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Aging of Sand – a Continuing Enigma?

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ABSTRACT

Sand aging, a process during which the engineering properties of clean sands such as stiffness, penetration resistance and liquefaction resistance may exhibit considerable improvement over periods of only weeks to months after deposition and/or densification by different ground improvement processes, has been shown over the past 30 years to be of considerable practical importance. Numerous examples from a range of projects are presented. Chemical, physical-mechanical, and microbiological processes are examined relative to their adequacy for explaining the observed behavior. 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. Information about the rate and magnitude of property changes during aging is summarized, and 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.

INTRODUCTION

Nearly 30 years ago I was involved in the Jebba Dam hydroelectric development on the Niger River, Nigeria, requiring the construction of a 42 m high earth and rockfill dam founded on a deposit of alluvial uniform clean sand with a depth to bedrock of up to 70 m. The medium to coarse silica sand has an average uniformity coefficient of 2.94, and the grains are sub-rounded to rounded, with occasional sub-angular to angular grains present. To prevent differential settlements and liquefaction of this sand that could result in cracking of the compacted clay upstream seepage blanket and dam's clay core, the loose zones in the upper 40 m of the sand were densified using explosive compaction and vibrocompaction. Several unexpected results were obtained that showed:

- The in-situ sand underwent a loss in penetration resistance as a result of mechanical disturbance. This "sensitivity" may be seen by the data in Fig. 1. A steel casing for installation of explosive charges was vibrated to a depth of 15 m. The force required to push the casing to the required depth during its initial insertion was considerably greater than when reinserted, and the force to push it in was even less following blast densification, even though surface subsidence confirmed densification of the sand. The three cone penetration test (CPT) records, located 1.5 m from the blast tube insertion point indicate similar penetration resistance reductions.
- The penetration resistance decreased immediately following blast densification, as may be seen by the comparisons in Fig. 1.
- Large increases in cone penetration resistance that developed over a period of several months following densification using explosive compaction (Fig. 2) and vibrocompaction (Fig. 3).
- Large increases in cone penetration resistance over a period of several weeks in a 10-m thick hydraulic sand fill placed to provide a working platform in the river, as illustrated by the CPT records in Fig. 4.

These findings are described in some detail in Mitchell and Solymar (1984) and Mitchell (1986). A tentative hypothesis was offered that accounts for the observations in terms of solution-precipitation-cementation reactions during the aging period. Since then, similar aging phenomena have been reported, and alternative hypotheses have been proposed for the responsible mechanisms by a number of investigators. These studies have provided much greater insight and clarification of the phenomena and controlling factors.

Irrespective of the precise mechanism(s) responsible for the property changes during sand aging, the changes are usually beneficial, resulting in decreased compressibility and increased stiffness, penetration resistance, and liquefaction resistance.

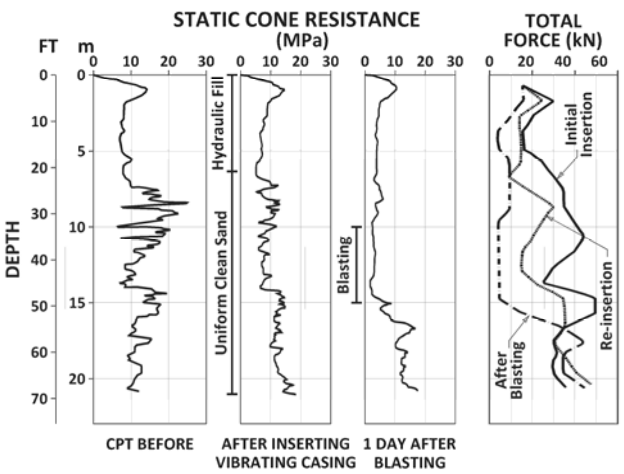

Fig. 1. Effect of sand disturbance on resistance to penetration

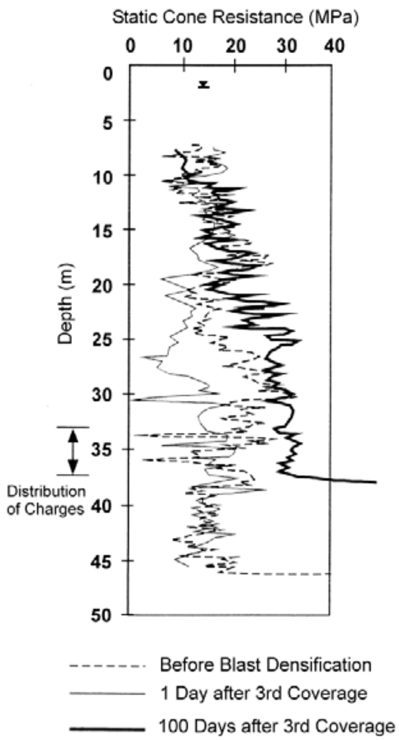

Fig. 2. Effect of time on the cone penetration resistance of sand following blast densification at the Jebba Dam site.

My objectives in this state-of-the-art and practice paper are to provide a comprehensive summary and illustration of the effects of sand aging over "engineering time"; i.e., periods of several years (as opposed to much greater geologic time), to examine the mechanisms responsible for observed aging effects and the factors that influence them, and to evaluate predictive methods for addressing the two questions likely to be of importance in practice: How much? and How fast?

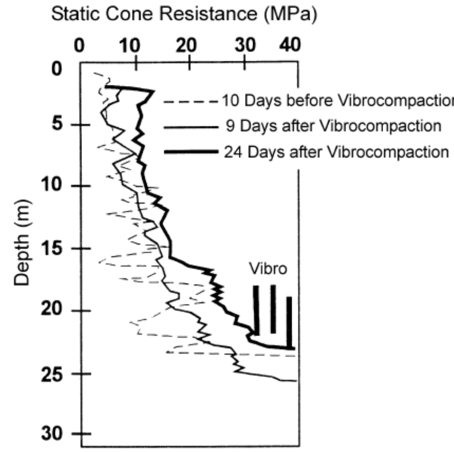

Fig. 3. Effect of time on the cone penetration resistance of sand following vibrocompaction densification at the Jebba Dam site.

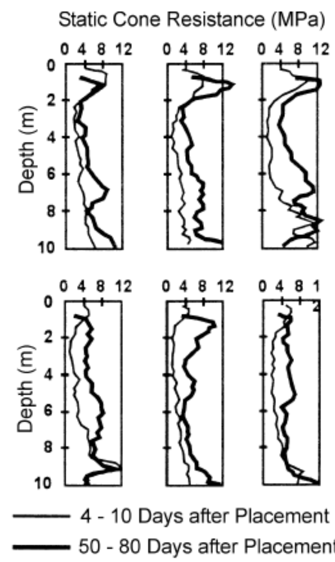

Fig. 4. Effect of time on the cone penetration resistance of hydraulic fill sand after placement at the Jebba Dam site.

ILLUSTRATIONS OF SAND AGING EFECTS

Numerous examples of sand aging as indicated by changes in stiffness, stress-deformation response, penetration resistance, and liquefaction resistance are summarized in this section. Both dry and saturated sands are considered; partly saturated sand behavior is not.

<u>Small Strain Modulus and Stiffness</u>

<u>Laboratory Measurements.</u> Test results obtained by Afifi and Richart (1973) and Anderson and Stokoe (1978) using resonant column testing showed that the low amplitude dynamic shear modulus of sands held under constant confining pressure increases with time. These observations

were quantified and generalized by Anderson and Stokoe (1978) using the following equations:

$$N_G = \frac{\Delta G}{G_{1000}} / \log(t_2 / t_{1000}) \qquad (1)$$

and

$$G_t = G_{1000}(1 + N_G \log \frac{t_2}{t_{1000}}) \qquad (2)$$

where N_G is the coefficient of shear modulus increase with time, t_{1000} is a reference time after completion of primary consolidation, taken as 1000 minutes, t_2 is some time of interest thereafter, ΔG is the change in small strain shear modulus from t_{1000} to t_2, and G_{1000} is the shear modulus measured after 1000 minutes of constant confining pressure. N_G is the increase in normalized shear modulus per 10-fold increase in time. Large increase in stiffness due to aging is indicated by large values of N_G. The aging effect increases with increasing plasticity index, as shown in Fig. 5 (Kokusho, 1987). The data in the figure have been supplemented by values of $\Delta G/G_{1000}$ for several sands compiled by Jamiolkowski (1996). Mesri et al. (1990) report that N_G for sands varies between 0.01 and 0.03 and increases as the soil becomes finer. Jamiolkowski and Manassero (1995) give values of 0.01 to 0.03 for silica sands, 0.039 for sand with 50% mica and 0.05-0.12 for carbonate sand.

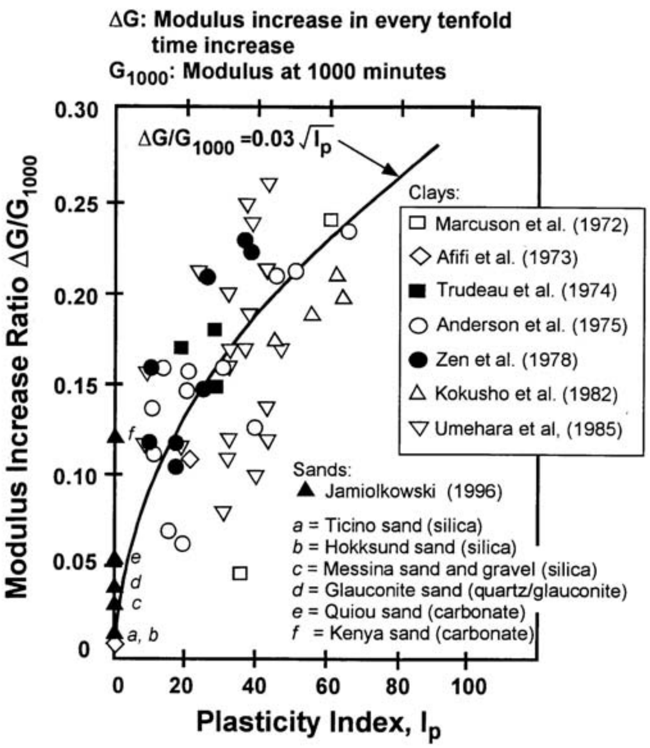

Fig. 5. Shear modulus increase ratios for clays (from Kokusho, 1987) and for sand (from Jamiolkowski, 1996).

Aging effects were studied by Daramola (1980) by means of triaxial tests on samples of saturated Ham River sand consolidated at 400 kPa for times up to 152 days. The results of these tests are shown in Fig. 6. The secant Young's modulus at 1.0% axial strain had increased by approximately 50 percent after 152 days. The peak strength was little changed by the aging process, although the strain at failure decreased with aging time.

In contrast to these findings, the results of drained triaxial tests by Human (1992) on dry samples of pluviated Crystal silica sand at a relative density of 78 percent and under a consolidation pressure of 150 kPa showed essentially no differences in stress-strain behavior over aging periods of up to 28 days. The initial shear wave velocities increased by about 4 percent over this period, however.

Howie, et al. (2002) determined small strain values of Young's modulus (*E*) for very loose samples of Fraser River Sand (D_{50}=0.27 mm, C_u=1.9) using triaxial compression tests in which samples were held under both isotropic and anisotropic confining pressures for periods up to 10,000 minutes prior to deformation by increasing the vertical to horizontal stress ratio, *R*. Observed time-dependent increases in shear modulus *G*, deduced from secant values of E by assuming linear elastic behavior, were consistent with equation (1) in that they increased approximately linearly with log time. However, the magnitude of N_G *was s*trongly dependent on the stress ratio under which the specimens were allowed to creep during aging and the axial strain at which the secant modulus was determined. Some of their results are given in Table 1. Two important findings from these tests are that (1) the initial moduli were higher for samples loaded isotropically and (2) the rate of stiffness gain, as indicated by N_G increased with increasing anisotropic stress ratio *R*. Furthermore, contractive volumetric strain decreased with aging time at all values of *R*.

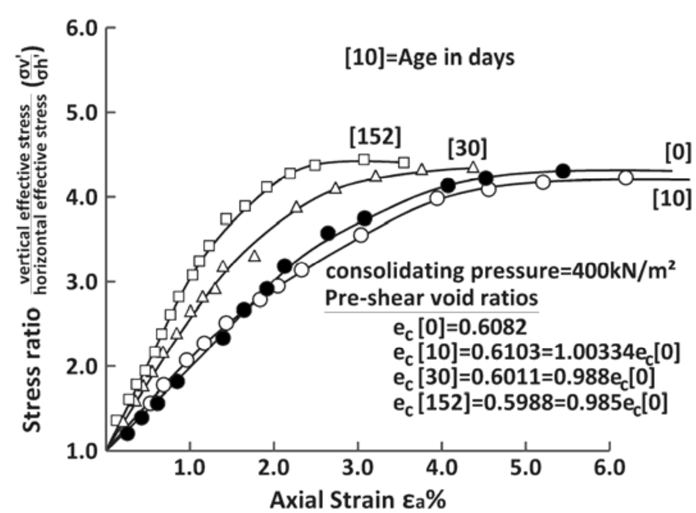

Fig. 6. Effect of aging on the stress-strain relationships for Ham River sand (from Daramola, 1980)

Table 1. Stiffness Increases in Loose Fraser River Sand during Aging (data from Howie, et al., 2002)

Stress ratio during aging	**G at 1 min. (MPa)**	**N_G t_p=1000 min**
G from E at 0.02% strain		
1.0	23	0.01
2.0	10	0.02
2.8	5.5	0.03
G from E at 0.1% strain		
1.0	14	<0.005
2.0	7	0.01
2.8	4	0.012

A laboratory testing program was designed by Baxter and Mitchell (2004) to study mechanisms responsible for aging effects under carefully controlled conditions. Measurements were made of the small strain shear modulus, electrical conductivity, pore fluid chemistry, and mini-cone penetration resistance after different periods of aging. Two different sands, a poorly graded fine subangular sand (Evanston Beach sand) and a poorly graded rounded fine to medium sand (Density sand) were tested, and aging effects up to 118 days were evaluated for different combinations of relative density (40 and 80 percent), temperature (25 and 40ºC), and pore fluid composition (air, distilled water, CO_2 saturated water, ethylene glycol). The samples were formed in fixed ring consolidometers and maintained under a vertical effective stress of 100 kPa. Variability in the data precluded detection of any consistent secondary compression beyond the first few days after application of the vertical stress. Increases in the small strain shear modulus were observed in most of the samples. Values of N_G for the different conditions are given in Table 2.

Table 2. Values of N_G for two sands (from Baxter and Mitchell, 2004)

EVANSTON BEACH SAND

Temp. (T)	Initial Relative Density (D_{ro})	***N_G*** Distilled Water	Ethylene Glycol	CO_2-Sat. Water	Dry
25^o C	40%	.017	.005	.018	---
25^o C	80%	.028	.008	.028	.005
40^o C	40%	.016	.001	---	---
40^o C	80%	.040	.013	---	---
DENSITY SAND					
25^o C	40%	-.006	.008	-.007	---
25^o C	80%	.003	.008	.003	.022
40^o C	40%	-.001	-.007	---	---
40^o C	80%	.010	.001	---	---

Field Measurements. There is only limited direct field evidence to show aging effects on stiffness properties of sands. Troncoso and Garces (2000) measured shear wave velocities using downhole wave propagation tests in low plasticity silts, having fines contents from 50 to 99 percent, at four abandoned tailing dams in Chile. The four sites are called Barahona, Cauquenes, La Cocinera and Veta del Agua and the aging times between abandonment and testing were 28, 19, 5 and 2 years, respectively. The tailing deposits at Barahona had a liquid limit of 41% and a plastic limit of 14%, whereas those at other three sites had liquid limits of 23-29% and plastic limits of 2-6%.

The shear modulus normalized by the vertical effective stress is plotted against the age of the deposit in Fig. 7. The ages of the deposits are expressed as the time since deposition. Although the soil properties vary to some degree at the four sites, very significant increase in stiffness at small strains can be observed after 10 to 40 years of aging.

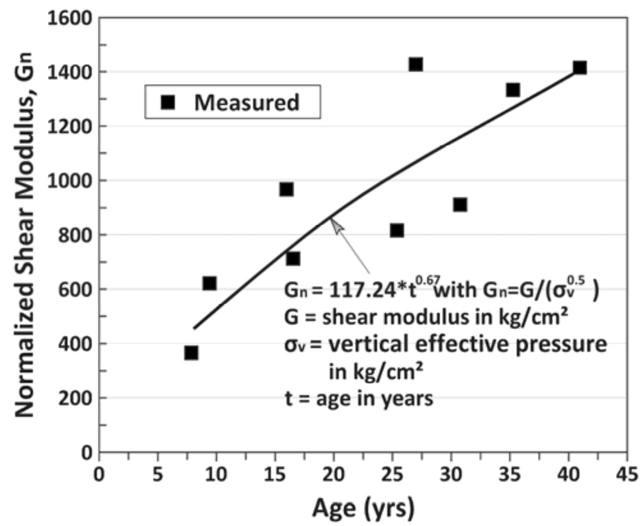

Fig. 7. Normalized shear modulus as function of aging of tailings (redrawn from Troncoso and Garces, 2000)

Shear Strength

Few results of direct measurements of the shear strength of aged sand specimens have been reported, with most evidence of the effects of aging being in terms of stiffness and penetration resistance increases. The latter, dependent mainly on both shear strength and horizontal stress, are discussed in some detail later in this paper.

The results obtained by Daramola (1980), shown in Fig. 6, indicate that in spite of the stiffness increases caused by aging, the peak strength as measured in triaxial tests was little affected. On the other hand, Al-Sanad and Ismael (1996) did laboratory direct shear tests on a fine to medium well graded silty sand containing about 20 percent calcium carbonate, dolomite, and gypsum. Samples were prepared to a relative density of 60 percent and aged under a vertical normal stress of 2 kPa for periods up to 6 months. The direct shear tests were done under normal stresses ranging from 40 to 120 kPa. The results of these tests are shown in Fig. 8. The following equation was developed by Al-Sanad and Ismael (1996) to relate the friction angle ϕ to aging time:

$$\phi = \phi_i + \frac{3.81t}{(2.82+t)} \qquad (3)$$

where ϕ_i is the friction angle at time = 0 and t is the time in months.

Penetration Resistance

The results of field penetration tests have provided the bulk of the data that show the beneficial effects of aging with respect to the engineering properties of sands. Cone penetration tests have yielded especially illustrative results as previously indicated in Figs. 1-4. In this section additional evidence of aging in terms of penetration resistance is shown, both by laboratory and field test results.

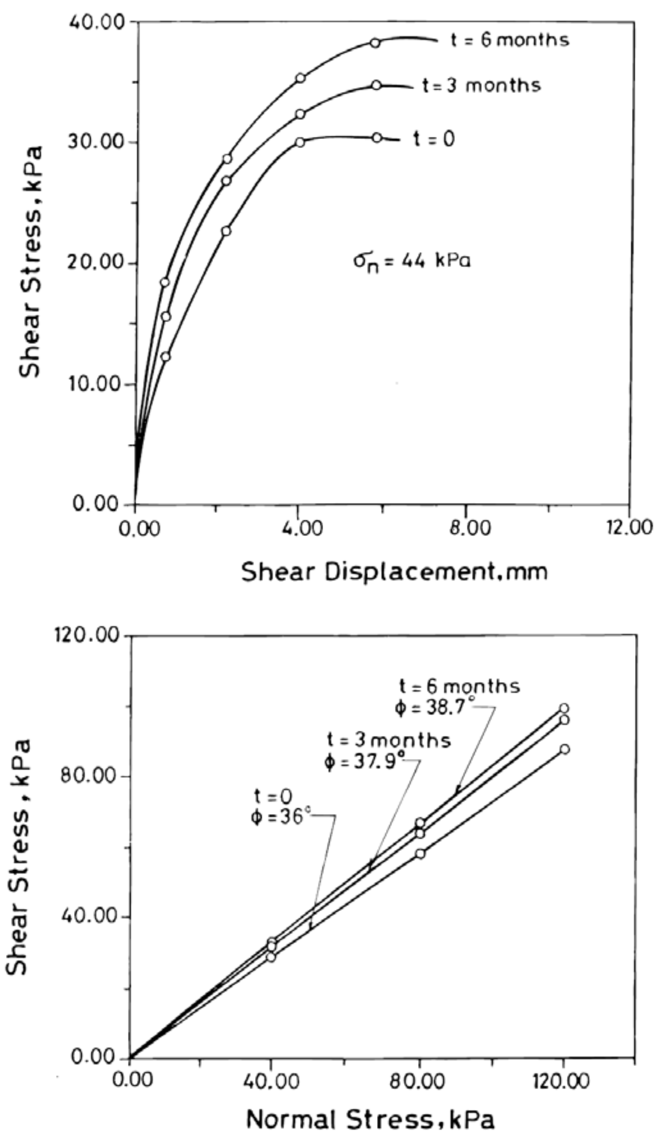

Fig. 8. Shear stress-displacement curves and failure envelopes for aged samples of a calcareous sand (from Al-Sanad and Ismael, 1996)

Laboratory Penetration Tests. A result similar to that shown in Fig. 1 was reported previously by Durante and Voronkevich (1955). Penetration test resistance values for a naturally deposited and undisturbed alluvial sand were compared with those for the same sand recompacted to the same densities. Their results are shown in Fig. 9. Durante and Voronkevich (1955) concluded that the sand in its natural state had a "cohesion" that was totally lost as a result of disturbance.

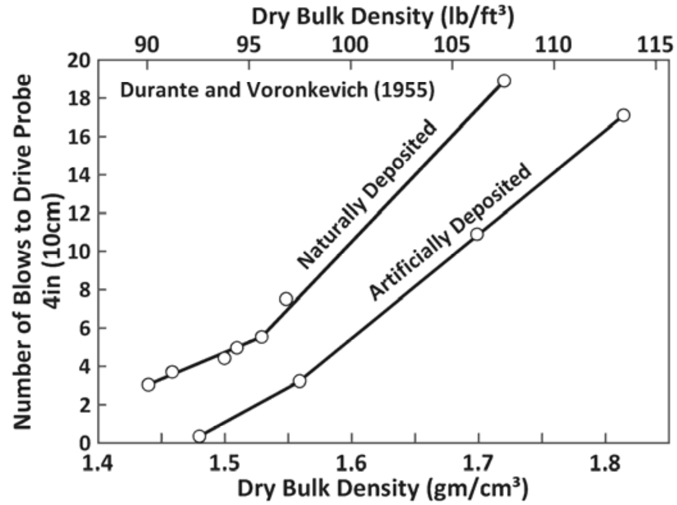

Fig. 9. Penetration resistance of naturally deposited and artificially sedimented sand (revised and redrawn from Durante and Voronkevich, 1955)

One of the first studies done in the laboratory that showed aging effects was that reported by Dowding and Hryciw (1986). Evanston Beach sand, a fine, poorly graded silica sand (D_{50} = 0.2 mm, C_u = 1.5), was placed to a depth of 660 mm in a 1 m diameter by 1 m high cylindrical liquefaction tank at a relative density of 50 percent. Explosive charges were set off in the center of the tank, and mini-cone penetration tests were done at radial distances of 100, 200, 300, and 400 mm at different times following the blast. The relative density ranged from 60 to 78 percent after blasting. Time-dependent increases in the penetration resistance were measured both in the densified samples and in a control test with no blasting. A summary of the test results is shown in Fig. 10. The results indicated that the penetration resistances increased with time in all tests, and were greater for the sand subjected to explosive compaction. The penetration resistance increases were greatest near the blast location. It may be seen also that close to the blast points the 1-day values were lower or the same as the pre-blast values.

Samples of a local river sand and Beaufort Sea sand were tested dry, in distilled water and in sea water by Joshi, et al., (1995). The sands were pluviated through air or water into cylindrical PVC cells, vibrated to the desired density, confined under a vertical stress of 100 kPa, and aged for two years. Several penetration tests were done in each sample using a 10 mm diameter penetrometer. Fig. 11, which is for penetration of aged Beaufort Sea sand in sea water, is typical of the results obtained. The magnitude of the penetration resistance increases relative to the initial values was in the increasing order of dry sand, sand in distilled water, and sand in sea water.

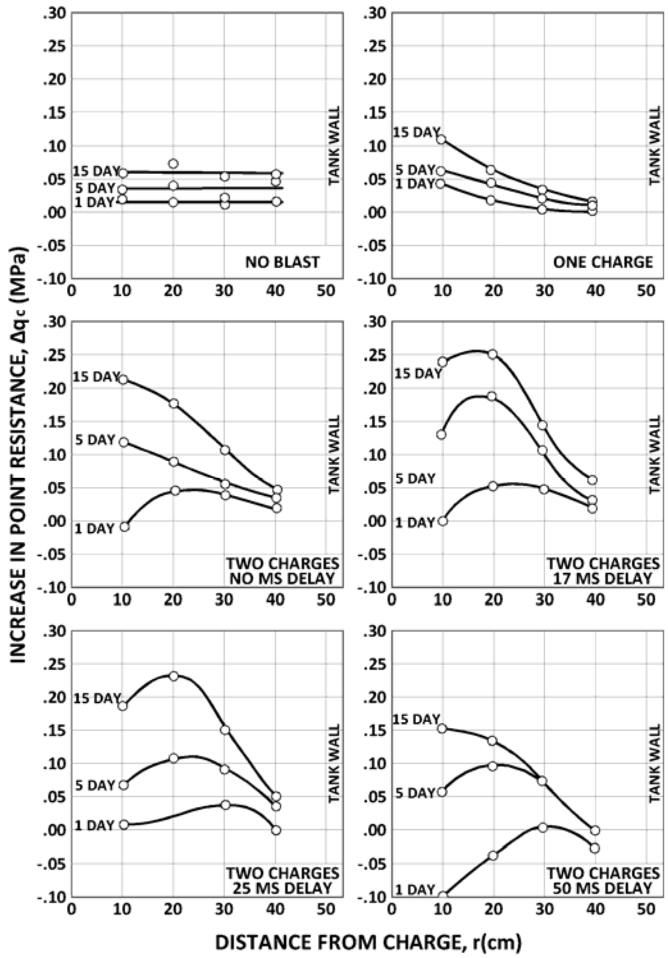

Fig. 10. The effects of aging on the post-blasting penetration resistance of Evanston Beach sand (redrawn from Dowding and Hryciw, 1986).

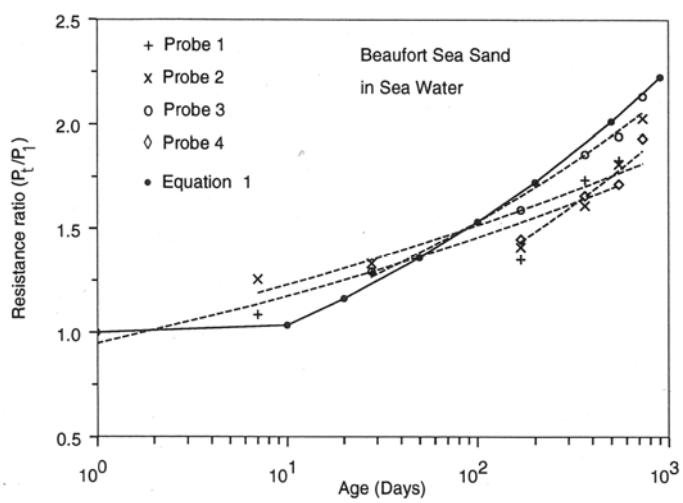

Fig. 11. Normalized penetration resistance for aged samples of Beaufort Sea sand in sea water. See Table 4 for Equation 1 (from Joshi, et al., 1995)

Despite the changes in stiffness and composition measured by Baxter and Mitchell (2004) in the test program described earlier, there were no corresponding increases in the mini-cone penetration resistance with time in any of the samples.

Field Penetration Tests. There are now many documented field case histories to illustrate significant increases in penetration resistance of sands as a result of aging. There are also cases where no significant time dependent increases have been detected. Denisov, et al. (1963) (cited by Schmertmann, 1991) were among the first to quantify the behavior. They determined the number of blows per 10 cm to drive a 74 mm diameter 60º cone using a 60 kgf hammer dropped 0.80 m into hydraulically placed, saturated quartz river sand fills. It was found that the number of blows/10 cm increased regularly from 2.1 at an age of 10-20 days to 4.4 at an age of 100-140 days.

At about the same time Mitchell and Solymar (1984) reported the relatively short term results shown in Figs. 1-4, Skempton (1986) published a comprehensive paper on the Standard Penetration Test in which the effects of much longer aging times were estimated. Skempton's conclusions are summarized in Table 3, where $(N_1)_{60}$ is the SPT N-value normalized to an effective overburden pressure of 1 atm, and D_r is the relative density.

Table. 3. Effect of Aging on the SPT Resistance of Normally Consolidated Fine Sands (values from Skempton, 1986)

	Age, years	$\mathbf{(N_1)_{60}/D_r^2}$
Laboratory Tests	10^{-2}	35
Recent Fills	10	40
Natural Deposits	$>10^2$	55

Increases in the cone penetration resistance, expressed as the ratio of q_c at time t to that immediately after deep dynamic compaction (DDC) of a 10 m thick layer of silty sand in Jacksonville, FL, are shown in Fig. 12 from Schmertmann (1991). At this site a 33 ton weight was dropped 105 ft. The different curves correspond to best fit lines for penetration resistance ratios after the number of drops indicated. More comprehensive discussions of this project and the aging results are given in Schmertmann et al (1986) and Schmertmann (1987).

Another example, illustrating time-dependent increases in penetration resistance following DDC (330 t-m/m^2) of a clean sand fill at the Pointe Noire deep sea harbor, Sept Iles, Quebec, is shown in Fig. 13, from Dumas and Beaton (1986). Not only was DDC very effective at this site, but the additional penetration resistance increase during the first 8 days following treatment was very significant.

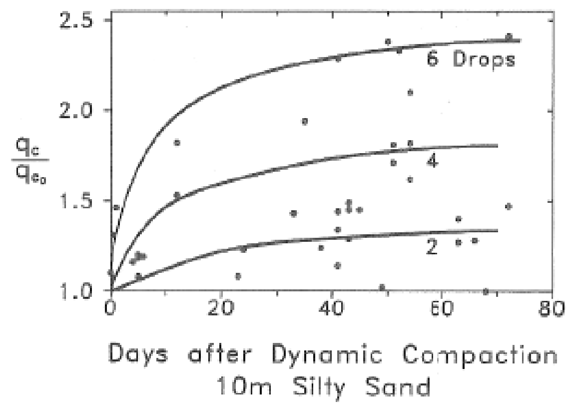

Fig. 12. Normalized increase in cone penetration resistance as a function of time after deep dynamic compaction. (after Schmertmann, 1987, 1991).

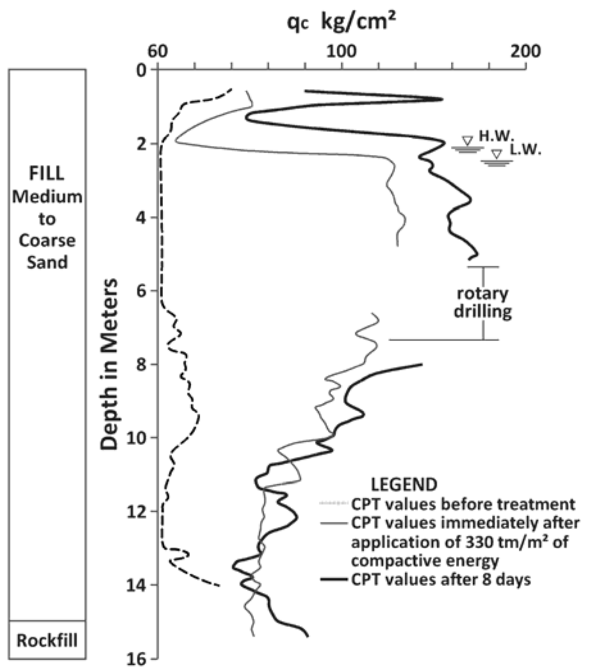

Fig. 13. Effect of DDC and aging on the penetration resistance of a 6-17 m thick medium-coarse clean sand loosely deposited by end dumping in water (redrawn from Dumas and Beaton, ASCE, 1988)

Ground improvement at Chek Lak Kok Airport in Hong Kong required densification of sand hydraulic fill (Ng, et al. 1996). Vibrocompaction was used, and CPT tests done up to 47 days following densification gave the results shown in Fig. 14.

The mobile caisson Molikpaq was deployed at four sites in the Canadian Beaufort Sea for offshore hydrocarbon drilling (Bruce and Harrington, 1982; Jefferies, et al., 1985; Jefferies and Rogers, 1993). The square open 70 m by 70 m center core of the 100 m wide octagonal structure was filled with hydraulically placed sand (D_{50}=0.3-0.4 mm, <8% silt). Because of potential instability under cyclic ice loading, blast densification of the 25 m deep sand was used for the deployment at Amauligak F-24 (Stewart and Hodge, 1988). Blasting produced an overall settlement of about 1 m, corresponding to a vertical strain of about 5 percent. Jefferies and Rogers (1993) report a significant post-blasting aging effect, with a typical example (from Rogers, et al, 1990) shown in Fig. 15 where CPT records for pre-blasting, 1.5 days after and 16 days after blasting are shown. The CPT resistance continued to increase for several weeks at a decreasing rate. These results contrast with those for a prior deployment of Molikpaq at the Tarsuit P-45 project where the sand was not densified after placement (Jefferies, et al. 1988). In that case CPT q_c values were constant over 10 months.

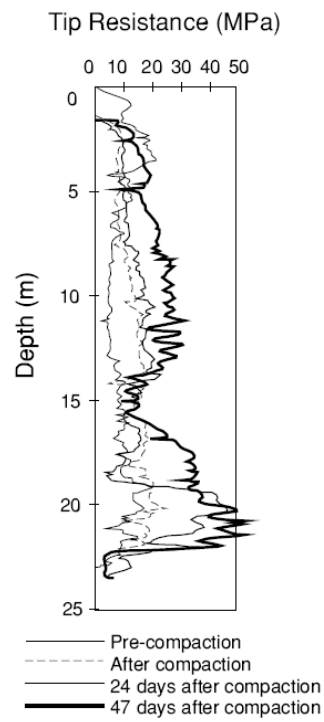

Fig. 14. Effect of aging on CPT penetration resistance at Chek Lak Kok Airport after densification by vibrocompaction (from Ng,et al, 1996)

The effects of blasting and time on the tip resistance, skin friction, and friction ratio at a site underlain by a 1.5 m thick layer of poorly graded medium to fine sand over a 3.6 m layer of poorly graded gravelly sand were studied by Charlie, et al. (1992). These materials were initially classified as dense to very dense. As might be expected, shortly after blasting (one week), the tip resistance and sleeve friction were less than the pre-blast values. After 18 weeks the normalized tip resistance had increased by an average of 12 percent, and the sleeve friction had decreased by about 40 percent relative to the one week values. After 5-1/2 years the tip resistance had increased by 211 percent, to a value greater than the pre-blast value, but the sleeve friction had decreased by 42 percent relative to the one week value. The high initial relative density was likely a major factor in the initial penetration resistance drops as a result of blasting. Charlie et al. (1992) suggest that decreasing horizontal stress with time may be the cause of the measured decreases in sleeve friction.

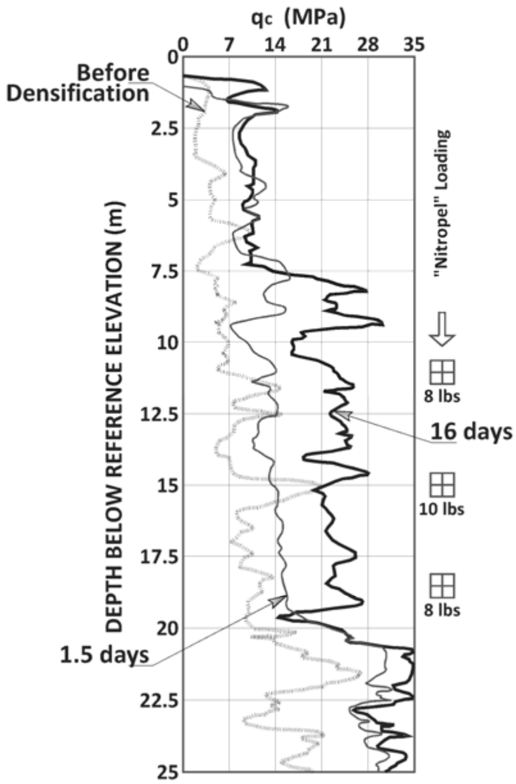

Fig. 15. Effect of time on the CPT resistance of Molikpaq sand core hydraulic fill after blast densification (data from Rogers, et al. 1990)

A full-scale blast-induced liquefaction test was done by Ashford, et al (2004) at the Treasure Island, CA, National Geotechnical Experimentation Site. A hydraulic fine sand and silty sand fill was placed during 1936-37 and was about 8 m thick at the test location. Settlement of up to 100 mm, equivalent to 2.5 percent of the liquefied thickness, was induced using two passes, three days apart, of 0.5 kg charges at 16 detonation points. About 85 percent of the settlement had occurred within the first 30 min after the blast. The cone penetration resistances two days after the second blast were less than prior to the first blast. However, Ashford, et al (2004) state that after a third blast using about the same charge pattern, the tip resistance between 2 and 5 m depth had increased to twice the pre-blast value and 2.5 times the value two days after the second blast.

There are other reported cases where increases in CPT resistance with time were either not detected or uncertain at best. Human (1992) reports on the results of investigation at the man-made hydraulically filled Bay Farm Island in Alameda, CA. CPT records were available prior to the 1989 Loma Prieta earthquake. Sand boils were observed above the 4 m thick fine silty sand fill following the earthquake. Four CPT's were done 4, 14, 30, 65, and 317 days after the earthquake. At 4 days the penetration resistances were the same as the pre-earthquake values. There were no consistent increases thereafter, and it was concluded by Human (1992) that observed increases were within the natural variability of the deposit. Gohl, et al. (1994) report on a blast densification test at a site in British Columbia where 2 to 3 m of random fill overlay loose sand. Blasting caused liquefaction of the sand as indicated by settlements up to 1 m and water discharging at the surface. Penetration test measurements at times up to 450 days after blasting were inconsistent in indicating resistance increases. Liao and Mayne (2006) used blasting to study the liquefaction and post-liquefaction behavior of sandy layers at two sites in the New Madrid Seismic Zone. While the blasting disturbance caused a short-term decrease in penetration resistance of about 10 percent, there was no significant increase over the next eight months. The authors speculate that the freshwater environment might have been responsible for this absence of aging effects.

Liquefaction Resistance. Aged natural deposits of sand are known to be more resistant to liquefaction under seismic loading than recent (Holocene) deposits (Youd and Perkins, 1978). Seed (1979) presented the data in Fig. 16 indicating that the cyclic shear strength, and, therefore, the liquefaction resistance of sands increases with time under stress.

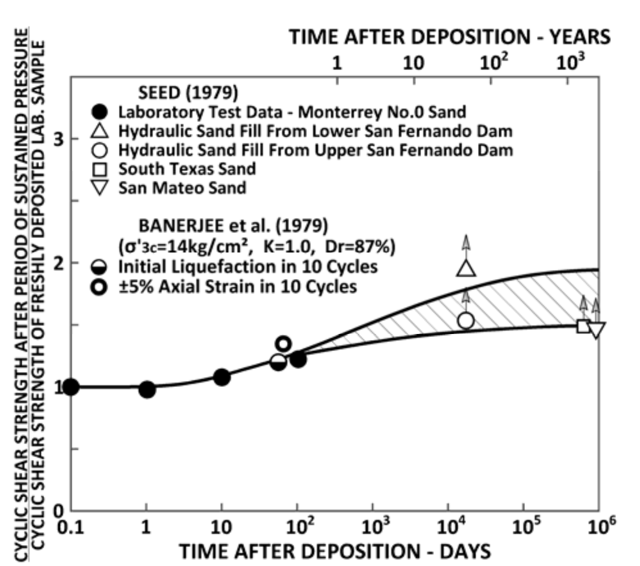

Fig. 16. Effect of time under confinement on cyclic shear strength (adapted from Seed, ASCE, 1979)

Arango, et al. (2000) supplemented the data and reanalyzed the curves in Fig. 16. Cyclic strength data were obtained for Miocene-age clayey sands from the Charleston, SC area and used to develop the relationships shown in Fig. 17. The strength gain factor indicates the cyclic strength relative to unaged samples and can be applied to field conditions where penetration tests do not properly reflect the true liquefaction resistance.

Leon, et al. (2006) analyzed data on aging effects on cyclic strength of the aged sand deposits of the South Carolina Coastal Plain. They found that for these materials the cyclic resistance ratio was some 60 percent greater than for the young Holocene sands used to establish the boundary curves of *CSR* vs $(N_1)_{60}$, *CSR* vs $(q_c)_1$, and *CSR* vs V_{s1}, currently used for assessment of liquefaction potential. In a discussion to Leon, et al. (2006), Monaco and Schmertmann (2007) suggest that the parameter K_D determined using the Flat Plate Dilatometer test (DMT) can provide a more detailed indication of the effects of aging on the *CSR* than can the CPT, thereby making it a better indicator of liquefaction potential.

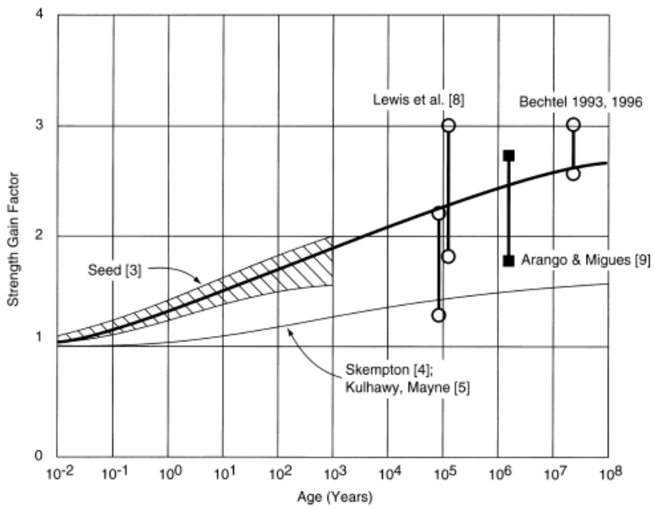

Fig. 17. Updated field cyclic strength relationship proposed by Arango, et al. (2000)

Pile Set-up. Much field data indicates that the load-carrying capacity of piles driven into sand may increase significantly over months, long after pore pressures have dissipated; e.g., Chow,et al. (1997, 1998); Bowman, (2002); Bowman and Soga (2005), and Jardine, et al (2006). The magnitude of the increase is variable, as may be seen in Fig. 18; however, most of it is due to increased shaft resistance rather than tip resistance. Piles driven into silts and fine sands set up proportionately more than those in coarse sands and gravels (York, et al, 1994). Both driven and jacked piles exhibit set-up, whereas bored piles do not. The mechanisms responsible for these time-dependent effects are considered in the next section of this paper.

MECHANISMS FOR SOIL PROPERTY CHANGES DURING SAND AGING

Several hypotheses have been advanced for why and how time-dependent property changes develop in sands over relatively short (days to months) periods after deposition or disturbance. These include (1) chemical precipitation and cementation, (2) micro-biological processes, (3) physical processes involving particle rearrangement, slow compression and stress changes, and (4) combinations of (1), (2), and (3). Each of these is described in this section and examined in terms of its consistency and ability to account for the aging effects on soil properties and behavior presented in the last section. Before doing so, however, it is helpful to summarize the key characteristics of the sand aging phenomena and processes that can be deduced from the numerous examples given in the last section. These include:

- Some existing natural deposits of clean sand exhibit "sensitivity" in the form of a strength loss when disturbed.
- Excess pore pressures generated as a result of disturbance or densification dissipate rapidly, usually in minutes, so continuing consolidation of the type common in clays is insignificant.

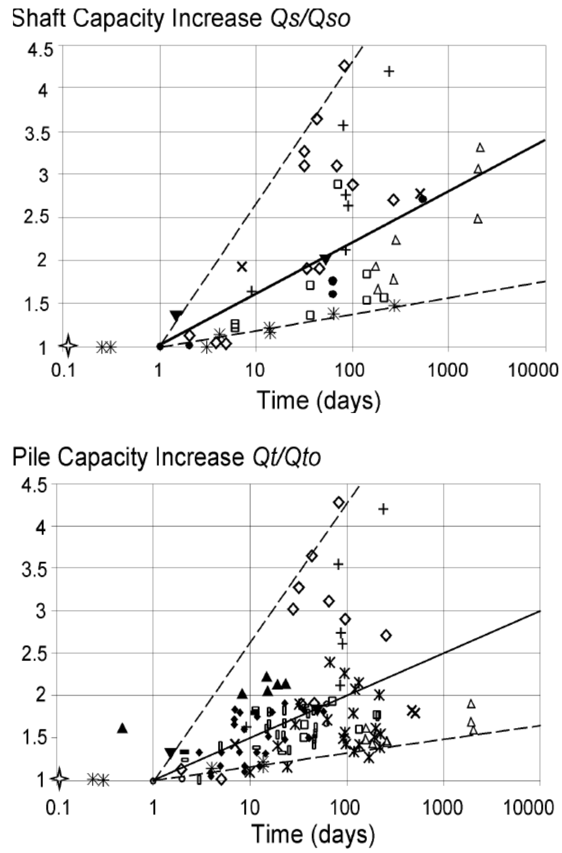

Fig. 18. Increase in shaft and total pile capacity with time for displacement piles in sand (from Chow, et al, 1998 and Bowman, 2002).

- The mechanical consequences of sand aging include increasing shear wave velocity, stiffness, strength, penetration resistance, liquefaction resistance and pile set-up with time.
- Aging effects have been measured in both dry and saturated sands, with the effects being greater in saturated sands. Partly saturated sands are not considered herein, owing to the complicating influences of capillary pressures that can be expected to change with disturbance, densification and time.
- In some cases there may be significant increases in shear wave velocity and initial modulus without change in the penetration resistance or measured strength.
- Increasing penetration resistance with time has been measured in the field following deposition of sand fills and sand densification by means of explosive

compaction, deep dynamic compaction, and vibrocompaction.

- In some cases, especially those involving sand densification using explosive compaction, the penetration resistance immediately following treatment may be less than that before blasting even though significant ground settlement has been measured.
- Compression during aging of sand samples in the laboratory under constant effective stress is very small, often undetectable.
- Continued settlement of the ground surface following rapid densification of sand deposits is very small, often not detected.
- There are some cases where the penetration resistance of sands has not increased with time following densification.
- Pile set-up in sands is primarily the result of increases in shaft resistance with time.
- Low-level (about 20 % of shaft capacity) one way cyclic loading accelerated the beneficial aging processes causing pile set-up in sands (Jardine, et al, 2006).

The dominant mechanisms in any case should be compatible and consistent with these observations.

Chemical Precipitation and Cementation

The large decreases in penetration resistance and force required to push a casing into the sands at the Jebba Project (Fig. 1) after disturbance followed by the significant time-dependent increases in penetration led Mitchell and Solymar (1984) to hypothesize that rupture and reformation of cementing bonds were the primary causes of the unexpected behavior. This initial concept was reinforced by reports that iron oxide coatings had been detected on some of the sand particles during earlier site investigations. In addition there were other findings in the literature that lent support to a cementation mechanism.

For example, Terzaghi referred to a "bond strength" responsible for a quasi-preconsolidation pressure in the field (cited by Schmertmann, 1991). This could be thought of as a type of cementation that would increase cohesion without affecting the friction angle. Denisov and Reltov (1961) showed that quartz sand grains adhered to a glass plate over time. They placed individual sand grains on a quartz or glass plate and measured the force necessary to move the grains when the plate was vibrated, as shown in Fig. 19. Dry grains were placed on the plate for varying times and then the plate was submerged, also for varying times, before the vibrations were started. The force required to initiate grain movement continued to increase with time as shown. The cementing agent was thought to be derived from silica gel precipitated at particle contacts. Increased strength could then be derived from crystal overgrowths caused by pressure solution and the compressive stresses. A difficulty with this hypothesis, however, is that pressure solution is unlikely at the temperature and pressure conditions; e.g., 0º C to 30º C and effective stresses less than 200 kPa or so, under which sand aging of the type discussed in this paper is important.

The laboratory aging study by Joshi, et al (1995), described in connection with Fig. 11, showed greater effects of aging in submerged samples than dry sand. Scanning electron micrographs of specimens aged for two years in distilled water and sea water showed precipitates on and in between sand grains. The precipitates on the river sand in distilled water were composed of calcium and possibly silica. For the same sand in sea water they were composed of sodium chloride.

The results obtained in the aging study by Baxter and Mitchell (2004), also described earlier, indicated significant dissolution of both calcium and silica during aging, and chemical analyses suggest that precipitation of carbonate and silica occurred in two tests. Electron microscope studies of samples after aging failed to indicate any evidence of precipitated material; however, detection of thin films by electron microscopy is known to be very difficult. The amount of ions in solution was greater for the Evanston Beach sand (which contained dolomite) than in the Density sand. The largest increases in dissolved Ca^{2+} and HCO^- occurred in samples of Evanston Beach sand with carbon dioxide-saturated water as the pore fluid. There was no corresponding increase in the concentration of dissolved silica for these samples.

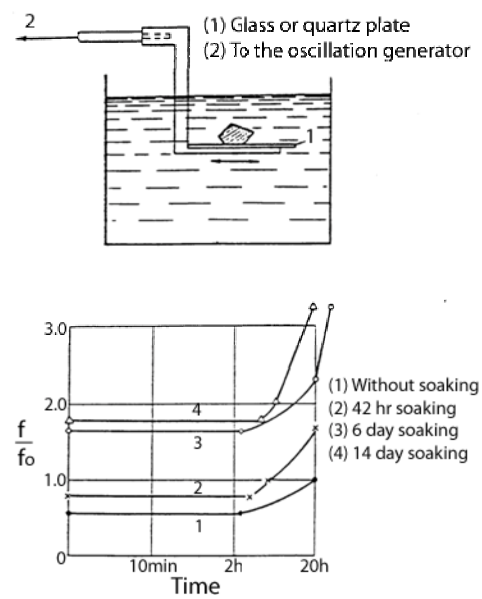

Fig. 19. Vibrating plate experiment results from Denisov and Reltov (1961). f/f_o is a measure of the bonding force between sand particles and quartz or glass plate.

The concentrations of SiO_2 in solution after aging at 25° C ranged from 12 to 28 ppm in distilled water and 16 to 36 ppm in carbon dioxide-saturated water. For the tests performed at 40° C, the SiO_2 concentrations ranged from 24 to 34 ppm. The known solubility of SiO_2 at 25° C is 11 ppm and at 40° C is 16 ppm, which suggests that the solutions were slightly supersaturated with respect to silica. Therefore, some small precipitation of silica might have been expected. Silica solubility reactions in water are described by Dove and Rimstidt (1994).

Predicted dissolution rates of quartz range from 10^{-13} mol m^{-2} sec^{-1} at pH 2 in 0.0001 m NaCl to 10^{-10} mol m^{-2} sec^{-1} at pH 12 in 0.20 m NaCl (Dove and Elston, 1992). Using the upper bound value of 10^{-10} mol m^{-2} sec^{-1}, and assuming a fine sand, a simple calculation leads to about 0.00005 kg of quartz per kg sand dissolved per year, a very small amount. Even if all this material then immediately precipitates to cement interparticle contacts, a significant strength gain in a year from this source seems unlikely.

Thus, the potential for silica cementation at levels sufficient to account for the observed strength increase in clean sands due to aging seems small. More cementation is likely if carbonates, alumina, or iron are present. Precipitated calcium carbonate is not particularly strong; whereas, AlOOH, $Al(OH)_3$. FeOOH, $Fe(OH)_3$, and alumino silicates, if formed, may provided more significant levels of cementation. However, the formation of these materials is more likely in evaporate deposits, where there are high concentrations in solution and periodic episodes of wetting and drying, than in continually saturated sands. Furthermore, the cementation hypothesis cannot reasonably account for the aging effects, predominantly increases in low strain modulus, that have been measured in dry sands.

For these reasons and others offered by Mesri, et al (1990), the original clean sand aging hypothesis by Mitchell and Solymar (1984) based on silica cementation does not seem tenable, at least in clean silica sands, as the major cause of the observed mechanical property changes resulting from sand aging over times of weeks to months. Mesri, et al (1990) make the observation that "--- a hypothesis that implies cementing bonds forming at grain contacts of clean silica sands in a matter of weeks to months would preclude the existence of any uncemented natural sand deposits."

Micro-biological Processes

"Microbial processes influence rock weathering, mineralization, soil formation and fabric, and soil grain surface properties. They can produce slime, gel, polymer, and biomass, cause pore and filter clogging, and change the deformation and strength properties of soils." This conclusion, taken from Mitchell and Santamarina (2005) reflects the pervasive nature of microorganisms in the geoenvironment and suggests that biological factors might play a role in property changes during sand aging. In fact, in a recent laboratory study DeJong, et al (2006) developed calcite cementation in Ottawa sand by introducing and nourishing *Bacillus pasteurii* under controlled conditions, resulting in increased initial shear stiffness and ultimate shear capacity compared to untreated specimens.

However, whether processes such as this could be a significant contributor to sand aging property changes of the type discussed herein is doubtful. They would require the initial presence or influx of large quantities of bacteria, as well as a source of energy for their reproduction beyond what is initially present at the time of deposition or densification of the sand. Continued permeation by waters containing organics and microorganisms might provide a source. Bacteria might adhere to charged mineral surfaces at small pore throats to bind grains through charged membranes, polysaccharide films, or biofilms if the nutrient level is high. It should be noted, however, sand aging has been observed under both flowing and static water conditions.

Bacterial mediation is known to accelerate geochemical redox processes, especially iron, sulfur and manganese oxidation; however they have little influence on silica and alumina solution and precipitation rates. Therefore, overall it seems reasonable to conclude that while under special conditions microbiological processes may play a role in sand aging; they are not likely to be the major cause of the phenomena being addressed in this paper.

Physical Processes

Subsequent to publication of the hypothesis by Mitchell and Solymar (1984) that chemical bonding and cementation were the likely causes of the observed sensitivity and time-dependent strength gains in the clean sand at the Jebba Dam project, Schmertmann (1987), Mesri, et al. (1990), Schmertmann (1991), and others proposed an alternative mechanism based on slow particle rearrangements and stress redistributions that accompany drained creep deformations; i.e., secondary compression, subsequent to dissipation of excess pore pressures.

Schmertmann (1987) suggests that ground improvement methods such as explosive compaction and vibrocompaction create zones of low lateral effective stress immediately around a blast or vibrocompaction point. Although experience indicates that vibrocompaction is more likely to increase rather than decrease lateral stresses, one can imagine that, somewhat similarly to blasting, reduced horizontal stress might exist around deep dynamic compaction drop-weight impact points owing to the cratering that develops. As penetration resistance is a strong function of lateral effective stress, subsequent increases in horizontal stress with time would result in penetration resistance increases, such as shown, for example, in Fig. 12 for DDC.

In addition, Schmertmann (1987, 1991) states that particle reorientations during secondary compression can result in significant increases in modulus and strength owing to increased effective friction. Mesri, et al (1990) conclude, based on careful analysis of the compression behavior of sands, evaluation of the time behavior of clean sand after densification, new laboratory test data, study of the data in Dowding and Hryciw (1986), Solymar (1984) and Solymar, et al (1984), and geological considerations, that "during drained aging (secondary compression) of clean sands, an increased stiffness and an increase in effective horizontal stress result from continued rearrangement of sand particles resulting in an enhanced macro-interlocking of sand grains and micro-interlocking of grain surface roughness." They propose an equation for the rate and magnitude of the CPT penetration resistance increase based on the compression and secondary compression indices, as is discussed later in this paper.

A key element in this explanation for the time-dependent increases in modulus, strength and penetration resistance is that they cannot be explained solely by the densification that occurs during secondary compression. As illustrated by Mesri, et al (1990), the void ratio decrease is much too small to account for their magnitude. This is readily confirmed by reference to established correlations between properties and relative density for sands; e.g., Kulhawy and Mayne (1990. The important consequences are the small slips and rearrangements of particles and the changes in normal and shear forces at interparticle contacts as the particles adjust to the new void ratio and stress conditions from their initial post-depositional or post-densification fabric. The process can perhaps be viewed in many ways as similar to thixotropic hardening in fine-grained soils (Mitchell, 1960) except that in clays, initially unbalanced physico-chemical forces of attraction and repulsion are responsible for small particle movements and fabric changes, whereas, in sands, gravitational forces and applied stresses provide the necessary driving forces for internal structural changes. These similarities between cohesionless soil aging (mechanical aging) and thixotropy in clays, as well as the differences, are noted also by Schmertmann (1991).

Support for the postulation that sand particles do rotate and displace during the creep and aging process was provided by Bowman and Soga (2003). Using resin injection techniques they showed that following the application of load in one-dimensional creep, particles aligned perpendicular to the load direction, but with time they rotated in space. Frictional slippage of weakly loaded particles occurred during the early stages of creep, but with time, the particles clustered together and formed strongly loaded columns. Bowman and Soga (2003) state that "the model may help to explain the complex volumetric creep response of dense soils and why dynamically densified soils 'age' with no detectable change in relative density."

On a macroscopic scale the consequences of aging according to this mechanism should be manifested by time-dependent increase in frictional resistance. Macro-interlocking of sand grains would result in greater dilatancy, and micro-interlocking of grain surface roughness would produce increased sliding resistance, both contributing to a higher total resistance to deformation. Volume change measurements were made by Daramola (1980) during the strength tests shown in Fig. 6, and they gave the results shown in Fig. 20 indicating that the dilatancy increased with sample age, consistent with this hypothesis.

A special type of triaxial strength test, termed the IDS test was developed by Schmertmann (1976) to provide a methhod for separating the frictional contribution (tan ϕ") and the cohesional contribution (c") to a soil's shearing resistance as a function of strain. By means of this test, Schmertmann (1991) was able to determine the effect of secondary compression aging on the strength components of kaolinite, with the results shown in Fig. 21. In keeping with the behavior reported in many similar tests on different cohesive soils, it is seen that the cohesive contribution mobilizes completely at low strain, whereas, the frictional contribution to strength develops continuously with strain throughout the test. Of particular interest is that the greater the aging period, the higher the value of frictional contribution, tan ϕ", whereas, the value of c" seems to bear no consistent relationship to age. This finding is also consistent with the physical aging process outlined above, although kaolinite differs from clean sand in that it is composed of much smaller particles and possesses plasticity. The author is not aware of IDS test results on samples of clean sands.

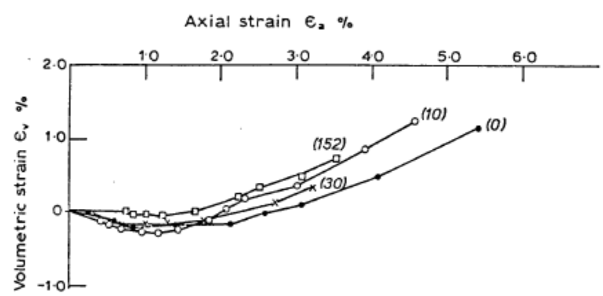

Fig. 20. Increased dilatancy with increasing sample age for Ham River Sand (from Daramola, 1980)

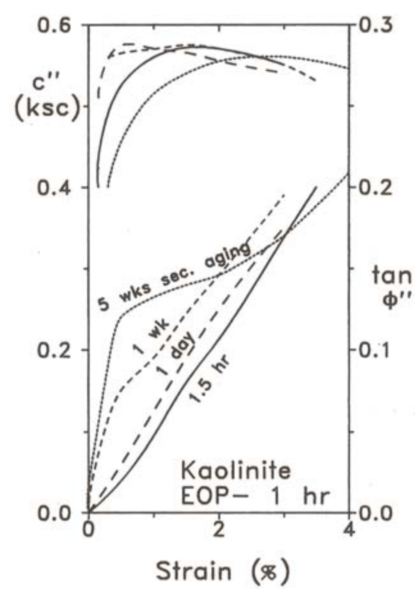

Fig. 21. Effect of aging on the rate of development of cohesion and friction during deformation of kaolinite (from Schmertmann, 1991).

A reasonable question now is whether sensitivity of in-situ sand deposits, such as shown in Fig. 1 and in other cases as a result of ground disturbance can be explained by the secondary compression-creep and stress redistribution mechanism. Sensitivity has been measured in almost all cases where densification by blasting has been done. A notable exception was the Moliqpak core densification (Rogers, et al, 1990). In this case the sand had just been placed, whereas, in the others, which were natural sand deposits, the sand had been aged for long periods. In those cases blasting destroyed the stiffened and more resistant structure that had developed by the grain rearrangements and stress redistributions. It is likely also that in the zone immediately around the blast points the effective confinement may be significantly reduced due to the blast gases and loosening. Collectively, these effects can result in lower penetration resistance even if overall there has been densification of the site. Significant sensitivity has not been observed at sites where vibrocompaction and deep dynamic compaction have been used because the nature of these processes, especially vibrocompaction, is such that sufficient positive compression has occurred adjacent to the probe and weight impact points that any strength loss by disturbance is immediately regaained.

A further test of the mechanical-physical processes explanation for aging is its adequacy in accounting for pile set up in sands. As noted earlier, the increased pile capacity with time is attributed primarily to increased shaft resistance with time after driving (setup is not observed with bored piles). A key finding was by Axelsson (2000) who reports increases with time of the horizontal stress acting on the sides of a 235 mm square concrete pile. Chow, et al (1997) considered the main process responsible for this to be "gains in the radial effective stresses acting on the pile shafts resulting from the relaxation, through creep, of circumferential arching established around the pile shafts during installation" (quoted from Jardine, et al, 2006). Reorientation of sand grains and possible cementing or micro-interlocking processes were also considered as possible contributing, but less important, factors that would result in increased sand stiffness, interface dilation, and strength with time.

Bowman (2002) and Bowman and Soga (2005) propose a somewhat different set-up mechanism based on dilatant creep and aging. The intensive sand shearing developed during pile installation results in contractive volume changes initially as the sand grains rearrange to distribute internal stresses. The volumetric creep strain then changes to dilatant, with a tendency towards volumetric expansion. However, the presence of the pile provides a kinematic constraint resulting in increased normal stress on the pile shaft, thereby increasing its load capacity. Among several types of supporting evidence cited by Bowman and Soga (2005) is that while pile set-up with time has been reported for silica sands, it has not been observed for piles driven into carbonate sands. The reason for this is that the much weaker carbonate sand particles crush under the high interparticle compressive stresses before significant dilatant stress increase on the pile can develop.

Finally, Jardine, et al (2006) note that low-level, one-way load cycling on a pile driven in sand may accelerate the rate of set-up as a result of creep acceleration under gentle vibration. This phenomenon is akin to rheopexy, a phenomenon observed in thixotropic materials, including saturated silts and clays, wherein gentle shaking accelerates the rate of gelation and stiffening.

<u>Conclusion</u>

Chemical precipitation and cementation, micro-biological, and physical processes have been examined relative to their importance as causes of the property and behavior changes associated with sand aging. Each may play some role, with its importance depending on many compositional and environmental factors. Overall, however, following deposition and/or densification, physical processes associated with particle rearrangements and internal stress redistributions that occur under the action of the new in-situ stress conditions play the dominant, if not the only significant role in producing the behavior illustrated in this paper. In essence the process may be viewed as a type of secondary compression wherein the sand structure adjusts to a new equilibrium under the new stress and other environmental conditions. The resulting changes in properties are considerably greater than can be accounted for by the small decreases in void ratio because the aging process induces a precompression-like effect.

RATE AND MAGNITUDE

From a practical viewpoint the two most important concerns for the geotechnical engineer interested in accounting for sand aging in practice are (1) how much change in a given property is there likely to be, and (2) how long will it take. Available information that addresses these questions is summarized in this section.

Information on the rate and magnitude of property changes with time of sand aging from several references is summarized in Table 4 and the following figures. Included in Table 4 are the relevant references, the property of interest, and the types of measurements or other basis of quantification or prediction. The normalized increment factor shown in Fig. 22 as a function of log time represents the increase in any specific property normalized in terms of an initial property value at a short reference time after deposition and/or densification. The curves show that (1) a specific property may as much as double during the first year and (2) there is a large variation among the reported results by different investigators and the predictions using the different equations that have been proposed.

As there is no fundamental reason that different sand property types should increase at the same rates during aging, the curves in Fig. 22 have been separated by type in the following figures. Aging relationships for initial shear modulus are shown in Fig. 23. The overall range for different sands can be large, as shown by the approximate upper bound curve developed from Jamiolkowshi and Manassero (1996), largely because of the high values of N_G for carbonate sands (see Fig.

5). For clean silica sands N_G is usually of the order of 0.03 or less and the experimental data from Human (1992), Howie, et al (2002), and Baxter and Mitchell (2004) are in this range.

Table 4. Information on Rate and Magnitude of Sand Aging Effects on Sand Properties

Property	**Reference**	**Type of Data**	**Predictive Equation**	**Comments**
Initial Shear Modulus, G_o	Anderson and Stokoe (1978), Mesri, et al (1990), Jamiolkowski and Manassero (1996)	Mainly Resonant column	$G_t = G_{1000}(1 + N_G \log \frac{t_2}{t_{1000}})$	$0<N_G<0.05$; t_{1000} is reference time of 1000 min.
	Human (1992)	Laboratory tests		G computed from Vs measured using bender elements
	Troncoso and Garces (2000)	Downhole shear wave velocity measurements at 4 tailings dams	$G_n = 117.24t^{0.67}$	See Fig. 7 for definition of terms and units
	Howie, et al (2002)	Triaxial tests after consolidation and aging at different σ_1/σ_3 values		G computed from Young's Modulus, E, at low strain
	Baxter and Mitchell (2004)	Laboratory tests in fixed ring consolidometers		G computed from V_s measured using bender elements.
CPT Resistance, q_o	Dumas and Beaton (1988)	Field CPT		Only data for one time
	Mesri, et al (1990)	Lab compression tests, lab blast tests by Dowding and Hryciw (1986), Jebba Dam data (Solymar, 1984), and post-DDC field data from Schmertmann, et al (1987)	$\frac{q_c}{(q_c)_R} = \left(\frac{t}{t_R}\right)^{C_D C_\alpha / C_C}$	$(q_c)_R$ is a reference q_c at reference time t > time for primary consol. C_C is compression index, C_α is secondary compression index, C_D is a parameter, in the range of about 3 to 20, dependent on densification in excess of compression under static loading.
	Schmertmann (1991)	CPT tests following deep dynamic compaction		q_c/q_o vs. time determined from Fig.14 in Schmertmann (1991)
	Charlie, et al (1992)	Field data from several sources	$(q_c)_N / (q_c)_1 = 1 + K \log(N)$	N in weeks, K in the range of 0 to 1 and dependent on soil type, densification method and temperature.
	Joshi, et al (1995)	Lab samples and a 10 mm dia. penetrometer	$P_t / P_1 = a(t)^b$	Example of fit to measured data shown in Fig. 11
SPT Resistance, N	Kulhawy and Mayne (1990)	Lab tests and field data from several sources	$(N_1)_{60} = D_r^{\,2} C_A C_P C_{OCR}$ $C_A = 1.2 + 0.05\log(t/100)$	D_r is relative density, t in years, C_P is 60+25logD_{50} (in mm), C_{OCR}=(OCR)$^{0.18}$
	Al-Sanad and Ismael (1996)	N-values from 4 to >9 in one year.		Calcareous sand
Static Shear Strength	Al-Sanad and Ismael (1996)	Lab direct shear tests	$\phi = \phi_i + \frac{3.81t}{(2.82 + t)}$	t is time in months; aging under only 2 kPa confining pressure
Cyclic Shear Strength	Seed (1979)	Cyclic loading in lab tests after different periods of confinement		See Fig. 16 for comparisons of lab data with samples from several project sites
	Arango, et al (2000)	Cyclic loading of clayey sand samples		Augmented Seed (1979) data – see Fig. 17
Pile Capacity	Chow, et al (1998)	Pile load tests	Linear vs. log time: 50% per cycle.	Approximates the average well

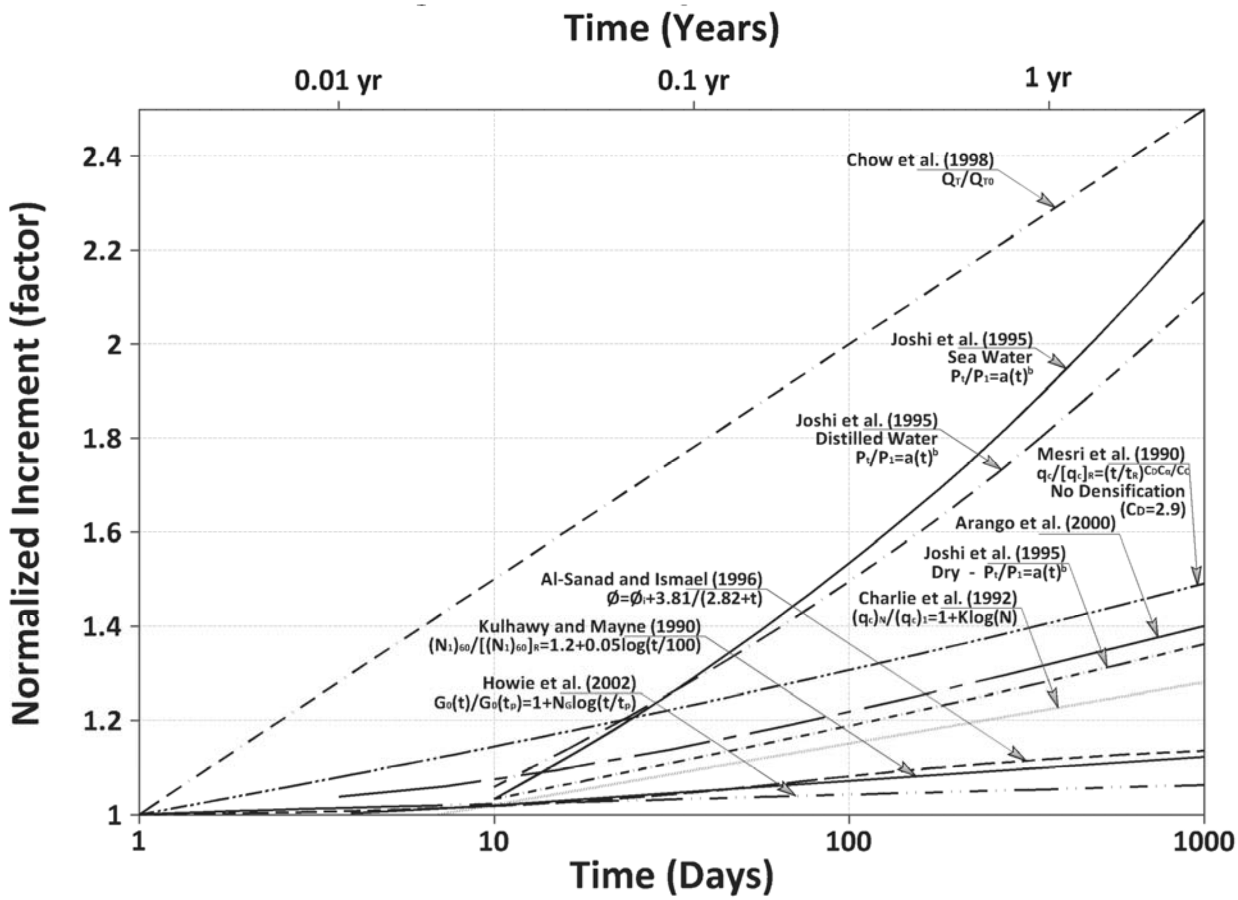

Fig. 22. Summary of measured and predicted rates of property increase as a result of sand aging.

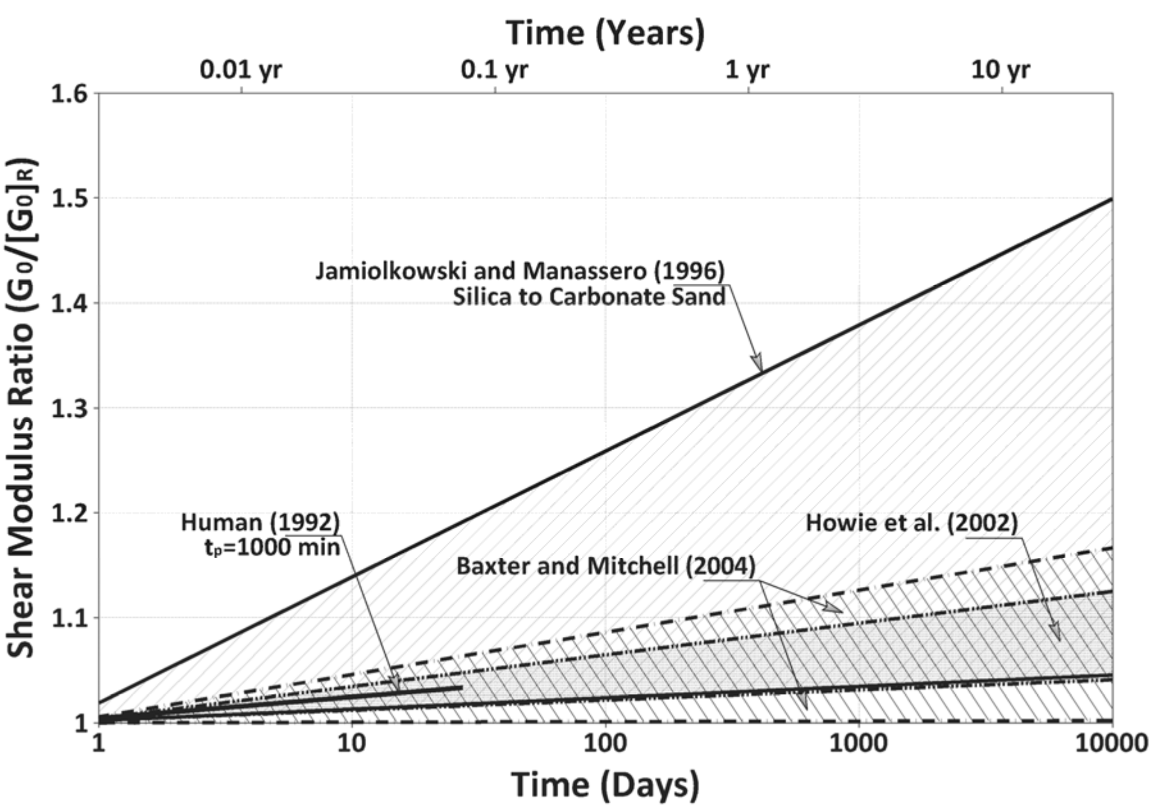

Fig. 23. Rate of initial shear modulus increase as a result of sand aging

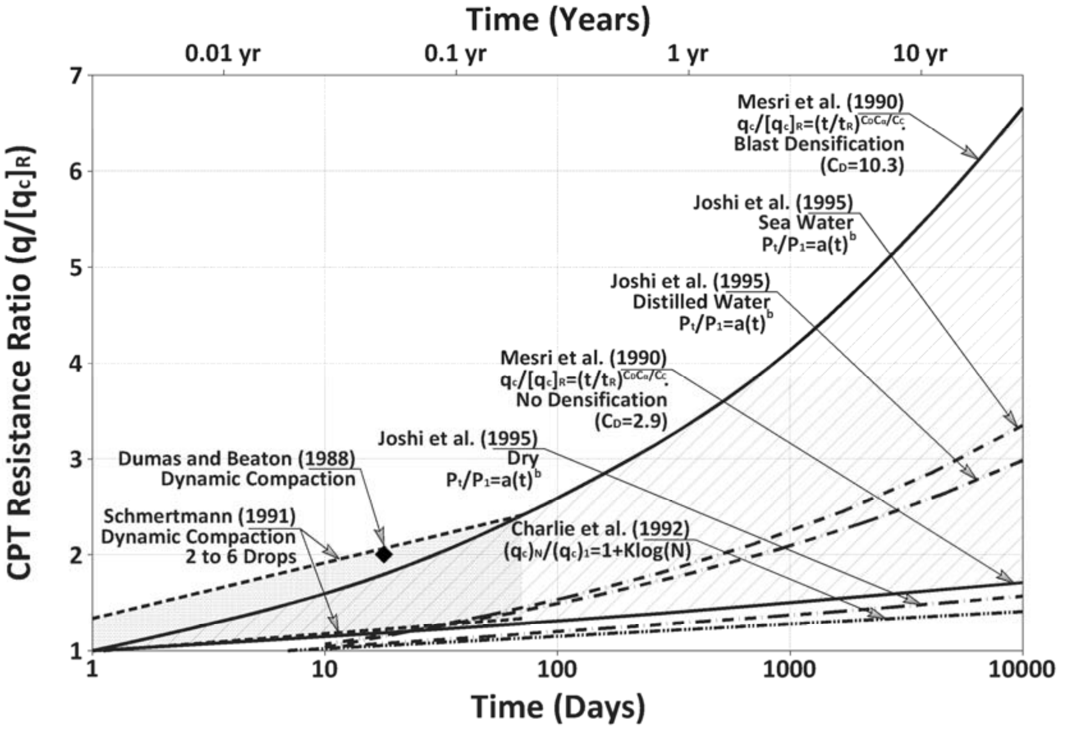

Fig. 24. Rate of CPT penetration resistance increase as a result of sand aging

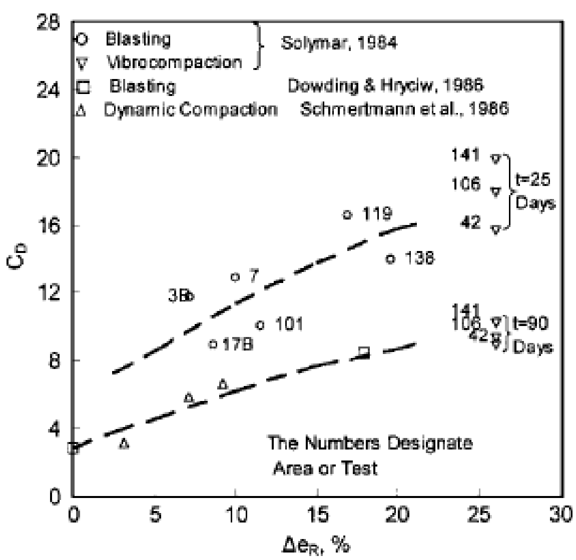

Fig. 25. Summary of C_D data for ground improvement projects (from Mesri, et al, ASCE, 1990).

Normalized curves for CPT tip resistance as a function of time are shown in Fig. 24. The range of measured and predicted values is large. The equation proposed by Mesri, et al (1990) brackets most of the results. The controlling parameters in this equation are the ratio of the secondary compression ratio to compression index, C_α/C_C and C_D, with C_D having by far the greatest effect. The bracketing values of C_D shown in Fig. 24 were back-calculated by Mesri, et al (1990) by fitting their equation, given in Table 4, to the aging data obtained from the case history measurements. The greater the potential for further particle rearrangement and redistribution of internal stresses following densification, the higher the value of C_D. This is further illustrated by Fig. 25 from Mesri, et al (1990), which shows back-calculated values of C_D as a function of increase in relative density, Δe_r, for several densification case histories. As Mesri, et al (1990) note, "The aging effect is most pronounced in clean sands that have been rapidly densified, thus producing a substantial potential for improved sand grain and surface interference that has not been fully realized by the time densification is completed." Reliable methods for computing or estimating C_D in advance on a project so far do not appear to be available.

Fewer data are available on the effect of aging on SPT N-values, Fig. 26. The Al-Sanad and Ismael (1996) data are limited and were obtained for a specially prepared field test section containing calcareous sand, which might develop cementation not likely to form in clean silica sand. The Kulhawy and Mayne (1990) relationship is an approximation based on a widely scattered set of values, most of them for sands with ages greater than about 50 years, well beyond the time period of interest in Fig. 26.

Only the data by Al-Sanad and Ismael (1996), Fig. 8, show a clear increase in static shear strength ratio. The rate of increase is shown in Fig. 27 where $\tan\phi_t/\tan\phi_{t1}$ is plotted as a function of time. Since static shear strength is a large strain property, the initial sand structure formed during aging is likely to be destroyed before the peak or ultimate strengths are reached, as illustrated, for example by Daramola's (1980) results in Fig. 6. On the other hand, the cyclic shear strength, also shown in Fig. 27, does show a continuing increase with time as it is typically based on the number of load cycles or stress level to reach 100 pore pressure ratio or a specified shear or axial strain. This difference, i.e., little or no improvement in strength values in static strength tests, but significant increases in cyclic strength, is reasonable. The static strength is an endpoint value, whereas, the cyclic strength as defined reflects the total energy needed to break down the sand structure.

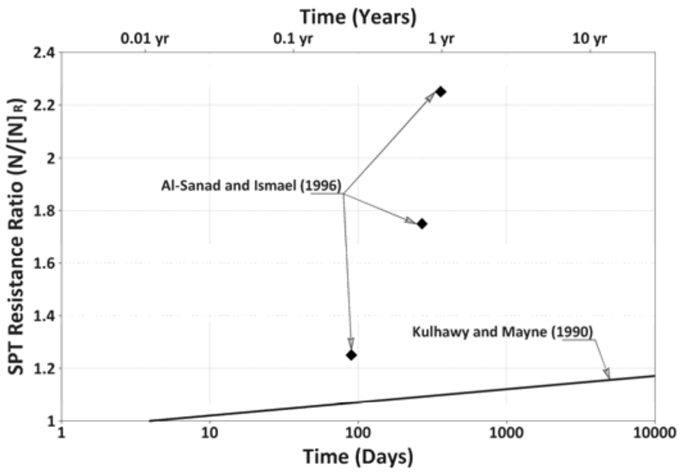

Fig. 26. Rate of SPT penetration resistance increase as a result of sand aging.

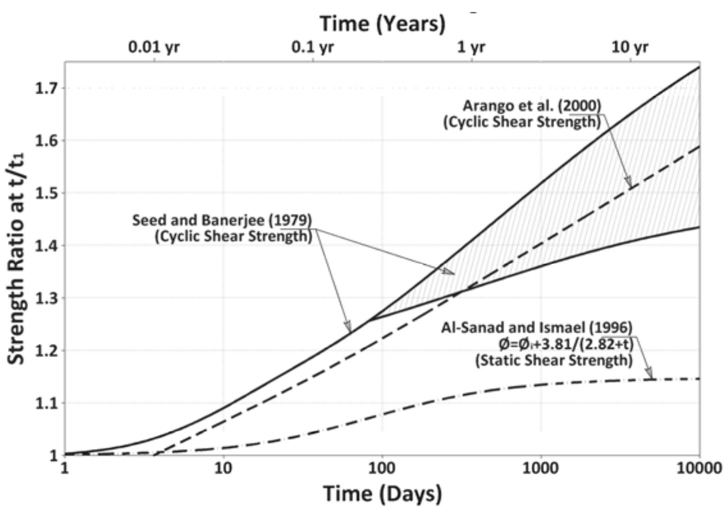

Fig. 27. Rate of cyclic strength increase as a result of sand aging.

The line labeled Chow, et al (1998) in Fig. 22 is taken from the average line, bottom plot in Fig. 18, for ratio of total driven pile capacity at time *t* to that at t_o, Q_t/Q_o, and indicates an increase of 50 percent per log cycle of time. As the increase in total capacity is attributable mainly to increase in shaft resistance, the rate of increase of shaft resistance ratio, Q_s/Q_{so}, is somewhat greater, as may be seen in the upper plot in Fig. 18.

<u>Effect of temperature</u>

The rates of most time-dependent phenomena in soils, including creep or secondary compression processes, are temperature dependent, with higher rates associated with higher temperatures. Detailed discussion of temperatures effects and their influences on "rate processes" can be found in Mitchell and Soga (2005). Limited data reported by Charlie, et al (1992) showed an approximate correlation between the coefficient *K*, and temperature in the following equation for CPT penetration resistance and time:

$$(q_c)_N/(q_c)_1 = 1 + K\log(N) \qquad (4)$$

In which *N* is the aging time in weeks and *K* is in the range of 0 to 1.

In a discussion of Charlie, et al (1002), Jefferies and Rogers (1993) provide a correction to one of the data points and suggested the relationship between K and temperature shown in Fig. 28.

Clearly, more data is needed to establish a strong correlation between temperature and aging process rates. Nonetheless, it does appear that temperature can be important, and knowledge of the effects of this important parameter may be useful in development of more fundamental relationships concerning both the mechanisms and factors controlling the rate and amount of property changes with time. Similarly, discrete element and rate process approaches such as developed by Kuhn and Mitchell (1993) for describing soil creep in terms of the sliding velocity of two contacting particles as a function of the ratio of the tangential to normal interparticle contact force components may be useful for further improvement of both fundamental understanding of the aging process and developing better predictive methods.

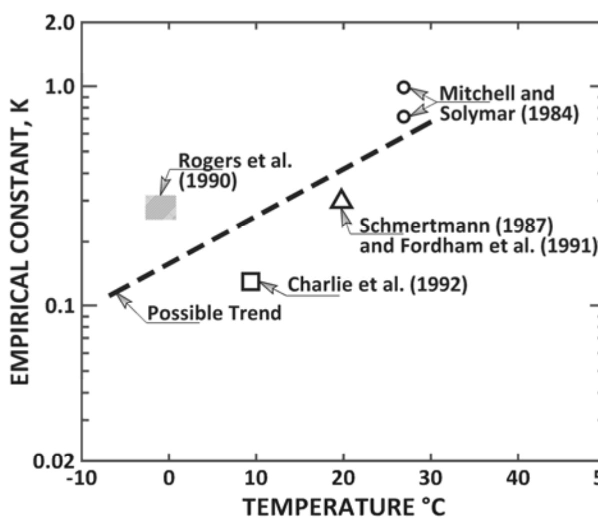

Fig. 28. Effect of temperature on the aging rate constant in equation (4) (modified from Jefferies and Rogers, 1993)

CONCLUSIONS

In this state-of-the-art and practice paper I have reviewed the phenomenon of sand aging wherein disturbed natural deposits of clean sand may exhibit a sensitivity or loss of strength and then, following re-deposition and/or densification show an increase in stiffness, penetration resistance, and liquefaction resistance. Illustrative examples are presented, possible mechanisms are examined, and quantitative information about the magnitudes and rates of property change are given. Several conclusions are warranted based on this review:

1. Aging and the associated improvements in stiffness, penetration resistance and liquefaction resistance are ubiquitous among freshly deposited and/or densified deposits of silica sands.
2. The observed pattern is that natural sands develop a structure over time following deposition and/or disturbance that can exhibit a metastability that is in many ways similar to sensitivity in clays.
3. Chemical and microbiological processes that lead to interparticle bonding and cementation are not the major contributor to property changes during sand aging.
4. Physical-mechanical processes involving particle rearrangements and internal stress redistributions under the action of the new in-situ stress conditions play the dominant, if not the only significant role in producing sand aging effects of the type described in this paper. The associated void ratio decreases play only a minor role in accounting for the property changes.
5. The initial stiffness of clean silica sands, as measured by the shear modulus will increase by a factor of about 1 to 3 percent per log cycle of time relative to that at an age of 1000 min. The shear modulus increase factor is greater, up to 10 percent, for carbonate sands.
6. The ultimate strength of uncemented aged sands does not increase significantly by aging owing to the breakdown of the aged sand structure during the relatively larger deformations required to reach failure.
7. Although in most cases the penetration resistance shows significant increases with time after disturbance and densification, a few cases have been cited where there was no significant increase. Most of these appear to be related to zones where the ground treatment has caused local loosening; e.g., by blasting or DDC cratering.
8. Driven pile set-up in sands can be explained as resulting from time-dependent increases in shaft resistance caused by particle reorientations and interparticle stress redistributions accompanying secondary compression-like creep deformations.
9. Whereas for silica sands the rates of low strain sheara modulus fall within a relatively narrow range (0.01 to 0.03 times the initial value for a 10-foled increase in aging time) the increment ratios for penetration resistance, liquefaction resistance, and pile set-up increases are (1) much greater, and (2) much more variable in magnitude.
10. Because property changes during aging seem to follow regular patterns with time and because they appear to occur more rapidly with increasing temperature, it may be possible to better quantify the behavior using rate process theory and discrete element methods.
11. Until more reliable methods for predicting the magnitude and rate of property improvement are available, site-specific determinations should still be used for assessment of the time-dependent aging effects for use in engineering practice.

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