THE MECHANICAL AGING OF SOILS

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(The Twenty-Fifth Karl Terzaghi Lecture)

ABSTRACT: Soils age. Their engineering properties often improve significantly during aging times of practical interest to engineers. This paper provides examples from research and practice. It includes a review of the triaxial IDS test, followed by examples of its use to investigate the question of whether the aging improvements result from frictional or cohesive effects. Contrary to most current thinking, the soil stiffening and strengthening appears entirely frictional in effect. The aging effects described appear mechanical, resulting from dispersive particle movements and internal stress arching under drained conditions. The paper concludes with suggestions for using these mechanical aging effects in practice.

INTRODUCTION

Everything on this earth has at least one thing in common—everything changes with time. All soils age and change. Mitchell (1986) discussed both surprising and important aging effects in his 1984 Terzaghi Lecture, with both positive and negative results from the viewpoint of a geotechnical engineer. However, this paper will describe primarily positive aging effects.

Aging obviously takes time. The writer defines "pure" aging as involving only the passage of time. During the passage of time, especially in the field, many events might take place that could confuse any insight into what would happen to a soil as a result of "pure" aging only. For examples of events—external horizontal or vertical stresses can change (As discussed in Schmertmann (1983), horizontal stresses may increase, decrease, or remain constant during "pure" aging. This paper does not discuss aging effects on stresses so as to limit it to a manageable scope and to reduce controversy and possible distraction from the soil structure aspects of aging.), ground-water levels can fluctuate, soils can swell or desiccate, freeze and thaw, and we can have biological attack and organic decay, or leaching, precipitation, earthquakes, and other ground movements, and chemical weathering. Nevertheless, the writer believes that sufficient information now exists that, taken together, demonstrates a pure aging effect of sufficient importance to have an impact on engineering research and practice.

A search of the literature shows many examples of the often significant effects of aging on soil engineering properties. This paper will show typically 50–100% effects. The writer will attempt, at least, to begin to organize some of the material related to this important subject. This material comes in part from his own experiences, shown here for the first time, as well as from the literature. He will try to show that aging effects over engineering times have a frictional and mechanical cause that appears to result from dispersive particle movements, possibly associated with internal stress arching.

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He will also suggest a possibly new theory for pore-pressure dissipation caused by aging effects and not the flow of water.

The paper presents many examples of aging improvement in soil engineering behavior, both from the lab and the field, for short and long aging times, and for clays, silts, sands, even gravel. These examples occupy about half of the paper. The writer then tries to answer the question as to whether these improvements in soil behavior result from an increase in the cohesive component of mobilized strength as a result of some form of cementation or internal bonding, as almost all references suggest, or from an increase in the frictional or mechanical component of strength? Finally, the writer presents data that give some insight about the soil-structure mechanics of engineering aging, and suggests practical applications of such aging.

The various examples herein include both engineering-aging times and geologic-aging times. Many aging effects can take place much more rapidly than geologic aging would imply. As shown in the following, the writer pictures engineering-aging times spanning times of interest to engineers—from a few days to 100 years. This compares with geologic aging, which spans thousands to millions of years. This paper provides evidence that suggests engineering-aging effects involve primarily the movement of particles and changes in the structure or fabric of a soil—or mechanical changes—this versus primarily changes in the particles themselves caused by chemical effects during aging over geologic time.

Examples of Aging Improvement

Preconsolidation

Let's start the examples of aging with the quasi-preconsolidation effect, referred to herein "aging preconsolidation." Leonards and his students first described this effect more than 30 years ago (Leonards and Ramiah 1960; Leonards and Altschaeffl 1964). They showed that otherwise normally consolidated (NC) soils develop an apparent approximate 40% increase in the preconsolidation stress, or an overconsolidation ratio (OCR) of 1.4, simply by letting the clay age in secondary compression. Perloff and Baron (1976) include a summary of these data. The literature has many others. For example, Zeevaert (1983) shows aging preconsolidation, which he calls "hardening," from a Mexico City clay and an organic silty clay. The writer will show more examples subsequently.

Initially, some controversy developed with the suggestion that the aging-preconsolidation effect observed in the laboratory results from a viscous, rate-of-strain, prestraining effect that would disappear under field-loading-rate conditions. To help refute this argument, Leonards (1972) presented the data in Fig. 1, which also serves here as an example of the aging-preconsolidation effect. After artificially sedimenting a clay for 46 days and then letting it sit in secondary for 100 days, rebound for 26 days, and age for another 100 days (small dots identify sedimentation curve), he used conventional one-day testing increments to reapply the loads with the results shown by the large points and heavy line. One can clearly see that the preconsolidation stress at point b increased about 40% compared with a, because of the aging. He demonstrated that secondary compression viscous effects alone could not explain the preconsolidation by pointing out that holding b for 1,000,000 days in secondary at the rate of secondary consolidation

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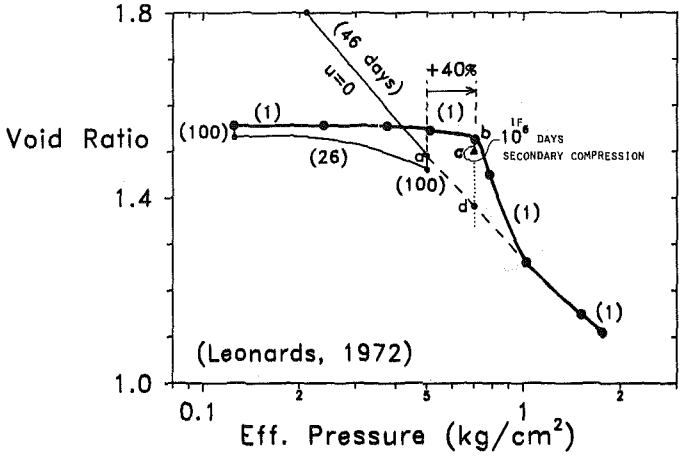


FIG. 1. Laboratory Sedimentation Example of Aging-Preconsolidation Effect

observed at that point would have only reduced the void ratio the slight amount shown to point c, rather than to d.

Leonards (1972) also noted that in his experience with natural clay deposits he had never encountered a supposedly normally consolidated clay that did not show an aging-preconsolidation effect. Fig. 2 shows one such clay deposit, as reported by Eide and Holmberg (1972). Although the site geology indicated normal consolidation, both triaxial and odeometer testing showed an OCR of about 1.5–2. The authors attributed this OCR to aging effects.

The writer searched for a well-documented example of a natural, normally consolidated clay in the classic sense that did not show an aging-overconsolidation effect. Few seem to exist. Fig. 3 presents a seemingly good example in an Italian clay, based on the data presented in Hansbo et al. (1981). The site was being considered for construction of a nuclear power plant, and the engineers involved thought they had very good quality soil tests. The average OCR equalled 0.94 from the many odeometer tests shown. The authors appeared to have presented an example of a truly NC clay. However, after the writer asked Jamiolkowski, one of the coauthors, to confirm this NC, he subsequently sent the unpublished data from the same site shown in Fig. 4. He had obtained better samples, and perhaps also did more modern testing, and obtained the new results, which showed an OCR averaging 1.20. He attributed the new OCR to a 20% aging-preconsolidation effect.

Recently, the writer also has seen more examples of clays that other engineers previously considered normally consolidated, but that recent research indicates have OCRs in the range of 1.2–1.5, most likely from aging. These all involve soft, postglacial Swedish clays, described in Larsson and Eskilson (1989a, b). Five of the nine sites described in these references involve organic clays with organic contents ranging from 5 to 35%. All the examples presented show an apparent aging-preconsolidation effect.

The writer presented this paper as the Terzaghi Lecture at the October 1989 ASCE Meeting in New Orleans and again two days later at Louisiana

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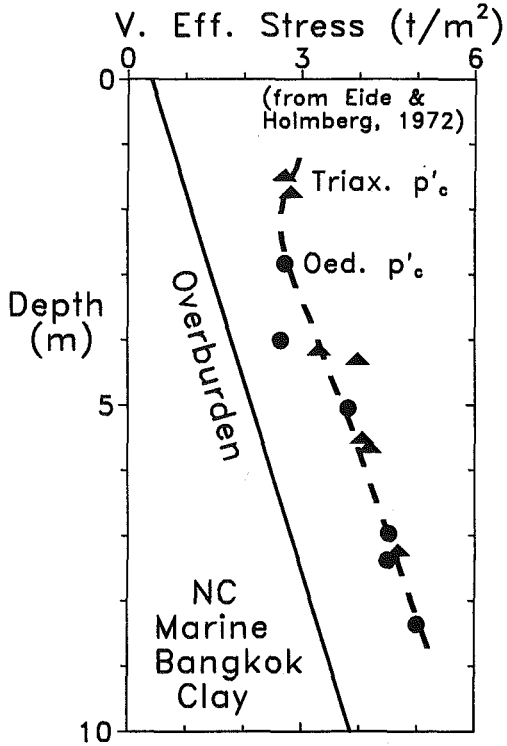


FIG. 2. Field Example of Aging Preconsolidation

State University (LSU) in Baton Rouge. He noted that the soft Mississippi River delta clays around New Orleans and Baton Rouge, typically normally consolidated from a geologic viewpoint, provide more, and local, examples of the aging-preconsolidation effect. Afterwards, L. Capozzoli (personal communication, 1989), with 33 years of practical geotechnical engineering experience in the area, reported that he has many times successfully taken advantage in practice of overconsolidation caused by secondary consolidation to support structures over these clays, with only small observed settlement. Judging by the writer's experience at one site in New Orleans, when using the Marchetti dilatometer in another supposedly normally consolidated clay, which self-boring pressuremeter and research quality sampling and laboratory tests by LSU showed had an OCR equal to 1.3–1.5 (M. Tumay, personal communication, 1989), some of the modern in situ penetration tests such as the dilatometer will detect this aging preconsolidation, while ordinary quality sampling and oedometer testing might not.

The writer now wonders, along with Leonards and perhaps many others, whether a natural clay aging at constant vertical effective stress in the field ever exists in a truly normally consolidated state? Perhaps most or all natural clays not presently undergoing active sedimentation show preconsolidation by aging. Perhaps a truly NC clay in the field exists primarily as a figment of our geotechnical imagination, conditioned by our common experience with

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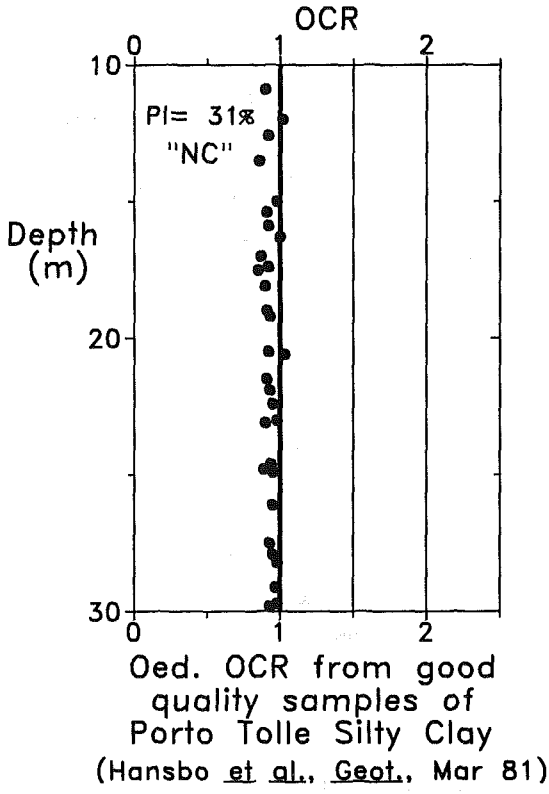


FIG. 3. *Example of High-Quality Laboratory Data Mistakenly Denoting Clay as Truly Normally Consolidated*

the laboratory testing of unaged samples.

Now consider the following demonstration, done perhaps for the first time by a test at the University of Florida on August 8 and 9, 1989, showing that the aging-preconsolidation effect can also occur in a sand. In this case, we used a clean (0.2% passing No. 200 sieve), commercial (30/65 sand, Felspar Corp., Edgar, Florida), and uniform (C_u = 1.6), air dry, medium (D_{60} = 0.45 mm, 85% between No. 20 and No. 50 sieves), quartz, SP or A3 sand. Fig. 5 shows a student in the final stages of setting up a plate-bearing load test on the surface of the sand in a special, large box built by and operated at the University of Florida, Department of Civil Engineering, and financed by Schmertmann & Crapps, Inc. As noted subsequently, demonstrating the aging-preconsolidation effect may require use of the special lateral stress control capabilities of this box.

The student in Fig. 5 appears to float over a large area of sand while setting up a vertical movement dial gauge on an 8-in.-diameter rigid plate placed on the sand surface. A bellofram with regulated air pressure applied the bearing load to the plate, with the load measured using an electric load cell reading in 1-lb increments. The system could maintain constant load on

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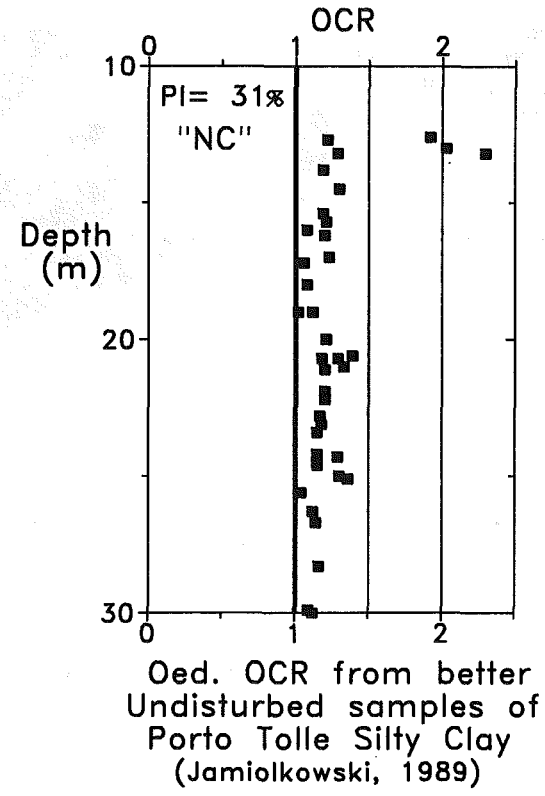


FIG. 4. Subsequent, Superior Data Showing Aging-Preconsolidation Effect in Same, Supposedly Normally Consolidated Clay

the plate within +/−2 lbf for at least several days.

Fig. 6 shows the plate settlement versus plate load for that part of the test that includes an 844-min interval of rest at an approximate constant load of 325 lb on the plate. The numbers between the points show the time interval of loading between those points in minutes. One can see that a distinct preconsolidation "bump" developed, giving an approximate 10% preconsolidation effect. This occurred in fresh sand, 0.7 m deep over a low-friction (δ = approximately 4°) base, shoveled air dry into the 2.4- × 2.4-m square box one day prior to the test. The temperature of the sand in the box in the vicinity of the plate varied from 83.2–83.4° F. Regulated air pressure acting through lateral air bags arorund the perimeter of the box first brought the whole mass of sand in the box to passive failure for a few minutes. The operator then unloaded the sand to the active state approximately a half hour before setting up and starting the plate test.

The sand, as originally placed by shoveling, had an average unit weight and relative density of approximately 96 pcf and 65%. We did not measure how these changed during the passive-active stress cycle, but believe they probably decreased. Several weeks later we repeated this test with another

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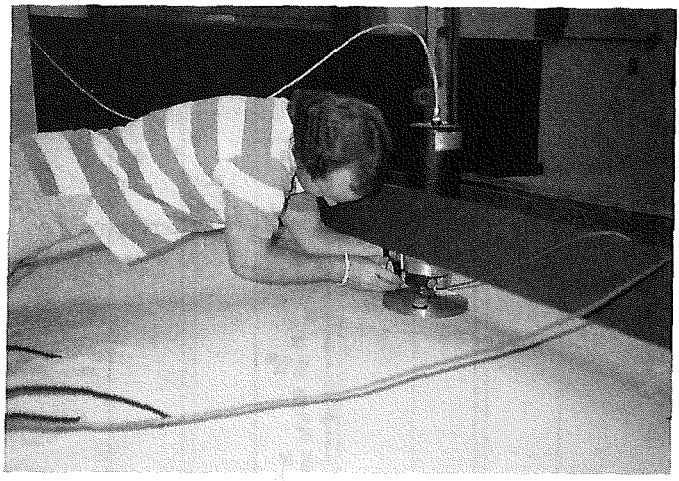


FIG. 5. Final Stages of Setting Up Fig. 6 Plate Load Test in Schmertmann & Crapps, Inc.—University of Florida Lateral Stress Sand Box

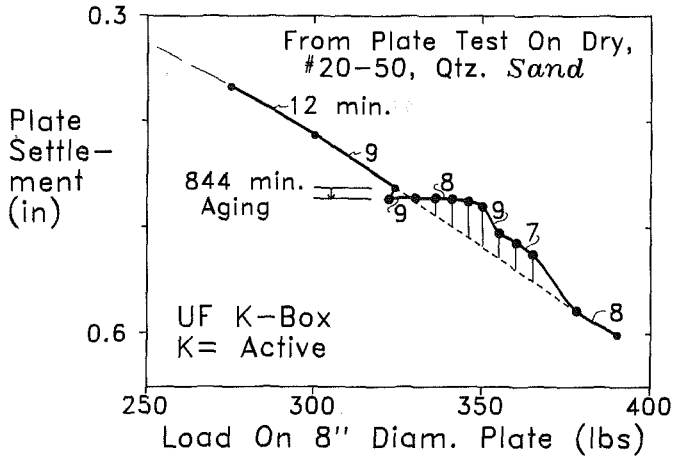


FIG. 6. Aging-Preconsolidation Effect Demonstrated for Plate Load Test in SC-UF Lateral Stress Box

fresh boxful of the same sand and got similar results. However, another aging test without the preliminary passive-active cycle failed to produce an aging bump for reasons not yet explored.

Modulus

A modulus increase with aging can also take place within compacted fills. Consider the data in Fig. 7 from a 30-ft-thick cohesive fill that supported a power plant. The writer had the opportunity to review very extensive, almost research quality, odeometer test results from samples of this fill. This figure

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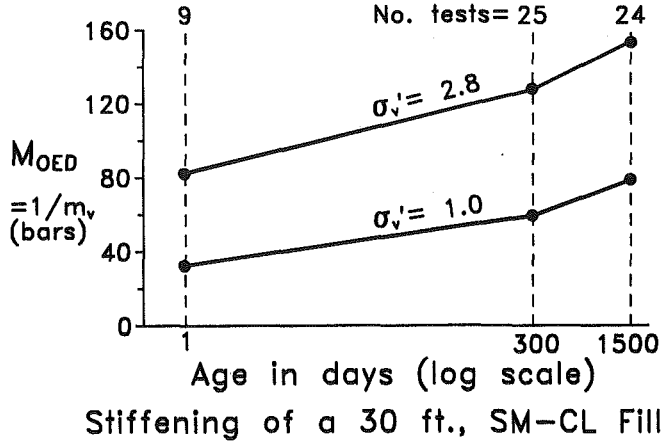


FIG. 7. Example of Aging Increasing Odeometer Modulus in Compacted, Cohesive Fill

shows an apparent and important increase in the one-dimensional compression modulus M with increasing age of the fill. The writer extracted these values by contouring through numerous data points to take out the effect of any variations in dry density. The figure shows the modulus increase versus time after compaction at two representative magnitudes of vertical effective stress. The higher exceeds all known surcharge effects at the site. Note that the modulus M increased about 100% during 1,500 days of aging.

Fig. 8 shows another example of a modulus aging effect in a cohesive fill. Seed et al. (1958) showed still another example from a compacted cohesive soil tested in the lab after only four days of aging. Their paper also suggests that cyclic loading may speed the aging process.

Most engineers working in the area of soil dynamics have known for a

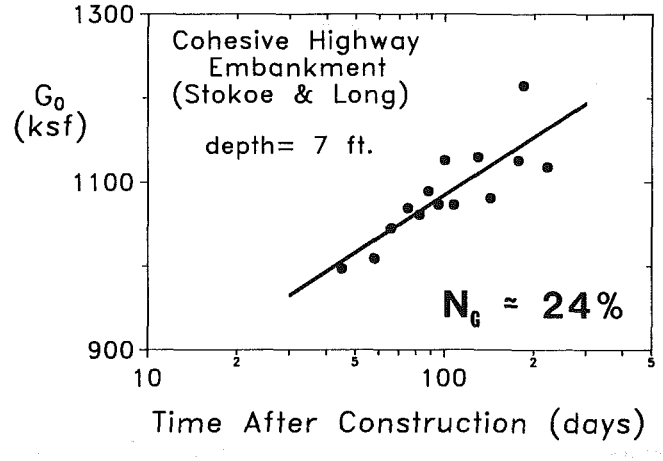


FIG. 8. Example of Small Strain Shear Modulus Increase in Field

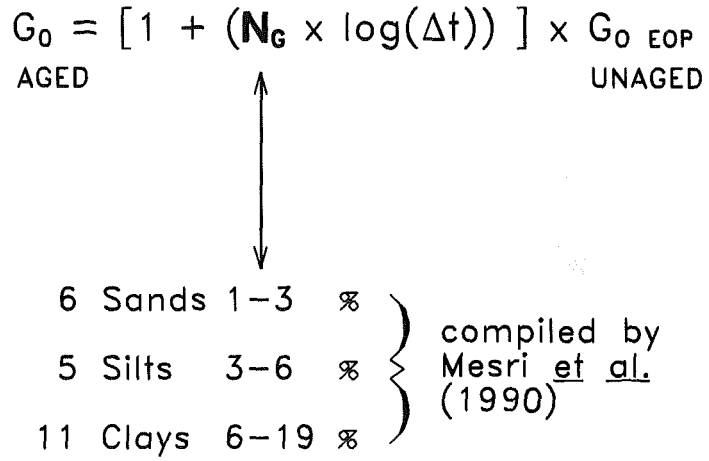


FIG. 9. Compilation of Laboratory Data Showing Small Strain Shear Modulus Increase with Aging Time in Secondary Compression

long time that the dynamic, small strain modulus increases with secondary compression aging of lab samples [see, for example, Hardin and Richart (1963), Stokoe and Richart (1973) and Anderson and Woods (1976)]. They also have established that this modulus, denoted G_o, increases with the log of secondary time as expressed by the equation in Fig. 9. The term N_G establishes the rate of increase. Mesri et al. (1990) compiled examples of this rate and kindly gave the writer a 1989 version of this paper that included this convenient compilation. Fig. 9 shows that this compilation has the N_G factor ranging from 1 to 19%, increasing as the soil becomes more fine grained. Others have compiled examples with N_G for some clays as high as 40%.

Note that even clean sands show the N_G aging effect. Also gravel: M. Jamiolkowski (personal communication, 1989) recently found an example with N_G = 2–4% in a Messina Strait sandy gravel. As a practical matter, engineers have to make major corrections to the typical lab values of G_o to predict the aged behavior in the field. They typically use a factor of 1.5–3.5 for clays (Anderson and Woods 1976).

As a demonstration that the G_o aging effect also takes place in the field, a student of K. H. Stokoe II, Long (1980), obtained the data in Fig. 8 from cross-hole tests using the same boreholes from 45–212 days after the completion of a cohesive fill. Both Stokoe and the writer know of only this example that demonstrates the aging increase of G_o in the field. These data suggest that $N_G \simeq 24\%$ for this compacted embankment, which seems similar to, and perhaps even greater than, laboratory results on similar soils.

Up to this point, the examples presented have shown only aging increases in moduli at relatively low strain—such as in odeometer tests and in tests for G_o. However, others have also noted very significant aging effects for higher strain moduli. Ladd (1964) gave the example in Table 1 of modulus increases in undrained testing of normally consolidated, sedimented samples

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TABLE 1. Normally Consolidated Vicksburg Clay

Secondary age (days) 1 > EOP (1)	E_{CIU}/σ_c'	
	FS = 3 (2)	FS = 1.5 (3)
3	175	110
10	230	135
60	300	210

Note: Source, Ladd (1964).

of Vicksburg clay. Note that the secant modulus increased approximately 70–90% during an aging time of 60 days at the strain levels corresponding to one-third and two-thirds of the undrained strength.

Fig. 10 from Daramola (1980) shows that this modulus increase occurs in saturated sands as well as clays. As with the clays, the modulus approximately doubled with a 152-day, drained, secondary aging time in triaxial cells. We will return to this example.

Strength

The cyclic load strength versus liquefaction also increases significantly with aging times, as shown by Fig. 11, first presented by Seed (1979). Based on laboratory and field data such as these, he recommended a 50–100% aging correction increase in laboratory-determined cyclic stress ratios to produce liquefaction before applying the results to the field.

Aging can also increase the strength of a clay, for example, as measured by the vane shear test. Bjerrum (1972) recognized this when he started separating the expected, normalized vane shear strength of normally consolidated clays into geologically young and geologically aged clays. Fig. 12 suggests an approximate 100% increase in strength at constant plasticity in-

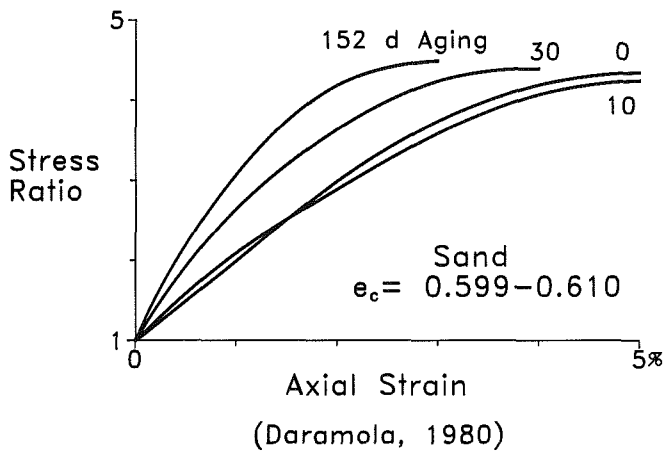


FIG. 10. Laboratory Example of Modulus of Sand Increasing with Secondary Compression Aging

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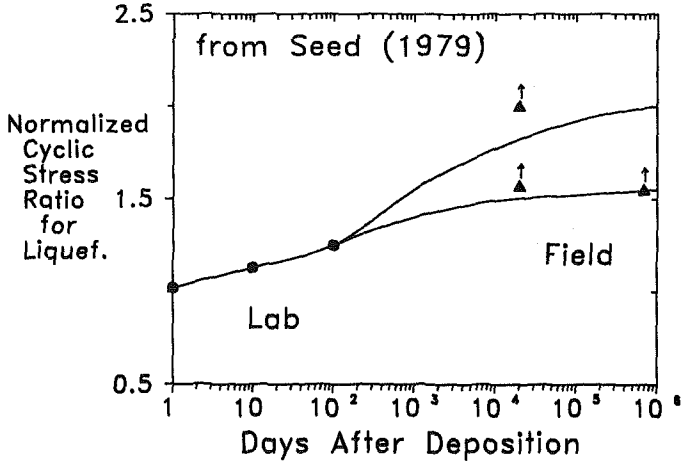


FIG. 11. Laboratory and Field Data Illustrating Increased Resistance to Liquefaction with Aging

dex (PI) due to the geologic aging. However, the strength-increase effect can also occur in clays over engineering aging time in the laboratory. The example in Fig. 13 from work by Yasuhara and Ue (1983) shows an approximate 50% increase in undrained, direct shear strength with secondary aging times varying from 30 min to 30 days. Hanzawa (1983) presented similar data.

As evidence that significant aging increase can occur over engineering times, and can occur in the field, consider the data in Table 2 collected by

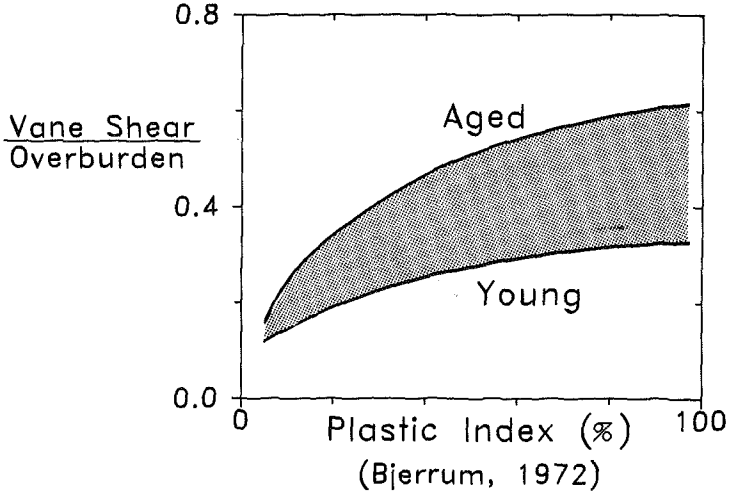


FIG. 12. Geologic Aging Increases Normalized Field Vane Shear Strength in Near-Normally Consolidated Clays

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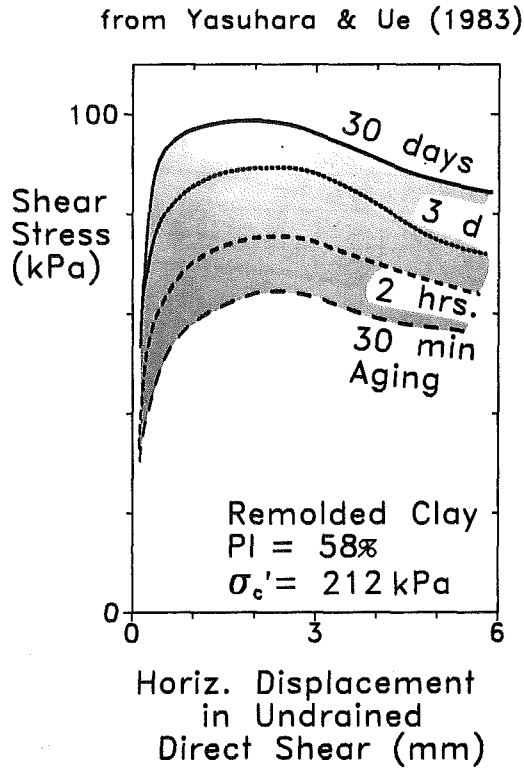


FIG. 13. Example of Laboratory Increase in Undrained Strength of Clay due to Aging in Secondary Compression

Osterman (1960), a former director of the Swedish Geotechnical Institute. The site involved a surcharge on a 5-m-thick clay layer, which compressed about 15% under the surcharge. After the end of primary, determined by pore-pressure and settlement measurements, the vane shear strength increased an average of more than 50% during an aging time of about 12 years.

The strength of sands, as measured by penetration tests, can also increase significantly with engineering-aging times. Consider the example in Fig. 14,

TABLE 2. Field Vane Tests (5-m Clay Layer, Sweden)

Date (1)	s_u (t/m^2) (2)	Note (3)
October 1945	1.15	Start surcharge
November 1947	1.9	After EOP
November 1956	2.1	
October 1959	2.9	50 + % Aging

Note: Source, Osterman (1960).

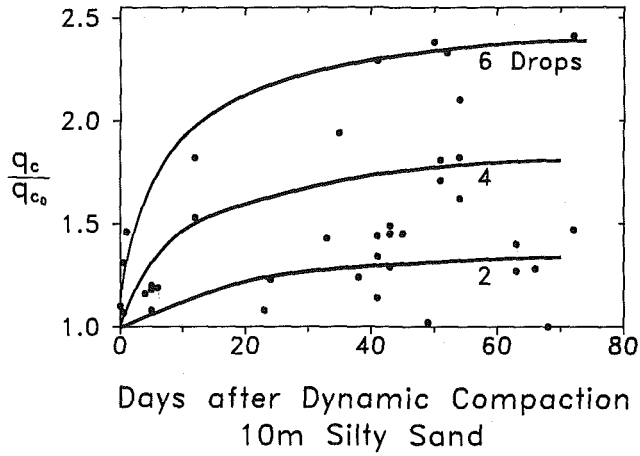


FIG. 14. Normalized Increase in Static-Cone Bearing Capacity after Dynamic Compaction

showing comparative static-cone test bearing capacity q_c after the dynamic compaction of a 10-m-thick silty sand layer in Jacksonville, Florida. The y-axis presents the ratio of q_c to q_{c0} immediately after the dynamic compaction. The x-axis gives the time after that compaction. The writer (Schmertmann 1987) previously used this figure in a Discussion of Mitchell and Solymar (1984), who presented similar data, and also in Schmertmann et al. (1986). Lukas (1986) also used this figure along with other data to demonstrate the general observation of modulus and strength-increase effects after dynamic compaction. See these references for more details and also alternative or supplementary thoughts about such aging. For example, increasing horizontal stresses may have contributed to the aging q_c increase. The engineers involved with the Fig. 14 data carefully monitored a test section with literally hundreds of electric static-cone soundings made before, during, and after repeated drops of the dynamic compaction weight. Each data point shown has a specific number of drops associated with it, which the interested reader can find in the aforementioned references. The writer estimated the average curves shown through the numerous data points.

The static-cone bearing capacity increased significantly for approximately 60 days, and increased with the number of successive drops per location used for the dynamic compaction. This compaction involved dropping a 33-ton weight 105 feet. The drops produce an indentation in the ground surface, known as a print. Presumably the disaggregation of the natural structure of the sand increased with the number of drops per print, and so did the relative aging improvement. It appears that breaking down structure may promote subsequent aging.

As a practical matter, the engineers involved used the q_c values as an acceptance criteria for the results from the dynamic compaction. They permitted the use of the curves in this figure to correct the q_c data obtained shortly after dynamic compaction to the values expected 60 days after compaction. They then used the 60-day values for acceptance or rejection.

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TABLE 3. Aging Effects in Hydraulically Placed, Quartz, River Sand Fills

Site (1)	Saturation (2)	Test (3)	Time after Placement (days) 10–20 (4)	20–40 (5)	40–50 (6)	60 (7)	100–140 (8)
A	100%	N[a]	2.1	2.2	3.1	3.3	4.4
			Time after Placement (years)				
			1.5	2.5	3.5	4.5	5.5
B	partial	γ_d (t/m^3)	1.54	1.54	1.56	1.54	1.57
	w = 4%	N	4.9		7.1		11.6
		E[b] (ksc)	121		173		380
		s_v[c] (ksc)	0.32		0.79		0.88

[a]N = Number of 60-kgf × 0.80-m hammer blows to drive 74-mm-diameter 60° cone 10 cm.

[b]E = Secant elastic modulus from settlement of 0.50-m^2 plate at 2-m depth loaded between 0 and 3.0 ksc.

[c]s_v = 55-mm ϕ, 110-mm-high vane shear strength at depth of 2 m.

Note: Source, Denisov et al. (1963).

Engineering aging also can increase the dynamic penetration resistance in sands, as shown in Table 3 taken from Denisov et al. (1963). Their dynamic cone resistances more than doubled in both saturated and moist hydraulic fills during the aging times investigated. Modulus and vane shear strength also more than doubled. But dry density increased only slightly because of secondary creep. Russian literature also shows the same magnitude of increases due to aging for SPT-type blow-count values. These Russian authors believed that these aging improvement effects resulted from a cementation bonding cohesion that developed between the sand particles during the engineering times involved. However, as the writer has found with other opinions about cohesion and aging, closer examination shows that the authors speculate about, but offer no proof of, the cohesion increase. This paper will return to investigate the cohesion versus friction aspects of soil aging.

The type of data shown in Figs. 10 and 11 led Skempton (1986) to propose that correlations of SPT N-values with relative density required correction for the effects of aging. DeCourt (1989) noted this and observed that the Bazaraa (1967) correlations, based on field data, if adjusted by the inverse of an aging factor, with this inverse equal to 0.4–0.6, would approximately match the SPT versus relative density correlations previously obtained by Gibbs and Holtz (1957) and Marcuson and Bieganouski (1977). Fig. 15 shows the comparisons, using their suggested correlation equations, from these three sets of data at two overburden pressure levels, namely 10 and 40 psi. The heavy dashed line shows the Bazaraa correlation adjusted by the DeCourt inverse 0.5 aging factor. One can see that the three correlations now approximately agree with each other. The writer believes that this explanation of aging helps greatly to explain a big mystery—namely, why the Bazaraa correlations did not fit well with the others.

The apparent success of this aging explanation reinforces the idea of the possible need to correct for aging effects when using the test results from

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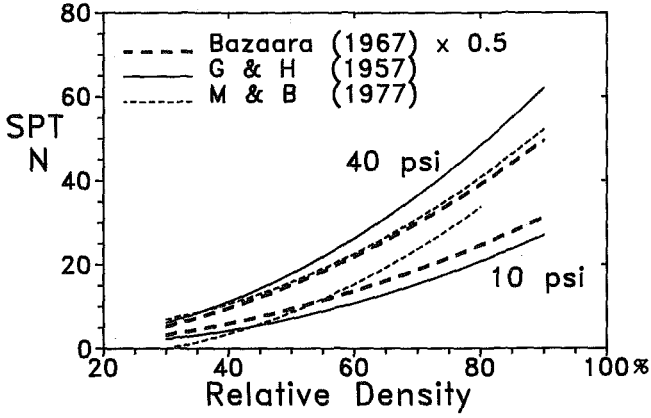


FIG. 15. Well-Known Field and Lab Chamber Correlations between N and Relative Density Match Better after Aging Correction

freshly prepared samples in large chambers versus applying such results to aged soil in the field.

Pile Capacity

Another very important aging-strength-gain effect involves the capacity of driven piles. Engineers know well that driven piles often exhibit significant setup or freeze effects. However, they may not know that the setup may result from not only a pore-pressure dissipation effect, but one that also involves the aging of the newly disrupted soils surrounding the piles—an aging that occurs at constant effective stress. Fig. 16 (Karlsrud and Haugen 1986)

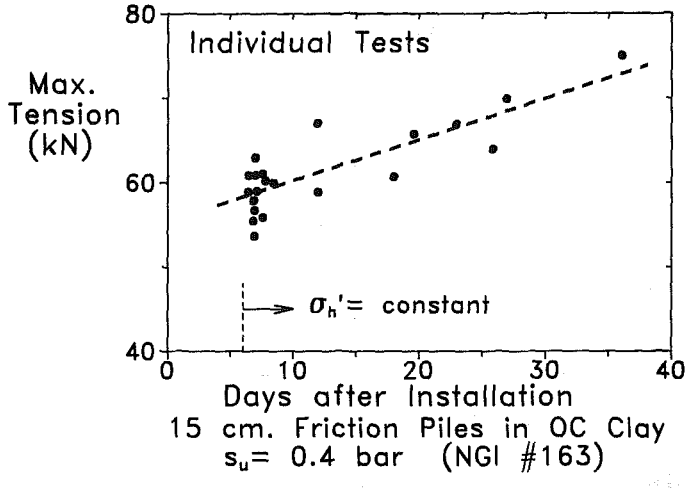


FIG. 16. Research Pile Tension Tests Show Pile Capacity Gain during Aging under Constant Effective Stress Conditions

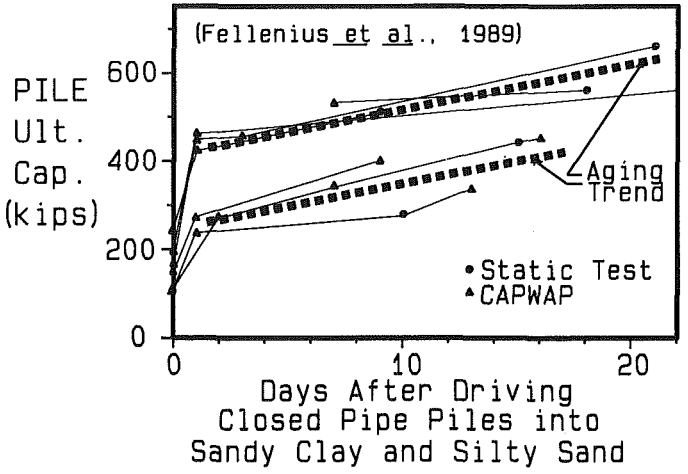


FIG. 17. Project Compression Pile Load Tests Show Capacity Gain with Time after Probable Full Excess Pore-Pressure Dissipation

shows an example from research performed by the Norwegian Geotechnical Institute. Each of the points involves a separate, driven pile tested in tension in an overconsolidated clay. Their total and pore-pressure instrumentation showed that the horizontal soil-pile interface effective stresses became constant after about six days following pile driving. The trend of the points clearly shows an increase in tension capacity at constant horizontal effective stress, which the authors attributed to aging. Their data show an approximate 30% increase in capacity after 30 days of aging.

Tavenas and Audy (1972) also reported an aging increase in the static load test capacities of 45 piles driven into sand with a permeability $k = 10^{-2}$ cm/s. Because excess pore pressures dissipated in a few hours but the piles gained an average 70% capacity in two to three weeks, they attributed the increase to changes occurring in the sand structure around the piles. Fig. 17 reports a similar experience and shows the results from a very recent series of production compression load tests and CAPWAP dynamic analyses. These data by Fellenius et al. (1989) appear to show a pile capacity increase with time similar to that in Fig. 16, with a 50% increase in 20 days. The sandy soils involved suggest a rapid pore-pressure dissipation, but the authors did not document pore-pressure effects.

Finally, Kehoe (1989) studied the pile-freeze effects at two bridge sites along the Gulf Coast in the panhandle of Florida. The soils involved varied from sands and clays to mixtures, often changing over short distances. The author used dynamic load test results, including CAPWAP analyses, to determine pile capacity and load distribution. The freeze effect increased capacities by an average of 58% and 200% in 11 days at the two sites, with half of this increase occurring in the first 100 min after driving. He speculated that this first half related to pore-pressure dissipation effects and the remainder to other aging effects. Perhaps most interesting, he determined that almost all the freeze increase occurred as an increase in side friction capacity. At the end of driving, the pile end bearing typically accounted for

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TABLE 4. Remolded Boston Blue Clay

Aging after EOP ≃ 200 min (1)	Number tests (2)	$\mathbf{A} = \Delta u/\Delta(\sigma_1 - \sigma_3)$ (3)
1,200 min	3	1.4
6,600 min	3	1.0

more than 50% of the then-total pile capacity, but increased only a little with time. Aging caused the piles to become primarily friction piles. It appears that the more the pile disrupts the natural soil structure, in this case the soil along the sides of the pile versus the soil below its tip, the greater the aging effects.

Pore Pressure

The writer now presents a theory, supported by subsequent data, that he has not seen previously in the literature. It concerns possible pore-pressure reductions caused by aging effects.

Consider the Skempton (1954) pore-pressure equation [(1)] shown below, with the well-known **B** and **A** coefficients

$$\Delta u = \mathbf{B}[\Delta\sigma_3 + \mathbf{A}(\Delta\sigma_1 - \Delta\sigma_3)] \quad (1)$$

$$\mathbf{A}, \mathbf{B} \propto \frac{C_s}{C_s + nC_{pf}} \quad (2)$$

$$\therefore \Delta u \downarrow \text{ when } C_s \downarrow \text{ due to aging} \quad (3)$$

The derivations by Skempton show both coefficients proportional to the compressibility ratio given by (2); namely, the compressibility of the soil skeleton C_s divided by the compressibility of the soil skeleton plus the porosity n times the compressibility of the pore fluid C_{pf}. When the pore fluid has a negligible compressibility compared with the soil skeleton, as in a 100% water-saturated soil, the ratio in this formula equals 1.0 for all C_s.

However, natural and artificial (such as compacted) soils may often contain a small amount of gas and, perhaps, have degrees of saturation somewhere between 95 and 100%. The compressibility of the pore fluid then becomes quite significant and of the same order of magnitude as the compressibility of the soil skeleton. We know from the previous examples that the soil skeleton stiffens and reduces compressibility with aging. With C_{pf} not negligible, (3) shows that if C_s decreases because of aging, then Δu would decrease because both the **A** and **B** pore-pressure coefficients would decrease while the total stresses remain constant. Eqs. (1), (2), and (3) form the basis for a reason for a pore-pressure dissipation not related to water flow, but instead related to aging effects, and which occurs in addition to that due to water flow. Consider the support given by the following examples.

The data in Table 4, extracted from Bea (1960), show that aging may decrease the pore-pressure coefficients, as postulated. A nearly 100%, but not fully 100%, saturated remolded (machine-extruded) Boston blue clay was aged for 1,200 min after a primary of 200 min. It then had a three-test average **A** value of 1.4 during the undrained loading to near-failure prior to

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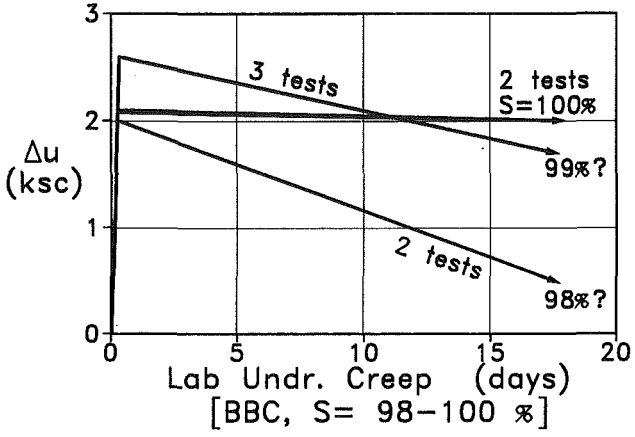


FIG. 18. Laboratory Example of Pore-Pressure Dissipation with Time under Undrained Creep Conditions

a creep test at constant deviator stress loading. The same soil, aged for 6,300 min, had an **A** value of only 1.0 for the same deviator loading, also from the average of three tests. In this case, it appeared that the additional aging from 1,200 to 6,300 min reduced the **A** pore-pressure coefficient by about 30%. Very careful testing gave these samples calculated degrees of saturation S between 97.6 and 100.4%. The small amounts of gas may have increased C_{pf} enough to produce the aforementioned aging decrease in **A**. Bea did not obtain the data to check **B** in these tests.

Similar remolded samples, when subjected to undrained creep at constant dead loading, showed a mysterious pore-pressure behavior we could not explain at the time. Fig. 18 shows this behavior, somewhat simplified. See Bea (1960) for the details. Two of the seven creep tests showed very little pore-pressure dissipation, as one might ordinarily expect. One of these two consisted of an undisturbed sample of Boston blue clay, which we knew had a degree of saturation of 100%. The other six tests had an after-consolidation range of calculated S = 97.1–100.5%. Three of the other tests showed a trend of more or less steady decrease in pore pressure, amounting to about a 25% decrease in 17 days. Another two tests showed a decrease trend of about 75% in 17 days. The writer now speculates that perhaps the 25% dissipation tests had a degree of saturation of about 99% and the 75% dissipation tests of about 98%. The increased compressibility of the pore fluid with reduced saturation would allow the decreasing compressibility of the soil skeleton with aging to have a progressively greater effect in reducing pore pressures by transfer of load from the pore water to the stiffening skeleton.

One might also explain the behavior described in Fig. 18 by speculating that the small gas content in the pore fluid slowly dissolved into the fluid, causing the pore pressures to drop during the undrained creep. A detailed examination of the test records does show an increase in the calculated S values before and after the creep phase of these tests. The writer calculated, from Bea's data, an average of 98.0% before and 98.9% after, for the five

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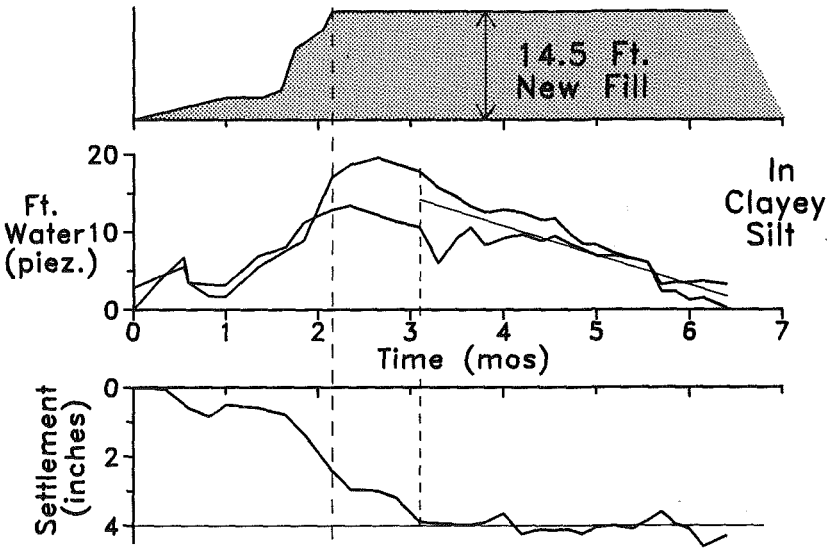


FIG. 19. Field Example Showing Pore-Pressure Dissipation with Time without Matching Consolidation Settlement

tests in Fig. 18, with decreasing pore pressure. However, the standard deviation of the error in the calculated degree of saturation probably equaled about 0.7%, introducing considerable uncertainty about the calculated 0.9% average increase in S.

Fig. 19 shows data from a possible field example of pore-pressure dissipation resulting from soil aging. In this case the Pennsylvania Department of Transportation placed 14.5 ft (4.4 m) of new highway embankment fill over a natural layer of sensitive, clayey silt. As shown in the figure, two vibrating wire piezometers at two elevations in the clayey silt showed the expected rise in pore pressure, followed by a gradual decrease during a period of some four months after completion of the fill. Note that the 14.5 ft (4.4 m) of new fill produced about 15–20 ft (4.6–6.1 m) of pressure head, which seems reasonable. But the settlement behavior, as shown at the bottom of Fig. 19, seemed to be quite different than one would normally expect from the pore-pressure decrease.

After an initial settlement, the settlement during the final three months appeared to remain essentially constant despite the continuing pore-pressure decrease of more than 10 ft (3 m) of head during that interval. Also, in this case the clayey silt supported a perched water table and it may not have been completely saturated. The writer speculates that the clayey silt layer aged and stiffened in secondary compression after the completion of the 14.5 ft (4.4 m) of new fill and that the pore pressures dissipated primarily as a result of this aging stiffening and in only a relatively minor way due to water flow. That would explain the dissipation without the settlement.

Summary

The writer has now shown many examples of the effects of aging, both in the lab and in the field, in clays and in sands. Table 5 summarizes these

TABLE 5. Summary of Examples

Effect (1)	Approximate % improvement (2)	Laboratory (3)	Field (4)
p_c, M	40–100	X	X
Liquefaction	50–100	X	X
G_0	50–200	X	X
E	50–100	X	—
s_u	50–100	X	X
q_c, N	30–140	—	X
Pile capacity	25–60+	—	(1 month)
Δu, A	40	X	X

Note: $\Sigma \simeq 50$–100%.

examples, which include the preconsolidation effect, the one-dimensional compression modulus, liquefaction, low- and high-strain moduli, undrained shear strength, CPT and SPT resistance in sands, pile capacity, and possible pore-pressure-decrease effects. The examples, from the lab and the field, natural and artificial soils, sands and clays, had a range of aging improvement of typically 50–100%. As a number to carry around when one thinks about aging effects in mineral soils, consider using 50–100% improvement.

Examples With No Aging Improvement

The introduction noted that many other phenomena could occur during aging times and mask or otherwise overwhelm only pure aging effects. The writer noted, in conjunction with the discussion of Fig. 3, that he also searched for cases where the aging effects described in this paper did not appear to happen. A comparison between the OCR data in Figs. 3 and 4 indicate that improved testing sometimes will show a previously unnoticed aging effect. However, it also remains possible that the beneficial effects of aging described herein may not always occur. The writer also noted this in conjunction with the data in Fig. 6, where the first attempt at a confirmation test, without the passive-active precycling, did not show an aging effect, while the next confirmation test with such a cycle showed the effect again.

Consider another example of nonoccurrence: Stokoe and his students (Stokoe and Richart 1973) attempted to provide the writer with another data point to add to Fig. 8 data from a Texas highway embankment, some 12 years after obtaining the data in Fig. 8. In the absence of the original boreholes used to obtain G_o from cross-hole testing, they attempted to get G_o at the 7-ft depth by interpreting surface Raleigh waves. The results proved disappointing because they showed a slight decrease in G_o during these 12 years. It remains unclear whether this occurred because of the different test method used, or because of changes in surrounding conditions (an adjacent lake level rose), or because a continued aging increase in G_o did not occur.

The writer also has encountered a few other examples that showed no aging effect. A highway embankment near Allentown, Pennsylvania, nearly failed during construction because of high excess pore pressures. Engineers obtained settlement and pore-pressure dissipation data over a period of eight and a half months after the last construction lift. Although not of research quality and volume, the available data showed neither a decrease in com-

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pressibility with time that was not attributable to pore-pressure dissipation, nor the suggested possible additional pore-pressure dissipation related to aging.

In another example involving Federal Highway Administration research on pile capacity in sands [see "Pile Group" 1987)], the report attributed time effects to pore-pressure behavior, specifically noting that they found no other aging effects. Still another example of nonoccurrence involved a compaction grouting test section to upgrade the liquefaction resistance of a dam in South Carolina ("Embankment" 1989). SPT measurements did not show an increase in N values in a silty sand between compaction points after aging for three years.

While it remains possible that more complete or better quality data in these cases might have been shown an aging effect on the types described herein, one also must admit the possibility that such aging effects may not always occur, for unknown reasons at this time.

Cohesion or Friction?

Let's now investigate the question, do the aging modulus and strength increases come from mobilizing additional soil friction, or from mobilizing additional soil cohesion, or from both? To help answer this question, the writer will show the results from a special type of triaxial test, denoted the "IDS test," he developed at the University of Florida for his doctoral work at Northwestern University. However, before showing results let's briefly review the IDS test.

Review of IDS Test

Schmertmann and Osterberg (1961) first described the IDS test and how it evolved from a desire to determine the Hvorslev components of strength as functions of strain. Schmertmann (1964) showed how the IDS-determined components measured more generalized versions of the Hvorslev components. The writer described and summarized much of his work with this test in Schmertmann (1976) and again presented a more limited description and summary in Schmertmann (1981).

The IDS test determines a basic soil frictional strength, to which the writer has given the symbol tan ϕ'', or just ϕ'', in this paper. Just as ϕ' represents a more basic strength parameter than ϕ because it takes out the effect of variable pore pressure, ϕ'' represents a still more basic strength parameter because it in addition takes out the effect of variable soil structure. c'' denotes that part of the mobilized shear strength not assigned to σ' tan ϕ''; c'' relates to c' and to c much as ϕ'' relates to ϕ', and to ϕ. However, as discussed subsequently, c'' includes envelope curvature effects as well as any basic cohesive bonding or cementation. The IDS test determines both ϕ'' and c'' as functions of strain. The following four schematic figures help describe in more detail, and in four steps, the philosophy and procedures for the IDS test:

First consider step one in Fig. 20, which shows the ordinary virgin consolidation loading and rebound unloading for compressible soil. Suppose one wanted to test two samples of this soil at two levels of effective stress but with the same structure. The rebound points, denoted R1 and R2, seem like the obvious place to choose the two samples because of the very small and,

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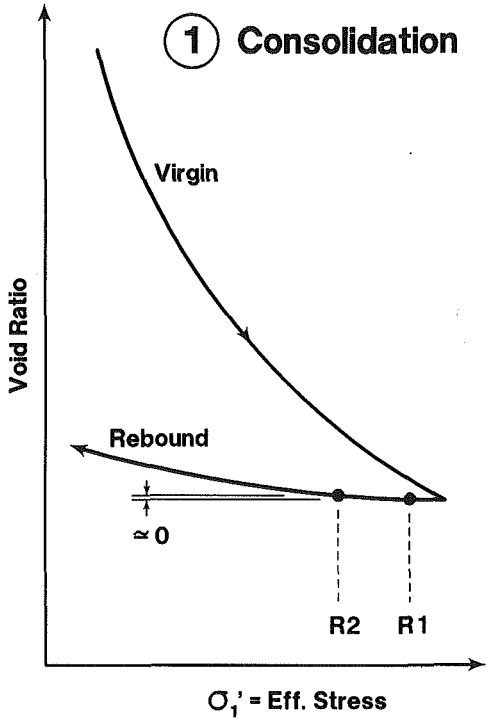


FIG. 20. Selection of Two Points in Soil's Stress History for Nearly Constant Structure at Different Effective Stress

we assume, negligible differences in void ratio, fabric, and other measures of structure, between these two points.

Let us now, in step two, subject each of these samples to a triaxial compression test in which we keep the major principle effective stress constant at the preselected σ'_{R1} and σ'_{R2} values throughout the strain range of the test. This procedure helps ensure that the stresses stay below the yield locus for the soil, thus minimizing the soil-structure changes that occur with the compressive strain. The IDS-test operator can maintain the constant σ'_{R1} or σ'_{R2} condition by directly controlling the sample pore pressures so that they always equal the imposed deviator stress. Fig. 21 shows the results in the form of two stress-strain curves.

The test at the higher effective stress σ'_{R1} would have a greater strength at all strains because of a higher mobilized friction resistance at the higher effective stress. At any strain, such as the ϵ_x shown, the two samples would have the two mobilized shear resistances shown, namely τ_{x1} and τ_{x2}. The two samples have essentially the same initial void ratio and structure at the start of compression, and the investigator brought them to the same strain ϵ_x. Therefore, they must have essentially the same void ratio and structure at the two mobilized shear stress points shown. Then they also would have essentially the same cohesion or mobilized bond strength. The difference in

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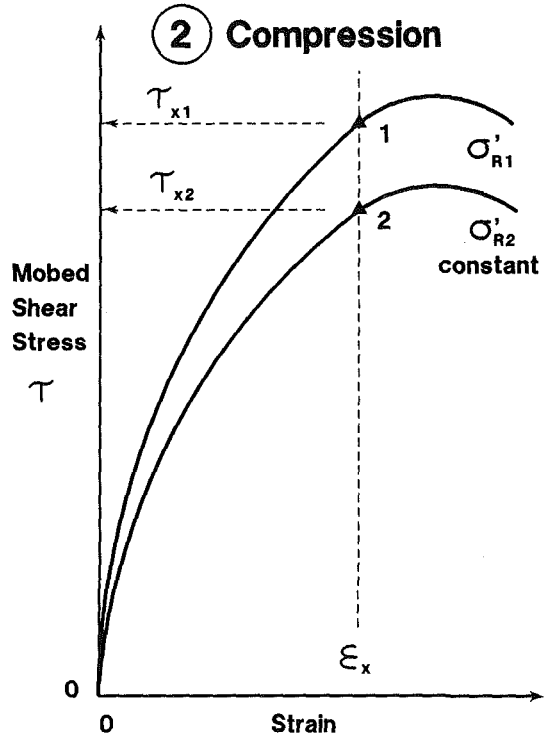


FIG. 21. Comparative Compression Tests having Same Soil Structure at Every Magnitude of Strain

mobilize shear resistance comes only from the friction component.

Schmertmann and Osterberg (1961) showed the evolution of this test from more than two comparison stress-strain curves to the two shown in Fig. 21. They then showed further that a single specimen also can generate such comparative curves if one suitably varies the controlled pore pressure to alternately produce σ'_{R1} and σ'_{R2} at intervals during the strain. The writer devoted an entire paper to demonstrating this in a variety of soils (Schmertmann 1962). The use of a single specimen to generate the two curves in Fig. 21 had two important benefits—it provided better assurance of the same soil structure at points one and two, and it permitted the testing of undisturbed samples and other soils wherein it might prove difficult or impossible to obtain pairs of initially identical samples.

Continuing with step three, each of points one and two have a Mohr's circle representation, as shown in Fig. 22. The common tangent, shown as 1-2 in Fig. 22, has a slope that defines the friction sensitivity of that soil structure, as represented by its slope at the angle ϕ''. As shown, a linear extrapolation of that slope to the effective stress origin defines the cohesion intercept component c''.

We now have the values of ϕ'' and c'' at ϵ_x, and for the final step four we can plot them as shown by the heavy points at the strain ϵ_x, shown in Fig.

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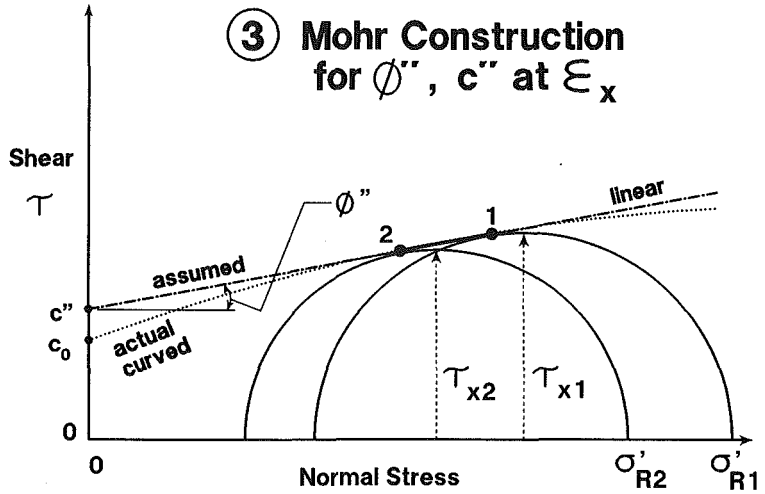


FIG. 22. Mohr's Circle Construction Defining ϕ'' and c'' at Each Strain in Tests Shown in Fig. 21

23. However, mobilized shear changes with strain, as shown in Fig. 21, and so does soil structure. We therefore should expect either or both ϕ'' and c'' to change with strain. One can use the same Fig. 21–Fig. 22 procedure illustrated for ϵ_x at all other values of ϵ from which we have enough data to define the two curves shown in Fig. 21. Doing this at strains both less than and greater than ϵ_x produces results such as those in Fig. 23. Connecting the points to provide smooth curves for each component—and remembering

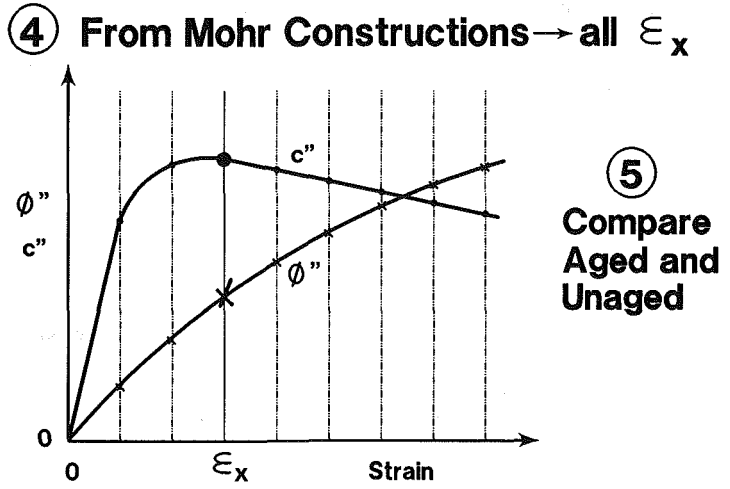


FIG. 23. ϕ'' and c'' Plotted as Functions of Strain, Based on Fig. 21 Tests and Fig. 22 Constructions

that at zero strain we have zero shear stress mobilized, and therefore both c'' and ϕ'' equal 0—allows one to produce the variation of ϕ'' and c'' with strain similar to Fig. 23. As a separate, comparative step five, one could now produce component mobilization curves, such as Fig. 23, for both aged and unaged samples of the same soil and compare them. Such comparisons might show at a glance which component has increased as a result of the aging. The writer will subsequently show such comparisons.

Unfortunately, one major complication first requires further explanation. If the Mohr envelope in Fig. 22 has curvature, such as illustrated by the light dashed line 1-2-c_o, then the actual mobilized bond shear strength for the ϵ_x structure equals the intercept, denoted c_o, a value equal to or less than c'' depending on the curvature. Without curvature, c'' equals c_o. Thus, tan ϕ'', or just ϕ'', denotes a basic parameter applicable over the 1-2 normal stress range tested, but c'' includes both the basic c_o and the effect of envelope curvature ($c'' - c_o$). If mobilized basic friction increased as a result of aging one would see an increase in ϕ''. If mobilized basic c_o increased, then c'' also would increase if envelope curvature remained constant. However, it remains possible that curvature might change, or that c_o might increase but remain undetected by a c'' increase because of experimental errors when c_o represents only a small fraction of c''. Based on the writer's experience, it seems unlikely that c_o can increase significantly without also increasing c''.

The skeptical reader might ask for some proof that the IDS test would detect an increase in basic bonding or cementation cohesion by an increase in c''. Figs. 24 and 25, extracted from Ho (1971), present such an example of c'' increasing and ϕ'' remaining the same. Ho prepared samples of glass spheres the size fine sand with and without a hydrocal (gypsum) cementing agent and tested them in compression using the constant-σ_3' version of the IDS test. Fig. 24 shows that the cementation produced a large increase in modulus and strength. Fig. 25 shows that the increase came entirely from an increase in c''. Tan ϕ'' even decreased slightly, probably because of a higher void ratio and therefore less particle interference and friction at a given strain in the samples with hydrocal. This example supports the writer's previously stated opinion that if fundamental cohesion c_o increased it would increase c'' as obtained in an IDS test. If c'' does not increase, it likely means that c_o also has not increased significantly.

Until this point, the writer has not mentioned Karl Terzaghi. However, he had the good fortune to meet Terzaghi on several occasions, and once spent several hours with him at the University of Florida in 1959, discussing the IDS-test research in progress then. Fig. 26 shows a photograph of the two of us during this discussion, taken by Frank Marshall, then a graduate student. Terzaghi expressed much interest in the research techniques to study the basic friction and bonding components of mobilized soil shear as functions of strain, and the results therefrom. He also suggested the work later described in Schmertmann (1962). Altogether, the writer and his students have performed approximately 500 research IDS tests of various types. Now, we will consider the results from some tests before and after various aging effects.

Comparing Aged and Unaged Soils

Let's first look at the results of IDS tests using nearly identical machine-extruded samples of Boston blue clay, with one group of samples aged one

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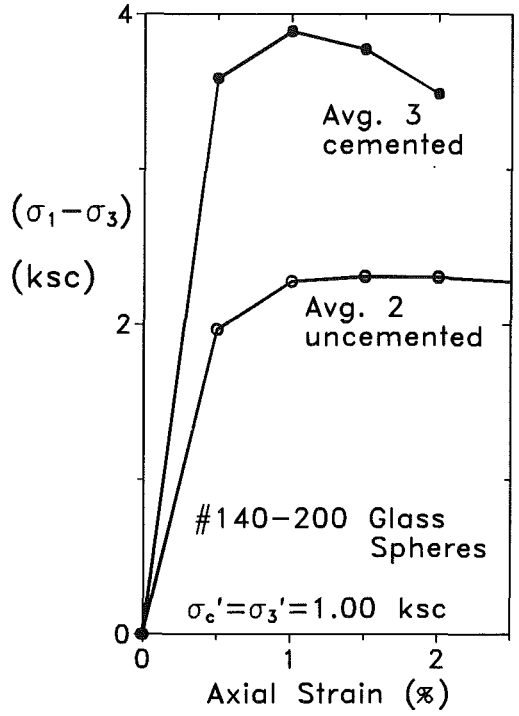


FIG. 24. Comparative IDS Compression Test Results using Glass Beads with and without Hydrocal Cementation

day and another five days. Bea (1960) has the details. As with the data shown previously, Fig. 27 shows that those aged five days have a higher compressive strength because of aging. Now consider the question, does that increased strength result from increased friction or increased cohesion? All test conditions remained the same except for the aging time.

Fig. 28 shows the IDS results for the c'' component separation as a function of strain. It shows that, if anything, the aged sample has a lower c'' than the unaged one. It does not appear to have gained cohesive bond strength. Now let us look at the ϕ'' component mobilization shown in Fig. 29. The reader can see that the strain mobilization of basic friction, ϕ'', has increased dramatically. These results suggest that the increased stiffness and strength from aging have resulted entirely from an increased friction capability that somehow developed within the Boston blue clay during the four additional days of aging.

The writer also performed a series of IDS tests on duplicate samples of extruded kaolinite, with all test conditions the same except for different times in secondary aging—ranging from 1.5 hr to 5 weeks. Fig. 30 shows the IDS component separations. It again shows that the cohesive component c'' remains either the same or, perhaps, even decreases as a result of the aging. Again, in sharp contrast, the ϕ'' friction mobilization increases dramatically with increased aging time, with the effects most dramatic at low strains.

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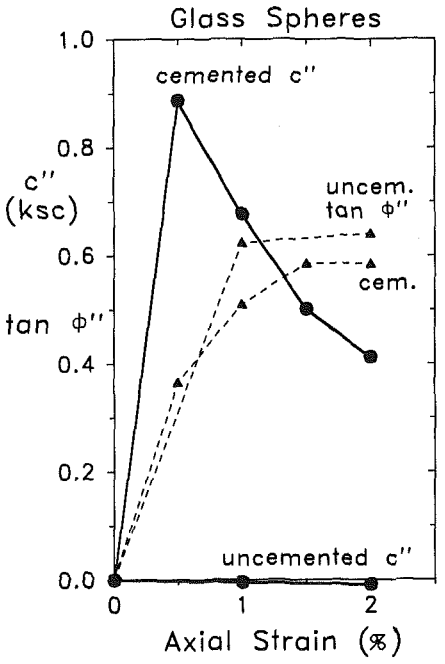


FIG. 25. ϕ'' and c'' Component Separation for Tests in Fig. 24

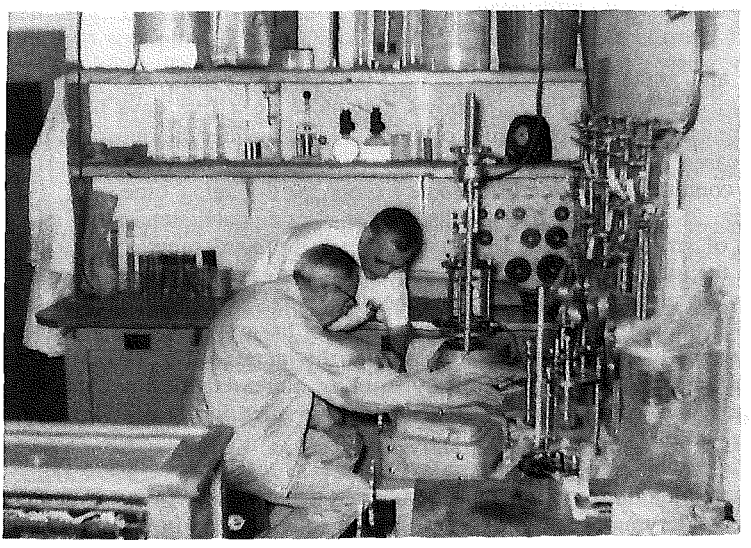


FIG. 26. Terzaghi and Schmertmann in 1959 Discussing IDS Research at University of Florida

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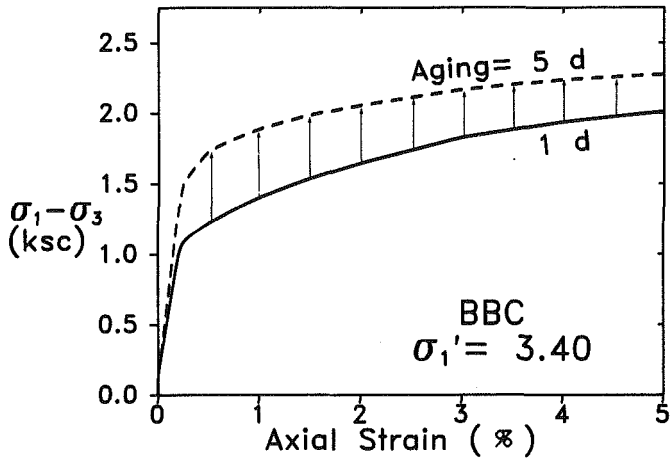


FIG. 27. Average Laboratory Compression Tests after One Day and Five Days of Secondary Compression Aging

One can also produce an aging effect by reducing the rate of strain. Fig. 31 shows the results from a comparative series of compression tests, wherein all the conditions remained the same except that the rate of strain varied by a factor of more than 5,000, with all tests fully drained. The figure shows that after about 1/2% strain the mobilized shear resistance increased with decreased rate of strain—perhaps a surprise because with viscous behavior we usually think of strength decreasing with reducing rate of strain. Nevertheless, this form of aging also increased strength.

Now consider the comparative component separations in Fig. 32 obtained from the tests in Fig. 31. They once again show that the cohesive component c'' either decreased or remained about the same, while the frictional com-

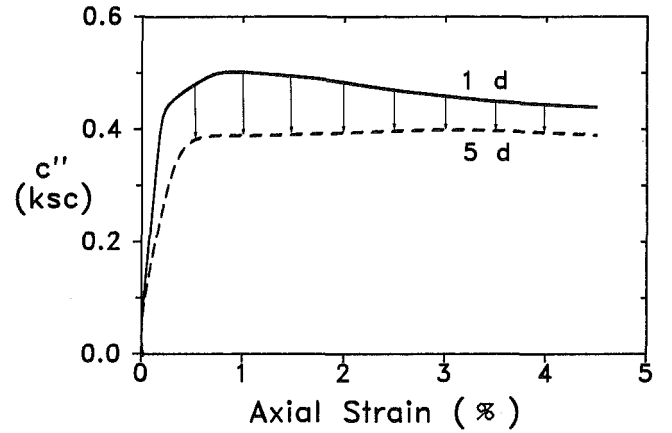


FIG. 28. Comparative c'' Components from Fig. 27 Tests

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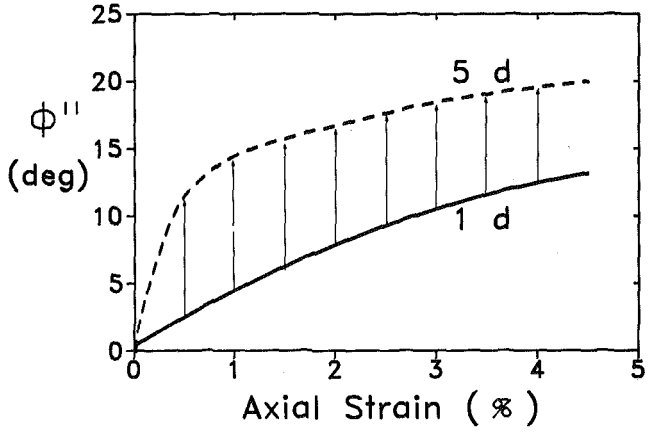


FIG. 29. Comparative ϕ'' Components from Fig. 27 Tests

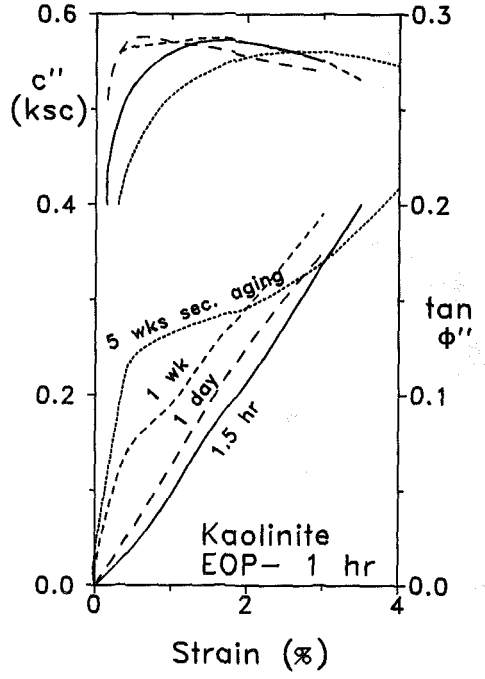


FIG. 30. Comparative IDS-Test c'' and Tan ϕ'' Components after Various Times in Secondary Compression Aging

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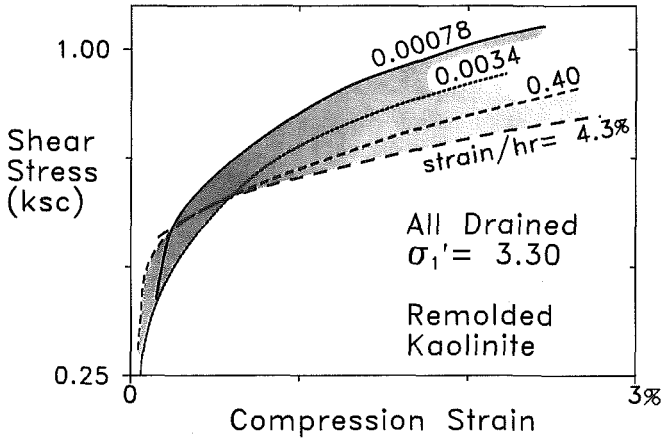


FIG. 31. Comparative IDS Compression Tests using Different Rates of Strain

ponent ϕ'' steadily increased with decreasing rate of strain. This type of aging, by reducing the drained strain rate, also produced a strength increase that was seemingly entirely due to an increased frictional capability of the structure of the kaolinite clay.

Recall the example of aging modulus increases in sands in Fig. 10. Let's also now look at the reason for these increases. Daramola (1980) also carefully measured the dilatancy behavior of the sands during the compression after the different aging times, with the results shown in Fig. 33. Although one can see some overlapping, possibly due to experimental error effects, the general trend seems clear: namely, that negative dilatancy reduces and positive dilatancy increases with an increase in aging time. The dilatancy component of the strength of sands clearly results from a mechanical effect.

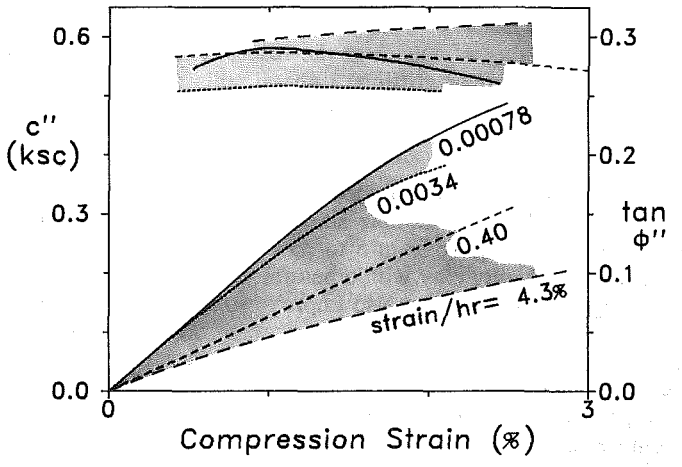


FIG. 32. Comparative c'' and tan ϕ'' Component Separations from Fig. 31 Tests

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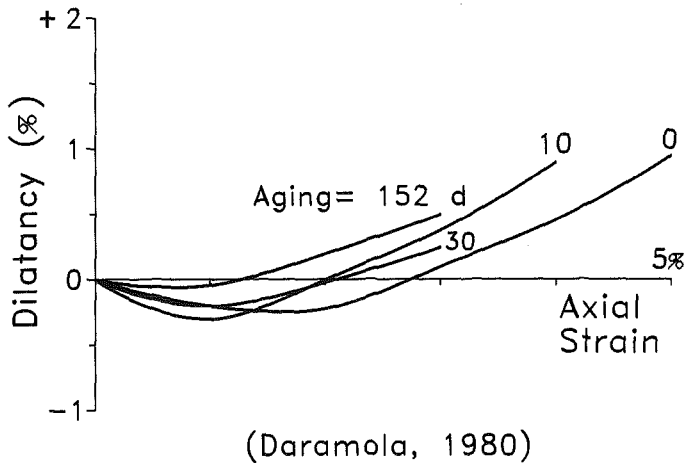


FIG. 33. Comparative Aging-Dilatancy Behavior for Sand Compression Tests in Fig. 10

The aging somehow allows the sand particles to interlock more effectively, which increases the effective stress-dependent component, and therefore the frictional component of a sand's mobilized shear resistance. With the data in Figs. 10 and 33, we have an example of aging effects increasing friction by using a test other than the IDS test to demonstrate the point.

Mesri et al. (1990) present a detailed review of various aging modulus and strength gains in clean sands, especially with respect to G_o and q_c. They also attribute these gains to increases in friction capability, not to cementation and cohesion. They make a strong argument that these gains result from increased micro-interlocking effects that result from the microshifting of particles during the strains of secondary compression.

Organic and Residual Soils

The examples of aging effects, and the research indicating its frictional nature, all involve natural or artificial mineral soils of sedimentary origin. The writer does not know to what degree highly organic or residual soils will exhibit aging improvement and can only speculate based on limited available data.

Concerning organic soils, the previous discussion of the aging-preconsolidation effect noted the five example sites in Larsson and Eskilson (1989), with organic contents ranging from 5 to 35%. Zeevaert (1983) had another example in an organic silty clay with a natural water content equal to 110%. These data suggest that aging can also improve soils with significant organic content. Very highly organic soils, such as peats, may or may not exhibit significant aging effects of the type described in this paper. Such soils usually have very significant secondary compression behavior. Because secondary compression and aging seem to be related, they may also show important aging effects. Of course, the additional aging effect of the possible decay of the organics in the soil might dominate their overall behavior.

Residual soils form, by definition, from the weathering of the parent rock.

This process, by itself, represents a deterioration of strength and compressibility properties with geologic time. It remains unclear whether the engineerirng-time aging effects described herein will also occur in a residual product that already has become a soil. The writer did test machine-extruded specimens of residual clay in a manner similar to that described by Figs. 27–30, and found similar results. Figs. 36 and 37 show data obtained from a residual clay. Perhaps once a material becomes particulate, whether of residual or sedimentary origin, it will show aging effects.

Locked Sands

The discovery of locked sands by Dusseault and Morgenstern (1979) and the subsequent work it inspired by others, for example Palmer and Barton (1987) and Barton and Palmer (1989), further supports the connection between aging and friction strength gain. During geologic aging of millions of years, such sands have developed little or no cohesive bonding but their density and interlock friction increased to the point that they became sandstones. They have a relative density of approximately 130% and a significant unconfined compressive strength due entirely to interlocking. For example, Dusseault and Morgenstern (1979) present data for a St. Peter sandstone from Minnesota that show a direct shear test Mohr envelope from block samples with a 0–100 kPa secant $\phi' = 71°$ and a $c' = 0$. The same samples show an immediate postfailure residual ϕ' equal to 33° and c' equal to zero. This gives a peak-to-residual tan ϕ' ratio = 4.5. The destruction of the natural interlocking along thin failure planes accounts for this approximately 80% decrease in shear strength.

With locked sands, we have a whole class of geotechnical materials with ϕ' increasing greatly and c' remaining zero over very long aging times. This apparent fact reinforces the plausibility that some increase in ϕ' can also occur over engineering times. Interestingly, although the ϕ' increase in locked sands comes from increased mechanical interlocking, Dusseault and Morgenstern report that the increased interlocking itself results from silica solution and reprecipitation on the quartz sand grains. We can see from this locked-sand example that chemical action that changes the surface of soil particles during aging may produce mechanical-frictional strength-increase effects.

All the aforementioned investigations and examples have convinced the writer that aging effects over engineering times occur mostly, and perhaps sometimes entirely, because of increases in the soil friction strength mobilization capability of the sands, silts, and clays involved. Frictional effects suggest mechanical effects and mechanisms.

Suggested Mechanisms

Let's now look at some of the evidence available to suggest or indicate the soil particle and soil-structure mechanism perhaps responsible for most of the aforementioned aging effects. As a start, consider "thixotropic" aging effects.

Thixotropy

Any discussion of soil aging effects and study of the related technical literature inevitably at some point leads to the term thixotropy. It does not

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appear to have an accepted, precise meaning in soil mechanics, although Mitchell (1960, 1976) has suggested one. The dictionary defines thixotropy as the property of certain gels that liquefy when subjected to vibratory forces, then solidify again when left standing. To most geotechnical engineers it has a general, somewhat vague meaning associated with strength regain with time after some type of remolding. Mitchell has focused his definition to include isothermal conditions and reversible behavior at constant composition and volume. ASTM D653 has long defined thixotropy as "the property of a material that enables it to stiffen in a relatively short time on standing, but upon agitation or manipulation to change to a very soft consistency or to a fluid of high viscosity, the process being completely reversible." This occurs in some clay suspensions, wherein, from the work of Mitchell (1960), they behave thixotropically because of alternating dispersion-flocculation-dispersion-etc., at constant volume.

No one, to the writer's knowledge, argues that sands exhibit thixotropic behavior. Therefore, the examples of the significant aging effects in sands given in Figs. 6, 8, 10, 11, 15 and Table 4 herein, and in Tavenas and Audy (1972), present a strong argument that something else important happens during soil aging that we cannot explain by thixotropy.

The profession has known about soil thixotropy more than 50 years. One can find the word used in connection with all manner of aging effects. Perhaps we have used it mostly to give a word to some aging phenomena we otherwise did not understand. The writer has not heretofore discussed it in this paper to avoid confusion with the more important, in his opinion, mechanical aging effects described herein.

The writer thinks of thixotropic effects as taking place in high void ratio clays or clay suspensions under conditions of very low effective stress. Under such conditions, as explained by Mitchell (1960), the attractive forces between particles cause particle flock or aggregate accumulations, which subsequently easily break apart (disperse) due to the small shear stresses of mechanical agitation or remolding. These effects probably dominate during the formation of soils, as at the seabed during the accumulation of a cohesive sediment. However, once significant effective overburden pressure accumulates, they begin to dominate behavior—including aging effects. The writer estimates this stress at approximately 0.2 tsf (20 kPa).

The energy that drives thixotropic aging has an internal source—the colloidal forces of attraction between clay particles. The energy that drives the mechanical aging effects presented and discussed in this paper have an external source—the effective stresses causing the volume decreases of secondary compression, or the shear strains of creep, or both.

Although thixotropic and mechanical aging probably occur for different reasons, with different sources for the driving energy and probably a different order of magnitude in the aging effects produced, they do have some things in common. Both result in soil structure or fabric changes, and both produce a stiffening and strengthening. Both also involve flocculation and dispersion. However, thixotropic aging occurs because of undrained flocculation and, as shown subsequently, mechanical aging probably involves drained dispersion. Thixotropic aging probably occurs much more rapidly. Mitchell (1960) also has presented evidence to show that thixotropic behavior reduces pore pressures—perhaps at least in part for the same basic reason previously suggested for mechanical aging. Considering the many qualitative

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similarities, one can see how thixotropy and mechanical aging might become confused.

To summarize, the writer postulates that thixotropic and mechanical soil aging occur simultaneously and that they have similar qualitative aging effects. But, they occur for different reasons, and thixotropic aging exerts a much weaker effect. Thixotropy probably completely dominates under high void ratio and very low effective stress conditions in clays. Mechanical aging probably completely dominates in sands at all stresses and in clays at the effective stress levels in most engineering practice. Let us now set aside thixotropy and continue by further investigating the likely mechanisms for mechanical aging.

Secondary Compression

Secondary compression, or the creep-type, logarithmic decay rearrangement of soil particles into slightly denser particulate structures, has a close association with aging effects. Aging and secondary effects seem to have a strong interrelationship. Mesri et al. (1990) explore this in detail for clean sands. They and others have attempted to ascribe some or all of the aging effects described herein to the density changes taking place during secondary compression.

The issue possibly boils down to this: Some believe that secondary compression density change determines a necessary and sufficient condition to describe aging effects. The writer believes the necessity because it provides the energy for the aging mechanisms. However, he doubts its sufficiency because density changes do not explain the magnitude of the aging effects. For example, as pointed out by Leonards (1972), in Fig. 1 if secondary density change effects alone explained the aging preconsolidation, then after 10^6 days compression the preconsolidation point b would have had a void ratio below point d on the dashed portion of the virgin compression curve shown, and not at the much higher void ratio point c, as extrapolated for 10^6 days compression.

Particle Interference

Consider again the evidence showing increasing dilatancy in aging sand given in Figs. 10 and 33. Soil aging, say in creep or secondary compression, always involves at least small strains, thus some particle movements must occur. The evidence presented in Fig. 33 shows increased interlocking effects, which, given the short times involved, probably result from small slippages of particles that produce a significant net increase in particle-to-particle interlocking. Mesri et al. (1990) reached the same conclusion. It appears that engineering-time aging effects in sand require some movement of the particles.

As further evidence of the need for significant particle movements to produce the aging effects, consider the results from the series of identical IDS compression tests performed many years ago using compacted kaolinite powder with a variety of pore fluids. After saturating the compacted kaolinite powder with the various fluids, and consolidating them all to the same isotropic compression and for the same time in secondary compression (one day), the writer subjected them to the same type and rate of compression and used IDS tests to separate the basic components. Fig. 34 gives the results for the c'' component and it shows no special order in the results—the vari-

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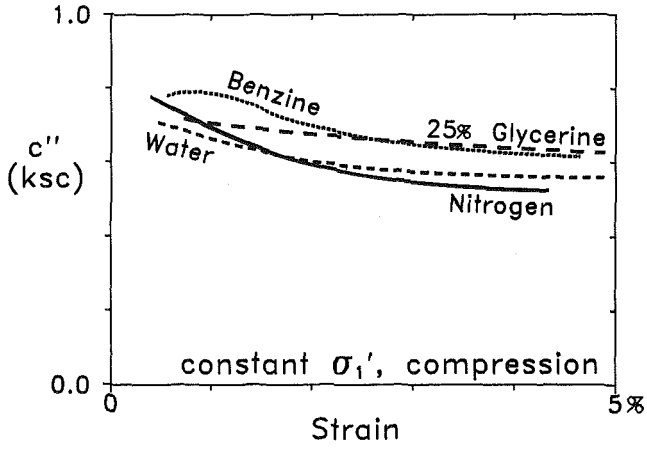


FIG. 34. Comparative c'' Mobilization Behavior from Compression Tests using Kalonite Powder Saturated with Different Pore Fluids

ability seems merely to reflect experimental variability. Now consider the friction component given in Fig. 35.

Fig. 35 shows a dramatic increase in the tan ϕ'' friction mobilization, at all strains tested, with a decrease in pore fluid viscosity and density. The writer interpreted this as follows: As the viscosity and density decreased the pore fluid interfered less with the particle movements associated with aging. Thus, a lower viscosity pore fluid and a smaller density of particles allowed more aging particle movements in a given time than with a higher viscosity and density. Less restriction of particle movements improved the rate-of-aging effects as shown by the increased mobilization friction capability of the soil. Schmertmann (1976) presents more details about these tests.

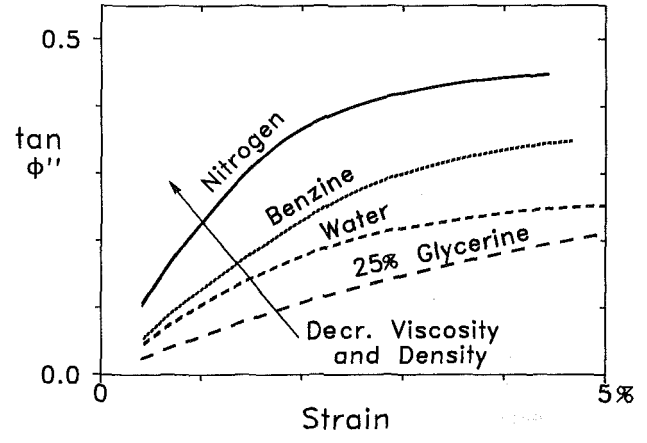


FIG. 35. Comparative Mobilized Tan ϕ'' Behavior from Same Tests as in Fig. 34

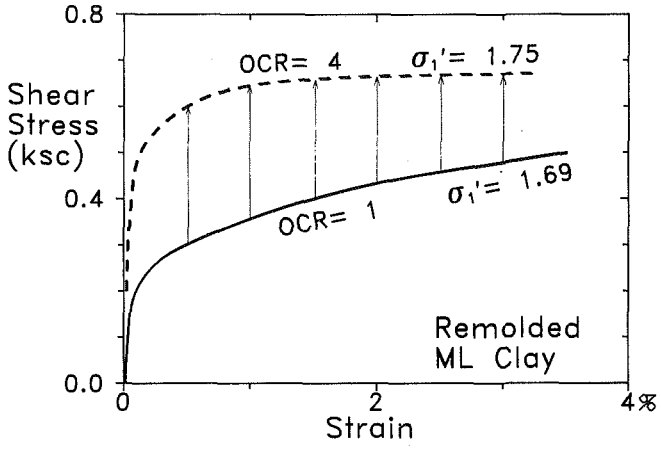


FIG. 36. *Comparative Compression Tests from Duplicate Specimens of Same Clay* Consolidated to OCR Ratios of 1 and 4

Clay Dispersion

Geotechnical engineers know well that the laboratory overconsolidation of a clay forces its structure through a yield condition, which causes the extensive particle movements associated with compression along the high compressibility virgin compression curve. This results in a much lower void ratio and a much stronger soil. Fig. 36 gives an example of the effect of an OCR = 4, using duplicate extruded samples from a low-plasticity residual clay. Although compressed at approximately the same effective stress values throughout the compression, the OCR = 4 sample has much greater strength. The writer then used IDS tests to separate and compare the basic components from each of the samples in Fig. 36. Fig. 37 shows the comparison.

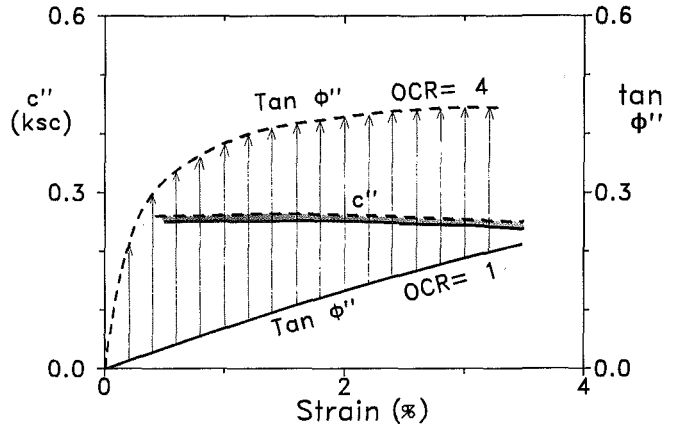


FIG. 37. Comparative IDS-Test c'' and tan ϕ'' Mobilization for Same OCR of 1 and 4 Tests in Fig. 36

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Fig. 37 shows that the c'' component has remained almost the same, while the tan ϕ'' component has increased dramatically as a result of the overconsolidation. We thus have a result similar to that produced by aging—namely, c'' remains about the same or may even decrease while ϕ'' increases dramatically. The interested reader can find many other examples in Schmertmann (1976). Note that these results reverse the concept that many engineers have about the effect of overconsolidation on a clay. They incorrectly believe that overconsolidation increases true cohesion because it reduces void ratio.

Geotechnical engineers usually think of the particle movements associated with the overconsolidation of clay as forcing the plate-shaped clay particles into closer, and therefore also more parallel, positions, or dispersing them, according to the terminology suggested by Lambe (1953). One can say that the OCR = 4 clay must have a more dispersed structure. By association with the similar c'' and ϕ'' effects, perhaps aging also tends to disperse structure?

To investigate the preceding question the writer deliberately dispersed machine extruded kaolinite samples by adding trace amounts of polyphosphate dispersants during the mixing before the extrusion. In this case, for a given consolidation pressure, adding the dispersant greatly reduced void ratio and decreased the PI from 21 to 4%. The writer then performed an IDS test similar to those done previously for nondispersed kaolinite when using different aging times, with the results first presented in Fig. 30. Fig. 38 repeats the tan ϕ'' part of Fig. 30 with the addition of the heavy line showing the tan ϕ'' mobilization of the dispersed kaolinite, but after only one day of secondary aging. One can see that the frictional mobilization with the one-day aged, dispersed kaolinite equalled or exceeded that after five weeks of secondary aging using the undispersed kalonite. It seems that the effect of aging produces similar c''-ϕ'' effects to that resulting from dispersants—again suggesting a linkage between aging effects with the drained, dispersive movement of soil particles.

As noted previously, Mesri et al. (1990) suggest a micro-interlocking mechanism for the increase in stiffness and strength of clean sands as a direct result of secondary compression aging. It seems possible, perhaps even likely, that the similar magnitude of aging-increase effects in clays could occur in part for this reason also. However, given the generally higher void ratios in clays, their generally plate-shaped particles, and the still unknown details of how their solid surfaces make contact or near-contact, it seems unlikely that only particle interference, or only micro-interlocking, can explain the entire phenomena. However, any increased interlocking explanation is consistent with the aging increase in tan ϕ'' documented herein.

The writer believes that the drained creep movements associated with aging, with the driving energy supplied by the in situ effective stresses, tend to disperse complicated structures such as those found in fine-grained soils. This dispersion then leads to an increased basic frictional mobilization capability of the soil. Perhaps as parts of the structural fabric of the soil stiffen due to the dispersion under the drained conditions during aging, the stresses arch to these stiffer parts. Or, the dispersive movements may occur primarily in the weaker, softer parts of the fabric and cause an arching stress transfer to the stiffer parts. Either way, this arching occurs internally at the particulate level with the overall, average effective stress remaining constant. The stiffer parts of the fabric probably also have greater strength, at least statis-

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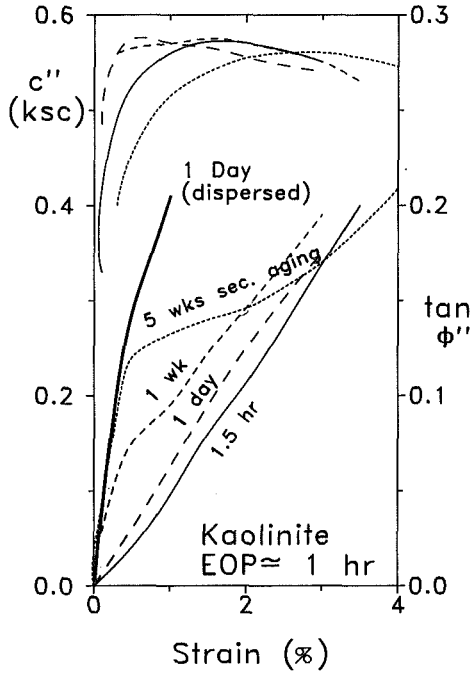


FIG. 38. Comparing Tan ϕ'' Mobilization of Dispersed Kaolnite with that of Relatively Undispersed Kaolnite Subjected to Various Times in Secondary Compression Aging

tically, thus increasing the overall effective stress stiffness and strength of the soil. The writer believes that the three possibly interdependent effects of dispersive particle movements, internal stress arching, and increased interlocking combine to produce the associated mechanical modulus and strength increases during engineering aging.

Conclusions and Practical Applications

1. The profession now has reports of significant aging effects occurring in virtually all types of soils, including dry sands.
2. The many examples herein demonstrate that soil aging over engineering times can cause a general 50 to 100% improvement effect in many key soil properties. Judging by the G_o and liquefaction research (Figs. 9 and 11), and secondary compression generally, these effects increase with time at a diminishing rate approximately linear with the logarithm of time.
3. The paper includes a possibly new pore-pressure dissipation theory that provides a mechanism for significant pore-pressure reduction and dissipation effects in nearly saturated soils. This dissipation does not result from hydrodynamic water flow. It results from the aging transfer of load from the pore fluid to the soil fabric skeleton.
4. Considerable evidence exists that indicates that most engineering-time age-strengthening effects result from increased basic soil friction, including dilatancy

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effects, and not from increased cohesion. The in situ effective stresses drive the mechanisms involved. These include grain slippage, soil-structure dispersion, increased interlocking, and a postulated internal arching of stresses. Thixotropic aging effects occur primarily at very low effective stresses and under undrained conditions in cohesive soils. Mechanical aging has a different cause.

5. The preceding mechanical explanations of the age-strengthening phenomena appear adequate to explain engineering-time effects. None of the examples showed evidence of chemical bonding or other cohesion effects. Increasing the basic soil friction ϕ'' over engineering times implies a mechanical origin of this aging phenomenon. This involves a definite, seemingly easily understood mechanical effect. As engineers learn more about what soils show this effect, and what if any soils do not, and in what circumstances, we can perhaps continue to expand our use of the beneficial effects of soil aging over engineering times.

6. Our degree of understanding and experience has already reached the point where we can use engineering-aging effects in practice, as in some of the current usage and possibilities suggested below:

a. Look more carefully, if warranted, for the aging-preconsolidation effect in so-called normally consolidated natural soils. More careful testing, perhaps involving higher quality sampling and lab testing or more sophisticated in situ tests, may demonstrate a usable preconsolidation effect.

b. Engineers usually do not correct many routine laboratory test results for any aging effects. Should they have an aging correction applied, similar to that now done routinely by some engineers for laboratory G_o and liquefaction stress ratio data?

c. In situ tests, after soil-structure–disruptive ground-improvement methods such as dynamic compaction, may require correction for aging effects before using them as criteria for acceptance or rejection.

d. Near-saturated soils may generate significant pore pressure under increased loads, but have it dissipate more rapidly than hydrodynamic theory would indicate, due to aging effects.

e. When appropriate, perhaps designs should include the beneficial effects of engineering aging. For example, allow an initial lower factor of safety in an embankment or other fill if a significant time interval will pass before it must perform under load.

f. When performing research, or transposing the results from research into recommendations for practice, consider the possible different aging effects between the soils tested in the research and those to which the recommendations will apply.

ACKNOWLEDGMENTS

Many engineers have made significant contributions to the subject of soil aging. The writer has acknowledged some in the text, figures, and references included in this paper. He has had to omit others because of length constraints. The writer's son, Gary Schmertmann, skillfully prepared the figures. Anita Cannon worked through many drafts to prepare the manuscript. Richard Karkowski performed the sand-aging tests noted in conjunction with Fig. 6 as part of a senior honors project in the Department of Civil Engineering, University of Florida. Department chairman Paul Thompson generously provided department space and faculty support, with faculty members John Davidson and David Bloomquist taking an active part in this research.

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Finally, the writer is indebted to his partner, David Crapps, for supporting the time and treasure involved in preparing this lecture and paper, and for his valuable suggestions made during the course of many informal conversations as the work on the paper progressed.

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APPENDIX I. REFERENCES

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APPENDIX II. NOTATION

The following symbols are used in this paper:

$\mathbf{A}, \mathbf{B}$ = Skempton pore-pressure coefficient;
c' = effective stress cohesion intercept;
c'' = IDS-test cohesion intercept;
c_o = IDS-test bond shear strength;
C_{pf} = compressibility of pore fluid;
C_s = compressibility of soil skeleton;
E = Young's modulus;
G_o = low-strain shear modulus (dynamic);
K = ratio σ'_h/σ'_v;
k = coefficient of permeability;
M, M_{oed} = one-dimensional, drained compression modulus;
N = hammer blow count for a given penetration of SPT sampler or cone;
N_G = % increase in G_o per log cycle of time;
n = porosity;
p'_c = preconsolidation effective stress;
q_c = static-cone test bearing capacity;
q_{co} = q_c at time = 0;
S = degree of saturation;
s_u = undrained shear strength;

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s_v = s_u from vane shear test;
t = time;
u = pore water pressure;
w = water content (dry-weight basis);
ϵ = strain;
γ_d = dry unit weight;
Δ = denotes change;
σ' = effective normal stress;
σ_1 = major principal stress;
σ'_1 = major principal effective stress;
σ_3 = minor principal stress;
σ'_3 = minor principal effective stress;
σ'_c = major effective stress at EOP consolidation;
σ'_v = vertical effective stress;
σ'_h = horizontal effective stress;
τ = shear stress;
ϕ = Mohr-Coulomb soil friction angle, total stress basis;
ϕ' = Mohr-Coulomb soil friction angle, effective stress basis;
ϕ'' = Mohr-Coulomb soil friction angle, from IDS test; and
$'$ = superscript to denote effective stress.

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