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SHEAR STRENGTH CHARACTERISTICS OF THE RECENT MARINE CLAYS IN SOUTH EAST ASIA

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SYNOPSIS

The shear strength characteristics of the Recent normally consolidated marine clays in the major deltas and coastal plains of South East Asia are reviewed. The undrained shear strength increase parameters, c_u/σ', are shown to increase with the distance from the coastline and are, generally, much higher than values quoted for the Recent marine clays in temperate climates. This is thought to be due to the pronounced secondary consolidation effect in these highly plastic and organic marine clays. A weathered crust has developed in surface horizons as a result of the weathering and leaching processes, and a classification into various crust types on the basis of undrained shear strength is proposed. Total and effective stress shear strength parameters at a number of sites in South East Aisa are summarized.

INTRODUCTION

The marine clays beneath the low, flat deltaic plains of South East Asia are a significant foundation material in the region. The deltaic plains cover an appreciable proportion of the total area (see Fig. 1) and are becoming, because of their high population densities, the foci of the urbanisation and industrialisation process which is now taking place thoughout the region. Unfortunately, however, the marine clays beneath the deltaic plains are extremely soft and the construction of the buildings, roads, airports, harbours, etc. required for this modernisation process presents considerable problems. This paper describes the shear strength parameters of the marine clays at the sites in South East Asia shown in Fig. 1, in the hope that a regional review will provide further insight into their strength characteristics.

GEOTECHNICAL PROPERTIES OF RECENT CLAYS IN SOUTH EAST ASIA

The geotechnical properties of the Recent marine clays in South East Asia have been described in detail by the Author (COX, 1968c) but they will be briefly described in this paper for the sake of completeness.

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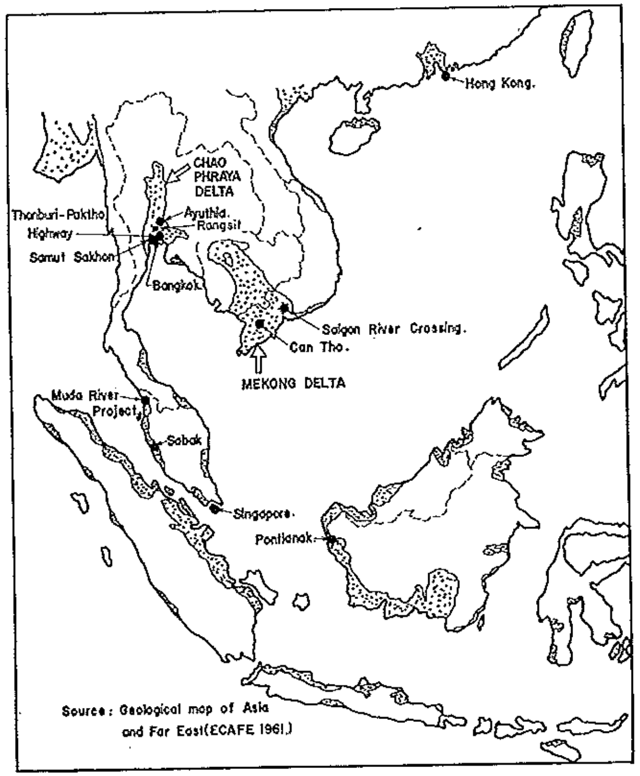
Formation

The marine soils are generally grey, soft, silty clays which vary from 10 to 30 m in thickness at the coastline to zero thickness away from the coast. A section through the Chao Phraya delta in Thailand is given in Fig. 2 to show the extent of a typical marine deposit in South East Asia.



Source: Geological map of Asia and Far East(ECAFE 1961.)

Fig. 1. The distribution of Recent clays in South East Asia.

The clays are derived from the material which is brought down by the major rivers in the area and are, therefore, of greatest lateral extent near these rivers. The rate of rise in sea level generally exceeded the deposition rate in the last 20,000 years and transgression of the sea over the land occurred until about 8,000 to 4,000 years ago, when little further rise in sea level

occurred. In this period sediments were deposited beneath the sea in what is called the marine zone. Land building began after this transgression sequence and extensive areas of deltaic and coastal plains have been formed which are up to 100 kilometres in width.

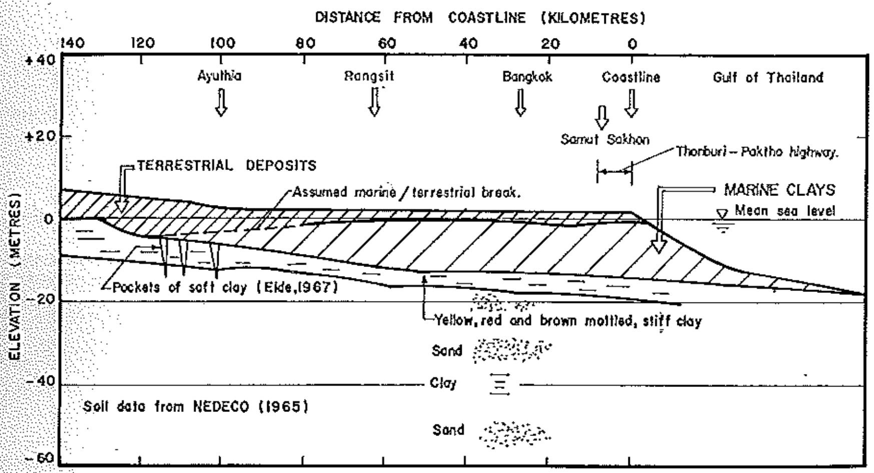


Fig. 2. Cross section through Chao Phraya delta, Thailand.

The marine clays are subject to wetting and drying cycles from tidal movements of approximately ± 1.0 m during emergence of the land (transition zone) and are then built up above mean sea level by the terrestrial deposition of silt and clay size particles in the annual flood waters (terrestrial zone). This terrestrial build-up is neutralized somewhat by secondary consolidation of the sediments, so that the natural surface level in most of the deltaic plains area is only 0.5 to 3.0 m above mean sea level.

The seasonal flood waters also leach the salt water from the pores of the marine clay. Values of the salt concentrations in the pore water of these clays are given in Fig. 3 for various sites in the Chao Phraya delta in Thailand. This diagram indicates that the depth and magnitude of leaching increases with the distance from the coast-line of the delta. This is to be expected, as the age of the deposit also increases with the distance from the coastline. The results indicate that leaching and the accompanying ion exchange reactions occur first in the crust (see profile for Samut Sakhon) but that, eventually, some leaching occurs throughout the whole soil profile (see profile for Bangkok).

The ground water table generally fluctuates from + 0.5 to — 1.0 m relative to ground level in the deltaic plains of continental South East Asia, which has a tropical wet and dry climate. A more constant water table occurs in the tropical wet climates of insular South East Asia, which does not have a pronounced dry season.

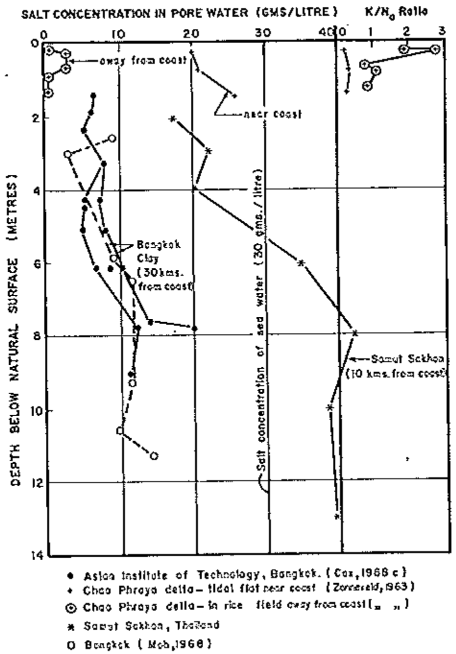


Fig. 3. **Salt concentration in pore water for Recent marine clays in Chao Phraya delta, Thailand.**

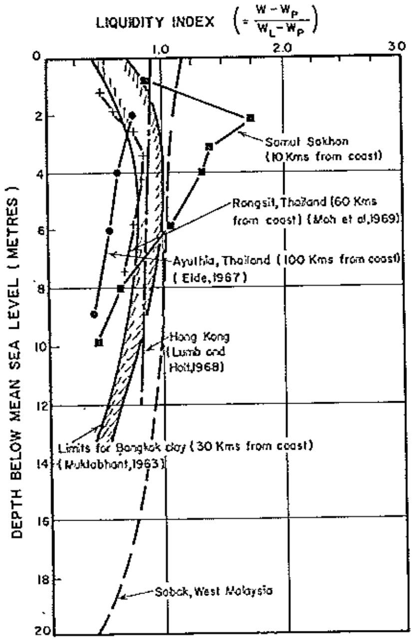


Fig. 4. **Liquidity index profiles for Recent marine clays in South East Asia.**

Index Properties

The clay percentage (< 0.002 mm) of the marine soils generally varies from 35% to 60%, the silt percentage generally varies from 40% to 60% and the sand percentage is generally less than 10% (COX, 1968c).

The moisture content values generally lie between 60% and 100% although values of 100% to 150% have been observed associated with high organic contents. The moisture content generally exceeds the liquid limit of the natural, undried material in young, immature deposits adjacent to the coast such as those found at Samut Sakhon in Thailand and at Sabak in Malaya (Fig. 4). The liquidity index values decrease at Bangkok (MUKTABHANT et al, 1963), some 30 kilometres from the coast, and decrease even further at Ayuthia (EIDE, 1967), some 100 kilometres back from the coastline (see Figs. 1, 2 and 4). This reduction in liquidity index with distance from the coastline is thought to be due to an increase in the liquid limit of the soil from weathering of clay minerals and to secondary consolidation of the sediments. Both of these effects increase with time, or with the age of the sediments, and, therefore, should also increase with distance from the coastline.

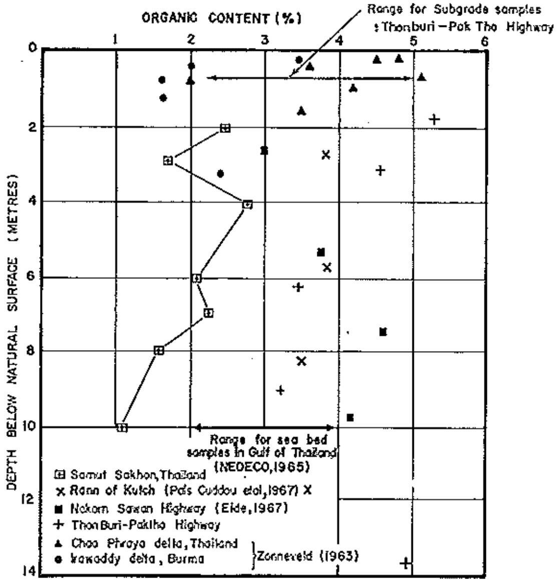


Fig. 5. **Organic content determinations for Recent marine clays in South East Asia.**

The marine clays are highly plastic with liquid limits ranging from 50% to 150%. The plasticity values generally plot close to the Casagrande A-line, indicating slight organic contents and a classification of CH/OH in the Unified System. Results of organic content determinations at several sites in South East Asia are shown on Fig. 5 and it is seen that most values lie between 2% and 5%. These organic contents are much higher than those quoted for the Recent post glacial marine clays in the temperate climates of the northern hemisphere.

The clay particles generally have a normal activity of between 0.75 and 1.25 but inactive clays are often found adjacent to the coast where weathering of the clay particles has not begun. Breakdown of the clay minerals begins in the organic acid environment of the marine clays (pH 4.5 to 6.0) and increases rapidly when subject to oxygen charged leaching waters, since this produces sulphuric acid and lowers the pH even further. Analyses of unweathered clay minerals in the Chao Phraya delta have indicated 60% to 75% illite, 10% of kaolinite and 5% to 20% of montmorillonite (NEDECO, 1965) but the percentage of illite decreases and the percentage of montmorillonite increases as weathering proceeds (COX, 1968c).

Consolidation Properties

An empirical correlation relating the compression ratio ($C_c / 1 + e_o$) of the virgin consolidation line to the percentage water content of the natural soil (w) has been proposed by the Author (COX, 1968c). This correlation is

$$\frac{C_c}{1+e_o} = 0.0045\,w \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

where C_c is the compression index and e_o is the intial void ratio. Casagrande preconsolidation pressures, or the critical pressures of BJERRUM (1967), have been determined from routine consolidation tests on several clays have been determined from routine consolidation tests on several clays *below* the crustal zone in South East Asia and these are shown in Fig. 6. The critical pressures (σ'_{vc}) generally exceed the overburden pressure (σ'_{vo}) and approximate to the value of $\sigma'_{vc} = 1.6\,\sigma'_{vo}$ given by BJERRUM (1967) for the Drammen clay in Norway. Some of the lower values could be due to sampling disturbances since the determination of the critical pressure is very sensitive to sampling techniques (BJERRUM, 1967).

It is evident from Fig. 6 that the marine clays are overconsolidated even below the weathered zone. This effect could be due to a fall in sea level since deposition but there is no other evidence for this (COX, 1970). The most likely explanation is that it is an apparent overconsolidation effect, similar to that described by BJERRUM (1967), which is due to a reduction in void ratio from secondary consolidation. Secondary consolidation of the marine clays in South East Asia should be quite pronounced because they are highly plastic and possess relatively high organic contents (shown in Fig. 5). This reduction in void ratio with time can be inferred from the decrease in liquidity index and the increase in shear strength (see later section)

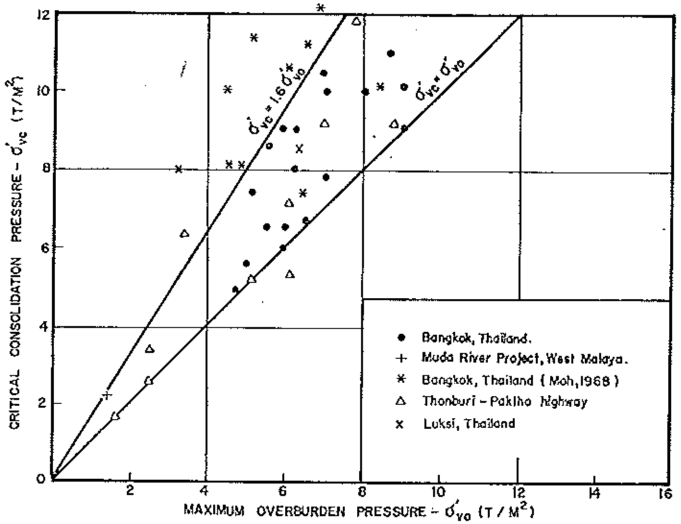


Fig. 6. Critical pressures for Recent marine clays in South East Asia.

with increasing distance from the coastline. A reserve resistance against further compression is present in these clays, therefore, if the induced stresses do not exceed the preconsolidation pressure. This effect is of great practical importance in the design of roadway embankments on these deltaic clays (COX, 1968a).

TOTAL STRESS SHEAR STRENGTH CHARACTERISTICS

The most common form of analysis used in stability problems on these clays is the '$\phi = 0$ analysis' (SKEMPTON, 1948) which is applicable immediately after the load is applied. In this analysis total stresses and a value of the apparent cohesion (c_u) with respect to these total stresses are substituted onto an assumed failure surface. The value of the apparent cohesion is determined by undrained compression tests on undisturbed samples by the vane apparatus (COX, 1967) or by the Dutch cone apparatus (BEGEMANN, 1965).

Below the Weathered Zone

A typical shear strength profile is shown in Fig. 7 (a), where it is seen that the strength is constant or decreases from the surface in the weathered zone but then increases linearly with depth. This section of the paper will concentrate on the shear strength properties below the weathered zone, i.e. below a depth of D_w.

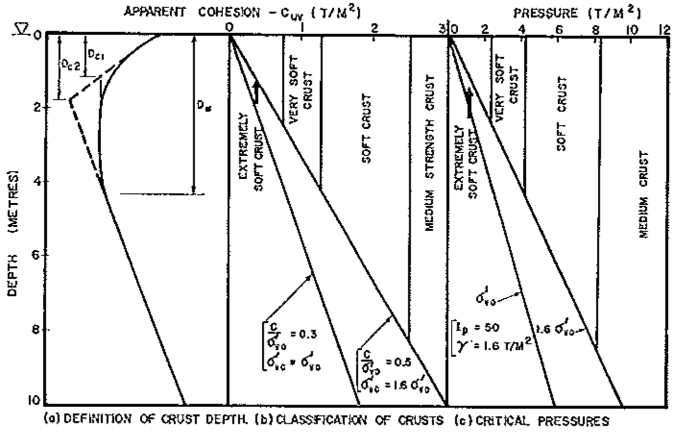


Fig. 7. Definition of depth, type and critical pressures in the weathered crust.

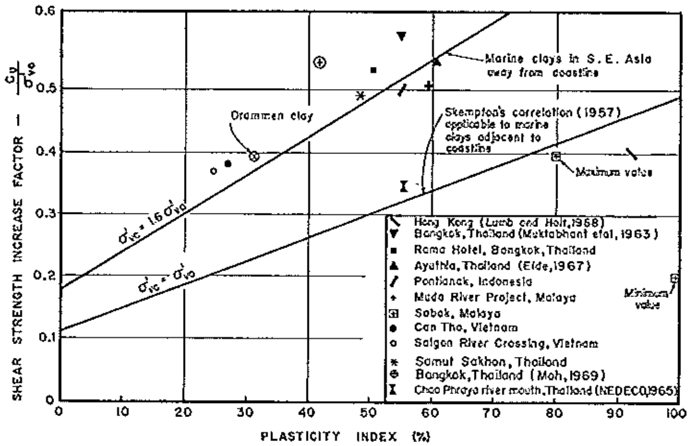


Fig. 8. Undrained shear strength relationships for Recent marine clays in South East Asia.

It can be shown from the growth rates of the major coastal plains in the region that primary consolidation of these marine clays should be complete

at all stages of the deposition process (COX, 1970), except for some fast growing plains in Indonesia. The growth of the coastline adjacent to the major rivers is much faster, however, and theoretical data indicates that the average degree of primary consolidation should only be from 30% to 70% at these locations.

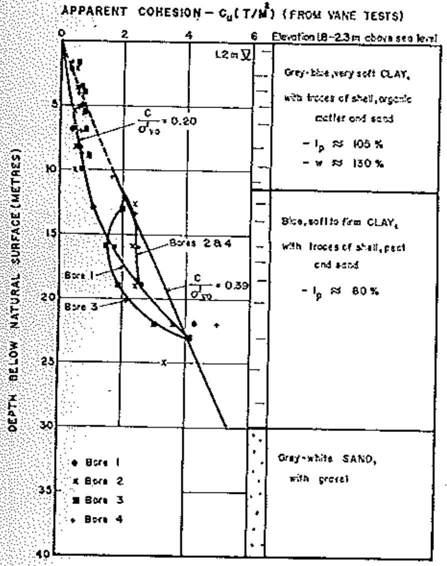


Fig. 9. Shear strength profiles for Recent clays at Sabak, Malaya.

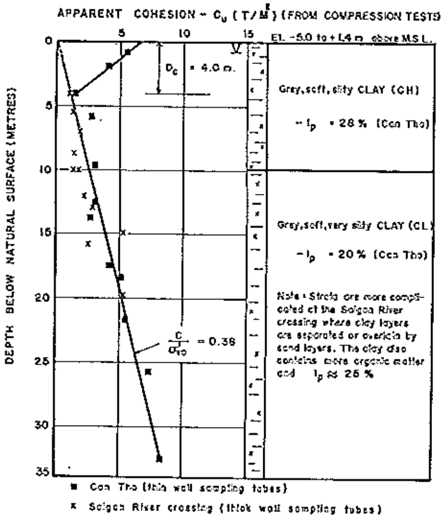


Fig. 10. Shear strength profiles for Recent clays in South Vietnam.

Figure 8 shows the variation in the shear strength increase factors, c_u/σ'_{vo}, with plasticity index for some marine clays of South East Asia. The shear strength profiles at Sabak in western Malaya (Fig. 9), which is located close to the mouth of the Surgai Bernam river, indicate that the sediments in this region could still be undergoing primary consolidation. The envelope of the maximum shear strength values with depth gives a value of c_u/σ'_{vo} equal to 0.39. This value plots very close to the correlation of SKEMPTON (1957) (see Fig. 8) and primary consolidation is probably complete at these points. There is a reduction in shear strength, however, in the middle of the lower clay layer and in the upper clay layer itself, locations where any residual excess pore pressures during the consolidation process would be expected. The shear strength increase parameter for the upper layer and the envelope of the minimum shear strength values both have a value of only 0.20 and this plots well below Skempton's correlation (see Fig. 8). The township of Sabak, where these vane tests were carried out, is located in a narrow isthmus of land between the Straits of Malacca and the Surgei Bernam river. It is possible, therefore, that the upper layer in this particular regional has been deposited very quickly and that primary consolidation is only 100% complete near the permeable boundaries of the clay layers. Similar profiles could be expected adjacent to river mouths in South East Asia and beneath the fast growing coastal plains of Sumatra and Java.

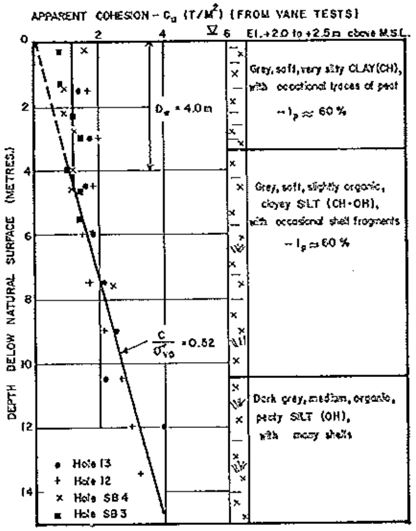


Fig. 11. **Shear strength profiles for Recent clays at the Muda River Project, Malaya.**

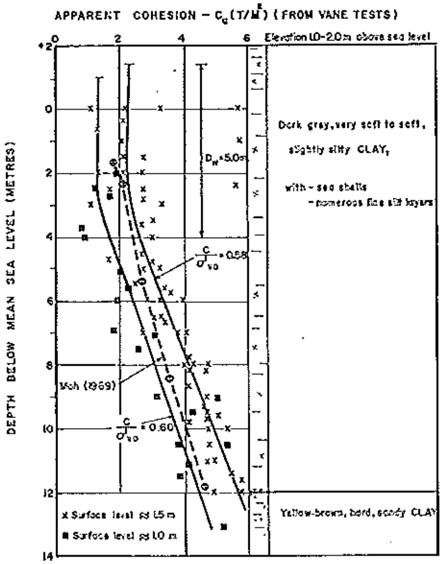


Fig. 12. **Shear strength profiles for Recent clays at Bangkok, Thailand (MUKTABHANT et al, 1967).**

Shear strength profiles are given for other marine clays away from the coastline in South Vietnam (Fig. 10), in Malaya (Fig. 11) and in Thailand (Figs. 12, 13 and, later, in Fig. 17). The shear strength increase parameters for these sites, together with the values derived from EIDE (1967) and LUMB and HOLT (1968) for other marine clays in the region, are plotted on Fig. 8. It is seen that, except for the values at the Chao Phraya River mouth and at Hong Kong, most of these parameters consistently fall way above the correlation of SKEMPTON (1957) for the marine clays in the temperate climates of the northern hemisphere. The data presented on Fig. 8 indicates that the shear strength increase parameter approximates to Skempton's correlation at sites adjacent to the present coastline, where primary consolidation is just complete, but increases beyond these values for sites away from the coastline, where the sediments are much older and have undergone secondary consolidation after deposition.

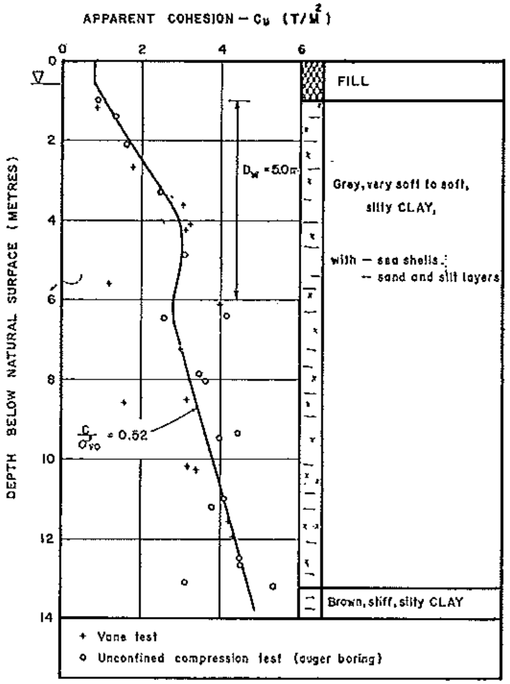


Fig. 13. **Shear strength profiles for Recent clays at the Rama hotel, Bangkok, Thailand.**

BJERRUM (1967) has found a higher shear strength increase parameter for the more highly plastic Dramen clay, which was laid down in a warmer, post glacial period under a more organic terrain. This clay has an organic content of approximately 1% and a shear strength increase parameter which plots very close to the correlation for the marine clays in South East Asia (shown in Fig. 8). The higher shearing strength for the Drammen clays is associated with a slight overconsolidation effect, since the preconsolidation pressure or critical stress derived from oedometer tests exceeds the overburden pressure by 60%.

These higher shear strengths and critical pressures in normally consolidated clays are considered to be due to a reduction in void ratio from secondary consolidation (BJERRUM, 1967) and would, therefore, be expected in the organic and highly plastic marine clays of South East Asia. The relationship of the critical stress to the effective overburden stress can be calculated from the shear strength data and calibration lines can be obtained. It is seen from Fig. 9 that the critical stress for the older marine deltaic clays in South East Asia is approximately 1.6 times the overburden pressure, i.e. $\sigma'_{vc} = 1.6\, \sigma'_{vo}$

The Weathered Zone

The shear strength characteristics of the weathered zone are much more variable than those below this zone since they are influenced by the length and nature of the environmental conditions after deposition.

Two *depths of crust*, D_{CI} and D_{C2}, and a *depth of weathering*, D_w, can be defined for a typical shear strength profile as shown in Fig. 7. RAYMOND (1967) has previously defined the crust depth D_{C2} but the crust depth D_{CI}, defined in this paper, has been found to be more useful in stability analyses in these marine soils. The *type* of crust can also be classified on the basis of the average vane shear strength in the weathered crust D_{CI}. Generally, the shear strength in the crust D_{CI} and the shear strength in the constant shear strength zone down to a depth D_w are the most common values required for stability analyses. It seems desirable to retain the same descriptive terminology for the crust types as normally used in describing cohesive soils, i.e. very soft, soft, medium, stiff and very stiff. A further category called *extremely soft*, and having an apparent cohesion of between 0 and 0.75 ton/sq.m is, however, added and the proposed classification is given in Table 1. It may be necessary, moreover, to subdivide, say, the soft crust into two sub-categories on any particular project to obtain further definition of crust types (e.g. a low range from 1.25 to 1.9 ton/sq.m and a high range from 1.9 to 2.5 ton/sq. m).

Table 1. Classification of crust types

Crust type	Average vane shear strength in depth D_{CI} (ton/sq.m)
Extremely soft crust	0 - 0.75
Very soft crust	0.75 - 1.25
Soft crust	1.25 - 2.5
Firm crust	2.5 - 5.0
Stiff crust	5.0 - 10.0

The crust type is often related to the use to which the land has been put. The investigations for the Thonburi - Pak Tho highway (see Figs. 1 and 4), which skirts the northern coastline of the Gulf of Thailand, showed that the crust was firm in the leached rice field areas, soft (1.9 to 2.5 ton/sq. m) in the uncultivated brushwood areas, and softer (1.25 to 1.9 ton/sq.m) in the saline mangrove, nipa and coconut plantations adjacent to the coast (COX, 1968b). The variation of shear strength in the weathered crust along the highway route and the correlation with land use is shown on Fig. 14.

The depth D_{CI} of the weathered crust in the deltaic plains generally varies from 1.0 to 4.0 m. The influence of the weathering process extends to greater depths, however, and can be felt up to 6.0 to 8.0 m below the natural surface. The depth of weathering at Samut Sakhon (6 m) is practically identical to the depth of the main zone of leaching, as can be seen in Figs. 3 and 17.

The nature of the weathering process in the surface layers has been investigated for the Norwegian marine clays by MOUM and ROSENQUIST (1957) and BJERRUM (1967). The depth of weathering in these clays also extends to 6 m and is attributed to the oxygen content of the leaching water. The oxidation process is said to weather the muscovite clay minerals to more active illite and montmorillonite, thereby increasing the liquid limit, activity and strength of the soil, and decreasing the liquidity index.

The weathering process should be particularly rapid in the South East Asian region since the high organic contents (Fig. 5) reduce the *pH* values in the clay deposits to 4.5 to 6.0 (DUDAL and MOORMAN, 1964) and produce an acid environment condition which fosters the decomposition of the clay minerals. The oxidation process lowers the *pH* of the soil even further and hastens this disintegration process.

The leaching process increases the shear strength in a further way by causing cation exchange reactions in this crustal zone. Initially, the clays are nearly saturated with calcium, magnesium, and sodium ions (see data of ZONNEVELD (1963) in Table 1 of his paper for unleached marine clays in South East Asia). Leaching removes the sodium ions and disintegration of the mica-type clay minerals liberates higher order cations such as potassium. The combined effect of the leaching and the weathering of the clay minerals results in an increase of the potassium/sodium ratio, from its value of approximately 0.2 when first deposited, to approximately 2.0 after weathering (BJERRUM, 1967). Values of the potassium/sodium ratio at several sites in South East Asia are given in Fig. 3, where it is seen that the ratio varies from 0.2 to 0.4 for the young soils in the mangrove swamps and on the tidal flats, and from 1 to 3 for the soils in the leached rice fields. These results appear to confirm the use of the potassium/sodium ratio as a

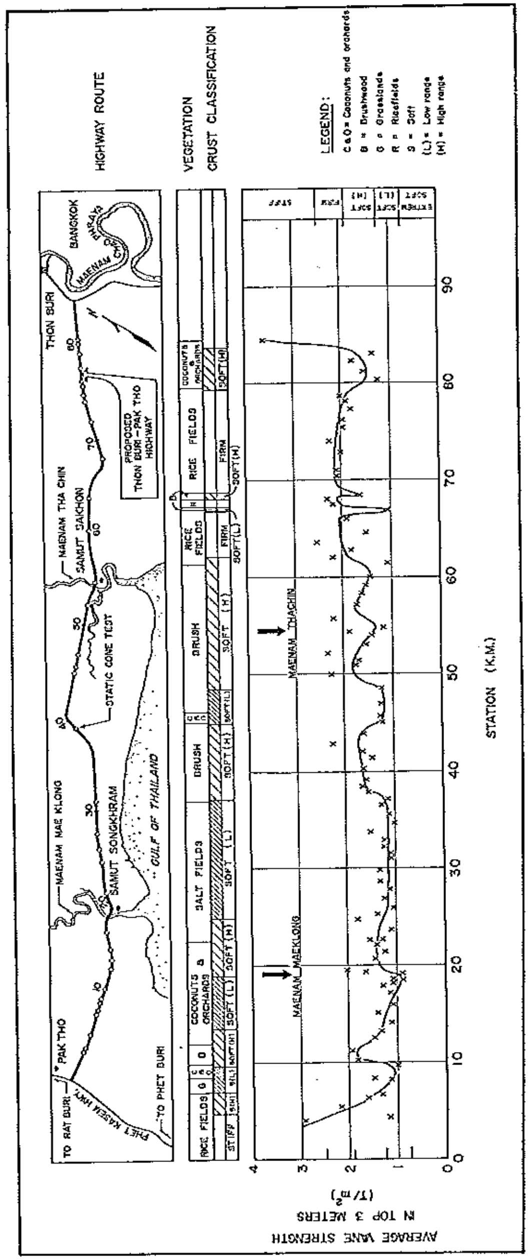


Fig. 14. Variation of crust type with vegetation along the route of the Thonburi - Pak Tho highway.

measure of the extent of weathering.

The Author (COX, 1968c) has suggested that the critical pressure profile can be approximately derived from the shear strength profile by the following expression.

$$\sigma'_{vc} = \frac{c_u}{0.11 + 0.0037\, I_p} \quad \ldots\ldots\ldots\ldots\ldots\ldots (2)$$

where I_p is the plasticity index (in %), σ'_{vc} is the critical pressure and c_u is the apparent cohesion with respect to total stresses. The critical pressures corresponding to the various crust types are plotted, from this expression, in Fig. 7 (c) for a soil with a plasticity index of 50%. It is seen that a uniform shear strength in the weathered zone, which is a very common condition, corresponds to a constant value of the critical pressure. This is to be expected if the development of the crust and of the critical pressures is due to the same ion exchange reactions during the leaching process.

Sensitivity

The true sensitivity of these marine clays is difficult to determine because the shearing strength of any sampled material has already been reduced by disturbances during the sampling operations. Strengths determined from the vane test probably gives the most reliable value for sensitivity. The vane sensitivity of the Bangkok clay is given by EIDE (1967) and by other sources as 8 to 10, which means that the clay is 'very sensitive' to 'slightly quick'. Further values measured on the Chao Phraya plain at Rangsit (MOH et al, 1969) and at Samut Sakhon are shown in Fig. 15. MISRA

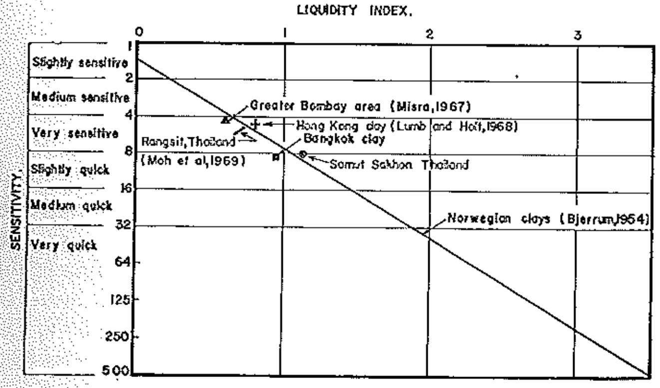


Fig. 15. Relationship between sensitivity and liquidity index.

(1967) and LUMB and HOLT (1968) each give vane sensitivities of 5 for the marine clays in the Greater Bombay area and in Hong Kong respectively. These values of sensitivity are plotted with their appropriate values of liquidity index on Fig. 15 and are seen to lie very close to the sensitivity-liquidity index relationships given by BJERRUM (1954).

The sensitivity of Bangkok clay tends to decrease towards the natural ground surface. This could be a reflection on the less saline environmental conditions during the deposition of these surface layers, since such conditions tend to give a more 'dispersed' and less sensitive structure.

TOTAL STRESS ANALYSES

The apparent cohesion, c_u, is used in the so called '$\phi = 0$ analysis' to determine the stability of footings, embankments, retaining walls, excavations, etc. The application of the $\phi = 0$ analysis in practice is very simple, since a single value of c_u is substituted onto assumed failure surfaces and a minimum factor of safety determined. The choice of the correct value of c_u to substitute in this analysis is an extremely complicated decision, however, and it will, therefore, be discussed in this section of the paper.

Relationships Between Shearing Strengths Determined by Compression, Vane and Cone Tests.

A detailed analysis of the theoretical and practical differences between the vane test and the compression test on 'undisturbed' samples has been given by the Author (COX, 1967). Theory shows that the compression test should give 10% to 25% higher values for the apparent cohesion than the vane test, provided that 'perfect' undisturbed samples are obtained (HANSEN and GIBSON, 1949). This has been confirmed in practice when high class sampling equipment has been used. Generally, however, disturbances during sampling result in the measurement of a compressive strength which is equal to or lower than the vane shear strength. The results given in Fig. 16 confirm this for the Bangkok clay. When good quality auger drilling is used, as at the Rama Hotel (Fig. 13), there is little difference between vane and compression test results. When wash boring drilling equipment is used (MUKTABHANT et al, 1963) the disturbance reduces the compression test strength in an unpredictable manner. The results of the soil investigations for the Thonburi - Pak Tho Highway seem to indicate that the unconfined compression test is unreliable when the vane shear strength is less than 1.5 ton/sq. m (300 lb/sq.ft) irrespective of the type of drilling equipment used (see Fig. 16). Similar results have been obtained by LUMB and HOLT (1968) who found that the Swedish foil sampler was the only reliable sampling method when the shear strength of the Hong Kong marine clay was less than 300 lb/sq.ft.

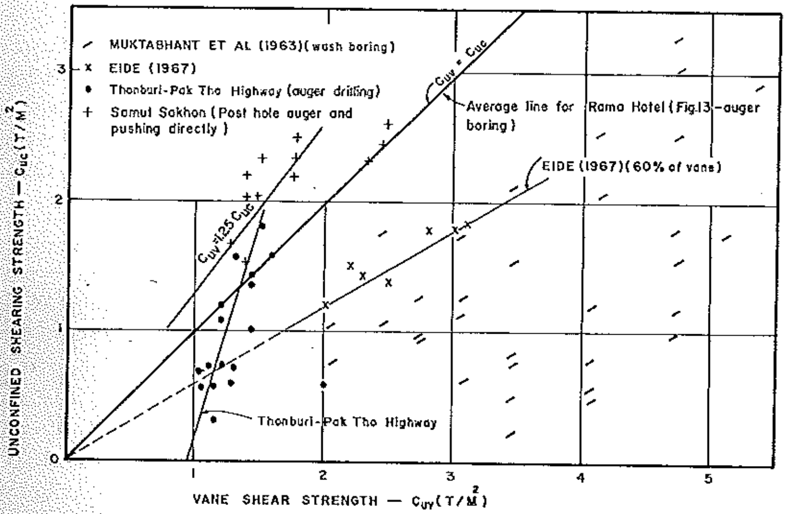


Fig. 16. Correlation between unconfined compression and vane shear strengths for Bangkok clay.

The results at Samut Sakhon, Thailand (Figs. 16 and 17) are particularly instructive. Thin-walled piston samples were taken by a variety of methods and compared with values obtained with a *Geonor* vane. Samples taken through the stem of hollow auger drilling equipment have comparable shear strengths to those measured by the vane test but better samples (i.e. higher shear strengths) are obtained by sampling at the bottom of a normal manually drilled auger hole. The best sample is, however, taken by pushing the piston sampling unit directly into the soil until the correct depth is reached and then taking the sample. The average sensitivity of the soil measured at this site was 4.5 by the vane and 8.0 by compression tests. The liquidity index varied from 1.6 at 2.0 m to 0.5 at 10 m (see Fig. 6) and the activity averaged 1.1. The piston sampling equipment was made in accordance with the specification recommended by KALLSTENIUS (1963).

The high values for the sensitivity and liquidity index of the Recent marine clays in South East Asia make good undisturbed sampling an extremely hazardous business. The results at Samut Sakhon indicate that sampling

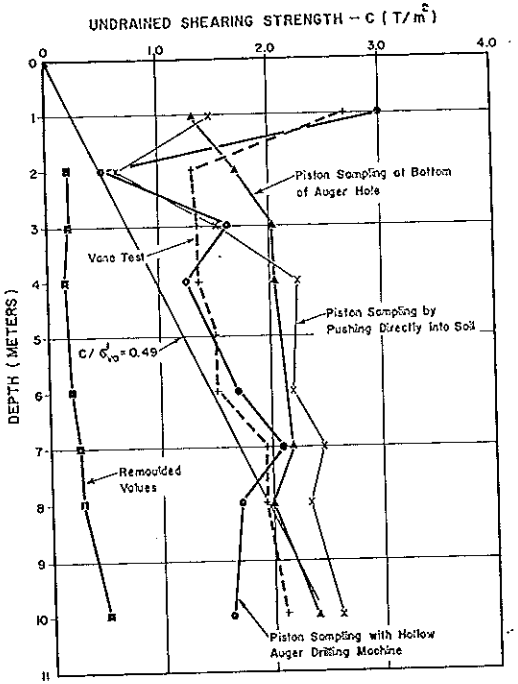


Fig. 17. Influence of sampling method on shear strength at Samut Sakhon, Thailand.

should be carried out with a thin walled piston sampler by sampling at the bottom of a post-hole auger hole or, even better, by pushing the piston sampler unit directly into the soil. Samples to depths of 10 m can be taken with portable sampling equipment by this method.

BEGEMANN (1965) has proposed a bearing capacity factor, or Dutch cone factor, of 14 to reduce the penetration resistance of the cone itself to a value of the apparent cohesion, c_u, with respect to total stresses. The influence of skin friction on the shaft of the cone apparatus must, of course, be allowed for, either by using Begemann's friction sleeve adjacent to the cone or by measuring the friction on a dummy rod. WESLEY (1967) compared cone resistance measurements against compression test values for several Indonesian clays. Very high cone factors were obtained but these could have been due to sampling disturbance and consequent reduction in the soil compressive strength; the upper envelope of the compression test results gave a cone factor of 18 to 20. PRASUTI (1965) correlated cone

and compression test results for the Bangkok clay and gave the expression

$$q_{(cone)} = 11.0\ c_u + 0.6 \qquad \text{kg/sq. cm} \ \ldots\ldots\ldots\ldots\ldots \ (3)$$

It appears, therefore, that the application of Begemann's cone factor of 14 will give a very approximate indication of the apparent cohesion, c_u. WISEMAN and SCHIFFMAN (1967) carried out penetration tests in the Bangkok clay with a flat disc on the end of a cased rod and deduced a bearing capacity factor of 10; vane test results were used as the basis for this correlation.

Relationship Between In Situ and Measured Shearing Strengths

The Author (COX, 1967) has previously summarized the available field records and has concluded that the vane shearing strength approximates to the *in situ* shearing strength derived from actual 'end of construction' failures in non-fissured, normally consolidated, saturated clays. Theoretically, the vane test should conservatively estimate the *in situ* shearing strength of the soil.

The $\phi = 0$ analysis has been critically reviewed in recent years and the conclusions of HANSEN and GIBSON (1949) have generally been confirmed. These investigators showed that the value of the apparent cohesion is not unique but that it is dependent upon the direction of the failure plane because of a reorientation of principal stresses during the loading process. Theoretical and experimental investigations have indicated that the lowest *in situ* undrained shearing strength occurs on a horizontal plane.

The effect of any reorientation of the principal stresses during the loading process on the undrained shearing strength has been considered in detail by LADD and VARALLYAY (1965). Any reorientation of principal stresses will lead to a reduction in shearing strength since this reorientation results in much larger changes in the principal stresses ($\Delta\sigma_1$ and $\Delta\sigma_3$) and much larger pore pressure changes, given by $\Delta u = B\,[\Delta\sigma_3 + A\,(\Delta\sigma_1 - \Delta\sigma_3)]$. The variation of the undrained strength with the direction of the failure plane and the reorientation of the principal stresses means that the value of the *in situ* apparent cohesion chosen for any stability analysis will depend upon the partcular stability problem being encountered. Several cases are discussed below.

Reasonable agreement between *in situ* strengths and strengths determined by vane tests and compression tests on 'perfect' samples should occur for a retaining wall in the active case, as no reorientation of principal stresses occurs (LADD and VARALLYAY, 1965). Smaller *in situ* strengths should be realized for a footing and for a retaining wall in the passive case, because reorientations of principal stresses do occur in these cases. It is suggested that the more conservative vane strength or the unconfined compression test strength on ordinary samples should be used in these instances. The unconfined compression test has probably been as successful as it has been in these particular cases because normal sampling disturbances have provided the required reduction in 'perfect' sample strength.

Table 2. Results of analyses of roadway embankment failures

Basis of Analysis	Safety Factor at Failure	
	Site 2	Site 3
No shear stresses at base of embankment (normal analysis)	1.36	1.49
Horizontal shear stresses at base of embankment from elastic, no arching stress distribution (PERLOFF et al, 1967)	1.21	1.27
Large horizontal shear stresses at base of embankment from full arching stress distribution (DAVIS and TAYLOR, 1962)	1.05	0.99

Very low *in situ* shearing strengths are realized beneath an excavation or at the toe of a slope, since the maximum possible reorientation of principal stress can occur in these cases. It is suggested that only 60% to 70% of the vane shear strength should be used in any analysis of base or heave failures. EIDE (1967) has analyzed several failures of roadway embankments in the Chao Phraya delta in Thailand and has found that the *in situ* undrained shear strength at failure was only 70% of the shear strength as determined by the vane test; i.e. the factor of safety at failure was 1.5. Similar results have been obtained in a trial embankment programme associated with the Thonburi-Pak Tho highway. Roadway embankments were built to failure at two sites along the highway (see Fig. 18) and total stress analyses carried out with the apparent cohesion, c_u, derived from numerous vane tests. Normal circular arc stability analyses gave high safety factors at failure, as for the cases considered by Eide (see Table 2). This method of analysis does not, however, take the inter-slice forces into account, which has the same effect as neglecting the horizontal shear stress between the embankment and the natural soil which acts outwards from the centre to the toes of the embankment. PERLOFF et al, (1967) have determined this spreading effect under elastic conditions, and the inclusion of these horizontal shear stresses lowers the computed safety factors at failure, as shown in Table 2. Settlement of the embankment causes arching of the embankment towards the toes and even higher horizontal shear stresses occur. The full arching condition has been analysed by DAVIS and TAYLOR (1962) and the use of these shear stresses has given the best estimate of the actual failure conditions on the Thonburi-Pak Tho highway.

Most embankment failures take place down the centre of the roadway because the total horizontal force between embankment and natural soil which is exerted on any failure surface is a maximum if this surface passes through the centre-line of the roadway embankment. Figure 19 shows this for an embankment under construction in the Chao Phraya delta.

Fig. 18. Failure of roadway embankment in Chao Phraya delta, Thailand, when embankment height reached 2.2 metres.

EFFECTIVE STRESS SHEAR STRENGTH PARAMETERS

The effective stress shear strength parameters, c' and ϕ', can be determined from drained triaxial tests or from consolidated undrained triaxial tests with pore pressure measurement.

The value of the apparent cohesion intercept, c', with respect to effective stresses is generally zero when the effective stress exceeds the critical pressure. If the effective stress does not exceed the critical pressure, which is often the case in the weathered crust, the clay will behave as if it is overconsolidated and a cohesion intercept will be measured. For this reason, MOH et al (1969) found c' values of 0.5 to 1.8 ton /sq.m in the weathered zone at Rangsit, Thailand.

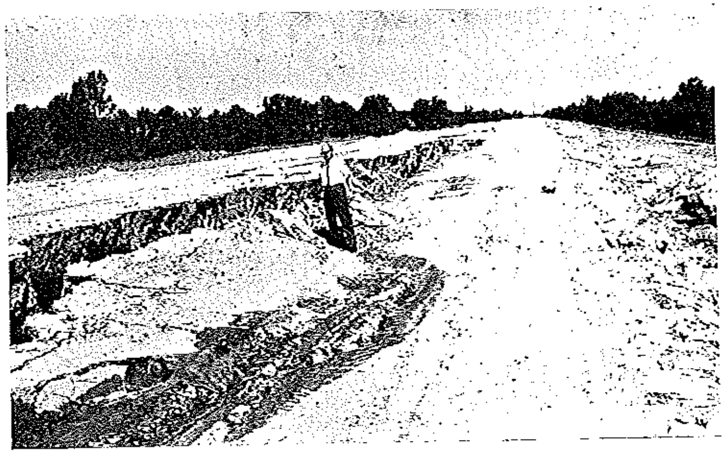


Fig. 19. Failure of roadway embankment near Bang Pakong River, Thailand, when embankment height reached 1.6 metres.

Values of ϕ', the angle of shearing resistance with respect to effective stresses above the critical pressure, are given in Fig. 20 for several marine clays in South East Asia. It is seen that most values, including those given by HOLT (1962) for Hong Kong marine clays, lie within the empirical correlations with plasticity index given by GIBSON (1953) and by BJERRUM and SIMONS (1960). The values of ϕ' generally lie between 20° and 25° and show a slight reduction with increasing plasticity index.

A value of ϕ'_e, the Hvorslev 'true angle of internal friction', has been calculated as 17° by the Author from the experimental data of WANG (1967) for Bangkok clay. The corresponding value of X in the expression $c_e' = X.\sigma_e'$ is 0.11; in this expression c_e' is the Hvorslev 'true cohesion' and σ'_e is the consolidating pressure.

The variation of the pore pressure parameter at failure, A_f', with the overconsolidation ratio is given in Fig. 21 for the Bombay marine clay (KULKARNI, 1967), for the Bangkok clay (WANG, 1967) and for the Rangsit clay (MOH et at, 1969). The values are seen to be very close to each other and to lie towards the upper limit of the envelope derived from triaxial tests on Oslo, London and Weald clays (SIMONS, 1960).

Actual *in situ* values of A_f calculated from the pore pressure response beneath a trial roadway embankment in the Chao Phraya delta (COX, 1968c) are shown in Fig. 22, together with the laboratory results of WANG (1967), for a similar marine clay at Bangkok, and SIMONS (1960). The *in situ* results are slightly lower than the laboratory results. It was found that there was very little difference between A_f values determined from beneath the centre-line and the side-lines of the embankment even though on the side-lines of the embankment there was some reorientation of principal stress directions from their original vertical position. The reorientation of stresses was not, however, very large (0° to 20°).

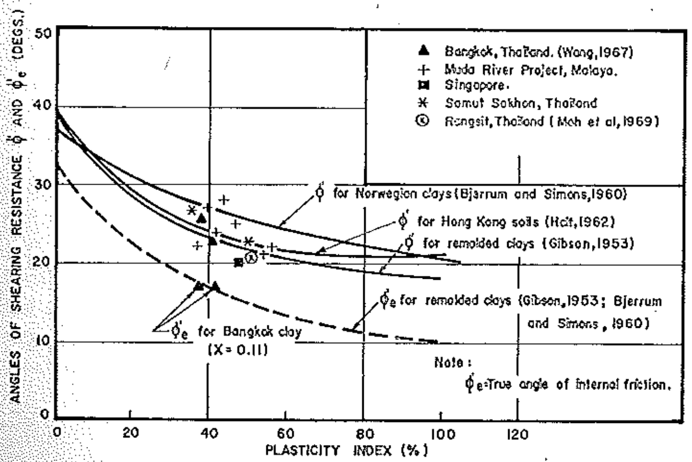


Fig. 20. Effective stress angles of shearing resistance for Recent marine clays in South East Asia.

Pore pressures have been observed to increase beneath roadway embankments under conditions of no load change. The analysis of one of the trial roadway embankments suggested that this increase was caused by the structural breakdown of cohesive bonds between particles when the average effective stress reached the critical stress. A similar breakdown at the critical stress with a consequent reduction in shearing strength has been observed in the laboratory by BJERRUM and WU (1961).

EFFECTIVE STRESS ANALYSIS

Undrained strength analyses may be too conservative if it is known that appreciable consolidation will take place during the construction period. Vane tests were carried out in the weathered crust before construction, and three months after construction, beneath the trial roadway embankment described earlier by the Author (COX, 1968c). The vane shear strength in this crust was found to have increased from its initial value of 1.2 ton/sq.m to an average value of 2.2 ton/sq.m in this time.

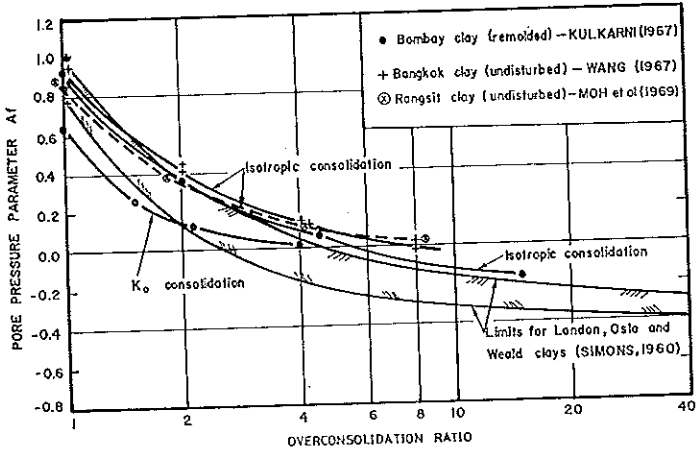


Fig. 21. Variation of Af with overconsolidation ratio for Recent marine clays in South East Asia.

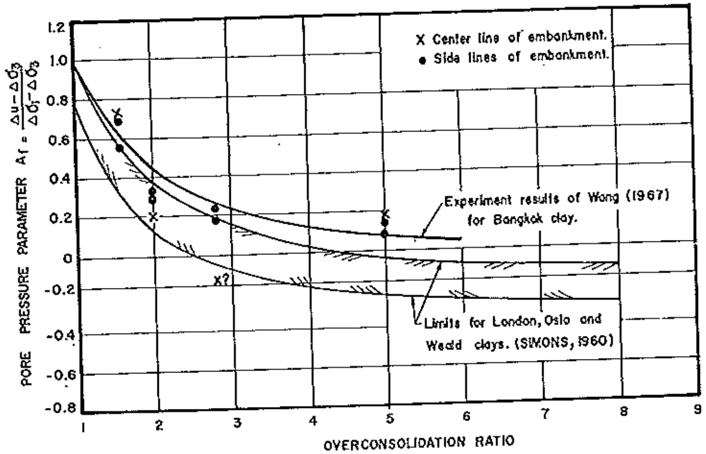


Fig. 22. Variation of Af with overconsolidation ratio as measured *in situ* and in the laboratory for marine clays in Thailand.

Control during construction of an embankment could be established by a series of vane tests before each loading stage, but it is generally easier to install piezometers and to use the measured pore pressures as an indication of whether failure is imminent. Effective stress analyses during the construction period could be carried out, but it is generally easier to determine a pore pressure profile which would exist at failure across the embankment, and to ensure that this profile is not exceeded during construction. Pore pressure profiles at failure for the two trial embankments described above are shown in Fig. 23.

Lateral strain measurements at the toe of an embankment are a good practical indicator of approaching failure conditions in the underlying subsoil. These strains generally increase immediately after loading and decrease, due to consolidation, between loading stages. When failure is imminent, however, it has been found that the lateral movements at the toes of the embankment increase between loading stages. This lateral movement occurred at the rate of over 0.8 cm/day as the Author's trial embankments approached failure.

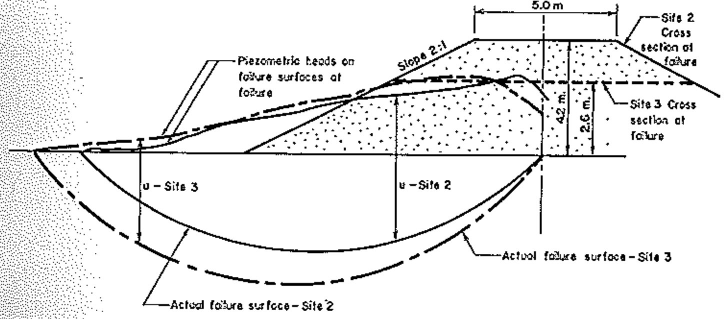


Fig. 23. Actual pore pressure profiles at failure for roadway embankments along the Thonburi-Pak Tho Highway, Thailand.

CONCLUSIONS

1. The Recent marine silty clays beneath the deltaic plains of South East Asia are slightly overconsolidated even though they have not been subject to greater overburden pressures than exist at present. This overconsolidation is thought to be due to the pronounced secondary consolidation of the sediments after deposition, since they have high plasticities and appreciable organic contents.

2. The undrained shear strength increase parameter, c_u/σ'_{vo}, is generally much higher for marine clays in the South East Asian area than for the post glacial marine clays in temperate climates. The undrained strength increases with the time from deposition because of the secondary consolidation effect, and the shearing strength and the shear strength increase parameter are both, therefore, found to increase with distance from the coastline.

3. The weathered zone extends to depths of 6 to 8 m and is thought to be due, as in the Norwegian marine clays, to ion exchange reactions and weathering of the clay minerals by the leaching water. The depth and strength of the weathered crust increases with the distance from the coastline, since the extent of the leaching process increases with the time from deposition. A classification for crust types is proposed.

4. Several failures of roadway embankments have occurred when the *in situ* undrained strength at failure was only 60% to 70% of the shearing strength as determined by the vane test. Analyses of actual trial embankment failures indicate, however, that the inclusion of horizontal arching forces between embankment and natural soil gives a reasonable approximation of the failure conditions.

5. Values of ϕ' generally vary from 20° to 25° and tend to decrease with increasing plasticity index. Values of A_f determined *in situ* beneath a trial roadway embankment were found to approximate to the values determined in the laboratory.

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REFERENCES

BEGEMANN, H.K.S. (1965). The friction Jacket as an Aid in Determining the Soil Profile, *Proc. Sixth Int. Conf. Soil Mech.*, Montreal, Vol. 1, p. 17.

BJERRUM, L. (1954). Geotechnical Properties of Norwegian Marine Clays, *Géotechnique*, Vol. 2, p. 49.

BJERRUM, L. (1967). Engineering Geology of Norwegian Normally Consolidated Marine Clays as Related to the Settlement of Buildings, *Géotechnique*, Vol. 17, p. 83.

BJERRUM, L. and SIMONS, N.E. (1960). Comparison of Shear Strength Characteristics of Normally Consolidated Clays, *A.S.C.E. Research Conf. on Shear Strength of Cohesive Soils*, Colorado, p. 711.

BJERRUM, L., SIMONS, N.E. and TORBLAA, I. (1958). The Effect of Time on the Shear Strength of a Soft Marine Clay, *Brussels Conf. on Earth Pressure Problems*, Vol. 1, p. 148.

BJERRUM, L. and WU, T.H. (1961). Fundamental Shear Strength Properties of the Lilla Edet Clay, *Géotechnique*, Vol. 10, p. 101.

COX, J.B. (1967). A Review of the Vane Test, *Civil Eng. Trans., Inst. of Engs. Aust.*, Vol. CE9, No. 1, p. 19.

COX, J.B. (1968a). The Design and Performance of Roadway Embankments on the Soft Recent Clays of South East Asia, *Fourth Aust. Road Research Board Conf.*, Melbourne. (Preprint).

COX, J.B. (1968b). Soil Investigations, Thonburi - Pak Tho Highway, *Technical Memorandum G4-3, General Engineering Company Ltd.*, Bangkok. (Unpublished).

COX, J.B. (1968c). A Review of the Engineering Characteristics of the Recent Marine Clays in South East Asia, *Research Report No. 6, Asian Inst. Tech.*, Bangkok.

COX, J.B. (1970). The Distribution and Formation of Recent Sediments in South East Asia, *Second South East Asian Conf. on Soil Eng.*, Singapore. (Preprint).

DUDAL, R. and MOORMAN, F.R. (1964). Major Soils of South East Asia, *Journ. Trop. Geography*, University of Malaya, Kuala Lumpur, Vol. 18, p. 51.

EIDE, O. (1967). Geotechnical Problems with Soft Bangkok Clay on the Nakhon Sawan Highway Project, *Conf. Eng. Inst. Thailand*, Bangkok, September, 1967.

GIBSON, R.E. (1953). Experimental Determination of the True Cohesion and True Angle of Internal Friction in Clays, *Proc. Third Int. Conf. on Soil Mech.*, Zurich, Vol. 1, p. 126.

HANSEN, J.B. and GIBSON, R.E. (1949). Undrained Shear Strengths of Anisotropically Consolidated Clay, *Géotechnique*, Vol. 2, p. 9.

HOLT, J.K. (1962). The Soils of Hong Kong Coastal Waters, *Symposium on Hong Kong Soils.*, Inst. of Civil, Mech. and Elec. Engs., Hong Kong, p. 33.

KALLSTENIUS, T. (1963). Studies on Clay Samples Taken with Standard Piston Sampler, *Proc. No. 21 Roy. Swedish Geotech. Inst.*, Stockholm, p. 210.

KULKARNI, K.R. (1967). The Effect of Overconsolidation on the Shear Strength Characteristics of Bombay Marine Clay, *Journ. Indian Nat. Soc. of Soil Mech. and Found. Eng.*, Vol. 6, p. 471.

KULKARNI, K.R., KATTI, R.D., MISRA, A.P. and PATEL, K.S. (1967). A Study of the Properties of Bombay Marine Clay in Relation to Certain Foundation Problems, *Proc. Third Asian Reg. Conf. on Soil Mech.*, Haifa, Vol. 1, p. 210.

LADD, C.C. and LAMBE, T.W. (1963). The Strength of Undisturbed Clay Determined from Undrained Tests, *A.S.T.M. Spec. Tech. Pub. 361*, p. 342.

LADD, C.C. and VARALLYAY, J. (1965). The Influence of Stress System on the Behavior of Saturated Clays During Undrained Shear, *Research Report R65-11*, Dept. of Civ. Eng., *Mass. Inst. Tech.*, Boston, Mass.

LUMB, P. and HOLT, J.K. (1968). The Undrained Shear Strength of a Soft Marine Clay from Hong Kong, *Géotechnique*, Vol. 18, p. 25.

McLELLAND, B. (1967). Progress of Consolidation in Delta Front and Prodelta Clays of the Mississippi River, *Marine Géotechnique*, University of Illinois Press, Chicago.

MOH, Z.C. (1969). Strength and Compressibility Characteristics of Soft Bangkok Clay, *Research Report No. 7, Asian Inst. Tech.*, Bangkok.

MOH, Z.C., NELSON, J.D. and BRAND, E.W. (1969). Strength and Deformation Behaviour of Bangkok Clay, *Proc. Seventh Int. Cont. on Soil Mech.*, Mexico, Vol. 1, p. 287.

MISRA, A.P. (1967). Geotechnical Properties of a Marine Deposit, *Journ. Indian Nat. Soc. of Soil Mech. and Found. Eng.*, Vol. 1, p. 231.

MOUM, J. and ROSENQVIST, I. Th. (1957). On the Weathering of a Young Marine Clay, *Proc. Fourth Int. Conf. on Soil Mech.*, London, Vol. 1, p. 77.

MUKTABHANT, C., TEERAWONG, P. and TENGAMNUAY, V. (1963). Engineering Properties of Bangkok Subsoils, *Report, Chulalongkorn University*, Bangkok.

NEDECO. (1965). Siltation, Bangkok Port Channel, *Report of Netherlands Engineering Consultants (NEDECO) to Government of Thailand*, Vol. II-The Field Studies.

PAIS CUDDOU, I.C.D.M, DESAI, M.D. and KHILNANI, K.S. (1967). Investigations of Marine Clays by Vane Test, *Proc. Third Asian Reg. Conf. on Soil Mech.* Haifa, Vol. 1, p. 231.

PERLOFF, W.H., BALADI, G.Y. and HARR, M.E. (1967). Stress Distribution Within and Under Long Elastic Embankments, *Highway Research Record No. 181*, Highway Research Board.

PRASUTI, S. (1965). Subsurface Soundings of Bangkok Subsoil by Static Cone Penetration, *M.Eng. Thesis No. 111, SEATO Graduate School of Engineering*, Bangkok.

RAYMOND, G.P. (1967). The Bearing Capacity of Large Footings and Embankments on Clay, *Géotechnique*, Vol. 17, p. 1.

SERAJUDDIN, M. (1967). Studies on Preconsolidation Characteristics of Some Foundation Clays in East Pakistan, *Pakistan Engineer*, Vol. 7, p. 215.

SIMONS, N.E. (1960). The Effect of Overconsolidation on the Shear Strength Characteristics of an Undisturbed Oslo Clay, *A.S.C.E. Research Conf. on Shear Strength of Cohesive Soils*, Colorado, p. 747.

SKEMPTON, A.W. (1948). The $\phi = 0$ Analysis and its Theoretical Basis, *Proc. Second Int. Conf. on Soil Mech.* Rotterdam, Vol. 1, p. 72.

SKEMPTON, A.W. (1957). *Discussion on* The Planning and Design of the New Hong Kong Airport, by H. Grace and J.K.M. Henry, *Proc. Inst. Civ. Eng.*, Vol. 7, p. 305.

WANG, W.S. (1967). Shear Strength Characteristics of Bangkok Clay, *M. Eng. Thesis No. 180, SEATO Graduate School of Engineering*, Bangkok.

WESLEY, L. (1967). The Dutch Penetrometer and its Use in Indonesia, *Proc. First South East Asian Conf. on Soil Eng.*, Bangkok, p. 223.

WISEMAN, G. and SCHIFFMAN, L. (1967). Some *In Situ* Tests on Bangkok Clay, *Proc. First South East Asian Conf. on Soil Eng.*, Bangkok, p. 231.

ZONNEVELD, I.S. (1963). Soil Formation in Deltaic Areas, *Water Resources Series No. 25, Proc. Reg. Symp. on Flood Control, Reclamation, Utilization and Development of Deltaic Areas*, ECAFE, Bangkok, p. 100.