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Special Lecture: Occurrence, Geotechnical Properties, and Special Problems of Some Soils of America

J.K. Mitchell (1), R.Q. Coutinho (2), Graduate Students

ABSTRACT

The soils of America range from very firm, rock-like materials through cohesionless gravels, sands and silts to soft organic and inorganic clays. They have diverse origins and challenge geotechnical engineers with a range of unique special problems. Several of these soils are considered in terms of their usual setting, geographical distribution, formational processes, compositions, classification and mechanical properties, and unique geotechnical problems that they cause. Brief case histories relating to the special characteristics of each soil are given.

INTRODUCTION

When it was first proposed by the Organizing Committee of the IX Pan American Conference on Soil Mechanics and Foundation Engineering that there be a special report on the soils of America, it seemed a very fine idea. However,

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(1) Cahill Professor of Civil Engineering, University of California, Berkeley, CA 94720 U.S.A.

(2) Associate Professor of Civil Engineering, Federal University of Pernambuco, Recife, 50000 PE, Brazil

J.R. Ash, M.R. Caruso, A.Y.W. Chen, D.K. Del Grande, D.G. Grubb, C.W. Herzog, T. Hino, T.D. Horton, C.S.S. Kao, T.C. Ke, S.M. Kelkar, S.W. Lai, S.A. Leigh, E.S. Lindquist, E.W. Medley, G.S. Nagle, K.R. Nelson, S.L. Pillsbury, W.E. Pratt, M. Prezzi, C. Robertson, V.S. Romero, S. Roy, E.Z. Rutkowski, R. Salgado R., P.J. Sandberg, D.D. Steele, S.A. Stephens, K.J. Susilo, B.B. Thapa, M. A. Tyler, W.A. Van Court, M. V. Wolski, K. Yamasaki, D.J. Young, and J.G. Zornberg

as the time for the Conference drew nearer the enthusiasm changed to panic with the realization that to cover such a vast subject in a manner that could provide useful and specific geotechnical information about each of the vast range of important soil types would be impossible. Even to prepare a paper that provides in-depth information about some of them would be a major undertaking. Just to develop a common framework for their discussion was a daunting task.

Accordingly, we followed the usual path of professors in a university - enlist the aid of bright students and turn the problem into a term project. Graduate course CE 273, Soil Behavior, was well-suited for this purpose, as soil formation, soil composition, soil structure and their relationships to geotechnical engineering properties and behavior are focal points of the course.

The class of 36 students in the Fall Semester of 1990 was divided into 12 teams, and each team was assigned to develop information for a project at a site with a different soil type that is common in the Americas. Reports were prepared that contained, to the extent possible, information on the following for each soil:

- General setting where the soil is found
- Geographical distribution in the Western Hemisphere
- Formational processes
- Usual compositions
- Index properties and classification
- Special fabric and structure characteristics
- Unique geotechnical properties
- Special geotechnical problems
- Summaries of two case histories; one from North America, the other from South or Central America
- Useful references

The resulting reports, augmented by some supplementary material considered by the authors to be particularly relevant, form the basis for the paper that follows, which focuses on nine of the 12 soil types studied. These soils are, in the order of their presentation: decomposed granite, laterite, volcanic ash soils, expansive soils, collapsing soils, loess, soft organic soils, quick and very sensitive clays, and carbonate sands.

DECOMPOSED GRANITE

Decomposed granite is formed by the in-situ decomposition and disintegraton of granitic rock outcrops, which may include not only granite, but also other closely associated plutonic igneous rocks such as granodiorite and quartz-diorite. The term "decomposed granite", as it is commonly used encompasses granitic rocks at all stages of weathering, without the products having been subjected to post-formational processes such as desiccaton and laterization. The properties of decomposed granite depend on the characteristics of the parent rock, climate, topography/drainage conditions, and the present stage of weathering.

Formation processes and distribution

Most granitic rocks have inherent fracturing along two roughly parallel sets of joints. Selective and progressive decomposition of unstable minerals in the granite causes a breakdown of the rock by spheroidal weathering, disintegration, and disagregation. Granitic rock may be weathered to a depth of 30 meters or more. Granitic rock weathering generally follows Bowen's reation series. Biotite decomposes first, producing limonite and clay minerals, and is followed by plagioclase feldspars. When part of the plagioclase has decomposed and breakdown of the orthoclase begins, the rock breaks into fragments of decomposed granite called gruss. When most of the orthoclase has weathered to clay minerals, kaolinite being the most common, the gruss crumbles to a silt sand, which typically contains mica flakes. Apart from some mechanical breakdown, the quartz fragments remain unaltered.

The sequence of horizons that is formed at any one site constitutes the soil profile and generally contains four zones, as shown in Fig. 1. The deepest zone consists of angular granitic blocks. The amount of residual debris is small, although the rock may be relatively highly altered. The next zone contains abundant angular to subangular core stones in a matrix of gruss and residual debris. The upper middle zone is the most variable part of the weathering profile and typically contains about equal amounts of rounded core stones, gruss, and residual debris. The topmost zone usually consists of a structureless mass of clayey sand with a highly variable grain size distribution. In tropical climates deep and intense weathering occurs because of the abundance of moisture and warm temperatures which speed up chemical weathering (Mitchell, 1991; Yapa, 1991). The irregular bedrock profile and variable depths to competent rock below the ground surface, as well as the presence of the core stones can present significant obstacles to excavation and foundation construction.

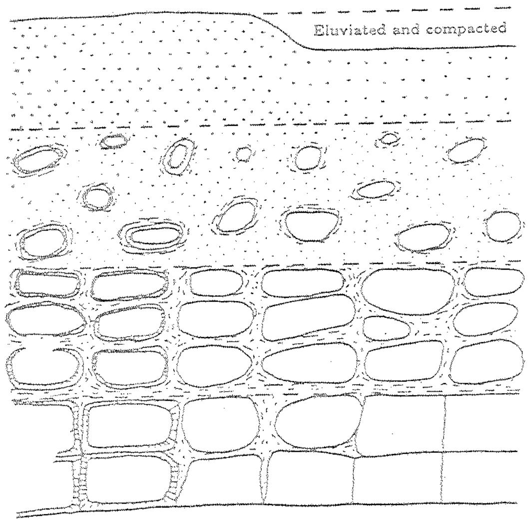

Fig. 1 Zones in a mature profile of decomposed granite.

The microfabric of decomposed granite is related to both the initial fabric of the parent rock and to the history of weathering; i.e., the degree to which feldspars have been weathered, the proportion of clay produced during the decomposition processes, and also the extent to which particles have been eluviated from the system (Massey et al,1989).

Granitic rocks and the soils associated with them are exposed over about 15% of the surface area of the continents (Twidale, 1982). Rocks of granitic composition are a major component of the ancient crystalline blocks or shields that form the heart of all the continents, as well as forming a significant portion of most fold mountains of orogenic (mountain building) belts (Fig.2). In the western United States granitic rocks are prominently exposed in the Sierra Nevada batholith of California and Nevada, the Idaho batholith of central and northern Idaho and western Montana, and in the Rocky Mountains of Colorado. In South America vast areas of decomposed granite soils are exposed within the Brazilian Shield. Granitic rocks are also exposed in the Andes Mountains along the west coast of South America and to a lesser extent in the orogenic belts which pass through Central America.

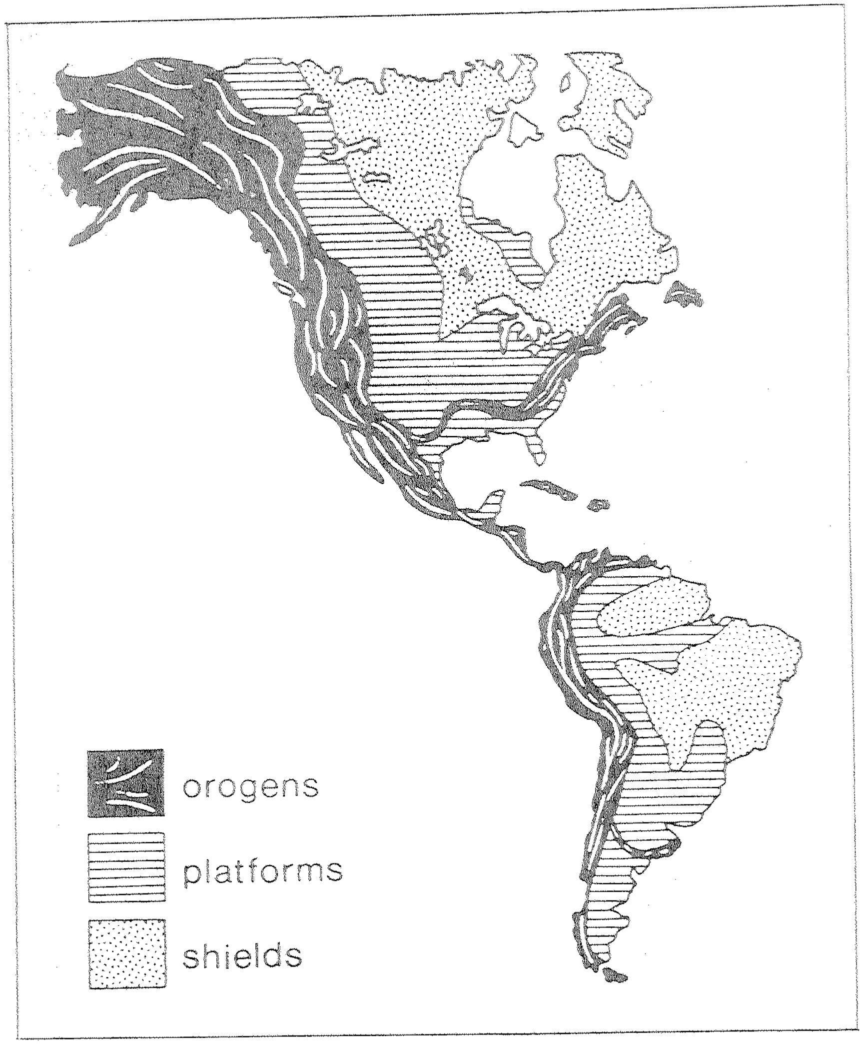

Fig. 2 Distribution of granitic shields orogens and platforms in the Americas (from Twidale, 1982).

Classification of decomposed granite

The characteristics of decomposed granite can not always be adequately represented by traditional systems, such as the Unified or AASHO Classification Systems. Different types of classification systems have been proposed for decomposed granite (Dearman et al,1978; Yapa, 1991). In the scheme proposed by Lee and de Freitas (1989) the granitic materials are classified into six grades, each defined by detailed visual descriptions of both chemical decomposition and physical disintegration of the material and a range of values of mechanical properties determined by simple field and laboratory tests. This system is summarized in Table 1.

TABLE 1. Classification of decomposed granite (after Lee and de Freitas, 1989)

Grade	Description	Compressive strength (MPa)		Hydraulic conductivity (m/s)
		Dry	Wet	
I	Fresh rock Sound minerals	125-260	80-240	Class 1 $\leq 10^{-9}$
II	Slightly weathered rock, Staining of some minerals	100-170	55-135	Class 1 $\leq 10^{-9}$
III	Moderately weathered - rock, some decomposition of feldspars	60-120	35-65	Class 2 10^{-7} - 10^{-9}
IV	Highly weathered, rock/soil, all feldspar decomposed	35-55	10-15	Class 3-4 $\geq 10^{-5}$ to 10^{-7}
V	Completely weathered - soil has original texture			Class 4 $\geq 10^{-5}$
VI	Residual soil - original texture gone			Class 2-3 $\geq 10^{-5}$ to 10^{-9}

For describing the mechanical properties of the "soil" portion of the profile, it is useful to subdivide the grades IV, V, and VI into smaller definable groups (Yapa, 1991).

Index properties

The main mineral constituents of a decomposed granite are quartz and feldspar. Quartz generally forms strong sand size particles; whereas feldspar may form weaker aggregates of sizes down to silty/clayey size depending on the level of weathering. There can also be dark minerals like biotite and hornblende. The proportion of each mineral present depends on the composition of the parent rock, climate, and process of weathering. The material generally falls into a classification between silty sand and sandy gravel.

Due to the friable nature of the material the results of sieve analyses for a particular sample depend strongly on details of test procedure. Particles of some deposits are strongly aggregated, and mechanical force or the long sieving time necessary to separate them can disintegrate softer feldspar grains (Miura and O'Hara,1979; Sandroni,1985; Phillipson and Brand,1989). The applicability of the Atterberg Limit tests has been questioned by many researchers because the portion finer than #40 in most decomposed granite deposits may be very small and hence may not be representative of the total soil behavior. This fine fraction may also have a high content of mica, which may give erratic or irreproducible results (Sowers and Richardson, 1983; Gidigasu,1985; Richards, 1985). Other difficulties in the determination of index properties in residual soils are considered in the subsequent sections on Lateritic Soils and Andosols.

The conventional specific gravity ($G_s=W/V_s\gamma_w$) does not reflect the intragranular voids of the microfractured grains, which can be significant in a decomposed granite soil. An apparent specific gravity (G_a) is defined by considering the water permeable intragranular void content(V_i): $G_a = W_s/(V_i+V_s)\gamma_w$; and $V_i = (W-W_s)/\gamma_w$; where V_s is the volume of solids; W is the weight of solids at saturated-surface-dry condition; W_s is the completely dry weight of the soil grains. Many reseachers have considered G_a as an better indicator of the degree of weathering than the conventional G_s. G_a has also been used to define effective void ratio $e_e = [G_a \times (1+e)/G_s] - 1$ that can explain better than conventional void ratio, the effects of weathering on mechanical behavior of the soil (Matsuo and Nishida,1970; Nishida and Kagawa,1972; Yapa,1991).

The specific gravity of feldspar grains (G_{sf}) has also been proposed as an index of the degree of weathering of decomposed granite (Matsuo and Nishida, 1968). Onodera et al (1976) suggested "Loss on Ignition" as a measure of the clay mineral content that is formed in decomposed granite during the process of weathering. The Loss on Ignition(W_{li}) measures the hydration water content of the soil: $W_{li}(\%) = (W_t$ at $1000^{\circ}C - W_t$ at $105^{\circ}C)\times 100/W_t$ at $105^{\circ}C$. As the amount of hydroxyl ions(OH^-) in fresh granite is less than 0.5%, a W_{li} value in excess of 0.5% is indicative of the level of hydration reactions that have occured during weathering. Other weathering indices for decomposed granite soils are given by Massey et al (1989).

Ranges of values of index properties of some decomposed granite deposits from different locations are shown in Table 2.

Important geotechnical properties

As the the term decomposed granite may refer to materials of wide ranging characteristics, only general ranges of properties can be presented, and it is important that each site should be evaluated on its own merits. Adequate measurement of mechanical properties requires samples of high quality, appropriate techniques, and test specimens that are large enough to include the effects of large size particles and soil fabric. It is also necessary to distinguish between undisturbed and remolded or compacted soil characteristics. Table 3 gives some ranges of mechanical properties of compacted samples of the materials whose index properties are shown in Table 2. Except for samples 1a, 1b, 2a, and 6a, which were compacted using standard Proctor, the maximum density and optimum water content values correspond to modified Proctor compaction.
The mechanical behavior of decomposed granite soils is more complex than of that conventional granular soils. Most decomposed granites show substantial particle breakage under relatively low loads. The possibility of the presence of mica flakes in some stage of weathering can increase the compressibility and reduce the shear strength. In addition, some of the original weakness of the parent rock such as joints, faults and slickensides, can become also weakness of the residual soil.

Particle breakage

Decomposed granite soil particles are more susceptible to breakage than many other types of soil particles because of: (1) the angularity of the particles; (2) the presence

Table 2. Index properties of some decomposed granite soils (Yapa, 1991)

No.	Location	Type	Grav.	Sand	Fines	G_s	LL	PI	Class.	Reference
1a	Colombia	Silty sand	0-25	50-75	0-25	2.7-2.75	30-50	0-15	V-SM	Li & Mejia (1967)
1b		Sandy silt		25-75	25-75	2.7-2.75	30-50	0-15	V-ML	
2a	Shasta Bally Batholith California USA	33R-1		40	60	2.72	26	6	CL-ML	USBR (1960)
2b		33R-2		50	50	2.71		NP	SM-ML	
2c		33R-3		48	52	2.73		NP	ML	
3a	Shasta Bally Batholith California USA	B2R-1	8	70	22	2.69		NP		Duffy (1990)
3b		B2R-4	15	73	12					
3c		B2R-5	7	74	19	2.72				
4a	N. Sierras California USA	Stockpile	5	75	20			NP		Macfarlane (1990)
4b		Native soil	18	64	18					
5a	Hong Kong	Granite-clayey	25	25	50	2.67	92	52	VI-CH	Gifford & Chan (1969)
5b		Granite-sandy	25	45	30	2.66	46	10	V-SM	
5c		Granite-dam	15	55	30		29	10	V-SC	
5d		Granodiorite	15	35	50		42	11	V-ML	
5e		Granodiorite	15	35	50		35	14	V-CL	
6a	Japan	Kanakuma	9	58	34	2.65	39	10	SM	Onitsuka & Yoshitake (1990)
6b		Kawakubo	24	66	10	2.71	35	10	SM	
6c		Saga	36	53	11	2.64		NP	SM	
6d		Washed saga	26	73	1	2.63		NP	SP	
7a	Japan	Sl. weathered		100		2.64				Nishida & Kagawa (1972)
7b		Hi. weathered		100		2.62				

Table 3. Ranges of properties of some compacted decomposed granites (Yapa, 1991)

No.	Class.	$(\gamma_d)_{max}$ (kN/m³)	w_{opt} (%)	c'(kPa)	ϕ' (°)	C_c	$c_v(10^{-8}m^2/s)$	$k(10^{-9}m/s)$
1a	V-SM	15.9-16.3	19-22	0-10	32-38			
1b	V-ML	14.8-15.5	23-25	0-10	32-38			100
2a		18.5	13.3	76	29			
2b		19.8	10.9					
2c		19.8	10.6					
3a				27.5	35	0.15	177-322	625-7640
3b				90	37			590-3640
3c				21	42		177-281	22-1146
4a				27.5	37.5			
4b				21	35			
5a	VI-CH	14.4	30.7	0	32	0.15	200	0.6-7
5b	V-SM	14	32.3	14	36.5	0.14	10000	60-216
5c	V-SC	18.4	12.3	21	37		11-66	
5d	V-ML	16	20.2					
5e	V-CL	16.8	18.6	17	31.3		9-22	
6a	SM	17	16	14-55	30-36			
6b	SM			21-34.5	30-35			
6c	SM	19	12			0.32(0.17)		
6d	SP			7-41	36-46	0.25(0.13)		
7a					37.7-41.7			
7b					37.7-41.7			

of minerals of different compressive strengths; (3) the high intragranular void content; and (4) the coarse, uniform grading. Externally applied stresses, soaking, and compaction increase particle breakage. Samples containing large particles are required to properly establish the effects of compaction on breakage of particles(Marsal,1973; Miura and O'Hara,1979; Onitsuka and Yoshiyake,1990; Feda, 1977;Yapa,1991). Comparison of particle size distributions before and after compaction or strength testing is the most common means for quantitative measurement of particle breakage. Several indices reported in the literature that are derived from these determinations are given by Yapa (1991).

Compaction characteristics

Data on the compaction characteristics of some decomposed granite soils are given in Table 3. Some decomposed granite soils have oversize particles; however, conventional compaction test equipment allows only particles up to 3/4 inch (18 mm). Scalping the oversize particles is known to significantly affect the maximum dry density determined in the laboratory (Siddiqi et al, 1985).

Repeated use of soil produces higher densities because of particle crushing. The gain in density is greater in samples with coarser particles and in samples with a high degree of weathering (Furukawa and Fujita, 1990). Thus to better represent the field conditions, specimens should not be repeatedly used in laboratory moisture-density tests. Specimens initially at natural moisture content produce higher densities than those which are gradually wetted from an oven-dried state. This effect is noticeably greater in highly weathered soils. In addition the optimum moisture content increases and the maximum dry density decreases with increased weathering.

Hydraulic conductivity

For a given void ratio the hydraulic conductivity decreases as the degree of weathering increases. This results from a decrease in void size as the degree of weathering increases. General values of mass hydraulic conductivity for the six grades of decomposed granite are indicated in the classification system given in Table 1. Typical values obtained for Grade V soil are on the order of 4×10^{-3} to 5×10^{-9} m/s, and for Grade VI soil on the order of 2×10^{-6} to 5×10^{-9} m/s (Li and Mejia,1967; Dearman et al,1978; GSGEWP, 1990).

Compressibility characteristics

Decomposed granite soils, like residual soils, may show a yield stress, sometimes called an apparent overconsolidation pressure, which reflects the presence of weak bonding and microfabric, either retained from the parent rock or generated during weathering. In this condition, the deformation of the in-situ decomposed granite soils will be small unless the yield stress is exceeded. Once the yield stress has been exceeded strains increase significantly (Vaughan et al,1988; GSGEWP,1990; see also the section on Compressibility of Lateritic Soils in this paper).

Values for the compression index, C_c, for Grade V granitic soils are in the range of 0.15 to 0.30. Consolidation is usually rapid (Dearman et al, 1978). Li and Mejia (1967) found that the total settlement of the decomposed granite soil foundation in the construction of some dams was about 4 percent of the overburden depth, and that more than 90 percent of the settlement took place during construction.

Collapse of soil stucture on wetting may occur, especially where ground water gradients elluviate the finer fraction producing an open structure (Dearman et al., 1978; Massey et al, 1989; GSGEWP,1990; section on Collapsible Soils in this paper).

Strength characteristics

The shear strength of decomposed granite soils is influenced by the microfabric and the variability of the void ratio which are inherited from the parent rock, by the weathering processes, and by partial saturation that can occur to considerable depth. Strength degradation (as measured by ϕ) can be significant in decomposed granite soils, even under low pressures, due to particle breakage and to polymineral composition (Miura and O'hara, 1979; Feda, 1977; Massey et al, 1989; Yapa, 1991).

Typical values from laboratory and in situ tests for the effective friction angle (ϕ) are between 30° and 40° for Grade V granitic soil, and between 20° and 40° for Grade VI soil. The effective cohesion (c) generally is in the range of 0 - 200 kPa for Grade V soil, and between 0 - 225 kPa for Grade VI soil, depending on the degree of saturation, and usually disappears when the soil is fully saturated, probably due to a reduction in capillary forces(suction). Grade VI shows more variability in strength properties, probably as a result of loss in granite texture and the coatiing of resistant grains clay (Lumb, 1962; Li and Mejia, 1967; Dearman et al, 1978; Brand et al, 1983; Massey et al, 1989).

Case history

Four earth dams, with heights varying from 25 to 55 meters, were succesfully constructed in the area of Medellin, Colombia, in the period of 1949 - 1965 (Li and Mejia, 1967). These dams are all of rolled residual soil earthfill and are founded on deep residual soil overburden.

An intrusive, late Cretaceous (about 70 million years old) igneous rock formation is present, in which the predominant rock is a quartz-diorite of medium to moderately coarse grains, composed largely of quartz, feldspar, hornblende, and biotite. The sub-tropical climate and heavy rainfall have produced a uniform residual soil overburden to an average depth of 20 to 30 meters. The gradation of the residual soil varies from clayey silt at the ground surface, through sandy silt, to coarse sand next to the rock crust. The mechanical properties and behavior of the decomposed granite foundation material were consistent with those cited earlier in this section. The index properties and the properties of the compacted material that was used in the fill are included in Tables 2 and 3. Li and Mejia (1967) believe, however, that the cohesion value in the field was considerably higher than shown by laboratory tests.

The dams were constructed entirely of decomposed granite soil, and the dam sections were essentially homogeneous. The most important and difficult problem encountered in designing and building these dams was the development of high construction pore pressure in the fill. In addition, the material was invariably wetter than standard Proctor optimum, requiring drying and limiting the fill placement to the 3-1/2 months dry seasons.

Measures were taken to guard against possible failures in the foundation and in the fill. Among them were: (1) No excavation into the in situ residual soil overburden was made in the immediate vicinity of the upstream site of the dam; (2) the principal waste areas were provided at both the upstream and downstream toes of the dam to serve as counter weight fills and as an impervious blanket (upstream); (3) geotechnical instrumentation was provided, which included an extensive piezometer system in the fill and settlement gages and surface settlement points; (4) frequent checking of the embankment stability during construction based on the measured pore pressures, and modification of construction practices to minimize the development of pore pressure.

Both laboratory and field tests, and the post-construction settlement measurements, indicated minimal particle breakage in the fill and collapse of the soil structure in the loaded foundation.

LATERITIC SOILS (FERRUGINOUS AND FERRALLITIC SOILS)

Whereas residual soils are the products of in-situ weathering of any type rock parent material, the term, "laterite", was originally given by Buchanan in 1807 to a fairly small group of red residual soils which harden irreversibly on exposure to air. The name has since been applied to many red, tropical and subtropical soils. Duchaufour(1982),in the French classification system (see also GSEGWP, 1990), distinguished three phases of increasing intensity of weathering in residual soil development in tropical areas: 1 - fersiallitic soils; 2 - ferruginous soils; and 3 - ferrallitic soils. The soils in phase 2 in transition to phase 3 are termed ferrisols. The materials that are commonly classified as lateritic soils and laterites generally correspond to the last two development phases. Herein, the term lateritic soils is used to refer specifically to the residual soils from crystalline rock in the development phases 2 and 3.

Formation processes and distribution

Parent material, climate, topography and the age of the land surface influence the characteristics of a residual soil (or determine the phase of residual soil development). In tropical regions rock weathering is intense and occurs to great depths. With abundant rainfall, high temperatures, good drainage and crystalline rock parent materials, the feldspars weather initially to kaolinite, hydrated iron and aluminum oxides are formed, and the more resistant quartz and mica particles may remain. As the weathering proceeds the content of kaolinite decreases and the hydrated iron and aluminum oxides, geothite and gibbsite, progressively alter to hematite and boehmite, Fe_2O_3 and $Al_2O_3 \cdot H_2O$, respectively. Because of the high iron concentration, the resulting soils are usually red in color. When dried,such materials may harden as a result of the cementing action of the iron and aluminum oxides, and they are commonly referred to as laterites.

Highly altered minerals in horizons near the surface (ferrallitic) often pass downwards to less altered (fersiallitic) horizons. Clay contents often decrease downward, and 1:1 layer lattice minerals (e.g. kaolinite) may change to 2:1 minerals (e.g. smectite), giving significantly

different engineering characteristics. The sequence of horizons at any one site constitutes the soil profile and a typical form is shown in Figure 3. In nature, boundaries between the layers are not always clearly defined.(Gonzalez de Vallejo et al.,1981; Mitchell and Sitar, 1982; GSEGWP, 1990).

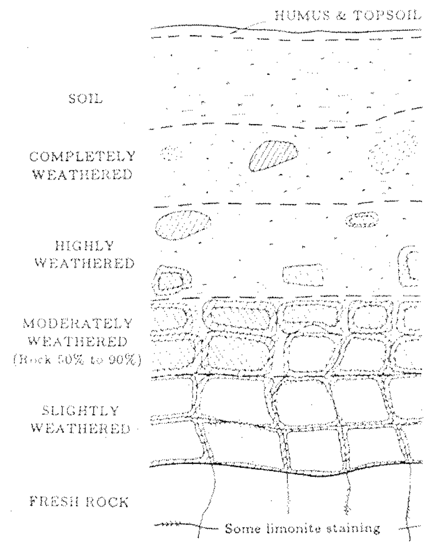

Fig. 3 Schematic diagram of tropical residual soil profile (from Little, 1969).

Duchaufour(1982) distinguished three phases of residual soil development in tropical areas (Table 4). They are characterized by increasing weathering of primary minerals, increasing loss of silica and increasing dominance of new clay minerals formed from dissolved materials.

Table 4. Residual soil phases (Duchaufour, 1982)

Phase	Soil type	Zone	Mean annual temperature (°C)	Annual rainfall (m)	Dry season
1	fersiallitic	mediterranean, subtropical	13-20	0.5-1.0	yes
2	ferruginous ferrisols (transitional)	subtropical	20-25	1.0-1.5	sometimes
3	ferrallitic	tropical	>25	>1.5	no

The macro structure of residual soils is primarily dependent on the weathering grade of rock mass (see Figure 3). Lateritic soils (grade VI) generally bear no resemblance to rock structure. However, they commonly have a granular structure,with particle aggregation into clusters that are cemented by iron or aluminum oxides. The soils may have hardened partially or extensively into pisolitic, gravel-like, or rock-like masses. They may also be relatively soft but with the properties of self-hardening after exposure. The result of the different micro structure possibilities is that lateritic soils may display significantly different physical properties depending on the stage of the weathering process (Mitchell and Sitar,1982; Townsend et al., 1982).

The lack of a standardized definition of "laterite" causes difficulty in delineating the geographical distribution of lateritic soils. Maps usually show regions of possible distribution based on climatic conditions, since climate is the principal factor controlling formation, and thus distribution. A simplified distribution of lateritic soils in the Americas is given in Figure 4. The main areas of distribution worldwide are located in North and Northeastern South America, Central Africa, and throughout Southeast Asia. A region of lateritic soils is also located in Southern North America. Recent studies have determined that most mature lateritic profiles formed during the late tertiary period, suggesting that they are distributed beyond the present day boundaries of the tropical zone.

Engineering classification and correlations

Traditional engineering soil classifications based on plasticity and grain size analyses have not always been effective in predicting the engineering behavior of lateritic soils. The classifications proposed for residual soils, including laterites, can be divided into four types (GSEGWP, 1990): (1) those that depend on environmental criteria and are useful in a geographical distributive sense; (2) those that employ strict pedological criteria, such as the silica-sesquioxide ratio; (3) those that merely extend the standard geotechnical soil classifications; (4) those that include a special parameter of particular relevance to a specific end use of tropical soil as an engineering material.

According to the GSEGWP(1990), none of these methods gives a satisfactory coverage for the variety of residual soil that may exist. They proposed a detailed engineering (genetic) classification of tropical residual soils based

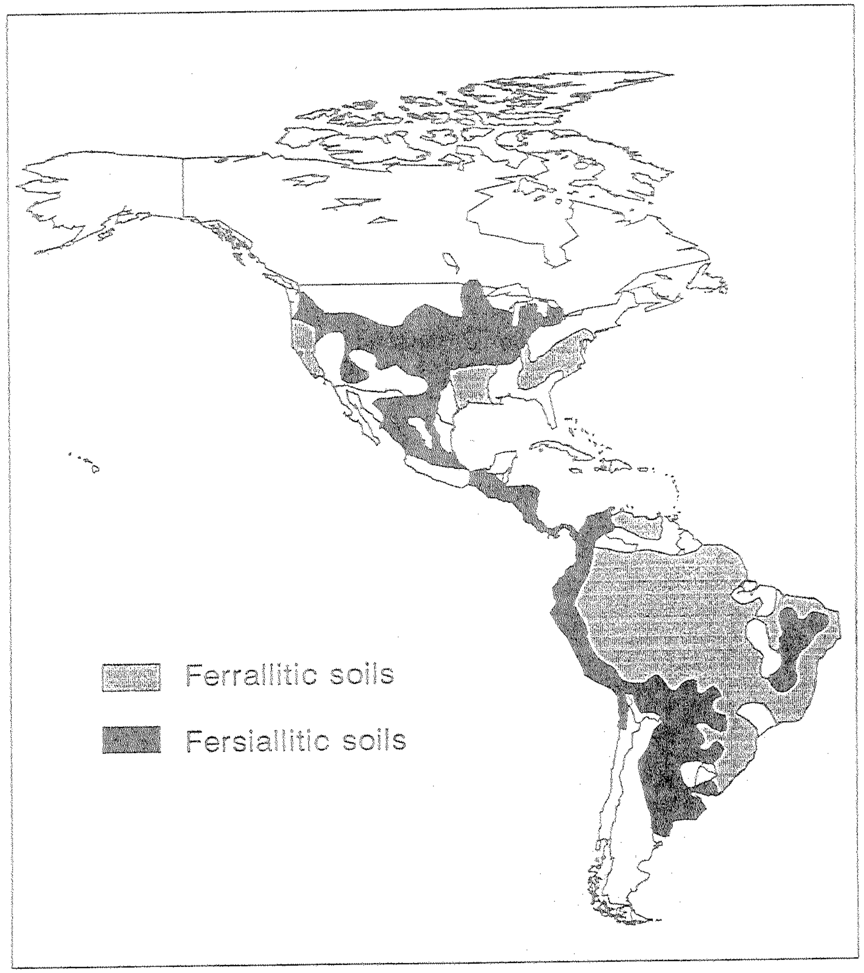

Fig. 4 Distribution of lateritic soils in the Americas (from GSEGWP, 1990).

on pedogenetic criteria and on the work of Duchaufour (1982).

It may be difficult to relate index parameters to engineering properties of lateritic soils, because both the composition and structure are important. The specific gravity of iron bearing minerals is high compared to most other rock and tropical soil forming minerals. Tuncer and Lohnes(1977) proposed a model for stages of soil formation and variation in engineering properties for lateritic soils based on variation in specific gravity and its relationship to engineering properties.

The silica-sesquioxide mole ratio, $K_r = (\%\ S_iO_2/60)/[(\%\ Al_2O_3/102)+(\%\ Fe_2O_3/160)]$ is commonly used to indicate the extent of weathering and to differentiate soil types. Values of $K_r < 2.0$ are typical of oxide-rich ferrallitic soils, ferrisols and some ferruginous soils, whereas fersiallitic and some ferruginous soils have K_r values > 2.0. The value of this ratio also depends upon the composition of the soil parent material and on the concentration either silica or sesquioxides by processes other than weathering (e.g. accumulation of Fe_2O_3 by podzolization of hydromorphic segregation). Hence the ratio does not correctly classify all soils in the French or other systems (GSEGWP, 1990).

More recently, an index parameter method of classification based on void ratios was proposed by Vaughan et al. (1988). The "relative void ratio", e_r is defined by the equation : $e_r = (e - e_{opt})/(e_l - e_{opt})$ where e, e_{opt}, and e_l are the void ratios at the natural state, optimum moisture content as determined by Test 11 of BS 1377:1975, and liquid limit, respectively. The relative void ratio is analogous to the liquidity index used for sedimentary fine-grained soils and can be related to the engineering properties.

Index properties

The range of particle sizes in a lateritic soil may be very large, from discrete, clay-sized particles to gravel-sized aggregates. Thick accumulation of laterite, a type of duricrust, formed under extreme conditions may behave like rock. Measured grain size distributions may have limited usefulness in lateritic soils, because they depend on the degree of drying and the treatment of the soil before and during the determination. The plasticity of a lateritic soil may be highly variable, depending on the presence of clay mineral (kaolinite) and particle aggregates. Younger, less weathered, lateritic soils(ferruginous soils) have a plasticity comparable to that of kaolinite clay. More

mature, well weathered soils(ferrallitic soils), that have more aggregates and less kaolinite content, are usually less plastic. A broad grouping of the plasticity properties of many lateritic soils is shown in Figure 5. It is known, however, that not all laterites fall within the range indicated. Plasticity increases as result of mechanical manipulation (mixing time) but decreases as result of drying. Probably the most important use of the index properties, including compaction characteristics, is in identifying the soils which change their properties as a result of drying, remolding and reusing(Townsend et al., 1971; Mitchell and Sitar,1982; GSEGWP, 1990).

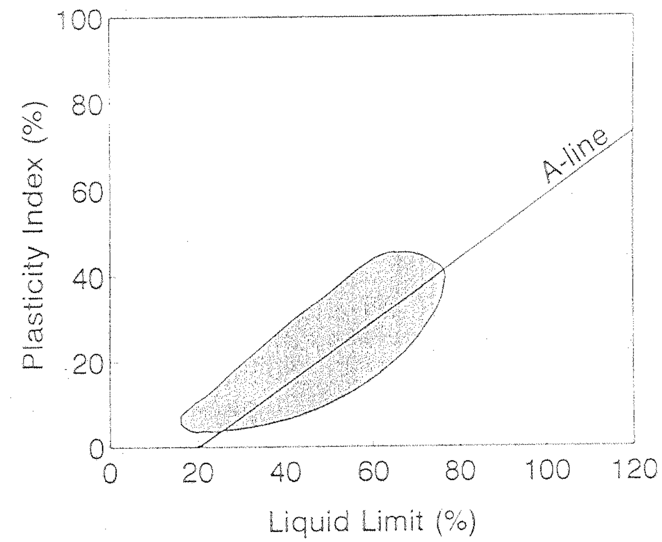

Fig. 5 Approximate range of Atterberg limits for lateric soils (modified from Mitchell and Sitar, 1982).

As the geotechnical properties of laterites are strongly dependent on the structure of the soil, it is necessary to distinguish between the undisturbed, compacted, and remolded states.

Compaction characteristics

The compaction characteristics of lateritic soils are influenced by gradation, crushing strength of the coarse fraction, method of pretreatment or sample preparation, mineral composition, and compactive effort. Consequently, the compaction characteristic may vary over a wide range. Values of dry unit weight range from a low of 13 kN/m^3 to a high of 22.8 kN/m^3, with most of the values falling between 17.3 and 22 kN/m^3 (De Graft-Johnson et al., 1969; De Graft-Johnson et al, 1972; Townsend et al, 1971). The corresponding optimum moisture contents range from 6 to 22 percent. The optimum moisture content increases and the maximum dry unit weight decreases with increasing clay size particle content (Gidigasu,1972).

Some effects of methods of sample preparation and laboratory procedure are illustrated in Figure 6. Oven-dried sample show higher maximum dry density and lower optimum moisture content than samples air dried to the point at which they are tested. Similarly, if the same sample is reused in all the tests, particle breakdown leads to a higher maximum dry density and lower optimum moisture content. An effort should be made to match compaction conditions used in the laboratory to those expected in the field.

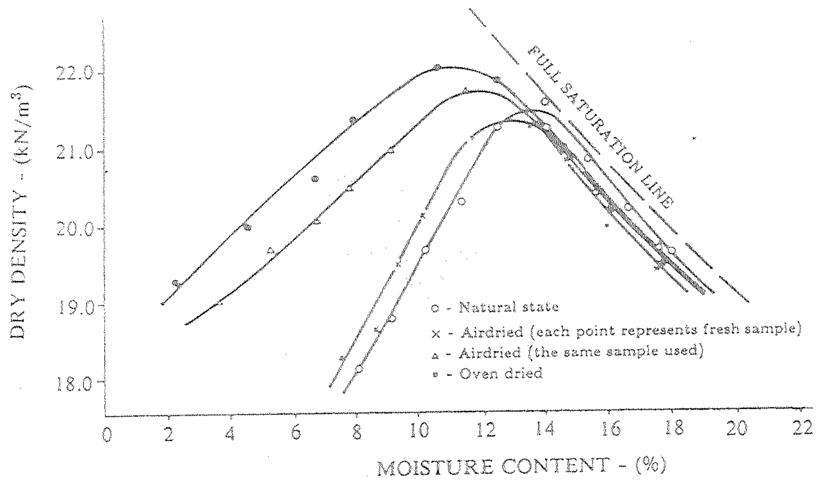

Fig. 6 Influence of methods of sample preparation and laboratory procedure on compaction characteristics of lateritic soils from Ghana (from Gidigasu,1974)

Hydraulic conductivity

Laterite soils often have an open structure and may have hydraulic conductivity values as high as 10^{-4} m/s. Hydraulic conductivity does not normally decrease significantly under loading until the yield stress (see Compressibility) is exceeded, when it may drop significantly. Owing to the structure, the hydraulic conductivities may be unacceptably

high for the foundations of water retaining embankments. In addition, the activity of termites and other insects can create large diameter and potentially dangerous seepage passages (Cadman and Buosi, 1985-see "Case History"; de Mello, 1988; GSEGWP, 1990).

The hydraulic conductivity of compacted lateritic soils is typically in the range of 10^{-8} to 10^{-10} m/s, and the coefficient of consolidation ranges from 10^{-5} to 10^{-7} m^2/s (Ola, 1980; Saunders and Fookes, 1970).

Compressibility characteristics

There is usually some inter-particle bonding in lateritic soils, and the soil exhibits a yield stress. This is defined as a stress or stress state at which there is a discontinuity in stress- strain behavior, and a decrease in stiffness. This yield stress is similar to that of an overconsolidated sedimentary soil, except that it is caused by chemical bonding between particles instead of precompression. The deformation of lateritic soils in situ will be small unless the yield stress is exceeded. Once the yield stress has been exceeded bonding is progressively destroyed as strains increase. The slope of a void ratio vs log stress plot from a one-dimensional or isotropic stress compression test after yield is a function of the yield stress and initial porosity of the soil,as shown on Fig. 7, rather than being an intrinsic function of the grading and mineralogy of the soil (Vaughan et al., 1988). Typical correlations are shown on Fig. 8. The state of an in situ soil according to the zones of Fig. 7 can be assessed by comparing in situ stress and void ratio with the compression line for the remolded and de-structured soil. The inter-particle bonding is particularly sensitive to disturbance during drilling and sampling and may easily be damaged. If there has been disturbance, then yield stress values determined from laboratory tests are probably underestimated (Vaughan et al. 1988; GSEGWP, 1990).

Vargas(1974) found the following relationship between the compression index, C_c, and the liquid limit for Brazilian laterites: $C_c = 0.005(LL + 22) \pm 0.1$. This relationship gives values very close to those that would be predicted using the relationship suggested by Terzaghi and Peck (1977) for temperate zone soils: $C_c = 0.009(LL - 10)$. There is no information given as to whether these values are for C_c or C_{cs} as defined in Fig. 7.

Even though some open textured lateritic soils can withstand reasonably large stress when partly saturated, they may collapse on wetting, even under low stress (Mitchell

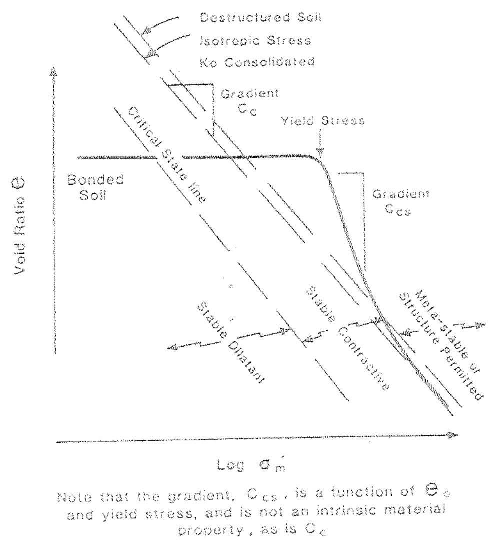

Fig. 7 Stress-void ratio state of a residual soil related to the possible states for a de-structered soil (from GSEGWP,1990).

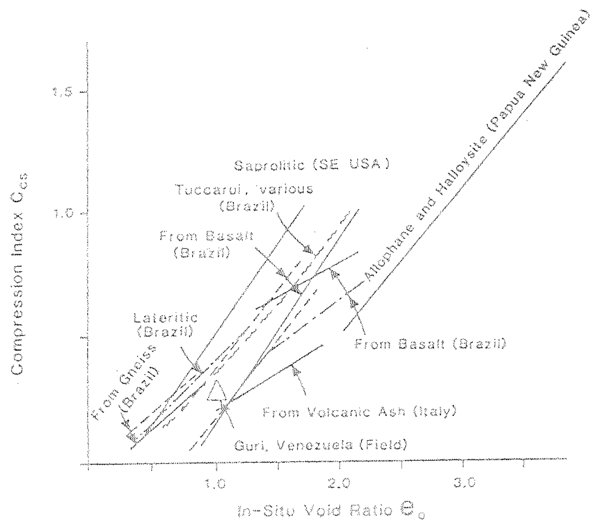

Fig. 8 Compression index C_{cs} vs. initial void ratio e_0 (Wallace,1973; Lacorda et al,1985; from GSEGWP, 1990).

and Sitar,1982; GSEGWP, 1990; see section Collapsible Soils).

Strength characteristics

The shear strength of lateritic soils is significantly influenced by: (1) the presence of bonding between particles, that gives both a component of strength and of stiffness; (2) widely variable void ratio which is a function of the weathering process and is not related to stress history; and (3) partial saturation, possibly to considerable depth(GSEGWP, 1990). The measurement of shear strength of tropical soils requires samples of high quality, and test specimens should be large enough to include large particles fabric elements.

Effective stress friction angles typically range between 20 and 30 degrees for the lateritic clays, and between 30 and 40 degrees for lateritic gravels (Townsend, 1985). Some representative values of strength parameters for lateritic soils are given in Table 5. The strength values obtained on tests performed on compacted specimens vary over a wide range. Reported values of friction angle range from about 18 to 41 degrees, with the majority of the results falling into a range from 28 to 38 degrees. The values of cohesion range from zero to well in excess of 48 kPa (Mitchell and Sitar, 1982).

Table 5. Strength parameters for some lateritic soils (soils 1-5 from Townsend, 1985; soil 6 from Vargas, 1974)

No.	Soil type & location	ϕ' (°)	c' (kPa)
1	Lateritic gravels, Africa	37.5	0 - 40
2	Lateritic clays, Africa	22.5	0 - 100
3	Lateritic soil, Panama	38	0
4	Granitic lateritic soil, Venezuela	21.5	20
5	Granitic laterite, Brazil	31	0
6	Lateritic soil, Brazil	23 - 33 Mean-28	0 - 59 Mean-24

Many in situ lateritic soils are in a partially saturated condition, and an increas in the degree of saturation causes a reduction in strength as a consequence of the decrease in soil suction and effective stress. An increase in moisture content due to saturation can also cause softening and solutioning of interparticle cementation also leading to a reduction of shear strength. However, Bressani and Vaughan (1989) found that the value of ϕ is not significantly altered by saturation. Methods for examining the effect of pore water suction on strength are discussed by Fredlund and Rohardjo (1985).

Case history

Three dams were built in various regions of the Amazon Basin, Brazil for electric power supply. Tubular cavities varying from millimeters to up to 30 cm in diameter were found at each site (Cadman and Buosi, 1985). Located primarily within the lateritic soils, they were found at depths of up to 30 meters. The tubular cavities were first discovered in 1978 at the Tucurui Dam site. Statistical analysis of water loss tests in boreholes in the foundation gave unexpected results. Test pits and trenches were dug to study the anomalies and revealed sub-vertical and sub-horizontal tubular cavities. Subsequent investigations at the other two dam sites (Balbina Dam and Samuel Dam), disclosed similar cavities.

Studies performed on undisturbed samples, including erosion tests, geochemical and morphological studies, and entomological studies indicated that the cavities were the result of termite activity during times of dry climate in the Quaternary period, probably more than 10,000 years ago.

Different treatments were utilized at each dam site to eliminate the risks of internal erosion and to avoid the problem of high percolation losses. At the Tucurui Dam, which has a maximum height of 100 meters, cutoff trenches, excavated to at least one meter below the active zone, were used to overcome the foundation problems. At Balbina Dam, which has a maximum height of 42 meters, it was decided to use injection of a clay-cement grout and small trenches in region with the largest cavity systems. At Samuel Dam, which has a maximum height of 51 meters, the tubular cavities occur only in the foundation area of the dikes and are of small diameter. Thus, it was considered unnecessary to have extensive special foundation treatments.

VOLCANIC ASH SOILS (ANDOSOLS)

Andosols are young, fine-grained soils derived from the weathering of recent volcanic ash material and occur in many parts of the world. The word "andosols" is from the FAO-UNESCO(1974) taxonomy and means dark soils.These soils have also been referred to as andisols and andept in United States; humic allophane soils or andisols in Japan; yellow-brown loams in New Zealand; and trumao or allophanic soils in Chile and Argentina. The unusual composition, structure and fabric that are developed in these soils mean that in the natural state they can have unusually high natural water content but are relatively stable; however, when disturbed, these soils can sometimes turn into semi-liquids, and cause construction and stability problems.

Formation and distribution of andosols

Andosols are young soils that developed from Quaternary or even Recent volcanic ash. Ages as short as 500 to 1500 years are cited. The high surface area of volcanic glass fragments and their lack of organized crystal structure contribute to their rapid weathering, which leads to formation of complex mixtures that can include glass, allophane, imogolite, halloysite, kaolinite, and masses of gel-like amorphous materials. In tropical climate, concentrations of iron and aluminum oxides occur. Montmorilonite may also develop during early stages of weathering of volcanic materials.

Allophane is an amorphous silicate soil mineral which has high porosity, high cation and anion exchange capacity, high water adsorption capacity, and very high chemical activity due to its high specific surface area. It may exhibit irreversible volume change behavior. Halloysite, when subjected to prolonged air drying or dried at a temperature greater than about 60°C loses part of its internal water. This dehydration is irreversible, and the properties of the soil change. Allophane and halloysite are most prevalent in wet climates (rainfall in excess of 1500 mm per year) in tropical areas. The fabric of volcanic ash soils commonly has a granular character, with particle aggregation into clusters that may be cemented by sesquioxides (aluminum oxides) (Gonzalez de Vallejo et al.,1981; Mitchell and Sitar, 1982; Morin, 1982; Townsend, 1985).

Volcanic ash soils are found close to active and recently dormant volcanos, which in the Americas are generally within mountainous regions approximately parallel to the western coast, in steep, rugged terrain. Andosols have also been found at sea level in various parts of the world. They are located in climates that range from wet, tropical to cool, semi-arid. Within the Americas, volcanic ash soils have been found in the northwestern United States, including Alaska and Hawaii, Central America including the Caribbean,and the western coast of South America. Figure 9 shows the distribution of volcanic ash soils in the Americas, as defined by FAO-UNESCO (1974).More detailed distributions are shown by Leamy et al.(1980) and Atlan(1990).

Index properties

The surface horizon is usually dark brown or black in color, and sometimes it is quite thick. The color is less dark in tropical climates, showing a more "vivid" dark brown or red brown. Volcanic ash soils have unusually high natural water content, high liquid limits and plastic limits and low plasticity indices. These soils typically plot below the A-line, as shown in Figure 10. Soils with relatively low natural water contents plot closer to the A-line than those with higher natural water contents. Plasticity increases as result of mechanical manipulation(mixing time) but decreases as result of drying (see example in Figure 10).

Measured grain size distributions have limited usefulness in volcanic ash soils, because they depend on the degree of drying and the treatment of the soil before and during the determination. Volcanic ash soils usually contain more than 5 percent organic carbon. In the surface horizon the organic content is high, with a average of 18-20 percent.

Soil classification systems for soils from temperate climates do not generally apply to young volcanic soils. Probably the most important use of the index properties, including compaction characteristics, is to help identify the soils which change properties as a result of drying and remolding (Morin and Todor, 1975; Leamy et al. 1980; Mitchell and Sitar,1982; Atlan,1990; Thrall and Bell,1989).

Mean values for index properties are given in Table 6. Natural water contents range from a low of less than 10 percent to a high of more than 300 percent, showing statistically three main groups, with the higher water content representing a small number of samples. Soils of the low water content group typically have high halloysite contents. High allophane percentages are typical of the higher water content groups.Liquid Limits range from 23 to 350 percent for samples initially in their natural condition, and 40 to 111 percent for previously dried samples. Similar results for andosols from Martinique were reported by Atlan (1990).

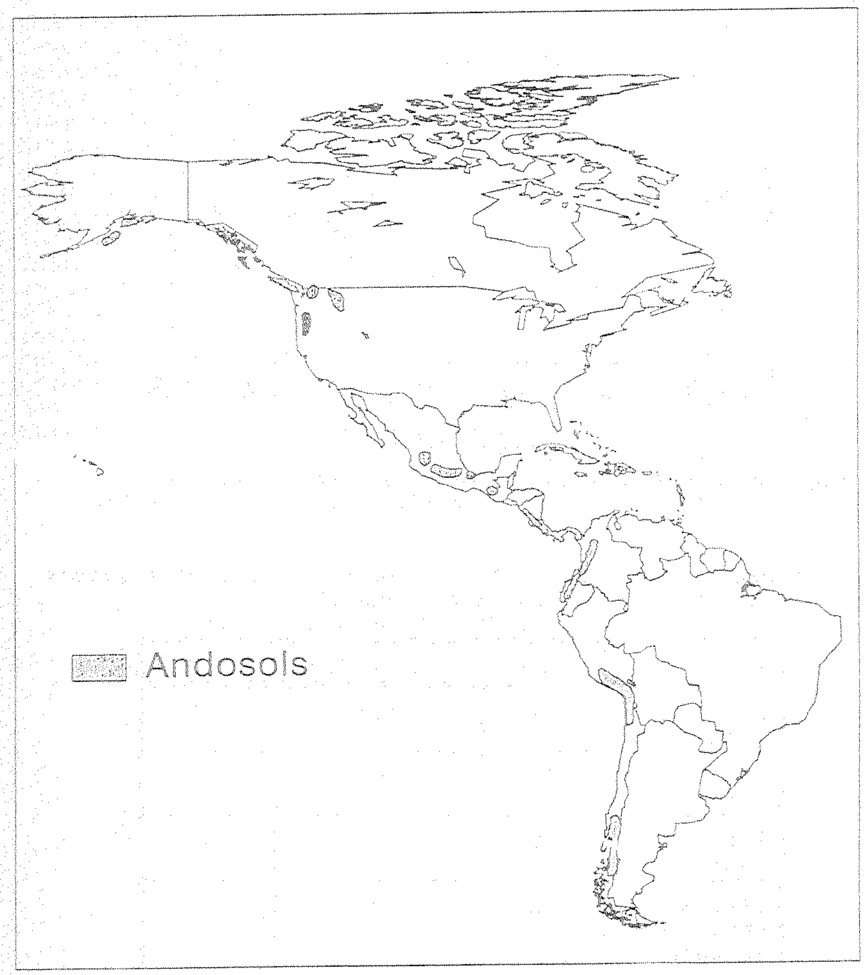

Fig. 9 Approximate distribution of andosols in the Americas (after FAO, 1974).

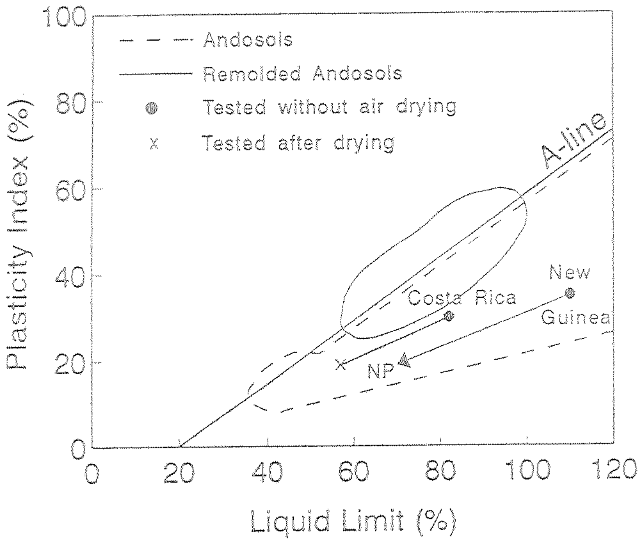

Fig. 10 Approximate ranges of Atterberg limits for andosols (modif. from Mitchell and Sitar, 1982).

Table 6. Mean values for index properties of andosols in their natural (undried) conditions (from Thrall and Bell, 1989)

Property	Low	Intermediate	High
Liquid limit (%)	64	116	180
Plasticity index (%)	15	38	64
Water content (%)	38	111	>200
Standard deviation in water content	12	35	

<u>Compaction</u>

The compaction characteristics of volcanic ash soils are influenced by gradation, crushing strength of the coarse fraction, method of pretreatment or sample preparation,

mineral composition, and compactive effort. Consequently, the compaction characteristics can vary over a wide range.

The maximum dry unit weights of andosols tend to be much lower than is typical of other tropical soils, such as laterite, or of temperate climate soils. For example, Matyas (1969) reports a range of values for the Sasumua clay from 9.5 to 10.8 kN/m^3 for material that has never been dried. The corresponding optimum moisture contents of 53 and 63 percent are significantly greater than usual for the other soil types. Oven dried samples have higher maximum dry density and lower optimum moisture content than samples that are only air dried before they are tested. Figure 11, from Atlan (1990), illustrates this for samples of allophane soils from Martinique. Halloysite soils show similar behavior with higher values for the maximum dry density and smaller values for the optimum moisture content after drying. Thrall and Bell(1989) found linear correlations between optimum water content and plastic limit for the low and high natural water content groups, in samples which have not been previously dried (Table 7).

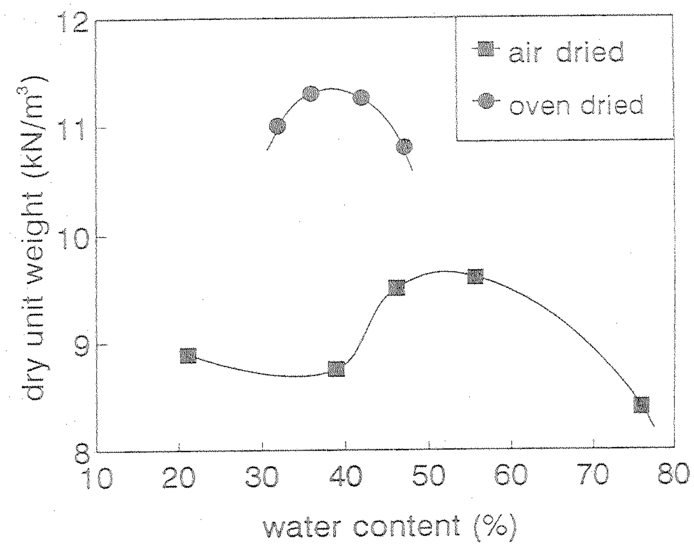

Fig. 11 Typical Proctor compaction curves for allophane soils (from Atlan, 1990).

<u>Hydraulic conductivity and compressibility</u>

The hydraulic conductivity and compressibility of volcanic soils are strongly dependent on the structure of the soil, and it is necessary to distinguish between the undisturbed and the compacted or remolded soil properties. The permeability of compacted tropical residual soils is typically in the range of 1x 10^{-8} to 1x 10^{-10} m/s, and the coefficient of consolidation ranges from 1x 10^{-5} to 1x 10^{-7} m^2/s (Ola, 1980; Saunders and Fookes, 1970). Wesley (1974) studied andosols from Java and obtained hydraulic conductivity values around 3 x 10^{-10} m/s for samples compacted at their natural water content. Published permeability data for undisturbed andosols are very limited. Most often it is assumed that andosols, like residuals soils, have an open flocculated structure, and may exhibit permeabilities as high as 10^{-4} - 10^{-5} m/s.

Often it is assumed that andosols are relatively less compressible than temperate zone soils due to cementation. Thrall and Bell (1989) found a logarithmic relationship between compression index and void ratio for the high natural water content group (allophane mineral) (see Table 7). The cementation in soils formed in regions with a distinct dry season can be weakened by saturation and lead to a collapse of the soil structure (Foss, 1973; Mitchell and Sitar, 1982; section on Collapsible Soils, this paper).

Strength

Despite their high natural water content, undisturbed andosols are unusually strong. Values of friction angle for andosols from Hawaii, and Kenya have been reported in the range of 27 to 57 degrees, with averages between 35 and 42 degrees. Cohesion values are highly variable, from 20 to 350 kPa. Moisture significantly influences the strength characteristics of these andosols: (1) the Hawaiian andosols weathered under conditions of higher rainfall have lower strengths; (2) andosols from Kenya show at least 50 percent reduction in cohesion and, in several cases, a corresponding drop in the friction angle of about 30 percent, as a result of saturation (Foss, 1973; Tuncer and Lohnes, 1977; Mitchell and Sitar, 1982). Wesley (1974) obtained values of friction angle in the range of 35 to 40 degrees, and cohesion values from 15 to 20 kPa for samples compacted at their natural water content.

Thrall and Bell (1989) in their statistical study found that the effective angles of internal friction (ϕ) for undisturbed andosols are divided into separate distributions, similar to the water content and Atterberg limit distributions. The low water content group corresponds to a ϕ' averaging about 38 degrees. The medium and high water content groups correspond to ϕ' averaging about 18 degrees. Their study indicated an average value of about 56 kPa for

Table 7. Property correlations for andosols (after Thrall and Bell, 1989)

Engineering vs. Index Property	Group	Andisol Regression Equation[a]	Crystalline Soil Regression Equation[b]
Optimum water content (w_{opt} in %) vs plastic limit (PI in %)	low w_n group	w_{opt}=0.77(PL+5.7) (0.78)	
	high w_n group	w_{opt}=0.46(PL+53.8) (0.72	w_{opt}=0.38(PL+16.5)[d] (McDonald, 1972)
Compression index (C_c) vs void ratio (e)	high w_n group	C_c=0.5(ln e + 0.48) (0.92)	C_c=0.75(e - 0.5) (Azzouz, 1976)
Effective angle of internal friction ϕ' (°) vs void ratio (e)	low w_n group	ϕ'=-7.30(e-7.4) (0.56)	ϕ'=-26(e-2.1)[c] (U.S. Navy, 1971)
	high w_n group	ϕ'=-1.02(e-23.8) (0.35)	
Undrained shear strength (S_u in kPa) vs natural water content (w_n in %)	low w_n group	S_u=4100 exp(-0.1w_n) (0.44)	S_u=355,000 exp(-0.22w_n) Sherif and Burrows, 1969)
	high w_n group	S_u=1100 exp(-0.03w_n)	

[a] The regression coefficient is in parenthesis.
[b] Reference for equation is in parenthesis.
[c] For ML soils.
[d] For soils of low plasticity

the cohesion intercept for all groups . The correlations obtained between effective angle of internal friction and void ratio are shown on Table 7. Correlations between undrained shear strength and natural water content are also given in Table 7.

Atalan(1990) points out that slopes in volcanic ash soils can be both unusually steep and high but that also unstable slopes can occur if smectite(montmorillonite) layers ("nappes") are located between the pedhologic B horizon (residual soil) and C horizon (saprolite).

Unique geotechnical features

Andosols are special materials from a geotechnical standpoint. This is because the extreme fineness of the weathering products of volcanic glass yields high specific surfaces and therefore low density and high water content. The microstructures soils are characterized by properties that change when dried or when it is manipulated. This is especially true for those soils formed in tropical climates, where the conditions favor the formation of allophane and halloysite minerals and cementing agents. The compacton behaviour in the field can be erratic and depends on the initial water content and the amount of remolding induced during compaction.

Case history

The nearly 200 ft high Mount St. Helens Sediment Retention Structure on the North Fork of the Toutle River in Southeastern Washington State is intended to collect sediment resulting from the 1980 volcanic eruption of Mount St. Helens and to prevent silting and flooding of the Cowlitz and Columbia Rivers downstream. The earth and rockfill materials for construction of the dam consist primarily of the soil and decomposed rock excavated at the site.

Some of the soils encountered by the contractor during the excavation, handling, stockpiling, and compaction of soil and rockfill from both the left and right abutment areas were the source of serious problems (Mitchell, 1989). Particularly difficult were the "old glacial drift and preeruption river alluvium" ("gravels") on the left abutment and the fine-grained sediment that overlies these gravels. The difficulties encountered by the contractor were reported as follows. The "old glacial drift" gravels on the left bank abutment that were expected to form a firm and stable base became soft and unstable during excavation, especially during wet weather. The material retained water and would not support construction equipment. Excavated materials stockpiled in disposal areas remained wet and unstable.

They continued to flow and creep as far as 500 ft horizontally and ultimately reached a stable slope of only two to five degrees. The fine-grained sediments unit above the gravels while appearing stable initially was equally unstable after excavation and disturbance. The natural water content was well above optimum for compaction and the materials could not be dried effectively in the cut, stockpile, or embankment.

It was determined that unique compositions and properties owing to the volcanic origin of the soil and the high rainfall climate were major factors responsible for the problems. Mineralogical analyses established that the clay fractions of samples from each of the materials causing the problems are dominated either by smectite or halloysite, and often by both types of clay. Expansive smectite is the major clay mineral component of the clay phase in the gravels; whereas, halloysite is most abundant in the fine-grained unit. Iron oxides, which can cement the structure in the undisturbed state, were also identified.

The geology, past construction experiences in the area, and the laboratory tests result all indicated the presence of unusual soils. The unexpected problems could have been anticipated and measures taken to minimize their impact on construction if the soil types had been properly identified and described prior to the start of construction.

EXPANSIVE SOILS

Expansive soils, which increase in volume when water is available, but shrink if water is removed, are a continuing source of problems in the design, construction, and maintenance of buildings, buried pipes, roads and airfields, canals, and retaining structures. Owing to the great amount of past experience and research on expansive soils that has been well-documented in the literature, this ubiquitous soil type in the Americas is only treated briefly herein. However, this is in no way intended to underestimate the importance of these difficult soils in geotechnical engineering.

The types and amounts of clay minerals are the main compositional factors which control swelling pressure and deformation. For a given soil composition, the density, initial moisture content and struture of the soil also significantly affect the swelling behavior. The smectite group of clay minerals (montmorillonite and others) has the highest water adsorption capacity and causes the greatest swell/shrink behavior. In addition degraded illites and mixed layer clays are sometimes somewhat expansive (Chen,1988; Arnold,1984).

Formation and distribution of expansive soils in America

Five geologic conditions are paramount in the formation of smectite (Tourtelot, 1974): (1) extreme disintegration of parent material; (2) strong hydration of precursor minerals; (3) restricted leaching which retains Mg, Ca, and Fe in the soil system; (4) variable oxidation-reduction conditions during soil formation; and (5) an alkaline environment. Condition (3) appears to be the most important. Any high alkali, aluminum silicate rock can be the source rock for smectite formation. Non-crystalline and poorly crystalline rock are best suited for clay formation. Volcanic glass and argillaceous lavas are prime candidates for montmorillonite genesis. Bentonites contain the greatest amount of expansive clays, with volcanic glass being the source material.

Expansive soils are found in many parts of the world, but predominate in the mediterranean and semi-arid regions where rainfall is moderate, precipitation is seasonal and there are high evaporation rates. Drainage must be sufficiently restricted to permit pore water salts to remain and become concentrated by evaporation. The topography may be flat,as in bentonite/marine shale deposits, or the grade may be steep, as in volcanic or orogenic settings, where slope stability can be a geotechnical hazard.

Figure 12 is generalized map showing expansive soils distributions in the Americas, compiled on the basis of climate, geology, and engineering experience. The distribution in the United States is fairly accurate, since most of the areas are defined from engineering experience. The distributions shown for Canada, Central and South America are less precise, as they rely on climate and geology to a greater degree. Regional areas of swell (RS) show areas where expansive soils are likely to occur. They do not indicate swelling soils over the entire area. Local areas of swell (LS) represent a more localized potential for expansive behavior.

Identification and unique geotechnical features of expansive soils

Indications of swelling potential may appear from the results of routine tests such as grain size analysis, Atterberg limits, and in-situ water content and dry density. The activity (the ratio of plasticity index to % finer than 2μm) provides a useful measure of expansion poten-

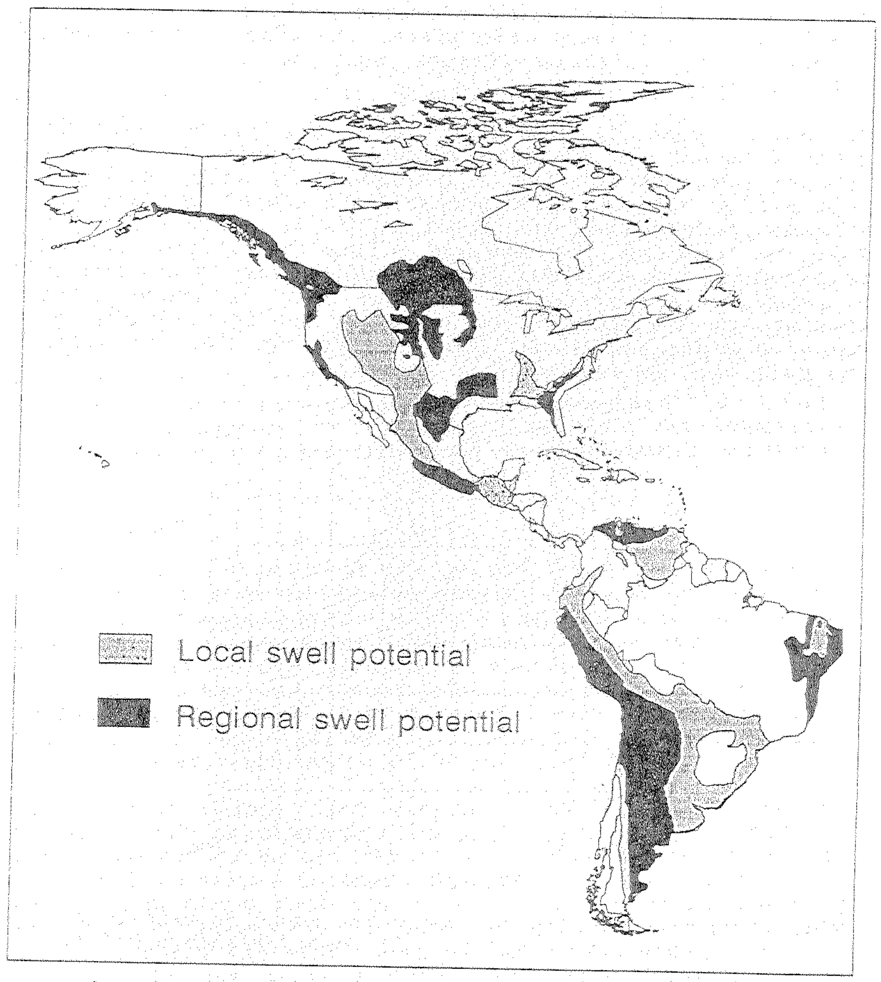

Fig. 12 Generalized map showing expansive soil distribution in the Americas.

tial. Several correlations, based on the above parameters, have been proposed for preliminary estimation of expansive deformations (Snethen,1984; Chen,1988). Any expansive soil has a lower and upper limit of water content between which swelling and shrinkage take place. Therefore, if movement is to occur, moisture changes must occur within this critical range.

Different types of oedometer tests are often used for quantification of swelling behavior. Figure 13 shows a typical test result for a procedure in which samples, initially at natural water content, are loaded to different stresses before they are given access to water.After the soil has reached its maximum volume,the sample can be reloaded and the swelling pressure determined.As the amount of swelling and shrinkage decrease with an increase of loading, it is important to determine the swell-shrinkage behavior under the actual load or range of loads to which the soil will be subjected in the field.

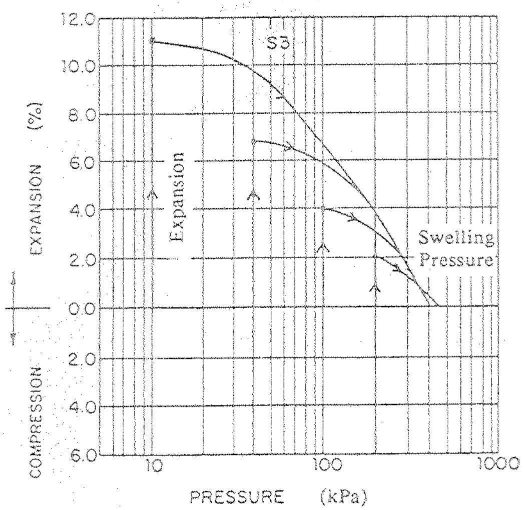

Fig. 13 Typical results for oedometer tests on samples of an expansive residual soil from Pernambuco,Brazil (from Signer et al, 1989).

Expansive soils, in general, have low hydraulic conductivity values and are highly plastic. However, when dried the soil can become highly fissured and fractured, thereby creating flow paths for water through channels and cracks. Also reduction in shear strength can occur in expansive soils which have been subjected to drying and wetting cycles. Thus, slope stability can be an important concern, especially in river valleys that have been cut through overconsolidated clay shales, in canals, and in other types of excavations.

Mitigation measures

Methods for mitigating the damaging effects of expansive soils include (Chen,1988; Stamatopoulos et al.,1989): (1) excavation with or without replacement of the soil by a compacted non-expansive soil; (2) flooding the in-place soil to achieve swelling prior to construction; (3) control of compaction water content and density; (4) mixing the soil with lime or cement before compaction; (5) using footings or piers that extend below the depth of seasonal moisture change; (6) using a bearing pressure that is high enough to balance the swell pressure; (7) use of procedures to minimize changes in the soil water content, including adequate drainage systems and surface impermeabilization of adjacent areas; (8) use of strutural mats and slabs that are resistant to differential movements; (9) use of deep foundations down to non-expansive material;and (10) treatment or replacement of the upper 1-3m with non-expansive soil (usually granular material) in embankment construction.

Case history

In the second phase of the Quiroz Irrigation Project in Northern Peru, field investigation and preliminary construction showed that the soils along the main canals and some auxiliary canals were potentially expansive (Montero, 1961). These soils could be a source of damage to the concrete lining and cause slope instability along the canals. The expansive clays developed swelling pressures as high as 7000 psi (48.2MPa) and swelled as much as 14.5 percent under confinement of 50 psi (344.5kPa).

To minimize damage from expansive soils, the designers decided that in dangerous zones the soil on the surface should be excavated and replaced with compacted non-expansive soil to make a suitable foundation for the lining. It was also proposed that the soils be saturated both during and after placement of the canal lining. In spite of these efforts, there was still some damage from expansive clays, although it was minimized. The repairs in the concrete lining, which included reinforcement with steel to prevent further cracking, were postponed for as long as possible to allow for maximum soaking of the soils after the water had been flowing in the canal.

COLLAPSING SOILS

Collapsing soils, also known as metastable or subsiding soils, are encountered in many areas of the world in a variety of climatic, geographic and geologic conditions. These soils are generally characterized by a sudden and large volume decrease at constant stress when inundated with water. Collapsibility depends primarily on the state, and the state is a function of the formational process and geological history. Many types of soil deposits may exhibit collapse, including loess, colluvium, mud or debris flows, residual soils, volcanic tuff, alluvial materials and man-made fills. Their collapse potential is usually a result of three conditions: (a) a partly saturated structure that is open and potentially unstable; (b) some force or material that bonds the intergranular contacts yet loses strength upon wetting, and (c) an applied or existing stress that is great enough to break down the metastable structure. With the exception of loess, the areal extent of collapsing soil deposits is usually not great. However, local occurences of collapsible soils are found throughout all the Americas.

Some examples of collapsing soils (see also section on Loess)

Residual collapsing soils have a metastable state characterized by honeycomb structure and partially saturated moisture condition that can develop after a parent rock has been throughly decomposed or while the decomposition is happening (Vargas,1973). Commonly, metastable residual soils form under condition of heavy concentrated rainfalls in short periods of time, long dry periods, high temperature, high evaporation rates, and flat slopes so that leaching of material can occur. There are two mechanisms of bonding in the metastable soil structure: soil water suction and cementation by clay or other types of fine particles (Dudley,1970). Residual soils derived from a wide variety of parent rocks can be collapsible; therefore, ranges of typical soil characteristics cannot be defined. The clay fraction of these soils is usually composed of kaolinite, though gibbsite and halloysite are also common (Lohnes and Demirel,1973; Prusza and Choudry,1979; Ferreira and Monteiro, 1985). Residual collapsible soils usually have low activity and low plasticity.

Transported and deposited soils exhibiting collapse behavior are most commonly silt, silty sand, silty clay, clayey sand with vaying degree of grading (Holtz and Hilf,1961). The composition of the soluble salts and fines that bind the grains together is very important in these soils. The most common clay minerals in the clay fraction are more site specific, depending on the parent material. Less

stable minerals, like montmorillonite, can remain in the soils. In silty sands,the binding force often results from the capillary tension of the pore fluid. Key factors influencing the potential collapsibility of a soil deposit are the in situ water content, density, saturation, percent passing the # 200 mesh sieve, and overburden stress.

Debris flows are slurry-like flows in which large grains ranging up to boulder size, are supported in a matrix of fine sediment and interstitial water. Debris flows can occur on gentle or steep slopes above or below water. Subaerial flows occur under many climatic conditions but are particularly common in arid and semi-arid regions where they are usually initiated by heavy rainfalls.The rapid deposition of the slurry produces a loose structure which is strengthened as bonding develops between grains by capillary forces. The content of clay particles is very important in the formation of these deposits (Clemence and Finbarr,1981). The maximum subsidence of debris flows in the San Joaquim Valley of California occurs when the clay content is about 12 percent.

Colluvial deposits are deposits of soil and rock that are transported by gravity without the effects of excessive water. These soils become collapsible in enviroments where the climate is characterized by alternating wet and dry seasons that cause a continuous process of leaching of the soluble salts and colloidal particles much like residual soils. Soils of this origin occur in more than 50% of the Sao Paulo State of Brazil (Ferreira and Monteiro, 1985).

Fluvial deposits in river channels that experience intermittent flow are rapidly solidified as the energy of the river decreases below the critical velocity for transporting solids. Flash flood deposits are typically well-graded and must contain a silt and clay fraction of 5 to 20 percent to exhibit collapsible behavior. Flash floods commonly occur in arid to semi arid regions where small watersheds are subject to sudden cloudbursts at infrequent intervals. A high evaporation rate in these climates means that rain does not soak into the ground very far before it is evaporated back to the atmosphere. This allows for partly saturated soils to exist in-situ even in river channels.

Identification and determination of collapsibility

The identification of collapsible soils and the prediction of the amount of collapse that may occur is very important in preventing damage to structures and foundations. The engineering behavior of collapsible soils is not well predicted by the Unified Soil Classification System or by the AASHO system alone, because the manipulation of the soil required for classification destroys the structure of the soil upon which the collapse behavior is dependent, and because collapse and swelling behavior depend on initial void ratio and effective stress.

Consequently, a number of other criteria has been used, and they are sumarized by Lutenegger and Saber (1988). Most are based on either density measurements or consistency limits, or combinations of the two. One criterion that is often quoted is shown in Figure 14 (Gibbs and Bara,1967). Such criteria are useful,but they may only be locally applicable, and recent review indicated that in most cases no single one criterion alone is accurate enough to predict collapsibility for all soils.

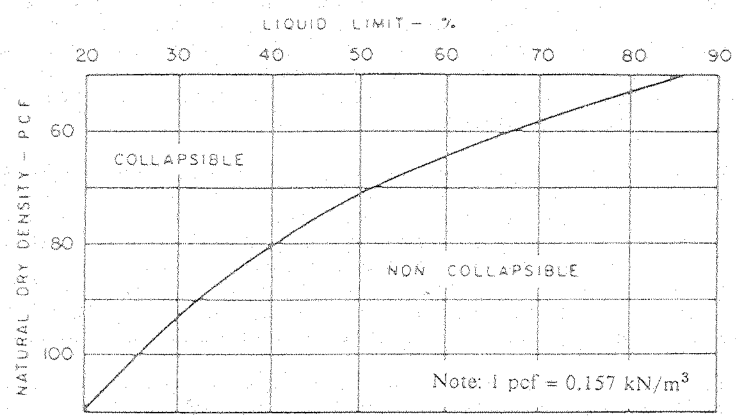

Fig. 14 A commonly used criterion for determining collapsibility (from Gibbs and Bara, 1967).

For direct quantification of the amount of collapse representative undisturbed samples are essential. Houston and Mostafa (1990) illustrate how the effects of common sampling techniques influence the response of cemented collapsible soils. Two types of oedometer tests are commonly used: the single oedometer collapse test and the double oedometer collapse test. The results of both tests are similar to the plots in Figure 15. The quantity Δe is used to assess the amount of collapse that can occur at any initial effective stress state,and an approximate degree of the severity has been proposed (see also GSEGWP, 1990): (1) slight, when the collapse potential is 2.0%; (2) moderate, when the collapse potential is 6%; and (3) severe, for collapse potential of 10% or more. Although distilled water is usually used to saturate the specimens, for some site

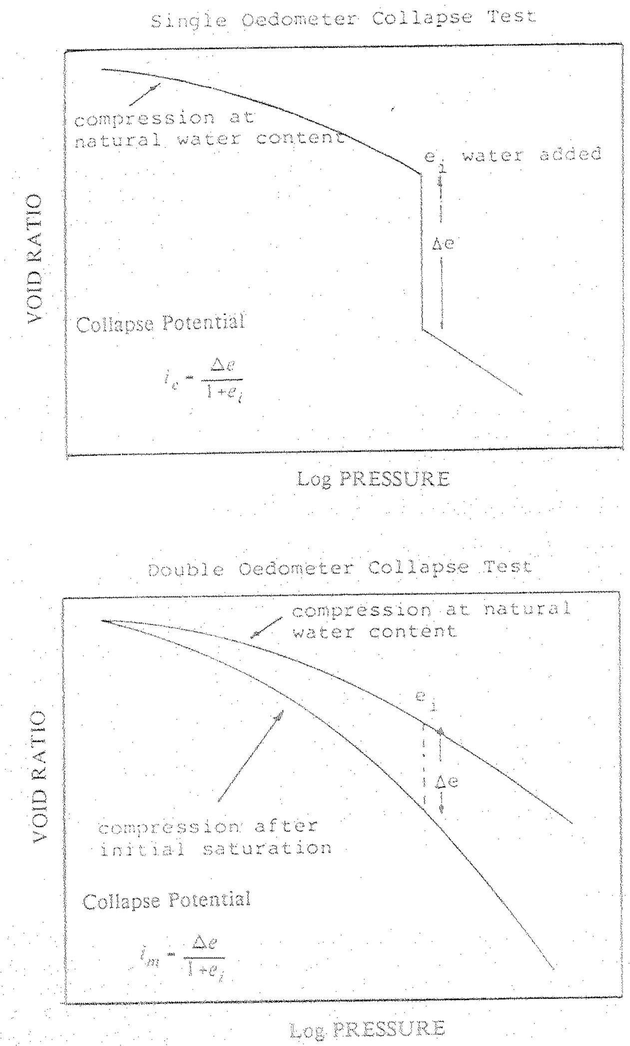

Fig. 15 Single and double oedometer collapse tests.

conditions the use of other fluids may be more appropriate (Reginatto and Ferrero, 1973; Mitchell,1976). The designer should be careful when using the results from oedometer tests as they may over estimate the actual amount of collapse in the field.

In situ tests and foundation performance observations are desirable to complement the results of laboratory measurements. A field collapse prediction procedure based on laboratory and field tests is given by Houston et al. (1988). It is important to note also that the magnitude of collapse depends on the amount of reduction in soil water suction (Maswoswe, 1985; Burland, 1985). Thus partial relief of suction stresses may result in only partial collapse, with more developing later if the suction is further reduced (GSEGWP, 1990).

The fact that collapse is accompanied by a reduction in strength must not be overlooked, because the overall stability of a site may be reduced.

Mitigation of collapsible soil problems

Methods used to mitigate collapse problems are (Holtz and Hilf, 1961; Clemence and Finbarr, 1981; and Houston and Houston, 1989): (1) partial or total removal;(2) ponding water, with or without infiltration wells and compaction of the soil to deliberately collapse the soil before the construction begins; (3) excavation and recompaction of the soil while wetting it; (4) avoidance or minimization of wetting; (5) driving displacement piles into the wetted soil to increase the compaction; (6) silt injection; (7)vibrocompaction and grouting; (8) dynamic compaction; (9) lime and cement treatment; (10) the use of either very flexible or very stiff structural systems; and (11) deep foundation systems to a firm, unyielding strata.

Case history

In some areas of the San Joaquin Valley of California adjacent to the Coastal Hills to the west, classic collapsible alluvial soils are found. The soil is unsaturated and has a high porosity, and the clay portion comprises a small percentage of the soil by weight. The steep mountainous terrain adjacent to gently sloping topography provided the mechanism for debris flows to be deposited during infrequent heavy rains, and an arid climate prevents this deposited material from becoming reworked or saturated which preserves the metastable structure. Some of the flash flood deposits in this area have subsided as much as 15 ft when thoroughly wetted (Clemence and Finbarr, 1981).

Ponding prior to canal construction was successfully used to reduce collapse potential of the collapsible soils found along the canal path (Gibbs and Bara, 1967). Delaying construction of berms after the soil was prewetted produced additional beneficial foundation settlements. Prewetting softened the ground surface and curing time was required before the concrete liner could be placed. In some areas it was possible to excavate, wet, and recompact the soil in order to remove its collapse potential. To determine safe slopes for the canal berms, triaxial shear tests were performed on representative specimens that were wetted for 24 hours before testing.

LOESS

Loess is an aeolian soil deposit encountered in large areas of the world. It is characterized by vertical eroded and cut slopes that are stable owing to cementation, lack of stratification, and vertical root holes left from grasses during accumulation of the deposit. Most loess is collapsible (see section on Collapsible Soils), and this property along with the vertical slopes are its most significant geotechnical features. The potential for collapse results from its loose partly saturated structure and the presence of soluble cementing materials. Three agents play important roles in the collapse of loess: wetting, added surcharge, and dynamic loading. The predominance of silt-sized particles makes loess susceptible to erosion, frost heave, and liquefaction.

Formational history and distribution of loess in America

Wind deposited loess covers 13 million square kilometers worldwide and is found in plateau regions, rivers valleys, and floodplains. Most loess deposits were deposited during the Pleistocene and post-Pleistocene epoch. Modern loess deposits are still forming in desert areas. Currently, loess exists at or just below the surface of the ground, and its thickness can vary from less than two meters in highlands to several hundreds of meters near river valleys.

Loess has its origin in sediments that are removed from weathered rocks by fluvial action. The fine material is then transported from these alluvial deposits by wind. Aeolian soils form in windy, dry areas where the wind, like water,sorts the transported sediments. Loess deposits are commonly thicker with coarser particles near the source and become thinner with finer particles downwind. This formational process usually produces an unsaturated soil of low density, high compressibility, and of poor bearing capacity. Loess is slightly to moderately cohesive due to small amounts of clay, usually smectite, and calcium carbonate that are usually present.

Once transported and deposited by the wind, the formation of loessial soils depends on the growth of grasses on the surface of the deposit, which leaves behind vertical root holes as the deposit thickens. Calcium carbonate is precipitated along the root holes. This,in part,is responsible for the vertical cleavage in loess. In addition, the loose structure of loess soils is strengthened by subsequent precipitation of water-mobilized substances, including inherent clay and other soluble cementing materials, such as ferrous oxides and calcium sulfate. Capillary tension, caused by low humidity, also contributes to the apparent cohesion of loess.

Loess covers about ten percent of the worlds'landmasses. In North America,loess can be found over much of the Mississippi River Valley and Nebraska. In South America, large deposits of loess are found in the Argentinean pampas. Locations of major loess deposits in America are shown in Figure 16. Large deposits also occur in Central China and Russia.

Special fabric and structure characteristics

Natural (undisturbed) loesses exhibit some special characteristics and unique mechanical behavior:

1. Loesses are very porous materials, and sometimes their macropores can be visible to the naked eye. In-situ porosity is between 40 percent and 60 percent, and occasionally up to 70 percent.
2. Despite their loose structure,in their natural state loesses are comparitively strong and stiff, and can stand in vertical cuts without support.
3. Upon wetting or when subjected to heavy surcharge or dynamic loading, loess can become unstable and collapse.
4. The vertical permeability of loess deposits is greater than that in the horizontal direction as a result of the secondary fabric; i.e., root holes. This is opposite to the permeability anisotropy in waterborne sediments.
5. An initially flat cut in loess will gradually slough until a nearly vertical cliff forms. There is a high gully-erosion potential related to the silt-sized particles and soluble cementing materials.
6. The shear strength of loesses may be anisotropic as a result of the root holes and tubular concretions.

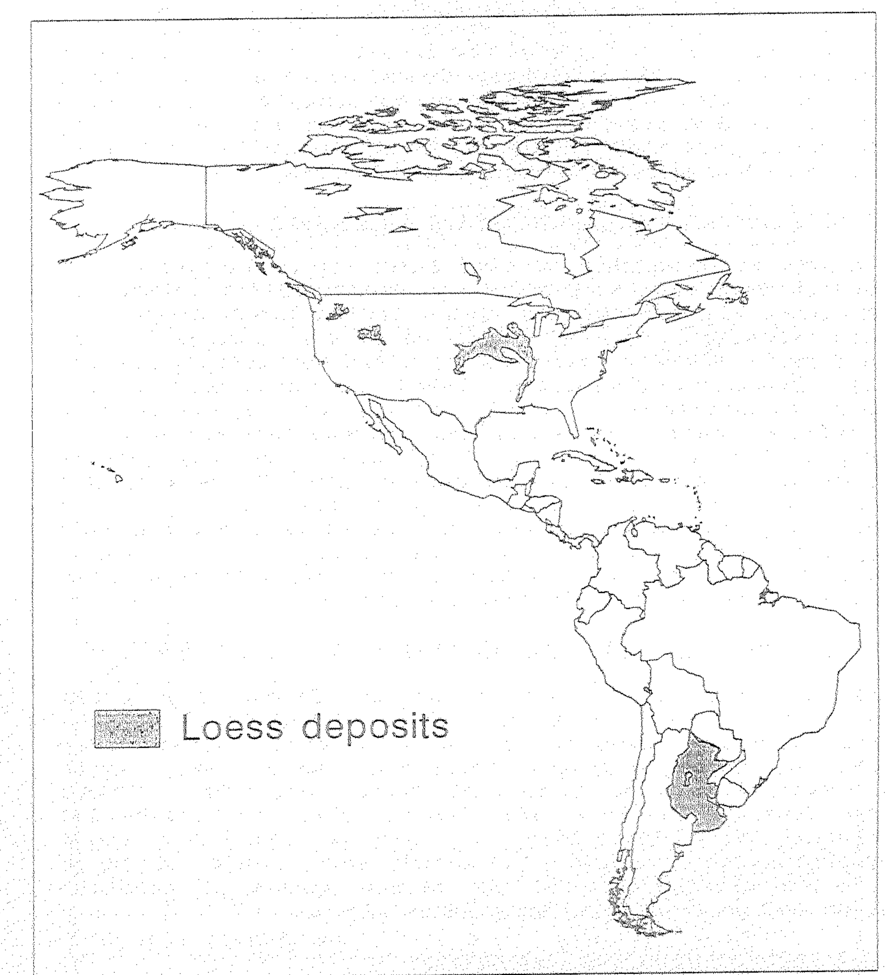

Fig. 16 General distribution of loess deposits in the Americas (from Rocca, 1985).

Wetting weakens the clay bonding (as a result of swelling), and can totally destroy the bonding by soluble cementing materials, resulting in a sudden loss of strength. Surcharge loading can increase the in-situ stress level beyond the strength of the cementing. Dynamic loading can fracture the structure. The effective stress can be reduced as a result of high air flow in a dry collapsible loess, or by increased pore water pressure in an initially wet loess, leading to liquefaction-type flows in both cases.

Identification and determination of collapsibility

A number of approaches for evaluating the collapsibility of loesses have been proposed; e.g.,Rocco (1985), Lutenegger and Saber (1988). They are classified in three groups: (1) identification, based on simple parameters, such as initial void ratio, density, water content, and consistency limits; (2) direct quantification, based on mechanical tests, mainly oedometer tests (see Figure 15), and (3) preliminary estimation based on empirical regressions for a given region. The criteria of the first group are only general rules for evaluating the collapsibility of loesses, and they may only be locally applicable. Terzaghi and Peck(1967) propose that the most reliable identification approach is to perform in-situ load tests, and to investigate the influence of variation in possible wetting sources and other relevant environmental factors.

Usual composition and typical ranges of index properties

The major mineral in loess is quartz. Carbonate minerals may be present in amounts as high as 40 percent by weight. Other minerals present in small amounts include clay minerals, mica, feldspars, hornblende,and pyroxene. Table 8 summarizes typical ranges of index properties of loess (see Terzaghi and Peck,1967; Bowles,1982; Krinitzsky and Turnbull,1967; Rocco,1985). It may be seen that these properties fall within a relatively small range, which is directly attributable to the narrow spread of particle sizes and well-defined formational history.

Important geotechnical properties

In general, a true loess behaves as a slightly cemented material in its initial in-situ state. After wetting and collapse however, it suddenly loses strength. When totally remolded (sometimes termed modified loess) it behaves much like other silty soils. The natural structure of a loess may not be totally destroyed by a single collapse event, however. A summary of geotechnical properties of a true loess is given in Table 9 (for more information see Terza-

Table 8. Summary of index properties of loess

Index property	Typical range
Particle size	Mainly silt with 0-15% clay size
Particle shape	Bulky, with tubular $CaCO_3$ concretions
Grain size distribution	Generally uniform
Natural water content	4 - 25 %
Liquid limit	25 - 55 %
Plasticity index	5 - 25 %
Unified classification	ML, CL, ML-CL
Specific gravity	2.6 - 2.8

Table 9. Summary of geotechnical properties of loess

Property	Value
Hydraulic conductivity, k_v, (m/s)	$\approx 1 \times 10^{-5}$
Collapse	f(e, w_i, c_i, EF)
Swell	Small
Compression modulus, E (MPa)	≈ 20 (D); < 2 (W)
Unc. comp. strength (kPa)	≈ 200 (D); < 50 (W)
Dry: ϕ (°) c (kPa)	≈ 34 ≈ 100
Wet: ϕ (°) c (kPa)	≈ 0 (During collapse) < 20
Dynamic properties	f(e_i, w_i, c_i, EF)
Erodibility (1)	Significant
Liquefaction potential	High

e_i = initial void ratio; w_i = initial water content; c_i = cement bonding; EF = environmental factors; D = dry; W = wet; (1) = dependent on clay present ghi and Peck, 1967; Turnbull, 1948; Feda, 1965; Milovich, 1969; Hunt, 1984; Gibbs et al., 1960; and Rocco, 1985).

Soil improvement

Deposits of loess are sometimes very thick.If the deepest layers collapse from overburden, then they act like a silt soil. However layers closer to the surface may pose a collapse hazard. Improvement techniques used to eliminate or minimize collapse potential are based on either modification of the initial fabric or control of the environmental factors (see Rocco,1985 and Evstatiev, 1988), and include: (1) dynamic compaction; (2) injection of clay grout under pressures of 150-350 KPa; (3) preponding water, sometimes using hydroblasting in order to increase efficiency, to deliberately collapse the soil before construction begins; (4) compaction of only those areas where a load is transmitted to the ground, between which collapsible loess remains; (5) use of soil piles; (6) injection of chemical stabilizers such as sodium silicate, including gas treatment, to augment the natural cementation; (7) avoidance or minimization of wetting; and (8) the use of a good drainage system and vegetation to control erosion and localized slope failure.

Case history

The Los Molinos - Cordoba Canal, used to conduct water to the city of Cordoba, Argentina, is 62 Km long and passes through three different geological formations:metamorphic rock,basalt and loess. The loess deposits are up to 80m thick, and the top 10 to 15m are collapsible. The loess soil is composed of silt with small percentages of fine sand, and it classifies by the Unified System as ML, and in some cases as CL-ML (Moll et al., 1979).

Large settlements and slope failures were observed in some sections. The sequential soil improvement techniques employed to mitigate the collapse hazards,including both "deep and shallow" techniques, were as follows:

1. Water saturation of the area using 5 cm diameter and 16 m deep boreholes, which caused an average settlement of 15 cm;
2. Hydroblasting, which increased the settlements by 27 cm to 75 cm;
3. Resaturation of the top 4 m, which produced a further settlement of about 10 cm; and
4. Dynamic compaction using a hammer of 1.4 tons dropped from 10 m, which caused a settlement of approximately 40 cm.

As a result of water saturation and blasting the dry unit weight was increased by a small amount, and the residual strength increased 5 times due to soil structure modification. As these deep techniques did not modify the first 2m of the deposit, the heavy tamping was then used to improve the loess throughout the upper 5m.

SOFT ORGANIC CLAYS

Soft organic clays are usually fully saturated and normally consolidated to slightly overconsolidated, with unconfined compressive strengths of 50 kPa or less. These clays are often mottled and grey to black in color, and sometimes dark greens or browns with visible non-decomposed organic matter or shells present in some of them. Organic clays often emit a distinctive odor of decaying organic matter. The organic matter content may range from very low to very high values, with true fibrous peats as an upper bound. The discussion in this paper is limited to slightly and moderately organic (peat is excluded) soil and to specific influences of the organic matter on their geotechnical properties.

The construction of a structure on soft organic clays poses many difficult problems. The low strength may make it difficult to insure the stability of the structure. Large settlements and deformations of the struture may result from the high compressibility. The low permeability of some of these materials may require long construction periods if stability is to be maintained. Construction cost may be high and require the use of complex and costly techniques of drainage and soil improvement.

Formation and distribution of soft organic clays

In an organic clay, the soil fabric that develops at the time of formation depends on the amount and type of clay mineral; the amount, type, and degree of decomposition of organic matter; and water chemistry. The clay and organic sediment load to a given quiet depositional enviroment determine the initial relative ratios between clay minerals and organic matter. Additional biogenic ativity in the depositional environment may increase or decrease the organic matter content with time after deposition. Within a given depositional region, the exact composition will vary according to the respective settling velocities of each clay mineral and the organic matter, which is a function of their abilities to be electrostatically destabilized. Anaerobic bacteria decomposing the organic detritus and other compounds within the organic matter-clay matrix can lead to transformation in the nature of the organic matter and the soil properties.

The agricultural soils groups most likely to contain appreciable quantities of organic matter are Entisols, Gleysols and Hystosols. Figure 17, showing large areal distribution of organic soils in the Americas, was developed from FAO-Unesco's surficial soil maps (FAO-Unesco,1974) and supporting documents. The information for North America was more consistent and easier to interpret than that for Central and South America, where subjective interpretation was required. In Brazil soft organic clays have been found in many places along the coastal areas. Figure 17 is only a general indicator of the distribution of soft organic clays. In fact, they may be encountered in almost any highly vegetated quiet water area. Soft organic clays deposits are often highly localized subunits of larger clay or peat deposits.

Identification of soft organic clays

The influence of organic matter on the soil properties is primarily a function of its type, amount, and degree of decomposition. There are many methods for determining the amount of organic matter in a soil. One method which is frequently used consists of oxidizing the organic matter with a mixture of sodium or potassium bichromate and sulphuric acid. Another method consists of burning the organic matter (loss on ignition). (See, for example, Broadbent, 1965; Al-Khafaji and Andersland, 1981). The degree of decomposition can be determined using the humification test of Von Post (see Perin, 1974). As the different methods do not all measure the same thing, the amounts of organic matter indicated by each test may differ significantly.

There has been a recent modification in the classification of organic fine grained soils within the "Unified Soil Classification System" (ASTM D2487-85,D2488-84,1989). An organic clay is a soil that would classify as a clay (the Atterberg limits plot above the A-line), except that its liquid limit value after oven drying is less than 75% of its liquid limit value before oven drying. In the Laboratoires des Ponts et Chaussees Classification (Magnan,1980), which is a revision of the Unified Classification, the organic content (OC) should be determined by tests, and the soils are classified as follows:(1) OC < 3% - inorganic soil; (2) 3% < OC ≤ 10% -slightly organic soil; (3) 10% < OC < 30% - moderately organic soil; (4) 30% ≤ OC - very organic soil. Moderately and very organic soils have a special classification developed based on the humification test of Von Post. Slightly organic soils are classified in

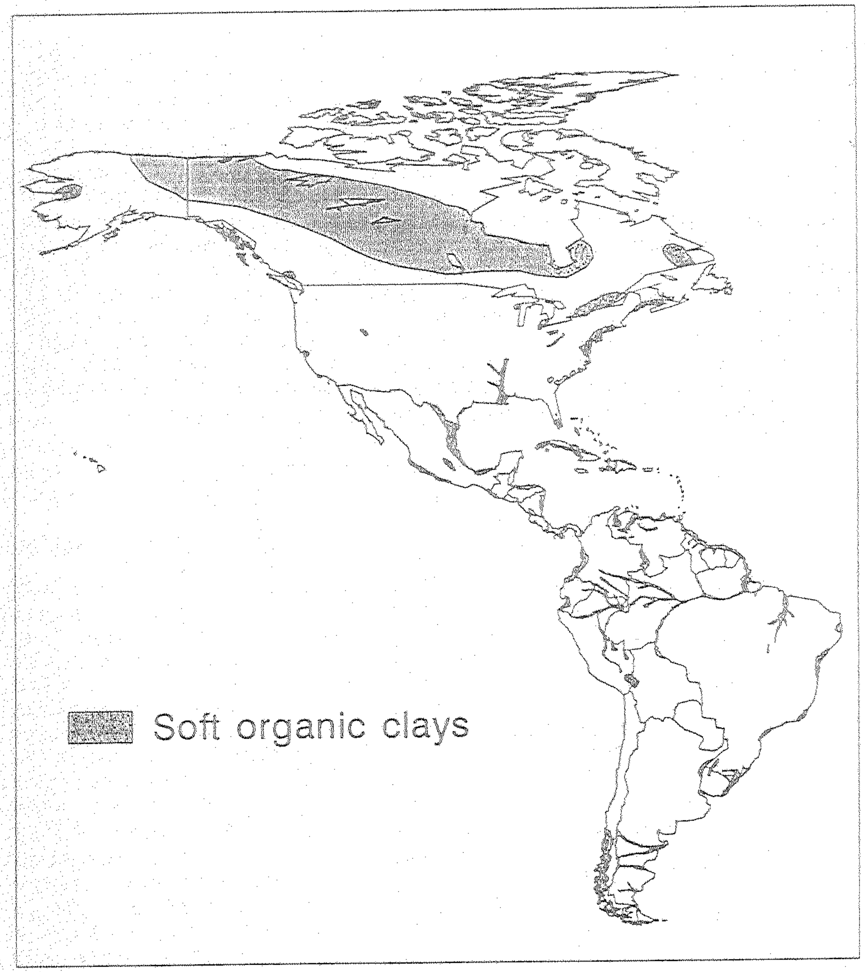

Fig. 17 Simplified map showing distribution of soft organic clays in the Americas (after FAO, 1974).

the same way as inorganic fine-grained soils using the plasticity chart, but the symbols are preceded by fO.

Index properties

The natural water content, the plastic and liquid limits, and the plasticity index of organic clays tend to be higher than inorganic clays of the same mineralogy due to the ability of the organic matter-clay matrix to absorb water. Odell, Thornburn, and Mckenzie (1960) illustrated this relationship by showing that small increases in the amount of organic matter can have large effects on these properties. The specific gravity of organic soils is lower than that of inorganic soils, because the less dense organic matter has a specific gravity of only about 1.4. As a consequence of low specific gravities and high natural water contents, the unit weights of organic clays are also low. The void ratio of soft organic clays can be very high. The influence of organic content on the classification properties of clays from Juturnaiba Dam site is illustrated in Figure 18 (Coutinho and Lacerda, 1987 and 1989).

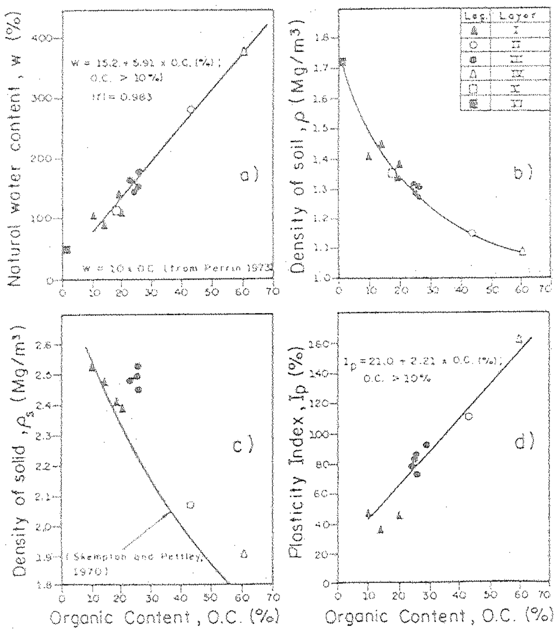

Fig. 18 Influence of organic content on characterization properties of Juturnaiba organic clay (from Coutinho and Lacerda, 1987).

Typical values of properties of some representative soft organic clays from the Americas are given in Table 10. With exception of Mexico City clay, which has much higher values, the water contents are in the range of 30 to 200 percent. Mexico City clay is a soft organic clay that is derived from volcanic rocks. Its unusual properties are more related to the high smectite clay mineral and siliceous diatom content than to organic content (Mesri et al., 1975). The liquidity index of soft organic clays is typically 1 or more.

Hydraulic conductivity

Little data is readily available on the hydraulic conductivities of organic clays. It is generally believed that the presence of fine organic matter (humus) reduces permeability (Mitchell, 1976). However, the presence of fibers (non-decomposed organic matter) can cause a contrary effect, as illustrated by pure fibrous peats which have a high initial hydraulic conductivity in their undisturbed state. Available information on soft organic clays has shown hydraulic conductivity values usually within the range associated with clays and silty clays (10^{-7} to 10^{-10} m/s), with the higher values usually for higher fiber contents.

Volume change behavior

The compression index values for the representative organic clays in Table 10 clearly indicate high compressibility. The compression index increases with increasing liquit limit and natural water content or void ratio (Kulhawy and Mayne, 1990). As the presence of organic matter further increases these index parameters, it will also serve to increase the soil compressibility. The value of coeficient of secondary compression (C_α) also increases with increasing natural water content; therefore, soft organic clays have relatively high values of C_α (Mesri and Godlewski, 1977). Mesri and Choi (1985) show that $C_{\alpha e}/C_c$ or $C_{\alpha e}/CR$ are constant for a given soil during both recompression and virgin compression. For the majority of inorganic clays, $C_{\alpha e}/C_c = 0.04 + 0.01$, and for organic clays and silts $C_{\alpha e}/C_c = 0.05 + 0.01$. Very organic soil can have higher values. The data in Table 10 are consistent with this, except for Mexico City organic clay which has a lower value. Figure 19 illustrates the influence of organic content on soil compressibility, showing average results for Juturnaiba organic clays. Typical values reported for the coefficient of consolidation (c_v) of soft organic clay are intermediate to low. In virgin compression it is common to find values in the range of 10^{-8} to 10^{-7} m^2/s.

Table 10. Geotechnical properties of some soft organic clays in the Americas (organic contents between 3 and 30%)

Clay	Organic Content-%	LL %	PI %	w_n %	G_s	γ kN/m^3	C_c	C_α/C_c	$(S_u)_{FV}$	Reference
Juturnaiba Rio, Brazil	5-30	90-190	40-100	80-200	2.3-2.6	12-14.5	0.6-2.3	0.03-0.06	8-35	C & L (1987,1989)
Portland, Maine, USA		50-85	20-50	30-80			15-25%(CR)		7.5-12.5 (lab)	Ladd et al (1969)
Sarapui Rio, Brazil	4-6	90-160	50-80	100-180	2.51	13.2	1.2-2.2	0.05-.055	5-15	Ram.(83),Lac.(77)
Atchafalaya LA, USA		142	104	110					5-50	Kauf.&Weav.(67)
Willard Dam Utah, USA		33-52	20-25	30-55					7.5-32.5 (lab)	Walker(1967)
Chicago area USA	8			47	2.57					Franklin (73)
New Orleans, LA, USA	10-17	75-130	55-80	60-200		13.8	0.7-1.1		15-45	Ahmed (84)
Mexico City clay	14	500	350	420-575	2.35		2-12	0.03-.035		Mesri (75)
Santa Helena Dam, Brazil		33-123					0.33-2.7			Garga (84)
Lornex Dam, BC, Canada		155	85	142					5-50	Burke&Smucha(80)
Itaipu Lagoon Rio, Brazil	10-20	110	65	100-110		13.3			4-5	Sandroni(84)
Recife soft clay, Brazil	3-10	≈ w_n	30-70	30-110	2.5-2.7	14-19	0.2-2.2	.043-.053	30-50	Coutinho(88)

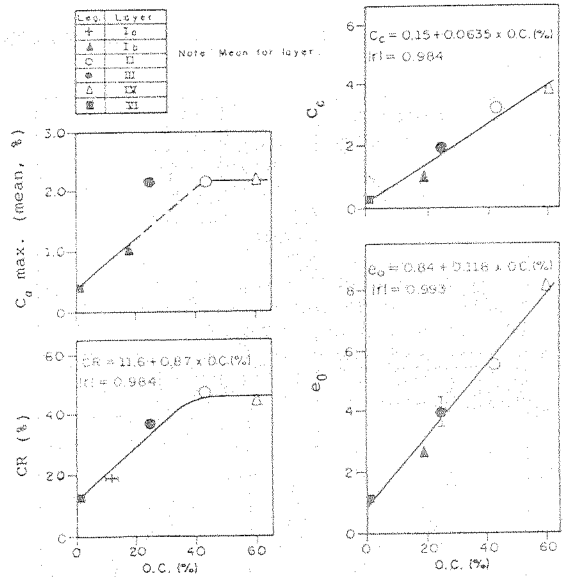

Fig. 19 Effect of organic content on compressibility Properties (from Coutinho and Lacerda, 1987).

Shear strength

Some studies suggest that the presence of organics can give reduced undrained shear strength (Mitchell,1976), while others have shown a contrary effect, especially in soil with non-decomposed organic matter (fibers) which can act as reinforcement. Thus the amount and the type of the organic matter and its relationship to the solid particles in the soil is very important in influencing the strength behavior (Andersland et al, 1981; Al-Khafazi and Andersland, 1981a). In soft organic clays the ratio of undrained shear strength to preconsolidation pressure S_u/σ_p has usually been observed to be higher and with more scatter than for inorganic clays (Larsson, 1980; Leroueil et al. 1990; Coutinho and Lacerda, 1989). Increase in the effective friction angle (ϕ'_{nc}) due to the presence of organic matter also has been observed by Andersland et al. (1981) for a mixture of kaolinite and fibers and by Coutinho and Lacerda (1989).

The ratio of undrained Youngs' modulus to shear strength E_u/S_u is smaller for highly plastic and organic clays than for convencional plastic inorganic clays (Ladd et al, 1977). The influence of organic matter content on strength and deformation parameters of the Juturnaiba organic clays is shown in Fig. 20.

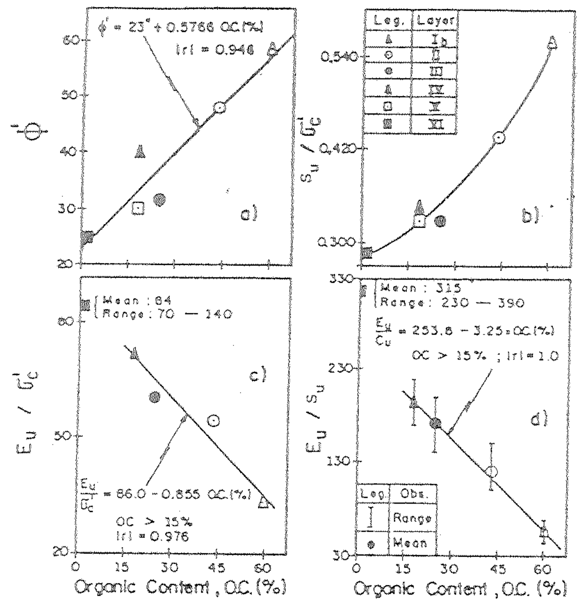

Fig. 20 Effect of organic content on strength of normally consolidated Juturnaiba organic clay (from Coutinho and Lacerda, 1989).

Special problems

Clays with high plasticity and/or organic content are susceptible to large initial settlement and undrained creep deformations (Foott and Ladd, 1981). This may necessitate the use of high factors of safety to maintain settlements and shear deformations within acceptable limits. The presence of layers of soft organic clays beneath structures may amplify small rock motions but damp very strong rock motions due to earthquake.

Case history

An instrumented test embankment was built on a typical soft organic clay of the Rio de Janeiro region (Ramalho-Ortigao et al, 1983). A summary of the geotechnical properties of the foundation soil, which had the thickness of 11 m, is given in Table 10. The test embankment had a base length of 80 m and a base width of 40 m. The instrumentation included piezometers, magnetic and horizontal wire extensometers, inclinometers, settlement plates, and surface markers. The fill material was a silty-sand residual soil with a unit weight ~ 18 kN/m^3; cohesion (c') = 10 - 20 kPa; and friction angle(ϕ') = 35°. Initially fill was placed to a depth of 0.5m. The embankment was then compacted in 30 - 40 cm lifts. Cracks began to appear when the embankment reached a height of 2.5m and embankment failure occured at a height of about 3m. Failure occured one month after the filling commenced.

Maximum observed settlements were of the order of 300 mm, while ground heave near the embankment toe was about 50mm. Maximum horizontal displacements at the toe were in the range of 300-400mm. Ground displacements did not extend more than 10m beyond the embankent toe. The field measurement data were used to stablish deformation limits for control of construction on Rio de Janeiro clay.

Stability analysis of the failure was carried out using total and effective stress methods. Several hypothesis for the foundation soil undrained strength and fill strength were tested using a total stress analysis. The obtained factor of safety (FS) was quite close to unity using mean field vane strength, suggesting the correction proposes by Bjerrum (1973) was not necessary for the Rio de Janeiro soft organic clay. Coutinho (1986) reached the same conclusion for a trial embankment built on Juturnaiba organic clays near to Rio de Janeiro city. Factors of safety well below one (~ 0.65) were obtained by effective stress analysis. The reasons for this are discussed elsewhere (Costa Filho et al, 1985; Almeida, 1985).

QUICK AND VERY SENSITIVE CLAYS

Sensitivity refers to the loss in undrained strength that may develop upon disturbance of the structure of an undisturbed clay. Most soft clays exhibit some degree of sensitivity. The term "quick clay" is applied to clay masses that are so sensitive that they may be rapidly transformed from an stiff coherent material to a viscous liquid.

Slides in quick clay may be initiated by some disturbance, such as traffic, the shock of a pile driver, an explosion, an earthquake, or toe erosion or excavation. The initial disturbance produces enough remolding to cause a local overloading followed by progressive failure. The static shear strains then continue to remold the clay, thereby reducing the shearing strength until it is ultimately negligible. The effect is cumulative, and a large mass may be transformed into a thick flowing liquid very rapidly.

Formational processes and geographical distribution

Most sensitive clays were formed during the last 5000 - 12000 years by sedimentation at a high rate in fresh or saline water. Clays of low to medium sensitivity are common in all regions of the world. The environment of sedimentation of soft clays can be marine, lacustrine, deltaic, coastal or lagoonal. However, nearly all quick clays and most very sensitive clays are found in areas that were covered by ice during the Ice Age and are of glacial marine origin. The salinity of the sedimentation environment led to a flocculated state, which, coupled with a rapid rate of sedimentation, generally led to a very open structure and correspondingly high water contents.

At least six different phenomena may contribute to the development of sensitivity (Mitchell and Houston,1969),as shown in table 11. Of the listed phenomena, leaching and

Table 11. Summary of the causes of clay sensitivity (Mitchell and Houston, 1969)

Mechanism	Approx. upper limit of sensitivity	Predominate soil types affected
Metastable fabric	slightly quick (8-16)	all soils
Cementation	extra quick (>64)	soils containing Fe_2O_3, Al_2O_3, $CaCO_3$, free SiO_2
Weathering	medium sensitive (2-4)	all soils
Thixotropic hardening	very sensitive*	clays, silty clays
Leaching, ion exchange, increase in mono-to divalent cation ratio	extra quick (>64)	glacial and post-glacial marine clays
Formation or addition of dispersing agents	extra quick (>64)	clays containing organics

* Pertains to samples starting from present composition and water content. Role of thixotropy in causing sensitivity in situ is indeterminate.

ion exchange seem to be associated with the most sensitive quick clays. Sometimes the quick behavior is a result of combined mechanisms.

Since their emergence above sea level about 3000 to 5000 years ago, as a consequence of isostatic uplift, leaching and diffusion have led to a general reduction in salinity of the pore water and and increase in the exchangeable sodium percentage of the adsorbed cation complex. As a result the physico-chemical interparticle equilibrium is changed so that the structure that was once stable has become metastable. In some cases some interparticle cementation develops in the undisturbed clay further protecting the undisturbed structure and strength. When disturbed, however, this metastable structure causes the clay to behave like a collapsing soil under undrained conditions.

Other clays can sometimes be very sensitive, including some lacustrine clays, especially varved clays, and some coastal and lagoon clays. Deposition in fresh water with adsorption of polyvalent ions can lead a flocculated structure. Subsequent addition or formation of organic dispersing agents contributes to the increase of sensitivity (Quigley, 1980; Brenner et al., 1981; Leroueil et al., 1990).

Most quick clays are Wisconsin and post-Wisconsin glacier in age, and they are commonly found in higher latitudes. The whole northern part of North America (Canada and part of the United States) was covered by the Wisconsin glaciation. Based on the maximum extent of Pleistocene glaciation, quick clays could exist in some areas of Chile, Argentina, and also in the Peruvian Andes (Snead, 1987; Liebling and Kerr, 1965). Sensitive clays are frequently found also in older river deltas, and in coastal areas not previously covered by ice. The deltas of Mississippi-USA, Orinoco-Venezuela, Amazon-Brazil are examples of large delta sedimentation environments. In the United States, the San Francisco Bay Mud and the Boston Clay are examples of clays formed in coastal areas. In Venezuela, Brazil and Argentina, many coastal areas are covered by soft clays. Recently, a few sites showing very high sensitivity have been found along the Brazilian coast. Local deposits of lagoonal and coastal clays derived from volcanic rocks are found in Mexico and Central America that have low to medium sensitivity. The very compressible and sensitive Mexico City clay was formed from volcanic ash and sedimentation into a lake.

Figure 21 shows the general distribution of quick and very sensitive soft clays in America. Deposits of quick clay also occur in Norway, Sweden, Japan and New Zealand.

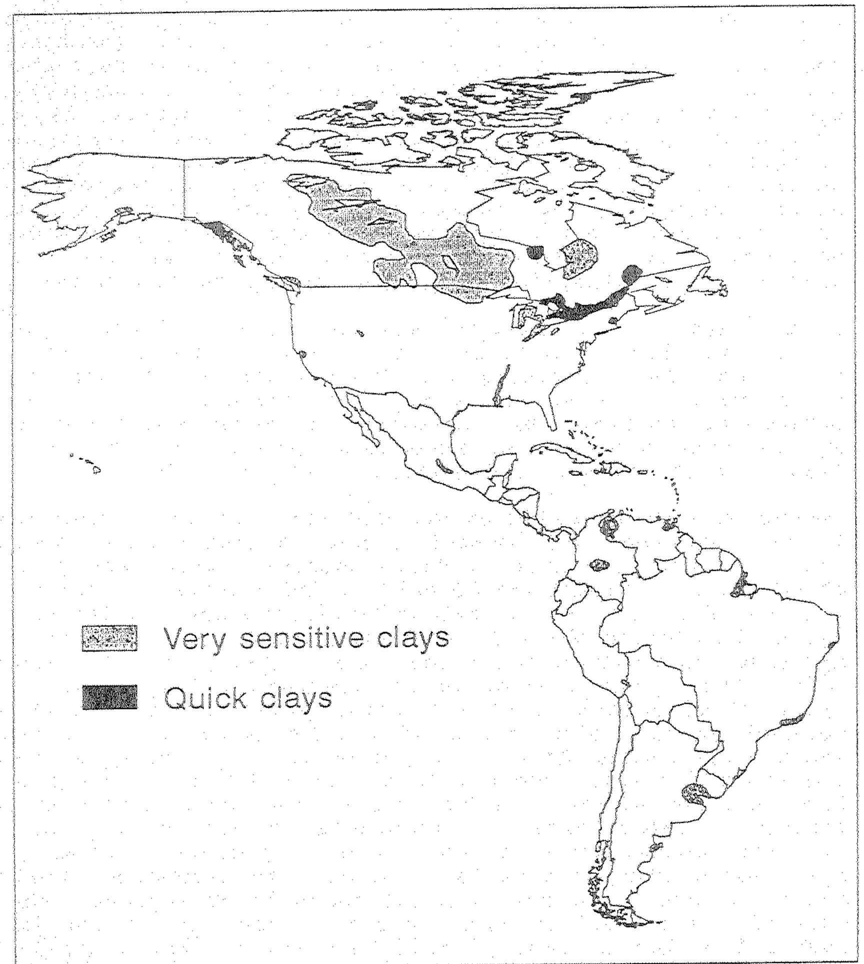

Fig. 21 General distribution of very sensitive and quick clays in the Americas.

Identification of sensitive clay

The liquidity index LI, defined as (water content-plastic limit)/plasticity index, is an excellent indicator of relative soil properties. Clays that have become highly sensitive by leaching, ion exchange, or dispersing agents have a liquidity index greater than 1.0. The general relationship between sensitivity, liquidity index, and effective stress can be seen in Figure 22. Values of liquidity index as high as 3 have been observed in some Canadian quick clays. High values of the pore pressure parameter $\bar{A}_f$ also indicate high sensitivity. Values of $\bar{A}_f$ up to 1.5 to 2.2 have been measured for some quick clays.

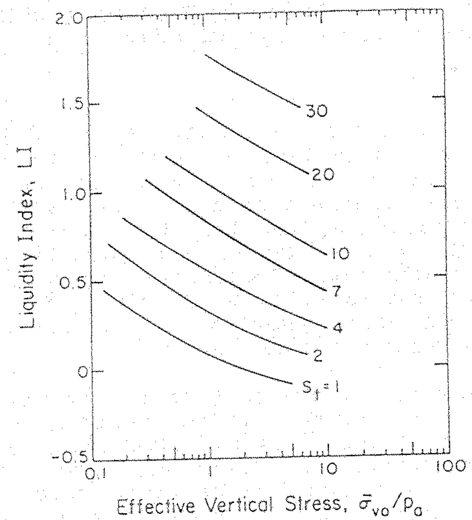

Fig. 22 General relationship between sensitivity, liquidity index, and effective stress (from Houston and Mitchell, 1969).

The measure of sensitivity (S_t) proposed originally by Terzaghi (1944) is the ratio of the peak undisturbed strength to the remolded strength as determined using the unconfined compression test. The remolded strength of highly sensitive and quick clays is so low, however, that test specimens cannot be formed. As the vane shear test is widely used for determination of the strength of soft clays in the field and provides a rapid method for determination of both undisturbed and remolded strength, quantitative expression of sensitivity in terms of vane shear strength is common. According to the classification system proposed by Rosenqvist (1953): $S_t \approx 1$ insensitive clay; S_t = 1-2 slightly sensitive clay; S_t = 2-4 medium sensitive clay; S_t = 4-8 very sensitive clay, and $S_t > 8$ quick clay, ranging from slightly quick (S_t = 8-16) to extra quick clay ($S_t > 64$). Other classification systems are given by Mitchell and Houston (1969).

Geotechnical properties

Typical values of properties of some representative highly sensitive and quick clays (as indicated by Liquidity Index values) are given in Tables 12 and 13. Other data are given in Table 10 in the section of this paper on Soft Organic Clays. Some soils listed there, like Juturnaiba organic clays and Sarapui clay from Rio de Janeiro, and Mexico City clay, are medium to very sensitive clays.

Index properties

Although the amount of clay size material (%< 2 micron) may be substantial (more than 50%) in quick clays, much of the fine material may be rock flour rather than clay minerals. The most common clay minerals in quick clays are illite and chlorite. Montmorillonite, vermiculite and mica are sometimes present. Quartz, feldspar, and hornblende are the dominate non-clay minerals. Carbonates are also sometimes present (Liebling and Kerr, 1965; Quigley, 1980). In less sensitive clays in South America, it is common to find kaolinite as the dominant clay mineral.

Conversion of sensitive clays to highly sensitive clays by leaching, ion exchange or dispersing agents is accompanied by a decrease in liquid limit, plasticity index, and activity, and an increase in liquidity index at constant effective stress. Once the liquid limit drops below the natural water content, the liquidity index LI becomes greater than 1 and the sensitivity increases to values above about 8 to 10. Values of activity as low as 0.15 have been reported. The usual classification of very sensitive and quick clays in the Unified System is CL-ML, CL, and CH.

Hydraulic conductivity

The relationship between hydraulic conductivity and void ratio for several soft sensitive clays is shown in Figure 23. From a practical point of view a relation, $\log k = \log k_0 - (e_0 - e)/C_k$ is excellent for initial void ratios less than 2.5 and for volumetric strains of practical interest in engineering problems. The permeability change index C_k is simply correlated to the initial void ratio,

Table 12. Properties of some sensitive clays (Tavenas et., 1983)

Site	Depth m	w %	LL %	PL %	PI %	LI	%< 2μ	σ'_p kPa	C_c
Champlain Sea clays									
St.Zotique	2.00 - 17.00	91 - 63	61 - 43	25 - 23	36 - 20	1.8 - 2.2	80 - 60	50 - 240	6.0 - 2.0
Ft. Lennox	6.10	79	70	22	48	1.2	81	180	3.0
St.Hilaire	9.50	69	55	23	32	1.4	71	125	4.0
Mascouche	3.80	61	55	24	31	1.2	76	290	2.8
Louiseville	2.90 - 26.00	79 - 60	71 - 59	27 - 25	44 - 34	1.2 - 0.8	77 - 85	80 - 300	3.7 - 2.2
Batiscan	5.50 - 20.50	80 - 71	35 - 54	22 - 24	17 - 31	2.6 - 1.5	77 - 91	80 - 190	2.2 - 4.5
St.Thuribe	6.90	52	44	22	22	1.3	44	195	1.2
St.Alban	1.90 - 7.80	90 - 40	53 - 28	25 - 18	28 - 10	2.7-2.0	78 - 31	40 - 100	2.5 - 1.2
Other Canadian clays									
B2	4.90 - 13.10	31 - 38	30 - 20	15 - 14	15 - 6	1.4 - 2.9	36 - 43	150 - 105	0.3 - 0.5
B6	2.80 - 13.40	53 - 29	24 - 44	14 - 25	21 - 9	2.1 - 0.9	76 - 51	130 - 180	0.7 - 0.3
Matagami	1.90 -10.30	108-48	74 -48	25 -28	49 - 20	2.3 - 1.4	91 - 65	55 - 90	5.6 - 1.2
Other clays									
Atchafalaya	20.80	65	99	37	62	0.5	76	160	1.1
Backebol	5.40	81	74	28	46	1.1	59	55	2.2
Lilla Mellösa	4.30	104	111	38	73	0.9	63	40	3.1

Table 13. Mean property values of some sensitive clays. (after Windisch and Young, 1990)

Clay	w_n %	LL %	PL %	PI %	LI	σ'_{vo} kPa	σ'_p kPa	$(S_u)_{FV}$ kPa	No. data points
ECM	57.5	48.5	23.7	24.8	1.61	80.9	212.3	57.5	610
CHA	62.8	53.4	25.1	28.3	1.49	83.6	202.7	55.8	391
BOJ	63.9	51.3	24.0	27.4	1.77	53.8	108.7	29.8	127
SCA	80.1	73.0	26.4	46.6	1.22	46.5	50.9	13.6	47

ECM - Eastern Canada marine clays
CHA - Champlain Sea clays
BOJ - Barlow-Ojibway lacustrine clays
SCA - Scandinavian clays

$(s_u)_{FV}/\sigma' = 0.27$ for ECM and CHA

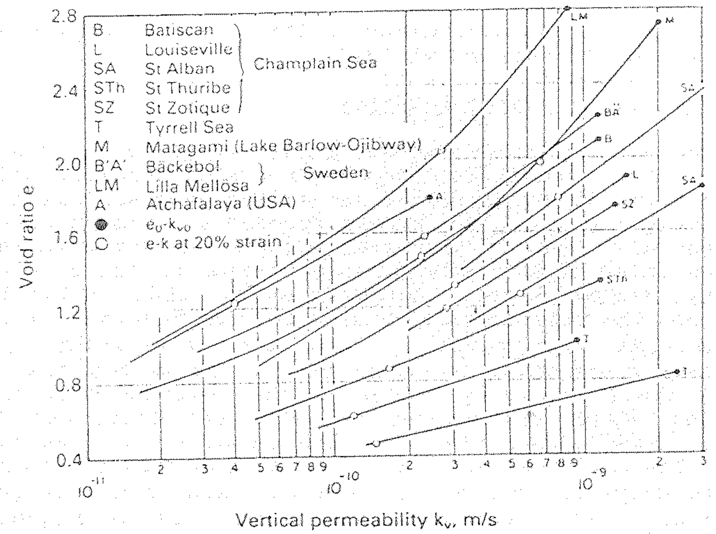

Fig. 23 Variation of vertical permeability with void ratio for various clays (from Tavenas et al, 1983/Leroueil et al, 1983).

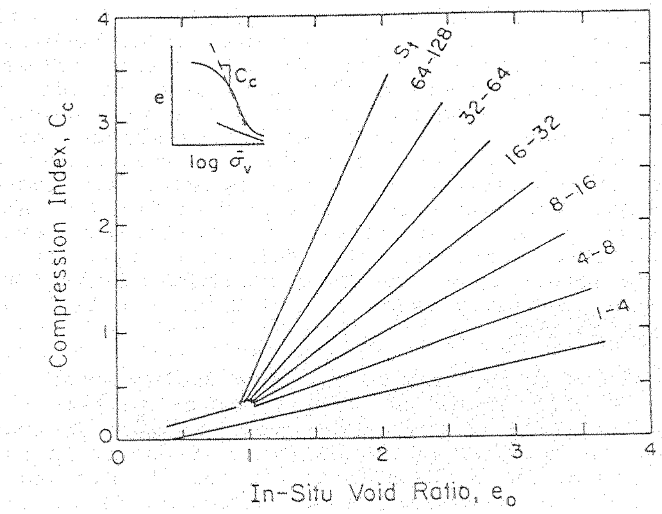

Fig. 24 General relationship between compression index, void ratio, and sensitivity (from Leroueil et al, 1983).

$C_k = 0.5\ e_0$. It was also observed that k is not appreciably anisotropic (Tavenas et al 1983).

Compressibility characteristics

The compression index values shown in table 12 clearly indicate that sensitive clays have a high compressibility in the normally consolidated range. The compressibility is relatively low until the stress exceeds the preconsolidation pressure, when it increases sharply. As the void ratio reduces under higher consolidation pressure, the compressibility becomes lower. Leroueil et al.(1983, 1990) have developed the relationship between the compression index C_c, the initial void ratio e_0, and the sensitivity S_t of sensitive clays shown in Figure 24. It can be seen that the higher the sensitivity higher is the compressibility. In addition, Mesri and Godlewski (1977) and Mesri and Choi (1985), have shown that for the majority of inorganic clays, the ratio between the secondary compression index C_α and the compression index C_c is unique and in the range of 0.04 ± 0.01. For organic soils this value can be higher as noted in the section of this paper on organic clays.

Shear strength

Some important principles and concepts of strength behavior in sensitive clays have been established by Houston and Mitchell (1969) and Leroueil et al (1983, 1990). When a saturated sensitive clay is remolded under undrained conditions, the structure is broken down and the pore water pressure increases, causing the effective stress to decrease, with a consequent reduction in the undrained strength. The higher the initial effective stress and the higher the water content, the higher the sensitivity. A unique correlation between the liquidity index and remolded shear strength has been found, as shown in Fig. 25. Based on this curve and on the definition of sensitivity, Leroueil et al.(1990) proposed the following equation for relating the sensitivity, the shear strength of the intact clay, and the liquidity index : $S_t = S_u(LI - 0.21)^2$, with the strength in kPa.

Highly sensitive clays tend to collapse during shear, thus generating a high value of pore pressure parameter $\bar{A}_f$, defined as the ratio of the excess pore pressure generated by the shear deformation to the deviator stress at failure. This rapid reduction in the effective stress is responsible for a reduced peak undrained shear strength. Thus, while the ratio S_u/σ'_c, in the normally consolidated range

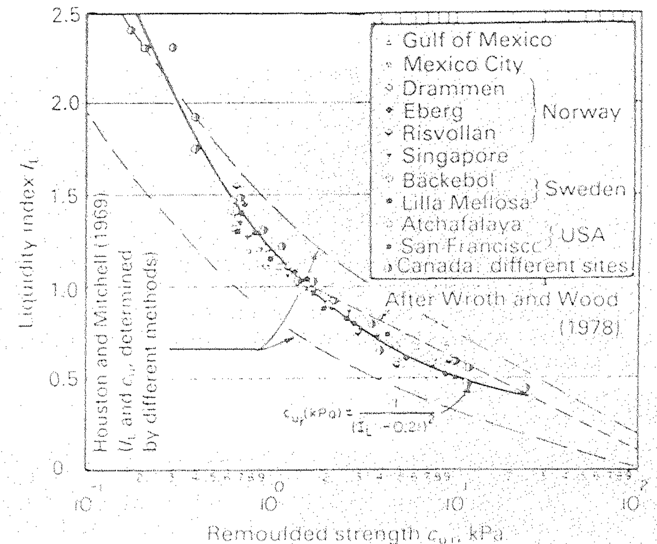

Fig. 25 Relation between remolded shear strength and liquidity index (from Leroueil et al,1990).

typically varies from 0.2 to 0.35 for most clays, its value for quick clays can be less than 0.1.

Because of the flocculated structure and cementation of most sensitive clays, very little strain is needed to break the bonds at particle contacts. Generally, the more sensitive the clay, the lower the strain at failure. A typical stress-strain curve for very sensitive clay shows a marked peak at a strain of only few percent followed by strain softening. Quick clays often exhibit brittle behavior during unconfined loading, fracturing into chunks at low strains, before turning into a fluid mass upon further working.

Treatment of sensitive clays

Various techniques have been used in sensitive soft clays to improve their stability and suitability as foundation materials. They include use of sand and prefabricated vertical drains, geotextiles, stone columns, lime stabilization, embankments on piles, electro-osmosis and dynamic compaction. For more information on the methods and their applications see Mitchell(1981) and Leroueil et al (1990).

A unique geotechnical property

The dominant feature of highly sensitive and quick clays is their very low remoulded shear strength. However, when samples of quick clays are stored after sampling, changes in the pore water chemistry can occur due to bio-chemical reactions that increase the divalent to monovalent cation ratio of the pore water and reduce the pH. This has relatively little effect on the undisturbed strength, but can cause significant increase in the remolded strength. Consequently, the results may not properly reflect the true in situ behavior. To minimize these aging effects, samples of quick clays should be tested as soon as possible after sampling or special storage precautions should be employed (Lessard and Mitchell, 1985).

Case history

At least 25 large landslides and innumerable smaller ones have occurred in the Champlain Sea and Lake Barlow- Ojibway area (Ottawa - St. Lawrence lowlands) of eastern Canada during the 20th century. More than 100 lives and 8,000 hectares of land have been lost. Soft to stiff overconsolidated deposits that are naturally cemented sensitive clays are common in this region (Mitchell and Klugman, 1979). Three modes of mass wasting have been observed:

1. Relatively small slides involving one or two rotational slips.
2. Intermediate sized failures involving 10^3 to 10^4 m^3 of soil, which form by multiple retrogressive rotational failure that leave a crater behind.
3. Earthflows that develop without rotational sliding and include stratigraphic units of varying geotechnical properties, geologic history, and slope height. These occur an average of once of every four to five years somewhere in eastern Canada. The largest documented earthflow involved 10^9 m^3 of soil.

The small rotational slips are usually triggered by high groundwater levels and/or toe erosion. Some minimum sensitivity is a prerequisite for retrogression, but the main factors are the undrained strength and the slope height. According to Mitchell and Klugman (1979), when the undrained strength S_u in the upland is less than 0.18-0.22 times the total overburden stress, retrogression is possible, and can create a very large earthflow in a short time period. When interspersed sands exist in layered silts and clays, hydrostatic pressure can cause mass wasting even though S_u > 0.25 times the overburden stress.

Slope monitoring instruments, including electric piezometers, tiltmeters, and micro-seismic/micro-acoustic sensors can be utilized to develop warning systems in areas of potential hazard. Research also showed, from existing landslides, that when the Stability Number N_s ($\gamma H/S_u$) is below 5, it indicates a generally safe site, but when N_s > 12 the site should be avoided. Slope stabilization techniques that have been used include slope grading and rockfill toe berms. Currently, the practice is to implement stabilization systems before failure occurs.

CARBONATE SAND

Carbonate sands are mostly organically produced in a shallow marine enviroment. Ideal conditions for carbonate growth are warm, shallow, clear water with normal marine salinities. Most carbonate sand is produced and deposited at the same location; therefore, the type and distribution of carbonate sand is predictable based in enviromental factors.

Carbonate minerals are very soluble under normal conditions so diagenesis, including dissolution and cementation, are very common. The fabric of carbonate sand differs markedly from that of siliceous sands because of intraparticle porosity. Particle angularity, particle structural weakness, variable cementation, and the unusually high void ratio all have important implications on the behavior of carbonate sands, with the high void ratio being the most important behavior-determining feature.

The most important geotechnical problems with carbonate sands have been in connection with the support of offshore structures using deep foundations.

Formation and distribution of carbonate sand

Modern carbonate sediments are found in five major groups of depositional enviroments: shallow water marine; deep-water marine; evaporitic basins; fresh water lakes and springs; and eolian. Of these the vast majority of carbonates are formed in the marine enviroment. Deep-water carbonates cover more than 50% of the sea-floor and comprise over two thirds of all calcium carbonate fixed by marine organisms (Scholle et al, 1983). While volumetrically less significant, shallow-water carbonates still produce very large quantities of carbonate sediments and are a source for deep-water carbonates. Most carbonate sand is produced in shallow-water.

Shallow-water carbonates form largely by biochemical precipitation of calcium carbonate by calcareous algae, planktonic biota, invertebrate organisms and other calcareous marine plants. Because photosynthesis by algae and other plants is responsible for a large portion of carbonate production, shallow-water carbonates are restricted to warm and clear water. Models for carbonate deposition on the shelf generally fall into two main categories: the carbonate ramp and the carbonate platform. The ramp model is typical of carbonate deposition in the Persian Gulf where the depositional surface decends into deeper water without a slope break. The platform model,where a slope break separates the shelf from a deeper basin, is applicable to many shelf enviroments in the Western Hemisphere. The main depositional enviroments of the carbonate platform are: shelf margin, open shelf, and tidal flat (Lloyd et al, 1986; Milliman et al., 1974).

Deep-water marine carbonates are deposited in areas below the strong influence of waves and tides in marine basins. Three sources of carbonate material are major contributors to deep-water carbonates (Pettijohn, 1975): pelagic organisms; carbonate debris transported from shallow water; and benthonic organisms. Carbonate debris transported from shallow-water is the main source of carbonate sand. Benthonic organisms are locally important, but generally occur only in restricted enviroments. Pelagic carbonate sedimentation is very widespread, but is typically fine-grained resulting in carbonate oozes. Widespread carbonate sand deposits are not common except immediately adjacent to the shelf margin.

Most shallow-water carbonate sands are deposited betwwen 30 degrees north and south of the Equator (Fig. 26). In North America, shallow-water carbonates are prevalent along the Florida coast, mixed with percentages of siliciclastic sediments. The largest carbonate sand accumulations in the Western hemisphere are in the Gulf of Mexico and in the Caribbean on the Great Bahamas Bank, fringing the Yucatan Peninsula and south along much of Central America, and along parts of the northern coast of South America (Ginsburg and James, 1974). Some shelf carbonates also occur along the west coast of Central America and north into the Sea of Cortez. Carbonate sand in South America is restricted to the Brazilian shelf, from the Amazon delta south to the Uruguay-Brazil border. Carbonate sand along the Brazilian coast is usually associated with coral reefs (Campos et al, 1974).

The principle areas of eolian carbonate deposits in the Western hemisphere are: Bermuda, Bahama Islands, Yucatan coast, Baja California, Cayman Island, Gilbert Islands,

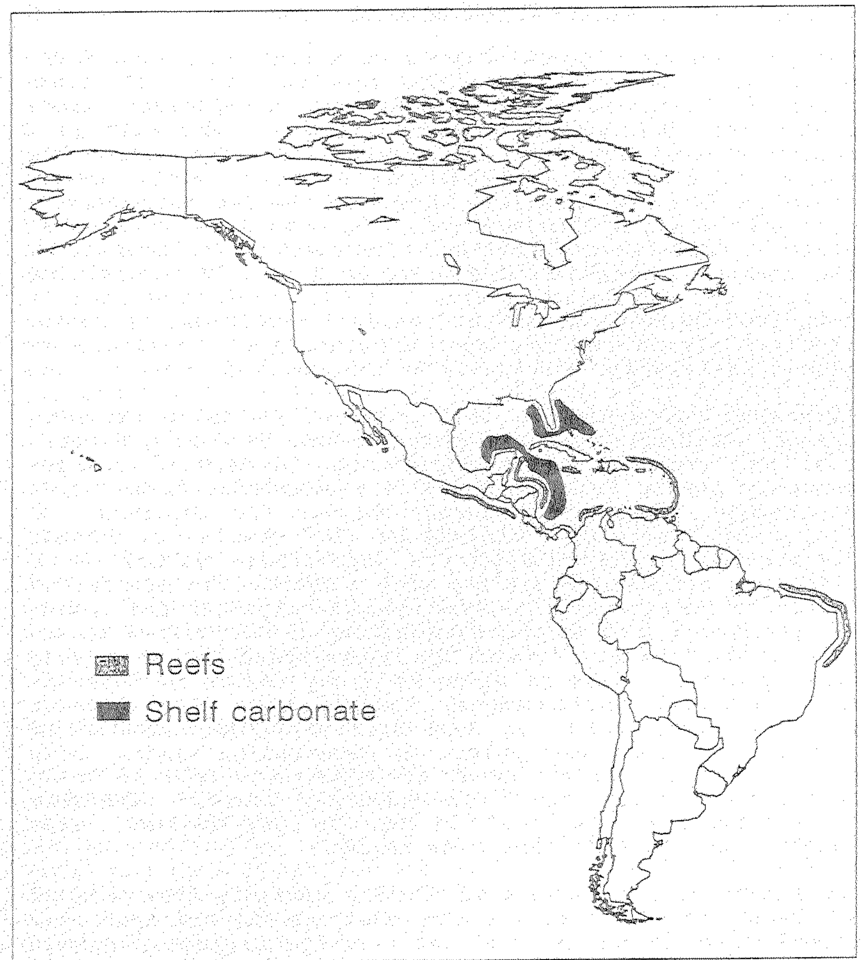

Fig. 26 Distribution of carbonate sands in the Americas (from Wilson,1975).

Christmas island, Great Salt Lake Desert, and the Peruvian coast (Mckee and Ward, 1983).

Composition, diagenesis and special fabric

The predominant minerals that form marine carbonate sediments are aragonite, calcite, and dolomite. Of these minerals, aragonite and calcite are considerably more common than dolomite in modern carbonates; dolomite is a common mineral in older carbonate rocks. The metastable forms of aragonite and calcite predominate in modern sediments but are virtually absent in the geologic record. Carbonate mineral hardness varies between 3 and 4 on Mohs' scale of hardness compared to 7 for quartz. The density of the common carbonate minerals varies from 2.72 for calcite to 2.94 for aragonite (quartz = 2.65). Carbonate sand is comprised of skeletal and non-skeletal particles. Of these, skeletal grains are the most frequent and widespread on modern shelves(see Scholle,1978; Wilson,1975).

Carbonate sediments are very susceptible to diagenesis. Changes can begin in loose grains before or during deposition and continue after deposition. Diagenetic changes commonly result from exposure to a subaerial fresh water enviroment, movement of water through the sediment and biological activity. Diagenesis can result in either cementation or dissolution of individuals grains in a carbonate sediment or of the pore network. The process of carbonate cementation involves complex physical and chemical phenomena such as the dissolution of carbonate, transport through mineral voids and reprecipitation as a void filling cement. Time or aging is a key factor in cementation, although environmental factors also play a major role. A warm, arid, or semi-arid enviroment tends to promote the rapid development of cementation bonds, while a cold, wet enviroment tends to reduce the rate at which cementation progresses. Consequently, marine carbonate sands should be expected to be, in the average, less cemented than desert carbonate sands.

The fabric of carbonate sand differs markedly from that of siliceous sands by the common presence of intraparticle porosity, which occurs as a result of the original morphology formed by the organism, as a result of boring organism, and by partial dissolution of carbonate grains. Skeletal fragments vary widely in shape, size, and the amount and patterns of intraparticle porosity. Carbonate particles range from nearly perfectly spherical ooids to highly elongate particles such as pelecypod fragments, and some of them can be broken forming angular particles. These geometric characteristics when associated with the inherent fragility of carbonate as a material, are responsible for the structural weakness of carbonate particles (Poulos, 1989).

Engineering classification of carbonate sand

Widely used geotechnical classification systems, such as the Unified Soil Classification System, are not fully adequate for carbonate sands (Celestino and Mitchell, 1983; Dutt and Ingram, 1990). Demars and Chaney (1982) (see also Datta et al,1982) suggest that the following factors should be considered to describe the behavior of marine soils: (1) carbonate content; (2) degree and uniformity of cementation; (3) grain size and grain size distribution; (4) Atterberg limits of the non-carbonate fraction; and (5) geological and compositional characteristics.

Several other classification system have been proposed and one of the more widely used system (Dutt and Ingram, 1990) was suggested by Clark and Walker (1977). Their scheme uses (1) grain size, separated in two groups, poorly indurated soils and indurated soil and rock; (2) particle composition in terms of percent carbonate regardless of mineralogy; (3) degree of induration or cementation; and (4) geologic origin of the carbonate component.

Noorany (1989) proposed an extension of the Unified Soil Classification System for marine sediments. His system divides marine sediments into three major groups: lithogenous, hydrogenous, and biogenous. Marine carbonate sands are classified as oolite sand, Ot,in the hydrogenous group, or as a bioclast sand with some gradation (BSW, BSP, BSM, BSC), in the biogenous group.

Geotechnical properties

The most important geotechnical problems associated with carbonate sands have been in connection with deep foundations for offshore platforms. The very high void ratio, the random cementation, and the high crushability of carbonate sands are responsible for low skin friction and end bearing capacity. The initial void ratio, e_0, which appears to be the most important factor controlling the behavior of carbonate sands (Poulos,1989), is a function of the effective stress state in-situ, particle angularity and presence of cementation. Values for e_0 for carbonate sands are in the range of 0.8 to 1.4, while for silica sands the usual range is 0.4 to 0.9.

There are also difficulties associated with determining the in situ properties of carbonate sands. Sampling is difficult, because inserting the sampler, especially by offshore percussion methods, breaks cementation bonds and may cause grain crushing. Sample trimming causes additional disturbance. Furthemore, changes in temperature, pressure, and carbonate concentration that occur during sampling may cause changes in sample cementation during retrieval (Beringen et al, 1982).

Crushability

The most widely used method to quantitatively measure the particle breakage is to compare initial and final particle size distributions after a loading cycle. Datta et al, (1979) defined the crushability coefficient as the ratio of the percentage of particles finer than the D_{10} size of the original sand after testing to the percentage of particles finer than D_{10} before testing. The crushability coeficient is a function of the last stress applied.

Compressibility and deformation characteristics

Compressibility indices of carbonate sands may be up to 100 times those of a standard reference silica sand (Murff, 1987). This compressibility is related to the low crushing resistance of the particles. Semple (1988) observed that the correlation between ultimate volumetric strain in drained triaxial tests correlates practically linearly with initial void ratio both for carbonate and silica sands.

Strength characteristics

Drained triaxial samples of carbonate sands show (Poulos, 1989) a volume reduction on shear, even at low confining stresses, and that the measured angle of internal friction decreases significantly with increasing confining stress. Several authors have also reported that carbonate sands are contractive even at very low values of strain (Golightly and Hyde, 1988; Airey et al, 1988).

Typical values for the angle of internal friction ϕ' are above 35° and are frequently greater than 50° (Murff, 1987). Hull et al. (1988) found, from triaxial tests with effective consolidation stresses varying between 100 and 400 kPa, values for peak ϕ' to be between 35° and 50° and ultimate ϕ' to be between 33.5° and 44° for a number of carbonate sands of Australia. The decrease in the peak friction angle over this range of consolidation stress was about 10%. Peak strains ranged from 7% to 36%, with 20% being a more typical value.

Morrison et al. (1988) performed direct simple shear tests on carbonate sands from the continental shelf, south of Mossel Bay, South Africa, and observed intense moduli

degradation in cyclic tests on samples maintained at constant height and a strain amplitude of 20% (20% degradation after the first cycle and up to 90% degradation at cycle number 20). In cyclic tests under constant vertical load the degradation was less intense than in constant height tests.

Owing to their high void ratio, carbonate sands can develop greater pore pressures during undrained shear than do normal quartz sands.

Implications for pile foundations

The mean peak values for the limiting skin friction, as determined by a number of pile load tests, varies from 13.3 to 20.3 kPa (Murff, 1987). Puech et al. (1990) found similar results in carbonate sands from the Bay of Matanzas on the northern coast of Cuba. For comparison, the API recommends a limiting shaft resistance of 80 kPa for silica sands.

There is now general agreement that low normal stress along the pile shaft is responsible for the low frictional resistance developed by piles in carbonate sands. The contractive behavior of carbonate sand, even at low confining stresses, promotes a partial separation of soil and shaft.

The unit end bearing capacity (q_p) of deep foundations in carbonate sands is inversely proportional to the sand compressibility, and it is typically lower than that of silica sands at similar relative densities. Some reference values are between q_p = 4 MPa for moderately compressible sands and q_p = 1 MPa for extremely compressible sands (Golightly and Nauroy, 1990; Puech et al, 1990). However, the mobilization of the end bearing capacity is progressive and requires displacement of the pile tip of 5 to 10% of the pile diameter. Somewhat higher values are given by Murff (1987). For example,values ranging from 6-8 MPa at 2-3 cm displacement to over 10 MPa for a displacement of the order of 12-13 cm were obtained in some tests.

Two alternative solutions have been used for the problem of low bearing capacity of piles in carbonate sand (Perkin et al, 1990; de Mello et al, 1989; Puech et al, 1990): (1) using close-ended piles instead of conventional open-ended piles; and (2) using open-ended piles with grouting. This method is very effective. Skin friction values up to 400 kPa can be developed, but it may be costly. The lateral capacity of piles in carbonate sands may be determined in the usual load-deflection curve (p-y) type analysis (Hagenar and Waanders, 1986).

Installation of piles in carbonate sands can also be a complicated task. Carbonate formations may contain large voids that may cause piles to sink, rendering the required pile length extremely variable. In addition, the presence of carbonate blocks in the sand may cause deviations in pile verticality during driving.

Case history

The Campos Basin Northeast Pole is an oil production system consisting of 7 fixed four-legged steel jackets, located on the Pargo, Carepeba and Vermelho sites, off the coast of the state of Rio de Janeiro, Brazil (de Mello et al., 1989). The water depth varies from 80 to 101 meters. The soil profile consists of an upper layer of silty fine to medium quartz sand that ranges in thickness from 13 to 28 meters, underlain by a skeletal coralline carbonate sand deposit, that occurs to depth between 97 and more than 115 meters below the sea floor. Cementation is variable, ranging from uncemented to highly cemented carbonate sand. The basic foundation design consisted of eight driven steel opened-ended pipe piles: four main 66 inch O.D. piles and four 80 inch O.D. skirt piles. All of the piles have approximately 70 m of penetration.

During the engineering phase of design, high values of unit skin friction were estimated from cone penetration tests (CPT) and laboratory tests of remolded samples of carbonate sand. However, prior experience suggested that lower values should be used with carbonate sands. An additional in situ test program was proposed to confirm the design parameters, and also to test the use of closed-end piles.

Side friction resistance of about 25 kPa was obtained by in situ pile pull out tests in the carbonate soils. This value is in accord with the values reported in the literature, and is about 65% of the initial estimated design value. Therefore, an additional four piles were required for each platform.

Comparison of open-ended steel pile and closed-end pile load tests showed that the closed-end piles had much greater bearing capacity in the carbonate sand. Driving closed-ended piles resulted in displacing enough sand to develop higher skin friction. Use of closed-ended piles on the Campos Basin platforms resulted in a cost savings of approximately U.S. $15,000,000 for the six platforms.

CONCLUSION

In the preceding pages we have described the formation, distribution, composition, and geotechnical properties and problems of nine of the many different soil types that are found in the Americas. All of them provide excellent illustration of the great importance of geologic history, composition, structure, and stress state as determining factors for geotechnical properties and behavior.

Three of the soils - decomposed granite, laterite, and volcanic ash soils (andosols) - are residual materials that are representative of successive stages of weathering of different parent materials under different climatic conditions. Three - expansive soils, collapsing soils, and loess - derive their volume change properties from both special compositions and metastable structures that are easily changed as a result of moisture content and stress changes. Two of them - soft organic soils and very sensitive clays - are sedimented fine-grained materials with behavior that is dominated by high compressibiity and low strength. Finally, carbonate sand is a biogenic material that is common to tropical and sub-tropical offshore areas.

From the assembled information on these very interesting, and sometimes very troublesome soils, it is clear that successful geotechnical engineering and earthwork construction require as much knowledge and insight about the materials and their properties as about the mechanics of soil. Thus, it is important to keep always in mind that the textbook teaching of soil and foundation engineering, in which we traditionally consider almost all soils as "sands" and "clays" or "cohesionless" and "cohesive", provides only a framework within which we are able to work. There can be great danger in attempting to force our real soils to fit our theories. On the other hand, successfully meeting the challenge of adapting our theories and methods to creative, safe, and economical design and construction on, in, and with the soils of America provides both the highest professional service to our clients and great personal satisfaction to us as engineers.

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