GEOTECHNICAL ENGINEERING IN MALAYSIA FROM A YOUNG CONSULTANT'S PERSPECTIVE: NEW DEMANDS, EXPECTATIONS AND CHALLENGES AHEAD

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Abstract : Geotechnical engineering is still rapidly evolving from the classical era of Coulomb and Rankine in the 18th and 19th century to the introduction of "modern" soil mechanics by Karl Terzaghi, Casagrande, Skempton, Roscoe and other famous geotechnical engineers in the 20th century. The development of geotechnical engineering in the 21st century is still at a rapid pace with the introduction of various advance tools for geotechnical engineers. The use of finite element method to analyse complex geotechnical problems is increasingly common and new challenges in the offshore industry and geo-environmental problems have presented new challenges to the geotechnical engineering community. In addition, the increasingly complex nature of the problems encountered has presented challenges to new generation of geotechnical engineers where interactions with various other specialists without civil engineering background require good communication skills. The increasingly "flat" world also presented unique opportunities for geotechnical engineers to tackle problems from all around the world from the problems of residual soils in tropical countries, sabkhas in arid regions and permafrost in cold climates. As such, proper understanding of fundamental soil mechanics is important as the codes of practices in different regions may be different but the fundamentals behind it is still the same. In this paper, the experience gained in Malaysia on the practice of geotechnical engineering consultancy is presented together with the expected demands, expectations and challenges ahead from a young consultant's perspective. It is hoped that the paper will stimulate opinions from the geotechnical engineering community in Taiwan for future exchange of technical expertise and to gain from each other's experience.

Key words : Finite element method, communication skills, fundamental soil mechanics, young consultant's perspective

1 INTRODUCTION

Geotechnical engineering is now a well-established branch of civil engineering and its scientific development can be traced to the early works of French engineers such as C.A. Coulomb who read his famous paper to the Academy of Science, Paris related to shear strength of masonry and soils, earth pressure, stability of arches and the strength of beams in 1773, A. Collin who made outstanding contribution to knowledge on the stability of clay slopes in 1846, British engineers such as William Jessop and Thomas Telford who contributed significantly to the practical aspects of soil mechanics and geotechnical engineering through their field works such as Caledonian Canal in 1810 and W.J.M. Rankine whose "A manual of civil engineering" is the standard text for at least half a century. The term 'soil mechanics' first came into general use after a series of articles by Karl Terzaghi had been published in 'Engineering News Record' (Skempton, 1979). The publication of the book Erdbaumeckanik by Karl Terzaghi in 1925 marks the emergence of classical soil mechanics with the introduction of effective stress theory and as such Terzaghi is widely regarded as the Father of Soil Mechanics. Subsequent to that, the Proceedings of the First International Conference on Soil Mechanics which was held at Harvard is published in 1936. The year also marks the emergence of soil mechanics as a discipline of civil engineering complementing the theory of structures and hydraulics. Thereafter, many outstanding works in soil mechanics and geotechnical engineering are made by famous engineers such as R.B. Peck, A. Casagrande, A.W. Skempton, A.W. Bishop, K.H. Roscoe, G.G. Meyerhof, N.M. Newmark, L. Bjerrum and many others. The term 'geotechnology' is probably first introduced by R. Glossop when he delivered the Eighth Rankine Lecture in 1968. Asian engineers have also contributed significantly to the practice of geotechnical engineering and some notable names whom the Author have come across include Kenji Ishihara, Chin Fung Kee, Za-Chieh Moh, A.S. Balasubramaniam, E.W. Brand and Lee Seng Lip.

Since then, geotechnical engineering has continued to progress and significant progress has been made in new areas such as application of finite element method to geotechnical engineering problems, understanding of pile behaviour and offshore geotechnics. The effect of globalization has also introduced new challenges to geotechnical engineers where understanding of fundamental soil mechanics is important to complement local experience for works in different parts of the world. In this paper, some experience on the application of new developments in soil mechanics and geotechnical engineering in consulting practice is presented. Some perspective on geotechnical engineering practice compared to experience in other parts of the world is also discussed from a young consultant's point of view.

2 FINITE ELEMENT METHOD IN GEOTECHNICAL ENGINEERING

The easy availability of powerful finite element software with user-friendly interface has encouraged the use of finite element method in geotechnical engineering. Currently in Malaysia, software packages such as PLAXIS, CRISP, etc. are commonly used by engineers of differing levels of experience and expertise especially for deep excavation works in urban areas. As such, various authors such as Potts (2003) and Wood (2004) had highlighted the importance of proper understanding of finite element analysis and also the coding and constitutive soil models used in the software. In an example quoted by Wood (2004) based on Schweiger (2003), the results of a benchmark problem analysed by different people using the same numerical analysis program (PLAXIS) and the same constitutive soil model with the same soil parameters are shown in Figure 1.

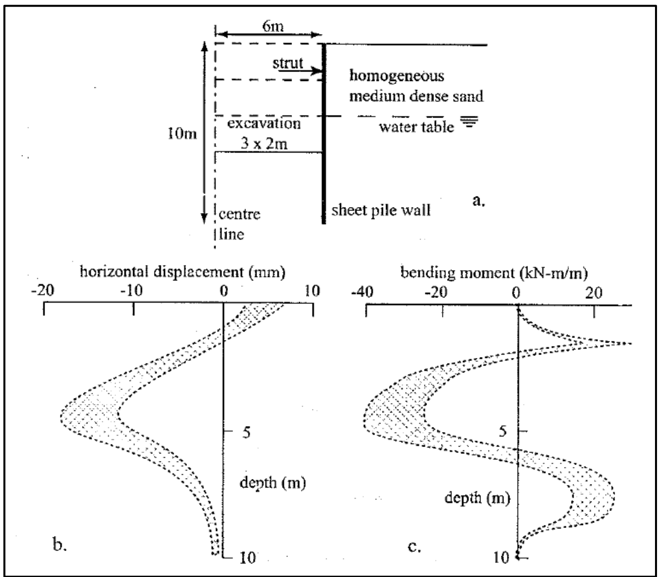

Fig. 1 Benchmark comparisons of results of numerical analysis of strutted sheet pile wall retaining dense sand using PLAXIS: (a) outline of problem analysed; (b) approximate range of predictions of horizontal displacement of walls; (c) approximate range of predictions of bending moment in walls (Wood, 2004).

As can be seen from Figure 1, there is scatter in the results, even for such a 'simple' excavation problem. As such, the designer should be aware of the following factors which may affect the results of FEM analyses:

a) Locations of the boundaries of the problem. The problem boundary should be located far enough away such that there is no stress rotation near the boundary. For undrained analysis, the extent of the model required will be greater.

b) Details of mesh. Higher order elements are to be preferred to simple elements, especially if high strain gradients are anticipated, or for failure analysis. More and smaller elements need to be placed where gradients are expected to be highest, and at regions of stress concentration (Wood, 2004).

c) Long, thin elements will lead to calculation instability. As such, the layout of the model and mesh should avoid long, thin elements.

d) Stages of construction. As soils are non-linear, history dependent materials, proper modelling of the soil at various stages from the past to its construction stages needs to be carried out.

e) Modelling of interfaces. Improper modelling or use of unconservative interface reduction factors may lead to dangerously unsafe design.

f) Use of suitable constitutive soil models to model different geotechnical problems.

g) Sensitivity of various soil parameters. For different constitutive soil models adopted in different FEM software packages, different soil parameters may have different effects on the analysis results. Some of the important parameters include:
   i. Shear strength parameters (c' and ϕ')
   ii. Stiffness parameter (E)
   iii. Coefficient of earth pressure at rest (K_o)
   iv. Wall-Ground Interface factor (δ)
   v. Permeability of soil (k)

   The amount of shear stress which can be mobilised at the wall-ground interface is governed by the wall-ground interface factor (δ) which can be significant for deep excavation.

   $$\delta = k \cdot \phi_{cv}'$$

   where ϕ_{cv}' = critical state angle of shearing resistance in terms of effective stress

   EC7 (EN1997-1:2004) recommendations are as follows:
   - for precast concrete or steel sheet pile, k should not exceed 2/3
   - for concrete cast against soil, k can be assumed as 1.0.

h) Geometrical data. In FEM analysis, the geometry of the model should reflect the actual field conditions closely. In Malaysia, the following criteria are normally observed in both FEM and manual calculation:

   a. Provision for Over-Excavation (Δa):
   The stability of the retaining wall depends on the passive ground resistance in front of the structure and therefore it is prudent to allow for over-excavation (Δa) in the design (EN1997-1:2004) depending on the site control.
   - for cantilever walls, Δa should equal to 10% of the wall height above excavation level, limited to a maximum of 0.5m;
   - for a supported wall, Δa should equal 10% of the distance between the lowest support and the excavation level, limited

to a maximum of 0.5m.

A smaller value of Δa can be used when the excavation surface can be controlled reliably throughout the excavation works. The over-excavation (Δa) provided in design is not meant for lack of control at site which is very important for all excavation works.

b. Water levels:
The selection of design groundwater level (free water and phreatic surfaces) should be based on information collected during subsurface investigation through monitoring of standpipes or other means. If the site is prone to flooding, as in many areas of Malaysia, the flood level should be taken into consideration depending on the permeability of the subsoil.

c. Surcharge:
The surcharge value should take into account the site conditions and control at site. Site conditions such as loadings from adjacent buildings, vehicles, services, etc. should be taken into consideration in the design. It is prudent to incorporate a minimum surcharge of 10kPa to cater for construction loads and unforeseen circumstances. During tender and construction, it is very important for the Contractor to be aware and follow the assumptions adopted by the designer to prevent causing problems to the retaining system due to uncontrolled stacking of materials (loading) on the retained side.

i) Constitutive soil models. In FEM analysis, proper understanding of the constitutive soil models is important in order to produce a safe design. Various soil models have been incorporated in commercial software packages ranging from elastic-perfectly plastic Mohr-Coulomb model to the Cam-clay model. For example, in the FEM computer program PLAXIS, there are various soil models for different application, i.e. the Mohr-Coulomb, Soft Soil Model and Hardening Soil Model. An example of incorrect use of soil models is best illustrated in the recent Nicoll Highway collapse (Yong & Lee, 2007). In the design of the diaphragm wall with internal strutting, the Contractor had used effective stress parameters (c' and ϕ') with the Mohr-Coulomb model to simulate the undrained behaviour of soft clay (known as Method A among PLAXIS users). This method overestimated the undrained shear strength of the marine clay as illustrated in the stress path diagrams (Figure 2). The undrained shear strength, c_u in the 'real' soil from the test is much lower than that predicted using Method A.

The consequence of using Method A with the Mohr-Coulomb model in the Nicoll Highway project led to an under-estimation of wall deflection, bending moment and strut forces in the design. The original design estimated a maximum deflection of 145mm whereas 450mm would have been computed if the lower c_u value in the 'real' soil was used (Yong & Lee, 2007).

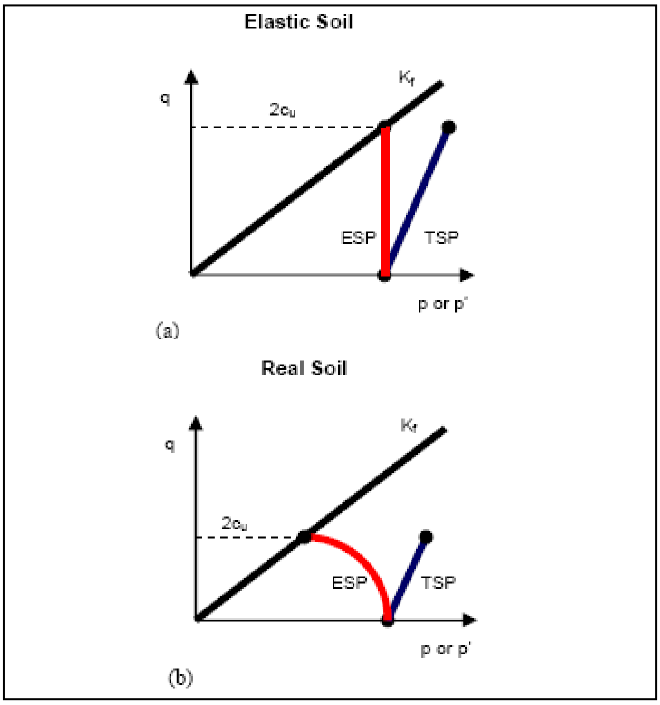

Fig. 2 Comparison of undrained strength (c_u) of a soft clay in consolidated undrained triaxial compression test: (a) stress path determined from finite element analysis using Mohr-Coulomb model with effective stress parameters (Method A) and (b) stress path of a real soil (Yong & Lee, 2007).

Even though there are incidences of improper use of finite element method with serious consequences (Figure 3), its use should not be discouraged in order to further advance the method. What is important is to set the proper "rules" for the use of finite element method (FEM) in geotechnical engineering and to prevent "abuse" of the method. Table 1 summarises some of the potential misuse of the method due to lack of proper guidelines or "rules".

Table 1 Potential misuse/abuse of FEM in geotechnical engineering

Potential Misuse/Abuse	Recommended Guidelines/ "Rules"
Inappropriate use of optimistic (without substantiation) stiffness parameters (E) to gain competitive advantage.	Design should be carried out for two scenarios, i.e. using conservative stiffness and more optimistic stiffness. In such a way, contingency measures are in place prior to actual construction in the event the

	stiffness is less optimistic than initially assumed. Such approach will also prevent affected parties to continually "push the boundary" in order to minimise cost while at the same time such approach will still encourage design innovation (within an acceptable risk framework). The validity of the assumptions made shall be verified based on instrumentation results with clear threshold established.
Inappropriate use of optimistic wall-ground interface, K_o and did not take into considerations provision for over-excavation and surcharge loading. Guidelines for use of FEM in geotechnical engineering are not covered in traditional codes of practice and as such, instances of ignoring such good practice are encountered in order to gain competitive advantage.	Clear guidelines in the use of appropriate wall-ground interface, K_o and provisions of over-excavation and surcharge loading should be clearly specified. Such approach will provide a "fair" approach as geotechnical engineers will compete based on design innovations without compromising public safety.
Use of lower factors of safety for design. This arises as guidelines for use of FEM in geotechnical engineering is not covered in traditional codes of practice and as such, explicit checking of factors of safety is sometimes ignored.	Clear sets of factors of safety (FOS) such as overall stability, structural design of wall, strut, etc. should be established. The use of FEM does not necessarily justify the use of lower factors of safety.
Inappropriate use of constitutive soil model.	Guidelines on the selection of appropriate soil models for specific geotechnical problems shall be established. For example, the inappropriate use of soil models is illustrated in the incident of Nicoll Highway Collapse. Some guidelines on the use of FEM in geotechnical engineering have been published by the European Geotechnical Thematic Network (GeoTechNet, 2005).

Fig. 3 Nicoll Highway collapse (COI, 2005).

In the practice of geotechnical engineering, variability such as those shown in Figure 1 is expected and is not confined to use of FEM only. For example, Figure 4 shows the variability associated with predictions of pile capacity and settlement and it demonstrates the variability of the prediction even for such relatively "simple" design of a pile.

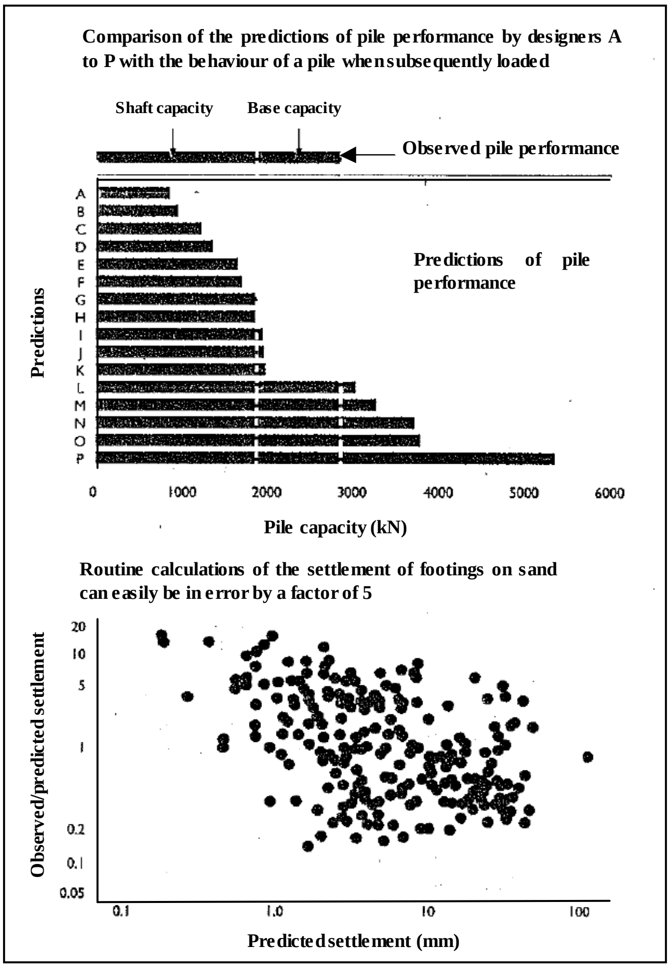

Fig. 4 Variability of geotechnical engineering predictions (Clayton, 2001).

However, incidences of foundation failure is rare and traditionally, geotechnical engineers have rightly and prudently carried out pile load tests at site to verify pile capacities. As such, similar approach for retaining wall design is appropriate where design verification must be carried out based on instrumentation results. In addition, the designer should be aware of the modelling techniques required for different FEM software and also their limitations. In this respect, some guidelines have been published by the European Geotechnical Thematic Network (GeoTechNet, 2005) and this represents the appropriate way forward for the application of FEM in geotechnical engineering. Some important guidelines highlighted by Geotechnet (2005) are summarised below:

a) For non-linear constitutive soil models, higher order elements should be used. As a minimum, quadratic elements for elastic-plastic analysis are suggested.
b) For excavations and retaining walls, in overwhelming majority of simple cases, linear elastic analyses are entirely inappropriate and can be misleading.
c) For embankments in soft clays, normally consolidated or slightly overconsolidated, the soft clays are often subjected to plastic deformations during excavation, and it may be necessary to select models taking account of the pre-failure plastic behaviour of the soil (hardening models).

On a final note, fundamental understanding of soil mechanics and theory of structures is important. While variability in predictions is expected as shown in Figure 1 but engineers should not get the behavior of the structures wrong! As such, a simple hand calculation should always be carried out together with FEM analysis as a check against the validity of the FEM analysis.

3 SOIL NAIL DESIGN

Soil nail as stabilization measure for distressed slopes and for new very steep cut slopes has the distinct advantage of strengthening the slope without excessive earthworks to provide construction access and working space associated with commonly used retaining system such as reinforced concrete wall, reinforced soil wall, etc. In addition, due to its rather straightforward construction method and is relatively maintenance free, the method has gained popularity in Malaysia for highway and also hillside development projects.

The basic concept of soil nailing is to reinforce and strengthen the existing ground by installing closely-spaced steel bars, called ‘nails’, into a slope as construction proceeds from ‘top-down’. This process creates a reinforced section that is in itself stable and able to retain the ground behind it. The reinforcements are passive and develop their reinforcing action through nail-ground interactions as the ground deforms during and following construction.

Various international codes of practice and design manuals such as listed below are available for design of soil nail:

a) British Standard BS8006: 1995, Code of Practice for Strengthened/Reinforced Soils and Other Fills.
b) HA 68/94, Design Methods for the Reinforcement of Highway Slopes by Reinforced Soil and Soil Nailing Techniques.
c) U.S. Department of Transportation, Federal Highway Administration (FHWA, 1998), Manual for Design and Construction Monitoring of Soil Nail Walls.

Chow & Tan (2006) discussed the various design methods available for design of soil nail and subsequently recommended the FHWA method (incorporating some good practices in HA68/94) as the method provides a complete design method for soil nail inclusive of other design aspects such as shotcrete, soil nail head, etc. which is important to ensure satisfactory performance of soil nailed slope but is often overlooked in design.

In this paper, a review of important design and construction issues will be presented especially on the importance of shotcrete face design for very high and steep slopes which is sometimes overlooked during design. An interesting investigation case history is also presented to illustrate some of the pitfalls of easy-to-use computer software.

The failure modes of soil nails can be categorized into the following:

a) Pullout failure
b) Nail tendon failure
c) Face failure
d) Overall failure (slope instability)

3.1 Pullout failure

Pullout failure as illustrated in Figure 5 results from insufficient embedded length into the resistant zone to resist the destabilizing force. The pullout capacity of the soil nails is governed by the following factors:

a) The location of the critical slip plane of the slope.
b) The size (diameter) of the grouted hole for soil nail.
c) The ground-grout bond stress (soil skin friction).

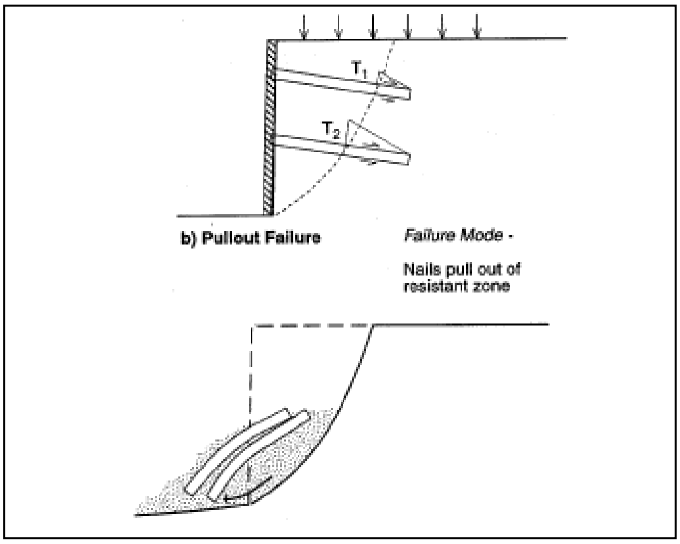

Fig. 5 Pullout failure mode (FHWA, 1998).

The location of the critical slip plane for the slope could be readily assessed from manual calculations or various commercially available slope stability analysis software with capability to include internal reinforcements (e.g. geotextile, ground anchors and soil nails). Resisting force for the soil nails based on the available bond length from the critical slip plane shall then be input into the stability analysis in order to obtain appropriate factor of safety. Some software have the capabilities to automatically update the resisting force based on the computed critical slip plane. If not, iterative analysis needs to be carried out to obtain the correct soil nail resisting force for slope stability analyses. The size (diameter) of the grouted hole for soil nail is usually in the range of 75mm to 150mm for commonly available drilling rigs. Therefore, for pullout failure, the responsibility between designers and constructors can generally be summarized as follow:

a) Designer: Determination of appropriate ground-grout bond stress and pull-out capacity based on critical slip plane. Some guidance on the determination of ground-grout bond stress is discussed in Tan & Chow (2004).
b) Constructor: To ensure diameter of grouted hole as specified by the designer is achieved at site and the hole is properly grouted throughout the nail length. (Grouting using tremie method filling from bottom up and non-shrink grout shall be used).

3.2 Nail tendon failure

Nail tendon failure as illustrated in Figure 6 results from inadequate tensile strength of the nails to provide the resistant force to stabilize the slope. It is primarily governed by the grade of steel used and the diameter of the steel. Typically a minimum nail size of 25mm is used as nail sizes smaller than 25mm may cause installation problems for moderate to long nail lengths due to their low stiffness. Besides specifying the appropriate nail size corresponding to the required resistant force, it is important that proper detailings with regards to corrosion protection of the nails are specified and properly executed at site. Some of the important considerations include:

a) Adequate cover for nails is provided by ensuring rigid spacers/centralizers at appropriate spacing. Figure 7 shows example of typical spacers used.
b) Corrosion protection on the nails using galvanized steel bars or by encapsulation inside a corrugated plastic sheath.

Therefore, for nail tendon failure, the responsibility between designers and constructors are:

a) Designer: Determination of required nail diameter, spacing of spacers/centralizers and corrosion protection requirements.
b) Constructor: To ensure spacers/centralizers are rigidly secured to the nail and corrosion protection carried out as per requirements.

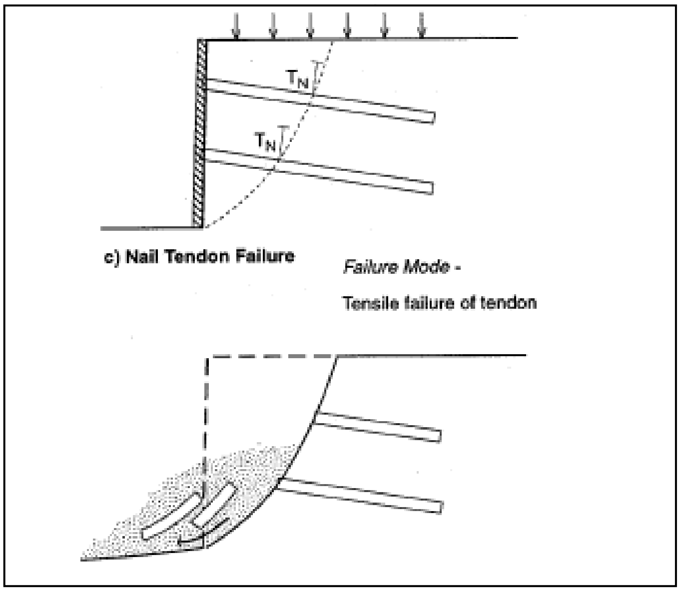

Fig. 6 Nail tendon failure mode (FHWA, 1998).

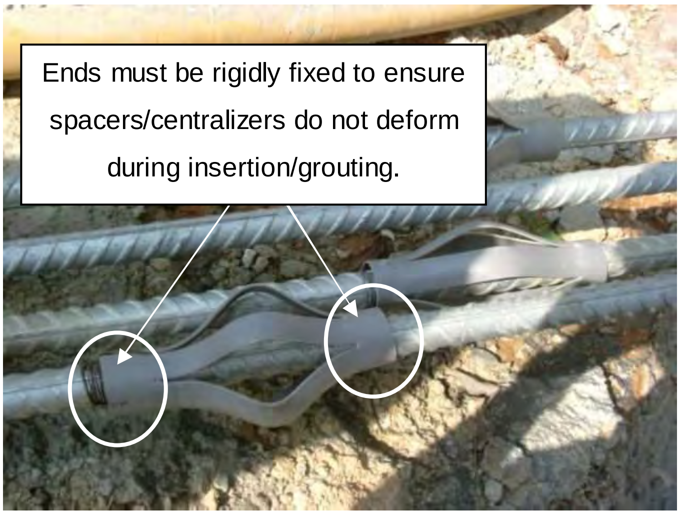

Fig. 7 Typical spacers/centralizers for soil nails.

Some of the common problems encountered at site include damage to the nails during transportation where the galvanized layers are being scraped off and also inadequate spacing between the nail and corrugated plastic sheath to form an effective grout protection layer. Figure 8 shows an example of such incidence where the very thin layer of grout cracked and peeled off upon insertion of the nails into the drilled hole. Generally, it is not recommended to use pre-grouted corrugated plastic sheath for soil nails in Malaysia due to lack of good quality workmanship and control at site. For soil nails that need to use corrugated plastic sheath, then larger diameter hole with the diameter of the corrugated plastic sheath at least three times the diameter of the steel bar or minimum of 75mm, whichever is larger should be used. In addition, a minimum grout cover between the sheath and the borehole wall should not be less than 12mm (FHWA, 1998) but commonly 25mm is recommended for practical purposes. Special care shall also be exercised during insertion of the pre-grouted corrugated soil nails to prevent bending and accidental knocking that could cause cracks to the grout and thus, loss of bonding between the grout and the steel bar (potential pullout failure).

Finally, the designer and constructor also have to ensure that the spacers/centralizers are rigidly fixed to the nails and do not deform during insertion and grouting (Figure 7).

Fig. 8 Grout cracked and peeled off from nail – ineffective corrosion protection.

3.3 Face failure

This aspect of failure mode (Figure 9) for soil nailing is sometimes overlooked as it is generally wrongly "assumed" that the face does not resist any earth pressure. For soil nailing works which involve slopes of relatively low height and gentle gradient, the earth pressure acting on the shotcrete face is relatively small and nominal shotcrete thickness and reinforcement is adequate.

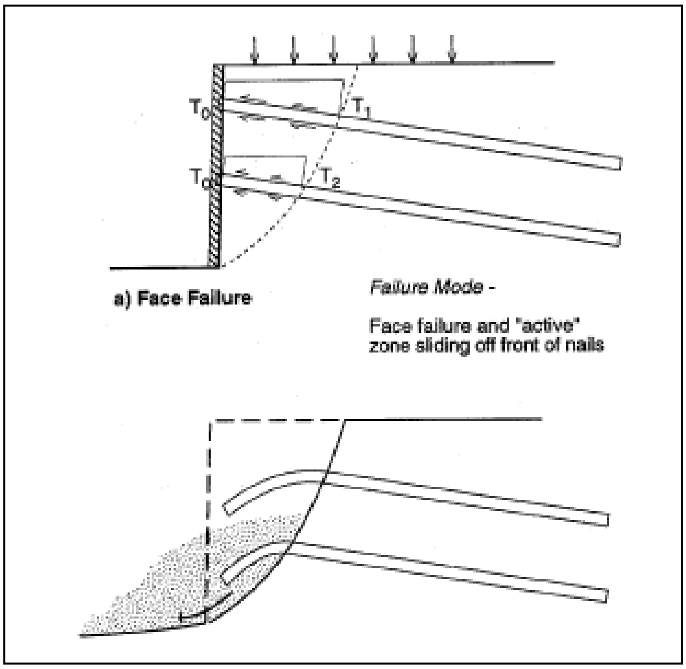

Fig. 9 Face failure mode (FHWA, 1998).

However, this assumption shall not be applied for all slopes and face failure is an important failure criterion that should not be overlooked. This is highlighted in the following clauses of BS8006 and FHWA's manual:

a) BS8006: 1995, Clause 6.7.3:
"Facings should be designed to accommodate the loads resulting from horizontal soil pressures and the corresponding reinforcement tension reactions developed in the connections between the facing and the reinforcement."

b) FHWA's Manual for Design and Construction Monitoring of Soil Nail Walls (pg. 95):
"The facing structural design requires provision of adequate concrete thickness, reinforcement and moment capacity to resist the earth pressures applied to the facing span between adjacent nail heads, and provision of adequately sized bearing plates to provide adequate punching shear capacity."

For example, for a slope of 10m high with global slope gradient of 45° (1V:1H) with soil properties of ϕ'=33° (c'=0) and nail spacing of 1.5m (vertical and horizontal). The active force acting at the bottom of the soil nailed slope is only about 6kN taking into consideration that 50% of the force is transferred to the nails due to arching effect and flexible shotcrete facing as per FHWA (1998).

However, for a slope of 40m high with global slope gradient of 76° (4V:1H) with similar soil properties and nail spacing, the active force acting at the bottom of the soil nailed slope is about 140kN which is more than 20 times larger than the earlier example. Such large active force acting on the shotcrete face should not be overlooked during design and the shotcrete thickness and reinforcement should be properly designed.

Figures 10 and 11 show examples of failure due to face failure where nails 'protruding' out from the slope after failure can be observed. It is interesting to note that FHWA (1998) recommendation of 50% of the active force transferred to the nails is based on results of field monitoring on typical nail spacing ranging from 0.75m to 1.8m. Therefore, for very high and steep slopes, large spacing of soil nails should be avoided unless reliable analyses on the stresses acting on the shotcrete surface is carried out and designed for it.

Similar to other modes of failure, the designer and constructor each have important roles to play to prevent face failure:

a) Designer: Adequate shotcrete thickness and reinforcement provided with proper detailings. Figure 12 illustrates example of improper detailing which will trigger potential face failure.

b) Constructor: To ensure shotcrete thickness and reinforcement as per requirements. A proper shooting technique by experience nozzleman and correct shotcrete mix are important to ensure shotcrete of good quality.

Fig. 10 Example of face failure (side view).

Fig. 11 Example of face failure (front view).

3.4 Overall failure (slope stability)

This aspect of failure mode is commonly analyzed based on limit equilibrium methods. The analyses are carried out iteratively until the nail resistant force corresponds to the critical slip plane from the limit equilibrium analysis. To carry out such iterative analyses, it is important that the nail load diagram (Figure 13) is established. From Figure 13, it can be seen that the nail load diagram consists of three zones, A, B and C. Zone A is governed by the strength of the facing, T_F and also the ground-grout bond stress, Q. If the facing of soil nails is designed to take full tensile capacity of the nail, then the full tensile capacity of the nail can be mobilized even if the critical slip circle passes through Zone A. However, to design the facing with full tensile capacity of nails instead of lower T_F is not economical for high slope (e.g. more than 15m). Zone B is governed by the nail tendon tensile strength and Zone C is governed by the ground-grout bond stress, Q.

From the diagram, it is clear that the mobilized nail resistance should not exceed the nail load envelope developed from the three failure criteria discussed earlier. Therefore, the nail resistance to be input into slope stability analysis should refer to the nail load diagram (Figure 13) corresponding to the available bond length for the critical slip plane (Figure 14).

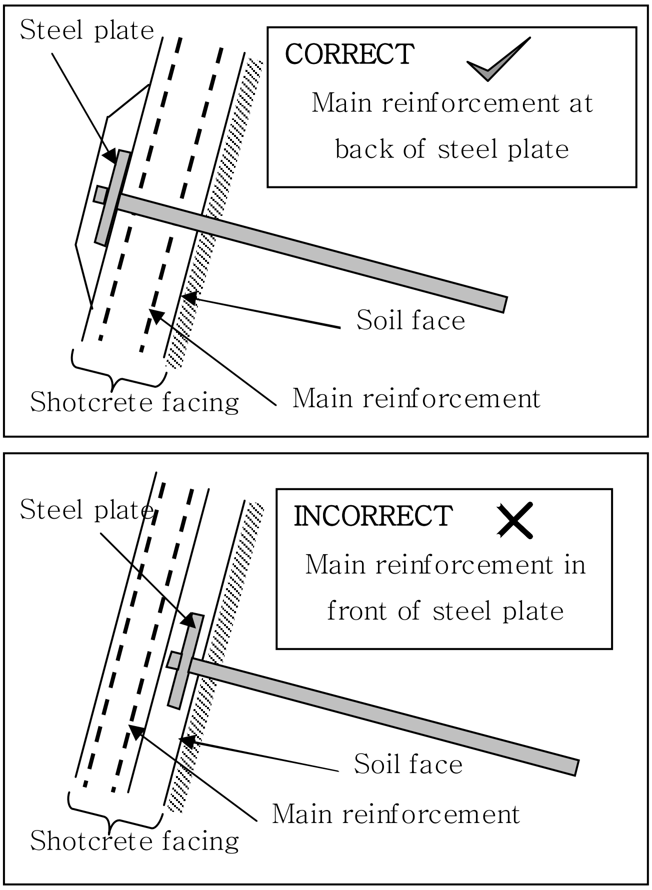

Fig. 12 Example of proper and improper detailings.

Some slope stability analysis software have the capabilities to automatically adjust the nail resistance based on the ground-grout bond stress and nail tendon tensile strength. However, extra caution needs to be exercised as some of the software do not cater for the reduction of nail load at Zone A and assumes strength of T_N. As illustrated in Figure 15, failure to cater for the reduction of soil nail resistance in Zone A may lead to overestimation of the available nail resistance in slope stability analysis for critical slip circle that passes through this zone.

3.5 Case history

A soil nail slope of approximately 20m high failed in August 2008 (Figure 16) and this prompted an investigation into the possible cause(s) of failure and review of the soil nail slope design. In this paper, only some 'interesting' findings are presented to emphasize the importance of proper understanding of principles of soil mechanics and geotechnical engineering. The investigation also demonstrates an example of 'improper' use of slope stability analysis software. Some of the main findings are summarized below:

a) Grout-soil skin friction in the original design has adopted higher values for soil nails in the upper parts of the slope compared to soil nails in the lower part of the slope. This is illogical as the subsurface investigation results clearly indicate that the soil strength is increasing with depths.

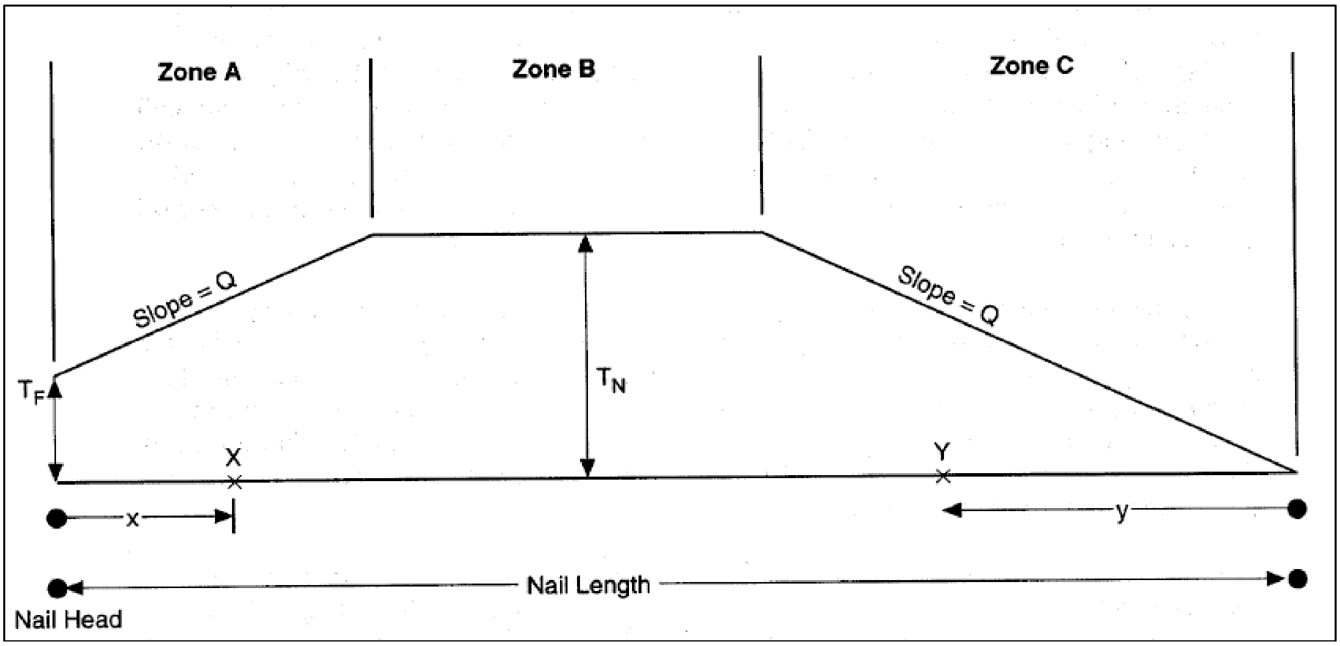

Fig. 13 Nail load diagram (FHWA, 1998).

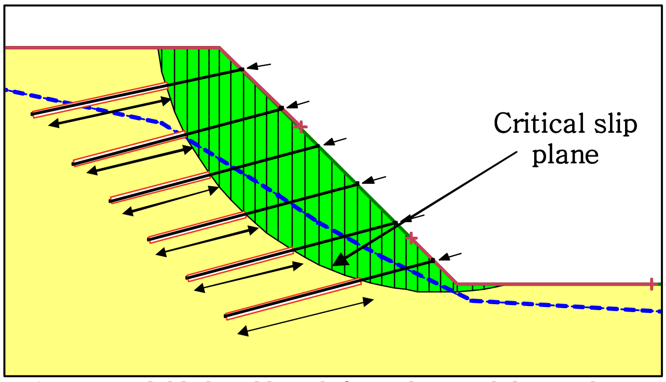

Fig. 14 Available bond length from slope stability analysis.

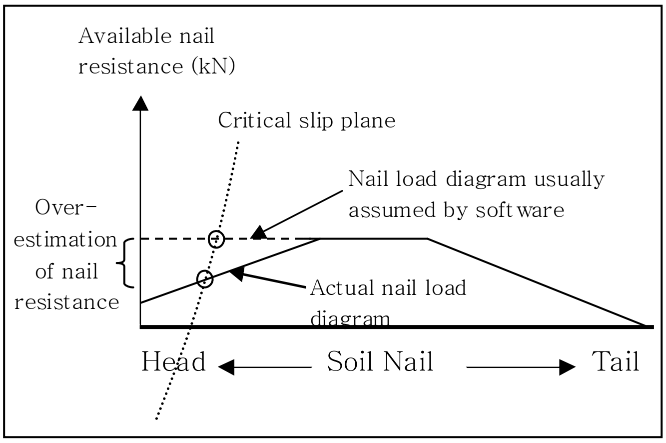

Fig. 15 Overestimation of nail resistance if shotcrete facing strength not taken into consideration.

Upon further investigation, it was found that the original design has ‘fixed’ the capacity of the soil nail to its structural strength and then the skin friction is back-calculated in order to match the structural strength of the soil nail. As such, with the longer soil nails at the lower parts of the slope, the ‘calculated’ skin friction is lower for the soil nails at the lower parts of the slope.

This is a serious error, demonstrating the lack of understanding of principles of soil nail where the grout-soil skin friction is an important parameter which affects the slope stability significantly!

b) The original slope stability analysis has reported a factor of safety (FOS) of 1.235 as shown in Figure 17. Based on the soil strength and other soil nail parameters adopted in the slope stability model, the reported FOS is doubtful based on the Author’s experience.

Upon further investigation, it was found that the reported FOS of 1.235 is not the most critical failure surface produced by the software but was chosen as it report adequate FOS of greater than 1.2. Independent analysis carried out by the Author using the exact same model and computer software and using the original (erroneous) grout-soil skin friction has produced FOS of only 0.835 as shown in Figure 18!

This simple case history highlights the lack of understanding of fundamental soil mechanics and geotechnical engineering and misuse of computer software. As such, it is important that engineering education focuses on fundamental engineering principles instead of rigid design procedures according to codes of practices. The importance of understanding of fundamental soil mechanics is further illustrated in the following section.

Fig. 16 Failed soil nail slope
(Note: Some backfilling works already carried out to stabilize the slope).

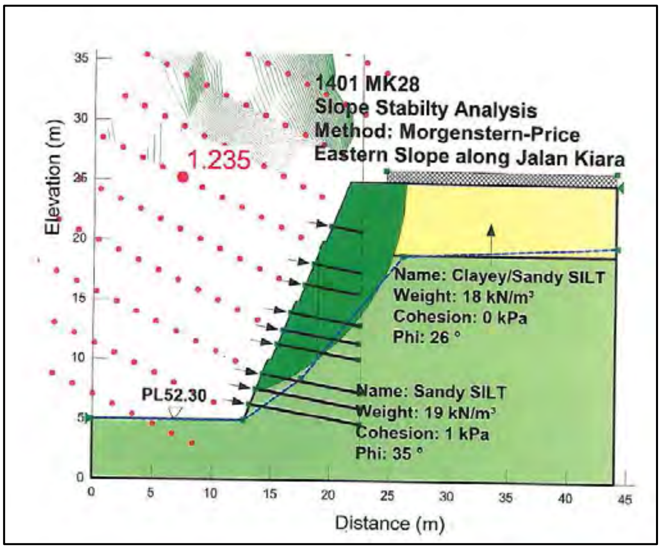

Fig. 17 Incorrect FOS reported in original design of soil nail slope.

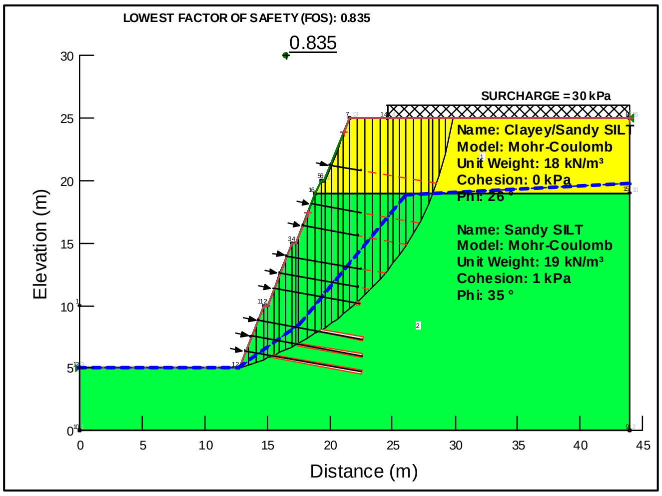

Fig. 18 FOS of only 0.835 obtained for most critical slip surface using the exact model and soil parameters of Figure 17.

4 EXPERIENCE IN TURKMENISTAN

Turkmenistan is part of the former Soviet Union which gained independence in the year 1991 after the collapse of the Soviet Union. It is a Central Asia country bordering the Caspian Sea between Iran and Kazakhstan.

In this paper, the information obtained is for the Turkmenbashy area, which is a port town situated approximately 535km from its capital, Ashgabat as shown in Figure 19 below. The proposed site at Turkmenbashy area experiences semi-arid weather condition with absolute minimum temperature reaching -22°C and the absolute maximum temperature reaching +45°C. Average monthly relative air humidity at 1 o'clock ranges from 71% (coldest month) to 39% (hottest month). The atmospheric precipitation within a year is only 136mm. The site also experiences dust storms with frequency in the order of 20 days within a year.

Fig. 19 Map of Turkmenistan
(Source: http://www.lib.utexas.edu/maps/turkmenistan.html)

4.1 Geological and geotechnical aspects

A notable feature which is common with arid and wind-swept landscapes are residual peaks of hard rock left upstanding and wind polished above the general level (Blyth & de Freitas, 1984). These features are termed inselbergs (or 'island mounts'). An example of inselbergs in the Turkmenbashy area is shown in Figures 20 and 21. Due to its dry climate, vegetation is scarce and therefore, any soil deposits are easily eroded by wind. This is in contrast to features in Malaysia where dense vegetation and heavy rainfall results in very deep zones of residual soil and weathered rock as shown in Figure 22.

Fig. 20 Inselbergs at Turkmenbashy.

Fig. 21 Inselbergs at Turkmenbashy.

Fig. 22 Residual soils in Malaysia.

The weathering process in regions of hot and dry climate is predominantly governs by the action of wind. Wind-blown sand (or eolian sand) grains, dominantly composed of quartz, become worn down to well-rounded, nearly spherical forms with frosted surfaces. The grains are poorly graded, i.e. of nearly uniform size, since wind of a given velocity cannot move particles larger than a certain diameter (Blyth & de Freitas, 1984). This characteristic is observed in the results of particle size distribution carried out on soil samples obtained from the site and shown in Figure 23.

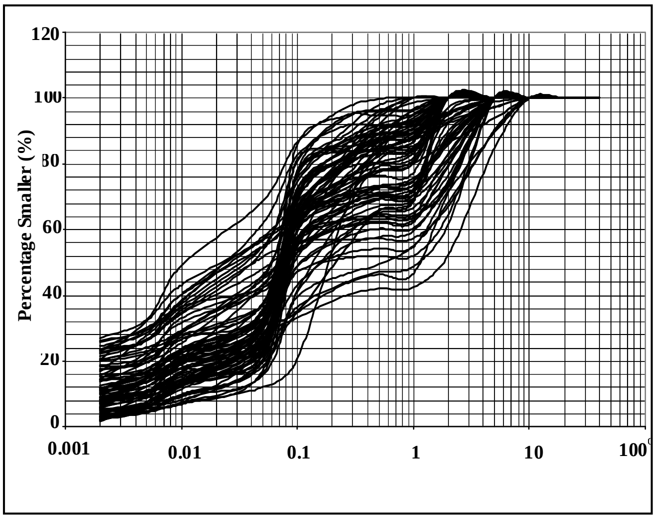

Fig. 23 Particle size distribution of wind-blown sand from Turkmenbashy area.

As shown in Figure 23, the grains are poorly graded and predominantly consist of grain size with diameter in the range of 0.05mm to 0.1mm. Typical landscape of the Turkmenbashy area is shown in Figure 24.

In the absence of downward leaching (e.g. from rain), surface deposits become contaminated with precipitated salts, particularly sulphates and chlorides (Bell, 2000). This is also confirmed from the preliminary geotechnical investigation carried out at the site and summary of the chemical test results are presented in Table 2.

Fig. 24 Typical landscape of Turkmenbashy area in summer.

Table 2 Results of chemical tests on soil samples.

BOREHOLE	DEPTH (m)	CONTENT (mg/l)	
		Cl^-	SO_4^{2-}
BH-1	1.0	208.80	2514.40
BH-2	0.5	475.60	1547.20
	1.0	336.40	2639.40
	2.0	406.00	2533.70
BH-5	1.0	353.80	2471.20
	5.0	336.40	2903.80
BH-6	1.0	411.80	2615.40
	2.0	359.60	2985.60
BH-9	0.5	359.60	2370.20
	1.0	226.20	2896.90
BH-10	1.0	400.00	2962.80

As can be seen from Table 1, the soil is very aggressive towards concrete made with ordinary Portland cement as its sulphate content is consistently more than 2280mg/l (expressed in SO_4^{2-}). This is in accordance with BS8110:Part 1 where concrete exposed to soil with sulphate content more than 2280mg/l (expressed in SO_4^{2-}) or 1900mg/l (expressed in SO_3) but less than 3700mg/l (expressed in SO_4^{2-}) or 3100mg/l (expressed in SO_3) is categorized as Class 3 where ordinary Portland cement with pfa or ggbfs, sulphate resistant cement (SRPC) or super sulphated cement (SSC) shall be used.

As such, the challenges posed to geotechnical engineers are completely different due to the different geological settings of Malaysia and Turkmenistan. In addition to the different geological settings, other challenges include different standard practices in Turkmenistan which essentially follows the former Soviet Union/USSR practice.

4.2 Soil investigation and design practice in Turkmenistan

The common soil investigation (SI) practice in Turkmenistan is rather surprising to the Author who is more familiar with British/American practice which is commonly adopted in Malaysia. The common soil investigation practice adopted in Turkmenistan is advancement of borehole my means of a drill on a make-shift army truck as shown in Figure 25.

Fig. 25 SI drill rig in Turkmenistan.

Collection of undisturbed samples is not common due to the sandy nature of the soil and inspection of the soil properties is carried out at-site where the operator placed the drilled materials on the ground as shown in Figure 26. Collection of disturbed samples for laboratory testing is carried out using the apparatus shown in Figure 27. Figure 28 shows the project geologist collecting disturbed samples from the drill auger.

Fig. 26 Drilled materials obtained during SI.

Fig. 27 Apparatus for collection of soil samples.

Fig. 28 Collection of disturbed samples from drill auger.

The initial reaction of the Author is that the SI carried out to common Turkmenistan practice is grossly inadequate and is of limited use. As such, the Author requested for another round of SI based on American/British standards to be carried out by Turkish contractors or international SI contractors such as Fugro. Naturally, the cost for the SI works quoted by the international contractors was very high and as such, the quantity of the SI had to be scaled down dramatically.

However, upon further review of the design practice adopted in Soviet Union/USSR for sandy materials (e.g. SNiP 2.02.01-83: Foundations of buildings and structures), it was found that the SI practice was generally sufficient for preliminary design based on Soviet Union/USSR's Building Standards and Regulations (SNiP).

For example, in SNiP 2.02.01-83, typical values of cohesion, c' and friction angle, ϕ' is given based on the types of soil, porosity ratio, e and for fine-grained materials, additional parameter of liquidity index, I_L is required (Table 1 from SNiP 2.02.01-83 is reproduced as Tables 3 in this paper). Preliminary values of design bearing capacity can also be assessed using similar set of soil parameters and the same approach is also adopted for pile foundations (SNiP 2.02.03-85). In fact, Soviet Union/USSR's practice for foundation design is based on limit state principles with partial factors of safety which is now being adopted by Eurocode. As such, it is important for engineers to approach design works in a foreign country by first understanding their common practice and then subsequently modify/improve the local practice to suit the project needs. The temptation to 'import' the entire system which we are familiar with should be resisted and in some cases, the approach may not be valid due to lack of local experience.

Therefore, for the project in Turkmenistan, the SI carried out by international contractors to American/British standards was significantly scaled down and its primary objective was to confirm soil parameters obtained from SI carried out using local practice and to obtain other parameters such as SPT-N values for liquefaction assessment.

In this project, a valuable experience gained is the importance of understanding of fundamental soil mechanics and geotechnical engineering principles. This is because even though the SNiP design approach is significantly different compared to British Standards, the underlying fundamentals and principles are still the same and a training based on fundamental understanding will produce a more 'international' geotechnical engineer who is not confined to its local code of practice.

Table 3 Standard values for cohesion, c' and friction angle, ϕ' (Table 1 from SNiP 2.02.03-85).

Standard values for the specific cohesion c_n, kPa (kgf/cm^2), angle of internal friction ϕ_n, degrees, and modulus of deformation E, MPa (kgf/cm^2) for sandy ground in Quaternary deposits

Type of sandy ground	Symbol for ground characteristics	Ground characteristics with a porosity ratio e equal to			
		0.45	0.55	0.65	0.75
gravelly and coarse	c_n	2(0.02)	1(0.01)	-	-
	ϕ_n	43	40	38	-
	E	50(500)	40(400)	30(300)	-
medium coarse	c_n	3(0.03)	2(0.02)	1(0.01)	-
	ϕ_n	40	38	35	-
	E	50(500)	40(400)	30(300)	-
Fine	c_n	6(0.06)	4(0.04)	2(0.02)	-
	ϕ_n	38	36	32	28
	E	48(480)	38(380)	28(280)	18(180)
Powdery	c_n	8(0.08)	6(0.06)	4(0.04)	2(0.02)
	ϕ_n	36	34	30	26
	E	39(390)	28(280)	18(180)	11(110)

5 COMMUNICATION SKILLS

In the preceding sections, discussions on the importance of understanding of fundamental soil mechanics are presented so that the next generation of geotechnical engineers can fully utilize the various advances in geotechnical engineering such as finite element method, new construction methods such as soil nail and to be able to practice on a global platform. However, besides technical skills, engineers should also be sufficiently trained in other soft skills such as communication skills.

It is the Author's opinion that because of the rigours and complexity of engineering studies, engineers often overlook other soft skills such as communication skills which are equally important in any civil engineering project.

A study carried out by the Carnegie Foundation for the Advancement of Teaching and Carnegie Institute of Technology in the 1930s showed that 85 percent of one's financial success, even in technical fields such as engineering, is due to communication skills (Lowndes, 2003). The importance of communication skills is perhaps even more important for geotechnical engineers due to the inherent variability and uncertainties associated with the ground. As such, current and future geotechnical engineers are expected to be:

a) Technically competent with strong grasp of fundamental principles of soil mechanics and geotechnical engineering.
b) Possesses an inquisitive mind with a spirit to continually improve oneself in both technical aspects and other social skills.
c) Possesses good communication skills and strong community spirit.
d) Ethical and professional.

From the above, the practice of geotechnical engineering is not expected to be easy and it should be rightly so. This is in line with increased public expectations to have safer, more economical and more environmental-friendly solutions. The future will provide more exciting challenges to geotechnical engineers with decreasing 'good' land, increasingly deep offshore exploration and increasingly complex geoenvironmental issues.

6 CONCLUSION

Various advances in the practice of geotechnical engineering have been made since the early development of soil mechanics and geotechnical engineering by pioneers such as C.A. Coulomb, K. Terzaghi, A.W. Skempton, etc. Recent advances include application of finite element methods in geotechnical engineering, new construction methods such as soil nail, jack-in pile, etc. and development of offshore geotechnics. The advances made have certainly benefitted the geotechnical engineering community and the general public but like all new advances, a few 'hiccups' are expected as we go through the learning curve. However, in order to minimize the 'hiccups', it is important that geotechnical engineers adopt a conscientious approach to the use of relatively new technologies such as finite element methods, with public safety always in their mind.

The understanding of fundamental soil mechanics and geotechnical engineering is important so that geotechnical engineers can fully utilize the various advances in geotechnical engineering. In this respect, universities should focus on teaching of engineering fundamentals instead of codes of practices.

On a final note, the demands, expectations and challenges ahead for geotechnical engineers will increase with time and as such, current and future geotechnical engineers are expected to be equally competent in both technical skills and also social skills such as communication skills. Geotechnicals engineer also have to continually improve themselves to develop or apply new geotechnical concepts and procedures. To quote Prof. Heinz Brandl (Brandl, 2004):

"(Geotechnical engineers) should never cross the borderline from a serious calculated risk to "Geo-gambling" or "Geopoker" in order to save money or win a bid (i.e. insufficient ground investigation, improperly reduced safety factors, lower quality of material and execution, etc."

"Whoever wants to move the world, has to first move himself" (Sokrates, 470-399 B.C.).

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