(G_o/p_a)/(q_t'/p_a)^{0.25}]$ and such an increase as a result of vibro-replacement is observed in Fig. 1. The estimated values of K_o before and after ground improvement indicate a significant increase in lateral stress. If G_o given by Equation [4] is substituted in Equation [5], the resulting expression includes the term $(V_s)^{4.33}$. The estimated K_o values will thus be very sensitive to changes or errors in V_s. However, there is still a good indication that an increase in lateral stress has occurred.

The above considers the effect of lateral stress on situ test results. The important issue of ageing of the sand has not been included in this discussion.

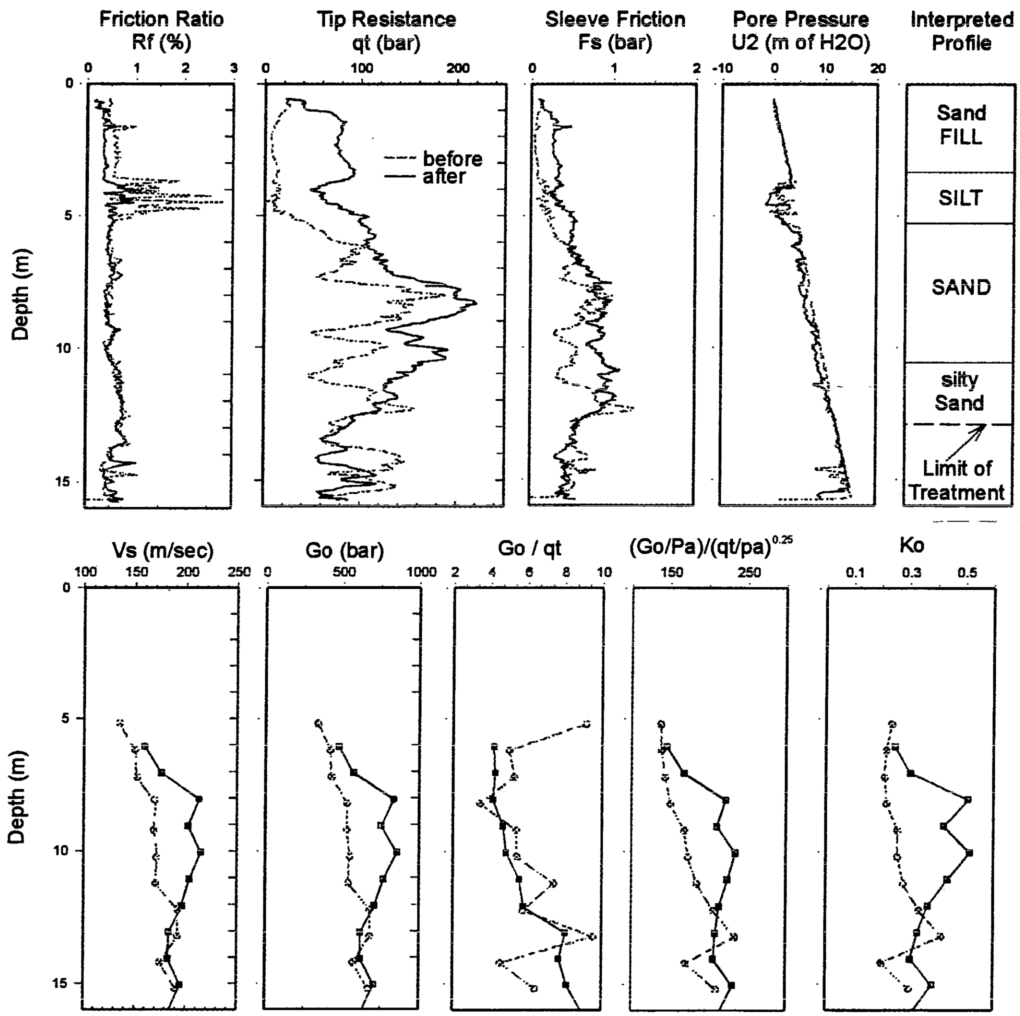

Fig. 1. SCPTU profiles before and after Vibro-Replacement

In situ testing - effect of ageing

Previous studies

Penetration resistance has been observed to change considerably with time after deposition. Skempton (1986) documented increases in N-value of fills with time as discussed by Jamiolkowski et al. (1988) and shown in Fig. 2. Fig. 3 from Troncoso and Garcés (2000) shows the ageing effect on normalized shear modulus, G_n interpreted from V_s measurements in Chilean tailings dams over a period of 45 years.

Much of the evidence for time effects on penetration resistance comes from the assessment of ground improvement. Both q_t and V_s have been observed to decrease in sands when measured soon after ground treatment and to then show an increase with time after treatment (Schmertmann 1991). Mitchell and Solymar (1984) observed an initial decrease in q_t immediately after ground treatment by explosive compaction. The tip resistance was then observed to increase considerably with time over a period of 11 weeks. Similar effects have been noted in other explosive compaction contracts. Time effects have also been observed with other types of ground improvement. Various mechanisms have been proposed for observed ageing effects but the phenomenon is not well understood. Several laboratory studies have been undertaken to investigate the factors contributing to ageing.

Dowding and Hryciw (1986) observed considerable increase in penetration resistance with time in a laboratory simulation of blast densification. Joshi et al. (1995) also found considerable increase in penetration resistance in their laboratory study. Baxter (1999) carried out an extensive study of the effect of ageing on penetration resistance and V_s, based on laboratory and model testing. He observed increases in V_s varying with soil type and pore fluid composition but failed to observe changes in penetration resistance. He suggested that some ground conditions had not been replicated in the model test.

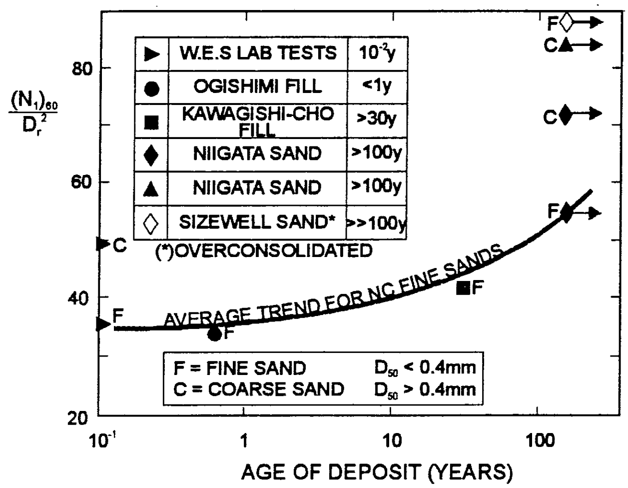

Fig. 2. Influence of ageing on Standard Penetration Resistance of NC sands (after Skempton 1986).

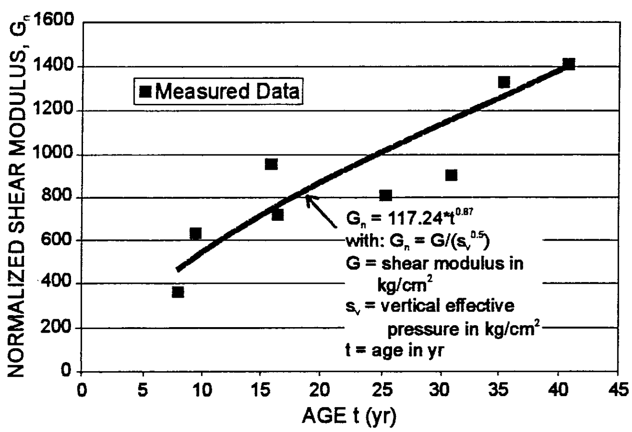

Fig. 3. Normalized shear modulus as function of age for tailings (after Troncoso and Garcés 2000)

Many other laboratory studies have shown an increase in V_s and, hence, G_o with ageing time (Anderson and Stokoe 1978, Mesri et al. 1990). Jamiolkowski and Manassero (1995) suggested that the effect of ageing on G_o, interpreted from resonant column tests or seismic wave velocities, may be approximated by the expression

$$[6] \quad \frac{G_o(t)}{G_o(t_p)} = 1 + N_G \log\left(\frac{t}{t_p}\right)$$

where t_p is the time to the end of primary compression (EOP), t is any time $>t_p$, $G_o(t)$ is G_o at time t, $G_o(t_p)$ is G_o at time t_p, and N_G is the slope of a plot of G_o expressed as a fraction of $G_o(t_p)$ versus the log of time. Mesri et al. (1990) found that N_G varied from 1 to 3 % for sands and increased as the soil became finer. Fahey (1998) suggested that this rate of increase was not sufficiently high to explain the difference between the measured stiffnesses of undisturbed and laboratory samples documented by Ishihara (1996).

Daramola (1980) studied the effects of ageing on the stiffness of dense Ham River sand in conventional triaxial testing. The stiffness was observed to be a function of D_r when samples were not aged for a long time. However, after ageing, time was observed to have great influence on stiffness and D_r was not the main factor controlling stress-strain response. Secant stiffnesses at strains less than 0.5% increased by 100% over three log cycles of time. Recent laboratory testing has provided new insight into the reasons for such observations.

Laboratory ageing of Fraser River sand

Shozen (2001) carried out a study of time effects on stress-strain response of very loose Fraser River sand for ageing periods up to 1000 minutes after consolidation. Fig. 4 shows the effect of ageing on conventional drained triaxial stress-strain curves obtained from tests on samples consolidated to three different stress ratios, ($R=\sigma'_1/\sigma'_3=1.0$, 2.0 and 2.8). A period of ageing results in a much stiffer response during the initial portion of the stress-strain curve but, the curves coincide beyond the initial stages of loading. This suggests that ageing increases initial stiffness but has little effect on larger strain properties, including strength.

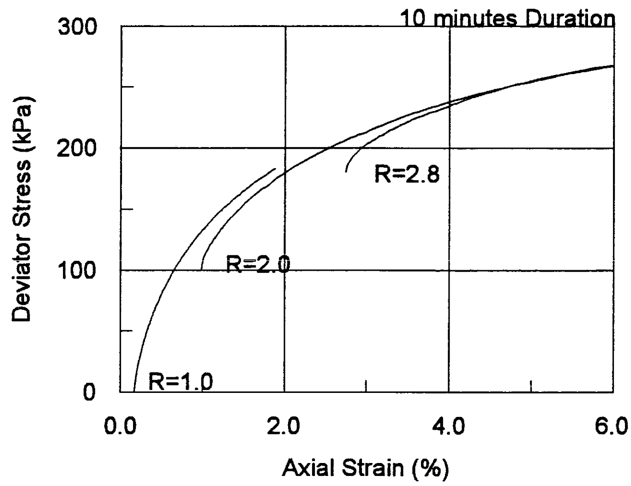

Fig. 4. Effect of ageing on triaxial stress-strain behavior at different stress ratios.

Fig. 5 shows the variation in stiffness with strain increment level for different periods of ageing under a stress ratio of 2.8. The increase in stiffness with time is larger at smaller strain levels. This results in the degree of strain softening increasing with ageing time. Resolution of the strain measurement apparatus was not considered reliable for shear strains below about 0.02% and so the effect of ageing on G_o ($\gamma<0.0001\%$) was not studied.

Fig. 6 shows the effect of stress ratio during ageing on the change in measured stiffness with time at shear strains of 0.03% and 0.15%, $G_{0.03}$ and $G_{0.15}$ respectively. Increases in stiffness with time are much greater at higher stress ratios. At any time after consolidation, isotropically consolidated samples are stiffer than samples consolidated

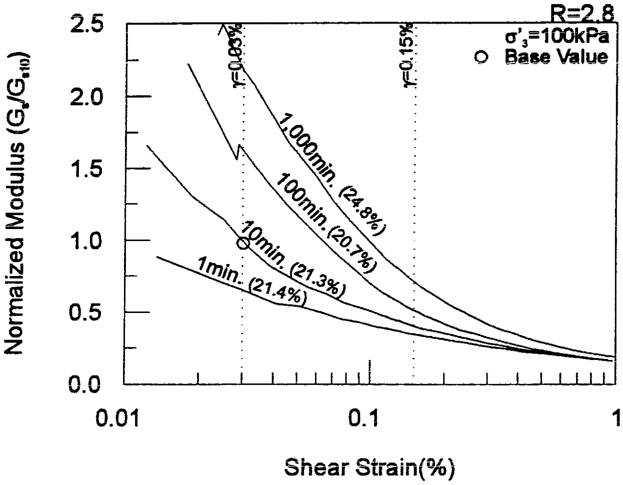

Fig. 5. Attenuation Curves of shear modulus for R=2.8 on conventional path.

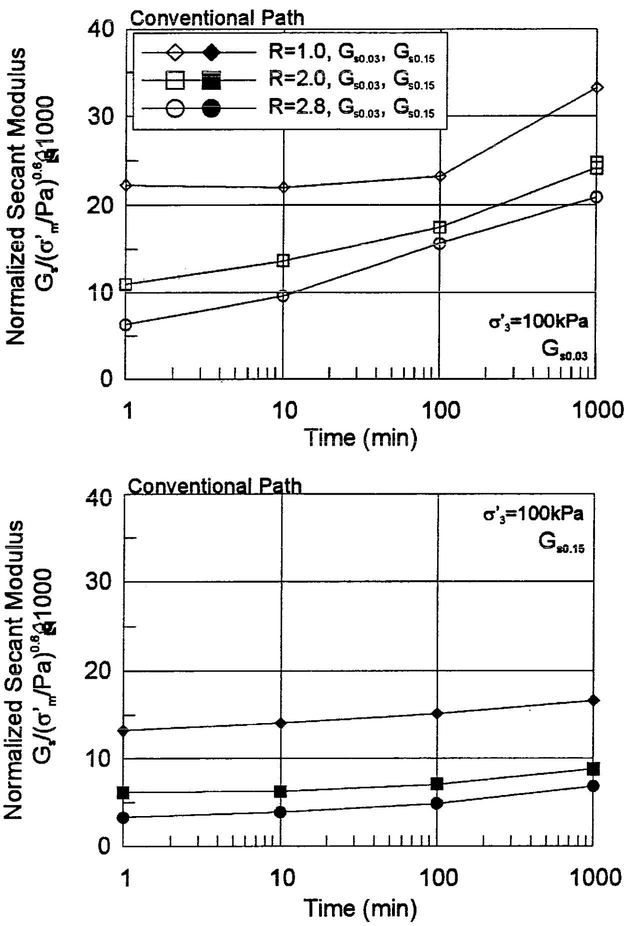

Fig. 6. Effect of Stress Ratio for Different holding durations.

at stress ratios of 2.0 and 2.8. Fig. 7 shows that very loose samples aged for 1000 minutes can be stiffer than medium dense samples that are unaged. Shozen (2001) indicates that the magnitude of the increase in stiffness with age is approximately the same for very loose and medium dense samples of Fraser River sand. As the unaged stiffness of the medium dense sample is greater, the percentage

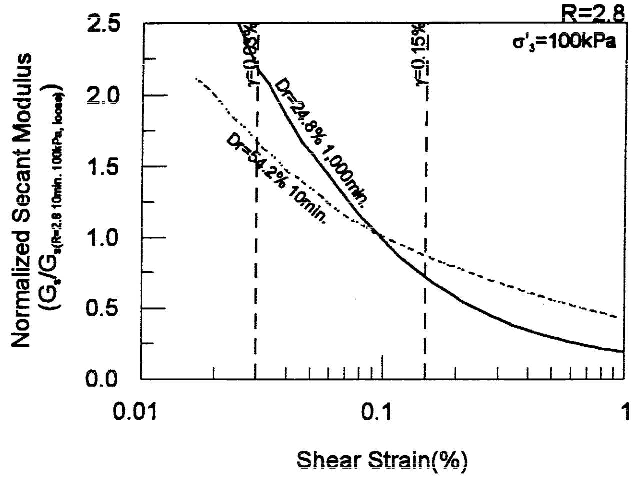

Fig. 7. Comparison between the effect of time and relative density on stiffness.

increase in stiffness with time is smaller, i.e. the N_G factor in Equation [6] will be smaller for denser samples.

For ageing times of up to 1000 minutes after completion of consolidation of loose and medium dense Fraser River sand, the following effects were observed:

- The rate of increase in stiffness with age is greater for smaller strain increment levels, i.e. N_G is larger for smaller strain levels;
- The rate of increase in stiffness with age reduces with increasing density, i.e. N_G is smaller for denser soils;
- The rate of increase in stiffness with age increases at higher stress ratios i.e. N_G is larger at higher stress ratios;
- There is no apparent increase in strength with time;
- Loose aged samples can be stiffer than younger denser samples.

From the above, it is clear that the measured secant stiffness of Fraser River sand is strongly affected by sample age and stress ratio during ageing in addition to the more commonly considered factors such as D_r; magnitude of strain increment; stress ratio; and stress path.

Implications of the observed ageing effects for interpretation of in situ test results

The above effects are only strictly applicable to samples aged for about 1 day. Attempts to extend the testing to 10,000 minutes or close to 1 week were inconclusive due to difficulties with test stability. However, stiffness did continue to increase. Based on observations during laboratory testing, K_o for unaged Fraser River sand is just less than 0.5. Robertson and Fear (1995) also found K_o=0.5 in field-testing in the Fraser delta. Therefore, ageing at R=2.0 may be reasonably representative of K_o conditions. The effect of age on K_o is controversial (Schmertmann 1983; Mesri and Castro 1987). Fig. 8 shows the increase in stiffness over time extrapolated at the same logarithmic rate of increase as for the first 1000 minutes at R=2.0. The graph suggests that for a very loose sand at a confining pressure of 100 kPa, $G_{0.03}$ would increase to 3.4 times its initial value over the first week, 4.4 times its initial value at

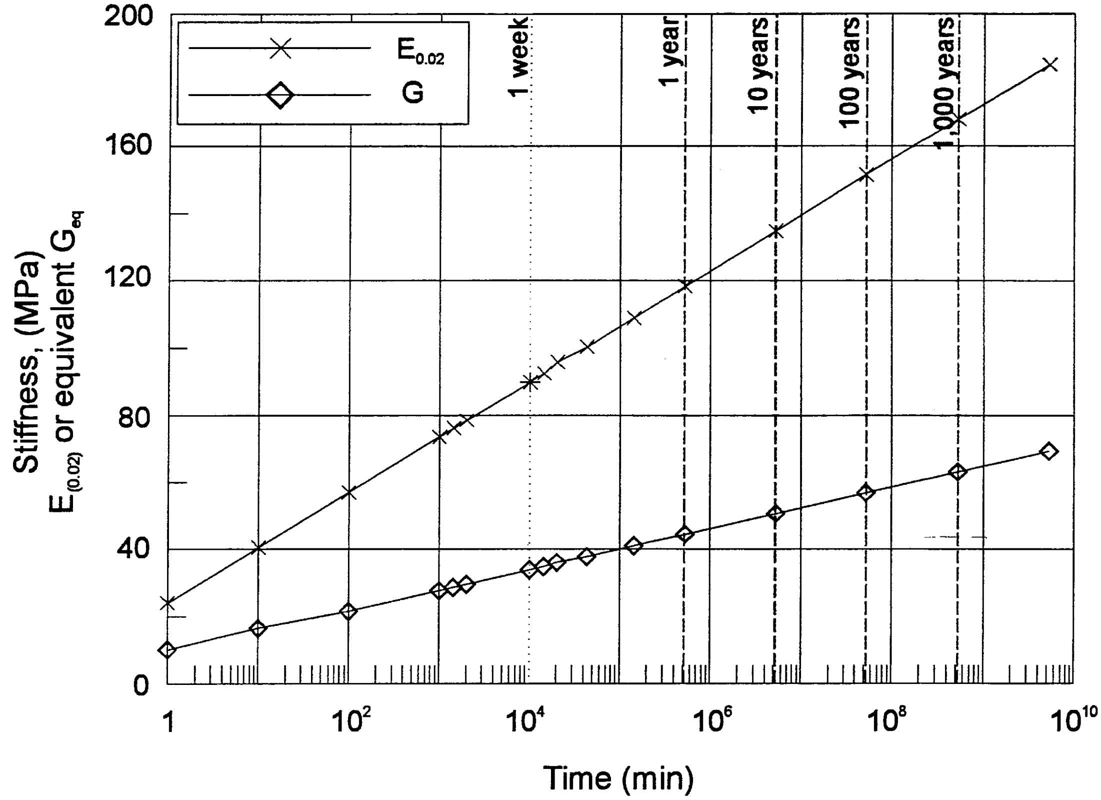

Fig. 8. Effect of ageing on $E_{(0.02)}$ and $G_{eq}=E_{(0.02)}/2(1+\nu)$ at R=2.0.

1 year and to 6.3 times its initial value at 1000 years. Denser sand would also experience an increase in stiffness but the percentage increase would be much less. For sand at a higher initial stress ratio, the rate of increase in stiffness would be much greater. For sand at a lower initial stress ratio, i.e. higher lateral stress, the rate of increase would be less pronounced.

The above observations suggest that tests responsive to small strain stiffness of sand should be more sensitive to time effects than those affected by the large strain response. Thus, V_s should be very sensitive to time effects. As q_t is conventionally considered to be a large strain index parameter, q_t should be insensitive to time effects. Experience suggests otherwise.

Monahan et al. (2000) used carbon dating of organics in Fraser River sands to show a linear increase in $(q_c)_1$ with age in Fraser River sand as shown in Fig. 9. $(q_c)_1$ is q_c normalized to a common stress level. For drained penetration, q_c is identical to q_t. The scatter is likely due to other factors such as density variations.

Baxter (1999) carried out numerical modelling to investigate the effect of an increase in G_o on q_t in sand. He used a model based on cavity expansion developed by Salgado (1993) with typical properties of Ticino sand. He found a 20% increase in G_o resulted in a 5.5% increase in q_t for both loose (D_r=30%) and dense (D_r=80%) sand. An increase in K_o of 33%, i.e. an increase in lateral stress but a reduction in stress ratio, produced a 17% and 9% increase

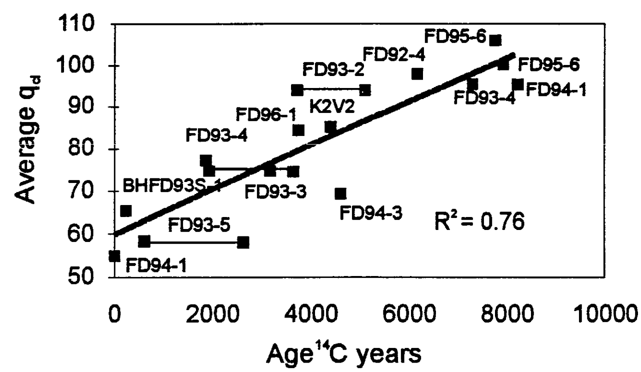

Fig. 9. Plot of average qc1 and 14C age of organic material in topset sand (after Monahan et al. 2000).

in q_t for loose and dense sand, respectively. This increase in lateral stress also caused a 6% increase in G_o. Numerical modelling of cone penetration in chamber tests by Ahmadi (2000) found a large effect of lateral stress and soil stiffness and a relatively small effect of shear strength on q_t. Although Baxter's laboratory cone tests failed to detect an increase in q_t due to ageing, despite an increase in G_o prior to cone penetration, this may have been because of the limited diameter of the test chambers. A small diameter chamber would not permit an extensive zone experiencing small strains to develop beyond the zone of high shear.

Based on these modelling results and the observation of considerable ageing effects on q_t, it must be considered possible that q_t is more affected by soil stiffness than is

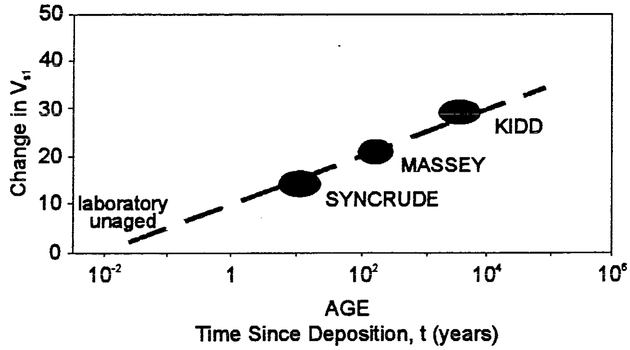

Fig. 10. Change in normalized shear wave velocity with age for uncemented sands (after Robertson et al.,

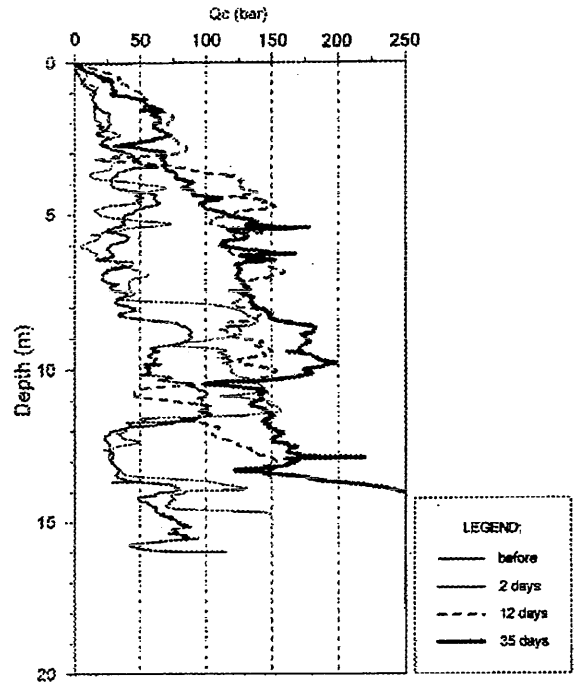

Fig. 11. Time dependence of cone tip resistance Qc

conventionally assumed. For fills placed in water by pipeline discharge, initial conditions should be close to K_o and correlations developed in similar unaged sands should be applicable for assessing soil properties. Considerable scatter should be anticipated, particularly in loose sands, because little attention is paid to the age of sand samples used to derive the correlations. For fills that are not pluvially deposited, the penetration resistance will be strongly influenced by the magnitude and distribution of lateral stress. Conventional correlations may not be reliable.

For geologically aged sands, such as those in the Fraser River delta, both V_s and q_t should be higher than for recent fills at a similar density. Robertson and Wride (1997) used field data to illustrate this effect for V_s (Fig. 10) and Monahan et al. (2000) illustrated the effect of age on q_t. (Fig. 9).

Disturbance to an aged sand will result in changes in density, stress conditions and stiffness. For example, after the first pass of explosive compaction, a decrease in penetration resistance has been observed despite an obvious increase in density indicated by observed settlement. Subsequent passes of treatment cause further increases in density and penetration resistance is observed to increase considerably over time after completion of treatment. Fig. 11 illustrates this ageing effect for a medium grained sand treated by explosive compaction (Gohl et al. 1996).

Explosive compaction uses widely spaced charges to subject the soil to cyclic loading. Based on Dobry et al., (1982), cyclic strains above a threshold strain cause settlement and pore pressure build up. In the extreme, liquefaction may occur. The initial set of charges destroys any structure set up by ageing, and the resulting settlement causes an increase in density. Depending on the relative effects of destructuring and density increase, V_s and q_t may go up or down. At points close to the charges, there may also be a change in lateral stress.

Ground improvement by Deep Dynamic Compaction (DDC) or vibro-replacement (stone columns), also causes widespread large deformations. In DDC, the major deformations are near ground surface and strains attenuate with depth.

Fig. 12 shows profiles of CPTU parameters and V_s measured before and 3 days after the first pass of treatment by DDC at a Fraser delta site. The first pass consisted of 15 drops of a 13.5 tonne mass from a drop height of 24 m. An attempt was made to locate the CPTU soundings as close to each other as possible to reduce the effects of site variability. There has been a marked increase in q_t at shallow depth but there is a reduction in V_s. Fig. 13 shows two V_s profiles measured before treatment and two after. V_s has been reduced by the disturbance caused by the impact of the mass. A large crater was created by the impact above the point of testing and so there is little doubt that some density increase occurred. It is also likely that the lateral stress was increased close to the impact point. In this case, the net result of destructuring and of increases in density and lateral stress has been an increase in q_t and a reduction in V_s. As is common with ground improvement studies, the site was not accessible for further testing due to construction activity.

In vibro-replacement, the in situ material is displaced sideways and additional material is introduced throughout the depth of treatment to form stone columns. Consequently ageing effects will be erased by the large strains but large lateral stresses are likely induced close to the points of treatment. The combined effect of increases in density and lateral stress and decreases in the age-induced stiffness gain should still result in a net increase in q_t and V_s after treatment. The data in Fig. 1 showed that for treatment by vibro-replacement, increases in both q_t and V_s

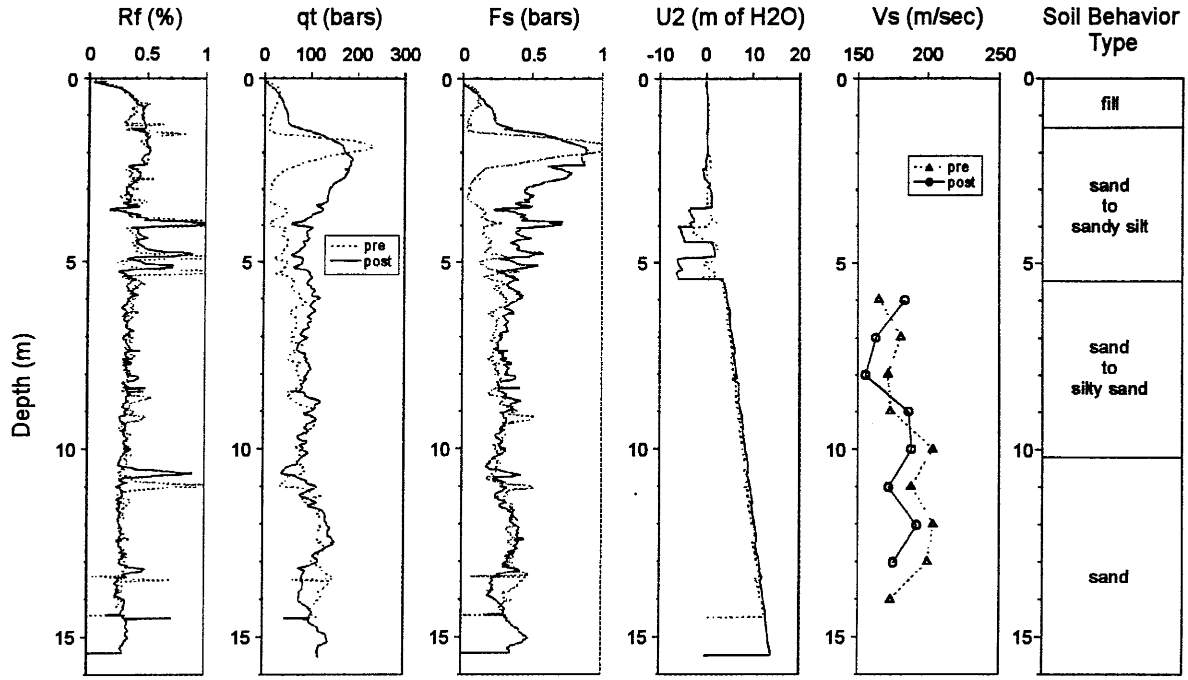

Fig. 12. SCPTU results before and after Dynamic Compaction

were observed within and slightly below the depth of treatment. Below this level, a decrease in q_t was observed due to the effects of disturbance by cone penetration during the original CPT sounding. Soundings were at the centroids of triangles of stone columns. The results are consistent with a large increase in lateral stress and an increase in density throughout the depth of treatment by vibro-replacement.

Conclusion

The method of fill placement affects its initial density, stress state and fabric. Correlations between in situ testing parameters and soil properties are largely based on the results of chamber tests and other laboratory testing carried out on unaged specimens or on specimens where little attention is paid to the potential effects of ageing. For fill placement conditions similar to those used to prepare samples on which correlations are based, such correlations may be applicable for assessment of the engineering behaviour of the fill. For other methods of fill placement and for the assessment of ground improvement, these correlations are inapplicable as the density, stress state and fabric are unlikely to be consistent with those of a normally consolidated soil. Such correlations will give misleading soil properties.

Ageing effects will change the engineering behaviour of soil with time after placement or ground improvement. Recent laboratory testing has shown that these effects are greater for loose soils and for high stress ratios than for

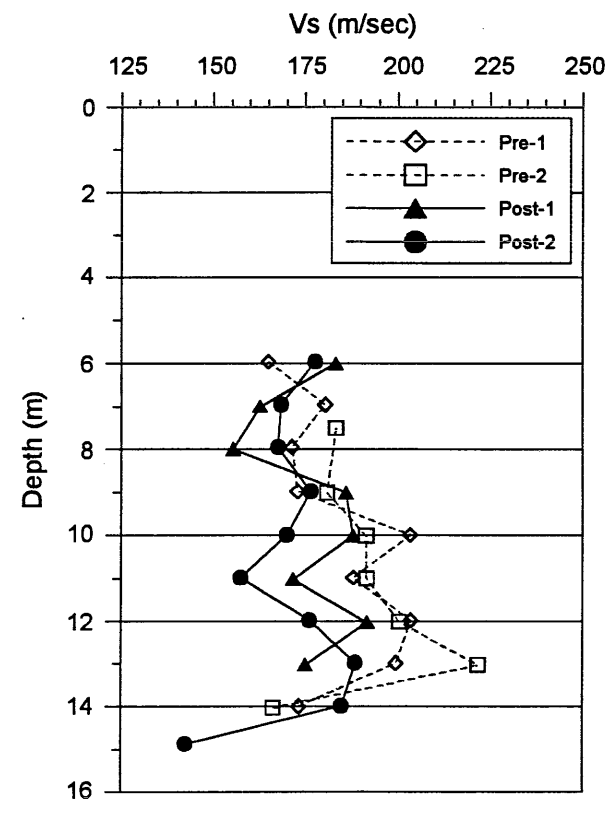

Fig. 13. Comparison of shear wave velocity before and after Dynamic Compaction

denser soils and lower stress ratios. For natural ground in which ageing has resulted in a substantial stiffening at small strains, a reduction in q_t and V_s may be observed

immediately after treatment due to destruction of the soil structure developed by ageing, despite an increase in density. Immediately after ground treatment of unaged fills, any changes in q_t or V_s should be the result of increases in density and lateral stress. Changes in q_t and V_s with time after treatment will depend on the density and lateral stress distribution achieved by ground treatment.
A more rational approach than penetration resistance or achieved density is required for the specification of fill properties and the targets for ground improvement. The stress strain behaviour under the proposed loading conditions is what matters.

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