Impact of geological conditions on ground improvement projects

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Abstract: Ground improvement projects are often carried out to replace or modify the natural geological formations of the ground and the success of a ground improvement project is greatly affected by the geological conditions of the natural ground or the soils to be improved. Therefore, a good understanding of the geological conditions and the geotechnical properties of the soils play a pivotal role in the design and implementation of ground improvement projects. In this paper, two case studies will be used to illustrate the effect of geological conditions on the selection of ground improvement methods, ground improvement procedures, and the design and implementation of ground improvement works. The first case is a large-scale land reclamation project at Changi East in Singapore, where consolidation of seabed marine clay and ultra soft slurry using surcharge and vertical drains was undertaken. The second case is ground improvement for the runway of an airport in China, where a layer of silty clay was improved by a combination of vertical drain and dynamic compaction methods. The unique geological features of these two projects and their influence on the ground improvement works are discussed. The ground improvement methods adopted are also presented.

Résumé: Des projets d'amélioration des sols sont souvent réalisés pour remplacer ou modifier des formations géologiques naturelles du massif de sol. Le succès d'un projet d'amélioration des sols est largement influencé par les conditions géologiques du massif naturel ou des sols à améliorer. Donc, une bonne compréhension des conditions géologiques et des propriétés géotechniques des sols joue un rôle déterminant dans la conception et réalisation des projets d'améliorations des sols. Dans cette communication, deux études de cas seront utilisées pour illustrer l'effet des conditions géologiques sur la sélection des méthodes d'amélioration des sols, sur la procédure d'amélioration des sols, sur la conception et la réalisation des travaux d'amélioration des sols. Le premier cas est un projet de récupération de terrain de grande échelle à Singapore, où l'amélioration de l'argile du fond marin et d'une boue ultra molle en utilisant des drains verticaux et le préchargement a été réalisée. Le second cas est l'amélioration des sols pour les pistes d'un aéroport en Chine, où une couche d'argile limoneuse a été améliorée en utilisant une méthode combinant les drains verticaux et la compaction dynamique. Les aspects géologiques uniques de ces deux projets et leur influence sur les travaux d'amélioration des sols sont discutés. Des méthodes d'amélioration des sols adoptées sont aussi élaborées.

Keywords: Compressibility, consolidation, geomaterials, shear strength, settlement, stability.

INTRODUCTION

Ground improvement often involves replacement or modification of natural geological formations. Therefore, careful consideration of geology and geological process is a key to the success of ground improvement projects. Consider land reclamation work as an example, it is a process of placing fill geomaterials on existing geological formations over a large area extent. The geological conditions will significantly affect the planning, design and implementation of a land reclamation and ground improvement project. The interplays between geology and the geotechnical work involved in ground improvement will be illustrated by two case studies. The first is a large-scale land reclamation project in Singapore and the second is ground improvement project for the runway construction of an airport in China.

IMPACT OF GEOLOGY

Geological age

Whenever foundation options are considered, the geological age of the formation plays a major role. The older geological formations are always found as competent foundation strata, providing stability with little deformation. Most foundation problems are generally encountered in Quaternary age young geological deposits. Unfortunately several major developments including land reclamations are frequently required on such quaternary deposits. However the proposed land reclamation areas are generally associated with a nearby quaternary deposit which could be used as a borrow source. Therefore understanding Quaternary geology is important for the engineering geologists and engineers who involve in the land reclamation projects.

Geological process and geomorphology

Geological processes such as deposition, erosion etc: control landforms of present day landscapes. Marine deposits, coastal and estuarine deposits have significant impact on land reclamation and ground improvement. These types of deposits are either loosely deposited or have a high compressibility depending upon the type of soil. Loosely deposited granular soil causes immediate settlement upon additional static load and liquefaction upon dynamic load; highly compressible cohesive soil causes significant primary and secondary consolidation upon application of static load. Estuarine deposits, old river channel, valley fill deposits, sand bars and levees generally have a complex formation and generally contribute significant differential settlement.

Geological history

Several geological processes such as deposition, erosion, extrusion, intrusion, metamorphism, tectonic movement, desiccation, sea level fall and rise etc: have been happening throughout the life of the earth. The history of these are important to the ground engineer as they determine the characteristic of earth crust; stability and deformation behaviour of these geological formations are much dependent upon these geological processes. For example, erosion is one of the major factors for overconsolidation; sea level rise and fall also cause overconsolidation but in addition they can create a desiccated crust, which has significant effect on the deformation of quaternary deposits.

Stress history

Stress history is one of the most important controlling factors of the strength and deformation behaviour of quaternary deposits. The magnitude of settlement is directly related to the stress ratio and stress history, which are generally given by the overconsolidation ratio. Therefore these are important parameters to predict the magnitude of settlement. In nature soil can be in a state of normally consolidated, overconsolidated and under consolidated conditions depending upon geological processes undergone and the age of the formation. In nature normally consolidated soils are rare since most behave as lightly overconsolidated due to aging even if there was no previous vertical stress greater than current stress. Significant overconsolidation can be found in a formation where erosion has removed the previous overburden stress. However this type of geological process creates a soil with varying degrees of overconsolidation as the removed stress is constant and the original effective vertical stress of the soil varies with depth. Even in a nature, under consolidated soil can be found in a recently deposited formation in which soil is still undergoing self-weight consolidation. This type of soil will contribute excessive deformation. This type of underconsolidated soil can also be found in man-made deposits such as waste pond, mine tailing etc. Desiccation is one of the factors causing overconsolidation. The rise and fall of sea levels are major cause of desiccation.

Organic content

In general fine grained soils deposited at a delta mudflat where mangrove are abundant or fine soils deposited in an old river channel where eroded soils carried growing plants contain significant amount of organic matter. Due to the fabric nature of organic matter the soils with high organic content have high void ratios and high moisture contents. These types of soil contribute significant amounts of primary and secondary consolidation settlements.

Soil anisotropy and micro fabric

In nature geological formations are rarely isotropic due to the process of deposition involving horizontal lamination and layering. This creates soils that are anisotropic in strength, compressibility and hydraulic conductivity. This means the strength, compressibility and hydraulic conductivity are different between the vertical direction and horizontal direction. In general greater strength with lower compressibility can be found in a horizontal direction whereas lower hydraulic conductivity can be found in the vertical direction. Micro fabric of soils was extensively described by Rowe (1972). Generally even in young deposits of nearly homogeneous soils there is a degree of anisotropy of 1.5 to 2 depending upon the process of deposition and the age of the formation. However Tanvenas et.al (1983) stated that permeability anisotropy of a homogeneous deposit is not significant.

Macro fabrics are known to exist in young alluvial and fluvial deposits due to inclusion of the small lenses of fine granular material in the fine-grained soil deposit. These inclusions of micro fabric affect the deformation and consolidation process of fined grained soil. This will be discussed in details in the later section.

GEOLOGICAL FEATURES AFFECTING MAGNITUDE AND TIME RATE OF SETTLEMENT

Generally land reclamations are carried out on geological formation using engineered fill. The magnitude of load imposed on the geological formation can be anything from 100 kPa to as high as 600 to 700 kPa depending upon the depth of the seabed and required final finished level or otherwise surcharge level. This magnitude of load can cause the underlying geological formation to settle significantly depending upon the type of soil, and its age, compressibility and stress history. The settlement will be either immediate or long-term depending upon the type of soil and thickness of the soil. This will be discussed in the later section.

Impact of geological features on the magnitude of settlement

Total settlement involves immediate settlement caused by elastic behaviour of soil, primary consolidation of low permeable soil and secondary consolidation of high organic content soil. The former type of settlement will occur in all types of soil and is unavoidable. However although magnitudes of such settlement depend upon the elasticity of the soil, they are usually small in magnitude. In addition, they have little or no technical impact as these settlements occur during construction and are not noticeable. However there could be a financial impact to the developer due to the volume losses. This has to be taken into consideration in the project costing by predicting the immediate settlement accurately.

The later two types of settlements only occur in low permeable soils and secondary consolidation is more significant in soils which contain significant amount of organic matter. In general suitable sites for land reclamation are selected where the area is underlain by a competent geological formation. These underlying formations are typically soils with low compressibility.

However these types of competent formations are rarely found near the urbanized areas where land reclamation is generally required. If the competent layer is overlain by a thin layer of compressible soft soil, it is common practice to remove this before the land is reclaimed in order to eliminate the excessive primary and secondary consolidation settlement. However when thick compressible layers are encountered it may not always possible to remove them due to the high cost of dredging, disposal and replacement. In such situations, ground improvement techniques are preferred options. This will be discussed in details in the later section.

Magnitudes of settlement are significant in the lightly or normally consolidated soils. Heavily overconsolidated soils contribute little settlement depending upon the magnitude of additional load. In the latter cases, the additional loads are generally much less than their preconsolidation pressures and hence settlements are only in the recompression range. Settlement in this range is much smaller than that occurs in the virgin range. In addition the consolidation rates are much faster in the recompression range. Therefore most settlement will occur during construction. The details on predicting of magnitude and time rate of settlement and method of reclamation design can be found in Bo & Choa (2004).

Secondary compression

Secondary compression generally occurs due to the rearrangement of soil particles rather than dissipation of pore pressure. Therefore secondary compression occurs without gain in effective stress and most of it occurs after the primary consolidation process. Although the magnitude of secondary compression is dependent upon organic content, it is also dependent upon the stress level. The secondary compression indices are known to decrease with increasing stress level. Therefore the magnitude of secondary compression can be minimised by prestressing the soil to the greater level (Bo & Choa 2004). This will be discussed in the later section.

Impact on time rate of consolidation

Time rate of consolidation is dependent upon the coefficient of consolidation, which in turn dependent upon the permeability, and compressibility of soil and thickness of the formation. The soils contain higher percentage of clay usually have low permeability. Lightly or normally consolidated soils have high compressibility. Therefore such soils have low coefficient of consolidation and when deposited in a thick layer, time to complete the primary consolidation can take few decades to century due to their low coefficient of consolidation and the longer drainage path. Therefore ground improvement process is required in order to eliminate the future settlement and in order to accelerate the consolidation process. This will be discussed in the later section. As coefficient of consolidation is increasing with decreasing compressibility, overconsolidated soils generally have high coefficient. Therefore consolidation process is bound to be faster and in many cases, it may not be necessary to improve the soils.

GEOLOGICAL FEATURES AFFECTING STABILITY OF EARTH AND RETAINING STRUCTURES

Major stability problems on engineered earth and retaining structures in land reclamation projects are undrained. It is a well-known characteristic of soils that the undrained shear strength is dependent upon the stress levels and their stress history. Normalized undrained shear strengths with respect to the effective vertical stress for normally consolidated or lightly consolidated soils are generally lower than 0.25 although slight variations can be found depending upon their plasticity. The value increases with increasing overconsolidation ratio. Therefore undrained shear strengths of normally or lightly consolidated soils are extremely low and construction of earth slope and retaining structures on such soils can have significant instability problems. The construction of sheet pile walls requires excessive penetration length due to the greater earth pressure on the wall. Therefore removal of soft soils to a suitable level and replacement with engineered fill or carry out appropriate ground improvement in order to improve the strength of the soft soils is required to be able to achieve stable structures.

GROUND IMPROVEMENT

As explained earlier, ground improvement is generally required in land reclamation projects carried out on highly compressible soils with low strength. Therefore improvement is required to improve the strength and to reduce or eliminate the settlement.

Improving strength to achieve stability

Stability problems in land reclamation projects are generally short-term and mainly related to the undrained characteristic of low permeable soils. Improving these types of soils can be achieved by either preloading or soil mixing methods. However preloading options are not favourable as the stability problems are at the edge of the reclaimed land. Soil mixing is usually an expensive option. Alternative options are provision of load transfer piles or stone, sand or cement columns. These options are much more attractive. The most simple and attractive option are excavation of low strength soils and replace them with well-compacted highly permeable soils. This type of granular key will not only provide the stability but also will eliminate large settlements.

As explained in the earlier section, the undrained shear strengths of clay are increasing with increasing overburden stresses, therefore the strength of such soil increases after consolidation under additional stress. Therefore the construction of an embankment or slope by stage construction allows the strength to improve due to consolidation. The selection of suitable stress can be made based on critical height (Bo & Choa 2004).

Ground improvement to eliminate future settlement

Eliminating future settlement is possible by preloading with the additional load equivalent to or higher than the future load to force the settlement of compressible soils to complete during construction. However in many cases preloading alone is not practical to complete the degree of consolidation required especially in thick clay deposits.

Consolidation times can be significantly reduced by installing a vertical drainage system which provides much shortage drainage paths and has the advantage of allowing pore water to flow through horizontal laminations which usually have a higher coefficient of consolidation than the vertical flow path (Bo et.al 2003, Bo & Choa 2004). The spacing of the vertical drains is based on the coefficient of consolidation due to horizontal flow and the duration of preloading allowed. With this technique it is possible to accelerate the consolidation process so that it is completed during the construction period. Although acceleration of the consolidation process does not reduce nor accelerate secondary compression, reduction of secondary compression can be achieved by preloading the deposit with much higher stress than the future stress as the coefficient of secondary compression reduces with increasing stress level (Bo & Choa 2004).

CASE STUDY 1: CHANGI EAST RECLAMATION AND GROUND IMPROVEMENT PROJECTS

Reclamation at Changi East was carried out to extend the land at the foreshore of the eastern part of Singapore. The area reclaimed is about 2000 hectares and it will be used for the future airport runway, taxiways and the terminal buildings. The depth of seabed at the reclamation area ranges between 2 metres and 15 metres being much deeper at the northern edge of the area. The lay out of the reclamation area is shown in Figure 1.

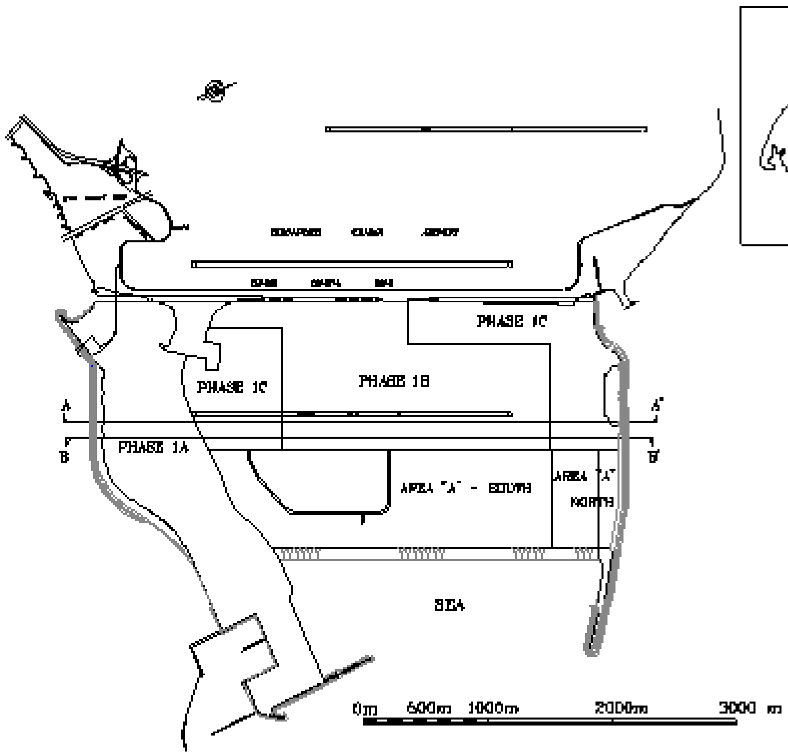

Figure 1. The layout of the reclamation area

Geology and ground profile

Generally the study area is underlain by Singapore marine clay in the two deep valley cuts of Old Alluvium. Figure 2 shows the typical soil profile from the north to the south. The Old Alluvium comprising of cemented silty clayey sand is exposed in the seabed at the central area close to the southern side of Singapore Island where some sand quarries can be found. Two eroded limbs were formed at the northern and the southern sides of the study area due to the erosion of the Old Alluvium. The northern valley cut is deeper than the southern one and its eroded surface of Old alluvium is sloping towards the northern side. The thickness of marine clay found in this valley ranges from 5 to 55m. Two marine members of the Kallang Formation, locally known as Upper Marine Clay and Lower Marine Clay, are found to be separated by a layer of either stiff silty clay or medium dense silty sand of 2 to 5m in thickness. This desiccated crust layer of silty clay, locally known as the 'Intermediate layer', was formed when the sea level dropped by 20 to 25 meter during two regressions between 10,000 and 20,000 years ago (Pitts, 1983). The desiccated silty clay and silty sand layers are found at levels varying from -10mCD to -28mCD. In some areas alluvial sediment of silty sand layer was deposited on top of the lower marine clay where the lower marine clay was exposed at the river mouth in the past.

In some part of northern most area of the study, the marine clay is directly underlain by granite rock at about 60m depth from the seabed. The granite belongs to a member of the intrusive Pulau Ubin Granite. The southern valley cut, which is shallower than the northern one, is also filled up with marine clay of 4m to 20m in thickness. In the southern part of the area, unlike the northern valley, no intermediate layer is found between the Upper and Lower Marine Clay. The marine clay found in this southern valley is more overconsolidated than those from the northern valley probably due to the removal of the top part of Upper Marine Clay and sand deposit in the past. Therefore the undrained shear strength of the marine clay in the southern area is much higher than that in the northern area. Some parts of the Old Alluvium in the study area are found to be overlain by soft slurry or clay and sand mixtures that are products of man-made activities.

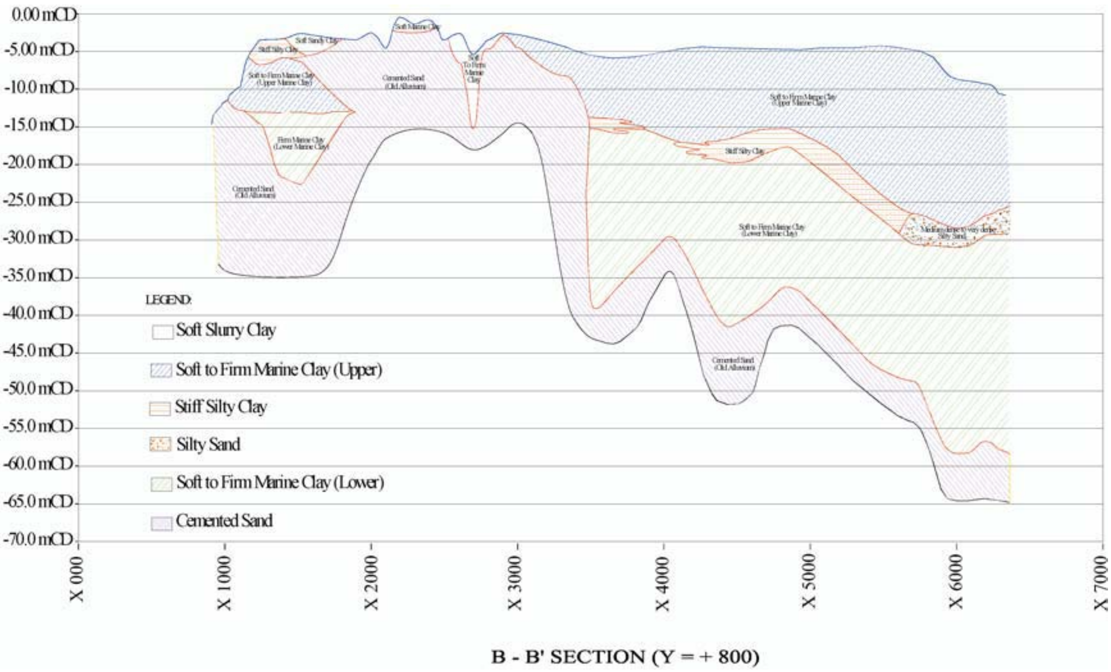

Figure 2. Typical profile along north-south line. The lowermost line drawn on the profile indicates the maximum depth of the boreholes.

Ground improvement consideration

The general profile of the area allows the designer to zone the area required for ground improvement and those not required. In general ground improvement is not required where the seabed is underlain directly by the old alluvial deposits. As the southern part of the area is underlain by the Lower Marine Clay, which has a greater degree of consolidation, most of this area does not require ground improvement since the additional load is lower than the yield stresses of the underlying soils. Therefore the yield stresses of the formations were carefully determined from the end of primary consolidation tests. Due to the high yield stresses and the higher overconsolidation ratio of the Lower Marine Clay in the southern part, ground improvement was not carried out at the southern part of the area. A comparison of overconsolidation ratio of northern and southern areas is shown in Figure 3. Details of the stress history of Singapore Marine Clay can be found in Bo et.al (2002).

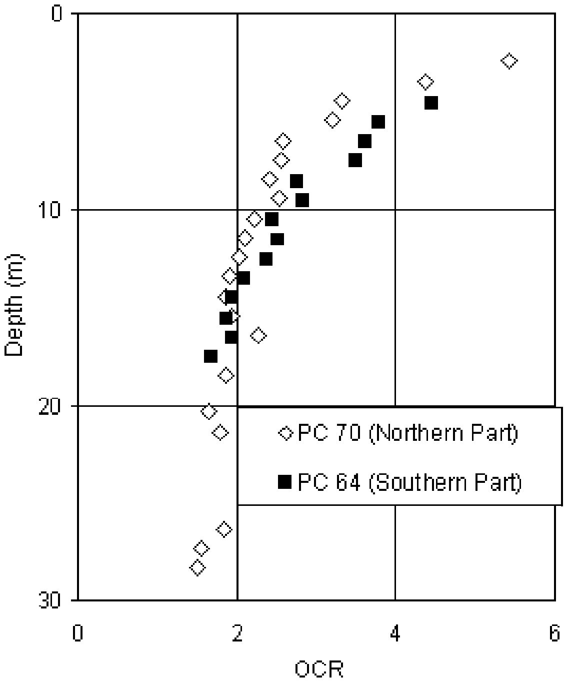

Figure 3. Comparison of OCR between northern and southern part

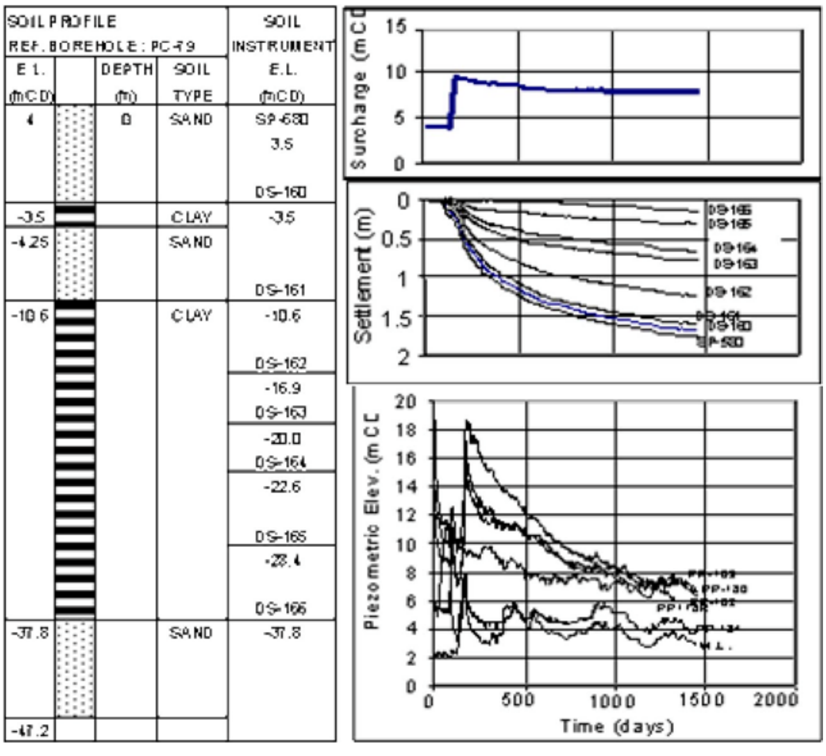

Figure 4. Settlement and pore pressure monitoring data during ground improvement

The spacing of the vertical drains was based on the coefficient of consolidation, thickness of the compressible layer, drainage conditions and the time allowed for the preloading period. However, only a few options of spacing between 1.1 and 1.8 m in square grid were applied in the project to achieve the required degree of consolidation by allowing various preloading periods. The height of the preload fill levels was based on the required future loads, possible future groundwater level changes, predicted settlements and allowable magnitudes of secondary compression. Where the location had a thicker compressible layer with greater predicted settlement, comparatively higher preload levels than others was placed. Appropriate extra fill was also placed at the location where the magnitude of predicted secondary compression was significant. This helps minimize the magnitude of secondary compression (Bo & Choa 2004).

Figure 4 shows some monitoring data from the northern part of the area. It can be seen in the figure that the settlements were completed within a short period and pore pressures were dissipated within the construction period due to the accelerated consolidation process provided by the vertical drainage system.

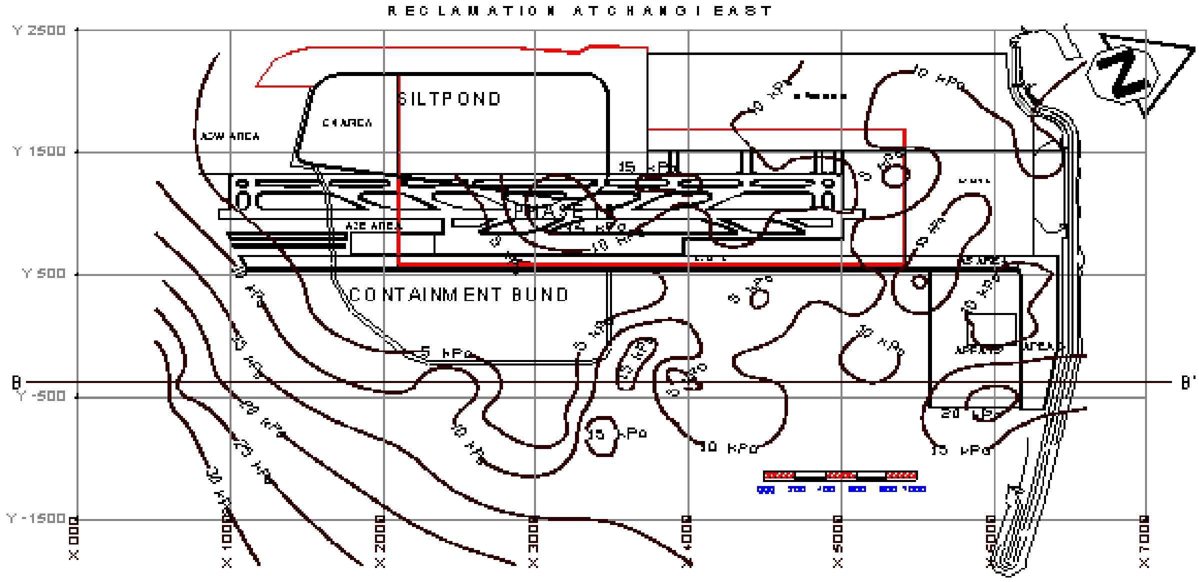

Figure 5 Variation of undrained shear strength at the seabed

Stability consideration

In general, the field vane shear strength of the marine clay ranges from 5 to 20 kPa at the seabed in the study area (Fig. 5). Comparatively high seabed shear strength values of 15 to 20 kPa are found at the northern most part of the area. However, in some parts of this area the seabed shear strengths were 8 to 12 kPa where the original seabed was overlain by dumped clay. The field vane shear strength ranges from 10 to 20 kPa at the seabed of central area. The seabed level of the central area varies from -3mCD to -5mCD. In general the field vane shear strength of marine clay at the seabed of the area in the northern valley cut gradually increased from the central area towards the northern most area (Fig. 5). Likewise, field vane shear strengths at southern area also increased towards the southern part. However the difference in the higher undrained shear strength found at the southern part is at the shallow seabed whereas that found in the northern part is at the deep seabed. In addition the c_u/σ' ratio of the southern Marine Clay is greater than that of the northern part. Therefore slopes at the southern part with less than 10 m of fill with more than 80% of the fill submerged under the water was well below the critical height. As such the provision of a sand key was unnecessary at the southern part.

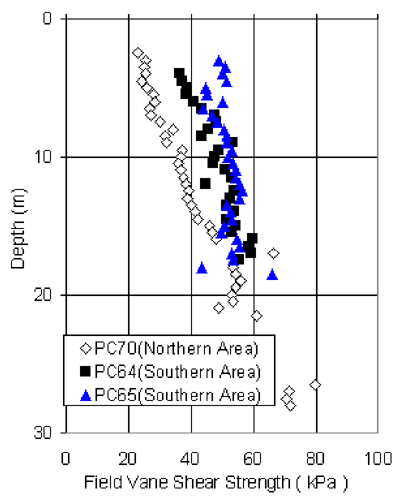

Figure 6. Comparison of undrained shear strength between the southern and the northern parts

In the northern part, undrained shear strength found at the deep seabed was lower than that found in the southern part and the c_u/σ' ratio was also lower than that of the southern part. Due to the excessive thickness of fill required, which was significantly higher than the critical height, the removal of soft soils and provision of a sandkey became necessary. The determination of excavation levels for the sandkey was based on the c_u/σ' ratio and the estimated future load. Figure 6 shows the comparison of undrained shear strength at the northern and the southern parts. A detailed description of the undrained shear strength of Singapore Marine clay can be found in Bo et.al (1999). Figure 7 shows typical profile of shore protection rock bund.

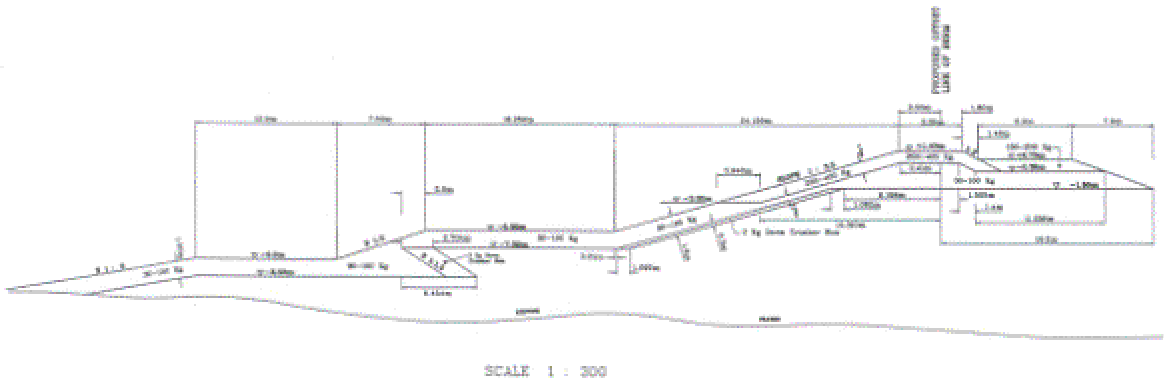

Figure 7. Typical Profile of Shore Protection Rock Bund

Stability of Retaining wall

The best location for the berthing jetty was in the southern part where the location was underlain by a more competent formation. However due to the necessity for a vertical wall with retaining height of average 10 m, excessive penetration of sheet piles was required and the provision of raker piles was also necessary as the cantilever wall approach was not feasible. Raker piles were generally anchored into the old alluvial formation with 70-degree inclination. Figure 8 shows typical design of sheet pile wall with raker piles.

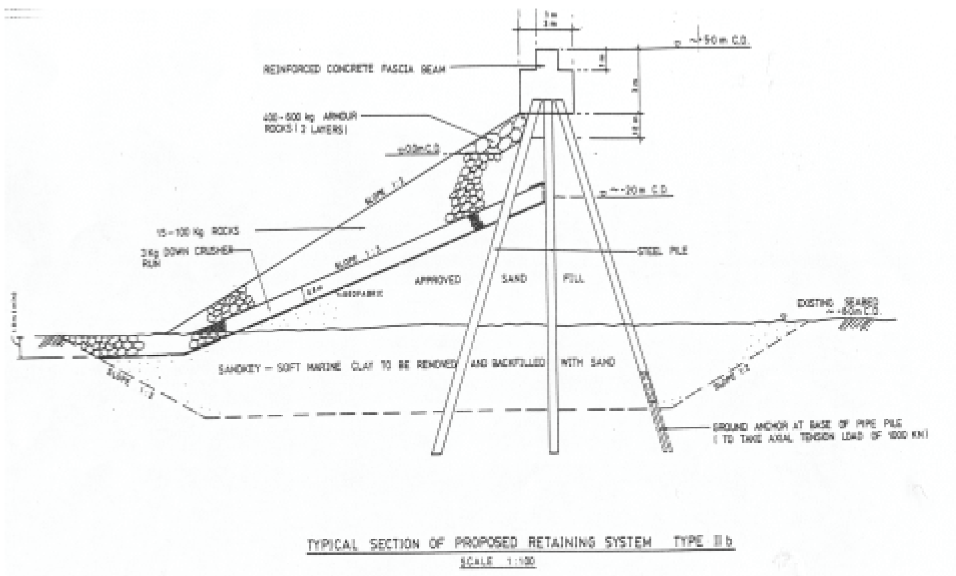

Figure 8. Typical profile of the sheet pile wall with raker piles

CASE STUDY 2: SOIL INMPROVEMENT FOR A RUNWAY IN CHINA

The construction of a new airport near Shanghai, China, required a soft ground site to be improved. The typical soil profile along the cross-section of the runway is shown in Fig. 9. The soil strata varied considerably across the runway and the thickness of the soft clay ranged from 2 to 7 m. The soft clay was recently deposited under a marine environment. The soil profile was divided into four layers. The first layer was a stiff crust of 0.8 to 1.6 m thick, and consisted of mainly silty clay. The second layer, ranging from 0.3 to 4.4 m thick, was mainly very soft, silty clay with a water content greater than the liquid limit. Both the first and second layers contained some roots and rotten vegetation. The third layer was a 0.4 to 5.1 m thick silty clay layer interbedded with organics. The average water content of this layer was as high as the liquid limit and the average undrained shear strength was about 20 kPa. The second and third layers needed to be treated. The fourth layer ranged from 1.5 to 13 m thick. It was mainly stiff sandy clay with the undrained shear strength varying from 50 to 80 kPa. The basic soil properties of each layer are given in

Table 1. The soils were normally consolidated except for the overconsolidated top thin crust. The water table was at 0.8 m below the ground surface.

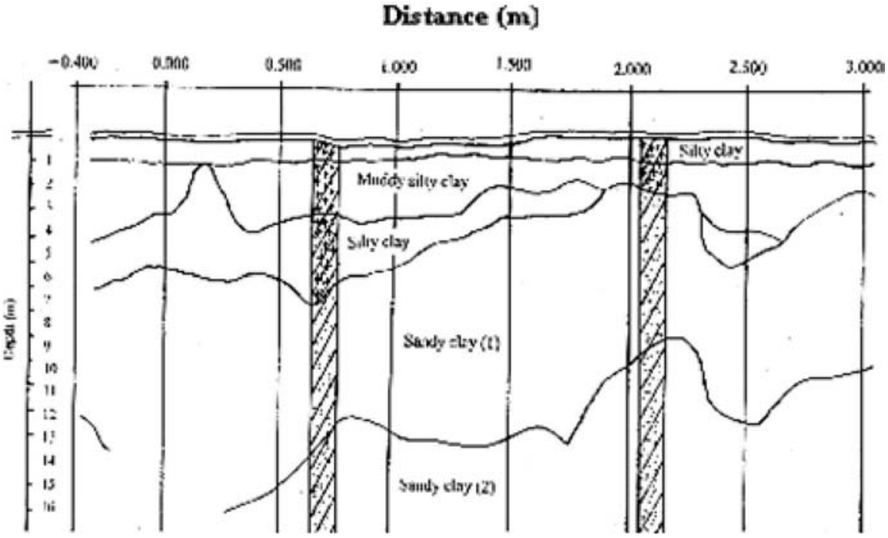

Figure 9 The soil profile along the runway

As the construction schedule was very tight, some conventional soil improvement methods such as surcharge preloading would be too slow. Deep cement mixing or stone column methods were considered not economical. The possibility of using dynamic compaction as the method of improvement was considered. It is generally believed that the dynamic compaction (DC) method is not suitable for fine-grained soils, particularly for soils with a plasticity index larger than 10 (Mitchell, 1981). Considering the dynamic compaction method for this project was based primarily the geological conditions of this site. There were two factors that favour the use of dynamic compaction despite the fact that the soil is fine-grained. 1). As shown in Fig. 9, although the thickness of the soft clay layer varied erratically, it was generally within 6 m which was within the influence zone of dynamic compaction. 2). As can be seen from Table 1, the plasticity index of the soil was low, between 13 to 14%, which is only marginally higher than the normal limit of 10%.

Table 1. Typical soil properties of each layer

Layer	***Soil Type***	Water Content (%)	Bulk Density (kN/m^3)	Initial Void Ratio	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	c_u by UU Test$^+$ (kPa)	Compression Index*
1	Silty clay	33.6	18.67	0.947	34.3	20.5	13.8	17.0	0.36
2	Muddy silty clay	37.6	18.25	1.047	32.8	19.8	13.0	20.8	0.43
3	Silty clay	35.1	18.47	0.979	35.1	21.6	13.5	21.9	0.47
4(1)	Sandy clay	33.4	18.71	0.916	38.3	25.3	13.0	-	0.26
4(2)	Sandy clay	29.8	19.04	0.843	-	-	-	-	0.22

+ Undrained shear strength c_u measured by unconsolidated undrained (UU) tests.
* The compression index is defined as the coefficient of compressibility measured for a load interval between 100 kPa to 200 kPa according to the Chinese Foundation Design Code, GBJ7-89.

Some pilot tests were carried out using different levels of compaction energy. Three levels of compaction energy: 520 kNm/m^2 (low), 810 kNm/m^2 (medium), and 1750 kNm/m^2 (high) were used in these tests. The details of the pilot testing results are reported in Zheng et al. (2004). The compaction was carried out using a flat cone shape steel hammer with the top and bottom diameters of 2.0 and 2.5 m respectively. The weight of the hammer was 120 kN. To facilitate the dissipation of pore pressure, vertical drains were installed. A sand blanket was also placed on the ground surface.

The field experiments showed that it was necessary to use vertical drains and a sand blanket to facilitate the dissipation of pore water pressure generated during compaction. Among the three levels of compaction energy used, the high level energy compaction method did not work at all. This was because the energy level was too high so that the soil structures were destroyed. This was indicated by the observations that the excess pore water pressures were much higher and the dissipation rate was much slower compared with the tests under low and medium energy levels (Zheng et al., 2004). The low and medium level energy compaction resulted in obvious improvement. However, with low level energy compaction, the effective depth of compaction was only 2 to 3 m, which was not deep enough for most projects. With the use of medium energy level (810 kNm/m^2), the effective depth of compaction can be extended to 5 to 6 m (Zheng et al., 2004).

Based on the pilot tests, two major factors that controlled the effectiveness of the DC method were identified: (i) the drainage system and (ii) the method of compaction, which includes the compaction energy and compaction procedure. The compaction energy has to be controlled within a limit so as not to destroy the structure of the soil. Once the structure of the soil is destroyed, a higher excess pore water pressure will build up and the dissipation will also be slower. Based on the pilot testing results, the following guidelines have also been drawn for compaction of soft clay ground (Zheng et al. 2004):

(1) A proper drainage system has to be installed before compaction. The use of PVD with sand blanket appears to be an effective drainage system.
(2) The compaction should begin with low compaction energy for the first pass and then increase the energy gradually for the subsequent passes. The rationale is to consolidate the topsoil to form a "hard crust" first. Once a "hard crust" is formed, larger compactive energy can be applied and soil at a deeper depth can be compacted. This is totally different from the procedure used for compacting granular soil in which higher compaction energy is suggested to be used for the first few blows to extend the compaction as deep as possible (Broms, 1991). A compaction scheme with compaction energy gradually increased from 500 to 800, and then 1600 kNm appears to be suitable for the compaction of soft silty clay.
(3) It is more effective to use more passes, but only 1 ~ 3 numbers of blows per pass for compaction.
(4) A resting time between each pass of compaction is required to allow the pore pressure to dissipate. With a properly installed drainage system, more than 80% of the excess pore water pressure can dissipate within 1 to 2 days for this case. Therefore, a resting time of 4 to 7 days appears to be sufficient.

Adopting the above guidelines, a compaction procedure using medium level energy has been established as follows: (1) Place a sand blanket and install PVDs at close spacing (2 m or less). (2) Compaction shall be carried out in four passes. The first pass uses a compactive energy of 500 kNm for one blow. The points of compaction are in a checked pattern with a spacing of 2 - 3 m between rows and prints. The second pass is compacted with 800 kNm for 3 blows. The compaction points are still in a checked pattern with a spacing 3 - 5 m between rows and prints. The third and forth passes are compacted with 1600 kNm for 3 blows at a spacing of 2 - 4 m between prints and rows. The second, third or forth pass of compaction should be carried out only after 80% of the excess pore water pressure generated from the previous compaction has dissipated.

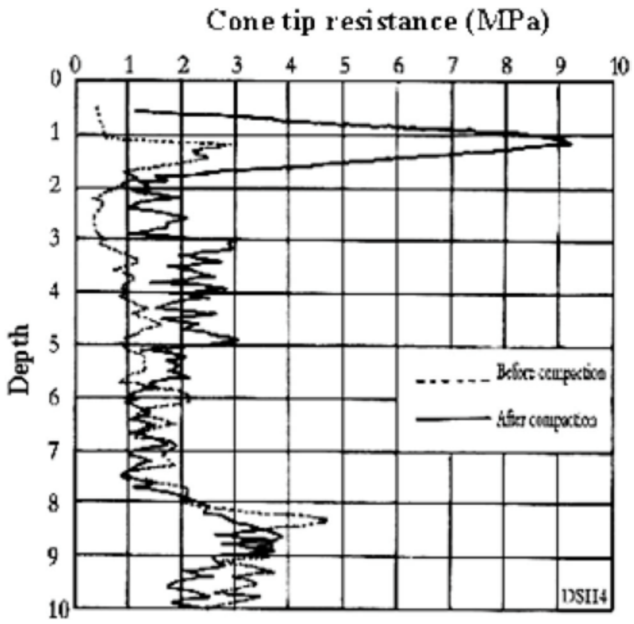

Figure 10 Comparison of the CPT tip resistance profiles before and after compaction

A comparison of CPT tip resistance profiles before and after compaction is shown in Fig. 10. Considerable increase in the tip resistance can be seen in the top 6 m. The study has shown that the DC method can be effectively used to treat soft ground if a proper drainage system is installed by using prefabricated vertical drains (PVDs) and a sand blanket, and the compaction energy is applied in a suitable way.

CONCLUSION

- Geological conditions such as age of the formation, geological processes and geomorphology are important indicators in planning and implementation of ground improvement projects.
- Geological history and stress history have significant impact on deformation and compression of foundation soils upon placing an additional fill load.

- Types of erosion, transportation and depositional environment are major causes of the inclusion of organic matters in the deposited soils. The greater content of organic matters in the soil causes large magnitude of secondary compression.
- Soil anisotropy and micro fabric features due to inclusion of sand lenses have positive effect on the time rate of consolidation.
- Stability of temporary reclamation slope and permanent shore protection structures is largely dependent upon the type and strength of soils. Construction of slopes and shore protection structures on the soils with high moisture content and low shear strength generally leads to an unstable condition.
- Ground improvement methods are available for improving strength of the soil to improve stability of the structures and also for accelerating the soil consolidation in order to eliminate primary and secondary consolidation settlement during the construction period.
- Prefabricated vertical drain and dynamic compaction combined with PVD were found to be effective in improving soft soils.
- This paper described the two unique case studies on reclamation and ground improvement in the Far East.

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