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MECHANICAL BEHAVIOR OF NON-TEXTBOOK SOILS

(Literature Review)

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16. Abstract

Traditionally, soil mechanics has focused on the behavior of two distinct types of geomaterials: clean sands and pure clays. Under the application of external loads, these two types of geomaterials represent and are conveniently associated with two extreme types of soil responses: drained and undrained behavior. The drained behavior of clean sands and the undrained behavior of pure clays have been covered extensively in most soil mechanics textbooks. In order to provide some insight into the mechanical response of additional materials, a literature review on the mechanical behavior of "non-textbook" soils (i.e. soils other than clean sands and pure clays) was carried out. The non-textbook soils investigated in this study were silty sands, clayey sands, silty clays, sandy clays, sandy silts, and cemented soils. The review focused on the following aspects of their mechanical behavior: (1) response to static loading; (2) response to cyclic loading; (3) compressibility, consolidation and creep behavior; (4) hydraulic conductivity; and (5) additional studies. Static response studies focused on both strength and stiffness properties of non-textbook soils. Investigations on the cyclic response emphasized the liquefaction resistance and, whenever available, the evolution of excess pore-pressure during cyclic loading. Whenever possible, an attempt was made to compile experimental protocols and theoretical frameworks used in the studies cited in the literature review. The literature review indicates that many aspects of the mechanical behavior of non-textbook soils have been studied in a somewhat superficial manner. A summary of the major observations regarding the mechanical behavior of non-textbook soils is presented. Topics meriting future research are identified.

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CHAPTER 1 – INTRODUCTION

Background

Soil mechanics is a new field of study that has developed within the framework of the scientific method starting in the first half of the 20th century (Goodman 1999). Its focus is on the motion of soil (either contained deformation or sliding of one soil mass with respect to another) or the flow of water through soil as a result of the action of loadings (boundary loads applied to soil masses across interfaces with materials such as concrete and steel; hydraulic gradients applied across soil masses by, for example, different water levels upstream and downstream of a dam). The knowledge of soil response to loading has largely been developed for clean sands (i.e., sands without any clays or silts) and pure clays.

There were many reasons why soil mechanics developed around pure clays and clean sands. The possibility of sample recovery from the ground without excessive disturbance (in the case of clays) and of preparation of reasonably homogeneous samples (possible for both clays and sands) figured as one of the most important. Additionally, each represents an extreme in soil behavior. Clays are very impervious; as a result, the application of stress states to clay produces porewater pressure increases in the short term. Only if and after water has had the opportunity to flow under the induced hydraulic gradient field, and the porewater pressures, as a consequence, have dissipated, do the increases in total stresses lead to increases in the effective stresses experienced by the soil skeleton. In sands, only combination of the highest rates of load application (such as those that develop during seismic events) and the smallest sand size particles (in which case the sand is referred to as fine sand) lead to undrained loading conditions. In most cases, drained loading conditions exist.

In the laboratory, drainage out of a sample during loading applications can be completely impeded by closing the drainage lines connecting the inside of a sample to the outside. In practice, most construction operations (whether construction of a highway embankment or a superstructure, such as a bridge or building) take place at a rate such that no significant drainage is possible in clays during load application. Besides, save for rare exceptions (such as excavations in heavily overconsolidated clays), clays consolidate and strengthen with time, so that the period during and immediately after load application is the most critical to stability. It follows that undrained loading is both relevant in the practice of engineering in clays and easily studied. Even the exceptions (such as the aforementioned excavations in overconsolidated clays) can be readily studied, although loading rates in laboratory tests may have to be very slow.

As is true of completely undrained loading, fully drained loading is also easily studied in the laboratory. Keeping drainage lines open produces drained conditions in sands even at relatively fast loading rates. So loading of sands and clays under conditions prevalent in the field has been studied extensively, and the body of knowledge about these two materials is vast and widely available in textbooks and articles (e.g. Bolton 1986; De Josselin de Jong 1976; Pender 1978; Roscoe and Burland 1968; Roscoe and Schofield 1963; Roscoe et al. 1958; Rowe 1962; Schanz and Vermeer 1996).

Cementation is another important factor affecting the behavior of real soils that is still insufficiently understood, even though an increasing number of studies have been carried out on this topic, specially in lightly cemented soils, over the last years (e.g. Gens and Nova 1993; Leroueil and Vaughan 1990). Most of these studies have been backed up by experimental investigations with either naturally or artificially cemented soils (Clough et al. 1989; Consoli et al. 2001; Coop and Atkinson 1993; Cuccovillo and Coop 1999; Huang and Airey 1998; Mitchell 1976b).

Project outline

In the practice of engineering, construction operations in silty or clayey sands and silty or sandy clays (which include both transported and residual soils) and in lightly cemented sands are common. However, the body of knowledge of how these materials respond to loading is rather limited (e.g. Alonso et al. 1990; Georgiannou et al. 1990; Guo and Prakash 1999; Kuerbis and Vaid 1988; Leroueil and Vaughan 1990; Salgado et al. 2000). Usual construction loadings are applied at rates that may create, in such materials, drainage conditions that are neither drained nor undrained. Analysis of foundations, retaining structures or slopes in such materials with parameters obtained from either drained or undrained tests is wrong and leads to either excessively conservative or potentially dangerous designs.

The problem addressed by this research is that of the loading of non-textbook materials. These materials behave differently for one of three reasons: (a) their hydraulic conductivity is not as high as that of sand nor as low as that of clay, leading to drainage during loading that is neither full nor completely impeded; (b) although sand-size, the soil is lightly cemented, and thus behaves differently from uncemented sand and may also have much lower hydraulic conductivity; (c) the degree of saturation is less than 100% but much higher than 0%, and so capillary tension may develop, leading to neither fully drained nor undrained loading conditions. The proposed research focuses primarily on (a) and (b).

The implications of the proposed research effort are twofold: (1) it will open the door for advanced experimental research in soils that, although plentiful in nature, have been poorly studied, and (2) it will raise a number of questions that will require additional research, both to explore behavior that has not been observed in classical soils and to better quantify the properties of non-textbook soils.

Research objectives

The main objective of the proposed research is to carry out a comprehensive literature review on the mechanical behavior of non-ideal soils. The detailed goals of the research are:

- Compilation of state-of-the-art experimental protocols utilized to study the mechanical behavior of non-textbook soils;
- Compilation of the existing theoretical frameworks that have been used to evaluate the mechanical behavior of non-textbook soils;
- Identification of potential flaws and gaps in the existing theoretical frameworks and experimental techniques that may constitute promising areas for future research on non-ideal soils behavior.

CHAPTER 2 – SILTY AND CLAYEY SANDS

Studies of sands containing fines have been triggered mostly by earthquake-related hazards that have occurred over the last two decades. Many liquefaction and other problems originated by seismic loadings have been associated with sand deposits containing some amount of fines.

According to the Unified Soil Classificaton System USCS (ASTM D 2487), silty and clayey sands are soils that contain at least 50% of particles larger than 4.75 mm and more than 12% particles smaller than 75 μm, by weight. Depending on the plasticity characteristics of the fraction smaller than 75 μm, a sand is classified as Silty Sand (SM) if the fines classify as Silt (ML) or Elastic Silt (MH), or as Clayey Sand (SC) if the fines classify as Lean Clay (CL) or Fat Clay (CH). Sands with 5 to 12% fines require dual symbols based on the USCS. In order to simplify the terminology used in this text, sands containing more than 5% fines (up to 50%) will be named either silty or clayey sands depending upon the fines plasticity characterisitics as defined above.

Terminology aside, there are three basic differences between these soils and clean sands. First, a careful choice of how to quantify density is required for a logical, consistent treatment of liquefaction of these soils. Secondly, analyses based exclusively on density and stress state are not sufficient for these materials, as the soil fabric (and thus specimen preparation) is a key determinant of their behavior (Leroueil and Vaughan 1990; Mitchell 1993). Lastly, not only the content of fines, but also their plasticity, needs to be properly accounted for. These differences are discussed in the sections below.

Strength and stress-strain response

The choice of an appropriate parameter to quantify soil density is an issue that has consistently arised in studies of the mechanical response of sands containing fines. This issue has been examined in more detail by few researchers (Kuerbis and Vaid 1988; Mitchell 1993; Salgado et al. 2000; Thevanayagam et al. 2002). The most common parameters in studies of the mechanical reponse of sands with fines are: (a) global void ratio e; (b) relative density D_R; and (c) intergranular void ratio e_G (Mitchell 1976a; Mitchell 1993) or skeleton void ratio e_{sk} (Kuerbis et al. 1988; Shen et al. 1977)[1].

Contradictory conclusions in early studies regarding the effect of fines on the shear strength and resistance to cyclic loading of sands can be attributed to the inappropriate selection of a soil density parameter for comparisons and analyses (Carraro et al. 2003; Polito and Martin 2003; Thevanayagam et al. 2002). There seems to be no consensus yet as to which parameter would be the most suitable to model the behavior of these materials. Sands containing fines can be deceptive in that they may have low global void ratios while having high susceptibility to undrained collapse (Thevanayagam et al. 2002).

Experimental evidence suggests that if global void ratio is used as the basis for comparison, sands containing nonplastic and/or low plasticity fines may show similar or decreasing shear strength with increasing fines content FC. If the skeleton void ratio is used instead, shear strength is relatively independent of FC (Finn et al. 1994; Thevanayagam 1998; Thevanayagam et al. 2002; Vaid 1994). These observations usually hold for fines contents less than the threshold fines content FC_{th}, usually between 25 and

[1] For silt-sand or clay-sand mixtures, e_G is calculated by assuming the volume occupied by the finer particles as voids; e_{sk} is a special case of e_G in which the finer and coarser particles have identical specific gravities.

35%, below which non-floating soil fabrics prevail (Salgado et al. 2000) and the soil behavior is mostly controlled by the coarser particles in an idealized two-sized particle packing (Thevanayagam et al. 2002). Figures 1 and 2 show schematic representations of non-floating and floating fabrics, respectively, for binary mixtures of spheres.

It is widely recognized that the method of specimen preparation strongly affects the stress-strain response of granular soils reconstituted in the laboratory. In the case of sands containing fines this effect is even more pronounced. The most widely used reconstitution techniques for sands with fines are: (1) moist tamping (MT); (2) air (or dry) pluviation (AP); (3) water (or wet) pluviation (WP) or slurry deposition (SD). Figure 3 shows the typical undrained triaxial stress-strain response of loose specimens of sand (with or without fines) at identical states for these three methods of specimen preparation.

Loose, saturated MT specimens of sands containing fines are typically the most contractive and show strain-softening response under undrained monotonic triaxial compression (Thevanayagam et al. 2002; Vaid et al. 1999). Vaid (1994) argued that the MT technique neither simulates the fabric of alluvial soil deposits nor guarantees specimen uniformity. Homogeneity of MT specimens is tacitly assumed, but limited direct evidence supporting the assumption is provided (Castro 1969; Mulilis et al. 1977; Vaid et al. 1999).

Vaid (1994) reasoned that, considering that the critical state line for certain sands has a very flat slope in the e - p' space (where p' is the mean effective stress), the sensitivity of undrained shear strength s_u to void ratio changes is high and a small error in e can result in a large variation in s_u. Hence, test results based on non-uniform MT specimens are of questionable validity. Furthermore, this technique enables specimen reconstitution at void ratios that may be too high and thus not even accessible to soil deposited under water (Kuerbis 1989; Vaid 1994). These observations suggest MT specimens may not be suitable for use in laboratory element testing if the goal is to simulate the response of liquefiable sands containing fines, such as found in alluvial deposits, hydraulic fills, tailings dams, and off-shore marine deposits.

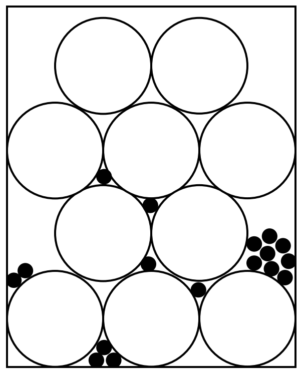

Figure 1 – Non-floating fabric

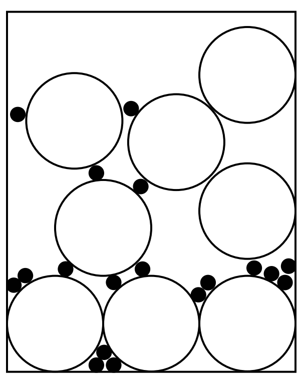

Figure 2 – Floating[2] fabric.

[2] A "floating" fabric is one in which, on average, the sand particles are separated by the fines (Salgado et al. 2000)

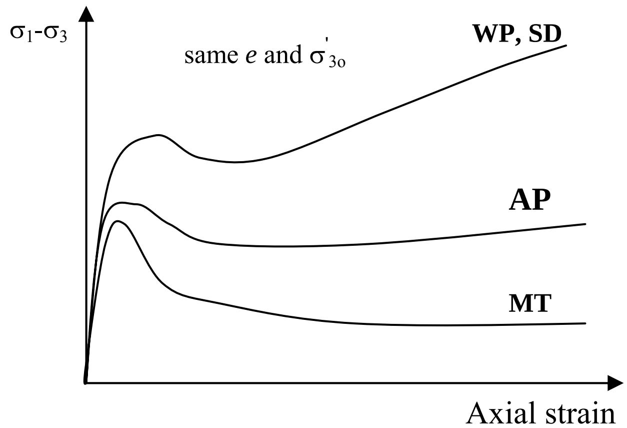

Figure 3 – Undrained stress-strain responses of sand specimens prepared by Moist-Tamping, Air Pluviation, and Water Pluviation/Slurry Deposition techniques (Modified after Vaid et al. 1999).

The fabric of loose specimens of silty sands obtained by AP has been shown to be highly contractive (Lade and Yamamuro 1997; Yamamuro and Covert 2001; Zlatovic and Ishihara 1997). Lade and Yamamuro (1997) attributed the highly contractive behavior of AP silty sands to the floating fabric obtained with their method of specimen preparation. Zlatović and Ishihara (1997) corroborated the conclusions of Lade and Yamamuro (1997) regarding the effect of fabric. Yamamuro and Covert (2001) performed drained and undrained monotonic triaxial tests on very loose specimens of Nevada sand with 40% silt. Their specimens resulted highly contractive, even at large axial strains.

Homogeneous specimens of uniform sand can be prepared by either air or water pluviation. Vaid and Thomas (1995) showed that the static response of WP sand specimens is dilative in triaxial compression, even in the loosest deposited state (as shown in Fig. 3). WP is preferable for it produces initially saturated specimens that are easy to replicate (Vaid and Negussey 1988) and have fabric and behavior similar to that of natural alluvial soils (Oda et al. 1978), but it cannot be used for sands containing fines because of particle segregation (Kuerbis and Vaid 1988).

Slurry deposition (SD) methods have been proposed to avoid segregation of the fines but still obtain the same type of fabric and stress-strain response as WP. Ishihara et al. (1978) presented a SD technique for silty sand and sandy silt but their specimens did not result very homogenous for *FC* between 30 and 80%. Kuerbis and Vaid (1988) developed a SD method that produces homogenous specimens (e.g. Emery et al. 1973) of well-graded sands and silty sands. Kuerbis et al. (1988) showed that SD silty sands prepared at similar states may be dilative in triaxial compression but contractive in extension. The observed response of their SD specimens was similar to that of WP sand specimens and natural alluvial sand specimens (e.g. Miura and Toki 1982).

Vaid et al. (1999) studied the undrained static response of sands using different specimen reconstitution techniques and showed that, at identical soil states (i.e., same initial void ratio and effective stress), the MT clean and silty sands are capable of developing classical flow liquefaction response but WP specimens are not (Fig. 3). Similar observations were also reported by Hoeg et al. (2000). Additionally, WP specimens are very uniform, in contrast with the non-uniformity that is usually observed in MT (Castro 1969; Mulilis et al. 1977). A direct comparison of the behavior of truly undisturbed clean and silty sand specimens retrieved by in-situ ground freezing and their corresponding reconstituted counterparts after consolidation to identical initial states was also made by Vaid et al. (1999). Their results showed that the fabric generated by WP closely simulates that of natural alluvial and hydraulic fill sand. Hoeg et al. (2000), who noted dramatic differences in undrained stress-strain-strength behavior between “undisturbed” and MT silt and silty sand specimens tested at the same soil state, agreed that water/slurry deposition is the specimen preparation method that best simulates the in-situ fabric of most liquefiable silty sand soils (alluvial deposits, hydraulic fills, tailing dams, etc.).

Salgado et al. (2000) investigated the effects of nonplastic fines on the small-strain stiffness and shear strength of sands. It was observed that the small-strain stiffness at a given relative density and confining stress level decreases dramatically with the addition of even small percentages of silt. Also, the addition of even small percentages of silt to clean sand considerably increased both the peak and the critical-state friction angles at a given initial relative density and stress state.

Experimental investigations of sands containing clay have been very few. Georgiannou et al. (1990) showed that, at the same e_G, the amount of strain softening and the strain to the phase transformation state in monotonic triaxial compression loading increased as the clay content of slurry-deposited, anisotropically consolidated clayey sands increased from 4.6 to 10%. Similar behavior was also observed for MT specimens (Chu and Leong 2002; Ovando-Shelley and Pérez 1997). On the other hand, Pitman et al. (1994) noted a decrease in undrained brittleness of MT kaolin-sand specimens with increasing kaolin content up to 40%. Their results also showed that variations of the host sand gradation appeared to have little effect on the monotonic undrained behavior of the mixtures at a given fines content. In extension, Georgiannou et al. (1990) observed that flow liquefaction takes place for clay contents up to 7.5%. Wood and Kumar (2000) showed that the stress-strain response of clayey sand specimens in drained and undrained triaxial compression tests remains unchanged for clay contents higher than 40%.

The undrained behavior under monotonic loading of anisotropically consolidated specimens of natural clayey sand reconstituted by SD was studied by Georgiannou et al.

(1991b). Their results showed that the peak resistance of the anisotropically consolidated clayey sands was mobilized at axial strains of about 0.1-0.3% in compression and 0.5% in extension.

Response to cyclic loading

As observed previously for monotonic loading, ample experimental evidence suggests that if global void ratio is used as the basis for comparison, sands containing nonplastic and/or low plasticity fines may show similar or decreasing cyclic resistance with increasing fines content *FC*. If the skeleton void ratio is used instead, cyclic resistance is also relatively independent of *FC* (e.g. Carraro et al. 2003; Chang 1987; Erten and Maher 1995; Kaufman 1981; Koester 1994; Law and Ling 1992; Polito and Martin 2001; Shen et al. 1977; Singh 1994; Troncoso and Verdugo 1985; Uyeno 1976; Vaid 1994). As noted before, these observations usually hold for fines contents less than the threshold fines content FC_{th}, below which non-floating soil fabrics prevail.

An overwhelming amount of evidence from earthquake-related case histories suggests that liquefaction-related hazards are strongly correlated with loose deposits of saturated sands containing fines, formed under aquatic modes of deposition such as alluvial soil deposits, hydraulic fills, and tailings dams (Adalier and Aydingun 2000; Boulanger et al. 1997; Cetin et al. 2002; Chang 1987; Holzer et al. 1999; Ishihara 1997; Ni and Lai 2001; Sancio et al. 2002; Seed and Harder 1990; Yoshida et al. 2001). Thus, the selected specimen reconstitution method, as discussed previously, should replicate the actual fabric of these materials as closely as possible.

Yamamuro and Covert (2001) performed undrained cyclic triaxial tests on very loose specimens of Nevada sand with 40% silt and observed that the cyclic resistance of such collapsible soils becomes notably small, even when they are partially saturated (e.g. Ishihara and Harada 1994).

Ishihara and Koseki (1989) pointed out that the liquefaction resistance of silty sands depends in an important way on the plasticity index *PI* of the fines fraction. They suggested that the liquefaction resistance of these sands increased only when the *PI* of the fines exceeded 10. Ishihara (1993) attributed the greater resistance to liquefaction of sands containing plastic silt to silt cohesion. In contrast, he observed, silty sands associated with tailings dams can be as liquefiable as clean sands due to the nonplastic nature of the fines (Ishihara 1985). Reporting data from earthquakes in China and Japan, Chang (1987) showed that soils containing either plastic or nonplastic fines are not exempt from liquefaction. This statement is supported by the numerous cases of liquefaction in sands containing both plastic and nonplastic fines observed in the recent Kobe, Japan (1995), and Adapazari, Turkey (1999) earthquakes, which led to shallow foundation failures, lateral spreading and landslides (e.g. Ishihara 1997; Yoshida et al. 2001).

Georgiannou et al. (1991a) studied the response of SD normally consolidated clayey sands to both cyclic and monotonic triaxial loading and concluded that the effective stress paths of the specimens loaded monotonically in triaxial compression and extension form a bounding envelope to the cyclic response in the p'-q space. Kuwano et al. (1995) showed that the cyclic resistance of reconstituted clayey sand specimens correlated well with their undrained shear strength obtained in triaxial extension, irrespective of the clay content and method of specimen preparation.

Since the earlier liquefaction studies originally developed for clean sands in the 1960's and 1970's, the profession has relied on semi-empirical SPT- and CPT-based liquefaction potential assessment curves such as the one presented in Figure 4. These curves have been commonly referred to the mean grain size D_{50} and the fines content FC. Regrettably, the available CPT database usually does not provide direct information about the plasticity of the fines present in the deposits tested in the field (e.g. Robertson and Campanella 1985; Seed and De Alba 1986; Shibata and Teparaksa 1988; Stark and Olson 1995); consequently, sands containing plastic and nonplastic fines were typically lumped together. The empirical CPT-based methods suggest that the presence of fines would increase the liquefaction resistance of sands for the same value of penetration resistance (Robertson and Wride 1998; Seed and De Alba 1986; Stark and Olson 1995; Youd et al. 2001), giving a sense of confidence that such materials would be unlikely to liquefy; however, extensive liquefaction was observed in deposits and structures containing these materials during the San Fernando (1971), Tangshan (1976), Chile (1985), Loma Prieta (1989), Kobe (1995), Kocaeli (1999) and Chi-Chi (1999) earthquakes. During those events, most of the liquefaction-related problems have been associated with the presence of sand layers containing various amounts of silt and clay.

Robertson and Wride (1998) recognized that soils containing nonplastic silt ($PI < 5$) should have lower CRR values than soils with plastic fines for a given FC, but their curves are still too high according to Seed et al. (2003). Seed et al. (2003) observed that the empirical CPT-based curves for sands containing fines proposed by Robertson and Wride (1997) provided values of liquefaction resistance that are likely to be unconservative. Based on a probabilistic analysis of existing data, Seed et al (2003) proposed new CRR-q_{c1} curves that are significantly to the right of (below) the Robertson and Wride (1997) curves.

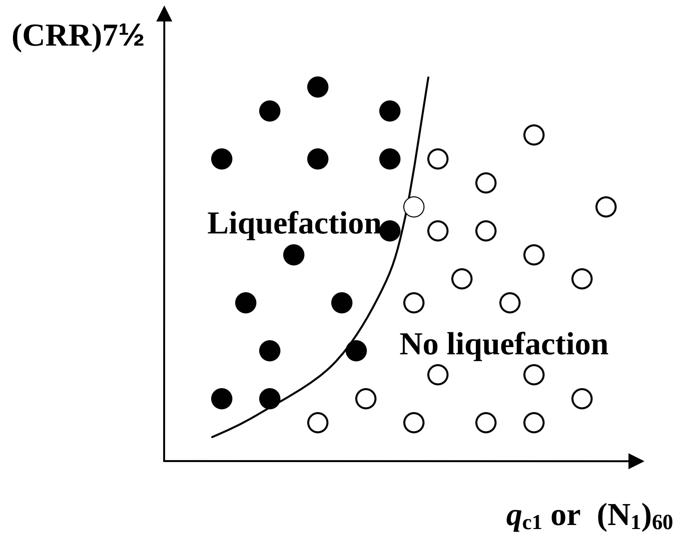

Figure 4 – Semi-empirical approach to evaluate liquefaction resistance.

Carraro et al. (2003) developed $(CRR)_{7.5}$-q_{c1} curves for both clean and nonplastic silty sands by combining a cone penetration resistance analysis and laboratory testing. The curve for clean sand was shown to be consistent with widely accepted empirical relationships; the curves proposed for both clean and silty sands were shown to be consistent with available field observations. The $(CRR)_{7.5}$-q_{c1} relationships for the silty sands depend on the relative effect of silt content on $(CRR)_{7.5}$ and q_{c1}. The cone resistance was shown to increase at a higher rate than liquefaction resistance with increasing silt content for the silty sand studied by these authors, shifting the $(CRR)_{7.5}$-q_{c1} curves to the right.

Compressibility, collapse, and creep behavior

Estimates of the coefficient of volumetric compressibility m_v and coefficient of consolidation C_v of sands with nonplastic fines have been presented by Thevanayagam et al. (2001). Carraro et al.(2003) used theoretical correlations between C_v and the hydraulic conductivity K to obtain estimates for the normalized cone penetration rate of sands containing up to 15% nonplastic fines.

The compressibility behavior in one-dimensional compression of a residual clayey sand derived from sandstone was investigated by Martins et al. (2001). Their results showed that no unique normal compression line could be identified for the soil they tested. Although the compressibility on first loading is similar to that for many sandy materials at states on their normal compression lines, the locations of the compression curves were found to be a function of the initial void ratio, and there was no convergence at higher stresses. Tests on a model clay-sand mixture indicated that this behavior is common to gap-graded clayey sands.

Hydraulic conductivity

The effect of plastic and non-plastic fines on the hydraulic conductivity of sands has been studied by a few authors. Carraro et al. (2003) used flexible-wall permeability tests to estimate the hydraulic conductivity K of silty sands containing up to 15% nonplastic fines. Other estimates of K were presented by Thevanayagam et al.(2001) for silty sands and by Sällfors and Öberg-Högsta (2002) for clayey sand soils.

Additional studies

Not available.

CHAPTER 3 – SILTY CLAYS

In the Unified Soil Classificaton System USCS, silty clays are classified as fine-grained soils with 50% or more passing through No.200 sieve (75 μm), inorganic, with *LL*<50, 4<*PI*<7 and plots on or above the “*A*” line (ASTM D 2487). Their basic engineering properties are discussed below.

Strength and stress-strain response

The variation of the undrained shear strength with depth of silty clays was firstly investigated using in situ vane tests and laboratory quick shear tests on samples of a Norwegian quick clay (Brinch-Hansen 1950). A hand-held vane device was used to determine the undrained shear strength of two residual silty clays of the Papua New Guinea highlands by Wallace (1973). This author also used shear box tests to characterize the stress-strain response of those soils.

Triaxial test results on silty clay specimens performed by Kjaernsli and Simons (1962) showed that the same strength envelope was obtained from consolidated drained triaxial tests, regardless the stress-path followed during shearing. This envelope is only very slightly higher than that obtained from consolidated undrained triaxial tests expressed in terms of effective stress. The authors investigated the stability of the natural soft silty clay slopes of the north bank of the Drammen River and showed that the effective stress analysis is the correct approach to long-term stability problems of those slopes. O'Reilly et al.(1989) carried out undrained triaxial tests at a range of strain rates on specimens of an anisotropically normally consolidated silty clay. Their results were used to formulate a simple model based on critical state soil mechanics concepts. In this model, the strain rate dependency is accounted for by introducing a viscous yield locus which exists simultaneously with the static yield locus. Monotonic strain-controlled undrained triaxial tests were also performed on reconstituted samples of a silty clay by Hyde and Ward (1985) and by Brown et al. (1975) to study the loading response of various overconsolidated silty clays.

A detailed study of the relationship between undrained shear strength and consolidation pressure of a glacial silty clay in the compression and extension shear modes was performed by Mesri and Ali (1999). In the compression range, for each consolidation-shear mode, the ratio of undrained shear strength to vertical consolidation pressure ratio, the ratio of shearing-induced pore water pressure to vertical consolidation pressure ratio, the friction angle mobilized at yield and the coefficient of earth pressure at rest were all constant and independent of consolidation pressure.

Response to cyclic loading

An investigation of the response of various overconsolidated silty clays to stresses applied by repeated load triaxial tests was carried out by Brown et al. (1975). Pore-pressures built up to constant positive values for lightly overconsolidated samples within about 104 cycles. For heavily overconsolidated samples, however, negative values were still building up in some cases even after 106 cycles.

Low frequency undrained cyclic triaxial tests were carried out on reconstituted samples of a silty clay by Hyde and Ward (1985). Their tests were carried out at overconsolidation ratios of 1, 4, 10 and 20, and the data were used to establish the stable state boundary surface. Results from repeated load tests on samples with overconsolidation ratios of 1 and 4 were compared with this boundary surface. Previous investigations have shown that the dry side of the state boundary surface for clays, sometimes known as the Hvorslev surface, forms an empirically determined stability criterion for cyclic effective stress paths of samples of all stress histories. Normally consolidated and lightly overconsolidated soils subjected to undrained cyclic loading become heavily overconsolidated, the movement of the effective stress path being governed by the build-up of porewater pressure. The rate of pore pressure development was shown to be a function of the number of cycles, the applied stress and the sample stress history. The combination of the stability criterion with the pore pressure relationship allows the prediction of the number of cycles to failure in terms of fundamental critical state constants and two further easily determined pore pressure parameters.

Guo and Prakash (1999) pointed out the lack of guidelines for evaluating the liquefaction potential during an earthquake of low plasticity silts and silty clays, materials commonly encountered in the central United States. The liquefaction behavior of these soils is not properly understood at present and is often confused with that of sand-silt mixtures. Porewater pressures and liquefaction in silt and silt-clay mixtures are discussed by the authors, and the influences of plasticity index on their cyclic strength are reviewed. The authors observed that considerable additional work is needed to fully understand the liquefaction behavior of these soils.

Compressibility, collapse, and creep behavior

The compressibility characteristics of two residual silty clays were determined using oedometer tests by Wallace (1973). The behavior of the soils was interpreted in terms of an idealized model of the soil structure. The soil is considered to consist of a coarse open skeleton of cemented rock mineral and aggregated clay particles surrounded by a viscous gel. At low applied stresses, when the cemented bonds in the soil skeleton are intact, it is the soil skeleton that determines structural behavior. At high applied stresses, the volumetric strain is directly proportional to the flexibility of the soil skeleton.

Oedometer and isotropic compression test results on partially saturated and fully saturated soils, including a silty clay, were presented by Jennings and Burland (1962). Their results indicate that the silty clay studied exhibit behavior which cannot be accounted for purely on the basis of effective stress changes below a critical degree of saturation as high as 90%.

Stress-controlled triaxial test data on the creep behavior of a coastal organic silty clay under undrained conditions at different isotropic consolidation pressures and stress levels were reported by Arulanandan et al. (1971). Their results were interpreted according to the critical state framework. Pore pressures developed during creep were shown to be due to the secondary compression effect. The magnitude of porewater pressure build-up during secondary compression was shown to be time and structure dependent. The behavior of a silty clay under creep loading has been shown to be very similar to that under repeated loading by Brown et al. (1975).

Hyde and Brown (1976) carried out a series of triaxial tests on reconstituted samples of a silty clay, using both creep and repeated loading regimes, in order to quantify the similarity in behavior under both loading modes in terms of the relationship between stress and plastic strain. The development of plastic strain and mean pore-pressure in both test modes was shown to be similar. The results were expressed in terms of plastic strain rate as a function of time, applied deviator stress and stress history. The sinusoidal stress pulse used in the tests was approximated by a step function which allowed the creep test results to be used to successfully predict the behavior under repeated loading.

Rao and Reddy (1998) studied the influence of physico-chemical factors on the swell/collapse characteristics of laboratory-prepared dry silty clay specimens. The compression behavior of the saturated silty clay specimens was dominated by the physico-chemical effects at all the experimental pressures. Comparatively, the physico-chemical effects dominated the volume change behavior of the dry silty clay specimens on wetting only at the lower stresses, whereas the mechanical effects prevailed at the higher stresses.

Hydraulic conductivity

Olsen et al. (1985) conducted permeability measurements with the flow-pump method on specimens of silty clays and other soils in a conventional triaxial system by introducing and withdrawing water at known constant flow rates into the base of a specimen with a flow-pump. The results showed that previously reported advantages of the flow-pump method, compared with conventional constant head and falling head methods, were realized for permeability measurements in conventional triaxial equipment. These advantages are that direct flow rate measurements are avoided together with the associated errors that arise from the effects of contaminants on capillary menisci and the long periods of time involved in flow rate measurements; permeability measurements can be obtained much more rapidly and at substantially smaller gradients; errors from the small intercept in the otherwise linear flow rate versus hydraulic gradient relationship, and also from seepage-induced permeability changes, can easily be recognized and avoided or minimized.

The hydraulic conductivity of a estuarine silty clay was studied by Leroueil et al. (1992). The in situ hydraulic conductivity was determined using pushed-in-place piezometers, self-boring permeameters and the BAT system. In the laboratory, oedometer cells, triaxial cells and a radial flow cell were used to determine the vertical and horizontal hydraulic conductivities and their variation with vertical compression. The results show a size effect and an anisotropy ratio varying with the fabric of the soil, but typically between 1.5 and 2. They also indicate that the pushed-in-place piezometers and the BAT system underestimate the hydraulic conductivity. The most representative profile seems to be given by the self-boring permeameter.

Hydraulic conductivity of thawed consolidated slurries of a silty clay from Lachute, Quebec, Canada, subjected to closed-system freezing at different temperatures ranging from –2 to –12°C were determined from constant-head permeability tests by Konrad and Samson (2000). The permeability index defined as the slope of the relation between log K and e was found to increase with decreasing temperature. It was also established that the ultimate permeability index was related to the temperature at which no further change in unfrozen water content occurs. For the silty clay studied, the permeability index increased from 1.4 for the unfrozen soil prior to freezing to a maximum value of 8 at a temperature of –12°C.

Rinaldi and Cuestas (2002) performed hydraulic conductivity tests on compacted specimens of a loess silty clay and determined the relationship between its electrical and hydraulic conductivities. Electrical conductivity was measured with a two electrode cell. It was concluded that the ohmic conductivity of compacted specimens depends mainly on the salt concentration in the pore fluid, and volumetric water content. The effect of compaction density was observed to be less significant.

Additional studies

Findings from a wide range of in situ tests and laboratory investigations on a very uniform deposit of soft silty clay were presented by Nash et al. (1992). The authors noted that, in contrast with many other soft clays, the material studied had high effective strength parameters and high undrained strength.

A trial using dredged clay as a reclamation material in the Tianjin Port East Pier project in China is described by Choa (1994). The author reported the use of the vacuum preloading method in combination with vertical drains to improve the hydraulically placed silty clay reclamation operations.

CHAPTER 4 – SANDY CLAY

Sandy clays are classified according to the Unified Soil Classificaton System USCS as fine-grained soils with 50% or more passing through No.200 sieve (75 μm), inorganic, with ≥ 30% particles by weight retained in sieve No.200. If *LL*<50, *PI*>7, and plots on or above the "*A*" line, the soil is classified as CL, Sandy Lean Clay. If *LL*≥50 and *PI* plots on or above the "*A*" line, the soil is classified as CH, Sandy Fat Clay. Systematic information on the mechanical behavior of these soils is very limited.

Strength and stress-strain response

Wallace (1973) studied two residual sandy clays using a hand-held vane device to measure the undrained shear strength and direct shear boxes to characterize their stress-strain responses. The authors reported undrained shear strengths ranging from 38 to 125 kPa and residual friction angles varying from 29 to 38^o for the soils tested.

Toll and Ong (2003) presented experimental data from constant water content triaxial tests on a residual sandy clay from Singapore, tested under unsaturated conditions with measurements of matric suctions. The data for critical-state conditions from these tests are presented within a framework previously proposed by Toll. The unsaturated critical state requires five parameters: M_a, M_b, Γ_{ab}, λ_a and λ_b. At high degrees of saturation (in a nonaggregated condition) $M_a = M_b = M_s$ (the saturated critical-state stress ratio). At lower degrees of saturation, the value of M_b drops considerably and reaches values near to zero at degrees of saturation below 60%. Conversely, the value of M_a appears to increase as the degree of saturation reduces. A similar pattern is observed for λ_a and λ_b. The functions relating the critical-state parameters to degree of saturation (or volumetric water content) can be expressed in normalized form by referencing them to the saturated state.

The results of a number of suction-controlled laboratory tests on sandy clay specimens were used to evaluate the ability of a constitutive model for partially saturated soils (Alonso et al. 1990). The model is formulated within the framework of hardening plasticity using two independent sets of stress variables: the excess of total stress over air pressure and the suction. The model is able to represent, in a consistent and unified manner, many of the fundamental features of the behavior of partially saturated soils. On reaching saturation, the model becomes a conventional critical state model. The model is intended for partially saturated soils which are slightly or moderately expansive.

Comparisons of numerical predictions of a numerical model of the stress-strain behavior of unsaturated soil with results obtained from suction-controlled experiments on sandy clays were presented by Thomas and He (1998). An extended critical state elastoplastic constitutive model based on net stress and suction is solved via the finite-element method.

Response to cyclic loading

Not available.

Compressibility, collapse, and creep behavior

Wallace (1973) tested two residual sandy clays using an oedometer apparatus and found these materials to be highly compressible when the applied pressure in the tests exceeded a certain critical pressure, ranging from 1.7 to 3.6 ton/ft^2. Compression indexes of the soils in their natural state were reported to vary between 0.79 and 2.3.

Hydraulic conductivity

Not available.

Additional studies

Not available.

CHAPTER 5 – SANDY SILT

Sandy silts (ML) classify according to the Unified Soil Classificaton System as fine-grained soils with 50% or more passing through No.200 sieve (75 μm), inorganic, with ≥ 30% particles by weight retained in sieve No.200, *LL*<50, *PI*<4 or plots below the "*A*" line, % sand ≥ % gravel, and % gravel is less than 15%. As pointed out previously for sandy clay soils, systematic information on the mechanical behavior of sandy silts is also scarce.

Strength and stress-strain response

Thevanayagam et al. (2002) performed monotonic static undrained triaxial tests to evaluate the undrained fragility and behavior of sandy silts and other soils. Results indicate that the mechanical behavior and collapse potential of sandy silts depend on intergrain contact density. In general, intergranular void ratios or equivalent intergranular void ratios are found to be better contact density indices to characterize the mechanical response of granular mixes than the global void ratio. The dominant mechanisms that affect the shear response for sandy silt are interfine contact density and friction. The strength of sandy silt is typically higher than that of pure silt at the same interfine void ratio. When compared at the same equivalent interfine void ratio, the behavior of all sandy silt specimens is similar to that of the host silt.

Response to cyclic loading

Laboratory undrained cyclic tests followed by dissipation were conducted to study the pore pressure generation. and post-liquefaction dissipation and densification behavior of sandy silts (Thevanayagam et al. 2001). Results suggest that the pore pressure generation rate for sandy silt is somewhat higher than that of sand but the authors observe that, due to the limited number of tests performed, this issue should be further studied.

Compressibility, collapse, and creep behavior

Not available.

Hydraulic conductivity

Not available.

Additional studies

Rollins et al. (1998) evaluated the influence of moisture content on dynamic compaction efficiency at six field test cells, each with a progressively higher average moisture content. The soil profile consisted of collapsible sandy silt, and average test cell moisture contents ranged from 6% to 20%. At each cell, compaction was performed with a 5 ton weight dropped from a height of 24.3 m. Compaction efficiency was evaluated using (1) crater depth measurements, (2) cone penetration tests before and after compaction, and (3) undisturbed samples before and after compaction. Crater depth increased by a factor of 4 as moisture content increased. The degree of improvement increased up to a moisture content of about 17% and then decreased. The optimum moisture content and the maximum dry unit weight are similar to those predicted by laboratory Proctor testing using energy levels comparable to those employed in the field. Maximum dry unit weight decreased with depth, while optimum moisture content increased before the compactive energy decreased with depth below the impact point.

CHAPTER 6 – NATURALLY-CEMENTED SOILS

The effects of structure and cementation are as important in determining engineering behavior of geomaterials as are the effects of initial porosity and stress-history, which are the basic concepts of soil mechanics (Leroueil and Vaughan 1990). According to Mitchell (1993), the term *fabric* should refer to the arrangement of particles, particle groups, and pore spaces in a soil, while *structure* should be used, preferably, to refer to the combined effects of fabric, composition, and interparticle forces, therefore, taking both fabric and its stability into account. Precipitation of material onto particle surfaces, at interparticle contacts, and in pores can develop *cementation*, given rise to a fabric of partly discernable particle groups (Mitchell 1993). Many soils contain carbonates, iron oxide, alumina, and organic matter that may precipitate at interparticle contacts and act as cementing agent. Upon disturbance, these cemented bonds may be destroyed leading to a loss of strength (Mitchell 1993).

Leroueil and Vaughan (1990) noted that while soil structure can arise from many causes, its effects follow a simple general pattern that involves stiff behavior followed by yield. This yield can be described in a similar way to that occurring due to overconsolidation, although it is a separate phenomenon. As it can be described in a general way, soil structure and its effects should be treated as a further basic concept of equal importance.

The effects of structure and cementation on the mechanical behavior of soils is reviewed below with emphasis given to studies on naturally-cemented soils. The effect of artificial cements is covered in the next chapter.

Strength and stress-strain reponse

The unique strength characteristic of naturally cemented soils is a yield curve, defined by breakdown of cementation, which is independent of the classical effective stress patterns of yielding or failure for soils (Sangrey 1972). As schematically illustrated in Figure 5, the stress-strain behavior is dependent on the position of the initial state of the soil relative to the yield locus of the bonding (Coop and Atkinson 1993; Leroueil and Vaughan 1990).

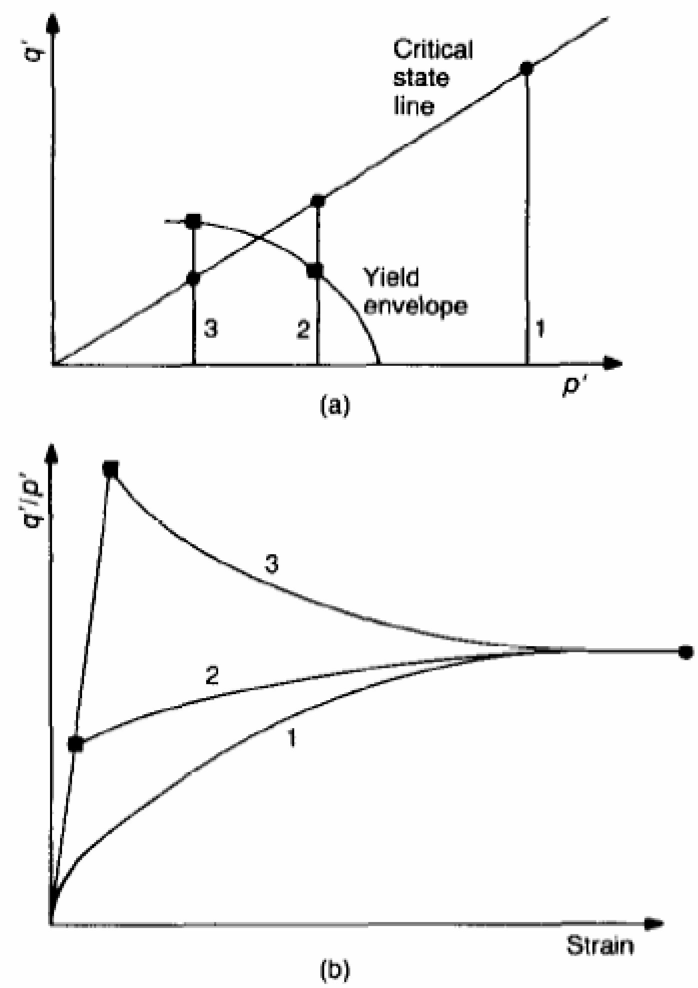

Figure 5 – Idealized behavior of cemented soils: (a) stress paths; (b) stress-strain behavior (After Coop and Atkinson 1993)

Despite its limitations, particularly the one related to the formation of localized shear bands, the triaxial apparatus has been the most used type of experimental device in studies of mechanical behavior of naturally-cemented soils. It has been used extensively to characterize the stress-strain response and obtain estimates for the shear strength of cemented clays (Bjerrum and Wu 1960; Sangrey 1972; Wong and Mitchell 1975), cemented sands (Cuccovillo and Coop 1997; Cuccovillo and Coop 1999; Ismael 2001), and other cemented soils (Airey and Fahey 1991; Fonseca et al. 1997; Kohata et al. 1997; Vatsala et al. 2001). Plane strain (Wong and Mitchell 1975) and shear box (Wallace 1973) devices have also been used to study the stress-strain response of naturally-cemented soils.

Since structure strongly affects the pre-failure behavior of cemented geomaterials (Cuccovillo and Coop 1997), most up-to-date triaxial apparatuses used to test these materials incorporate some sort of on-sample, local-strain measurement device (Clayton et al. 1989). Local measurements of strains provide for much more accurate and reliable determinations of the small-strain stiffness of cemented soils (Bressani and Vaughan 1989; Cuccovillo and Coop 1997). Non-linear stiffness models such as the one proposed by Kohata et al. (1997), which is based on elastic characteristic moduli for a wide variety of stiff geomaterials, require reliable assessment of small-strain stiffness parameters.

Degradation of the cementitious bonds results in a progressive transformation of structured sands into a frictional material (Cuccovillo and Coop 1997), giving rise to changes in the yield stress and shear stiffness that contrasts with the strain-hardening behavior of unbonded granular soils. As bonding degrades, the variation of the shear stiffness with state depends on which structural feature is predominant, fabric or bonding.

In a subsequent study (Cuccovillo and Coop 1999), the previous authors suggest that structure should be considered as an element of the nature of a sand in addition to properties such as mineralogy, particle shape and grading. While bonding results in a cohesive mode of shearing, it is shown that when fabric dominates, the shearing behavior remains predominantly frictional, although the dilation rates and peak strengths may be very much higher than for the reconstituted soil at the same stress-volume state. Additionally, they also state that it is not necessarily the position of the state of the soil relative to the critical-state line that distinguishes strain-hardening and strain-softening behavior, but the proximity to the boundary curve determined in isotropic compression.

A model for bonded or cemented soils within the framework of hardening plasticity was presented by Vatsala et al. (2001) based on the concepts that (1) the strength of a cemented soil can be considered to be made up of two components, the usual strength of the soil skeleton and the strength of cementation bonds; and (2) soil deformation is associated with the component of stresses on the soil skeleton (excluding the bonds), as in a reconstituted soil, whereas the cement bonds offer additional resistance at any given strain level. Wong and Mitchell developed a plasticity theory to describe the post-yield stress-strain behavior of sensitive cemented soils based on an experimentally defined non-associated flow rule (Wong and Mitchell 1975).

Other experimental devices such as the unconfined indirect tension, unconfined compression (Sangrey 1972), and Torvane (Wallace 1973) have also been used to estimate, respectively, the tensile, compressive, and shear strengths of naturally-cemented soils.

In-situ full-scale load tests on naturally-cemented soils were performed in a limited number of studies. In these studies, the effect of foundation geometry and material such as circular concrete footing (Fonseca et al. 1997), circular steel plates (Consoli et al. 1998), drilled shafts (Ismael 2001; Walsh et al. 1995), and pile groups (Ismael 2001), on the load-settlement response have been investigated. Walsh et al. (1995) reported that in naturally-cemented soils significantly higher deflections are required to mobilize the peak skin friction along the pile shaft.

Response to cyclic loading

Results from several undrained cyclic triaxial tests on undisturbed samples of both cemented and uncemented calcareous soil have been reported by Airey and Fahey (1991). The more cemented soils sustained higher cyclic stresses and, depending on the load level, showed either elastic behavior with no pore pressure build-up, eventual stabilization after some pore pressure build-up, or steady state pore pressure build-up and eventual failure. The effect of frequency on the response in the range 0.1 to 10 Hz was investigated but, since no dramatic differences in performance could be detected, no firm conclusions could be drawn regarding the effect of testing rate.

Compressibility, collapse and creep behavior

The compressibility of structured sands has been claimed to be a key parameter, particularly to petroleum engineers since most new oil and gas reservoirs are discovered in sands and weak sandstones (Coop and Willson 2003). While analyzing an extensive amount of triaxial data generated by the oil industry, these authors state that, when present, cement bonds between particles appear to be weaker than the particles themselves, so that the major yield point during compression is still associated with breakage of the particles rather than of the cement bonds.

Oedometer tests have also been used to determine the compressibility characteristics of residual, naturally cemented soils (Wallace 1973).

The effect of soaking and leaching on the compressibility and collapse of a salt-cemented soil has also been examined in the laboratory (Abduljauwad and Al-Amoudi 1995). Results indicate that the percolation of distilled water through this soil causes destruction of the natural cementation, leading to collapse and increase in settlement. Soaking produces practically negligible collapse; leaching causes a significant collapse due to the softening, dissolution and effusion of salts from the soil skeleton.

A generalized power formulation of specific volume as a function of stress which describes the virgin compression of a wide range of soils has been presented (Haan 1992).

Hydraulic conductivity

Abduljauwad and Al-Amoudi (1995) showed that the permeability of a salt-cemented soil was increased by percolation of distilled water. Dissolution and leaching of halite, gypsiferous and calcarenite cements occurred, leaving the quartz particles covered by a thin loose mat of illitic clay with large voids.

Additional studies

Coleman et al. (1964) reported that a red clay soil from Nyeri, Kenya (which occurred immediately beneath the top soil and was seasonally air dried in situ), possessed unusual properties, including an excessively high moisture sorption, increase in index values with prolonged mixing and abnormally low optimum dry density for B.S. compaction. The explanation of these properties was shown to lie in the mineralogy and structure of the soil. By use of colour studies it was shown that the metahalloysite and quartz in the soil are coated overall and cemented with haematite.

CHAPTER 7 – ARTIFICIALLY-CEMENTED SOILS

The addition of cementitious products to soils affects their mechanical behavior in a way that is similar to that observed for naturally-cemented soils. Yet, while the fabrics of both naturally- and artificially-cemented soils can be similar, their structures may differ a great deal. This is because structure encompasses the stability of the soil fabric (Mitchell 1993), as pointed out in the previous chapter.

In general, naturally-cemented soil structures are more unstable than those generated by soil stabilization techniques. The extent to which the original properties of a soil change with cementation depends upon the amount and nature of cementation incorporated into the soil. Slight changes are usually achieved by soil improvement techniques while more significant modifications require the use of soil stabilization methods.

Although a universally-accepted classification system for cemented geomaterials is not available yet, basic index tests such as the unconfined compression and tension tests may be used to assess the degree of cementation of geomaterials (e.g. Mitchell 1976b). Typical ranges for these parameters are provided by Huang and Airey (1998).

In addition to the intrinsic characteristics of the cementation, another distinction between naturally- and artificially-cemented soils is related to their density state. While naturally cemented soils may occur at void ratios substantially higher than those that would be expected for a similar uncemented material, stabilized soils are usually compacted and/or densified during preparation, therefore existing in more compact states.

Several cementitious products have been used in experimental studies of artificially-cemented soils. The most common ones are Portland cement, lime[3], calcite, and gypsum. But there is very little information in the geotechnical literature specifically regarding the influence of the type of cement on the engineering behavior of artificially-cemented soils.

Ismail et al. (2002b) carried out triaxial testing results on samples treated with different types of cement, namely Portland cement, gypsum, and calcite. Their results showed that, despite having the same unconfined compressive strength q_u and density, the effective stress paths and post-yield response of these materials may be significantly different, mainly because of the different volumetric response upon shearing. Portland cement specimens showed ductile yield and strong dilation afterwards; while calcite and gypsum-cemented samples exhibited brittle yield, generally followed by contractive behavior.

A summary of studies on artificially-cemented soils focusing on the effect of one particular type of cementitious agent on the mechanical response is presented below.

Strength and stress-strain reponse

- Portland cement:

Some studies have either focused on the effect of addition of Portland cement on the unconfined compressive strength q_u of sandy, clayey, gravelly, and other soils (Clare and Pollard 1954; Croft 1967; Newill 1961; Sherwood 1957; West 1959) or used q_u as an index parameter in correlations with other cemented soil properties (Puppala et al. 1995;

[3] A comprehensive summary of stabilization techniques of soils with lime was published by the Transportation Research Board TRB. (1987). *Lime stabilization: reactions, properties, design, and construction*, National Research Council, Washington, DC.

Schnaid et al. 2001). In these studies, it was usually found that q_u increases with increasing curing temperature, curing time, and cement content. On the other hand, q_u decreases with increasing clay, sulphate, and organic content; it also decreases with increasing compaction delay. The soil composition, particularly regarding the type of clay mineral present in the soil, was found to affect q_u as well.

Statistical correlations between unconfined compressive, indirect tensile, and flexural strengths of soil-cement mixtures have also been developed (Doshi and Guirguis 1983).

The influence of both the degree of cementation and the initial mean effective stress on the stress-strain response of a silty sand stabilized with Portland cement was studied by Schnaid et al. (2001). Based on their drained triaxial compression and unconfined compression tests results, it was observed that: (1) the degree of cementation correlated well with q_u; (2) the triaxial shear strength could be expressed as a function of the friction angle of the nonstructured material and q_u. A logarithmic formulation was used to express the relationship between static deformation moduli and axial strain amplitude in axisymmetric conditions.

Consoli et al. (2000) studied the stress-strain response of the same material tested by Schnaid et al., but submitted the specimens to two different curing conditions: with and without confining stress application. They concluded that: (a) the stress state acting during the cementing process strongly affects the mechanical behavior; (b) confining stress application caused a reduction in stiffness with increase of the initial mean effective stress for specimens cured without confining stress; opposite behavior was observed for specimens cured under stress; (c) curing under stress increased the friction angle but did not affect the cohesive intercept. The authors suggest that naturally-cemented samples should be taken from different depths and tested under their respective confining stresses or the triaxial tests should be restricted to confining stresses below those causing bonding degradation.

Drained and undrained triaxial response of saturated samples of a silty soil weakly cemented by a Portland cement – fly ash slurry was studied experimentally by Lo and Wardani (2002). Special zero effective stress tests were conducted to measure, directly, the contribution of bonding between grains to the enhanced strength and stiffness. The cemented soils were initially less dilatant than their respective parent soils but eventually became more dilatant than the parent soils. The shear strength data followed a curved failure surface that merged back, at high stress, into that of the parent soil. This feature can be captured by a failure function that models the contribution of cementing agent to strength as two parts, true bonding and increase in dilatancy rate at failure. Both parts degrade with an increase in confining stress, but at different rates.

Using piezo-ceramic bender elements, Baig et al. (1997) showed that confining pressure has a negligible effect on the measured maximum shear modulus G_{max} of artificially-cemented sand specimens. The main variables that determine the moduli of the cemented sand are the percentage of cementation, and to a lesser degree the relative density of the sand skeleton. The authors also suggest that resonant column tests may be inadequate to assess G_{max} on these materials due to coupling problems between the specimen and the end platens.

Mechanical-shear and electromagnetic waves were also used in the laboratory to monitor the setting and hardening of cement, bentonite-cement slurries, and attapulgite-cement slurries (Fam and Santamarina 1996). Increases in shear-wave velocity reflected the rise in effective stress due to consolidation, the decrease in double-layer repulsion, and the higher rigidity of the mixtures as a result of cementation.

- Lime:

Early investigations of the suitability of clayey soils for stabilization with hydrated lime were reported by Newill (1961) based on unconfined compressive strength results. The authors studied in the laboratory the properties of two typical red clays with the main object of determining the parts played by the clay mineral – whether hydrated halloysite or metahalloysite – and the coating of iron oxide on the soil particles.

The results of a laboratory study on the influence of standard Proctor and West African standard compactive efforts as well as compaction delays up to 3h on the compaction and strength characteristics of lateritic soil treated with a maximum of 8% lime were presented by Osinubi (1998a). Their results showed that the compaction and strength properties of the lime-treated soil decreased with increases in compaction delays. The determination of properties of lime-treated soil at no compaction delay defines optimum properties of the soil-lime mixtures, while compaction and strength properties determined following compaction delays define the minimum that can be achieved in the field for the specified elapsed times between mixing and compaction.

A series of triaxial compression tests were performed on uncemented, artificially cemented and destructured specimens of gravely sand to sandy gravel with some cobbles (Asghari et al. 2003). Hydrated lime was used as the artificial cementitious agent. During shearing, the cemented specimens undergo dilation at confining stress lower than 1000 kPa while the uncemented and destructured samples show contraction. The shear strength increased with increasing cement content but the influence of the cementation decreased with increasing confining stress. Test results indicate that the failure envelope for cemented samples is curved and not linear.

Unconfined compression and tensile tests, and saturated drained and undrained triaxial compression tests with local strain measurement were carried out to evaluate the stress-strain reponse of a silty sand stabilized with carbide lime – fly ash mixtures (Consoli et al. 2001). The addition of carbide lime to the soil-fly ash mixture caused short-term changes due to initial reactions, inducing increases in the friction angle, cohesive intercept, and stiffness of the mixtures. Under the effect of initial reactions, the maximum triaxial stiffness occurred for specimens compacted on the dry side of the optimum moisture content, while the maximum strength occurred at the optimum moisture content. After 28 days, pozzolanic reactions magnified brittleness and further increased triaxial peak strength and stiffness; the maximum triaxial strength and stiffness occurred on the dry side of the optimum moisture content.

- Calcite:

A technique of cementing porous materials artificially using calcite was presented by Ismail et al. (2002a). The technique depends on flushing a mixture of chemical solutions through a porous medium, leading to precipitation of calcite due to reaction of the solution ingredients. It was found that the strength of a calcite-treated material increases with: (a) intrinsic strength of the individual grains, (b) density, (c) decreasing particle size of the host grains, (d) pre-coating of grains with calcite, and (e) roundness and non-angularity of soil grains.

- Gypsum:

The triaxial response of artificially cemented carbonate sand was investigated at confining pressures of up to 9 MPa by Coop and Atkinson (1993). Their results show that an important effect of cementing is a reduction in specific volume resulting from the increase in fines content. This influences both the stress-strain response and the peak strength at strains beyond those required to fracture the cement bonding. Comparisons between the behavior of cemented and uncemented soils should, therefore, be carried out on samples with similar gradings. For cemented samples it is possible to identify a yield curve outside the state boundary surface of the uncemented soil. A framework for the behavior has been defined which depends on the relative magnitudes of the confining pressure and cement bond strength.

Huang and Airey (1998) studied the influence of inherent variability in density and degree of cementation on the properties of cemented materials by producing artificially cemented specimens at various ranges of dry unit weights and gypsum cement contents. Tests were performed to investigate the index strengths, the behavior in isotropic and K_0 compression, and the standard triaxial compression response from tests over a wide range of confining pressures. The effects of the bonding were found to be significant only for stresses below an apparent preconsolidation stress. The strength and stiffness increased with increasing density and cement content, but the influence of the cementation decreased with increasing density.

Response to cyclic loading

- Portland cement:

Cementation is known to increase the liquefaction resistance of a sand and can be a critical factor in engineering decisions about site response (Clough et al. 1989). Using both triaxial and cubical cyclic shear devices, these authors tested weakly-cemented sand specimens over a wide range of cementation levels and unit weights. The cubical shear apparatus was used to allow assessment of possible stress concentration effects in the triaxial device and to investigate loading under multiaxial stress conditions.

Cyclic triaxial and resonant column tests on artificially-cemented sand specimens were carried out by Saxena et al. (1988). Their study showed a good correlation between dynamic moduli and liquefaction resistance for the materials tested.

Chang et al. (1990) applied microstructural continuum modeling to study the mechanical properties of cemented sands under low amplitude oscillatory loading. The effect of cement in sand was accounted for by considering adhesion in addition to friction at the inter-particle contact. Under the assumption that cemented sands are statistically isotropic random packings of equal spheres, a closed form relationship was derived for the initial shear modulus of cemented sands under isotropic confining stress. Also, the secant moduli and damping ratios were computed numerically for various strain amplitudes.

- Calcite:

The deformation characteristics of artificially cemented calcareous soil subjected to undrained cyclic triaxial loading were investigated at different confining pressure and cyclic stress levels by Sharma and Fahey (2003a). It is observed that the deviator stress and the deviatoric strain at yield reduced with increasing number of cycles for cemented sand due to progressive degradation of bond, which results in significant decrease in stiffness. On the other hand, a strain-hardening effect is observed in uncemented sand and this results in increasing yield stress and strain with progressive number of cycles. A linear relationship between degradation index and number of cycles is observed for cemented sand. This relationship has been synthesized in the form of an empirical equation by modifying a previously proposed equation for cohesive soils. This empirical equation was further used to evaluate the fatigue life of soils by adopting a failure criterion.

The mechanism controlling the cyclic shear strength of cemented calcareous soils was investigated by Sharma and Fahey (2003b) based on the results obtained from monotonic and cyclic triaxial tests on two different types of calcareous soil. Undrained cyclic triaxial tests performed on artificially cemented calcareous soils with different loading combinations showed that the effective stress path moved towards or away from the origin, due to the generation or dissipation of pore pressure with progressive cycles. Previous investigations have shown that the peak strength envelope or the state boundary surface or the critical state line forms a boundary beyond which effective stress paths during cyclic loading cannot exist. However, in this study it was observed that the maximum stress ratio η_{max} obtained from monotonic tests defined the boundary for the cyclic tests. Based on the information obtained from this study, an approach for evaluating the cyclic shear strength is proposed. It was observed that the modified normalized cyclic shear strength had a strong linear relationship with the logarithm of the number of cyclic to failure irrespective of confining pressure, type of consolidation and stress reversal.

Compressibility, collapse and creep behavior

- Portland cement:

The formation of a cemented sedimentary silty sand deposit in which cement bonding occurs after burial and under geostatic stresses was simulated in the laboratory by Rotta et al. (2003). Isotropic compression tests were carried out on artificially-cemented specimens, consolidated to the uncemented normal compression line after a 48h curing period. The results showed the importance of the void ratio during cementation and also of the degree of cementation on the compressive behavior of the cemented soil, and demonstrated that the variation in yield stress with void ratio and cement content is dependent on the curing stress and independent of the stress history.

Hydraulic conductivity

- Lime:

Laboratory investigations on a residual lateritic soil treated with up to 8% quicklime were carried out by Osinubi (1998b) in order to evaluate the effect of lime content, curing period, and compactive effort on the permeability of lateritic soil-lime mixtures prepared at various maximum dry densities and corresponding optimum moisture contents. The permeability of uncured specimens (standard Proctor) increased to a maximum at 4% lime content and decreased with increasing lime content. For the cured condition, the permeability of all the lime-treated specimens increased with curing age up to 14 days and decreased with curing age beyond 14 days. Consoli et al. (2001) have also reported estimates of K for lime-stabilized silty sands.

Additional studies

- Portland cement:

Expansive behavior seems to be a key aspect of the behavior of artificially-cemented materials, particularly those improved and/or stabilized with Portland cement.

Rollings et al. (1999) reported that a 3.5-km (2.2 mi) section of a road in Georgia developed unexpected transverse bumps within 6 months after construction. The source of the bumps appeared to be expansion within the cement-stabilized base course. Laboratory examination of samples from areas showing distress revealed the presence of ettringite, a calcium sulfo-aluminate whose formation can be accompanied by severe expansion. The calcium and alumina needed to form ettringite were available from the portland cement and the stabilized soil's clay minerals. The source of the sulfur was identified as the well water that was mixed with the cement-stabilized base. Similar problems were pointed out by Scullion and Harris (1998) who reported that three cement-treated base (CTB) pavements constructed around 1990 near Houston, Texas, showed severe pavement deterioration after 3 to 4 years. The primary cause was determined to be chemical deterioration that resulted in destruction of the cement matrix. Extensive ettringite was found in one material. In one case deterioration was caused by the growth of two phases exhibiting different morphologies: hydrated calcium aluminum silicate with a radiating needle morphology and ettringite, which showed a bladed morphology in these samples. Practical recommendations to prevent recurrence of the problem were (a) moisture absorption as acceptance testing for coarse aggregate and (b) elimination of designs placing different stabilized materials on top of one another. The laboratory suction test that was developed is recommended as a screening test for molded CTB specimens. Sherwood (1957) investigated the influence of the chemical properties of a clay on its strength when stabilized with cement. A boulder clay, having a low clay and organic content and no sulphate, could be satisfactorily stabilized with 10% of ordinary Portland cement. Neither of the other layers could be stabilized, however, owing to their high clay and sulphate contents. Because of the high sulphate contents of the samples, further tests were made to find the effect of sulphates on cement-stabilized clay. These tests showed that the effect of sulphates only became apparent when the clay-cement was immersed in water, when 1.0% of calcium sulphate or 0.75% of magnesium sulphate (expressed as SO_3) incorporated in the clay led to complete disintegration of the clay-cement mixture. When sulphate-free clay-cement mixtures were immersed in aqueous sulphate solutions as little as 0.2% sulphate (as SO_3) in the surrounding solution caused disintegration. The substitution of sulphate-resistant cement for ordinary Portland cement in these experiments did not effect much improvement.

Sulfate attack of cement-stabilized soils is a relatively infrequent problem, but it is highly destructive when it occurs. Currently, there are no firm criteria for identifying when sulfate attack of a cement-stabilized soil is a potential problem nor are there established methods of preventing the attack (Rollings et al. 1999).

Sinkhole development in a profile with a weakly cemented sand overlain by an uncemented sand and underlain by a karst limestone was studied using 51 reduced-scale stress-correct centrifuge models by Abdulla and Goodings (1996)

Cone penetration testing in very weakly cemented sands with unconfined compressive strengths of 60 kPa or less was investigated in a calibration chamber by Puppala et al. (1995). The tip resistance and the sleeve friction are both found to increase with cementation. The friction ratio is found to be indifferent to the increase in cementation at very weak cementation levels and vertical effective stresses of 50–300 kPa. Tip-resistance values at vertical effective stresses of less than 100 kPa increase over two times and over four times the uncemented values at peak cohesion intercepts of 10 kPa and 30 kPa, respectively. Although the effect of confinement gradually overshadows the influence of any weak cementation, tip resistance at a vertical stress of 300 kPa can still be 15–25% and 40–45% more than the uncemented values at peak cohesion intercepts of 10 kPa and 30 kPa, respectively.

- Lime:

Clare and Cruchley (1957) mixed samples of unweathered clay from ten geological formations with 1, 2, 4, 7, and 10% by weight of hydrated lime, at moisture contents near the liquid and plastic limits. The samples were stored in air-tight containers for periods of 1, 6, and 12 months. These, together with a series of freshly prepared samples of similar compositions, were used for the determination of the liquid and plastic limits of the mixtures and for establishing the suction/ moisture content relationships under wetting conditions. Estimations were also made of the amount of lime taken up by the soil, and of the pH (hydrogen-ion concentration) of the mixtures in aqueous suspension. The results of the present experiments support the view that the immediate effects are due to flocculation of the clay particles, as a result of the replacement of adsorbed hydrogen ions by calcium ions between the natural pH of the soils (7.0-8.5) and that of the mixtures (12.0-12.5). This is followed, during storage, by bonding of the particles by calcium silicates and/or aluminates having cementing properties, and formed by chemical reaction between the lime and the clay.

CHAPTER 8 – CONCLUSIONS AND RECOMMENDATIONS

The main issues related to the mechanical behavior of non-textbook soils that were identified in this literature review are summarized below. The type of non-textbook soils most related to the issue addressed will be identified accordingly in each of the following recommendations.

- The selection of a single parameter that may be used to evaluate the density state for a wide range of geomaterials is an issue that still has not been resolved and should be further investigated. This has been shown to be particularly true for silty sands, clayey sands, and sandy silts.
- The development of a suitable parameter that incorporates and properly quantifies the role played by soil structure into mechanistic analyses of geomaterials seems to be an open issue as well, particularly for materials such as cemented soils, silty sands, and clayey sands.
- Constitutive modeling of cemented/structured geomaterials is a topic that shows great potential for future research. The literature review carried out on this topic suggests that the majority of the models currently available are unable to capture and integrate into the same framework all the relevant aspects of the behavior of these materials. This limitation is particularly evident with respect to foundation design in/on/with cemented soils.
- The effect of sulphates on the degradation and expansive behavior of cemented geomaterials, particularly those artificially-cemented with Portland cement, has been pointed out in a few studies. Such a problem may have serious implications in pavement management systems constituted by a significant percentage of

stabilized bases and stabilized/improved subgrade soils; the mechanisms that control this phenomenon have been unsatisfactorily studied to this date.

- Most aspects of the mechanical behavior of sandy silts, sandy clays, and silty clays have been studied in a very limited manner and also show potential as prospective research areas. Silty soils and glacial tills are additional examples of non-textbook geomaterials commonly encountered in Indiana that also present great potential for future studies of mechanical behavior.

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