

3.0 PROPOSED MECHANISMS FOR AGING EFFECTS IN SANDS

3.1 Introduction

Despite the increasing number of examples that illustrate the effects of aging on sand properties, the mechanisms behind aging phenomena are not well understood. The state of the knowledge is that several researchers have proposed hypotheses to explain these effects, and some tests have been performed to support their ideas. However, no strong evidence has been presented in support of any of the hypotheses.

An understanding of the mechanisms that cause aging effects is of direct practical importance in the design and evaluation of ground improvement. If the mechanism(s) responsible for aging effects in sand can be determined, then predictive models can be developed. Thus far, any methods that have been proposed to predict aging effects (Charlie et al. 1992; Joshi et al. 1995; Schmertmann et al. 1986), as described in chapter 2, are based on empirical approaches. The large variability in the quality of data as well as incomplete information about site conditions presented in chapter 2 makes the use of such empirical approaches difficult to apply to new cases. A fundamental model could help engineers account for the important factors influencing aging effects, and to better use time dependent increases in strength in ground improvement designs.

Six hypotheses from the literature are described. Three are mechanical in nature, two are based on chemical bonding, and one is a biological mechanism. Evidence supporting each mechanism is presented followed by a critical discussion of the hypotheses.

3.2 Schmertmann's Hypothesis

3.2.1 Hypothesis

After investigating a number of examples and hypotheses to explain aging effects in both clays and sands, Schmertmann (Schmertmann 1981, 1983, 1987; Schmertmann et al.

1986) postulated in his Terzaghi lecture (Schmertmann 1991) that aging effects are the result of frictional mechanisms as opposed to chemical bonding or cementation. In his arguments, he made no distinction between the phenomenon in sands and clays and considered the mechanisms responsible to be the same for all soils. Schmertmann considered as possible mechanisms thixotropy, secondary compression, dispersive particle movements, internal stress arching, and increased interlocking. Thixotropy was discounted as a possible mechanism because the small attractive/repulsive forces between charged particles thought to be responsible for thixotropy (Mitchell 1993) are negligible in sands. Secondary compression was also discounted because, although aging effects develop during secondary compression, Schmertmann did not believe that the corresponding density changes were sufficient to explain measured increases in strength. He postulated that aging effects in sands are the result of the interdependent effects of dispersive particle movements, increased interlocking, and internal stress arching; and each mechanism is described below.

3.2.2 Dispersive Particle Movements

Schmertmann believes that, during aging, small particle movements occur which lead to a more stable soil structure. These movements result in an increase in stiffness and a decrease in the compressibility of the soil. He suggests that the movements that occur during aging are akin to the particle movements that occur during mechanical overconsolidation, where it is known that platy particles will tend to orient themselves perpendicular to the direction of the major principal effective stress. In this case, the resulting fabric is more dispersed, meaning that there are more face-to-face parallel arrangements of the platy particles. This is the type of particle movements meant by this hypothesis.

3.2.3 Increased Interlocking

Schmertmann postulated that the dispersive particle movements mentioned above would result in an increased micro interlocking at the particle contacts, which might also contribute to the property changes during aging. To understand micro interlocking, visualize

a contact point between two particles on a microscopic level. Even smooth particles at small scales have an irregular and rough shape. Small particle movements or grain slippage could cause the two particles to fit together in a more stable arrangement, thus providing an increased resistance to shear. The strength due to the micro interlocking would be a function of the normal stress at the contacts and would cause an increase in the frictional component of strength. During shear, this interlocking would manifest itself as an increase in stiffness and dilatancy.

3.2.4 Internal Stress Arching

Schmertmann also hypothesized that aging effects might be caused by uneven stress distributions within the soil skeleton. The idea is that there are stiffer elements, or zones, within the soil that carry more stresses than adjacent weaker elements, creating an arching effect. This phenomenon has been observed in the laboratory in studies using photoelastic material to simulate soils. This experimental technique enables observation of stress distributions within objects that have complicated geometry or loading conditions. This method relies on the fact that the stress distributions within certain transparent materials are visible when illuminated with polarized light (Post 1989).

Figure 3.1 shows an example of how the stresses from a constant boundary load are transmitted through a collection of photoelastic disks (Drescher and De Josselin De Jong 1972). In this figure, the faint circles of different sizes are the photoelastic disks that represent a granular soil. The dark, spidery chains that connect some of the disks are called load chains; they show the actual pathway of stresses through the collection of disks. The thickness of the load chains is related to the magnitude of the stresses in that area. Note that there are clearly zones where high stress concentrations exist as well as zones where very little stress is transmitted. Numerical simulations (Kuhn 1987) using discrete element models have shown similar load transfer behavior in granular assemblies, as shown in Figure 3.2.

As small particle movements or grain slippage occurs during secondary compression, Schmertmann believed that there may be a redistribution of stresses within the soil skeleton, with high stress load chains shedding stresses to other particles. This redistribution may also decrease local shear stresses between particles and increase normal stresses between particles, resulting in an overall stiffer soil structure.

3.2.5 Evidence Supporting Hypothesis

Schmertmann's hypothesis relies heavily on the results of IDS tests. The IDS test is a modified triaxial test that was developed to show how the frictional and cohesive components of shear strength develop with increasing shear strain. The "D" stands for the portion of strength that is stress dependent and "I" stands for the stress independent component of strength. Some further background information is needed to interpret the results of these tests.

IDS tests are based on the underlying assumption that small changes in void ratio do not affect the fabric, and behavior of overconsolidated soils. Figure 3.3 shows two identical samples that are consolidated to the same stress, but then unloaded to different stress levels. If the difference in void ratio is very small between the two samples then Schmertmann assumes that they will have the same fabric and behavior. If both samples are sheared at their respective confining stresses, then at any value of axial strain the Mohr circles for both tests can be plotted. This is shown in Figure 3.4. From the Mohr circles at a given value of strain, an envelope can be drawn and the values of the stress dependent component of strength (ϕ'') and the stress independent component of strength (c'') can be determined. If this is repeated for different values of axial strain, then the development of ϕ'' and c'' throughout the shearing process can be plotted, as shown in Figure 3.5.

In an effort to remove the effect of variability between samples, Schmertmann actually performs IDS tests on a single sample, and achieves the two stress strain curves by cy-

cling the confining stress back and forth during shear, as shown in Figure 3.6. Details of the test procedure and experimental setup can be found in Schmertmann (1976).

According to Schmertmann, IDS tests accurately separate the components of “friction” and “cohesion.” This is an important assumption, because IDS tests are not commonly performed tests and not universally accepted as valid. Schmertmann believes that, during aging, an increase in friction would indicate that a mechanical mechanism is responsible for aging effects, whereas an increase in cohesion would indicate that bonding or cementation is responsible.

Figure 3.7 shows the results of four IDS tests that were performed to investigate the effects of aging on samples of kaolinite. The samples were prepared identically but were subjected to different amounts of secondary compression (1.5 hours, 1 day, 1 week, and 5 weeks). For each of these tests, Figure 3.7 shows the development of the frictional component and the cohesive component of strength with increasing strain. The results show that, with time, the frictional component of strength increased and was mobilized at smaller values of strain. In contrast, the cohesive component of strength did not change significantly with time. To Schmertmann, this provided proof that aging effects are primarily a mechanical phenomenon.

There are questions, however, regarding the validity of the IDS tests to study aging effects, given that the effective confining stress is cycled back and forth during shear. If, during aging, weak cementation were to develop, this cyclic loading may destroy those bonds. To address this issue, Schmertmann presented the results of two tests conducted on glass spheres. In one of the samples the spheres were artificially cemented. The IDS test results correctly showed an increase in c'' due to the cementation with no corresponding increase in ϕ'' . However, artificially cemented glass spheres may not be representative of naturally occurring cementation. It is possible that a natural process of cementation would be fundamentally different than that typically simulated in the labora-

tory. For example, artificially cemented specimens are often prepared by mixing sand with a known quantity of dry cement and then saturating the sample (Clough et al., 1981; Rad and Tumay, 1988). In natural soils, cementation may not occur so uniformly. Dissolution would likely occur at particle contacts where local areas of high stresses exist or from the soluble fractions of the sand's mineralogy. Any subsequent precipitation, and thus possible cementation, might then initiate at the contacts between particles in localized areas of lower stresses. Therefore, uniformly cemented sands in the laboratory may not be representative of the aging process in nature.

To support his hypothesis that dispersive particle movements cause an increase in the frictional component of strength, the results shown in Figure 3.7 were compared to another test of a kaolinite sample mixed with trace amounts of polyphosphate dispersants. The purpose was to compare the results of an aged sample with that of a chemically dispersed sample. The results of this test are shown in Figure 3.8 along with the results of two of the tests from Figure 3.7 for comparison. The results show that the frictional component ϕ'' of the dispersed sample is very similar to the ϕ'' of the sample aged for five weeks. To Schmertmann, this showed that dispersive particle movements during secondary compression were responsible for the increase in friction.

Schmertmann did not report the void ratios of the two samples, and it is not known whether the structure of the aged sample studied after the test. Chemical dispersants can drastically alter soil properties such as compressibility and strength, and Schmertmann did not show conclusively that the two samples had the same structure. Without this knowledge, it is difficult to conclude that the increase in ϕ'' for the aged sample was in fact due to an increasingly dispersed soil fabric.

In addition, the idea that dispersive particle movements at a given void ratio lead to a stronger stiffer soil runs contrary to the current understanding of flocculated and dispersed structures in soils. Given the same conditions, a flocculated soil will have a

higher stiffness and shear strength than a soil with a dispersive structure. Soils tend to flocculate, or clump together, when the thickness of the diffuse double layer decreases. The diffuse double layer is a measure of the electrical field around a clay particle; when its thickness decreases, positively charged ends of clay particles can orient themselves with the negatively charged faces of other clay particles. In dispersed structures the thickness of the diffuse double layer is relatively larger, the repulsive forces between particles are larger, and there is less physical interaction between particles. The result is that the particles orient themselves in a more face-to-face, or parallel, arrangement, which during shear allows the particles to slide past each other, resulting in lower values of shear strength and stiffness. Thus, it is difficult to see how the development of a dispersed structure with time would result in an increase in the frictional component of strength.

In fact, experimental studies on thixotropic hardening of fine-grained soils have shown that, during aging, soils will actually tend towards a more flocculated structure. It is thought that this gradual change in structure may be responsible for some of the sensitivity of natural soil deposits (Mitchell 1993).

It is important to note that no IDS tests on aged specimens of sand were reported. In fact, Schmertmann used the results from tests on clay, and extrapolated to form a hypothesis including the behavior of sands.

As evidence that increased interlocking may be responsible for aging effects, Schmertmann relied on the drained triaxial test results from Daramola (1980) which were presented in section 2.5. In these tests, sand specimens that were aged for up to 152 days showed a clear increase in modulus and a slight increase in dilatancy. Schmertmann, however, did not present any direct evidence linking Daramola's results to an increased interlocking mechanism. No supporting evidence or data was presented for the internal stress arching mechanism.

3.2.6 Summary

Schmertmann's work does not provide conclusive evidence that aging effects in sands are the results of a mechanical mechanism. His work provides three hypotheses to describe mechanical mechanisms for aging. The evidence presented in support of these hypotheses relies heavily on IDS tests, which are unusual tests. It is not widely accepted that the c'' and f'' values from the IDS tests accurately separate a soil's chemical bonding and frictional strength. Results of IDS tests on aged clays suggest that there is an increase in the frictional component of strength of clays during aging; however, the tests do not confirm that it is due to an increased dispersed soil fabric.

Without decreases in void ratio, dispersive particle movements would not appear to lead to an increase in stiffness given the current understanding and definition of dispersive clay structures. In addition, these movements do not seem to be applicable to sands since they are not charged particles.

Most importantly, all of Schmertmann's tests were run on clay samples; no tests on sand samples were reported. Finally, no direct evidence was presented in support of the increased interlocking mechanism or the internal stress arching mechanism.

3.3 Mesri's Hypothesis

3.3.1 Introduction

Mesri et al. (1990) performed a study specifically on the aging effects in clean sands; i.e., sands with little or no fines. Examples of these effects on the small strain shear modulus and cone penetration resistance from the literature were presented, and the authors postulated that the mechanism responsible for the aging phenomenon is due entirely to frictional resistance. Empirical equations were proposed which estimate the increase in both

the small strain shear modulus and the cone penetration resistance with time as a function of the soil's behavior during secondary compression.

3.3.2 Hypothesis

Mesri et al. (1990) proposed that aging effects in sands are due to an increased frictional resistance that develops during secondary compression. According to the authors, this increased resistance does not occur solely from the change in density that occurs during drained secondary compression. Rather, it is due to a continued rearrangement of particles resulting in increased macro interlocking of particles and micro interlocking of surface roughness. These mechanisms cause an increase in both stiffness and horizontal effective stress, which, in turn, cause increases in properties such as the small strain shear modulus and the cone penetration resistance.

3.3.3 Evidence Supporting Hypothesis

Mesri et al. presented no direct evidence to support their hypothesis of an increased interlocking during secondary compression. An argument against thixotropy and cementation as mechanisms, however, was made using the drained triaxial test data from Daramola (1980). As discussed in section 2.5, specimens of Ham River sand that underwent secondary compression for up to 152 days had an increased modulus up to approximately 3% axial strain. According to the authors, any mechanism involving bonding or cementation would result in a brittle stress-strain response and any property increase would be destroyed at strains less than 3%. Thus it was postulated that another less brittle mechanism must be responsible for the increase in stiffness, and the frictional mechanism was proposed.

In support of the increased interlocking hypothesis, an equation was presented to predict the increase in cone penetration resistance with time. The equation accounts for the decrease in compressibility that occurs during secondary compression and the effect of densification due to ground improvement. The aging phenomenon was first related to a

time-dependent increase in the constrained modulus, which is $M = \Delta\sigma'_v / \Delta\varepsilon_v$ where no lateral strain is allowed. The constrained modulus at the end of primary consolidation can be written as

$$M_p = \frac{\sigma'_{vo}}{0.434 \frac{C_c}{(1+e_o)}} \quad (3.1)$$

and during secondary compression it can be written as

$$M = \frac{\sigma'_{ve}}{0.434 \frac{C_r}{(1+e_o)}} \quad (3.2)$$

where

$\Delta\sigma'_v$ = Change in vertical effective stress

$\Delta\varepsilon_v$ = Change in vertical strain

σ'_{vo} = Initial vertical effective stress

σ'_{ve} = Equivalent consolidation pressure corresponding to the void ratio at time t greater than t_p

C_r = $\Delta e / \Delta \log \sigma'_v$ is the recompression index

C_c = $\Delta e / \Delta \log \sigma'_v$ is the compression index

e_o = Initial void ratio

Δe = Change in void ratio

The equivalent consolidation pressure is related to time by

$$\frac{\sigma'_{ve}}{\sigma'_{vo}} = \left(\frac{t}{t_p} \right)^{C_\alpha / C_c} \quad (3.3)$$

where

t = time $> t_p$

t_p = time at the end of primary consolidation

C_α = $\Delta e / \Delta \log t$ is the secondary compression index

The exponent in Equation 3.3 relies on the assumption that, for a given soil, there is a unique value of the ratio of secondary compression index to compression index ($C_\alpha/C_c = \text{constant}$) that accounts for an increase in stiffness due to secondary compression alone.

This assumption has been advocated strongly by Mesri (Mesri 1977; Mesri 1983; Mesri 1987). Although there is still controversy over its fundamental validity (Martins and Lacerda 1989; Yasuhara 1989), values of C_α/C_c have been found to range between 0.015-0.03 for at least nine different sands in the laboratory (Mesri 1990) over a stress range of 50-3000 kPa. Considering that C_α/C_c for sand appears to be approximately 0.02, for practical purposes it can be considered constant.

In order to account for the effect of densification due to ground improvement, an empirical parameter, C_D , was introduced. Including the new parameter, C_D , Equations 3.1, 3.2, and 3.3 can be combined to show the increase in constrained modulus due to secondary compression and ground improvement, and can be written as

$$\frac{M}{M_p} = \frac{1}{\frac{C_r}{C_c}} \left(\frac{t}{t_p} \right)^{C_D C_\alpha / C_c} \quad (3.4)$$

Next in the analysis is to relate the constrained modulus and the horizontal effective stress to cone penetration resistance. It was assumed that horizontal effective stress increases during secondary compression and this increase can be written using the coefficient of lateral earth pressure and equation 3.3 as

$$K_o = \frac{\sigma'_h}{\sigma'_{vo}} = K_{op} \frac{\sigma'_{ve}}{\sigma'_{vo}} = K_{op} \left(\frac{t}{t_p} \right)^{C_D C_\alpha / C_c}$$

or

$$\frac{\sigma'_h}{\sigma'_{vo}} = K_{op} \left(\frac{t}{t_p} \right)^{C_D C_\alpha / C_c} \quad (3.5)$$

where K_o is the coefficient of lateral earth pressure at any time, and K_{op} is the coefficient of lateral earth pressure at the end of primary consolidation.

The question of how the horizontal effective stress changes during secondary compression was posed in a technical note published by Schmertmann (1983). Since then, horizontal effective stresses have been shown to increase during secondary compression in laboratory oedometer tests on clays (Mesri and Castro, 1987) and in numerical simulations (Kavazanjian and Mitchell 1980; Kuhn 1987). To date no evidence has been presented to show that a similar increase of K_o occurs in sands.

Combining equations 3.4 and 3.5 yields

$$\frac{M}{M_p} = \frac{1}{\frac{C_r}{C_c}} \frac{\sigma'_h}{K_{op}\sigma'_{vo}} = \frac{1}{\frac{C_r}{C_c}} \frac{\sigma'_h}{\sigma'_{hp}} \quad (3.6)$$

where σ'_{hp} is the horizontal effective stress at the end of primary consolidation. Rearranging Equation 3.6 yields

$$\frac{M}{M_p} \frac{\sigma'_{hp}}{\sigma'_h} = \frac{1}{\frac{C_r}{C_c}} \quad (3.7)$$

At this point, Mesri et al. (1990) assume that there is “a direct relationship between q_c and $(M \cdot \sigma'_h)^{1/2}$.” In order to get Equation 3.8 into a form where this assumption can be applied, the following steps can be taken:

$$\frac{M}{M_p} \frac{\sigma'_{hp}}{\sigma'_h} \frac{(\sigma'_h)^2}{(\sigma'_{hp})^2} = \frac{1}{\frac{C_r}{C_c}} \frac{(\sigma'_h)^2}{(\sigma'_{hp})^2} \quad (3.8)$$

$$\frac{(M\sigma'_h)^{1/2}}{(M_p\sigma'_{hp})^{1/2}} = \frac{1}{(C_r/C_c)^{1/2}} \frac{\sigma'_h}{K_{op}\sigma'_{vo}} \quad (3.9)$$

It appears at this point that Mesri assumed the compression and recompression indices for sand to be approximately equal. The relationship $q_c \propto (M \cdot \sigma'_h)^{1/2}$ can now be introduced, and Equation 3.10 can be written as

$$\frac{q_c}{(q_c)_p} = \frac{\sigma'_h}{K_{op} \sigma'_{vo}} = \left(\frac{t}{t_p} \right)^{C_D C_\alpha / C_c}$$

or

$$\frac{q_c}{(q_c)_p} = \left(\frac{t}{t_p} \right)^{C_D C_\alpha / C_c} \tag{3.10}$$

where $(q_c)_p$ and t_p are values of cone penetration resistance and time at the end of primary consolidation or some other reference time. Mesri believes that Equation 3.10 is developed from fundamental mechanical principles and that it can be used to predict aging effects in sands.

Two case studies from the literature, presented in section 2.6.2 (Dowding and Hryciw 1986) and section 2.6.1 (Mitchell and Solymar 1984), were used to calibrate Equation 3.10 by adjusting the densification parameter, C_D , until predicted values matched well with the field data. The resulting values of C_D are summarized below.

Table 3.1 C_D Values back-calculated from the literature.

Type of Densification	C_D values	Notes
Secondary compression only	2.9	Blast densification in calibration chamber (Dowding and Hryciw 1986)
Blast densification and secondary compression	8.5	
Vibrocompaction	16-20	Jebba Dam Project (Mitchell and Solymar 1984)
Blasting	9-17	

Mesri et al. (1990) did not state the reason for their choice of the assumed relationship $q_c \propto (M \cdot \sigma'_h)^{1/2}$. Mitchell and Gardner (1975) cite that the relationship between the constrained modulus and the cone penetration resistance is commonly assumed to be written as $M = \alpha q_c$ where α is a constant. More recently, Lunne et al. (1997) recommend

$$\begin{aligned} M &= 4q_c & q_c < 10 \text{ MPa} \\ M &= 2q_c + 20 \text{ (MPa)} & 10 \text{ MPa} < q_c < 50 \text{ MPa} \\ M &= 120 \text{ MPa} & q_c > 50 \text{ MPa} \end{aligned}$$

Thus there are significant differences between the relationship assumed by Mesri et al. (1990) and other relationships published in the literature.

The parameter C_D was introduced to account for the effects of ground improvement. The method used to calculate C_D involved fitting a curve through existing $q_c/(q_c)_p$ data and back-calculating C_D . Thus, the value of C_D should be equal to 1 for cases with no ground improvement. However, based on data from Dowding and Hryciw (1986), Table 3.1 shows that a C_D value of 2.9 was calculated for the case with secondary compression only. Given this fact and the discussion above, it appears that the C_D parameter must account for factors in addition to the effects of ground improvement. As such, Equation 3.11 should be considered to be a completely empirical relationship.

3.3.4 Summary

Mesri et al. (1990) do not provide conclusive evidence that a mechanical mechanism, namely an increased interlocking during secondary compression, is responsible for aging effects in sands. They rely heavily on Equation 3.10 to support their hypothesis, and a number of assumptions were made in its development. These assumptions include

1. C_α/C_c is constant for a given soil
2. Horizontal effective stresses increase with time
3. C_r/C_c for sands is approximately equal to unity
4. $q_c \propto (M \cdot \sigma'_h)^{1/2}$

The last one differs significantly from other published relationships between cone penetration resistance and constrained modulus. In addition, the ground densification parameter, C_D , appears to be a curve fitting parameter used to match the results of Equation 3.10 with field data.

3.4 Bonding Hypothesis

3.4.1 Introduction

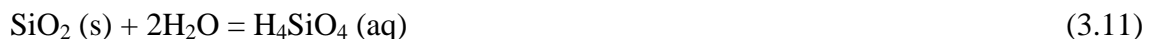
Historically, the most widespread theory used to explain aging effects in sands has involved some type of chemical bonding between particles. Terzaghi originally referred to a “bond strength” in connection with the presence of a quasi-preconsolidation pressure observed in the field (Schmertmann 1991). Generally, this bonding mechanism has been thought of as cementation, which would increase the cohesion of a soil without significantly affecting its friction angle.

Before presenting the bonding hypothesis, a brief review of the relevant chemical concepts is warranted. This discussion will focus on the dissolution and precipitation of two very common constituents in sands: silica and calcium carbonate.

3.4.2 Dissolution/Precipitation Reactions

When a solid is immersed in a liquid, there is a chemical reaction with the liquid that transforms a portion or all of the solid from the solid phase to the aqueous phase. This is called dissolution. How much of the solid will dissolve into solution is governed by its solubility. If the dissolved material exceeds a certain amount, some of the aqueous material will come out of solution back into the solid phase. This is called precipitation.

The dissolution reaction of silica, SiO_2 , in water can be expressed by



where

SiO_2 = silica (solid phase)

H_2O = water

H_4SiO_4 = silicic acid (aqueous phase)

The solid phase of SiO_2 can be found in different crystalline structures, such as quartz and cristobalite, with quartz the most common form found in nature. SiO_2 can also be found in a non-crystalline form called amorphous silica.

Quartz is the most stable form of SiO_2 , and as such it is the least soluble in water. Conversely, amorphous silica is the least stable and is the most soluble form of SiO_2 . As such, the solubilities of quartz and amorphous silica provide an upper and lower bound on the solubility of silica in water (Dove and Rimstidt 1994). The solubility of both forms increases with temperature, as shown in Table 3.2. Solubility data is often reported in units of parts-per-million (ppm), molality (moles/kg), or mg/L. Parts-per-million is roughly equivalent to mg/L, and so, for convenience, Table 3.2 lists the solubilities in both ppm and molality.

Table 3.2 Solubility of quartz and amorphous silica for different temperatures.

SiO_2 (aq) from Quartz (Rimstidt 1997b)			SiO_2 (aq) from Amorphous Silica (Langmuir 1997)		
T (°C)	ppm	molal (moles/kg)	T (°C)	ppm	molal (moles/kg)
0	5	0.00008	0	67	0.0011
25	11	0.00018	25	116	0.0019
50	21	0.00035	50	183	0.0030
100	61	0.00101	100	372	0.0062

Table 3.3 presents typical silica concentrations in the groundwater of different rock formations.

Table 3.3 Solubility of SiO₂ in some natural systems (Langmuir 1997).

	SiO ₂ (aq)	
	ppm	moles/kg
Pyroclastic Volcanics	~85	0.00142
Fresh Basalts	50	0.00083
Plagioclase Feldspars	~50	0.00083
Marine Sandstone	~15	0.00025
Limestone and Dolomites	~9	0.00015

The results in Tables 3.2 and 3.3 represent equilibrium, or steady state, concentrations. The rate of quartz dissolution and precipitation, however, is very small, as shown in Figure 3.9. This figure also suggests that saturation with respect to amorphous silica can occur in several days, whereas it may take several weeks to become saturated with respect to quartz.

Like quartz, carbonate minerals such as calcite (CaCO₃) and dolomite (CaMg(CO₃)₂) are very common in soils and are the weathering products of limestone and dolomite rocks, respectively. The dissolution/precipitation reactions are more complex than for silica, and are complicated by the fact that carbonate solubility is heavily dependent on the partial pressure of CO₂ gas in the groundwater. As the partial pressure (and thus the concentration) of carbon dioxide in the water increases, the solubility of the carbonate also increases. The median solubility of calcite in natural groundwater is approximately 50 ppm, or 0.0012 moles/kg (Langmuir 1997).

Unlike quartz, however, the solubility of calcite decreases slightly with increasing temperature. Table 3.4 shows theoretical concentrations of Ca²⁺ from calcite for two different temperatures and three different partial pressures of carbon dioxide. Note that the partial pressures are written as the logarithm of the pressure. Thus, the partial pressure increases by one order of magnitude going from log P_{CO₂} = -2.5 to log P_{CO₂} = -1.5.

Table 3.4 Ca²⁺ concentrations, in ppm, for varying temperature and partial pressure of CO₂ (Langmuir 1997).

T °C	log P _{CO₂}		
	-3.5	-2.5	-1.5
10	25	55	127
25	20	44	100

The solubilities of silica and calcium carbonate are both dependent on the pH of the water. For silica, the solubility is relatively constant for pH values below 8. As the solution becomes more basic, however, the solubility increases dramatically, as shown in Figure 3.10. The solid line represents the total amount of silica ions in solution. The dashed lines show that at higher values of pH, dissolution reactions different from Equation 3.11 govern the silica solubility.

The solubility of calcium carbonate increases at both low and high values of pH. This is shown in Figure 3.11 for a temperature of 25° C and a total pressure of 1 atm. As in Figure 3.10, the dashed lines represent different chemical reactions that result in different species of carbonate in solution.

3.4.3 Hypothesis

There are two different hypotheses associated with bonding as a basis for aging effects in sands. The first is the more traditional hypothesis for clean sands and centers around silica-water interactions. It is believed that silicic acid gel films that develop on particle surfaces can act as a cementing agent. The formation of these films may be promoted by ground improvement, which, similar to physically grinding the sand, may create a disturbed layer of amorphous silica on the surface of the particles (Henderson et al., 1970). The amorphous silica, which has a relatively high solubility compared to quartz, will go into solution. It can then subsequently precipitate at the particle contacts and act as a bonding agent (Mitchell and Solymar 1984).

The second bonding hypothesis focuses not on silica, but on the more soluble fractions of a soil skeleton such as carbonate minerals. It is thought that when the soil is saturated, these soluble fractions will dissolve quickly into solution. Subsequently, the solutes in solution would then precipitate at the particle contacts, thus causing the measured property increases.

3.4.4 Evidence Supporting Hypothesis

There is little evidence that cementation of sand particles by silica precipitation occurs after ground improvement. Denisov and Reltov (1961) obtained results that may support this hypothesis, as discussed in section 2.7. In conjunction with a study on the destructive effect of water on structured soils such as loess, it was found that the intensity of disintegration, or destructurement, of loess when in contact with water was greatly affected by sample disturbance. This led the authors to postulate that as the natural structure of a soil develops, the points of contact between particles become less accessible to water due to bonding between the particles. These bonds were postulated to be the result of the formation of silicic acid gels, which coat the particles as a film and act as a cementing agent. To support this idea, the authors cite a study by Grebenshchikov (1937) that showed that such a gel forms at the surface of both natural and artificial silicates due to hydrolysis (the chemical reaction with water).

Denisov and Reltov (1961) developed an experimental program to test the silicic acid gel hypothesis. This involved placing individual sand grains on a vibrating quartz or glass plate and measuring the force necessary to move the grains, as shown in Figure 2.7. The dry grains were allowed to sit on the plate for varying times and then the plate was submerged, also for varying times, before vibrating began. It was found that the force required to move the sand grains continued to increase up to about 15 days of soaking, as shown in Figure 2.8. The authors believed that these results showed experimental evidence of silicic acid gel film formation.

Unfortunately, it is very difficult to interpret the importance of Denisov and Reltov's experiments. The vibrating plate test is not commonly performed on soils, so there is nothing to compare these results with. There is very little description of the test setup and procedure, and the results are not discussed in detail. For example, there was no explanation of the increase in shear force during the 20 hours of dry contact only; it is unlikely that this increase was caused by a precipitation reaction. Furthermore, the results suggest that the aging effects increased significantly simply because of an additional 18 hours of dry contact (from 2 hours to 20 hours). It is not clear how this could have a significant effect on the measured shear force. Therefore, although Denisov and Reltov (1961) presented evidence of possible bonding, it is not convincing evidence that a chemical mechanism is responsible for aging effects in sands.

Mitchell and Solymar (1984) suggested that precipitation of amorphous silica may have been responsible for the large increases in penetration resistance measured at the Jebba Dam project discussed in section 2.6.1. The work of Denisov and Reltov and the high solubility of amorphous silica compared to quartz was cited to support this hypothesis. However, no direct evidence was presented.

Silica may precipitate out of solution at particle contacts whenever the concentration of silica is in excess of its solubility, i.e. supersaturated conditions. Depending on the environment, silica may precipitate into different crystalline forms or as amorphous silica. As mentioned previously, quartz and amorphous silica represent an upper and lower bound in terms of structure and solubility.

Quartz saturation occurs at approximately 11 ppm at 25° C. There are some natural systems where silica concentrations in solution exceed quartz saturation; however, the rate of quartz precipitation is of significance in terms of geologic time only, and not in times relevant to geotechnical engineers.

An example of this can be found in the “locked sands” in portions of England, as reported by Palmer and Barton (1987). These sands in the undisturbed state show a high degree of interlocking and can be sampled as intact blocks. In some cases, the measured relative densities are greater than 100%. It is hypothesized that, over thousands of years, precipitation of quartz has caused infilling of the voids, resulting in increased angularity and decreased porosity of the sand. In these sands, however, there is no evidence that there is bonding between particles. Rather, the infilling resulted in an increased interlocking of the sand matrix.

Thus, for the dissolution and precipitation of silica to be the cause of aging effects in sands, the precipitate would have to be amorphous silica. Given the high solubility of amorphous silica shown in Table 3.2, the concentration of silica in solution would need to be greater than approximately 120 ppm to precipitate as silicic acid gel. As mentioned above, it is unlikely that concentrations this high would be found in natural systems at low temperatures such as those existing in the ground (i.e. 10° C).

More recently, evidence supporting the hypothesis regarding the dissolution and precipitation of soluble fractions was presented by Joshi et al. (1995). Details of their laboratory testing program and penetration test results are shown in section 2.6.14. To complement the penetration tests performed in 36-cm diameter rigid wall cells, small (1.25-cm diameter, 1.90 cm high) samples were submerged, loaded, and allowed to age for two years. After aging, the samples were allowed to air dry and scanning electron micrographs (SEM) were taken of the sand grains. These are shown in Figure 3.12 for two different sands: a local river sand and Beaufort sea sand. The micrographs of the samples submerged in distilled water and seawater, shown in Figure 3.12 (e), (f), (g), and (h), clearly showed the presence of precipitates connecting the sand grains. The micrographs of freshly deposited sands, shown in Figure 3.12 (a) and (b), and sands with dry aging, shown in Figure 3.12 (c) and (d), showed no evidence of precipitates. Using energy dispersive x-ray analysis in conjunction with the SEM, the composition of the precipitates

for the river sand was determined. In distilled water, the precipitates were composed of calcium and possibly silica. For the sample in seawater, the precipitates were composed of calcium, silica, sodium, and chlorine.

The scanning electron micrographs presented by Joshi et al. (1995) appear to show for the first time evidence of some sort of cementation occurring during aging. Not only was precipitate actually observed with an electron microscope in between sand particles of aged specimens, but also the composition of the material matched the soluble fractions of the sand. In the case of the specimens submerged in seawater, the composition of the precipitate also contained dissolved material from the pore fluid (sodium and chlorine).

Unfortunately, there was no analysis of the pore fluid composition to check the amounts of dissolved silica and carbonate minerals in solution. This would have confirmed that supersaturated conditions existed to initiate the observed precipitation. In addition, it is unclear why sodium chloride would precipitate at all given its very high solubility in water.

As a final note, it is interesting that, as discussed in section 2.6.14, increases in the mini-cone penetration resistance were observed in all the aged samples, including those having dry sand. No precipitation was observed in the dry specimens, therefore an additional factor besides a chemical mechanism must be at least partly responsible for the measured increases. It is not known whether another mechanism, such as interlocking, might be responsible, or whether something about the test setup affected the results.

3.5 Other Potential Mechanisms

3.5.1 Introduction

Although the mechanisms proposed by Schmertmann and Mesri and the bonding mechanism are the most cited mechanisms attributed to aging effects, there are other hypotheses

that should be considered. As with the others, there is little or no evidence that these mechanisms are responsible for the observed property changes in sands. The potential mechanisms discussed in this section include blast gas dissipation, biological activity, and pressure solution.

3.5.2 Blast Gas Dissipation

Increases in penetration resistance following blast densification have led some researchers to suggest that the gradual dissipation of blast gases into solution is responsible for aging effects in sands (Dowding and Hryciw 1986). The principal gases that are generated during blasting are carbon dioxide (CO₂), nitrogen (N₂), and water vapor (H₂O) (Persson et al. 1993). In a saturated soil deposit, the water vapor quickly condenses to a liquid. The carbon dioxide is more soluble than the nitrogen gas, and it also goes into solution rather quickly.

There are two possible effects of the blast gases on the sand that might affect the penetration resistance. When the carbon dioxide dissolves in solution, carbonic acid is formed which then dissociates into bicarbonate, as shown below.



where

CO₂ = carbon dioxide

H₂CO₃ = carbonic acid

H⁺ = hydrogen ions

HCO₃⁻ = bicarbonate

The generation of hydrogen ions results in a temporary decrease in pH around the vicinity of the blast. As shown in section 3.4.2, the solubility of silica is independent of pH for

pH values less than 8. The solubility of carbonate, however, increases dramatically below a pH of 7. It is possible that the dissolution of CO_2 will increase the amount of carbonate material in solution in the sand, which will then precipitate out as the pH of the system increases with time.

Another possible effect of blast gases is that the degree of saturation would, for a time, decrease in the vicinity of the blast. The gradual increase in the degree of saturation as blast gases dissipate might be responsible for some of the observed aging effects. Hryciw (1986) investigated this effect to see if increasing degrees of saturation could account for the measured increases in penetration resistance discussed in section 2.6.2. Figure 3.13 shows a plot of a normalized tip resistance versus the degree of saturation of the sand. From about $S = 70\%$ to 100% , the penetration resistance increased approximately 10% . However, this increase was not large enough to account for the increases measured with time that are shown in Figure 2.17.

3.5.3 Biological Activity

Traditionally, it was believed that microorganisms and bacteria existed primarily in the vadose zone in soils, and that there was very little biological activity at greater depths. More recently, however, there has developed an awareness that biological activity in soils below the water table can be just as great as in surficial soils. Researchers have found that certain bacteria found commonly in soils can create cementing agents to glue sand particles together. Bacteria such as *Bacillus pasteurii* and *Sporosarcina ureae* produce, as a waste product, ammonia. This reacts with the surrounding water to form ammonium hydroxide, which then causes carbonate minerals in solution to precipitate. Bacteria such as these are currently being studied as a natural way of filling cracks in stone masonry and statues (Stocks Fischer 1997).

There is also some evidence that other organisms have the ability to cement sand grains. Small invertebrate organisms (approximately 0.05 mm in size) such as gastrotrichs and

turbellarians have been found in some beach sands below the water table. These animals secrete epoxy-like substances, which enable them to stick to sand grains (Morell 1995).

Work done by Martin et al. (1996) has shown in the laboratory that slime forming microorganisms and biopolymers can be used to decrease the hydraulic conductivity and increase the shear strength of soils. As discussed in section 2.5, bacteria such as *Alcaligenes eutrophus* and the biopolymer Xanthan gum were mixed with silty sands, and triaxial tests were performed. The hydraulic conductivity of the samples decreased by a factor of 100 and there was a 50% increase in the shear strength.

Although it is now understood that bacteria and organisms exist at depth in soils and can cause some property changes, there is no direct evidence to show that biological activity may be responsible for aging effects in sands.

3.5.4 Pressure Solution

Pressure solution refers to the dissolution of minerals directly at the contact points between particles. High contact stresses cause a localized increase in solubility, and it has been suggested that the solution around the contacts can become supersaturated. As the dissolved minerals migrate away from the contacts, precipitation will occur. However, pressure solutioning of silica is very difficult to study because, as discussed in section 3.4.2, the rates of dissolution are too slow for practical purposes. In order to observe significant pressure solution in the laboratory, tests must be performed at stresses in excess of 25 MPa and at temperatures greater than 270° (Renton et al. 1969). Neither condition is representative of typical conditions in the field, and it is unlikely that pressure solution is responsible for the measured property increases in sands.

3.6 Summary and Conclusions

From the preceding sections, the main question with regard to the mechanisms responsible for the aging of sands appears to be “is it mechanical or chemical?” Schmertmann

offers evidence that aging in clays causes an increase in the frictional component of strength without affecting the cohesive component of strength. This would suggest a mechanical mechanism, which was attributed to dispersive particle movements, increased interlocking, and internal stress arching. For sands, however, there is no evidence that either the frictional component of strength increases in the same manner as with clays, or that the mechanisms of increased interlocking and internal stress arching even exist. Furthermore, Schmertmann's evidence relies on the results of tests that are not commonly used in geotechnical practice.

Mesri et al. (1990) suggest that aging of sands is due to increased macro and micro interlocking. However, no direct evidence is presented for either mechanism. They present an empirical equation to predict an increase in cone penetration resistance due to aging, which was presented in section 3.3. The equation uses a relationship between the cone penetration resistance and the constrained modulus that is significantly different from other relationships published in the literature. In addition, the empirical constant does not have a firm conceptual basis and is used primarily to fit the model of the data.

If the mechanism is chemical in nature, it appears likely that it is not due to precipitation of silica at the particle contacts. The rate of quartz precipitation is too slow for the time periods of interest, and the amount of amorphous silica that would need to be in solution for precipitation to occur is too high for most natural systems. However, it is possible that other substances, such as calcium carbonate, may precipitate at particle contacts.

It should be noted that if the mechanism is chemical, then aging effects would be a site specific phenomenon, dependent heavily on the chemistry of the soil and the groundwater. In contrast, if a mechanical mechanism is responsible, then the effects of aging on sands would be a more general phenomenon. Recent evidence presented by Joshi et al. (1995), however, suggests that aging effects may be caused by both effects, with me-

chanical mechanisms dominating in dry sands and both chemical and mechanical mechanisms occurring in saturated sands.

Other mechanisms, such as blast gas dissipation and biological activity are interesting hypotheses. They may contribute to aging effects, but they can not fully explain the wide range of conditions where aging effects have been observed. Very little research has been done on the effect of these phenomena on soil behavior.

The state of the art regarding aging effects in sands is that a number of hypotheses have been proposed to account for observed property changes both in the field and in the laboratory. Some tests have been performed, but to date there is no unambiguous evidence as to what causes aging effects in sands, and there are many more questions than answers.

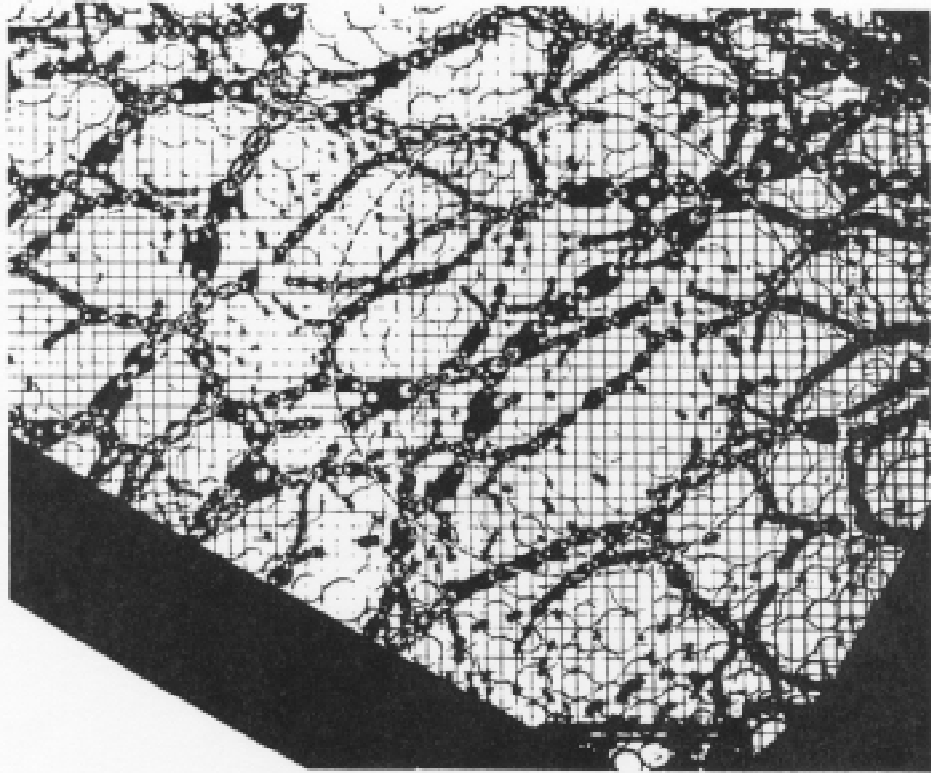


Figure 3.1 Development of load chains in a granular assembly of photoelastic disks
(Drescher and De Josselin De Jong 1972).

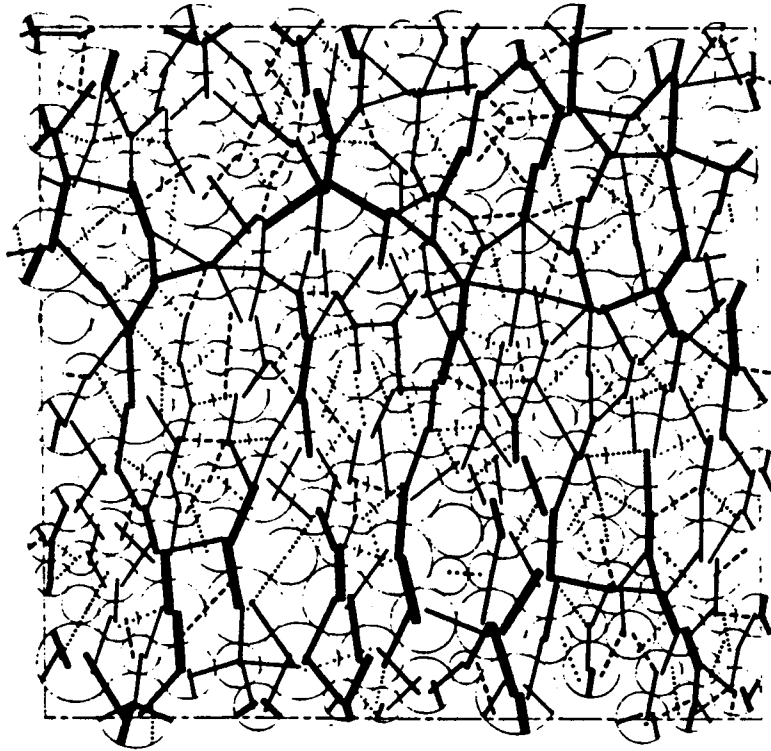


Figure 3.2 Numerical simulation of load chain using discrete element modeling (Kuhn 1987).

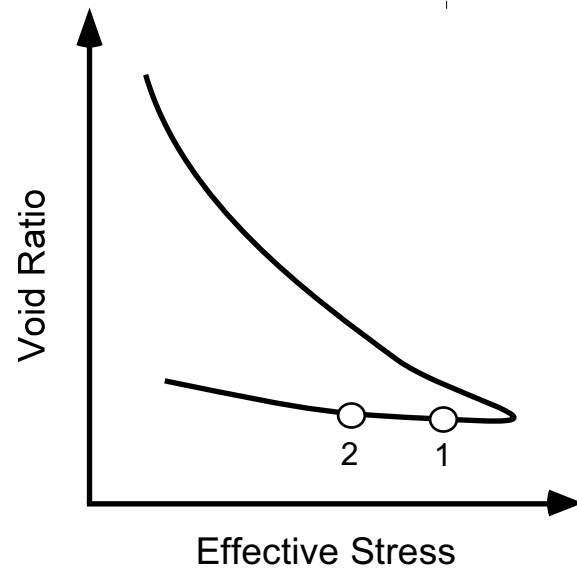


Figure 3.3 Two samples at different stress states but having "identical" structures (after Schmertmann 1991).

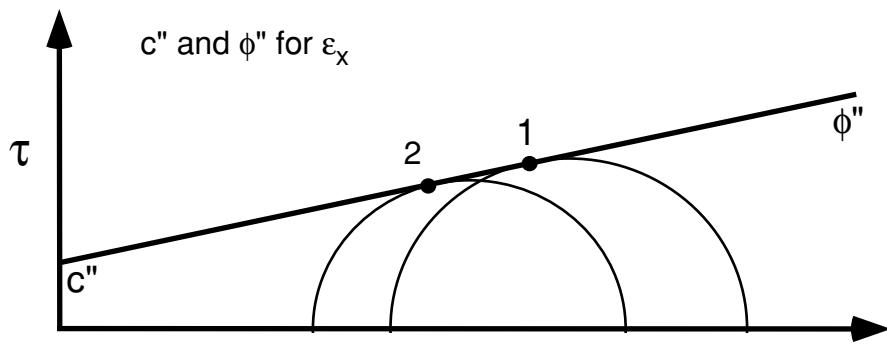
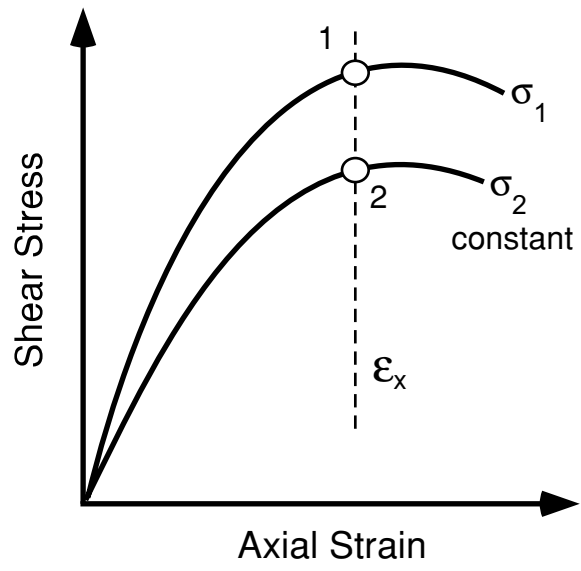


Figure 3.4 Mobilized c'' and ϕ'' for an IDS test during shear (after Scmertmann 1991).

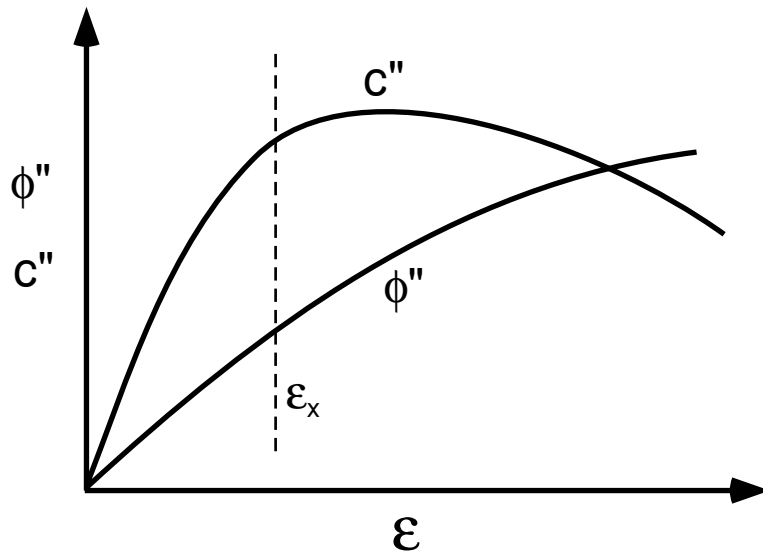


Figure 3.5 Mobilization of c'' and ϕ'' as a function of strain
(after Schmertmann 1991).

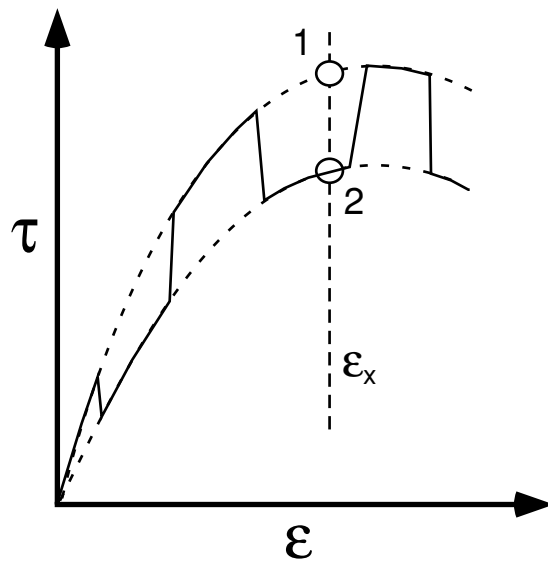


Figure 3.6 Conceptual stress-strain curve for IDS test using one sample.

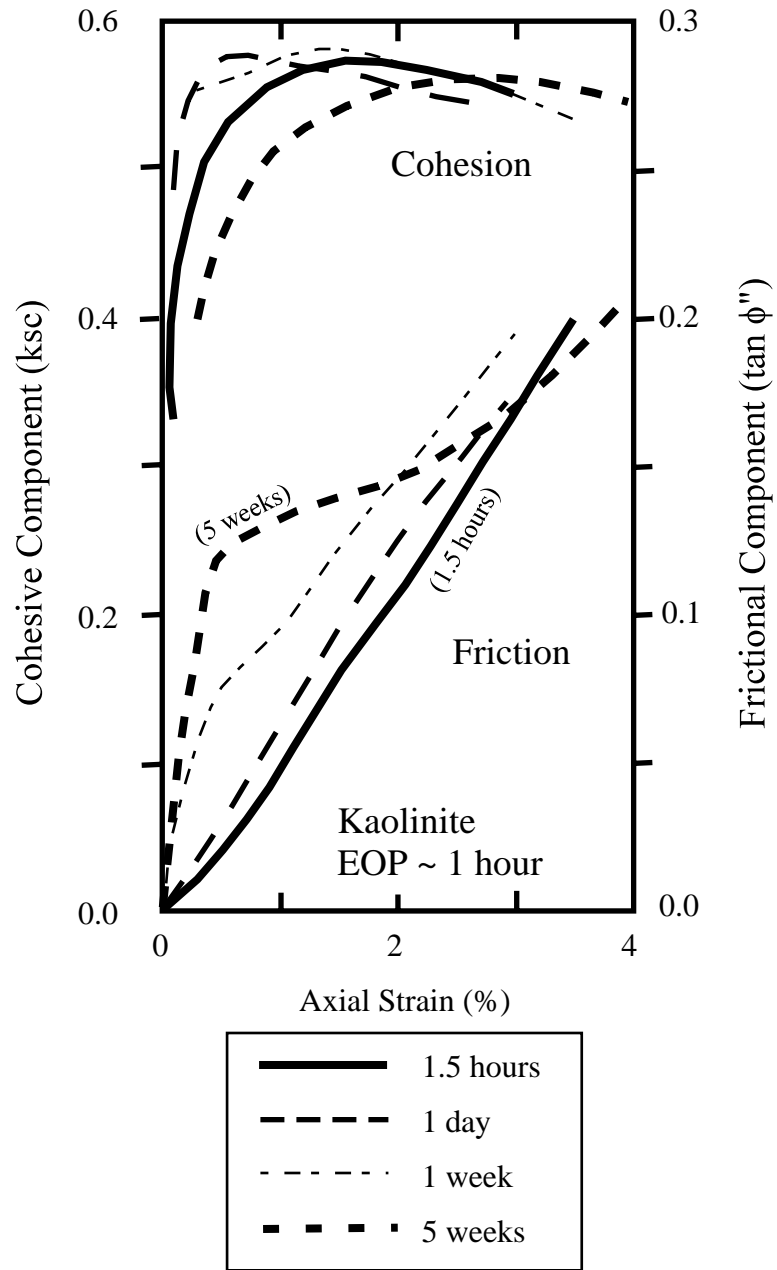


Figure 3.7 Development of the frictional and cohesive components of strength with strain during IDS triaxial tests (after Schmertmann 1991).

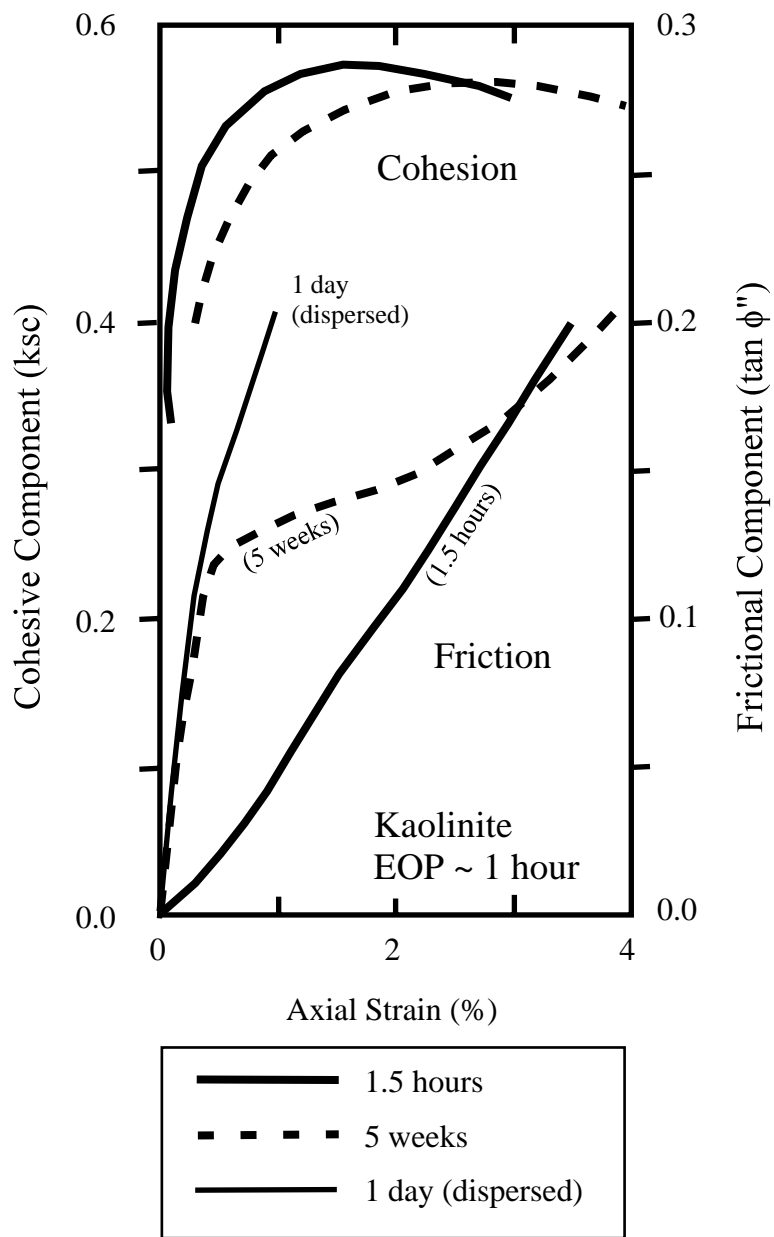


Figure 3.8 IDS test results comparing a sample aged for five weeks to a sample mixed with dispersants and aged for one day (after Schmertmann 1991).

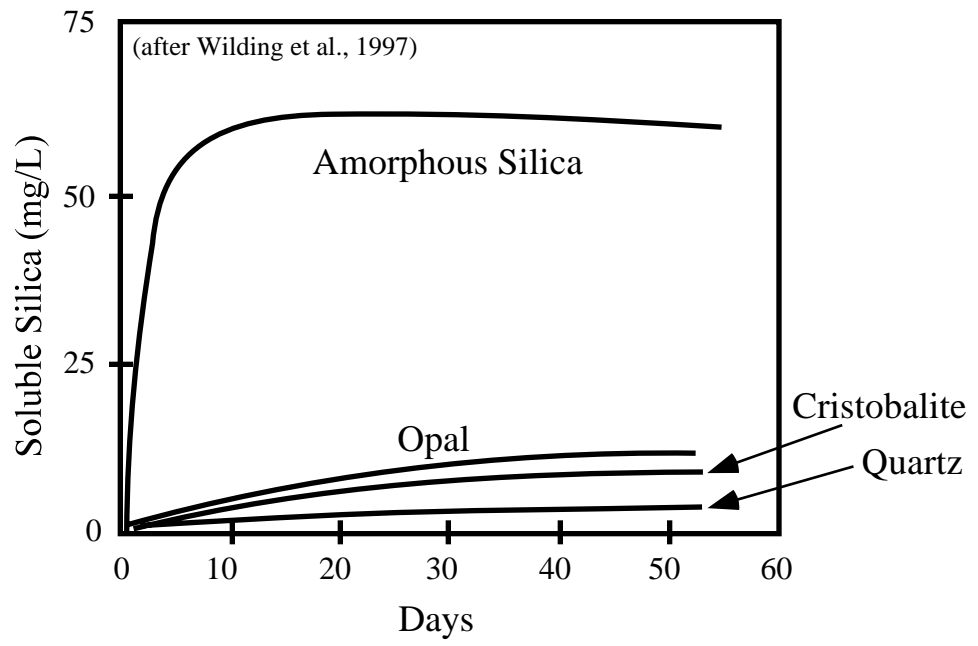


Figure 3.9 Dissolution of silica with time (after Wilding et al. 1977).

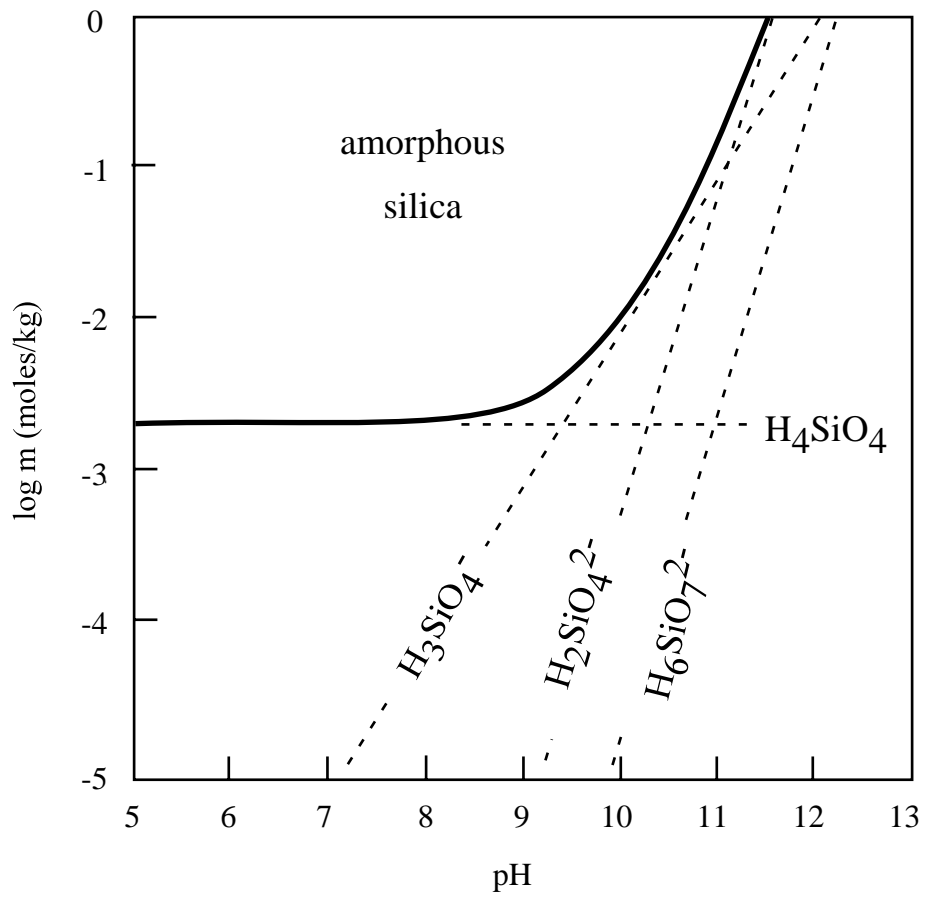


Figure 3.10 The effect of pH on the solubility of silica (after Dove and Rimstidt 1994).

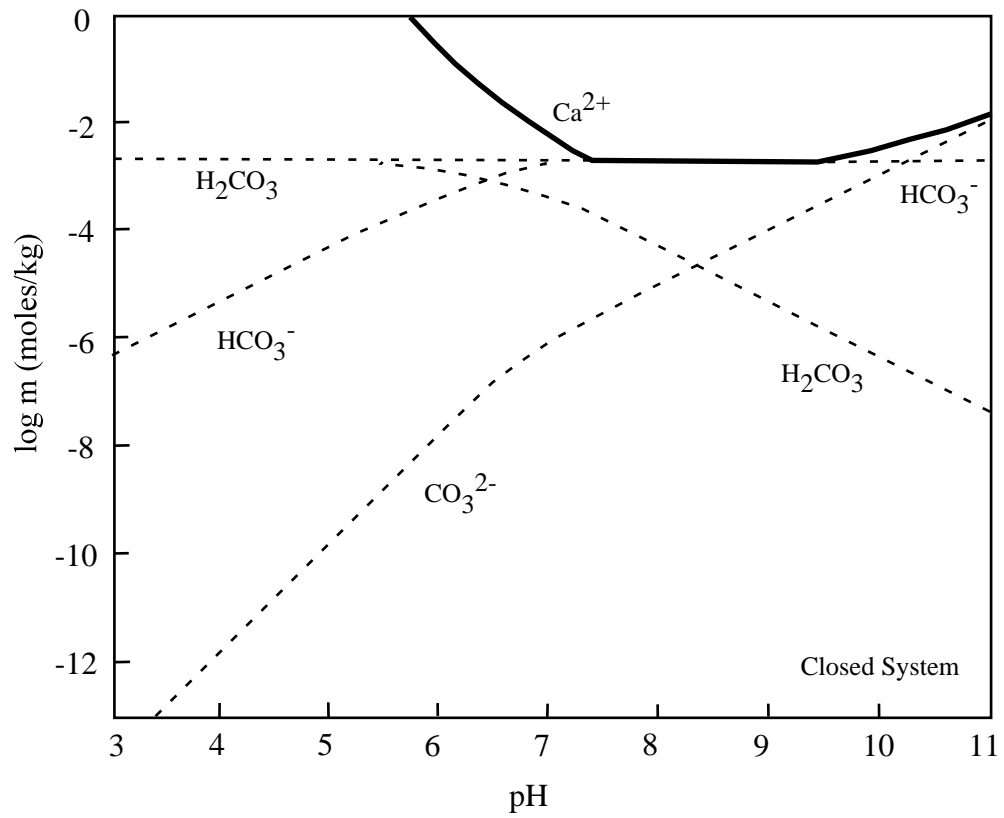


Figure 3.11 The effect of pH on the solubility of calcium carbonate (after Langmuir 1997).

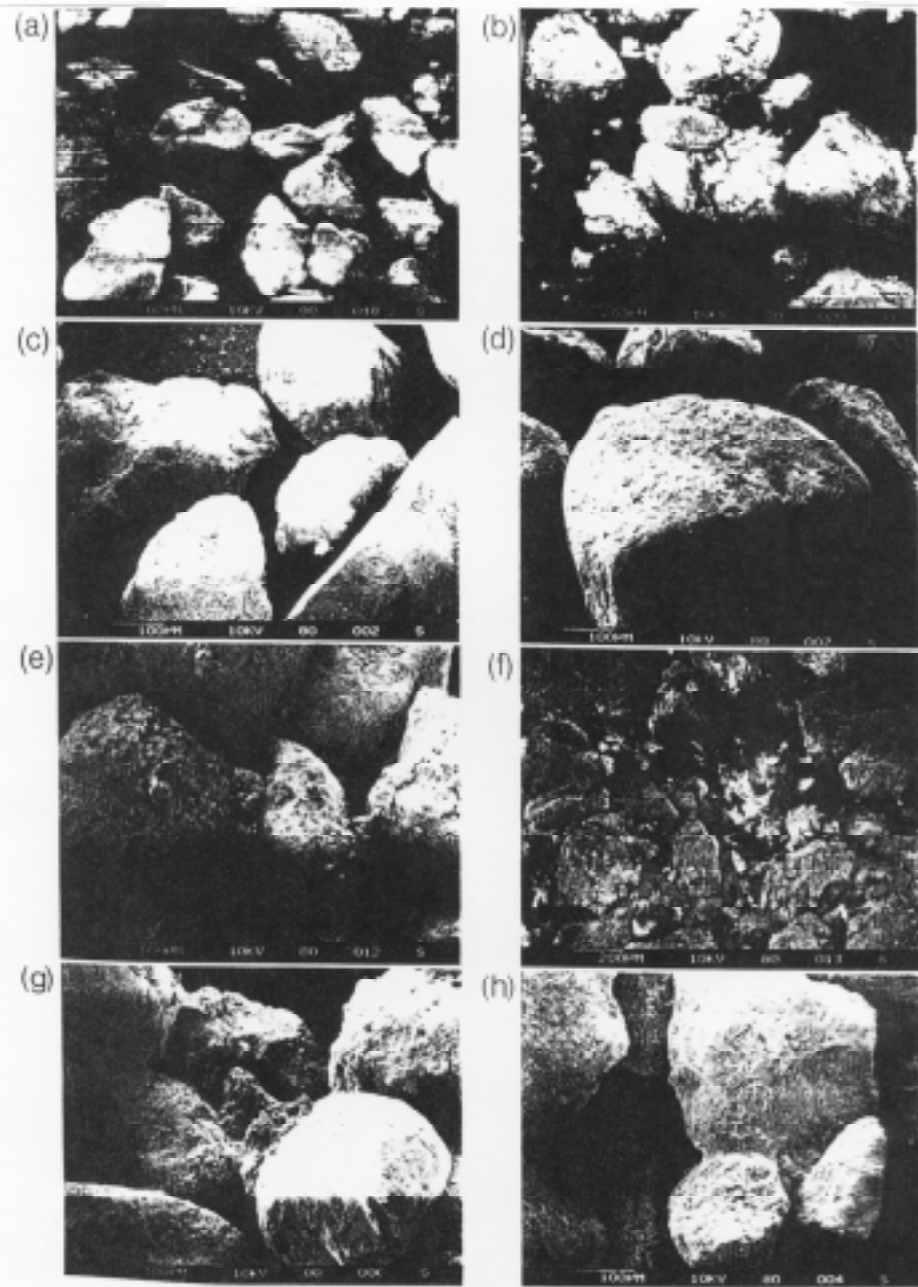


Figure 3.12 Scanning electron micrographs showing the effect of aging on (a) freshly deposited river sand, (b) freshly deposited Beaufort Sea sand, (c) river sand after dry aging, (d) Beaufort Sea sand after dry aging, (e) river sand aged in distilled water, (f) Beaufort Sea sand aged in distilled water, (g) river sand aged in seawater, (h) Beaufort Sea sand aged in seawater (Joshi et al. 1995).

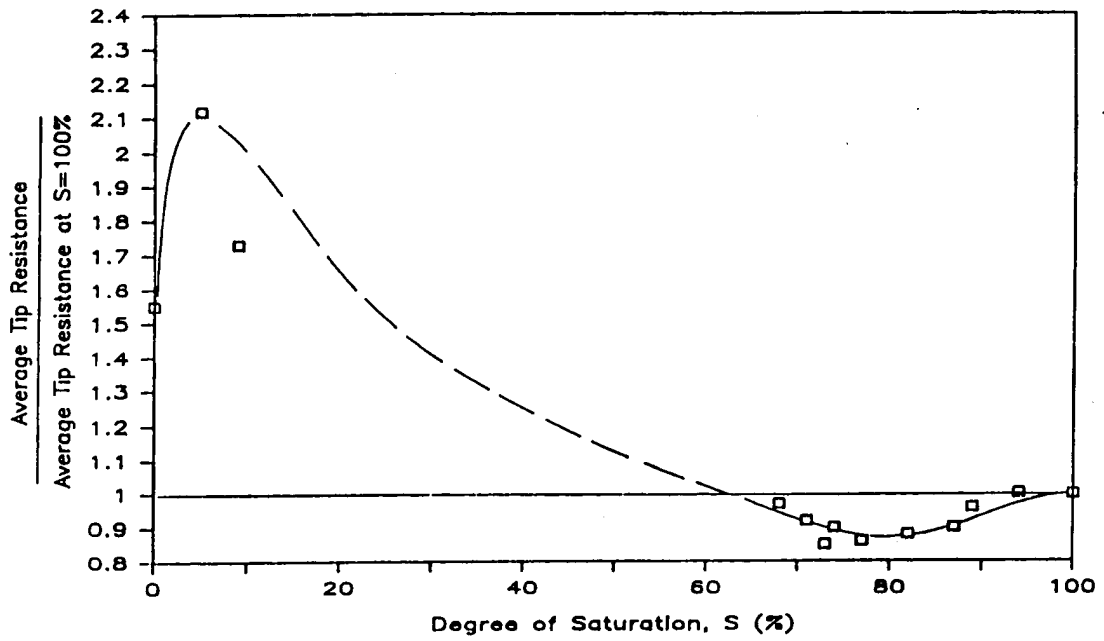


Figure 3.13 Effect of the degree of saturation on cone penetration resistance (Hyrziw 1986).