Geotechnical Advancements in Hong Kong since 1970s

15 May 2007

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THE HONG KONG INSTITUTION OF ENGINEERS 香港工程師學會
Geotechnical Division, Hong Kong Institution of Engineers

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Geotechnical Engineering Office
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Proceedings

Geotechnical Advancements in Hong Kong since 1970s

15 May 2007

Hong Kong

Joinly Organised by:
Geotechnical Division, Hong Kong Institution of Engineers
Hong Kong Geotechnical Society
Geotechnical Engineering Office, Civil Engineering and Development Department,
Hong Kong SAR Government

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FOREWORD

2007 is a special year for the geotechnical profession in Hong Kong. 35 years ago (1972), the two most disastrous landslides in the last century occurred in Po Shan Road and Sau Mau Ping, killing a total of 138 people. After these incidents, the Government started to recognize the importance of geotechnical engineering. Another serious landslide in Sau Mau Ping four years later eventually triggered the establishment of the then Geotechnical Control Office 30 years ago (1977). The status and importance of geotechnical engineering have since then been increasing. With the growing of the number of engineers practicing in the geotechnical field, the Geotechnical Group of the HKIE was set up in 1980 and later the HKIE Geotechnical Division was formed 20 years ago in 1987 to provide a venue for development of our professional excellence. Therefore, year 2007 should be a good time for geotechnical professionals in HK to reflect on their experience and achievements in the past.

This Geotechnical Division Annual Seminar with the theme of "Geotechnical Achievements in HK Since 1970s" will provide an excellent opportunity for local geotechnical experts and practitioners and others to impart on the advancements of geotechnical engineering practice in Hong Kong over the past 3 decades and to discuss what we could do for further development of geotechnical engineering practice to meet Hong Kong's need.

On behalf of the Geotechnical Division, I would like to thank the HKGES and Geotechnical Engineering Office, CEDD for jointly organising this seminar, the authors of papers, the sponsors and all those who have helped to make this event possible. In particular, I am grateful to the members of the Organising Committee of the Seminar : Ir Prof. Charles Ng, Ir Prof. George Tham, Ir Y S Au-Yeung, Ir Dr Eric Li, Ir Dr Gavin Toh, Ir Ringo Yu, Ir Philip Chung, Ir Anthony Lam, Ir William Wong, Dr Johnny Cheuk, Ir Crystal Ho and Ms Amy Yu for their excellent and dedicated work in making this seminar a great success.

(Albert Ho)
Chairman,
Geotechnical Division (2006/07 Session)
The Hong Kong Institution of Engineers
May 2007

Message From Ir Dr Greg C Y Wong

Past Chairman of Geotechnical Division (1987-88)

To me there are three important milestones in the development of geotechnical professionals in Hong Kong. The first important milestone was the establishment of the Geotechnical Division in 1987/1988 session making the administration of geotechnical engineering matters in HKIE independent and in the hands of geotechnical engineers. The second one was the establishment of the Geotechnical Discipline in 1988 making geotechnical engineering a distinct engineering practice in Hong Kong and clearly label for the public who are the geotechnical professionals. The third milestone was the Registration of Geotechnical Engineers RGE under the Building Ordinance making building related geotechnical works required by Law to be the responsibility of geotechnical engineers. It does not mean there are no achievements other than these and many people have contributed in many ways to the development of the geotechnical profession in the past 30 years. Dr. E-Brande and Dr. A-Malone are not writing this congratulatory message, but their contribution is enormous.

Ir Albert Ho asked me to write my issues of interest as Chairman. Unfortunately the issues I can recall are generally not interesting and certainly unpleasent. I recalled the many passing comments about my up splitting civil engineering when the Geotechnical Group requested to be allowed to form its own Division. After I became Chairman of the new Division, 16 members called an SGM and passed a resolution to form the Geotechnical Discipline. Again I went to the HKIE Council and as expected faced more ridicules. There were suggestions from other disciplines that our next step was to form a Hong Kong Geotechnical Society separate from HKIE (which we did in 2003). There was talks about we wish to open HKIE to geologists (which we tried with MHKIE but no RPE(G) scheme but failed). There was then also a comment that the geotechnical engineers' real target is to have RGE (which we did after trying for 10 years, we started in 1995 and got it in 2005). I cannot forget the EGM called by 60 HKIE members passing a resolution to withdraw the HKIE's President letter of support to Secretary of Works for RGE.

All those who resisted geotechnical engineers' development effort would now agree that there is no ground for their concern.. The establishment of a distinct geotechnical community in Hong Kong has not caused a harm to any group of people and had not taken away anything from any other engineering sector. On the contrary, the growth and development of the geotechnical community has strengthen the HKIE, benefit the public by enhancing the technology and professional practice of geotechnics in the territory. The growth of our geotechnical community is the model of many South East Asian countries and the mainland. I think the relentless effort of Chairmen and Committees of past years, as well as all the geotechnical engineers in HKSAR had contributed in achieving growth of the geotechnical community, enhancement in practice and high status in international standing.

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20th Anniversary –
Founding of the HKIE Geotechnical Division

正德利用厚生惟和

Our heartfelt congratulations to the 20th Anniversary of the HKIE Geotechnical Division.

The setting up of the Geotechnical Group in 1980, the Geotechnical Division in 1986, the Geotechnical Discipline in 1988 in HKIE and the Register of Professional Engineers (Geotechnical) under the Engineers Registration Ordinance in 1990 marked the formal recognition of valuable contributions of Geotechnical Engineers in the construction industry. Recently, the establishment of the Register of Geotechnical Engineers under the Buildings Ordinance in December 2005 is another step forward for Geotechnical Engineers to take up a pivotal role in enhancing the quality and safety of geotechnical works in the private sector.

Let's work together to bring Geotechnical Engineering practice in Hong Kong to another new era.

水火金木土穀惟修

(John Bruce MASSEY)
Chairman, 1993/1994

(CHAN Kin Sek, Raymond)
Chairman, 1995/1996

(CHAN Yun Cheung)
Chairman, 1998/1999

(PANG Pui Loi, Richard)
Chairman, 2001/2002

(MAK Shu Hei)
Chairman, 2004/2005

興土木 利民生 齊拓展 創明天

We bring the best engineering to life

Congratulatory Message

As a Past Chairman of the Geotechnical Division, I am very pleased to see that the Geotechnical Division has been growing from strength to strength since it was first established as the HKIE Geotechnical Group in 1980 and then as the HKIE Geotechnical Division in 1986.

The development and advancement of the geotechnical profession in Hong Kong in the last 30 years can be briefly divided into two stages. The development and advancement in the period from 1970s to the early 1990s were mainly driven by the need of safe slopes, and from early 1990s to present day driven by the requirements of large infrastructure projects (e.g. Chek Lap Kok Airport and associated road and rail networks) as well as the need of safe and green slopes.

This year the theme of the Geotechnical Division Annual Seminar is "Geotechnical Advancements in Hong Kong since1970s". The seminar will have a variety of papers to present the wide range of works undertaken by the geotechnical profession in Hong Kong in the last 30 years. I wish everyone of you would find the seminar useful and inspiring.

Ir Y C Koo
Past Chairman (1996 to 1997)
Geotechnical Division, HKIE

Acknowledgements

The Organizing Committee would like to express sincere thanks to the following sponsors for their generous support of the Seminar:

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Piling Design for Hong Kong – an External Viewpoint

H. G. Poulos
Coffey Geotechnics, Sydney Australia, & University of Sydney

Abstract: This paper presents an external perspective of the methods of pile design in Hong Kong and reviews both prescriptive methods and methods based on engineering principles. Comments are offered on some of the limitations of prescriptive methods and some of the design practices which are commonly employed. An example is described in which the results of prescriptive design procedures are compared with those employing methods which make use of a rational design procedure and which incorporate the results of recent research into pile behaviour in Hong Kong conditions.

1 INTRODUCTION

Piling has been used very widely in Hong Kong to provide support for buildings and bridges over many decades. It has been customary to design piles as end bearing piles that derive most, if not all, of their resistance from end bearing on relatively unweathered rock. This approach has often led to the use of very long piles whose length may exceed 100 metres or more. Moreover, the complex nature of the geology of Hong Kong has led to the adoption of relatively conservative parameters for pile design, and this conservatism has persisted, despite the considerable amount of research that has been undertaken to explore the behaviour of piles in Hong Kong soils and to develop more appropriate shaft friction and end bearing parameters for pile design.

This paper will present an external perspective of the methods of pile design in Hong Kong. Both prescriptive methods and methods based on engineering principles are considered and some of the shortcomings of the former are discussed. An example will be described which compares the results of conventional prescriptive design procedures with those employing methods which make use of a rational design procedure and which incorporate the results of recent research into pile behaviour in Hong Kong conditions. Attention will be focussed on the design of conventional piles under axial loading.

2 PRESCRIPTIVE DESIGN PROCEDURES

Pile design practice in Hong Kong has been dominated for many years by the recommendations of the Buildings Department (BD) and its forerunners. A key design document has been PNAP 66 which prescribes safe end bearing pressures for bored piles on various grades of rock and which provides a series of design stipulations, including the requirement that every pile within a group of piles shall have a computed factor of safety of at least 2 against bearing capacity (geotechnical) failure. Recently, this document has been revised (PNAP 2004), and this document now refers to the Code of Practice for Foundation Design (CPFD) which was published in 2004 (HKBD 2004). Current pile design in Hong Kong appears to now be largely influenced by this pair of documents.

Table 1 summarizes the BD presumptive design parameters for piles bearing on rock. In using the presumptive design method, the following stipulations are made:

1 The minimum socket length is 0.5m for categories 1(a) and 1(b), and 0.3m for categories 1(c) and 1(d).
2 Negligible lateral loads exist at bearing level.
3 total core recovery should be proved to a depth of at least 5m into the specified rock category.
4 The use of the prescriptive values does not preclude the requirement for consideration of the settlement of the structure.

Table 1. Summary of BD presumptive pile design values

Category	Description	Allowable end bearing capacity kPa	Allowable skin friction kPa
1(a)	Grade I rock	10,000	700
1(b)	Grade II rock	7,500	700
1(c)	Grade III rock	5,000	700
1(d)	Grade IV rock	3,000	300
2	Intermediate (SPT $N \geq 200$)	1,000	-
3a	Non-cohesive soil – v. dense (SPT $N \geq 50$)	250 *	-
3b	Non-cohesive soil – dense (SPT = 30-50)	250 *	-
3c	Non-cohesive soil – medium dense (SPT = 10-30)	50 *	-
3d	Non-cohesive soil – loose (SPT = 4-10)	< 50 *	-
4a	Cohesive soil – v. stiff (s_u >150 kPa)	300	-
4b	Cohesive soil – stiff (s_u = 75-100 kPa)	150	-
4c	Cohesive soil – firm (s_u = 40-75 kPa)	80	-

* Values shown are for submerged soils. For dry soils, the values can be doubled.

In addition to the use of the values shown in Table 1, there are a number of other requirements that are usually followed, including the use of the "45-degree rule" which typically is expressed as follows (Pickles *et al.*, 2004): "If additional loads are transmitted to the adjacent piles due to the 45 degree load spread and the total pile loads exceed the allowable bearing capacity of the adjacent piles, the founding levels for the bored piles shall be founded at the level outside the 45 degree load spread from the toe of the other bored piles".

From an external perspective, the following comments may be made about the use of prescriptive pile design:

Its use facilitates acceptance of the foundation design and thus it is efficient from the point of view of time.

It generally provides a very conservative foundation solution, and so is not efficient from the point of view of direct foundation cost and may also be inefficient from the viewpoint of time because of the need for additional piling.

Its use may inhibit innovation and may also engender an unwarranted degree of confidence in the foundation outcomes, regardless of the construction techniques employed. Such confidence may have played some role in the piling difficulties which have beset Hong Kong over the past 10 years or so.

3 DESIGN VIA ENGINEERING PRINCIPLES

The CPFD allows the use of rational design procedures for calculating the ultimate capacity of pile foundations, and requires a sound engineering approach which should include:

1 The reasonable interpretation of the results of the site investigation;
2 An assessment of the test results obtained from in-situ or laboratory testing;
3 An analysis based on the laws of physics and recognized engineering principles, taking into account the ground conditions and foundation geometry, or else an established empirical method proven with adequate correlation.

Valuable information on pile performance and design in Hong Kong conditions has been presented by many authors, including Davies & Chan (1981), Ng *et al.* (2000a, 2001a,b,c, 2003), Pratt *et al.* (2000), Littlechild *et al.* (2000), Lo & Li (2003), Yang *et al.* (2006) and Zhang *et al.* (2006).

A document which encapsulates such principles and design methods has been published by the Hong Kong Geotechnical Engineering Office GEO (1996). Very recently, a revised document has been released which summarizes the principles of pile design and incorporates the results of a considerable amount of research that has been undertaken in Hong Kong and elsewhere (GEO 2006). This document can be considered to contain information which is state-of-the art and which facilitates the design of piles based on sound engineering principles, rather than stipulating a series of prescriptive design rules.

Among the issues that are covered in GEO (2006) are the following:

– Ultimate Skin friction
– End bearing
– Settlement
– Group effects
– Negative skin friction
– Piled raft foundations.

Brief consideration is given to each of these issues below.

3.1 *Ultimate skin friction*

GEO (2006) considers cohesionless soils and cohesive soils separately and provides suggestions for both driven and bored piles. The effective stress (β) method is recommended for cohesionless soils, with the ultimate skin friction f_s being given as:

$$f_s = \beta \cdot \sigma_v' \quad (1)$$

where σ_v' = vertical effective stress
β = shaft resistance coefficient.

Typical ranges of values of β are shown in Table 2 for various soil types and methods of pile installation. Users are warned that the effects of bored pile construction may have adverse effects, and that design assumptions should be verified via load tests on instrumented piles. However, approval of load test has to be obtained from BD and this normally takes up to 3 months or so. This time delay effectively discourages engineers from using a process of design based on engineering principles. Thus, the use of the prescriptive design approach will normally allow approval from BD to be obtained more easily and quickly, and therefore, very few engineers are willing to adopt foundation designs based on engineering principles.

For piles in cohesive soils, either an effective stress or a total stress approach can be employed. In the former case, consideration should be given to the effects of the pile construction process on the value of β in Equation 1.

For the total stress approach, the ultimate skin friction f_s is given by:

$$f_s = \alpha \cdot c_u \quad (2)$$

where α = adhesion factor
c_u = undrained shear strength.

Figure 1 shows recommended values of α as a function of undrained shear strength c_u. The line denoted by Nowacki *et al.* (1992) is applicable to bored piles, while the equations from the API method are recommended for driven piles.

Table 2. Typical Values of β for Piles in Cohesionless Soils

Pile type	Soil type	β
Driven small-displacement piles	Saprolites	– 0.4
	Loose-medium dense sand	0.1 – 0.5
Driven large-displacement piles	Saprolites	0.8 – 1.2
	Loose-medium dense sand	0.2 – 1.5
Bored piles & barrettes	Saprolites	– 0.6
	Loose-medium dense sand	– 0.6
Shaft grouted bored piles & barrettes	Saprolites	0.2 – 1.2

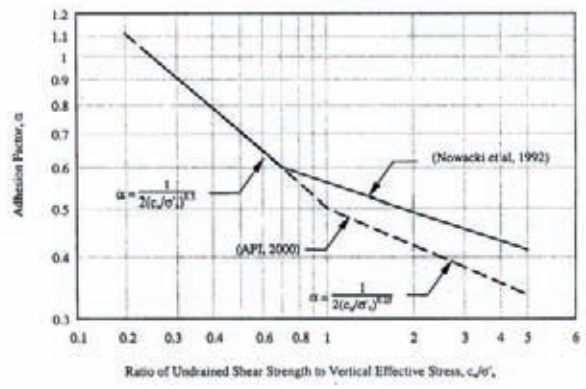

Fig. 1. Design Values for Adhesion Factor α for Piles in Clay

For rock socketted piles, on the basis of pile load tests in Hong Kong, f_s can be related approximately to the unconfined compressive strength of the rock, σ_c, as follows:

$$f_s = 0.2\ (\sigma_c)^{0.5} \quad (3)$$

Ng *et al.* (2001b) have presented the results of field test which relate the skin friction at a settlement of 1% of the shaft diameter to the rock weathering grade. This relationship is reproduced in Figure 2.

Finally, f_s can be correlated very approximately with the SPT-N value, and Table 3 summarizes ranges of the ratio of f_s / N, where N = uncorrected SPT. The values in Table 3 are lower than those reported for some other soils, but are based on back-calculations from pile load test data in Hong Kong, and are thus considered to be applicable to those soils. Caution should be exercised when using these relationships for large values of SPT.

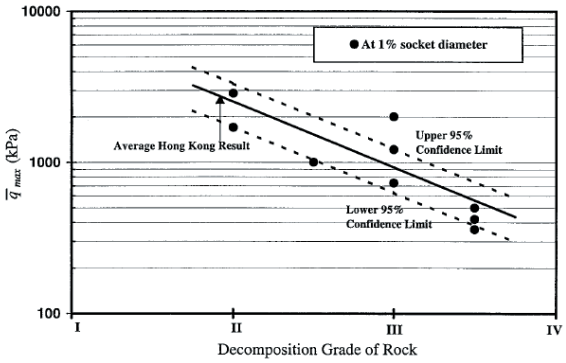

Fig. 2. Shaft Resistance versus Rock Grade for Hong Kong Granitic and Volcanic Rocks (Ng *et al.*, 2001b)

Table 3. Typical Ranges of f_s / N for Piles in Hong Kong Soils

Pile type	Soil type	Range of f_s / N values (kPa)
Bored	Sandy granitic saprolites	0.8 – 1.4 ($N \leq 200$)
Driven small-displacement	Sandy granitic saprolites	1.5 – 2 ($N \leq 80$)
Driven precast prestressed concrete	Sandy granitic saprolites	4 – 7 ($f_s \leq 250$ kPa)
Large displacement piles	Saprolites	4.8 ($N \leq 60$)
Post-grouted minipiles	Saprolites	5 ($N \leq 100$)

For piles subjected to uplift, the ultimate skin friction is often less than that for compression, because of the Poisson effect arising from the "stretching" of the pile. De Nicola & Randolph (1993) provide an excellent basis for assessing the ratio of ultimate skin friction in uplift to that in compression. In the absence of information that is required in this approach, it is suggested that a reduction factor of 0.75 be applied for piles in sand or rock, while a factor of unity can generally be adopted for piles in clay (unless the clay is very stiff). For pile sockets with shallow embedment into rock, consideration should be given to the mechanism of cone pullout, although this mechanism is likely to be most relevant when there is little or no overburden above the rock surface.

All the above methods of assessing the ultimate skin friction appear to be consistent with common practice world-wide and have the advantage that they incorporate the results of load tests carried out in Hong Kong. As with all design methods, appropriate engineering judgement needs to be used when selecting design values of f_s.

3.2 *Ultimate end bearing pressure f_b*

GEO(2006) adopts the approach suggested by Poulos and Davis (1980) for piles in cohesionless soils, in which the ultimate end bearing capacity f_b is given by:

$$f_b = N_q \cdot \sigma_v' \quad (4)$$

where N_q = bearing capacity factor which is a function of effective stress friction angle ϕ', and is shown in Fig. 2,
σ_v' = vertical effective overburden stress at level of pile toe.

If ϕ_1' is the in-situ effective friction angle of the soil (in degrees, prior to installation), then it is suggested that ϕ' (in degrees) in Fig. 3 be estimated as follows:

For driven piles: $\phi' = 0.5(\phi_1' + 40)$ (5a)

For bored piles: $\phi' = \phi_1' - 3$ (5b)

Alternative approaches have been proposed by Fleming et al (1992) for piles in sand, in which the stress-dependency of frictional and volumetric characteristics of the sand are taken into account.

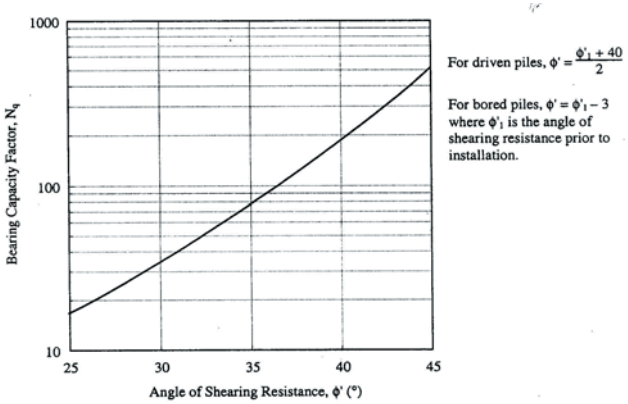

Fig. 3. Factors for End Bearing Capacity

For piles in cohesive soils, it is customary to relate f_b to the undrained shear strength in the vicinity of the pile toe c_{ub}, as follows:

$$f_b = N_c \cdot c_{ub} \quad (6)$$

where N_c = bearing capacity factor.

For a pile of length L and diameter d, N_c can be approximated as follows:

$$N_c \approx 6 + L/d \qquad \leq 9 \quad (7)$$

For bored piles founded in rock, a number of studies have indicated that f_b may have very high values and may be several times the unconfined compressive strength, q_c. Zhang & Einstein (1998) have reviewed a considerable amount of data and have suggested the following the following approximation:

$$f_b \approx 4.8(q_c)^{0.5} \text{ MPa} \quad (8)$$

Alternative approaches for estimating the allowable end bearing pressure are presented by GEO (2006). These approaches require consideration of the defects within the rock mass.

3.3 *Settlement*

The settlement of a single pile can be estimated by various methods, including load transfer methods, elastic continuum theory and numerical methods (finite element or finite difference). An advantage of elastic continuum methods is that they do not necessarily require a complex computer program, but can be evaluated by hand, by spreadsheet or by a mathematical program such as MATHCAD. Use can be made of charts such as those presented by Poulos & Davis (1980) or of the closed-form solutions of Randolph & Wroth (1978). The latter equations are reproduced in GEO (2006).

The main challenge in using these methods (and indeed any of the other available methods) is in assessing the relevant deformation parameters of the soil, in particular the Young's modulus E_s or the shear modulus G_s. These values depend on soil type, the method of pile installation and the level of stress or strain, and it is common to employ empirical correlations with such parameters as SPT-N or undrained shear strength c_u. While a number of correlations have been proposed, it is prudent to employ local correlations for soil types and geological conditions relevant to the project in question. Table 4 summarizes some correlations, some of which are employed commonly in Hong Kong, for the drained (long-term) Young's modulus E_s' for axial loading. For lateral loading, smaller values of E_s' are generally relevant, because of the larger soil strains developed by lateral loading. The author commonly adopts a reduction factor of 0.7 to allow for this effect.

Table 4. Typical Correlations for Drained Young's Modulus E_s' for Axially Loaded Piles

Pile type	Soil type	Correlation	Remarks
Bored	Saprolites, sandy soils	E_s'/N = 0.8 to 1.2 MPa	For preliminary design
Driven	Granitic saprolites	E_s'/N = 3.5 to 5.5 MPa	Based on field load tests
Bored	Clays	$E_s' = 200\ c_u$	Lower bound value
Driven	Clays	$E_s' = 400\ c_u$	Average value
Bored	Rock	$E_s' \approx 0.06e^{0.05RMR}$ (in GPa) (RMR=Rock Mass Rating %)	Conservative relation, based on field data for RMR between 15% and 80%

It should be noted that the shear modulus G_s of decomposed materials is highly non-linear, even at small strains (Ng *et al.*, 2000; Ng & Leung, 2006). Thus, an initial estimation of potential strain level to be developed may be a useful starting point to assist in engineering design or to verify empirical prescriptive design values.

3.4 *Group effects*

There are three main effects that need to be considered when designing axially loaded pile groups:

1 There may be a potential for a reduction in group capacity when piles are closely spaced, because of the development of the "block" mode of failure. This tends to occur mainly for friction pile groups in clay soils.
2 If a moment acts in addition to the vertical loading, there may be a potential for the group capacity to be adversely affected and for some of the piles to be in tension.
3 There is likely to be an increase in settlement due to interaction among the piles through the soil (sometimes termed "pile-soil-pile interaction".

It is common to consider the efficiency η of a pile group as the ratio of capacity of the pile group to the sum of the capacities of the individual piles comprising the group. Table 5 summarizes some of the information available for estimating group efficiency.

Where there is a significant moment and lateral load acting on the pile group, the author has employed a numerical non-linear pile group analysis, DEFPIG (Poulos 1980) to assess the overall stability of the pile group. For conventional factor of safety design, the vertical load, lateral load and the moment are increased (assuming a constant ratio of loads and moment) until the pile group becomes unstable. This load combination can then be related to the applied loads to assess the factor of safety against failure. For limit state design, the pile capacities in compression and uplift are reduced by an appropriate geotechnical reduction factor (typically 0.5 – 0.7) and then the ultimate limit state combinations of load and moment are applied. The group is deemed to be stable (and therefore adequate) if the analysis indicates that the group does not collapse.

In the design of pile groups, it has been customary in Hong Kong, and also in some other countries, to apply a factor of safety against axial (geotechnical) failure of each individual pile within the pile group. Most methods of pile group analysis indicate that the load in the corner piles is greater than the load in the inner piles of the group, due to interaction effects. The customary design approach responds to this situation by increasing the capacity of the outer piles or by increasing the number of piles to reduce the pile loads. The author believes that such the traditional approach of designing for a specified factor of safety for each pile within the group may be inappropriate and excessively conservative, and GEO (2006) also agrees, stating: "It is incorrect to design for this load re-distribution by increasing the capacity of the outer pile in order to have the same factor of safety as a pile loaded singly". There will inevitably be some load redistribution to the inner piles if the load on the outer piles tends towards the ultimate value, with a corresponding increase in settlement. The key factor of importance is the overall load capacity of the group, rather than the load capacity of each individual pile, and the fact that one or more piles may reach the geotechnical ultimate capacity will not be significant if the following conditions are satisfied:

1 The group as a whole has an adequate factor of safety against failure;
2 Each individual pile has an adequate factor of safety against structural failure;
3 The settlement of the pile group is within the allowable limit.

Table 5. Recommendations for Pile Group Efficiency Estimation

Case	Group efficiency η	Remarks
Driven piles in loose to medium dense sand	1.0	η may be considerably greater than 1: adopt 1 for design
End-bearing piles on rock, dense sand, or gravel	1.0	Base resistance is not much effected by group action, even at small openings (Meyerhof 1976)
Bored friction piles in sand	0.67	For "customary spacings": i.e. 3 ± 1 diameters (Meyerhof, 1976)
Friction piles in clay - cap above surface	Lesser of $P_B/\Sigma P_u$, or 1.0	Terzaghi & Peck (1967). Make allowance for any soft layers below base.
	$1 / \{1+(\Sigma P_u / P_B)^2\}$	Poulos & Davis (1980). Make allowance for any soft layers below base

Note: P_B = ultimate load capacity of block containing piles and soil
ΣP_u = sum of ultimate capacities of individual piles.

For rapid practical estimation of group settlements without recourse to a computer, there are at least three convenient methods which may be employed: the Settlement Ratio method, the equivalent raft method, and the equivalent pier method.

In the Settlement Ratio method, the group settlement S_G is related to the single-pile settlement as follows:

$$S_G = R_s \cdot S_{iav} \tag{9}$$

where S_{iav} = settlement of single pile at the average load of a pile in the group, and R_s = settlement ratio. S_{iav} can be estimated either by calculation (e.g. via elastic theory) or from the results of a pile load test on a prototype pile.

Theoretical values of R_s for various pile groups in homogeneous and non-homogeneous soil profiles have been presented by Poulos (1977a, 1979b) and Butterfield and Douglas (1981). A particularly useful approximation for the settlement ratio has been derived by Randolph (Fleming *et al.*, 1992):

$$R_s \approx n^w \tag{10}$$

where n = number of piles in the group, w = exponent depending on pile spacing, pile proportions, relative pile stiffness, and the variation of soil modulus with depth. For typical pile proportions and pile spacings, Poulos (1989) has suggested the following approximate "rules of thumb": $w \approx 0.5$ for piles in clay, and $w \approx 0.33$ for piles in sand.

The equivalent raft method suggested by Tomlinson (1986) involves the representation of the pile group by a raft located at an appropriate depth below the surface, while the equivalent pier method (Poulos & Davis, 1980) involves the replacement of the pile group by an equivalent pier consisting of the piles and the soil between them. Poulos *et al.* (2002) demonstrate that both of the latter approaches are capable of providing adequate practical estimates of group settlement.

3.5 *Negative friction*

Negative skin friction is an aspect of pile design that often causes considerable confusion. For example, it is sometimes considered to cause a reduction in pile capacity, whereas in reality, the geotechnical capacity of the pile is unlikely to be affected. Two important effects do however arise from the "drag-down" effect of the soil settling past the pile:

1 There is an increase in axial load within the pile, which has the potential to compromise the structural integrity of the pile;
2 There is an increase in settlement of the pile.

Because of the latter phenomena, it is often necessary to reduce the allowable load on the pile, in order to keep the axial stresses adequately below the strength of the pile and/or to limit the settlement of the pile.

The downdrag force is generally computed from an effective stress approach, for example, as summarized by GEO (2006), while the additional settlement can be estimated as set out by Poulos (1997). For piles within a group, the maximum downdrag force induced by negative skin friction is generally decreased as compared with a single isolated pile. GEO (2006) recommends applying a reduction of 10-20% to the single pile downdrag force for a pile group consisting of at least 5 piles at "customary" spacing (presumably 2.5 – 4 diameters centre-to-centre).

Lee & Ng (2004) have undertaken finite element analyses to examine group effects of downdrag force development in piles. Fig. 4 shows the computed distributions of downdrag force in a single pile and in piles within a 25-pile group. In the latter, position a is at the corner, position c is at the centre and position b is half-way between the centre and the corner. The reduction in downdrag force due to group effects, especially for piles near the centre, is clearly demonstrated in this figure.

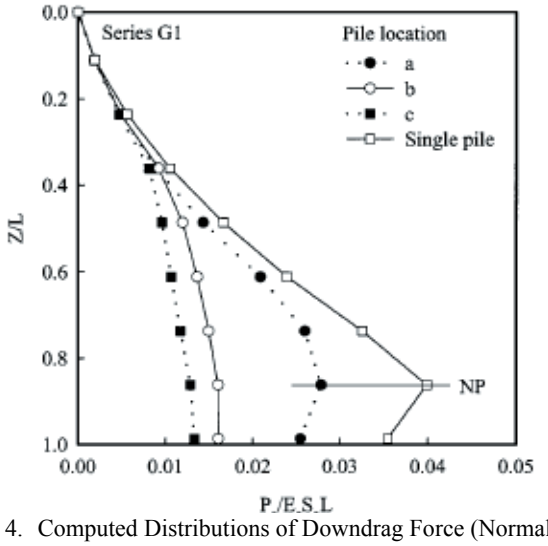

Fig. 4. Computed Distributions of Downdrag Force (Normalized) for Single Pile and 25-Pile Group (Lee & Ng, 2004)

As a means of practical design, the author has found that both settlement and capacity requirements can generally be satisfied if the portion of the pile within the "stable" zone (i.e. that portion of the pile which is not settling) is designed to have a factor of

safety against axial failure of the order of 1.25 against the action of the applied vertical loads (dead plus live) plus the downdrag force in the pile at the top of the "stable" zone.

3.6 *Piled raft foundations*

Piled raft foundations involve the use of both the piles and the raft to provide stiffness and load capacity for the combined foundation system. They have been used extensively in some countries to provide an economical and effective foundation system, especially when the raft can be demonstrated to provide a reasonable amount of load capacity and stiffness. For example, many high-rise buildings in Frankfurt are supported by piled raft foundations, because the subsurface conditions consist of stiff clay, while buildings in the United Arab Emirates and in the Gold Coast of Queensland Australia are underlain by relatively dense and/or cemented sand deposits which provide very suitable ground conditions for piled rafts. They may also provide a useful solution to the problem of low-rise buildings on very soft clays, for example, as demonstrated by Tan *et al.* (2004, 2005).

Some methods of analysis have been suggested by Poulos (2001), while three dimensional finite element methods have also been used extensively by Katzenbach *et al.* (1998, 2000). An issue which concerns some foundation designers is that, if the ground supporting the piled raft settles, the soil will move away from the raft and the entire load will then be carried by the piles, thus causing additional settlements. While this is a valid concern, it need not necessarily rule out the use of piled rafts as the piles within the foundation system can be designed to withstand the loads without excessive settlement, while the raft provides an additional reserve of resistance to the piles. It must be borne in mind that, if there is a tendency for excessive settlement of the piles, then the raft can re-contact the soil and the raft will then become operative again. Thus, as long as the settlement, differential settlement and ultimate capacity of the foundation system can be shown to be adequate, it is possible to design a piled raft system even in settling ground conditions. At the very least, taking account of the raft capacity can reduce the number of piles that are required.

Table 6. Assumed Parameters for Typical Soil Profile

Material	Thickness	Average SPT-N	Average c_u	f_s	f_b	E_s
	m		kPa	kPa	kPa	MPa
Fill	8	15	-	15	-	15
Marine Deposit	5	-	10	10	-	1
Alluvium	10	20	-	20	-	20
EW Rock (V)	17	-	-	200	-	200
SW Rock (II)	Large	-	-	500	22.5	500

4 EXAMPLES OF THE CONSEQUENCES OF PRESCRIPTIVE VERSUS ENGINEERING DESIGN

In order to examine the possible consequences of using a prescriptive approach, rather than a modern engineering principles approach to design, two typical cases will be examined below:

- A building founded on driven steel H-piles
- A building founded on large diameter bored piles.

Fig. 5 illustrates the ground conditions, which are typical of some sites in Hong Kong, and which are assumed to be the same in each case. The geotechnical parameters are shown in Table 6.

RL at bottom of geo-model (m)	Description of Geo-Unit	E_v (MPa)	E_h (MPa)	f_s (MPa)	f_b (MPa)
-8	Fill	15	10.5	.015	-
-15	Marine Deposit	1	0.7	0.01	-
-23	Alluvium	20	14	0.02	-
-40	Extremely Weathered Granite (V)	200	140	0.2	15
-60	Slightly Weathered Granite (II)	500	350	0.05	22.5

Fig. 5 Assumed Ground Profile for Pile Design Examples

The building footprint is illustrated in Fig. 6, and is a simplified representation of a typical high-rise building in Hong Kong. To simplify the discussion here, only dead and live loads will be considered, and these loads are assumed to be as follows:

Dead load = 420 MN

Live Load = 84 MN.

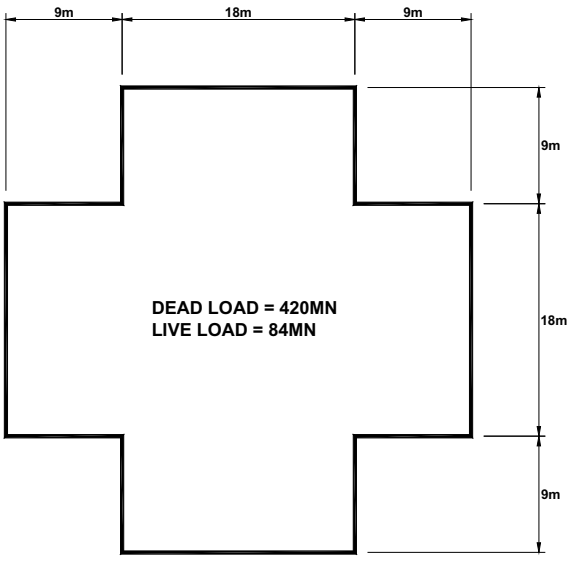

Figure 6 High-Rise Building Footprint for Pile Design Example.

4.1 *Case 1: Steel H-Pile Foundations.*

In this case, the foundation elements are 305×305×186 steel piles. Using the parameters in Table 6, and assuming that the steel H-pile operates as a square pile incorporating the soil between the flanges, the following capacities are computed:

Ultimate structural capacity = 5925 kN (assuming 250 MPa steel yield strength)

Ultimate geotechnical capacity = 7130 kN.

If a factor of safety of 2 is adopted for both structural and geotechnical strengths, then the structural strength governs and the allowable load per pile is 2960 kN.

Considering first the prescriptive design approach, a group reduction factor of 0.85 needs to be applied. The number of piles necessary to resist the dead plus live loads is then:

$n_1 = (420 + 84)/(0.85*2.96) = 200.$

Consider next the rational design approach. The governing capacity is again the structural capacity of 2960 kN, and it may be argued that no group reduction factor is necessary, both because of the nature of the soil deposits into which the piles are driven, and also because the pile capacity is governed by structural, rather than geotechnical, considerations. The necessary number of piles is then:

$n_2 = (420 + 84) / 2.96 = 170.$

Thus, the use of the rational design method allows a reduction in the number of piles from 200 to 170.

A group settlement analysis using the program DEFPIG shows that the average settlement of the groups is as follows:

200 pile group: 32.5 mm

170 pile group: 35.5 mm.

Thus, in this case, the use of a rational design approach leads to a group with 15% less piles than the prescriptive design, and which settles only 3mm more than the 200 pile group obtained from the prescriptive design.

It should be noted that, in this example, the design has not involved the requirement that the maximum computed pile load should not exceed the allowable value of 2.96 MN. If this requirement is imposed, then in the case of the prescriptive design, the necessary number of piles rises from 200 to about 300. The computed settlement reduces to about 26 mm, so that compared to the results of the rational design method, the extra 130 piles required leads to a reduction in settlement of less than 10 mm. It therefore seems clear that the imposition of a limit on individual pile loads within a large pile group leads to a design which is overly conservative and which performs little better than that described from a rational approach. A similar conclusion can be derived from the work of Mandolini & Viggiani (1997).

4.2 *Case 2: Bored Pile Foundations*

It will be assumed that the bored piles have a shaft and base diameter of 1.2m and that they will be socketted 1 m into the slightly weathered (Grade II) granite, and that the nominal concrete strength is 45 MPa. If an allowable strength of 0.4 times this value is used, with a further factor of 0.80 being applied for the concrete being below the water table, the allowable structural strength is $45.0 \times 0.4 \times 0.8 \times \pi \cdot 1.2^2/4 = 16.286$ MN.

Considering first the prescriptive design, and ignoring the friction along the socket shaft, the allowable end bearing capacity on the Grade II rock is 7500 kPa. The allowable load per pile is then $7500 \cdot \pi \cdot 1.2^2 /4 = 8480$ kN = 8.48MN. This is less than the allowable structural capacity of 16.286 MN, and so geotechnical failure governs the design. A group efficiency factor of 0.85 is again applied, so that the necessary number of piles is:

$n_3 = (420 + 84) / 8.48 \times 0.85 = \underline{70}.$

If the traditional approach is adopted in conjunction with the prescriptive approach, whereby each pile in the group has to have a load which does not exceed the allowable geotechnical load, then it may be necessary to use more than 70 piles. The number of piles will depend on the approach taken to compute the distribution of load within the group. If a simple "rivet group" approach is taken in which pile-soil-pile interaction is ignored, then (ignoring wind and seismic loadings), each pile is equally loaded and so 70 piles are required. If, however, use is made of a pile group analysis such as DEFPIG, then it is found that a significantly larger number of piles is required. Via a process of repeated calculation, it has been found that a total of about $\underline{101}$ piles would be needed to reduce the maximum load within a pile in the group to the allowable value of 8.48 MN. The DEFPIG analyses indicate the following computed average settlements:

70 piles: 27.6 mm

101 piles: 24.5 mm.

Thus, the use of an extra 31 piles (44% more than the original design) leads to a reduction in settlement of only about 3 mm.

The rational approach will now be considered.

Using the parameters in Table 6, the following capacities of each pile are as follows:

Ultimate geotechnical capacity: 41,540 kN.

If a factor of safety of 2.5 is employed in this case, the allowable geotechnical capacity is 16,616 kN. This is slightly larger than the allowable structural capacity of 16.286 MN, and so the latter value governs the design. If the group reduction factor is again not applied because of the nature of the soil profile, and also because the structural strength is the governing factor, the required number of piles is now:

$n_4 = (420+84)/16.286 = \underline{31}.$

It was found necessary, because of the configuration of the building, to have 33 piles, and the DEFPIG analysis for this case indicates that the average settlement is now 38.5 mm.

Table 7 summarizes the results of the comparative analyses carried out, and the following conclusions can be drawn:

1 The overall factor of safety against geotechnical failure for the prescriptive designs is much larger than is normally considered economical in foundation engineering practice.
2 The imposition of the requirement that all piles in the group be designed to have a specified safety factor leads to an even less economical design (an extra 31 piles in this case).
3 The reduction in settlement with the use of the extra 31 piles is only about 3mm, a very poor reward for an increase of 44% in the number of piles.
4 The rational design method, which requires less than half the number of piles than the prescriptive design, would lead to an average settlement of about 39 mm which is generally quite acceptable for most buildings.

Table 7. Summary of Computed Performance of Bored Pile Groups

Design approach	Number of piles	Average settlement mm	Overall factor of safety against geotechnical failure
Prescriptive – no requirement for individual pile safety factor	70	27.6	5.77
Prescriptive – individual pile load does not exceed allowable	101	24.5	8.32
Engineering Principles – no requirement for individual pile safety factor	33	38.5	2.72

On the basis of this example, it may be concluded that, while the prescriptive design approach is certainly safe, it leads to an unnecessary degree of conservatism. This conservatism is compounded if the design further requires all piles within the group to have a specified factor of safety or to have computed loads which do not exceed the allowable value.

5 CONCLUSIONS

The prescriptive design procedure that is widely used in Hong Kong is convenient and reduces the number of decisions that the foundation designer must make. However, there is an economic "downside" to the use of this approach, in that there can be a significant increase in the number of piles required for the foundation and hence in the construction duration and programme. In the example considered, when steel H-piles are used, the differences in outcome between the prescriptive and rational design procedures are not great, and are a result of the application of a group reduction (or efficiency) factor, which is considered to be inapplicable in a rational design process because the pile design is governed by structural strength considerations. In the case of bored pile foundations, a greater difference is found between the outcomes of the prescriptive and the rational design approaches. If no requirement is placed on the computed load within the piles in the group, the required number of piles from the prescriptive design approach is still more than twice the number that would be assessed from the rational design approach. If a requirement is imposed that all piles must have a computed axial load which does not exceed the allowable value, then the required number of piles in the prescriptive approach is about three times that from the rational approach. The penalty paid by the use of the rational approach is a somewhat larger average settlement, but the computed value for the latter approach is still well within acceptable limits.

Prescriptive design methods have a useful place in the design of foundations for relatively low-rise buildings and lightly loaded structures, where economy of foundations is less important than the slightly greater element of risk involved in using a rational design method. However, for major buildings and structures, the situation differs, and it is likely that foundations which are designed by rational principles, and constructed with careful supervision by the designer, are likely to be significantly more economical than those designed by prescriptive methods which often pay little heed to actual mechanisms of behaviour.

Clearly, there are a number of areas in which research in Hong Kong could have a beneficial effects on pile design and which could increase confidence in the use of rational design procedures. These include:

1 Carefully controlled and monitored tests on model and full-scale piles and pile groups, which would enable further data to be obtained on pile skin friction, end bearing and stiffness.
2 A detailed study of the effects of construction imperfections on pile behaviour, and methods by which such imperfections can be controlled and reduced.
3 More detailed measurements on the settlements and differential settlements of buildings. This would enable an improved assessment to be made of the relationship between predicted performance and measured behaviour, and assist in developing more reliable procedures for estimating building settlements.
4 The potential for the use of piled raft foundations in Hong Kong, and the conditions under which this type of foundation can provide economical and effective support for high-rise buildings. They may also provide a useful solution to the problem of low-rise buildings on very soft clays, for example, as demonstrated by Tan *et al.* (2004, 2005).

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Engineering Geological Practice in Hong Kong

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Abstract: Engineering geology provides the link between geology and engineering through the gathering of engineering geological data that allows the formation of geological models which can be used to identify geological hazards and uncertainty, plan effective ground investigation, and define blocks of ground and geological structures in an engineering context to facilitate geotechnical risk assessment and design. The amount of engineering geological input required for a particular project varies depending on geological factors such as rock type, geological structure, weathering and geomorphology, as well as engineering considerations such as the type of scheme and the construction method adopted. Over the last 30 years or so, the importance of engineering geology to the timely, cost effective and safe completion of civil engineering works has been demonstrated. However, the need to improve engineering geological practice in Hong Kong has been recognised. Improvements in this regard will also enhance geotechnical practice and increase its long-term value to the society of Hong Kong.

1 INTRODUCTION

The ground in Hong Kong has the potential to be geotechnically complex as a result of geological variations. However, this complexity is rarely random or unpredictable, but is the result of genetic and process-related geological and anthropogenic factors that have contributed to the present-day ground conditions. Much of this complexity can be anticipated, identified, understood and quantified through the application of sound engineering geological principles. As only a tiny fraction of the volume of ground which will affect or be affected by the proposed works can usually be directly observed or tested during a site investigation, the risk of 'unforeseen ground conditions' has the potential to increase with geological complexity.

Chan & Kumaraswamy (1995) report that 'unforeseen ground conditions' was cited as the most significant factor in causing construction delays to civil engineering works in Hong Kong. Unforeseen ground conditions have also been cited as major factors in a number of large man-made slope failures in Hong Kong (Wong & Ho 2000; Ho et al. 2003). Two of the main contributing factors relevant to engineering geological practice were (i) the presence of "adverse geological features" and/or "adverse groundwater conditions", and (ii) the use of an over-simplified geological and/or hydrogeological model which does not adequately cater for safety-critical geological features in the ground.

Whilst over the last 30 years or so much local knowledge and experience with respect to engineering geological practice in Hong Kong has been documented, mostly in the form of conference proceedings and study reports, this information has not been systematically consolidated in a form suitable for dissemination to the geotechnical profession in Hong Kong. In late 2004, the Geotechnical Engineering Office (GEO) of the Civil Engineering and Development Department (CEDD) commissioned the preparation of "Engineering Geological Practice in Hong Kong" (GEO 2007) to provide a compendium of knowledge and experience, based primarily on a review of relevant literature and current practice to demonstrate the importance of engineering geology to the timely, cost effective and safe completion of civil engineering works. This paper summarises some of the key aspects presented in the document.

2 EXISTING GUIDANCE

Many of the geotechnical problems that affect Hong Kong are related to weathering processes. These problems are further compounded by structural weakness in the original rock being retained in the resulting soil mass, potentially giving rise to a very complex material. As a result many international standards for description and classification are not readily applicable to Hong Kong and the GEO (named GCO before 1991) has worked on producing more appropriate systems, which have subsequently been adopted internationally. A rock mass weathering scheme was developed for igneous rocks, to allow entire weathering profiles to be classified. In parallel with this a material classification scheme was developed, allowing the systematic description of materials and enabling typical engineering properties to be assigned on a reliable basis. This work cumulated in the publication of Geoguide 3 "Guide to Soil and Rock Descriptions" (GCO 1988).

Limestone and marble weather by dissolution, yet most rock mass classifications do not include degree of dissolution as a key parameter. A marble rock mass classification system, based on the Marble Quality Designation (MQD), has been proposed by Chan (1994) and Chan & Pun (1994) to facilitate the zoning of pure marble rock masses for interpretation of the dissolution process and assessment of suitability for foundations. The MQD uses two main parameters derived from drillhole records, namely, RQD (Rock Quality Designation) and marble core recovery ratio.

Other technical publications and Technical Guidance Notes (TGN) published by the GEO give guidance on standards for geotechnical engineering in Hong Kong, some of these also

cover engineering geological issues and practice. TGN 1 (GEO 2005a) provides a list of publications which are used as de facto standards. The TGNs are updated regularly, primarily in response to improvements in geotechnology, better understanding of local geological conditions, and geotechnical lessons learnt both in Hong Kong and elsewhere.

The importance of engineering geology in slope engineering and the need for improved assessment and design practices have been highlighted by many authors, e.g. Wong & Ho (2000), Campbell & Parry (2002), Ho et al. (2003) and Martin (2003). Most of the key issues highlighted emphasise the need for enhanced application of the core engineering geological skills of observation and interpretation to help produce realistic ground models to enable geotechnical hazards to be recognised so that they can be managed. However, these skills have their roots in developing an understanding of the Earth through long experience and are very difficult to codify in an effective manner due to their wide-ranging scope and partially implicit nature. Therefore, good engineering geological practice requires that the existing guidance and reference documents are interpreted and adapted as necessary in response to the site-specific conditions and requirements of the project at hand.

3 MODEL APPROACH

3.1 Introduction

Fundamental to good engineering geological practice is the need to systematically develop geological, including geomorphological and hydrogeological, models to facilitate engineering designs. These models should be updated continuously throughout the design and construction processes to increase awareness of potential geological uncertainties and geotechnical hazards, to facilitate checking and design verification, and to form the basis of geotechnical risk analysis and management frameworks that are becoming increasingly required by clients, contractors, and insurance underwriters for large projects.

In order to provide a framework for the input of engineering geological work, a three-step approach comprising ‘geological’, ‘ground’ and ‘design’ models, based on local and international recommendations is recommended. The degree to which these steps are applicable to a specific engineering project and the level of engineering geological input required will depend on the nature and scale of the engineering works and perceived geotechnical risks. However, the development of a geological model is the first step towards the assessment of geotechnical risks for most engineering projects.

3.2 Geological Model

The concept of geological models is not new. GCO (1987) states “Before commencing ground investigation, all relevant information collected should be considered together to obtain a preliminary conception of the ground conditions and the engineering problems that may be involved.” The importance of the geological model has been recognised as one of the key components of geotechnical design in BD (2003): “it is always a good practice to first formulate a preliminary geological model based on existing information obtained from a thorough desk study. The ground investigation fieldwork should then be planned with the objective of refining and confirming the geological model and the parameters to be used in the design, and identifying the various uncertainties involved as far as possible”.

The term “geological” model refers to a geological model that characterises the site, i.e. it focuses on geological, geomorphological and hydrogeological features and characteristics that are relevant to the engineering project (Fookes 1997). A site may for instance be geologically complex; however, this does not necessarily imply that it is also geotechnically difficult for the engineering application. The focus of the model will also depend on the nature of the project.

How this model is actually presented can vary depending on the complexity of the site and the nature of the works being undertaken. In its simplest form a geological model can be constructed from an interpretation of a geological map or a site reconnaissance (Fig. 1). It is good practice to continually refine and update the model during the ground investigation and construction phases, with reviews undertaken by suitably skilled personnel. Such reviews can reduce the possibility of errors and misinterpretations which could have an adverse impact on the relevance and effectiveness of the site investigation, design and construction methodology.

The geographical extent of the model will depend primarily on the type of proposed works and the hazards that may be relevant. For example, when considering landslides, the extent of the model may have to be widened to include nearby terrain with similar geomorphology. To assess the effects of tunnelling or deep excavation on hydrogeology, the extent of the model may also need to extend a considerable distance from the works site.

3.3 Ground Model

The ground model builds on the geological model and embeds the range of engineering parameters and ground conditions that need to be considered in the design (Knill 2002). The ground model refines the geological model by defining and characterising bodies of ground with similar engineering properties, and identifies boundaries at which changes in geotechnical conditions may occur. Engineering geological input assists in ensuring as far as practicable that the ground model reflects the ground conditions indicated by the geological model. Such input is useful in ensuring that stability-critical or performance-critical features such as faults, dykes, discontinuities and hydrogeological boundaries are considered and, if necessary, incorporated. This enables critical features to be targeted for more detailed ground investigation, testing and characterisation if necessary.

The ground model gives due consideration to the possible ranges of material and mass properties. Environmental factors such as the groundwater regime, contamination, in situ stress conditions, and qualitative estimates of the possible ground and groundwater response to the changes in environmental conditions imposed by the proposed works may also need to be considered.

The ground model should include plans and sections through critical areas to indicate the possible range of ground conditions. It should convey an understanding of these conditions, geotechnical hazards and areas of uncertainty that is commensurate with the nature of the proposed engineering works. For example, a ground model for a slope engineering project will need to focus on stability-critical features, while a ground model for a foundation engineering project will need to focus on features that will affect the type and design of foundations.

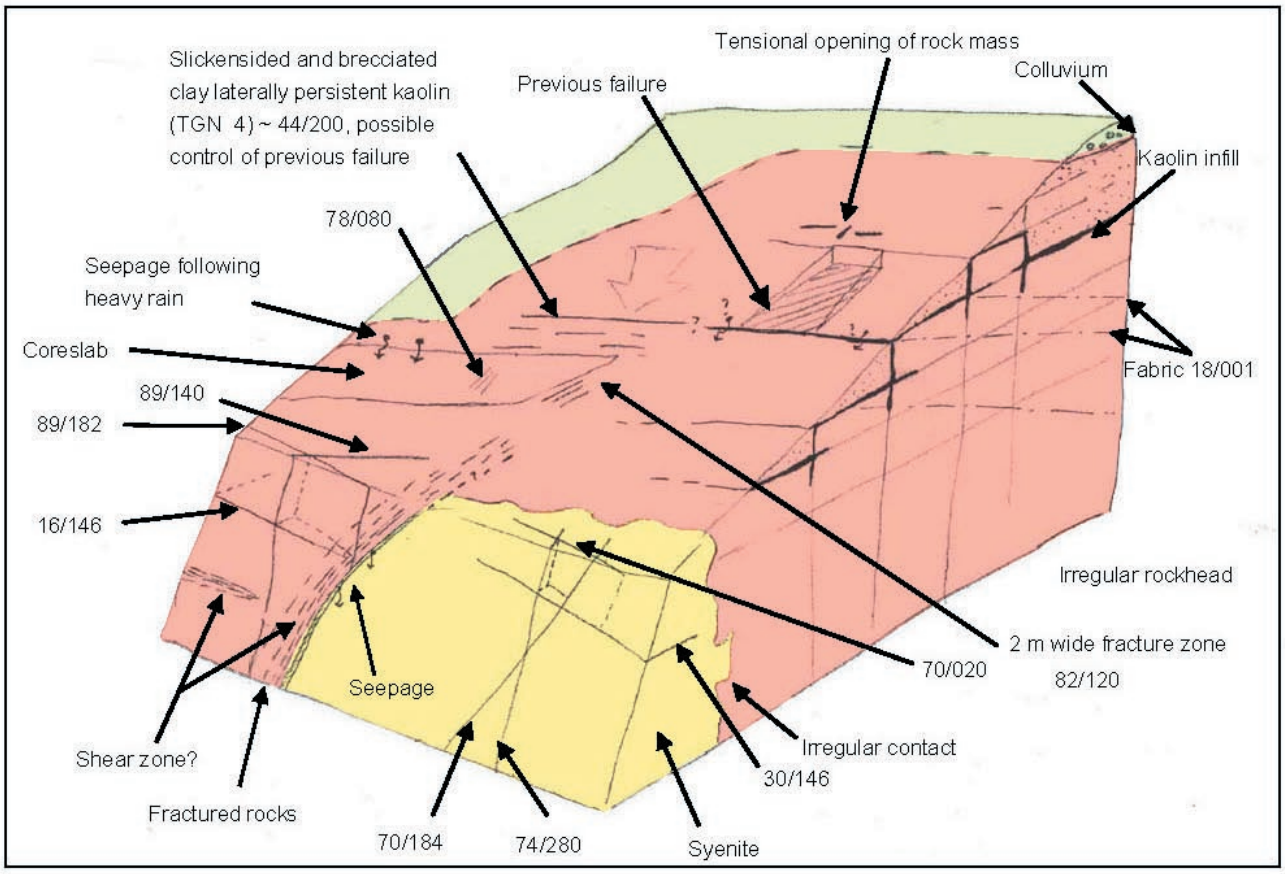


Fig. 1. Example of a geological model based on site reconnaissance (Parry et al. 2004).

For large projects where the basic details of the proposed works are known or can be adequately estimated, any geotechnical uncertainty can be incorporated into preliminary risk registers which can then be used during the design stage to target further investigations. These registers can be audited and traced by the design team throughout the rest of the investigation and design process as part of the overall risk management strategy. This approach can also be adapted to suit the needs of smaller projects, depending on the nature and consequences of the perceived risks.

3.4 Design Model

The design model is concerned primarily with assessment of the response of the ground to the proposed works and vice versa for use in geotechnical assessment or engineering design. Design models for empirical, prescriptive and quantitative designs depend on the engineering application, degree of conservatism in the empirical/prescriptive models and the level of geotechnical risk.

An example of an empirical design approach is the assessment of allowable bearing capacity for foundations on rocks based on presumed values derived from empirical correlation (BD 2004). In this case the ground model would typically comprise a series of plans and sections indicating the variations in decomposition grade and percentage of core recovery, based on the results of the ground investigations. The ground model could be used for preliminary purposes to identify the level at which the ground may satisfy the requirements of the foundation design.

The design of soil nailed slopes in accordance with Wong et al. (1999) provides an example of a prescriptive design approach. In this case, the geological models and ground models are first constructed to provide an initial check on whether the slope satisfies the geotechnical and geometrical qualifying criteria for the application of the prescriptive design methodology.

Unless the design is based on empirical or prescriptive approaches, some method of numerical analysis is required. Knill (2002) considers that the steps which need to be taken to convert a geological model, through the ground model, to the design model (i.e. Knill's "geotechnical model") will require refinement to meet the requirements of the selected method of engineering analysis. During the conversion, engineering geological input is essential to ensure that the actual conditions are represented as accurately as possible in the eventual analysis. The design model therefore incorporates and simplifies the main elements of the ground model so that a representative range of ground conditions can be defined for use within a suitable design framework.

3.5 Application

The typical development and application of the model approach for a major project is shown in Fig. 2. Although the chart depicts a linear progression from one activity to the next, there is normally considerable overlap and iteration in practice. Engineering geological input is particularly effective from the planning and feasibility stages, through to the stage when all site investigation data has been interpreted and incorporated into the design models. Engineering geological mapping of exposed ground during construction also assists in confirming

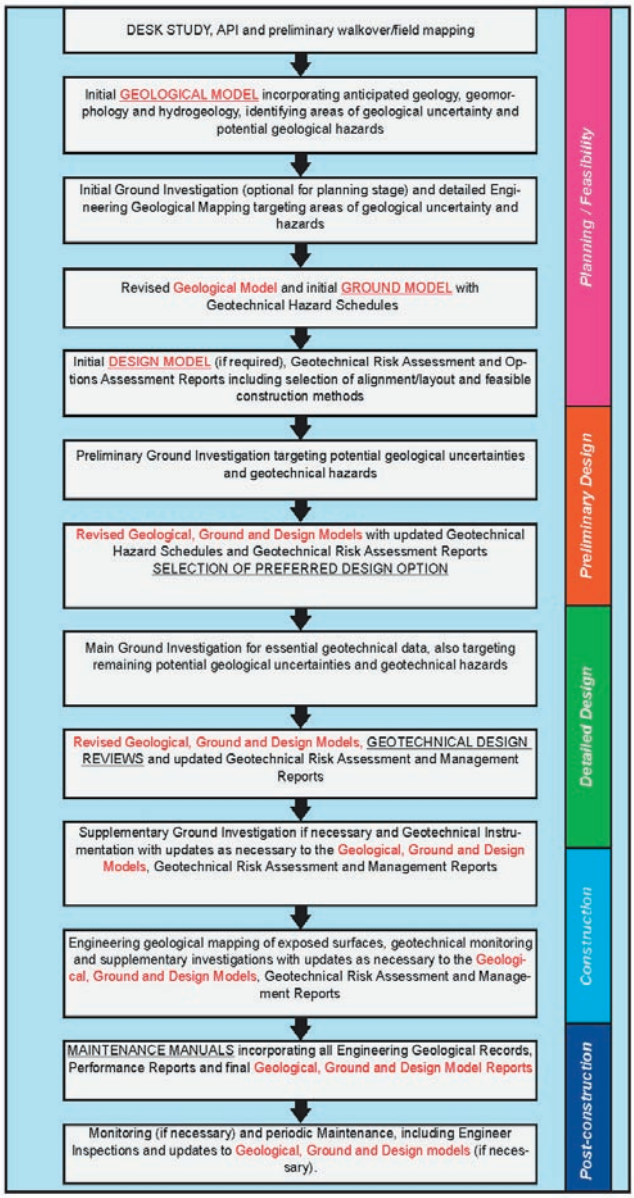


Fig. 2. Typical development and application of the 'Model Approach' for a major project (GEO 2007).

the ground conditions to facilitate verification of the design assumptions, particularly where the final design is based on the 'Observational Method' (GEO 2005b).

Application of an appropriate level of engineering geological skill and perspective usually enables a large percentage of the geotechnical characteristics of the area of interest to be anticipated at an early stage. Timely identification of areas of uncertainty and potential hazards enables subsequent ground investigations to be efficiently focused, thereby reducing costs and the risk that 'unforeseen ground conditions' may be encountered during construction.

4 GEOLOGICAL PROCESSES

4.1 Introduction

The distribution of geological units in terms of stratigraphy and lithology in Hong Kong is reasonably well documented by geological maps and memoirs (Sewell et al. 2000, Fyfe et al. 2000). Similarly, there is a wealth of geotechnical data on the engineering properties of the main igneous rock types. However, variations in geotechnical properties within the main rock types, arising from geological processes either during formation or post formation, is less well understood. The key geological processes that affect the engineering characteristics of most rocks and soils in Hong Kong include (i) tectonics, (ii) metamorphism and hydrothermal alteration, (iii) weathering, (iv) geomorphological processes, and (v) hydrogeological processes. Understanding these processes, their evolution over geological time, their spatial relationships and their effect on the engineering properties of different rock and soil types is key to the development of geological models for engineering purposes.

4.2 Tectonics and Tectonic Structures

Sewell et al. (2000) provides a summary of the regional tectonic setting of southeastern China including Hong Kong. During the late Jurassic and early Cretaceous Periods the plate margin was much closer to Hong Kong. This resulted in a period of intense volcanic activity with associated granitic intrusions, and also in the main pattern of faults evident today (Fig. 3).

Tectonic structures include faults, folds, metamorphic fabrics such as foliation and cleavage, and tectonic joints. These structures reflect the response of the rock mass to in situ stress over geological time. Most faults, metamorphic fabrics and joints are discontinuities that have much lower tensile strength than the intact material. Therefore, discontinuities have a major effect on the engineering properties of rock masses.

Key engineering geological issues include (i) the effect of past and present regional tectonic settings on the formation of geological structures and in situ stress, (ii) zones of deep weathering along some of the major faults and their engineering implications, (iii) the geotechnical influences of different types of faults, (iv) the development and significance of discontinuities, including the response to stress-relief from natural and man-made sources, and (v) preferential groundwater flow. A summary of types, occurrence and geotechnical significance of discontinuities, including those that are not of tectonic origin, is given in Hencher (2000).

4.3 Metamorphism and Hydrothermal Alteration

The location of igneous intrusions and hence the potential areas of contact metamorphism are well documented. Dynamic metamorphic effects are widely found in the northwest and northern New Territories and are associated in part with fault movement (Sewell et al. 2000). Knowledge of metamorphism and hydrothermal alteration and skilled interpretation of their spatial relationships with other geological structures facilitate the development of realistic geological and ground models in areas where such altered rocks may be present.

The key engineering geological issues associated with these processes include (i) granular recrystallisation resulting in a stronger material structure, e.g. hornfels and marble, (ii) formation of foliation resulting in anisotropic material properties relative to the alignment of the fabric, (iii) alteration or concentration of minerals such as with greisenisation. Furthermore, hydrothermal alteration involves mineralisation, replacement or alteration of existing rocks by mineral-rich fluids which tended to concentrate near the boundaries of the plutons and within major joints and faults.

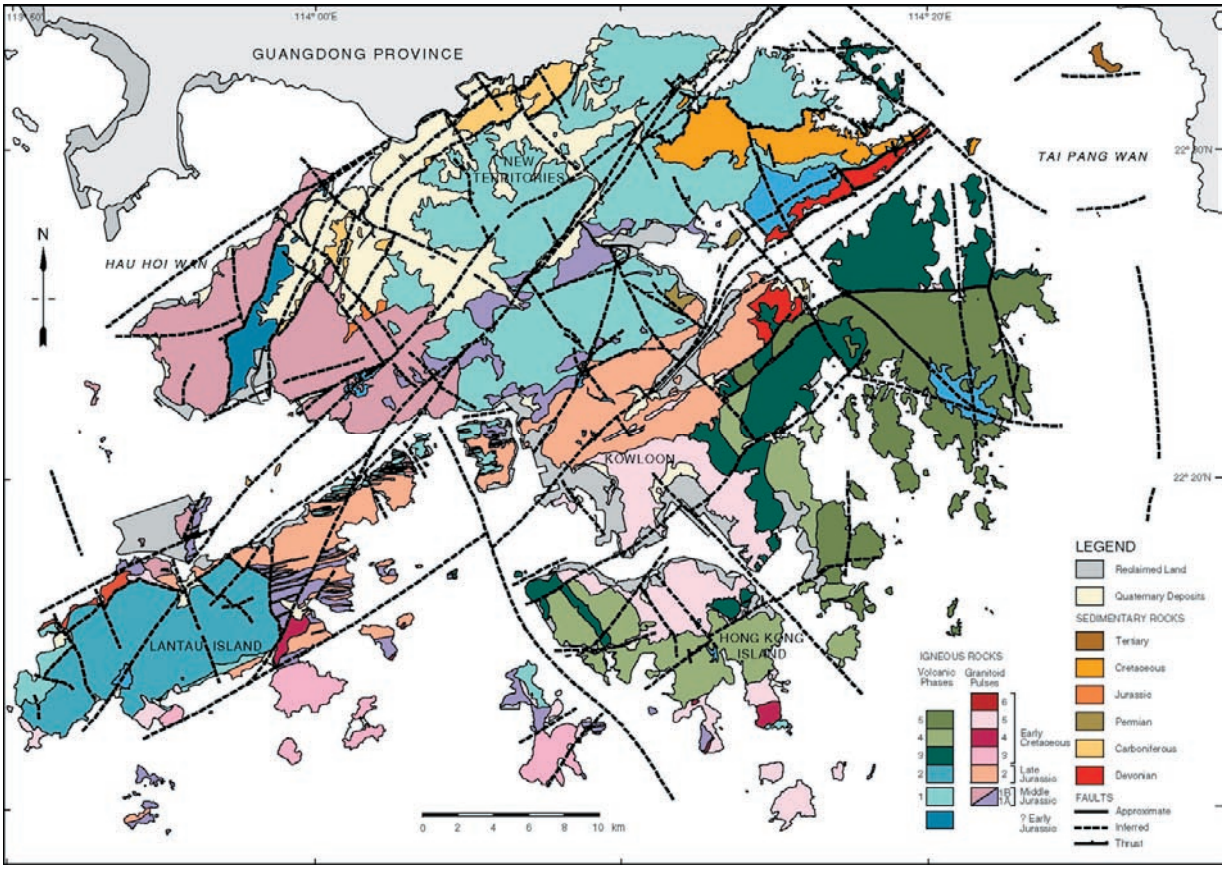


Fig. 3. Simplified geological map of Hong Kong (Fyfe et al. 2000).

4.4 Weathering

The two main components of weathering are mechanical disintegration and chemical decomposition, which can give rise to complex weathering profiles. Therefore, knowledge of weathering processes, and skills in interpreting their development in relation to structures and differing rocks types, are essential for realistic geological and ground models to be formulated.

Material and mass weathering classification systems have been developed to characterise the variability of weathered in situ rock masses for geotechnical design purposes. Accounts of weathering and the development of weathering classification systems that are relevant to rocks in Hong Kong can be found in GCO (1988), Martin & Hencher (1988) and BSI (1999). The GEO has also undertaken a considerable amount of work on the chemical and mineralogical aspects of weathering as these also affect geotechnical properties (e.g. Campbell & Parry 2002).

The key engineering geological issues associated with weathering include (i) decomposition of the original minerals to low strength clay minerals, (ii) growth of pore spaces, causing increases in porosity and possibly in permeability, and with reduction in grain bonding, thereby decreasing material strength, (iii) growth of microfractures, (iv) retention of geological structure and fabric in saprolite, which may result in heterogeneous variations in mass shear strength and permeability, (v) concentration of clay minerals along discontinuities, particularly in saprolite close to interfaces between rock and soil, (vi) variations in weathering intensity and depths giving rise to difficulties in defining rockhead, and (vii) the presence of corestones and heterogeneous masses giving rise to difficulties in estimating mass shear strength, deformability and permeability.

The term ‘rockhead’ as used in engineering is the level at which the engineering parameters of the rock mass satisfy the design parameters for the project. These requirements vary considerably, for example rockhead can signify the depth to which the ground can be excavated mechanically without blasting, or it can signify the top of rock with a required bearing capacity. As such, engineering rockhead is project and site specific and its determination can be one of the most critical issues for construction purposes (Fig. 4). Major variations in engineering rockhead level are commonly caused by geological structures whereas the presence of corestones may lead to irregularities in the rockhead profile.

4.5 Geomorphological Processes

Geomorphological processes encompass all forms of surface erosion and deposition including colluvial, fluvial and coastal processes. These processes have shaped the present-day topography and are of fundamental importance in understanding the engineering geological characteristics of the Hong Kong landscape. Geomorphological mapping places the site and its surroundings in a hierarchical framework that integrates morphology (form), process, materials and age (GEO 2004). This helps the practitioner to interpret the influence of lithology, structure, materials and processes on past and current landform development, thus allowing the formulation of geological models to predict future behaviour.

The key engineering geological issues are (i) identifying the various processes currently active and those which have affected the terrain in the past, and (ii) assessing if the results of

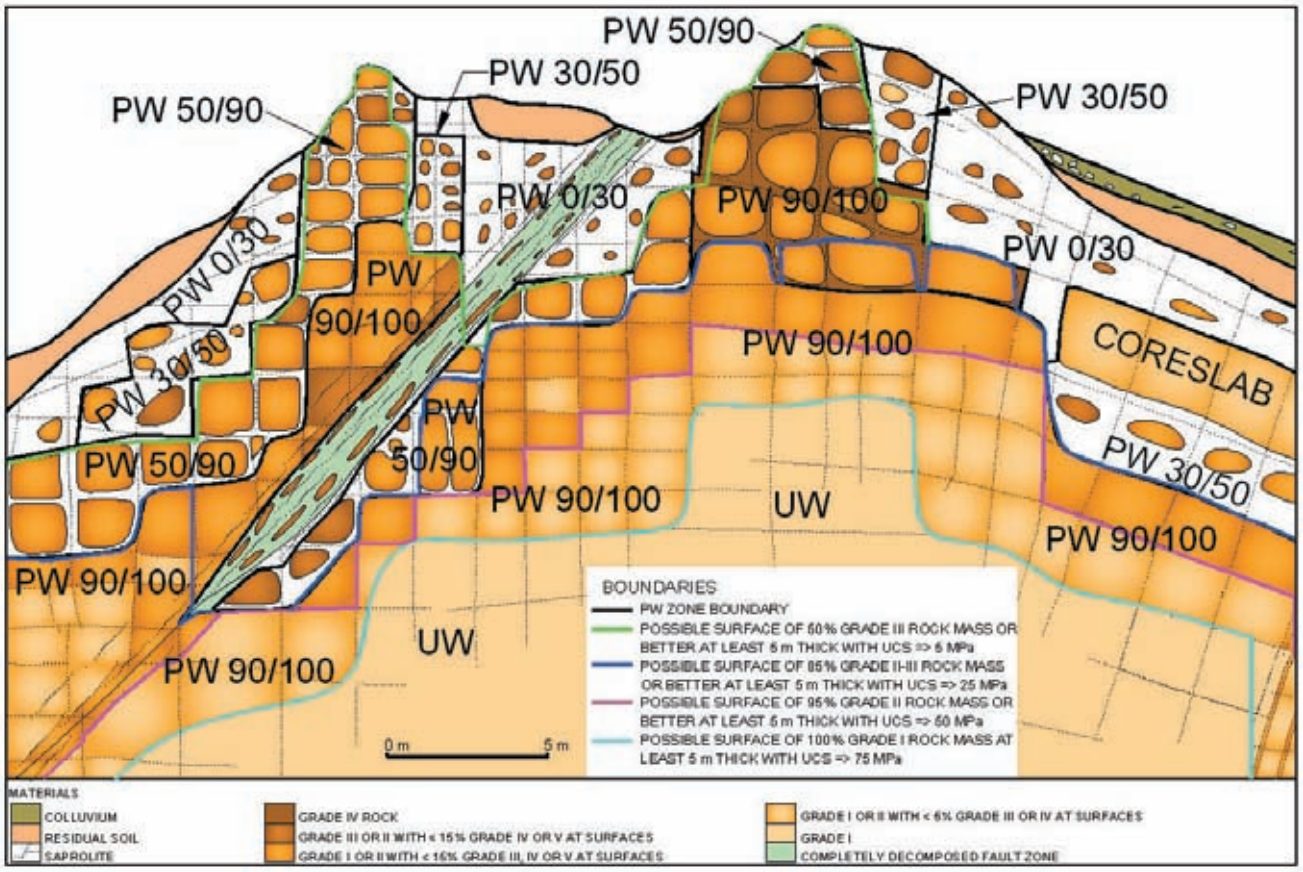


Fig. 4. Schematic partial weathering (PW) scheme applied to a mass exposure and other possible boundaries based on engineering requirements (after GCO 1988).

these processes could affect the engineering project in question. The recognition of active processes (e.g. erosion, transport and deposition) related to progressive deterioration, such as weathering, changes in the hydrogeological regime and slope movements, can be facilitated by geomorphological mapping. Progressive deterioration can lead to increased water ingress and modify subsurface water flow conditions in soil pipes and joints, thus changing the potential for hazard on a local scale.

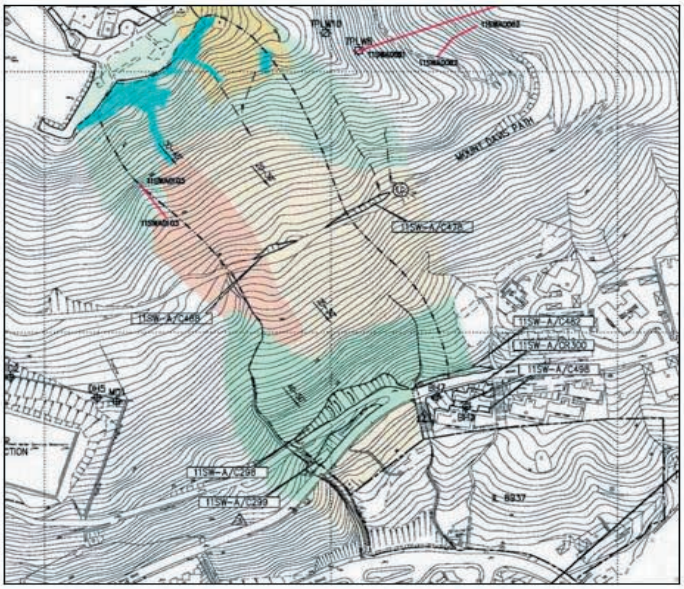
Mass movements (e.g. landslides, debris flows, rockfall and boulder falls) have played a significant part in forming the present-day landscape of Hong Kong. Evidence of mass movement such as degraded, amphitheatre-shaped depressions in hillsides and large colluvial lobes near the base of hillsides can be seen in many places in Hong Kong. In many cases debris may be absent or it may not be possible to link the debris present with the source area. Consequently considerable skill, and often detailed mapping, is required to determine whether such features are degraded large landslides or the result of the coalescence of a number of smaller landslides or erosional features.

4.6 Hydrogeological Processes

Hydrogeology is of major geotechnical importance in Hong Kong, with uncertainty regarding the groundwater regime often being a key issue in many types of engineering applications, such as slope stability, deep excavations and tunnels. The key engineering geological issues are (i) heterogeneous and discontinuous geological materials with complex contrasts in permeability, such as soil pipes, perching and damming of groundwater, and (ii) settlement of unconsolidated deposits (e.g. new reclamation) in response to groundwater abstraction or flow into deep foundation and tunnel excavations during construction.

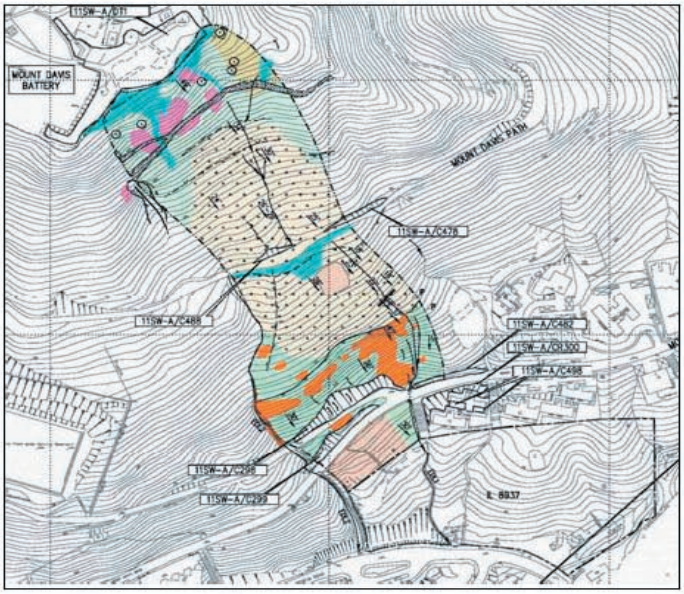
A large amount of uncertainty may exist due to the heterogeneous nature of the ground, the impracticality of defining it in detail and potential future changes in environment. The variability of hydrogeological characteristics is primarily due to the geological origins (e.g. dense saprolite vs bouldery colluvium) and the subsequent effects of the geological processes. Furthermore, the groundwater regime is affected by environmental influences which may be caused by natural phenomena, e.g. annual and variations in rainfall, and man-made influences, e.g. changes in vegetation cover due to construction, cultivation, hillfires.

A number of groundwater studies in Hong Kong (e.g. GCO 1982; Li et al. 1995) have demonstrated that the piezometric response time to individual rainstorms generally increases with depth, with sharper responses of shallow perched water tables in colluvium or thin saprolite overlying shallow rock being common.



Preliminary Geological Model

- Desk study information, particularly a detailed API, permits a preliminary classification of the site in terms of morphology, material (lithology and regolith), human activity, previous evidence of hazard, etc.
- This includes looking beyond the site boundary to ensure the setting of the site has been adequately considered.
- The preliminary geological model is used to direct the field work and optimise the site investigation.



Detailed Engineering Geological Model

- An amended and more detailed model of materials, morphology, history and process is generated after fieldwork and ground investigation.
- In the case of this site, more detailed information was obtained on the distribution and classification of regolith and lithology, slope gradients, seepage zones and human activity.
- The information in the detailed engineering geological model permits identification of potential hazard.

Potential Natural Terrain Hazards

- Areas susceptible to potential hazard are identified. The areas and potential hazards can be rationally evaluated using desk study, field and ground investigation information.
- The detailed nature of each hazard becomes the input information for any subsequent hazard assessment if a risk has been identified.
- In this example, the upper slope offered little risk given its distance from the facility. Detailed analysis of the lower slope also indicated little risk to the facility.

Fig. 5. Progressive development of geological models for a natural terrain hazard study (GEO 2007).

Relatively rapid and large responses in thick weathering profiles and colluvium can also occur where a network of relatively open joints, fissures or soil pipes allow rapid infiltration and conduct flow towards zones of lower mass permeability (Sun & Campbell 1999).

Rock mass is often regarded as being less permeable than saprolite, but there is some evidence for zones of more permeable rock partially confined by less permeable weathered rock mass, resulting in upward hydraulic gradients (GCO 1982). Jiao (2000) has also raised the possibility of partially confined groundwater having contributed to the delayed response and deep-seated failure of some large cut slopes in Hong Kong. In general, the groundwater regime in saprolite can be complex, with primary porosity (soil material) and secondary porosity systems comprising networks of relict discontinuities, fissures and soil pipes. The secondary porosity may result in a transmissivity much higher than the primary system. Conversely, geological features such as clay-infilled relict discontinuities may result in lower permeability and lead to local perching or retardation of slope drainage (Au 1990). Hydrogeological uncertainty can have major effects on the reliability of geotechnical designs and engineering performance both during and after construction.

5 ENGINEERING APPLICATIONS

5.1 Introduction

Engineering applications in Hong Kong which require engineering geological input include natural terrain hazard assessment and mitigation works, site formation, slope engineering, foundations, deep excavations, tunnels and caverns, marine works and reclamation, landfills and contaminated land, and natural resource assessment. The following examples are used to illustrate the need to produce realistic geological and ground models to identify and address the key geotechnical issues which are most relevant to engineering application.

5.2 Natural Terrain Hazard Assessment

Natural terrain hazard studies are an increasing component of engineering practice in Hong Kong as new development extends into steeper terrain and existing development is assessed for potential risk. A fundamental aspect in assessing natural terrain hazards is understanding the geological and geomorphological processes that currently operate, as well as those that have operated in the past (Ng et al. 2003). Although these processes are complex, they can be interpreted using engineering geological principles to formulate a geological model such that potential natural terrain hazards arising from them can be identified and assessed.

The formation of a geological model provides the basis for identifying the likely geomorphological and geological controls on the location, type, magnitude, frequency and runout characteristics of potential hazards, and generally comprises two distinct components of (i) mapping and assessment of terrain characteristics and interpreting how the landscape at a site evolved, and (ii) evaluation of the natural terrain instabilities in the area to develop an inventory of potential hazards for the site. The hazard assessment builds on the geological model and hazard models in order to assess the potential risk to facilities in question, particularly by the quantification of hazard location, magnitude, frequency and mobility (Parry et al. 2006).

The identification of a suitable design event for implementation of risk mitigation works requires careful engineering geological judgement. The key value of utilising an engineering geological approach is to ensure the range of design events is consistent with the information derived from the geological and hazard models. An example of the development of a geological model is given in Fig. 5, where preliminary terrain characteristics were evaluated and refined by API and field mapping, which then formed the basis for development of a hazard model.

5.3 Slope Stability

Intense urban development in the hilly terrain, combined with thick weathering profiles and heavy, seasonal rain, contributed to some notable slope failures with associated loss of life such as the Sau Mau Ping fill slope disasters in 1972 and 1976 and the Po Shan Road disaster in 1972 (CEDD 2005). The Government of Hong Kong established the Geotechnical Control Office in 1977 (now GEO) with the main aims being to progressively improve slope safety and geotechnical practice in Hong Kong. Over the last 30 years or so, improvements in geotechnical control, slope management and safety awareness have substantially reduced the overall rate of fatalities resulting from landslides when compared to the rate of increase of urban development (Malone 1998; Chan 2003).

There are inherent variability and uncertainties in the geological and hydrogeological conditions of slopes in Hong Kong. Ho et al. (2003) document the key lessons learnt from studies of failures of man-made slopes together with observations from reviews of investigation and design practice based on examination of over 100 slope design or assessment reports. Their findings indicate that the most important factor with regard to major failures is the adoption of an inadequate geological or hydrogeological model in the design of slopes, with the main problems being associated with adverse geological features and adverse groundwater conditions. Martin (2003) report that evidence from systematic landslide investigations, together with earlier case histories, shows that most of the sizeable (>50 m^3) landslides and all the large landslides (>500 m^3) in cut slopes have failure surfaces formed wholly or partly along discontinuities in saprolite or less weathered rock. Assessment of the geological structure is therefore important in the investigation, design and construction of cut slopes.

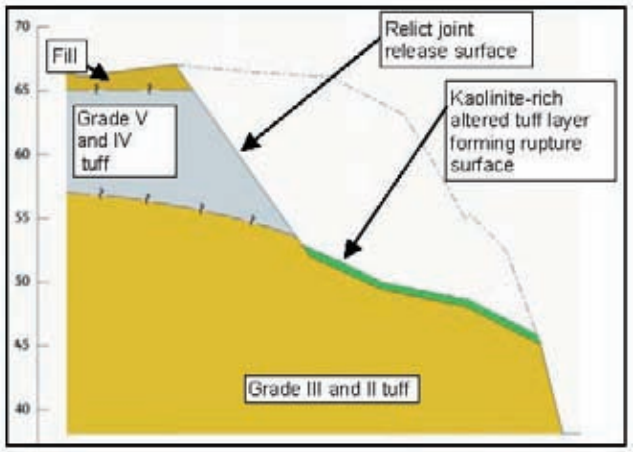


Fig. 6. Section through Fei Tsui Road landslide (GEO 1996).

Fig. 7. Location of the 1995 Fei Tsu Road landslide in 1977.

An example involving adverse discontinuities and associated development of perched water level is the Fei Tsui Road Landslide which occurred in August 1995. The landslide involved the failure of about 14,000 m^3 of highly and completely decomposed tuff along a laterally persistent, kaolin-rich seam dipping between 10° and 20° out of the slope (GEO 1996). Extensive, steeply-dipping, kaolin infilled relict joints formed the back-scarp to the landslide (Fig. 6). Shear box testing on the kaolin rich tuff layer yielded average and lower-bound shear strength parameters of $c' = 0$, $\phi' = 29°$ and $c' = 0$, $\phi' = 22°$ respectively. Back analysis of the landslide indicated that a perched water table only 2 m above the rupture surface yielded a factor of safety of 1.0 if the operative angle of friction on the kaolin seam was assumed to be 28°. A photograph of the slope taken in 1977 (Fig. 7) clearly indicates the exposed and unprotected nature of the kaolin-rich seam, but in several studies undertaken before the failure, its true implications for slope stability had not been recognised (GEO 1996).

5.4 Tunnels

Underground construction is capital intensive, with the overall cost, programme and risk of adverse consequences being heavily dependent on the ability to characterise and manage the ground conditions adequately. Timely recognition of the ground conditions and the determination of appropriate measures to deal with them are central to the degree of success of all tunnelling projects (GEO 2005b). Errors in estimating the percentage of tunnel requiring heavy support or the extent of groundwater control measures can result in large differences between anticipated and actual costs and construction programmes.

Although engineering geological input is essential for all types of tunnelling, the input can vary depending on the stage of the project, the ground conditions and the tunnelling methods employed. Engineering geological input required to develop geological and ground models for tunnels include assessment of (i) geological structures and identification of potentially adverse geological conditions, (ii) the relevant characteristics of the ground, which may include the use of rock mass classifications to aid selection of excavation methods, temporary support types and permanent lining types, (iii) mixed ground interfaces associated with variable rockhead profiles and corestone-bearing profiles, and (iv) the impacts on the local and regional hydrogeology, during and after construction.

The KCRC DB350 Tai Lam Tunnel serves to highlight how geological, ground and design models were developed using an observational method where a major fault zone was excavated at a depth of about 400 m in a 14 m-span tunnel. The example illustrates the translation of engineering geological data available at the tender stage into a series of design models and compares the as-built conditions with the range of conditions assumed for design.

The log of the water tunnel in the vicinity of the Sham Tseng Fault zone indicated that the zone was composed of several faults, with the largest being about 45 m thick. The log indicated that very adverse tunnelling conditions might be encountered when passing through the fault zone, where the much wider railway tunnel would probably need to be driven using incremental excavation and support techniques.

The design and construction strategy adopted was to develop the initial geological and ground models based on 'geomechanical' interpretations of the existing information from the water tunnel log, assuming a range of conditions (Fig. 8). These assessments were also tempered by an engineering geological knowledge and perspective of the typical nature of brittle-ductile fault zones at depth and the pictorial depiction of the fault zone on the log, which indicated that conditions might be much better than those inferred from a straight 'geomechanics' interpretation of the water tunnel log. Fig. 9 shows the comparative range of difficulty in tunnelling that might be expected for the ranges of possible ground conditions based on Fig. 8 and the more optimistic engineering geological assessment.

Horizontal coring was carried out as the fault zone was approached. Conditions were much better than indicated by the water tunnel log. Probing ahead established that the fault zone was composed of discrete, narrow faults separated by zones of relatively competent rock. The largest individual fault was only 8 m thick. The conditions as-encountered were very similar to the more optimistic conditions initially assumed from a general engineering geological knowledge of brittle-ductile fault zones at depth. The differences in implications with regard to relative tunnelling difficulty are considerable. The actual ground conditions revealed in the KCRC Tai Lam Tunnel are in line with the interpretation based on engineering geological perspective (Figs. 8, 9).

6 THE FUTURE OF ENGINEERING GEOLOGY

This paper and its supporting references demonstrate that the application of good engineering geological practice can enhance the sustainability and success of many geotechnical applications in Hong Kong. Taking slope engineering as just one example, a good understanding of the engineering geology of the ground is essential to enhance and sustain slope safety and increase the value of slope engineering and natural hazard assessment to the general public. Therefore, the future of geotechnical engineering in Hong Kong will be influenced in no small measure by the ways in which engineering geology is perceived, utilised and practiced.

As previously indicated, the core engineering geological skills which are necessary to produce realistic ground models are not easy to define or codify, due to their wide scope, partially implicit nature and dependence on the experience of the practitioner at both local and international levels.

These aspects, which even engineering geologists find difficult to deal with (Knill 2002; Baynes & Rosenbaum 2004), need to be better understood to promote the contribution of engineering geology to the geotechnical engineering community and the general public.

What are engineering geologists and what do they do that is different from other geotechnical professionals? In Hong Kong whilst the term "engineering geologist" is commonly specified, there is no agreed description for what this comprises. A Hong Kong definition was suggested that an engineering geologist should be a person with a geological first degree, a Masters degree with significant engineering content and experience in a responsible capacity confirmed by a relevant professional qualification (Parry 2004).

In essence, the purpose of an engineering geologist in Hong Kong is to investigate and interpret the ground conditions relevant to an engineering project so that the ground can be reliably characterised and any potential geotechnical hazards identified to facilitate the management of geotechnical risks. This requires appropriate academic training and experience with focus upon understanding the ground in both geological and engineering contexts. It also requires skills in communicating the significant engineering and environmental elements of the geological model to others involved with the project. As in most other professions, the future of engineering geology in Hong Kong depends upon a sustainable supply of practitioners with suitable academic training, experience and competency.

The move to modular based courses and the amalgamation of Geology Departments into broad "schools" has resulted in undergraduate courses not providing sufficient depth in the core components of geology (Clarkson 2004). As a result today's earth science graduates may lack some of the basic geological skills upon which the profession of engineering geology is built. This can place graduates entering the industry or about to commence a

45 m THICK GRANODIORITE FAULT ZONE WSD TUNNEL DESCRIPTION (3.5m dia. tunnel)	Param.	Q-SYSTEM Interpretation				Notes	
		Worst	Poor	Typical	Best	Poor	Best
Decomposition Grade "Moderate and Highly Weathered (Zones III-IV)" It is not known whether Grade IV is confined to seams or not - although the "and" in the weathering description tends to favour Grade III material with highly weathered seams.	Grade =	III-IV	III-IV	Close to Grade III	III	Grade III-IV With Grade IV Seams	Grade III With Grade IV Seams
Strength "Weak" - but fracturing and Grade IV seams have probably influenced description. **Note: This assumption is very critical!**	Intact Unconf. Comp. Strength (MPa) =	8.00	8.00	12.00	16.00	Weak to Moderately Weak	Moderately Weak to Moderately Strong
Discontinuity Spacing "Laminar, brecciated, crushed, fractured. Narrow to moderately narrow discontinuity spacing (including secondary schistosity)"	RQD =	10.00	10.00	10.00	10.00	RQD = 10	RQD = 10
Joint Sets / Structure Depends on how rock mass description is viewed. Crushing & brecciation may be confined to seams and the seams and secondary schistosity which will strike directly across tunnel may blank out all other sets. Therefore may be few effective sets with orientation of shears most favourable for stability.	Jn =	15.00	15.00	10.50	6.00	Brecciation, crushing and fracturing assumed to dominate "sugar cube"	Brecciation, crushing and fracturing assumed to be confined to seams normal to to tunnel "Two sets plus random"
Joint Roughness No joint roughness description available. However, assumed to be "no rock wall contact when sheared" to "slickensided undulating"	Jr =	1.50	1.50	1.25	1.00	"rock wall cont. before 10cm shear" "slickensided undulating"	"no rock wall contact when sheared"
Joint Infilling Description limited to "extensive quartz veining, kaolinite veneers". Range interpreted to vary between "zones or bands of disinegrated or crushed rock and clay" and "softening clay infillings"	Ja =	8.00	6.00	5.00	4.00	"Softening clay mineral fillings" but 0.75* Ja to allow for favourable orientation of shears	"Bands of crushed rock and clay" but 0.5* Ja to allow for favourable orientation of shears
(RQD*Jr)/(Jn*Ja)	Q' =	0.13	0.17	0.24	0.42		
"Geological Strength Index"	GSI =	25	28	31	36	Calculated from Q'	Calculated from Q'
Intact mi Value Discontinuity spacing indicates material is better than "Phyllite" (mi = 10). For Granodiorite, the minimum mi values for "normal" Grade III-IV and Grade III may be 22 and 25 respectively. For worst Grade III-IV a value of 16 may be appropriate.	m_i =	16.0	22.0	23.5	25.0	Grade III-IV	50% Grade III-IV to 100% Grade III
Poisson's Ratio based on laboratory testing of weathered intact HK rock specimens	v=	0.35	0.35	0.33	0.32	-	-

Fig. 8. Geomechanics interpretation of the water tunnel log (GEO 2007).

post-graduate course in engineering geology at a distinct disadvantage that can be only partially overcome with much training and experience over a relatively long period of time.

In addition to fundamental knowledge, engineering geology demands skilled observations, interpretations and analyses, often based on limited information. Engineering geology is akin to an apprenticeship, where such skills are acquired by appropriate training and supervision from more experienced practitioners. The sustainability of engineering geological practice therefore depends on the availability of the more experienced and the amount of time that they have to train and supervise. However, in Hong Kong, graduates commonly work under engineers who may have only a few years of experience with little or no knowledge of engineering geology. In such cases, the graduates have limited opportunity to develop their geological skills.

As a result of the current system many graduates end up as geotechnical technicians, carrying out logging, data gathering and simple analysis but lacking the skills to do engineering geological tasks such as generate engineering geological models and then take these forward for the purpose of geotechnical design. If these areas are not acknowledged and acted upon by the geotechnical profession, the future of engineering geology and by implication, the future of geotechnical engineering, will be compromised.

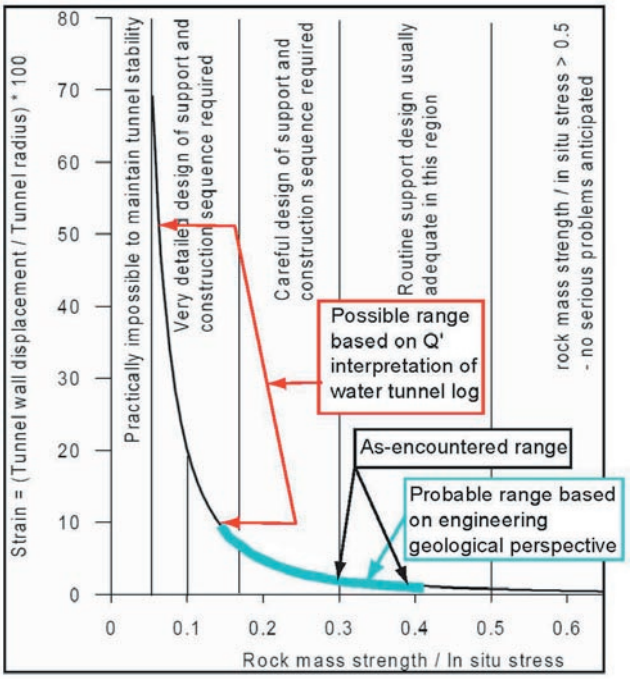


Fig. 9. Graphical representation of relative tunnelling difficulty (GEO 2007).

7 CONCLUSIONS

Engineering geological practice is primarily concerned with the determination of geological and hydrogeological conditions to facilitate ground engineering with respect to the recognition and management of geotechnical risk. This requires the application of geological knowledge and skills to define and communicate the potential and actual variations in ground conditions that are relevant to Hong Kong.

Only a fraction of the volume of ground which will affect or be affected by the proposed works can usually be directly observed or tested during a site investigation. Therefore, the risk of 'unforeseen ground conditions' has the potential to increase with geological complexity. In such situations, engineering geological knowledge of the site setting and critical interpretation of other available data can help define the ground conditions in three dimensions, so as to facilitate the resolution of key geotechnical issues relevant to different engineering applications.

The need to improve engineering geological practice in Hong Kong has been recognised. Improvements in this regard will also enhance geotechnical practice and increase its long-term value to Hong Kong's society.

ACKNOWLEDGEMENT

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A Review of Site Investigation Techniques Introduced to Hong Kong in the last 30 years

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Abstract: The introduction of new site investigation practices and techniques to Hong Kong has largely been governed by the nature of the construction projects being undertaken at the particular time. Techniques that may have been widespread in other parts of the world have usually been first introduced on large construction projects. Some equipment, such as the Mazier Sampler, which was found to be suitable for local conditions have been adopted by local practitioners and have now become widely used. Other techniques have been tried but for various reasons have been deemed to be unsuitable for local conditions subsequently have only been sporadically used. (E.g. hollow stem augers, wireline drilling). This paper reviews the techniques that have been introduced in the last thirty years highlighting which ones have become widespread mainstream techniques and explaining why some have only been used rarely and largely on prestigious jobs.

1 INTRODUCTION

From 1970 – 1990 the major driving force behind the new techniques introduced was the slope stability issues addressed by the GEO. These initiatives lead to the introduction of triple tube core barrels, foam flush, higher levels of supervision and the introduction of codes of practice. These measures generally increased the quality of samples being recovered.

From 1990 to present day the majority of techniques introduced were to satisfy the requirements of the large infrastructure projects such as the Chek Lap Kok Airport Project and associated road and rail networks. Examples of techniques introduced on these projects include the use of Cone Penetration Tests to assess ground conditions for reclamation works and High Pressure dilatometer tests undertaken for the KCR West Rail Project. The techniques on these projects were generally introduced to either speed up or reduce the costs of construction.

Other techniques such as the acoustic and optical televiewer together with have been introduced simply because of advances in technology.

It is likely that future techniques and methodologies introduced into Hong Kong and the Pearl River Delta will become increasingly influenced by large scale energy generation projects and the remediation of contaminated land. This could see the introduction of such technologies as sonic drilling and environmental cone penetration tests.

The following sections will summarise the advances made in the following areas:-

- Drilling and Sampling
- In Situ Testing
- Geophysical Testing
- Data Management

A brief description of the techniques will be given together with examples of projects in which the techniques were introduced or widely used.

2 DRILLING AND SAMPLING METHODS AND PRACTICES

Site investigation field works in the 1970s usually comprised drillholes, trial pits and surface strips. Drillholes were advanced into ground by surging and drilling the casing down, and water was normally used as the flushing medium. Trial pits were commonly used because they allowed a full assessment to be made of soil texture together with other features such as fill and colluvium boundaries. They also enabled the carrying out of field density tests and collection of block samples. As man-made slopes in Hong Kong were usually protected against surface erosion by a surface layer of cement-lime stabilized soil, known as “chunam”, it was a common practice in site investigation field works to remove strips of the surface protective material to examine the exposure to obtain relevant geological information. The Standard Penetration Test (SPT) was frequently conducted in drillholes for assessment of subsoil profiles and liner samples were usually obtained in conjunction with the SPT in helping to interpret the results. Most developers failed to recognize the benefits of carrying out a proper site investigation, and were therefore reluctant to spend money on more sophisticated site investigation techniques to achieve better quality results.

After disastrous landslides occurred in June 1972 and August 1976, the Geotechnical Control Office (GCO, renamed Geotechnical Engineering Office in 1991) was formed in July 1977 by the Hong Kong Government to regulate planning, investigation, construction, monitoring and maintenance of slopes in Hong Kong. Since the establishment of the GCO, technical standards, guidelines and model specifications for the investigation, design and construction of slopes have been published. Apart from setting safety standards, significant advances have been made on other basic aspects concerning slope stability in saprolitic soils, including improved site investigation

techniques, research and development activities on assessing the shear strength of saprolitic soils, using appropriate geological / hydrogeological model in stability analysis, etc. Guidance on site investigation for slope stability assessment and design is given in the Geotechnical Manual for Slopes (GCO 1979, revised in 1984) and the Guide to Site Investigation – Geoguide 2 (GCO, 1987), which have been widely adopted by the geotechnical profession in Hong Kong.

2.1 Drill Flush Fluids

One of the significant advancements in site investigation techniques is the use of air-foam as flushing medium in obtaining undisturbed samples in saprolitic soils. When sampling saprolitic soils, a serious problem encountered in the traditional core drilling was contamination and scour of the core caused by the drilling fluid. Much effort was made in the early 1980's to overcome this problem. After many drilling trials, it was found that by using air-foam as the flushing medium, excellent core quality and recoveries could be obtained, even for colluvium, one of the most difficult soil types to sample (Phillipson & Chipp, 1982). Consequently, air-foam flushing medium is used when high quality sampling is required.

Polymer drill fluids are widely used for taking rotary core soil samples in many parts of the world but have only been intermittently used in Hong Kong. Polymer drill fluids of varying viscosities can be used to obtain high quality samples of soil and clay. The main advantage over foam flush is that they can be used to any depths even below the water table. Foam flush has a maximum practical limit of around 50m and becomes largely ineffective below the water table.

One project which successfully user polymer drilling fluids was described by Fletcher et al. (2000) during which intact samples of clay, silt and loose sand sequences were recovered from depths of up to 150m using polymer drill fluid. This work was undertaken for a development site at Tung Chung New Town (Proposed Tower 5 Site 3) an area which was designated to be underlain by complex ground conditions comprising xenoliths of marble, other weathered metasedimentary rocks, granitic saprolites, karst infill deposits and possible voids. The ability to recover continuous intact samples (which were split open) allowed the true nature of the geological structures within the cavity fill sediments and overlying soils to be revealed.

2.2 MAZIER CORE BARRELS

Mazier triple tube retractable core barrels were introduced in Europe in the 1960's as described by Cambefort and Mazier (1961) and Mazier (1974).

The Mazier sampler was sporadically used on projects such as at Kai Tak and the High Island Reservoir in the 1960's but did not become commonly used as the minimum required standard until the late 1970's.

The Mazier sample was found to be ideal for sampling the saprolitic soils encountered in Hong Kong. Its effectiveness relies on a non rotating thin wall seamless steel sampling tube which protrudes in advance of a tungsten inset rotating core bit. The amount of inner barrel protrusion is controlled by a spring placed in the upper part of the core barrel. The inner barrel cutting shoe also comes in three lengths with the longest being used for the softest formations and shortest for the hardest formations. The inner barrel is fitted with a liner which was originally made of brass. The liner enables the sample to be sealed at the ends with max and then transported to the laboratory for testing. The liner material has evolved over the years brass was initially replaced by opaque UPVC and in recent years clear ABS plastic has been used enabling the geologist to record strata boundaries and discontinuities without disturbing the sample.

2.3 Large Diameter Piston Samples

In order to obtain high quality samples which were large enough to contain representative fabric features such as sand laminations, large diameter (254mm) piston samples were taken during the ground investigation for the Container Terminal 9 Development in 2000 and in 2003 on Contract No. HK/13/02- Wan Chai Development Phase 2 (Refer to Plate 1). The large diameter samples were used for Rowe Cell Consolidation tests which were conducted in a hydraulic cell enabling the pore water pressure to be measured during the test. The ability to measure the porewater pressure allows a more detailed investigation of the consolidation properties to be made and therefore produces more reliable parameters. However a multi-increment test can take several weeks to complete at considerable expense. The relatively high cost of obtaining the samples coupled with the high laboratory test cost means that this type of sampling and testing has only been used on large scale reclamation projects.

Plate 1. 300mm diameter Piston sampler

2.4 Vibrocoring

The Vibrocoring technique was introduced into Hong Kong on a large scale for the Terminal 9 Development Project in 2000 where more than 600 samples were taken. Since then it has been a routinely used sampling technique used on marine ground investigation projects for reclamation works, pipeline routes, bridges and piers. The main end use of the samples is for contamination testing to satisfy EPD requirements with regard to

the disposal of dredged sediments. Other sizable projects in the last decade have included the Penny's Bay Reclamation Stage 1 ground investigation during which more than 2400 samples were taken. The technique has also been used on land to obtain quality control samples of reclamation fill, notably for the Chek Lap Kok project in the mid 1990's and on many subsequent reclamation projects.

Many slightly different vibrocoring sampling systems have been used by contractors in Hong Kong. The basic principle is that a steel sample tube fitted with PVC plastic liner is driven into the sea bed and soil is held inside the liner with the aid of a "basket" catcher fitted inside the removable sampler cutting shoe.

The sampler barrel can be driven into the seabed in several different ways. Most commonly a floating percussive drill head is used, which is either hydraulic or electric. Casing is lowered to the seabed with a conventional top drive marine drilling rig and then the sampler is attached to drill rods and lowered inside the casing. The sampler barrel often sinks its entire length without any additional driving force. Should extra force be required the sampler barrel is vibrated until either full penetration is achieved or hard stratum is encountered.

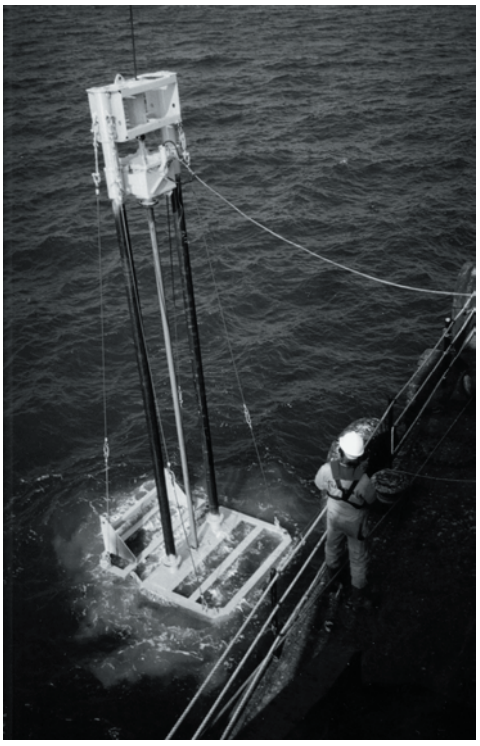

Plate 2. High Performance (HPC) Vibrocore unit with inclination and penetration measurements

A much quicker sampling method especially in deep water is to use a seabed sampling frame as shown in Plate 2. This system comprises a frame which holds the sample barrel and vibrating head. The whole frame is simply lowered to the seabed. When the frame encounters the surface of the marine sediments it stops but the sampler barrel continues to advance with aid of the percussive head until it encountered hard strata or it has penetrated the full 3.00m, 6.00m or 9.00m. The Fugro High Performance Vibrocore unit has the facility to record the depth of penetration and inclination of the sampler barrel. The barrel is then hoisted back into the frame and then lifted onto the deck of the drilling barge where the sample liner is removed and cut up into sections as required by the engineer. This type of sampler has the advantage that it is much quicker to deploy (especially in deep water) and it is more energy efficient as the vibrating head is close to the sampler barrel. Its main disadvantage is that it can only be used to sample to maximum depths of 9.00m and any slight obstruction or significant inclination of the seabed will result in low sample recovery. This sampler produces Class 2/3 samples suitable for environmental contamination testing and soil classification testing.

2.5 Vertical Wireline Drilling

Vertical wireline drilling was first introduced into Hong Kong in the 1980's on such projects as the Tate's Cairn Tunnel ground investigation and has been sporadically used on tunnel site investigation since then. The wireline drilling system is used for deep drillholes because it is supposed to be more efficient.
A conventional drillhole is cased and then a sampler barrel is lowered into the drillhole on the end of rods. In the wireline system the outer barrel of the sampler is attached to drill rods of the same outer diameter. The inner barrel of the sampler is then lowered inside the drillrods until it latches onto the outer barrel. The inner barrel is then released and the wireline is retrieved. The sampler barrel is then drilled into the rock. On completion of drilling an "overshot" is lowered inside the drillrods until it latches onto the inner barrel and then subsequently hoisted to the surface. This wireline system is more efficient because it is much quicker to rise and lower the inner barrel using a winch than it is to retrieve and install the conventional drill rods and sampler barrel. However because the wireline barrel houses relatively complex rotating and latching mechanisms the "kerf" area of the corebit is much larger than a conventional T2101 or TNW core barrel. This large kerf area means that the drilling times are often longer and the core bits are more expensive.

For this reason many contractors in Hong Kong prefer to use conventional drilling systems even in drillholes to depths of 300m. Obviously as the hole gets deeper the "tripping time" (time taken to pull out and put back the core barrel) for the conventional system gets longer and longer, so the deeper the drillhole the more efficient the wireline system becomes.

Due to the high cost of the initial investment and core bits coupled with high maintenance costs it is unlikely that wireline drilling will become more prevalent in the current market conditions.

2.6 Horizontal Directional Drilling

Horizontal directional drilling was first used in Hong Kong for the Cheung Tsing tunnel project in the late 1980's. This investigation involved drilling a horizontal drillhole along the line of a proposed tunnel. The direction of the drillhole was controlled by inserting wedges in the bottom of the hole to deviate the hole along a predetermined course. The main benefit of this technique is that the ground conditions can be established along the full length of the proposed tunnel intersecting any faults or fractured ground sections. Vertical or inclined drillholes along the length of a tunnel only sample a very small percentage of the ground conditions present along the length of the tunnel. For the Cheung Tsing project two holes were drilled, one from each and of the tunnel. One was 1000m long end the other 500m long. These holes used the wedging method and were surveyed with a single shot Eastman Camera.

Drilling techniques in the oil and mineral exploration industry have advanced considerably in recent years. There are several tools such as under reamers, bi-center reamers and mud motors used to steer drillholes. However all these techniques suffer from one common problem in that no core is recovered over the sections of drillhole where the "steering" is taking place. One tool that has overcome this problem is the Devico DeviDrill™ wireline steerable core barrel.

In 2003 Devico equipment was used to drill two horizontal directional drillholes to a length of 1152m and 532m for the Eagles Nest Tunnel project.

This Eagles Nest Tunnel project was the first time the Devico steerable core barrel was used in Hong Kong. This system has the following advantage over traditional methods.

- Wireline operation during directional drilling
- Core can be recovered in steering section
- Operates with an electric multishot survey tool which is much quicker than the Eastman Camera
- Direction of steering is measured while drilling
- Compared with a mud motor a high pump capacity is not required.

The contractor undertaking the Eagles Nest project estimated that the production rate was doubled using the new technologies.

Since the Eagles Nest Project the same equipment has been successfully used on the Po Shan Road slope drainage project where two holes were drilled to lengths of 252m and 310m and for the current Tsuen Wan drainage tunnel project where one hole has been drilled to 1000m and another in progress is now approximately 600m in length.

2.7 Portable Triple-tube Sampler

As a result of a pilot research project instigated by the CEDD in 1998 a portable triple tube sampler was developed which could be mobilised to a slope rapidly without the need for extensive scaffolding. The sampler itself was similar to a Mazier barrel except it was shorter and lighter producing 500mm long 74mm diameter samples. The initial development research project is described in detail by Chan (2000). The actual rig used to take the samples in the initial project was a modified electric drill. Since this project, portable triple tube sampling has been specified in many ground investigation term contracts. Individual contractors have now developed their own mini drill rigs which are commonly hydraulically powered. An example of such a rig is presented in Plate 3.

Chan (2000) concluded that considerable savings in time and cost could be achieved using the portable sampler and that disruption due to road closures could be minimized as the components can be mobilized by hand.

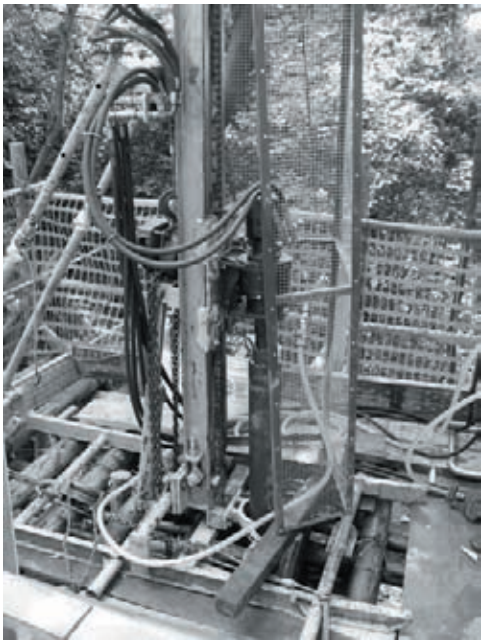

Plate 3. Portable Triple Tube Sampler Rig

3 INSITU TESTING

3.1 Cone Penetration Testing

3.1.1 Mechanical Cone Testing

The cone penetration test (CPT) which was developed in Holland in 1934 was originally used as a means of locating and evaluating the density of sand layers within the soft deltaic Dutch sediments. The manual CPT was introduced into Hong Kong in the late 1970's. This early equipment used a 60° cone with face area of 10cm^2 which was hydraulically pushed into the ground at a constant speed of 2cm per second using a special mobile hydraulic penetrometer rig.

The cone resistance and side friction was measured using mechanical methods. The mechanical cones are very rugged, simple to use and maintain and can give reliable results provided the equipment is properly maintained and the test is carried out with care. However the method of advancing a mechanical cone is complex and if not done correctly it can lead to serious errors.

3.1.2 Electric Cone Testing

Electric cone penetration tests were undertaken for the Tin Shui Wai development in the early 1980's (See Plate 4a & b). The electric cones proved to be simpler and faster to use than the mechanical cone and less prone to errors as the load measurement is close to the point of application. Generally the electric cone is more accurate and the results are more repeatable. Also there is better delineation of thin strata because readings are taken more frequently.

Plate 4a. Electric Cone Penetrometer Rig in Tin Shui Wai 1983

Plate 4b. Electric Cone Analogue data acquisition system in Tin Shui Wai 1983.

The electrical cone can be filled with many instruments allowing the following parameters to be measured as standard:

1) Cone inclination
2) Pore Pressure
3) Cone Resistance
4) Cone Friction

Other parameters or tests that can be undertaken include.

1) Soil resistivity (used in contamination studies)
2) Ground Vibration using three component geophones (seismic cone)
3) Gamma Ray Backscatter (for density determination)
4) Pressuremeter values
5) Sound (Acoustic Penetrometer)

3.1.3 Piezocone

Piezocones (Fig. 1) were also introduced into Hong Kong in the 1980's and have now become the minimum industry standard. This type of probe was commonly used in the late 1980's and 1990's as an exploratory tool prior to undertaking the large reclamation for the building of new towns such as Tin Shui Wai and Tseung Kwan O.

A piezocone has porous element fitted at various positions on the cone with a pore pressure transducer mounted in a cavity behind it. Consideration of the porewater pressure enables the piezocone to be used for profiling, identifying soil types, measuring the static pore pressure and also to determine insitu consolidation characteristics using a porewater pressure dissipation test.

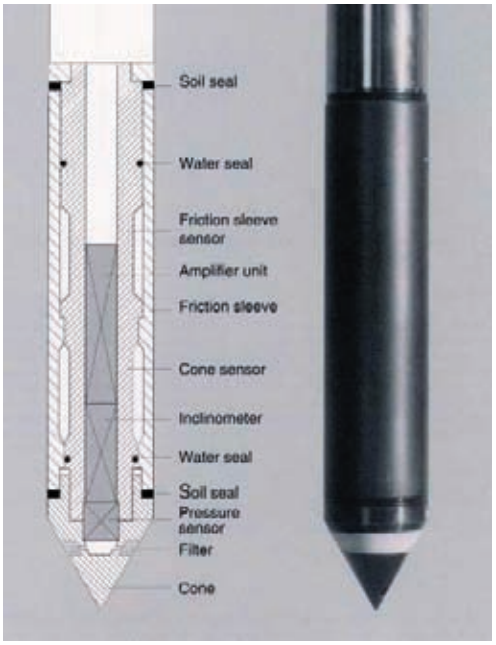

Fig. 1. Piezocone Penetrometer

Vlasblom (1999) described how 3500 CPT tests were used to define the level of the "Base of unsuitable material" or theoretical dredge level which was used to calculate the volume of material which had to be removed prior to placing the fill upon which Chek Lap Kok airport was to be constructed.

The piezocone was also used on the Chek Lap Kok project as a tool for checking the compaction of the fill material. After field trials a specification was devised in order to assess the suitability of the compacted reclamation fill.

The compaction of the fill was deemed to be acceptable if it fulfilled the following criteria.

- the friction ratio was less than 45%;
- the deviation between pore pressure and hydrostatic pressure was no more than 0.2 bar;
- the cone resistance was not less than 4MPa; or
- The fines content in the gradings were less than 20% when the vibrocore samples were taken.

Non compliance of any tests was based on a continuous length of 1.00m or 10% of the total test length, should more than one non-complying interval of less than one metre have occurred.

3.1.4 Wheeldrive Seacalf

The Seacalf system was developed by Fugro in 1972 for carrying out offshore geotechnical investigations for jack up rigs, production platforms pipe lines and other offshore structures in water depths from 10m to 500m.

The "Wheeldrive Seacalf" unit (Plate 5a) is a further development of the Seacalf which was introduced into Hong Kong

for the Chek Lap Kok trial embankment in the mid 1980's.

The penetration force for the heeldrive Seacalf is provided by a wheeldrive system (Plate 5b) in which four wheels grip the test rod and when rotated move it up or down. The wheels are driven from a power pack mounted on a seabed frame. The frame is 3.4m high and its base is 3.00 m x3.00m and weighs approximately 100 kN in air. For extra reaction this can be increased to 260kN by the addition of lead ballast blocks. The unit can be set up to provide 100kN (10 tonnes) or 200 kN (20 tonnes) of penetration thrust by using one or two wheeldrive systems.

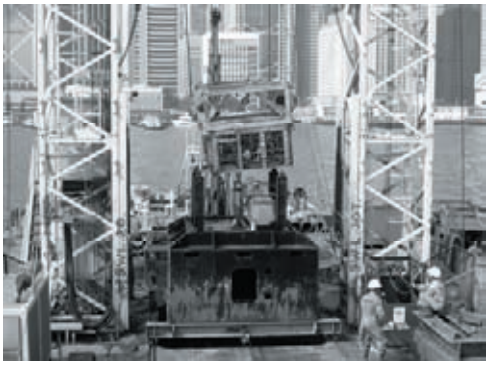

Plate 5a. Fugro 20T Wheeldrive Seacalf Unit

Plate 5b. Wheel drive mechanism

3.1.5 Seismic Cone Penetration Testing

The seismic cone penetration testing was undertaken for the Tung Chung New Town Development in 1995. This test is a reliable and cost effective technique to determine the insitu seismic wave velocities.

The seismic test method consists of measuring the travel times of body waves propagating between a wave source on the ground surface and an array of geophones in an in-situ seismic cone penetrometer. These body waves comprise shear or secondary waves (S-waves) and compressional or primary pressure waves (P-waves).

The seismic wave velocities give an indication of ground characteristics, such as low-strain shear modulus and Poisson's ratio. These parameters are relevant for assessing the response of dynamically loaded foundations and for earthquake studies. In particular, earth-quake resistant design of structures according to the Eurocode 8 standard require classification of stratigraphic profiles on the base of seismic wave velocities. In addition, Eurocode 8 recommends Cone Penetration Tests (CPT) for stratigraphic profiling and for assessment of soil liquefaction potential. Seismic wave velocities also allow correlation of seismic profiling data and geotechnical borehole and CPT results.

3.2 Pressuremeter Testing

The borehole pressuremeter was originally developed by Menard in the 1960's in France but was not widely used in Hong Kong until after 1980, but it is now routinely undertaken on many site investigation projects.

The test is a load test carried out in a drillhole. An inflatable cylindrical probe is installed at the base of a drillhole within a soil or weak rock mass. The method of installation of the probe is determined by the material characteristics. The test pocket can be formed by either a displacement sampler in soils or a core barrel in weak rocks. The equipment comprises a probe, a control unit, a pressure source (a compressed gas cylinder) and coaxial tubing to deliver pressurizes gas and liquid to and from the control unit and the probe. Once the probe is in place the probe is submitted to equal increments of increasing pressure. The probe volume changes are recorded at 30 and 60 seconds after each pressure stage is reached. The pressure volume data is plotted to determine the limit pressure P_L (used to calculate the bearing capacity) and the pressuremeter modulus (used to evaluate absolute and differential settlements for specific foundation designs).

The use of the a displacement sampler or core barrel to form the test pocket results in stress relief in the test pocket which in extreme cases causes the borehole to collapse. In order to minimize the stress relief the self boring pressuremeter was developed in the UK in the 1970's. Self boring pressuremeter tests have been undertaken in Hong Kong on several projects since the late 1990's. One such project was Contract No. HK/13/02 the Wan Chai Development Phase 2 ground investigation in 2003. A total of 129 tests were undertaken at 46 marine drillhole locations. The tests were carried out in soft marine and variable alluvial deposits as well as some tests in granite derived saprolite. Two types of equipment manufactured by Cambridge In-situ were used, a conventional Self Boring Pressuremeter (SBP) and a Weak Rock Self Boring Pressuremeter (WRSBP). The equipment is essentially the same but the WRSBP is fitted with a stronger membrane and the cutting system is arranged to drill slightly oversized. The SBP equipment used on this project is shown in Plates 6a and 6b.

Plate 6a. Self Boring Pressuremeter Drill head

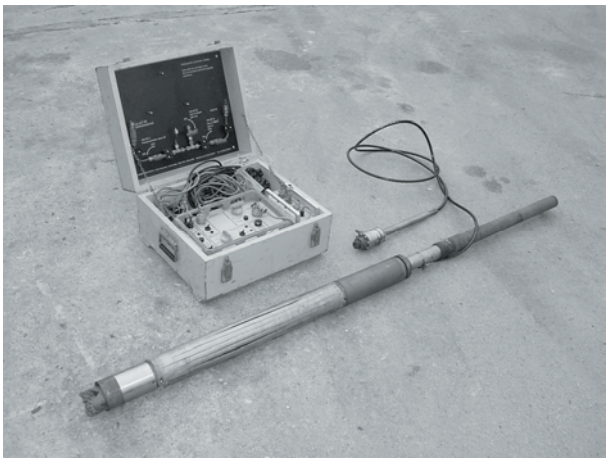

Plate 6b. Self boring Pressuremeter

A research project was reported by Schnaid et al (2000) on a series of tests undertaken on a site in Kowloon Bay adjacent to the old Kai Tak Airport Site. A total of nine drillholes were undertaken in which Standard Penetration Test (SPT), Self boring pressuremeter (SBP) and Marchetti dilatometer (DMT) tests were undertaken.

The loading and unloading portions of the SBP curves were analysed and results compared with those from other tests. The following constitutive parameters were obtained for the granitic saprolitic encountered at the Kowloon site:

i) Values ranging from 40MPa to 100MPa in the low-strain range (γ=0.01 to 0.1%). For higher strain ranges to should be selected from within the range of 22-40MPa
ii) The peak plane strain friction angles were in the range 33°-40°, varying with depth
iii) The critical state friction angle ϕ'_{cv} was close to 31°
iv) K_o was in the range 0.4-0.6

3.3 High Pressure Dilatometer and Goodman Jack Tests

High Pressure Dilatometer and Goodman Jack Tests were undertaken as part of the West Rail advance pile test study reported by Hill and Plumbridge (2006).

These tests were used to derive in situ modulus values which were correlated against values obtained from full scale pile loading tests. As a result of this testing programme Arup successfully achieved the following:

- Bearing capacities 50% higher than normally adopted presumptive bearing values for use in rational design;
- Approval to use shaft grouting and the enhanced shaft friction capacities associated with it; and
- Justification of higher ground stuffiness parameters than normal prescriptive values for laterally loaded piles.

3.4 Hydrofracture Testing

This test involves the high pressurization of an isolated borehole section which is pressurized until the wall rock fractures or pre-existing fractures open. After the test equipment is removed from the hole the trace of the induced or stimulated fracture is orientated with an impression packer or an acoustic televiewer.
In situ stress measurements in Hong Kong have been taken using the hydraulic fracturing technique since 1990 for more than 12 sites involving site investigations for railway tunnels, water scheme projects or caverns. This has involved some 250 tests conducted in some 27 drillholes drilled up to depths of 200m. This set of data has been described as the best hydrofracture stress data base by Rummel (2002). Part of this data was analysed by Klee et al (1999). The analysis indicated that although the tests were performed in fractured and unfractured crystalline rocks and the topography varied widely the results yielded a consistent orientation of the maximum horizontal stress of N108 ± 28°.

3.5 Optical and Acoustic Televiewer Survey

The Acoustic Borehole Televiewer was first developed for the petroleum industry in the late 1960's to enable a more accurate assessment of formation fracturing and drillhole inspection.
The current tools for site investigation imaging are developments of the original tools which were required to be slimmer, shorter and lighter than the original versions without any loss in resolution or accuracy.

Plate 7. Acoustic Televiewer Probe

The acoustic televiewer (Plate 7) was introduced to Hong Kong in the mid 1990's as an alternative to the traditional impression packer discontinuity survey tool. The probe is lowered down the drillhole on a wireline attached to a powered winch incorporating a depth encoder. Upon reaching the base of the hole the logging parameters are set on the portable computer and the televiewer is withdrawn at a rate of around 1.00m per minute. The probes work by emitting an acoustic signal which is reflected by the borehole walls. The travel time and amplitude of the return signal is dependant on the size of the borehole at that point and the material reflecting the signal. Proprietary logging software produces a coloured plot (Plate 8) of travel time and amplitude from which the presence of discontinuities can be interpreted. The dip and direction of the discontinuities are obtained by fitting a sine wave to each of the fractures and the signals are orientated using a built in magnetometer. The probes typically sample at rate of 3000 samples per second and if logging at a speed of around 1.00m per minute they can take a full scan of the drillhole wall every 1-2mm depending on the probe type.

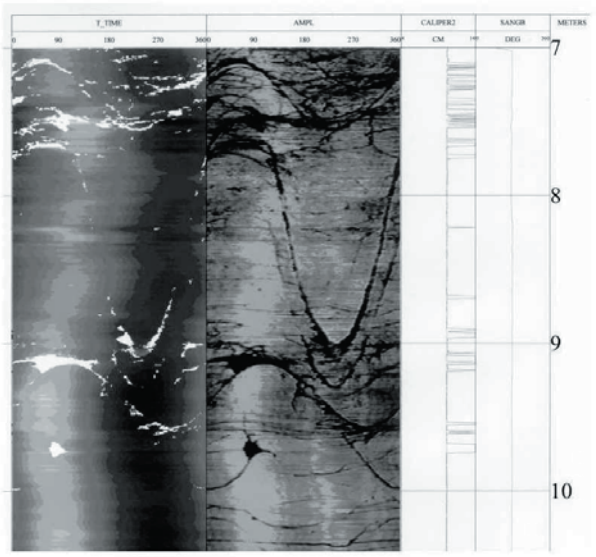

Plate 8. Acoustic Televiewer Record

The main advantages of the televiewer over the impression packer are:

- Wireline operation means the whole process is much quicker.
- Much more accurate. There are many processes involved with the impression packer in transferring the north orientation line onto the film and then from the film to the rock core. It is very easy to make mistakes.
- Much less labour intensive.
- Fracture orientation is an automated process.
- Can be used in inclined drillholes

If care was not taken it was possible with some early acoustic probes to misalign the magnetometer and the acoustic emitting sections of the probe, which resulted in consistent orientation errors. A couple of unfortunate incidents lead to a general mistrust of the acoustic televiewer data in the late 1990's. However this problem has been eradicated from the latest generation of probes and is now very uncommon to be asked to use an impression packer to undertake borehole discontinuity surveys.

One unavoidable limitation to the acoustic televiewer technique is that it requires the drillhole to be full of fluid to enable the acoustic signal to be transmitted. In highly fractured ground it can often be problematic to maintain the water level high enough to carry out the survey. Recent power advances and price reductions in portable computers enabled equipment manufacturers to produce optical televiewer probes which use most of the same equipment and software as the acoustic probe.

The optical probe produces a digital optical image and can be used in dry drillholes. Its disadvantage is then when testing below the water table the groundwater must be clear to obtain any results. The optical televiewer was introduced to Hong Kong in the late 1990's and is relatively frequently used to this day.

4 ADVANCES IN GEOPHYSICS

Geophysical and hydrographical methods are usually reliant upon electronic systems in some form, for data acquisition, data processing and data presentation. So it is not surprising that with the rapid advance of digital technology, particularly in terms of ever increasing processor speeds, available data storage capacities and decreasing component sizes, some geophysical and hydrographical systems have benefited significantly in performance. This applies especially to those marine techniques that generate large volumes of data, such as side scan sonar, multibeam echo sounding and to a lesser extent single channel reflection seismic methods. For these methods, the advances in digital technology and software development have enabled a wider spectrum of data acquisition coupled with real time data processing, enhanced digital display, and importantly, the facility for continuous data storage in digital form. The large volume digital storage has facilitated post acquisition data re-processing which was not available previously with printed paper records.

Commercial software development, particularly for PCs, has led to substantial improvements in the way data are presented, whether by filtered data profiles, shaded image maps or even 3-D rotating visualisations, all of which help to convey the required information between the geophysicist and the engineer, too often a weak link in the chain.

With one or two exceptions, onshore geophysical acquisition methods have not changed quite to the same degree as marine methods, at least in part because land data tend to be collected as discrete sample points and the site conditions of Hong Kong often remain an overriding limiting factor. Some advances have been made in instrument design though, equipment generally becoming more efficient and portable. For example, measurement of resistivity sounding data is now typically performed on multi-channel digital recording equipment which, coupled with modeling software enables continuous sectional profiles of the ground to be generated. Electromagnetic (EM) ground conductivity methods have also been introduced to Hong Kong in recent years, for shallow ground contamination and void studies, although the method is not a new development. Refraction seismic survey, the principal onshore geophysical method for geotechnical investigations remains relatively unchanged.

The advances that have taken place have done so against a backcloth of survey prices that have plunged over the last several years and even now remain stubbornly low. In fact whilst margins are low globally outside the oil geophysical sector, there is little doubt that the present levels of geophysical pricing in HK are barely sufficient to support the necessary capital investment for the

latest technologies.

Summarising, there have been some significant technical improvements especially in methods that generate high volumes of data, and this has been largely on the back of developments in digital hardware and commercial software. Fundamentally, the techniques themselves have not changed greatly.

5 DATA MANAGEMENT

During the 70's and early 80's all data from ground investigations such as drillhole records, in situ test results etc. was simply typed up on to printed base forms by the contractor (with hand-drawn or stuck-on legends) and presented in a report for use by clients and consultants. Beside the labour intensive method for the contractor the problem for the end users was that data contained in the report would then be copied out for use in interpretation and design with the opportunities for errors in copying.

By the Mid 80's various software programs were becoming available that allowed the fast input and repeated output of data to produce drillhole records. However the lack of industry standards and the incompatibility between systems used by the contractors and the end users of the data did not allow for easy transfer of the data collected from the ground investigation works. Such data would still require inputting by copying from the submitted reports.

The major change was the advent of more complex software which operated as a database rather than a word processing package. A wide range of the data produced by a ground investigation could be input and printed logs and records output and the database used to extract data for use in other software for design purposes etc. The other advantage was the requirement to only input the data once thereby reducing the workload for checking.

The report production software which has been most widely adopted in Hong Kong is gINT which became the industry standard for GEO works in the early 90's.

The Association of Geotechnical and Geoenvironmental Specialists (AGS) based in the UK recognized the need for a simple format to transmit digital data without the requirement for the sender and receiver to have identical word processing or report production software. A working party was set up and published the First Edition in 1992 and this was adopted by GEO in 1993 as the medium for digital data exchange in ground investigation term contracts.

Following the success of the First Edition in the UK the updated Second Edition was published in 1994. The Second Edition was widely used in Hong Kong by client organisations, consultants and contractors. During its use the Second Edition was subject to many interpretations and adaptations throughout the Hong Kong ground investigation industry as users modified the format in varying ways to suit the local conditions. The Third Edition addressed the need for changes to the UK based format to take into account the variations required for Hong Kong and in 2000 was adopted by the GEO for both ground investigation fieldwork and laboratory testing in term contracts.

The adaptation of gINT to support AGS and output data in the format was begun with Second Edition in the mid 90's but due to the variations which occurred in Hong Kong the results required extensive manual checking. By 2000 the use of AGS Third Edition and advances in the gINT software for AGS production with built-in checking system had greatly improved the efficiency of production of useable AGS data by contractors and increased the confidence of the end users in the transmitted data.

Further improvements to the usability of gINT have taken place regularly since, generally to ease the data input and to allow the output of a wider range of report forms, drawings and other data representations by the increasing compatibility with specialist software including GIS.

Another attempt at automating site investigation report production has been the introduction of PDA drillhole logging tools for geologists. Many proprietary versions have been introduced to the world wide market in the last decade and some have been developed in house by local contractors. The first version in Hong Kong used a Psion Walkabout PDA which was very robust and enjoyed a relatively successful honeymoon period and was sold to many of the local contractors. However the development costs were high and technology moved on so fast that the Psion became quickly outdated. A second generation tool incorporating a wireless modem meant that the geologist could transmit his logging back immediately it was finished rather than go back to the office to download the data via cable. The general comment from local engineers is that the logging produced via the logging tools is too prescriptive and often of insufficient detail. These tools are ideal for foundation predrilling contracts that require preliminary logs the day after the drilling is completed.

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Laboratory Testing of Soils and Rocks in Hong Kong's Universities

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Abstract: Geotechnical laboratory testing has historically been used to identify basic properties of soil and rock. Some standardised tests have become so widely accepted that they form an essential component of a comprehensive ground investigation. With the advances in laboratory testing techniques, more sophisticated testing equipment has been devised. These new testing systems not only facilitate the study of the complicated behaviour of geomaterials, but also help engineers to obtain project-specific parameters for their design calculations. This paper describes the development of laboratory testing techniques in Hong Kong. The discussion focuses on strength and stiffness of soil and rock, which have seen the most significant advancement over the last 30 years. Some important soil/rock properties observed through these state-of-the-art testing techniques are also presented.

1 INTRODUCTION

Unlike other construction materials, the quality and properties of geomaterials vary significantly depending largely upon the geological processes through which the soil/rock has gone. This has made geotechnical laboratory testing particularly important as it serves as a platform for defining the design parameters; no standard properties of a geomaterial can be adopted until it has been tested in a laboratory. Many tests have therefore become essential components of a comprehensive ground investigation. The standard measurements obtained in these conventional tests have also been routinely adopted as input design parameters.

Although the procedures in these common laboratory tests have been well established and standardised, some of the testing conditions are tremendously simplified from the true field conditions. With the advances in laboratory testing techniques, more sophisticated testing systems have been invented over the last 30 years. These systems share the same goal of exploring the complicated behaviour of geometerials, under conditions as close to the field situation as possible. This paper presents the development of laboratory testing techniques in Hong Kong. Among the wide range of properties covered by laboratory testing, the study of shear behaviour, including the strength and the stiffness, has seen the most remarkable advancement. These aspects are the focus of this paper. Some important soil/rock properties observed through these state-of-the-art testing techniques are also presented.

2 TESTS ON SATURATED OR DRY SOIL

2.1 *Shear strength*

2.1.1 Conventional shear box and triaxial tests

The study of shear strength of Hong Kong soils was pioneered by Professor P. Lumb at Hong Kong University (HKU) (Lumb, 1962, 1964, 1965). In his paper describing the development of a new soils laboratory (Lumb, 1954), he reported the details of the two shear boxes housed in the laboratory and the plans to install a triaxial compression testing machine. To date, these two types of tests are still the most widely accepted shear strength tests in Hong Kong as well as other parts of the world. The historical development of the two testing techniques is briefly described below.

Direct shear test is the oldest and simplest way to evaluate soil shear strength. According to Head (1982), the earliest attempt to measure soil shear strength was made by the French engineer Alexandre Collin in 1846. In his apparatus, a 40 mm × 40 mm sample was subjected to double shear under a load applied by hanging weights. The earliest version of a shear box apparatus was devised by Bell (1915). Nevertheless, the modern form was actually designed by Professor A. Casagrande at Harvard University in 1932 (Rutledge, 1935). Since then, different versions based on this prototype have been developed (e.g. Cooling & Smith, 1935; Gilboy, 1936; Golder, 1942; Bishop, 1948).

Triaxial tests are considered more versatile than shear box tests. Although the technique was introduced more than 70 years ago (early published results can be found in Terzaghi (1932)), the principal features remain roughly the same except that many of the instruments have been superseded by modern electronic devices. Detailed descriptions of the conventional triaxial apparatus and the standard procedures are given in Bishop & Henkel (1962) and Head (1982). The conventional way of measuring the shear strength of a soil is to carry out several single-stage triaxial tests on "identical" specimens to derive the shear strength parameters, c and ϕ. In order to overcome sample variability, multi-stage tests may sometimes be conducted. References to multi-stage triaxial tests are given in Kenney and Watson (1961) and Lumb (1964).

Despite the obvious advantage of testing the "same" specimens, a study conducted by Pun & Ho (1996) suggested that the shear strengths from the single-stage tests were higher than those of the multi-stage tests, presumably due to progressive loss of inherent structure and residual bonding. Due to this reason, single-stage tests are more preferably to multi-stage tests.

Since the establishment of Geotechnical Control Office (GCO) (now Geotechnical Engineering Office (GEO)) in 1977, significant amount of research has been carried out to quantify the undisturbed shear strength of Hong Kong residual soils using the shear box and triaxial testing techniques (e.g. Massey 1983; Gan & Fredlund, 1996). Limited tests were also carried out on reconstituted samples in an attempt to interpret the soil behaviour using the critical state soil mechanics framework. A set of generalised shear strength parameters for completely decomposed granite (CDG) was reported by Pun & Ho (1996).

Dead loads are sometimes used for applying the axial stress in a triaxial compression test – a test referred to as dead load test. Brand (1981) considered that the stress path in a dead load test simulates that of soil elements in a slope subject to an increase in pore-water pressure, and therefore the shear strength parameters so obtained are more relevant than those measured from conventional triaxial compression tests. It was noted in the Working Party Report on Residual Soils (GSL, 1990) that the strain required to reach failure in a dead load test is typically much smaller than that in a conventional triaxial test and that a reduced destruction of bond may be responsible for a generally higher measured shear strength in a dead load test.

In an attempt to study the possibility of static liquefaction failures in loose fill slopes in Hong Kong, several studies were conducted on loosely compacted CDG and completely decomposed volcanic (CDV) to examine the loss of shear resistance of these extremely loose materials under undrained shearing (Gray, 1980; Law *et al*., 1997; PWCL, 1998). Apart from using recompacted specimens, these tests were operationally identical to conventional consolidated undrained tests.

2.1.2 Stress path controlled triaxial tests

More advanced stress path controlled triaxial tests were carried out at the Hong Kong University of Science and Technology (HKUST) to shed light on the collapse behaviour of loosely compacted CDG, which was postulated to be the cause of static liquefaction behaviour (Ng *et al.*, 2004a). Details of the automated triaxial system can be found in Li *et al*. (1988). The tests considered a decreasing mean effective stress path under constant deviator stress conditions (CQ). The stress path was imposed under drained conditions by increasing the pore pressure at a very low rate with the top and bottom drainage lines connected to a pore pressure controller, whereas the applied total vertical and horizontal stresses were held constant throughout the tests. This type of stress path was intended to mimic the loading of soil elements within a slope subjected to a slow increase in pore water pressure or a reduction in soil suction due to rainfall or a rising groundwater table.

The results of the CQ tests shown in Fig. 1 suggest that the loosely compact decomposed granite exhibited structural collapse behaviour. The collapse was characterised by a sudden large development in both axial and contractive volumetric strains. The mobilised friction angle at collapse (ϕ'_{col}) ranged from 31.8º to 38.7º, which are lower than or equal to the critical state friction angle of 38.7º (M=1.58) depending on the initial stress ratio ($\eta_c=q_c/p'_c$) (Fig. 2). Moreover, the measured ϕ'_{col} are consistently higher than the mobilised friction angle of the instability line (28.1º) obtained in conventional consolidated undrained tests (Fig. 1).

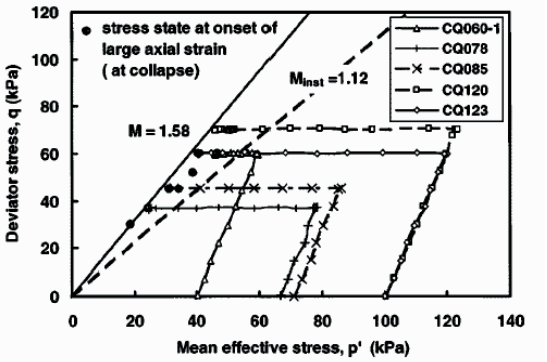


Fig. 1. Effective stress path of constant deviator stress (CQ) tests (after Ng *et al*., 2004a).

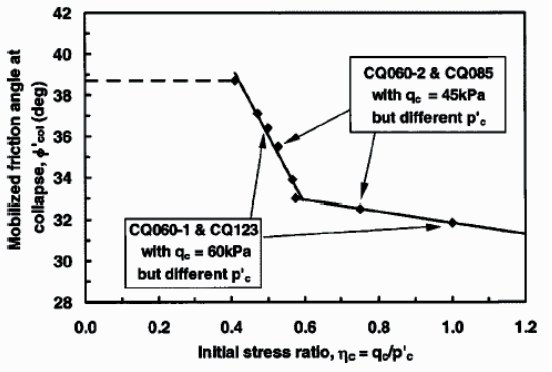


Fig. 2. Effect of initial stress ratio on mobilised friction angle at collapse of CQ teets (after Ng *et al*., 2004a).

2.1.3 True triaxial machine

The conventional triaxial machine assumes that either the minor principal stress (during extension) or the major principal stress (during compression) equals the intermediate principal stress. A true triaxial system (TTS) that allows the three principal stresses to be applied independently was purchased and installed in the Soil Mechanics Laboratory of Hong Kong Polytechnic University (PolyU) in 1999. The soil specimen is a brick-shaped one with a height of 150 mm and a cross-section area of 70 mm by 70 mm.

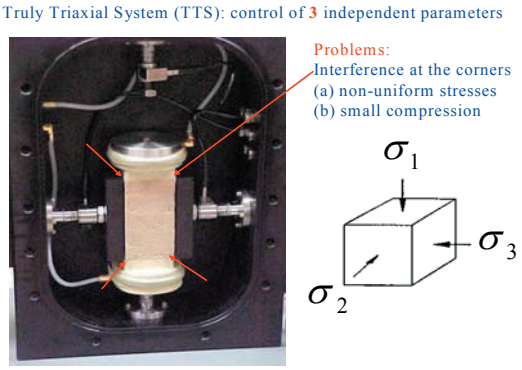


Fig. 3. The corner interference problems with the true triaxial cell using the old loading plates.

The vertical and lateral stresses are applied using rigid plates and oil jacks. They represent the major principal stress and the middle principal stress respectively. The minor principal stress is applied by oil pressure in the chamber when the door is closed (see Figs 3 and 4). The TTS can control three independent variables; for example, the vertical stress σ_z, the lateral stress σ_y and the other lateral stress σ_x (see the 3-D specimen in Fig. 3).

However, using the old loading plates, the top cap and the bottom cap will be in contact with the two side rigid plates during compression of the soil specimen (Fig. 3). The manufacturer suggested leaving a suitable gap between the top (or bottom) cap and the side plates, which would lead to highly non-unifrom stresses and strains inside the spoil specimen. The old plates are therefore not suitable for soil testing.

In fact, the design of a loading system for true triaxial testing has been a very challenging research topic with a long history. The earliest design was the Cambridge type of six rigid loading plates. This design had the following problems:

(i) Difficulty in installation of a soil specimen, and
(ii) Non-uniform stresses and strains inside the specimen.

Due to these problems, the Cambridge type loading device is not commonly used nowadays. Another type consists of six flexible loading bags. The bag is a rubber bag filled with de-aired water. However, two bags next to each other shall be in contact. When a soil specimen is compressed, the two bags will affect each other except that a special measure is taken to hold the bags. But this will cause non-uniform strains. When the compression is larger, the problem becomes more serious.

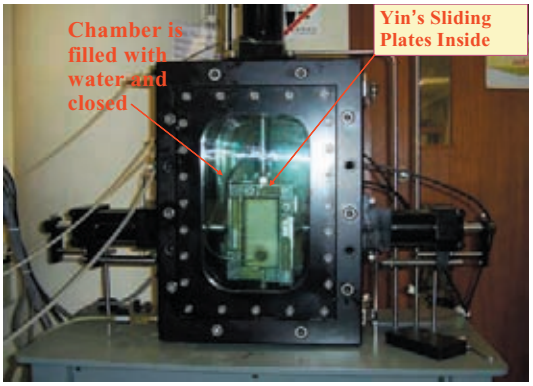


Fig. 4. The true triaxial cell with Yin's (2006a) sliding plates and a soil specimen in a rubber membrane during testing.

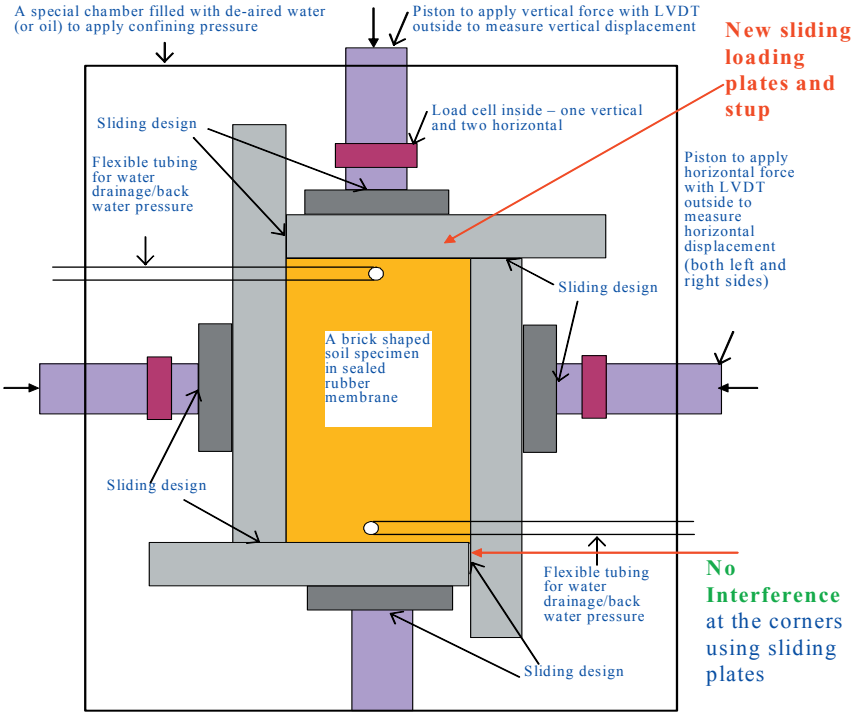


Fig. 5. A schematic view of the innovative four sliding rigid loading plates and two flexible loading faces (Yin, 2006a).

To solve the above problems, an innovative loading system was designed by Yin (2006a) (Figs 4 and 5). The new design consists of only four sliding rigid plates (top and bottom; left and right). A brick-shaped soil specimen is enclosed in a rubber membrane of the same shape and size. The whole soil specimen in membrane and the four sliding plates are all enclosed in a chamber as shown in Figs 4 and 5. Thus, the front and the back faces of the brick-shaped specimen are loaded by water pressure inside the chamber. This TTS with the new loading plates have been used to test CDG (20 tests done) and a foam (15 tests done). The compression strains were up to 50%.

2.2 *Shear stiffness*

Stress-strain behaviour of soil is highly non-linear. In order to measure soil stiffness at very small strain level ($\gamma < 10^{-5}$), Shirley & Hampton (1978) developed a shear wave transducer utilising piezoceramic bender elements to measure the shear wave velocity, V_s, in a soil specimen, which can be used to calculate the shear modulus by:

$$G_0 = \rho \cdot V_s^2 \qquad (1)$$

where ρ = soil density.

For studying stiffness anisotropy, the techniques of multidirectional shear wave velocity measurement can be adopted (Thomann & Hdyciw, 1990; Jamiolkowski *et al*., 1995; Viggiani & Atkinson 1995), in which bender elements are installed in different directions in laboratory testing equipment. Recent advances have also enabled the measurement of the velocity of horizontally propagated shear waves with vertical and horizontal polarisation ($V_{s(hv)}$ and $V_{s(hh)}$).

One important consideration in the assessment of small-strain stiffness is accurate measurement of the strain level. Measurements of very small strains can be made by local strain transducers. Different types of transducers are available and detailed reviews of these devices are given in Scholey *et al.* (1995) and Yimsiri & Soga (2002).

To measure small-strain stiffness of decomposed soils in Hong Kong, a computer-controlled triaxial stress path apparatus equipped with local strain transducers and bender elements was developed at HKUST (Ng *et al.* 2004b; Ng & Leung, 2006a, b) (Fig. 6). Local axial and radial strains at the mid-height of the specimens were measured by Hall Effect transducers. A mid-plane pore pressure probe was used to measure the pore pressure at the mid-height of the specimens. Three pairs of bender elements were used to measure the shear wave velocity in three orthogonal planes of a soil specimen.

Using the apparatus shown in Fig. 6, a series of tests were conducted on different types of decomposed soils. The results obtained from completely decomposed tuff (CDT) are presented in Fig. 7 for illustration. Block and Mazier specimens were subjected to isotropic and anisotropic effective stresses. The stress ranges considered for isotropic loading were 80 kPa to 400 kPa and 100 kPa to 400 kPa for Block and Mazier specimens respectively. For anisotropic loading, the effective stresses ranged from $p' = q = 80$ kPa to $p' = q = 400$ kPa, while the stress ratio ($\eta = q/p'$) was kept constant at $\eta = 1.0$, corresponding to $K = \sigma'_h/\sigma'_v = 0.4$. Shear wave velocities in three orthogonal plances, $V_{s(hv)}$, $V_{s(hv)}$, and $V_{s(hv)}$ were measured to determine the inherent anisotropy of the material. By plotting the shear modulus against the product of the effective stresses in the corresponding shear plane, the influence of effective stress on shear stiffness can be taken into account.

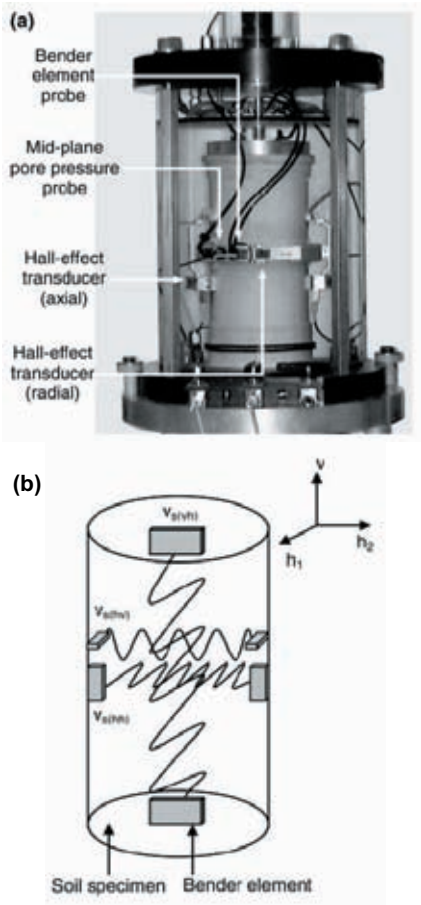


Fig. 6. Triaxial apparatus with multidirectional shear wave velocity and local strain measurements (after Ng and Leung, 2006a, b).

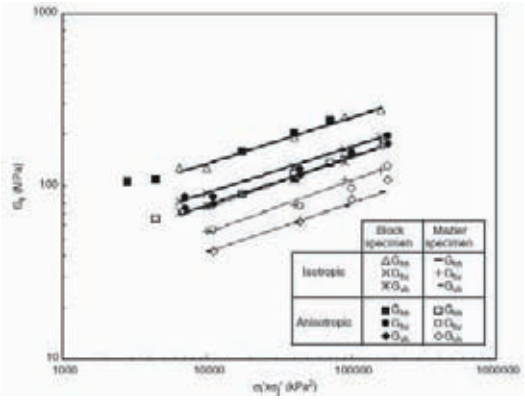


Fig. 7. Variation in shear modulus (G_{ij}) with product of effective stresses ($\sigma'_i \times \sigma'_j$) of CDT in block and Mazier specimens (after Ng and Leung, 2006a).

The results plotted on Fig. 7 suggest that CDT shows stiffness anisotropy, with the shear modulus in the horizontal plane (G_{hh}) higher than that in the vertical plane (G_{hv} or G_{vh}). The results obtained from anisotropic stress states are generally consistent with the data obtained in isotropic stress states after considering the effective principal stresses in the shear planes. The stiffness ratio between the shear modulus in the horizontal and vertical planes

(G_{hh}/G_{vh}) ranges from 1.26 to 1.56. The Mazier specimens consistently show a lower degree of anisotropy and lower shear moduli, G_{hv}, G_{vh} and G_{hv}, presumably due to sample disturbance.

The stiffness parameters derived from the laboratory investigation were utilised in numerical analysis to look at the influence of the inherent stiffness anisotropy on ground deformations around typical multi-propped deep excavations. The computed results showed that maximum wall deflection and ground settlement due to the pumping of groundwater prior to any excavation were 8% and 19% greater, respectively, than those of an isotropy analysis. The maximum wall deflection and ground settlement because of the combined effects of the pumping and recharging of groundwater inside the site and the subsequent multistage excavation were 15% and 10%, respectively less in the anisotropic analysis. Details of the analyses can be found in Ng *et al*. (2004b).

2.3 *Dynamic properties*

2.3.1 Resonant column test

Resonant column devices are widely used to measure dynamic properties. Nevertheless, conventional resonant column testing has the shortcomings of the prestraining effect due to a large number of loading cycles applied for a single measurement (Drnevich & Richart, 1970; Anderson & Richart, 1976; Vucetic, 1994) and the difficulties in controlling the loading frequency or strain rate which cause loading rate effects (Kim, 1991; Zavoral & Campanella, 1994).

To overcome the above problems associated with resonant column testing, an innovative control system has been devised by Li *et al*. (1998). The system employs the concepts on energy injection and virtual mass. In the system a steady-state vibration is maintained by feeding back a torque proportional to the angular velocity of vibration such that the energy loss due to damping is automatically compensated, and a pre-specified resonant frequency is reached by feeding back a torque that is proportional to the angular acceleration such that the inertia term in the equation of motion matches the stiffness of the specimen for the specified resonant frequency. Calibration tests were conducted on an aluminium rod (Li *et al.*, 1998) which showed that the feedback control system was capable of producing continuous and automatic measurement of modulus and damping properties under clearly defined test conditions. With the fully automatic operation, the number of cycles can be minimised, implying minimal disturbance to the soil specimen under testing. Details of the principle of the new system can be found in Li *et al.* (1998).

2.3.2 Spectral ratio method (SRM)

An alternative way to obtain the damping ratio of soil is to use the spectral ratio method (SRM). Wang *et al.* (2006) described a study in which bender elements were used as sensors to measure the damping ratio of soil using such a method. The SRM is based on the fact that the total decay of the wave amplitude with distance across the specimen can be expressed as:

$$ln\left[\frac{YW_1}{YW_2}\right]=\left[\frac{2\pi D}{V_s}(r_2-r_1)\right]f-ln\left(\frac{H_{R1}{}^{-1}(f)}{H_{R2}{}^{-2}(f)}\right)-ln\left[\left(\frac{r_1}{r_2}\right)^{\beta}T\right] \quad (2)$$

where D = damping ratio;
V_s = shear wave velocity;
YW_1, YW_2 = windowed receiver signals in the frequency domain at receivers 1 and 2, respectively;
H_{R1}, H_{R2} = transfer functions of receivers 1 and 2, respectively;
r_1, r_2 = distances between the source and receivers 1 and 2, respectively;
β = geometric spreading factor, and
T = transmissivity coefficient of the propagating wave.

The damping ratio, *D*, is therefore the slope of Eq. (2) if the spectral-ratio ln $[YW_1/YW_2]$ is plotted against frequency. The accuracy of the measurements using the above method is, however, adversely affected by two factors (i) the near-field effect, and (ii) the different transfer functions of the two receiver bender elements. To avoid the near-field effect, Wang *et al.* (2006, 2007) proposed to set the sensors far enough from the source such that $r_1/\lambda \geq 2$ and $r_1/r_2 \geq 2$, where λ is the wavelength. The benefit of these settings has been verified by physical and numerical experiments, although the arrangement has the obvious trade-off of receiving weaker signals from a practical point of view. To minimise the second effect, the SRM is modified by the introduction of the self-healing technique (Wang *et al*., 2006; Mok *et al.*, 2006) which is demonstrated in Fig. 8.

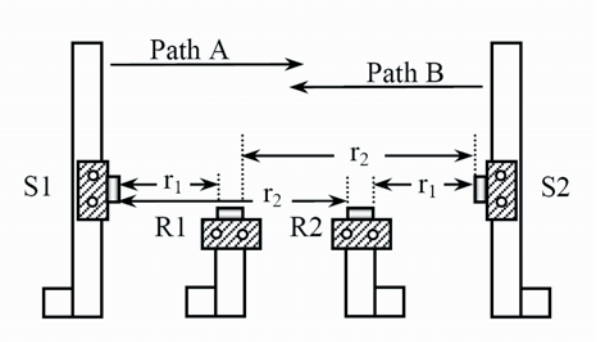
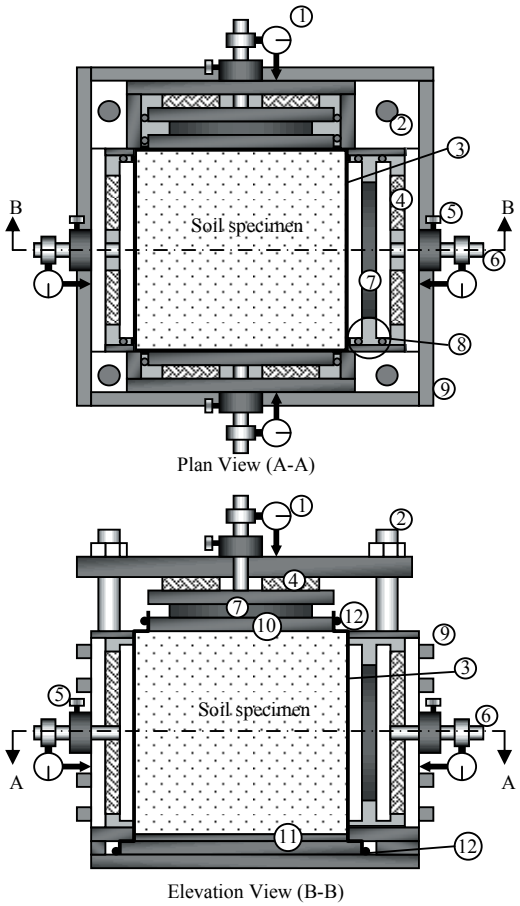


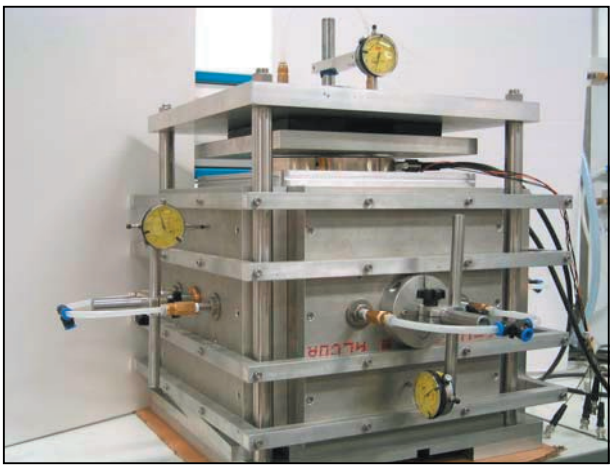
Fig. 8. Configuration of bender elements in self-healing technique (after Mok *et al.*, 2006)

With two independent measurements from two opposite wave propagation directions (Paths A and B), the dependency of the transfer functions in Eq. (2) can be eliminated, yielding:

$$ln\left[\frac{YW_{1A}\cdot YW_{2B}}{YW_{2A}\cdot YW_{1B}}\right]=\left[\frac{4\pi D}{V_s}(r_2-r_1)\right]f-ln\left[\left(\frac{r_1}{r_2}\right)^{2\beta}T_AT_B\right] \quad (3)$$

where YW_{1A}, YW_{1B} = windowed receiver 1 signals in frequency domain for paths A and B, respectively;
YW_{2A}, YW_{2B} = windowed receiver 2 signals in frequency domain for paths A and B, respectively, and
T_A, T_B = transmissivity coefficients for waves propagating in path A and B, respectively.





1. Dial gauge
2. Supporting rod
3. Membrane
4. Air bag
5. Locking screw
6. Guide rod
7. Load cell
8. Bearing system to reduce friction during loading/unloading
9. Reinforced bar
10. Top plate (with drainage holes)
11. Bottom plate (with drainage holes)
12. O-ring

Fig. 9. True triaxial apparatus at HKUST.

Using the SRM with the self-healing technique and careful considerations of the near-field effect, the damping ratio of dry Toyoura sand has been measured in a tailor-made true-triaxial apparatus (Fig. 9) (Wang *et al.*, 2006). The initial density of the sample is 1.6 g/cm^3, corresponding to a relative density of 85.6%. The inside dimensions of the apparatus are 300 mm × 300 mm × 300 mm. The confining pressures are applied by pressurising the air bags, which transfer the stresses to the specimen by moving the aluminium loading plates that are placed on the sides of the specimen. The unique feature of the true triaxial apparatus is the ability to apply independent stresses in the three principal stress directions. This allows anisotropic stress states to be applied to the specimen. Fig. 10 compares the measured damping ratios by SRM integrated with the self-healing technique and those from the resonant column test at the strain level less than 10^{-6}. The two measurement techniques yield not only a comparable trend, i.e., a weak dependence on confinement, but also very similar damping ratios. The average damping ratio in loading and unloading is 0.65% and 0.52%, respectively, for resonant column tests; 0.65% and 0.47%, respectively, for measurements in the true triaxial apparatus.

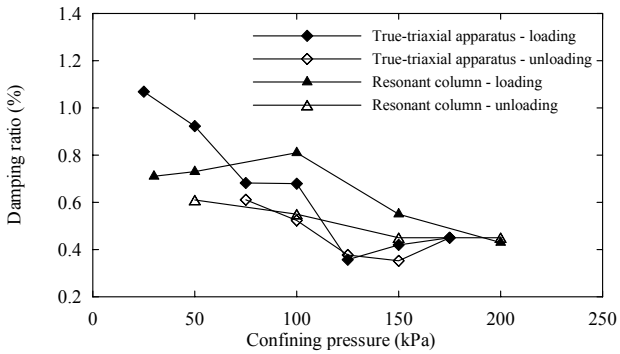


Fig. 10. Comparison of measured damping ratios from the experiment conducted in the true-triaxial apparatus and those obtained from the resonant column tests under the same state of stress (after Wang *et al.*, 2006).

2.4 *Wave-based characterisations*

Soils are particulate media and relevant physical principles behind the macro-scale engineering properties originate from "particle interactions". However, their discrete nature and multiple scales facilitate the unavoidable effect of “measurement on the measurand”. Therefore, the low-energy wave propagation offers unique possibilities to study such materials with minor disturbance. Both mechanical and electromagnetic wave-based techniques can provide complementary information to “hear” and “see” into the soil.

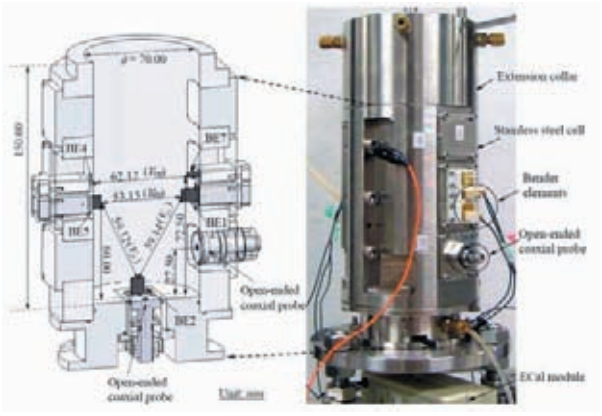


Fig. 11. The details of the tailor-made stainless steel cell (Wang & Dong, 2007a).

Wang & Dong (2007a) uses both mechanical and electromagnetic wave-based techniques to characterize the sedimentation process of clays. The details of the tailor-made cell used in their study are presented in Fig. 11. This cell has a diameter of 70 mm and a height of 150 mm with an extension collar of 60 mm. The bender element sets (BE) capture the variation of the shear wave velocity, V_s, during the sedimentation process in different polarization and propagation directions and the open-ended coaxial probes measure the change of the complex relative permittivity, κ^*, ranging from 10 MHz to 3 GHz. Within this measurement frequency range, the bound water polarization (the relaxation frequency, f_{rel}, is about 10 MHz) and the bulk (or free) water polarization ($f_{rel} \approx 20$ GHz) are prevailing. The spatial, double layer, and the electrode polarization, which gain relevance around the range of *kHz*, are not significant and can be ignored. Hence, the measured complex dielectric spectrum can be described by two Cole-Cole spectral functions plus the DC conduction loss:

$$\kappa^*(\omega) = \kappa_\infty + \frac{\Delta\kappa_w}{1+(j\omega\tau_w)^{\alpha_w}} + \frac{\Delta\kappa_b}{1+(j\omega\tau_b)^{\alpha_b}} - j\frac{\sigma_{DC}}{\omega\varepsilon_0} \quad (4)$$

where subscripts w and b represent the bulk and bound water, respectively;
asterisk signifies a complex number;
ω is the angular frequency;
κ_∞ is the permittivity due to ionic and electronic resonance (i.e., the permittivity at $\omega = \infty$);
$\Delta\kappa$ is the relaxation strength;
σ_{DC} is the DC conductivity;
ε_0 is the permittivity of the free space (= 8.854×10^{-12} *F/m*), and
τ is the relaxation time, and
α ($0 < \alpha \leq 1$) is the Cole-Cole parameter related to the distribution of the relaxation time.

Fig. 12 presents the experimental results. The velocities, V_{VH} (S-wave polarized in the vertical direction and propagating in the horizontal direction) and V_{HH}, continue to increase as sediment consolidation develops. The two velocities, V_{VH} and V_{HH}, are almost identical over the entire process because the randomly packed edge-to-face flocculation yields isotropic fabric associations. This open card-house structure also results in relatively higher relaxation strength of the bulk water, $\Delta\kappa_w$, owing to higher water content. Sediment consolidation gives rise to a decrease in the bulk-water relaxation strength but to an increase in the bound-water relaxation strength (owing to decreasing void ratio and increasing particle content).

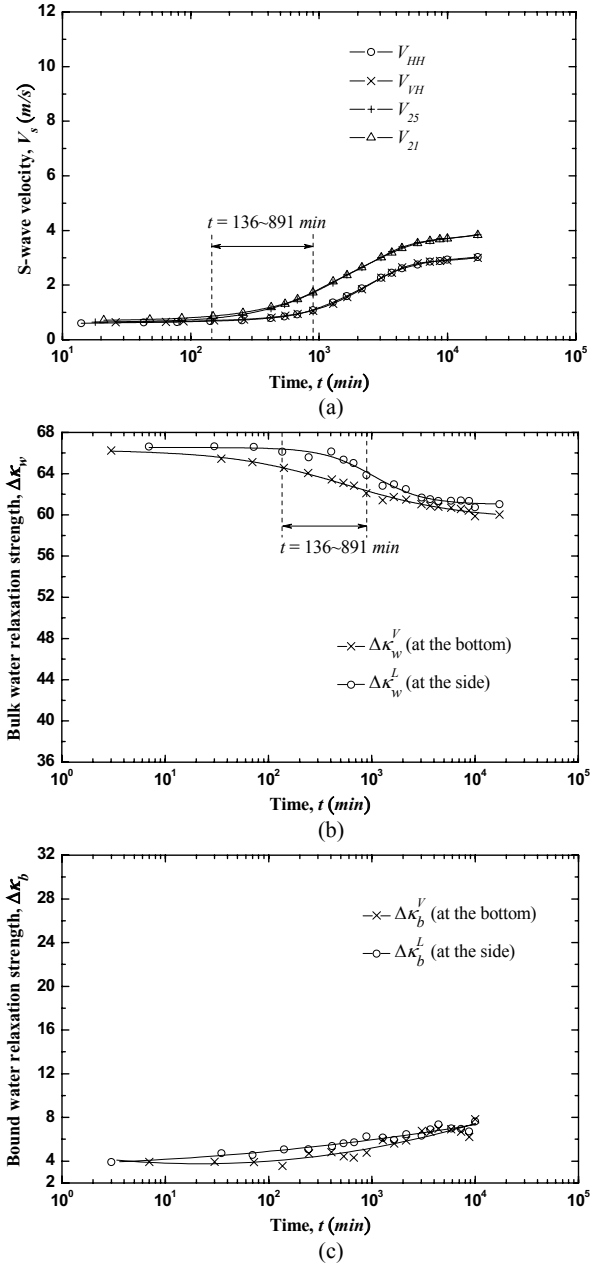


Fig. 12. Evolution of (a) the shear wave velocity, (b) the bulk water relaxation strength, and (c) the bound-water relaxation strength during sedimentation (after Wang & Dong 2007a).

Wang & Dong (2007b) uses a slim-form probe, which has an outside diameter of 2.2 mm and a length of 200 mm, to carry out local dielectric measurements ranging from 10 MHz to 3 GHz, by which the spatial variability along the sample depth can be explored. Fig. 13 illustrates the testing arrangement. The slim-form probe together with an Electronic Calibration module (ECal, Agilent 85093C-M0F) is tightly fixed onto a loading frame (AMETEK EZ50, Lloyd Instruments). This loading frame allows the slim-form probe to be steadily and accurately penetrated into the specimen in a rate of 5 mm/min. The sample is put into the stainless cell and the local dielectric measurement is car-

ried out in an interval of 5 mm. Fig. 14 presents the results of the local dielectric measurements in Leighton Buzzard sand; the bottom of the sample is set as level zero and the sample surface is marked as a dotted line. The relaxation strength of bulk water, $\Delta\kappa_w$, is directly related to the volumetric water content and therefore to the local porosity under the saturated condition. The conductivity of the sample, σ_{mix}, is relevant to the pore quantity and directional features of the interconnected pores, i.e. the tortuosity. The tortuosity, δ, here is defined as the actual travel distance divided by the length of the shortest path, which can be derived from σ_{mix} and the conductivity of the pore fluid, σ_{pf}. In Fig. 14, the small variation in $\Delta\kappa_w$, is consistent with the findings in published results: the wet pluviation method can produce a more homogeneous sample (i.e. lower variations of local void ratios). The relatively better graded Leighton Buzzard sand shows a higher tortuosity because the smaller particles fill the voids that are formed by larger particles and therefore the conduction path becomes more tortuous, i.e. a higher δ. Testing results of different types of soils, such as kaolinite with different fabric associations, and pore size estimation by the measured $\Delta\kappa_w$ and mixing rules can be found in Wang & Dong (2007b).

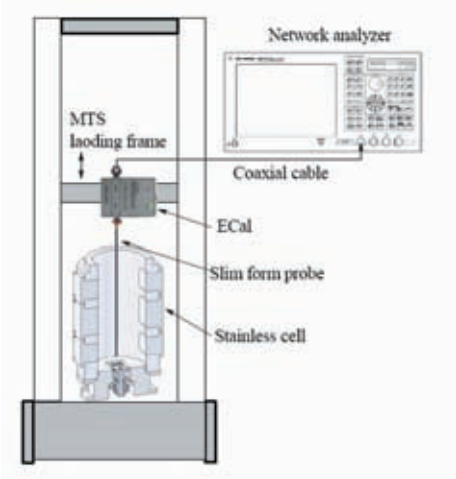


Fig. 13. Experimental setup of spatial variability characterisaion using a slim-form probe (Wang & Dong, 2007b).

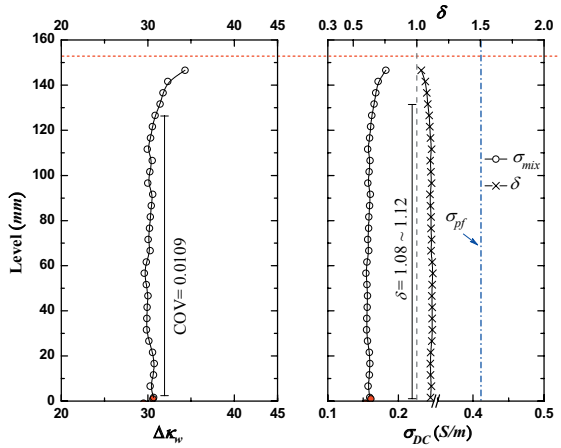


Fig. 14. Results of the local dielectric measurements in the Leighton Buzzard sand: (a) the relaxation strength of bulk water, $\Delta\kappa_w$; (b) DC conductivity of the sediment, σ_{mix}, and the pore fluid, σ_{pf}, and the tortuosity, δ. Note that COV is the coefficient of variation (after Wang & Dong, 2007b).

3 TESTS ON UNSATURATED SOIL

3.1 *Shear behaviour*

3.1.1 Modified shear box and triaxial testing

Research work on the shear strength of partially saturated soil in Hong Kong dates back to the 1960's (Lumb, 1965). Tests in the early days were conducted without adequate control and measurement on the pore-air and pore-water pressures. Although the increase of shear strength with decreasing degree of saturation was observed consistently, detailed interpretations of the results were ambiguous due to the unknown stress states at failure.

Fredlund and Morgenstern (1977) proposed to use two independent stress state variables to explain unsaturated soil behaviour. This is a milestone in unsaturated soil mechanics research as the proposal clearly defines the essential measurements in laboratory testing if one wants to understand the behaviour of partially saturated soils. Among the three possible combinations, the most widely used state variables are the net normal stress, $\sigma - u_a$, and the matric suction, $u_a - u_w$, where σ is the total stress and u_a and u_w are the pore air and pore water pressures, respectively.

With the appreciation of the need to independently control and measure pore-air and pore water pressures, modified shear box and triaxial devices have been devised. Fig. 15 shows the modified shear box developed at HKUST. The apparatus, originally designed by Gan *et al.* (1988) and modified by Zhan (2003), utilises the axis translation technique proposed by Hilf (1956). By applying a high air pressure, the water pressure can be maintained at a positive value even at a high matric suction such that cavitation can be avoided. The shear box is equipped with a high air-entry value of 500 kPa ceramic disk through which pore water pressure is controlled. Pore air pressure is applied at the top of the specimen step by step until the target value is reached. Shearing is carried out when the suction reaches an equilibrium state with negligible volume change.

Fig. 15. Suction controlled direct shear box for unsaturated soil testing (after Ng & Zhou, 2005).

Ng & Chiu (2001, 2003) reported a study on the behaviour of loosely compacted unsaturated decomposed granite and volcanic soils using a modified triaxial apparatus. The automatic triaxial apparatus combined suction and stress-path control facilities (see Section 2.1.2). The machine was equipped with three water pressure controllers and one air pressure controller. Suction was applied to a specimen through one water pressure controller and one air pressure controller. Pore-water pressure was applied or measured at the base of the specimen through a porous filter,

which had an air entry value of 100 kPa. Positive pore water pressure was maintained by using axis translation principle. Pore-air pressure was applied at the top of the specimen through a sintered copper filter.

Two different types of stress paths were considered in Ng & Chiu (2001, 2003), namely constant water content and constant deviator stress with a decreasing suction (CQS). In the constant water content tests, the soil specimen was sheared under a drained condition in the pore-air phase and an undrained condition in the pore-water phase. The CQS tests, on the other hand, simulated a stress path of a slope element subjected to rainfall infiltration. Fig. 16 illustrates the results obtained from the CQS tests conducted on anisotropically consolidated specimens. The change in axial strain due to reduction in suction is presented in Fig. 16(a). In all the tests, except the one at very low net mean stress of 25 kPa (i.e. test ua1), a small axial strain of < 4% was mobilised as the suction decreased from an initial value of 150 kPa to ~80 kPa. As the suction continued to decrease, the rate of increase in the axial strain accelerated towards the end of the test. The variation of volumetric strain with suction plotted in Fig. 16(b) suggests that unsaturated CDV changes from contractive to dilative behaviour with increasing applied net mean stress. This type of behaviour cannot be explained by classical elasto-plastic constitutive models, and has led to the development of more advanced state-dependent soil models (Chiu & Ng, 2003).

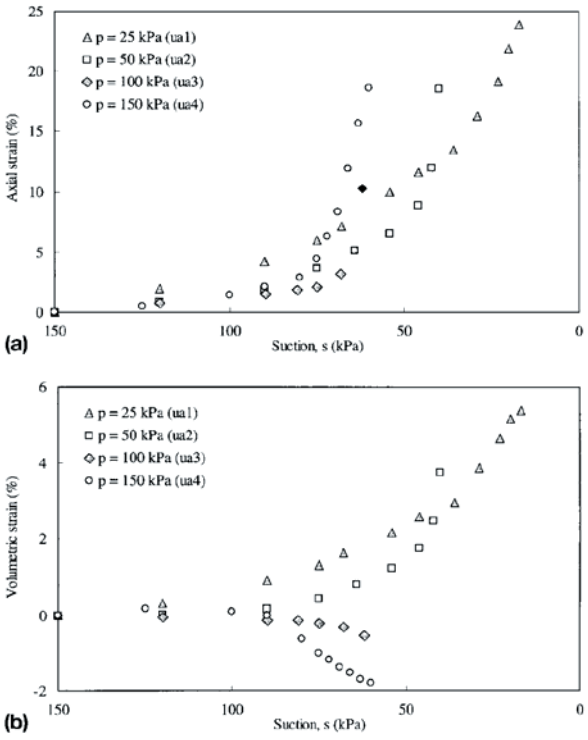


Fig. 16. Triaxial wetting tests on unsaturated CDV simulating rainfall infiltration: (a) axial strain vs suction, and (b) volumetric strain vs suction (Ng & Chiu, 2001).

3.1.2 Advanced systems for volume change measurement

One major challenge in unsaturated soil testing is accurate measurement of the total volume change if air drainage is allowed. In a fully saturated soil, the volume change in the soil can be implied from the volume of water expelled from or draw into the specimen, assuming that water is incompressible. The volume change in the air phase in an unsaturated soil requires specially-designed equipment. The use of an additional inner cylindrical cell sealed to the outer cell base (i.e. a double-cell) in a modified triaxial cell was explored by Bishop and Donald (1961) and Wheeler (1986), limitations of the two types of the so-called double-cell are discussed by Ng *et al.* (2002).

A new simple system, incorporated into a triaxial apparatus, for accurately measuring overall total volume changes in unsaturated soil specimens has been developed in HKUST (Ng *et al.*, 2002). The basic principle of the measuring system is to record changes in the differential pressure due to changes in the water level inside an open-ended, bottled-shaped inner call caused by the volume change in the specimen and inside a reference tube using an accurate differential pressure transducer. Details of the measuring system are shown in Figs. 17.

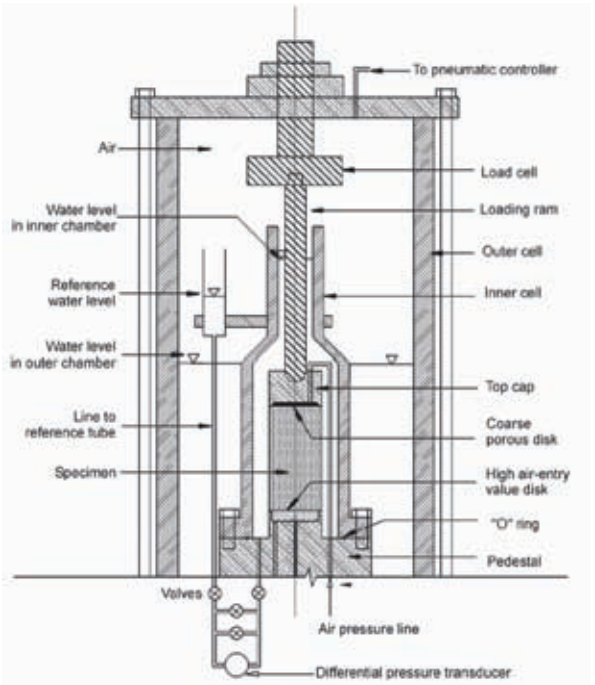


Fig. 17. A new total volume measuring system for unsaturated soils (after *Ng et al.*, 2002).

To improve the accuracy and sensitivity of the system, the following steps have been taken:

(1) The inner cell is designed to be open-ended and bottled shaped. During testing, the volume change in the specimen will be reflected by a change in water level within the bottle neck which has a small (314 mm^2) cross-sectional area to ensure sensitivity.
(2) The open-ended design ensures identical applied pressure in the inner and outer cells. This is different from the closed system described by Wheeler (1986) and Sivakumar (1993).
(3) The material used in the inner cell wall is aluminium, which exhibits negligible creep, negligible hysteretic effects and absorbs less water than an acrylic cell wall.
(4) A high accuracy (0.1% full-scale) differential pressure transducer with a measurement range from -1 to 1 kPa is used. For a typical specimen of 38 mm diameter and 76 mm in height, the estimated accuracy is equivalent to 0.04% of the volumetric strain.

(5) To minimise the potential expansion-compression of various connecting tubes due to application of cell pressure, bronze tubes are used.
(6) De-aired water is used inside the inner cell and the reference tube and a thin layer of paraffin is added to the surface to minimise evaporation of the water as suggested by Sivakumar (1993).
(7) To reduce diffusion, two layers of membrane sandwiching a layer of greased tin foil are used to seal the specimen. Slots are cut in the foil to reduce reinforcing effects on the specimen.
(8) The reference tube is fixed onto the inner cell wall so that the tube moves together with the inner cell, which is fixed on the bottom pedestal during testing.
(9) The internal diameter of the reference tube is enlarged at the top so that its cross-sectional area becomes the same as that at the bottle neck, thus the amounts of water evaporation from the two sources are the same.

For similar purpose of yielding improved measurement of volume change in unsaturated soils, a Double Cell Triaxial System (DCTS) was set up at PolyU (Yin, 2001, 2002, 2003). Fig. 18 shows the schematic diagram of the apparatus. The main features of the apparatus include:

(1) The inner cell is totally enclosed within the outer cell. De-aired water is used to fill both the inner cell and the outer cell.
(2) Both outer cell and inner cell are subjected to the same magnitude of the cell pressure σ_{i_cell} (inside) = σ_{o_cell} (outside).
(3) Because of the same water pressure σ_{i_cell}, both the wall and top cap of the inner cell will have negligible deformation. This will avoid errors caused by the inner cell deformations.
(4) The inner cell water pressure σ_{i_cell} is equal to outside cell water pressure σ_{o_cell}, the hydraulic gradient along the piston at the inner cell top cap (from Point 1 to Point 2 as shown Fig. 19) is therefore zero. Thus no water flow will occur along the gap between the piston and the inner cell top cap. This will avoid errors due to water flow/leaking at the gap. In fact, an O-ring is used at the gap.
(5) A submersible electric load cell is placed inside the inner cell and used to measure the vertical load on the soil specimen directly. This will avoid the error due to the friction between the piston and the cell caps (inner and outer cell caps).
(6) All water volumes are measured by electric volumemeter. All data, such as vertical load, pore water pressure, and volume changes are collected automatically by a personal computer.

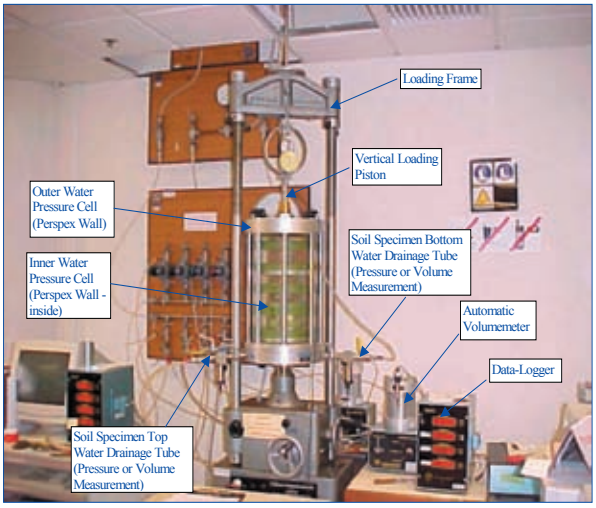


Fig. 19. The Double Cell Triaxial System (DCTS) at PolyU.

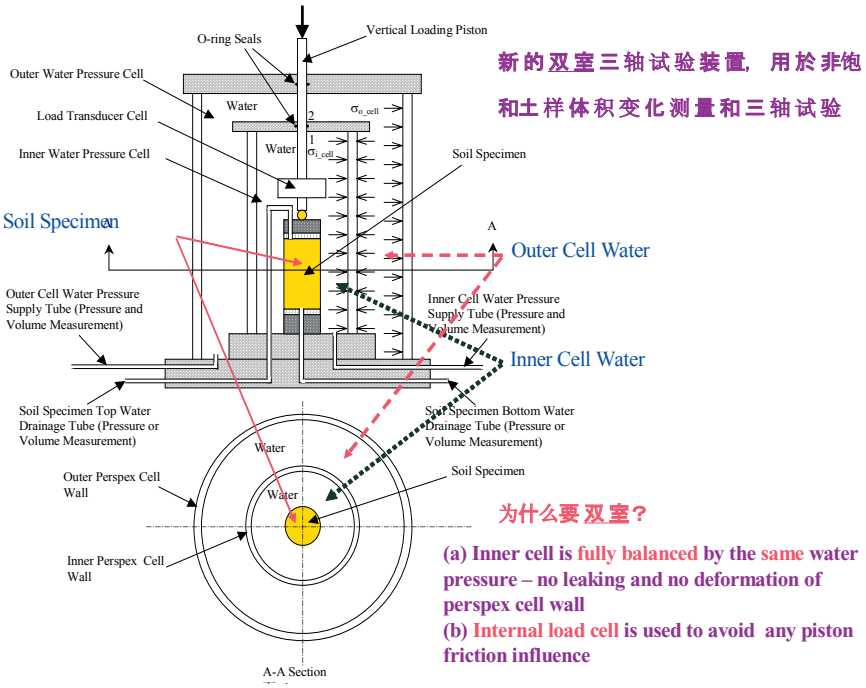


Fig. 18. Schematic diagram of the Double Cell Triaxial System (DCTS) (Yin, 2001, 2002, 2003).

The DCTS has been set up in PolyU as shown in Fig. 19. The outer cell has an internal diameter of 230 mm, a height of 425 mm and a wall thickness of 8 mm. The inner cell has an internal diameter of 90 mm, a height of 235 mm and a wall thickness of 6 mm. The load cell has dimensions of thickness of 30mm and diameter of 65 mm. The axial load piston has a diameter of 20 mm. The standard size for a soil specimen is diameter of 50 mm and height of 100 mm.

3.2 *State-dependent soil-water characteristic curve (SDSWCC)*

The water storage capacity is an important property of unsaturated soils, and is represented by the soil-water characteristic curve (SWCC) which plots the relationship between matric suction and volumetric water content (or degree of saturation). Very often, the SWCC of a soil is determined in a laboratory using a pressure plate apparatus at zero vertical and/or confining stress. In addition, the volume change of the soil specimen in the course of suction change is generally assumed to be zero. In order to overcome these two major deficiencies of the conventional technique, a stress-dependent volumetric pressure plate (SDSWCC) was invented at HKUST (Ng and Pang, 2000a, b).

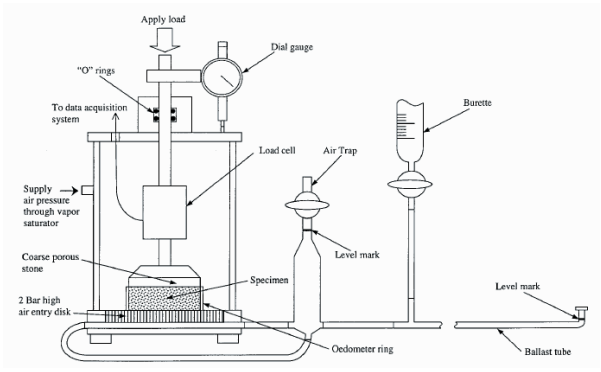


Fig. 20. Schematic diagram of stress-dependent volumetric pressure plate extractor (after Ng and Pang, 2000a).

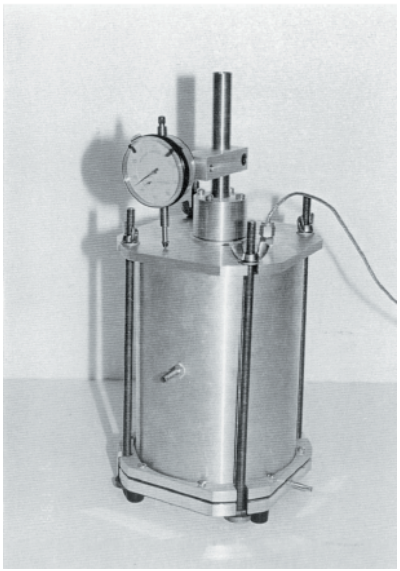


Fig. 21. Assembled stress-dependent volumetric pressure plate extractor (after Ng and Pang, 2000a).

Fig. 20 depicts the schematic arrangement of the stress-dependent volumetric pressure plate extractor, whilst the assembled set-up is shown on Fig. 21. An oedometer ring equipped a high air-entry ceramic plate at its base is located inside an air-tight chamber. Its rigid wall is used to maintain the K_0 stress conditions. Vertical stress is applied through a loading frame to a soil specimen inside the oedometer ring. The air-tightness of the chamber is ensured using rubber O-rings at the openings. To eliminate the error due to side friction between the loading piston and the O-ring, a load cell is attached near the end of the piston inside the airtight chamber for determining the actual vertical load applied to a soil specimen. The total volume change of the specimen is measured from the vertical displacement of the soil specimen using a dial gauge.

Similar to the conventional volumetric pressure plate extractor, the pore air pressure is controlled through a coarse porous stone together with a coarse geotextile located at the top of the specimen. The pore water pressure is controlled at atmospheric pressure through the high air-entry ceramic plate mounted at the based of the specimen. Additional components, including a vapour saturator, air trap, ballast tube and burette, are employed for the purpose of studying hysteresis of the SWCCs associated with drying and wetting of the soil.

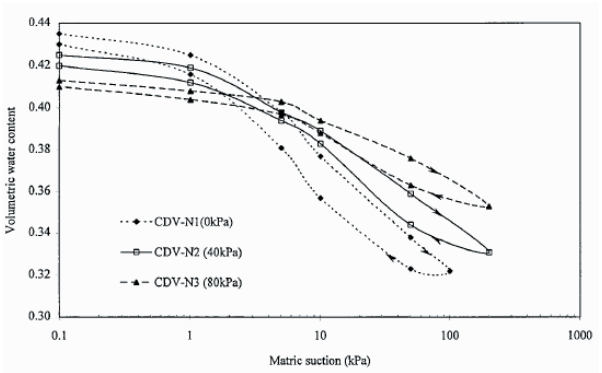


Fig. 22. Effects of stress state on SWCCs of CDV (Ng and Pang, 2000a).

Using the stress-dependent volumetric pressure plate extractor, the SWCC at different normal stresses was measured for CDV. The results shown on Fig. 22 demonstrate that there is a general tendency for the soil specimen subjected to a higher stress to possess a larger air-entry value. This is presumably caused by the presence of a smaller average pore size distribution in the soil specimen under the higher applied load. In addition, there is a marked hysteresis between the drying and wetting curves for all soil specimens. However, the size of the hysteresis seems to be independent of the applied stress for the stress range considered.

3.3 *Shear stiffness*

Investigation into the shear stiffness of unsaturated soil requires modifications of the equipment described in Section 2.2 to control the applied suction. Such modifications developed at HKUST are depicted in Fig. 23 (Ng and Yung, 2007; Ng and Xu, 2007). A dual channel automatic pressure controller that can control pore air pressure and pore water pressure independently was added to the triaxial testing system so that the technique of axis translation can be employed. Similar to other equipment for unsaturated soil testing described in this paper, air pressure was

controlled through a coarse low air-entry value corundum disk placed on top of a soil specimen, whereas water pressure was controlled through a high air-entry value (3-bar) ceramic disk sealed to the pedestal of the triaxial apparatus.

In order to incorporate both a high air-entry value ceramic disk and a bender element, the conventional pedestal of the triaxial cell was modified. Besides, a spiral-shaped drainage groove of 3 mm wide and 3 mm deep connected to the water drainage system was carved in the base pedestal. It serves as a water channel for flushing air bubbles that may be trapped or accumulated beneath the high air-entry value ceramic disk as a result of air diffusion during long periods of testing. More details of the system are described in Ng and Yung (2007).

Using the equipment shown on Fig. 23, Ng and Yung (2007) investigated the anisotropic stiffness of a reconstituted CDT at various matric suctions ($u_a - u_w$ = 0, 50, 100 and 200 kPa). The results at different values of net mean stress ($p - u_a$), where p is the mean total stress, are plotted in Fig. 24. The measured shear moduli increase with the two stress state variables, matric suction and net mean stress. The shear moduli increase non-linearly with increasing matric suction, especially within the air-entry value of the reconstituted CDT which is roughly 50 kPa. The rate of increase of the stiffness reduces as the matric suction exceeds this value.

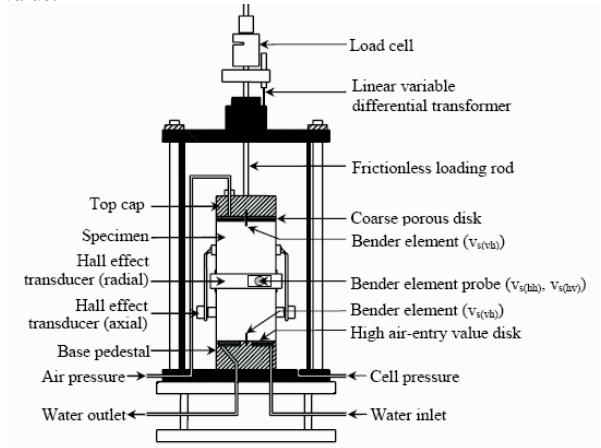


Fig. 23. Triaxial apparatus with multidirectional shear wave velocities and local strain measurements for unsaturated soils (after Ng and Yung, 2007).

The observed behaviour may be explained by the fact that the soil specimens remained essentially fully saturated at matric suction less than the air-entry value and therefore bulk water effects dominated the soil stiffness responses (Mancuso *et al*., 2002). Any increase in suction is practically equivalent to an increase in mean effective stress as in saturated soil behaviour, leading to a stiffer response. Once desaturation occurs at the air-entry value, air-water meniscus (or contractile skin) begins to form at contact points of soil particles. Although there will be increasing tensile forces from the meniscus which holds the soil particles tighter, the beneficial effect will level off due to progression reduction of the radius of the meniscus when matric suction increases beyond 100 kPa (Mancuso *et al.*, 2002).

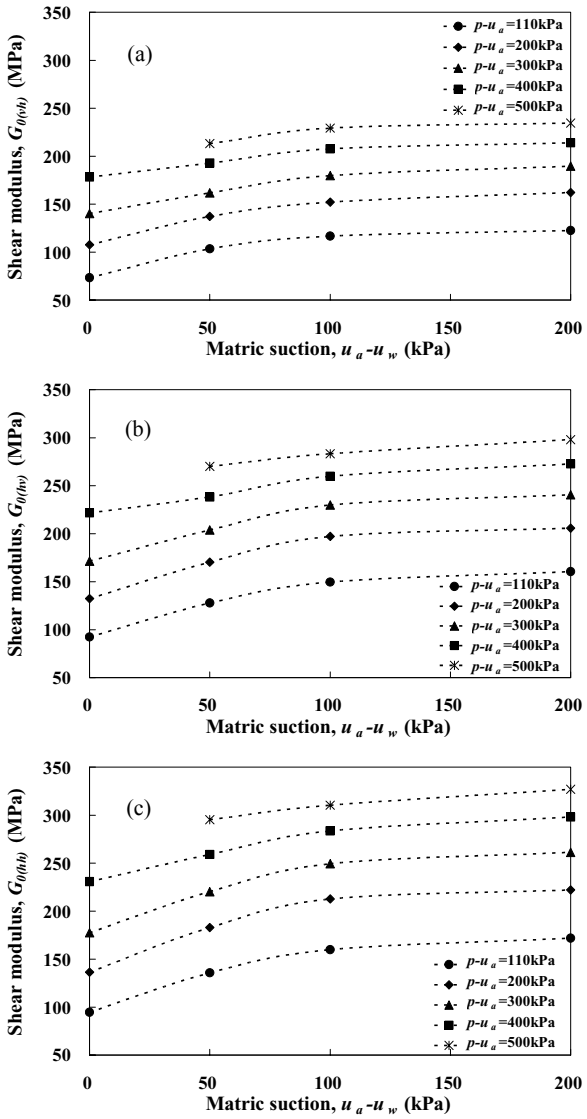


Fig. 24. Shear modulus of unsaturated CDT: (a) G_{vh}; (b) G_{hv}, and (c) G_{hh}.

4 ROCK TESTING

4.1 *The Rock Engineering Research Centre at HKU*

In 1994, the Department of Civil Engineering of HKU was awarded a grant of HK$10M by the Research Grants Council of the HKSAR government to establish a centre focusing on research in rock engineering. With this grant, the Rock Engineering Research Centre was established in 1997. To facilitate the activities of the Centre, it is equipped with a comprehensive set of MTS Rock Testing System (Fig. 25). It consists of a computer controlled, servo-hydraulic compression machine, load frame, hydraulic power supply, triaxial cell, confining pressure subsystem, digital controller, test processor and a PC workstation. It is also equipped with a pair of end platen containing ultrasonic sensors for the determination of wave velocities when a specimen is loaded. Two linear variable differential transducers are used to measure the axial deformation. On the other hand, the circumferential deformation is measured by an extensometer. The System

has a maximum axial capacity of 4600 kN and confining pressure up to 140 MPa. Pore pressure of not more than 140 MPa can be applied axially.

In addition, a MISTRAS 2001 is available to record the acoustic emission (AE) activities (Fig. 26). The system is a fully digital, multi-channel, computerised, acoustic emission system that performs AE waveform and signal measurement and stores, displays and analyzes the resulting data. It consists of an IBM compatible PC, an AEDSP-32/16 card and the MISTRAS software, sensors, preamplifiers and cables.

Fig. 25. MTS Rock Testing System at HKU.

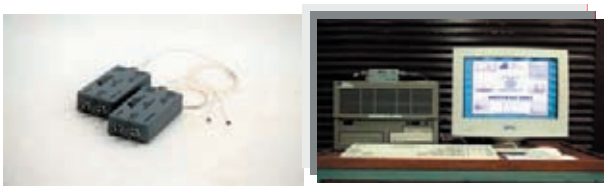


Fig. 26. MISTRAS 2001 for recording the AE activities.

With such equipment, the Centre has the capability to study the behaviour of rock specimens under different loading conditions. Fig. 27 shows that typical stress-strain curves for local granite under uniaxial compression (Zhou, 1999). Furthermore, direct tensile strength of rocks can be determined by using the fixture as shown in Fig. 28. A typical stress-strain curve of granite plates in the direct tensile test is depicted in Fig. 29 (Liu, 2000). The curve is almost linear before the peak stress is reached, then drops vertically from the peak point. Such a curve is quite typical for brittle materials. It is difficult to obtain the residual strength experimentally as the clamps with universal joints and chains have low rigidity. However, it is believed that one can have a better trace of the post-peak behaviour if the loading rate is slower.

The behaviour of the rock under sustained load was also studied (Lin, 2006). In the test, a specimen was loaded to a predetermined load level and the load was maintained until the specimen failed. Fig. 30 shows the evolution of dilatancy under different confining pressure. Specimen tends to become more stable under higher confining pressure. Specimen with stress ratio higher than 1.15 can fail in a few hours and all phases can be observed under uniaxial condition. Specimen with stress ratio of 1.28 failed after about 18 days under the confining pressure of 10 MPa. Specimen with stress ratio of 1.32 was still quite stable and had no sign of failure even after 14 days under the confining pressure of 30 MPa; only the primary and secondary phases were observed. Specimen No.10-RT-3 and Specimen No.30-RT-4 have a similar stress ratio of 1.31. The former one failed in about 4 days while the latter one is still quite stable for 14 days.

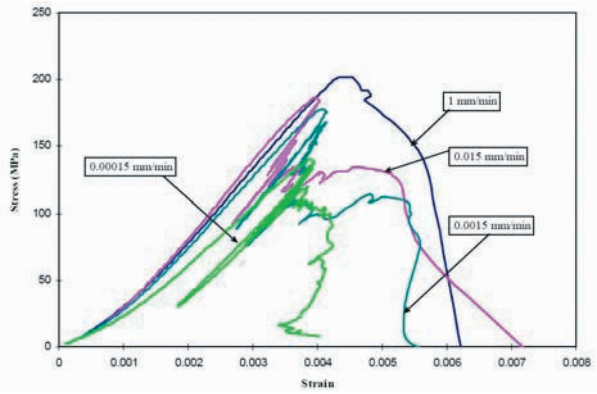


Fig. 27. Typical stress-strain curves of granite.

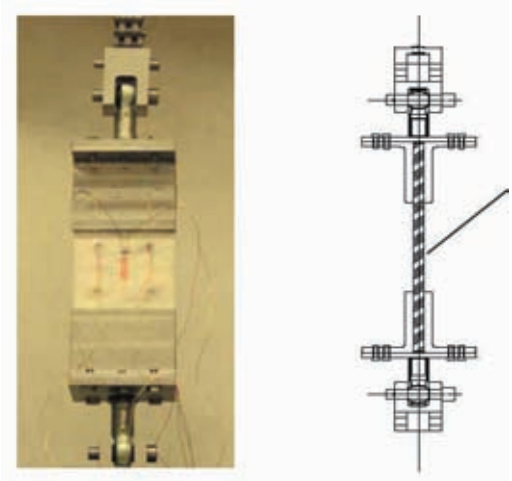


Fig. 28. Set-up for direct tensile test.

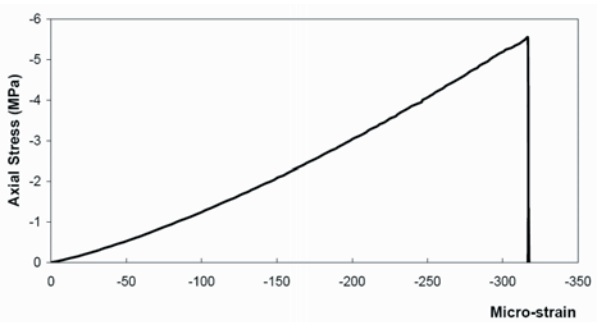


Fig. 29. Stress-strain curve of granite under direct tensile test.

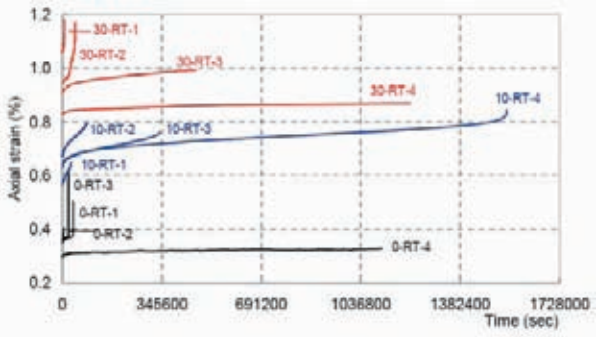


Fig. 30. Creep axial strain under different stress levels and confining pressures.

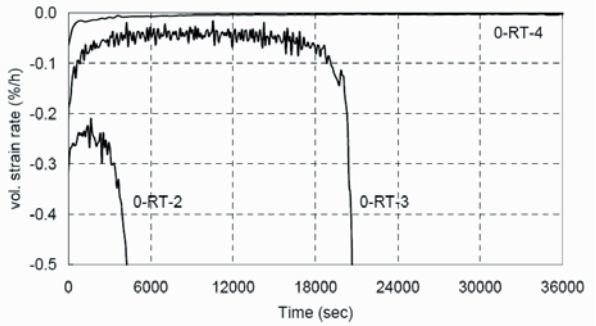


Fig. 31. Volumetric strain rate varies with time under different loading levels.

Fig. 31 summarises the volumetric strain rate of a series of tests under uniaxial condition. The initial strain rate of Specimen No.0-RT-3 is high and then decreases gradually until it reaches a minimum strain rate at the end of primary phase. The minimum strain rate remains constant during secondary phase. In the tertiary phase, the strain rate increases suddenly and ends in failure. Stress ratio of Specimen No.0-RT-4 is lower than Specimen No.0-RT-3, the strain rate decreases gradually to the minimum value and remains constant; no acceleration of strain rate was observed during the testing period. The stress ratio of Specimen No.0-RT-2 is higher than Specimen No.0-RT-3 and the strain rate decreases quickly to the minimum value and then accelerates to the failure.

Acoustic signals were generated by micro-fracturing within the specimen when stressed. Fig. 32 shows the relation between volume change (or dilatancy) and acoustic emission activities. During primary phase, acoustic emission activity decays with time and the inelastic volumetric strain rate decreases accordingly. During secondary phase, the acoustic emission activity levels off and the volumetric strain rate remains constant. During tertiary phase, the acoustic emission activity accelerates rapidly with time and the volumetric strain rate accelerates gradually until the specimen fails.

As the waveforms of acoustic emission from six sensors attached to the specimen are recorded simultaneously, the arrival time and the amplitude of the first motion can then be determined. Hence, the location of acoustic emission source can be computed by the arrival time differences. The crack type and crack direction can also be determined from the amplitude of the first motion. Therefore, the detailed information about the damage process in both space and time can be obtained to predict where and when the damages occur.

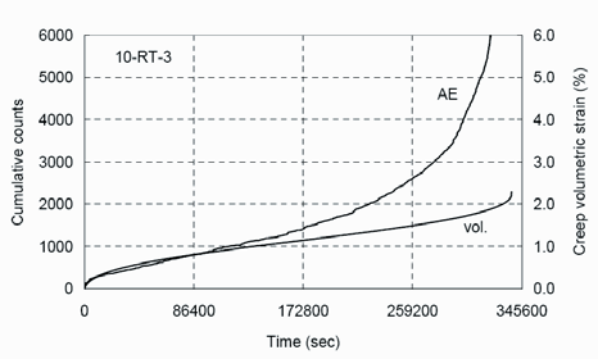


Fig. 32. Dilatancy and micro-cracking during creep.

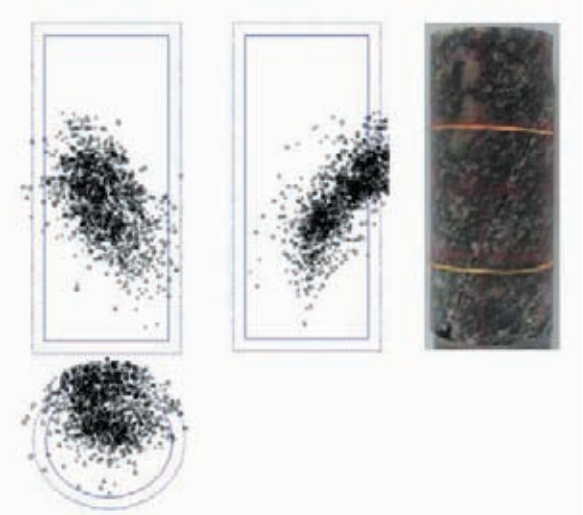


Fig. 33. Orthographic projections of specimen tested under high confining pressure.

Fig. 33 shows the orthographic projections of specimen tested under confining pressure of 30 MPa. The right projection shows the front view of the specimen, while the left projection is viewed from left to right. The bottom plot is a view from the top of the same specimen. Each symbol represents one acoustic emission event. Different crack types were represented by different symbols. Symbol × represents shear crack; symbol □ represents tensile crack; symbol Δ represents mix mode. It is interesting to find that most tensile cracks occur on the circumferential surface of the specimen. On the other hand, shear cracks occur mainly inside the specimen while a mix mode crack nucleation zone between tensile crack zone and shear crack zone.

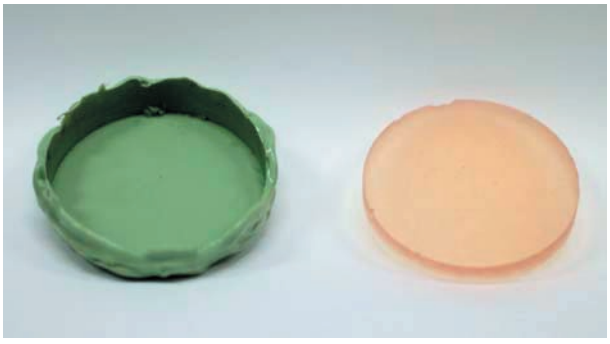


Fig. 34. Double Replica Method

A special replica method (Li, 2001) was also developed by the Centre to study the growth and propagation of cracks. The method consists of two steps. The first step is to make a cup-shaped replica (Fig. 34) from a ground or polished rock surface using a silica-based impression material (President Light Body). This material has properties that satisfy the basic requirements of replica material, such as tear strength, good compatibility with rock surfaces, and stable dimensions after consolidation. As the replica is rubbery, it can be easily removed from the rock surface immediately after setting. In fact, it can be directly used to observe and analyze under SEM after coating. As expected, it will give a negative rather than a positive image.

To obtain a positive image under SEM and allow optical transmitting observation, the second step is to make a semi-transparent replica from the cup-shaped replica by employing TAAB 812 Embedding Resin. This resin replica (Fig. 34), which is hard after curing, is finally used for observation under optical microscope and SEM.

A comparison between the micrographs of the resin replica and the actual rock surface is made in Fig. 35. The micrographs indicate that due to the very good compatibility of the impression material with both the rock surface and the resin replica, there is almost no difference between the replica image and the actual rock image if the magnification is not higher than 3000X. Moreover, cracks that are wider than 0.2μm can be effectively copied. This accuracy is sufficient for microcrack study, as the required magnification is usually in the hundred orders.

The double replica method is also used to obtain images of the cracks on the surfaces of the specimen. For this purpose, the replicas of the unstressed specimen surfaces are first made as references for later comparison. As one would like to have replica images of the rock surfaces at different levels of loading before rupture, it is necessary to increase the load in a number of loading-unloading cycles. If the stress level is continuously increased in every successive loading cycle, then the specimen is very likely to suddenly fail, and the "expected" crack patterns, which can show the gradual cracking development of a specimen, may not be obtained. Therefore, it is more appropriate to use crack propagation other than stress levels as the controlling factor for determining the maximum load in each cycle. This is achieved by carefully monitoring the sound that is produced during the test, which serves as an indirect indication of crack initiation or propagation. Whenever a sound that is emitted by cracking of the granite specimen is picked up, the specimen is unloaded immediately and surface replicas are made. Thus, a series of surface rep licas that reveal a series of crack patterns associated with subsequent cycle loading can be obtained. These replicas can give a representative picture of the gradual cracking process of the granite plate. It must be pointed out that one only has to unload the specimen if double replica images are required, otherwise one can continue to load the specimen after images of each surface are captured by the digital camera and video. In the study, the loading cycle is designated as Cycle 1 when stress- induced cracks are first produced, and the subsequent cycles are designated as Cycle 2 and Cycle 3, etc.

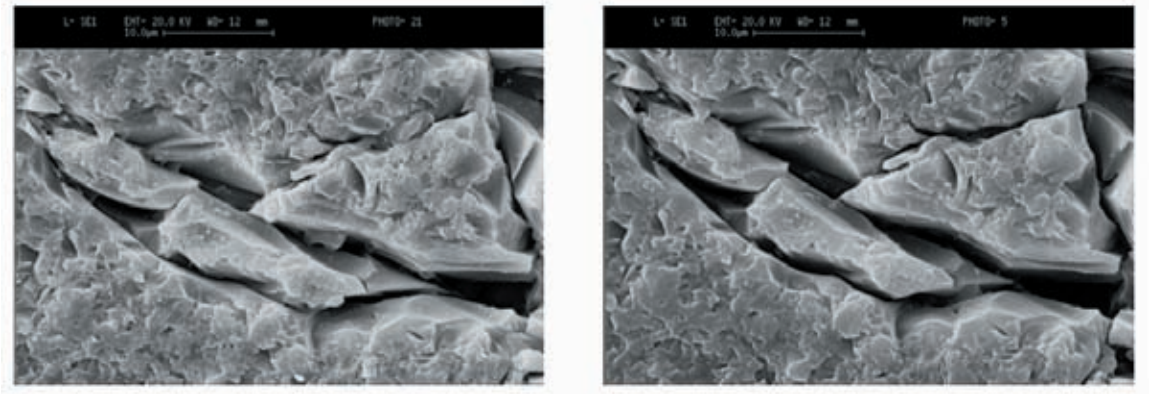


Fig. 35. SEM images (3000X) of: (a) rock surface, and (b) resin replica.

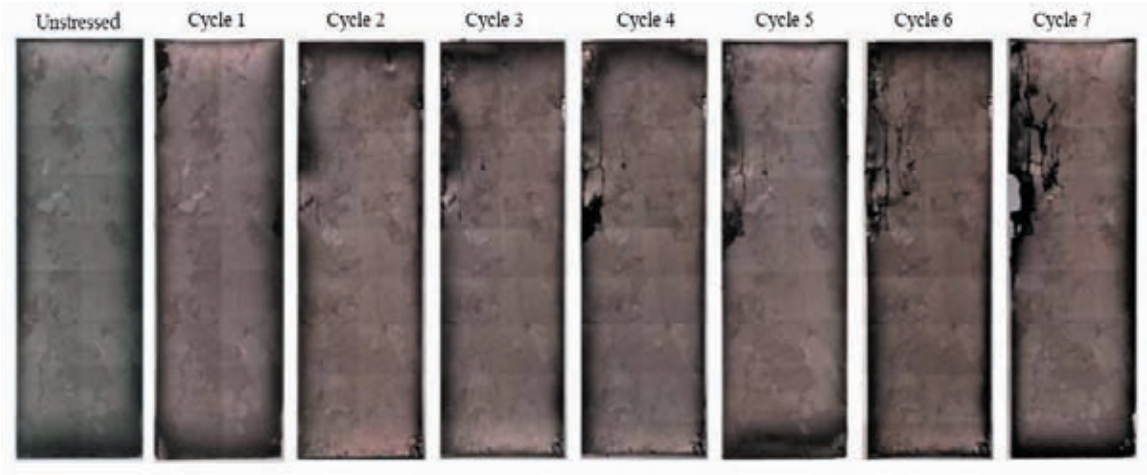


Fig. 36. Replica images showing crack propagation in the specimen surface (loading is along the vertical axis).

From Fig. 36, one can conclude that failure takes place and develops locally. Whenever cracks appear in a particular area, their propagation and coalescence are the major activities of the subsequent cracking process. The other area of the specimen remains essentially unchanged until the final collapse occurs. This is because rock is a naturally heterogeneous material, and flaws such as pre-existing microcracks, micropores, and grain boundaries, etc., induce local stress concentration when the specimen is loaded. Once the value of this stress concentration exceeds the strength of the local place, failure will occur. Immediately after the occurrence of local cracking, the stress concentration transfers to the tip of the crack and promotes the failure to develop continuously in the area. This is thought to be the general process of local failure of rock material.

4.2 *Rock testing activities at PolyU*

PolyU houses a number of rock testing systems that contribute significantly to the understanding of rock behaviour. A servo-controlled direct shear test system is available to study the shear strength of rock joint and rock bridge study (Fig. 37). The testing system was manufactured by MTS Corporation. It consists of four parts: (1) direct shear unit; (2) control panel; (3) Test Star II software and (4) the power and hydraulic supply system. The direct shear unit consists of the upper and a lower shear box; two actuators which is used for applying the normal and shear forces and three LVDTs (two in horizontal and one in vertical direction) to measure the displacement of the shear box during the shearing testing. The control panel is used to control the movement of the actuators. The Test Star II software is to control the experiment procedures either in force control mode or displacement control mode. The maximum normal loading and shear force capacity of the direct shear system are 254kN.

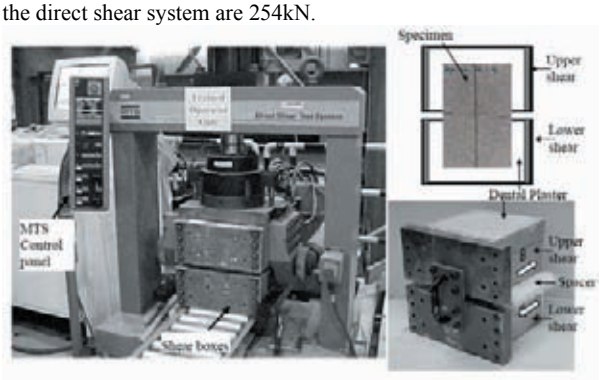


Fig. 37. MTS direct shear testing system at PolyU.

The shear strength of rock specimen containing pre-existing fractures with rock bridge was studied using the MTS direct shear system (Fig. 37). The pre-existing fractures can be created by a circular blade cutting at the two sides of the specimen. For the internal pre-existing fracture, it can be created by gluing the two specimens together (Fig. 37). The effect of material, the separation of fractures and the applied normal load on shear strength of rock bridge can be studied. Fig. 38 shows the progressive shear and dilation behavior of different types of rock containing two pre-existing fractures with vertical separation. The shear strength of tuff is higher than the granite and marble specimens. Fig. 39 shows the normalized shear stress of the three types of rock against bridge length and bridge angle. From Fig. 39, it can be concluded that normalized shear stress increases with the bridge length, but slightly decreases when the bridge angle increased. The results of rock bridge study can be applied on the stability study of rock mass containing discontinuous joint.

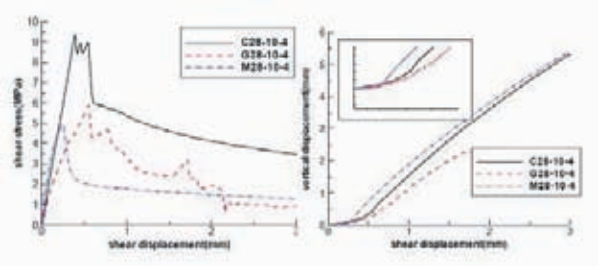


Fig. 38. Progressive shear and dilation behavior of different types of rock containing two pre-existing fractures with vertical separation.

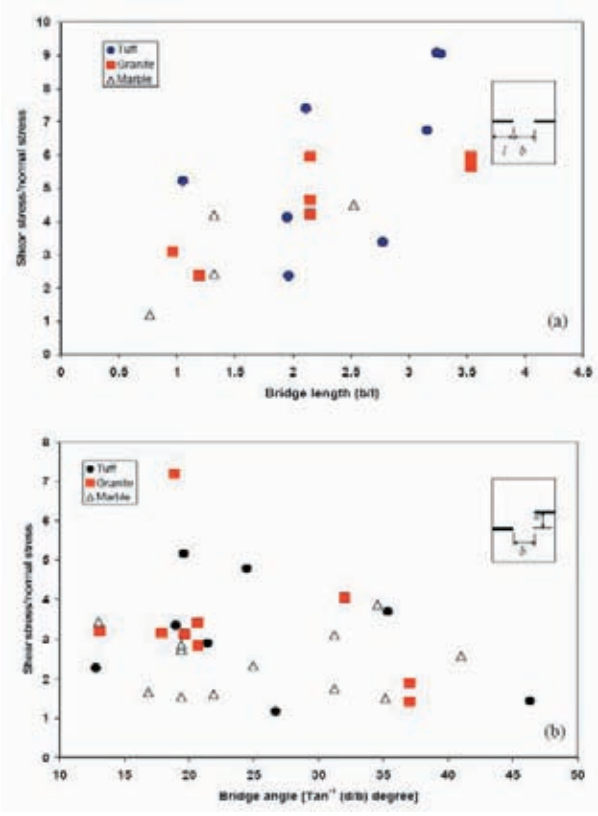


Fig. 39. The normalised shear stress of the three types of rock plotted against: (a) bridge length, and (b) bridge angle.

Under compression loading, crack can initiate and propagate from the tips of a 2-D pre-existing fractures. It is the so-called wing crack (Fig. 40a). However, for rock specimen containing a 3-D type surface pre-existing fracture, four types of cracks can be observed: wing crack, anti-wing crack, petal cracks and compressive cracks (Fig. 40). Anti-wing crack is the tensile crack but grows in the opposite direction of the wing cracks (Figs. 40b and 40c). Compressive crack is the crack burst out from the fracture tip. Petal cracks grow along the edge of the pre-existing fracture (Fig. 40d). These four types of crack can be found on the rupture of the ground after earthquake induced. However, the nucleation and propagation of these crack is not fully understood. To understand crack propagation and nucleation of fracture, strain gages and AE measurement can provide useful information on such kind of study.

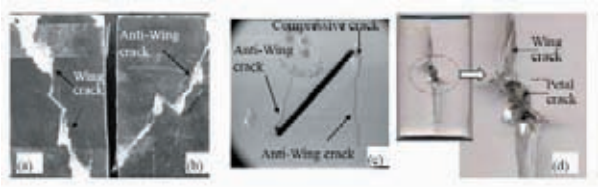


Fig. 40. Four types of crack induced from the tips of a 3-D type pre-existing fracture in real rocks: (a) Wing crack (tensile cracks); (b) anti-wing crack; (c) compressive crack, and (d) petal crack (captured from PMMA specimen).

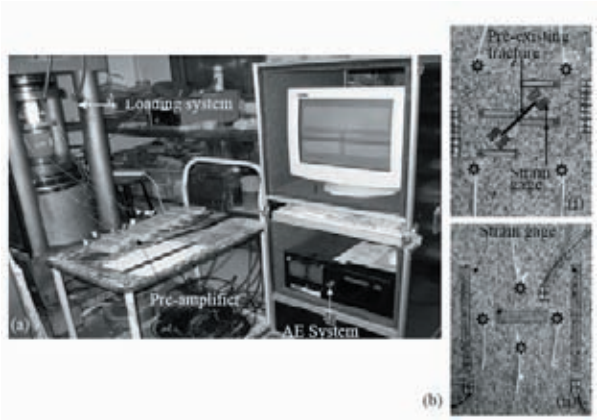


Fig. 41. (a) The loading equipment and AE system at PolyU; (b) the rock specimen containing a pre-existing fracture with strain gauges and AE sensors mounted on the (i) front face and (ii) back face of the test specimen (the symbol ✲ is the AE sensor).

An AE system is also available in PolyU to study crack growth mechanisms from pre-existing fractures (Fig. 41a). The AE system is developed by the Physical Acoustic Corporation (PAC). The system contains 8 channels. The sensors are the PICO sensor with the diameter of 2mm and the resonant frequency in the range from 200 to 750 kHz. The AE events are recorded by using the MISTRAS-2001 AE recording software. The recorded sample rate can be up to 1MHz per each channel. The number, the magnitude, the frequency and the location of the AE events can be acquired by using the AEwin software.

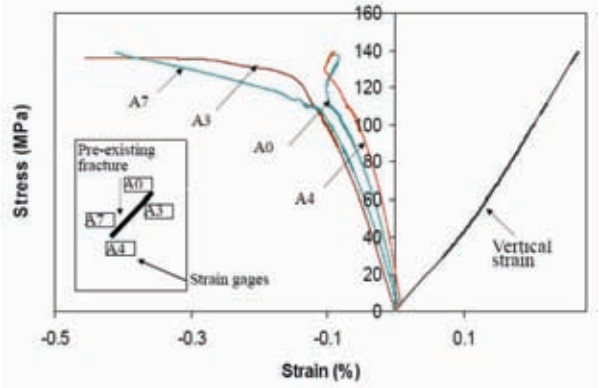


Fig. 42. Stress-strain curves of rock specimen containing a 3-D type pre-existing fracture under uniaxial and biaxial compressions.

The pre-existing fractures can be created by using a circular cutting blade. Strain gages and AE sensors can be mounted on the front and the back surface of the specimen (Fig. 41b) to measure the deformation around the pre-existing surface fracture and to detect the location of the AE events during crack propagation under compressions. Fig. 42 shows stress-strain curves of rock specimen containing a surface pre-existing fracture under compressions. Fig. 42 indicates that the when a tensile crack propagate, the strain values near by the tensile cracks are reduced. The AE record (Fig. 43) indicates that the energy for wing crack and anti-wing crack initiation is very low. But when the crack grows, large amount of the AE energy released and the accumulative curve increases rapidly. Most of the high energy is released from anti-wing cracks and compressive cracks. The growth of anti-wing cracks is 4 times faster than of the wing cracks. This phenomenon has not been reported by any researcher.

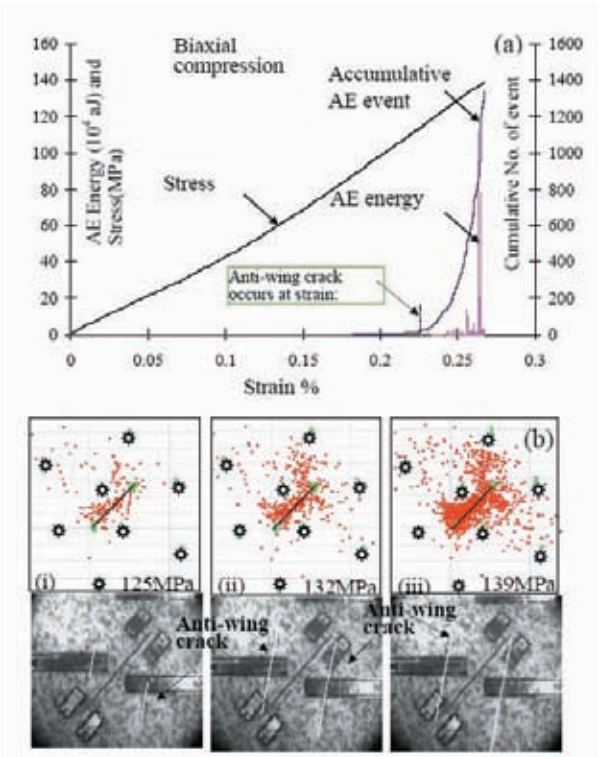


Fig. 43. (a) Stress, AE energy and AE events (accumulative) plotted against strain value under compressions, and (b) AE events and the captured photo during the process of crack growth.

Thin rock panels are commonly used as an exterior cladding wall in high-rise building construction. However, it has been found that such rock panels in exterior cladding may develop tensile cracking and unstable failure, due to stress concentrations induced by periodic wind load and sunshine, with a failure mode that is sensitive to environmental effects. This problem can be examined by subcritical crack growth (SCG) phenomenon, but it is found that none of existing design codes of rock panel accounts for SCG. Thus, a four-point bending equipment is designed and available to investigate the SCG in rock panel under different environmental conditions such as water, acid and air.

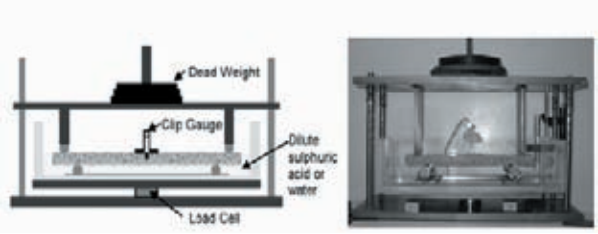


Fig. 44. Schematic diagram and experimental setup of rock specimen subjected to four-point bending test.

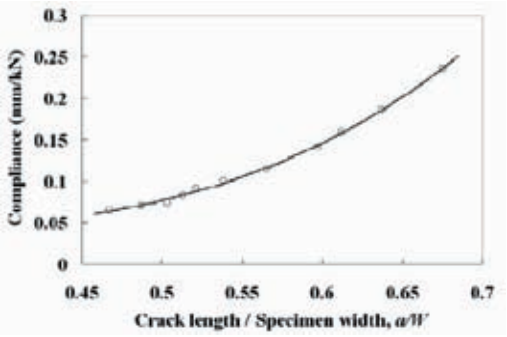


Fig. 45. Compliance calibration curve of rock specimen.

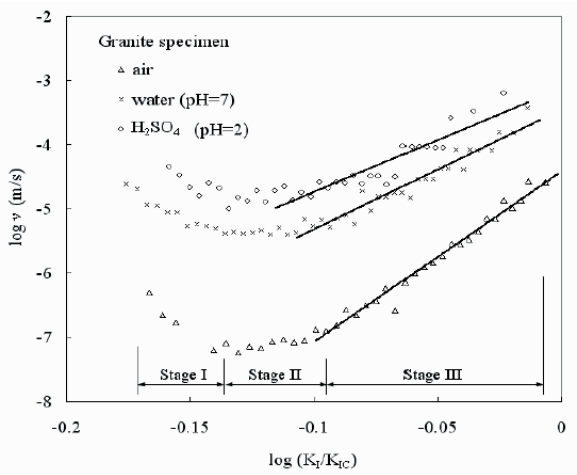


Fig. 46. log (K_I /K_{IC}) – log v curve for granite obtained from four-point bending test.

A pre-existing fracture is created by using a circular cutting blade for subcritical crack growth (SCG) study. The experimental setup of four-point bending test is shown in Fig. 44. The dead weight of pre-determined value can be applied to the specimen is applied. A clip gauge is fixed at the two side of the pre-existing fracture to monitor the opening displacement of the fracture mouth (CMOD) under the applied loading. The growth of crack length is monitor by a microscope while the microscope is connected to a video recorder to record the growth process of the crack. A compliance calibration between the opening displacement of the fracture mouth (CMOD) and crack growth length is plotted in Fig. 45. The calibration curve shown on Fig. 45 can be applied on the same type of rock specimen to predict the length of the growth crack according to the record of CMOD. The stress intensity factor K_I and fracture toughness K_{IC} of the rock under four point bending can be calculated. The log-log plot of the crack growth rate v versus normalized stress intensity factor K_I /K_{IC} of four-point bending test on rock specimens under air, water and dilute sulphuric acid (pH value of 2) can be studied and are shown in Fig. 46. Three stages are observed in rock specimens under different testing conditions. In Stage I, the crack growth rate v decreases after the application of pre-determined dead weight has been just completed. In Stage II, v becomes roughly constant over a certain range of K_I/K_{IC}. In Stage III, v increases and the crack propagates towards to the lower edge, splitting the specimen into two pieces. The subcritical crack growth indices n of the rock specimen is determined by the linear fitting slope in Stage III. Fig. 46 concludes the pH value of the aqueous environment can influence the crack growth rate. Under acidic condition, crack growth rate v becomes faster than that in air condition. Moreover, the time to failure of rock becomes shorter when the pH value decreases. The study of subcritical crack growth (SCG) can be used to predict the life of the rock panels.

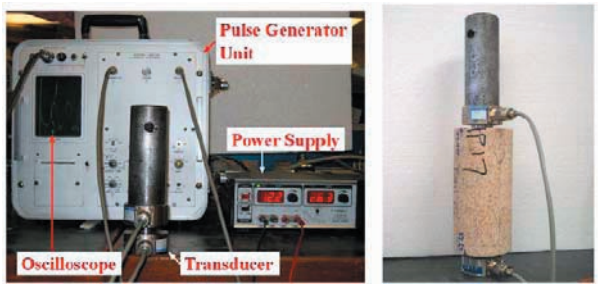


Fig. 47. Apparatus for pulse velocities measurement and the setup for wave velocities measurement.

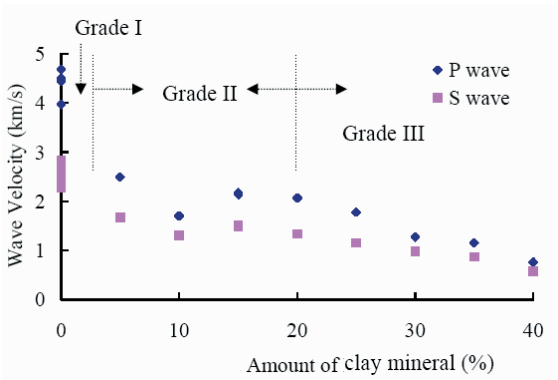


Fig. 48. Relationship between wave velocities and weathering grade of rock specimens with percentage of clay minerals.

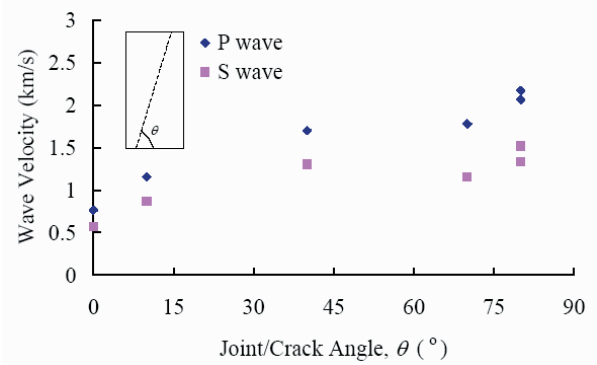


Fig. 49. Relationship between wave velocities and joint/crack angle of the specimen.

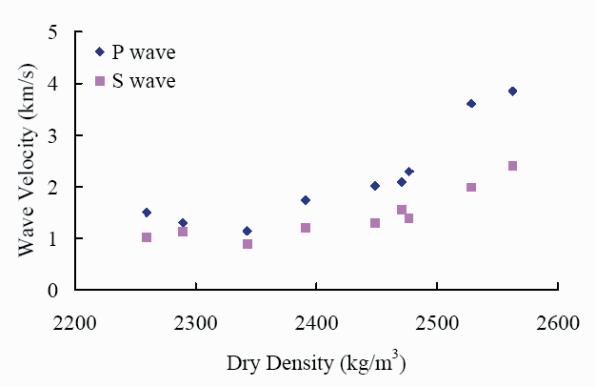


Fig. 50. Relationship between wave velocities and dry density of the specimen.

A pulse velocity measurement system is available in PolyU to study the effect of physical properties on pulse velocities and elastic constants. Fig. 47 shows the apparatus for pulse velocities measurement and experimental set up. Fig. 48 shows the influence of degree of weathering with percentage of clay minerals on wave velocities. The clay mineral is the product of the chemical weathering. Wave velocity will slow down when passing through the clay material. Fig. 48 shows the influence of pre-existing fractures of the rock specimen on wave velocities where the specimens have the same weathering grade. Both P and S wave velocities increase when the angle of the fractures increased. In addition, wave velocities increase with the dry density of rock (Fig. 49). In general, the smaller dry density of the rock specimens contains fractures (joints, or fissures) or voids. Wave velocities slow down when passing through such kinds of the specimens. Pulse velocity is a non-destructive tool to detect the degree of weathering, or fissuring of the rock.

5 CONCLUSIONS

Laboratory testing is inevitably a crucial component in all geotechnical projects. Many testing methods have been widely accepted and routinely adopted as the standard way of deriving the material and design parameters. Notwithstanding the popularity of these conventional tests, the importance of understanding the inherent simplifications or assumptions associated with the testing method can hardly be overemphasised.

In this paper, the development of laboratory testing techniques in Hong Kong over the last couple of decades has been outlined. Many of these developments have demonstrated the complicated behaviour of soil and rock. With the help of these new techniques and continuous advancement in technology, it is the wish of the authors that our understanding of the behaviour of geomaterials will be improved, leading to safer and more economical geotechnical designs.

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Centrifuge Modelling in Engineering Practice in Hong Kong

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Abstract: Recently there has been a rapid development of the geotechnical centrifuge modelling technology in many countries worldwide. Centrifuge testing has become the preferred method of physical experiments for many geotechnical and geo-environmental applications. It is anticipated that the applications of centrifuge technology will continue to increase and flourish in the Far East in particular. In this paper, the principles and the major applications of geotechnical centrifuge modelling are reviewed. Moreover, the development of an 8.4m diameter state-of-the-art geotechnical centrifuge facility in Hong Kong is introduced. This 400 g-ton geotechnical centrifuge facility in Hong Kong is equipped with unique and advanced capabilities including the world first in-flight bi-axial shaker, an advanced 4-axis robotic manipulator and the state-of-the-art data acquisition and control system. A number of industrially funded projects are presented to illustrate how centrifuge model tests can be used to reveal some unusual physical phenomena and to assist engineering designs in Hong Kong.

1 INTRODUCTION

According to a survey conducted by the British Geotechnical Society in 1999, centrifuge modelling was ranked fifth in the list of the most important developments in British geotechnics over the last 50 years (see Fig. 1). The results were based on responses to a questionnaire sent to 68 geotechnical experts from academia, consulting, contracting and research organisations in the U.K. It is clear from the survey that centrifuge modelling plays a key role in modern geotechnical engineering research and applications.

Following the 10th International Conference on Soil Mechanics and Foundation Engineering in Stockholm in 1981, a Technical Committee on Centrifuges was set up to broaden communications between all parties interested in this area of activity and to promote awareness of the techniques worldwide. The Chairman of the first Technical Committee (TC2) was Professor Andrew Schofield of the University of Cambridge. Since the formation of the Technical Committee, a series of regular (3-4 years) major international conferences on geotechnical centrifuge modelling was organised and held at Manchester (U.K.) in 1984, at Paris (France) in 1988, at Colorado (US) in 1991, at Singapore in 1994 and at Tokyo (Japan) in 1998. Since 2002, TC2 has widened its scopes to cover all physical modelling activities and organised its extended international conference on physical modelling in geotechnics in Newfoundland (Canada). In 2006, a formal conference numbering system was agreed and the 6th international conference on physical modelling in geotechnics was subsequently held in the Hong Kong University of Science and Technology (HKUST).

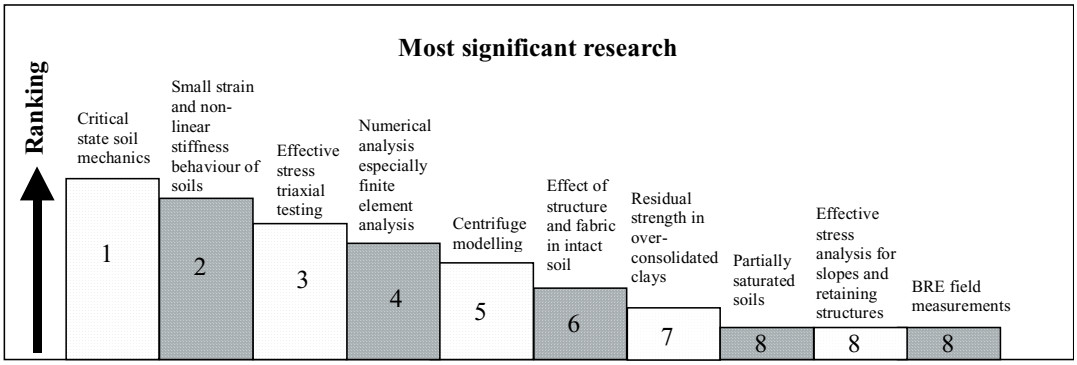

Fig.1 The great and the good of 50 years of geotechnics (From Ground Engineering, July 1999)

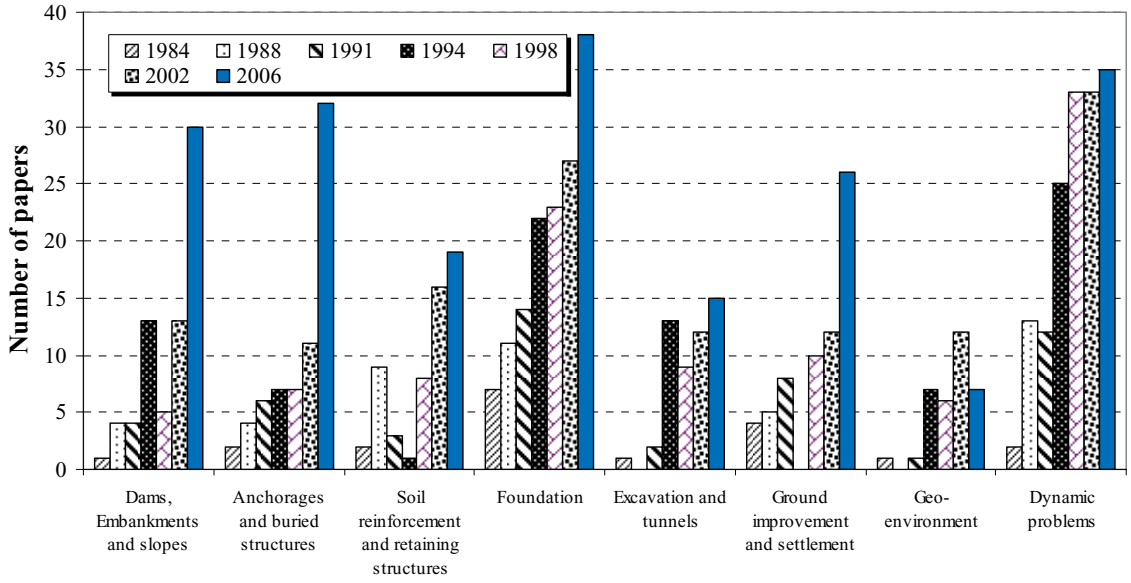

Fig.2. Major subject areas covered in the conferences.

Fig. 2 shows the number of major subject areas covered in the last seven international conferences. There is no doubt that there has been a significant increase in the applications of centrifuge modelling technique to tackle various geotechnical and geo-environmental problems. Over 300 delegates from 30 countries attended the international conference held in HKUST. It is anticipated that a significantly large number of participants will attend the 7th international conference to be held in Zurich, Switzerland, in 2010.

Geotechnical centrifuge modelling has become an alternative modelling tool to complement more conventional numerical analysis and field testing or monitoring (Ng *et al*., 2003). The inter-relationships between these three approaches are illustrated in Fig. 3. These three approaches are definitely not mutually exclusive. On the contrary, they are complementary with each other as no approach is perfect for every geotechnical or geo-environmental problem in terms of quality of result, time and cost. Ideally, two or three approaches should be performed to calibrate results against each other and to verify any conclusion drawn. Small-scale geotechnical model tests under only one-gravity (9.81 m/s^2) can be very misleading as soil behaviour is stress-dependent. These tests should therefore be treated with caution.

Fig.3 The inter-relationships between Field Monitoring, Numerical and Centrifuge Modelling

In this paper, the brief history, principles and major applications of geotechnical centrifuge modelling are reviewed and described. Moreover, the development of a state-of-the-art geotechnical centrifuge facility in Hong Kong is introduced. A number of industrially funded centrifuge applications are presented to illustrate how centrifuge results can be used to reveal some unusual failure phenomena and deformation mechanisms and hence assist engineers in their designs.

2 BRIEF HISTORICAL DEVELOPMENT OF GEOTECHNICAL CENTRIFUGE MODELLING

Although the germ of an idea in using a centrifuge to study engineering problems started as early as in 1869 in France (Phillips, 1869), the actual applications of the centrifuge modelling in civil engineering were not reported until in the 1930s. Bucky (1931) reported the study of the integrity of mine roof structures in rock where small rock structures were subjected to increasing accelerations until they ruptured at Columbia University in the US. Apparently independently, there was a significant early development of geotechnical centrifuge modelling in Russia (former USSR). The first high-profile publication in English was presented by Pokrovsky & Fiodorov (1936) at the first International Conference on Soil Mechanics and Foundation Engineering at Harvard. They described the study of soil pressures and deformations by means of a centrifuge at the laboratory for Physics of the Military-Engineering Academy of the USSR. After the second world war, little more was heard about the applications of centrifuge modelling from the USSR following the isolation of the Soviet block behind the so-called “Iron Curtain” (Craig, 1995). Incidentally, centrifuge activity for civil engineering applications waned in the face of a developing American preference for mathematical modelling in the age of the digital computer between the 1950s and 1960s (Cheney, 1988). However, it is believed that use of the centrifuge in the military applications was still very active during the long era of the Cold War between the US and USSR. For examples, it was used for estimations of the size of craters formed by nuclear explosions and planetary impact

of large bodies in the USA and by megaton bombs in the USSR (Schofield, 1998). However, details of the military applications are generally not available in the public domain.

Apart from his significant contributions to the development of the well-known Critical State Soil Mechanics between the 1950s and 1960s (see Fig. 1), Professor Andrew Schofield of the University of Cambridge had become aware of the early centrifuge work in the USSR and he initiated a series of centrifuge experiments to investigate slope instability problems, using a 2.7m diameter existing centrifuge originally built for aerospace testing at Luton Airport in the 1960s. Notable early Cambridge research students in centrifuge modelling include Avgherinos (1970) and Endicott (1971). Subsequently, Professor Schofield moved to the University of Manchester of Science and Technology in 1968 and built a 3m diameter centrifuge there. At Cambridge University, late Professor Ken Roscoe decided to develop a 10m diameter geotechnical centrifuge, which was completed in 1973 (Schofield, 1980). One year later, Professor Schofield re-joined the Soil Mechanics Group at Cambridge and took the responsibility for developing and operating the large centrifuge for more than 20 years. Under his outstanding leadership and strong influence, geotechnical centrifuge modelling has flourished not only in the U.K and Western Europe, but also in Australia (Randolph, 1991), Canada (Philips *et al*., 1994) and the US (Ledbetter *et al*., 1994).

In the East, the first geotechnical centrifuge was constructed under the leadership of Professor Mikasa at Osaka University in 1964 (Kimura, 1998) for validation of his history on the consolidation of soft clay and for studies on bearing capacity and slope stability. Although there were only 5 centrifuges in Japan before 1985, the number and variety of the centrifuges have increased dramatically over the last 15 years. Totally, there were 37 geotechnical centrifuges in 1998 in Japan (see Fig. 4). A number of new centrifuges have also been developed in the last few years. One of existing centrifuges is upgraded with a bi-axial shaking table. It can be seen from Fig. 4 that not only the universities and national research institutes have fully committed to the centrifuge modelling technology, but the private sector including contractors and consultants have also fully embraced the potential of this powerful experimental approach and technology.

Although some centrifuges had been used for structural applications in China since mid-1950s, two purposely-built centrifuges for geotechnical applications were constructed at the Nanjing Hydraulic Research Institute and the Yangtze River Scientific Research Institute in 1982 and in 1983 respectively (Wang, 1988; Zhu *et al*., 1991). Currently there are about 8 active geotechnical centrifuges in China including the newly developed one at Tongji University. The size of the centrifuges ranges from 10 to 450 g-tons. In addition, at least two more new centrifuges at Zhejiang University and the Changsha University of Science and Technology are being planned and developed. In the Far East, a number of geotechnical centrifuges have also been developed in the last 15 years such as the one in Singapore and Taiwan. Recently two new large geotechnical centrifuges have been developing in South Korea. One belongs to the Korean Government and the one is owned by one national university. Both of these centrifuges will be equipped with a bi-axial shaking table and a robotic manipulator.

3 THE HONG KONG GEOTECHNICAL CENTRIFUGE FACILITY (GCF)

One of the largest and most advanced geotechnical centrifuges in the world was established on the campus of HKUST and the GCF was officially opened in April 2001 (Ng *et al*., 2001). This 400 g-ton geotechnical centrifuge facility is equipped with unique and advanced capabilities including the world first in-flight bi-axial (2D) shaker, an advanced 4-axis robotic manipulator and the state-of-the-art data acquisition and control system. Figures 5 and 6 show an elevation view of the centrifuge equipped with the world's first bi-axial shaking table (Shen *et al*., 1998; Ng *et al*., 2001) and the 4-axis robotic manipulator (Ng *et al*., 2002a), respectively.

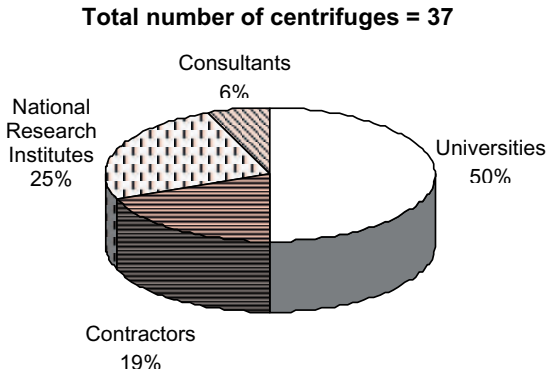

Fig.4. Breakdown of ownership of centrifuge in Japan (Kimura, 1998).

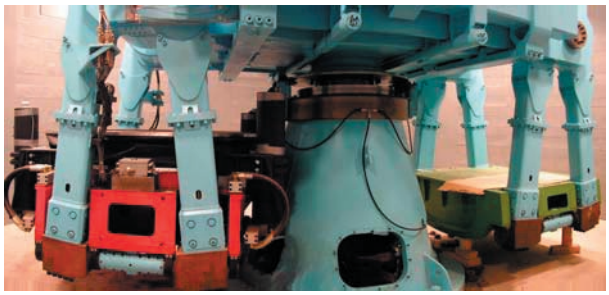

Fig. 5. The HK centrifuge and the 2D shaker (Ng *et al.*, 2001).

Fig. 6. The 4-axis robotic manipulator (Ng *et al.*, 2002a).

The centrifuge arm is supported on a vertical drive shaft running on a pair of pre-loaded tapered roller bearings. It is driven by a 350 hp Vector Drive Variable Speed AC motor coupled to the lower end of the vertical drive shaft through a right angle

speed reducer gearbox. The 400 g-ton centrifuge has a rotating arm of 4.2 metres nominal radius. Fig. 7 compares the size of the centrifuge in Hong Kong with other major centrifuges in the world. It is clear that the Hong Kong centrifuge is one of the largest in the world.

In total, the Hong Kong centrifuge has three swinging platforms. Two platforms are identical and are designed for non-shaking tests. Each of these 'static platforms' can accommodate a model of up to 1.5 m x 1.5 m x 1 m in size and up to 40,000 N in weight. The third platform comprises the bi-axial shaker and associated structural supports, hydraulic manifolds and reaction mass. The shaker slip-table can accommodate payloads of up to 0.6 m x 0.6 m x 0.4 m and up to 3000 N in weight. Table 1 summaries some key specifications for the centrifuge. Note that for static tests, the centrifuge can be operated at up to 150 g whereas for dynamic tests, the bi-axial shaker is designed to operate at up to 75 g.

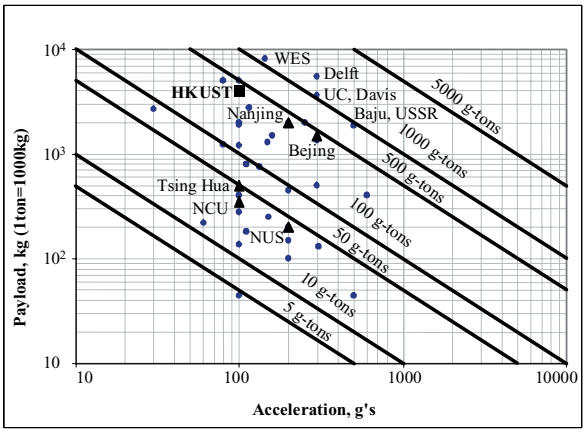

Fig. 7 Capacities of major geotechnical centrifuges in the world

Table 1. Technical specifications for the centrifuge.

Key item	Specification
Payload capacity	400 g-tons
Arm radius	4.2 m to the base of the swinging platform
Maximum acceleration	150 g (Static tests) 75 g (Dynamic tests)
Payload size	1.5 m x 1.5 m x 1 m for static tests, 0.6 m x 0.6x 0.4 m for dynamic tests

The centrifuge drive uses a vector drive to precisely control the acceleration, constant running speed, and deceleration of the AC motor driving the centrifuge. A computer is used to control the centrifuge and the shaker. Besides controlling the speed of the centrifuge, the computer is used to monitor operational parameters such as imbalance forces in the arm, temperature of the main bearings, hydraulic fluid temperature and pressure, and status of safety interlocks.

A fluid rotary joint is provided for supplying pressurized air, water, and hydraulic fluid to ancillary test devices mounted on the centrifuge during spinning. An electrical slip ring assembly is provided for transmitting electrical power and signals to and from centrifuge instrumentation, for control of various test devices and acquisition of experimental data. Some specifications for the fluid rotary joint and the slip ring assembly are given in Table 2.

In the main laboratory area, a 20 tonne capacity overhead gantry crane is available to move the pre-cast concrete panels above the centrifuge enclosure and to load and unload the centrifuge model containers. The crane is also used to interchange the static platform and shaker when required. The unused platform or shaker is stored in a recess in the floor of the centrifuge enclosure. The centrifuge is monitored using CCTV cameras and microphones, and an intercom is used to communicate between the centrifuge chamber and control room during model checkout. The hydraulic power supply is located below the main laboratory area in a room adjacent to the centrifuge.

Table 2. Specifications of slip ring and rotary joint assembly.

Key item	Specification
Slip rings	32 for analog signals, 8 for analog return, 16 for power
Co-axial cable channels	8 for video, and high frequency equipment, 4 high quality for digital signals (computer network)
Air ports	2 at 700 kPa, 0.05 m^3/min
Water ports	2 at 1400 kPa, 40 liters/min

3.1 *The bi-axial shaker*

As shown in Fig. 5, the HKUST centrifuge incorporates a unique bi-axial servo-hydraulic shaker (the first in the world), to be used for modelling earthquake-induced engineering problems (Ng *et al*., 2001; Ng *et al*., 2004). In consideration of the facts that earthquake motions are multi-directional in nature, HKUST decided to develop a bi-axial shaker in order to simulate more realistic earthquake motions in two horizontal directions simultaneously.

Because large shaking forces in two directions are possible with this shaker, development of the centrifuge and the shaker was carried out simultaneously, with the shaker designed as an integral part of the centrifuge. This integrated approach to the design was adopted in order to isolate the shaking forces from the centrifuge to as large an extent as possible and to produce high quality shaking motions.

To facilitate installation and maintenance, and to permit operation of the centrifuge at accelerations greater than 75 g for static tests, the shake table and its bucket form a single assembly that is removable from the suspension arms and replaceable by an optional static bucket.

With a payload weight of 3000N, the total moving weight (payload and shake table hardware) is about 10,000 N. To optimise the dynamic behaviour of the in-flight shaker, a large reaction mass (4000 kg) has been incorporated into the design. Several key technical specifications of the shaker are listed in Table 3. More details of the shaker are given by Ng *et al*. (2001).

Table 3. Technical specifications for the shaker.

Key item	Specification
Shaking direction	Two prototype horizontal directions
Maximum shaking acceleration	35g
Maximum shaking velocity	750 MM/SEC
Shaking frequency	0-350 Hz

3.2 *The 4-axis robotic manipulator*

An advanced and state-of-the-art 4-axis robotic manipulator has been developed for simulating details of construction activities such as soil nailing, pile driving, tunnelling and excavation in-flight (Ng *et al*., 2002a). As shown in Figure 6, the robot incorporates a standard tool changer and four tool adopters to permit interchanging tools without need for stopping the centrifuge. It can operate either in accordance with a sequence of pre-programmed instructions, or in response to real-time commands from an operator, in a "fly-by-wire" mode of operation. Key specifications are listed in Table 4. In a centrifugal field of 100 g, the equivalent prototype load capacities of the driving mechanisms are ±5 MNm torque, and ±10 MN, ±10 MN, and 50 MN forces in the x, y and z directions, respectively, which are sufficient for modelling common foundation problems. Another important feature of the robot is its capability to transmit fluid pressure and electrical signals to the tool through the tool adapters. This feature introduces test capabilities driven by pneumatic/hydraulic power or electrical power, and flexibility in control and signal transmission.

Table 4. Key specifications for the HKUST robotic manipulator.

	X-axis	Y-axis	Z-axis	θ-axis
Stroke	1.008m	0.839m	0.305m	270 °
Maximum Speed	6.67 cm/s	6.67 cm/s	3.47 cm/s	10 °/s
Accuracy	1.0 mm	1.0 mm	1.0 mm	1.0 °
Load Capacity	1000N	1000N	5000N	5 N-m

3.3 *Data acquisition and control system*

A new state-of-the-art distributed data acquisition and control system has been developed to support sophisticated modelling activities at GCF. The system uses multiple networked data acquisition servers to provide high-speed real time data acquisition on a large number of input channels. Custom-designed computer-controlled signal conditioning provides adjustable amplification of low-level signals from measurement transducers at the source, providing signals of very high signal-to-noise ratio and exceptionally high quality. A dedicated file server is used to archive experimental data. In addition, a relational database is used to store all information required to accurately interpret data acquired during testing.

By using the networked data acquisition and control system, clients, researchers and practitioners located at anywhere in the world are able to view experimental data in nearly real-time and to interact with the centrifuge operator while a test is in progress via the internet.

4 BASIC PRINCIPLES OF GEOTECHNICAL CENTRIFUGE MODELLING

For almost all applications in civil engineering, the major and fundamental difference in engineering behaviour between soil and steel (or concrete) is that the behaviour of the former material is strongly dependent on stress level and stress path, whereas the latter is generally not. This implies that any laboratory and field test involving soils without correctly modelling stress level and stress path will likely produce misleading or incorrect results. How to create a correct stress field effectively and economically for physical modelling of geotechnical problems has been a major challenge for engineers and researchers for many years.

4.1 *What is geotechnical centrifuge modelling?*

The basic principle of centrifuge modelling is to recreate the stress conditions, which would exist in a full-scale construction (prototype), using a model on a greatly reduced scale. This is done by subjecting the model components to an enhanced body force, which is provided by a centripetal acceleration of magnitude *ng,* where *g* is the acceleration due to the Earth gravity (i.e. 9.81 m/s^2). Fig. 8 shows a plan view of a model earth dam rotating at a constant angular velocity (dθ/dt) in a centrifuge test. For a typical static model test, $r(d\theta/dt)^2$ provides the artificial "gravitational" acceleration *ng*, which is used to increase the body force of the model, where *r* is the distance from the centre of rotation to any point in the model.

Stress replication in an *n*th scale model is achieved when the imposed "gravitational" acceleration is equal to *ng*. Thus, a centrifuge is suitable for modelling stress-dependent problems. Moreover, reduction of time for model tests such as consolidation time can be achieved by using a reduced size model.

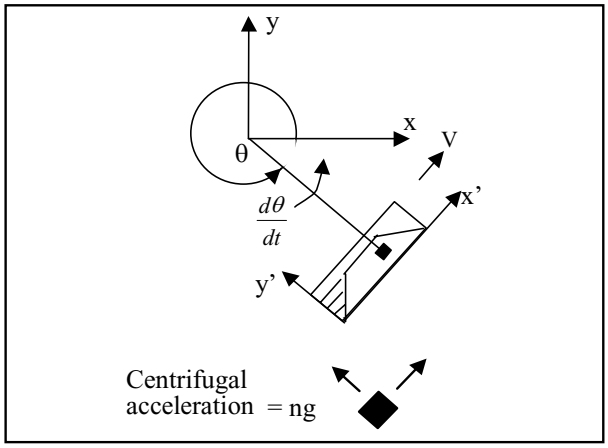

Fig. 8. A plan view of model earth dam in a centrifuge test.

4.2 *Scaling laws*

For centrifuge model tests, model laws are generally derived through dimensional analysis, from the governing equations for a phenomenon, or from the principles of mechanical similarity between a model and a prototype. Some commonly used scaling laws are summarised in Table 5.

It can be readily deduced from Table 6 that the stress level of a 100m tall slope can be correctly modelled by using a 1-metre slope model when it is subjected to an elevated "gravitational" acceleration of 100g (i.e., *n=100*). Also, a four-hour centrifuge modelling at 100g can correctly simulate a prototype soil settlement problem consolidated for more than 4.5 years (i.e., 4xn^2 or 4x100^2 hours). Substantial time reduction and hence cost savings by adopting the centrifuge modelling technique are obvious.

4.3 *Modelling of models*

One of the major criticisms of geotechnical centrifuge modelling or any physical model test conducted in both laboratory and in the field is the so-called "model and particle size effects". This is because there are two basic assumptions made in the majority of

model tests. Firstly, soil with grains small enough to behave as a continuum in a prototype will also behave as a continuum in a small laboratory or field test. Secondly, side boundary effects of a model test in a laboratory or in the field are either negligible or do not dominate behaviour but are quantifiable. Verifications of any model or particle size effects on centrifuge test results or scaling laws are commonly carried out using a technique called "Modelling of models" (Ko, 1988). The concept of modelling of models can be illustrated in Fig. 7, in which the model size is plotted logarithmically against the gravity level at which the centrifuge model is tested. All the models that are tested at 1-g conditions are assumed to be at full-scale (or prototype). Consider a 10m prototype, represented by A1. It can be modelled at the 10^{th} scale by a centrifuge model of 1m size (A2), or at the 100^{th} scale by a model of 0.1m (A3), etc. A2 and A3 are not only models of the same prototype, A1, but they are also models of each other. In the absence of the prototype, A2 and A3 can be compared with one another and, by using the scaling relations, their behaviour should both extrapolate to the same projected prototype performance, if there is no model size effect. It should be noted that in passing that A2 and B2 are not models of one another. On the other hand, B1, B2 and B3 are equivalent model tests. A factor of 2 to 5 is usually reasonable and practical between models.

Table 5. Some common scaling laws for centrifuge tests.

Parameter	Scale (model/prototype)
Acceleration	N
Linear dimension	1/n
Area dimension	$1/n^2$
Volume dimension	$1/n^3$
Stress	1
Strain	1
Mass	$1/n^3$
Density	1
Unit weight	N
Force	$1/n^2$
Bending Moment	$1/n^3$
Bending Moment / unit width	$1/n^2$
Flexural stiffness / unit width	$1/n^3$
Time (dynamic)	1/n
Tim (consolidation/ diffusion)	$1/n^2$
Time (creep)	1
Pore fluid velocity	N
Concentration	1
Velocity (dynamic)	1
Frequency	N

If the same model soil in terms of type and size is used in all the verification tests, particle size effects, if any, can also be quantified. Goodings & Gillette (1996) conducted 61 centrifuge model tests of granular slopes brought to failure to assess boundary and particle size effects on model behaviour. Particle size effects are negligible if the model failure "surface" is more than 30 times the particle size used. In investigating of the effects of model width, it may need to exceed model slope height by a factor of 5 for shallow wide slope failure to avoid adverse boundary effects. Their study clearly implies that if centrifuge model tests are conducted properly, model and particle size effects can be minimised to a negligible and acceptable level.

5 PRINCIPAL APPLICATIONS OF CENTRIFUGE MODELLING

According to Ko (1988), at least four principal applications of geotechnical centrifuges can be identified.

5.1 *Modelling of prototype*

Modelling of prototype is an obvious and direct application of the centrifuge modelling technique to tackle actual engineering problems. This centrifuge modelling technique is particularly useful when the construction of a full-scale test for earthquake or rain-induced slope failures under well-controlled conditions and for the behaviour of piles subjected to wind/wave storms for offshore oil explorations are almost impossible or prohibitively expensive.

Although field conditions have to idealised and only important features are duplicated properly in the model, the idealisation required for centrifuge modelling is generally less restrictive than assumptions routinely accepted for other methods of analysis such as finite element analysis.

Common applications of geotechnical centrifuge modelling to assist in understanding and designs include slope instability problems, ground movements due to tunnelling and excavations, influence of contaminant transport and earthquake resistance. Both qualitative and quantitative analyses are possible from the test results.

5.2 *Investigation of new phenomena*

Centrifuge modelling has been successfully applied to the study of various phenomena that are not well understood and extremely difficult if not impossible to be studied. Typical examples include plate tectonics, crater formations by nuclear explosions, various earthquake-induced events and soil liquefaction, and transportation of contaminants in soil. Behaviour of soil nails in loose fill slopes subjected to various rainfall conditions can also be investigated.

5.3 *Parametric studies*

Parametric study in geotechnical centrifuge modelling is an example where efforts for performing physical model experiments are best rewarded. Normally, a major effort is necessary to design and manufacture the first model, while the actual testing and small variations in the model are relatively easily performed. By varying some model parameters (geometry, loading and boundary conditions, rainfall intensity or soil type), the sensitivity of the test results to these variations can be evaluated and the most critical parameters can be identified. This leads directly to the possibility of generating useful design charts. Examples include bearing capacity of footings on slopes, critical design parameters in flow processes and capacity of laterally loaded pile groups.

5.4 *Validations of numerical methods*

Any modelling technique, physical or numerical, demands the acceptance of simplifications and assumptions. In many cases, the numerical techniques are still limited to two-dimensional problems for various reasons, while centrifuge modelling does not impose this restriction. On the contrary, it is easier to model a three-dimensional than a two-dimensional plane strain problem. For investigating any complicated geotechnical problem, it

would be ideal to perform both numerical analyses and centrifuge model tests. The results from these two techniques can then be compared.

Moreover, the success of numerical modelling requires a representative constitutive soil model which can capture essential features of soil behaviour and it also demands good quality of input model parameters. Very often, model parameters do not bear any physical meanings or are difficult to determine in the laboratory. This will certainly cast serious doubts on the computed results and hence computer-based design methods. For example, numerical modelling is not sufficient for the design of large jack-up structures or spud-can foundations for off-shore oil platforms subjected to combined vertical, horizontal and moment loading. Centrifuge model tests are carried out to verify numerical predictions and assist in designs (Dean *et al*., 1992).

In fact, any full-scale field test or case history may not offer good data for numerical modellers to calibrate their models and analyses. This is because the actual ground conditions, in-situ permeability, degree of saturation, soil homogeneity and boundary conditions are normally not known for certain to anyone involved. Computed results, which "match" with observed or measured field behaviour, may be fortuitous as a result of compensating errors. On the contrary, any centrifuge model is made by human beings and so all parameters required by any numerical modeller would generally be known. The centrifuge test results are therefore ideal for numerical modellers to verify and calibrate their models and modelling procedures. In principle, numerical models and procedures of any use should be capable of capturing essential features of centrifuge tests.

Ideally, both numerical and centrifuge modelling should be carried out to investigate the same prototype problem and verify the modelled results against each other.

6 EXAMPLES OF THE USE OF GEOTECHNICAL CENTRIFUGE MODELLING FOR DESIGNS

6.1 *Modelling ship impact on the bridge foundation at Container Terminals 8 and 9 (CT8 and CT9)*

To assist in the design of the Stonecutter Bridge foundation at CT8 and CT9 for the Highways Department of the HKSAR, Ove Arup and Partners Hong Kong Ltd (Arup) commissioned 16 centrifuge model tests to assess the loading on the bridge foundation and penetration into a rip-rap revetment, which is designed to protect the foundation (Lee & Peiris, 2004; Bezuijen *et al*., 2006). Based on the scaling law given in Table 6 and the design container vessel of 155,000 tons, an overall view of the centrifuge model set-up and details of the revetment, model ship, rail system and some instrumentation are shown in Figs 9 and 10, respectively.

The ship model was accelerated by means of a hydraulic plunger. The plunger stopped before the ship hit the revetment. The test programme started with 4 'proving tests' without the foundation. The remaining 12 tests, the 'impact tests' were performed after evaluating the results of the proving tests. The proving tests were performed with revetment CT9. Fig. 11 shows the penetration that was measured in the proving tests converted to prototype values. It appears to be proportional to the square of the velocity. Although there was some scatter in the individual tests, the same line was found when fitting the penetration to the velocity for the CT9 tests in the impact test programme. The angled tests and the test on the CT8 revetment have a higher penetration. The penetration for the angled tests was measured along the line of penetration. This means that when 1 cm of penetration is measured in a 30 degrees angled test, the penetration perpendicular to the revetment is only 0.87 cm. However, even when corrected for this difference it appeared that the penetration is quite large for the angled tests. It was concluded that a different deformation pattern (compared with the perpendicular tests) leads to relatively high penetration.

Table 6. Scaling rules (Bezuijen *et al*., 2006).

Parameter	Model/Prototype	Unit
Length	1/N	m
Velocity	1	m/s
Acceleration	N	m/s^2
Time	1/N	s
Volume	$1/N^3$	m^3
Mass	$1/N^3$	kg
Force	$1/N^2$	N
Pressure	1	kPa

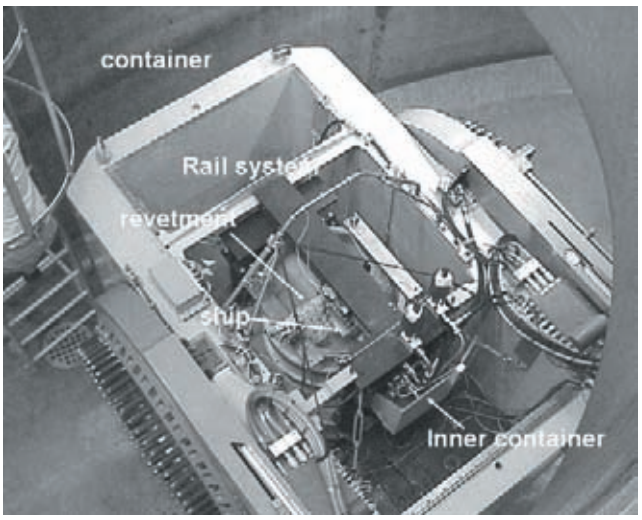

Fig.9. Overview of the model in the centrifuge (Bezuijen *et al*., 2006).

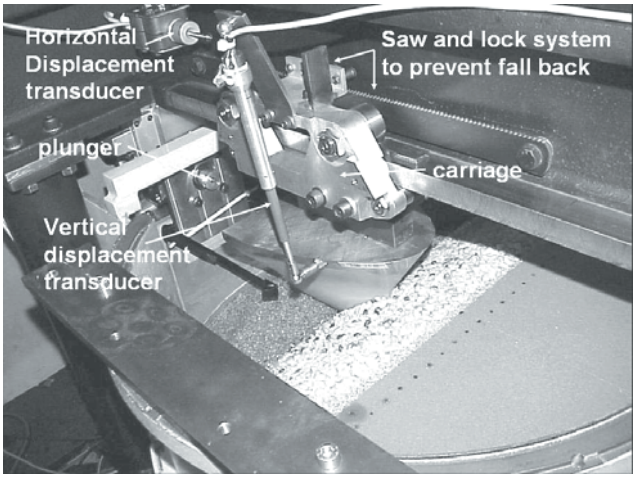

Fig. 10. Detail of the model after a test (Bezuijen *et al*., 2006).

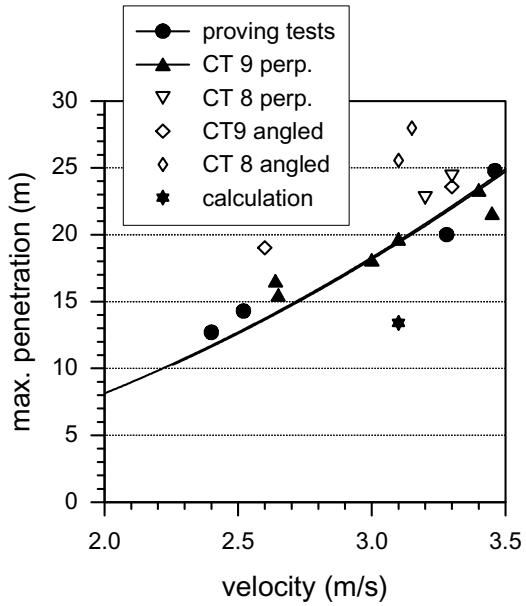

Fig. 11. Penetration of the ship into the revetment as a function of the impact velocity (prototype) (Bezuijen *et al*., 2006).

Based on the centrifuge model tests, it was founded that penetration of the ship into the revetment increases with the square of the impact velocity. An impact out of the center of the foundation leads to a rapid reduction of the loading on the foundation. An angled impact leads to hardly any reduction of the loading. No recordable permanent movement of the foundations due to impact was registered. The experiments could be simulated with a 3-D finite element calculation with reasonable accuracy. Having validated the numerical model, it was then used in the later design stages of the foundation to explore other changes such as pile group dimensions, configurations and removing the boundary and size effects from the physical model.

6.2 *Investigations of the mechanisms of potential static liquefaction of loose fill slopes*

Commissioned by the Geotechnical Engineering Office (GEO) of the Civil Engineering and Development Department of the HKSAR, a series of centrifuge model tests have been carried out to investigate potential static liquefaction mechanisms of loose fill slopes and the behaviour of loose fill slopes stabilized using soil nails (Zhang & Ng, 2003; Ng, 2005; Zhang *et al.*, 2006; Ng *et al*., 2007).

6.2.1 Static liquefaction of loose SAND fill slope

In order to investigate the possibility and mechanics of "static liquefaction" of loose fill slopes in Hong Kong, centrifuge model tests were conducted on both loose sand and completely decomposed granite (CDG) fill slopes subjected to either rising ground water table or rainfall or both. Fig. 12 shows an instrumented loose sand fill slope model with pore water pressure transducers (PPTs) (Zhang & Ng, 2003). The initial slope angle and relative compaction was about 29° and 68%, respectively. During the test conducted at 60g, the 18m-height (prototype) slope was destabilised by rising ground water from the bottom of the model (Zhang & Ng, 2003; Ng, 2005). The loose sand slope liquefied statically during the test (see Figs 13a and b for before and after the static liquefaction). Fig. 14 shows the measured sudden increases in the excessive pore water pressure ratio ($\Delta u/\sigma'_v$) within about 25 seconds (prototype) at various locations in the slope during the test. As shown in Figure 6.4, the completely liquefied slope inclines at about 4° to 7° to the horizontal after the test. The observed fluidization from in-flight video cameras and the significant rise in excessive pore water pressures during the test clearly demonstrated the static liquefaction mechanism of the loose sand fill slope. It should be noted that measurement of sudden and significant rise of excessive pore water pressure is essential to "prove" the occurrence of static liquefaction of loose fill slopes, if no video recording is available.

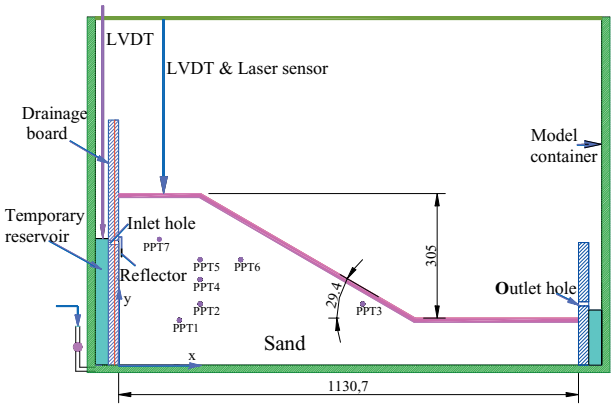

Fig. 12. Centrifuge model of a loose sand fill slope subjected to rising ground water table at 60g (from Zhang & Ng, 2003). Dimensions are in model scale.

6.2.2 Excessive settlement failure of loose CDG fill slope

Centrifuge model tests on CDG fill slopes subjected to rising of ground water and rainfall were also carried out by Ng *et al.* (2002b) and reported by Ng *et al.* (2003). The main objective of these tests was to investigate static liquefaction in loose CDG fill slopes, and the use of soil nails to stabilise these loose slopes. Fig. 15 shows a model package of a loose CDG fill slope equipped with an in-flight soil nailing system. The relative compaction of the fill model slope was less than 80%. At 60g, the 300mm high 45-degree model slope was equivalent to an 18m high slope at prototype. Fig. 16 compares the displacement vectors of the loose CDG fill slopes obtained from two model tests, one without and one with the use of soil nails. The soil nails were installed in-flight at 60g and it can be seen that the use of soil nails could substantially reduce soil movements by at least a factor of 5. No sign of static liquefaction of the slopes were observed during and after the tests. Similar findings are also reported by Take *et al.* (2004) from their independent centrifuge model tests using the same loose CDG fill in Cambridge University and a large-scale field test carried out by HKU (Tang & Lee, 2003). The slope angle of the field test was 33°. The height and width were 4.75m and 9m respectively. It was constructed by end-tipping method and resulted in a loose state with an initial dry density of 70% to 75% of the maximum dry density. Only excessive settlement of the slope was recorded and no sign of liquefied flow slide was observed in the test (Tang & Lee, 2003).

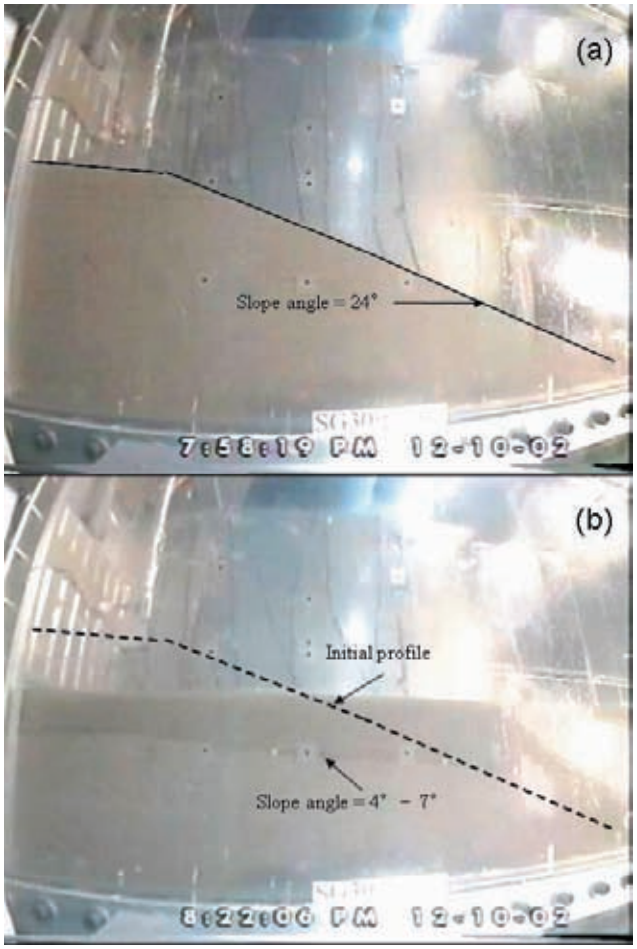

Fig. 13. Slope profile in a loose sand fill test (a) before rising ground water table; (b) after static liquefaction (from Zhang & Ng, 2003).

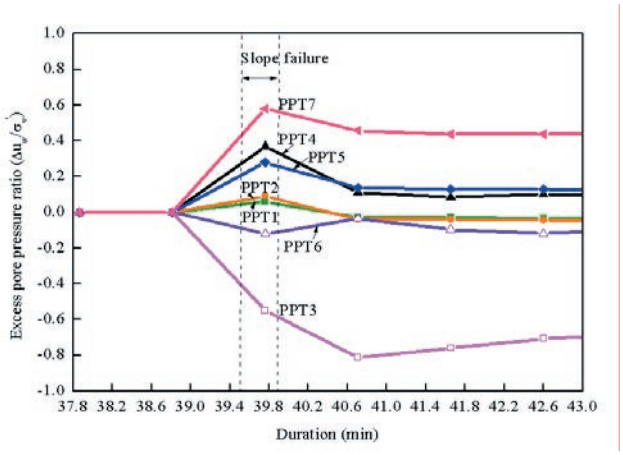

Fig. 14. Measured sudden and substantial increases in pore water pressure at seven locations inside the slope (Ng *et al*., 2007).

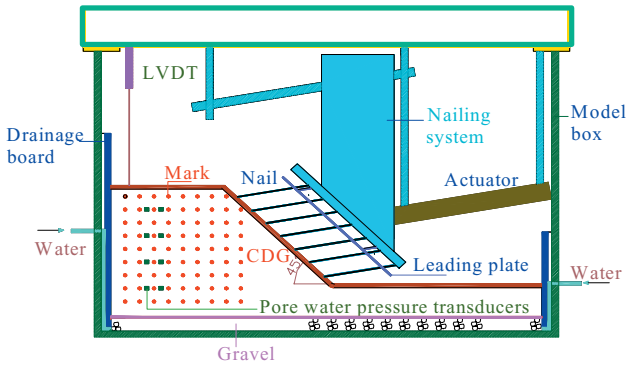

Fig. 15. A typical centrifuge model slope package equipped with an in-flight soil nailing system (from Ng *et al.*, 2002b).

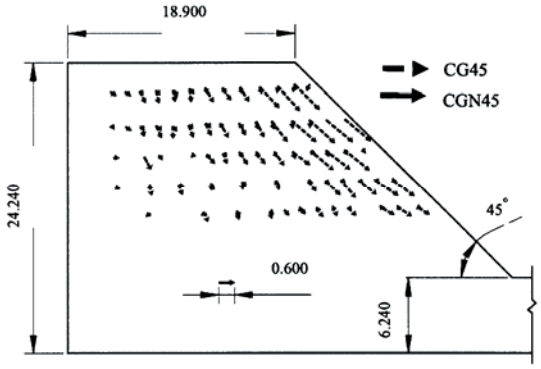

Fig. 16. Comparisons of measured soil displacements without (CG45) and with soil nails (CGN45) in two centrifuge tests using CDG loose fill at 60g (dimension in metres at prototype scale) (from Ng *et al.*, 2002b).

6.2.3 Dynamic centrifuge tests on loose CDG fill slope

To further investigate the possibility of liquefaction of loose CDG fill slopes, uni-axial and bi-axial dynamic tests were carried out (Ng *et al.,* 2004b; Van Laak & Ng, 2005). Fig. 17 shows the configuration of a typical model slope and its instrumentation. A rigid rectangular container having internal dimensions of 712 x 432 x 440 mm was used. The insides of the container were lined with a lubricated plastic membrane to minimize friction of the soil with the container walls. Five pairs of miniature accelerometers were installed in the embankment. Each pair was arranged to measure soil accelerations in two horizontal directions (i.e. X- and Y-direction). In addition, three accelerometers (ACC–T–X, ACC–T–Y and ACC–T–Z) were fixed on the bottom plate of the container in two horizontal orthogonal directions and one vertical direction to monitor the input motions. Miniature pore pressure transducers were installed in the soil near the accelerometers to record pore water pressures during shaking. On top of the embankment, three LVDTs were mounted to measure the crest settlement, and one LVDT and one laser displacement sensor were used to measure crest horizontal movement.

To correctly simulate the dissipation of excessive pore pressures in the centrifuge tests, sodium carboxy methylcellulose (CMC) powder was mixed with distilled de-ionized water to form the viscous pore fluid which was then used to saturate the CDG model embankment. After constructing the model embankment, the soil was vacuum-saturated with the viscous CMC solution. Small surface settlement was noted during the saturation.

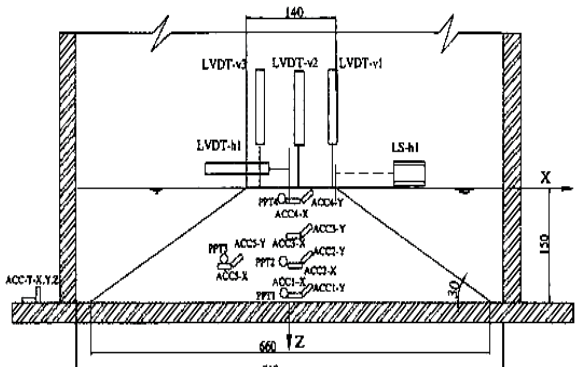

Fig. 17. Configuration of model slope and instrumentation (from Ng *et al*., 2004; Van Laak & Ng, 2005).

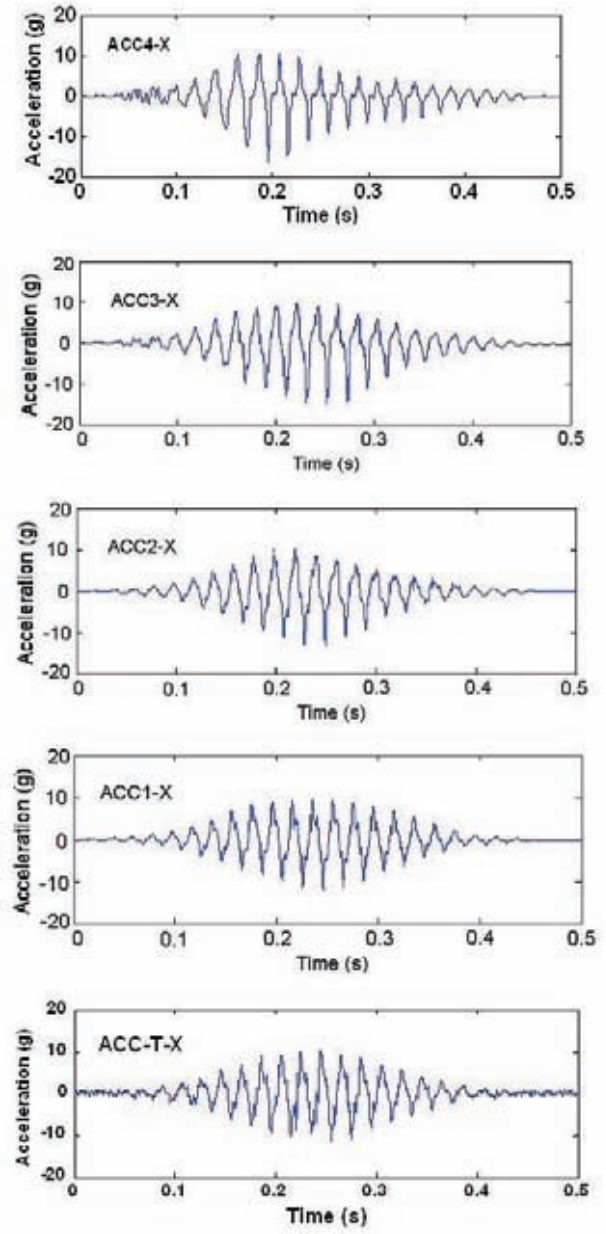

Fig. 18. Acceleration time-histories for uni-axial test M1D-0.3 (from Ng *et al*., 2004; Van Laak & Ng, 2005).

The acceleration time histories recorded by the accelerometers along the middle of the embankment are shown in Fig. 18. Relatively small magnitude acceleration was generated in the Y-direction and Z-direction; the shaking was predominantly uni-directional in the X-direction.

Fig. 19 shows the measured excess pore pressure ratios at different depths in the middle section of the embankment. An enlarged figure shows the measured pore responses during shaking (duration ~ 0.5 s). It can be seen that the higher the elevation of the model, the smaller the measured excess pore pressure ratio. This was probably due to the difference in the path of pore pressure dissipation or caused by the larger dilation tendency of the soil at a lower mean effective stress at shallower depths. The peak excess pore pressure occurred at about 0.33 s (i.e. 12.5 s in prototype) and dissipated to zero at about 10 s (6.3 m in prototype) after the triggering of the earthquake. The measured maximum excess pore pressure ratio was 0.85 at PPT1. No sign of liquefaction or flowslide was observed during the test.

Based on the static and dynamic centrifuge model tests at HKUST (Zhang & Ng, 2003; Take *et al*., 2004; Ng *et al*., 2004; Zhang *et al*., 2006) and the field test carried out at HKU (Tang & Lee, 2003), and numerical modeling of loose fill slopes (Cheuk *et al*., 2005), it may be concluded that statically liquefied flow slides is hard, or may even be unlikely, to occur in loose CDG fill slopes in Hong Kong. A preliminary design guide was published by the Geotechnical Division Committee of the HKIE for the use of soil nails in loose fill slopes (HKIE, 2003).

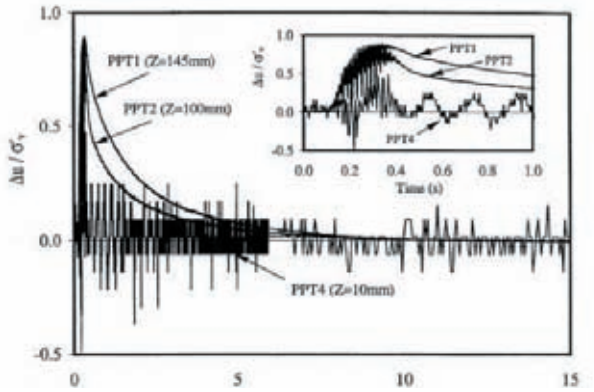

Fig. 19. Measured excess pore-water pressure ratios for uni-axial shaking test M1D-0.3 (from Ng *et al*., 2004; Van Laak & Ng, 2005).

6.3 *Environmentally upgrading of existing shallow loose fill slopes*

The Housing Department of HKSAR has been actively looking for innovative methods to preserve the environment by minimizing the need of felling tree when improving / upgrading loose fill slope. A series of centrifuge model tests was commissioned to investigate possible failure mechanisms of loose fill slopes. Fig. 20 shows the instrumented centrifuge model to study any potential static liquefaction behaviour loose CDG fill slope subjected to rising ground water table. The initial fill density was 66%. This model was used to simulate a 1.5m thick, 24m high layered fill slope when it was tested at 60g. In addition to laser sensors (LS) installed for monitoring soil surface movements, pore pressure transducers (PPT) were installed to measure excess pore water pressures during the tests. Effects of layering were considered by titling the model container during model preparation (see Fig. 21). Figs 22 and 23 show the occurrence of non-liquefied flowslide during the test and measured excessive pore water pressure

during two failures in the test, respectively. Based on the observed failure mechanisms and the small excessive pore water pressures measured, it was concluded non-liquefied flowslide of loose CDG fills slopes could occur but static liquefaction did not happen in the slopes.

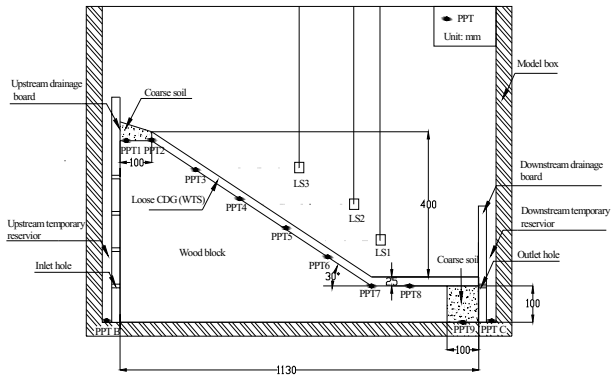

Fig. 20. Model package of an instrumented thin fill slope.

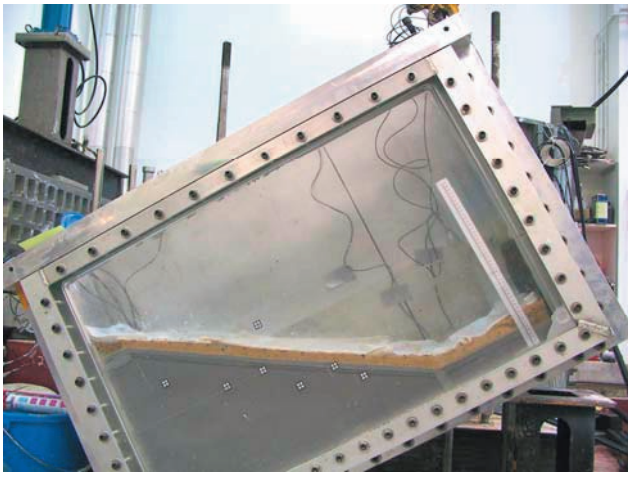

Fig. 21. Tilting of container by an angle of 30º for simulating effects of soil layering.

Fig. 22. Model top view showing non-liquefied flow slide.

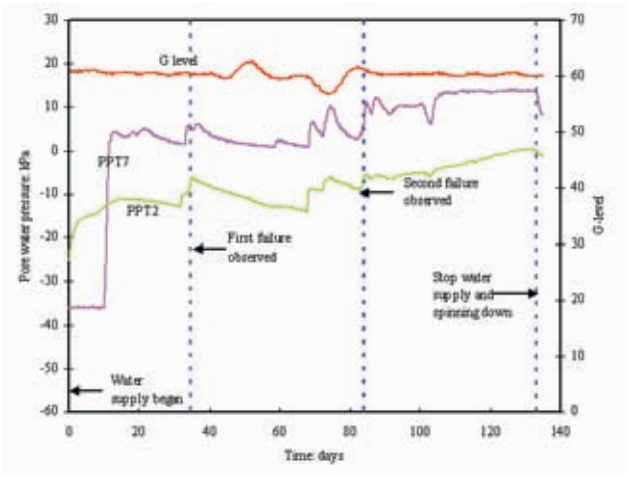

Fig. 23. Variations of measured pore water pressure at the crest (PPT2) and at the toe (PPT7) of the slope with time.

6.4 *Effect of soil nail heads in stabilising slopes*

Commissioned by GEO, a series of centrifuge model tests have been carried out in a research project to study the effect of soil nail heads in stabilizing slopes (Ng, 2005; Zhou *et al*., 2006). The model slopes were made of compacted CDG fill. Fig. 24 illustrates the instrumented model of a nailed slope centrifuge test (CGN65_30) in model scale. A similar model was used for an unreinforced slope model (CG65_30) except that no soil nails were installed. The soil used in the experiment was sieved CDG from Beacon Hill, Hong Kong with particles larger than 2 mm removed. D_{10} and D_{50} were found to be 8 μm and 600 μm, respectively. The specific gravity of the CDG was 2.62. The maximum dry density was 1845 kg/m^3 and the optimum water content was 14.2% by the standard Proctor compaction test. From undrained triaxial compression tests, the measured critical state friction angle for recompacted CDG was 40º (Shi, 2004). As expected, no true cohesion was recorded for re-compacted specimens.

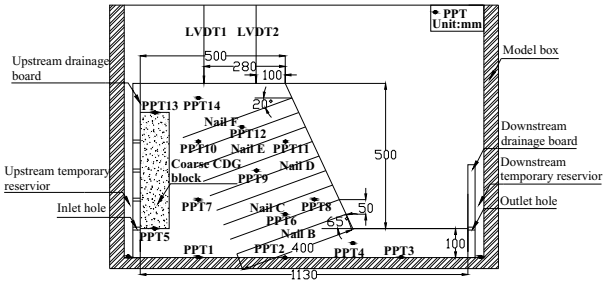

Fig. 24. A nailed slope model on the model scale (CGN65_30) (Zhou *et al*., 2006).

A horizontal bed of CDG mass was first formed by moist tamping layer by layer. The mass of soil required in each layer was calculated based on a dry density of 1753 kg/m^3 (relative compaction 95%) at a water content of about 16% (wet of the optimum). Fourteen miniature Druck PDCR-81 pore pressure transducers (PPTs) were installed in the model to measure changes in pore water pressures during testing. A rectangular block of coarse CDG was placed to facilitate the seepage of water into the slope mass. Silicon grease was used on both sides of the container to reduce side friction and preferential water flow at the interfaces. After the model was formed, the horizontal bed of soil was cut to

form a slope 500 mm high with a slope angle of 65°. The dimensions of a 1/30th-scale steep slope model were chosen to represent the prototype behaviour of a 15 m high steep slope when tested at 30 g.

After the slope model was formed, holes were drilled for inserting nails into the slope mass at 1 g. Fig. 25 shows the locations of five instrumented soil nails (Nail B to Nail F) at the middle of the slope. The arrangement of strain gauges on each nail is shown in Fig. 26.

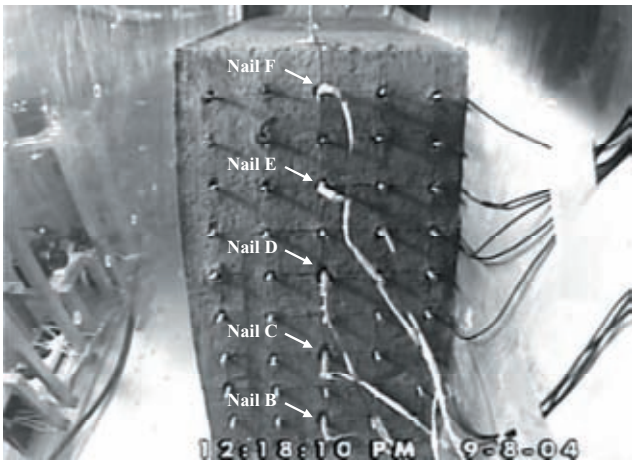

Fig. 25. Positions of instrumented soil nails (front view) (Zhou *et al*., 2006).

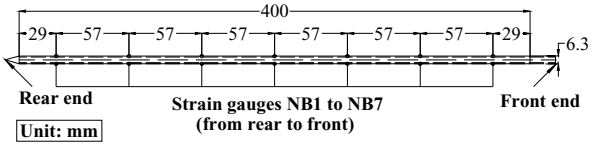

Fig. 26. Arrangement of strain gauges in Nail B (Zhou *et al*., 2006).

When the g-level in the centrifuge was slowly increased to the target value of 30 g, rising water table was simulated. A solenoid valve, which controlled water flowing from the standing pipe to the upstream temporary reservoir, was opened. Water flowed from the standing pipe into the upstream temporary reservoir because of the total head difference and it seeped into the slope through the coarse CDG block. The groundwater in the slope mass started to rise slowly. The water level in the upstream temporary water reservoir was increased up to the crest of the slope and the water was kept at this level until the end of this stage. At the downstream side, the water level was kept within 3 m (in the prototype) of the model base. The supply of groundwater was stopped when no further local failure, excessive settlement and significant sign of distress was observed after over four months of seepage in prototype time.

Test results of the unreinforced (CG65_30) and nailed slopes (CGN65_30) in terms of settlements at LVDT1 and LVDT2 versus g-level are shown in Fig. 27. As expected, settlements in both tests increased with increasing g-level. In the unreinforced model, CG65_30, settlement at LVDT1 increased smoothly with the g-level. Settlement at LVDT2 increased smoothly until 23 g (Point A). After point A, the faster rate was an external sign of the internal distress of the model slope, although no obvious failure sign was observed during this stage. Otherwise, the curve would possibly extend like the dashed line AB in Fig. 27. The difference in the measured responses by LVDT1 and LVDT2 was due to the fact that LVDT1 and LVDT2 were located outside and inside the Rankine active failure zone, respectively. In contrast, settlements at LVDT1 and LVDT2 in the reinforced model CGN65_30 increased with g-level smoothly from 1 g to 30 g. This means that this model was still very stable due to the installation of the soil nails. The settlement at LVDT1 in the reinforced model was greater than that in the unreinforced model at the same g-level. One possible reason for this was the existence of gap (i.e. 0.7 mm in model scale) between nail and soil around it. Assuming that all gaps were completely closed at 30 g, an additional settlement of 0.19 m was required. The measured difference at LVDT1 between the reinforced and unreinforced models was 0.12 m. When the additional settlement of 0.19 m at LVDT1 in the reinforced model was subtracted, settlement at LVDT1 in the reinforced model would be smaller than in the unreinforced one.

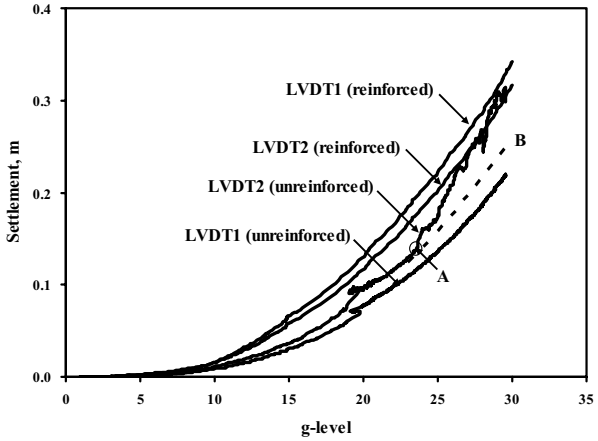

Fig. 27. Settlements at LVDT1 & LVDT2 with g-level (Zhou *et al*., 2006).

Fig. 28 shows a typical response of axial nail forces with g-level. Nail B was located in the middle of the lowest row of nails in the reinforced model (CGN65_30). NB1 to NB7 indicate changes of the axial nail forces along Nail B from its rear end to front with g-level. All axial nail forces are converted to the prototype scale based with a scale factor of $1/N^2$ (model/prototype). The positive axial nail force in this figure means a tensile force. It was found that below 10 g during the closing up of the gap around each nail, all axial nail forces along Nail B were zero. After the g-level was higher than 10 g, tension along Nail B except NB7 near the front end started to be mobilized. With the increase in g-level, the mobilized tensile force increased non-linearly. When the g-level reached 30 g, the maximum tensile force of 117 kPa was mobilized at NB4, whereas the least tensile force of 1.2 kPa was mobilized at NB7.

Fig. 29 shows measured axial nail force distributions in the model (CGN65_30) when the g-level reached 30 g. Zero axial force was assumed at both ends of each nail since no nail head and facing was used in the test. It was observed that almost all strain gauges recorded tensile force except at NE7. A relatively small compression force of 4.2 kN was recorded by NE7. As expected, axial forces were not distributed uniformly along each nail. Each nail mobilized its maximum tensile force at 1/6 to 1/2 a nail length away from its rear end. Among all the nails, the highest tensile force of 165 kN was mobilized at NC2. In contrast, the lowest maximum tensile force of 39 kN was mobilized at NF4.

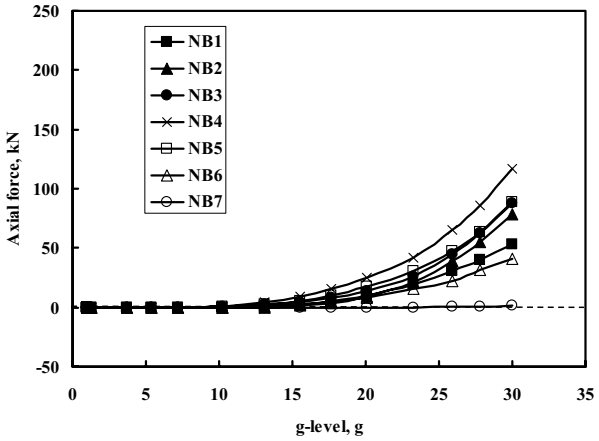

Fig. 28. Axial forces along Nail B with g-level (CGN65_30) (Zhou *et al*., 2006).

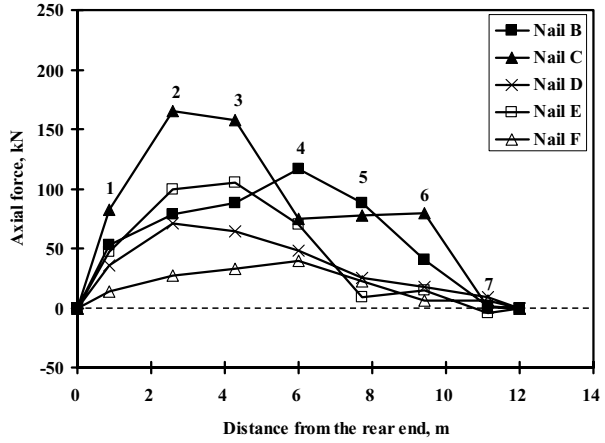

Fig. 29. Distribution of axial forces in each instrumented nail at the end of rising g-level (CGN65_30) (Zhou *et al*., 2006).

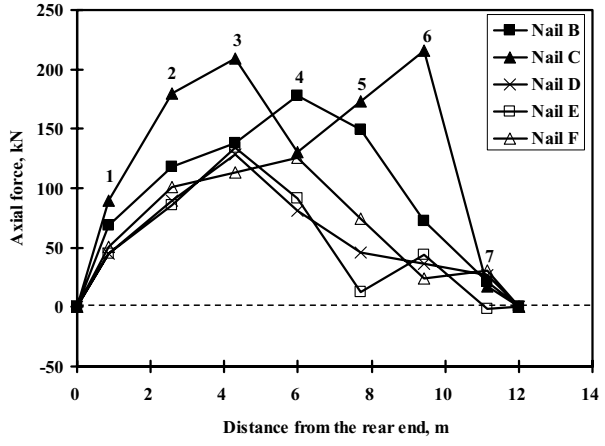

Fig. 30. Distribution of axial forces in each instrumented nail when the groundwater stopped rising (CGN65_30) (Zhou *et al*., 2006).

Fig. 30 shows measured axial nail forces when the groundwater stopped rising. Compared with those at the end of the rising g-level (Fig. 29), a similar axial nail force distribution was observed in each nail with the tensile force in each nail was mobilized further. A greater maximum tensile force of 216 kN, which was 31% larger than that at the end of the rising g-level, mobilized at Nail C. The maximum axial nail forces in nails D and F was mobilized to almost the same higher values as those in Nail E. Possibly this was because of the occurrence of crack at the crest during this stage. The measured maximum tensile force at Nail F increased by about 220% as compared with that recorded at rising g-level. The largest mobilized maximum tensile force in each nail shown in Figs 29 and 30 did not occur at Nail B, which was located at the lowest row. This is in good agreement with numerical predictions reported by Shiu & Chang (2005).

This study is still in progress. Further centrifuge tests on model slopes installed with soil nails with nail heads will be carried out.

7 CONCLUSIONS

There is no doubt that competition will increase in every sector of society because of globalization. How to enhance the competitiveness in each country and region will be the key for success. To increase the competitive edge will very often require a significant advancement in technology. Geotechnical centrifuge technology certainly offers some distinct advantages over many traditional methods of analysis, modelling and design. By possessing the largest number of geotechnical centrifuges in world (37 centrifuges in total in 1998, among which 25% of them owned by contractors and consultants and another 25% of them owned by national research institutes), the Japanese have already been making the best use of the geotechnical centrifuge to improve their construction technology and validate their innovative designs to compete worldwide.

The Hong Kong state-of-the-art geotechnical centrifuge equipped with a unique bi-axial shaker and an advanced robotic 4-axis robotic manipulator (second of its kind in the world) should be able to assist the construction industry to advance its technology and to improve its competitive edge globally.

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Geotechnical Practice of Reclamation in Hong Kong

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ABSTRACT: This paper presents a historical account of the reclamation practices adopted in Hong Kong to create new land for the development needs, tracing back to the early methods used in the 19th Century to current practice. The paper closely follows the evolution of the various reclamation practices, such as uncontrolled end-tipping of waste material and excavation spoil to form land used in the early days, the use of bottom dumping of Construction and Demolition (C & D) fill from barges to minimise the formation of excessive mud-waves , and eventually developing into the current practice of controlled filling by hydraulically placed sand fill together with the adoption of prefabricated vertical band drains to expedite the consolidation of the soft deposits beneath the seabed, attempting to reduce the residual settlement of the newly formed land. Nevertheless, problems are still inherent in the current practice; such as significant on-going settlements, slower than predicted rates of consolidation, and reclamation instabilities leading to the generation of mud-waves. Current practices are critically reviewed and improvement measures are recommended in order to overcome or minimize these problems and issues.

1 INTRODUCTION

Up until recent times, reclamation has been the major formation process for winning land for infrastructures and housing developments in Hong Kong. Reclamation in the Territory started very early in the 1850 mainly along existing shorelines. This method of winning land was also used in the 1950s for forming the runway of the old Kai Tak Airport. It can be seen that up to 1976, about 2000 hectares of land had already been reclaimed for these developments. And between the 1970's and 1990's, extensive land formation by reclamation had been undertaken by filling into the existing Shing Mun River in Sha Tin and Ma On Shan and the various bay areas including Tuen Mun and Tseung Kwan O for the development of new towns in order to relieve the surging demand on housing and infrastructure from the growing population of the Territory. At the same time, reclamations were also used to form the various container ports in the Kwai Chung area in order to cater for the rapidly growing container traffic in the Pearl River Delta. The largest reclamation of all is the formation of the platform for the new airport at Chek Lap Kok in the early 1990s involving 938 hectares of newly formed land. The most recent reclamation was the land formation and infrastructure works in Penny's Bay for the Disneyland Development, which was completed in 2005. Another phase of

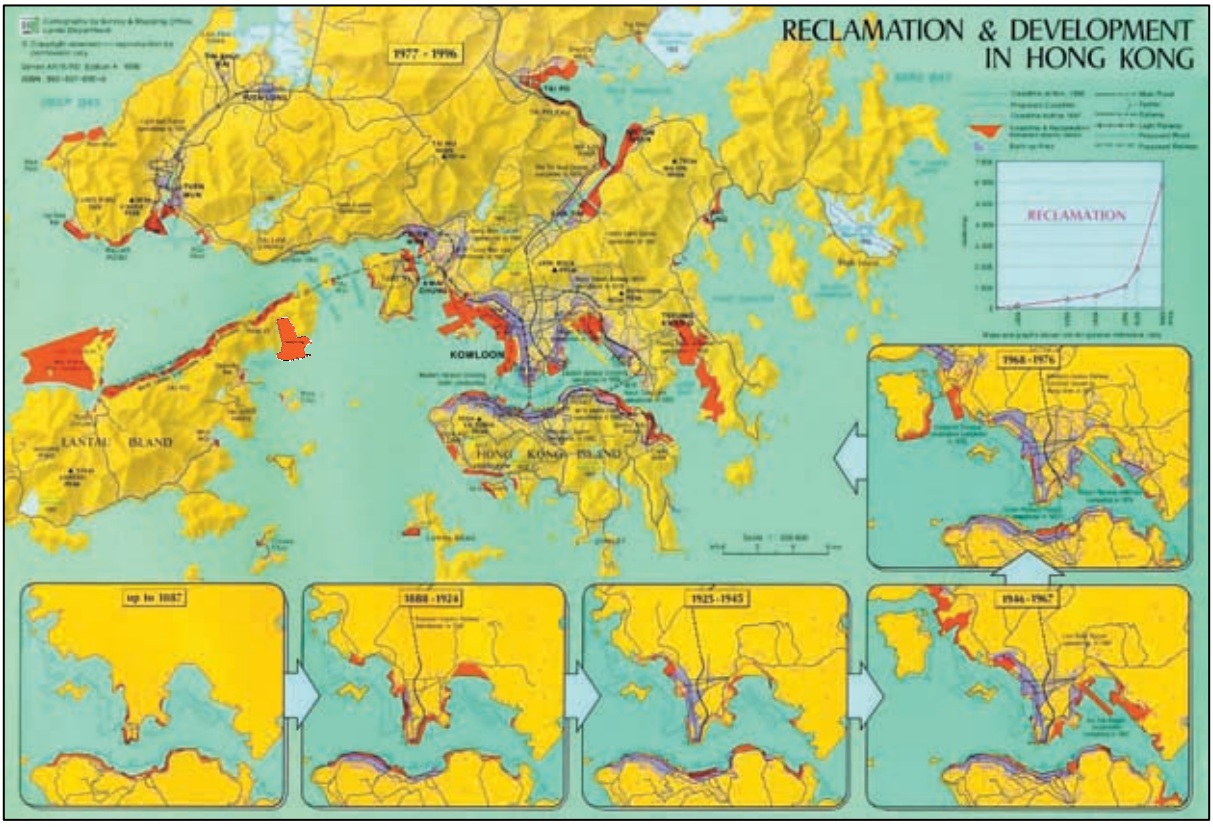

Fig. 1. Progress of reclamation in Hong Kong since 1850s (Courtesy of the Lands Department, HKSAR Government).

reclamation is currently in progress at Penny's Bay.

During these times, the method of reclamation has been gradually adapted with changes, in order to overcome the problems relating to filling over thick deposits of soft muds. Some of the reclamations conserved the mud in place, with or without ground treatment, whereas others removed partly or completely the existing soft deposits to eliminate these problems. The filling materials also changed with time from construction waste and land fill to mostly hydraulically placed sand fill.

A large number of publications have been prepared relating to these reclamations and their performances. Kwong (1997) provided a summary of projects in Hong Kong that had used the drained reclamation method. Detailed knowledge of the geotechnical properties of the marine mud has also been documented in many past works. Endicott (2001) discussed a large number of Hong Kong case histories and indicated that use of vertical drains in combination with temporary surcharge loading can effectively increase the rate of primary consolidation.

In the dredged reclamation method, all the highly compressible sediments are removed by dredging and replaced with sandy fill materials, thus significantly reducing total settlement permitting faster construction of the reclamation, and rendering long-term settlement small and more predictable. Construction of Container Terminals, CT1 to CT9 and the Chek Lap Kok Airport platform provides good examples of dredged reclamations.

The reclamation history and development in Hong Kong can best be summarized by Fig. 1.

2 DEVELOPMENT OF RECLAMATION METHODS AND TECHNIQUES OVER THE YEARS

Early Reclamation (1880 - 1950s)

Since the establishment of Hong Kong in 1841, reclamation projects have created an extensive developable land mass along the shorelines of Hong Kong. In the 1950's, the convenient way of reclamation was the uncontrolled end-tipping of fill into a seabed area enclosed by rubble mound 'seawalls'. Fill materials, which consisted of any available material ranging from building debris to refuse, were placed directly onto the seabed and the newly formed land was allowed to settle over time. The random filling activities would create significant disturbances to the soft marine mud deposits and led to the creation of large mud-waves ahead of the reclamation front, which subsequently accumulated into large pools of re-worked soft mud. On many occasions, the mud pool broke out above the water line. These areas of large thickness of mud generated significant settlements and also differential settlements across the newly formed land.

Fig. 2. End-tipping of excavation spoil at Sha Tin

In the period of 1950s to 1960s, the change in the economy of Hong Kong from an entreport to a manufacturing centre prompted more reclamation to provide land to fulfil the additional demands for industrial use. Nevertheless, the reclamation method used was still primarily end-tipping (Fig. 2). More thought and considerations were however given for reclamation for important facilities such as the Kai Tak Airport. With the thicker deposits of soft clay encountered offshore, larger settlements and longer consolidation times were required. In 1952, reclamation works for the Kai Tak Airport runway out into Kowloon Bay was started. The project had to deal with the soft marine clay below the seabed, which was up to 10 m thick. Based on estimates of strength and compressibility of the marine clay, it was decided to dredge the mud beneath the runway to eliminate future settlement of the runway and below the seawalls for the stability of these boundary seawalls, but to leave the marine mud in place at the less critical areas of the airport.

In the period of 1960s to 1970s, the same basic technique of building boundary seawalls and end-tipping within them was still being used. As usual, mud-wave formation was a major problem. It needed a number of years for the reclaimed land to consolidate prior to development. By the 1980s, where the mud was left in place, vertical prefabricated band drains were introduced and they were inserted into the mud to speed up the draining of the internal pore water to consolidate it under the reclamation load (which is now termed as a 'drained reclamation'). The usefulness of the drained method was demonstrated by the construction of test embankments at Chek Lap Kok in the early 80s, and many reclamation projects using 'drained' technique were completed satisfactorily.

Some of the more notable cases which actually formed the milestones of change of practice, in the authors' view, are further discussed as follows:

Reclamation at Sha Tin – New Town Development and Facilities (1960s to 70s)

Reclamation at Sha Tin to provide land for many of the facilities including the racecourse and the sewage treatment works commenced in the late 1960s and early 1970's (Fig. 3). Also, as

Fig. 3. View of reclamation works in progress in Sha Tin in 1976/1977.

part of Government's Decentralization Policy to re-locate the population from the congested districts in Hong Kong and Kowloon Peninsula to new towns in the New Territories. New Developable land was required in the New Territories. Major reclamation works in development of new towns including Sha Tin, Ma On Shan and Tuen Mun were undertaken. At the time, the common practice of reclamation was to end-tip (Fig. 2) the fill directly onto the soft mud to reclaim land. It was tempting to adopt this filling process as it was very convenient to win filling materials from excavating from the adjacent hillsides as borrow areas, and pushing them into the water to fill up the land. These activities had pushed the soft mud around generating pools of reworked mud of relatively little strength. Attempts to push mud offsite for removal by progressive end tipping were unsuccessful resulting in large thickness of very soft mud (almost a slurry) beneath the fill in places. The uncontrolled reclamation method has created many latent problems that needed long term maintenance efforts. For example, large voids have been discovered at the underside of some of the building structures and facilities (which are founded on piles) due to settling ground, which have also created significant elevation differences (Fig. 4). The problem is not unique with facilities, but was also found at some of the building developments formed at about the same time in the area. Large residual settlements were found causing major maintenance issues to these developments. These problems have demonstrated the weakness of the reclamation approach, which was appropriate at the time, but it needed substantial improvement.

Fig. 4. Ground settlement in Sha Tin.

Ma On Shan Area 77 (1980s to 90s)

Following on the formation of the Sha Tin New Town, reclamation for the Ma On Shan New Town was carried out as a continuation of the new town development programme (Fig. 5).

Taking on the earlier reclamation experience of Sha Tin New Town, the reclamation method was improved. Recognizing end-tipping method would cause stress concentration below the steep reclamation edge which had caused mud upheaval at the leading edge, an improved filling process was required to ensure even spreading of the fill. The reclamation technique adopted specified fill materials to be placed in layers on the seabed to achieve an even spread of surcharge on the sea mud. A filter geotextile was placed on the seabed, prior to filling works, to avoid mixing and the loss of fill material into the soft mud. A relatively flat leading edge of about 1 in 15 gradient was maintained at the reclamation front (leading edge) to ensure

Fig. 5. Ma On Shan reclamation - work in progress.

stability during filling. Land-sourced fill materials were loaded into bottom dump barges at a temporary jetty. Underwater filling was progressively built up in 1 m to 2 m thick layers by bottom dumping. Due to cost considerations, it was decided that no ground treatment to the mud would be implemented, and such works would be left to the future land users. In addition to the land-sourced fill, fill materials were also derived from construction and demolition (C & D) waste materials, which did cause problems with respect to quality of material and the rate of supply. With the bottom-dumping method, it was found difficult to ensure and control the placed thickness of the material, and the operation often resulted in local overfilling (mounding) near the point of discharge from the barge, and minor mud pockets were still unavoidable. At the same time, the lack of alternative disposal sites for C & D materials at the time in the Territory forced the reclamation site to receive C & D material at a rate much higher than the placement rate of fill using the bottom dumping method. Therefore it was necessary to stockpile the C & D materials in the form of high mounds above 10 m on the recent reclamation over untreated soft mud. Since this method had employed no strengthening measures for the soft mud, stability of the reclamation was just marginal with the stockpiles C & D material rising up to such great height. In 1989 a major failure occurred in Area 77 in Ma On Shan when such an embankment was constructed to stockpile the fill material. A large area of the reclamation disappeared below the sea and marine mud heaved-up and re-surfaced above the sea level at a considerable distance (about 200 m away) from the newly reclaimed shoreline. The failure measured about 150 m × 200 m in plan with a volume of about 200,000 m³ of soft mud. The outward edge of the reclamation was displaced laterally by about 8 m to 10 m.

Back analysis of the failure indicated that rapid loading of the embankment was the major factor contributing to the failure and unless additional means were adopted to accelerate the consolidation of the mud, the rate of construction should be controlled. The failure clearly indicated that the strength enhancement of the soft mud would certainly be of benefit in reclamations and future reclamation method should take that into account.

Tuen Mun New Town Development (1970s -80s)

In Tuen Mun, from 1975 to 1986, some 240 hectares of land were reclaimed from Castle Peak Bay. Marine mud in excess of 15 m in thickness was encountered in parts of the reclaimed area. Due to the problem of mud disposal, it was decided to limit the dredging of the marine mud and its replacement by fill material to

those areas where stable foundations were required - for seawalls, river walls and breakwaters. From the earlier experience of reclamation, it was recognized that end tipping from the shore would result in large scale disturbance and cause mud-waves ahead of the leading edge. Therefore, it was decided that a thin layer of granular material be placed on top of the mud, on the seabed, as a protection blanket, prior to placing end-tipped fill materials.

Some 65-70% of the total area of reclamation was completed successfully using this technique without serious disturbance to the underlying mud. However, it was envisaged that for such an area, the thick layer of marine mud will take years to achieve the primary consolidation which could result in ground surface settlements of the order of about 1.6 m.

In the late 80's some of the areas, such as Area 14 of Tuen Mun, situated at the southern part of the earlier reclaimed areas, was developed on a reclamation programme to limit residual settlement. The construction method and technique employed vertical drains to accelerate consolidation of the marine mud with geotechnical instrumentation installed to monitor reclamation performance. The mud was 8 m to 12 m thick.

The various land-sourced fill resources available locally had exhausted by the mid 1980's and marine transportation of land-sourced fill from other sources further away was required. Decomposed granite was used as reclamation fill, and placed by bottom dumping from barges, where the water depth permits, and by grabs from flat bottom barges where the draught was insufficient for bottom dumping. Once above water, end-tipping and dozing in layers was carried out. Nevertheless, as Area 14 reclamation was carried out mostly in shallow water, some level of disturbance to the soft mud at seabed was considered inevitable.

Once a stable reclamation platform was formed above the waterline, a granular drainage layer was placed and vertical band drains were installed from land plant. Geotechnical instruments were also installed to monitor the performance of the drained reclamation, which was found to be successful in controlling the consolidation of the mud and hence the settlements (Hadley, 1992).

Tseung Kwan O Reclamation (1980s - present)

The development of Tseung Kwan O (TKO) New Town at the latter stages of the new town development relied upon ground treatment of the soft mud for reclaiming the bay area in order to cope with the urgent need for new land. The TKO New Town is built mainly by reclaiming TKO bay (Fig. 6) and formation of platforms on either sides of a long narrow inlet. TKO has been fast expanding since its first population intake in 1988 in the public housing estates in Po Lam and Tsui Lam, turning a small fishing village and ship building industrial area in the 1960s to a major new community of about 340,000 people today.

Fig. 6. Overview of Tseung Kwan O reclamation.

The reclamations for TKO were undertaken by installing vertical band drains into the marine mud from marine plant and surcharging within selected infrastructure corridors. The marine mud was only dredged along the seawall foundations and along some of the major drainage culvert alignments. The thickness of the soft sediments ranged from 5 m to 15 m. The vertical band drains were generally installed from marine plant (Fig. 7) after placing a 1 m thick sand drainage blanket on the seabed, which permitted consolidation to commence early once filling started underwater. A geotextile was placed on the soft mud before filling commenced to prevent the loss of the marine sand drainage blanket into the soft mud. Initially, supply of filling materials, were derived from the site formation works on the adjacent hills, which were gradually taken over by Public Fill materials (Fig. 8) and hydraulic sand fills in selected area. The reclamation was designed to achieve 95% consolidation of the soft mud within a period of 9 to 12 months after completion of filling. Surcharge of 5 m to 10 m high were placed to minimize residual settlement along service corridors.

Fig. 7. Installation of vertical band drains in TKO.

Fig. 8. End-tipping of public fill in TKO.

Monitoring of the early reclamation at TKO has recorded a range of settlements with many in the range of 1 to 2 m for reclamations nearer to the shorelines. In most cases 90% of the settlement occurs within 12 months. When reclamations proceeded into deeper water, larger settlements and longer periods of consolidation were encountered.

Fig. 9. View of the slip failure at the TKP Stage I Landfill in 1982.

In March 1984, a major failure (Fig. 9) occurred at the TKO Landfill site, which involved about 500,000 m³ of materials (GCO, 1984). The containment bunds for the landfill were founded on untreated mud, which could not attain the strength to support the landfill. The collapse had submerged 3.6 million tonnes of construction and demolition waste and a horizontal displacement of about 20 m of the edge of the landfill site.

Pak Shek Kok Reclamation (1990s- present)

Pak Shek Kok Reclamation is a reclamation project targeted to use public fill sources as the supply of filling materials to form a new reclamation area immediately offshore from Pak Shek Kok, Tolo Harbour (Fig. 10). The total area is approximately 117 ha.

Fig. 10. Pak Shek Kok reclamation.

The project has been implemented in three stages under three separate contracts. Stages one and two of the project have been carried out in phases to provide about 17 hectares of land for the development. Stage three was designed to provide another 17 hectares of land for future housing development.

The reclamations were carried out by leaving the 6 m to 12 m of marine mud sediments in place. A layer of non-woven separator geotextile was placed on the top of the soft mud before placement of a 1 m to 2 m thick sand blanket. Vertical band drains were installed from marine plant. Public fill was placed by bottom dumping from barges and end-tipping.

During the course of reclamation for Stage 1, squeezing of the underlying marine mud was identified and 'mud-waves' were created. The extent of the mud-wave problem covered both the worksites of Stages I & II. The problem was most likely due to locally overfilling during marine dumping, in order to accommodate the large volume of public fill being received on site. The intake of public fill was then temporarily suspended. The mud wave was partially removed and confined within a layer of heavy duty geotextiles and vertical band drains were re-installed from land plant into the disturbed and displaced mud zones.

Subsequent reclamation was carried out by replacing the layer of non-woven geotextile with a layer of heavy duty geotextile both as reinforcement and as separator. The sand blanket was installed in two phases with the objective of confining the mud below to avoid lateral movement. After the vertical band drains were installed from marine plant, public fill was placed initially by bottom dumping from barges in a controlled manner. The fill was deposited in a grid pattern to form earth bunds so that the possible formation of mud movement could be confined. End tipping from land by trucks was then carried out.

Once the reclamation was above the sea level, piezometers and settlement plates were installed to monitor the behaviour of the soft muds. Surcharge was then applied to accelerate the rate of consolidation and to reduce residual settlement. To-date five stages of reclamation at Pak Shek Kok have been completed successfully using this method.

There are also many other reported drained reclamations undertaken during various times, such as the Island East Corridor at Sai Wan Ho (Siu *et al.*, 1988), Lai Chi Kok Bay (Bramall *et al.*, 1993), Yau Ma Tei Typhoon Shelter Reclamation, Siu Ho Wan MTR Depot (De Silva *et al*., 1998), etc., where similar reclamation techniques were used.

Dredged Reclamations

As can be seen from the sections above, reclamation over soft mud is inherent with problems. They can only be minimized and contained with appropriate measures but cannot be completely eliminated. For many important facilities and fast track developments, decisions have been made to partly or to completely remove the soft mud to acquire a higher level of confidence in controlling the residual settlements and meeting the construction programmes. These projects include the reclamation for the Chek Lap Kok Airport platform, the various Container Ports in Kwai Chung, and the Penny's Bay Development for Disneyland. Dredged reclamations are technically simpler and are much more reliable in minimizing long term settlements and delivery of project on time, but are not favoured from an environmental standpoint due to disposal of mud, and the need for more fill resources.

3 USEFUL OBSERVATIONS AND LESSONS LEARNT FROM PAST RECLAMATION PRACTICE

As observed from the reclamations of various periods, when different filling techniques had been used, the performances of the reclamations will vary, apart from the difference in engineering properties of the sub-soils, with the reclamation methods adopted and also the types of fill materials used.

3.1 *Reclamation fill*

As described in the sections above, the use of fill materials had changed with different reclamation methods and with time (the period when reclamation was carried out). Different types of fill

materials have their own specific characteristics, which then affect the performance of the reclamation.

Use of spoil and demolition waste in early reclamations

The early reclamations along the existing shorelines during the 50's were carried out by dumping construction waste comprising excavated earth fill and building construction waste directly on to the seabed. The major advantages of doing so were two-pronged: (1) to dispose off easily and conveniently the waste products generated from site formation works and demolition of older buildings; and (2) to generate new land in land-scarce Hong Kong. In these early days there were virtually no controls on the type of materials that were dumped and how it was dumped. These fill materials were heterogeneous, containing soil, and rocks of gravel size to boulder size (as large as 2 to 3 m), together with wastes comprising concrete, steel, wood, glass, other metal objects, rubber tyres, refuse and occasionally household appliances and other waste materials. Most often, the demolition waste was transported by truck and end-tipped directly on to the seabed. A designated area was opened to receive construction waste and spoil over a long period of time until the designated area was filled-up. Then the newly formed land was left unattended for a few years before it was taken up for development. Due to the presence of obstructions within the fill, major difficulties were commonly encountered when attempting to carry out subsequent engineering works including drilling, installation of pile foundations, sheet pile cofferdams, diaphragm walls and also when attempting to install vertical band drains to treat the soft mud.

Due to the heterogeneous nature of the fill which had no compaction, the compressibility is generally larger compared to that of undisturbed residual soil. Even though the rates of settlement immediately after reclamation were high the settlements had essentially ceased by the time the development of the sites were completed. Therefore, no major settlements had been recorded on developments on the earlier reclamations where marine mud was not displaced (Lumb, 1976).

Use of borrow materials

As indicated in the last section, most of the new town developments had their own designated borrow areas as the initial fill source for the reclamation. which was the most inexpensive way for acquiring fill. Like the land reclamation in the 50's, the earlier phase of the new town developments such as the Sha Tin New Town also adopted end-tipping as the most common method to fill up the land. The quality of the fill materials would be more uniform compared to construction waste; nevertheless they could still contain large boulders and rocks from the excavation, which became obstructions to subsequent construction works.

The major problem encountered with the end-tipping approach was that it generated significant displacement of the very soft to soft seabed sediments (particularly in marine mud), causing the generation of mud-waves and entrapping large pockets of thick marine mud. The mud pool caused local instabilities when differential loads had to be placed and the uneven thickness of the displaced soft mud subsequently resulted in large on-going differential settlements on the reclaimed land. Due to these incumbent problems, this method of land filling was gradually abandoned.

Another issue is the settlement originating from the fill layer due to creep. Measurements obtained from these reclamations indicate that the creep settlement of the fill is generally small. However, if appliances such as washing machines and refrigerators had been dumped with construction waste, the corrosion and collapse of the void spaces could cause ground subsidence, particularly if they are located close to the ground surface. Similarly, if refuse had been used, decaying of refuse can also cause significant settlements.

Table 1. Coefficients of secondary compression, $C_{\alpha\varepsilon}$, of fill (construction spoil & marine sandfill) from CLK airport project.

Fill Type		Type A	Type A/B	Type C
		Blasted granite rock up to 2 m in size	Decomposed granite & rock up to 300 mm in size	Marine Sandfill
Average		0.35%	0.5%	0.6%
Range	1st Yr	0.5% - 1.0%	0.5% - 2.0%	0.5% - 1.0%
	After 1st Yr (up to 3 yrs)	0.2% - 0.5%	0.2% - 1.0%	0.5% - 0.8%

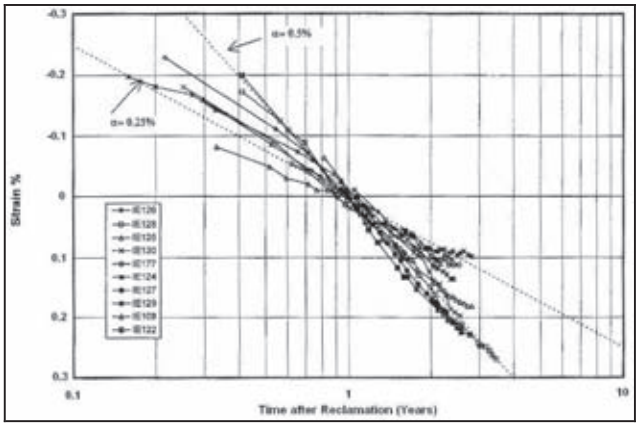

Fig. 11. Settlement of fill at CLK airport platform (after Pickles *et al.*)

The first few container terminals, CT1 to CT5, built in the 1980s used excavation spoil or fill sourced from borrow areas in Hong Kong. Therefore, the fill comprised residual soils, completely decomposed to highly decomposed rock (generally Granite and Volcanic Tuff) with core stones, boulders and rock fragments derived from Granite and Volcanic Tuff. The fill materials were end tipped onto a dredged seabed where the soft sediments (marine mud and soft alluvial clay) were removed. When the reclamation was above water, the top layers of the fill were compacted using rollers. These terminals have functioned very satisfactorily over the years, even though the terminal equipment such as Rubber Tyred Gantries (RTGs) and the container stacking yards are sensitive to differential ground settlement, particularly within the first few years after completion when the settlements are generally the largest. These observations confirm that the settlements arising from the fill materials from borrow sites when used in reclamations are generally small. At the Chek Lap Kok (CLK) Airport Platform construction, a combination of fill types was used. They ranged from up to 2 m in size boulders and rocks won from CLK and Lam Chau islands, to crushed rock and to marine sandfill dredged from Outer Deep bay and Urmston Road. The marine sandfill was placed as a capping layer over a filter geotextile placed above coarser fill layers. The fill was placed on a fully dredged seabed where all soft sediments were dredged. The extensometers that

were installed in Type A/B fill (As-blasted rock up to 2 m in size and excavated soil and rock with rock fragments up to 300 mm) and monitored over a period of about 2 years indicated coefficients of secondary compression, $C_{\alpha\varepsilon}$, ranging from 0.2 % to 1.0 % (see Table 1 and Fig. 11 below). Generally the creep rates are larger in the first year (up to 2%). The coefficients of secondary compression, $C_{\alpha\varepsilon}$, values determined from extensometers installed in various types of fill are summarized in the following sections.

Use of Public Fill

In the mid-1990s, the construction and demolition waste was re-classified by the Public Fill Committee (PFC) of the Public Fill Management section of the Fill Management Division of CEDD. Public Fill was thus defined as "Fill arising from construction, excavation, renovation, demolition and road works and comprising of rocks, concrete, asphalt, rubbles, bricks, stones and earth". Unlike in the past where the Construction and Demolition waste (C and D Material) contained all types of waste such as steel, wood, plastic, glass etc., in Public Fill these undesirable materials had been removed making Public Fill more suitable for reuse in reclamations and site formations. Inspection was also made more stringent at the public dump sites to control the size of boulders included in fill, which were generally limited to no more than 300 mm.

Public Fill has been used extensively in reclamations. These reclamations include TKO Town Centre, Tiu Keng Leng, Area 137 TKO, Ma On Shan, and Pak Shek Kok. Spider magnet type extensometers installed in Public Fill in TKO afforded an opportunity to monitor the settlement behaviour of Public Fill. When monitored over a period of 9 months, the settlement in fill ranged from 0 mm to 15 mm corresponding to coefficients of secondary compression, $C_{\alpha\varepsilon}$, of up to 1.0 %, with an average value of about 0.5% in the first 5 years after placing fill.

The Public Fill was either placed in layers using bottom-dumping (with barges) or by end-tipping from trucks when there were already a sufficient fill thickness placed underwater above the seabed as protection to prevent heaving of mud.

Vertical band drains were used at some of the sites and they were installed from barges and through a marine sandfill drainage layer placed on a filter geotextile laid on the seabed. These added measures helped to arrest some of the mud wave formations. Mud waves were still occasionally found but of a lesser extent compared with the direct end-tipping method. However, due to the pressing demands of the public dump programme, significant amount of Public Fill was incorporated within a short duration – not allowing sufficient time for the soft mud to gain strength by consolidation. Often, stockpiling of fill in the form of high mounds was needed to accommodate the large volume of incoming fill within the land already created, which could subsequently cause the reclamation to fail. These are the very reasons that caused the failures at Ma On Shan Reclamation and at the TKO Landfill Site.

Use of Hydraulic Sand Fill in Reclamations

Experience had shown that the use of land fill materials had difficulties in controlling the reclamation quality and the supply could be at time unreliable. With the exhaustion of available borrow areas, in the late 1980's the use of land sourced excavation spoil in reclamation works diminished and was replaced by the use of hydraulic sandfill. The many benefits and advantages that were associated with the change are:

(a) With the increased reluctance to dredge the marine muds (the soft mud to be left-in-place and treated with vertical band drains), it was necessary to place the fill in a controlled manner, in small layers in order to prevent failures of the soft mud. It was not possible to tightly control the placement method and the layer thickness by bottom dumping land sourced fill. Whereas, hydraulically placed sand can achieve the thickness requirements to acceptable accuracies.
(b) Since hydraulic sandfill is sourced offshore from marine sand pits, the sand is won by cutter suction dredgers and transported to site and placed by spreader barges or by pumping. Therefore, there was no need to double handle the material (if sourced from land the material must be trucked to a barging point to place in a barge, which then transports it to the required location and deposited the fill by bottom dumping).
(c) The rates of filling are therefore much faster with marine sourced sandfill compared with land sourced fill- hence the land can be ready much earlier and there was a lesser need to stockpile on land.
(d) Obstructions such as large boulders, rock fragments are not present in the reclamation.
(e) It is possible to densify the reclamation fill in order to provide a much denser 'fill raft'. The dense fill layer will further reduce on-going creep settlements and differential settlements, in addition to providing a good foundation layer.

Therefore, wherever possible hydraulically placed sandfill sourced from marine sources were used for reclamation. In fact, marine sand was first used for the runway and taxiways of the Kai Tak Airport and also for the filling in of Causeway bay for the construction of the Victoria Recreation Park. The extensive use of marine sand started with the construction of the container terminals. The first terminal to use hydraulic sandfill was CT6, where the marine mud was dredged and replaced with hydraulic marine sandfill, which was not densified, but surcharged. However, interestingly, after about 5 years of operations at the terminal, the pad footings at the corners of the container stacks at CT6 were reset due to ground settlement. The differential settlements though were not large, were in the order of about 25 mm to 50 mm., since the container stacks were found to be 'rocking' due to uneven ground. The settlements are most likely to have originated from the alluvium and the hydraulic sandfill, even though similar problems were not encountered at the earlier terminals where land sourced fill was used. It is acknowledged that some long term settlements due to creep do occur in as-placed hydraulic sandfills since the as-placed relative density is low, ranging generally from about 25% to 40 % (based on post filling CPT data), and also surcharging alone would not help in increasing significantly the relative density of a hydraulic sand.

Other major projects where hydraulic sandfill has been used are, Container Terminals 8 and 9 (CT8 and CT9), TKO Third Industrial Estate (Area 87), West Kowloon Reclamation, North Lantau Expressway Reclamation (NLE), Central Reclamation Phase II and III, Hung Hom Bay reclamation and at Penny's Bay. Amongst them deep vibro-compaction of the hydraulic sandfill was carried out at CT9, Penny's bay and along road alignments in the West Kowloon Reclamation. At CT9 and at Penny's Bay the relative densities achieved after vibro-compaction ranged from 70% to 85%.

3.2 *Dredged reclamations*

As indicated in the last section, dredged reclamations had been used where afteruse facilities on a reclamation would be sensitive to settlements. The amount of dredging will generally depend on the tolerable settlements of the facilities to be built, but generally all of the compressible very soft to soft marine muds and alluvial clays and silts are removed. Piezocone penetration tests and cone penetration resistance values are commonly used to identify the dredge levels of soft clays and silts. The values of cone penetration resistance, q_c, values used to determine the dredged base elevations for reclamations generally range from 0.5 MPa to about 1.2 MPa.

Past experience has shown that even after dredging sometimes significant on-going settlements can still occur. These settlements have arisen from the compression of the medium stiff to stiff alluvial layers and also from the residual soils and fill. Generally these settlements occur within a relatively short period of time following reclamation, but on occasions the settlements can continue over a longer period of time. A case in point is the settlement of the MTRC's Advance Tunnel Unit (ATU) at the Central Reclamation where the Tunnel Unit settled by more than 400 mm, even though the Unit was founded predominantly on Completely Decomposed Granite.

Significant on-going settlements, larger than 700 mm at some locations, have been experienced on the dredged reclamation at CLK Airport platform. The residual settlements predicted for CLK, from January 1997 to 2040 ranged from 200 mm to 600 mm (Pickles *et al*, 1998). A major settlement component is that arising from primary and secondary consolidation of the compressible strata of the thick CLK Formation. Also, a substantial contribution, up to about 50 %, is due to the creep settlement of the reclamation fill (Plant *et al*, 1998). Even though laboratory tests indicated coefficients of vertical consolidation, c_v, ranging from 1.5 m²/yr to 4 m²/yr (average of about 2 m²/yr – see Fig. 13), the values back calculated from actual piezometer data range from 4 m²/yr to 30 m²/yr with an average of about 16 m²/yr for the Alluvial clay of the CLK Formation (Plant *et al*, 1998). The larger field coefficients of vertical consolidation, $c_{v\,(field)}$, are most likely, due to the presence of interbedded sand layers and lenses, which effectively reduces the drainage distances for the dissipation of pore pressures. Therefore, in assessing primary consolidation settlement, it is important to investigate the presence of sand layers and lenses within the Alluvial Formation. Piezocone penetration test is a relatively inexpensive and a reliable tool to investigate them.

The investigations undertaken for the unusual settlements experienced in TKO also afforded a good opportunity to study the consolidation characteristics of the alluvial deposits of the CLK Formation since a series of spider magnet type extensometers and piezometers were installed to cover the site. These data indicate a relatively rapid rate of consolidation of the CLK Formation compressible layers at TKO. The consolidation of these materials was complete within about 1.5 to 2 years. The rapid rates of consolidation can once again be attributed to the presence of sand layers within the alluvium. The settlement contribution from the alluvium was estimated at about 250 mm under the reclamation load. The seawalls in Tseung Kwan O, with soft mud all removed, have settled between 200mm and 500mm due to the combined effects of the seawall and reclamation load and due to the groundwater drawdown effect caused by the SSDS tunnel construction. These settlements had occurred rapidly and had stabilized within 1 to 1 ½ years from the commencement of groundwater drawdown. These observations demonstrate the effective drainage conditions of the alluvial stratum prevalent in TKO bay.

Other important dredged reclamations are the Central Reclamations and the reclamations for the container terminals. Generally, no significant settlements have been reported from the terminal reclamations. However, locally the southern seawall of CT8 had settled close to about 1 m by 1997, within 4 years of construction. As mentioned before, at the Central reclamation, the MTRC advance tunnel unit of the MTRC Cross Harbour Immersed Tube Tunnel, which was placed under the seawall as entrusted works, settled more than 400 mm. The ground conditions here were alluvium and thick profile of completely decomposed granite, with the bedrock encountered at depths of more than 80 m. These results show that significant on-going settlements can arise from both the alluvium and also from residual soils and completely decomposed rock.

3.3 *Drained reclamations*

Stability

One of the major problems when reclaiming new land over very soft to soft mud is the interim stability during reclamation. Many a failure of the seabed has taken place in the past when placing fill on the soft marine mud sediments. Amongst them the more notable incidents are failures at Ma On Shan in 1987 and the Junk bay tip in 1984. These two failures have been thoroughly investigated by the Geotechnical Engineering Office. Both these failures had occurred when placing fill, in order to increase the fill height on already reclaimed land with a very shallow leading edge. The failure surfaces of both these cases were not very deep but relatively long, with the mud heaving at distances of about 20 m at TKO and at 200 m at Ma On Shan.

Seabed instabilities and mud-waves had occurred on many other reclamations where vertical band drains were installed from marine plant prior to filling (i.e. Pak Shek Kok, Tseung Kwan O, West Kowloon). It is clear in these cases the strength gain expected of the marine mud has not been attained. The reasons are many. However, the most likely reason is that the band drains had not functioned effectively to drain the pore water in the mud, possibly due to the disturbance created by the continuous displacement of the soft mud as the fill was being placed (see Fig. 12). At some reclamations the drainage blanket placed before the drains were installed, may not be sufficiently thick to restrain the underlying mud or provide adequate surcharge to

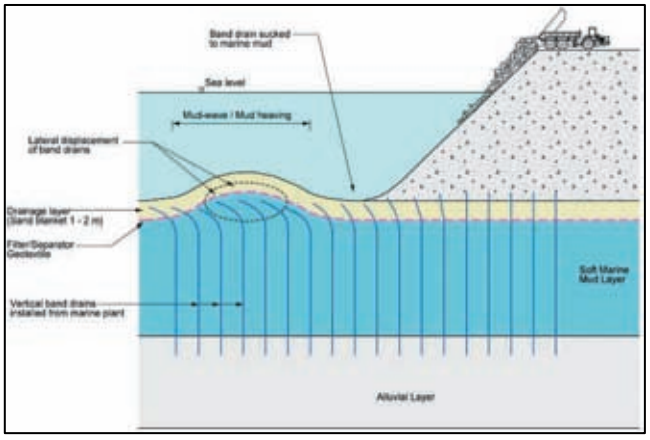

Fig. 12. Sketch demonstrating the loss of effectiveness of pre-fabricated vertical band drains due to disturbance during filling operations.

strengthen the underlying mud, and any displacement and heaving of the mud could damage the drains or bury the tops of the band drains in the displaced mud and out of the drainage blanket rendering them ineffective (see Fig. 12) by not having a drainage outlet. It is also doubtful whether the thin capping layer was adequate to prevent local punching of fill materials into the soft mud during bottom dumping. It is clear in these cases that the mud strengths were not sufficient to withstand the fast rate of fill placement. Formation of local mud pockets would weaken the overall stability of the reclamation. It is also difficult to maintain a gentle leading edge for ensuring stability by bottom dumping of land-sourced fill materials.

Similarly, end-tipping of Construction and Demolition waste and Public Fill has caused failures of the soft seabed in numerous occasions leading to the generation of large mud-waves and mud upheavals. These failures have been experienced at Ma On Shan, Sai Yin Pun, Pak Shek Kok and at TKO (see Figs. 9 and 10), as reported in the earlier sections. Since the end-tipping leads to a steep angle of repose both above and below the water line, the large load differential, created by the steep leading edge, cannot be sustained by the low shear strength of the mud and the capping layer. On many occasions the volume of Public Fill entering a reclamation site is large that it would be necessary to stockpile it or overfill close to the reclamation edge, which had led to failures (Ma On Shan). Therefore, end–tipping of Public Fill needs very careful planning and scheduling.

Slow dissipation of excess porewater pressure

One of the major design issues for drained reclamation is the prediction of the consolidation rate of compressible strata, Significant deviation from the prediction could lead to serious contractual implications, particularly on the contract completion, on reclamation contracts with ground treatment using vertical band drains. Slower than predicted rates of dissipation of excess pore pressures of the muds could cause major delays to the subsequent infrastructure works planned on the reclamation. The predominant reason for inaccurate prediction is very often due to optimistic evaluation of the coefficients of consolidation, c_v and c_h adopted in the vertical band drain design. The common design approach is to determine the coefficients of vertical consolidation, c_v, of the very soft to soft compressible marine mud and alluvial clay using one-dimensional oedometer consolidation tests. Then a factor, generally between 2 and 4 is applied to derive the field coefficient of horizontal consolidation, $c_{h(field)}$, based on the presumption that the horizontal coefficient is always larger than the vertical. This is not always a valid assumption in most situations, for the marine muds encountered in Hong Kong. ROWE Cell consolidation tests with radial drainage conducted on 70 mm and 250 mm diameter samples have shown coefficient of horizontal consolidation, c_h, values very similar to c_v values. Occasionally, when reclamations are carried out very close to the shoreline or within small bays that are fed by streams running down hilly terrain, larger c_h values can be expected due to the constant supply of sand and gravel from the streams depositing within the marine mud forming thin and thick interbedded sand layers and lenses, which would significantly increase the c_h value of the marine mud as well as the overall field coefficient of horizontal consolidation, $c_{h\ (field)}$. A case relevant to this observation is the Sai Wan Ho reclamation, where the rate of consolidation as observed from field measurements indicated $c_{h\ (field)}$ values of about 4 to 6.5 m²/yr; which was better than expected.

Whereas for many other cases where filling had been carried

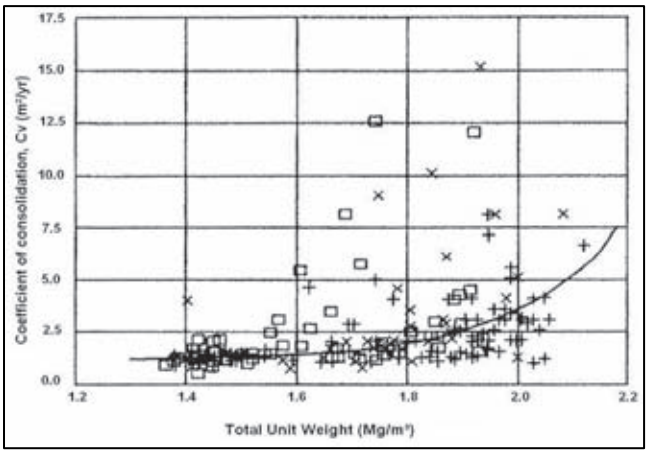

Fig. 13. Coefficient of vertical consolidation, c_v vs unit weight of clay soils of CLK (including upper soft clay)

out in deeper water, when the $c_{h\ (field)}$ value was back calculated from settlement data or from pore pressure dissipation data they are either equal to or only slightly higher than the coefficient of vertical consolidation, c_v, determined from standard oedometer tests. In some occasions, the $c_{h\ (field)}$ values were in fact less than c_v. The back-analysed field coefficient of horizontal consolidation, $c_{h\ (field)}$, from the CLK Trial Embankment data is about 2 m²/yr (back calculated using the long term settlement records from the trial embankment corresponding to vertical band drain spacing of 1.5 m.- Maunsell (1992) - SETY Report). The average coefficients of vertical consolidation, c_v of the marine mud (Upper Soft Clay) at CLK determined from oedometer tests is 1.3 m²/yr (Fig. 13). The average value of $c_{h\ (piezo)}$ determined from piezocone dissipation tests however, was 4.3 m²/yr for the Upper Soft Clay and 6 m²/yr for the Lower Clay using the Baligh and Levadoux (1980) method (Plant *et al*, 1998). It is also worth noting that the $c_{h\ (field)}$ is much lower than $c_{h\ (piezo)}$. Despite the observed low values, it must be pointed out that, in fact, these results are from the Trial Embankment - where the installation works had been undertaken with care and under close supervision on the workmanship and the quality of drains used; hence on the supervision and the workmanship aspects, it may not be a true reflection of a general work site in Hong Kong.

The review of the reclamation at Siu Ho Wan on the northern shore of Lantau, where the MTRC's Siu Ho Wan Depot is located, also revealed low field coefficient of horizontal consolidation $c_{h\ (field)}$ when they were back-calculated from the actual, measured field settlement data. They range from 0.4 m²/yr to 2.4 m²/yr with a mean value of about 1.3 m²/yr (De Silva *et al.*, 1998 - see Fig. 14). Since the consolidation was not complete at the time of

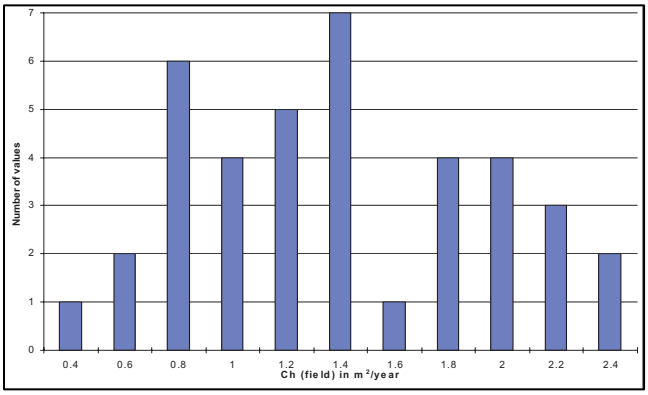

Fig. 14. Histogram of $c_{h\ (field)}$ back-calculated by settlements at Siu Ho Wan depot during ground treatment.

hand-over of the site (after site formation undertaken by the NLE site formation contractor) to MTRCL for the construction of the Depot, significant ground settlements were experienced after the site was delivered to MTRCL. Therefore, additional ground treatment works had to be carried out in spite of the tight construction schedule at hand. These measures included installation of stone columns at selected locations, and placing additional surcharge. However, even after execution of additional treatment works significant settlements were experienced by the buildings at the Depot, all of which were founded on shallow footings or rafts (see Fig. 15).

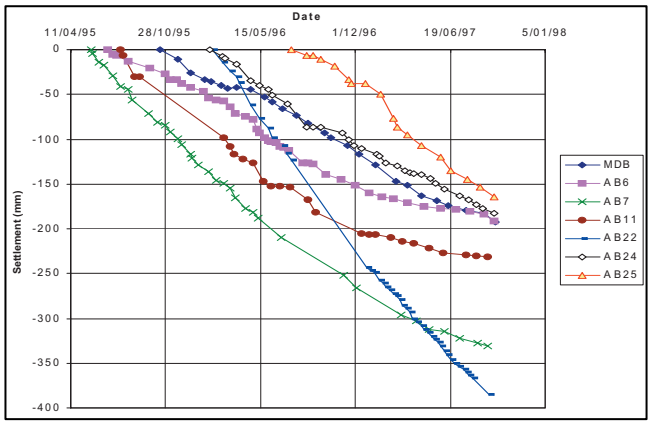

Fig. 15. Building settlement plot at Siu Ho Wan Depot (buildings founded on footings and rafts).

Also, it should be noted that the $c_{h\ (field)}$, values interpreted can heavily influence the overall drain performance. Very often, the drains work very effectively initially with high $c_{h\ (field)}$ values interpreted based on instrumentation results. However, the drain performance can drop sharply at the latter stages of the consolidation process resulting in much lower $c_{h\ (field)}$ values. The reasons for significant drop in the drain efficiency are many:

(a) the smearing of soil around the band drain as it is pushed-in with the mandrel;
(b) some drains not functioning due to the top of the drains (as a result of ground displacement) are fully embedded in mud (Fig. 12), when drains are installed from marine plant;
(c) high confining stresses on the drains restricting flow, particularly in types of drains without a suitable drain core;
(d) clogging of drains due to infiltration of fines into the drain across the filter, and due to inappropriate filters used or alteration of the filter resulting from the filter fabric stretching under the forces experienced in the installation process, and
(e) entrapped air within the band drain, in the filter fabric and at the interface between the band drain and the soil (a space of a few centimetres exists between the mandrel and the band drain which may not be fully filled with water when the mandrel is withdrawn) can impede the water flow into the band drain.

'Stagnant' Pore Pressures

Associated with the slow rates of excess pore pressure dissipation is the phenomenon of 'stagnant' pore pressures in the marine muds treated with vertical band drains. This phenomenon, has been observed in many reclamations in Hong Kong including Tuen Mun New Town, CLK Trial Embankment, TKO reclamation and also overseas, such as in Singapore Changi Airport Reclamation (Choa *et al.*, 1979). The 'stagnant' pore pressures are more common in thick deposits of marine clay/mud and the magnitude of the excess 'stagnant' pressure is generally larger, greater the clay/mud thickness. These pore pressures are in fact locked-in excess pore pressures or excess pressures that are dissipating very slowly indeed that they can be considered 'stagnant' for all practical purposes (see Fig. 16). The cause for this 'stagnation' or the slow dissipation rate is thought to be the 'well resistance' of the vertical band drains. The hydraulic head (and hence the hydraulic gradient) is inadequate for flow to occur through the band drains, or the hydraulic gradient is too small to cause an adequate flow rate for the excess pressures to dissipate. Generally some 'stagnant' residual excess pore pressures will remain at the tail end of the primary consolidation process because the well resistance will increase with time as the band drains deteriorate and the tortuousity of the drains increase with large ground settlement. In addition the reduction in the interstitial pore volume due to the significant secondary consolidation settlement effects could also contribute to the stagnant pressures at the end of the primary (EOP) consolidation stage.

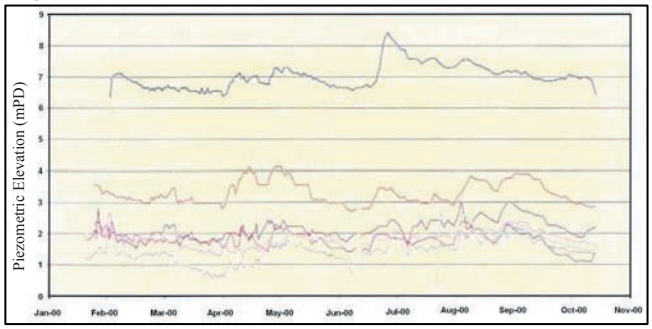

Fig. 16. 'Stagnant' pore pressures observed at TKO.

On-going long term settlement

On-going long tem settlement is a common problem in many drained reclamations (on-going settlements in undrained reclamations are expected and are not dealt with here). The settlement of reclamations where vertical band drains have been installed to accelerate the primary consolidation, can still continue for many years after completion of reclamation, up to even 15 to 20 years. Some of the projects where such on-going settlement had been experienced are; the early reclamations in TKO, TKO Town Centre, TKO Industrial Estate (see Figs 17 and 18), Ma On Shan (areas with treatment - see Fig. 19), CLK Airport Trial Embankment, Siu Ho Wan Depot (see Fig. 15) (De Silva *et al.*, 1998), North Lantau Expressway (MGSL, 1995), Lai Chi Kok Bay Reclamation (Bramall & Raybould , 1993), Castle Peak Bay Reclamation (Hunt *et al.*, 1982), and Yau Ma Tai

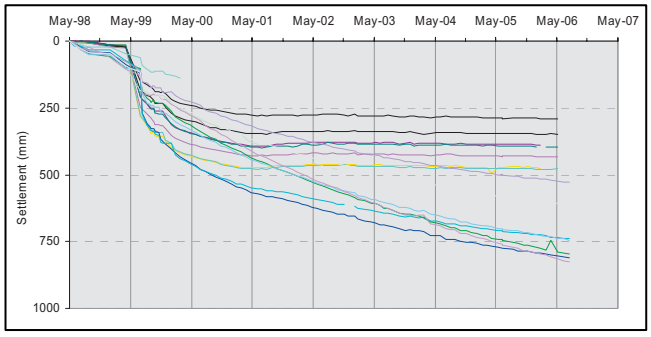

Fig. 17. Settlement records of Tseung Kwan O town centre zone B

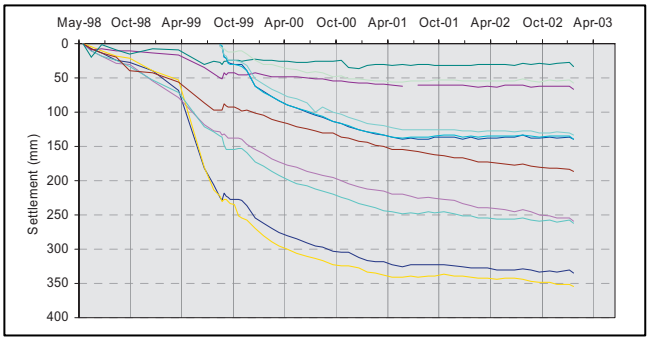

Fig. 18. Settlement records of Tseung Kwan O industrial estate.

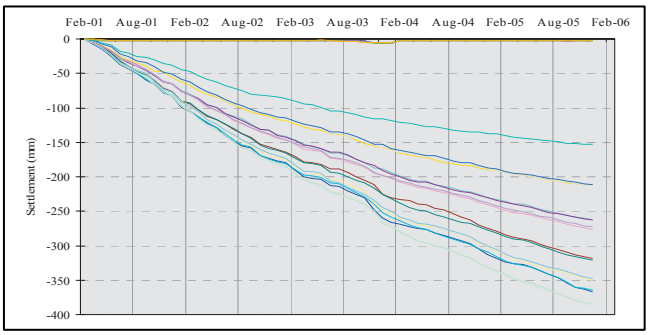

Fig. 19. Settlement records of Ma On Shan area with treatment.

Typhoon Shelter Reclamation (Endicott, 2001). However, long term (over 5 years) settlement monitoring records are generally not available from majority of these projects. But other evidence such as ground settlement around piled structures, broken water main connections to buildings etc. can show these long term settlement effects.

The on-going settlements could result from a number of factors: (1) On-going primary consolidation of the underlying thick deposits of alluvial clays; (2) on-going residual primary consolidation of the marine mud and the slow dissipation of the 'stagnant' pore pressures; (3) secondary consolidation of marine mud and the alluvium; and (4) creep settlement of reclamation fill, residual soil and saprolite.

The settlement contribution from the alluvial clays is generally complete in about 2 to 3 years after reclamation, except when the alluvial layers are thick and comprise soft clays. At certain locations in Hong Kong, such as at the western corner of Lantau Island (CLK and Tai O) where the decomposed rock is encountered at elevations of about –50 to –60 mPD, the sediments underlying the Hang Hau Formation, are geologically slightly older, but yet comprise soft to medium stiff compressible marine clay. These lower marine sediments if not treated, will also cause significant on-going settlements.

It has been demonstrated that the prediction of the performance of drained reclamation is highly sensitive to the interpreted consolidation characteristics of the soft deposits. It is imperative to closely examine the regional geology of the area and depositional history of the sediments when developing the engineering geological and the geotechnical engineering ground models of the site for the design, particularly relating to the interpretation of consolidation parameters. A very detailed study of the sedimentary history has already been done by Professor Wyss Yim of Hong Kong University, and would be invaluable in updating the geology of these areas.

However, in majority of the cases the on-going settlement is due to the residual primary consolidation of the marine mud and/or due to the secondary consolidation of the same deposit. For a given material the ratio, $\frac{C_{\alpha\varepsilon}}{C_c}$ (Mesri & Choi, 1977, Mesri & Castro, 1985), is a constant and investigations indicate that for marine clays of Hong Kong the ratio $\frac{C_{\alpha\varepsilon}}{C_c}$ is generally about 0.065 (Lo & Chan, 2004). Since $C_c^{'}$ $(=\frac{C_c}{(1+e_o)})$ for marine clay, considering the average slope of the e versus log(p) curves, generally ranges from about 0.15 to 0.3, the $C_{\alpha\varepsilon}$ ranges from about 0.01 to 0.02 (1% to 2%). Hence, large settlements can also occur due to secondary consolidation. However, research has shown that $C_{\alpha\varepsilon}$ can be about 3 to 7 times this value at stresses that are 1.5 to 2 times the pre-consolidation pressure (Mesri & Castro, 1987; Lo & Chan, 2004). Therefore, if the marine mud is normally consolidated under the reclamation fill load, at the time of completion of reclamation, then any additional fill placement or construction of infrastructure such as roads etc., can further increase the load causing initially the primary consolidation settlements and then much larger secondary consolidation settlements, which can be as large as 300 mm to 1,400 mm (3 to 7 times increase of $C_{\alpha\varepsilon}$). However, it is unlikely that the stress would increase by 1.5 to 2 times; but nevertheless, any increase of load can increase $C_{\alpha\varepsilon}$ significantly since the rate of increase of $C_{\alpha\varepsilon}$ is rapid (see Fig. 20) when the pre-consolidation pressure, $\sigma_c^{'}$ is exceeded. Therefore, the logical way to overcome these large secondary consolidation settlements and the possible additional primary settlements is to over-consolidate the compressible layers under the reclamation. As the working stresses on the compressible layers are less than the pre-consolidation pressure, $\sigma_c^{'}$, the coefficients of secondary consolidation, $C_{\alpha\varepsilon}$, reduces dramatically. This design approach, which was developed by the second author of this article in 1995, was adopted in the detailed design of the reclamation for the Container Terminals 10 and 11. The surcharge design was to achieve an over-consolidation ratio (OCR) of at least 1.2. This reclamation design philosophy has been adopted by Maunsell since 1995.

The settlement contributions from the various types of fill materials have been discussed in detail in the sections above.

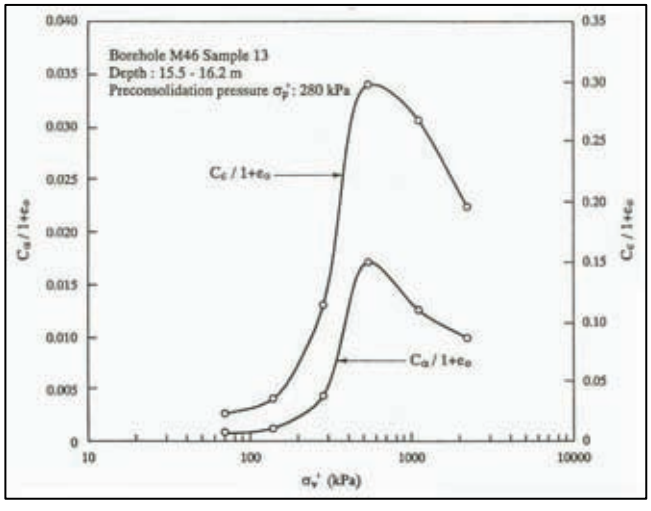

Fig. 20. Variation of $C_{\alpha\varepsilon}/(1+e_0)$ and $C_c/(1+e_0)$ with consolidation pressure.

4 PROPOSED MEASURES TO IMPROVE RECLAMATION PRACTICE

General

In summary, the problems encountered in reclamations as discussed, particularly relating to drained reclamations could include: (1) Instability and failures of the soft seabed leading to the generation of mudwaves and other large scale failures; (2) consolidation of the marine mud taking longer than predicted at the design stage; (3) 'stagnation' of pore pressures leading to significant on-going settlements; and (4) larger on-going settlements occurring over a long period of time after reclamation. Some possible improvements to the practice, in terms of design of reclamations and quality control during construction are discussed in the following section.

Seabed instability

Since the seabed sediments of a drained reclamation are very soft to soft, comprising most often of marine muds, in order to maintain the stability the reclamation leading edges must be very gentle with slope angles ranging from 1V:20H to about 1V:10H. It has been observed in the past that such gentle slope angles cannot be easily achieved with end-tipping from trucks or by bottom dumping from split barges. Placing fill by pumping it hydraulically has made significant improvements, yet it is still not possible to reliably achieve the limiting gradient all the time, which require that the discharge pipes are constantly moved about. Another more reliable way of controlling the placement of fill is to place it with spreader pontoons or barges. Only sandfill can be placed in a controlled fashion with spreader pontoons or barges, which have sophisticated computer controlled discharge mechanisms to discharge the sandfill.

Another additional measure to prevent instability is to ensure a thick capping layer is placed first, using the above-mentioned techniques, in thin layers with no leading edge, to provide enough counter-weight against heaving of the seabed. Only upon completion of the capping layer, the designed leading edge of the filling front, either by tipping or bottom dumping, can be placed. Also, a sufficient thickness of the capping layer can provide the necessary surcharge to consolidate the mud and to achieve an adequate strength gain. In general, only when the capping layer is in place, land fill or Public Fill can be placed. End tipping can only be carried out for the final placement of fill, when a sufficient thickness of fill has already been built up and the draught is not adequate to place the Public Fill from bottom dumping barges.

Time to Completion of Primary Consolidation of Marine Mud

In order to make a reliable estimate of the time required for completion of consolidation (which is defined as 95% consolidation), it is necessary to have reliable and accurate consolidation parameters that are applicable in the field, and that at the same time are appropriate to the ground treatment method that is proposed to accelerate the consolidation process; e.g. installation of vertical band drains.

It is often difficult to assess accurately the consolidation characteristics of the soft deposits only by undertaking laboratory tests on soil samples. A good understanding of the engineering geological setting of the proposed reclamation site is warranted, and it will help enormously in identifying general characteristics of the marine mud and the underlying alluvial sediments that are present in the area to be reclaimed. For example, for reclamation close to the shoreline and in the vicinity of a stream course or an ephemeral stream, it is likely that the marine deposit contains a higher percentage of silts and sand and also many interbedded layers and lenses of sand and gravel. The higher sand content in the mud will increase the coefficients of consolidation of the mud and the permeable interbedded sand layers will significantly increase the field coefficient of horizontal consolidation, $c_{h\ (field)}$. Correspondingly, their compressibility will be lower. Therefore, it is important to undertake a detailed engineering geological assessment of the proposed reclamation site and the surroundings as part of the ground investigation programme. The ground investigation shall be detailed and shall comprise drillholes, piezocone penetration tests, vibrocores and detailed geophysical surveys. The laboratory tests shall include small and large diameter ROWE Cell radial drainage tests and Constant Rate of Strain (CRS) consolidation tests, in addition to the standard laboratory tests.

The field parameters such as the field coefficient of horizontal consolidation, $c_{h\ (field)}$, can be affected by the type of plant used for ground treatment, the type of band drains used and also the height of surcharge that is applied. Hence in order to make an accurate prediction of the rate of consolidation, an accurate determination of the field coefficient of horizontal consolidation, $c_{h\ (field)}$, that correspond to the site concerned, would be most desirable. The most reliable way to determine this parameter, and the magnitude of the settlements, is to carry out a field trial using the same plant and equipment, band drains and the surcharge that is proposed for the project. However, the difficulty here is, this information in general cannot be obtained at the design stage, and it can only be obtained during construction.

Therefore, the approach to follow is to carry out the design based on the field coefficient of horizontal consolidation, $c_{h\ (field)}$, values back calculated from previous but similar sites of similar geological setting and where similar equipment, plant and band drains have been used. At the construction stage, the parameters used at the design stage will be verified with a full scale field trial, and if necessary the design parameters will be revised to suit. In order to follow this process it is necessary to allow sufficient float in the construction programme and contract provisions in the contract documents to accommodate the variations on the ground treatment design, when they are identified during construction.

'Stagnation' pore pressures

One of the main reasons for the stagnation of pore pressures is the well resistance of the band drains as the band drain performance deteriorates over time, and as the consolidation proceeds. It is also very important to select good quality and durable band drain products that can survive the rigours of the installation and to last the design consolidation period. The slower rates of excess pore-water pressure dissipation, at times, are also related to the effectiveness of the vertical drains.

When selecting the type of band drain, the performance of the drains must be checked with respect to the following: (1) flow through the drain under confinement stresses and the tortuosity (which is dependent on the total ground settlement - 15 %, to 20% of the soft mud thickness) that the band drain will be subjected to under the proposed surcharge; (2) the tensile strength of the drain to confirm that it will survive the forces it will be subjected to during installation; and (3) the performance of the filter and the drain core after stretching – the stretching that the band drain will undergo during installation. The field trial will be very useful in

assessing the overall performance of the band drains. The flow characteristics of band drains can decrease significantly when the strain due to settlement is above 15 % (for band drain lengths of about 10 m (Lo, 1998). A comprehensive quality control test schedule is essential to ensure that quality drains are delivered to site.

At the construction stage, the reclamation should be adequately instrumented to monitor the reclamation and the consolidation behaviour. Vibrating wire piezometers should be installed in all compressible layers. The dissipation of the pore pressures should be observed closely to identify stagnation of pore pressures, which can commonly occur in thicker soft deposits. One way of alleviating stagnation is to re-install band drains into the mud in critical areas. However, now the installation will need to be undertaken from top of the surcharge and possibly through Public Fill, and pre-drilling may be required. Another more practical option is to increase the surcharge and create a greater hydraulic gradient to expel the water. Therefore, provisions to undertake these measures should be considered in the contract documents.

Large on-going settlement after reclamation

As explained in the previous sections, the on-going settlements are predominantly related to the remaining residual component at the tail-end of the consolidation process (the final 5 % to 10 %, but yet the settlements can be large) and due to end-of-primary (EOP) secondary consolidation settlements. Therefore, the way to minimise these settlements is to achieve 100% consolidation under the expected working loads, at the end of construction and also to achieve a degree of over-consolidation under the working loads in order to reduce the coefficient of secondary consolidation, and hence the magnitude of secondary consolidation settlements. Generally for an economic design an over-consolidation ratio (OCR) of 1.2 can achieve significant reductions in the coefficients of secondary consolidation, $C_{\alpha\varepsilon}$.

5 RECOMMENDATIONS FOR A BETTER PRACTICE

In order to implement the various design considerations discussed above, appropriate measures shall be in place at different stages of the project. Recommended measures are presented in the following sections.

5.1 *Ground investigation*

A detailed ground investigation is essential. The investigation should not be limited to geotechnical and geophysical investigations and surveys. It should also include an engineering geological assessment of the proposed reclamation area and the surrounding areas, to reveal salient features characteristic to the depositional history of the area to be reclaimed.

The geophysical investigation can comprise a bathymetric survey; a side scan sonar survey, a seismic reflection survey and a magnetic resonance survey (where metal objects such as shipwrecks etc need to be identified).

The geotechnical investigation shall at best include drillholes, piezocone penetration tests and vibrocores. Insitu tests shall include field vane shear tests, piezocone dissipation tests, permeability tests, SPTs (in stiff and dense strata) and preferably self boring pressuremeter tests with holding tests (dissipation tests) in soft muds. The laboratory tests should include Constant Rate of Strain (CRS) consolidation tests in mud and the alluvial clays, ROWE Cell radial consolidation tests on standard 70 mm diameter samples and on large (250 mm) diameter samples of soft mud where vertical band drains, stone columns or sand compaction piles will be used to treat the mud. These tests will be carried out in addition to the conventional tests. The consolidation tests shall be of long duration in order to better determine the coefficient of secondary consolidation, $C_{\alpha\varepsilon}$.

If vertical band drains are envisaged to be used on the project, it is recommended that a series of appropriate band drain types are also tested with the soil samples retrieved from the site, to determine the flow, filter and discharge characteristics of the drain types which are identified to be used.

5.2 *Reclamation design approach*

Design parameters

In order to obtain accurate parameters for prediction of rate of consolidation, the field and laboratory data should be closely analysed and the design parameters applicable to the site shall be established with due care. The laboratory soil parameters of the mud, the consolidation parameters such as c_v, $c_{h\ (pressuremeter)}$, $c_{h\ (piezo)}$ and $c_{h\ (radial)}$ and C_c, shall always be compared with other similar sites where the field consolidation parameters such as field coefficient of horizontal consolidation, $c_{h\ (field)}$, , have been back-calculated. Based on a detailed review of all these parameters, and due thought given to the engineering geological setting of the site, a learned engineering judgment should be made on the design consolidation parameters, particularly the field coefficient of horizontal consolidation, $c_{h\ (field)}$ of the soft muds to be adopted for the captioned site.

The consolidation parameters of the alluvium (or the lower marine sediments) shall not be overlooked and also shall be determined with care, giving due thought to the information gathered from the CPT profiles (which are useful to identify the interface, or the base of soft alluvial deposits), vibrocore logs, drillholes, the engineering geological setting of the site and back analyses results from similar or adjacent sites.

Since the secondary consolidation plays an important role in the long term settlements, accurate coefficients of secondary consolidation and secondary compression parameters, $C_{\alpha\varepsilon}$, shall also be determined from long term consolidation tests.

Design philosophy

In general the design approach should be aimed to achieve more than 100 % primary consolidation under working load conditions, as far as possible. In areas that are sensitive to settlement and where on-going settlements must be minimized, the soft mud is to be consolidated to achieve a degree of over-consolidation of at least 1.2 at any depth within the mud layer under the working load conditions after completing the reclamation. Additional surcharge will be required to achieve the over-consolidation of the mud.

In order to maintain the stability of the seabed, all temporary leading edges shall achieve factors of safety of at least 1.2. The reclamation fill placement methodology shall be dictated by this requirement. If placement of surcharge would compromise the factors of safety then methods such as vacuum preloading shall be considered.

In reclamations where the residual settlement criteria are very stringent or where the time available for ground treatment is very short, other more reliable ground treatment approaches such as Stone Columns, Sand Compaction Piles or Deep Cement Mixing should be adopted.

5.3 *Construction stage*

Tender evaluation

Stringent controls must be incorporated in the tender documents pertaining to the use of vertical band drains for drained reclamations. These shall include a suite of appropriate tests to be carried out on the prefabricated vertical band drains that the tenderer is proposing to use. These test results shall be reported in the tender submissions.

Despite how carefully the design has been prepared, mud performance could still deviate from the design, given the variability of the ground for the large extent of reclamation. Provisions in the tender documents shall be included for remedial ground treatment works if those proposed do not perform exactly as required.

Construction Stage

It is warranted that the contractor shall carry out additional tests on the vertical band drains he intends to use, using bulk soil samples retrieved from the site, to confirm the performance of the drains under the site conditions. Routine assurance tests to verify the specified drain properties would also be necessary to ensure the quality of the site deliveries. In addition a field trial shall also be included in the contract, and shall be carried out prior to site wide installation. The trial should be adequately instrumented.

The reclamation site shall be fully instrumented to closely monitor the consolidation/creep behaviour of the soft muds, the soil strata below the mud and the reclamation fill. The instruments shall comprise, vibrating wire piezometers, extensometers, inclinometers at leading edges and seawalls, deep settlement markers, surface settlement markers and observation wells.

The instrumentation data shall be reviewed and analysed rigorously at regular intervals during consolidation in order to fully understand the behaviour of the various sub-strata. It needs no emphasizing the importance of closely reviewing the site data to ensure that the degrees of consolidation and the over-consolidation ratios have been achieved before instructing the removal of surcharge upon completion of ground treatment works. Early removal of surcharge in the past had proven to cause more subsequent delays and render the need for 'painful' and expensive corrective measures. Post ground treatment ground investigations would be useful to confirm the achievement of the design requirement and provide data to improve the practice in future.

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Geotechnical Instrumentation and Monitoring in Hong Kong - Past, Present and Future

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Abstract: Geotechnical instrumentation has been installed and monitored in many Hong Kong civil engineering projects over the past thirty years. This paper provides an overview of the history of the use of geotechnical instrumentation over this period, and placing the technical developments in instrumentation and monitoring techniques into the context of the major Hong Kong civil engineering projects during this period. The paper looks at the current state of the art in geotechnical instrumentation and monitoring, and concludes with a review of new techniques that it is considered may become more common in future years.

1 INTRODUCTION – THE EARLY YEARS

The widespread adoption of geotechnical instrumentation and monitoring as an engineering tool in Hong Kong over the years has developed in response to the availability of increasingly reliable and affordable instrumentation and data acquisition equipment, and the growing understanding of local engineers that instrumentation and monitoring can offer real benefits in terms of safety, cost savings and increased engineering knowledge.

The three major areas where instrumentation and monitoring have been used, and hence the drivers of advances in the state of the art in Hong Kong have been slope monitoring for landslip prevention, monitoring for excavation and lateral support works during the construction of deep basements, and monitoring associated with the construction of railways (in particular tunnels).

Geotechnical instrumentation first started to be used significantly in Hong Kong during the 1970s and 1980s, a time when the Government was committing resources to improving slope safety, the Mass Transit Railway was under construction, and the commercial districts of Hong Kong Island and Kowloon were beginning to be filled with high-rise buildings.

During these early years, the instrumentation used was predominantly mechanical rather than electrical, such as pneumatic piezometers. Instruments were manually read, requiring significant levels of manpower. Commercially available instruments in those days did not lend themselves to easy automation, and furthermore, automated data acquisition was relatively expensive.

Software, storage technologies, and computing capability were not at a stage where it was practically and economically viable to store and process large quantities of data within a reasonable time period. In the late 1980s, a typical computer used for data acquisition purposes (such as a Digital Equipment Corporation MicroPDP-11/23) would typically have a memory capacity of less than one megabyte, and if a hard disk was installed, it might have a storage capacity of a few tens of megabytes at the most (Currie & Solomon, 1992). Such computers cost over a hundred thousand Hong Kong dollars at the time. Although introduced in 1981, the IBM personal computer and its successors were not yet at a stage of development in terms of processor speed and memory capacity where use for data acquisition was viable. Similarly, stand-alone datalogging systems were not commercially available, although expensive and bulky open-reel multi-channel instrumentation tape recorders such as the Racal Store series (available in 4, 7 and 14 channel versions) were sometimes used.

In the 1970s, several major and fatal slope failures focused the public and political attention on the need for an improvement in the management of geotechnical works by the Government. As a result, the Geotechnical Control Office (GCO) was set up by the then Public Works Department (now the Geotechnical Engineering Office of the Civil Engineering and Development Department) in mid-1977, as a control organization to provide continuity throughout the whole process of investigation, design, construction, monitoring and maintenance of slopes in Hong Kong (Knill *et al*., 1976; Yeung, 2005).

Much of the initial work carried out by the GCO was concentrated on the inspection, registration, categorization, prioritization, upgrading and maintenance of slopes (Chan 2000, 2003; Lam *et al*., 2001). This work involved the first significant usage of geotechnical instrumentation in Hong Kong, primarily simple equipment such as tensiometers, standpipe and Casagrande piezometers, rain gauges etc (Vail & Attewill 1976; Wong *et al.,* 2006). GCO started to install tensiometers in selected slopes and the data collected were transmitted to their office by fixed phone lines. However, the tensiometers took tremendous manpower to maintain them in a functional state.

The setting up of the GCO encouraged overseas geotechnical instrumentation manufacturers to set up agencies or representative offices in Hong Kong. With the increasing market in geotechnical engineering in Hong Kong, major international geotechnical consultants and contractors also began to set up offices in Hong Kong in the late 1970s and early 1980s, which in turn lead to further demand for geotechnical instrumentation.

Following construction of the MTR system in the 1970s and 1980s, instrumentation became important to help to ensure that MTR structures were not affected by construction activity taking place within the vicinity of MTR structures. Buildings Department PNAP77 defined a “railway protection zone” covering 30m either side of affected structures (Lai, 1989). Any construction planned to take place within the railway protection zone has to be submitted to MTRC for approval, and MTRC railway protection engineers will specify the necessary precautionary and monitor-

ing measures to be taken to ensure that several key parameters affecting the MTR structures (such as differential movement, total movement, vibration levels etc) do not exceed predefined limits.

The commercial districts of Hong Kong and Kowloon saw many high-rise buildings with deep basements constructed during this period. Perhaps one of the most well known was the Norman Foster-designed HSBC headquarters building in Central (Fig. 1). With a construction cost of around HK$5.2 billion, it was the most expensive building in the world on completion in 1985. The building is some 180m tall, with 47 floors, and a 20m deep, four-level basement.

Extensive design studies were carried out to predict the effects of construction on the adjacent roads and buildings (Fig. 2), and an instrumentation programme was adopted to verify these predictions, and act as a warning system in case predictions were exceeded (Fitzpatrick & Wilford, 1985). A total of 41 piezometers and standpipes, 23 inclinometers, and almost 90 ground settlement markers were installed, together with tilt monitoring points on the facades of adjacent buildings. The instruments were all read manually, at frequencies ranging from weekly to every six hours, which became a full-time job for the two monitoring contractors employed to carry out this work. In the event, settlements were in many cases significantly lower than predicted, and the adjacent buildings showed no signs of distress.

Fig. 1. HSBC Headquarters, 1 Queens Road Central.

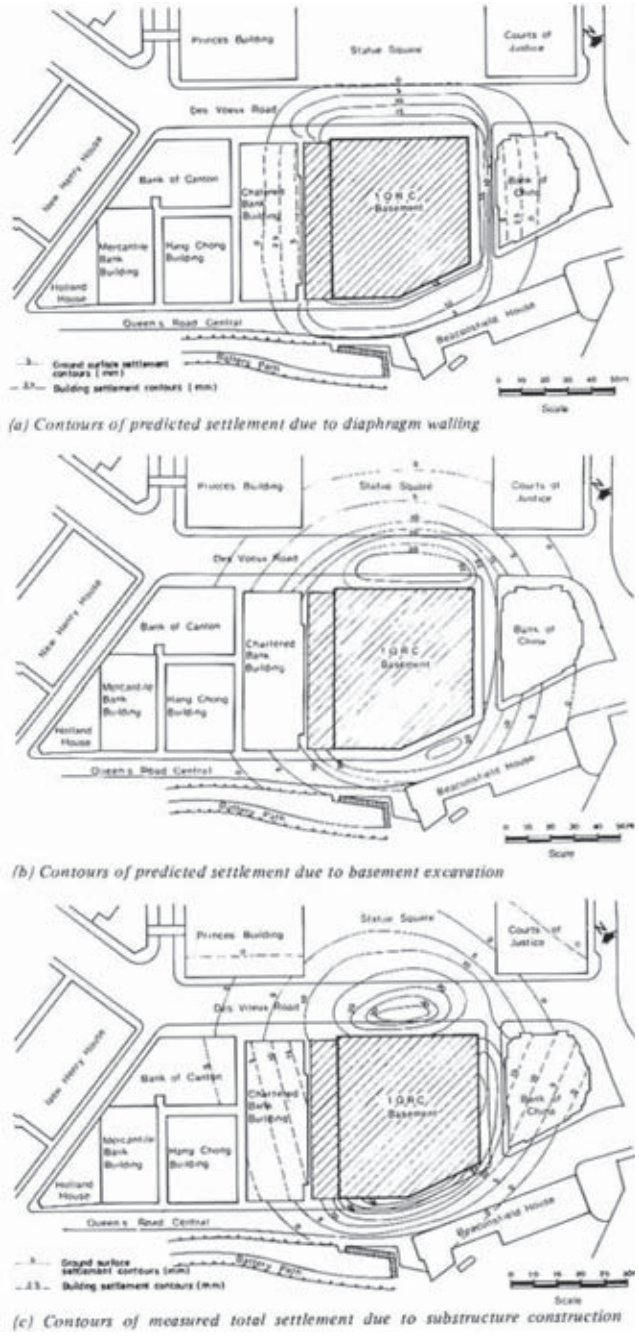

Fig. 2. HSBC Headquarters – comparison of predicted and measured settlements (from Fitzpatrick & Wilford, 1985).

2 MAJOR INFRASTRUCTURE DEVELOPMENTS IN THE NINETIES

By the late 1980s the contribution of instrumentation in major construction works had been recognised in the geotechnical community - the HKIE Geotechnical Division's ninth Annual Seminar in 1989 had the theme "Recent Applications of Instrumentation", and the fifteenth Annual Seminar in 1995 had the theme "Instrumentation in Geotechnical Engineering". It is interesting to observe that even in 1989, the philosophies and concerns were very similar to those today (Blacker, 1989) – selection of suitable and reliable instruments, designing appropriate

monitoring schemes, and application of automation to reduce the effort required to acquire and process readings.

The early 1990s saw the development of the new Hong Kong International Airport at Chek Lap Kok, and the associated road and rail developments required to connect the new airport to the existing transportation network (the "Airport Core Programme projects").

The development of the airport platform and the associated infrastructure links was a major geotechnical engineering undertaking with significant instrumentation and monitoring content. Because of the tight construction timescales, instrumentation schemes were developed to determine the stability of the reclaimed land, evaluate the effectiveness of the ground improvement work, and monitor construction activities, in order to ensure as soon as possible that the site formation was safe and ready for the next phase of construction.

In the airport and related developments, instrumentation was typically used (Wong & Wood, 1998) to:

- characterise initial site conditions,
- verify design assumptions and predicted performance,
- monitor the effects of construction, and to control how quickly construction can safely proceed,
- to enforce quality of workmanship
- to provide early warning of impending failure
- to provide evidence to defend against claims of construction-related damage.
- to monitor in-service performance.

Many different types of instrumentation were installed in the Airport Core Programme projects, however they can be broadly grouped into the following categories:

- pore water pressure (ground water)
- deformation and settlement
- load, strain and vibration induced by excavation and blasting.

Wong & Wood (1998) estimate that almost 2,500 geotechnical instruments were installed across the ten Airport Core Programme projects, plus more than 5,000 settlement points of varying types, monitored by conventional optical survey techniques.

Although much of the instrumentation installed during this period was still of the traditional manual reading type, electrical instruments capable of being read automatically began to become more common, prompted by the wider commercial availability of standalone dataloggers suitable for use with geotechnical instrumentation, such as the DataTaker DT615.

For example, vibrating wire piezometers were used in place of standpipe or pneumatic piezometers; in-place inclinometers were used in place of torpedo inclinometers; electrolevel sensors were used in place of tilt plates, and vibrating wire crack meters replaced mechanical strain sensors. In many cases, the automatically read sensors also offered better accuracy and repeatability than the traditional sensors that they replaced.

The extensive use of geotechnical instrumentation in the Airport Core Programme projects significantly advanced the state of the art in Hong Kong, and allowed local designers, contractors and instrumentation specialists to build a significant degree of expertise in the acquisition and interpretation of large quantities of data from automatically logged instrumentation installed in inaccessible locations.

The airport platform reclamation at Chek Lap Kok was heavily instrumented to monitor settlement during the construction phase (Barwell *et al.,* 1998). Almost 650 instruments (piezometers, extensometers and inclinometers) were installed across the reclamation, along with almost 3,700 settlement markers. The instrumentation was typically installed in clusters comprising an extensometer surrounded by a group of piezometers. Clusters were spaced on a grid of 400-500m (200-250m in places where settlement was predicted to be greater, or the underlying geology was more variable).

Monitoring of the geotechnical instrumentation installed at the airport platform for the purposes of monitoring of in-service performance (mainly settlement) has continued to the present (Luk, 2002).

With the significant volume of monitoring data acquired during the construction of the airport platform came the realisation that in large-scale monitoring projects data management was a key factor that needed to be addressed if the instrumentation and monitoring was to add significant value to the project as a whole. By data management, we mean the ability to safely store and retrieve large amounts of information, and to allow timely circulation of such information to concerned stakeholders. The data management approach used for the airport development was semi-automated, involving large numbers of interlinked spreadsheet workbooks to process and present the data in standardised formats (Tosen *et al.,* 1996; Barwell *et al.,* 1998). This approach was very successful and the experience gained resulted in a set of guidelines and recommendations in several areas.

Towards the end of the 1990s, the Hong Kong construction industry was dominated by rail projects (KCRC West Rail, Ma On Shan Rail, and East Rail Extensions projects, the MTRC Tseung Kwan O extension), and medium sized reclamation projects (Disneyland, Container Terminal 9, Lamma Power Station Extension, North Tsing Yi Reclamation).

During the Airport Core Programme works, major clients and their designers developed a deeper appreciation of the benefits of high-quality instrumentation. As the airport-related projects were completed, the experience and appreciation of instrumentation that had been gained on these projects was carried forward to the new projects.

Particularly on the KCRC West Rail project, many of the lessons in geotechnical instrumentation and data management learned from the airport project were taken into account from the onset of the project. These included:

- Awarding an advance instrumentation contract in order to establish baseline readings before construction started
- Insisting that the construction contractors use specialist instrumentation contractors.
- Development of instrumentation and monitoring plans, with action plans to be initiated when readings exceeded certain predefined trigger levels (the so-called "alert", "action" and "alarm" levels).
- Commissioning a major centralised database system (Solomon *et al.,* 2001) as part of the advance instrumentation contract to allow KCRC's engineering team and the various construction contractors' online access to the latest readings taken from the instrumentation stored on site, and to give more flexibility and expandability than the spreadsheet approach used.
- Networking the various construction sites and project design offices together to ensure that instrument readings are available to site and office staff immediately they are entered into the database.

The late 1990s and early 2000s saw other advances in geotechnical instrumentation and monitoring:

Major programmes of full scale pile load tests were carried out by KCRC and their contractors on both the West Rail and Ma On Shan Rail projects. These tests were intended to verify the performance of non-conventional (for Hong Kong) pile designs

and saw the introduction of novel techniques such as fully automated load control, Osterberg Cell testing, and the monitoring of pile shaft strain sensors in the period between concrete placement and load testing.

Automatic Deformation Monitoring Systems (ADMS) using either robotic total stations and prisms or electrolevel beams were used for the first time for tunnel convergence monitoring when excavation was taking place adjacent to the tunnel (Rasmussen *et al.,* 1995; Cooper *et al.,* 2001).

Developments in networking, communications and data management technologies were combined to produce fully automated systems that could automatically read large numbers of sensors, check the data, alert users to any unexpected reading by SMS text message or email, and display the readings as engineering data without human intervention. An example is the geotechnical instrumentation system used for the MTRC project to link Tsim Sha Tsui Station to the new KCRC East Tsim Sha Tsui Station with underground walkways (Chan *et al.,* 2003). In this project, the existing MTRC tunnel linings in the vicinity of the excavation were heavily instrumented with a large ADMS system (four total stations and 280 prisms), 250 strain gauges plus other minor instrumentation.

Fig. 3. ADMS system (robotic total station and prisms) in MTR tunnel (from Chan *et al*., 2003).

The instrumentation was read automatically by computer every two hours, resulting in over 6,000 separate readings per day being generated, automatically checked for exceedances and stored in a database. Without this high level of automation the overall project as a whole probably would not have been practical, and would have required extensive changes in the nature and sequence of construction, as it would not have been practical to make manual instrument readings during working hours, while construction was taking place, due to the fact the instrumentation was installed inside live railway tunnels. Similarly, the volume of data generated by the frequent reading schedule was too much for sensible and timely manual analysis and checking.

3 THE PRESENT STATE-OF-THE-ART

With a few exceptions, the geotechnical instruments in common use in Hong Kong have not changed significantly in the past 15 to 20 years, although there has been a shift from mechanical instrumentation to electrical instrumentation.

Although much of the geotechnical instrumentation in use in Hong Kong is still manually read, there has been a significant drive towards automation, particularly on large-scale projects. Some types of sensor are difficult to automate, for example magnetic extensometers, and torpedo inclinometers, although it should be noted that it is often possible to measure the same parameter with a different type of instrument that may be read automatically.

Developments in computers and data communications have pushed prices down and capabilities up such that many things are now practical that would have been prohibitively expensive or technically complicated to achieve before.

Multichannel dataloggers with large memory capacities are common. Vibrating wire piezometers are starting to be replaced by Automatic Groundwater Monitoring Devices (AGMDs). These are "smart" piezometers containing built in dataloggers and memory. An example is the LevelTROLL series of piezometers manufactured by In-Situ Inc, which are less than 20mm in diameter yet contain a battery that will power the device for up to 5 years and an internal memory that will hold up to 6 million individual readings.

The ready availability almost all over Hong Kong of mobile phone services has made it easy to wirelessly transmit sensor readings off-site in real time, making it possible to monitor site conditions even in situations where it is not practical or safe to have personnel at the instrument locations, for example on a remote slope during a heavy rainstorm, or in a live railway tunnel. This type of approach is also very efficient, as one technician sitting in a central location may take readings from instruments at several sites without the need to leave the office or travel between sites.

A good example of an instrumentation system that can be read remotely is given in Chan *et al.* (2004). This was for a research project to investigate the properties of construction waste (public fill) when used for the construction of foundations for seawalls and breakwaters.

Two heavily instrumented steel caissons of 3m diameter were installed in the middle of the test zone (Figs. 4 & 5), and were monitored during the placement of the fill and thereafter.

Fig. 4. Instrumented caisson with remote reading instrumentation (from Chan *et al.,* 2004).

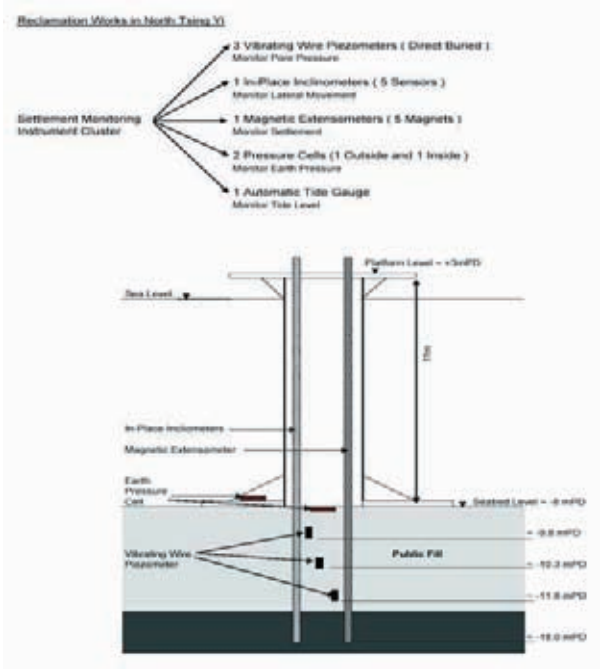

Fig. 5. Schematic of caisson instrumentation layout (from Chan *et al.,* 2004).

With the caissons installed in a marine environment, it was obviously extremely inconvenient to take regular readings manually, not to mention that having to use a boat to travel to the caissons to do so would have disrupted the progress of the vessels placing the fill.

However, by reading the instruments with a datalogger and wirelessly sending them back to an off-site location via a GSM modem, the measurement exercise became practical, and useful results were obtained.

As well as wireless transmission of sensor readings off site, it is also now practical to connect sensors wirelessly to a data logger. This can both enhance system reliability, as there are no vulnerable cables to be damaged (or requiring expensive protection), and reduce costs for installation and maintenance – one datalogger can serve sensors spread over a wide area, and once the sensor and wireless transmitter is installed, the sensor can begin operating, and there is no need to spend time installing long cables, possibly in conditions where access is difficult or hazardous.

The fact that almost everybody now has access to the internet has meant that the internet is a logical channel for the distribution of sensor readings to stakeholders, as information can be made available immediately it is posted, with no need to wait for collation and distribution of paper reports, and no geographical restrictions - one can check the web for the latest readings from your site from another continent.

In fact, it is now practical (and very desirable) to have a fully automated process, with no human involvement in the measurement chain. Sensors are read at regular intervals by a datalogger which transmits the readings wirelessly off-site to a receiving database, which after automatically checking the readings against pre-set thresholds and issuing email or SMS text message alerts if any thresholds have been exceeded, will post the readings on a website where they can be viewed or downloaded by registered users located anywhere in the world.

An example of a current project using all of the state-of-the art techniques mentioned in the preceding paragraphs is the slope monitoring being carried at Po Shan Road, in Mid-Levels, as part of risk mitigation measures carried out under the the LPM programme. The LPM programme, which has driven many innovations in geotechnical instrumentation, provides for the risk-prioritised screening and upgrading of the man-made slopes constructed before the establishment of the GEO.

The slope at Po Shan Road has been the subject of much work over the years, and it was one of the slopes affected by major landslips in the early 1970s (Vail *et al*., 1976) that lead to the setting up of the GEO.

In the current project, two horizontal drainage tunnels are being driven into the slope, with 200 sub-vertical drains being drilled up through the tunnel roof into the slope (Fig. 6). These drains are intended to lower the water table to reduce the risk of major landslides.

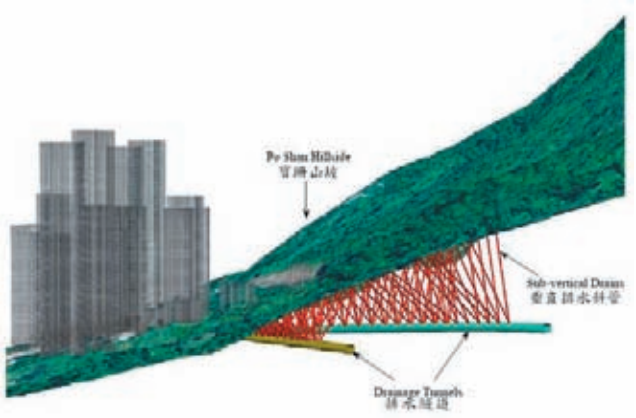

Fig. 6. Schematic of risk mitigation measures at Po Shan Road (from http://www.hkie.org.hk/engarchive/).

In order to monitor the affects of the work on the groundwater during and after construction, some 50 piezometers are installed over the slope (Fig. 7).

Fig. 7. Po Shan Road wireless piezometer station (enclosure and antenna).

These sensors are connected to each other over a self-organising wireless network, and the readings hop from station to station until they reach a central data logging system at the foot of the slope which is connected to an off-site database via a wireless GPRS link.

Each piezometer is read every five minutes, and the readings are sent back to the off-site database every ten minutes. Readings entered into the database are checked automatically, and then made available on a website (Fig. 8), from where they may be checked by registered users (Fig. 9).

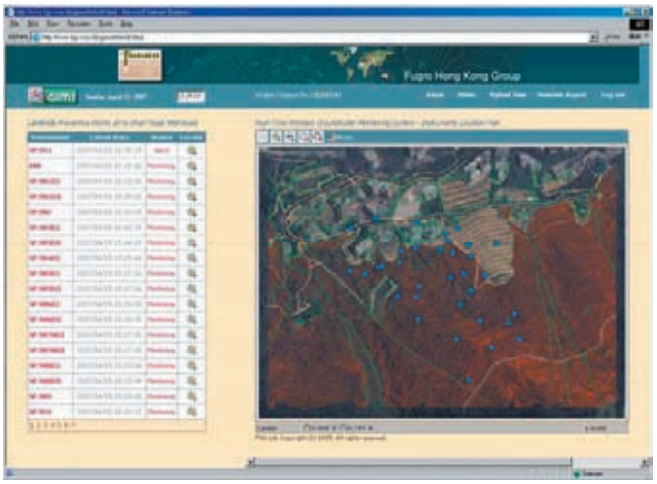

Fig. 8. Po Shan Road website – site plan and instrument status.

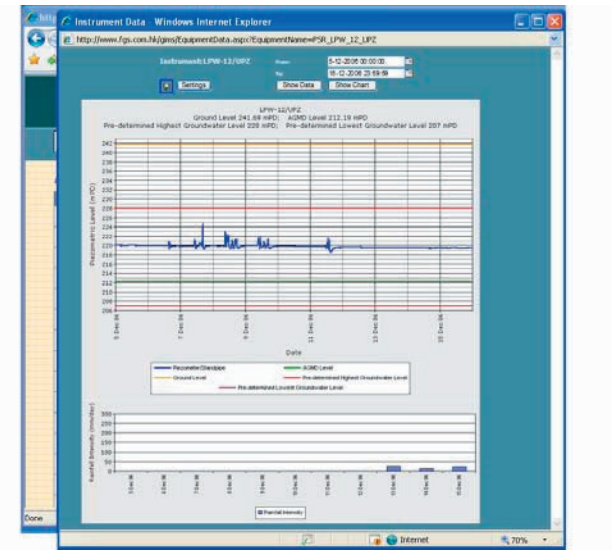

Fig. 9. Po Shan Road website – sensor readings.

As well as using instrumentation for construction monitoring on site, it is possible to use geotechnical instrumentation during the drilling process. An example is the Drilling Progress Monitor (DPM) developed by Hong Kong University (Yue *et al*., 2004). This is a system to monitor the installation of soil nails. Real-time monitoring of the soil nail drilling process can provide useful information on the properties materials encountered in the drilling process, and thus verify the design assumptions in-situ.

The complete drilling process can be monitored automatically by the DPM through real-time sequential measurements of the following parameters:

- position of the swivel drill chuck;
- rotation of the shank adaptor associated with the swivel drill chuck;
- pressures of the two compressed airflows for the thrust motor;
- pressures of the two compressed airflows for the rotation motor; and
- pressure of the compressed airflow for the hammer percussion through the swivel drill chuck

The collected data is transmitted to the office by wireless transmission for further analyses and integration with other available information (Fig. 9).

Although research on correlating the properties of geologic materials being drilled and the drilling parameters measured is still ongoing, instrumented drilling is becoming an indispensable tool in the classification of materials along the drilled hole (Gui *et al*., 2002; Yue *et al*., 2004).

Non-destructive evaluation of the integrity and dimensions of the installed soil nail is another major research area. Many different techniques, including sonic echo method, mise à la masse method, magnetometry, electro-magnetic induction method, time domain reflectometry, etc., have been evaluated (Cheung, 2003). The techniques of magnetometry, time domain reflectometry, and surface wave – reflectometry appear to be most promising (Lo *et al*., 2005; Tang & Yeung, 2006).

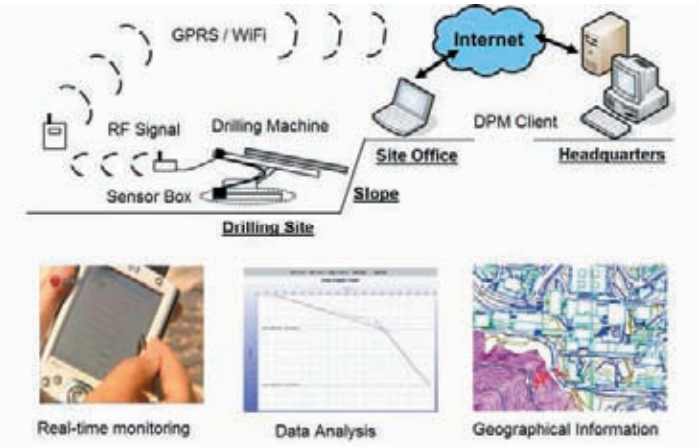

Fig. 10. Hong Kong University drilling progress monitor.

4 WHAT DOES THE FUTURE HOLD?

It is always difficult to predict the future with any degree of certainty, however it is probably safe to say that geotechnical instrumentation and monitoring will continue to keep pace with technology, as it has in the past thirty years.

It is likely that remote sensing technology to monitor elevations such as airborne laser scanning and satellite synthetic aperture radar (InSAR) will be refined to a degree that millimetre level readings will become practical and useful for geotechnical purposes. Research work has already been done at Hong Kong Polytechnic University on the application of InSAR to settlement monitoring of reclaimed land (Liu *et al.,* 2001).

Similarly, global positioning systems will continue to improve, and with the refurbishment of the Russian GLONASS sys-

tem and the launch of the European GALILEO system augmenting the original United States system, enough satellites will be visible at any one time to allow GPS to finally become a practical tool for positioning in Hong Kong's "urban canyons". Work is also being done on "local positioning systems" using similar concepts to GPS but using short range terrestrial references instead of satellites.

It is likely that the new CIRIA guidelines and the resulting increase in interest in the use of the Observational Method for excavation and lateral support projects in Hong Kong will lead to an increase in the amount of geotechnical monitoring that is carried out for construction control purposes (Chan & Solomon, 2005; Solomon & Chan, 2006).

Improvements in low-power, low-cost wireless networking technologies such as UWB, Bluetooth and ZigBee will lead to more wireless sensors and dataloggers and less expensive and vulnerable cables being installed on construction sites.

Data presentation techniques can be expected to keep up with developments in geographical information systems, and as the use of commercial internet mapping services such as Google Earth become more common, users will similarly expect more and more of their geotechnical instrumentation information to be available via internet websites in a similar map-based format.

New and improved methods of measuring geotechnical parameters will become available, and gradually move from being interesting academic research concepts to being commercially available

Currently making this transition are MEMS sensors (micromachines fabricated from silicon using the same techniques as used to fabricate integrated circuits) – MEMS-based inclinometers are becoming available, and offer the benefits of being cheaper and more robust than previous accelerometer based sensors. Optical fibre sensors continue to be improved, and offer benefits in terms of resistance to electromagnetic interference and the fact that one fibre can measure in several locations over its length, which may be many metres long, although at the present time the cost of the data logging equipment required for optical fibre sensors normally outweighs many of the advantages.

One technology that is currently in the research stage is the application of grid-based tactile sensors to the measurement of soil stresses.

Grid-based tactile pressure sensors were originally developed at the Artificial Intelligence Laboratory of MIT for the analysis of dental occlusion (Hillis, 1981; Purbrick, 1981; Podoloff & Benjamin, 1989). A sensor consists of two thin, flexible polyester sheets that have conductive strip patterns deposited on them. Typically one sheet has a row pattern while the other has columns. A force-sensitive coating is applied over these conductors, and when the two sheets are placed together, a grid of sensing cells is formed by the intersection of the rows and columns. The combined thickness of the two sheets is less than 0.1 mm. This type of sensor is now used in the medical field to measure dental, foot, and orthopedic pressures. Industrial applications of the sensors include the measurement of contact stresses of robot hands, tyre treads, and pinch roller pressures.

The University of Hong Kong is researching the application of these sensors to the study of soil arching, and the improvement of total pressure cells currently used to measure normal stresses in the soil. However, total pressure cells have difficulties measuring stresses in soils due to the huge contrast in stiffness between the cell and the surrounding material and the assumption of uniform stress distribution within the cell. The contrast in stiffness can cause errors in the measured stress due to soil arching across the face of the cell. It is hoped that the application of the grid-based tactile pressure sensors will give new insights into the soil-arching issues that take place.

5 CONCLUSIONS

From the simple manual instruments used in the 1970s to the sophisticated fully automated systems of today, instrumentation and data acquisition has come a long way in the past thirty years, keeping pace with the developments in that have taken place in the world of electronics, computers and telecommunications, and it is certain that this will continue.

Since the 1970s, Hong Kong has come from a position where geotechnical instrumentation was a novelty and expertise had to be imported from overseas, to being a world leader in the field, with Hong Kong experience and capabilities in the application of geotechnical instrumentation being sought after overseas, particularly on reclamation and railway projects.

The key advantages of instrumentation are the same now as they were 30 years ago, and it is safe to say that the benefits that instrumentation brings in terms of increased safety, reduced construction costs and enhanced engineering knowledge will continue to be sought after by Hong Kong engineers.

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Nuturing Young Geotechnical Engineers

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Abstract: The appointment of Professor C.A. Middleton as the Chair Professor in Civil Engineering in 1912 began a new chapter in education of civil engineering with geotechnical engineering as its integral component in Hong Kong. Though the initial scale was small, it provided an opportunity for talents in this region to receive formal engineering education without travelling to other countries. The development was slow and momentum only started to pick up in the nineteen fifties when Hong Kong moved from an entrepot to a light industrial centre. The transformation of economy of Hong Kong to a finance and service centre since the late nineteen seventies initiated another major expansion in education; shifting from elitist to mass education. This paper attempts to present the historic facts of the various stages of the development and discusses the challenges/opportunities ahead of us.

1 DAWNING PERIOD: PRE-WAR

The establishment of the University of Hong Kong owed much to the then Governor Sir Fredrick Lugard who personally rallied for the support of politicians and academics back in United Kingdom. In his letter to Lord Curzon of Oxford University, he made his case strongly by arguing that "*what China wants are Engineers, Surveyors and Electricians to run her new railways and Engineering works, and medical men of Chinese race*" (Lugard, 1903-1913). His effort eventually paid off when the British Government (the Crown) approved his proposal. But without generous contributions by local donors, the dream of having a local university could not have come true. In his address during the opening ceremony of the University of Hong Kong, Sir Frederick reiterated that the University should "*be eminently practical, in order to fit its graduates for sphere of utility*" (Lugard, 1912). With such a view, the University, when established, consisted of only two faculties: namely Medicine and Engineering. Other faculties such as Arts were added at a later date. It is interesting to note that Tsinghua University, St John's University and Tongji University were also established, around almost the same time frame, by Americans and Germans in Beijing and Shanghai.

The University of Hong Kong was modeled on the British civic university system (the Birmingham model) (Eddacott, 1962) admitting students after their matriculation. It was the first university in the Far East providing English medium education. It is well-known that the Hong Kong College of Medicine, with Dr Sun Yat Sen as one of its first graduates, had been providing medical training for local Chinese before the establishment of the University and this College was absorbed into the University as its Faculty of Medicine. What is less often mentioned is that a technical institute, Li Shing Scientific and Industrial College, had served the community with technical training when there was no local university. It offered evening courses with the financial support from Mr. Li Shing. The detailed information of the courses is not available but it is believed that the Institute aimed to provide training in modern science and industry training such that its graduates were "*independent workers and not mere 'hands' to be always under the direction of foreigners*" and used their "*hands as well as their brain*" (HKPU, 2002). Nevertheless, the Institute had played a vital role in providing formal technical training in the early days.

Initially, the Faculty of Engineering only offered specialization in civil engineering. Mechanical and electrical engineering were added at a later stage. The programme was a four year one with students receiving general engineering training in the first three years and then specializing in either one of the three streams in the fourth year. It also emphasized workshop experience and all engineering students had to receive high standard practical training in the Ho Tung Workshop and Peel Engineering Laboratory. The final examination papers were sent to special assessors in the University of London for assessment and degrees were only conferred, after confirmation by the assessors, to students who had reached the standard of the University of London. Such arrangement has evolved into the present external examiner system in which external examiner(s) is (are) appointed for each degree programme. Each term of the external examiner is three years. The duties of such examiners are to ensure the standard of the programme, review the curriculum and advise on the development of the curriculum.

The University attracted not only local students but also students from the Mainland and the Far East (Malaysia, East Indies, Siam and Japan). The first batch of students graduated in 1916 and there were 12 engineering students. Up to and including 1932, 175 engineering students had graduated and 99 of them were in civil engineering. These graudates not only fulfilled the local demand for engineers but also contributed to infrastructure construction, such as railways and roads, in China. Records also show that our graduates were found in the Far East including Malaya, Borneo, French Indo-China, India, Singapore, Burma (Clark, 1988).

The Faculty also started offering a post-graduate degree in Master of Science in Engineering in the nineteen twenties. According to the records, the first post-graduate degree was conferred on Mr. S. B. Ahmed in 1929. It is interesting to learn that his thesis title was "*Theory of earth pressures as applied to retaining walls*" (Clark, 1988). It is unfortunate that the copy of his thesis was lost.

2 EVOLUTION PERIOD: AFTER THE SECOND WORLD WAR – 1972

During the Second World War, teaching in the University was interrupted as most students and staff either had left Hong Kong or were in internment camps. After the War, the Government

decided to restore the University. The following excerpts from an article by Stock (1962) summaries the state of the University after the War:

The effects of war upon the University were diverse and far-reaching. The building, if not destroyed, had been devastated by looters. The student body had been scattered throughout China and, except for a handful of medical students, none needed to return to complete the education. Some members of the pre-war staff had reached retiring age, others had died, others elected not to return; there was only a nucleus of the teaching staff available to carry on work. Endowment funds invested in China had disappeared, leaving the University in desperate financial straits. But more serious than all these material considerations was the doubt that had arisen in the minds of people in and out of Hong Kong of the wisdom of re-establishing the University at all.

The Faculty of Engineering, which had a heavy reliance on workshops and laboratories, suffered the most. Its dire state was best described by Professor S Mackey (1962):

Due to shortage of funds and staff, many subjects of fundamental importance to the civil engineers were neglected in the existing course. After leaving the University, engineering graduates were faced with the alternatives of continuing their studies or of restricting severely the range of their professional activities. To qualify for corporate membership of the Institution of Civil Engineers they had to show evidence of seven years' practical experience, of which three should be served in apprenticeship under a corporate member of the Institution; and furthermore they had to pass the qualifying examinations set by the Institution itself.

The University recalled Professor F.A. Redmond to re-establish the civil engineering programme. His job and that of his successors had never been easy as the existence of the Faculty of Engineering was under pressure from the Government and the University. The Keswick's (1952) report tabled to the Legislative Council on 17 September 1952 cast doubt on the need of engineering graduates.

Most them wanted to take full university degree courses but there are few openings for civil engineering graduates in Hong Kong and fewer still for graduates in mechanical and electrical engineering.

The Jennings & Logan (1953) Report, a report of an internal review conducted by the University in 1953, stated that "*the question whether Engineering or Architecture should or should not be taught in Hong Kong is almost exclusively to be decided by the demand for graduates*" and it went further to recommend that the University should set up a committee to consider the continuance of the Faculty of Engineering. Under such pressure, the curriculum underwent major revamp to meet the United Kingdom professional institutions' requirements. The future of the Faculty was cleared only when the degree of civil engineering was accorded recognition by the Institute of Civil Engineers in 1955, and the Institute of Structural Engineers a year later. The accreditation of civil engineering programmes by the United Kingdom Institutes went uninterrupted before we gradually turned to the Hong Kong Institution of Engineers for accreditation after the Government decided to recognize corporate members of the Hong Kong Institution of Engineers. In addition to the effort of the staff of the Department, contributions of various persons and companies, in particular Sir Lawerence Kadoorie, had also helped the restoration of the teaching facilities as Government funding was very limited at that time.

After the War, the programme was still four years. The introductory courses were restricted to the first year and followed by three years of civil engineering subjects. Later, the first year became optional with exemption given to the majority of students. It was only in 1957 that it was converted to a three year programme.

The teaching in geotechnical engineering started to emerge and develop into a structured format after Mr. P. Lumb joined the Department in 1954. Mr. Lumb was promoted to become the first professor in geotechnical engineering in 1976. The contributions of Professor Lumb were not restricted to undergraduate teaching. He established the Soil Mechanics Laboratory in the Annex between the Peel Engineering Laboratory and Ducan Sloss Building (Fig. 1). The two buildings housed the Department until 1982 when it moved to the Haking Wong Building. This laboratory became the training ground of many local leaders in the geotechnical engineering field including a Pro-Vice Chancellor and six Presidents of the Hong Kong Institution of Engineers. His report on the settlement of buildings in Mong Kok is always treated as one of the early records of geotechnical works conducted by local academics. The numerous papers published by him and his students "*have become standard and indispensable reference materials for geotechnical engineers working in Hong Kong*" (Li, 1991).

Fig. 1. Engineering buildings in the nineteen fifties.

The University resumed the research degree programme at the master's level (MSc (Engineering)) and introduced the doctorate degree (Doctor of Philosophy) after the War. The first post-war MSc (Engineering) degree was conferred in 1953 and the first PhD degree was conferred in 1966. The recipients were Huang Wei Hong and Chen Loh Kwan, respectively. Dr. Chen was a teaching staff in structural engineering of the Department of Civil Engineering. The titles of the MSc theses in geotechnical engineering are listed in Table 1. It is interesting to note that some topics of their research still remain as hot ones nowadays.

Table 1. Thesis titles of MSc in geotechnical engineering.

	THESIS TITLE	YEAR
1	*An investigation of pore-water suction in Hong Kong soils* – Tao Wing Fai	1964
2	*Infiltration of water in unsaturated soils* – Wong Hong Yau	1966
3	*Tensile strength of remoulded soils* – Tong Yuk Lun	1966
4	*Analysis of pile loading tests in Hong Kong* – Tsui Kam Kwong	1968
5	*Anisotropic strength of clay soils* – Lee Kwok Wing	1968
6	*Numerical solution of three-dimensional consolidation* – Wong Chak Yan	1968
7	*A study of the stress and strain relationships in heterogeneous soil* – Luk Wang Kwong	1968
8	*A study of clay minerals in Hong Kong soils* – Lee Chak Fan	1970
9	*Pore-water suction in Hong Kong soil by psychrometric measurement* – Wong King Keung	1970
10	*Influence of strength variability on the safety of slopes in cohesive-frictional soils* – Law Kum Tim	1971

3 EXPANSION PERIOD: 1972-NOW

This period represented rapid expansion of higher education in Hong Kong. In the early nineteen seventies, there were two major developments in education.

In 1972, the Master of Philosophy was introduced as the research degree at the master's level and the MSc (Engineering) was reserved for taught post-graduate masters. The taught master's programme targeted young practising engineers. They were required to take courses in the evening and complete a dissertation. The first cohort of MSc (Engineering) programme graduated in 1976.

In the same year, the Hong Kong Polytechnic was established in place of the Hong Kong Technical College. As pointed out by Sir S.Y. Chung, the Polytechnic was "*a wholly vocationally-oriented institution, listens, and responds, to the problems, advice, and requirements of Hong Kong commerce, industry and government*" (Bailey, 1982). The Polytechnic mainly provided education from certificate to higher diploma level and the Department of Civil and Structural Engineering is one of the departments under the Division of Construction and Land Use. Upon graduating from the higher diploma, students could opt for an additional year leading to Associateship satisfying the requirements for corporate membership of the professional institutions. Though the number of students going for associateship was not large, the University of Hong Kong had since then lost its monopoly in offering courses reaching corporate membership level in civil engineering.

Engineering education at the degree level remained almost unchanged in the next ten years. In 1984, the Hong Kong City Polytechnic was established but its programmes did not include civil engineering. Furthermore, a number of technical institutes were established under the Vocational Training Council. These institutes were providers of technician level training. It was the obvious intention of the Government that The University of Hong Kong should be responsible for the degree education in engineering whereas the two polytechnics and the technical institutes provided different levels of technician training.

In the early nineteen eighties, the Hong Kong Government conducted two reviews (Topley Report and Llewelyn Report) on higher education (Rong, 2002). Both reviews had far-reaching implications in the development of education in Hong Kong and we saw major changes in higher education.

In view of the fierce competition from our neighbours, the Government decided to establish a third university, the Hong Kong University of Science and Technology. With the generous donation from the Hong Kong Jockey Club, the building of the campus started in 1987 and the recruitment of undergraduate students commenced in 1991. The first intake was 700 and the student number increased to 7000 within five years. Degree programmes in civil engineering were also offered in the new university.

Another move by the Government was to promote the two polytechnics to universities (the two polytechnics were renamed Hong Kong Polytechnic University and Hong Kong City University) in 1994 but only Hong Kong Polytechnic University has a civil engineering programme. Since then, the civil engineering degree has been offered by three, instead of one, institutions. In addition, the Hong Kong City University started offering programme in Modern Structures a few years ago. The programme was re-named "Structural and Geotechnical Engineering" this year. In 1995, the Department of Earth

Science was established in the University of Hong Kong and it is offering courses in geology at undergraduate and master's level (Master of Applied Geoscience).

The rapid increase in student numbers created pressure on employment for the graduates and the Government finally decided to freeze the student intake number for the degree programmes to about "*18% of the 17-20 cohort*" (Sutherland, 2002). Though the 2001 Policy Address by the Chief Executive of the SAR outlined one of the goals for education in Hong Kong being "*to increase the number of post-secondary places, so that 60% of senior secondary school leavers can attain post-secondary education*" (Sutherland, 2002), the increase was mainly at the sub-degree level (associate degree and higher diploma). The number of civil engineering undergraduate students increased slightly in the late nineteen nineties but the enrollment number for the three civil engineering programmes has re-adjusted recently and it has remained at about two hundred and ninety for the last three years. The variation of student numbers in the nineteen nineties only reflected the answers of the three universities to the increase in interest in the programmes by high school students and demand for graduate engineers by our construction industry.

In the same period, the MSc (Engineering) programme had expanded. A programme in Geotechnical Engineering was added to the general civil engineering programme offered in The University of Hong Kong in 1998. Table 2 summarized the intake numbers in the last ten years. We would also like to point out that training at post-graduate level in geotechnical engineering is also provided by other universities and their student populations in the master course should be nearly the same as ours. From the table, one can easily note that the number of intakes has dropped in the last three years. The reasons for the drop could be many but one can be the new Government policy of not supporting taught master's degree programmes by public fund and the MSc (Engineering) programme, as a result, has to be converted to self-supported mode.

On the other hand, the Government has placed more emphasis on research and established the Research Grants Council in 1991 to oversee and fund research activities of the universities and higher education institutions. Table 3 tabulates the funding that the Civil Engineering, Surveying, Building and Construction group received in the recent years (RGC, ??). Though an exact breakdown of the share was not available, the geotechanical discipline should have a fair share of the funding. In addition, the members of the discipline have also been very successful in securing supports for large research endeavour from the Government. With the support of the Central Allocation Vote of the University Grants Committee, The Geotechnical Centrifuge Facility and Rock Engineering Centre were established in the Hong Kong University of Science and Technology and The University of Hong Kong, respectively. These Centres are equipped with state-of-the-art facilities serving the needs of Hong Kong and nearby regions in centrifuge model testing and rock engineering testing. The study in loose fills and vacuum preloading are good examples of large scaled joint research activities of three universities in the geotechnical field. These research projects provided good opportunities for training young graduates for their higher degree pursuits and the graduates are now serving the local as well as international community as researchers, academics and practicing engineers. Research grants from other sources, including the Hong Kong Jockey Club, Innovation and Technology Fund and Croucher Foundation, are also available. But support from the local construction industry is limited. With the increase in funding, one can see a significant increase in the number of research degree students. In addition to local students, some are from China and countries in South Asia etc.

Table 2. Enrollment in MSc (Eng) in Geotechnical Engineering.

YEAR	NUMBER OF STUDENTS ADMITTED
1998	27
1999	50
2000	35
2001	35
2002	37
2003	50
2004	38
2005	29
2006	17

Table 3. Earmarked grant for research (HK$).

2004-2005	**2005-2006**	**2006-2007**
28.999M	26.325M	35.961M

4 CHALLENGES

In the 2001 Policy Address, the then Chief Executive promised "*to make significant investments in education to prepare each one of us for advent of the knowledge-based economy*" (Rong, 2002), and his successor, Mr. Donald Tsang, is equally committed to investing in education. Education has taken up a considerable share of our annual expenditure and spending on higher education is over 35% of our education budget. In the last decade, we have seen a transformation from an elitist education to a mass education and an increase in research activities. Our education system will still continue to reform and improve. The major thrust in our education in the near future will be the introduction of the new 3+3+4 system, replacing the present 5+2+3 system, in 2012. Such a change will have impact on both university as well as high school systems. As they will have spent one year less in their schooling, students are expected to be less prepared and less mature when entering the university. The universities are given the responsibility to make up for such deficiencies with an extra year of university education so that students can receive a world-class education to prepare them for their future development and career.

Since the accession of China to the World Trade Organization, globalization in China and Hong Kong as its special administrative region is unavoidable, and it is going to happen at a rapid pace. Hong Kong has advantages in its geographical location as it is "*within five-hour flight from more than half of the world's population*" (Rong, 2002). Such an excellent location could provide huge opportunities for expanding to a developing and untapped market. But it is equally true that it attracts competition. Attracted by low cost, industry together with its related jobs have shifted to the Mainland since the late nineteen seventies. For our profession, we also see a trend of operation moving across the border or to neighbouring countries. What we

could anticipate is that such cross border activities will increase, and the universities are faced with the challenge of training engineers with an international outlook and knowledgeable in our own culture and history. Our strength is bi-literacy (Chinese and English) and tri-lingualism (Putonghua, Cantonese and English). The universities understand its importance and are putting in their utmost efforts on training students with high level of fluency in the two written languages and the three spoken languages. To provide opportunities for students to gain an international outlook, all higher institutes are working hard on exchange programmes. The exchanges cover China, United Kingdom, United States, and Australia etc, and it is going to expand into other countries and regions. Now, only a small percentage of the students have the chance for exchange within their study period but the exchange programme is anticipated to expand after the introduction of the four year system. Besides outgoing exchanges, we also see a gradual increase in foreign students, from as far as Sweden, spending a year in our universities as exchange students. The liberation of China and de-colonization of South East Asia had deprived Hong Kong of sources of non-local students. Recently, the universities have resumed recruiting non-local undergraduate students from China and other countries as the University Grants Committee (UGC) has relaxed the non-local student quota from 4% to 10%. Such developments have enhanced the international flavour of the local universities and raised the awareness of students on different cultures.

Though the Government has not been supportive, the universities believe that our graduates should be entitled to better "rounded" training in both engineering as well as law, business studies, economics etc. With minimal Government support, double and dual degree programmes have been offered. These programmes include double degree in civil engineering and law and dual degree in technology and management, and take four to five years to complete depending on the nature of the programmes. Such programmes are funded by the UGC for the first three years as the normal bachelor's degree programmes, and the students have to be self-funded for their subsequent years of study. From the admission figures (application numbers and quality of the students), it has proven that these programmes are very popular among our high school students. In addition, the flexibility embedded in the credit unit system has allowed students to overload and complete the major-minor mode of study within three years. The broadening of the curriculum will enable our next generation to become better prepared for the fast-changing society. However, the greatest challenge in devising new multi-disciplinary programmes is how to maintain the high level of technical competency without over-stretching our students.

Another key change in education is the emphases on the importance of 'life-long' learning. The 1996 Committee's Report (UGC, 1996) highlighted the situation:

> *The increase rapidity of change in both employment patterns and the nature of society has made this "one-shot" philosophy of higher education no longer tenable. Many people will need, either for their own purposes or those of their employer, to participate in higher education on a recurring basis throughout their lives."*

It continued to explain that

> *The very large number of small enterprises in Hong Kong are mostly unable to provide in-house training for their employees except where the worker learns by performing the task or by watching a colleague. If they engage in staff development beyond specific on-the-job training, it will usually be through HEIs, government training authorities, or commercial or professional institutes.*

In this respect, the universities and the profession have been working hard to provide training for engineers after their graduation from their first degree in the form of seminars, workshops, continuing professional development courses etc. It is encouraging to see that more cross-border activities have been co-organized in recent years by our HKIE and sister institutions in this region for their members. However, it is disappointing that the Government has ceased to support coursework master's as it believes that "*this should be largely self-funding*" (Sutherland, 2002). "This" refers to the group of programmes which primarily enhance the career prospects of an individual and a taught master's is considered to be one of them. But one has to commend the Government for the increase in research funding as it provides opportunities for our young generation to pursue higher degree study. Nevertheless, our spending on research is still relatively low when compared to our competitors (UGC, 1996), and the participation and support of industry, not only the construction, is limited (World Economic Forum, 2005-2006). Unless the Government and the industry recognize the importance of research in enhancing our competitiveness by investing in our research, we will be losing out in capturing the high-end market which requires advanced knowledge and specialized technical know-how.

A hundred years ago, the wisdom and persistence of Sir Frederick brought us the first local university and together with it the first civil engineering department. Since then, Hong Kong education has evolved and developed, especially after the War. The contribution of education to our prosperity is well reckoned and it cannot be overstated in today's technology intensive era. While expanding in *quantity*, it is equally important to maintain *quality*. Only quality curricula and students can guarantee high quality graduates for our profession. With high school leavers more incline to study business and finance as indicated in the recent admission statistics, we must strive to attract bright youngsters to join our engineering profession by promoting the social status of our profession and imparting positive images of our profession to the community. Failing this, our profession will be in a down-spin and dwindle into a secondary one.

5 THE EPILOGUE

Like Sir Frederick, only a hundred years later, we are again at a crossroads. We are not asked to establish a university but we have to design a new curriculum for engineering education to meet the expectations of today's society. Blunders in the reform will lead to repercussions which the community cannot afford to experience, and they can only be avoided with all stakeholders involved in the reform process. With the support of members of the profession who have the best understanding of the needs of the industry and the profession, we are confident that we can design a curriculum that best serves the society.

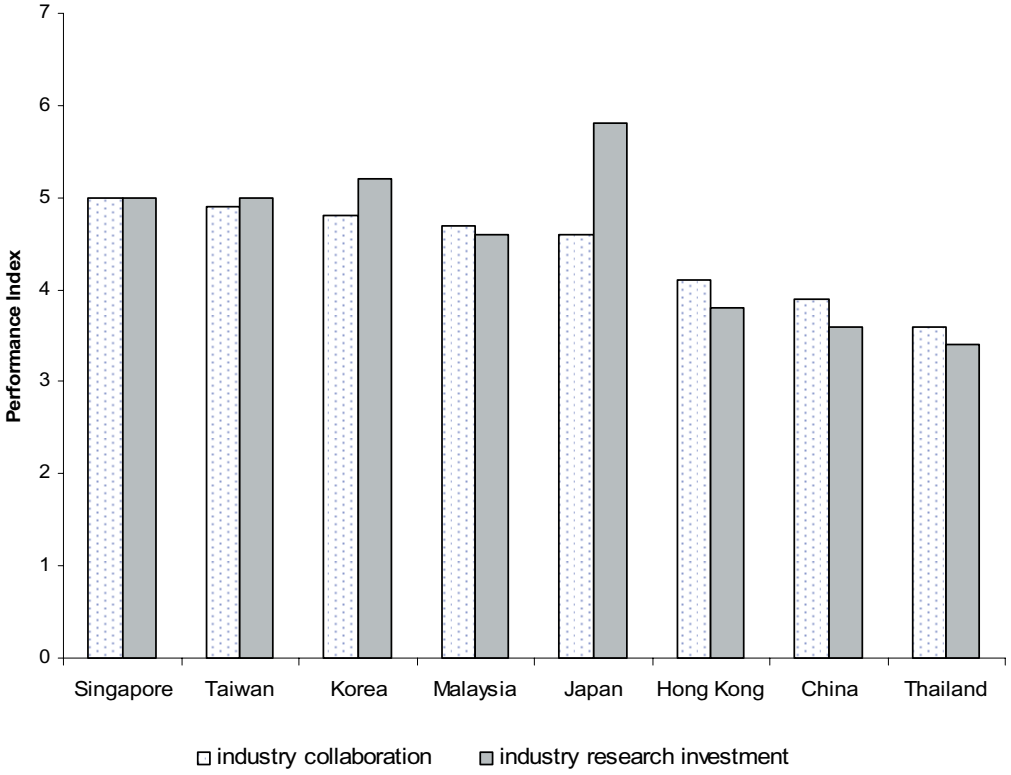

Fig. 2. Industry research indices.

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Review of Last 30 Years of Geotechnical Engineering in Hong Kong 1976 to 2006

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ABSTRACT: During the last 30 years, Hong Kong has been a focus of geotechnical activities with probably more geotechnical engineers per square metre of territory and more technical publications per engineer than in any other part of the world. There have been extensive developments in the technology, in standards of practice, in dissemination of information, and in control. The papers for this Annual Seminar are invited to address the developments in geotechnical engineering that have taken place in Hong Kong over the preceding 30 years. This paper is aimed at setting the context of the seminar by providing a retrospective summary of the state of art of geotechnical engineering in Hong Kong 30 years ago, an overview of the developments that have taken place as described in more detail in each of the locally invited papers, and the prospects for the future.

1 INTRODUCTION

During the last 30 years, Hong Kong has been a focus of geotechnical activities with probably more geotechnical engineers per square metre of territory and more publications per engineer than in any other part of the world.

Certainly for those of us who have been fortunate enough to work here life has been a veritable Pandora's Box of interest and activity.

Hong Kong has become a centre of geotechnical excellence. Spearheaded by Government's initiative through the Geotechnical Engineering Office (GEO), geotechnical engineers in Hong Kong are world leaders in the fields of Slope Safety, Geotechnical Control, Landslip Warning System, Public Education and Warnings.

Progress in these and in other fields of geotechnical engineering is highlighted in the other invited papers.

Prior to 1976 life in Hong Kong was quite different. The economy had been in recession and budgets for civil engineering were limited. Civil Engineers did their own site investigation and geotechnical engineering was virtually unknown. Slope failures during severe rainstorms were common place and fires in squatter areas were of greater concern because they resulted in more fatalities than landslips.

Subsequent to 1976, an overall substantial improvement in the economy, a drive to re-house the poor, and a second slope failure with multiple fatalities at Sau Mau Ping changed it all.

2 GENERAL PERSPECTIVE IN 1976

2.1 *Economy and expenditure on infrastructure*

1976 marked the end of a period of negative inflation. A fixed price contract in 1975 was a promise of big rewards. Leading up to 1976 expenditure on infrastructure was limited. There was a number of interesting civil engineering projects with significant ground engineering, as summarised below. However there was virtually no interest in slopes. I recall that I was advised at the time to "keep out of slopes, there is more than enough interesting geotechnical work to do in civil engineering".

2.2 *Water supply*

The 1960's saw a massive increase in the population due to immigrants from China. There was an extreme shortage of water. Rationing was effected by standpipes in roads operating for nominally four hours supplying silty rusty water for four days each week as described by Tam (2001). A supply of water from China was negotiated and local reservoirs were converted from collection to storage. They were supplemented by new reservoirs at Plover Cove, Tai Lam and at High Island, as described by Vail *et al.* (1976). The latter gave its name to the scheme. This scheme not only involved new reservoirs but joined all the reservoirs to new conduits from China and a central treatment works in Sha Tin. WSD access roads were cut across a lot of otherwise undeveloped hillsides. Major feats of civil engineering of the scheme were the dams and water supply tunnels. The two dams at High Island were under construction in 1976.

2.3 *BOT contracts*

Characteristically, the people of Hong Kong are ever alert to opportunities. In the 1970's the use of Build Operate and Transfer (BOT) contracts was novel globally. In Hong Kong, BOT was adopted for the First Cross Harbour Tunnel involving private enterprise in providing public roads at a profit. The construction adopted immersed tube technology. In 1976 it had been recently completed and was earning revenue.

2.4 *Tunnel engineering*

Colonisation of Hong Kong in the 1800's included construction of the Kowloon to Canton Railway, the last link of a system which extended through China and thence across Russia to Europe and the English Channel. The KCRC railway in the New Territories included one long tunnel under Beacon Hill and several short tunnels.

Tunnels for the MTRC underground railway had not yet been built.

The water supply system included some 40 km of water tunnels that were nearing completion as described by Tam (2001).

The tunnels were mostly "straight lines from portal to portal" and were constructed by drill and blast.

Road tunnels were few. The first Lion Rock Road Tunnel, was motivated by the need to provide a water conduit from Sha Tin to Kowloon, and was in operation. The road tunnel beneath Kai Tak, built by cut and cover, was built but not open in 1976. A trial tunnel for the Aberdeen Tunnel was under way adopting rock mechanics principles for design including measurements of insitu stress as described by Twist & Tonge (1979).

2.5 *Underground railway*

Just before 1976 negotiations with an international consortium to build all of the Mass Transit Railway system for a lump sum dissolved when faced with a rise in the price of fuel worldwide. The scheme was reduced to the Modified Initial System (MIS) which was tendered locally in 1975. The tenderers included some international contractors and some local contractors with no prior experience of design and construct for underground railway work in Hong Kong. Tenders were returned within budget and 1976 saw the breaking of ground for the first underground station at Choi Hung. The subsequent completion of the MIS on time, and the substantial early operating revenue enabled early commencement of the subsequent lines.

The MIS experience involved tunnelling in mixed ground beneath the water table using air pressure with open shields. Silicate grouts were brought into Hong Kong and the tube-a-manrchette process became widely adopted to control ground water.

Building the MIS was not without problems. Subsidence occurred due to lowering of ground water and building damage was extensive in area but controlled in severity as described by Morton *et al.* (1980). This early experience led to controls on dewatering and subsidence that are still effective today.

2.6 *Reclamation*

Hong Kong has always been short of flat land. Since the mid 1800's new land had been reclaimed by filling the sea. One of the first engineered reclamations was the runway for Kai Tak Airport that had been built in the 1960's as described by Henry *et al.* (1961). Subsequently at Kwai Chung Container Terminal, Berths 1 to 3 were also built on reclamation.

The common method of reclamation was by end tipping. Some reclamations, such as Wanchai North, provided useful dumps for dry fill although abuse of these facilities included disposal of used truck tyres and unwanted cut-off lengths from concrete piles.

Dry fill was obtained from adjacent hillsides to form platforms for development and reclaiming by end tipping was adopted for Sha Tin New Town. This was seen to be a win-win situation since the old borrow areas were formed for residential development. For the racecourse the consequence of the end tipping onto soft mud was substantial mud waves. There were subsequent major problems for the adjacent sewage treatment works too. Economising on the use of materials included the use of dredged mud from the trenches for the sea walls as fill. The consequence of disposal of dredged mud on site and of mud waves was ongoing settlement of as much as 8 m over 16 years necessitating regular reconstruction of sections of the racecourse, as advised by Halliday (1996).

The planning of extensive reclamations, especially for New Towns, required a lot of ground investigation over water. Ground investigation over water for new reclamation was greatly facilitated by the adoption of marine seismic survey. The archiving of records of marine seismic survey by Messrs EGS ltd. commencing at about 1976 can be considered to be one of the first GIS databases in Hong Kong.

2.7 *Basements*

In 1976 basements were seldom deep. The first deep basement, which was 6 floors deep, for the New World Centre, Tsim Sha Tsui, had been excavated for construction in 1976. Not only was it the first deep basement, it also had adopted novel techniques for Hong Kong of diaphragm walling with tie back ground anchors for temporary support as described by Craft (1983). Very quickly ground anchors became popular including use for stabilisation of slopes. Their popularity was short lived. Subsequent failures of ground anchors in service resulted in a ban on their use imposed by the Building Authority (1978).

2.8 *Slopes*

From early times it had been accepted that the combination of steep slopes and severe tropical rainstorms in wet seasons results in slope failures. These were not considered to be of serious concern until there was a rainstorm disaster with fatal landslips in 1972. Government appointed a Commission of Inquiry who produced an Interim Report which mainly concerned the failure of a filled slope at Sau Mau Ping, published by Hong Kong Government (1972a), and a Final Report which included other failures but mostly concerned a large failure of a deep excavation on sloping ground and a natural hillslope above at Po Shan Road, published by Hong Kong Government (1972b).

Until 1976 there was very little engineering work done on slopes. Planning of slopes for Government projects generally adopted rules of thumb, such 10 on 6 for roadside cut slopes, and 1 on 1.5 for fill slopes. For private building works it was generally accepted as good engineering practice to build cutting slopes at 50 deg or filed slopes at 35 degrees without design as reported by Hong Kong Government (1972b).

At the time, generally, fill slopes were formed by end tipping from trucks and allowing the soil to fall to what was described to me at the time as "a safe angle of repose". Compaction of fill was limited to civil engineering works such as the formation for road pavements. Few, if any, privately owned filled slopes were compacted. The concept of loose fill being subject to liquefaction was not widely appreciated in Hong Kong despite the failure of tipped fill slopes at Aberfan in Wales not many years before in 1966 as reported by H.M.S.O. (1967). In the Interim Report of the Commission of Inquiry in 1972, the failure at Sau Mau Ping was attributed to infiltration of rain water and a contribution from a burst water main. There was no comment on the need for compaction of fill.

It was a practice that some cuttings in strong saprolite, or even in colluvium, were left uncovered and vegetated, as illustrated by Huntley & Randall (1981). Some slopes had proved to be erodible and an economical surfacing called chunam was adopted. Chunam was prepared on site by mixing local soil with lime and cement and in the 19780's it was plastered on the majority of new soil slopes, including cuttings and fill slopes. Roadside cuttings often slipped. Volumes of debris were generally not great and, with some exceptions, the consequences were not severe.

Observations of incidences of slope failures by Lumb (1975) noted few failures in dry seasons or early in wet seasons. This led him to propose a concept of slopes drying out in the winter and

"ripening" due to ingress of water with corresponding reduction in strength during the wet seasons. He considered that many failures occurred during periods of heavy or severe rainfall after wetting, softening and "ripening". He proposed the "wetting band" concept which models the direct infiltration of rain water into soil and downwards seepage resulting in a rise in the ground water levels. This concept is still in use today for estimating ground water levels for evaluating the stability of slopes.

2.9 *Testing*

Several geotechnical testing laboratories were operating. However the results being produced in some cases were of concern. For example, many times triaxial tests on samples of completely decomposed granite were reported to have been drained with each stage lasting several days. They were reported to have an effective cohesion of typically over 30 psi (over 210 kPa) and an effective angle of friction of less than 10 degrees. In many cases such results were not credible and were indicative of poor quality of testing. Subsequent investigation led one to question in some cases whether or not the testing was actually carried out as reported since the number of tests exceeded the number of apparatus. For a brief period I stopped ordering triaxial testing from commercial laboratories until I was able to implement adequate quality control. Some thought that designing soil slopes without laboratory tests was irresponsible; I considered that it was irresponsible to spend money on tests when I dismissed the results as unreasonable.

Even as late as 1986 I raised the rhetorical question "Do the test results relate to the performance of the prototype?" noting that strength tests were often no more than index tests as reported by Cooper (1986).

2.10 *Hand dug caissons*

Prior to 1976 versatile and cheap labour were used to construct hand dug caissons for foundations. Shafts typically of 1 to 1.5 metres in diameter taken to hard rock provided large diameter piles, called hand dug caissons, of very high load bearing capacity. The method was robust since it permitted visual examination of the formation and probing by drilling beneath the formation level. With minimum site works, a team of men and women, teams with two men digging and one woman hoisting out the spoil, were able to work on many caissons at once, and they could achieve a rate of progress of about 0.75 m per day in each shaft.

In 1976 the technique was adopted for building underground walls as an alternative to diaphragm walls. Up to 1,000 caisson workers were engaged on MTR construction work alone. Very quickly the method was adopted for constructing retaining walls on sloping sites.

When drilling into rock, fine dust was created. Protection from dust was not enforced in most cases. Sadly, many of these workers had chronic pneumoconiosis and within 15 years hundreds of caisson workers had died. Eventually Building Authority (1993) banned the use of hand dug caissons in all but exceptional circumstances.

2.11 *IT*

In 1976 the term "IT", information technology, had not been conceived. Logarithm tables and slide rules were still in use by engineers. Mechanical calculators were still in use in offices. In 1975 electronic hand calculators with one memory and four functions came onto the market and, although they were competitively priced at the time, they cost more than my salary for a month.

Computers were cumbersome and were not generally available in design offices. Electronic programmable calculators were scarce and the few that were available were in much demand. Engineers had to write their own programs as reported by Beattie & Chau (1976).

2.12 *Interest group*

In Hong Kong there were few geotechnical engineers and even fewer engineering geologists. Much of the geotechnical work was conducted by civil engineers.

There was a geotechnical engineering interest group who met on an ad hoc basis. AS I recall, they included Peter Robertson (Fugro), Tom Yamashita (Intrusion Prepakt), Henri Marchini (Bachy Soletanche), Alan Beattie (Binnie), Paul Tong (Enpak), Richard Maher (Enpak), Stanley Elliot (Scott Wilson), Michael Guildford (Scott Wilson), and myself.

Peter Lumb was teaching geotechnical engineering at the University of Hong Kong. A perceptive man, he had withdrawn from the generally adopted deterministic approach of measuring and applying soil properties and he advocated a statistical approach well in advance of other praticioners as illustrated in Lumb (1979).

3 ENGINEERING CHANGES SINCE 1976

Subsequent to 1976 there have been two major changes which have driven the development of geotechnical engineering in Hong Kong. These are the economic growth and the recognition of the need for Geotechnical Control.

3.1 *Economic growth and investment in infrastructure*

Subsequent to 1976 the economy of Hong Kong generally improved, albeit with periods of recession, and in the last 30 years there has been massive investment in infrastructure; firstly to meet the social needs, and secondly to develop Hong Kong into a World Class City.

In the mid 1970's, faced with repeated fires with multiple fatalities in squatter areas, Government recognised the need for safe economical housing for masses of people and embarked on an extensive programme of public housing and the necessary infrastructure such as roads, water supply, sewerage and drainage and a new airport. This resulted in a lot of geotechnical engineering. The key to housing was a series of New Towns, including Sha Tin, Tai Po, Tuen Mun, Tseung Kwan O, Fanling, Shek Wu Hui, Kwai Chung, providing in all accommodation for some 3 million people.

In the private sector, there was booming investment in real estate and in capital works such as container terminals and in power generation and transmission.

The massive investment by Government in the urban fabric, as well as private development, coupled with geotechnical control imposed by Government, has resulted in a growth of geotechnical engineering in Hong Kong both with respect to numbers of geotechnical engineers and the depth of development of the technology which ranks highly worldwide in terms of numbers of publications and, due to the small land area, must hold the record for numbers of engineers per hectare of land area. Notwithstanding

the major construction works, the focus of the geotechnical activities, as reflected in the numerous publications during the last 30 years, has been on geotechnical control and slope safety.

3.2 *Geotechnical control*

In 1976, memories of rainstorm disasters of 1972 with 138 fatalities had been put somewhat out of mind. Then, in the wet season, disaster struck again and fill slopes at Sau Mau Ping flowed as mud avalanches and caused the death of 18 more people. In 1976 an international Commission of Inquiry was appointed and they identified the significance of the disasters as reported by Hong Kong Government (1976). On the technical side, they recognised the need for compaction of fill and recommended fill within 4 m of the ultimate face should be compacted to 95% of standard density. They also identified the need for geotechnical control in Hong Kong. In response, Hong Kong Government established the Geotechnical Control Office (GCO) in 1978.

The evolution of geotechnical control since 1978 and the achievements of the GCO and subsequently of the Geotechnical Engineering Office (GEO) are described in other papers in this seminar. However the impact of the GCO can not be understated.

Highlights to mention here include establishing the slopes safety system and establishing standards relating to investigation, testing, design, and construction practices.

These objectives have been achieved through a broadly based structured plan of providing basic studies of geology and regional studies, gathering information relating to landslips, establishing the Hong Kong Geological Survey (HKGS), including publication of geological maps and making available memoirs; the establishment of the massive archive of borehole data managed by the Geotechnical Information Unit (GIU); checking of private building works submissions; cataloguing and prioritising slope features, and implementation of upgrading; and the slopes safety warning system.

Early tasks were to improve practice including Geotechnical Control for Buildings, for private building works, and to check geotechnical design for public works. A major study was conducted on the slopes of the Mid Levels. This study recommended special measures which are still enforced by the in Scheduled Area Number 1 under the Buildings Ordinance by the Building Authority (1982).

Publication of standards has included the genesis many of the Practice Notes for Authorised Persons and Registered Engineers, Technical Guidance Notes, and six Geoguides which have sold widely at home and overseas and a myriad of other publications. The latest publication under preparation is Engineering Geological Practice in Hong Kong

Through progressive improvement, testing of soil and rock has evolved from being so poorly conducted as to be meaningless, through a perception that "strength tests are index tests", via accreditation of testing laboratories, to include wide access to research quality testing on a commercial basis that is available today.

Tackling problems with slopes has progressed over the 30 year period. To tackle the problems with slopes, GCO and GEO have conducted many investigations and cataloguing of slope failures. Since 1996 teams have conducted systematic investigations of landslides with the objective of increasing knowledge about geological causative factors. They have instituted a slopes warning system, they have registered slopes and prioritised them for study and upgrading. This has led to economies of prescriptive design, a detailed programme for implementation and, successful treatment of hundreds of slopes.

Studies have been conducted and the results published with the general objective of improving standards. These include area studies of terrain evaluation and land use, regional geological studies, a register of slope features (walls, cuttings and fill slopes) which includes some 53,000 features, an inventory of landslides, recently enhanced to include some 105,000 recent and relict landslides in natural terrain, and many technical studies. This general approach has facilitated and includes the basis for Quantified Risk Assessment for man made features and natural slopes. Risk based slope management is excellent and represents state-of-the-art in applied risk management.

An achievement of GEO has been the new approach to slope safety, as described in other papers. The focus has been public awareness and on Landslip Warning. Today the Hong Kong Slopes Safety Management System is amongst the best in the world and the GEO specialists are recognized amongst leading experts in the hazard management community.

In the light of the large investment in infrastructure and the limited reserves, GEO provided a fill management service which identified and managed resources of filling materials and deposit areas for unwanted materials as described by Whiteside (1988). Without this service the rapid development of the last 20 years would not have been feasible in the time available.

3.3 *Slope stability assessment*

As described above, before 1976 slope failures were a frequent and an inevitable consequence of steep slopes and heavy rainfall. Until 1976 there was very little engineering work done on slopes. Roadside slopes often slipped but generally the volume of debris was not great and the consequences of the failures were not severe.

In 1972 there were several concurrent landslip disasters. At Po Shan Road failure of a major slope resulted in 67 fatalities and the total collapse of an occupied high rise apartment block. At Sau Mau Ping a fill slope failed and a Temporary Housing Area was partly buried with debris and 71 people died as reported by Hong Kong Government (1972a). In 1976 disaster struck again at Sau Mau Ping where a filled slope failed as a mud avalanche which is now described somewhat confusingly as "liquefaction" and 18 people died as reported by Hong Kong Government (1976).

Since 1976 the studies of slope stability have been quite extensive. These include area studies, compilation of a register of slopes and records of landslides, and territory-wide measurement of rainfall. The information gathered has permitted a statistical approach to the correlation between the occurrence of landslides and rainfall.

Statistical correlation linked failures with intense short duration rain. A criterion of 70 mm/hr has been adopted as "intense rainfall" affecting failure of slopes. Weather forecasting is now linked with rainfall monitoring and the issuance of rainstorm warnings depending on hourly intensity of rainfall. The currently used landslip warning system relies on correlation of 24 hr rolling rainfall with observed landslide frequency. It has been suggested that correlation based on soil moisture balance might be effective.

Although failures are associated with rainfall, studies of the phenomenon of infiltration have been few. Lumb's wetting band concept, was based on a rate of advance of wetting band for fine soil of the order of 1 m in 11.5 days (permeability $k = 1 \times 10^{-6}$ m/sec). This slow rate of infiltration and seepage is often over-

looked even though Lumb observed that slopes dried out in the dry season, and he described the slopes as "ripening" during the wet season, i.e. a period or periods of wetting resulted in softening, until a heavy rain storm brought the slopes down.

The performance of slope surfacing has been investigated through experience by trials with different surfacing. In some instances the same slopes have been surfaced successively with chunam, then with shotcrete, now with geotextiles and selective planting.

Loose fill slopes were addressed as a priority. Different methods have evolved. Loose fill has been re-compacted, or treated with dynamic compaction, or with soil nails, and, to a limited extent, with vegetation.

Technical studies of the performance of slopes during periods of rain are few.

Data on groundwater levels has been obtained from piezometers and standpipes. Data has been obtained in the mid levels since 1978 for the Mid Levels Study as reported by GCO (1982). Monitoring of ground water levels in the Mid Levels continues. Numerical modelling was initiated by Brian Leach & Herbert (1982) and modelling is continuing at Hong Kong University by Jiao (2006).

There have been few instances of controlled field experiments with irrigation on slopes, measuring infiltration, studying changes in partial saturation, modelling and observing deformation of slopes prior to failure. In the late 1970's suction was thought to play a major role in slope stability. A few studies were carried out using tensiometers as illustrated by GCO (1984). However depletion of suction during rainstorms was observed in slopes that had computed factor of safety less than unity that did not fail as reported by Sweeney (1980).

During recent years, with the accreditation of testing laboratories and the expansion of tertiary education including geotechnical testing laboratories in the universities, quality testing can now be obtained. This has included tests on soil samples at low stresses and following different stress paths as appropriate to stresses in slopes and on soil which is only partially saturated. However experience in this area is still limited and at present the stability of only a few slopes have been studied making use of such test data.

3.4 *Railways*

In 1976, KCRC operated diesel locomotives on mainly a single track. This line has since been duplicated including cutting back slopes, replacement tunnels beneath Lion Rock and elsewhere and electric traction and is now called the East Rail. Tunnelling was essentially by drill and blast. A new line from Kowloon to the border has been built. West Rail includes both cut and cover and mined underground sections. Work is in progress on the Kowloon Southern Link.

In 1976 the first sections of the MTRC were under construction for the Modified Initial System. Subsequently the Tsuen Wan Line, the Island Line, Kwun Tong Line, Airport Rail and Tung Chung Lines, and Tseung Kwan O lines have been added. The system includes 39 underground stations and 17 others at grade or on viaducts. Altogether in Hong Kong there are now some 113 km of railway, mostly underground.

In 1976 MTRC tunnels below the water table were commenced with open shields and air pressure. Control of inflow of ground water in soils with low porosity required the use of multi-stage grouting and water soluble silicate grouts. The tube-a-manchette system was successfully introduced.

The first underground tunnels in rock were of 5.2 m diameter and constructed by drill and blast. Some of the tunnels experienced such little inflow that relief drains and drained assumptions were adopted for the design of the permanent linings. Since then tunnel diameters have been increased, the widest span at Tai Koo Station cavern has a span of 22 m as described by Sharp et al. (1986). Tunnel boring machines (TBMs) having been introduced for cable tunnels and then for sewer tunnels, were then adopted for construction of MTR tunnels. It is noteworthy that for Quarry Bay Relief tunnels granite was encountered of such good quality that suggested the use of no lining but, in the event, it was not adopted.

More recently, enclosed tunnel boring machines have been used of up to 9.6 metres diameter EPB for mixed ground on West Rail Contract DB320. Currently a slurry shield of 8.1 metres diameter is in use for the South Kowloon Route, Contract KDB200.

There has been increasing congestion of urban areas and complexity of underground works. This has resulted in more difficult and costly construction. For example, QBR mining beneath North Point Station close to occupied tracks, and the extension of Tsim Sha Tsui Station has involved excavating immediately above operating tunnels. These projects have provided fascinating opportunities of work for geotechnical engineers.

Current planning for the MTRC includes the West Island Line with proposed mined stations in densely populated areas.

3.5 *Highways*

In 1976 the first bore of Lion Rock Road tunnel was operating. Since then the Lion Rock Tunnel has been duplicated and many other road tunnels have been constructed including the Aberdeen, Tseung Kwan O, Cheung Ching, Tates Cairn, Shing Mun, and Tai Lam Country Park Tunnels. Technical developments include increasing spans. Tunnel drives with spans of up to 15 metres are now common and spans of up to 60 metres occur at intersections of ventilation tunnels with road tunnels.

For the Aberdeen Tunnel, the presence of thick completely decomposed monzonite forming the slopes at the southern portal could not be mined due to the inflow of groundwater and later involved extensive grouting to stabilise the hill side using silicate grout and the tube-a-manchette process.

Several road tunnels have been built under BOT franchise. Bored tunnels under BOT franchise are at Tate's Cairn and Tai Lam Country Park. Working for a contractor, who is also a stakeholder in the franchise, hones the skills of the designers to produce cost effective designs and construction. For example, the major cut slope to the west of the southern approach to the Tai Lam Tunnel was subject to a re-profiling optimisation during construction to best suit the ground conditions as encountered. For the north portal of the same tunnel, NATM has been used in soft ground permitting rapid excavation beneath a live water supply channel.

Predicting ground conditions for tunnelling has progressed from the early days of reliance on aerial photographs and experience. Facilitated by the use of computers, large amounts of data concerning the ground conditions as encountered during construction and data on the performance of the contractor can be stored, retrieved and analysed. Quantified estimates of expected rock quality and types of tunnel support are feasible and are becoming progressively more reliable as the data base expands. GEO has recently opened a GIS database for which the first layers are of tunnel data. At present collecting data electronically is

at the option of the engineer whereas collected data should be a national resource.

The highways of Hong Kong include extensive networks of viaducts. There are too many to name. These have included challenging ground conditions for piling with irregular and, in places, steeply inclined rock head, boulders above rockhead, and deep weathering below rockhead. During the last 30 years these ground conditions have been encountered many times and often in congested urban areas.

Although work on construction of new highways has been technically demanding, the largest use of geotechnical manpower for highways has been the upgrading and maintenance of slope features alongside roads.

3.6 *Submerged tube tunnels*

In 1976 the first cross harbour tunnel had been opened to traffic on a BOT contract. Subsequent immersed tube tunnels include the MTRC MIS line, the Eastern Harbour Crossing, for road and rail, the Western Harbour Crossing and the Airport Rail and Tung Chung Line MTRC tunnel. Geotechnical engineering is required not only for the design of the casting yards and the cut and cover approaches but it is fundamental to the successful control of settlements, the joining of the units, and the eventual smooth profile as described by Suraj de Silva *et al.* (1997)

3.7 *Reclamation*

At about 1976 reclamations were formed largely by end tipping of dry fill. Minimal control over tipping of fill and disposal of dredged soft mud as fill material led to displacement of mud as mud waves and excessive ongoing settlement.

During the last 30 years reclamations have been widely adopted and techniques have improved. Generally, reclamations have made use of controlled tipping, including the use of barges or hydraulic pumps for placement of lower layers, geotextiles and wick drains as described by Endicott (2001).

With better control the reclamations have been more successful. However, the disposal of refuse on reclamation at Tseung Kwan O led to a failure including underlying soft mud in 1984. This failure was subject to an engineering investigation which, for the first time in Hong Kong, involved a detailed study of properties of landfill and consolidation of soft to very soft clays as reported by GCO (1984).

The need for extensive reclamation for new town and container port development could not be met using land based fill and it was necessary to identify, and win, marine sand for fill. The use of marine sands for fill in reclamations was initiated by private development for container terminal at Kwai Chung. Terminal 6 in 1987as reported by Wragge-Morley (1988).

In order to cope with the demand for fill and to steward the limited resources of marine sands Government conducted a search for fill throughout all coastal waters and established a Fill Management Committee to manage the resource of some 600 million cubic metres of sand and to allocate sufficient fill and dumping areas for major projects. The offshore prospecting was conducted mostly by marine seismic survey.

Studies for a new international airport studies included a trial reclamation and embankment with different drains at Chek Lap Kok as described by Fung *et al.* (1984). This trial was monitored for over 10 years yielding a valuable source of quality data on the longer term behaviour of reclamations over soft mud with a variety of vertical drains.

However a decision to accelerate the construction of the airport taken at about 1990 led to adopting a largely dredged reclamation by removing a large proportion of the soft mud from the seabed. The site investigation included some 3,300 cone penetration tests and marine seismic profiling. The site preparation is the largest reclamation in Hong Kong. It was carried out in only 28 months and involved a large fleet of dredgers to win some 89 million cubic metres of marine sand and to dredge from site and from borrow areas some 100 million cubic metres of marine mud. The reclamation included dry fill obtained from the lowering of Chek Lap Kok Island and other sources totalling 118 million cubic metres as described by Plant *et al.* (1998).

Generally applications of ground treatment to reclamations in Hong Kong have been limited in number. For the reclamation for Castle Peak Power Station, in order to reduce subsidence during service, dynamic compaction was adopted. However this technique has not been widely adopted in Hong Kong. The reclamation for the Airport Rail depot encountered problems with settlement and was subjected to various types of ground treatment as described by Suraj de Silva *et al.* (1998).

3.8 *Sewerage collection and treatment plant*

Secondary sewage treatment plants have been provided for New Towns.

Tolo Harbour Effluent Export Scheme involved a TBM drive from Tai Po to Tseung Kwan O. Whereas these projects have included tricky foundations on newly reclaimed land, the main area for geotechnical input has been underground.

GCO/GEO commissioned studies by Ove Arup and Partners (1990) for development of underground space. Of the sites studied two have gone ahead. The cavern for treatment works at Stanley was successful as reported by Tam (1994). As was the cavern for West Island Transfer Scheme for refuse.

Deep sewer tunnels have been built. Plans for the harbour area to transfer sewage to a large plant for treatment at Stonecutter's Island are under phased implementation. The construction of the Stage 1 tunnels of depths as low as 140 m below the sea was the subject of dispute and arbitration. The claims concerned tunnelling conditions and control of inflow of water. The data from the construction for the completion of the tunnels has been retained on an electronic data base which is available (on application). It is being processed for use on new tunnel projects. This data is a valuable source of information on rock quality, rates of inflow, on amounts of ground treatment to control the inflow, amounts of materials used and rates of progress. Such empirical data coupled with a geological model offers the opportunity to estimate expected conditions for new tunnels of a similar type on a statistical basis.

3.9 *Power*

In addition to buried cables, transmission of electrical power has made extensive use of towers and suspended cables. These have included some very steep locations for towers and some challenging site formation works. In urban areas TBM tunnels have been used for electricity and gas supply as reported by McFeat Smith (1992). These tunnels have provided further experience and a broader data base for machine bored tunnels.

3.10 *Professional practice*

Changes in professional practice during the last 30 years are described in other papers.

In 1976 there was an interest of a handful of people. Now HKIE includes a Geotechnical Division, a Geotechnical Discipline, and interest groups. In addition there is the Hong Kong Geotechnical Society HKGS, Geological Society of London Kong Kong Regional Group GSLHK, Association of Geotechnical and geo-environmental specialists AGSHK, Hong Kong Branch IMMM, Tunnel Society. HKIE counts 4250 members of the Geotechnical Division of all grades.

Regarding registration and control, in 1976 there was no GCO/GEO, and no registration of geotechnical engineers. There was no independent checking, and no accredited laboratories. There were programmable calculators but no approved programmes. Now there is registration of geotechnical engineers. A general registration as a Professional Engineer, RPE(GEO), has been in place since 1990. A register controlled by the Building Authority for private building construction, RGE, has been in place since 2004.

Since 1978, GCO commenced checking and now GEO checks all private geotechnical works and, recently, all tunnels. Laboratories are accredited and computer programmes for use for geotechnical submissions are required to be approved for use.

3.11 *Technical advances*

In 1957, the geology of Hong Kong had been mapped by Allen and Stevens (1971) and was published at 1:50,000 scale. During the last 30 years the geology has been studied in considerably greater detail and maps are available at 1:5,000 scale. Some of the rocks have been re-classified and marble in the North West New Territories has been found to be far more extensive that previously thought. Deep zones of weathering have been identified at Ma On Shan and at North Lantau.

Extensive records of slope failures have been compiled. Quantified Risk Assessment has been established for slope features and natural slopes in Hong Kong. However although slopes fail in the wet season, technical advances in the study of groundwater and its contribution to slope deformation leading to failure have been quite modest.

Whereas the GIU has collected a massive library of old borehole records, IT has provided for instant access to lots of data, on slope failures, rock conditions and tunnelling performance etc. GIS provides the means to quickly access such data bases. Examples of data from tunnels are referenced above.

The use of Geographic Positioning System (GPS) facilitates location in the field and ortho-rectified aerial photos with contours can be down loaded in the field for immediate reference.

3.12 *IT*

In 1976 engineers used slide rules. If they were lucky they could get access to a programmable calculator. Now powerful software, which is user-friendly, is readily available. Computer programs are registered for use.

Every engineer has use of a PC and some have palmtops for use in the field.

With the construction of data bases, access to enormous amounts of data is made easy.

The user has to click on an icon and it all happens with instant gratification.

The downside is that the user becomes taken up in a virtual world of simulated reality which is fine for computer games. The diligent engineer has to check the source, accuracy and reliability of the data. This takes time and care. He also needs to understand what he can do with the data and how the software that he has operates. A danger is a technical program with default parameters. The user gets an answer but not necessarily the right one.

The reality of computers is that they are only an aid, a very powerful aid, to save the engineer from performing numerous detailed calculations.

The misuse of data bases and software is a big risk for engineering practice.

3.13 *Litigation*

Not all changes have been for the good. Hong Kong used to have a reputation for being non litigious. In 1975 there was negative inflation and a fixed price contract promised big rewards. However, tightening of the market leads to fierce competitive tendering and awarding contracts on the basis of the lowest tendered sum means accepting the tender with the lowest provision for the unexpected. In practice it turns out that this does not meet the Employer's objective of a secure price for the works and no claims. The substantial claims for civil engineering works usually are based on changed conditions. A clause in the contract requiring the contractors to accept risk on changed ground conditions often does not work out in practice. For example, for the first stage of the deep sewer tunnels in Hong Kong, the claims were reported to be of the order of a billion dollars and were settled for $750 million as reported in the press by SCMP (2001). Other newsworthy recent litigation has included prosecutions in relation to offences whilst constructing piles that were short of the requirements, as reported in the press by SCMP (2003). These piling contracts were signed at a time of low bidding prices too.

Although Hong Kong has become recognised worldwide as a centre for good arbitration, litigation can be extensive and expensive. Costs for litigation are high and there is a need for Alternative Disputes Resolution in order to speed up the process of resolution and to mitigate costs.

The practice of expert witnesses in Hong Kong has been severely criticised by the judiciary. Hong Kong experts have been branded as "cowboys", and there is need for procedural reform as reported in the press by SCMP (2002).

4 WHAT LIES AHEAD?

4.1 *Technical*

Although over the last 30 years great improvements have been brought about in geotechnical engineering, areas of uncertainty, or lack of understanding, remain.

At present I can identify two technical areas of uncertainty that merit research and are under study. Both involve groundwater. They are the hydrogeology affecting slope stability and the hydrogeology of constructing deep tunnels.

4.2 *Study of groundwater in slopes*

The coincidence of heavy or severe rainstorms and slope failures in Hong Kong is well recognised. However the effects consequent upon rainfall such as infiltration, sub-surface flow, changes of saturation, changes of suction, and groundwater pressures have

not been investigated in detail. For purposes of development works typically piezometers and standpipes are measured for only limited periods and there is little synthesised data. It is poignant that the inaugural meeting of the HKIE Geotechnical Group in 10th May 1980 featured a paper by Endicott (1980) on the analysis of piezometer data and rainfall records; however currently there are relatively few publications on the subject, and little scientific insight.

Current practice includes design of groundwater rises in slopes for assumed extreme conditions relying on the wetting band concept which is now 30 years old.

There have been few instances of controlled field experiments with irrigation on slopes, measuring infiltration, studying changes in partial saturation, modelling, and observing deformation of slopes prior to failure.

The currently used landslip warning system relies on correlation of 24 hr rolling rainfall with observed landslide frequency. It has been suggested to investigate correlation based on soil moisture balance.

Associated topics include the development of appropriate constitutive models. Testing and modelling conditions of partially saturated soils, at low stresses and under different stress paths or strain control.

Further insight and verification of conceptual models could be achieved by centrifuge testing.

In my opinion the response of slopes to infiltration and groundwater is an important area for research since an understanding of the process of slope response to rainfall is fundamental to understanding how and why slopes fail.

4.3 *Groundwater in deep tunnels*

The variable nature of groundwater flow through hard igneous rocks, and the generally limited knowledge of the condition of the rock around a tunnel, lends itself to statistical analysis provided that a sufficiently large data base can be assembled to reliably determine the probabilities of occurrence. Data is available for limited rock conditions. Expansion of the current data set by collecting and classifying field data from construction should enable reliable estimation of groundwater conditions when tunnelling. Likewise geological conditions can be determined and tunnelling performance, including rates of advance and the need for temporary supports or ground treatment to control inflows, can be calibrated against rock conditions.

Such data should be gathered, as a matter of course, for all tunnelling projects and made available for analysis and further use.

4.4 *GI*

In Hong Kong a lot of ground investigation is carried out. Techniques that are adopted are generally simple ones. Ground investigation for the Study of the Mid Levels, reported by GCO (1982), introduced foam in 1978 but there have been hardly any improvements in drilling and sampling since then. Excepting for directionally controlled coring over long distances, such as 500 m or more, tens of thousands of boreholes carried out to specifications that have not changed much in nearly 30 years. New techniques and standards as used overseas could be adopted.

4.5 *Complexity*

30 years ago Hong Kong was in need of development. Now Hong Kong is a developed city with an infrastructure that ranks highly in the world. Whereas in 1976 Nathan Road could be dug up for cut and cover construction for the MTR, the complexity of our urban architecture is such that new underground railways, have to wind in and out of underground obstructions, including existing underground railways, and disruption at ground level has to be minimised. Geotechnical engineering has become correspondingly complex and geotechnical engineers will continue to be needed.

5 COMMERCIAL

As a result of the large amount of geotechnical work in Hong Kong a large number of geotechnical engineers have built up. Also the infrastructure of Hong Kong has been supplied, and largely completed, to a good standard. It is likely that the capacity of the geotechnical engineers exceeds the expected ongoing workload and their continued deployment will depend on earning projects overseas or on the mainland. In order to make use of skills learnt in Hong Kong, there should be a drive to export technology and to import work.

HKIE and GEO have already held meetings on technical exchange with engineers in Malaysia. Geotechnical engineers from Hong Kong have many exchanges with engineers in the mainland.

6 CONTRACTUAL

An objective of contracting ground engineering construction work is to satisfactorily complete construction within time and within budget. Experience has shown that geological risk, especially for deep bored tunnels, can invoke very high consequential costs or additional time. Reduction of geological risk involves both acquisition and synthesis of more data, and the provision of controls.

IT provides for retention and handling large amounts of data. GIS provides a basis for retrieval and assessment of the data. GEO has just commissioned a GIS for geotechnical data of which the first data entries are for a series of bored tunnels. This procedure provides the opportunity to build up a sufficiently large empirical database for statistical analysis to be meaningful. Prediction of rock condition, rock quality, and tunnelling performance such as rates of advance, number of probe holes to be grouted etc. can then become progressively more reliable.

Experience has shown that competitive tendering and lump sum payments does not provide for high risk and, for geotechnical work especially underground, very large claims, sometimes in excess of the value of the contract can arise.

Current debate concerns proposed use of re-measurement for geotechnical work including extension of time appropriate to any increases of work. The objective is that the Employer pays for the work that is done and therefore assumes the risk of changed conditions. Also under consideration is the use of a Geotechnical Baseline Report (GBR) which completes the item description for work ordered within the contract and therefore provides a baseline from which to identify changed conditions. The objective is to more clearly define the work envisaged by the Contract and thereby more uniquely define changed conditions.

7 CONCLUSIONS

For the last 30 years Hong Kong has been a boom town for geotechnical engineering. Major development works for Government and for the private sector have involved extensive ground engineering. Hong Kong has been an excellent place to work and it has become a centre of excellence in certain areas especially in slope safety control. The technology, standards and practices of geotechnical engineering have developed at a rapid pace. Much experience has been gained. For the future, there will be less need for new infrastructure. However the new projects will be typically more complex, requiring more planning, monitoring, and careful construction and will provide new challenges for geotechnical engineers.

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Development of Deep Excavation Practice in Hong Kong

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Abstract: Since the mid 1970's, a considerable number of projects involving deep excavation in soils have been successfully constructed in Hong Kong. These were either basements for building developments or for civil engineering projects such as underground railway stations and cut-and-cover tunnels. A good deal of local experience has been gained and built-up, improving and advancing the engineering practice for such projects. This Paper looks back at the development of the practice over the last three decades and discusses some of the issues involved. The importance of well-monitored case histories is emphasized.

1 INTRODUCTION

The mid 1970's saw the start of construction of the Modified Initial System of the Hong Kong Mass Transit Railway and also some deep building basements. Since then, more than 20 underground railway stations have been constructed, some involving excavation depths of 15 to over 25 m using the cut-and-cover method. A number of building basements also involved excavation in excess of 20m. Many of these projects have been in typical coastal reclamation areas, with ground conditions characterised by fill and marine or alluvial deposits overlying a granitic residual soil profile, with a high groundwater table. A wide variety of retaining walls and lateral support systems have been successfully used including steel sheet pile walls, hand-dug caisson and secant pile walls, and diaphragm walls. Excavations have been carried out with temporary shoring system using bottom-up or top-down construction methods. Many of these excavations have been in densely developed urban areas in close proximity to other buildings, major busy roads with congested and sensitive utilities. Deep excavations inevitably cause movements in adjacent grounds. The overall primary design consideration for deep basements is to control ground movements to ensure satisfactory performance of adjacent structures and facilities during construction.

Ground movements are very much dependent upon the construction including the type and sequence of forming the perimeter basement walls, excavation and lateral support methods, construction time and quality of construction. Also, the configuration of the basement in plan and in depth will affect the deformation of the ground. It is common to use cast-in-situ concrete diaphragm walls to provide temporary and permanent supports for basements in excess of about 12m in depth. Depending on the layout of the basements, ground movements have been successfully minimized in the past for deep excavations either by stiff shoring systems subject to pre-loading or using proprietary supports for bottom-up excavation, or top-down method of construction, or in some recent cases, circular permanent wall cofferdams.

For many building basement projects, high groundwater table outside results in a need to relieve the water pressure at the bottom of the constructed basement by a permanent pumped drainage blanket. Also, as part of the structural design requirements, it is common practice and economical to make use of the permanent basement walls to carry part of the vertical design loads. In addition to satisfying the stability and groundwater cutoff requirements for excavation, the depth of walls will have to achieve an adequate load bearing capacity, and in most cases, the walls have to be extended down onto weathered rocks to achieve the required allowable bearing pressures, usually 1.2 to 3 MPa. The cutoff effectiveness will be further enhanced by the common practice of toe grouting into the underlying 5 to 10 m of rocks. Furthermore, the secondary structural design requirements on water tightness of the basement walls are to be satisfied by using sufficient thickness of the walls and an adequate amount of reinforcements in the walls. The past and current design practices have nevertheless achieved satisfactory geotechnical and structural integrity of the basement walls, providing an important element of ground movement control for deep excavations.

2 A HISTORICAL ACCOUNT

The New World Centre in East Tsim Sha Tsui waterfront area is believed to be the first deep basement constructed with diaphragm walls in Hong Kong (Engineering News Record, July, 1977). The 4-level basement at a depth of 14 m was constructed from 1975 to 1977. The perimeter walls involved the use of 1m thick diaphragm walls up to 28 m deep together with 6m deep continuous toe grouting. During excavation, the diaphragm walls were supported by four levels of inclined anchor tie-backs with a minimum 7.6 m long anchorage length. A basement underslab drainage system was used for the first time in Hong Kong to relieve the hydrostatic uplift pressure over the entire basement area. The construction of the two-level, 9.8 m deep basement of Sun Hung Kai Centre in Wanchai in 1978 also involved the use of diaphragm wall with anchor tie-backs outside the site. (Cheng & Cheng, 1980). Anchors were seldom used since.

The construction of the Mass Transit Railway started in 1976. At Chater Station (now called Central Station) (Fig. 1), Davies & Henkel (1980) reported that the construction of the station box caused considerable movements in the surrounding grounds which had been attributed to the combined effect of diaphragm walling, dewatering and excavation. The ground movements also resulted in relatively large settlements of adjacent sensitive buildings. Their work examined the behaviour of the grounds caused by panel excavation and the station box bulk construction. The use of increased slurry head and recharge wells were

reported to be successful in minimizing ground movements. Measurements from this project, as well as others from other projects of the MTR Modified Initial System, formed the initial and limited data on which the design of subsequent projects was based.

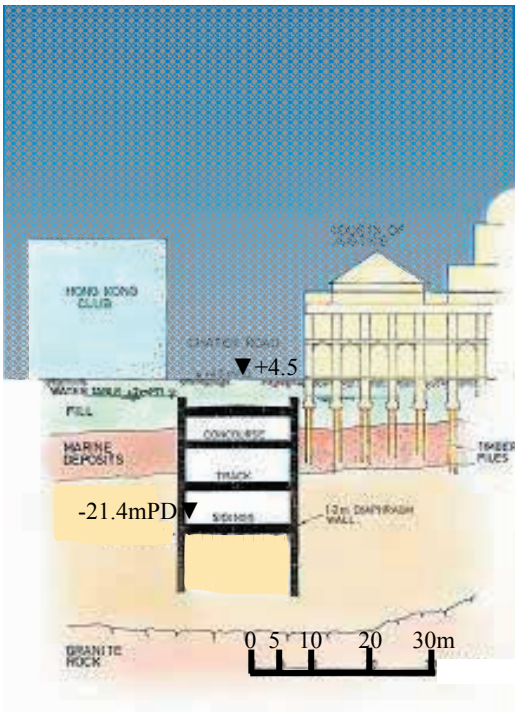

Fig. 1 Chater Station

From early to late 1980's, most commercial developments on Hong Kong Island involved tall buildings on top of deep basements which were usually constructed by top-down method using diaphragm walls to minimize total construction period. Typical examples of these developments include deep basements for the Hong Kong and Shanghai Bank Headquarters Building in Central (Fig. 2), MTRC Wanchai Station (Fig. 3) and Taikoo Shing Cityplaza Site B (Fig. 4).

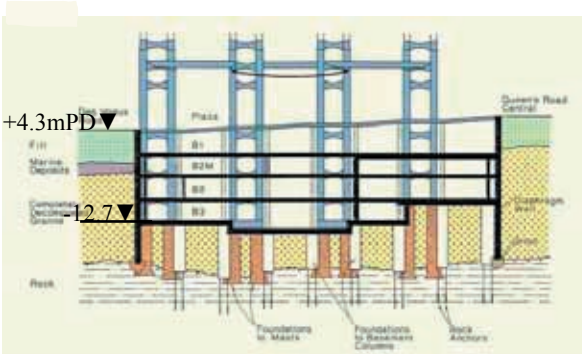

Fig. 2 Basement of Hong Kong and Shanghai Bank Headquater

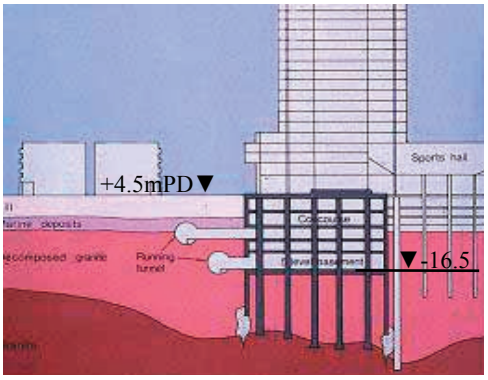

Fig. 3 Wanchai Station

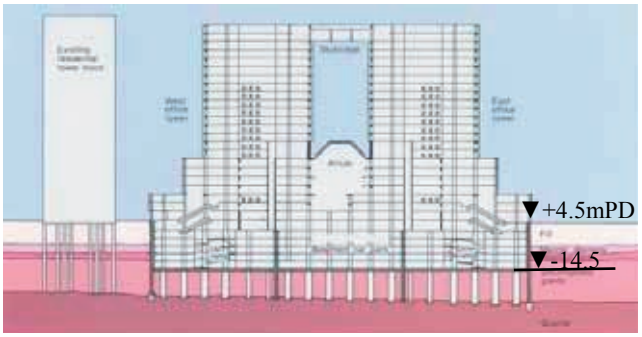

Fig 4 Basement for Taikoo Shing City Plaza Site 'B'

All these sites were located in reclaimed land and the ground would have possibly been affected by earlier construction activities. Following the experience gained from Chater Station, high quality monitoring data were collected during construction of these basements (Humpheson *et al.*, 1986; Davies 1987) to better understand the behaviour of the ground. Another example was the Citibank Plaza basement in Garden Road, Central. In this case, separate cofferdams were used allowing the core of the tall buildings to start bottom-up ahead of the top-down excavation for the rest of the site (Fig. 5). Assessment of ground movements at other sites based on the Chater Station data tended to be conservative, due to variations of ground conditions and previous foundations construction activities nearby. These field monitoring cases were used to back-analyse the basement walls in an attempt to improve the prediction of the performance of excavation.

Fig. 5 Citibank Plaza

In early 1990's, the design and construction of Dragon Centre (Fig. 6) in Sham Shui Po district in the heart of Kowloon

Peninsula created a challenge and afforded an opportunity to the geotechnical engineer. The project involved a 9 storey tall retail complex on top of a 5 level basement for private car and lorry parking, to a depth of 27m below ground, which was considered to be the deepest building basement at the time. The site is located on old reclaimed land and a study of the area suggests that the site and its surrounding area of 100m from its boundaries the ground had not been affected by any construction dewatering such as caused by had dug caissons, MTR underground stations or other excavations. The basement was constructed virtually in a 'virgin' ground with the layer of residual soil (Completely Decomposed Granite, CDG) being reasonably homogeneous. A full scale pumping test was carried out on completion of the perimeter diaphragm walls. The high quality data were back-analysed to establish effective soil stiffness profiles for better prediction of the performance of the deep basement during excavation. Using the back-analysed stiffness profile, close match with measured wall deflection profiles was achieved for each subsequent stage of excavation (Lui & Yau, 1995). Since then the recommended soil stiffness profiles for granitic soils have been widely used to analyse and design of similar deep basements in Hong Kong. The Authors are of the opinion that the monitoring from Dragon Centre site provided the best data and guidance for subsequent design of deep basements and for calibration of new analytical tools.

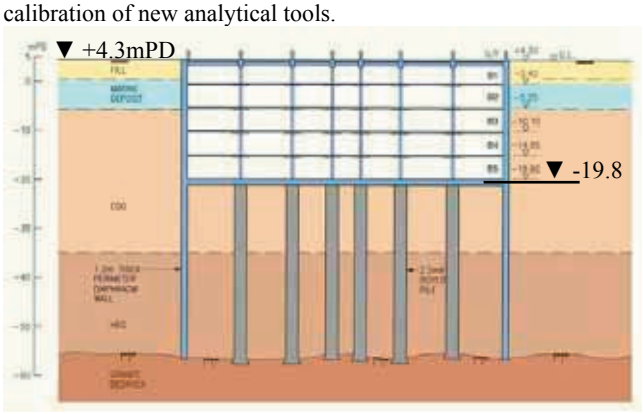

Fig. 6 Cross-section of basement at Dragon Centre

Since mid 1990's and up to most recently, a wide variety of deep basement projects designed with similar practice has been successfully completed e.g. Lee *et al.* (2001); Sze & Young (2003); Sze & Lo (2005). It is worth noting that some recent deep basements have been designed with the use of increasingly large diameter circular diaphragm wall cofferdams to achieve very stiff perimeter walls and to create a clear site for bottom-up construction. Typical examples of these projects include deep basements for Cheung Kong Centre, 36 m diameter (Fig. 7), International Finance Centre Two at Hong Kong Station, 50m diameter (Fig. 8) and International Commerce Centre at Kowloon Station, 76 m diameter (Fig. 9).

Fig. 7 Basement Excavation at Cheung Kong Centre circular diaphragm wall cofferdam, 36 m diameter

Fig. 8 Basement construction at International Finance Centre Two, circular diaphragm wall cofferdam, 50 m diameter

Fig. 9 Basement excavation at International Commence Centre, circular diaphragm wall cofferdam, 76 m diameter

External ground settlements from such construction have been very small. Similar circular diaphragm will cofferdam 40m diameter was used at the Stonecutters Island sewerage treatment plant works (Fig. 10).

The use of circular diaphragm wall cofferdam requires almost zero supporting system. Much saving on construction programme could be achieved with the provision of unobstructed space for construction of basement structures. Recent cut-and-cover station projects such as those for the KCRC East Rail Extension (Fig. 11), West Rail (Fig. 12) and Lok Ma Chau Spur Line (Fig. 13) (Sze & Lo, 2005) made use of early experience to achieve design economy.

Fig. 10 Stonecutters Island sewerage treatment plant, circular diaphragm wall cofferdam, 40 m diameter

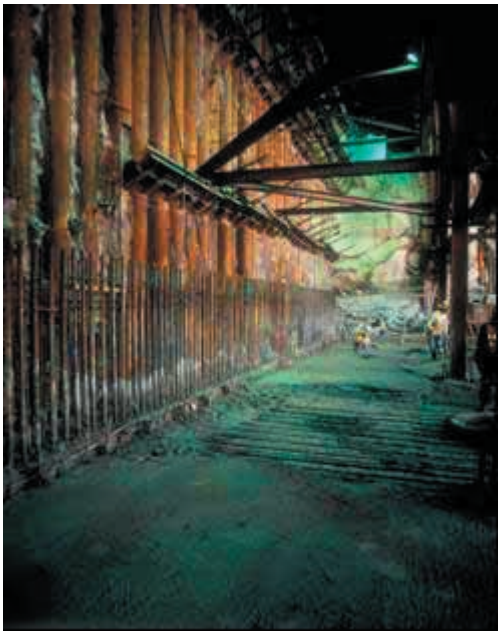

Fig. 11 KCRC East Rail Extension Tsim Sha Tsui East Station

Fig. 12 West Rail Tsuen Wan West Station

Fig. 13 Lok Ma Chau Spur Line

In the case of the Tseung Kwan O Station constructed in under-consolidated soft ground, an innovative scheme was adopted to minimize excavation movements by installing precast supports in preformed traverse trenches as buried props before excavation started (Ho *et al.*, 2001, Fig. 14). Another innovation on construction technique involved the use of precast diaphragm wall panels. The deep basement (excavation depth 13 meters but not entirely in soils) for the Admiralty underground car park in Queensway constructed in the early 1990's (Fig. 15) is an example. Each panel was prefabricated off site with common structural thickness of 400 mm, much thinner then the minimum conventional thickness of 800 mm for cast in-situ diaphragm walls available at the time. Although the precast panels were installed in a slurry trench of nominal 800 mm wide, much saving in permanent concrete walls had been achieved.

Fig. 14 Installation of pre-cast prop for Tsueng Kwan O Station

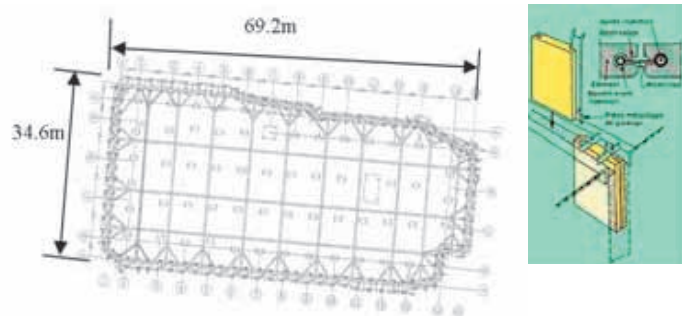

Fig. 15 Precast diaphragm wall at Admiralty Carpark

3 GROUND MOVEMENTS

Records thus far show that failures of the retaining system due to inadequate design relating to stability are extremely rare (Pappin *et al.*, 2004) and in the densely built-up urban areas of Hong Kong, ground movements caused during the various construction stages of an excavation and their potential effects on adjacent properties and facilities tend to be of primary concern.

The total ground settlement at the end of construction is made up of three components, viz those due to (1) installation of the wall, (2) dewatering for excavation, and (3) bulk excavation of the soil enclosed by the retaining wall (Fig. 16).

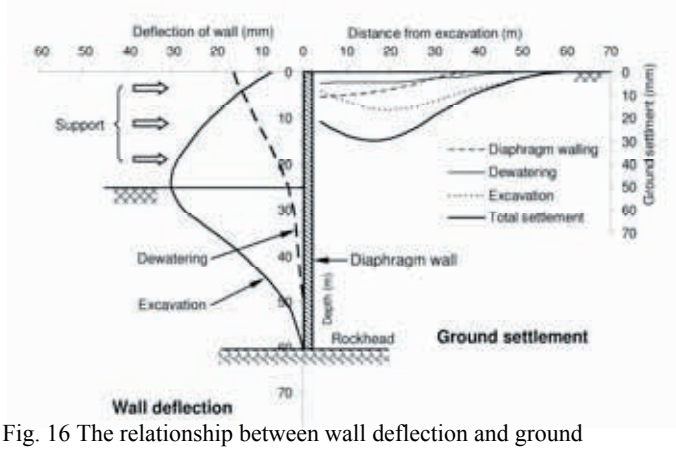

Fig. 16 The relationship between wall deflection and ground movements

3.1 *Ground settlements due to wall installation*

Significant ground settlements resulting from diaphragm wall installation was first reported by Davies & Henkel (1980), although field study of the subject was published earlier (Stroud & Sweeny, 1977). One important observation was that the total ground settlement caused by excavation and concreting of a whole length of diaphragm wall is much larger than that caused by excavating a single panel. This was explained in terms of the breakdown of the arching effect that exists during excavation of a single panel. The zone of ground settlement also extended as a result to a distance of one time the trench depth or more. Following this early project, other ground settlement measurements were taken from other sites in the early 1980's. One example is shown in Fig. 17, for the Taikoo Shing Cityplaza Site B (Ove Arup & Partners 1983), which illustrates the point. Cowland & Thorley (1984) summarized some of the observations available at the time. A version of the measurements summarized is shown in Fig. 18. The art of estimating such settlements remains largely unchanged since the 80's, and is entirely empirical. The practice has been to relate magnitude settlement to the net slurry pressure in the trench, i.e. the total slurry pressure minus the pore pressure in the soil. The swelling of CDG at low stress levels was deemed responsible for the significant settlements observed. Here, the stress history of the CDG at a particular site is clearly directly relevant. In order to mitigate the potential contributors, designers have been adopting a minimum trench width together with alternate trench excavation sequence where appropriate to minimize settlement of sensitive structures. Where necessary, the workmanship of the contractor is also closely supervised through proper control measures including density of bentonite slurry, bentonite flocculation, and trench opening time.

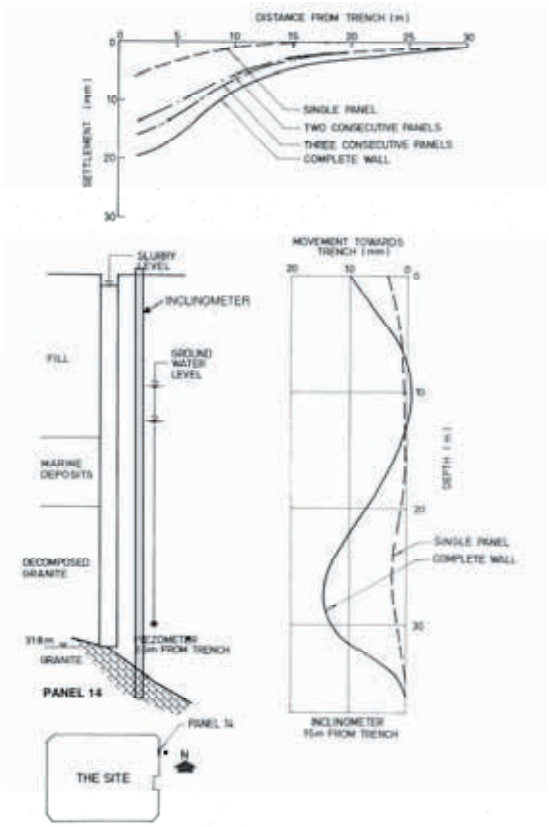

Fig. 17 Monitoring of Ground Movements due to Diaphragm Wall Panel Excavation in Taikoo Shing Project

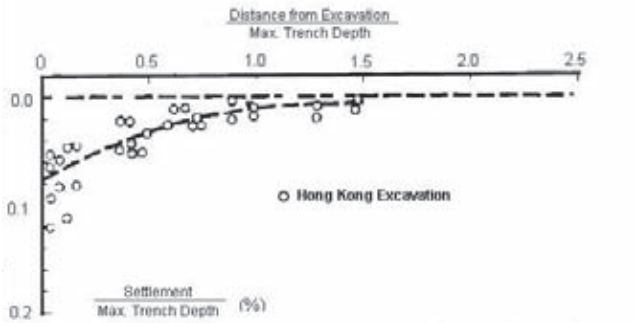

Fig. 18 Summary of measured settlements caused by the installation of concrete diaphragm walls (after Clough & O' Rourke (1990))

Where sensitive structures are located at close proximity of the site, it is common practice to use secant bored pile walls in lieu of diaphragm walls in order to better control movements. The bored pile walls are to be installed carefully and, if necessary, ground treatment including compensation grouting is carried out to further protect the adjacent buildings. The deep basement for URA project K2 presently known as Langham Place in Mongkok is an example (Fig. 19a & 19b). Close monitoring is always essential and review of monitoring data by experienced geotechnical engineers is extremely important to forewarn any possible unexpected movements and to revise construction sequence or to add strengthening works as necessary. More case histories are clearly needed to improve the state-of-the-art.

Recent work on difficult wall excavation for Tseung Kwan O Station (Cowan *et al.*, 2001) is related to the soft under-consolidated clays.

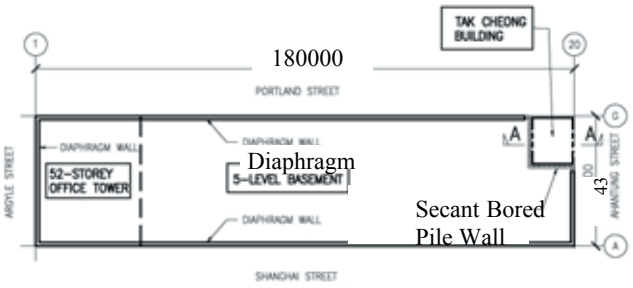

Fig. 19a Langham Place basement layout

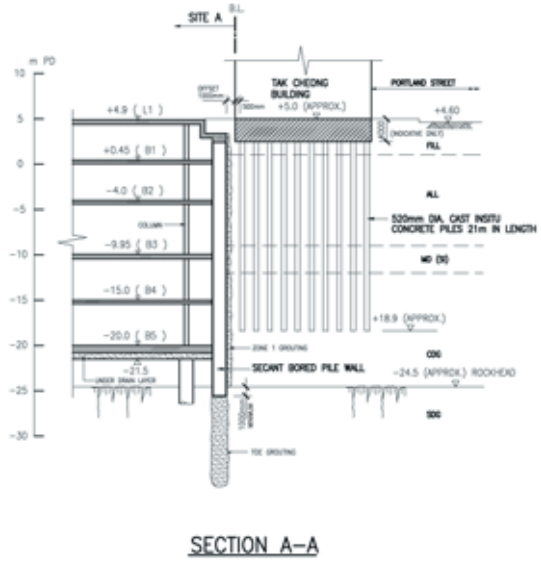

Fig. 19b Langham Place basement section showing existing building foundations

3.2 *Ground settlements due to dewatering*

Settlements occurs due to seepage-induced groundwater level drawdown in the soil. Excavations for urban building sites in Hong Kong where protection of adjacent properties was required often involved diaphragm walls extended to rockhead either for groundwater cut-off and/or vertical load bearing purposes. Additional precautions of silicate grouting is often performed well below the toe level of the wall to minimize drawdown external to the site. A full scale pumping test is often performed, drawing down the water level within the box, to test the effectiveness of the cut-off. Davies (1987); Lui & Yau (1995) show that seepage drawdown can cause settlement outside the excavation at considerate distance. It is worth noting that such test causes inward wall deflection due to consolidation which in turn causes ground settlement outside.

Excavation in deep soil profiles for underground railway stations and cut and cover tunnels often have the diaphragm walls stopping short of rockhead. For example, as in the case of Chater Station, the predicted drawdown settlement outside may be too large and a system of groundwater recharge had to be installed to control the situation.

3.3 *Ground movements during bulk excavation*

Chan (2003) reviewed a chronological series of projects by Arup constructed over a period of 25 years or so. For the range of typical ground profiles, the observed maximum lateral wall movements from these well-controlled excavations with stiff wall and good propping systems fell between 0.15 and 0.3% of the final excavation depth, H (Fig. 20). Observed ground surface settlements have been smaller than those deduced from the corresponding wall deflection. Lui & Yau (1995) reported that the ratio of maximum ground settlement to maximum wall deflection being maintained at 0.5 is considered adequate for practical application (Fig. 21).

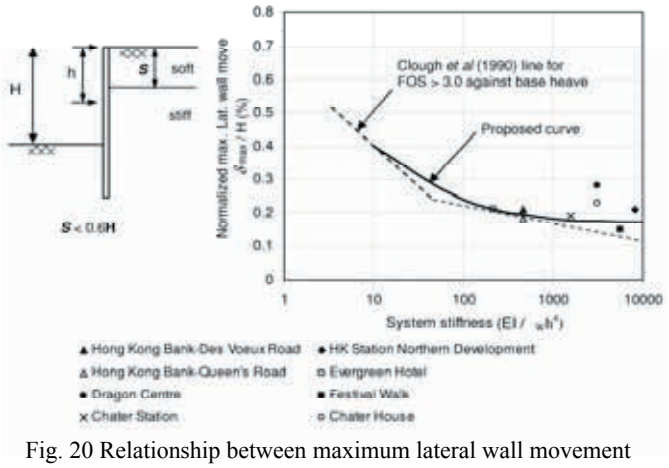

Fig. 20 Relationship between maximum lateral wall movement and system stiffness

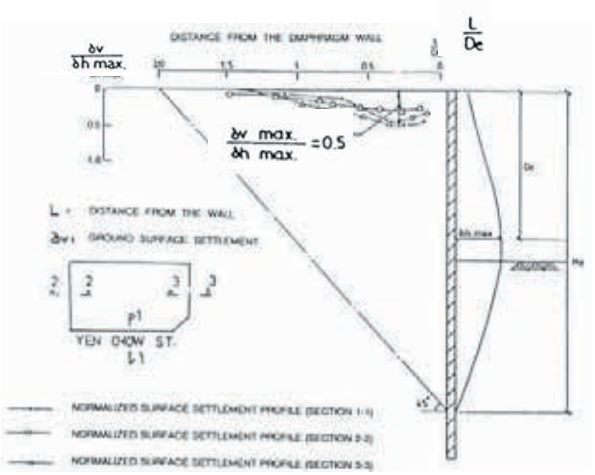

Fig. 21 Normalized ground settlement profile due to basement excavation

Recently Leung & Ng (2007) presented a more comprehensive review of 14 published cases on Hong Kong projects. The paper suggests a useful simple categorization of ground profiles and analyses both the observed lateral wall deflections and the corresponding ground surface settlement. Results from their analyses are summarized as follows:-

Ground Profile	Mean Value Max. Wall Deflection	Mean Value Max. Ground Settlement
SPT N-value ≤30 at half excavation depth	0.23% H	0.12% H
SPT N-value >30 at half excavation depth	0.13% H	0.02% H

Overall, the movement measurements from Hong Kong soil profiles are found similar to those from other sedimentary soils.

4 METHOD OF ANALYSIS

Excavation designs involve limit equilibrium and soil-structure interaction analyses. Over the years, increasingly sophisticated computer programs have become available capable of detailed simulation of the construction process.

4.1 *Sub-grade reaction method*

This method was used from the mid 1970's to late 80's, in which the retaining wall is simulated as a beam and the soil is modelled as a series of horizontal springs, the stiffness of which is either linear elastic or non-linear incorporating strain-softening and plastic yield. This kind of analysis computes wall movements, bending moments and prop loads. Props are generally modelled also as springs, which may be not ideal in representing real support conditions. Many basements (e.g. Humpheson *et al.*, 1986) and MTR stations (Endicott *et al.*, 2002) have been designed with this method but it became obsolete by the early 1990's.

4.2 *Pseudo finite element method*

From the 1990's this method is commonly applied in Hong Kong as in the flexible retaining wall design programs such as WALLAP and *Oasys* FREW (Pappin *et al.,* 1986). FREW is a relatively advanced program, enabling a quick analysis of a complex excavation problem. It has the capabilities to:

- model the installation of wall, and the insertion and removal of strut or anchors
- model the construction sequence
- model dewatering
- model the continuum nature of the soils with finite elements in a simplified manner.

The wall is represented as a line of nodal points, simulated as a beam and the soil is modelled as an elastic continuum. The soil stiffness is specified by means of the stiffness of the elastic continuum with the soil stiffness matrices developed from pre-stored stiffness matrices pre-calculated by the finite element program *Oasys* SAFE. The program calculates the force imbalance at each node imposed by that stage and calculates displacement and soil stresses using the stiffness matrices. Three stiffness matrices relating nodal forces to displacements are developed, one representing the wall in bending and the others representing the soil on each side of the wall. Like the sub-grade method, wall movements, bending moments and prop loads are calculated, but it does not give ground movements around the wall. Props are modelled as springs with specified stiffnesses. It is noteworthy that there may be still some difficulty in representing real support conditions, especially where moment restraint is provided.

The method has inherent approximations and is not a complete or exact solution. Its successful use is dependent on appropriate selection of design parameters. Careful back-analysis of well-monitored cases (e.g. Chan 2003) has helped to give confidence to FREW's extensive application to excavation problems in Hong Kong over the last 15 years.

4.3 *Finite element and finite difference methods*

Using finite elements programs such as SAFE, CRISP, PLAXIS, or finite differences program such as FLAC, it is possible to model:

- soil behaviour through employing complex soil constitutive models
- actual construction sequences
- affected structural and support details
- consolidation and dewatering (including seepage)
- two dimensional or three dimensional problems.

These methods require the user to have significant and specific experience of the particular software package being used and experience of modelling the ground conditions and construction sequence envisaged. Modelled results give ground movements as well as wall movements, bending moments and prop loads can also be obtained.

Many of these and other advanced computer programs offer a variety of soil constitutive models from simple linear elastic to the more advanced ones such as the strain-dependent stiffness BRICK model (Simpson 1992) and the critical state models. In Hong Kong, analysis of excavations using SAFE with linear elastic-perfectly plastic (Mohr-Coulomb failure criterion) have been used occasionally. Malone *et al.* (1997) presented an interesting back-analysis of the Dragon Centre case, testing the effectiveness of the BRICK model. Comparison of wall deflections predicted by this finite elements analysis with BRICK model and pseudo-finite elements FREW analysis with the measured profiles (Fig. 22) suggests that it is adequate to achieve reasonably close match from simple FREW analysis using back-analysed E' values from dewatering of the box. Finite elements analyses have been used mainly to study effects of movements on adjacent sensitive building foundations and tunnels. Up to now, full finite elements or finite differences analyses are not used for routine design analysis. Unless the insitu initial stress state and appropriate soil parameters can be ascertained, it is generally better to use a relatively simple analysis calibrated with experience.

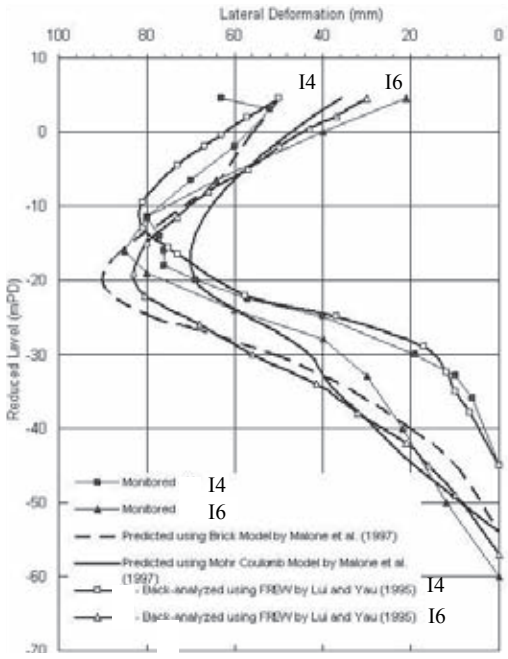

Fig. 22 Comparison of different predictions for the deflection of diaphragm walls at I4 and I6 in Dragon Center

4.4 *Recent development*

The use of global factor of safety for the design of excavation and lateral support works has been the norm in the local industry until the promulgation of the CIRIA Report C580 design approach (Gaba *et al.,* 2003) by the Buildings Department and Geotechnical Engineering Office in 2005 with a two-year trial period. The report not only gives a clear and unambiguous design method of excavation works, the resulted geotechnical design is also consistent with the structural design which is already based on limit states partial factors. In some cases, the use of the method will result in more economical design without sacrificing the overall safety and robustness (Pappin & Tham, 2005).

The application of the C580 method using limit states partial factors has been simplified by the rapid advances in the soil-structure interaction analysis tools mentioned earlier. However, the use of this design approach was reported only occasionally economical for the civil engineering projects so far.

Application of the limit states partial factors method is an international trend that the local industry should follow. The trial period for use of the C580 approach has been further extended in January 2007 to encourage the more practitioners. The authors believe that there are several measures which would assist the transition. Firstly, since the C580 design approach was developed elsewhere where the ground conditions and construction practice are somewhat different from those encountered in Hong Kong, it is considered beneficial that a specific in-depth study should be carried out to extend coverage to local specific ground conditions and practice. Secondly, the partial factors as stated in Geoguide 1 (GEO 1993) should also be reviewed in respect to the design of excavation works. Lastly, case histories based on the use of the method should be collected and shared to give confidence.

5 DETERMINATION OF SOIL PARAMETERS

5.1 *Initial stress state*

It is necessary to assign a value of the coefficient of earth pressure at rest (i.e. corresponding to zero horizontal strain) K_o, to each of the soil layers at the beginning of the soil-structure interaction analysis. Whereas K_o value of saturated reclamation granular fill, marine and alluvial deposits can be reasonably estimated at (1 - sin ϕ'), relevant for normally-consolidated soils, the corresponding value for soils formed by insitu weathering such as CDG is by no means easily obtained. Practitioners have nevertheless found it reasonable to use K_o = 1 - sin ϕ' in their analysis, as if it was 'normally-consolidated'. In their back-analysis of the Dragon Centre basement construction in CDG, Lui & Yau (1995) found a K_o value of between that given by (1 – sin ϕ') and K_a, the active pressure coefficient given by Caquot and Kerisel (1948), gives good fit to observed wall deflection profiles. This was thought to be due to the stress relief which took place in the soils adjacent to the wall panel when being excavated, resulting in a lower value of the coefficient of earth pressure at rest, perhaps closer to K_a. The use of K_o = 1-sin ϕ' for CDG at the beginning of the analysis remains the current practice.

5.2 *Strength and deformation parameters*

Methods of determining parameters for normally-consolidated soils used in the analyses are well-established in the literature (e.g. GEO 1993) and can be based on both laboratory tests and from insitu tests, i.e. the Standard Penetration Tests (SPT) 'N' values. For the Hong Kong residual soils, the angle of internal friction, ϕ', and cohesion, c' are usually from standard laboratory single-stage triaxial tests. For the effective stress Young's Modulus, E', of CDG, the practice has been to correlate with SPT 'N' values without depth correction. Historically, the relationship, E' (MPa) = $f \times N$, was built up in the early 1980's from very limited back-analysis of case histories of building settlements, plate loading tests, pile tests and so on. Early experience in Central, e.g. Davies & Henkel (1980) used a 'f' value of about unity. Later on in the 80's and 90's, full scale pumping tests inside excavation boxes provided additional data which tended to give a higher 'f' value. Lui & Yau (1995) summarises this observation, and Chan (2003) addressed the same issue on the basis of back-analysis of excavation stage wall deflection profiles of some of the cases mentioned earlier in this paper.

Recently Ng & Leung (2006) reported a series of small strain laboratory tests on Hong Kong residual soils. The paper also reported results from Cambridge self-boring pressuremeter tests (SBPM) at a Yen Chow Street site, in addition to those from Kowloon Bay site on similar granitic soil by Ng *et al.* (1999). Interpretation of these data in terms of stiffness at small strains relevant to excavation conditions lends support to higher 'f' values discussed above.

6 CONCLUDING REMARKS

The art of the practice for deep excavation and related support design and construction has been under development in Hong Kong since the mid-1970's. In the past 30 years, a number of projects were well-monitored, providing valuable information aiding the rational improvement of the practice. The authors have chosen to mention a particular series of examples they are familiar with but there are others available, recently summarized and analysed by Leung & Ng (2007).

Until recently, designs have been almost exclusively based on traditional limit equilibrium analysis with global safety factors, coupled with the necessary soil-structure interaction analysis for the design of the structural aspects of the wall and support system, as well as to estimate wall deflection profile which in turn is used to estimate ground settlement. By and large such practice, with care taken during construction, has been successful from a safety standpoint. Over the years there has been tremendous advancement or refinement made in the area of soil-structure interaction analytical tools. With these powerful tools at our disposal, we can analyse complex geometry and construction details and situations. Since 2005, the modern limit states design approach of CIRIA Report C580 has been in trial use in Hong Kong. It provides a modern design framework consistent with that for structures and shows promise for economic designs (Sze & Lo, 2005).

In reality, the advance in sophistication of the computing tools is not matched by our ability to obtain appropriate soil parameters for their input. To start with, we do not know the appropriate initial stress state of the soil close to the wall. We derive relevant soil parameters by empirical correlation with site investigation SPT 'N' values. Practitioners are familiar with the difficulties they routinely confront when trying to determine the SPT "design profile" from considerably scattered data. Both the quality of the site investigation and the natural spatial variations of residual soil profiles must be recognized. The effect of "stress history" caused

by previous nearby construction activities on soil stiffness is also important.

What we know therefore is that when we use certain soil parameters empirically derived with our familiar computing tool, we are likely to be on the safe side. Ground movements cannot be accurately calculated although past records allow us to establish the likely consequences in typical situations. The estimated movements must be viewed in the light of their consequence or effects on the buildings or utilities one is trying to protect. For example, differential movements are more important than total movements. In assessing effects of ground movement on sensitive buildings, we often resort to or forced to use of full finite element analysis, modeling the whole boundary value problem at hand. Here there are still problems, e.g. with modeling adjacent foundations especially when in two-dimension, not to mention the difficulties in soil parameters mentioned above.

Our confidence in the use of the available techniques in predicting and controlling the performance of excavation projects relies heavily on the experience gradually built up from 'calibrating' the analysis using tools such as FREW with well-monitored cases in similar grounds. Regular review and modification of predictions with measurements for each stage of construction would enhance our confidence in controlling performance of excavation. As construction activities move to the less-familiar areas, such as the more recent 'young' reclamations with thicker fill and marine deposits combined with deep rockhead, care must be taken and innovations would be required of more economic design which is to be achieved through shortening wall embedment depth.

The authors are of the opinion that the state-of-the-art in Hong Kong in the design and construction of deep excavations can best be described as a case-history-based empirical practice. Observations in themselves are only sufficient for a robust design if the design situation fits within the range of previous experience (Chan 2003). To extrapolate to other complex situations and/or to achieve more economical design, the employed analytical method and process must be validated by back-analysis of relevant cases to give confidence. The importance of collecting and publishing more detailed monitoring cases cannot be overstated.

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Development and Statutory Control of Pile Foundations for Private Buildings in Hong Kong since the Seventies

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Abstract: Since the economic and population boom in the 1970s, the size and height of private developments in Hong Kong have rapidly increased, and buildings in excess of 30 storeys have now been commonplace. Over the past three decades, numerous types of pile foundations have been developed and evolved to cater for the increasingly taller buildings. This paper recaps the history and technical background of eight distinctive pile foundation types, including Driven Cast-In-Place Concrete Pile, Driven Steel H-Pile, Precast Prestressed Spun Concrete Pile, Hand-Dug Cassion, Large Diameter Bored Pile, Minipile, Socketed Steel H-Pile and Pakt-In-Place Pile. Each of these pile types has once gained popularity in Hong Kong. Amongst them, some have been phased out due to various reasons, and some are still in use to date.

1 BACKGROUND

Before the 1960s, most buildings constructed in Hong Kong were tenement buildings with typical heights of 3 to 6 storeys. Most of these buildings were founded on shallow footings or China fir piles imported from southern Asia. Such timber piles were usually 200mm in diameter, 4m to 6m long, and driven in close spacing. The piles were driven by small steel hammers to achieve a certain set. The strength of such foundation systems is mainly derived from pile raft actions.

From the late 1960s, Hong Kong had experienced continuous economic and population growth. Such changes induced enormous demands for building developments. In response to the demand on residence, the government began to develop the first batch of satellite towns, including Tsuen Wan, Sha Tin and Tuen Mun. As the economy and population continued to boost throughout the 70s and towards the 80s, development of other new towns such as Tai Po, Fan Ling, Sheung Shui, Yuen Long and Tin Shui Wai continued.

Apart from the geographical expansion of areas, the heights of buildings have also increased since the 1960s. From the 3 to 6-storey buildings commonly built in the 1950s, buildings of more than 20 storeys started to appear in the 1960s. Height of buildings continued to increase over the past three decades; nowadays, skyscrapers in excess of 80 storeys become more and more common.

One of the most important contributions to the realisation of such high-rise buildings as we see today is perhaps the advancement of foundation technology. From the era of shallow footings and China fir piles of the tenement building to the present when large diameter bored piles were commonly used in many developments, numerous types of foundation techniques have evolved over the past few decades. Some of them were still in use to date and some have become obsolete. The following sections of this document will introduce the history and technical background of a number of foundation types that once have been or still are commonly used in Hong Kong.

2 DRIVEN CAST-IN-PLACE CONCRETE PILE

A number of Driven Cast-In-Place Concrete Pile types were popular in the 1970s and early 1980s. They were typically used for buildings about 20 storeys high. A common characteristic of these pile types is that they involve driving a casing using mechanical hammers or alike to the desired depth and filling of concrete within the casing to form the pile shaft.

The most common size of this pile type in Hong Kong is 610mm in diameter, the capacity of which under compression is 1461kN, derived from the design principle that the allowable compressive stress on the pile cross-section is 5MPa. To gain sufficient strength, empirically these piles are driven to a stratum with SPT-N value between 80 to 100 blows.

There were a few patented pile types that may be classified into this category:

Franki Pile

Franki Pile was developed by a Belgian engineer Frankignoul. Size of these piles range from 275mm to 700mm. The depth of piles is typically less than 20m, and can be up to 30m in extreme cases.

The piles are installed by driving an open-ended steel tube with a long cylindrical internal hammer. Initially a temporary gravel plug is secured at the base of the tube. By hammering onto the gravel plug, the steel tube penetrates into ground. Upon reaching the desired depth, cables will be used to hold the steel tube in place, and subsequent hammering actions break the gravel-plug. Dry concrete is then placed in the tube and hammered into the ground to form a large bulb until refusal is reached. The pile shaft is then formed by successive charges of concrete. During the process of concreting, the steel tube is gradually retrieved while hammering onto the dry concrete continues to enhance its compaction.

In calculating the required set for driving the steel tube, the Dutch Formula as presented below is used:

$$R_w = \frac{P^2 H}{(P+Q)nq} \quad (1)$$

where R_w = Allowable bearing capacity of pile (in kN);
P = Weight of hammer;
H = Drop of hammer;
Q = Weight of steel tube;
n = Factor of safety (normally taken to be 6), and
q = Penetration of tube per one last blow (in mm).

Since Frankie Piles involve the forming of a gravel plug and concrete bulb at the base, their founding levels are generally higher than that of the other variations of this pile type, such as the Vibro Piles as discussed in the following section.

Vibro Pile

Construction of the Vibro Piles involves driving a steel tube with a cast-iron shoe of slightly larger diameter to the desired depth. The tube is then filled with concrete and is connected by extracting links to the hammer, which makes alternative upward and downward blows on the tube. On the upward blows, the tube is extracted for a short distance and the concrete therefore slumps outwards against the walls of the hole. On the downward blows, the mass of concrete contained in the tube moves downwards, thus compacting the concrete below the tube and forcing it into contact with the soil. The extracting and tamping blows are performed in rapid successions in aim of keeping the concrete "alive", thus preventing it from rising with the tube during the upward blows. Upon completion, the cast-iron shoe is left in place.

In calculating the required set for this pile type, the Hiley Formula as presented below is used:

$$R_u = \frac{NWH}{S + \frac{1}{2}(C_1 + C_2 + C_3)} \quad (2)$$

where R_u = Ultimate bearing capacity of pile = 2 x Allowable capacity (Factor of safety = 2);
W = Weight of hammer;
H = Effective drop of hammer;
N = Efficiency of hammer blow = $\frac{W + Pe^2}{W + P}$
P = Weight of piling tube, cap and shoe;
e = Coefficient of restitution;
S = Set per blow;
C_1 = Temporary compression in the cap of pile and the driving head;
C_2 = Temporary compression in the pile, and
C_3 = Temporary compression in the ground.

In summary, Driven Cast-In-Place Piles were popular during the 1960s and 1970s due to their main advantage that they are low in cost. However, limitations such as low in bearing capacity, not being able to penetrate boulders and being large displacement piles thus causing high levels of vibration have caused them to phase out in the mid-1980s.

3 DRIVEN STEEL H-PILE

In the late 1970s, Driven Steel H-Piles started to gain popularity for their higher capacities and being low displacement piles. The Grade 50B H-sections in compliance with BS4360, possessing yield strengths between 345MPa and 355MPa, were most commonly used. Two popular sizes, namely 305 x 305 x 110kg/m and 368 x 368 x 174kg/m, possess maximum compressive strengths of 1491kN and 2298kN respectively. Such capacities follow the design principle of 0.3 x yield strength (Ys) (Clause 7.4.6.3.1 of C.P. 2004:1972).

Empirically, the desired depth of steel H-piles is usually at a saprolite stratum with SPT-N = 200 blows.

In the mid-1980s, Grade 55C steel was introduced in Hong Kong. Such materials possess yield strengths in range of 415MPa to 450MPa depending on their thickness. This brings an advantage in strength of more than 20% over the Grade 50B materials. Approval to the use of Grade 55C steel piles was first given in 1984 for a development project in Des Voeus Road Central. It may also be worth to note that BS4360 is due to be phased out upon the fully implementation of the Code of Practice for the Structural Use of Steel 2005 on 1 September 2007, and the Grade 55C steel to BS4360 should then be replaced by Grade S460 steel to BS EN 10025 : 2004, or equivalent, in accordance with Clause 3.1.1 of the new steel code.

A number of commonly used sizes and their capacities are listed in Table 1.

Table 1. Size and capacity of commonly adopted steel H-piles in Hong Kong.

H-Section	Maximum Thickness	Allowable Load
305 x 305 x 186kg/m G55C	25.6mm	2951 kN
305 x 305 x 180kg/m G55C	24.8mm	2958 kN
368 x 368 x 223kg/m G55C	30.5mm	3548 kN

305 x 305 x 186kg/m piles were one of the commonly used sizes. As can be seen from the above table, although being slightly larger than the later available 305 x 305 x 180kg/m piles, possess lower strength. This is due to the fact that the thickness of the latter is below 25mm, thus as stipulated in BS5950, the yield strength of which does not require further reduction. This is the reason why the 305 x 305 x 186kg/m piles have been generally replaced by the 305 x 305 x 180kg/m piles.

Due to a higher carbon content compared to Grade 50B, Grade 55C steel piles introduced welding problems when they were first used. To get over this problem, initially pre-heating was required prior to welding.

Before the introduction of steel H-piles, it was the general assumption in foundation designs that piles possess no bending capacities and are only subject to axial loads. Lateral resistance of the foundation systems were gained solely from the passive resistance on the vertical faces of the pile caps. In cases where high lateral resistance is required, extension of the pile caps around the sides, or so-called drag walls, would be designed.

In contrary, steel H-piles have the advantage of taking bending moments. The maximum allowable combined stress due to axial load and bending moment to be taken by steel H-piles is not to exceed 0.5 x yield strength (Ys). Considering the axial capacity is based on 0.3 x Ys, it follows that driven H-piles have the minimum capacity of 0.2 x Ys to resist solely the bending

moments induced by lateral loads. This attribute, together with being small displacement piles, have given driven steel H-piles great advantages over driven cast-in-place piles.

4 PRECAST PRESTRESSED SPUN CONCRETE PILE

Precast Prestressed Spun Concrete Pile was first used in Hong Kong in 1980 at a site in Wan Chai. The reason for such piles to be prestressed is to enhance their bending capacity to facilitate lifting and handling.

These piles are made of specially mixed concrete with compressive strength of more than 78.48MPa (800 kgf/cm2). Such high strength is obtained by compacting the concrete under centrifugal force thus achieving a lower water cement ratio, as well as by curing using a two-stage steaming method called autoclave. Each pile is made of individual precast pile sections that are normally 12m in length. Connection is made by welding at the steel joint plates at the ends of each section. Steel conical pile shoes are usually installed at pile ends.

Common sizes of this pile type include 500mm diameter with 100mm thick walls and 500mm diameter with 125mm thick walls. The maximum compressive capacities of the two types are 2300kN and 2700kN respectively. With similar capacities to 368 x 368 x 174kg/m Grade 50B driven H-piles, this pile type had the competitive edge over steel H-piles in terms of price.

In view of the large displacement of this pile type, the pile driving sequence should be carefully arranged. Normally, driving is commenced for piles in the central portion of the site, and progressed in an outward direction towards piles in the peripheral areas of the site.

Limitation of this pile type is that they are not suitable for sites where underground boulders and other obstructions exist, since the piles may crack under excessive hard driving.

5 DRIVING EQUIPMENT AND STATUTORY CONTROL ON DRIVEN PILES

The installation of driven piles involves driving using typically diesel, hydraulic and drop hammers. Such machineries induce vast amount of nuisance in terms of noise and air pollution. To minimise such adverse effects to the public, the Government drafted and first implemented the Noise Control Ordinance in 1989.

The ordinance stipulates that percussive piling is prohibited between 7 p.m. and 7 a.m. and on holidays unless specifically exempted by an order. Percussive piling during the daytime may only be carried out in accordance with a Construction Noise Permit, normally only 3 hours per day is permitted in residential areas.

To further tighten the control, the use of noisy diesel, pneumatic and steam hammers was banned from 1 October 1999 through the Technical Memorandum on Noise from Percussive Piling. Exceptions are for use in the Northwest New Territories and Ma On Shan areas, or specifically Scheduled Area Nos. 2 and 4.

Advantageous over the driving construction techniques in terms of nuisance, Jacked Piles were introduced in the early 2000s. This construction technique involves sinking of steel H-piles using hydraulic jacks instead of driving, thus much less nuisance would be created. However, in general the final set of the piles still requires the use of hammers.

One of the controls for driven piles is the requirement of driving tests. The purpose of the tests is to determine the length of pile and give a guide for future driving. Therefore the test should be carried out on the first piles. Test piles should generally be driven at a location near a borehole where the soil characteristics are known and are representative of the ground conditions of the site. A penetration record should be kept for every 0.5m of penetration for the entire length of pile.

Another statutory control on driven piles is the requirement of proof tests by imposition of test loads. The current required quantity of this test is 1 test per 100 numbers of working piles.

Other statutory controls on driven piles include the followings:

(i) The minimum pile spacing should be at least 1-pile perimeter, and
(ii) A group factor should be applied on the capacities of piles driven in close spacing.

6 HAND-DUG CAISSON

Hand-dug caissons were first introduced in the 1960s. The typical diameter of this pile type ranges from 1m to 3m.

Construction of this pile type involves hand excavation. A circular shaft of approximately 900mm deep will first be excavated. The vertical face of the shaft is temporarily supported by the arching action of soils. A circular steel mould with a smaller diameter is then erected within the shaft and concrete is cast between the steel mould and soil to form a concrete ring, or so-called caisson ring. The typical ring thickness is between 100mm and 150mm. The above procedures can usually be completed within one day. On the next day when the caisson ring is set, the same procedures are repeated further down to form another ring. The shaft continues to progress downwards at a rate of about 1m per day until the desired founding level is reached.

In order to optimise the load capacity of piles, often a “bell-out” is formed close to the pile founding level. It is simply an expansion of the pile shaft diameter towards the base such that the bearing area is increased. When caisson foundation was first used in Hong Kong, there were no restrictions on the size of bell-outs, and was only limited by practicability.

For caissons founding on soil, the desired founding level is at a stratum with SPT-N value of approximately 240, the allowable bearing stress on which is 1200 kPa. For caissons founding on Grade III or better rock, the allowable bearing stress is 5000 kPa.

In order to obtain a broader picture on the depths of possible founding strata, predrill holes were often sunk close to the pile locations. In recent years, it has even become mandatory to carry out predrilling for each working pile.

After the caisson is formed, the Engineers may enter the shaft to check its integrity and the compliance of the founding materials. Pneumatic drilling tests using hand held pneumatic hammers may also be carried out. The rate of penetrating 4.5m below the founding level is recorded. By proper calibration, the integrity of the rock stratum can be revealed.

After the integrity and the founding materials are checked, concrete is poured from the base of the caisson such that the inner portion, or so-called the hearting of the pile is formed.

It is stipulated in the Buildings (Construction) Regulations that a 0.8 factor should be applied to the concrete strength if casting is carried out under water. However, this rule is often not applied in the design of caisson foundations because even if the piles are founded below the groundwater table, dewatering will always be

carried out during the excavation of the shaft and before concreting.

In cases where there is a high level difference between the groundwater level and the pile founding level, pre-grouting at typically 1m c/c spacing may be carried out around the caisson to reduce the permeability of the ground, thus minimising the drawdown of groundwater. In some cases, pumping tests may also be required to demonstrate the effectiveness of the grouting.

The capacity of caisson foundations is controlled by both the ground bearing capacity and the structural strength of the caisson shaft. In determining the shaft strength, normally the caisson rings are not considered. However for the sake of cost effectiveness, some designers may regard the rings as part of the pile shaft. In such cases, the quality of the concrete that formed the rings requires stricter control.

Other design assumptions for caisson foundations also evolved over the decades. In the early years, caissons were designed to have no reinforcements but only short dowel bars at the pile heads for connection purposes. Such piles were assumed to take axial loads only. As mentioned in a preceding section, the lateral resistance of such foundation systems was gained from the passive soil resistance on the vertical faces of the pile caps or extended drag walls. It was later evolved that reinforcements are provided for the full length of the piles, and the design of the piles follow that of an RC structural column, which may also resist bending moments and shear forces.

Hand-dug caissons have a number of advantages over other pile types. Perhaps the most obvious is their sizes and therefore their enormous capacities. Caissons are suitable for sites on hilly terrains or with bouldery grounds, as excavation using hand-dug method would impose minimal stability concerns to nearby slopes or sensitive structures. They are suitable for sites with difficult access or insufficient working areas, as they virtually require no mechanical plants at all during excavation. Their large capacities also render them suitable for top-down constructions.

Apart from foundation works, hand-dug caissons can also be used in site formation works as cantilevered retaining walls, or so-called caisson walls. In some cases, designers have combined the two functions of hand-dug caissons as both retaining structures and piles, and take advantage of the benefit of a good combination of bending moment and axial load, as is often practiced in the design of structural columns.

Hand-dug caissons also have a number of limitations. Dewatering is required if groundwater level is above the base of pile. If peripheral pre-grouting is not carried out, settlements may occur on nearby grounds. For the same reason, caissons are not suitable for coastal reclamation sites. They are also not suitable for sites underlain by cavernous marble.

Amongst others, perhaps the worst disadvantage of hand-dug caissons is that they introduce enormous health and safety hazards to the labours working within them. Some accidents occurred in the past include dropping of objects into the caisson and sudden geotechnical failures within caissons such as piping or heaving. Furthermore, many caisson workers face long-term health problems such as pneumoconiosis caused by working under high concentration of silica dusts from the drilling and hammering within the confined caissons.

Because of the adverse health effects brought by the construction of hand-dug caissons, the Government legislated under the Buildings Ordinance Section 16(1A) in 1995, that the Building Authority is empowered to refuse giving approval to any plans of building works that include hand-dug caissons. Exemption may only be granted if:

(i) The depth of the hand-dug caisson does not exceed 3m and the diameter of the inscribed circle of the hand-dug caisson is not less than 1.5m; or
(ii) The use of a hand-dug caisson is the only practical construction method or there is no other safe engineering alternative.

7 LARGE DIAMETER BORED PILE

Bored Piles were first introduced to Hong Kong in the 1970s. The typical diameter of this pile type ranges from 1m to 3m. The construction of bored piles involves firstly oscillating a steel casing into ground. Soil material within the casing is usually extracted using large grabs operated by cranes. The purpose of the casing is mainly to avoid collapse of the bored hole. Sinking of the steel casing and extraction of soil within the casing progress alternatively until harder material is reached where the casing can no longer penetrate. To further extend the bored hole, chisel is deployed to crush the hard materials until the desired founding material is reached. The pile founding level may often be a few metres below the rockhead to form a “rock socket”. Bell-out can also be formed at the founding level such that a larger bearing area is achieved. One drawback of the above construction method is the high level of vibration created during chiselling, which may bring adverse effects to surrounding sensitive structures.

In 1976, Reversed Circulation Drilling (RCD) was introduced to Hong Kong by Mitsuri. Kowloon Bay MTR station (Telford Garden) was the first site where this technology was adopted. The advantage of this technology is that it uses the self-weight and rotary motion of the drill to crush hard materials. Crushed materials are then retrieved by circulation of fluids within the drillhole. The vibration and noise induced under this method is much less than that from chiselling.

Unlike the hand-dug caissons, construction of bored piles normally does not involve dewatering within the shaft. In order to check the integrity of the completed bored hole and bell-out, divers were employed in the earlier times. Nowadays, ultrasonic tests such as the KODEN test are usually used to perform the same task.

Same as for hand-dug caissons, predrilling for every working bored pile has become mandatory in recent years. Before then, evaluation of the founding materials relied heavily on GI data and observation of the cuttings extracted during RCD excavation or chiselling. A heavy plump bob tied to a scale or a piece of string would also be dropped around the perimeter of the completed bored hole to ensure the founding material had certain hardness. However, this exercise of course would not provide any accurate information.

After checking the integrity of the completed bored hole, concrete is injected into the bored hole using a concreting pipe that extends to the base of the bored hole, or so-called by tremie method. The steel casing is usually extracted along with concreting until the shaft is formed.

A variation of this pile type is the barrettes, which adopt the same construction method as for diaphragm walls. Instead of using a temporary steel casing, a trench is excavated into ground using crane-operated grabs under bentonite slurry. Owing to this construction method, barrettes are usually rectangular in cross-section. After the excavation is completed and reinforcement cages installed, concrete is cast by tremie method. The bentonite slurry is displaced upwards during concreting until fresh concrete

is seen at the cut-off level. Since no dewatering is required during their construction, barrettes are suitable for sites where sensitive structures that are prone to settlements caused by dewatering exist. However, barrettes have the limitation that relatively large site area is required for the placement of bentonite silos.

The bearing capacities of bored piles depend on the nature of the founding stratum. As stipulated in the Code of Practice for Foundations published by the BD in 2004, the presumed allowable bearing capacity on rock with weathering grade of Grade III or better and with total core recovery (TCR) of higher than 85% is 5000kPa. For piles founding on Grade II rock with TCR greater than 95% and Grade I rock with TCR of 100%, the presumed allowable bearing capacities are 7500kPa and 10000kPa respectively. For uplift resistance, the presumed frictional bond between the pile and rock within the rock socket is 350kPa and 700kPa under permanent and transient loads respectively.

Before the implementation of this Code of Practice, the capacity of the bored piles under compression may solely be gained from end bearing on rock. The frictional resistance between the pile and rock within the rock socket may not be combined with end bearing, and can only be used to resist uplifting loads. It is now provided in the Code of Practice that the two resistances may be combined, with the restriction that the rock socket length considered for this purpose may not be longer than two times the pile shaft diameter. The presumed allowable frictional bond between the pile and rock for this purpose is 700kPa.

Other design principles for bored piles also evolved over the years. Similar to hand-dug caissons, when bored pile was first introduced, no steel reinforcement was used. The piles were only to resist axial compressive loads gained from the strength of the concrete shaft. Steel reinforcements were later also used to form the pile shaft, providing it with tensile, bending and shear resistances. Bored piles and barrette piles founded in soil have also evolved in the 1980s. Shaft grouting technique to enhance the shaft frictional resistance was introduced in 1997.

Statutory controls for bored piles include the adoption of a 0.8 reduction factor on the concrete cube strength for piles founding below groundwater level. As stipulated in the Building (Construction) Regulations, this is to take into account the effect of casting concrete below water. Core-drilling at the concrete/rock interface is also required for each of the completed piles as set out in PNAP 66. The core-drilling should be carried down to at least 1m below the interface.

The major advantage of bored piles over hand-dug caissons is that they post far less health and safety hazards to the workers, since all operations are carried out by mechanical plants. However for the same reason, they are considerably more costly. Another limitation of bored piles is the requirement of a larger working area to cater for the mechanical plants.

8 MINIPILE

The materials used in constructing minipiles consist of steel casings, steel reinforcement bars and cement grout. As suggested by their name, minipiles are small in size typically between 200mm and 300mm in diameter, and relatively low in capacity.

The construction method of this pile type involves first drilling a hole using ODEX method. This refers to drilling by pneumatic hammering and rotating actions. With sufficient rotating speed, an eccentric arm in the drill bit is flung out to increase the diameter of the drillhole. This allows the steel casing to propagate with the drilling, so as to prevent the drillhole from collapsing. When the rockhead is reached, a socket will be formed using down-the-hole method, with which the steel casing will not further propagate. Upon achieving the desired rock socket length, the drill bit is extracted and steel reinforcement bars bundled together will be installed into the drillhole. Using centralizers, the reinforcement bars are kept at the centre of the circular drillhole to achieve maximum cover. The final step is to fill the entire drillhole from the base with cement grout. The steel casing is left in place to provide extra protection to the pile.

Various sizes of steel casings are available. One of the most common types is the 219mm diameter hollow section. Together with four numbers of T-50 reinforcement bars, the maximum capacity under compressive load is 1374kN, which is based on the design principle that the allowable compressive stress of the steel bars is 175MPa. Neither the steel casing nor the cement grout may be considered to contribute to this capacity.

The capacity of this pile type is also controlled by the frictional bond between the pile and the rock within the rock socket. The allowable bonding stress under compressive loads is 700kPa, and is 350kPa and 700kPa under permanent tensile loads and transient tensile loads respectively. Since the piles are relatively slender, the buckling capacity of the pile under compression may also be controlling.

Minipiles are not designed to resist bending moments in view of their limited bending capacity. Lateral resistance of a pile cap is provided by inclusion of raking piles, where the lateral shear and bending moments exerted on the pile cap are resolved and resisted by the axial capacity of the raking piles.

Similar to hand-dug caissons and bored piles, predrilling for minipiles has also become mandatory in recent years. Predrilling should be carried out at intervals that the tip of every pile should be within 5m horizontal distance from a predrilling hole. The depth of predrilling should be at least 5m into the category of rock specified for forming the rock sockets, or the designed rock socket length of the nearest pile, whichever is the deeper.

Another statutory control for minipiles is the requirement of post construction proof drilling. Proof drilling should be sunk at least 5m below the as-built top level of the rock socket, or the as-built rock socket length of the nearest pile, whichever is the deeper. The current required quantity of proof drilling is 2 proof drills for sites with 100 or less piles, and 1% of the total number of working piles for sites with more than 100 piles.

The advantage of minipiles is that they only require relatively small mechanical plants. As such, they are suitable for sites in hilly terrains or sites with accessibility difficulties. However, due to their relatively low capacities, minipiles are normally not sufficient to support tall buildings. Because of these characteristics, minipiles are popular for low-rise developments in hilly terrains.

9 STEEL PILE IN PRE-BORED HOLE – SOCKETED STEEL H-PILE

Socketed Steel H-Pile was introduced in Hong Kong in the 1970s. This pile type is very similar to minipiles in terms of both materials and the construction method used. Instead of using bundled reinforcement bars, socket steel H-piles use steel H-sections as the main structural member, from which the pile capacities are gained.

The construction of this pile type also involves adopting the ODEX and down-the-hole methods as mentioned in the previous section to form a desired rock socket. After forming the drillhole, a steel H-section is lowered, and the drillhole is backfilled with cement grout. The major difference from minipiles is that the steel casing is normally extracted along with grouting.

Different sizes of steel casings and steel sections are available in the market for this pile type. One of the most common steel casing sizes is the 610mm diameter hollow section, which is sufficient to cater all sizes of the H-sections from the 305mm and 356mm series with adequate cover.

The major difference in terms of design principles between minipiles and socketed H-piles is that socketed H-piles can resist bending moments. The bending capacity is gained from the steel H-sections. The allowable axial working stress or the combined axial and flexural stresses should not be greater than 50% of the yield stress of the steel H-section. This provides this pile type a 20% advantage in terms of pure compressive capacity over driven H-piles, the allowable compressive stress of which, as previously mentioned, is 30% of the yield strength of the section.

Other statutory controls on socketed H-piles such as predrilling and proof drillings are the same as those imposed on minipiles.

10 PAKT-IN-PLACE (PIP) PILE

Pakt-In-Place piles were first approved by BD in the early 1960s. Construction of this pile type involves drilling using auger method. Upon the desired depth is reached, cement grout is injected into the drillhole through a nozzle at the tip of the drilling rod. Along with the grouting, the drilling rod is rotated in the reversed direction and therefore gradually retrieves from the drillhole. Soil trapped between the auger flights of the drilling rod is extracted along with the drilling rod.

One of the most common pile sizes of PIP piles is 610mm in diameter, with a compressive capacity of 1461kN, which is derived from the design principle that the allowable compressive stress on the pile cross-section is 5MPa.

The capacity of the pile is also controlled by the ending bearing and friction between the pile and soil. The allowable shaft frictional stress is 5 x N / 3 kPa, while the allowable end bearing stress is 5 x N kPa, where N refers to SPT-N value.

The major advantage of this pile type is that its construction method does not involve the use of pneumatic tools and hence is virtually vibration free. It is therefore suitable for sites in urban areas where sensitive structures exist in close proximity. However, the limitation of this pile type is that it is very ineffective and costly in overcoming underground obstructions and boulders. Furthermore, it relies heavily on GI data, as its capacity is determined solely from SPT results.

11 CONCLUSION

Since the economic and population boom in the 1970s, the size and height of private developments in Hong Kong have rapidly increased, and buildings in excess of 30 storeys have now been commonplace. Over the past three decades, numerous types of pile foundations have been developed and evolved to cater for the increasingly taller buildings. This paper recaps the history and technical background of eight distinctive pile foundation types, each of them had once gained popularity in Hong Kong. Amongst them, some had been phased out due to various reasons, and some are still in use to date.

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Three Decades of Foundation Development

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Abstract : The foundation industry in Hong Kong is highly regulated. One could argue that this is necessary considering the astonishing pace of our building industry. However, one important observation that has become apparent over the year is lack of advancement. This paper gives an overview of the foundation development in the last three decades with emphasis on deep foundations. Some issues in the design and construction practices are highlighted and discussed in appraising the degree of advancement. Finally, suggestions are put forward on the discussed issues with an intent to attract more fruitful discussions and debates within the industry for further advancement.

1 INTRODUCTION

Some degree of development of the design and construction practices of foundations was noted following the publication of the Code of Practice for Foundations (CoPF) by the Buildings Department in 2004 (BD 2004) and a new revision of Pile Design and Construction (GEO Publication No. 1/2006) by the Geotechnical Engineering Office (GEO 2006) of the Civil Engineering & Development Department in 2006. These documents take into account the latest testing data and other advances of design and practices into consideration.

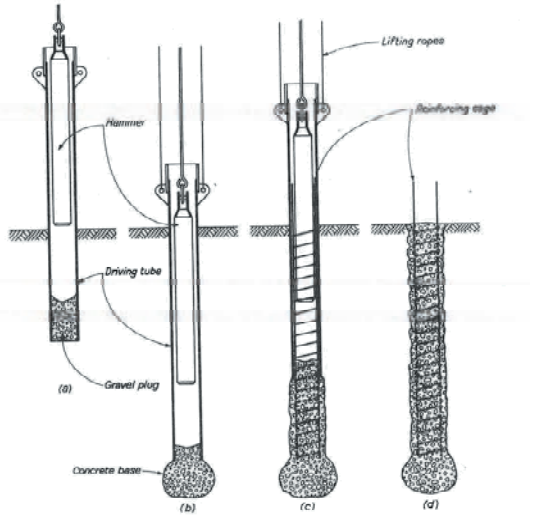
Nevertheless, looking back 30 years, it appears that although the construction techniques improved remarkably in achieving deeper and bigger piles, the design practices have advanced very little. The uses of simple, albeit conservative, empirical approaches are still the norm in routine designs.

This paper provides a review of the development of foundation practices since 1970s with emphasis on deep foundations. In the process, some key issues that may or may not have been brought up but other writers previously are discussed, such as bearing stress, pile driving formula, pile-raft foundation system, static load test as well as loading test criteria. The authors also give some suggestions to the further development of the foundation industry, with an aim to promote more rational approaches.

2 SEVENTIES: THE STARTING POINT

A snapshot of the state of the foundations industry more than twenty years ago may be found in the Symposium on Piling (1981). Timber piles started to fade out in early 70s and a wide variety of proprietary foundation systems have been very common at the time, these included Franki piles, Vibro piles, Pressure piles and Pakt-in-Place (PIP) piles.

The former two types of piles are usually formed by driving a temporary casing, usually 600 mm in diameter and backfilling with plain concrete and extracting the temporary casing in stages during concreting, see Figs. 1 and 2. The top few metres of length, usually 6 m or so will be placed with dowel bars or starter bars to provide connections with the pile cap. Pile lengths are usually determined by driving the casing to refusal which is normally corresponding to SPT 'N' values of about 40 to 60 blows per foot. Pile lengths are usually less than 20 m. Pile lengths are usually determined to match the structural capacity of the pile and trial pile loading tests are carried out to confirm the design before working pile installation.



a. Driving piling tube
b. placing concrete in pile tube
c. compacting concrete in shaft
d. completed pile

Fig. 1. Construction technique of Franki piles.

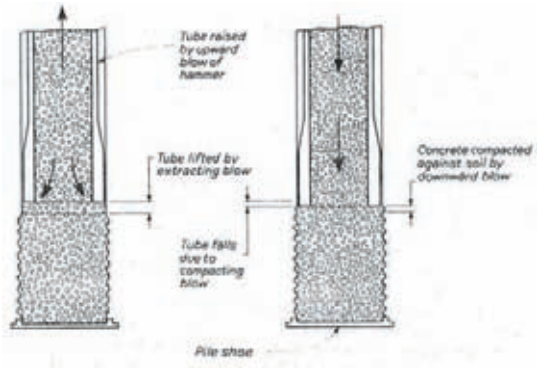


Fig. 2. Compaction of Vibro piles.

Pressure pile and PIP piles, see Figs. 3 & 4, are either excavated by auger or boring drop tubes in a diameter of about 500 mm. Boring is usually stopped at soil layers with SPT 'N' values of about 40 to 50 blows per foot at lengths of normally at about 15 to 20 m. The holes are backfilled with either cement sand mixture or grade 25 concrete under pressure. The pressure is applied by either an elevated head of about 15 to 18m for cement sand grout or by air pressure through a pressure cap screwed onto the top of tube during concreting. This process is

aimed to enhance the shaft friction by pressure grouting or concreting. Again starter bars are placed at the top few metres, say not exceeding 6m in length to connect to the pile cap. The working capacity is about 1000 to 1500 kN per pile. Out of these four pile types, only PIP piles are still in use nowadays.

Fig. 3. Pressure piling in congested site

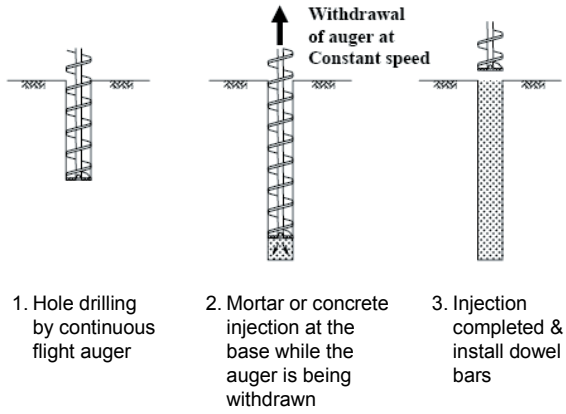


Fig. 4. Construction method of PIP piles

Other typical piling systems at the time also include bored piles (diameters up to 1 or 1.5 m), hand-dug caissons and driven concrete piles. The former two types were either founded into HDG or bedrock whereas the termination of the driven piles was based on dynamic formula. These foundation systems were used with special attention to the availability of plants and particular requirements on the structures at the time.

During the Seventies in Hong Kong, the foundation design other than shallow footings has always been relied on the piling contractors, who were normally responsible for the design and construction of the foundations up to the pile cap level. The architects and structural engineers were rarely concerned on the foundations and quite often foundation works were awarded to the lowest tender including the cost for pile caps.

3 EIGHTIES

Upon entry to early eighties, massive implementation on infrastructures including highways, port works, MTR, LRT and public and private buildings in urban and new town developments had demanded for substantial foundation construction.

The use of conventional foundation systems including bored piles, caissons (see Fig. 5) and driven piles designed by the engineer increased extensively. Due to the large volume of works, the demand for more economical foundations using improved design requirements emerged as a challenge to geotechnical engineers at the time. A lot of research works, sophisticated ground investigation techniques and instrumented pile loading tests were used to obtain enhanced design principles including higher presumptive allowable bearing pressures on soils and rocks, combination of shaft friction and end bearing capacity for piles and caissons and lateral loading capacity of piles, such as Fraser & Kwok (1986), Powell *et al.* (1988) and Lam *et al.* (1991). It was recognized that obtaining approval from the Building Authority for designs of these piling systems was not an easy matter at the time, but some of the projects were successful in designing piles with enhanced principles as compared with the statutory requirements.

Fig. 5. Hand-dug caisson construction

The improvement on the construction techniques, plants and experience gained had resulted in the introduction of new piling systems. In some cases, difficult foundations which were resulted from using bigger pile/caisson diameters, longer lengths and penetrating through more complex ground conditions without adequate previous knowledge. A new piling system using precast prestressed concrete hollow piles (commonly called Daido piles) with or without surrounding steel casing was introduced and were extensively used for civil and public housing block projects as shown in Fig. 6. There were a number of studies on the performance and quality control on Daido piles conducted at the time, such as Lee (1983), Evan (1987) and McNicholl (1989).

Fig. 6. Daido pile installation

Minipiles, as shown in Fig. 7, were also introduced to Hong Kong at the time and were successfully used for low rise structures and basements for particular purposes (Bruce & Yeung, 1984).

Fig. 7. Minipiles construction adjoining existing building

Driven piles were pitched using diesel hammers, as illustrated in Fig. 8, and final set was achieved using drop hammers. The works on understanding the dynamic interaction between hammer, capping and cushion materials and ground conditions using stress wave theory on pile driving were carried out. Hiley formula has been widely used and the driving performance is confirmed by trial piling using stress wave theory and loading tests although some writers had doubt about its applicability in the ground conditions in Hong Kong (Gammon 1981). Other problems including provisions to deal with corrosion protection measures to H piles, pile shoes to deal with bouldery grounds, piling to refusal or false set and soil/piles heaving have been investigated and satisfactory measures implemented. The requirement of using driven piles in new reclamation had triggered a series of field measurement of driving and loading performances of the piles such as Choi *et al.* (1981) and Permchitt *et al.* (1988) and assessment of the negative skin friction, such as Wong (1981) and Davies & Chan (1981). Different forms of pile shoe strengthening methods have been discussed by Houghton & Wong (1990) to tackle and overcome the pile damage problem under hard driving and an example is shown in Fig. 9.

Fig. 8. Diesel hammer for pile driving work

Fig. 9. Example of pile shoe strengthening work

Bored piles and hand dug caissons were designed as end bearing piles usually on rocks without any contribution to shaft friction. Presumptive allowable bearing pressures stipulated in the Building (Construction) Regulations were followed. For some individual projects, well documented instrumented loading tests on bored piles and caissons were obtained and these test results show that in contrary to the usual assumptions of end bearing, a substantial portion of the load is carried by shaft friction on piles. An attempt was made to correlate the skin friction with the shaft deformation which could provide guidance to increase the pile load carrying capacity by inclusion of shaft friction in addition to end bearing pressure. Although this was not successful in promoting the rational design of piles to include shaft friction, a few projects had since been successful in using higher presumptive end bearing pressures of 7.5 MPa in Grade II rock instead of 5 MPa as stipulated in the code at the time (Thomas 1984).

A variety of problems associated with construction of bored piles and had dug caissons was recognised and these include base heave or softening, collapse of side, presence of loose materials in the base and on the side, settlements associated with loss of ground or construction dewatering. In addition, the integrity of the piles might be affected by concrete aggregate segregation during pouring, the presence of inclusions in the concrete or necking and soft toe on piles or bearing materials inferior to that specified. Verification of founding materials for large diameter piles was discussed by Irfan & Powell (1985). In addition to the full depth coring on selected completed piles, the importance of integrity testing (sonic) of the pile was emphasized (Tijou 1984).

4 NINETIES

Part of the Yuen Long and Ma On Shan areas was underlain by karstic marble defined as Scheduled Areas, as shown in Fig. 10. The marble clasts are generally not interconnected and the dissolution of these marble clasts is variably localized resulting in honeycomb weathering of the rock but sometimes formation of large cavities could also be found. The existence of large cavities encountered may pose geotechnical concerns to the foundation design. Since 1980s and especially in early 1990s, there were demands on the development of high-rise residential buildings in these areas. The planning, design and construction of engineering works for sites in the scheduled areas is subject to geotechnical control as stipulated in the Buildings Ordinance (HKG 1990) and PNAP161 (1993). Chan (1996) and Kwong *et al.* (2000) provided overviews of much of the experience gained in these karstic regions and the regulatory framework which is in place to control development in them.

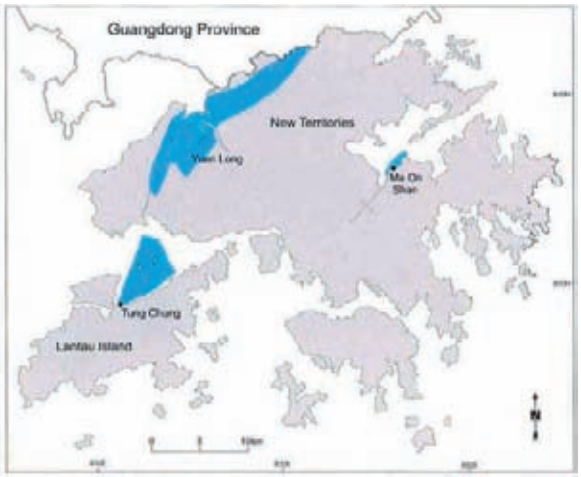


Fig. 10. Areas underlain by karstic marble

In earlier days, the foundation systems in dealing with karst formations was mainly shallow foundations or other alternatives, such as Pakt-in-Place piles (PIP) or prestressed precast tubular concrete piles (Daido) as discussed in Houghton & Wong (1990) and Kwong *et al.* (2000) in order not to overstress the underlying marble and hence limit the chance of causing collapse into the cavities. To achieve a high loading capacity, the foundation systems had been gradually changing to those able to be installed below the cavities onto sound bedrock, such as driven steel H-piles with pre-boring (Blake *et al.*, 2000) or large-diameter bored piles (Lee & Ng, 2004) as illustrated in Fig. 11.

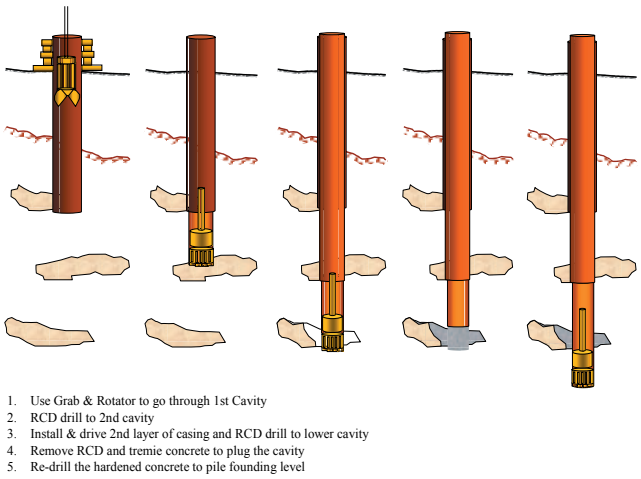


Fig. 11. Construction technique of large diameter bored pile in marble area

The marble class was commonly evaluated on every 5m of rock core by means of the 'Marble Quality Designation' (MQD) index (Chan *et al.*, 1994b) and the site classification was appraised in accordance with GEO Report No. 29 (Chan 1994a). Emphasis is placed on the adequacy of ground investigation to ascertain the extent of cavities within the marble area. A certain degree of redundancy is recommended to be applied to the foundation design in these circumstances. The rationale is that should there be a 'failure' of a local pile, a transfer of the structural loads through the pile cap to adjacent sound piles will occur. The surrounding piles will share this load to ensure that no single pile is over-stressed. There is no fixed rule in determining the redundancy factor although there are some recommendations provided in Chan (1994a).

Hand-dug caissons had been used for many years and have had a disturbing history of high accident rate and health hazards to workers and the use was banned in 1995 following the enactment of the Buildings (Amendment) Ordinance 1995. Nowadays, hand-dug caissons could only be approved under exceptional circumstances, e.g. no viable alternative in terms of foundation constructability. Specific historical information regarding hand-dug caissons is contained in Mak (1993) and Mak *et al.* (1994).

Since 1998, the Government of Hong Kong SAR has been actively phasing out the use of diesel hammers due to the high noise level and pollutant exhaust gases associated with them. They have now been replaced by hydraulic hammers, as illustrated in Fig. 12. Nevertheless, the final set was still done by drop hammer since the conservative parameters adopted in Hiley formula does not match with the highly efficient hydraulic hammer which resulted in either overdriving of the piles or the final set criterion could never been achieved. The comparison of hammer efficiency between a hydraulic hammer and a diesel hammer with the similar theoretical potential energy is shown in Fig. 13.

Fig. 12. Hydraulic hammer for pile driving

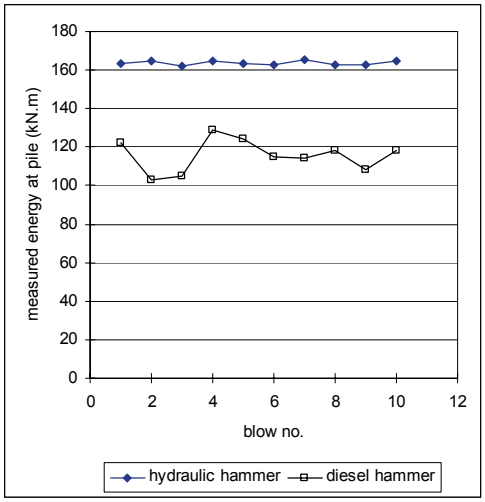


Fig. 13. Comparison of hammer efficiency (with input energy = 220 kNm)

Due to the vast development for airport core projects and other major infrastructures, one major advance in foundation engineering in the 90s was the testing data obtained from the full-scaled tests on end-bearing and rock socketted large diameter bored piles, plain and shaft grouted friction bored piles and barrettes (HKIE 2000). The most comprehensive tests being the load testing programme carried out for KCRC West Rail Phase I project in the mid 90s with an aim to achieve economic and rational design by justifying higher end-bearing capacity in different grades of rocks, combined use of rock socket friction and end bearing as well as introduction of shaft grouted technique to Hong Kong in enhancing shaft capacity of friction piles. These had resulted in project specific rational design of pile foundations and allowed 50% increase in presumptive end-bearing stress onto different grade of rock and the applicability of shaft grouted barrette in the ground conditions of Hong Kong. As part of the testing programme, bored piles were also laterally tested to investigate the response of the pile foundation in resisting horizontal forces, it was concluded that the conventional assumption on the soil stiffness and the application of conventional group reduction factor on the lateral soil subgrade was over-conservative (Plumbridge *et al.*, 2000).

The foundation industry was in a turmoil time after the short pile scandals discovered in 1997 which had resulted in a large extent of remedial works/piles to be carried out and in an extreme case, some newly completed housing blocks were demolished. The impact of these incidents to the industry was that it had triggered a series of changed on the construction and supervision practices on foundation works. These included the tightened quality site supervision of foundation works (PNAP242 2000), ban of forming bell-out of bored piles using chisel and replace by reverse circulation drill (RCD), as shown in Fig. 14. It is also mandatory to carry out interface coring on end-bearing piles. The use of divers in verifying the dimensions of bell-outs were no longer permitted and replaced by echo-probing test (commonly called Koden test, see Fig. 15) or other methods.

Fig. 14. RCD bell-out drillbit

Fig. 15. Koden test to check pile verticality and shape of bell-out

In view of the scandal of short Daido piles in some public housing estates and doubtful integrity during hard-driving, the clients and government authorities were hesitated in using this type of piles. Since late 90s, although it was not formally banned it was seldomly used for medium or high rise developments.

5 WHERE ARE WE NOW AND WHAT DO WE NEED?

The following are quotations from two publications:-

Faber (1981):

"The design of piled foundations has hardly developed at all in the last 20 years in Hong Kong in spite of the large volume of practical experience and research from other places which is evident from the extent of the literature."

Li *et al.* (2000):

"There has been relatively little change in the design practice of deep foundations in Hong Kong since the early 80's, although significant advancement has been made in the construction techniques".

It can be seen that the foundation design practices in Hong Kong have not really advanced significantly as compared to other major industries in the last five decades. So where are we now and what need to be further developed?

5.1 New government publications

In recent years, two new government publications have been released, namely the Code of Practice for Foundations (BD 2004) and the GEO Publication No. 1/2006-Foundation Design and Construction (GEO 2006).

The CoPF consolidates the practice commonly adopted in Hong Kong and supersedes the design requirements stipulated in the various practice notes. Foundations designed in accordance with the code are deemed to satisfy the Buildings Ordinance and related regulations. Some of the rational approaches in foundation design have been explicitly addressed in the code such as the alternative approach for design of negative skin friction with separate consideration on structural and geotechnical pile capacities and the combination of rock socket friction and end bearing capacity for rock socketed piles.

GEO Publication No. 1/2006 provides an update of the experience gained and improvement made in the practice of foundation design and construction over the last 10 years in the territory as well as around the world. The publication provided a wide spectrum of technical considerations to various issues related to pile design and construction.

5.2 Bearing stresses

Apart from formalization of the design values adopted in common practices, CoPF introduced a new category of founding materials and denoted as Category 1(a). This category has a presumptive allowable bearing stress of 10MPa and requirements are Grade I rock with 100% total core recovery and no weathered joints and minimum uniaxial compressive strength of rock material (UCS) not less than 75MPa. The same bearing stress had been proposed three decades ago for the development of the Castle Peak Power Station but was considered conservative when compared to Buildings Codes elsewhere at that time (Thomas 1984). Lee *et al.* (2005) had pointed out that it is of high risk to the client, designer as well as the contractor to design for foundations to comply with the requirements for this category of founding material. It can only be reasonably used as contractor's alternative once all the predrilling for each individual pile has been satisfactorily carried out. However, with the demand of fast track construction that is typical in Hong Kong, the necessity of the late change in design would render this alternative option unattractive.

Higher bearing stress has been adopted but only on isolated cases, such as Arup (1999). However, in practice, the justification of the "rational" design to the authorities' satisfaction still results in a long process and is so expensive (usually involving the performance of preliminary full scale pile tests at every site where the design method is to be adopted) that the benefits of less conservative design usually outweighs by the time and cost to justify it which becomes unattractive to most private developments.

5.3 Combination of rock socket friction and end bearing capacity

CoPF has allowed the use of combined rock socket friction with end-bearing capacity for piles formed in Category 1(c) or better rocks with a limitation on the socket length cannot exceed twice the socket diameter.

The capacities of rock sockets in relation to the rock strength were documented comprehensively by Hill *et al.* (2000). They reported several full-scale loading tests for the West Rail project in which Osterberg cells were used at the base of the pile. In this form of testing, the stresses developed in the pile base and the rock socket can be acquired separately. From these tests, the ductile behaviour of the rock socket was illustrated. Similar strain-hardening behaviour was also reported by Fraser & Ng (1990) for bored piles in granitic rock and Zhan & Yin (2000) for bored piles socketed in volcanic rocks. Ng *et al.* (2001) reviewed from various publications the pile loading tests conducted in bored piles socketed in rock and came to similar conclusions. Such behaviour is important in allowing the mobilisation of shaft resistance in carrying foundation loads together with the end bearing resistance.

The local experience indicated that shaft resistance could be mobilised in rock sockets longer than three times the pile diameter (maximum ratio tested so far is 2.92). In order to provide an effective alternative to practitioners to opt for the use of combined socket and end bearing, instead of the use of bell-out, a further relaxation of the allowable ratio of socket length to pile diameter is considered justified, based on local test data.

5.4 Use of Shaft-grouting

The concept of post construction shaft grouting to enhance frictional capacity at the soil and pile interface for small replacement piles was first introduced to Hong Kong in early 1990s with the first case of application being used for minipiles embedded in completely decomposed granite (CDG) as reported by Lui *et al.* (1993). After that, full-scale instrumented trial shaft-grouted bored pile and barrettes were carried out under the KCRC West Rail Phase I project in 1997 and 1998 as reported by Plumbridge *et al.* (2000). It was evident that the ultimate shaft friction capacities measured from the test piles in CDG, completely decomposed rhyolite and metasiltstone could achieve up to 2 to 3 times the capacity compared to those without shaft grouting and concluded that the shaft grouting technique is more controllable in friction barrette construction. The installation of shaft grout tubes with a steel cage is shown in Fig. 16 and a coring through the vertical interface giving an overview of the grout intrusion is shown in Fig. 17.

Fig. 16. Shaft grout tubes and steel cage

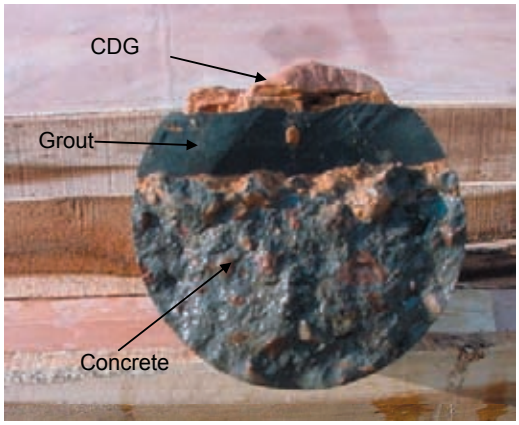


Fig. 17. Coring along soil/pile vertical interface

Since then shaft-grouted friction barrette, upto an allowable capacity of 40 MN, had been adopted for a number of major infrastructure and private projects notably KCRC West Rail Nam Cheong Station and Mass Transit Railway Corporation (MTRC) Kowloon Station Mega Tower Project (Chan *et al.*, 2004b). The use of shaft grouting technique has been gradually getting accepted and there are more reported case histories such as those reported by Chan *et al.* (2004a) and Sze *et al.* (2007).

5.5 *Use of base grouting*

There are many case histories of using base grouting technique to improve the load carrying capacity characteristic of frictional replacement piles elsewhere in the world as well as in mainland China. The operation is similar to shaft grouting except that the tube-a-manchette grout pipes are installed at the pile base. The grouting action can compact loose materials deposited at the pile base combined with possible rises along the shaft (Teparaksa *et al.*, 1999). The applicability of this technique in replacement piles in Hong Kong ground conditions shall be further explored.

On the other hand, the industry had been suffered from problem of segregation and soil inclusion at the pile base of large diameter bored pile even with improved concreting procedure. The remedial works usually comprise drilling, high pressure flushing and followed by pressure grouting and core testing. In long piles, these proved to be difficult and unnecessary time consuming and hence costly. With the tube-a-manchette grout pipes pre-installed at the pile base with the steel cage, base grouting technique appear to be a feasible contingent measure for carrying out such remedial works in quicker and easier manner.

5.6 *Alternative approach for negative skin friction*

Piles installed through compressible materials can experience negative skin friction along the pile shaft due to the downward movement of the surrounding soils resulting from the consolidation of the soft deposits. There are various simplified approaches to estimate the negative skin friction (NSF). It is also a customary to check the NSF on the structural and geotechnical capacities of the pile foundations. An alternative approach was therefore suggested in the CoPF to check NSF for structural integrity only since it is considered that when geotechnical failure occurs, the pile will settle accordingly and the relative displacement at the pile-soil interface will eliminate the effect of NSF on the pile shaft. It is also explicitly stated in the code that the test load for static load testing of the piles shall be twice the applied load plus NSF, which implies that the NSF is considered as ultimate load.

Nevertheless, the practice is still not yet standardized amongst different government authorities and the conventional practice is still in use, e.g. ArchSD (2003), in which NSF as considered as part of design load and hence a factor of safety has to be applied which is illogical as the NSF is often estimated from the ultimate shaft resistance using the simplified approaches as discussed by Li *et al.* (2000).

5.7 *Jack-in pile*

Jack-in pile is a new type of pile system in Hong Kong used over the past few years. The system has been adopted in a limited number of projects in Hong Kong, all involving in steel H-piles. The main limitation on its application to the developments in urban area is the size of the jacking machine, see Fig. 18, making it difficult to install the piles near the site boundary.

Li *et al.* (2003b) reported a case history of using jack-in steel H-pile, in which the jack piles were loaded initially to minimum 2.2 times the design pile capacity until the settlement rate was less than 5mm in 15 minutes during pile installation in order to accelerate the rate the pile creep and to satisfy the residual settlement criteria of the Buildings Department during subsequent static load test. At present, there is still not a well-established set of termination on which jacked pile can be based upon to satisfy the current pile acceptance criteria of BD. In this regard, the experience and technical specification of jack-in piles in mainland China and elsewhere can be borrowed.

Fig.18. Heavy jacking machine to provide reaction force

5.8 Hiley formula vs wave equations

Hiley formula has been criticised for its incorrectness as well as its applicability in the ground conditions of Hong Kong (Gammon 1982, Hannigan *el al.*, 1998 and HKCA 2004). The situation becomes worse for ultra long piles and might result in unrealistic final set value and caused damage to the piles. In this circumstance, the use of wave equation becomes more appropriate, as reported by Ng *et al.* (2000). Hussein & Goble (2004) gave a comprehensive history on the development and application of wave equation. Today, the most commonly used programmes are based on WEAP (Goble & Rausche, 1976), TTI (Hirsch *et al.*, 1976) and TNOWAVE (TNO Reports 1985-1996). Fig. 19 presents the basic principle of dynamic pile testing method which applies to the one-dimensional stress-wave theory with electronic measurements and numerical calculations to simulate the pile driving process.

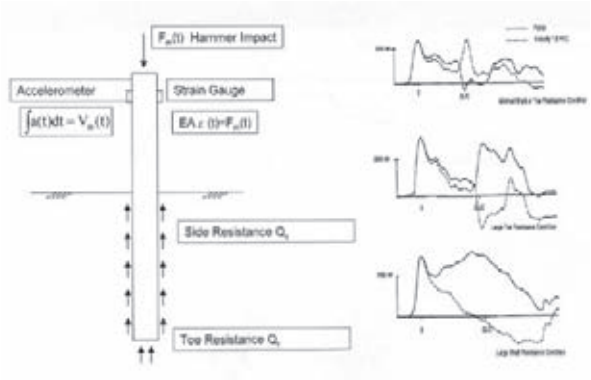


Fig. 19. Basic principle of dynamic pile testing

Terzaghi (1942) commented that pile driving formulae continued to enjoy great popularity among practising engineers because these formulae reduced the design of pile foundations to a very simple procedure. Unfortunately the price to the industry is high. After diesel hammers had been abandoned, highly efficient hydraulic hammers have been used to pitch the piles, which has reflected an advancement in the construction technology, it is unreasonable to continue using outdated drop hammers in final setting of piles simply because unwilling to review the conservative coefficients in the Hiley formula.

ArchSD has adopted hydraulic hammer in final setting after evaluation made in early 2000. However, the use of drop hammer is still a norm in private projects.

The use of hydraulic hammers in final setting requires measurement of energy transfer to pile head, which indeed involves wave equation analysis. It is therefore logical to seek for further development in the use of the wave equation approach.

5.9 Static loading test

The requirement of performing a static proof load test on piles deriving pile capacity other than end-bearing had been emerged more than three decades ago. By that time there were no rock socketed piles such as minipiles and pre-bored H-piles.

Since introduction of rock socketted minipiles and pre-bored H-piles, static load test is stipulated. In early days of introducing the rock-socketted H-pile, there were occasionally reported failure cases and most of them appeared to have a slippage of the bond between the steel and grout.

A series of laboratory tests was conducted by Wang *et al.* (2005) to investigate the load transfer mechanism along socketed H-piles. It was observed that the average mobilised shaft resistance between the steel and grout interface was about 680 kPa (ultimate) which is substantially lowered than the presumed value advised by the Buildings Department which is deemed to be 960kPa under imposition of the proof test load. This ultimate bond strength was, however, greatly increased to 1950 kPa by welding shear bars to the steel section.

Shear bars has now become a standard detail for ArchSD projects (ArchSD 2003). In private projects, it is also common practice that either the designers specifies the use of shear studs/bars in the contract drawings or contractors would incorporate such detail by themselves, as shown in Fig. 20.

For these types of piles, despite from carrying out the predrill holes in determining appropriate rockhead levels for individual piles, as part of the quality control, post construction drillholes are also required to verify the as-built socket lengths.

The authors opine that the bond strength between the steel and grout was a critical factor to the load-carrying capacity of rock-socketed piles and it was well controlled by introducing a simple detail. With abundance test data for the socket friction in all the major rock types being available, the need of static load test on these piles founded into bedrock is becoming a formality and should be reviewed.

Fig. 20. Socketted Steel H-pile with shear studs

Pile Dynamic Analysis (PDA) testing is essentially a high strain test where, in the case of driven piles, dynamic loading is imposed on the pile head by the driving hammer. The load causes a wave to evolve for a finite duration along the pile's length. Measurements of the in-pile stress wave are recorded through the use of strain gauges and an accelerometer mounted near the pile head. The force-velocity plots, measured by the strain gauges and accelerometer, represent the in-pile stress waves over the length of the pile. Detailed interpretation of results from the force-velocity plots allows assessment to be made on driving stress, hammer performance, pile integrity, and pile capacity with CAPWAP analysis.

Fung *et al.* (2004) had reviewed the accuracy of CAPWAP analysis in determining the pile capacities in comparison with more than 100 nos. static load test results. They suggested that CAPWAP analysis is a fairly accurate method for capacity prediction. Having reviewed 300 cases of CAPWAP and static load test data elsewhere in the world, Likins & Rausche (2004) also concluded that CAPWAP usually gives conservative results

as compared to the reserve strength of the pile. Nevertheless, PDA is still normally restricted to the detection of pile defects, monitoring of the driving stress and measuring the hammer efficiency, despite dynamic pile testing methods have been extensively investigated in the last few decades and are now widely used around the world in evaluating pile capacity, for example for off-shore structures, where there is practical problem in setting up testing frame and performing accurate movement measurement under the extreme weather.

5.10 Residual settlement requirement in static load test

Although there is an apparent relaxation of CoPF on the residual settlement during a static load test by introducing an alternative criterion of 25% of total pile head movement at the peak test load, virtually it has no impact to the small diameter piles.

The rationale for the requirement on residual settlement in a static pile loading tests has not been clearly laid out. If a pile is designed to have a factor of safety of 2.0, it is no doubt that substantial movement will occur when the pile is subjected to a test load of two times the working capacity of the pile which matches with its ultimate capacity. The imposition of the residual settlement becomes illogical. Fraser & Ng (1990) opined that if a pile can satisfy such small residual settlement criterion, the pile has to perform relatively elastic and it implies the actual factor of safety in the pile capacity far exceeds the design intent of 2.0.

There are occasions where piles are reported to have failed the criterion on residual settlement while satisfying the maximum pile head settlement. The recovery of the pile head settlement may be restricted by the 'locked-in' stress in soils, as a result of reversal of shaft resistance upon removal of test load as reported by Fraser & Ng (1990) and Fellenius (2002). This phenomenon becomes evident with driven piles of long pile length and it causes unnecessary hard driving of the piles in order to comply with such requirement. Ng *et al.* (2001) had also reviewed the acceptance criteria of load test around the world and observed that the criteria used by various regulatory bodies in Hong Kong appeared to be stringent ones particularly on the residual settlement requirement. A further developed acceptance criteria for local practice is desired.

5.11 Foundation design for areas underlain by karst related deposits

A Technical Guidance Notes No. 26 (GEO 2005) was issued to provide technical guidance on design principles and requirements for foundations in areas underlain by marble and marble-bearing rocks. It proposed to limit the increase of vertical effective stress at the marble surface to an insignificant value generally in the order of less than 3% to 10% so as to prevent the collapse of any cavities in the marble rock due to imposition of foundation load. This is in line with the recommendations established by Meigh (1991).

Driven pile construction in marble areas is still associated with high risk of pile damage, as illustrated in Fig. 21. The case study given by Sze (2006) indicated that the percentage of damage for driven steel H-pile with a pile working capacity of 2950 kN could be as high as 20% for a site in Tin Shui Wai even with reinforced pile shoe and another example is a case study in Ma On Shan using the same foundation system with a pile damage rate of about 5% as reported by Wightman *et al.* (2006).

Fig. 21. Pile damage due to hard driving

One of the reasons causing pile damage was associated with high stress along the pile during hard driving onto the sound marble in achieving the final set criteria of refusal, i.e. 10 mm/10 blows. Sze (2006), Shek (2004) and Li & Lam (2001) observed that the peak driving stress of a steel H-pile driven onto rock could reach about 85% of the yield strength of the steel pile and cautioned the use of an unduly conservative penetration limit that may overstress and damage the piles reported the similar findings. Li & Lam (2001) proposed a relaxation on the final set criterion for pile driven to refusal. A critical review is required.

In the Designated Area of North Lantau, however, the sound engineering bedrock was found to be in excessive of 150m below existing ground making that the construction of conventional pile foundation extremely difficult. A recent experience of a trial installation for a 3m diameter bored pile to a depth of 155m below existing ground was constructed with extremely long construction period and high cost, Wightman *et al.* (2006). Relocation of the proposed development is not always feasible at the construction stage. It is therefore necessary to get Geotechnical Engineer involved at the outset of master layout planning stage to ensure the proposed development to be located within the less risk areas. It is desirable to have more feasible foundation options in addition to the conventional driven steel H-pile or end bearing bored pile for such a difficult ground condition.

Recently, a successful experience was recorded of using shaft-grouted barrette with a dimension of 2.8m x 0.8m in karst-related deposits for a residential development in Tung Chung area, which provides an allowable capacity of up to 22.5 MN per pile (Sze *et al.*, 2007). The barrette was constructed using a hydrofraise as shown in Fig. 22. Instrumented site trials had demonstrated that the performance of shaft-grouted barrette is well within the tolerable limit of the current statutory requirement. The foundation cost is competitive with the typical foundation system such as driven steel H-pile.

Fig. 22. Construction of shaft grouted friction barrette

5.12 Piled raft foundation system

A piled raft foundation system takes into account the contribution of both the piles and the cap acting as a raft footing in carrying the imposed load. Poulos (2001) summaries the different design philosophies for piled raft foundations:-

1 Piles are mainly designed to take up the foundation loads and the raft only carries a small proportion.
2 The raft is designed to resist the foundation loads and piles carry a small proportion of the total load. They are placed strategically to reduce differential settlement.
3 The raft is designed to take up majority of the foundation loads. The piles are designed to reduce the net contact pressure between the raft and the soils to a level below the pre-consolidation pressure of the soil.

Piled raft foundation system is able to achieve an economic design without sacrificing the serviceability requirement in the settlement performance if approaches (ii) and (iii) are used. Piled raft foundation has received considerable attention overseas as well as in mainland China (Gong & Zeng, 2002). An example of piled-raft modeling for China Central TV headquarters at Beijing is illustrated in Fig. 23.

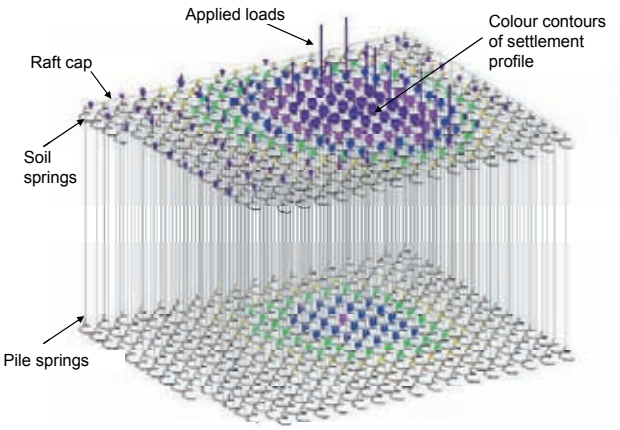


Fig. 23. Piled-raft foundation modelling.

It has not been used in Hong Kong mainly because the prescribed large factor of safety for pile foundations has limited the degree of mobilisation of the bearing capacity from soils underneath the raft cap under working load condition and results in little difference from conventional foundation design for suspended slab approach (similar to approach (i) as described above. This is in contradiction to the principle of composition foundation in which pile is playing the role of settlement reducer and hence much lower factor of safety on capacity can be accepted. It is envisaged that if the implementation of the CoPF continues to be rigidly followed, many of the sustainable pile alternatives such as the piled-raft system would not be able to develop in Hong Kong.

5.13 Hand-dug caisson re-visited

Hand-dug caisson was banned in view of its hazardous working environment to the workers primarily in terms of health but perhaps in safety as well. Nevertheless, the topography in Hong Kong still demand such a construction technique or similar alternatives. The safety issue with hand-dug is manageable through better technical specification and better regulatory control. A set of highly supervised trial had been conduced in Hong Kong to look into the mitigation on silicon dust affecting the health of workers and has devised suitable measures to handle this to the satisfaction of the industry.

Sze & Lo (2007) reported the use 3.0m diameter bored pile wall to retain more than 40m deep excavation into a hillside for a residential development at Kowloon. This option evolutes after the rejection from Lands Department on the initial scheme of smaller sized wall with temporary tie-back protruding into the Green Hatched Black Area of the site. In order to construct the bored pile construction, a temporary steel platform of 160m x 17m was erected at some 30m above the adjacent public roads to support the heavy plants, as shown in Fig. 24, which could be a threat to the public safety. Nowadays, large diameter bored pile wall is also a common retaining wall for the road widening in a number of trunk roads. These bored piles are also required to be constructed over the steep hillside.

Fig. 24. Temporary steel platform for bored pile construction over hillside

The authors opine that hang-dug caisson should still be an appropriate alternative that can be adopted more, given that there are proper solution to handle both safety and health hazards to the workers. The lost of the once prevailing experience in this trade would soon vender this technique obsolete.

6 CONCLUSIONS

Hong Kong has usually been characterised as a dynamic city with sacred lands and hence high premium. The pace of development is usually fast especially in mid 90s and hence time usually takes precedence over cost effectiveness of foundation systems. The development of foundations also tends to move to high capacity piles. The concentrated load on individual piles inevitably results in less redundancy towards defects or less tolerable to a violation of design assumptions. The adoption of simple and perhaps over-conservative systems seems to be meeting the above needs in the last few decades. As a result, foundation design practice in Hong Kong has been lagging behind to many places and innovative designs are limited.

Kulhawy (2000) remarked that many codes are very well-intentioned but they should be considered as living documents. Ideally they should reflect current knowledge and its evolution in a proper manner. To the benefits of the industry and finally to the society, rational foundation design is an irresistible path. Engineers and the regulatory authorities are urged to be objective and versatile in order to make meaningful advancements.

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Developments in Tunnel Engineering in Hong Kong

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ABSTRACT: Many tunnels have been constructed successfully in Hong Kong. These underground structures cater for water supply, mass transport (such as railways and roads), drainage, conveyance of sewage and electrical cables, as well as for underground space development. Techniques for site investigation, construction, design and risk management have been developed progressively, and engineering standards and practice have evolved. This paper summarizes the historical geotechnical achievements and recent developments in the area of tunnel engineering in Hong Kong.

1 INTRODUCTION

The population of Hong Kong has increased steadily from about 2.2 million in 1950 to about 7 million today. Intense urbanization and infrastructure development, combined with limited land availability and a growing awareness of environmental issues has driven Hong Kong's need to develop its underground space. This has resulted in the construction of numerous tunnels and associated underground structures.

This paper summarizes the developments in tunnel engineering in Hong Kong. It is based largely on a review of published papers, including those given in the proceedings of the 26th Annual Seminar of the Hong Kong Institution of Engineers (HKIE) Geotechnical Division (GD) held on 12 May 2006 on Cavern and Tunnel Engineering. Reference has also been made to the recent paper prepared by the HKIE GD Working Group on Cavern and Tunnel Engineering on tunnels and caverns in Hong Kong (HKIE, 2007).

Although this paper will focus on the geotechnical achievements since the 1970s, some earlier tunnels will be mentioned briefly where they are of historical or technical interest. A review of the developments in cavern engineering in Hong Kong has been covered by Chan & Ng (2006) and will not be repeated here, except that some of the developments presented in this paper are also relevant to caverns.

For the purpose of this paper, 'tunnel works' comprise tunnels, shafts, caverns and associated underground facilities, however constructed. The rock and soil descriptions given in this paper follow the recommendations of Geoguide 3: Guide to Rock and Soil Descriptions (GCO, 1988), which is the standard commonly adopted in Hong Kong.

2 GEOLOGY OF HONG KONG

The geology and general pattern of the main inferred faults in Hong Kong is shown in Figure 1. The abundance of massive hard crystalline volcanic and granitic rocks makes Hong Kong particularly suitable for tunnelling and underground development.

The faults can play a significant role in controlling the engineering geological and hydrogeological properties of the rock mass, a good understanding of which is vital for the design and construction of tunnel works. While weathering, structural geology, and the extent and nature of the superficial deposits govern the requirements for tunnel support to ensure ground stability, hydrogeology plays an important role in respect of assessment of groundwater ingress into tunnels during their construction, the drawdown of groundwater pressures outside the tunnels and the consequential settlement of the ground and the facilities that the ground supports. There can be significant uncertainty in the hydrogeology within the groundmass in Hong Kong.

Discussion on rock material and mass weathering characteristics, development of ground models, and the key engineering geological issues related to tunnel works in Hong Kong is given in GEO (2007a). An interactive on-line facility to view the available geological information is now available (GEO, 2007b).

3 TUNNELS IN HONG KONG

About 47% of the land in Hong Kong is greater than 100 m above sea level, and 12% exceeds 300 m. There are 32 peaks higher than 500 m, three of which rise above 800 m. Tunnels have been built through many of these hills including Beacon Hill (457 m), Eagles' Nest (312 m), Lin Fa Shan (578 m), Lion Rock (495 m), Ma On Shan (702 m), Needle Hill (532 m), Smugglers' Ridge (337 m) Tai Mo Shan (957 m) and Tate's Cairn or Tai Lo Shan (577 m). The limitations on flat land have necessitated the construction of tunnels to support Hong Kong's built environment. Other than the rock tunnels through the hills, urban development has led to many tunnels being constructed in soft ground in the low lying urban and sub-sea areas, as well as a number of immersed tube tunnels.

The earliest tunnels were water supply tunnels. Tai Tam Reservoir, which was constructed in the late 1880s, included a 2.2 km tunnel to transfer water to Victoria. The 2 km long Shing Mun Tunnels were built in 1926. From the mid 1950s to the mid 1970s,

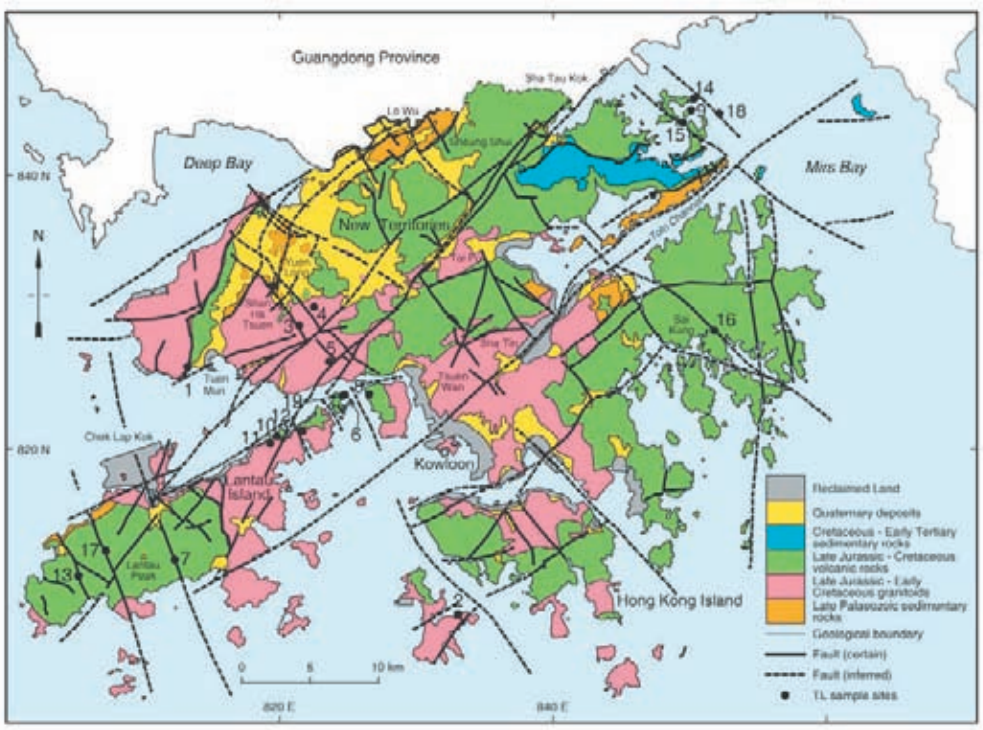

Fig. 1. Simplified geological map of Hong Kong (from Sewell et al., 2000)

about 128 km of tunnels were constructed as part of the various water schemes, including Shek Pik, Tai Lam Chung, Plover Cove and High Island. Since the early 1980s, a number of aqueduct schemes have been implemented, with about 53 km of tunnels completed. By December 2006, more than 199 km of water supply tunnels were constructed. The longest water supply tunnel is the 20.2 km long main tunnel for Plover Cove Stage I. This was completed in 1971. The tunnels for the Tai Po to Butterfly Valley Fresh Water Scheme, totalling 14 km long, were completed in 2003. The main tunnel has a maximum ground cover of 600m, and is the deepest tunnel below ground surface in Hong Kong.

Five railway tunnels were constructed between 1906 and 1910. These included the 2.2 km Beacon Hill railway tunnel (single-track) opened in 1910, as part of the Kowloon-Canton Railway. This tunnel was replaced by a twin-track tunnel in 1978. The old tunnel is now being used for accommodating a gas pipeline. To meet the increasing needs of mass transportation in Hong Kong, the MTR Corporation (MTRC) Limited has built many railway tunnels since its establishment as a statutory body in 1975. This saw the introduction to Hong Kong of soft ground tunnelling techniques, including use of tunnel shields and compressed air, in the late 1970s. From 2003 to 2006, the Kowloon-Canton Railway Corporation (KCRC) has completed a number of railway tunnels as part of their railway network upgrading. The 5.5 km long Tai Lam Tunnel, the longest railway tunnel in Hong Kong, was completed in 2003.

Tunnels have also been built to provide a cost effective road transportation system for its dense urban development. The first Lion Rock Tunnel, opened in 1967, was of dual 2-lane and 1.4 km long, connecting the New Territories and the urban area. This was the first road tunnel in Hong Kong. By December 2005, more than 44 km of road tunnels had been constructed in Hong Kong. The 4.0 km long Tate's Cairn Tunnel, being the longest road tunnel in Hong Kong, was opened in 1991. There are three cross-harbour road tunnels, of 1.9 km to 2.25 km long; all of them have significant sections of immersed tube. The Eastern Harbour Crossing, completed in 1989, is the only cross-harbour tunnel with road and rail. The Western Harbour Crossing, completed in 1997, is the first, and only, 3-lane cross-harbour road tunnel in Hong Kong.

The first sewer tunnel in Hong Kong is the 1.8 km Tseung Kwan O Sewer Tunnel completed in 1986. The 9.1 km North West New Territories Sewerage Tunnel was completed in 1992. The tunnels completed in 2001 under the Harbour Area Treatment Scheme (HATS) (previously known as the Strategic Sewage Disposal Scheme) Stage I project are at depths of 75-145 m below the sea level, unprecedented in Hong Kong. By December 2006, more than 55 km of tunnels were constructed for sewers and drains in Hong Kong.

In the past, the normal practice for installation of underground electrical cables has been by cut and cover trench excavation on a section-by-section basis (typically 300 m to 450 m long). In recent years, the Government of the Hong Kong Special Administrative Region (HKSAR) has strengthened control over excavation works in public roads, especially in carriageways, to minimize inconvenience to the public. This, combined with the desire to avoid overhead high voltage cables, has led to an increased use of cable tunnels and other trenchless methods of tunnelling to lay the cables (Hui *et al.*, 2002). Since 1988, the Hongkong Electric Co. (HEC) Limited has constructed about 11 km of cable tunnels on Hong Kong and Lamma Islands. CLP Power (Hong Kong) Limited has built about 4 km of cable tunnels in Kowloon and the New Territories. Since the late 1980s, hundreds of cable duct crossings have also been constructed using hand or machine-based trenchless excavation methods including pipe jacking and the horizontal directional drilling (HDD) technique.

Apart from railway and cable tunnels, a number of other private tunnels have been constructed. These include a few small drainage and sewage tunnels, tunnels for seawater cooling pipes, a number of pedestrian subway tunnels, a privately developed and operated road tunnel and two vehicular tunnels for the Hong

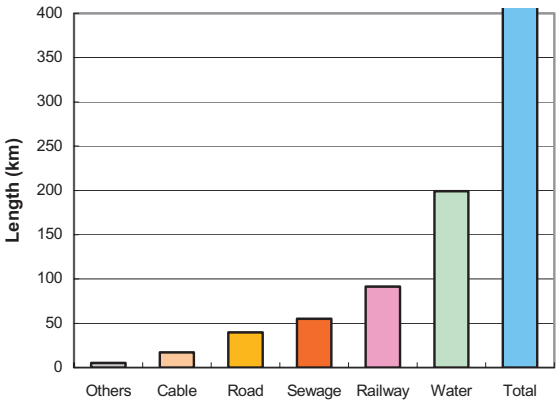

Fig. 2. Length of Tunnels Constructed in Hong Kong as of December 2006

Kong International Airport. A 2.6 km long man-accessible tunnel for a gas pipe was completed at Braemer Hill by the Hong Kong and China Gas Co. Limited in 1994.

A 1.15 km long, main access tunnel and ten 27 m long adits were built in 1997, for the Kau Shat Wan Explosives Depot managed by the GEO. In 1998, 100 m and 80 m long tunnels and deep shafts (76-89 m deep) were constructed for the glory hole construction under the GEO's project on rehabilitation of the Anderson Road Quarries.

By the end of December 2006, more than 400 km of tunnels had been constructed in Hong Kong (Figure 2). Further information on these tunnels is given in HKIE (2007).

At the time of writing, the construction of the Eagle's Nest, Nam Wan and Sha Tin Heights highway tunnels, which are part of the Route 8 project, have just been completed for opening in 2007. The MTRC/Swire Queensway pedestrian subway will open in early 2007. The construction of the KCRC Kowloon Southern Link railway tunnels, the CLP Castle Peak cable tunnel, a funicular tunnel for the Ocean Park Redevelopment project, and the Po Shan Landslip Preventive Measures drainage tunnels have commenced. Design of three major urban drainage tunnels, the second stage of the HATS sewage transfer tunnels, and the MTRC Island West extension is on-going, and planning is being carried out on a number of mass transit railway, high speed railway, road and water supply tunnels. The current and planned projects involving tunnel works in the next 8-10 years amount to project estimates of over HK$50 Billion (US$6.4 Billion).

4 SITE INVESTIGATION

When undertaking site investigation for tunnel works projects, Hong Kong's geotechnical engineers benefit from having ready access to up-to-date geological maps and memoirs, a comprehensive collection of aerial photographs and the Government's Geotechnical Information Unit, which houses factual borehole data and laboratory test results as well as interpretive geotechnical reports for all public works projects and many private projects. An interactive on-line facility to view the available geological information is now available (GEO, 2007b). In addition, mainly as a result of the work done by the GEO over the last 30 years, local standards for site investigation and laboratory testing have been developed and promulgated, e.g. Geoguide 2: Guide to Site Investigation (GCO, 1987) and Geospec 3: Model Specification for Soil Testing (GEO, 2001).

A nucleus of very competent and experienced private-sector ground investigation contractors and testing laboratories provides the equipment and expertise necessary to facilitate ground investigation for large and complex tunnel projects. The Public Works Central Laboratory (PWCL) managed by the GEO develops and provides specialist soil and rock testing facilities. Recently, Cerchar testing facilities have been set up to assess the abrasiveness of rock. The Cerchar test results provide information for the assessment of wear of Tunnel Boring Machine (TBM) cutters. Apart from the PWCL, some of the local testing laboratories and universities can also carry out a range of soil and rock tests for the design of tunnels.

Within the geotechnical profession and amongst major clients, the need for quality site investigation is generally recognized. Also, due to the contractual arrangements for many tunnel works projects, where work is typically done on a lump sum basis, without the provision for claims as a result of unforeseen ground conditions, fairly extensive pre-contract site investigation is normally undertaken for all new tunnel projects, especially sensitive projects within urban areas, to help mitigate the contract risks.

For example, boreholes at 25 m spacing, or less, were drilled in sensitive areas for the Kowloon Southern Link railway project currently being constructed. For tunnels situated at great depth (>200 m) beneath mountainous terrain or situated beneath the seabed, the cost of direct methods of ground investigation is relatively high. The main investigations commonly comprise vertical and inclined drillholes at wide spacing (about 500 m spacing, but much closer spacing at the portals and close to urban areas), to below the tunnel level, combined with relatively cheap geophysical surveys and where possible directional drilling or coring techniques. In all cases, satisfactory interpretation of site investigation data and the development of appropriate engineering geological and hydrogeological models for alignment and permanent works design, geotechnical risk assessment and design of risk mitigation works, is a critical issue.

The decision on whether to disclose the interpreted information compiled at the design stage (e.g. engineering geological sections along and transverse to the tunnel alignment) to the tenderers, or to rely on tenderers making their own interpretation of the factual data, is a contentious contractual matter, but one (among many issues) that can have an impact on the contract price, the potential for contractual claims and the overall success of the project. The industry recognizes that the Code of Practice for Risk Management of Tunnel Works (ITIG, 2005), which has to be complied with in the procurement of insurance for projects involving tunnel works, requires the project client to carry out adequate site investigation and to prepare (or have prepared on its behalf by a competent agent) "ground reference conditions" (see Section 9). The rationale being that the client, during the preliminary and detailed design stage should have followed a lengthy and iterative process of development of an engineering geological model based on desk studies and often two or more phases of ground investigation and associated testing. This generally produces a large body of data which would be difficult for the tenderers to interpret fully given the short time normally allowed for tendering.

Site investigation for tunnels now typically includes:

1 desk study (including review of available ground investigation and testing data and as-built records of nearby tunnels), site reconnaissance, alignment studies, aerial photographic interpretation, and geological and engineering geological mapping along the tunnel alignment,

2 initial ground investigation, including topographical survey, vertical and inclined boreholes as well as sampling and testing (in particular, permeability tests in rock), borehole logging, laboratory testing, and installation of instrumentation,
3 geophysical surveys and associated calibration boreholes, where required,
4 interpretation of geotechnical data and collation into a comprehensive geotechnical assessment report containing engineering geological sections along and transverse to the tunnel alignment and showing the positions of shafts and portals, and key ground and groundwater parameters required for design and risk management purposes, and
5 additional pre-contract and/or construction stage site investigation and interpretation to refine the engineering geological model and the geotechnical risk assessment.

A major constraint on ground investigation (including installation of geotechnical instrumentation) is access, especially within urban areas, where land ownership and the need to minimize disruption to traffic often severely limits the choice of sites to locate drilling plant and equipment. In rural areas, access permission and traffic disruption is less of a constraint, but the hilly terrain often means difficult physical assess and, as the depth of the tunnel can often exceed 100 m below the ground surface, the cost and benefit of drilling deep boreholes become a major consideration in deciding the extent of ground investigation. These constraints often mean that ground investigation tends to focus on the tunnel portal areas and/or at shaft locations in order to identify the depth of rockhead and to examine the extent of the weathering profile in such areas. Typically, a combination of vertical and inclined boreholes are drilled at the portals and at intervals along the tunnel alignment; inclined boreholes tend to be used to confirm the location, extent and nature of faults and other critical discontinuities.

Horizontal boreholes along the actual alignment of the proposed tunnel have also been drilled from portal locations and have proved particularly useful and cost-effective, as each metre of borehole directly relates to the ground conditions that will be encountered in the actual tunnel.

Early examples of horizontal boreholes reported by McFeat-Smith (1982, 1985) using rotary drilling rigs include 55-100 m long boreholes for the investigation of the Western Aqueduct Tunnels and the Lei Yue Mun Portal of the MTR Island Line, and boreholes up to 200 m long for the Pak Kong to Tseung Kwan O Tunnel. Varying the thrust and rotation speed of the drilling equipment essentially steered the direction of these holes but the control was very limited and, for the 200 m borehole, the position accuracy was about 15 m from the intended target alignment.

More recent examples, using a wedge (also called a 'bent subframe') installed at the end of rods to control direction of the holes, were undertaken for the Cheung Tsing Tunnel (up to 1,000 m long boreholes, using wireline drilling techniques) and the Route 3 Tai Lam Tunnel (boreholes over 1,000 m long) (McFeat-Smith, 1985; McFeat-Smith & Ichikawa, 1993; McFeat-Smith, 1996; Lloyd, 1998).

A similar technique was used for two horizontal holes, 730 m and 800 m long, drilled from shafts at the non-excavating ends of tunnels during construction of the HATS Stage I project. Further enhancement has come about through the use of a steerable drilling head and associated continuous directional monitoring equipment (sometimes referred to as horizontal directional drilling, but, in fact, the technique is not limited to drilling along a horizontal plane).

HDD was initially used for the installation of water pipes (Tam 2000), but its use has also been extended to ground investigation for tunnel projects. For example, two directional drillholes, 516 m and 1,151 m long, using the Devidrill system were undertaken for the recently completed Eagle's Nest Tunnels. Another two drillholes, 214 m and 310 m long, were also recently completed using the same system for the current Po Shan Road Landslip Preventive Measures drainage tunnels. The technique is currently being used for ground investigation for the 5.1 km DSD Tsuen Wan drainage tunnel. These drillholes provided a near complete geological core of the tunnel in advance of excavation. Apart from better directional control, a key advantage of using a steerable head is that it can help overcome site access problems; for example, under the HATS Stage 2A project, which is currently under design, it is planned to drill curved directional boreholes from the ground surface down and along the alignment of the proposed tunnels. In contrast, for the HATS Stage I project, horizontal boreholes along the tunnel alignment could only be installed after construction of deep shafts down to the tunnel level.

Recent advances in directional coring technology involve the use of the wireline-operated steerable core barrel and the electronic multi-shot drillhole survey instrument. Both techniques have reduced significantly the time required for hole steering and hole profile survey operations. The result is a significant improvement in drilling progress and thus lowers the drilling cost. The next important development is the use of wireline-operated in situ test equipment and measuring instruments including those for the core orientation test and the water inflow test.

Geophysical survey techniques, principally seismic refraction surveys, have been used for many tunnel projects, mainly to give a preliminary assessment of the subsurface conditions, including the depth of the weathered profile, thickness of buried alluvial or colluvial channels, and the location and thickness of major structural discontinuities such as faults along the tunnel alignment. For example, a seismic refraction survey was undertaken for the Western Aqueduct Tunnels and, more recently, for sub-sea sections of the HATS Stage I project and the Po Shan Road drainage tunnels. Seismic refraction is generally successful on land, provided that the area is not masked by seismic 'noise' and data is carefully interpreted (there is potential for misinterpretation in corestone-bearing weathering profiles).

Lumb (1991/1980) reported that 'sparker' seismic reflection surveys were used successfully for the first time in the investigation for the first immersed tube Cross-Harbour Tunnel. However, biogenic gas within marine mud can severely affect the reliability of survey data; it has hampered the recent investigation for the HATS Stage 2A project, where about 50% of the offshore survey area is masked by 'gas blanking'.

McFeat-Smith *et al.* (1986) reported on the use of cross-borehole, borehole-to-surface and tunnel face-to-borehole tomographic acoustic measurements. However, Lovegrove (1986) reported that acoustic tomography had failed to identify the location of a major fault for a project in Hong Kong.

Micro-gravity surveys have been undertaken to map offshore geological features, to locate disused tunnels, to investigate adverse geological conditions in a karstic limestone area (Kirk *et al.*, 2000) and to assess the weathering profile of the rock mass and the rockhead level for the design of station caverns and tunnels for a railway project (Ng & Wardall, 2005). This technique has proved suitable in areas with easy site access, relatively constant site topography and where there is sufficient contrast in the density between rock and soil derived from in situ rock weathering. Magnetic surveys have principally been used to map offshore geological features but have also been used to assist in the planning of further investigation for tunnel projects by detecting metal objects on or at shallow depth below the seabed along the pro-

posed tunnel alignment. So far they have not been carried out specifically for ground investigation for tunnels.

Given the dense urban environment in many parts of Hong Kong, assessment of the effects of new tunnel works on facilities sensitive to ground collapse, settlement or vibration, including buildings, slopes, roads and utilities, is a primary consideration in tunnel projects. Experience has shown that groundwater drawdown as a result of excavation within the weathered profile, i.e. at relatively shallow depths, can result in fairly localized and reasonably predictable ground settlement, which, in urban areas, still needs to be carefully assessed, monitored and controlled. Deep tunnels within rock, under the sea or land, but in the vicinity of urban development, can pose a more complex set of problems, which need to be assessed as part of site investigation.

During the site investigation stage for tunnels, particular attention has to be paid to the assessment of hydrogeology and, in rock, rotary coring, packer or Lugeon tests (Houlsby, 1976) and impression packer tests are routinely undertaken to obtain a measure of the amount of fracturing in the rock mass, its connectivity to water heads and its permeability, and hence an indication of the potential for large quantities of water to flow towards the tunnel with associated drawdown of piezometric levels during excavation.

For the HATS Stage 2A project, a wireline-operated pump-down packer system with double packers (Solexperts, 2006, 2007) will be used to carry out water inflow tests in holes formed by directional drilling. Unlike the Lugeon tests, where water is injected into the ground under high pressure which may affect the hydrogeological conditions, testing with pump-down packer tools in the directional drilled hole is a 'production test' (called the 'drill stem test', developed for the petroleum industry in the 1980's, see Delouvrier & Buehler, 2003) with water drained out of the ground under the water head at the section under test. The test conditions aim to simulate the hydraulic conditions during tunnel excavation. The wireline tools with on-line data acquisition and real-time interpretation allow the tests to be carried out during the drilling phase, thus saving drill rig standing time. During the test, the piezometric pressure as well as the transient pressure, temperature and flow rate during the test sequence when water is made to flow from the ground into the test section or vice versa are measured. The test results are interpreted and analyzed using aquifer models to estimate the initial and steady state water inflow rates into a future tunnel.

Ground vibration during tunnel excavation is generally a function of the method of construction, and to protect life and property, vibration limits and monitoring requirements for sensitive receivers are typically imposed during construction. To support the blasting assessment for a drill and blast tunnel, which has to be undertaken at the design stage and also refined at the construction stage of the project, the site investigation needs to obtain sufficient data for the assessment of blasting effects, in particular, the ground and groundwater conditions. The baseline conditions of sensitive receivers in the vicinity of the tunnel, including the portal slopes, are also required to be obtained for the assessment of their stability, movement and vibration and for setting the blasting control limits.

For design of tunnel support measures, the relevant rock mass parameters, soil parameters and information on groundwater pressures at the tunnel level are required to be assessed. Soil parameters and fluctuations in groundwater pressures at the tunnel level are required to be determined along the whole alignment in soft and mixed ground for the assessment of pressures to be applied at the face of earth pressure balance (EPB) and slurry (pressure balance) TBMs, in order to maintain tunnel face stability and to control ground movement (both settlement and heave). The relevant soil and rock properties required for the design of ground treatment (such as grouting) and groundwater control measures (such as recharge wells) are also needed.

In the investigation for the Aberdeen Tunnel in the mid-1970s, an almost full-length pilot tunnel of 2.4 m by 1.5 m was constructed (Chappell & Tonge, 1975; Twist & Tonge, 1979). This was carried out due to concerns over lack of previous experience of tunnelling works at such depth in Hong Kong, coupled with uncertainties regarding the contact zones between granite, deeply weathered monzonite and volcanic rocks near the south portal. The pilot tunnel construction provided the contractor with high quality information for pricing purposes, and inability to drive the pilot tunnel from the south portal without extensive ground treatment highlighted the adverse ground and groundwater conditions. This led to adoption of a remeasurement contract based on a schedule of rates for equitable reimbursement of tunnel costs in this difficult section of the tunnel.

The magnitude and direction of the in situ stresses in the rock mass can be important considerations in the design and construction of rock tunnels, e.g. high horizontal stresses normal to the tunnel axis are usually beneficial for roof stability. In situ stress measurements using both hydrofracturing and over-coring techniques have been carried out in Hong Kong. The results indicate that the principal horizontal stress in rock is in excess of the vertical stress at depths of less than about 150 m (Klee *et al.*, 1999; Free *et al.*, 2000; Ng & Wardall, 2005).

For compressed air or slurry operation, soil parameters and information on the groundwater regime are required to be assessed for use in reviewing the possibility of air or slurry leakage and also in checking against ‘blow out’ failure or excessive ground heave at locations with a shallow soil cover to the tunnel. Similar information is also required for use in grouting design.

In 2005, the GEO issued a Technical Guidance Note, TGN24, on site investigation for tunnel works to document best practice (GEO, 2005a). The preparation of this TGN had significant input from members of the HKIE GD Working Group on Cavern and Tunnel Engineering. It supplements the existing guidance given in Geoguide 2 (GCO, 1987) and Geoguide 3 (GCO, 1988). All of the GEO TGNs (GEO, 2007c), including TGN1, which lists all the documents being used as de facto geotechnical standards in Hong Kong, can be found at the CEDD website. Two other useful guidance notes for ground investigation for tunnels have been issued by the Association of Geotechnical and Geoenvironmental Specialists (Hong Kong) (AGS (HK), 2007).

5 CONSTRUCTION

Tunnels in Hong Kong are built using a number of construction methods. These include use of drill and blast, shields and compressed air, sprayed concrete lining, ground freezing, TBMs, trenchless methods such as pipe jacking or directional drilling, immersed tube and cut and cover methods.

5.1 *Drill and blast*

Drill and blast has been in use since the early development of Hong Kong, and because of the abundance of good quality rock this is by far the most commonly used method.

Black Powder (a mixture of potassium nitrate, carbon and sulphur) is the earliest known blasting explosive in Hong Kong. It was used in mining activities including construction of mining

tunnels and adits before and after World War II. Hong Kong ceased using Black Powder for rock blasting in 1974.

Nitroglycerine (NG) based explosives such as dynamite was reportedly used for the construction of the Tai Tam Tunnel in the period 1884-1887. It was probably introduced to Hong Kong in the 19th century. It was used in the Ma On Shan Mine and in the KCRC Beacon Hill Tunnel, Aberdeen Tunnel, Lion Rock Tunnel and the Plover Cove Water Scheme. Because of high cost, sensitivity to friction and shock, and high inflammability, use of NG-based explosives has declined since World War II and the last known application was in the MTR Island Line in 1985.

Ammonium Nitrate and Fuel Oil (ANFO) was introduced to Hong Kong in 1965. It was mixed in two plants at Ma Yau Tong between 1965 and 1997. Bulk ANFO manufactured by a mobile manufacturing unit (MMU) is still used in Hong Kong.

Water gels and slurry explosives are high density aqueous explosives containing AN and FO in both solution and suspension and thickened to a gelled water consistency. Sensitization is partly by chemicals and partly by bubble entrapment. Water gel, manufactured by a MMU, was first introduced to Hong Kong in the High Island Reservoir project in 1973. It was manufactured in cartridges in the Stonecutters Island Plant between 1979 and 1994 and was then imported into Hong Kong for the Airport Core Projects until 2000.

Emulsion explosives are minute droplets of a concentrated solution of ammonium nitrate and other oxidizers dispersed in oil or wax by using emulsifiers. They were imported into Hong Kong in 1986 in cartridge form, and are manufactured locally by a MMU as pumped emulsion. They have been used in many projects including the Cheung Tsing Tunnel, and currently dominate the commercial explosives market in Hong Kong.

The first full-face blasting for a 3-lane road tunnel, of 17 m span and 10 m high, was for the Cheung Tsing Tunnel, which was completed in 1997. Computer-controlled semi-robotic jumbos were first used in Hong Kong for the Route 3 Country Park Section Tai Lam Tunnel in 1998. The locations of blast holes are uploaded onto the on-board computer and, once the location and orientation of the jumbo are determined by laser survey, the hole drilling is carried out automatically, with the boom on the jumbo moving to the pre-set locations and drilling the correct length for each hole. The automated process reduces the risk of human error and hole-to-hole setup time, improves the accuracy of drilling, reduces overbreak, enhances the safety of the plant operators, and improves the accuracy of control of blasting vibrations.

There is regulatory control on the use of explosives in Hong Kong. The Commissioner of Mines, who is the Director of the CEDD, regulates the manufacture, storage, conveyance and use of explosives by means of a licensing system in order to ensure that the safety and security issues are adequately addressed. The tunnels to be constructed are often in close proximity to very densely developed areas. Therefore, the risks and potential problems associated with the use of explosives for rock blasting, especially ground vibrations, gas pressures causing blowout at locations of shallow rock cover, and the effects of noise and air overpressure need to be carefully assessed together with the safety of construction personnel, as is the need to manage the logistics and safety and security issues relating to site magazines and transportation of explosives from the Government-managed explosives depots to the works site.

In the construction of the Black Hill Tunnels, expanding agents were used to break up the rock, in order to mitigate the ground vibration and noise near the western portal (Tunnels & Tunnelling International, 2002). In this method, holes of 2 m long were drilled and then these were packed with a proprietary product, S-mite. Although it was effective, the method was very slow, taking 12-24 hour rounds, and therefore the expanding agents were only used on a few metres of the tunnel drive. The rest was excavated using drill and blast.

A chemical blasting method, which involves use of small cartridges containing explosives and gas, was used for splitting the rock at vibration-sensitive areas in the MTRC Quarry Bay Congestion Relief Works project.

A more recently introduced non-explosive method has been the use of hydraulic handheld splitters. These are used in sets of four or more, inserted into 1 m long pre-drilled holes to split the rock into manageable fragments. Although this method remains comparatively slow, it is proving to be the most efficient non-explosive method to date, having been used on a number of projects in Hong Kong including the recent CLP Kwai Chung cable tunnel, the HEC Lamma cable tunnels and the KCRC Lok Ma Chau Spurline railway tunnels.

5.2 *Developments in drill and blast and tunnel support technology*

There have been significant developments in drill and blast and tunnel support technology in tunnelling works.

New generation CAN-bus (Controller Area Network, comprising small on-board computers linked by a single cable) drill rigs are now commonly used since their introduction to Hong Kong in the 1990s. Medium and large boomer type drill rigs have been used wherever suitable, e.g. as in the MTRC Pak Shing Kok, KCRC West Rail and the Route 8 road tunnel contracts. The programmable computer control system on the rigs provides for real-time monitoring and adjustment of drilling parameters (indicators of changes in geology and ground-rig interaction) and drilling progress.

Specialist drilling equipment has also been developed. This, together with two rigs working side by side at the tunnel face where possible, has resulted in over 50% improvement in tunnel production rates when compared with traditional drilling equipment. The larger of these machines can be programmed for fully robotic drilling, further reducing the drilling time and increasing position and orientation accuracy of the drilled holes. The accurate blast hole drilling and the programming of blasting sequence achieved using computer control have allowed optimal blasting to be achieved. This has brought about reductions in overbreak and time and cost savings. With profiling equipment added, it is possible to identify areas of underbreak earlier for timely remedial actions.

There have also been major advances in shotcreting technology over the last 20 years (e.g. Bergfors & Coates, 1990). Availability of specialist admixtures, use of computer control dosing equipment, and rigorous quality control and testing procedures developed have allowed compressive strengths of over 60MPa to be achieved consistently. The development of wet-spray techniques has led to improved health and safety on site. Shotcrete is now used not just for providing temporary support, but also as a final permanent lining in some projects, e.g. in the MTRC Tung Chung Line at Lai King, Tseung Kwan O Extension and Disneyland Resort Line projects, the DSD Kai Tak Transfer Scheme shafts, the CLP Kwai Chung cable tunnel, and the MTRC/Swire Queensway Subway.

Different types of bolting systems have been intorduced for temporary support. These include split-sets (typically used in mining), grouted steel bar dowels, threaded bolt and expansion shell systems, combination mechanical/grouted bolts, and water-inflated bolts (e.g. Swellex). Water-inflated bolts, although fairly

expensive, allow installation within minutes of the bolt being placed and load to be taken instantly. Due to their simplicity of installation, they are flexible and efficient to apply, with good quality control. They were first used in Hong Kong in 1988 on the MTR Island Line at North Point, and have been used in a number of projects since.

5.3 *Tunnel shields and compressed air*

Shields and compressed air were introduced in the 1970s for the construction of the MTR tunnels. Some shields had a backhoe mounted below the central deck for excavation of soil at the lower half of the face. Compressed air construction had not previously been used in Hong Kong for tunnel construction; hence trial tunnels were carried out (Haswell & Umney, 1978; Haswell & Campbell, 1983). This was also the first time that such tunnels were constructed in soft ground with marine clays in the crown. The trials showed that the tunnels could be successfully constructed using compressed air, or in free air after pre-treatment of the ground by injection of a cement grout mixed with bentonite clay, followed by a sodium silicate gel with an organic hardener.

The combined cement bentonite and chemical grouting was carried out using a two-phase sleeve pipe system ('tube-à-manchette'), and was found effective for a range of soils including soils derived from in situ weathering of granite, alluvium, marine clays and fill. The experience on grouting from the MTR Modified Initial System is documented by Morton & Leonard (1980), Cater & Shirlaw (1986) and Shirlaw (1987).

In the compressed air drives, normally the air pressure had to be increased at rock/soil interfaces where high groundwater pressures were expected or encountered to provide support to the excavation face for stability. Where necessary, face grouting was carried out to stabilize the soft ground or boulders, mainly at the tunnel crown. Except for a few metres below the ground surface, soft ground above the tunnels were grouted before the tunnel drives, to strengthen the ground for stability and for minimizing ground deformation, and to prevent leakage of air from the tunnel face.

An advanced variant on compressed air tunnelling, using an air plenum shield, was adopted for construction of two cable tunnels, viz. Tuen Mun and then Tze Wan Shan, both completed in 2005. The machine has a front chamber which supports the ground by a combination of compressed air and the bund of excavated material. The excavation is made by a turret-mounted excavator arm. The excavated material is removed via a screw conveyor. Under suitable ground conditions, the tunnel remains unpressurized, thus minimizing the need for compressed air working and reducing the risks from loss of ground support pressure at the tunnel face.

5.4 *Sprayed concrete lining (SCL)*

The SCL method of soft ground tunnelling was used extensively on the MTR Island Line for initial drives, shield chambers, platform extensions, passenger adits and a siding tunnel. It was typically used for short sections of tunnel constructed through soft ground and through mixed faces of soft ground and rock. The length of each advance in the soil and mixed face sections was typically one metre, reducing to 600 mm if the ground conditions were poor. Shotcrete was applied almost immediately after excavation. Rolled steel rib supports were installed for every advance. Soil nails/rock bolts were also used in some of the early SCL sections on the Island Line, but this practice was largely abandoned for SCL in soft ground as water leakage at the nail/bolt hole locations caused dewatering settlement.

The SCL sections included the extensions to the Central Station platforms, which were constructed through fill, marine sand, colluvium and weathered granite (Grades II to V), with only about 4.5 m cover over the 7.6 m diameter tunnels. The tunnels were driven with low pressure compressed air after pre-treatment by silicate grouting (Cater & Shirlaw, 1986). After grouting, some boreholes were drilled to check the thickness of the grouted zone, and some samples were taken for UCS tests. Packer tests were carried out in some boreholes to check the permeability of the grouted zone.

More recent examples of the use of SCL in soft and mixed ground have been on the MTRC Tseung Kwan O line Pak Shing Kok tunnels where a 110 m long, 6.5 m diameter soft ground section of the tunnels passing beneath Wan Po Road was excavated using a heading and bench method and the MTRC/Swire Queensway pedestrian subway tunnel which was excavated by drill and blast and soft ground tunnelling methods, using forepoles and face bolts where a highly variable rock/soft interface was encountered at the excavation face (Lo *et al.*, 2001).

5.5 *Ground freezing*

Ground freezing was first used in the HATS Stage I project to treat the soft marine deposits behind the tunnel eye opening in the diaphragm wall of a 13.2 m diameter pipe jacking shaft at Kwun Tong. It was also used for the construction of the 3.6 m span cross passages in the KCRC Lok Ma Chau Spurline Tunnels (Storry *et al.*, 2006a).

5.6 *Hard rock TBMs*

In the early to mid-1990s hard rock TBMs were introduced. The first one was that used for the Hong Kong Electric's Nam Fung Road to Mount Parker Cable Tunnel in 1993. Other examples include the one for the Braemer Hill to Wong Nai Chung Gap Road Tunnel for a gas pipeline in 1994, one for the Siu Ho Wan to Silvermine Bay Aqueduct Tunnel in 1996, one for the Tolo Harbour Effluent Export Scheme in 1997, the TBMs used for HATS Stage I from 1995 to 2001, two TBMs for the MTRC Quarry Bay Congestion Relief Works in 2001, two for the Tai Po to Butterfly Valley Fresh Water Tunnel in 2003 and one for the Chi Ma Wan cable tunnel in 2006.

5.7 *EPB and slurry TBMs*

The pipe-jacked slurry TBM was first used in the Fanling Trunk Sewer project in mid-1989 (McFeat-Smith & Woods, 1990). Many other such machines have been used since, e.g. one for the Central, Western and Wan Chai West Trunk Sewer Tunnels in 2000 and four for the Wan Chai East and North Point Trunk Sewer Tunnels in 2005.

The open and closed compressed air mode EPB TBM was used for the first time in Hong Kong in the KCRC West Rail Kwai Tsing Tunnels in 2003. This TBM, being the largest tunnelling machine ever used in Hong Kong at that time, weighed 1,500 tonnes and measured 8.5 m in diameter and 100 m in length including back-up decks, and was capable of excavating through hard rock as well as soft and mixed ground (Asia Engineer, 2000). The same TBM, with a modified cutterhead of 8.75 m in diameter, was used for the Lok Ma Chau Spurline Tunnels, recently completed for opening in 2007. A 3.6 m diameter EPB TBM was also

used for the CLP cable tunnel at the KCRC Hung Hom Freight Yard in 2006.

The Kai Tak Transfer Scheme in 2004 was the first use of a large diameter (5.15 m) slurry mixshield TBM. This incorporated a compressed air cushion in the excavation chamber and air locks to allow cutter changes and maintenance. An 8 m diameter slurry mixshield TBM has recently commenced excavating tunnels for the KCRC Kowloon Southern Link project. This weighs 800 tonnes and is 78 m in length including back-up decks.

5.8 *Innovative/special techniques in TBM construction*

On the Quarry Bay Congestion Relief Works project, the contractor adopted an innovative approach and modified a 5.3 m diameter hard rock TBM for the construction of the running tunnels. The cutterhead diameter was increased to 6.2 m by installing extensions and purpose-designed cutter discs at the perimeter. This allowed the contractor to disassemble the outer cutter discs inside the tunnel, and then retrieve the TBM by moving it back down the bored tunnel, thus saving the cost of constructing a retrieval shaft. In effect the machine became a retractable TBM. It was then rotated in the launch chamber to complete the opposite drive in the other direction.

The same method of retracting a TBM was used on the HATS Stage I project to remove an abandoned TBM from beneath Victoria Harbour 600 m back to the launch shaft.

Two drainage tunnels are being constructed above Po Shan Road under the Government's Landslip Preventive Measures Programme. The tunnels are to be excavated using a hard rock TBM. Due to environmental constraints and site access problems (the land being in Country Park ground), a receiving shaft for disassembling the TBM cannot be constructed nearby. To overcome this, the TBM parts will be disassembled, pulled back by a winch and roller system and retrieved from the first tunnel bore, before the tunnel lining is constructed. The TBM will then be reassembled close to the entrance of the first tunnel to undertake the second tunnel bore. The construction of a receiving shaft is thus saved. Given land and access constraints in urban areas of Hong Kong, retractable types of TBMs could be in demand for future tunnel projects.

In the Lok Ma Chau Spurline project, due to high groundwater table, the close proximity of a river channel to the TBM launching shaft and the relatively small ground cover to the tunnel crown, a watertight cut-off wall was constructed around the launching shaft area using slurry wall techniques to facilitate the TBM launching. Cement bentonite grouting was also carried out above the tunnel crown level in the same area. These measures enabled the TBM to operate in open mode for the initial break-in excavation from the launching shaft. A grout seal between the segmental lining and the ground is formed at the back of the shield before the cutterhead breaks out of the treated ground, thus allowing full EPB mode to be deployed and tested before the cutterhead reaches the water-bearing ground. However, as the alignment of the tunnel also passed beneath the river channel, a thick concrete slab had to be constructed across the river on both sides of an open drainage channel, to provide sufficient surcharge loading to prevent blow out ground failure during the commencement of the EPB mode excavation. As with many projects using TBMs, especially EPB and slurry machines, engineering a successful launch of the TBM is often a critical point in the construction process.

5.9 *Cut and cover*

Cut and cover methods are also commonly used for the construction of tunnels and underground stations.

In the MTR Modified Initial System project, the cut and cover tunnels were constructed within strutted sheet pile cofferdams (McIntosh *et al.*, 1980). Only one section, close to sensitive buildings, was constructed within a braced cofferdam using a proprietary packed-in-place (PIP) pile system. In the PIP system, the wall was formed by contiguous piles constructed using a continuous flight hollow shaft auger mounted on a crawler, with bentonite slurry as ground support for soft ground.

The station walls were of different types of construction, including diaphragm walls (constructed in slurry-trenches), sheet pile walls, steel double-U section piles in pre-augered holes (Berlin pile wall system), anchored king pile walls (Tokyo wall system), secant-pile walls (formed using Benoto piling rigs) and interlocking hand-dug caissons.

Excavation after wall formation was from the top down or from the bottom up, depending on the contractor's choice, but with heavy bracing to restrict movements. Ground anchors were used but internal strutting was more common. In some cases preloading of the struts were carried out to minimize the lateral movement of the walls. Where the walls were not contiguous, grouting was usually carried out behind the walls to a sufficient depth to provide a groundwater cut-off.

Since the late 1970s, the range of cut and cover methods mentioned above have been commonly used for tunnels and associated underground construction in Hong Kong.

In the Lantau Airport Railway project, piles were installed for forming the retaining wall of the cut and cover Central Subway using specialist 'down-the-hole hammers' of 800 mm diameter. This was the first time such equipment and of such a size was used in Hong Kong (Crighton & Budge-Reid, 1998).

More recently the press-in piling technique has been used to reduce ground vibration and associated effects on adjacent facilities (Goh & Li, 2004; Storry *et al.*, 2006b).

5.10 *Risk control and mitigation*

Risk control and mitigation techniques used during construction of tunnel works in Hong Kong include:

1 probing ahead of the tunnel face (for drill and blast, open TBM and pipe jacking methods),
2 provision of reliable compressed air to balance groundwater pressures (for compressed air-assisted excavation and compressed air work during cutter changes and maintenance),
3 dewatering (where this will not cause problems with the induced settlements),
4 pre-grouting of the ground ahead of the excavation to limit water inflow and, where necessary, to enhance the strength and stiffness of the ground,
5 provision of internal structural supports such as shotcrete, rockbolts, dowels, steel ribs and lagging, and invert slab,
6 excavation with top heading and benching, and additional support by forepoling and steel ribs,
7 provision of an impermeable strong structure around the ground opening to prevent collapse, excessive ground deformation and water inflow during TBM break-in and break-out and cross passage excavation, e.g. slurry wall, jet grouting and ground freezing,
8 proper specification of slurry properties for the soil types anticipated and slurry treatment, including testing requirements prior to and during excavation, for slurry TBM,

9 proper design and reliable control of earth pressures or slurry pressures as well as of excavation chamber inflow/outflow rates to limit face loss and prevent ground instability for EPB TBM or slurry TBM respectively, including during launching, excavation and non-excavation periods, and proper design and testing of the seals and other safety-critical components,
10 recharge well systems to maintain piezometric heads,
11 relocation of existing structures, structural bracing, compensation grouting or underpinning,
12 monitoring and control systems, including real time, automated alerts and prediction modelling,
13 emergency contingency plans, and
14 training and competence assessment of personnel performing safety-critical works.

Site monitoring is often carried out, especially where there are sensitive receivers. This can include monitoring of groundwater levels/piezometric pressures, water inflow, ground settlements, differential settlements, lateral movements (using inclinometers), tunnel convergence, strut loads, tilts of structures, vibrations of the ground and structures, etc. Monitoring is now a combination of manual and digital readings. Geotechnical data acquisition and management systems are now available in Hong Kong for capturing, processing and reporting the data in real time. The data, data trends and reports are accessible by action parties and major stakeholders via the Internet using a web browser.

For emergency preparedness, provision is made for plant, equipment and operatives to be available to carry out speedy emergency repair in the unexpected event that a facility were affected by excessive settlement. This is a contingency measure and is only satisfactory where closure of the facility affected during the repair period is acceptable to the owner and the public.

6 DESIGN

6.1 *Rock tunnels*

The early tunnels were designed based on experience and precedence. Many of these tunnels had initial (temporary) tunnel support provided by shotcrete (or sprayed concrete), rock bolts, steel ribs and lagging. Several approaches/methods based on rock mass classification systems have progressively become available. These have been adopted by practitioners in Hong Kong for the design of initial tunnel support measures. They include the New Austrian Tunnelling Method (NATM), the Norwegian Method of Tunnelling adopting the Q-system, and the method adopting the IMS Rock Mass Classification System.

The NATM approach, developed by Rabcewicz, Müller & Pacher in the 1950's to 1960's (Rabcewicz, 1964), is based on the concept of a closed load-bearing ring of reinforced rock mass. It is a form of observational method in the sense that the ground supports (shotcrete, usually mesh- or fibre-reinforced, and any necessary rock anchors, bolts, dowels or steel ribs and lagging) are provided to suit the observed movements of the ground. Its use requires the control of deformation of the rock mass before the permanent lining is installed. The deformation must be limited to prevent overstressing the surrounding ground, but be sufficient to mobilize the shear strength of the rock mass around the underground opening and thus reducing the loads on the lining. Field monitoring of deformation (tunnel convergence), and sometimes also the stresses acting on the lining to verify design predictions, as well as controlled construction form an integral part of the NATM approach.

NATM relies on the ground standing up unsupported for a long enough period to apply the shotcrete and the necessary supports. Where the ground is very poor the method allows for a reduction in the length and/or width of excavation to improve stand-up time and reduce excavation and support construction time. Some movement will, however, occur during this phase, primarily in the crown. The tunnel face is usually supported by a bench and a dumpling to reduce movement, and a coating of plain shotcrete is applied to reduce deterioration of the excavation face.

NATM was first reported to have been used in the design of the Plover Cove Water Scheme water tunnel (Ford & Elliot, 1965). It was also reported to have been used in the design of the High Island Scheme water tunnel (Vail *et al.*, 1976), the Lok Fu station and the running tunnels in rock between Wong Tai Sin and Diamond Hill on the MTR Modified Initial System (Asian Building & Construction, 1977), and the Aberdeen road tunnel (Twist & Tonge, 1979).

In the mid to late 1970's, an adapted form of NATM (see Sprayed Concrete Lining section below) was used for the support of the soft and mixed ground sections in the MTR Modified Initial System tunnels in conjunction with the use of compressed air or dewatering to control groundwater (Edwards *et al.*, 1980; Haswell *et al.*, 1980).

The Q-system, with six parameters to characterize the ground, was developed by the NGI (Norwegian Geotechnical Institute) (Barton *et al.*, 1974). It was updated in the early 1990s based on a study of 1,050 cases of main road tunnels constructed and the developments in rock support technology over the previous ten years (use of fibre-reinforced shotcrete) (Grimstad & Barton, 1993; Barton & Grimstad, 1994). This method allows the initial support measures to be designed based on detailed engineering geological mapping and analysis of the geological features encountered during construction. Field instrumentation and monitoring of tunnel convergence, tunnel lining strains and stresses as normally implemented in NATM is not carried out unless the rock mass is of poor quality or the span of the excavation is large. This could achieve significant savings in time and cost when compared with the use of NATM, especially where the tunnel is driven in good rock conditions.

The Q-system was first reported to have been used in the design of the Tseung Kwan O (Junk Bay) road tunnel (Matson & Robinson, 1984). It was also reported to have been used in the design of the Tate's Cairn Tunnel (Matson & Porter, 1990), the Route 3 Tai Lam Tunnel, the MTRC Quarry Bay Congestion Relief Works Tunnel and the KCRC Tai Lam Tunnel (Lo *et al.*, 1999). Use of fibre-reinforced shotcrete (or fibrecrete), as opposed to mesh-reinforced shotcrete, is now commonplace in the use of the Q-system. Murfitt *et al.* (2006) discussed the possible misinterpretation of RQD values for the estimation of Q-values.

The IMS rock mass classification system was developed by McFeat-Smith in the mid-1980's based on experience gained from the excavation of 50 km of rock tunnels in Hong Kong (McFeat-Smith *et al.*, 1985). The IMS rock classes are based on rock joint spacing, weathering grade of the rock and thickness of fault gouge encountered. The IMS system has been applied for the determination of initial tunnel support requirements in massive, strong siltstones, granites and volcanic rocks. It was first used for the Western Aqueduct tunnels, which were largely unlined. It was subsequently used in 20 other projects including the Tolo Harbour Effluent Export Scheme (McFeat-Smith, 1992), for payment purposes.

The permanent lining of rock tunnels is usually designed to withstand the pressures from the rock mass with due account taken of the discontinuities and soils that may be present, and the

groundwater pressures. Often, in hard rock tunnels above sea level where water inflow can be effectively controlled to a minute quantity through grouting, the groundwater pressure in the rock is relieved through a drainage system of adequate capacity surrounding the permanent lining. This allows a nominal maximum credible groundwater pressure distribution to be adopted for the tunnel lining design.

Continuum analysis assuming an isotropic linear elastic medium such as that by Muir Wood (1975) and Curtis (1976) and the formulae given by Duddeck & Erdmann (1982) are used for preliminary design of tunnel linings. Detailed computer analysis is often carried out, e.g. for critical tunnel sections and for sections involving multiple tunnel-tunnel and tunnel-structure interaction. Types of computer programs used include distinct element code such as UDEC, 3-dimensional finite element code such as SAP2000 (Lo *et al.*, 1999), or finite difference code such as FLAC (Salisbury & Hake, 2004).

Endicott *et al.* (2000) reported the use of FLAC analyses to design the initial supports for the three Route 3 Tai Lam highway tunnels (north portals) and to predict the ground settlement induced by tunnelling. The tunnels are 15-17 m span, formed within Grades V and VI granite and granodiorite. The construction sequence and tunnel interaction effects were modelled. The settlement monitoring indicates reasonable agreement between the measurements and the analytical predictions. The challenge of such numerical modelling is to obtain sufficient ground and groundwater data (in particular, discontinuity data) to formulate a reliable ground model for such analyses.

For drill and blast tunnels, the constraints for blasting are assessed by conducting a blasting assessment. The key objective of the blasting assessment is to confirm that blasting is feasible and can be carried out safely without causing unacceptable risk to life and property. In the blasting assessment, the effects of the proposed blasting on nearby sensitive receivers are evaluated and the need for precautionary and protective measures is identified. The blasting constraints and the technical standards for precautionary and protective measures are specified in the contract documents. The pre-tender blasting vibration assessment is normally carried out based on available blasting wave attenuation relationships. During construction, site-specific attenuation relationships developed from blasting vibration monitoring data (obtained from trial blasts and production blasts) are used to update the blasting assessment and review the precautionary and protective measures required. Some guidance on blasting assessment and blast design is given in Geoguide 4 (GEO, 1992).

Where a tunnel may intercept permeable zones in the rock mass that are hydraulically connected to a water head, an assessment of the water inflow into the tunnel and its potential consequences is carried out. Excessive inflow could cause construction difficulties (e.g. inefficient or impossible working conditions, flooding of the heading and tunnel, and problems for concreting the lining). It could result in erosion and instability of the rock mass around the tunnel. It could also lead to drawdown of piezometric levels in the overlying soil, leading to ground settlement. If the settlements or differential settlements are large, it could result in damage to buried and surface facilities.

The guidance on estimation of rock tunnel water inflow given by Heuer (1995) has been used. McFeat Smith *et al.* (1999) presented water inflow rate data obtained from a number of rock tunnel projects in Hong Kong and correlated these to the IMS rock classes. Practical guidelines on water inflow control measures are also given. Lo & Cheuk (2006) examined the water inflow rate data for a tunnel project in Hong Kong and compared them with the water inflow rates obtained from directional drilling carried out before construction and those obtained from advance probing during construction.

The approach adopted locally in managing the risk of excessive water inflow is by specifying in the contract for probing ahead to be carried out or at least requiring probing ahead equipment to be available to be used where needed during construction. The specification normally stipulates that sufficient probe holes of suitable length should be drilled ahead of the excavation face. The number of probe holes to be drilled would depend on the size of the tunnel and the nature of the ground encountered, in particular the spacing of discontinuities. Also, the specification normally requires grouting or other suitable ground treatment to be carried out where the water inflow from individual probe holes or the cumulative inflow from the tunnel face and the unlined sections over a specified length (say 25 m) exceeds a specified limit. For an open or shielded rock TBM, experience has shown that it is advisable to specify that the TBM should have facilities for probing ahead as well as facilities for grouting over 360 degrees from the tunnel face. A limit on the water inflow through the permanent lining is usually specified depending on the watertightness requirement of the facility in service.

For rock tunnelling, the most commonly used grouts are cement-based. The termination criterion is usually based on grouting pressure considerations to achieve penetration into the groundmass for reducing its permeability and, where necessary, improving its strength and stiffness, but to prevent ground heave. Monitoring of the grout intake is undertaken and grouting may have to be terminated to prevent excessive grout loss and any undesirable effects due to excessive areas being grouted. The extent and quality of the grouting are checked by new probe holes prior to resuming the excavation cycle.

Grouting technology was put to the test when OPC, microfine and ultrafine cements were used for pre-grouting the unlined rock tunnels at the HATS Stage I project against pressures of about 15 bars. Use of suitable grout and grouting techniques was successful in controlling water inflow into the submerged rock tunnels (Sjostrom, 2004). Post-grouting was found to be far less effective than pre-grouting (Grandori *et al.*, 2001).

6.2 *Sprayed concrete lining (SCL)*

In the MTR Island Line, shotcrete of 100 mm to 200 mm thick was provided to form the temporary lining for soft or mixed ground. The actual thickness depended on the actual ground conditions encountered. Soil-structure interaction analyses were carried out for determining the spacing and section size of the steel rib supports. Although pre-grouting was carried out (normally chemical grouting to form an annulus of 2.5 m to 3.5 m around the tunnel extrados, and sometimes the tunnel core was also treated by cement bentonite grout), the strength improvement due to grouting was ignored in the analyses, as this could not be quantified. The soils were modelled as elastic soil springs. At-rest earth pressures and groundwater pressures (based on piezometric monitoring data) were applied and the structural forces were obtained from the analyses. Sophisticated computer programs for soil-structure interaction analysis have become available since then and these are now being used in practice where needed. Lee et al (2006) provided useful comments on the use of soil models when assessing the effects of tunnelling on adjacent structures.

6.3 Ground freezing

The design aspects of ground freezing, both to strengthen the soft ground and to prevent water inflow, in order to facilitate excavation of the cross passages in completely decomposed volcanic rock are outlined in Storry *et al.* (2006a).

6.4 *EPB and slurry TBMs*

For EPB and slurry TBMs, it is important to select carefully the range of operating pressures in the excavation chamber and control the rate of extraction of material from the ground. The aim is to prevent ground collapse, limit the 'volume loss' in order to keep ground deformation within acceptable levels, and to prevent blowout failure, at locations with a low ground cover. In a recent KCRC project, reference has been made to the BTS-ICE (2005) publication on close-faced tunnelling machines for guidance on the safety aspects of TBM control.

6.5 *Trenchless techniques and immersed Tube*

For pipe-jacked tunnels, the pipe jacking forces are required to be estimated. McFeat-Smith & Herath (1994) and Wang *et al.* (2006) presented field measurements of the resistance encountered during pipe jacking through a range of soils at different depths, and compared them with the predictions obtained using empirical design methods. A high resistance was often found to be associated with large ground settlements.

The design aspects of some immersed tube tunnels in Hong Kong are presented by Haswell *et al.* (1980), Matson (1987), Silva *et al.* (1998) and Yang *et al.* (2006).

6.6 *Cut and cover*

In the 1980s, design of cut and cover excavations in Hong Kong, including for those for tunnels, was mostly carried out using the first edition of Geoguide 1: Guide to Retaining Wall Design published in 1982. In 1990, GCO Publication No. 1/90 (GCO, 1990) was published, and its guidance has since become widely adopted. The GCO Publication recommends that design against instability of the excavation be carried out based on the global factor (of safety) method. To manage the risk of drawdown of piezometric levels and possible hydraulic failure due to high groundwater pressures outside the excavation, grouting is sometimes carried out to provide a groundwater cut-off, depending on the ground and groundwater conditions and the type of retaining wall used. Where perimeter grouting is carried out, a pumping test is performed by pumping water out of the excavation while checking the piezometric drawdown outside to verify the effectiveness of the grouting works. The second edition of Geoguide 1: Guide to Retaining Wall Design (GEO, 1993) was published in 1993. This is one of the first guidance documents in the world that introduce the use of limit state partial factor method for retaining wall design.

In 2004, as a result of feedback from the profession the GEO conducted a review of the limit state partial factor method for excavation design with input from the industry and the Buildings Department. This resulted in the introduction of a set of guidance notes by the Building Authority and the GEO for the trial use of CIRIA Report No. C580. The aim is to provide an alternative design method to the designer in order to increase the chance of achieving an economic design. Pang *et al.* (2005) outlined the background to the introduction of CIRIA Report No. C580 for trial use and summarized the geotechnical computer programs commonly used for excavation design in Hong Kong. Since the commencement of the trial use period in January 2005, the profession has so far not used the C580 method for a combination of reasons, principally because of concern over tight programme and high liquidated damages in the event delay in completion of contract. The Building Authority has recently extended the trial for three more years.

The deep groundwater cut-off due to grouting and the construction of permanent walls for the tunnels, coupled with extension of the tunnel works to the ground surface, exacerbates a possible damming effect of these works on the moving groundwater. Such damming effect on the stability of slopes upstream is considered in the design.

Where the tunnel is in shallow ground with a high groundwater table, the mechanism of floatation or uplift is also checked.

7 GROUND MOVEMENTS CAUSED BY TUNNELLING

References to literature on ground movements caused by tunnelling are given in the Bibliography on the Geology and Geotechnical Engineering of Hong Kong (GEO, 2007d). Due to space limitation, only a few cases of significant ground movements caused by tunnelling works are highlighted below.

During the construction of the MTR Modified Initial System, at one site the slurry trench for a diaphragm wall was very close to an old masonry building, the Supreme Court, founded on strip footings and short timber piles. Horizontal movements of about 100 mm towards the slurry trench occurred, caused by the formation of the trench alone and the stress relief in the soil, before any bulk excavation started, and these movements seriously affected the building (Davies & Henkel, 1980). Subsequent ground movements were reduced but not eliminated by shortening the panel length of slurry trench, so as to generate some support from arching action of the soil. Other mitigation measures were also adopted such as increasing the effective slurry pressure by raising the height of guide walls and slurry head. The MTR tunnels went through mixed ground, passing from soft to hard and back again, and machine boring could not be used. In soft ground, tunnel shields and compressed air were necessary to permit safe excavation, but settlements at ground level of up to 200 mm were recorded. Face collapse occurred on several occasions on passing from strong, fresh rock into decomposed granite.

An incident of excessive groundwater drawdown and significant settlement caused by tunnelling occurred in 1984 (Cowland & Thorley, 1985). This involved the construction of a seawater cooling tunnel for the Hongkong Bank in Central. The tunnel was at about 70 m below ground and was largely constructed in Grade II granite. The maximum drawdown of piezometric head recorded in the overlying fill and marine deposits was about 2 m, while the maximum drawdown in the Grades IV-V granite below was 25 m, located approximately 100 m to the west of the tunnel axis. The settlement pattern closely followed the drawdown pattern, with a total settlement of up to 100 mm, resulting in superficial damage to buildings and pavements. It appears that:

1 the marine deposits acted as an aquitard preventing recharge from the sea to the underlying weathered granite,
2 the deep diaphragm walls of a building provided a significant barrier to hillslope recharge,
3 the modest inflow into the tunnel drained the Grade V granite layer,

4 a good hydraulic connection existed between the Grade V granite and the underlying rock mass (as suggested by the rapid drawdown response), and
5 preferential drainage paths existed in the weathered granite which was only intersected by the tunnel in a few specific locations (as shown by the offset of the drawdown trough).

The HATS Stage I project carried out in the mid to late 1990s involved the provision of a system of deep tunnels, with a total length of 23.6 km. The tunnels are for transporting sewage from a number of catchments around Victoria Harbour to a treatment works on Stonecutters Island. Construction commenced in early 1995. The tunnels were excavated largely beneath the sea. During construction, serious problems arose due to heavy water inflow into sections of the tunnels (McLearie *et al.*, 2001). In addition, the water inflow led to extensive drawdown of the piezometric heads; in one section, significant ground settlement occurred up to 1.8 km from the tunnel, resulting in major public concern. The extent of drawdown was related to the occurrence of structural discontinuities within the rock mass as well as the consolidation characteristics of the overlying weathered rock and alluvial deposits. There are similarities in terms of the hydrogeological control and mechanisms of piezometric drawdown and ground settlement between this case and the case mentioned above.

For the HATS Stage I project, despite the fact that less than 5% of the tunnels went through difficult ground, there was significant delay in completion of the project and a substantial increase in cost. The extent and magnitude of the socio-economic impacts led to a review of the project by the Director of Audit (2004) and the Public Accounts Committee (LegCo, 2004). In essence, the delay and cost increase were attributed to problems in constructing the sewage tunnel system: difficult ground conditions, stringent water inflow specifications and major breakdowns in machinery, which resulted in the forfeiture of the two original tunnel contracts.

8 TUNNEL FAILURES

Relatively few tunnel failures have occurred in Hong Kong. Some of the incidents caused construction difficulties, while a few had led to serious consequences (see GEO (2007e)). Because of contractual and other reasons, there are few cases reported, especially recent cases, and those reported are usually of such scale or seriousness that they cannot escape public attention.

Problems were encountered during the construction of the Aberdeen Tunnel, which runs from Happy Valley at the north to Aberdeen at the south (Cochrane, 1984; Twist & Tonge, 1979). Construction of a pilot tunnel commenced in 1974, followed by a main tunnel contract in 1977. Tunnelling through granite from the north portal went smoothly, but at the south portal a 200 m wide dyke of decomposed monzonite caused severe engineering problems because of groundwater inflows. A short drive was found to be impossible to construct and was abandoned. Extensive grouting by the 'tube-à-manchette' method, using low pressure cement grout followed by a low viscosity sodium silicate grout, and deep well pumping (5-10 m into fresh rock or 5 m below tunnel level) was required before the portal could be formed, and this delayed the works considerably.

During construction of the second Beacon Hill railway tunnel in 1978, strong granite was found some 90 m from the south portal, as had been anticipated by the site investigation, and the contractor switched from soft ground tunnelling to full-face hard rock tunnelling using a drilling jumbo. Rock conditions deteriorated drastically and from 260 m to 620 m the face was in highly decomposed, moderately decomposed, completely decomposed or fresh granite, which made progress very difficult. A face collapse occurred at 600 m when the main groundwater table was encountered. At 1,100 m, after going through 48 m of hard rock, another zone of completely decomposed granite was encountered and caused a face collapse at a section where the overburden above the tunnel was at least 400 m thick (Lumb, 1991/1980). This sequence of events illustrates the great difficultly in estimating depths of decomposition in the granite without probing ahead.

Clay & Takacs (1997) reported two cases of tunnel collapse. The first incident was associated with construction of the MTR Modified Initial System in 1977. In this case the contractor was driving by drill and blast through sound granite towards an area of reclamation. The contractor was required to build a chamber for the installation of a shield and airlock, for the continuation of the drive through soft ground. Unfortunately the geology was interpreted in such a way that led the contractor to decide to continue with the drill and blast, just one round too far. The blast penetrated the shallow rock cover, allowing the water-bearing fill above to flow into the tunnel, opening a hole in the road above the tunnel.

The second incident was associated with the MTR Island Line. In this case the contractor was driving a station tunnel through weathered granite overlain by a fill layer. He had to treat the ground above the station tunnel from a pilot tunnel, as well as the ground above the running tunnel that joined the station tunnel. At both locations ground treatment was carried out, but unfortunately there was a gap between the ground treatment areas, through which the ground flowed into the tunnel, opening a hole in the road above. This illustrates the importance of checking the extent and quality of the grouting prior to excavation. In this incident, which occurred at Hennessy Road on 1.1.1983, 1500 m^3 of soil flowed into the tunnel, creating a void with a plan area of 100 m^2 and 30 m deep beneath the road surface. More than 150 people were evacuated.

Other notable failures in Hong Kong, reported in the newspapers, include the collapse of MTR Island Line tunnel sections at Shing On Street near Sai Wan Ho station (23.7.1983) and Shau Kei Wan (16.12.1983). In the first incident, there was an inflow of soil of about 40m^3 into the tunnel during tunnelling. This occurred through a small area about 1 m behind the excavation face (Cater *et al.*, 1986b). A high pressure fresh water main beneath the road slab burst, causing flooding and erosion, and a void was formed. A building settled by about 66 mm and tilted, and more than 400 people were evacuated. In the second incident, about 150 m^3 of soil flowed into the tunnel, leaving a void under the road during construction of the Sai Wan Ho Station. Significant ground settlement occurred and the water supply was disrupted. In all cases, emergency works and major remedial works had to be carried out, and much effort had to be spent in dealing with the public safety concern.

In 1995, a collapse occurred at a section of the Lantau Airport Railway drill and blast tunnel on East Lantau.

In 1996 a TBM production shaft for the HATS Stage I project in Chai Wan flooded due to excessive groundwater inflow and had to be abandoned. It was later backfilled and capped.

In 2004, during construction of a cable tunnel, an EPB TBM converted to work in open mode was flooded. The TBM was excavating ground which had been pre-grouted using tube-à-manchette techniques. The tunnel and cofferdam was flooded when water flowed in at the TBM face from the ground surrounding the grouted annulus, which is in close proximity to an existing buried seawall. The water inflow and flooding of the tunnel

caused a delay of two years while a rescue cofferdam was constructed for a replacement EPB TBM to be launched. The project is now completed.

A number of other minor collapses and ground failures have occurred. However, these are generally not reported in detail and often only anecdotal evidence reaches the public domain.

It is of interest to note that quite a number of tunnel failures worldwide have occurred outside the normal working hours of senior professional staff. This highlights the importance of provision of qualified supervision staff with access to senior professionals over such periods, as tunnelling is quite often a 24-hour 7-day operation and failure could occur even during non-excavating periods. It is important that there is sufficient professional input at all times, to identify the geotechnical and tunnelling method-related hazards and the associated risk of collapse and excessive deformation, and to ensure implementation of effective risk control measures in a timely manner during construction.

9 GEOTECHNICAL CONTROL AND RISK MANAGEMENT

9.1 *Recent developments*

Systematic identification and management of construction risks is becoming increasingly common practice in many areas of the construction industry and, in some jurisdictions, has become a regulatory requirement as part of construction site safety requirements. In Hong Kong, the "Tang Report" (CIRC, 2001) on Construct for Excellence pointed towards the need to adopt similar practices in Hong Kong's construction industry.

With regard to tunnel works, a review was carried out by the HKSAR Government in 2004 to examine the implementation issues of the HATS Stage I project, after experiencing the construction problems associated with major water inflows (see Section 7). The review was led by the Environment, Transport and Works Bureau (ETWB) with input from Government departments including DSD and GEO. During this process, the GEO carried out a review of the technical literature on the subject of risk management of tunnel works. This indicated that many problems in tunnelling, particularly those that resulted in a direct impact on the public, were due to inadequacies in the management of geotechnical risks. These geotechnical risks were either, as Barton (2004) has suggested, often a result of an unexpected combination of factors or the unexpected magnitude of a single factor. Such incidents resulted in significant losses to clients, contractors, consultants and the insurance companies worldwide.

Internationally, because of the tunnel failures experienced in recent years, the insurers' perception was that the tunnelling industry had an inconsistent approach to risk management, to the extent that it threatened to withdraw the provision of insurance coverage to the tunnelling industry as a whole (Mellors & Southcott, 2004). As a way forward, the insurers worked with the tunnelling industry to develop a "Joint Code of Practice for Risk Management of Tunnel Works", which was published by the Association of British Insurers and the British Tunnelling Society in 2003. This became the forerunner of the international code entitled "A Code of Practice for Risk Management of Tunnel Works" published by the International Tunnelling Insurance Group (ITIG, 2005) in late 2005.

The Code of Practice emphasizes the importance of risk management in all stages of a project, i.e. project development, construction and contract procurement, design, and construction stages. It promotes identification of hazards and the associated risks during all four stages, preparation of risk registers, cascading the registers throughout the project to ensure that all parties are aware of the previously identified hazards and risks, continuous review and updating of the registers throughout the project, and identification of a party to be responsible for managing each element of risk. It highlights the need for the project client to take proactive action and responsibility in risk management. It also requires the project client to carry out adequate site investigation and to prepare (or have prepared on its behalf by a competent agent) "ground reference conditions" for projects involving tunnel works.

The KCRC has followed the key recommendations of the Joint Code of Practice in the implementation of its Kowloon Southern Link project, which is currently under construction. The project involves construction of 3 km of running and station tunnels, using both cut and cover methods and a large diameter slurry TBM. KCRC has incorporated relevant provisions of the Code into the contract documents for this project.

In 2005, a technical guidance document, TGN25, on geotechnical risk management was issued by GEO (GEO, 2005b). The preparation of this TGN also had significant input from members of the HKIE Working Group. TGN25 refers to, and has incorporated the essential elements of, the International Code of Practice for Risk Management of Tunnel Works. It also refers to an administrative instruction issued by the HKSAR Government, ETWB TC(W) No. 6/2005, on Systematic Risk Management, which applies to public works projects with cost estimates exceeding HK$200M (US$25.6M); geotechnical risk management for tunnel works being an integral component of the systematic risk management for the overall project.

In the same year, the ETWB also issued ETWB TC(W) No.15/2005 on Geotechnical Control for Tunnel Works. Under this circular, the GEO audits the geotechnical aspects of design submissions on government tunnel works and the adequacy of the project's site supervision and geotechnical risk management provisions. The GEO will also conduct site audits on the implementation aspects. The scope of these audits is on risk to public life and property. This parallels the GEO's existing role to provide a geotechnical advisory service to the Building Authority on private tunnel works controlled under the Buildings Ordinance.

From the HKSAR Government's perspective as a regulator, a key aim of geotechnical control and risk management in tunnel works is to ensure that the works do not affect adversely public life or property. To help achieve this, it becomes the responsibility of the project client, with due advice from the project manager and an experienced geotechnical professional, to ensure that adequate resources are provided and an adequate system is in place for the management of geotechnical risks in the construction of such works. The implementation details should take into account the level of risk to life and property.

Being the HKSAR Government's in-house geotechnical advisor, the GEO has set up a Panel on Tunnels, chaired by a Deputy Head, to provide a one-stop service to advise government departments on the geotechnical aspects of tunnels works. In order to provide an effective advice, the Panel coordinates the input from expertise within the GEO in the various fields, including geology, engineering geology, ground investigation, laboratory testing, geotechnical risk management and blasting regulatory control. A Consultative Group on Blasting Regulations has also been set up to work with the industry to further improve blasting practice including for tunnel projects.

Compliance of the Code of Practice is effectively mandatory if the client wishes to transfer some of the contract risk to the insurance market. TGN25 advises that insurance of the risk does not

remove the need, or reduce the responsibility of the client, to ensure safety is properly managed. In complying with the Code, insurers expect availability of ground reference conditions. These may be put together by the geotechnical professionals preparing the contract, or by the tenderers as part of tender submissions.

Apart from those arising from geotechnical hazards, some geotechnical risks are construction method-related. Consequently, evaluation of the tunnel alignment, layout design and construction methods is an important step in managing such risks, in that exclusion of particular layout designs or construction methods could avoid specific risks. In some cases, the contract may need to exclude designs or construction methods that are not acceptable based on risk management considerations.

The background to the recent developments in the geotechnical control and risk management for tunnel works in Hong Kong is given in Pang *et al.* (2006).

9.2 Current practice

In Hong Kong, the current practice is that the geotechnical risks of a tunnel works project with respect to public safety are examined in a geotechnical assessment or geotechnical risk assessment report. A blasting assessment report is also prepared if rock blasting is to be carried out. The risk of ground collapse and the risks associated with excessive ground deformation, ground vibration and/or groundwater inflow and drawdown, as well as effects on life and property are assessed. The geotechnical risk assessment is conducted from the early project planning stage, so that adequate funding and time are allocated to manage the risks during design and construction.

The geotechnical risks during construction are managed by careful contract specification, clearly indicating the performance measures and limits, allowing for the major items of risk mitigation works anticipated and contingency measures, and putting in place a quality management system with experienced supervision personnel to ensure timely provision of adequate ground support and ground treatment, and effective monitoring and review of the construction effects and risks to life and property. For works for which the design responsibility is assigned to the contractor (e.g. Design and Build contracts), the pre-tender reference design is required to be carried out to a good level of detail, taking into account the geotechnical risks, and providing for robust risk mitigation works to be carried out and paid for under the contract should these be found to be necessary by the Engineer or Supervising Officer during construction.

In undertaking the geotechnical risk assessment, existing buildings, structures and other facilities affected are surveyed, studied and classified in terms of their condition and time of construction, and for buildings and structures an assessment of the potential damage is undertaken. KCRC has adopted the 'slight damage' category (as defined by Burland *et al.* (1977) and modified by Boscardin & Cording (1989)) in their contracts for the Kowloon Southern Link project. The contracts require that the allowable limits set by the authorities and major owners should not be exceeded. Contract performance/action limits, normally expressed in terms of Alert, Action and Alarm levels with corresponding actions defined under the contract, are also specified together with the methods of measurement. The validity of these limits is subject to confirmation by the responsible design professional, for risk control purposes.

The assessment of ground settlements due to construction of the tunnels and the associated temporary works, and its effects on adjacent buildings and other facilities, is usually carried out using the three-stage risk assessment approach recommended by Burland *et al.* (2001), allowing different degrees of geotechnical input and analyses to be applied depending on the level of risk and the stage of the project.

The assessment of piezometric drawdown and its effects is carried out where the tunnel is to be constructed under a groundwater table. This is done using conventional consolidation theory. The aim is to arrive at allowable piezometric drawdown levels and allowable water inflow limits along the tunnel alignment. Account is taken of the site-specific data on hydrogeology and structural, stability and maintenance conditions of the sensitive receivers. Reference is made to the known past fluctuations in groundwater pressures and ground settlement and vibration levels.

Local data on settlements due to construction of bored tunnels (including the temporary walls) and their effects were reported by Davies & Henkel (1980), Morton *et al.* (1980a,b), Howat & Cater (1983), Cater *et al.* (1984), Budge-Reid *et al.* (1984), Cowland & Thorley (1984, 1985), Cater & Shirlaw (1985), Cater *et al.* (1986a,b), Thorley *et al.* (1986), Stenning *et al.* (2001), Norcliffe *et al.* (2002), Salisbury & Hake (2004), Mok (2006) and Wang *et al.* (2006). These cover a wide range of ground conditions and construction methods. Guidance is also given in GCO Publication No. 1/90 on the assessment of ground movements due to wall construction, dewatering and bulk excavation in the construction of deep excavations.

In the case of drill and blast, the assessment of the effects of ground vibration is carried out using available attenuation relationships of blasting waves, site-specific data on ground and groundwater conditions, and information on the structural, stability, and maintenance conditions of the facilities affected. Local data on blasting wave attenuation were reported by Smith & Morton (1986), Clover (1986), Troughton *et al.* (1991), Sekula & Johansson (1998), Zou (2002) and Murfitt & Siu (2006a,b). For slopes, blasting effects on slope stability are assessed. The guidance given in GEO Report No. 15 (Wong & Pang, 1992) is commonly used.

In managing the geotechnical risks, clients now provide site supervision with significant input from geotechnical professionals with tunnelling and where appropriate blasting and TBM construction experience. Contractors are required to provide competent persons under the contract for safety-critical works and operations.

9.3 *Contract risk*

Other than management of geotechnical risks with respect to life and property, the tunnelling industry and client organizations in Hong Kong have recently been reviewing the contract risk sharing mechanisms in tunnel contracts, including the sharing of geotechnical risks. The current situation is that the geotechnical risks in tunnel contracts are largely allocated to the contractor. The industry is looking for more equitable risk sharing to be achieved. There is currently much discussion on whether the client should provide interpretive geotechnical reports and ground reference conditions to the tenderers. The use of Geotechnical Baseline Reports and, if they are to be used, what parameters should be used for setting the baselines, and what methods of measurement should be adopted to achieve consistent and repeatable remeasurement of cost and time when differing geotechnical conditions are encountered during construction, is being further debated.

It is of interest to note that in the tunnel contracts for the MTR Modified Initial System, borehole information together with details of the assumptions made by the Engineer in preparing the outline (reference) design were provided to the tenderers. This included assumed positions of soil/rock interfaces, the approximate

locations where soft marine soils would be encountered and the need for compressed air and/or ground treatment (Haswell *et al.*, 1980).

In some of the past rock tunnel projects, the costs of major items of tunnel support and groundwater control works are re-measured and paid for, e.g. the Tseung Kwan O road tunnel (Matson & Robinson, 1984), the Tolo Harbour Effluent Export Scheme (McFeat-Smith, 1992) and the CLP Kwai Chung cable tunnel.

On the KCRC Kowloon Southern Link contracts, KCRC provided all geotechnical data available (Geotechnical Data Reports), interpretations (Geotechnical Basis of Design Reports) and risk assessments (Existing Buildings and Structures Reports, Ground Movement Prediction Reports and Geotechnical Instrumentation Reports) to the tenderers and required the tenderers to provide their geotechnical interpretations and risk assessments which were then reviewed in the tender assessment exercise of each contract.

10 CONCLUSION

Many tunnels have been successfully constructed in Hong Kong. Techniques for site investigation, construction, design and risk management have progressively been developed, and engineering standards and practice, in particular in relation to geotechnical risk management, have evolved. The establishment of a Panel on Tunnels within GEO and a blasting consultative group comprising GEO staff and representatives from the professional bodies and the industry, both chaired by a Deputy Head in GEO, and the setting up of a Working Group on Cavern and Tunnel Engineering in the HKIE Geotechnical Division in recent years have resulted in synergy of expertise in different sectors and technical fields and effective consolidation of local knowledge and experience. This will be beneficial to the further development of tunnel engineering in Hong Kong.

With continuing demands for new and replacement public and private facilities, there is potential for significant tunnel and underground space development in Hong Kong in the future. The future development of projects is likely to be influenced greatly by factors such as availability of suitable surface land for new and replacement facilities, and environmental, traffic and transport impact considerations.

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Digital Technology in Geotechnical Engineering

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Abstract: Digital technology offers an opportunity for advancing geotechnical practice and enhancing the capability and efficiency of geotechnical work. This paper describes a number of novel digital technologies, which have notable potential for geotechnical application in Hong Kong. These include digital photogrammetry, Geographic Information System (GIS), Interferometric Synthetic Aperture Radar (InSAR) and Light Detection and Ranging (LIDAR). Their principles, capability and applications are presented. These technologies have been identified by the HKIE Working Group on Application of Innovative Technology in Geotechnical Engineering as novel techniques that warrant strategic attention.

1 INTRODUCTION

Significant advances have been made in digital and related novel technologies over the years. The technologies have become more readily accessible, with improved capability and reduced cost. Hong Kong is one of the parties taking the lead in applying novel technologies to geotechnical work (Wong 2001), and some of these applications are recognized as the state-of-the-art that has brought about enhanced geotechnical capability (SSTRB 2003 & 2004).

In recognition of the importance of novel technologies and their potential application, a Working Group on Application of Innovative Technology in Geotechnical Engineering was set up by the Geotechnical Division of the Hong Kong Institution of Engineers (HKIE) in mid-2003, with members from academic institutions, Geotechnical Engineering Office (GEO), and geotechnical practitioners in Hong Kong. The Group is tasked with developing knowledge and promoting geotechnical application of innovative technologies, which would help the geotechnical profession to meet new challenges in Hong Kong and to lead the professional excellence in the region.

The Group has identified the following eight strategic subjects for examination and promulgation of their geotechnical application:

(a) construction control and technology;
(b) digital photogrammetry;
(c) Geographic Information System (GIS);
(d) Global Positioning System (GPS);

Fig. 1. Stereoscope used in conventional aerial photograph interpretation.

(e) instructmentation;
(f) Interferometric Synthetic Aperture Radar (InSAR);
(g) Light Detection and Ranging (LIDAR); and
(h) new geotechnical subjects, including environmental geotechnology, natural terrain landslide hazard and quantitative risk assessment.

Items (b), (c), (f) and (g) above directly involve the use of novel digital technologies. Their technological principles, capability and applications are highlighted in this paper. Item (d) above is partly related to GIS and LIDAR application, and is also described in such context in this paper. Particular reference is made to three other papers authored by members of the Working Group. These papers give a detailed account of selected application cases, which include ground movement detection using InSAR (Ding et al. 2004a), mobile GIS mapping (Ng et al. 2004) and boulder identification using image analysis techniques (Shi et al. 2004).

2 DIGITAL PHOTOGRAMMETRY

Hong Kong has a comprehensive collection of historical aerial photographs that are accessible to the public. These aerial photographs have long been an important source of information for geotechnical studies. Conventionally, aerial photograph interpretation (API) and photogrammetric analysis are carried out by skilled personnel using stereoscope and stereo-plotter (Fig. 1). The work can now be undertaken by digital means via digital photogrammetry, with improved efficiency, resolution and analytical capability. It is carried out by digitizing a pair of aerial photographs with the use of a high precision and high resolution scanner, processing the digitized data together with the available control point data by an advanced digital photogrammetry algorithm, and displaying the stereo-images and processed data on a computer monitor (Fig. 2). Standard, off-the-shelf hardware and software packages are available for digital photogrammetric analysis and presentation of the results. The set-up cost is around HK$0.5 million. After setting up, the digital photogrammetry system is neither costly nor difficult to operate, particularly for personnel with API or surveying experience.

Digital photogrammetry has a number of notable applications to geotechnical work. These include:

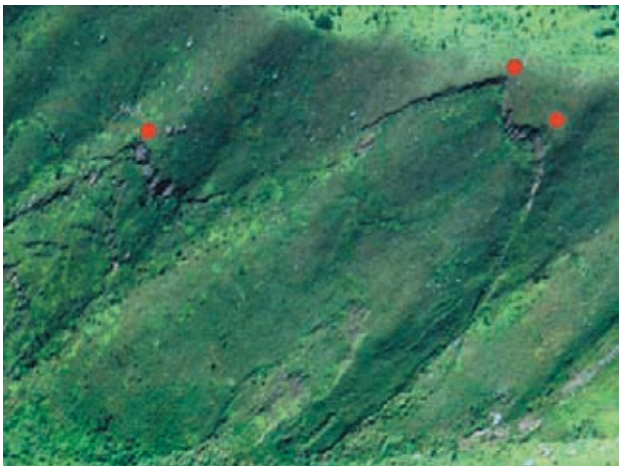
Fig. 2. Digital photogrammetry workstation (Note: stereo view displayed on computer monitors).

Fig. 3. Aerial photograph interpretation by a team using digital photogrammetric technology.

(1) Stereo visualization and API
With the ability to display 3-D stereo images on a computer monitor, stereo views of the present and past conditions of a site can be generated by the available historical aerial photographs and stereo visualization is easy, even for people without API training. Furthermore, it is feasible for API to be done and evaluated jointly by a team (Fig. 3), which greatly facilitates communication and discussion. Good resolution can be achieved, e.g. up to about 0.1 m for vertical aerial photographs taken at 4,000 feet. Remote sensing and image analysis techniques can be applied to the digitized images for terrain evaluation and feature identification. This provides an objective, consistent and less operator-dependent means of analyzing the aerial photographs, which supplements conventional API using stereoscopes.

(2) Surveying and measurement
Comprehensive data on topography and feature dimensions can be derived from a pair of aerial photographs by digital photogrammetry, without the need for detailed land surveying work. A spatial accuracy of about 0.5 m to 1 m is achievable in Hong Kong, and this can be further improved with the use of low-flight photographs and additional ground control points. This accuracy can meet the need of topographic survey and measurement for many routine geotechnical applications, e.g. slope design. In this respect, digital photogrammetry offers an efficient and inexpensive means of remote surveying and measurement. This is particularly useful for geotechnical work that covers a large area, e.g. assessment of natural terrain hazards, and for circumstances where field measurement is dangerous or not possible, e.g. in areas with access problem or unstable slopes.

(3) Movement monitoring
Movement monitoring normally requires a higher degree of accuracy than that commonly required for visualisation and measurement purposes. Hence, digital photogrammetry using conventional aerial photographs (4,000 feet or above) would have limited use in movement monitoring, unless where the movement to be measured is large. Fig. 4 shows an example of such application undertaken by the GEO at a large scale landslide site at Ma On Shan. Use of low-flight aerial photographs (e.g. taken at 500 feet) or terrestrial photogrammetry can give a better accuracy (e.g. Hansen & Lichti 2002, Li & Mo 2003), and there is scope for technological development in this area.

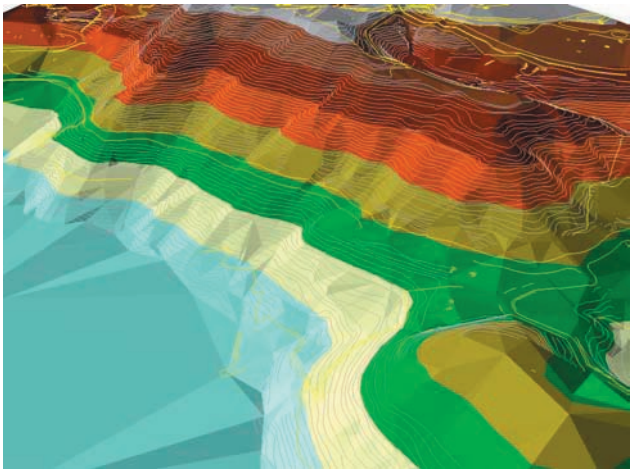


(a) Ma On Shan landslide site

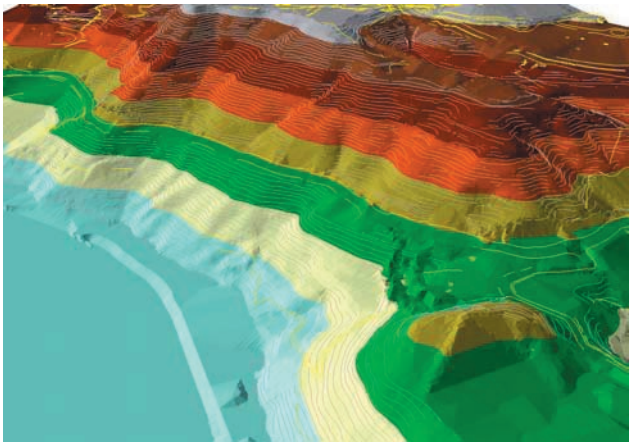
(b) Marker installed on site for use as reference point in digital photogrammteric analysis

Fig. 4. Slope movement monitoring at Ma On Shan by digital photogrammetry.

(4) Compilation of digital terrain model
Digital terrain model (DTM) is a specific surveying and measurement product of digital photogrammetry that **has**



(a) 5 m-grid DTM



(b) 2 m-grid DTM with enhancement (Note: roads and man-made slopes are correctly modelled in the DTM)

Fig. 5. Enhanced 2 m-grid digital terrain model (DTM).

important 3-D GIS and virtual reality application. Hong Kong has a full set of 1:1,000 Land Information Centre topographic maps, which can be used to generate DTM. Digital photogrammetry can provide supplementary DTM for areas where elevation data (i.e. contour lines and spot heights) are missing, or where more accurate DTM is required, or where the DTM of the site conditions in the past is required. However, in generating DTM by digital photogrammetry, due account should be taken to map the ground surface in area where vegetation is present. The GEO has completed a project to upgrade the HK-wide 5 m-grid DTM to 2 m-grid. Improvement has also been made in the provision for breaklines, such as drainage lines, roads and slopes, to remove artifacts and enhance the accuracy of the DTM in modeling ground features (Fig. 5). This DTM will enable a more realistic representation of the terrain components and morphology, as well as improve the accuracy of geotechnical and other spatial assessments that require the use of 3-D topographic data.

(5) Production of ortho-rectified images

By application of digital photogrammetric techniques, conventional aerial photographs can be converted into ortho-rectified images. Such images are true to scale and position-accurate, which can supplement or even replace survey plans (Fig. 6). Ortho-rectified images contain rich visual details of the ground at the time of taking the aerial photographs and are most suited for use in field reconnaissance and mapping. A list of the HK-wide ortho-rectified images available in the GEO is given in Table 1. As ortho-rectified images are in digital and geo-referenced format, they can be integrated into a GIS together with other spatial data for a range of novel GIS and remote-sensing applications. These include: geotechnical field mapping, GIS data mining, feature recognition and extraction, change detection and monitoring, and virtual reality. A notable application to mapping of boulders using image processing techniques is reported in Shi et al. (2004).

3 GEOGRAPHIC INFORMATION SYSTEM (GIS)

3.1 Development of Capability

Geographic Information System (GIS) has been adopted in Hong Kong for several years in recording and managing geotechnical data. By nature, most geotechnical data contain spatial attributes on their geographic location (x, y, and z) and on the geometry of the ground/object (e.g. point, line or polygon) represented by the data. Managing the data in GIS would register the spatial attributes and permit the use of the attributes in GIS-related application, resulting in improved capability and efficiency.

(a) Ordinary aerial photograph
(Note: not to scale and position-inaccurate)

(b) Ortho-rectified image
(Note: true to scale and position-accurate)

Fig. 6. Conversion of ordinary aerial photograph to ortho-rectified image.

Table 1. List of HKSAR-wide Ortho-rectified Images held in GEO.

Type of Image	Year	Section 1.01 or	Col Section 1.02 esolution
Satellite images (LandSat, SPOT 40+ & IKONOS sets)	1987 onward	Color	10 m +
Low-flight aerial photographs	1963 (1)	Black and White	0.1 - 0.5 m
	1973/74 (1)	Black and White	0.5 m
	1982 (1)	Black and White	0.5 m
	1993 (1)	Black and White	1.0 m
	2000 (2)	Color	1.0 m
	2001 (2)	Color	1.0 m
	2002/3 (2)	Color	0.5 m
	2004	Color	1.0 m
	2005	Color	0.5 m
Infra-red images	2000 (3)	Infra-red	0.35 m
	2000 (4)	False-color	0.35 m

Notes: (1) ortho-rectified by GEO
(2) ortho-rectified by Lands Department
(3) compiled by Chinese University of Hong Kong
(4) converted by GEO

Given its geotechnical mandate, since early 1990s the GEO, with support from practitioners, has been developing geotechnical spatial datasets and GIS capability and promoting GIS application to geotechnical work in Hong Kong. In the early years, the development and application work focused on the following areas:

(1) Compilation of GIS datasets, including conversion of existing data into GIS format and collation of new data. Important GIS datasets that are frequently used in geotechnical work (Fig. 7) include:

(a) Topographic and land information maps covering the whole of HKSAR in various scales, up to 1:1,000 scale
(b) 1:20,000 scale geological maps covering the whole of HKSAR and 1:5,000 scale for some development areas
(c) A catalogue of 57,000 registered man-made slopes including slope related information
(d) An inventory of over 100,000 historical natural terrain landslides
(e) An inventory of about 2,000 large (scar > 20 m wide) natural terrain landslide features
(f) An inventory of landslide incidents reported to the GEO since 1984
(g) 5-minute rainfall data since 1985 from about 100 nos. of raingauges
(h) Ground investigation data
(i) 5 m-grid and the updated 2 m-grid DTM

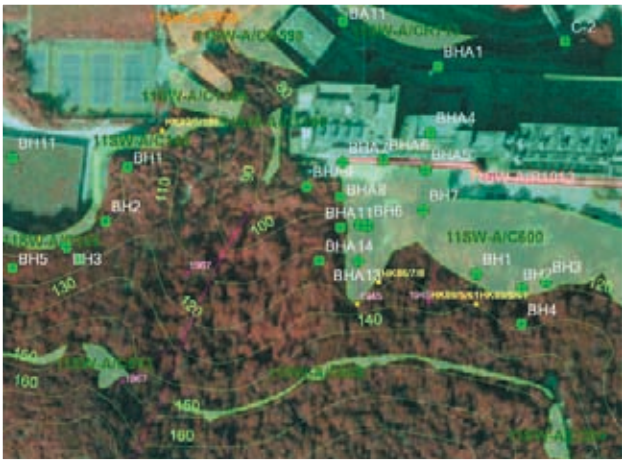


Fig. 7. Selected geotechnical GIS datasets (Note: ortho-rectified infra-red image of year 2000, contour lines, registered man-made slopes, historical natural terrain landslides, reported landslide incidents and ground investigation data as shown).

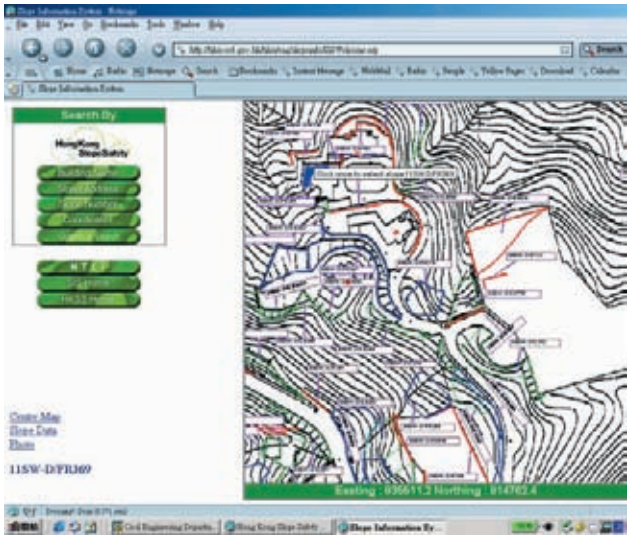


Fig. 8. Slope Information System.

(j) About 50 sets of ortho-rectified images covering the whole of HKSAR (Table 1)

(2) Development of GIS systems and capability, such as:

(a) Setting up software and hardware systems
(b) Developing GIS skills among professional and technical users, typically through short courses and on-the-job training
(c) Integrating GIS datasets and setting up centralized database for enhanced system management
(d) Acquiring Intranet, Internet and mobile GIS capabilities for 'enterprise-based' and other functional GIS applications, which involve use of specialized GIS modules, e.g. Spatial Database Engine and Internet Map Server for different scalable GIS applications. This development was however confined to key GIS users, such as the GEO and the Jockey Club Research and Information Centre (JCRIC) for Landslip Prevention and Land Development in the University of Hong Kong.

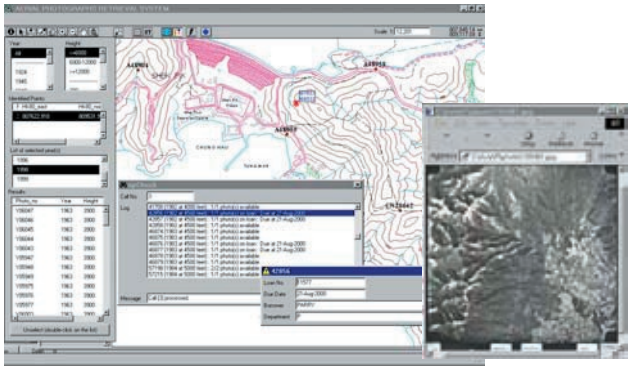
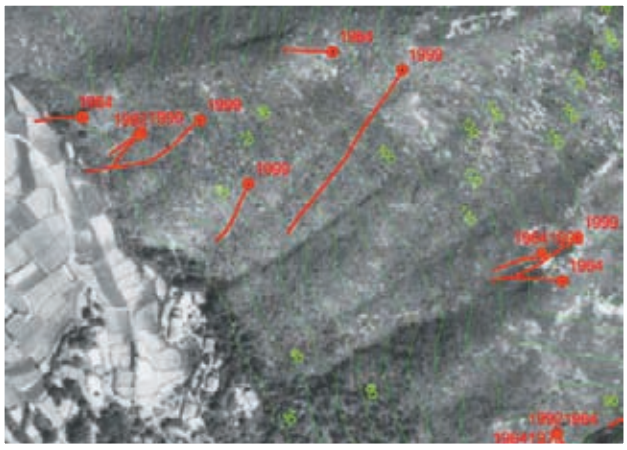
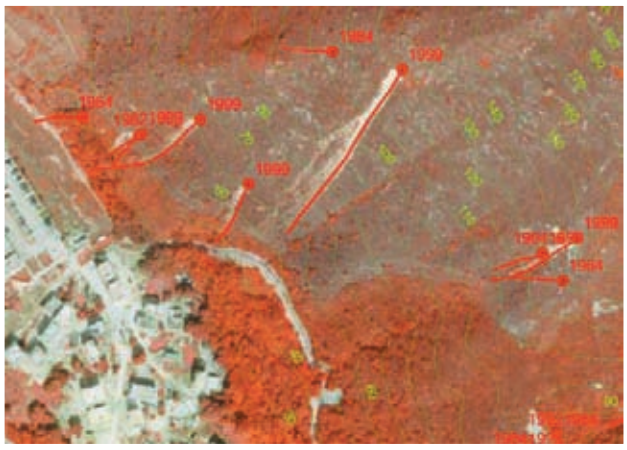


Fig. 9. Aerial Photograph Management System (Note: the system supports spatial search of aerial photographs covering the site concerned from over 120,000 nos. aerial photographs held in GEO's Aerial Photograph Library)

(3) GIS applications, which mainly focused on data management and information services. A number of GIS systems were set up in Hong Kong in the late 1990s and early 2000s, e.g. GEO's Slope Information System (Fig. 8), Fill Management System, Geological Modeling System and Aerial Photograph Management System (Fig. 9), and the Comprehensive Ground Information System and Electronic Mark Plant Query System of JCRIC. These GIS systems are primarily used for data management and information services, with basic GIS search and browsing functionalities provided to users, without using a GIS software to interact with the systems. The Slope Information System, in particular, is widely used by geotechnical practitioners and the public. The System has won many awards, including the prestigious 'Geospatial Achievement Award - Certificate of Merit' in 2003.



(a) Site in 1963



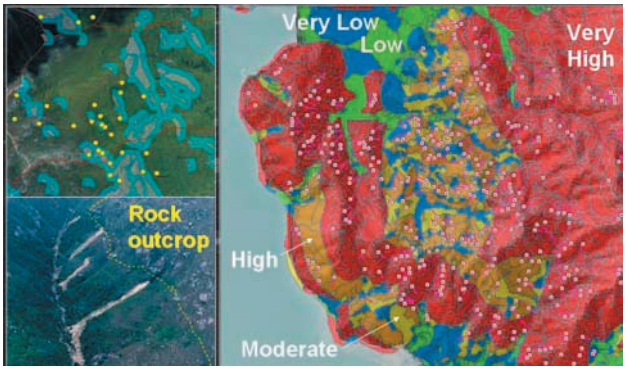
(b) Site in 2000 (Note: there are recent landslides on the natural hillside and new developments below the hillside)

Fig. 10. Use of GIS in geotechnical desk study.

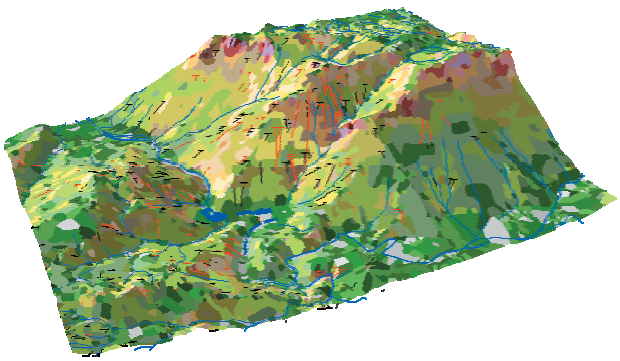
3.2 Application Trend

While GIS data management and information services remain important, the trend in recent years is to adopt more advanced GIS functionality to deal with geotechnical issues in Hong Kong. This is possibly the result of building up of GIS capability among the geotechnical profession as well as increased demand for use of GIS in solving geotechnical problems. Some notable applications are described below:

(1) Advanced GIS search, browsing, editing and publication
This is performed by more skilled GIS users via the use of GIS tools, say, in a geotechnical desk study to examine the available geotechnical data, review the site history and assimilate the key information for presentation (Fig. 10). Advanced GIS search would enable users to query and retrieve data that meet certain prescribed criteria or geographic relationship. Examples of such application include delineation of area of deep rock weathering using ground investigation data, search of man-made slope features for stability assessment, and identification of sites affected by historical natural terrain landslides.

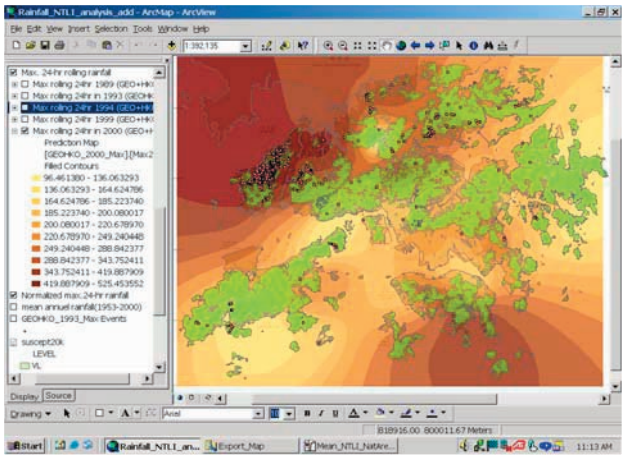


(a) GIS analysis and correlation of terrain with different levels of landslide susceptibility

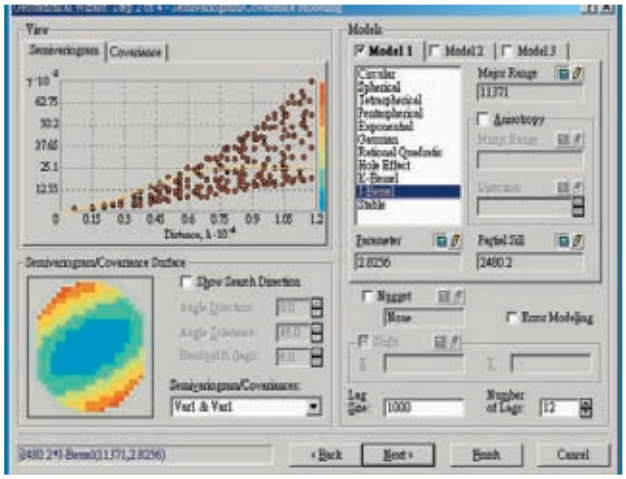


(b) 3-D visualization of historical landslides and terrain with different slope gradients

Fig. 11. GIS-based natural terrain landslide susceptibility analysis.



(a) Year 2000 maximum rolling 24-hr rainfall and natural terrain landslide locations

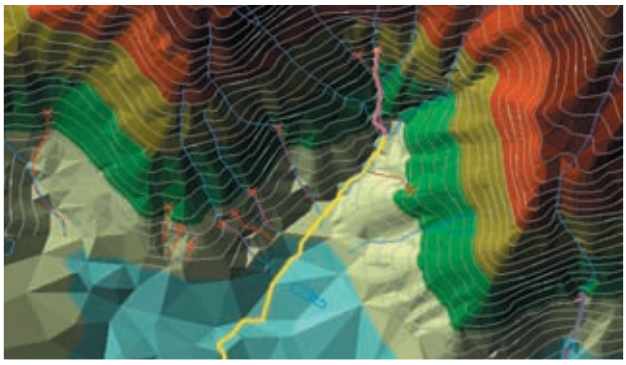


(b) GIS-based geo-statistical analysis

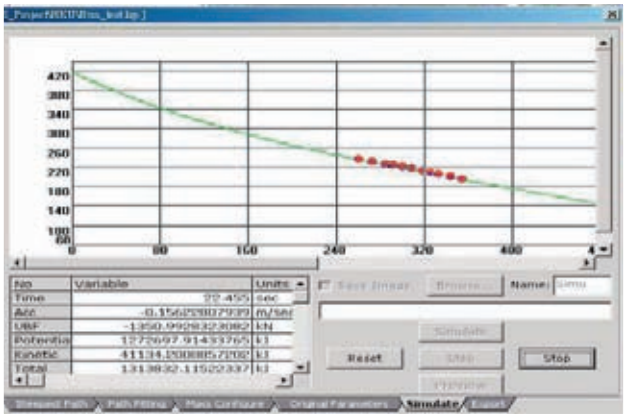
Fig. 12. GIS analysis of natural terrain landslide-rainfall correlation.

(2) GIS analysis

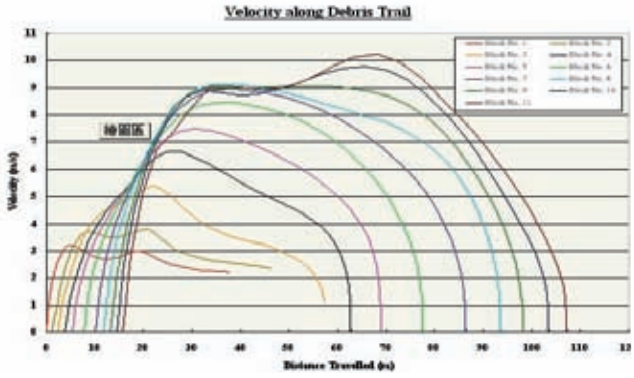
GIS analysis can be performed efficiently to examine the relationship and correlation among different spatial data, which are difficult to analyze using conventional means. It is now used commonly in natural terrain landslide susceptibility analysis (Fig. 11), as part of Natural Terrain Hazard Study (e.g. Evans & King 1998; Dai & Lee 2002; OAP 2003; Halcrow 2003; Wong 2004). GIS analysis also offers a unique capability in geotechnical research and development work involving spatial analysis of geotechnical data. Fig. 12 shows a recent example where the correlation between natural terrain landslide density and rainfall intensity in Hong Kong has been established using GIS analysis together with GIS-based geo-statistics (Ko 2003).

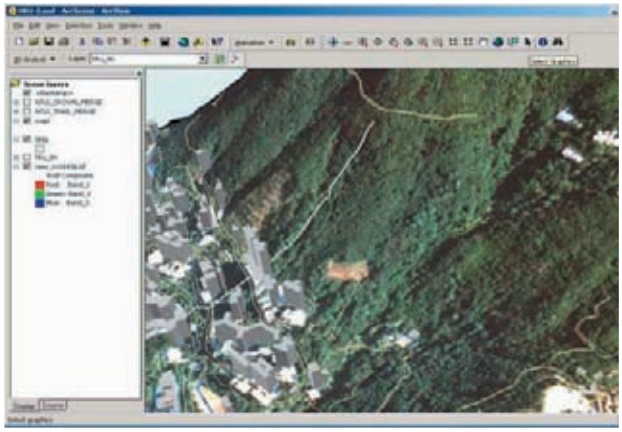


(a) Generation of debris flow path



(b) Generation of debris flow path





(c) Presentation of results

Fig. 13. GIS modeling of debris mobility.

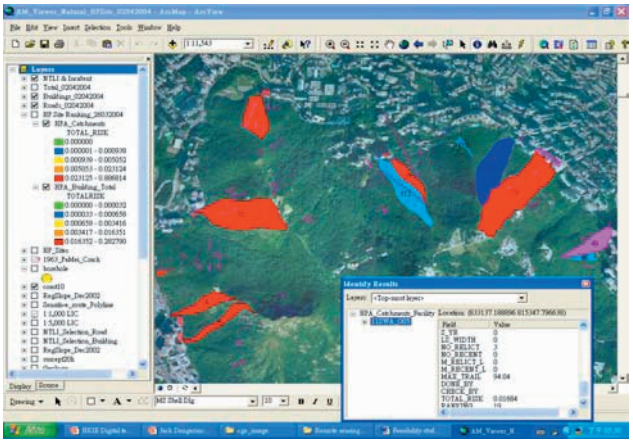


Fig. 14. GIS-based landslide quantitative risk assessment (Note: hazard and consequence models are incorporated into the GIS for calculation of risk using the relevant spatial data of the catchments and facilities at risk; catchments and facilities with different levels of calculated risk shown in different colours).

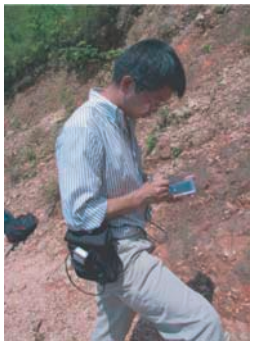
(3) GIS modeling

In addition to GIS analysis that is principally applied to assessing the spatial relationship among the data, performing GIS-based geotechnical analysis and numerical modeling based on application of engineering principles and governing physical laws has become increasingly important. Such application integrates engineering analysis with GIS, and is a powerful modeling tool, particularly for dealing with geotechnical subjects that involve the analysis of the geographic and engineering attributes of a large amount of spatial data. Development of GIS modeling applications requires GIS programming input from skilled personnel. Two geotechnical examples of such application in modeling the mobility of landslide debris and natural terrain landslide quantitative risk assessment are shown in Fig. 13 and 14, respectively.

(4) Mobile, location-based application

Geotechnical professionals spend considerable time and effort in field work, which is an important component of geotechnical practice. GIS can now be brought to site and applied to field work, by uploading the relevant datasets onto a mobile GIS platform that operates on a pocket computer. When integrated with a Global Positioning System (GPS) for detecting the spatial location on site, a mobile GIS system can guide on-site navigation to the point of interest (Wong 2001). In addition, the spatial data relevant to the site can be retrieved for location-based applications. The GEO has pioneered the development of a GIS-GPS mobile mapping system that also incorporates the use of ortho-imagery. The system is equipped with wireless telecommunication via the Internet with GEO's GIS Internet Map Server, for GIS data transfer for use in geotechnical field work (Fig. 15). This system was granted an international 'Special Achievement in GIS Award' in 2002 in recognition of its technological advances and benefits to the engineering field. Details of the system and its application are described in the paper by Ng et al. (2004).

(a) Uploading GIS data to the mobile mapping system



(b) On-site navigation, data retrieval and mapping using the mobile mapping system

(c) GIS data and ortho-rectified image together with GPS location shown on pocket computer

Fig. 15. Geotechnical mobile GIS mapping system.

(5) 3-D visualization and virtual reality applications

3-D GIS technology is maturing. Production of 3-D topography, city model, etc. has become relatively easy using the available spatial data, DTM and ortho-rectified images. Geotechnical professionals with GIS skills can now generate 3-D ground model and visualize the site conditions (Fig. 16) on their desktop computers. This facilitates geotechnical studies, e.g. in reviewing historical landslides and terrain evaluation (Wong 2001). Virtual reality animation and computer 'fly-through' can be produced for presentation and evaluation purposes without the need for input from specialists in 3-D computer graphics.

4 INTERFEROMETRIC SYNTHETIC APERTURE RADAR (InSAR)

Interferometric Synthetic Aperture Radar (InSAR) is an emerging remote sensing technology that could measure ground displacements with millimeter-level accuracy. Each pixel of a Synthetic Aperture Radar (SAR) image contains information on the phase of the signal backscattered from the terrain surface. By applying interferometry to a pair of SAR images of an area, the geometry of the two slightly displaced, coherent observations

of the surface would give their phase difference, which is a function of the surface height. Through repeated observations, it is possible to measure the surface displacement if ground movement has occurred over the observation period.

InSAR has been successfully applied to measurement of surface movement over large area induced by earthquakes (e.g. Massonnet et al. 1993), volcanic activities (e.g. Lu et al. 2002), ground subsidence (e.g. Fielding et al. 1998), etc. Some cases of application to detection of slope movement have also been reported (e.g. Strozzi et al. 2001). It is potentially a promising low cost, high accuracy remote sensing technique for geotechnical use, particularly for ground and slope movement detection.

Fig. 16. 3-D virtual reality model of Tsing Shan Foothills.

InSAR has been tried with encouraging results in Hong Kong in detection of reclamation settlement (Ding et al. 2004b). In collaboration with the Hong Kong Polytechnic University, the GEO has recently completed a trial application of InSAR for detection of slope movement and ground deformation at four selected sites in Hong Kong. The trial study shows that the available satellite-based SAR images are not very suitable for reliable InSAR analysis for the typical urban setting in Hong Kong (Fig. 17). Further information on InSAR application in Hong Kong is reported in Ding et al. (2004a).

A realistic appreciation of the current limitations of the technique and good awareness of its development potential are essential to successful application. In this respect, the following constraints on using of satellite-based InSAR in Hong Kong are note-worthy:

(a) Many of the available SAR images have large Doppler Centroid Frequency and low coherence, particularly for long time-span. These result in poor quality interferograms, which affect the reliability of InSAR analysis.

(b) SAR images have low spatial resolution (each pixel typically > 10 m). Hence, the interferogram only shows the average condition for each pixel and any surface movement that may be measured is at best representing the overall movement in the pixel. Hence, InSAR is more suitable for use in spatially extensive sites that are without drastic changes in surface profile and significant variations in ground movement over short distance. If the site to be measured is small in size (e.g. a relatively small-sized man-made slope) or if local changes in ground profile and ground movement are significant, it is difficult to apply satellite-based InSAR.

(c) InSAR results can be seriously affected by noises, such as those arising from geometric distortion, atmospheric effects and temporal decoration. Geometric distortion can be a serious problem for applying InSAR to steep terrain in Hong Kong. Also, the relatively humid environment and presence of thick vegetation imply that atmospheric effects and temporal decoration would be more significant in Hong Kong, than in other countries that have a more favorable site setting for successful use of InSAR.

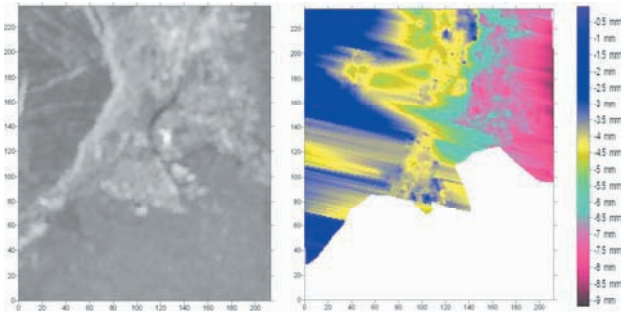


Fig. 17. Trial InSAR application for detection of slope movement in Hong Kong (18.3.1996 to 9.11.1999 3.6-year InSAR results at Tsing Shan Foothills, which are significantly affected by 'noises', as shown).

The quality and availability of SAR images would improve as more satellites are launched. Development of airborne and land-based InSAR would reduce noise effects and enhance the accuracy and spatial resolution of InSAR results. InSAR technologies, e.g. use of reflectors, filters and permanent scatter techniques, are evolving. The GEO has installed nine corner reflectors (Fig. 18) in three trial sites in Hong Kong. This provides an opportunity for testing the improvement that may be achieved and for developing the long-term potential for geotechnical application of InSAR in Hong Kong.

5 LIGHT DETECTION AND RANGING (LIDAR)

Light Detection and Ranging (LIDAR) is a promising remote-sensing tool for scanning surface topography by measuring the direction and time of sending and receiving coherent laser beams to the objects of interest. Land-based LIDAR capability is available in Hong Kong. Such land-based LIDAR system, commonly denoted as 'laser scanner' in Hong Kong, has the capability of measuring 3-dimensional point clouds of objects within about 300 m along the line-of-sight. The laser scanner emits thousands of laser beam pulses per second for measuring a 'window' of 3-dimensional surfaces. The positional accuracy is within 6 mm in a 50-m range. The point clouds, apart from providing spatial information on their x, y, and z coordinates, contain an intensity signal of the laser reflection and hence present a 3-D false-color digital model of the scanned object.

Laser scanners have been used in topographic surveys, which is particularly useful where physical access to the survey site is difficult or dangerous, e.g. a new landslide scar (Fig. 19).

Similar application of laser scanner in surveying a full-scale test of fill slope failure has also been reported by Kwong (2003).

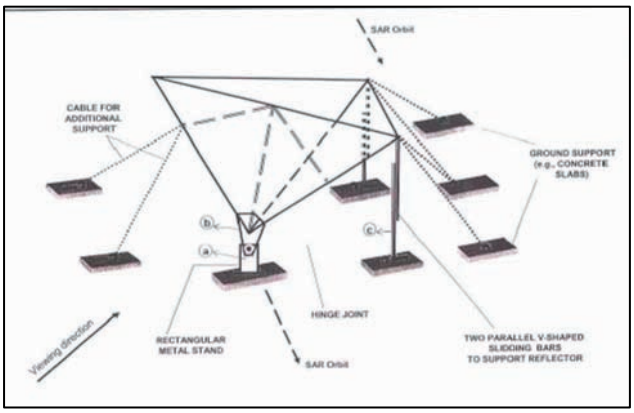


Fig. 18. Corner reflectors installed for InSAR application.

LIDAR can be used in many novel geotechnical applications apart from topographic survey, such as:

(a) Construction of high-resolution DTM – given the high sampling density, DTM produced by LIDAR can enhance the quality and supplement the DTM produced from the available topographic maps and from digital photogrammetry.

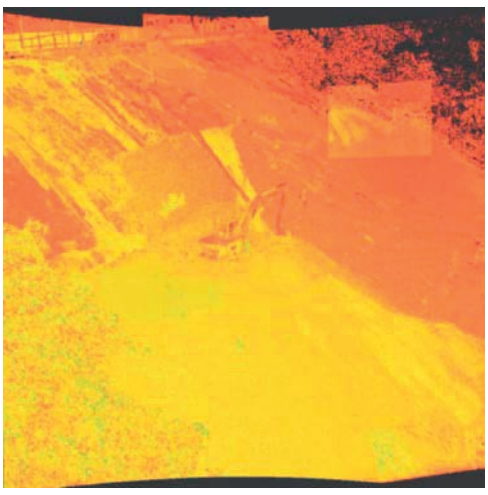
(b) Compilation of 3-D digital models of slopes, debris-resisting barriers, structures and other geotechnical features (Fig. 20) – this assists construction monitoring and provides an accurate and detailed virtual reality records for use in future maintenance and modification work.

(c) Movement monitoring of slopes and structures (Fig. 21) – movement can be detected and monitored by comparing LIDAR results obtained at different time; there is however a need to register the key data of common observation points in different LIDAR surveys to ensure accuracy and efficiency.

(d) Rock slope mapping and rock joint survey (Fig. 22) – by judiciously matching and analyzing the LIDAR point clouds.

Airborne LIDAR, which can survey a large area efficiently and at competitive cost, is subject to active technological development in other advanced countries where there is a demand for high-resolution topographic data. Airborne LIDAR is typically performed by mounting a high-powered LIDAR at the bottom or sides of a plane/helicopter to scan the ground features along the flight path. The instrument is bundled with accurate onboard differential GPS (DGPS) to register the reference plane and positioning during the LIDAR survey. It has been used in regional topographic mapping and construction of DTM, typically with an accuracy of ±0.15 m in height value. An important recent development is the use of a multi-return LIDAR to measure multiple returns for each laser pulse that covers several feet in diameter on ground. With the use of an advanced numerical algorithm, the last returns that come from the ground surface are extracted by filtering other returns from vegetation and building structures (a technique known as 'virtual deforestation') (Fig. 23). Hence, the system has the capability of mapping the ground surface of vegetated terrain (Haugerud & Harding 2003). The technique has been used to produce fine-scale topographic maps and DTM typically with grid size of about 1 m. This allows landslide geomorphology to be interpreted and landslide maps to be produced to a resolution that cannot otherwise be achieved by using conventional aerial photographs. It is recognized as one of the most important remote-sensing tool for obtaining high-quality digital elevation data and for landslide hazard mapping (National Research Council 2004).

(a) Landslide site

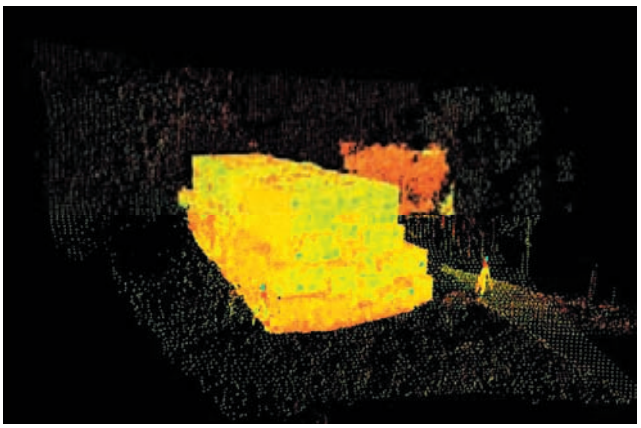


(b) LIDAR point clouds

Fig. 19. LIDAR survey of a landslide site.

The GEO conducted a pilot airborne LIDAR survey in December 2006 for the Hong Kong Island, to assess the 'virtual deforestation' capability in heavily vegetated area and the ability to detect slope features behind tall buildings. The Lands Information Centre of the Lands Department served as the technical adviser to the project and provided support by providing ground targets for calibration of the LIDAR equipment prior to the survey and enhancing the DGPS signal from the GPS reference stations from 1 signals per 5 seconds to 1 signal per second for post-processing of aircraft position. The technical requirements of the survey were sampling interval at 1.3 m, and horizontal and vertical data accuracies at 0.3 m and 0.13 m respectively. Through competitive tendering, an aerial survey and mapping service provider from Australia was awarded the survey contract which included deploying multi-return LIDAR equipment for the survey and processing of the data acquired. The data (Fig. 24) was received in March 2007 and data verification and evaluation is being carried out. The survey results will be used to plan future survey for the remaining parts of Hong Kong. It is anticipated that airborne LIDAR will become more widely used in Hong Kong and will bring about enhanced remote sensing capability that would facilitate geotechnical assessment and design, including studies of natural terrain landslide hazards.

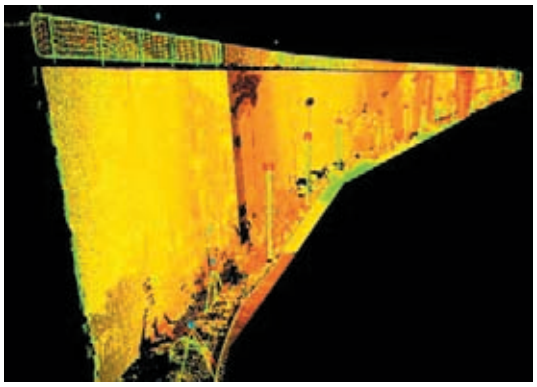
(a) Debris-resisting barrier



(b) 3-D digital LIDAR model

Fig. 20. 3-D digital model compiled by LIDAR.

(a) Retaining wall



(b) 3-D digita LIDAR model

Fig. 21. Use of LIDAR in monitoring movement of a retaining wall.

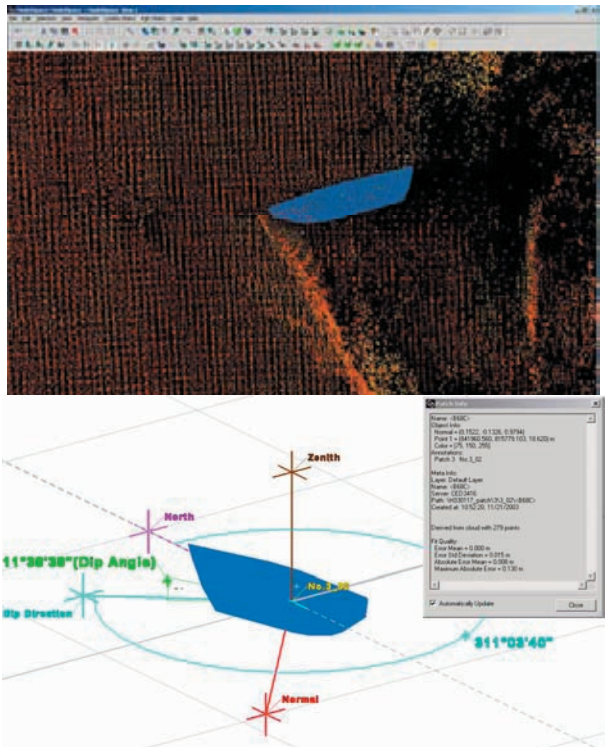


Fig. 22. Use of LIDAR in rock joint mapping and survey (Note: the blue area is a joint plane on a rock face with the joint orientation measured by LIDAR).

The GEO conducted a pilot airborne LIDAR survey in December 2006 for the Hong Kong Island, to assess the 'virtual deforestation' capability in heavily vegetated area and the ability to detect slope features behind tall buildings. The Lands Information Centre of the Lands Department served as the technical adviser to the project and provided support by providing ground targets for calibration of the LIDAR equipment prior to the survey and enhancing the DGPS signal from the GPS reference stations from 1 signals per 5 seconds to 1 signal per second for post-processing of aircraft position. The technical requirements of the survey were sampling interval at 1.3 m, and horizontal and vertical data accuracies at 0.3 m and 0.13 m respectively. Through competitive tendering, an aerial survey and mapping service provider from Australia was awarded the survey contract which included deploying multi-return LIDAR equipment for the survey and processing of the data acquired. The data (Fig. 24) was received in March 2007 and data verification and evaluation is being carried out. The survey results will be used to plan future survey for the remaining parts of Hong Kong. It is anticipated that airborne LIDAR will become more widely used in Hong Kong and will bring about enhanced remote sensing capability that would facilitate geotechnical assessment and design, including studies of natural terrain landslide hazards.

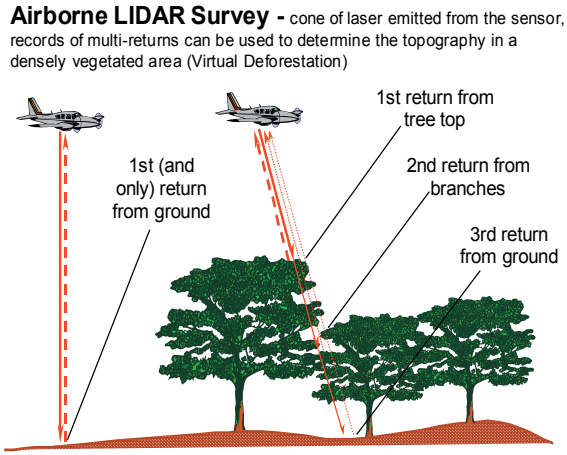


Fig. 23. "Virtual deforestation" capability of airborne LIDAR survey.

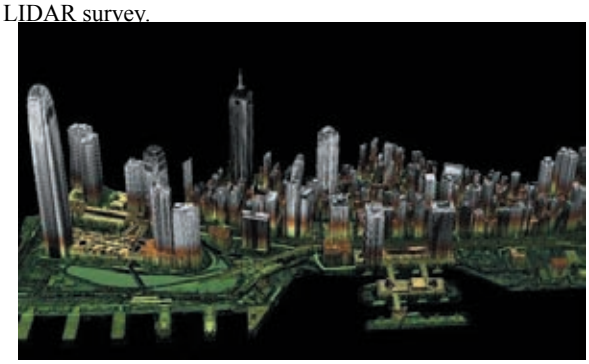


Fig. 24. Preliminary survey data from LIDAR over the coastline of Central.

6 CONCLUSIONS

Hong Kong has been actively pursuing the development and geotechnical application of novel digital technologies. This has led to improved capability and efficiency. The geotechnical profession should continue to maintain awareness of the technological development and opportunities for geotechnical application. Digital photogrammetry and GIS are examples of novel digital technologies that have successfully been integrated into our geotechnical practice. InSAR and LIDAR are emerging techniques that have notable potential. Development and customization work, which will further enhance their capability and suitability for geotechnical application, is being undertaken in Hong Kong and elsewhere.

ACKNOWLEDGEMENTS

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Research and Engineering Practice of Risk Assessment and Management

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Abstract: Decisions in geotechnical engineering have to account for risk. Engineers need to evaluate the impact of risk, qualitatively or quantitatively, on the reliability level of their design. Risk is a combination of probability of failure event and the failure consequences. Risk assessment and management for slopes has been practiced in Hong Kong for decades. It has become part of Hong Kong's slope engineering practice and is playing an important role in landslide risk management in Hong Kong. A brief review of the research and engineering practice of risk assessment and management for slopes is presented. Research efforts should continue to keep abreast with the latest advances in the field and to provide insights and results that help address current and future geotechnical challenges.

1 INTRODUCTON

In his Terzaghi Lecture, Casagrande (1964) called the attention of the geotechnical profession to recognize the existence of geotechnical risk and uncertainties, and to deal with them systematically. Not until the late sixties, reliability approach was introduced, most notably by Wu & Kraft (1970), to quantify the sources of geotechnical uncertainties and to study the effect of those uncertainties on the performance of a slope. Whitman (1984) presented, in his Terzaghi Lecture, an excellent summary of the works on geotechnical reliability up through the early eighties. In conclusion, he stated that "We must proceed to perform risk analysis in geotechnical engineering, despite of inadequate data and the criticism regardless of what we did, and learning how to do so in a meaningful and responsible manner is still a major challenge to the profession." After two decades since Whitman's Terzaghi Lecture, risk assessment has been evolved as a practical tool for managing geotechnical risk and uncertainties. However, the challenge to the geotechnical community of Hong Kong still exists. This paper aims to summarize the major development and application of risk assessment in geotechnical engineering, in particular slope engineering, in the past few decades and to highlight the challenges to the geotechnical community in Hong Kong.

2 LANDSLIDE RISK MANAGEMENT PRACTICE IN HONG KONG

2.1 *Landslide problem in Hong Kong*

Hong Kong has a population of some seven million within a small land area of about 1,100 square kilometers. As a result, it has a high concentration of developments on steep terrain in close proximity to man-made slopes and natural hillsides, which are susceptible to landsliding during heavy rain (Fig. 1). Until 1970s, man-made slopes were generally formed with little geotechnical engineering input in the design and construction process. Slope design was generally based on empirical rule-of-thumb, such as 55^{o} steep for cut slopes and 35^{o} steep for fill slopes; fill slopes were generally not compacted to an acceptable standard. The scale of landslide problem is reflected by a death toll of over 470 people since the late 1940s. These un-engineered slopes posed a significant landslide risk to the community.

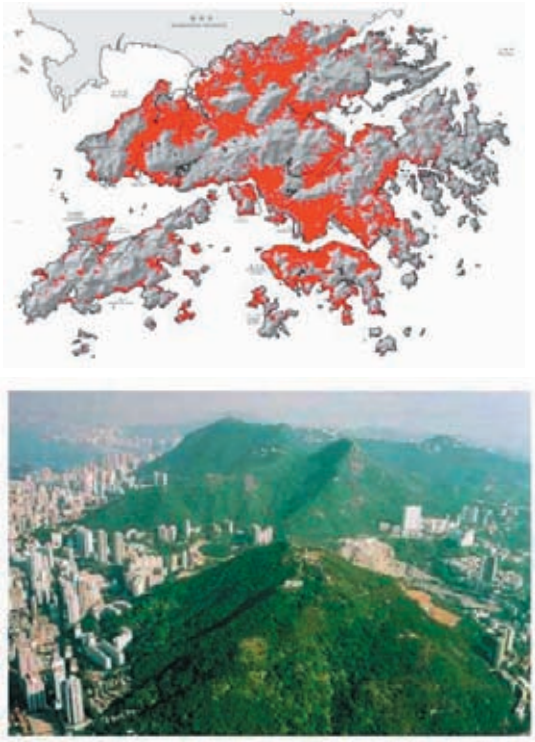

Fig.1. High concentration of developments in Hong Kong mingled with man-made slopes and natural hillsides.

2.2 *Evolution of slope engineering practice and landslide risk management in Hong Kong*

As a result of a number of disastrous landslides in the 1970s, the Geotechnical Control Office (GCO, renamed as Geotechnical Engineering Office, GEO, in 1991) was established in 1977. Over the years, a comprehensive Slope Safety System has been developed and implemented to combat landslide problems in a holistic manner (Malone, 1998; Chan, 2000).

Hong Kong is now renowned for its urban slope engineering and landslide risk management that facilitates safe and sustainable developments in a hilly terrain. The slope engineering and landslide risk management practice in Hong Kong has advanced considerably with time. Broadly speaking, it may be classified into three notable stages:

(1) Empirical slope design before 1977: Slope design and construction were based on rules of thumb, such as 55° steep for soil cut slopes (Fig. 2), and 35° steep for fill embankments. There was little geotechnical input, except for critical facilities, such as dams. About 39,000 sizeable man-made slopes were formed in this period. The vast majority do not meet the current safety standards, and are particularly vulnerable to landslide at times of heavy rainfall. The legacy of a large number of un-engineered man-made slopes brings about a long-term problem.

(2) Geotechnical slope design and landslide prevention from 1977 to mid-1990s: In the aftermath of several disastrous slope failures (Fig. 3), the GEO, which is currently part of the Civil Engineering and Development Department of the Government of the HKSAR, was set up in 1977 as the central body to regulate geotechnical engineering and slope safety in Hong Kong. Man-made slopes formed after 1977 are subject to geotechnical design and checking, to ensure that they meet the required safety standards. A total of about 18,000 sizeable man-made slopes have been formed since then. The GEO also operates a Landslip Preventive Measures (LPM) Programme, to systematically assess the stability of the pre-1977 man-made slopes according to their ranked order of priority and upgrade substandard Government slopes to the required design standards (GCO, 1984). The conventional deterministic approach of slope stability analysis was adopted by geotechnical practitioners in slope design. Landslide prevention was primarily aimed at, and based on, achieving the required design factor of safety (Table 1), with the risk management concept being implicit in the strategy adopted. Risk consideration, if done explicitly, was made in a qualitative manner.

(3) Enhanced landslide risk management since mid-1990s: In the past ten years, Hong Kong has pioneered the development and implementation of an explicit risk-based approach and strategy, in addition to the deterministic approach, for slope assessment and landslide risk management. The risk-based methodology embraces a holistic consideration of the likelihood of landslides and the adverse consequences. It can be applied in a qualitative or quantitative framework. The quantitative applications, in particular, have been instrumental in formulating the overall slope safety strategy for Hong Kong, as well as managing the landslide risk at individual vulnerable sites. This framework aligns slope engineering and landslide mitigation with other fields that practice state-of-the-art risk management in a more explicit manner.

Fig. 2. An un-engineered cut slope formed in Hong Kong before 1977.

Fig. 3. A man-made slope failure at Sau Mau Ping in 1972.

Table 1. Design standards for man-made slopes in Hong Kong.

Consequence-to-life Category	Required Minimum Factor of Safety
1 (e.g. affecting buildings)	1.4
2 (e.g. affecting sitting-out areas)	1.2
3 (e.g. affecting country parks)	> 1.0

Note: Applicable for 10-year design groundwater conditions

2.3 *Impetus for quantified landslide risk management*

Landslide risk is a measure of the chance of occurrence of slope failure causing a certain amount of harm (e.g. fatalities and economic losses), and can be quantified as the product of the probability and consequence of failure. ***Landslide risk assessment*** is the process of identifying the landslide hazards and estimating the risk posed by the hazards. ***Landslide risk management*** comprises an assessment of the landslide risk, deciding whether or not the risk is tolerable and exercising appropriate control measures to reduce the risk where the risk level cannot be tolerated. In a more global context, landslide risk management also refers to the systematic application of management policies, procedures and practices to the tasks of identifying, analyzing, assessing, mitigating and monitoring landslide risk.

Slope engineering and landslide mitigation have always involved some form of risk assessment and management. This was often done through the use of engineering judgement. The more formal risk assessment and management principles emerged

in the 1970s. It was confined to qualitative applications in the early years, but has been extended into Quantitative Risk Assessment (QRA), particularly since the mid-1980s. In the early 1990s, risk management became well established in the hazardous industries as a preferred approach for addressing risk issues to meet the expectations of the parties involved (Royal Society, 1992). Since about the mid-1990s, QRA has been formally applied in assessing and managing landslide risk, notably in Australia (Australian Geomechanics Society, 2000), France (Leroi, 1996) and Hong Kong (Wong *et al*., 1997).

Whilst the value of quantified landslide risk management is recognized by the geotechnical profession and by many slope-related agencies and regulators, some are more advanced in terms of implementation than others. Hong Kong has played a leading role in the development and application of quantified risk management, which brings about the following advances in slope engineering and landslide risk management:

(a) There are considerable uncertainties associated with the geological and groundwater conditions in the tropically weathered rocks in Hong Kong. These uncertainties are often difficult to effectively address in a deterministic slope assessment. However, they can be analyzed systematically and evaluated rationally using a quantified risk framework.

(b) The large number of potentially hazardous, old man-made slopes that await retrofitting call for application of risk management techniques and principles to assess the scale of the problem, optimize resources allocation, prioritize follow-up actions, and evaluate risk mitigation strategy.

(c) A risk-based approach facilitates communication of the nature and level of landslide risk with the stakeholders, including the resources allocations and the general public. This is particularly important in Hong Kong as the public have a high expectation of slope safety and in general they wish to be informed of the hazards that affect them.

(d) In the early years, the LPM Programme primarily dealt with large substandard man-made slopes affecting occupied buildings. As many of these slopes have been upgraded, slopes of relatively smaller size or affecting different types of facilities are being treated under the Programme. Furthermore, increasing attention is required on the mitigation of natural terrain landslide hazards affecting developed areas. QRA provides a structured framework for formulating a coherent landslide risk mitigation strategy. It also provides a scientific basis for evaluating the risk management options at individual sites, particularly those affected by natural terrain landslide hazards, which cannot normally be effectively dealt with by means of conventional slope stabilization measures (Wong, 2003).

Quantified landslide risk management is a novel technical development. Ho *et al*. (2000) described some notable issues and possible misconceptions in the use of QRA, as well as examples of its application to slope safety problems in Hong Kong. A number of factors are essential to the significant progress made in the formulation of quantified landslide risk management methodology and pioneering of its implementation to address landslide problems in Hong Kong. These include:

(a) Systematic landslide investigation and compilation of landslide records in Hong Kong (Wong & Ho, 2000) have provided considerable insight into the mechanisms and causes of landslides, as well as data for use in QRA.

(b) A series of technical development work undertaken in Hong Kong was instrumental to formulating appropriate landslide QRA methodology (Wong, 2001), e.g. assessment of landslide debris mobility (Wong & Ho, 1996), quantification of landslide consequence (Wong *et al*., 1997), quantification of landslide frequency (Wong & Ho, 1998), and formulation of natural terrain study and risk assessment methodology (Ng *et al.,* 2003; Wong, 2003).

(c) A set of landslide risk guidelines (ERM, 1998), benchmarked against those already adopted for Potentially Hazardous Installations in Hong Kong, has been formulated. The risk guidelines stipulate the tolerable risk criteria for natural terrain landslides and boulder falls in respect of both ***Individual Risk*** and ***Societal Risk***. The Individual Risk criteria apply to the annual probability of fatality for the most vulnerable person affected by the landslide hazard, and the maximum allowable limit is 10^{-5} in the case of a new development, and 10^{-4} for an existing development. The Societal Risk criteria apply to the total risk-to-life posed to the affected community by the landslide hazard, which is expressed as an F-N curve (see Fig. 4).

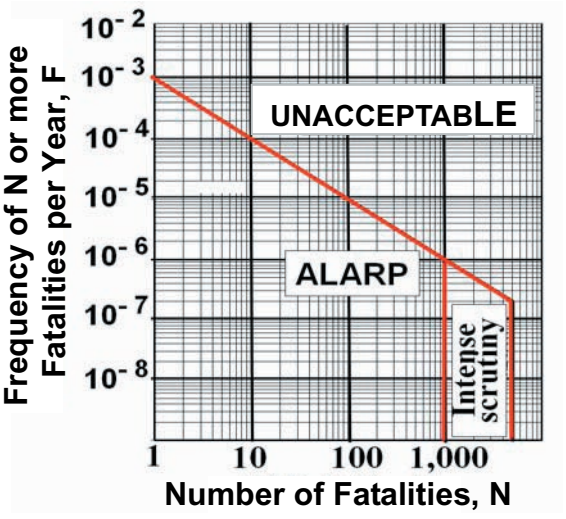

Fig. 4. Societal risk criteria adopted in Hong Kong for landslides and boulder falls from natural terrain.

Quantified risk management is applied to the formulation of the overall slope safety strategy and management of landslide risk posed by individual sites. These involve the use of global QRA and site-specific QRA respectively. The state-of-the-art of QRA applications, together with a review of the professional practice and development trends, is presented by Wong (2005). Three selected examples are given below to illustrate different types of application in practice.

2.4 *Examples of QRA application in slope engineering*

(a) Global risk management for landslides from old man-made slopes

The GEO has compiled a comprehensive Slope Catalogue, which registers the 39,000 pre-1977 and 18,000 post-1977 man-made slopes. The overall risk of these slopes, in terms of the annual potential loss of life (PLL) that may theoretically be caused by landslides from these slopes, was assessed in 1997, 2000 and 2004. The assessments serve to identify the scale of the landslide hazard, diagnose the risk distribution, formulate risk reduction strategy and targets for the LPM Programme, and provides a basis for monitoring the progress of the risk reduction programme.

The methodology adopted in the above risk assessments was described by Wong & Ho (1998). The key findings are summarized below:

(1) The risk profile of the pre-1977 man-made slopes as at 1997 (Fig. 5) shows that risk is unevenly distributed among the slopes. About 50% of the overall risk is derived from approximately 10% of the slopes that have the highest potential risk. This emphasises the importance of adopting an appropriate risk-based ranking system for prioritizing landslide preventive actions on the slopes, so as to ensure that the risk reduction effort is expended in a cost-effective manner and that maximum risk reduction would be achieved in the shortest possible time. Wong (2005) describes the slope ranking methodology used in Hong Kong in comparison with those adopted in other countries.

Table 2. Risk distribution according to slope type.

Slope Type	Un-engineered Man-made Slopes Formed before 1977		
	Cut Slopes	Fill Slopes	Retaining Walls
Percentage of Slopes	50%	25%	25%
Proportion of Total Risk	75%	13%	12%
Ratio of Total Risk	6	1	1
Average Ratio of Risk per Feature	3	1	1

(2) The global distribution of risk from different types of man-made slope features was established (Table 2). The average risks of a cut slope, fill slope and retaining wall are in the ratio of 3:1:1. This forms a basis for allocation of retrofitting resources to different slope types under the LPM Programme, as incorporated into the ranking methodology (Wong, 2005).

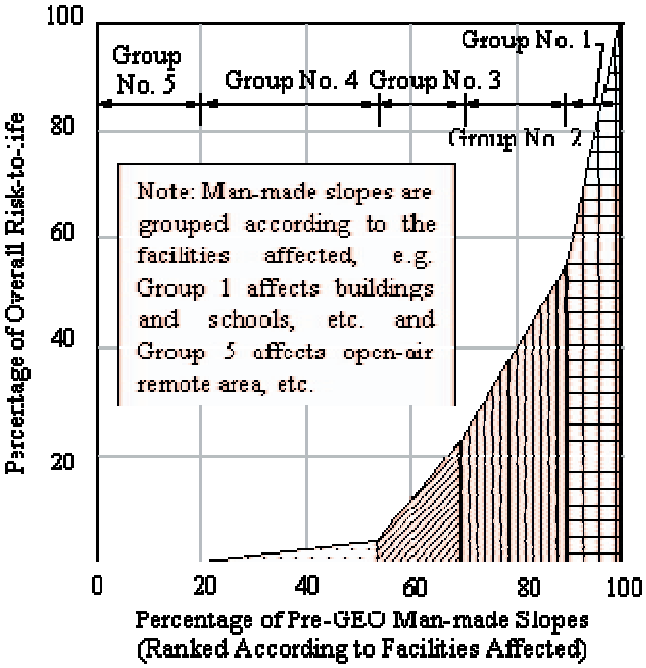

Fig. 5. Risk profile of un-engineered man-made slopes in Hong Kong in 1997.

(3) The risk profile has been formally adopted as quantified risk reduction targets pledged by the HKSAR Government. The LPM Programme was tasked to upgrade about 10% of the pre-1977 slopes by year 2000, and another 10% by 2010. The pledged reduction targets entail: (i) by the year 2000, the overall landslide risk from the pre-1977 slopes would be reduced to 50% of the level in 1977; (ii) by 2010, the risk would be further reduced to 25% of the level in 1977 (Works Bureau, 1998).

(4) QRA has also been applied to evaluate and monitor the performance of the LPM Programme. An assessment made in 2000 found that the risk of the pre-1977 slopes has reduced from 21.8 PLL/year in 1977 to 10.3 PLL/year in 2000, i.e. about 53% risk reduction was achieved (Cheung & Shiu, 2000). An update of the assessment made in 2004 for progress monitoring indicated that the risk of the pre-1977 slopes would be further reduced to about 4 PLL/year by 2010 (Lo & Cheung, 2005). At this rate of risk reduction, the Programme is operating at about HK$ 15 million per statistical life saved.

(b) Site-specific landslide risk management for existing buildings

The residential buildings at Shatin Heights are flanked by a 35° steep hillside that has a history of failure (Fig. 6). In 1997, six landslides occurred on the hillside and three developed into debris flows that affected the buildings. Geotechnical assessment revealed that the landslides were rain-induced shallow failures involving the surface layer of colluvium and completely weathered granite.

The landslide risk was quantified with the use of formal QRA techniques (FMSW, 2001). The societal risk was found to be 5.7 x 10^{-3} PLL/year, and the F-N curve was within the unacceptable zone. The individual risk at the vulnerable part of the site exceeded 10^{-4}, which was also unacceptable. These confirmed the need for risk mitigation in order to meet the prevailing risk

standards. From an examination of the possible risk mitigation options, an optimal risk mitigation strategy was established. This included improvement of surface drainage provision and surface protection at the crest of the hillside, provision of debris-resisting barriers and local slope stabilization. The cost of risk mitigation was about HK$ 6 million, which would result in mitigation of about 80% of the societal risk. After risk mitigation, the individual risk distribution (Fig. 7) and F-N curve (Fig. 8) would be well below the unacceptable zone. The risk mitigation was evaluated based on the As Low As Reasonably Practicable (ALARP) principle, and was found to be justified from cost-benefit consideration. The risk mitigation works were completed in 2004.

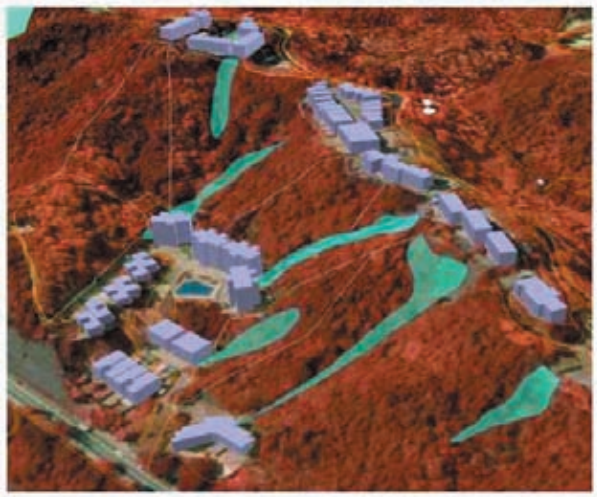

Fig. 6. Residential buildings affected by landslide hazards at Shatin Heights.

Fig. 7. Individual Risk at Shatin Heights

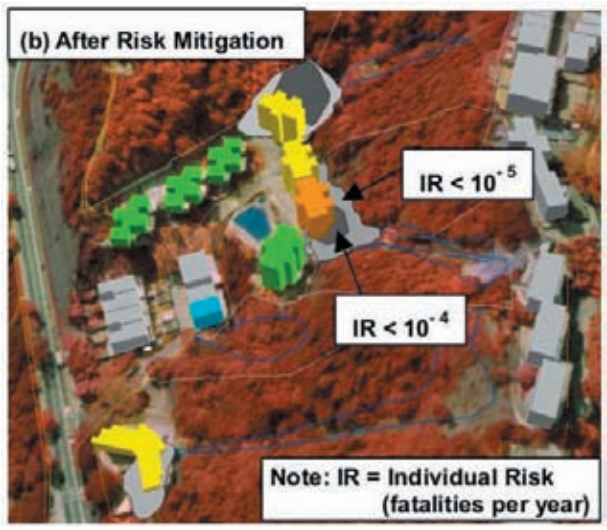

Fig. 7(con't). Individual Risk at Shatin Heights.

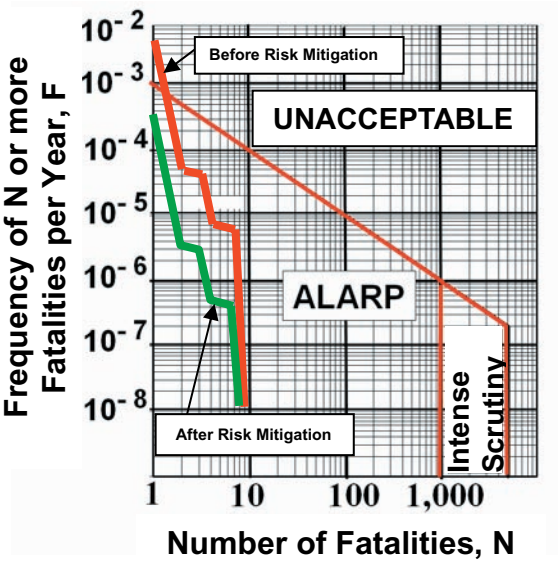

Fig. 8. Calculated F-N curves for Shatin Heights.

(c) Site-specific Landslide Risk Management for Development Planning

In 2004, the Ling Pei area in Tung Chung was being planned for low-rise residential development. The development comprised construction of 76 nos. of 3-storey houses at the toe of a hillside, where natural terrain landslides had previously occurred (Fig. 9). QRA was conducted to establish the risk of natural terrain landslides and the required risk management strategy, as a guide for development planning. The QRA is presented by Wong *et al*. (2004). The individual risk at the planned development was found to range from 3.3 x 10^{-7} to 8.9 x 10^{-6} (Fig. 10), which is within the maximum permissible level of 10^{-5} for new developments. The societal risk for the planned houses was 1.8 x 10^{-4} PLL/year, and the corresponding F-N curve was entirely within the ALARP zone (Fig. 11). The results indicate that while the hillside is moderately susceptible to natural terrain landslides, the risk posed to an individual in the planned houses is tolerable. However, the high concentration of the planned houses implies

that the total risk to the community could be of concern. The maximum justifiable expenditure for risk mitigation assessed based on application of the ALARP principle is about HK$ 0.7 million. At this order of maximum expenditure, extensive slope stabilization measures (e.g. soil nailing) and provision of heavy debris-resisting structures would not be practical. Instead, two possible risk mitigation schemes that are commensurate with the maximum justifiable expenditure were recommended (Fig. 12). The total cost of the planned houses is about HK$ 250 million. Provision of the recommended risk mitigation measures would amount to about 0.3% of the total cost.

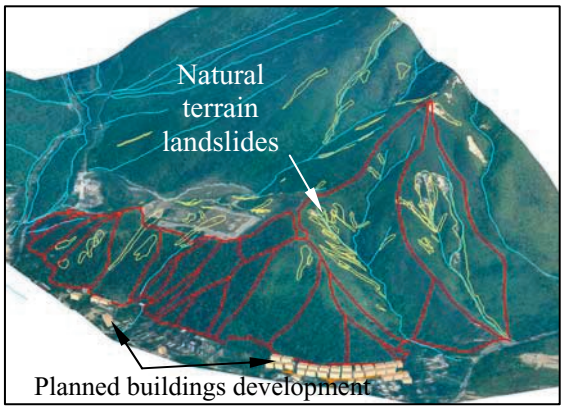

Fig. 9. Planned building development at Ling Pei.

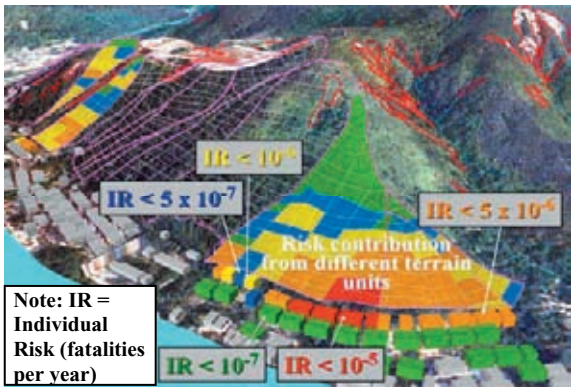

Fig. 10. Individual Risk at planned buildings at Ling Pei.

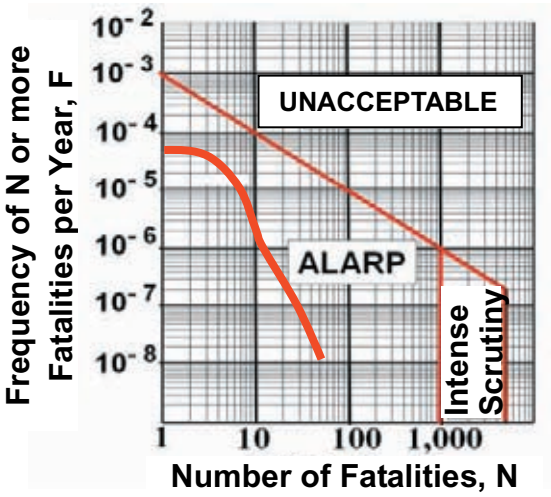

Fig. 11. Calculated F-N curve for Ling Pei.

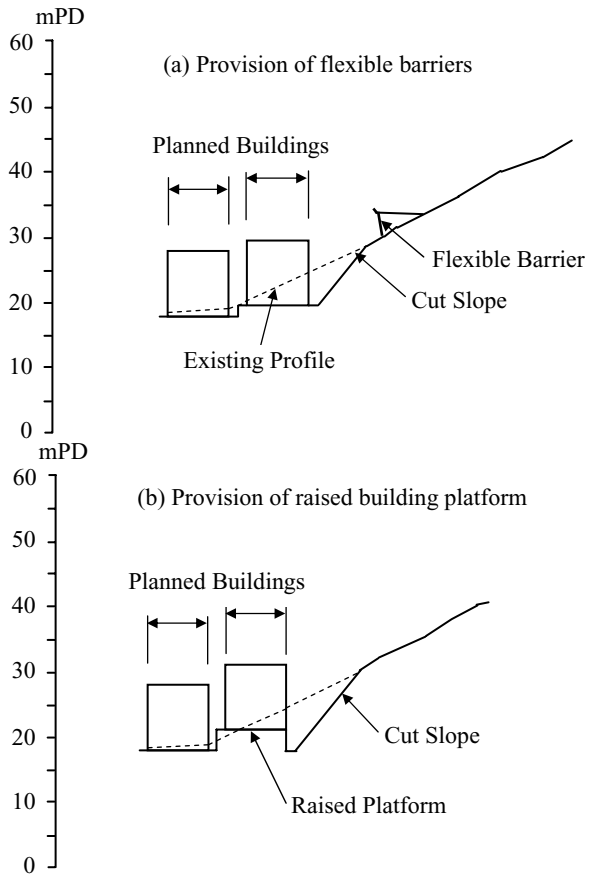

Fig. 12. Risk mitigation options at Ling Pei.

3 RESEARCH ON RISK ASSESSMENT OF LANDSLIDES

3.1 *Probabilistic assessment of slope failure*

There has been a rapid growth of the use of risk assessment tools in the field of geotechnical engineering in the past two decades. This also brings about some notable research and development findings, in particular in the field of geotechnical reliability and the use of probabilistic methods (Tang, 1993). With the advancement of technology, there is also an increasing trend of using digital technique in the subject areas. For example, Dai *et al*. (2000) and Zhou *et al*. (2002) have used the geographical information system (GIS) technique to study the statistical relationship between landslides and their causative factors at regional level, and to produce landslide susceptibility maps. In practice, various techniques, such as use of precedence performance statistics, probabilistic modeling and direct subjective judgment, can be used to formulate the failure frequency model. Among these techniques, use of historical landslide statistics, if available, is probably the most common method to determine the probability of slope failure. This probability is usually assumed to be time independent, and constant probabilities in each unit of time such as annual failure probabilities are commonly used in risk assessment. In other words, a Bernoulli model has been adopted intrinsically in the risk estimation. As pointed out by Cheung & Tang (2005a), some factors may lead to deterioration of a slope, and hence its performance and failure probability become a temporal variable.

These factors include degradation of slope surface cover, blockage and damage of surface and sub-surface drainage systems, reduction of soil shear strength due to weathering of slope forming materials and seasonal fluctuation of pore pressure, and others. Difficulty in characterizing the statistical variability of these factors further complicates the matter particularly when historical data covering a long observation period are not available. As a result, the reliability of deteriorating slopes at various service stages is generally not explicitly considered in risk assessment. Indeed, if historical performance data over a considerable long period are available, the reliability of a slope within a service period of k years can be expressed directly from the observed data as a product of the conditional probabilities of the slope performing well for k consecutive years. This is commonly referred to as the Product-Limit (PL) method in reliability analysis (Kaplan & Meier, 1958). In light of this, Cheung & Tang (2005a) presented a procedure for establishing a probabilistic model for determining the time dependent failure probability of slopes using observed performance statistics. The probability estimates using the proposed procedure (Weibull model) and those using the conventional Bernoulli model were compared with the PL estimates. Fig. 13 suggests that the estimates obtained from the Weibull model are more close to the PL estimates than those from the Bernoulli model. To illustrate the application, the Weibull model was used to determine the failure probability of a sample of 482 pre-1977 un-engineered soil cut slopes. These 482 cuts were classified according to the geometry bound envelopes, S1 to S4, as described by Brand (1985). In general, soil cuts of type S1 are more prone to failure than those of type S2, and so on. Out of the 482 cuts, 198 are found to be types S1 and S2, and the remaining 284 cuts belong to types S3 and S4. Fig. 14 shows the failure probability of different slope geometrical type over different service periods.

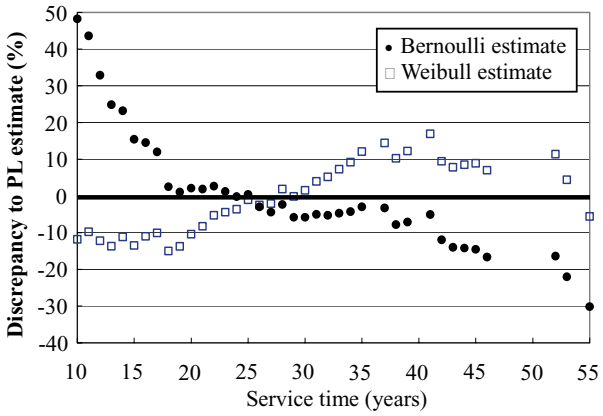

Fig. 13. Comparison of probability estimates using different probabilistic models (Cheung & Tang, 2005a).

As most of the landslides in Hong Kong are rain-induced in nature, it may be useful to characterize the probability of slope failure against different rainfall intensities. Cheung & Tang (2005b) developed a method to obtain probabilities for different rainfall intensities and service periods. In Hong Kong, a territory-wide automatic raingauge network has been established since 1984 (see Fig. 15). Rainfall data are recorded automatically at 5-minute intervals. In the analysis, a maximum rolling 24-hour rainfall, i_{24max}, has been used as the parameter for rainfall-landslide correlation. The parameter of i_{24max} is used as past studies (e.g., Brand *et al*., 1984) have indicated that it correlates well with the occurrence of landslides.

Fig. 14. Failure probability of different slope geometrical type (Cheung & Tang, 2005a).

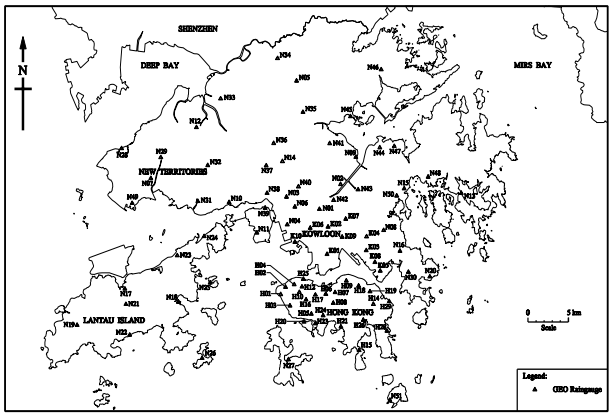

Fig. 15. Locations of GEO raingauges.

During the period of 1984 to 2002, there were a total of 208 rainstorms, with a maximum of 18 in 1997 and a minimum of 5 in 1999. To facilitate the establishment of rainfall-landslide correlation, the territory has been divided into a grid of 1,600 cells; among which 860 cells fall on land. Each cell has a plan area of 1.5 km by 1.2 km. Whenever a rainstorm occurs, the rainfall data recorded at the raingauges will be used to interpolate the rainfall intensity at each of the 1,600 cells. For each of the 208 rainstorms, the geographical characteristics of the rainfall will be represented by the spatial distribution of the maximum rolling 24-hour rainfall, i_{24max}, among the 1,600 cells. Each cell will have its own i_{24max}. For instance Fig. 16 illustrates the distribution of i_{24max} for the rainstorm that occurred between 1 and 4 July 1997 where the rainfall intensity is divided into three categories, viz. 0-200 mm/day, 200-400 mm/day, and > 400 mm/day. With reference to the same grid of cells and the locations of the slopes, the some 20,000 pre-1977 un-engineered soil cuts can be distributed to the appropriate cells as indicated in Fig. 17, where the number represents the number of slope in each cell.

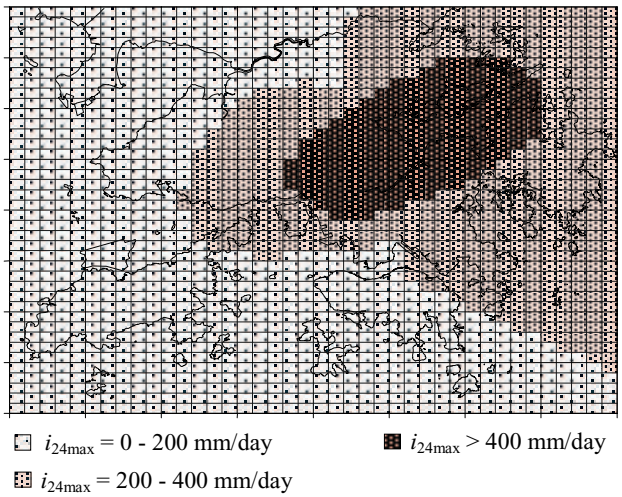

Fig. 16. Distribution of the maximum rolling 24-hour rainfall among the cells for the rainstorm of 1 - 4 July 1997 (Cheung & Tang, 2005b).

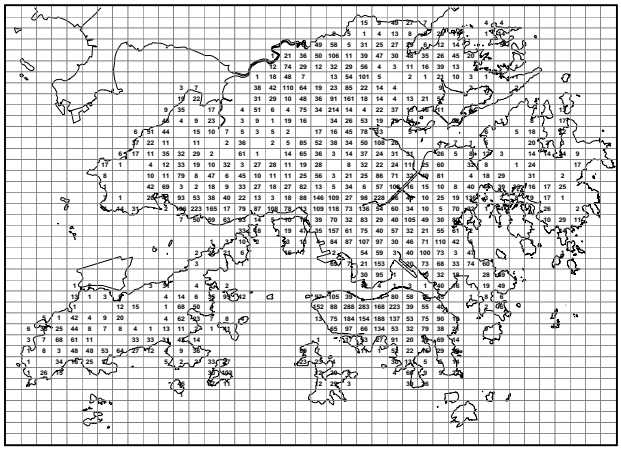

Fig. 17. Spatial distribution of pre-1977 un-engineered soil cuts in the territory (Cheung & Tang, 2005b).

Data on the performance history of the pre-1977 un-engineered soil cut slopes in the period of 1984 to 2002, including the time of formation and the time of failure, have been retrieved. The youngest and oldest slopes were 7 and 31 years old, respectively, in 1984 (the beginning of the availability of comprehensive rainfall data). By the end of 2002 (the end of observation period), these slopes were 26 and 50 years old, respectively. More than 1,100 landslides were reported over the observation period. For each of the rainstorms, relevant cuts and landslides in response to the three rainfall intensity categories from each of the small cells have been identified and grouped. The number of slopes hit by rainstorms (called 'rainstorm hit' thereafter) is then grouped according to slope age and rainfall intensity. The grouping of rainstorm hits for the 208 rainstorms under different categories of rainfall intensity and slope ages is shown in Fig. 18. Similar grouping has been carried out to the failures as in Fig. 19. Based on the set of data in Figs. 18 and 19, the conditional annual probability of failure for *t*-year-old slopes subjected to a given rainfall intensity can be estimated as follows:

$$\hat{h}(t, i_{24\max}) = \frac{\text{Number of } t\text{-year-old slopes that failed at a rainstorm of intensity } i_{24\max}}{\text{Number of rainstorm hits of intensity } i_{24\max} \text{ on } t\text{-year-old slopes}} \quad (1)$$

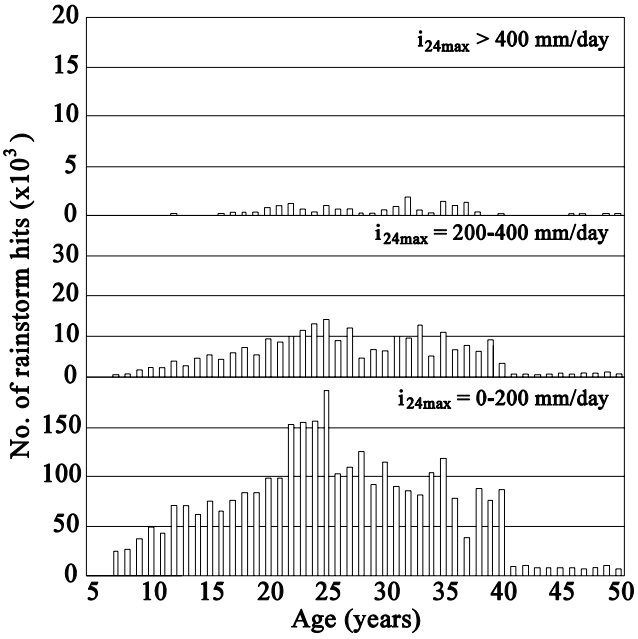

Fig. 18. Number of rainstorm hits on pre-1977 un-engineered soil cut slopes in the period of 1984 to 2002.

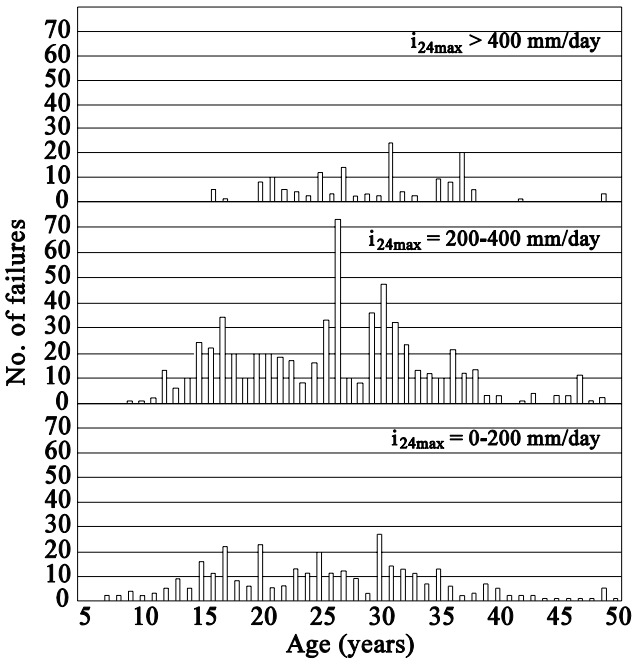

Fig. 19. Number of pre-1977 un-engineered soil cut failures in the period of 1984 to 2002.

One can base on Eq. (1) to determine the failure probability for a group of slopes under different rainfall intensities and slope ages. The model has been tested using the rainfall data in 2003. The number of landslides for each of the six rainstorms in 2003 predicted by the model is compared with the actual values in Table 3. Note that more data are needed to affirm the reliability of the model and the sensitivity of the prediction to the rainfall intensity classification.

Table 3. Predicted and actual number of pre-1977 un-engineered soil cut failures in 2003 (Cheung & Tang, 2005b).

Rainstorm Period	No. of cells on land under rainfall intensity			Actual no. of failures	Predicted no. of failures
	0-200 mm/day	200-400 mm/day	>400 mm/day		
7-10 Apr.	860	0	0	1	2
4-7 May	230	601	29	38	43
9-16 Jun.	590	270	0	18	23
21-26 Aug.	860	0	0	4	3
1-4 Sep.	860	0	0	3	2
14-17 Sep.	826	34	0	2	5

The failure probabilities of slopes as presented earlier are at best indicative of the overall performance of a large number of slopes of similar characteristics (e.g., geometrical types S1 to S4). These may facilitate "global" landslide risk assessment and perhaps provide some initial estimates of slope reliability. However, such models do not have adequate resolution for reliability assessment of individual slopes, particularly when important data such as geology, geometry, engineering parameters and underground water regime are not considered. Examples of application of probabilistic method to individual slope assessment are available in the literature (e.g. Yucemen & Tang, 1975; Vanmarcke, 1977; Li & Lumb, 1987; Christian *et al*., 1994; EI-Ramly *et al*., 2002; among others). Low & Tang (1997) also developed a procedure where the probability of failure of a specific slope can be estimated rapidly using a simple spreadsheet programme. However, a key limitation of the conventional probabilistic model is that the failure probability is determined on the basis of certain assumed deterministic and probabilistic models. For example, one may establish the performance of a slope based on limit equilibrium method and use a first-order reliability method to determine the probability of slope failure (e.g. Ang & Tang, 1984). Such model is simply an abstraction of the state of nature and is subject to model error. The probabilities obtained in a conventional model are therefore "calculated" or "notional", and do not fully reflect the actual reliability of the slopes.

Recently, much research work has been carried out to enhance probabilistic assessment of slope failure. Observed performance of slopes can help to calibrate a calculated probability for model errors. For example, the failure probability associated with a geotechnical structure such as a slope whose calculated reliability index is β, i.e. $P(F \mid \beta)$, can be updated with known performance records (Juang *et al*., 1999; Cheung & Tang, 2000; Juang *et al*., 2001). For a slope of similar characteristics whose calculated reliability index is β, the updated probability of failure can be determined as:

$$P(F \mid \beta) = \frac{f_F(\beta)P(F)}{f_F(\beta)P(F) + f_{NF}(\beta)P(NF)} \quad (2)$$

where $P(F \mid \beta)$ is the probability of slope failure for a given value of β , P(F) is the prior probability of slope failure, $P(NF)$ is the prior probability of no slope failure, $f_F(\beta)$ is the probability density function of reliability index at a value of β for slopes with past failure records, and $f_{NF}(\beta)$ is the probability density function of reliability index at a value of β for slopes without past failure records.

Based on Eq. (2), Cheung & Tang (2005b) determined the probability of failure of a slope of various ages whose calculated reliability index is β. For example, the reliability estimate associated with a calculated β over the next 50-year service for a slope of different ages can be updated as in Fig. 20. Note that the statistical methods as described earlier can be used to estimate the prior probability of failure.

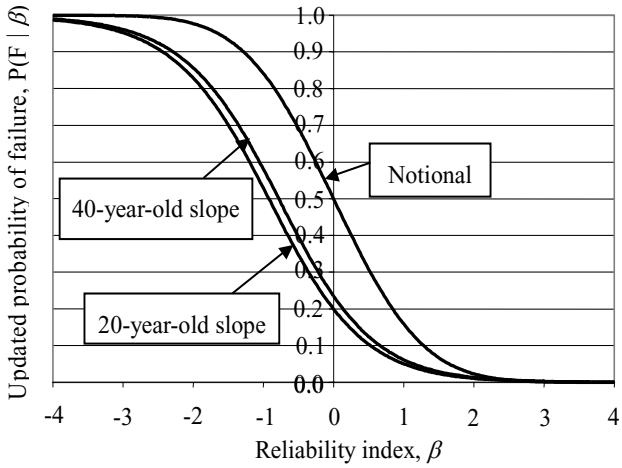

Fig. 20. Updated probability of failure of pre-1977 un-engineered soil cut slopes of different ages for a given reliability index β over the next 50-year service (Cheung & Tang, 2005b).

Research efforts have been given to improving the characterization of uncertainties associated with rainfall-induced landslide reliability analysis, to incorporate the use of more rigorous geotechnical models and to enhance the reliability evaluation. Zhang *et al*. (2003; 2005) identified sources of uncertainty that influence seepage and deformation in unsaturated soil slopes. These include uncertainties in soil properties, sampling and testing error, surface cracks (Hu, 2000; Li & Zhang, 2007) among others. Zhang *et al*. (2005) investigated uncertainties in soil porosity, θ_s, saturated permeability k_s, the parameters a and n in the Fredlund and Xing soil-water characteristic curve (SWCC) model and the shear strength parameters M_{cs}, M_{col}, Γ and λ in a critical-state constitutive model based on measurements from field and laboratory tests. Fig. 21 shows the frequency diagrams and probability density functions of $\ln(k_{sat})$ and porosity, θ_s based on measurements on soil samples from a slope at Sau Mau Ping.

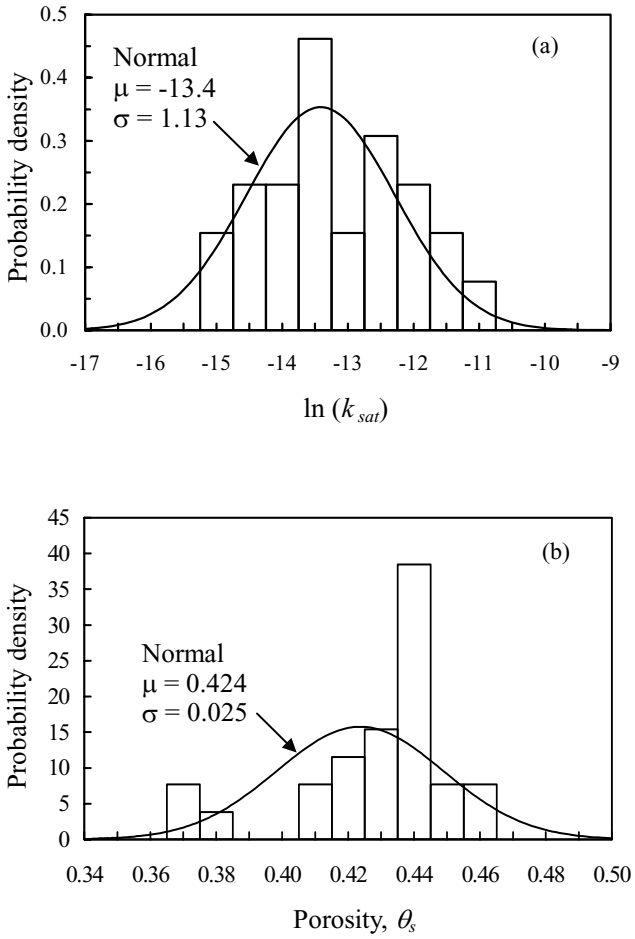

Fig. 21. Frequency diagrams and probability density functions: (a) ln(k_{sat}); (b) porosity θ_s (soil samples from a slope at Sau Mau Ping) (Zhang *et al*., 2005).

Reliability analysis is generally based on the use of a geotechnical model. As assumptions are always involved in a given model, use of a better analytical geotechnical model can reduce the model uncertainties and hence potentially improve the prediction of the performance. Based on a nonlinear elastic coupled formulation for seepage and deformation of unsaturated soils, a coupled hydro-mechanical analysis program was developed. Numerical solutions of the coupled formulation are obtained using a finite element partial differential equation solver, FlexPDE. A finite-element-based slope stability program FESSA is also developed to calculate the safety factor of the slope based on stresses and pore-water pressure distributions obtained from the coupled numerical analysis. Coupled numerical modeling and slope stability analysis are conducted for two slopes. The first is the slope at Sau Mau Ping, where a disastrous landslide occurred in 1976 after about 30 hours of heavy rain. The other slope is the Kadoorie test slope where a field test was conducted to study the response of the instrumented nailed loose fill slope under artificial rainfall conditions. Comparisons of the field measurements and the calculated results were made. Reliability analyses were conducted using a stratified sampling technique to generate random samples of input random variables. For a slope similar to that of Sau Mau Ping, the movement of the wetting front, the slope deformation, and the factor of safety could be tracked as infiltration progressed. Different sets of simulated input parameters gave different scenarios of safety factor. The variability in the simulated factors of safety was found to be much larger towards the latter part of the rainstorm as the uncertainties in the SWCC parameters began to play a more significant role. It was also observed that the degree of correlation between SWCC parameters could affect the uncertainty in the predicted reliability. Although the mean factor of safety may not differ very much, significantly greater uncertainty in the simulated factor of safety will exist if the parameters are uncorrelated (Fig. 22). This suggests the importance of obtaining suitable values of the correlation coefficients from experimental data for use in the reliability analysis.

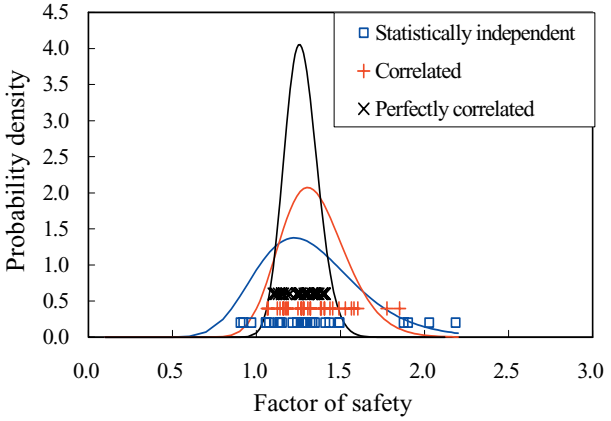

Fig. 22. Effect of correlation among SWCC parameters on safety factor of a soil slope under long-term rainfall with density of 10^{-8} m/s.

In the reliability analysis, it is essential to include the uncertainties implicit in the prediction model. However, if the prediction model is calibrated using physical model tests, uncertainties of the tests will be introduced into the estimated model error. Zhang (2005) proposed a Bayesian framework to evaluate the model error of a prediction model using information from physical model tests. The procedures of model calibration are illustrated by using the information from the field test in the Kadoorie slope and two centrifuge tests to calibrate the coupled numerical modeling program and FESSA. The results illustrate that if the preparation of a centrifuge model is carefully conducted and the variability of soil properties in the centrifuge model is very low, the calibration can be very efficient. In other words, the test will yield better assessment of the model bias and error. It is also found that when modeling the same prototype, if the quality of sample preparation can be maintained among different sizes of centrifuge model, a centrifuge model of a larger size will give a more efficient calibration than small-sized centrifuge model. This information may be important in designing the centrifuge test for the purpose of model calibration. Once the model error statistics are obtained, they may be used to update the reliability of a new slope. For example, Fig. 23 shows that with a successful field test, the safety factor of a new slope is expected to be higher and the predicted factor of safety less uncertain than those without the test. The corresponding reliability is shown to increase from 85 to 98 percent, or reliability index from 1.055 to 1.978.

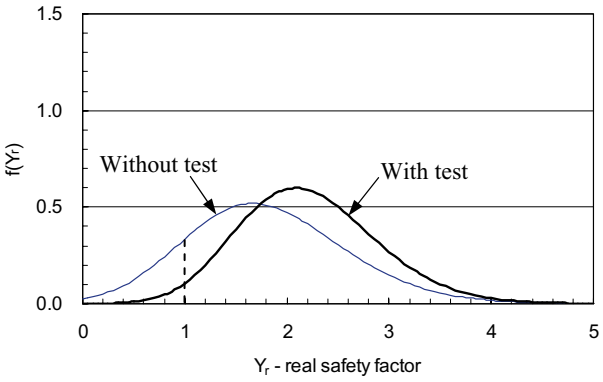

Fig. 23. Predicted distributions of the real safety factor for the new slope with and without information from a field test (Zhang, 2005).

There is an increasing use of soil nails to strengthen a slope against sliding failure. Assessment of the reliability of nailed slopes requires statistical characterization of the pull-out strength of soil nails. Zhang (2005) studied some pull-out data from field and laboratory tests, which showed that for nails in both cut and loose fill slopes, the ratio of measured to estimated pull-out strength generally exceeds 1.0 in submerged condition. Large variability was also observed in the pull-out test data. Through reliability analysis, reliability-based design charts may be developed to facilitate the design of nailed slopes. An example of reliability-based design chart for nailed fill slope is shown in Fig. 24. If a safety level corresponding to a reliability index of 3.0 is required, which corresponds to a notional probability of failure of about 3% and, a factor of safety of about 1.3 will be required. Based on these, one can find the required nail length for the hypothetical slope shown in the chart.

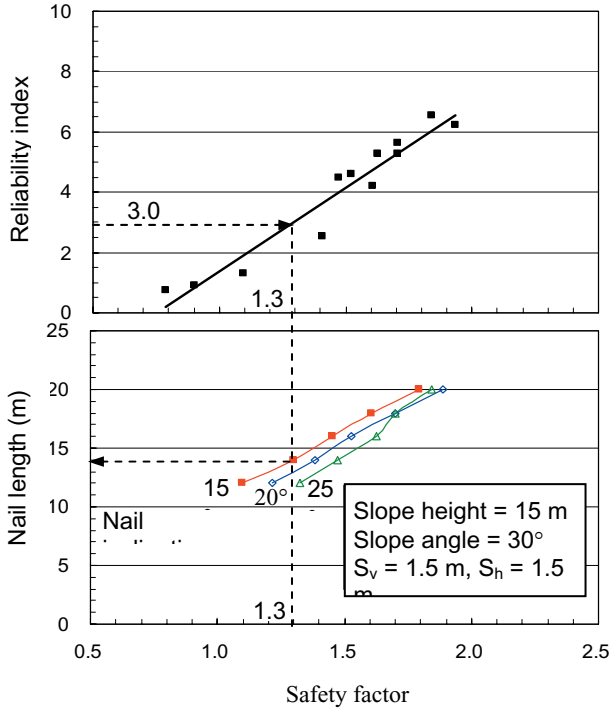

Fig. 24. Reliability-based design for a nailed loose fill slope based on relationship of reliability index and deterministic safety factor.

3.2 *Application to risk management*

A better estimate of the failure probability of a slope, such as those evaluated previously, is useful to geotechnical design option selection within the framework of landslide risk management. The choice among geotechnical design options is conventionally based on the deterministic approach with the balance between safety and economy taken into consideration. Under this approach, the probability and potential cost of failure are intrinsically included in the prescribed minimum factor of safety, and uncertainties are usually considered in a subjective manner. The use of the reliability method allows geotechnical engineers to handle uncertainties more explicitly and systematically, so that different design options can be compared and the optimal option can be chosen objectively. Cheung & Tang (2001) outlined the reliability-based method for slope design option selection. Emphasis was given on how the method handles the uncertainties in slope design and the potential consequences of failure. A schematic flow chart showing the design option assessment is given in Fig. 25.

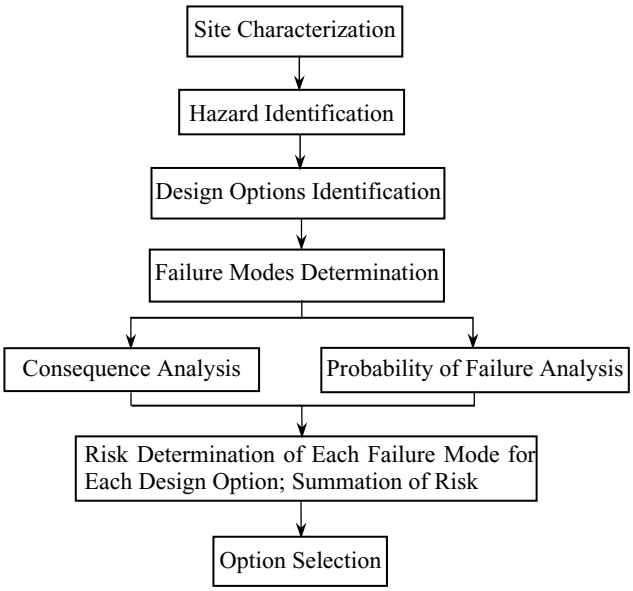

Fig. 25. Schematic flow chart for design option assessment (Cheung & Tang, 2001).

The method of scenario tree can be used to facilitate the identification of various combinations of design/decision alternatives, potential outcomes and quantitative risk analysis. An example on its potential use in charting out the design or improvement strategy for slopes in the Three Gorges River Zone is shown in Fig. 26 (Tang *et al*., 2006). The factors considered in the analysis include safety factor adopted for slope design, rainfall intensity, reservoir drawdown frequency, effectiveness of landslide warning system, and their impact on respective economic and fatality/injury losses. Table 4 compares the investment and the potential risk loss associated with three assumed levels of design safety levels. The table contains the key summary information to facilitate design decision. To extend further, the summary table can be expanded to the case of coupled mitigation strategies. An investment-risk analysis like this would facilitate and provide information for consideration in risk management.

Design scenario tree for Three-Gorge slopes

Abbrevations	Elements at risk				Vulnerability				Total slopes
E: effective	Economical Loss		Person		With warning		Without warning		
DFS: design factor of safety	Min.	Max.	Min.	Max.	Fatality	EL	Fatality	EL	
WDD: water draw down	50	100	50	300	0	0.4	0.3	0.7	225
SS: slope stability	Note: 1. The risk estimated here is on an annual base								
WS: warning system	2. The economical loss and inverstment are in terms of *million RMB/year*								
SN: scenario number	3. Class II slopes are considered								
EL: economical loss									

Input branches							Risk				Investment
Water table	Design F.S	Rainfall	Water Draw Down	Slope Stability	Monitoring system	Scenario No.	Economical Loss Min.	Economical Loss Max.	Fatality Min.	Fatality Max.	(Million/year)
					effective	1	0.036	0.073	0	0	
				failure 0.028%	0.6 effective						
			Yes 0.050		0.4 not effective	2	0.042	0.085	0.018	0.109	
		normal 0.95		no failure 99.972%		3	0	0	0	0	
					effective	4	0.002	0.003	0	0	
				failure 0.00007%	0.6 effective						
			No 0.950		0.4 not effective	5	0.002	0.004	0.001	0.005	
	1.25			no failure 99.99993%		6	0	0	0	0	182.74
					effective	7	0.002	0.004	0	0	
				failure 0.032%	0.6 effective						
			Yes 0.050		0.4 not effective	8	0.003	0.005	0.001	0.006	
		extreme 0.05		no failure 99.968%		9	0	0	0	0	
					effective	10	0.000	0.000	0	0	
				failure 7.34E-07	0.6 effective						
			No 0.950		0.4 not effective	11	0.000	0.000	0.000	0.000	
				no failure 99.99993%		12	0	0	0	0	
					effective	13	1.067	2.133	0	0	
				failure 0.83%	0.6 effective						
			Yes 0.050		0.4 not effective	14	1.244	2.489	0.53	3.200	
		normal 0.95		no failure 99.17%		15	0	0	0	0	
					effective	16	0.188	0.376	0	0	
				failure 0.008%	0.6 effective						
			No 0.950		0.4 not effective	17	0.219	0.439	0.094	0.564	
175m	1.15			no failure 99.992%		18	0	0	0	0	
					effective	19	1.223	2.446	0	0	
				failure 0.91%	0.6 effective						
			Yes 0.050		0.4 not effective	20	1.427	2.854	0.612	3.669	91.37
		extreme 0.05		no failure 99.09%		21	0	0	0	0	
					effective	22	0.011	0.022	0	0	
				failure 8.39E-05	0.6 effective						
			No 0.950		0.4 not effective	23	0.013	0.025	0.005	0.032	
				no failure 99.992%		24	0	0	0	0	
					effective	25	11.514	23.029	0	0	
				failure 8.98%	0.6 effective						
			Yes 0.050		0.4 not effective	26	13.433	26.867	5.76	34.543	
		normal 0.95		no failure 91.02%		27	0	0	0	0	
					effective	28	7.680	15.359	0	0	
				failure 0.315%	0.6 effective						
			No 0.950		0.4 not effective	29	8.960	17.919	3.840	23.039	
	1.05			no failure 99.685%		30	0	0	0	0	
					effective	31	12.826	25.652	0	0	
				failure 9.50%	0.6 effective						
			Yes 0.050		0.4 not effective	32	14.964	29.928	6.413	38.479	9.14
		extreme 0.05		no failure 90.50%		33	0	0	0	0	
					effective	34	0.431	0.862	0	0	
				failure 3.36E-03	0.6 effective						
			No 0.950		0.4 not effective	35	0.503	1.005	0.215	1.292	
				no failure 99.664%		36	0	0	0	0	

Fig. 26. A scenario tree for landslide risk assessment (Tang *et al.*, 2006).

Table 4. Comparison of risk and investment for selecting acceptable design strategy (Tang *et al*., 2006).

Design safety factor	Economical loss		Fatality		Investment
	Minimum	Maximum	Minimum	Maximum	(Million)
1.25	0.09	0.17	0.02	0.12	182.74
1.15	5.39	10.78	1.24	7.47	91.37
1.05	70.31	140.62	16.23	97.35	9.14

4 CONCLUSION

Decisions in geotechnical engineering have to account for uncertainties and potential consequences. Engineers need to evaluate the impact of uncertainties on the reliability level of their design. Risk is a combination of probability of failure event and the failure consequences. Risk assessment and management for slopes has been practiced in Hong Kong for many years. It has become part of Hong Kong's slope engineering practice and is playing an important role in landslide risk management in Hong Kong. A brief review of the engineering practice of risk assessment and management for slopes, and the research work conducted locally by academic institutions is presented. Research efforts should continue to keep abreast with the latest advances in the field and to provide insights and results that help address current and future geotechnical challenges.

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Partnering with the Community to Reduce Landslide Risk in Hong Kong Over the Past Thirty Years

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Abstract**:** Landslides are one of the most common natural hazards in Hong Kong. The serious landslide tragedies in the 1970s, which resulted in some 150 fatalities, prompted the establishment of a central geotechnical control organization, now the Geotechnical Engineering Office (GEO), 30 years ago to formulate and implement a comprehensive Slope Safety System to tackle the unique landslide problem in Hong Kong. Initially, the key strategy relied heavily on Government own efforts and concentrated on engineering means of geotechnical control and slope works. However, Government alone cannot solve the slope safety problem in Hong Kong. Experience over the past 30 years shows that community support and participation are essential to bring about substantial landslide risk reduction in Hong Kong. The United Nations has long been advocating building partnerships with the community as an important strategy for reduction of natural disasters. This paper gives a distil of our experience in the development of proactive community involvements in major landslide risk reduction measures in Hong Kong, including land use planning, non-development clearance of squatters, promotion of slope maintenance and education and advice on public response to landslip danger.

1 INTRODUCTION

With a population of about 7 million over a land area of about 1,100 square kilometers, Hong Kong is one of the most densely populated cities in the world. Over 60% of its land being on hilly terrain, Hong Kong has a substantial portion of the urban developments located on or near steep hillsides (Fig. 1), resulting in the creation of some 57,000 sizeable man-made slopes. Coupled with an annual rainfall of 2300 mm on average, the challenges that we have faced with regard to slope safety are rather unique in the world.

Fig. 1. Development of a city on hilly terrain (Hong Kong).

Landslides are one of the most common natural hazards in Hong Kong. Over the past century of Hong Kong history, landslides have occurred regularly. In the last 60 years, more than 470 people died as a result of failures associated with man-made slopes. Newspaper records of landslide fatalities and other impacts on the community date back much earlier to the 19th century (GEO 2005).

The Hong Kong history of frequent landslides culminated in two very serious disasters on the same day in 1972. On 18 June 1972 in Sau Mau Ping Estate in Kowloon, a 40 m high road embankment of loose fill collapsed, killing 71 people. This was followed a few hours later by the collapse of the hillside above a steep temporary excavation on Conduit Road in the Mid-Levels area of Hong Kong Island which triggered a landslide that demolished a 12-storey residential building and killed 67 people (Fig. 2). Four years later, another severe rainstorm hit Hong Kong and brought down three fill slopes in Sau Mau Ping Estate again, killing 18 people (Fig. 3). To prevent the recurrence of similar major disasters in Hong Kong, Government established in 1977 a central control organization, namely the Geotechnical Control Office (now the Geotechnical Engineering Office (GEO)), to regulate hillside development and the design, construction and maintenance of slopes. Since 1977, the GEO has developed a Slope Safety System to meet the community needs for a high standard of slope safety in Hong Kong and it has now evolved into a very comprehensive System (Chan 2005), which is highly regarded by geotechnical practitioners and natural hazard managers worldwide.

Fig. 2. Landslide at Po Shan Road in 1972 caused 67 casualties.

Fig. 3. Landslide at Sau Mau Ping In 1976 caused 18 casualties.

2 KEY STRATEGIES FOR REDUCING LANDSLIDE RISK

2.1 *Initial strategies for reducing landslide risk*

The Slope Safety System has continued to evolve in the last 30 years. When the System was first set up in 1977, Government faced the imminent problems of an effective system to control the geotechnical standards of land development and the existence of many substandard old man-made slopes (i.e. Pre-1977 slopes). Therefore, in the initial period, the key strategy relied heavily on Government own efforts to reduce landslide risk through two principal engineering measures, namely geotechnical control of new slopes (Post-1977 slopes) and retrofitting of old slopes (Pre-1977 slopes). While geotechnical control is to contain the increased risk arising from new developments by ensuring that all new slopes (i.e. Post –1977 slopes) are designed and constructed with proper engineering input, the retrofitting programme, namely Landslip Preventive Measures (LPM) Programme, is to progressively reduce the risk of old man-made slopes.

2.2 The importance of partnering with the community

With the unique challenges in slope safety facing Hong Kong described in Section 1 above, it is evident that Government will not be able to solve the slope safety problem alone. Experience over the past 30 years shows that community support and participation are essential to bring about substantial landslide risk reduction. This is in line with the International Strategy for Disaster Reduction as advocated by the United Nations (UN). In the International Decade for Natural Disaster Reduction Programme Forum Geneva, July 1999, Mr Kofi Annan, the then UN Secretary General, remarked that "*Prevention policy is too important to be left to Governments and internal agencies alone. To succeed, it must also engage civil society, the private sector and the media.*" (UN/ISDR 2004).

Over the years, the Slope Safety System has evolved into a very comprehensive one encompassing principal landslide risk reduction measures as follows.

- Upgrading and maintenance of Government slopes
- Geotechnical control of new developments
- Land use planning
- Clearance of squatters on hilly terrain
- Promotion of private slope maintenance
- Promotion of public response to landslide danger

The key strategies for reducing landslide risk are shown schematically in Fig. 4. Landslide risk which is a product of consequence and probability of slope failure could be lowered either by improving slope stability or by reducing landslide consequences or by both. New hillside developments could lead to a natural increasing trend of overall landslide risk (the dotted line in Fig. 4) if nothing is done. It is important to contain the increased risk arising from new developments by proper land use planning and geotechnical control. Risk from existing slopes could be reduced through slope upgrading and maintenance both by Government and private owners. Where slope stabilization is not a practical solution such as in squatter villages (see Section 4 below), landslide risk could be reduced by clearance of squatter structures to remove the consequence of slope failures. The adverse effects of landslides could also be minimized if the public takes personal precautions during heavy rain (Section 6).

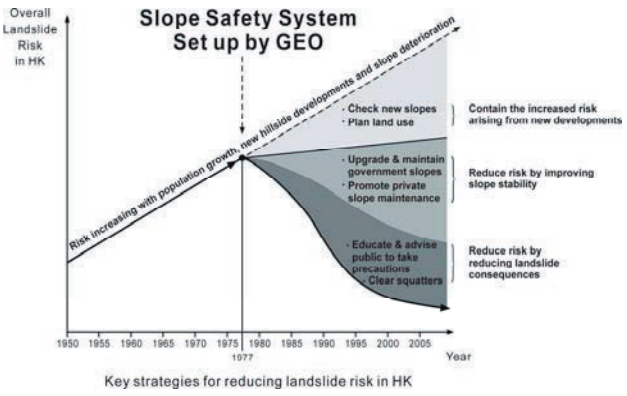

Fig. 4. Key landslide risk reduction measures.

The role and degree of the community involvement required for these risk reduction measures are summarized in Table 1, and elaborated further in Sections 3 to 7.

Table 1. Key landslide risk reduction measures.

Key landslide risk reduction measures	Degree of community involvement
Upgrading and maintenance of Government slopes	Mainly Government actions
Upgrading and maintenance of private slopes	Mainly private actions with assistance from Government
Geotechnical control of new developments	Mainly Government regulatory actions on public works and works by developers and professionals in the private sector
Land use planning	Government regulatory/planning actions with community support
Non-development clearance of squatters on hilly terrain	Support from squatters in accepting Government re-housing arrangements
Promotion of public response to landslide danger	Community support by proactively responding to Government landslip warnings and advice

3 LAND USE PLANNING

As part of the risk management strategy, careful planning of the use of land could be a cost-effective means to reduce natural disasters by mitigating the potential impacts of natural hazards in the planning process.

As Hong Kong has about 60% of its land lying on hilly terrain, a substantial portion of our urban developments have to be located on or near steep hillsides leading to increase in overall landslide risk. As it is best to reduce landslide risk arising from new developments as early as possible before concrete proposals are developed, the GEO provides geotechnical input in the early planning stage to identify geotechnical constraints on land developments and on the suitability of the land for particular purposes. With proper planning of the land use in hilly areas, the scale of the required landslide risk mitigation measures could be substantially reduced through special non-works provisions or allowance could be specified, such as designation of part of the land adjoining hillside as non-building areas or change of the location of the proposed buildings to avoid hazardous areas underlain by difficult ground conditions. In some cases, the scale of the required mitigation measures may be so extensive as to render the proposed developments economically not viable and it may be necessary to give up the proposed development or change the proposed use of the land, such as changing from a residential development to an open playground.

Community support is needed for implementing Government land use planning. However, the community is often inattentive to the landslide hazards they face, underestimates the level of risk and overestimates their ability to cope with the potential disasters. When land development proposals are turned down or restrictions imposed on the use of the land on slope safety grounds, strong reaction from the proposers and the community may result. It is important for Government to let the public understand the rationale behind the decision and solicit their support. The proposed usage of land and the associated restrictions imposed on geotechnical grounds, if accepted by the community, could be more cost-effective when compared with expensive landslide mitigation measures, which could be totally disproportionate to the development costs and benefits.

Some examples illustrating how planning of land use could contribute to the containment of landslide risk are discussed in the following sections.

3.1 Modification of proposed land use

The subject site is located in the foothill of Castle Peak at Tuen Mun Area 19 of the New Territories. The site was originally planned for a major public housing development. However, in 1990, a large scale debris flow (about 19,000m^3 debris originated at the top of the natural terrain overlooking the site) travelled over 1 km and reached the proposed site area (Fig. 5). Ground investigation shows that the site is underlain by weathered andesite of low shear strength. In view of the weak andesite and the high risk of further debris flow from the natural terrain, the land use of the site was subsequently changed from housing developments to a golf driving range and a highway (Tuen Mun Foothills Bypass), which traverses the toe of the hillside (Fig. 6).

Fig. 5. Large scale debris flow in 1990.

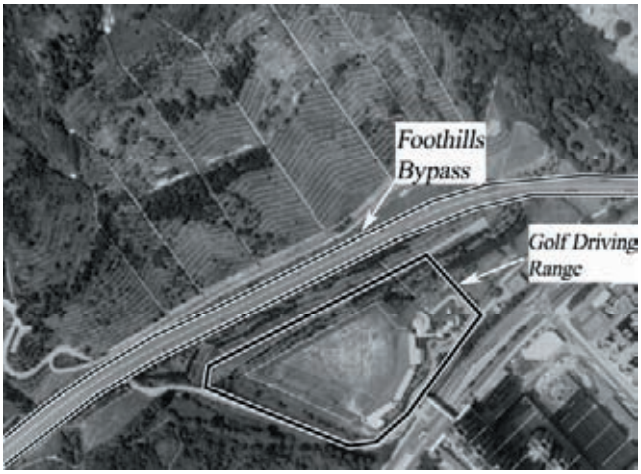

Fig. 6. Tuen Mun Foothills Bypass and a golf driving range have been finally developed.

3.2 Prohibition of a village house development

The site is located in a village area in Sai Kung of the New Territories and directly overlooked by steep natural terrain (Fig. 7). A number of small village houses were proposed in 2006. Owing to the close proximity of the site to steep natural terrain, a preliminary study showed that the proposed development may be subject to an unacceptable landslide risk and construction of landslide risk mitigation measures would be required. However, the estimated total costs of these measures were found to far out-weight the economic benefits of such small house development. The development proposal was turned down by Government with effort paid to win the community support by explaining to them the reasons behind the objection to the proposed development and seeking their understanding of Government decision.

Fig. 7. Proposed small house development on hillside.

The above two examples highlight the importance of land use planning in minimizing landslide risk due to new hillside developments in Hong Kong.

4 REDUCING LANDSLIDE RISK TO SQUATTERS

In Hong Kong, due to severe shortage of low cost housing, particularly during the period of 1950s to early 1980s, many squatter structures were constructed throughout the territory, frequently on steep hillsides, which were unsuitable for development from the perspective of slope safety. Squatter structures in hilly areas are particularly vulnerable to landslide hazards and even relatively minor slope failures could be disastrous. A typical squatter area on hilly terrain is shown in Fig. 8. Many casualties arising from landslides in squatter areas occurred, e.g. in 1982, more than 700 landslides were reported in squatter areas, resulting in 23 fatalities.

Fig. 8. Typical squatter area on hilly terrain.

In Hong Kong, stabilization of slopes in squatter areas is normally not practical, since, in many cases, works cannot be carried out without first demolishing the structures, in order to provide access and working space. Once the squatter structures are cleared, the risk to the residents is removed. Furthermore, due to the close proximity of squatter structures to the slopes, there could be problems associated with construction safety and subsequent maintenance. In the case of squatter areas in large areas of natural terrain, works needed to adequately stabilize the terrain could be very costly, time consuming and environmentally undesirable and are grossly disproportionate to the cost of clearance and re-housing of the squatters at risk. It has been Government policy to reduce landslide risks to squatters through clearance of the affected squatter structures and offer of re-housing to eligible squatters rather than by stabilizing the slopes. Since 1984, Government has an ongoing programme to reduce the vulnerability of squatters to landslide hazards through "non-development clearance" (NDC) (i.e. clearance based solely on safety considerations and not on development grounds).

4.1 Non-development clearance (NDC) programme

Under the NDC programme, geotechnical engineers from the GEO conduct terrain classification mapping, aerial photograph interpretation and reference to records of landslide casualties to prioritize squatter areas for NDC studies. Systematic inspections are then carried out and through expert judgment, squatter structures which are especially vulnerable to landslides during periods of heavy rainfall are identified and appropriate clearance recommendations made.

The conditions of a typical squatter area before and after non-development clearance are shown in Fig. 9 & 10.

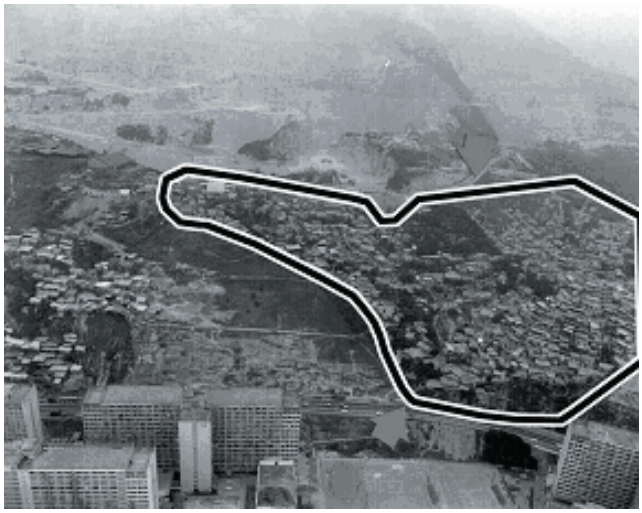

Fig. 9. Squatter area before clearance.

Fig. 10. Squatter area after clearance.

4.2 Categories of squatter structures for NDC

Squatter structures exposed to unacceptably high landslide risk are currently classified into NDC Category 1 "immediate and obvious danger", and Category 2 "especially vulnerable to landslides during periods of heavy rainfall" (in other words, not immediately dangerous, but liable to become dangerous at some unpredictable time in the future during heavy rain). Clearance of Category 1 squatter structures is compulsory and would be backed up by forced eviction where necessary. So far, there has been little problem in dealing with Category 1 squatter structures. In practice, Category 1 structures are mostly identified during periods of heavy rainfall when evidence of landslide danger has been reported to the GEO, in the form of a landslide or other obvious signs of distress. Such evacuations are rarely resisted because the danger is visually obvious and self-evident to the occupants. In contrast with Category 1 structures, squatter structures recommended for NDC Category 2 will be cleared through persistent persuasion of the squatters to leave on a purely voluntary basis due to the very strong resistance and undesirable social consequences of forced eviction.

4.3 Local community support for reduction of landslide risk to squatters

Local community support is essential for the success of the NDC programme. In particular, Government has to make the affected squatters understand the potential landslide danger and to persuade them to accept the re-housing offer. For those squatters who have not yet accepted re-housing offer, Government repeatedly explains to them the potential danger they are facing with a view to persuading them to move. Meanwhile, sustained public education is ongoing to alert squatters to the landslide risk and to solicit cooperation from these local communities to take necessary precautionary measures during heavy rain. Notices are posted on structures which have been recommended for NDC, and advisory letters are issued to the occupants warning them that they are assessed to be at risk from landslides and advising them to move to temporary shelters during inclement weather. The GEO has also been erecting warning signs in areas with undischarged NDC recommendations. Before each wet season, warning leaflets, which provide guidance to squatters on measures and precautions to safeguard themselves from landslide danger during rainstorms, and particularly when the Landslip Warning is issued and broadcast in TV and the radio, are distributed to residents in all squatter areas.

Since 1984, about 75,000 squatters on steep hilly terrain have been cleared either through NDC or development clearance. The risk to squatters from landslides in Hong Kong has been reduced significantly as is evident from the landslide casualty statistics. However, about 10,000 squatters recommended for NDC have still chosen to stay-put. It is important for Government to continue the persuasion effort and public education in order to win support from local squatter communities to further reduce the landslide risk in these areas.

5 PROMOTION OF PRIVATE SLOPE MAINTENANCE

5.1 *Importance of action from private slope owners*

As with other types of engineering structures, man-made slopes need regular maintenance to prevent deterioration. Experience shows that the lack of maintenance is a major contributory factor to many landslides in Hong Kong. Poorly maintained slopes, even those that were originally designed and constructed to adequate engineering standards, can deteriorate to the point where they may fail, possibly resulting in loss of life and damage to property. Fig. 11 gives some examples of landslides due mainly to lack of maintenance.

Fig. 11. Landslides caused by lack of maintenance.

In Hong Kong, there are about 57,000 man-made slopes registered in the Government Catalogue of Slopes. Government has a systematic programme to maintain all its 39,000 Government slopes (i.e. two thirds of the total slopes in Hong Kong) and to retrofit high priority old slopes under the LPM Programme. To ensure the continued stability of all man-made slopes, private owners have to take responsibility for the 18,000 private slopes (i.e. one third of the total). Before the commencement of systematic public education on slope safety in 1992, the majority of the public considered that Government was responsible for all slopes in Hong Kong. To address this misunderstanding, a key objective of the public education programme is to let private owners know that the responsibility for maintaining private slopes lies with private owners and not Government. Also, owners should understand that most landslides are caused by lack of maintenance and they should take prompt actions to regularly maintain their slopes in accordance with the published Hong Kong standard, Geoguide 5 – "Guide to Slope Maintenance" (GEO 2003).

5.2 Measures to motivate owners to take responsibility for their slopes

A clear demarcation of maintenance responsibility of slopes is an important component of any slope safety system. Understanding owners' difficulty of interpretation of special lease conditions related to slope maintenance, Government carried out a US$ 10 million project – "Systematic Identification of Maintenance Responsibility of Slopes in the Territory" (SIMAR) by engaging estate surveyors and lawyers to identify and set up a register of slope maintenance responsibility for all registered man-made slopes in Hong Kong. The information on maintenance responsibility has been uploaded to a Government website (http://www.slope.landsd.gov.hk/smris/), providing free access to private slope owners and the public anytime at home or in their offices.

To ensure that prospective home buyers know their slope maintenance responsibility, developers are required to include in their sales brochures for new flats clear statements, spelling out the flat owners' maintenance obligations for slopes together with a site plan showing the slopes under their responsibility. The maintenance responsibility of slopes should also be stated clearly in the relevant clauses and site plans in the Deed of Mutual Covenant, which forms part of the land lease. Prospective flat

purchasers are encouraged to check their liability for slopes before they decide to buy the properties and as part of their briefing of buyers of special conditions in the lease, solicitors will advise the buyers of slope maintenance requirements (if any).

To further encourage private owners to maintain their slopes, Government has set up a loan scheme, namely the Building Safety Improvement Loan Scheme, administered by the Buildings Department, to provide loans to individual owners who may wish to obtain financial assistance for slope works.

Other on-going public education activities to motivate community actions on slope safety will be discussed in detail in Section 7.

6 PROMOTION OF PUBLIC RESPONSE TO LANDSLIDE DANGER

6.1 *Importance of personal precautionary measures during heavy rain*

In Hong Kong, with the hilly terrain, heavy rainfall and intense development, it will be an impossible task to achieve zero landslides despite our strenuous efforts to rectify old substandard slopes and carry out regular maintenance of slopes. Statistics shows that there is still an average of 300 landslides reported each year. A cost-effective way of reducing adverse effects of the landslides is for the public to take personal precautionary measures during heavy rain to protect themselves and their family members against landslide danger.

6.2 Public warnings of landslide risk

Most landslides in Hong Kong are caused by heavy rainfall. By using a combination of real-time rainfall data and rainfall forecasts from the Hong Kong Observatory (HKO), and based on GEO's study on the landslip/rainfall relationship, the HKO in consultation with the GEO issue a public warning, known as the Landslip Warning (Fig. 12) to alert the public when it is considered that many landslides will occur. When the Landslip Warning is issued, local radio and television stations will broadcast the Warning to the general public at regular intervals, together with advice on the precautions that should be taken. The operation and details about the Landslip Warning are well documented elsewhere (GEO 2006).

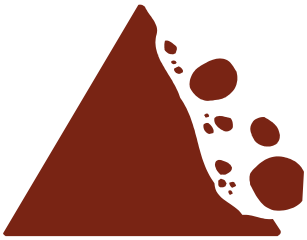

Fig. 12. Landslip Warning Signal.

A review of the landslide fatalities since 1985 has indicated that more than 90% of the landslide fatalities occurred when the Landslip Warning was in force. Therefore, the Landslip Warning has proved to be effective and reliable in giving forewarnings of landslide danger to the community, but there is certainly a need for extra effort to educate the general public to take the necessary precautions.

6.3 Promotion of Community Participation to Protect Themselves

The GEO recent studies reveal that 80% of landslide fatalities in the past 20 years were associated with squatters, pedestrians and motorists and this highlights the importance to devote greater effort and devise specific warning messages for these special groups of the community.

6.3.1 Squatters

Special warning signs (Fig. 13) have been erected in squatter areas to warn those squatters which have been found to be especially vulnerable to landslide risk during heavy rain. During a Landslip Warning, these squatters are advised to make immediate arrangements to move to a safe shelter. Government will open temporary shelters for the public including anyone whose dwelling is endangered by unstable slopes or boulders.

Fig. 13. Special warning signs for squatters.

6.3.2 Pedestrians

When the Landslip Warning is in force, the public should keep away from slopes. Pedestrians should avoid walking or standing close to steep slopes, especially where landslip warning signs are erected (Fig. 14). People should also cancel non-essential appointments and to stay at home or in a safe shelter.

Fig. 14. Warning signs for pedestrians.

6.3.3 Motorists

When the Landslip Warning is in force, motorists are advised to avoid driving in hilly areas or on roads with landslip warning signs (Fig. 15).

Fig. 15. Warning signs for motorists.

It is our belief that many landslide fatalities in the past could have been avoided if the community had taken heed of our Landslip Warning and took personal precautionary measures. It is crucial that the public responds positively to landslip warning messages promulgated by Government.

6.4 Landslide emergency service

As landslides are inevitable in Hong Kong, Government is committed to providing an effective and efficient response to all emergency situations which threaten life and property by establishing an Emergency Response System (ERS) managed by the Security Bureau. Under the ERS, the GEO is required to maintain a 24-hour, year-round service to provide geotechnical advice to other Government departments on actions to be taken in case of danger arising from landslides. To safeguard public safety, geotechnical engineers of the GEO may give advice on closure of roads (Fig. 16) and evacuation of buildings as an immediate precautionary measure, which may inevitably result in significant social disruption to the public. Therefore, in deciding on the most appropriate emergency measures, geotechnical engineers have to exercise professional judgment and common sense in striking a good balance between the estimated risk of any possible landslides and the inconvenience caused by the recommended measures. The successful implementation of these measures always requires the understanding and support from the affected local communities.

Fig.16. Road closed as immediate emergency measure.

Where a high demand for emergency service is anticipated, such as when a Landslip Warning or Typhoon Signal No. 8 or above is issued, the GEO will immediately mobilize its Emergency Control Centre (ECC) (Fig. 17) to deal with the expected large amount of requests for emergency advice. The ECC, headed by a Senior Geotechnical Engineer, is normally manned by 12 members of professional and technical staff on a rotational basis, working in 8-hour shifts. If needed, additional emergency teams can be called in to increase the resources for dealing with large number of landslide incidents.

Fig.17. GEO Emergency Control Centre.

In Hong Kong, managing public communications during landslide crisis is as important as handling the physical emergency operations. The community always expects very fast and effective Government response to their questions, particularly at times of large number of landslides with serious consequences, such as casualties, mass evacuation of buildings or closure of major roads. Understanding the importance of crisis communication, the GEO has prepared a Crisis Management Plan under which a Crisis Communication Centre (CCC) will be set up to deal with questions and queries from the media and other stakeholders when a severe landslide event has developed into a crisis situation. The CCC will be headed by a senior officer at Deputy Head (GEO) level and supported by a team of professional and technical staff. The CCC will also be provided with adequate IT equipment to ensure that the centre will always have a fast and effective access to all up-to-date landslide situation reports and slope-related information. The centre will closely monitor the relevant news and the public opinions in newspapers, radios and TVs and prepare appropriate response to address the community concerns. Past experience shows that poor performance in crisis communication can cause the public to lose their confidence in Government action. Therefore, proper handling of crisis communication is essential for Government to solicit community support during a crisis. Since the setting up of the Crisis Plan in 1995, GEO has operated the CCC two times, with the first time in August of 1995 in response to a number of serious fatal landslides, including the major landslide at Shum Wan Road (Fig. 18) involving two deaths. The CCC was triggered again in August 1999 following the landslide incident in Shek Kip Mei (Fig. 19), involving a mass evacuation of about 700 people.

Fig.18. Two people killed in Shum Wan Road landslide (1995).

Fig.19. Shek Kip Mei Landslide (1999).

7 STRATEGY AND ACTIONS FOR PARTNERING WITH THE COMMUNITY

With experience gained over the last 30 years, the GEO has developed a proactive and comprehensive strategy to motivate community actions to reduce landslide disasters in Hong Kong. The key strategy is to partner with the community to promote public awareness and response in slope safety through public education, publicity, information services and public warnings, as follows.

- Community education to maintain public awareness of landslide risk
- Information services and training
- Community advisory services
- Partnering with non-Government organizations (NGOs)
- Partnering with the media

7.1 Community education to maintain awareness of landslide risk

7.1.1 Importance of maintaining constant vigilance about landslide risk

In Hong Kong, public awareness has a great bearing on the attitude of the community towards landslide risk. At times of high level of awareness, the community will support Government slope safety initiatives and call for more resources to be allocated on landslide reduction programmes. It is also easier to win public understanding and support of Government proposed restrictions on land use, NDC and social disruptions/inconvenience resulting from slope works and landslide emergency measures. Owners will be more willing to spend money and effort to maintain and upgrade their slopes. People will also take heed of Government warning messages and take personal precautions during heavy rain. On the contrary, when the public does not consider slope safety to be a problem, people tend to pay little attention to slope maintenance/upgrading and Landslip Warnings and landslip warning signs, and may unnecessarily expose themselves to landslide risk. It is our challenge to maintain public awareness at reasonably high level even at relatively uneventful years with respect to landslides.

7.1.2 Television announcements of public interest (TV-API)

To assist us in formulating the best means to promote different slope safety messages, the GEO has, since 1997, appointed universities to carry out annual public opinion surveys to assess the effectiveness of various means of promulgation of the key messages. The survey results clearly show that broadcasting of television announcements of public interest (TV-API) is the most effective way to convey the slope safety messages (Fig. 20). Accordingly, dedicated effort has been given to produce interesting and impressive TV-APIs, approximately one new API in every two years. Seven TV-APIs with the main themes focused on slope maintenance and personal precautionary measures have been produced and broadcast.

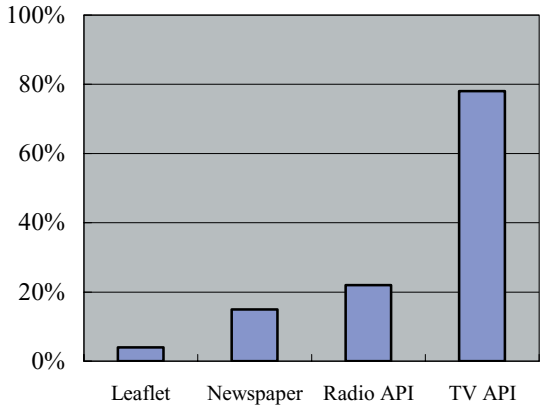

Fig. 20. TV-API to be the most effective means of publicity.

To ensure high-quality APIs, professional film production companies have been engaged to produce commercial-like 30-second short films using modern filming techniques with dynamic sound effects. A wide variety of filming techniques and styles are adopted to arouse maximum possible attention from the community. To ensure timely delivery of the right messages, API promoting slope maintenance is broadcast more frequently in the dry season to remind slope owners to complete all maintenance works before the onset of the rainy season. During the rainy season, the API on Landslip Warning and personal precautionary measures is broadcast to alert the public to weather conditions and Government warnings. To ensure the maximum possible penetration into the community, the APIs are also shown in the regular roving exhibitions on slope safety in schools and popular shopping centres.

7.1.3 Publicity programmes on targeted groups

As some of the slope safety messages are more relevant to a particular group of people, it is more effective and logical to plan

special publicity programmes targeted at different groups of people in the community. Some examples are discussed below:

7.1.4 Students

Students have long been targeted as one of our main focuses in the public education programme. In fact, the subject of landslides has been incorporated into the geography curriculum of secondary schools in Hong Kong. To facilitate teaching in class, the GEO has produced a teaching kit on landslides (Fig. 21) which includes workbooks, worksheets, VCD and cassettes. To supplement classroom learning, geotechnical engineers from the GEO also give regular school talks and hold exhibitions in primary and secondary schools (Fig. 22). Recently, exhibitions have been extended to various universities. Through these activities, students, especially primary students and junior secondary students, can act as "slope safety ambassadors" to bring slope safety messages back home to parents and other family members. Their role is reinforced through assigned homework and projects where students are encouraged to discuss and seek assistance from their family members.

Fig. 21. Teaching kit for secondary school students.

Fig. 22. School talk and exhibition on slope safety.

7.1.5 Children

"Children are pillars of the future society" and education is best to start with children. The GEO well recognizes the importance of providing suitable education and stimulation to this group of the community. Our publicity strategy to children is to focus on arousing awareness by means of interesting photos, films, games, etc. To this end, the GEO has produced cartoon books, cartoon VCD, flash cards, chesses, children umbrellas and stationeries of all kinds (Fig. 23) and developed a number of interactive computer games. The principal objective is that children would learn our slope safety messages through entertainment and games.

Fig. 23. Examples of publicity materials for children.

7.1.6 Morning walkers (unauthorized cultivation)

Since 2000, there has been an increasing concern on the adverse effects of unauthorized cultivation in hillsides on slope stability. To discourage unauthorized cultivation mainly by morning walkers, the GEO has included this subject in our on-going publicity and public education programme.

A poster and a leaflet have been produced for this purpose and District Lands Offices and District Offices have been distributing these promotional materials. The leaflets are handed out directly to the public during roving exhibitions. The GEO also regularly distributes pamphlets, handy fans and towels to morning walkers with slope safety messages against unauthorized cultivation (Fig. 24).

Fig. 24. Poster and handy fans distributed to morning walkers.

7.1.7 Activities to encourage participation from the community

To encourage active participation from the community, the GEO has organized open competitions targeted at students and general public. Examples include competitions on "Best Computer Animation of Notable Landsides in Hong Kong", "Best Slogan and Bookmark Design on Reducing Natural Disasters", "Best Landscaped Slope Awards" and "Building the Highest Slope" (Fig. 25). The GEO also organizes regular roving exhibitions at popular shopping centres (Fig. 26). Quiz games are held in the exhibitions and souvenirs with slope safety messages are given to the winning participants.

Fig. 25. Building the highest slope competition.

Fig. 26. Roving Exhibition.

7.1.8 Special themes of exhibitions

In addition to roving exhibitions, the GEO organizes major exhibitions with special themes once every two years. Three examples are given below:

Exhibition on Soil and Slope (2002)

In this exhibition, we focused on the use of scientific methods to describe the many facets of Hong Kong soils and slopes (Fig. 27). Opportunity was also taken to promote general understanding of geological and geotechnical knowledge. The exhibition was specifically arranged in the summer vacation and at the Hong Kong Science Museum attracting over 75,000 visitors, mainly students/children and their families.

Exhibition on the History of Landslides in Hong Kong (2004)

As reflected in the results of public opinion survey, there was a significant drop (about 10 %) in the level of public awareness of slope safety and of the importance of slope maintenance during the years 2002-2004. This drop was likely due to the reducing number of landslides and associated casualties and social disruptions in the relatively dry years. Such trend of decreasing public awareness of landslide danger was alarming. The GEO therefore stepped up efforts to raise landslide risk awareness both in Government and in the community at large. As exhibits of past major landslides are very effective in reminding the public of the landslide danger, a major exhibition on the "History of Landslides in Hong Kong" was organized in 2004 with display of photographs of landslides in Hong Kong which dated back to 1898 (Fig. 28). In addition, a lucid and concisely edited video showing notable landslides in the past was broadcast during the exhibition. In view of the success of the exhibition, the old photographs collected were put together to produce a history book entitled "When Hillsides Collapse – A century of Landslides in Hong Kong" (GEO, 2005) as a permanent record for maintaining public awareness of landslide problems in Hong Kong.

Fig. 27. Exhibition on soil and slope.

Fig. 28. Exhibition and book on landslide history in Hong Kong.

Exhibition on "Safer Living – Reducing Natural Disasters" (2005-2006)

In response to the call of the International Strategy for Disasters Reduction (ISDR) under the United Nations, the GEO organized the "Safer Living – Reducing Natural Disasters" campaign jointly with the Security Bureau, Hong Kong Observatory and other relevant Government departments and NGOs with shared vision. The campaign aimed at enhancing public understanding of common natural hazards, e.g. landslide, flooding and typhoon, so that appropriate response actions could be taken to reduce natural disasters for a safer living. The 12-month campaign covered a full spectrum of publicity activities (Fig. 29). Major exhibitions on natural hazards in Hong Kong were held in popular shopping centres and the Hong Kong Science Museum. TV special programmes and documentaries were produced. Lectures and talks targeted for general public were organized. A major international seminar was organized in October 2005 with speakers from world-renowned organizations such as the United Nations, World Meterorological Organization and International Federation of Red Cross. Interesting and dynamic demonstrations of rescue operations performed by the Hong Kong Red Cross, Auxiliary Medical Services, Civil Aid Services and Fire Services Department were also arranged. All these activities were well received by the public.

(a) Launching ceremony

(b) TV episodes

(c) Tropical cyclone name nomination contest

(d) Popular science lecture

(e) Major exhibition

(f) Rescue drill demonstration

(g) International seminar

Fig. 29. Some activities held under "Safer Living" campaign.

7.2 *Information services and training*

Slope maintenance starts with information. The GEO has set up a computerized Slope Information System (SIS), containing the digital data of slope-related information for some 57,000 sizeable man-made slopes registered in the Catalogue of Slopes. The SIS is a Geographic Information System (GIS) that integrates different types of datasets in spatial and textual attributes. It provides a valuable source of up-to-date slope information to assist engineers, slope owners and property managers to arrange for necessary slope inspections and works. The slope information is disseminated in a web-based framework that provides a convenient and free access to the general public through the Internet (Hong Kong Slope Safety Website http://hkss.cedd.gov.hk) (Fig. 30). The SIS is one of the largest and most comprehensive slope information databases in the world and has won high regards from geotechnical practitioners both local and worldwide. The Website also contains a wealth of information and advice on slope maintenance, personal precautionary measures during heavy rainstorms, progress of Government slope works, landslide investigations, natural terrain landslide hazards, etc. The net result is that the public is able to have a good grasp of slope safety issues and this greatly helps to win their good support and participation to enhance slope safety in Hong Kong.

Fig. 30. Hong Kong Slope Safety Website.

Fig. 31. Examples of VCD, pamphlets and brochures.

Pamphlets, information notes, information sheets, VCDs, brochures, information kits etc. are prepared to disseminate slope safety messages (Fig. 31).

The GEO also provides training on slope maintenance to various professional bodies that may be involved or have an interest in slope safety issues. For example, seminars are provided to the Hong Kong Federation of Insurers, the Hong Kong Institute of Surveyors and the Hong Kong Association of Property Management Companies.

7.3 *Community advisory services*

Many slope owners are laymen and may not possess the required information, knowledge or expertise in arranging for slope inspection and works. The GEO has been proactively providing various kinds of technical assistance and advisory services to help owners to maintain their slopes.

A dedicated unit, known as "Community Advisory Unit" (CAU), has been set up in the GEO to assist private slope owners to discharge their slope maintenance responsibility through direct community outreach advisory services. The CAU has the following principal functions :

1 organize slope safety and maintenance seminars and talks for private slope owners and bodies involved in slope maintenance;
2 provide the meet-the-public service by our geotechnical engineers to answer queries on slope safety matters (Fig. 32);
3 meet private owners who have received statutory Dangerous Hillside Orders to advise them on how to proceed with the necessary slope upgrading works (Fig. 33);
4 meet Owners' Corporations and Mutual Aid Committees to advise them on how to undertake slope maintenance works (Fig. 34).

Fig. 32. Meet the public service.

Fig. 33. Provide assistance to private slope owners.

Fig. 34. Meeting with Owners' Corporations.

7.4 Partnering with non government organizations (NGOs)

Promoting slope safety messages with NGOs is an effective and win-win public education strategy. By partnering with the community, Government secures supports from different influential NGOs, which share the common vision with us – "reducing natural disasters by education". Recent examples include the cooperation with the Hong Kong Red Cross (HKRC) in organizing the "Safer Living – Reducing Natural Disasters" campaign which fulfills one of the missions of the HKRC, i.e. to "deliver impartial and quality care to protect life and health and to enhance the capacity of vulnerable people to live a safe and dignified life". The GEO has also joined hands with the HKRC in providing community service to residents in remote villages (Fig. 35). Our common theme is to encourage "maintenance"- regular maintenance of body health and slopes alike. The responses from the villagers, especially the aged group, were very good.

Another example is the joint efforts with local green groups. In 2004, the Friends of the Earth (Hong Kong) and the China Polar Museum Foundation assisted us in organizing the "Best Landscaped Slope Awards". The main objective of the Awards was to promote public awareness in slope safety and aesthetics and arouse slope owners' interest in providing landscape treatment to their slopes. We also received supports from the Green Power in some of the activities held under the "Safer Living – Reducing Natural Disaster" campaign.

Fig. 35. Community activity jointly arranged by HKRC and GEO.

7.5 Partnering with the media

The media is powerful and influential in promulgating key slope safety messages to the community. News, feature stories and editorials have a major bearing on the degree of the community support of Government slope safety initiatives. It is important for us to develop good relationship with the media. Understanding the media's need for "news", the GEO has been proactive in supplying information and materials to the media to prepare features stories, such as new technologies and innovative methods of slope stabilization. A dedicated unit in the GEO has been set up to ensure prompt and effective response to the media questions, particularly after occurrence of serious landslides. Regular press conferences and media briefings are organized to promulgate important slope safety messages (Fig. 36). To ensure the media has a good understanding of slope safety policies, regular contacts with journalists are maintained for discussion about the various functions of the Slope Safety System (Fig. 37).

Fig. 36. Regular press briefing.

Fig. 37. Visit of journalists to GEO.

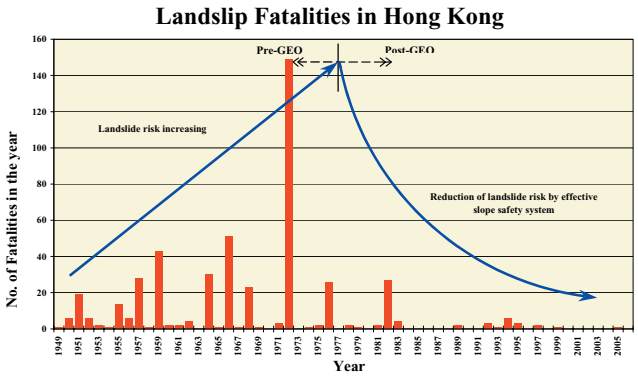

Fig. 38. Landslide fatalities.

8 CONCLUSIONS

As a result of the serious landslide disasters in 1970s, Government established the GEO in 1977 to formulate and implement effective strategies for reducing landslide risk in Hong Kong. A Slope Safety System was then set up and has continued to evolve over the last 30 years. Initially, Government relied heavily on its own efforts alone and concentrated on engineering means, e.g. geotechnical control of new developments and retrofitting of substandard old slopes under the LPM Programme. However, with the hilly terrain, intense rainfall and intense developments, Government action alone cannot solve the Hong Kong slope safety problem and community involvement is essential to further reduce the landslide risk. As with the United Nations advocating community participation in combating natural hazards, the GEO has developed a proactive and effective strategy to motivate community actions in its key landslide risk reduction measures, including land use planning, non-development clearance of squatters, promotion of private slope maintenance and community education and advice on public response to landslide danger. The strategy has been successfully implemented through sustained public education programmes, comprehensive information services, proactive advice and assistance, and partnering with stakeholders including the media and relevant non-Government organizations. Experience over the past 30 years has shown that the strategy for partnering with the community is a win-win situation to reduce the overall landslide risk in Hong Kong as reflected by the significant decrease in landslide fatalities (Fig. 38). Despite the success in the past 30 years, Government and the community should not be complacent. It is most important that everyone always maintains vigilant about the landslide risk and plays his/her part diligently to maintain the highest standards of slope safety in Hong Kong.

ACKNOWLEDGEMENTS

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Design Practice and Technical Developments of Soil Nailing in Hong Kong

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Abstract: Soil nailing has been used in Hong Kong since mid 1980s. The technique has gained popularity because it offers effective and economic reinforcing system for a variety of ground conditions. Principles of soil-nailed system including failure modes and nail-ground interactions are highlighted and discussed. Current design approach and method are presented. Considerable experience and knowledge of the mechanisms and behaviour of soil nailed structures have been gained in recent years through systematic research and development studies. The studies include field tests, site trials and monitoring, numerical and physical modeling, and laboratory tests. They lead to the development of many technological advances in the design and construction. Study results are presented along with the advances made.

1 INTRODUCTION

In upgrading sub-standard slopes in early years, options like retaining structures, sub-surface drainage and cutting back to reduce slope angle were preferred. Later, new techniques of strengthening the ground insitu were introduced to minimize the need for excavation and construction of retaining walls. These developments led to the use of soil nails. In Hong Kong, soil nails are generally in the form of steel bars installed by the drill and grout method without prestressing. In early 1990s, some practices for design of soil nails for upgrading existing slopes were summarised by Watkins & Powell (1992), which soon became the norm for soil nail design.

With more existing slopes and retaining walls need to be upgraded each year under the Landslip Preventive Measures (LPM) Programme of the Hong Kong Special Administrative Region, the soil nailing technique has gained popularity since mid 1990s. The technique is now widely applied to stabilize man-made slopes, and sometimes applied to retain deep excavations. Although many soil nails are installed each year and the performance of the soil-nailed slopes is generally satisfactory, an improved understanding of the load transfer mechanism and the mechanical behaviour of soil nails is warranted in order to identify room for rationalisation of the design practice and to ensure the safe application of innovative materials and construction technology. To this end, a number of soil nail related studies have been undertaken systematically since late 1990s. The studies involve field load tests, site trials, numerical and physical modeling, and laboratory tests. They have brought about technological advances in respect of design and construction. New design and construction guidelines have been developed, e.g. GEO Technical Guidance Notes (TGN) numbered 19 and 23 (GEO 2004a; GEO 2006). This paper gives an overview of local soil nail design practice and recent technical developments.

2 MERITS AND LIMITATIONS OF SOIL NAILING

The concept of soil nailing involves creating a stable block of composite material by strengthening the insitu ground with soil nails. This requires that the soil nails are installed at close spacing, both horizontally and vertically. The soil nailing technique has a number of merits:

- It is well suited for cramped sites with difficult access because of the use of relatively small and mobile drilling rigs.
- It is a flexible method that can cope with frequent variations in ground conditions during construction.
- It can generally result in time and cost savings compared to conventional techniques of cutting back and retaining wall construction.
- It causes less environmental impact as no earthworks is required and existing trees can be retained.
- It is more robust than the conventional slope stabilization measures of cutting back (Ho *et al.*, 2002).
- It renders ductile failure mechanism of a slope, thus providing warning signs before failure.

Like every other stabilization technique, soil nailing has its limitations:

- The presence of utilities and structures can limit the extent of soil nails.
- Unless agreement is obtained from owner of adjacent land, soil nails cannot extend beyond the lot boundary. This places restriction on the layout of soil nails.
- Special corrosion protection measures such as corrugated plastic sheathing are needed in aggressive ground and they could be costly.
- The presence of high groundwater table can lead to construction problem.
- Nailed excavations may result in relatively large horizontal displacements and cause damages to nearby structures or utilities.
- Soil nailing in soft clay can have problems of creep and very low nail capacity.
- Sites with highly fractured rocks and open joints or voids may limit its application due to potential grouting problem.

3 PRINCIPLES OF SOIL-NAILED SYSTEM

3.1 *General*

Soil nailing is an in-situ reinforcement technique used for enhancing the stability of slopes, retaining walls and excavations. The nails interact with the ground to support the stresses and strains that would otherwise cause the unreinforced ground to fail. The internal stability of a soil-nailed structure is usually considered in terms of two zones (i.e. active and passive zones) separated by a potential failure surface (see Fig. 1). The region in front of the potential failure surface tends to detach from the slope and is defined as the active zone. The region behind the

potential failure surface, that remains more or less intact, is the passive zone. The two-zone concept is only a convenient idealism for limit equilibrium model. In reality there is a complex shearing zone subject to shear distortion (CIRIA 2005). The following description of nail-ground interaction is based on the idealized two-zone system.

The ground has the potential either to move a small amount as a coherent mass, or to flow past the proximal end of the nail if it is not adequately restrained by the nail head, and the soil-nail friction within the active zone.

The shear stress exerted on the surface of soil nails within the active zone is directed outwards and has a tendency of pulling out the nails. The shear stress exerted on the surface of soil nails in the passive zone is directed inward and prevents the pulling out of the nails.

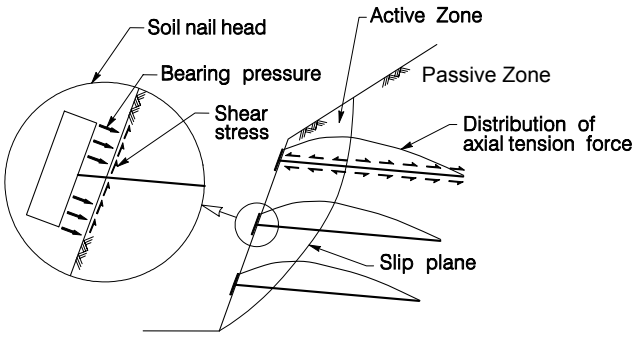


Fig. 1. Load transfer mechanism of soil nailed structure.

3.2 *Modes of failure*

The failure mechanisms of nailed structures can broadly be classified as external failure and internal failure.

The external failure modes refer to the development of potential failure surfaces passing beyond the soil nails. The soil-nailed ground mass essentially remains as an integral body. The failure can be in the form of sliding, rotation, bearing failure (for nailed excavation), or other form of loss of overall instability (see Fig. 2a).

Internal failure modes refer to failures within the soil-nail ground mass. In the active zone, these could be:

- failure of the bulk of the ground mass
  - ground disintegrates and 'flows' around nails
  - bearing failure behind nail head
- failure of ground between nail heads
  - washout or erosion
  - local sliding failure between nail heads
- structural failure of nail element
  - tensile failure of the nails
  - shear and bending failures of the nails
- structural failure of facing/head
  - bending/punching shear
  - nail-head/facing connection
- In the passive zone, the failure mode is mainly:
- pullout failure
  - pullout failure of nail along soil-grout interface
  - pullout failure nail along bar-grout interface

The various internal failure modes are illustrated in Fig. 2b.

3.3 *Nail-ground interaction in the active zone*

Soil nail acts as a structural element which provides load-transfer into the ground. Forces are developed in a nail through the interaction among the ground, the nail element and nail head (Fig. 1). The reinforcing action of the nail element is achieved through two fundamental mechanisms of nail-ground interaction. They are: (i) the nail-ground friction that leads to axial tension or compression in the nail (see Fig. 1); and (ii) the bearing pressure exerted by the ground on the nail element that leads to the development of shear stresses and bending moments in the nails (see Fig. 3). In these two mechanisms, the interactions between the ground and the nails are complex and the forces developed in the nails are influenced by many factors such as the size of the nail head, the bearing capacity of the ground to resist reaction force from the nail element, relative stiffness of the nail element and ground, and the tensile strength, inclination, shear strength and bending capacity of the nail element.

The development of stresses and strains in the active zone is resisted by the soil shear strength and the strength of the nail element under combined loadings of tension, bending and shear. When there is a small ground movement in the active zone, in particular at the shearing zone where the active zone moves downwards relative to the passive zone, the nail element will experience both axial and lateral strains. The axial strain will mobilize tensile forces, and the lateral strain will mobilize shear force and bending moment in the nail element. If the nail element is aligned with the direction of the tensile strain of the soil, the predominant action of the nail element is in tension and the shear force and bending moment induced in the nail are small. The tensile force improves the shearing strength of the soil by: (i) reducing the driving force on the soil; and (ii) increasing the normal stresses on the failure plane of the soil and consequently increasing the frictional resistance of the soil (Jewell & Wroth, 1987).

If the nail element is placed normal to the potential shear surface of the soil, bending moment and shear forces will be the dominant actions in the nail. In this case, the soil nail becomes a dowel element. The nail-ground lateral interaction will be as follows (Tan *et al.*, 2000):

3.3.1 Elastic soil - elastic nail

When there is small ground movement initially in the active zone, in particular at the shearing zone, both the soil and the nail element will be stressed in the range of elastic state. The elastic state will be maintained if equilibrium is reached.

3.3.2 Plastic soil - elastic nail / Elastic soil - plastic nail

If equilibrium cannot be reached in the stage of elastic-soil-elastic-nail, the ground movement in the active zone will continue until either the soil or the nail element, or both of them reaches plastic state. The state to be reached depends on the relative stiffness of the soil and the nail element. If the nail element is much stiffer and stronger than the soil, the soil will yield first when its bearing capacity is reached (Fig. 4(a)). If the soil is much stiffer and stronger than the nail element, the nail element will yield (formation of a plastic hinge) or rupture (brittle failure) when the yielding or rupture point with respect to the combined action of tension, bending, and shear of the nail is reached (Fig. 4(b)). A plastic hinge will be formed in the nail element only if it is ductile, otherwise it will rupture.

3.3.3 Plastic soil - plastic nail

If equilibrium for the lateral ground-nail interaction still cannot be reached, ground movement in the active zone will further

increase until both the soil and the nail element reach plastic state. In this case, the soil reaches its bearing capacity and yields; and the nail element either yields with a formation of plastic hinge or ruptures, depending on whether the nail element is ductile or not (Fig. 4(c)).

It has been demonstrated by means of laboratory tests (e.g. Pedley (1990), Jewell & Pedley (1992), Bridle & Davies (1997)), numerical analyses (e.g. Shiu & Chang, 2006; Smith & Su, 1997) and monitoring of in-service and test nailed structures (e.g. Plumelle *et al.*, 1990; Gässler 1997) that under the working conditions, the mobilized shear and bending resistances of soil nails are small. Further discussion is given in Section 6.3 below.

3.4 *Interaction between nail head and ground*

The ground movement in the active zone is resisted by nail elements as well as nail heads. The resistance that can be provided by a nail head depends on the stiffness of the head and the soil underneath, and the shear strength of the soil.

The head-ground interaction is affected by the direction of the resultant compressive and shear strains developed beneath the nail head in response to the ground movement in the active zone. If the resultant strain is close to a direction perpendicular to the base of nail head, the head-ground interaction will mainly be in the form of bearing mechanism. The mean effective stress in the soil behind the nail head will increase due to the confinement effect of the nail head. The shear strength of the soil will increase correspondingly. This is illustrated by the results of numerical analysis shown in Fig. 5. The earth pressure acting on the nail head will mobilize tension in the nail element. If the resultant strain is in a direction that significantly deviates from the normal of the base of the nail head, the head-ground interaction will be a combination of bearing and sliding mechanisms.

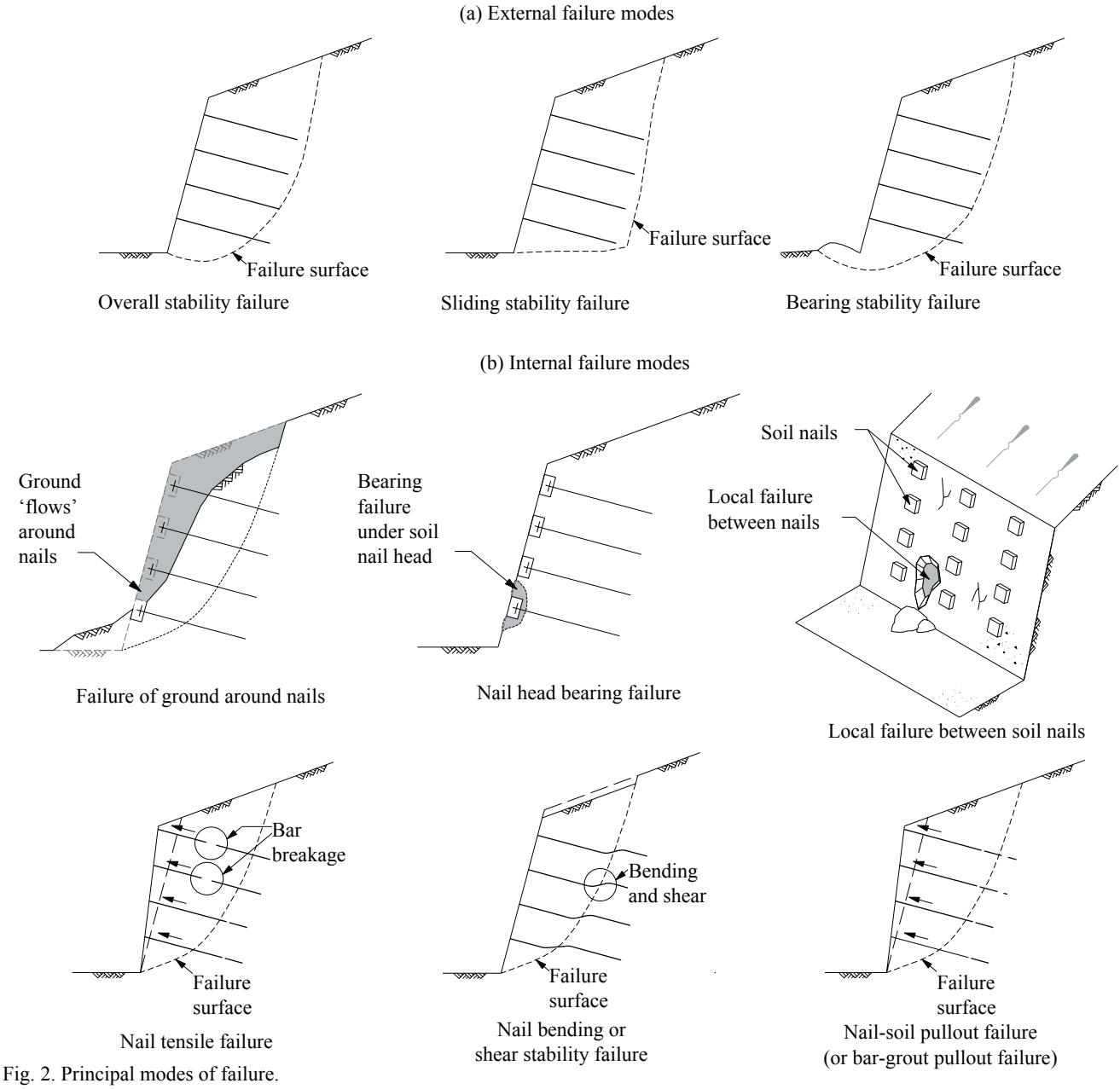
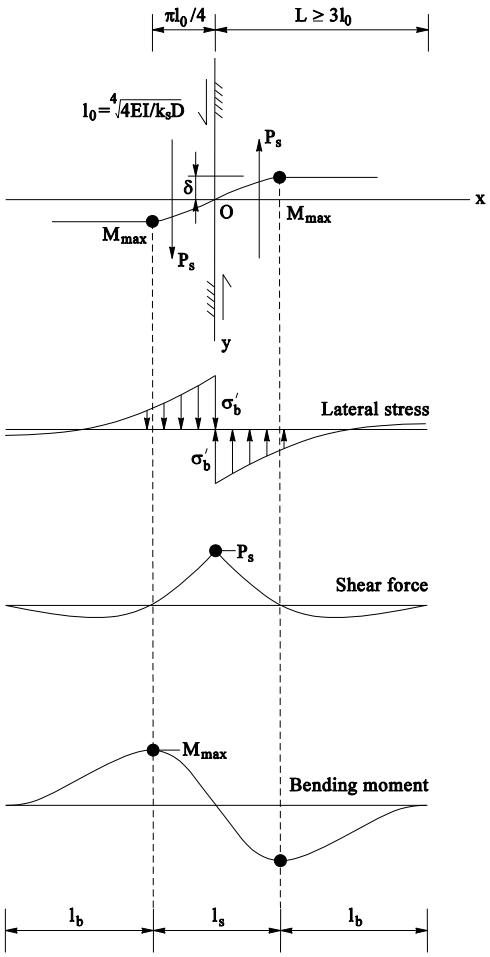


Fig. 2. Principal modes of failure.

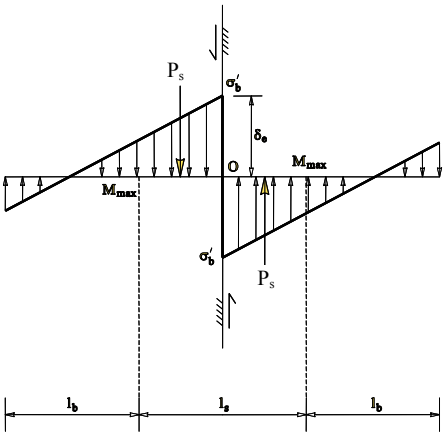


Legend:

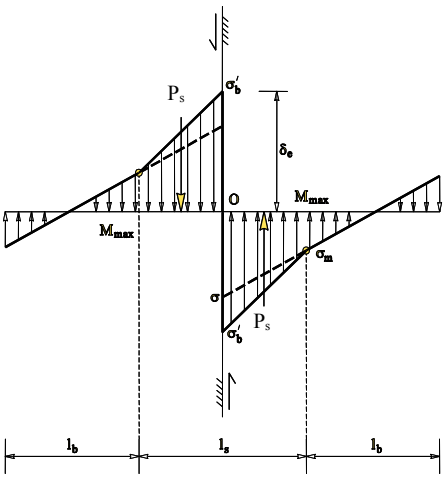
k_s	Coefficient of subgrade reaction	l_o	Transfer length of nail
δ	Lateral displacement of nail	E	Modulus of Elasticity of nail
I	Nail moment of inertia	D	Nail diameter
M_{max}	Maximum bending moment in nail	P_s	Shear force in nail
δ_b'	Maximum bending moment in nail		

Fig. 3. Nails subjected to bending moment and shear force (after Schlosser (1982)).

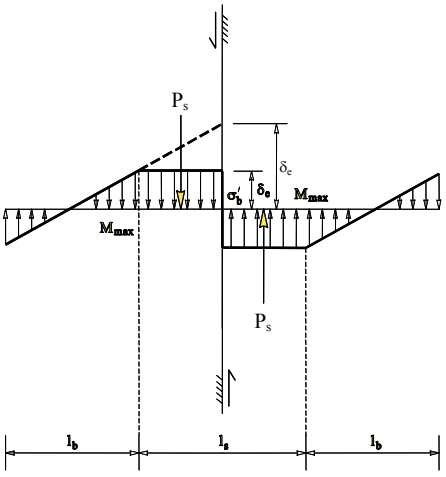
The mobilization of frictional force along nail element in the passive zone depends on many factors. Nevertheless, the basic principles are contact stress and interface friction. Immediately upon the formation of a drillhole in the ground for nail construction, the radial stress at the drillhole face drops to zero, irrespective of the overburden pressure. The hole remains stable by soil arching. Subsequent grouting will restore a certain level of the radial stress in the soil around the hole. The contact stress at the ground/grout interface depends on the pressure exerted by the grout. As it is common practice to grout up the drillhole under gravity, the contact pressure at the drillhole face is small compared to the overburden pressure. This seems to imply small bond strength at the ground/grout interface. In reality, the drillhole face, which is formed by percussive drilling, is fairly irregular and rough. Upon pulling of the soil nail, shearing will occur within the ground mass in a finite zone surrounding the nail element. If the soil is dilative, the effect of restrained dilatancy will come into place. The effect of this can be significant and can lead to high soil-nail friction.



(a) plastic soil - elastic nail

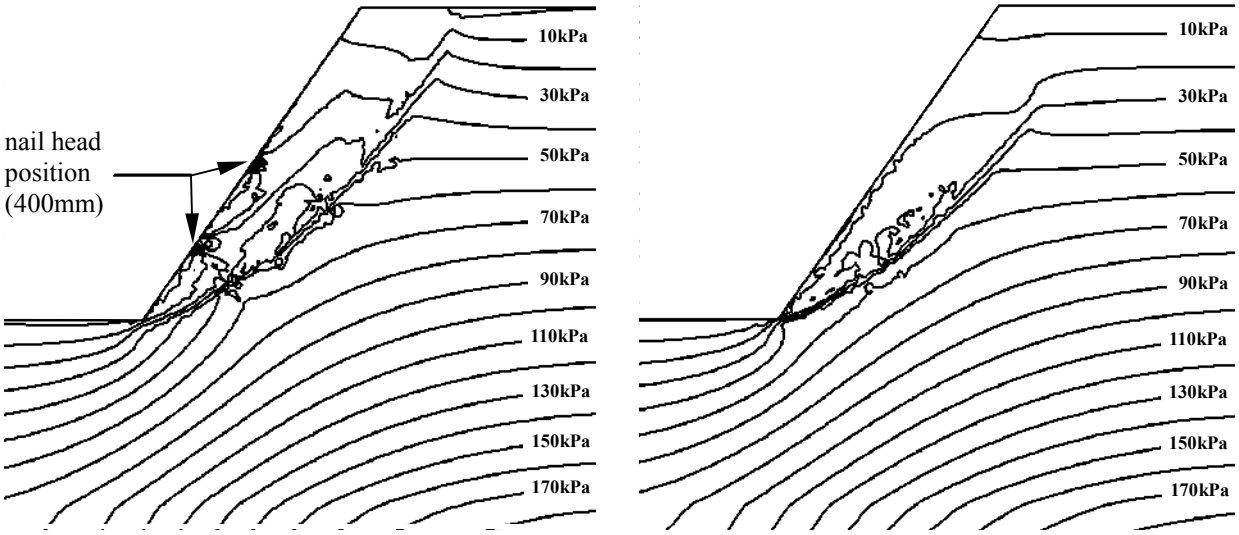


(b) elastic soil - plastic nail



(c) plastic soil - plastic nail

Fig. 4. Progressive development of stress in lateral nail-ground interaction (after Tan *et al.* (2000)).



(a) Contours of mean effective stress in soil nailed slope (b) Contours of mean effective stress in unreinforced slope

Fig. 5. Contours of mean effective stress in (a) soil nailed slope, and (b) unreinforced slope.

4 DESIGN APPROACH AND METHOD

4.1 *Analytical design approach*

The current soil nail design approach is essentially a combination of global safety factor approach (permissible stress design) and partial safety factor approach. The common design sequence is to determine the most critical potential failure surface for the unreinforced slope, determine the stabilization force required to provide the required global factor of safety to the slope, then provide this force by the action of soil nails. The diameter, length and spacing of the nails are determined to provide adequate partial safety factors against different internal failure modes.

The global safety factor to be used for a nailed slope is the same as that for the design of unreinforced slope as recommended in the Geotechnical Manual for Slopes (GCO 1984). Although not explicitly stated, the global safety factor deems to cover uncertainties related to ground model, shear strength of soils, design groundwater level etc, which exist in the design of unreinforced slopes.

For the design of stabilization works to existing retaining walls using soil nails, the minimum required global factors of safety against sliding, overturning and bearing of the retaining walls as recommended in the first edition of Geoguide 1 (GCO 1982) are to be followed. This usually results in a few rows of widely spaced and long soil nails even for tall retaining walls. Strictly speaking, the walls designed in this way behaves more like a tieback wall and it does not fully satisfy the soil nailing concept of reinforcing the insitu ground (using closely spaced soil nails) to form a stable block of composite material.

Three partial safety factors are applied to cover uncertainties related to the design of soil nail elements. They are summarized in Table 1.

The requirements for the soil nail design are given in GEO Technical Guidance Note No. 23 (GEO 2006).

The sizes of nail heads are to be determined by one of the following three methods:

- design table derived from numerical analysis (see Section 6.1 below);
- the lower-bound nail head design method adopted from the one given by UK Department of Transport (1994);
- prescriptive design approach (see Section 4.2 below).

Table 1. Partial factors of safety

Modes of failure	Partial factors of safety
Bond failure at grout-soil interface	1.5 on weathered granite and volcanic rocks; and 2.0 on others
Bond failure at grout-bar interface	2.0
Tensile failure of steel bar	1.5 on yield strength of bar

Note: The partial factors of safety are applied to soil nail with reinforcement in the form of high yield steel bar.

The structural design of nail heads follows recommendations stipulated in relevant structural design codes. Details for the design of soil nail heads are given in GEO (2004b).

The concept of limit state also applies to the current design approach. A limit state is typically defined as: "any limiting condition beyond which the structure ceases to fulfill its intended function" (Day 1997). Soil nail design for slopes is mainly carried out for the ultimate limit state, i.e. design against possible failure modes. As soil nails are passive reinforcing elements, certain ground movements in the active zone are inevitable in order to mobilize axial tensile force, shear stress and bending moment in the nails. Designs for serviceability limit state are performed in cases where there is concern on ground movement (e.g. nail excavation in close proximity of structures/utilities).

The design against possible failure modes can be carried out using the analytical method. Limit equilibrium methods (LEM) of slices are routinely used. Shiu *et al.* (2007) have reviewed the use of limit equilibrium methods for soil nail design. They cautioned that the behaviour of soil nailed structure is a strain compatibility problem and the effect of nail inclination cannot be accounted for in LEM. Furthermore, it is possible to define a wide variety of nail length patterns that satisfy stability requirements but that may not satisfy serviceability requirements (Shiu & Chang, 2005). Users of LEM computer programs should

recognize the potentially erroneous results and interpret the results carefully. It is important that only methods that consider both moment and force equilibrium, such as the Morgenstern and Price method, are used in soil nail design. Although LEM involves assumptions and has certain weaknesses, the method does provide a useful and practical technique for the analysis of slopes (both unreinforced and reinforced).

4.2 *Prescriptive design*

Apart from the analytical approach, soil nails can be designed prescriptively for stabilization of existing cut slopes and retaining walls. Prescriptive measures are pre-determined, experienced based and suitably conservative modules of works prescribed to a slope feature to improve its stability without detailed ground investigation and design analyses. Use of soil nails as prescriptive measures for upgrading soil cut slopes have been successfully applied since the publication of GEO Report No. 56 (Wong *et al.*, 1999). The scope of application of prescriptive measures has now been extended to include improving stability of substandard concrete or masonry retaining walls (Lui & Shiu, 2004). Prescriptive design of soil nail heads is given in GEO (2004b).

4.3 *Use of soil nails in loose fill*

Guidelines on the design of soil nails for the stabilization of loose fill slopes are given in the publication "Soil Nails in Loose Fills Slopes - A Preliminary Study" by the HKIE Geotechnical Division (HKIE 2003). The main design recommendations are as follow:

- Steady state shear strength should be adopted for loose fill in the design.
- Global stability should be provided for by bonding soil nails into a competent stratum.
- Local (near surface) stability should be provided for by a concrete grid structure covering not less than 50% of the slope surface and connecting soil nail heads. Soil nail spacing should not be more than 1.5m horizontally and vertically.
- The grid structure should be designed to withstand bending moments and shear forces generated by the loose fill it is retaining. It should be adequately founded on a competent stratum.
- The potential of leakage from water-carrying services should be duly considered.

5 AESTHETIC CONSIDERATIONS

In addition to safety, designers should pay due regard to slope appearance in their designs. In the past, vegetation covers were provided only to gentle slopes and hard covers such as chunam and shotocrete were used on steep slopes. As cut slopes stabilized by soil nails are usually steep, most soil-nailed slopes constructed in the early 1990s had a hard cover. Improvement in detailing of surface covers, involving the use of an erosion control mat in conjunction with a steel wire mesh, has allowed the provision of vegetation covers to slopes with a gradient up to 55° and sometimes 60° (Fig. 6). Many soil-nailed slopes in Hong Kong have been successfully vegetated using this technique (Fig. 7).

Where the provision of vegetated surface cover on a slope is practically not feasible, hard landscape treatment is generally provided to improve its appearance. Engineers are now more knowledgeable in landscape design concept than before. The commonly used methods are masonry block facing, ribbed or other patterned concrete finishes, toe planters, colouring and planter holes, coupled with suitable retention of existing vegetation (Chan 2005).

Comprehensive technical guidelines on landscape treatment and bio-engineering for man-made slopes and retaining walls are given in GEO Publication No 1/2000 (GEO 2000).

6 RECENT TECHNICAL DEVELOPMENT

6.1 Effect of soil nail head

Soil nail heads used in slope stabilization works in Hong Kong are usually in the form of isolated reinforced concrete pads. To investigate the effect of soil nail heads on stability of nailed slopes, numerical simulations have been carried out using the two dimensional finite element code, Fast Lagrangian Analysis Continua (FLAC). Fig. 8 shows the slope model used. Strength reduction technique (Dawson *et al.*, 1999) is employed to compute the factors of safety (FoS). In the simulations, nail heads of different sizes are modeled in plain strain. The slope without any soil nails (i.e. unreinforced) has a minimum FoS close to 1 (Shiu & Chang, 2004). Based on the FLAC analysis, Fig. 9 shows the relationship between the calculated FoS of the model slope and nail head sizes. The FoS increases from 1 for the unreinforced slope to 1.2 for the nailed slope with no nail heads. Substantial increases in the FoS are obtained with nail head sizes from 400 mm wide to about 800 mm wide. The trend of increase levels off for nail head sizes larger than 800 mm wide. It shows that nail head can have significant effect on the stability of a nailed slope.

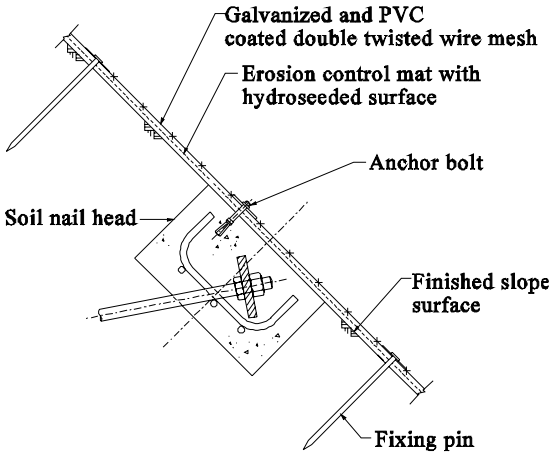


Fig. 6. Fixing details of steel wire mesh and erosion control mat on slope face with soil nail heads.

Fig. 10 compares the axial tensile forces developed in soil nails without nail heads with those in nails with heads of 800mm wide. It shows that for the nails with no nail heads, no tensile force is developed at the front end of the nail (Fig. 10(a)); but for the soil nails with nail heads, large tensile forces are mobilised in the nails at the connections to the nail heads (Fig. 10(b)). The large mobilised tensile forces in the latter case indicate significant interaction between nail heads and the ground, which greatly increase the stability of the slope.

A series of centrifuge tests has been conducted in the Geotechnical Centrifuge Facility of the Hong Kong University of

Science and Technology to investigate the reinforcing effect of soil nails and nail heads (Zhou *et al.*, 2006). Fig. 11 shows an instrumented model used in one of the nailed slope centrifuge tests. The test results support the results of the numerical simulations that nail head can substantially improve the stability of nailed slopes.

Fig. 7. Details of the use of steel wire meshes in conjunction with non-degradable erosion control mats.

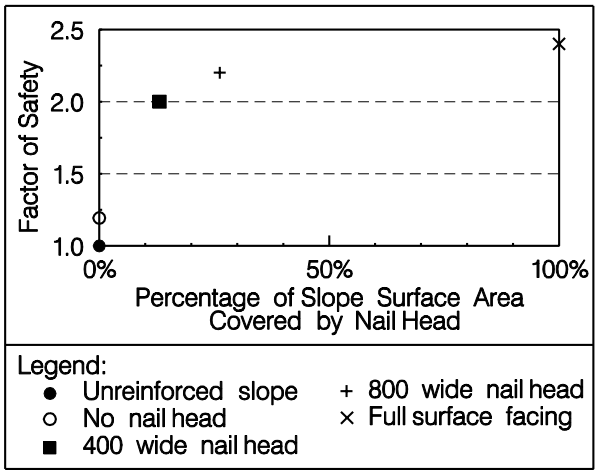


Fig. 9. Relationship between factor of safety and nail head size.

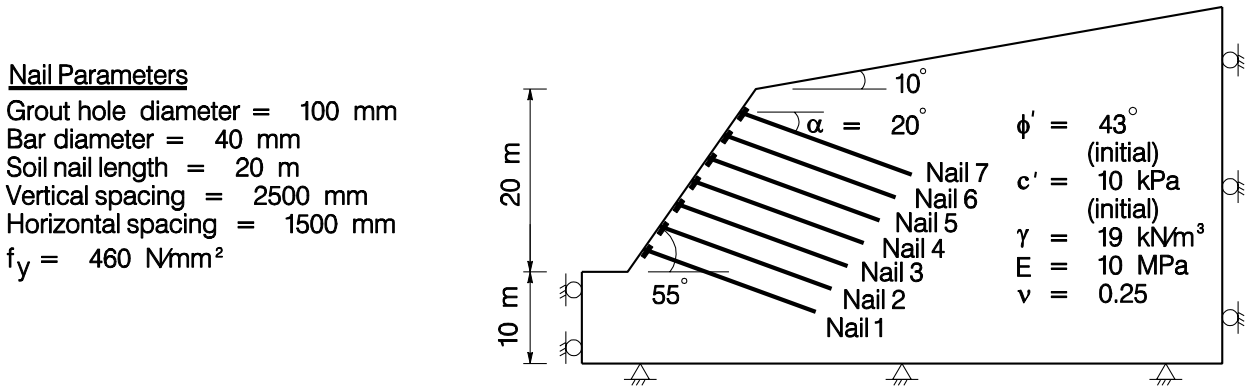


Fig. 8. Geometry and material parameters of model slope.

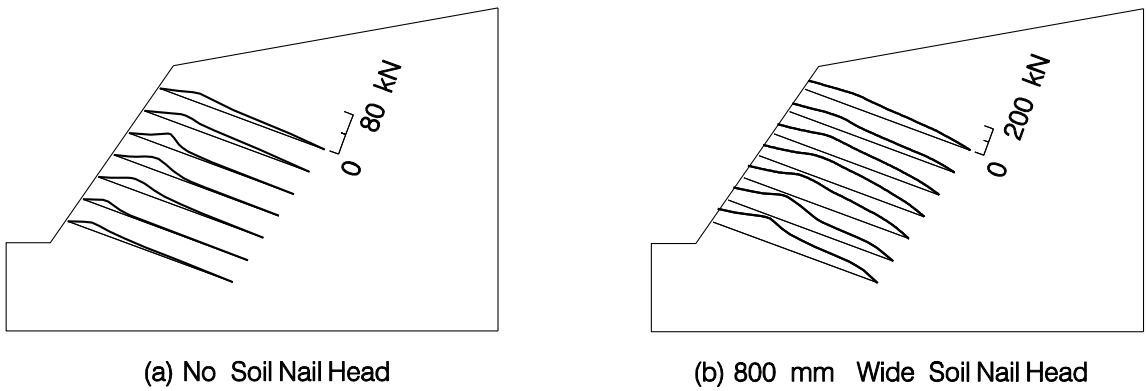


Fig. 10. Variation of axial nail forces for (a) 800 mm soil nail head and (b) no nail head.

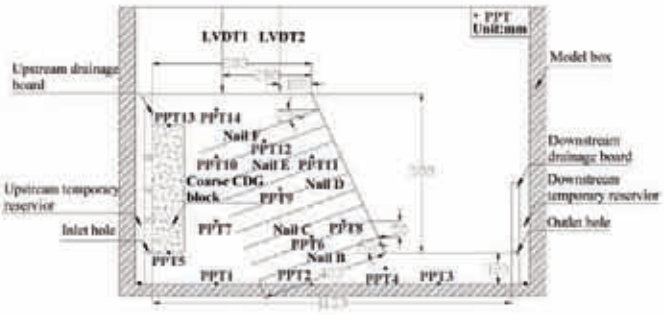


Fig. 11a. Set-up of a nailed-slope model in centrifuge test.

Fig. 11b. Side view of the slope model in centrifuge test.

Fig. 12. Typical shear strain plot.

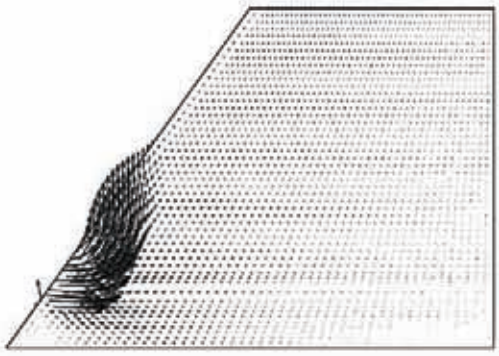


Fig. 13. Typical displacement vector plot.

Numerical analysis has also been performed to examine the bearing capacity failure of square soil nail heads. A small slope model of 5 m in height was used and various slope angles were considered. In the analysis, the nail head was pushed into the ground by a nail force to simulate the situation of soil moving out from a slope and pressing against the soil nail head. The nail forces used are determined from the allowable tensile strength of steel bars. Figs. 12 and 13 show respectively the shear strains and the displacement vectors at the point of bearing failure for a 600 mm × 600 mm nail head on a 45° slope. Typical results of the analyses in terms of c'-ϕ' envelope for limit equilibrium (i.e. when bearing failure occurs) are plotted in Fig. 14. In this plot, the nail head loads are expressed as diameters of steel bars. Details of the study are given in Shiu & Chang (2004). A number of the plots have been developed for different slope angles and nail head sizes. Knowing the shear strength parameters of the soil, the steel bar diameter and the slope angle, a designer can determine the size of nail head from these plots. A design table has been derived from the plots and it forms the basis of the design method derived from numerical analysis as mentioned in Section 4.1 above.

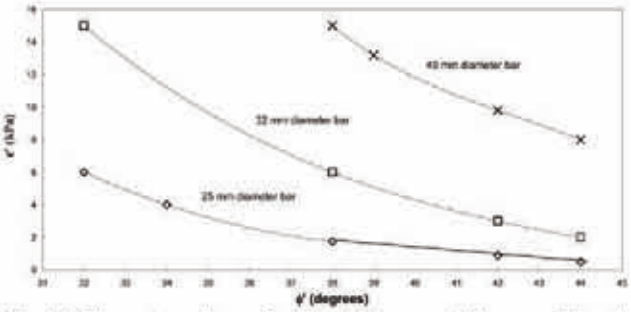
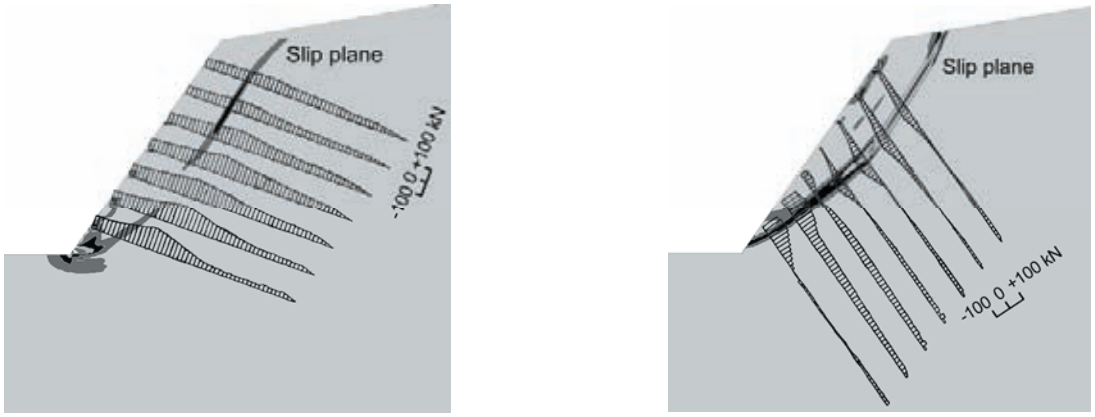


Fig. 14. Shear strength required for 600 mm × 600 mm nail heads on a 45° slope to mobilise allowable tensile strength of nails of specified diameters.

6.2 *Effect of nail inclination*

Unlike the reinforcement in reinforced fill structures, which are placed in horizontal direction, soil nails can be installed in the ground at various inclinations. In congested sites, soil nails are sometimes steeply inclined. Different nail inclinations can produce different effects on the behaviour of nailed structures. Studies of these effects have been conducted by means of numerical simulations (Shiu & Chang, 2005) in which the two-dimensional FLAC with strength reduction technique was employed.

Tensile or compressive axial forces can be developed in soil nails and this depends on the nail inclination, α, which is the angle of the soil nail below the horizontal. Fig. 15(a) shows that for soil nails with a small inclination of 20°, tensile forces are developed in all the nails. On the other hand, Fig. 15(b) indicates that when the nails are inclined steeply at an inclination of 55°, compressive forces are developed in the top four nails whereas tensile forces are mobilized only in the bottom three nails. Tensile nail forces can improve stability whereas compressive forces can have opposite effect. Increases in FoS (ΔFoS) due to the soil nails were calculated for different nail inclinations. Fig. 16 shows the relationship between the calculated ΔFoS and nail inclinations (α) for the model slope. The ΔFoS is close to 1 with little variations for the range of α between 0° and 20°. The ΔFoS decreases quickly as α increases beyond 20°, reflecting that the reinforcing effects of the nails reduce rapidly with increasing nail inclinations. At α = 55°, the value of ΔFoS is almost zero.



(a) $\alpha = 20°$ (b) $\alpha = 55°$

Fig. 15. Axial force distribution in nails for (a) $\alpha = 20°$ and (b) $\alpha = 55°$.

Behaviour of nailed structures is a strain compatibility problem. A nail force develops through the interaction among the deforming soil, the soil nail and nail head. An important point here is that depending on the nail inclination, compressive forces rather than tension forces can be mobilized in soil nails. This contradicts the common design assumption used in limit equilibrium methods that only tensile forces are developed in soil nails. The limit equilibrium methods do not consider strains and displacements, and as a result, they may give rise to invalid results in calculating nail forces and factors of safety of nailed slopes with steeply inclined nails. The development of compressive force in soil nails should be considered in such cases. The use of limit equilibrium methods for soil nail design is discussed in the paper by Shiu *et al.* (2007).

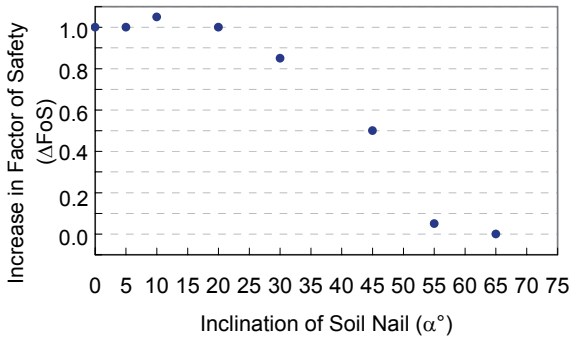


Fig. 16. Variation of increase in factor of safety against inclination of soil nails.

6.3 *Effect of bending stiffness of soil nail*

Steel soil nails can sustain shear forces which may also enhance the shear strength of soil. The development of shear forces in nails involves a mechanism which is dependent on the bending strength of the nail, the soil bearing strength, and the orientation and shear deformation of reinforcement. To study the effect of bending stiffness of the nails on nail forces and displacements, numerical simulations have been conducted using the slope model shown in Fig. 8 (Shiu & Chang, 2005). Nails with different inclinations have been investigated. For each nail inclination, the FoS of the nailed slope, the tensile forces, shear stresses and bending moments developed in the nails are computed.

The maximum axial force developed in a nail is T_{max}. Fig. 17 shows the total of the maximum tensile forces mobilised in all the soil nails (ΣT_{max}) at limit equilibrium condition of the slope model. The maximum shear force in a nail at the location where the shear plane intersects the nail is Ps_{max}. The total of the maximum shear forces (ΣPs_{max}) mobilized in the soil nails at limit equilibrium condition of the model are also plotted in Fig. 17. The value of ΣPs_{max} rises steadily with increasing nail inclination (α). The rise is small, from 31 kN/m at $\alpha = 10^o$, to 76 kN/m at $\alpha = 55^o$. In contrast, the value of ΣT_{max} decreases rapidly with increasing nail inclination. For small nail inclinations, ΣT_{max} is much larger than ΣPs_{max}. Comparing between Figs. 16 and 17, it can be noted that both ΔFoS and ΣT_{max} generally decrease with increasing nail inclinations. This similarity illustrates that ΔFoS is strongly influenced by the nail axial force. The ΔFoS is not sensitive to the mobilized shear resistances in the nails. The modeling results show that small shear forces are mobilized in soil nails and they have little effect on the factor of safety of the slope, except at very steep nail inclination where dowel action starts to play a role. The contribution from bending and shear to the calculated factor of safety of a slope is therefore generally ignored.

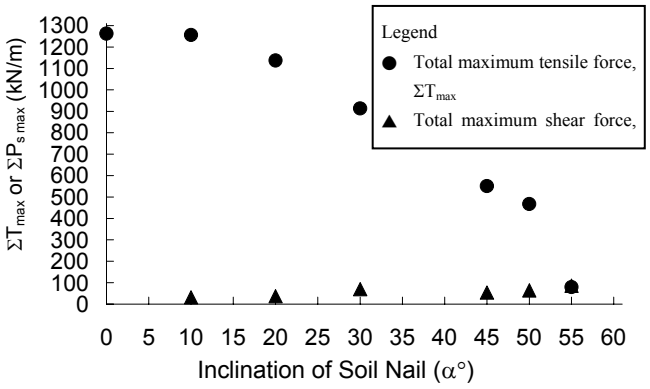


Fig. 17. Variation of total maximum tensile force (T_{max}) and total maximum shear force ($P_{s\,max}$) with nail inclination (α).

Soil nails are not effective in providing dowel action. For that purpose, other types of structural element should be considered, e.g. large diameter piles. Nevertheless, steel soil nails have large shear ductility. As a result of the mobilization of shear and

bending ductility at large deformations, a nailed structure tends to exhibit ductile failure rather than sudden failure.

6.4 *Pullout resistance of soil nails*

Pull-out capacity is a key parameter for the design of soil nails. At present, methods for estimating pullout capacity are not unified as reflected by the many approaches used in different technical standards and codes of practice, such as effective stress method (GEO 2006; CIRIA 2005), empirical correlation with SPT N values (JH, 1998), correlation with pressuremeter tests (Clouterre 1991), and correlation with soil types (FHWA 2003). The merits and limitations of the various methods are summarized in Table 2. The effective stress method is adopted in Hong Kong.

Table 2. Merits and limitations of the methods for determining ultimate pull-out resistance

Method	Merits	Limitations
Empirical Correlation	Related to field performance data; can better account for influencing factors.	Need a large number of field data and take a long time to establish a reasonable correlation; a general correlation may not be applicable to all sites.
Pull-out Test	Related to site-specific performance data.	Need to carry out a considerable number of field pull-out tests during the design stage; not feasible for small-scale project; time consuming.
Undrained Shear Strength	Based on soil mechanics principles; easy to apply.	Generally not suitable for Hong Kong; many factors that affect the pull-out resistance are not accounted for.
Effective Stress	Based on soil mechanics principles; easy to apply.	Many factors that affect the pull-out resistance are not accounted for.
Pressuremeter	Related to field performance data; can better account for influencing factors.	Need a large number of field data to establish a reasonable correlation; a general correlation may not be applicable to all sites; pressuremeter test is not common in Hong Kong.

Field pull-out test data have been collected from LPM contracts since 2004. Improved test set-up and procedures, which include minimising friction loss along a test nail, imposing better control on the length of the grouted portion and increasing the test load by using large bar size, were adopted in these tests. A total of 914 test results were collected. About 84% of the tests were conducted in granite or volcanic saprolite. The rest were conducted in other types of material such as fill, colluvium and moderately decomposed rock.

Most (423 nos.) of the pull-out tests were only tested to 90% of the yield strength of steel (T_p), i.e. not reaching the ultimate pull-out resistance (T_{ult}). Fig. 18 shows the plot of the ratio of the field to the estimated pull-out resistance against the overburden pressure of those tests which have reached the ultimate pull-out resistance. Fig. 19 presents the same plot with all the test data (542 nos., i.e. those reaching either T_{ult} or T_p). The field pull-out resistances are generally several times higher than those estimated using the effective stress method, but the safety margin (i.e. $T_{ult\ (field)}$/ $T_{ult\ (estimate)}$) gradually decreases when overburden pressure increases. Some of the field pull-out tests (26 nos.) were carried out under saturated condition, and the results do not show particularly low pull-out resistance when compared with other pull-out tests carried out under dry condition of the same overburden pressure and similar soil shear strength.

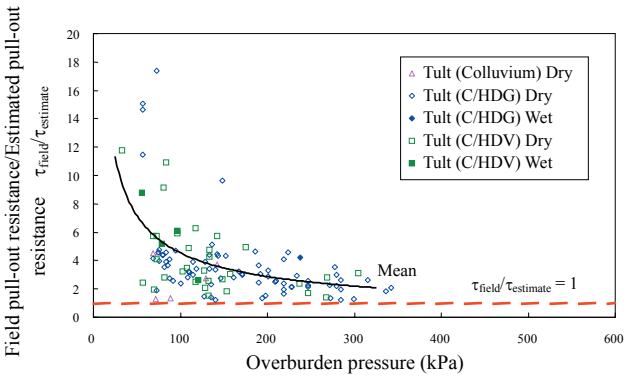


Fig. 18. Plot of field (T_{ult}) to estimated pull-out resistance against overburden pressure

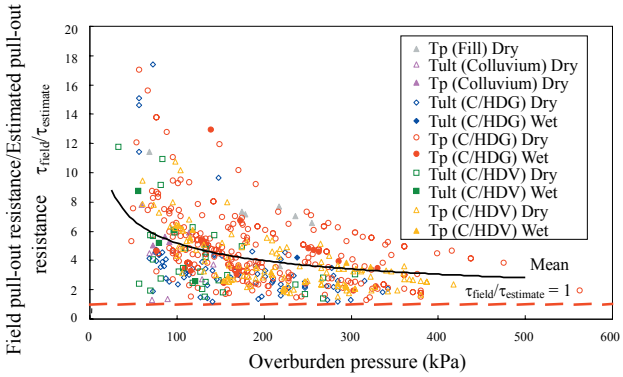


Fig. 19. Plot of field (T_p + T_{ult}) to estimated pull-out resistance against overburden pressure

Many factors that affect the pullout resistance of a soil nail are difficult to be quantified in a design. The current effective stress method does not account for factors including soil arching, restrained soil dilatancy, soil suction, roughness of drillhole surface, over-break, etc. All these factors except soil arching tend to result in higher pullout resistance than the design value. Field pullout test data support this hypothesis in that the actual pullout resistance is generally higher than that estimated using the effective stress method.

Laboratory pullout tests have been carried out in recent years by a number of researchers to investigate the development of soil/nail interface shear, such as Lee *et al.* (2001), Pradhan *et al.* (2003), Junaideen *et al.* (2004), Chu & Yin (2004). They provided useful information on the behaviour of soil nails in pullout in different types of compacted and loose fill. Most recently, Yin & Su (2006) studied the effects of hole drilling process, overburden pressure, degree of saturation of the soil, and grouting pressure on pullout resistance. Compacted fill of

completely decomposed granite was used in the tests. The following observations were made from the study: (a) the drilling process during soil nail installation led to stress reduction in the soil around the drillhole and the pullout resistances of the nails were not dependent on the amount of vertical surcharge applied if gravity grouting was adopted; (b) the peak pullout strength of soil nail in fully saturated soil was lower than that in partly saturated soil; and (c) pullout resistances of the nails increased with an increase of grouting pressure.

6.5 *Potential effect of blockage of subsurface drainage by soil nailing works*

Soil nails installed in the ground may impede groundwater flow and as a result dam up the water level. To study the significance of this effect, a number of numerical models were set up in both 2-D and 3-D for various geological settings, subjected to infiltration (Halcrow 2005). Typical nail spacings of 1m to 2 m were adopted in the models. Fig. 20 illustrates an example of computed flow nets and water table distributions for a slope under three conditions: (a) without soil nails; (b) soil nails with excessive grout loss, and (c) soil nails with no grout loss.

Results of the numerical modeling show that under typical conditions where there is little grout loss during the grouting operation, there should be no significant blockage of the drainage paths. It is also found that the influence of soil nails on groundwater flow can be significant if excessive grout escapes laterally to affect large volumes of the country rock. Therefore, measures should be taken to avoid excessive grout loss. Where excessive grout loss occurs during installation of soil nails, the cause should be investigated and, if necessary, measures taken to monitor rises in hydraulic head and to take action to drain the ground upstream of the nails.

6.6 *Long-term durability of soil nails*

Durability is an important aspect of soil nailing system. The long-term performance of soil nails depends on their ability to withstand corrosion attack from the surrounding ground. To enhance understanding of the subject, a review of the current state of practice of corrosion protection in different parts of the world has been carried out (Shiu & Cheung, 2003). The review also included a survey of the chemical properties of common Hong Kong soils and an assessment of their corrosion potential. In addition, soil nails of different ages were exhumed from the ground and they revealed that localized corrosion could occur even if hot dip galvanization was provided, particularly in areas where voids existed in the cement grout (Fig. 21). The review has resulted in the development of an improved corrosion protection system.

It is now a design requirement that different levels of corrosion protection are to be employed in accordance with the design life and aggressivity of the ground (GEO 2005). A scoring system is used to categorize the ground into four different levels of aggressiveness: “non-aggressive”, “mildly aggressive”, “aggressive” and “highly aggressive”. The scoring system is based on the physical properties and chemical characteristics of the soils, see Tables 3 and 4. For soil nails to be installed in soils classified as “non-aggressive” or “mildly aggressive”, the protection includes the provision of hot-dip galvanizing and a 2 mm sacrificial thickness on the radius of the steel bar. For “aggressive” or “highly aggressive” soil, corrugated plastic sheath in addition to hot dip galvanization should be provided (GEO 2006).

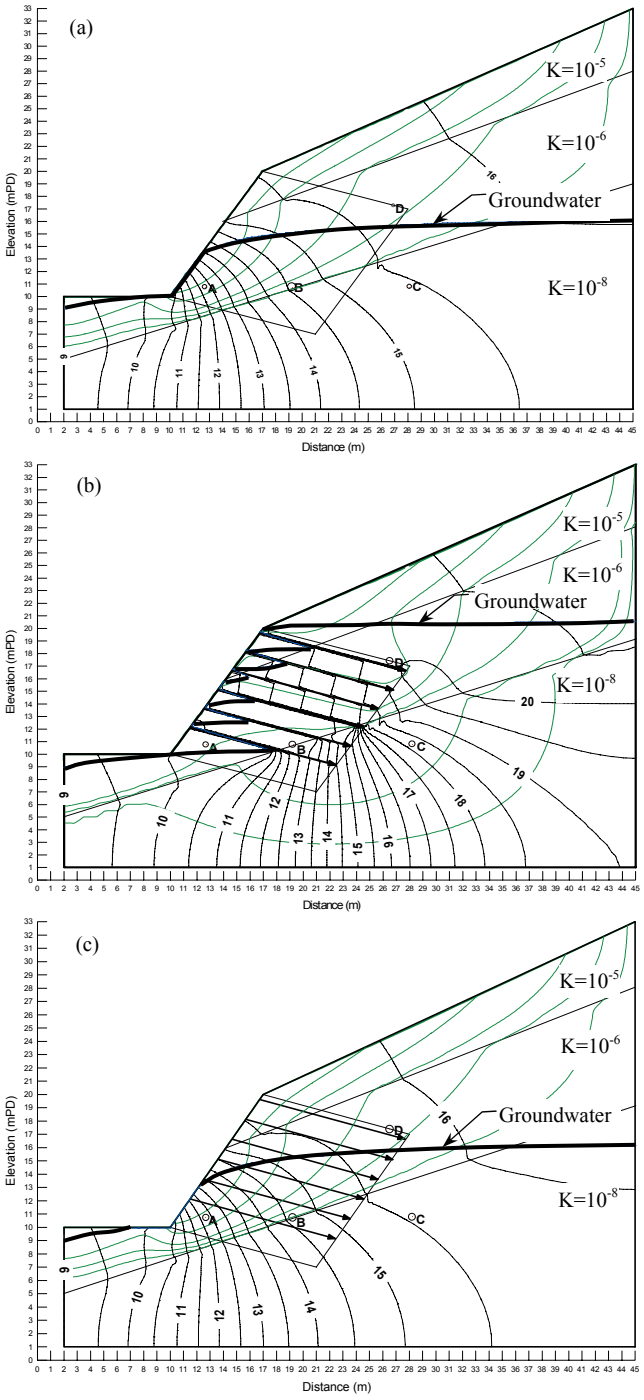


Fig. 20. Flow Patterns in a slope (a) without soil nails, (b) soil nails with excessive grout loss, (c) soil nails with no grout loss (after Halcrow (2005)).

Table 3. Soil corrosivity assessment scheme.

Property	**Measured Value**	**Mark**
Soil Composition	Fraction passing 63 μm sieve ≤ 10 %, and PI of fraction passing 425 μm sieve < 2, and organic content < 1.0 %	2
	10 % < Fraction passing 63 μm sieve ≤ 75 %, and fraction passing 2μm sieve ≤ 10 %, and PI of fraction passing 425 μm sieve < 6, and organic content < 1.0 %	0
	Any grading, and PI of fraction passing 425 μm sieve < 15, and organic content < 1.0 %	-2
	Any grading, and PI of fraction passing 425 μm sieve ≥ 15 and organic content < 1.0 %	-4
	Any grading, and organic content ≥ 1.0 %	-4
Resistivity (ohm-cm)	**≥ 10,000**	**0**
	< 10,000 but ≥ 3,000	**-1**
	< 3,000 but ≥ 1,000	**-2**
	< 1,000 but ≥ 100	**-3**
	< 100	**-4**
Moisture Content	**≤ 20%**	**0**
	> 20%	**-1**
Ground-water level	Above groundwater level and no periodic flow or seepage	1
	Local zones with periodic flow or seepage	-1
	At groundwater level or in zones with constant flow or seepage	-4
pH	6 ≤ pH ≤ 9	0
	5 ≤ pH < 6	-1
	4 ≤ pH < 5 or 10 ≥ pH > 9	-2
	pH < 4 or pH >10	See Note 1
Soluble Sulphate (ppm)	**≤ 200**	**0**
	> 200 but ≤ 500	**-1**
	> 500 but ≤ 1,000	**-2**
	> 1,000	**-3**
	(Water soluble sulphate as SO_3)	
Made Ground	**None**	**0**
	Exist	-4
Chloride Ion (ppm)	**≤ 100**	**0**
	> 100 but ≤ 300	**-1**
	> 300 but ≤ 500	**-2**
	> 500	**-4**

Note 1: If pH value is less than 4 or greater than 10, the ground should be classified as aggressive regardless of the results of other test items.

Note 2: "Made ground" refers to man-made ground associated with high corrosion rate such as non-engineering fill with rubbish, organic matters, etc.

Table 4. Classification of corrosivity of soil.

Classification of soil corrosivity	Total mark from the soil corrosivity assessment scheme
Non-aggressive	**≥ 0**
Mildly aggressive	-1 to -4
Aggressive	-5 to -10
Highly aggressive	≤ -11

Fig. 21. Localized corrosion in exhumed soil nail.

To overcome the problem of corrosion of metallic reinforcement, the feasibility of using non-metallic soil nails has also been examined. Field installation trials have demonstrated that carbon fibre reinforced polymer reinforcement (CFRP) can be an alternative to steel bar in soil nailing works (Fig. 22). The CFRP is highly corrosion resistant. The fibres in CFRP composites are carbon in nature, and the matrix is a resin. The CFRP reinforcement is lightweight and as such it greatly eases the installation works, especially at cramped slopes behind buildings.

The CFRP reinforcement has high tensile strength. Fig. 23 compares the typical stress-strain behaviour of a CFRP reinforcement with that of a high yield steel reinforcement. The brittle behaviour and low bending capacity of CFRP are concerns that need to be carefully considered. As such, CFRP is not yet ready for wide and general application. Despite this, a set of interim design and construction guidelines has been developed in order to facilitate trial use and gain more insight and experience (Cheung & Lo, 2005). Further laboratory tests and investigations are being carried out on this innovative use of the material in soil nailing.

Fig. 22. Typical circular section and rectangular section of CFRP.

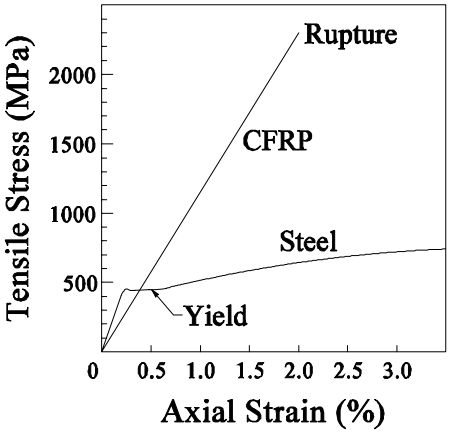


Fig. 23. Stress/strain curves of typical high yield steel bar and CFRP bar.

7 PERFORMANCE OF SOIL-NAILED SLOPES

7.1 *Post-construction monitoring*

Field monitoring of nailed slopes and excavations provides information on the behaviour of the feature for verifying design assumptions and parameters and for enhancing understanding on the load transfer mechanism of soil nails. The majority of soil nailing applications in Hong Kong are for stabilizing marginally stable slopes, and field monitoring is usually not undertaken. There are cases involving use of soil nails for supporting deep excavations where instrumentation was installed for monitoring the performance of the nailed excavations during and after construction (e.g. Shiu *et al.*, 1997; Yim & Yuen, 1998).

Shiu *et al.* (1997) reported the results of field monitoring in a 13.5 m high cutting to angles up to 80°. The instrumentation systems included an inclinometer casing for monitoring lateral deformations of the nailed excavation during and after construction. Nine soil nails in the most critical section of the slope were instrumented with vibrating wire strain gauges to measure the distribution and changes of nail forces during different stages of excavation. Fig. 24 shows the nailed excavation and locations of strain gauges.

Fig. 25 illustrates the lateral displacements recorded during different stages of excavation. The lateral displacement increased as the excavation depth increased. The maximum displacement at the top of the nailed slope at the end of construction was 13mm which is equal to about 0.1% of the height of excavation.

Fig. 26 shows the strain measurements (expressed as forces) from the nail on row 7 over the monitoring period. The responses of the strain gauges to the subsequent lifts of excavation are indicated clearly as rapid increases in forces. Following the completion of construction, there was a slow increase in force for a short period of time and no further noticeable changes thereafter.

The distribution of nail forces along the nails on rows 3 and 7 and their responses to excavation lifts are shown in Fig. 27. The effect of advancing excavation was significant on row 7 but much less noticeable on row 3. The stress distribution in row 3 was rather uniform and did not increase appreciably with depth of excavation. Similar observations can be made on other nails at the upper part of the slope, indicating that the upper nails (rows 1 to 4) did not have substantial contribution to the retaining force of the nailed slope. Unlike the upper nails, row 7 carried a distinct peak force. The forces mobilized in this nail were small initially but they increased substantially with subsequent lifts of excavation. Other nails in the lower part of the slope also showed the same pattern of changes in forces. This illustrates that they contributed significantly to the stability of the nailed slope. Fig. 28 presents the distribution of axial forces along each soil nail at the end of construction.

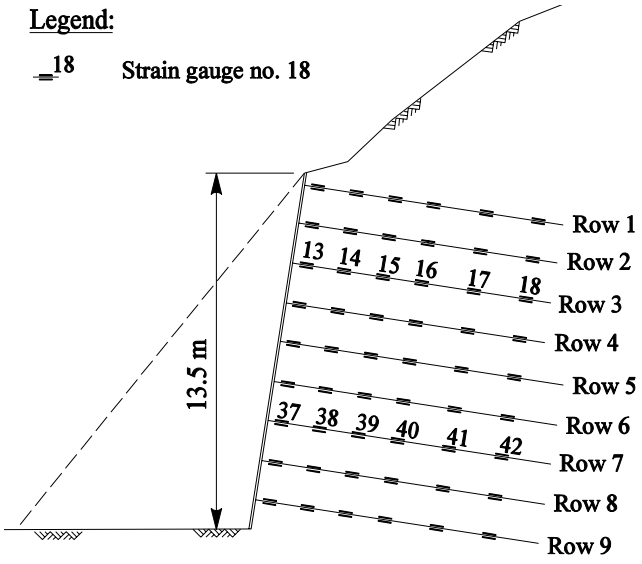


Fig. 24. Critical section of the soil nailed slope.

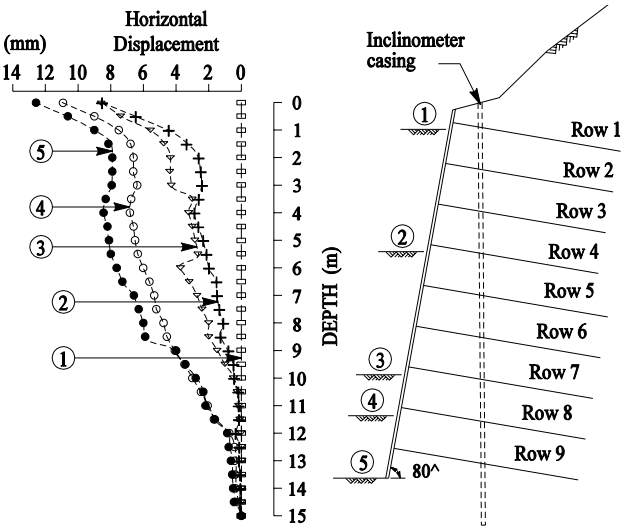


Fig. 25. Lateral displacements of the nailed excavation.

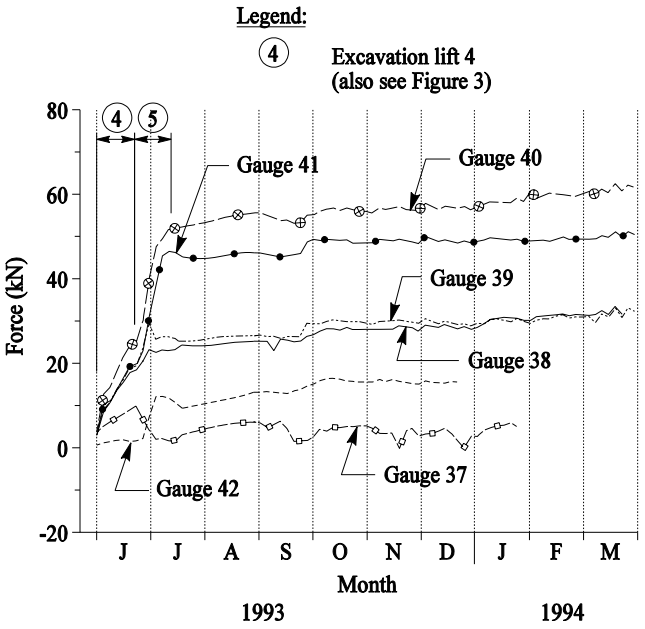


Fig. 26. Nail forces against time for row 7.

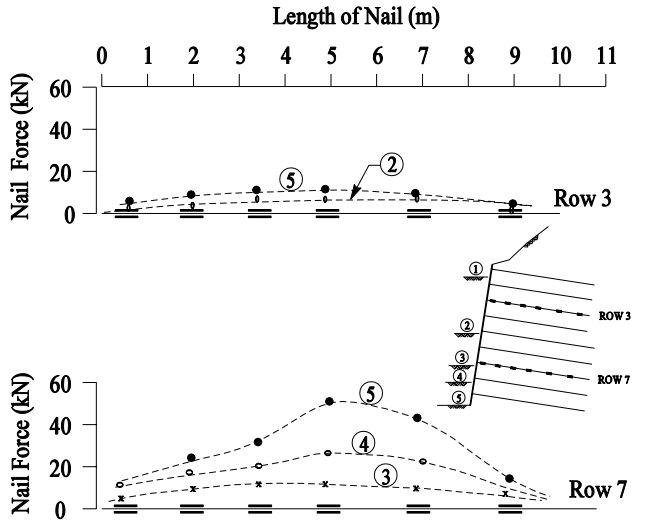


Fig. 27. Responses of soil nails (rows 3 & 7) to excavation.

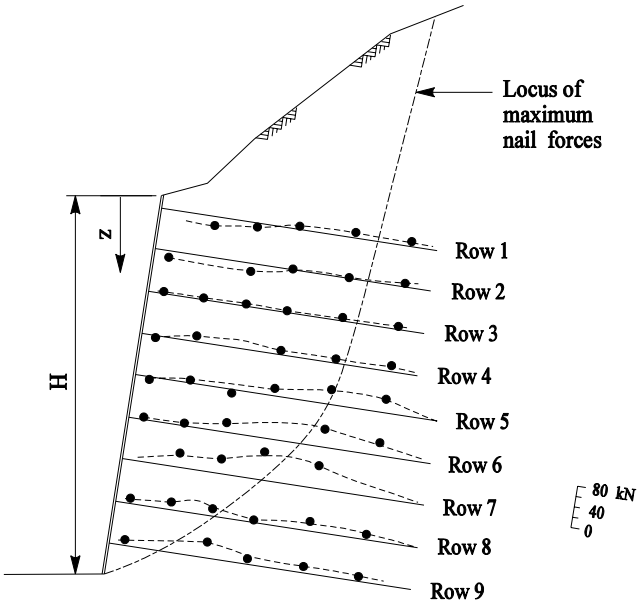


Fig. 28. Distribution of nail forces at the end of construction.

Fig. 29. Shallow failure at a completed soil-nailed slope.

7.2 *Landslides at soil-nailed slopes*

Whilst soil nailing is considered as a robust solution for enhancing slope stability, landslides do occur in soil-nailed slopes. Since 1993, a total of 31 such landslide incidents have been reported to the Geotechnical Engineering Office of the Civil Engineering Development Department. Of these, 24 incidents occurred at completed slopes and 7 occurred in temporary slopes. Those on completed slopes were all minor landslides (minor landslide being defined as one with failure volume less than 50 m^3; major landslide being one with failure volume of 50 m^3 or more), with sizes ranging from less than 1 m^3 to 35 m^3.

The landslides on completed slopes all involved shallow failures or surface erosions in the active zone (Fig. 29). The common factors contributing to the landslides were inadequate slope protection, inadequate drainage provisions or presence of adverse geological or hydrogeological conditions. There was no report of external failure or passive zone failure. There were no failures at soil-nailed slopes with a hard cover neither.

The sizes of landslides on temporary soil-nailed slopes were much larger (Fig. 30). There were three major landslides on temporary slopes, with failure volume ranging from about 50 m^3 to 1,700 m^3. A common factor in these major landslides was that soil nail heads had not yet been constructed at the time of failure.

8 CONCLUSIONS

Systematic research and development studies have brought advances in soil nailing technique. Development and advancement in the technology will continue. This can open up more opportunity for its applications and enable constant improvement in understanding the behaviour of soil nail structures, allowing use of new materials and cost-effective designs with fewer contingencies. The technological advances enable safe and durable nailed structures to be designed and constructed.

Fig. 30. Major failure at a temporary soil-nailed slope without nail head.

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Geotechnical Failures in Hong Kong

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Abstract: This paper presents a selection of notable geotechnical failures that have occurred in Hong Kong since the 1970s. It addresses three subject areas, viz. landslides, problems associated with deep excavations and the 'short pile' scandal in the 1990s. The impact of the failures on the local professional practice is highlighted. The paper does not seek to provide an exhaustive or complete list of the major incidents but rather gives an overview of the range of problems that have occurred.

1 INTRODUCTION

Osterberg (1990) diagnosed the causes of failures in geotechnical engineering as follows:

1 design errors (e.g. mistakes in analysis or use of incorrect or invalid design methods or design assumptions),
2 lack of design input during construction (e.g. communication problems, non-compliance with drawings and specifications, inadequate supervision, lack of design review to account for actual ground conditions, etc.),
3 inadequate ground investigation (including inappropriate interpretation of information),
4 unknown/unforeseen ground conditions.

Osterberg (op cit) emphasized that item (d) above should not be used lightly following failures as a key factor unless comprehensive ground investigations have been properly carried out and supervised prior to the failure. In fact, 'unforeseen' ground and/or groundwater conditions are not necessarily unforeseeable in many instances in hindsight. Osterberg (op cit) opined that ignorance and the failure to learn from past failures have contributed to many of the failures in geotechnical engineering in practice.

In his paper entitled "The Human Factor in Failures", Sowers (1991) summarized his experience over 48 years with the study of some 500 civil engineering failures, the majority of which were geotechnical in nature. Without giving details, he classified the reasons for the failures into three groups:

1 conditions beyond prevailing knowledge (which was responsible for 27% of the failures),
2 ignorance of prevailing knowledge (responsible for 33% of the failures), and
3 failure to use prevailing knowledge (responsible for 40% of the failures).

In Sower's judgement, almost three-quarters of all failures were the result of human shortcomings and only a quarter were due to "conditions beyond prevailing knowledge". This message is of a similar theme to that of Osterberg (1990).

Brand (1991) challenged the contention by Sowers (1991) that "prevailing knowledge" was adequate to have prevented three-quarters of the failures he examined. With the use of cuttings in residual soil profiles as an example, Brand (op cit) opined that these represent the 'difficult' end of the spectrum of stability problems, because of the heterogeneity of the groundmass, the difficulty in ground investigation, laboratory testing of shear strength being of limited value largely due to scale effects and the extreme difficulty in predicting the critical groundwater pressure distribution that could be brought about by intense rainfall. He noted that most of the failures are a consequence of over-simplification. He also contended that the conclusion by Sowers that "three-fourths of the failures are the result of human failure" was harsh.

The precise role played by the individuals in a given failure can only be examined on a case-by-case basis. The essence is to ensure that the best practices are duly established and strictly followed, and that technical knowledge is advanced on a continuous improvement basis. There is no doubt that detailed analyses and extensive discussions of serious geotechnical failures are invaluable and important lessons can be learnt from past failures with a view to preventing similar recurrence.

The Hong Kong SAR Government has been transparent in the investigation of failures in geotechnical works and the findings are promulgated widely to the profession. In this regard, it would be incumbent for the practitioners to seek to keep abreast of the lessons learnt and appreciate the rationale for changes in technical standards and guidance on professional practice.

This paper reflects on a selection of notable geotechnical failures in Hong Kong since the 1970s. It addresses three subject areas, viz. landslides, problems associated with deep excavations and the 'short pile' scandal, and highlights the impact of some of the 'milestone' failures on the evolution of local geotechnical engineering practice.

2 SLOPE FAILURES

2.1 *Landslide studies in Hong Kong*

The occurrence of serious landslides has largely shaped the evolution of the slope safety system and practice in Hong Kong.

Early work on landslide studies was carried out by Lumb (1975). However, he made no reference to geological or hydrogeological factors in the failures and his focus was largely on the influence of water ingress into an unsaturated soil. For example, Lumb (op cit) made the following observations:

"natural slopes show no signs of creep",
"the failures are all 'first-time slips'", and
"no significant seepage is ever noticed from the scarp after slip".

As noted by Hencher (2000), these statements have not stood the test of time as factors that are important to slope failures in tropically weathered profiles have become better understood through the accumulation of knowledge from studies of landslides over the years.

Since its establishment in 1977, the Geotechnical Engineering Office (GEO; renamed from Geotechnical Control Office, GCO, in 1991) of the Hong Kong SAR Government has always taken significant landslides seriously. Annual reports on rainfall and landslide statistics have been compiled and published by the GEO since 1984. Detailed studies of significant landslides have been carried out from time to time since the 1980s.

In the 1990s, the GEO undertook forensic investigations of the fatal landslides, such as the 1992 Baguio Villas landslide (Chan *et al.*, 1996), the 1992 Kennedy Road landslide (Chan *et al*., 1996), the 1993 Cheung Shan Estate landslide (Chan et al, 1996), the 1994 Kwun Lung Lau landslide (GEO 1994) and Castle Peak Road landslide (Chan *et al.*, 1996), the 1995 Fei Tsui Road landslide (GEO 1996a) and Shum Wan Road landslide (GEO 1996b), the 1997 Kau Wah Keng landslide (HAPL 1998a) and Ten Thousand Buddhas' Monastery landslide (HAPL 1998b). The investigation reports were submitted to the Coroner's Court and the lessons learnt were widely promulgated to the profession.

Apart from the study of the individual landslides, overall studies of landslides in an area triggered by a specific rainstorm have also been undertaken. Examples of such area studies include the approximately 350 reported landslides on Hong Kong Island arising from the 8 May 1992 rainstorm (Evans 1992), and the 300 or so reported landslides on man-made slopes (Wong & Ho, 1995) and more than 800 landslides on natural hillsides on Lantau Island arising from the 5 November 1993 rainstorm (Wong *et al.*, 1998).

2.2 *Some notable landslides in the 1970s*

In 1972, rain fell almost every day from early May, becoming more intense in mid-June, with a daily rainfall of over 200 mm from 16 to 18 June 1972.

The two most catastrophic landslides in the recent history of Hong Kong occurred on 18 June 1972, which together brought about more than 130 fatalities. Prior to that time, slope formation works in Hong Kong were largely done in an empirical manner with no geotechnical input (except for embankment dam construction). In general, slope failures up to then tended to be looked upon as an inevitable outcome of heavy rain. However, public outcry over the large number of deaths of the two fatal landslides led to the appointment of a Commission of Inquiry, charged with making recommendations to avoid such disasters.

The first fatal landslide occurred shortly after 1 p.m. in the Sau Mau Ping Resettlement Estate following a heavy downpour at about 11 a.m. with a maximum hourly rainfall of about 70 mm. The failure involved the sudden collapse of about 6,000 m^3 of fill material from a 40 m high road embankment inclined at 34°. The mobile debris slid down "like a carpet" and inundated a Licensed Area for temporary housing (Fig. 1) and destroyed many huts resulting in 71 fatalities and 60 injuries.

According to the interim report by the Commission of Inquiry (Hong Kong Government 1972a), there were 384 residents in the licensed huts and ten others in unlicensed huts within the Licensed Area prior to the landslide. According to the eye-witnesses accounts, small slips started to occur on the embankment some 20 minutes before the fatal landslide,

Fig. 1. The 18 June 1972 Sau Mau Ping landslide

bringing down soil debris onto the Licensed Area. In addition, the slope had suffered major surface erosion since its construction in the mid-1960s. The building platform for the housing estates, together with the road embankment, in the Sai Mau Ping area was constructed in the 1960s. The road embankment that failed in 1972 was completed in 1964 by depositing fill in a 60 m wide valley. The decomposed granitic fill, obtained from the adjoining site formation works, was specified to be deposited in 3 ft. (approximately 1 m) thick layers. However, in practice the fill material was largely deposited by end-tipping, which was commonly adopted in the construction industry in the 1960s. As a result of lack of compaction, the fill materials were in a loose state.

The Commission of Inquiry concluded that the landslide was "due primarily to softening of fill material caused by infiltration of rain-water mainly through the sloping face, as a result of an exceptional long and intense rainstorm" (Hong Kong Government 1972b). It also established that no fault was found with the manner in which the design and construction of the embankment was carried out.

Some eight hours after the fatal Sau Mau Ping landslide, at about 9 p.m. on 18 June 1972, a massive landslide (about 20,000 m^3) up to about 10 m deep occurred on a steep hillside above Po Shan Road in the Mid-levels area (Figure 2), killing 67 people and injuring 20 others. A 13-storey residential block was knocked down by debris impact. The slope failure was triggered by the collapse of a steep cutting at the toe of the hillside within a private building lot, which had been left to stand for some eight years. The excavation within the private lot began in 1964 to form a building platform and a steep cutting was formed below Po Shan Road, which remained unprotected following suspension of works then. Excavation resumed in early 1971, further deepening the cut and resulting in a number of minor slips. The process gave rise to signs of distress in the area in the form of small landslides and cracking of road pavement. On 16 June 1972, more extensive cracking was noted on Po Shan Road. In the morning of 17 June 1972, a large landslip occurred on the steep cutting within the building lot, distorting the sheet piling at the toe that had been installed in March 1972. A section of Po Shan Road settled some 1.8 m, getting even worse at noon and

Fig. 2. The 18 June 1972 Po Shan Road landslide

reaching as large as 4.5 m. Some material from the hillside above slipped onto Po Shan Road. In the morning of 18 June 1972, the sheet piling within the private building lot continued to distort. At about 5 p.m., a large landslide occurred within the private lot and the debris flowed across Conduit Road, demolishing the rear wall of No. 11 Kotewall Road, a two-storey house. Eventually, the fatal landslide occurred at about 9 p.m. The Commission of Inquiry (Hong Kong Government 1972b) concluded that three unfavourable factors combined to cause the landslide, namely the nature of the material forming the hillside, the exceptional intense rainstorm and the steep cutting in the private building site. The incident highlighted the marginal stability of steep hillsides and the severe consequences to life and property due to lack of geotechnical control.

Following the two fatal landslides in 1972, a civil engineering section with ten professional posts was set up in the then Buildings Ordinance Office (BOO) of the then Public Works Department (PWD), to assist the building surveyors and structural engineers by checking geotechnical aspects of development submissions. Soon afterwards, a consultant was commissioned to support the section in the vetting of submissions. Since that time, the traditional permissible angle rules for cut slopes were no longer automatically acceptable and designs for major earthworks required justification by use of soil mechanics analysis, based on a ground investigation report. In 1973, another consultant commenced systematic slope stability studies in phases on a territory-wide basis, culminating in recommendations on proposed actions to rectify potentially dangerous slopes and retaining walls. The same consultant published in the same year a guide to site investigations and earthworks, dealing with the causes of landslides and their prevention.

On 25 August 1976, another very destructive landslide occurred in the Sau Mau Ping Resettlement Estate following heavy rain associated with Tropical Cyclone Ellen. A total of four landslides took place, all resulting from the collapse of the side-slopes of highway embankments formed of earth fill. Three of these turned into mud avalanches, the most hazardous of which occurring on the face of a 35 m high embankment above an occupied public housing block. The 4,000 m^3 landslide debris slid down as "a large sheet" and inundated the ground floor of the public housing block. Eighteen people were killed and 24 seriously injured. The Governor immediately convened a panel of six geotechnical experts, four from overseas, to report on the causes of the landslide and to propose measures to prevent similar disasters in future. The technical investigation (Hong Kong Government 1977) concluded that the loose fill, placed by end-tipping, was liable to liquefy under intense rainfall leading to mobile debris with a long runout. The failure mechanism comprised an undrained failure involving the collapse of a metastable structure of the saturated, or near saturated, loose fill.

As a result of the recommendations of the Independent Review Panel for Fill Slopes (Hong Kong Government 1977), a consultancy was initiated to investigate all fill slopes in Hong Kong. The panel further recommended establishing a control organization within the Government to provide continuity throughout the whole process of investigation, design, construction, monitoring and maintenance of slopes in Hong Kong. As a result, the Geotechnical Control Organization was set up in July 1977, which comprised an independent Geotechnical Control Office (GCO) in PWD and a Geotechnical Control Branch in the BOO. Recompaction of the top 3 m of loose fill slopes was implemented as a prescriptive measure to retrofit the substandard fill slopes at the recommendation of the Independent Review Panel for Fill Slopes. Thus, following a series of 'milestone' landslides in the 1970s, Hong Kong has swung from a pragmatic acceptance of landslides effectively as acts of God to an acute awareness of geotechnical problems and concerted efforts were initiated to combat and mitigate such natural hazard.

2.3 *Some notable landslides in the 1980s*

The severe rainstorms in May and August 1982 brought about some 700 reported landslides. Although there were no disastrous events of the Sau Mau Ping or Po Shan Road type, the total loss of life amounted to 27, of which 23 persons were occupants of squatter huts. The 1982 rainstorms brought about a general recognition of the need for concerted effort to tackle the landslide risk to squatters. The GCO commenced a comprehensive study of squatters on hillslopes of Hong Kong Island and Kowloon, where much of the squatter population were then located, and where most of the recorded landside casualties had occurred. This study used the newly-established terrain classification database to zone Hong Kong into three categories of relative potential hazard with respect to squatters. The hazard zonation provided, for the first time, a rational basis for prioritized inspections of the squatter areas on hillsides, which enabled the then GCO to draw up safety clearance recommendations under the Non-Development Clearance (NDC) Programme.

In addition, the GCO selected ten significant landslides in 1982 for follow-up studies (Hencher 1983). The scope of the studies comprised desk studies, field mapping and back analyses, but no post-failure ground investigation works were carried out. The causes of the majority of the landslides were attributed to complex geological conditions and/or unanticipated groundwater condition. The studies noted that inadequate ground investigation data cannot be compensated by means of a higher factor of safety, "as the major problem appears to be in investigation and interpretation rather than in numerical analyses".

The 30 July 1987 major landslide at Cho Yiu Estate resulted in the temporary evacuation of 24 flats in a residential block due to concern about possible loss of support to the building foundation. Fortunately, no casualties were caused. The landslide (1,200 m^3) involved the failure of a 15 m high engineered soil/rock cut slope. Subsequent investigation (Siu & Premchitt, 1988) established that the failure occurred largely on adversely orientated relict joints with weak kaolin infill, which formed both potential failure surfaces and preferential groundwater flow paths. The failure was primarily rain-induced, but blockage of

the surface drainage system was thought to have contributed to water ingress into the groundmass. Massey & Pang (1988) noted that comparison of this failure with previously studied landslides (Hencher et al, 1984) revealed the following common factors:

1 Large areas of the failure surface coincided with sets of relict joints, many of which contained relatively weak infill materials, and the existence and significance of which were identified only after failure had taken place.
2 Observations of groundwater seepage soon after failure indicated that water was flowing along preferential flow paths in the soil mass, such as relict joints and subsurface erosion pipes.

The subject cut slope was previously assessed by consultants as having an adequate safety margin. The post-failure investigation (Siu & Premchitt, 1988) established that inadequate allowance was made for the influence of relict geological structures in the past assessment. The landslide also highlighted the importance of proper engineering geological input in slope stability assessments and the vulnerability of non-robust designs, such as unsupported cuts, to the uncertainties in heterogeneous ground conditions.

On 31 August 1988, one day before the start of a new school term, a major landslide (800 m^3) occurred on an unsupported soil cut behind the Island Road Government School in Aberdeen during the construction of landslip preventive works. The debris travelled rapidly for a distance of about 30 m, reaching the school complex but fortunately with no casualties. The subsequent detailed landslide investigation (Irfan 1989) revealed that both the man-made slope and the natural terrain above, together with the cuttings in the vicinity, had a long history of instability dating back to the mid-1940s. The investigation highlighted the importance of identifying adverse geological structures or pre-existing rupture surfaces during the investigation stage or during construction reviews.

2.4 *Some notable landslides in the 1990s*

On 11 September 1990, a massive channelised debris flow occurred on the eastern flanks of Tsing Shan above Tuen Mun (Figure 3). The failure was apparently triggered by relatively light rainfall, with a corresponding return period of less than 3 years (King 1996). The debris flow initiated as a relatively small rock topple and rock/soil slide (350 m^3) on the steep upper slopes of Tsing Shan. The debris flowed over an exposed sheeting joint and entered the streamcourse below, developing into a channelised debris flow along the steep drainage line infilled with loose bouldery colluvium. Given considerable entrainment along the drainage line, the volume of debris reached 19,000 m^3, making it the largest mobile natural terrain landslide in recent time in Hong Kong. The runout distance of the landslide mass was approximate 1 km, with the debris encroaching onto a proposed building platform. The estimated peak velocity of the debris exceeded 16 m/sec. Had the housing development proceeded as originally planned, the consequences could have been very serious.

This failure highlighted the marginal stability of steep hillsides in Hong Kong and the potential hazard of low-frequency, high-magnitude natural terrain landslides, which could result in grave consequences if they were to occur close to densely developed areas. The significant entrainment was a result of the specific combination of the site settings. The 1990 Tsing Shan debris flow led to the initiation of an R&D programme on natural terrain landslides, culminating in, inter alia, the compilation of an inventory of natural terrain landslides and the promulgation of guidance on natural terrain hazard studies. An area study of the Tsing Shan Foothills was also undertaken subsequently in the early 2000s (MFJV 2002), which advanced the knowledge in respect of susceptibility and hazard analyses.

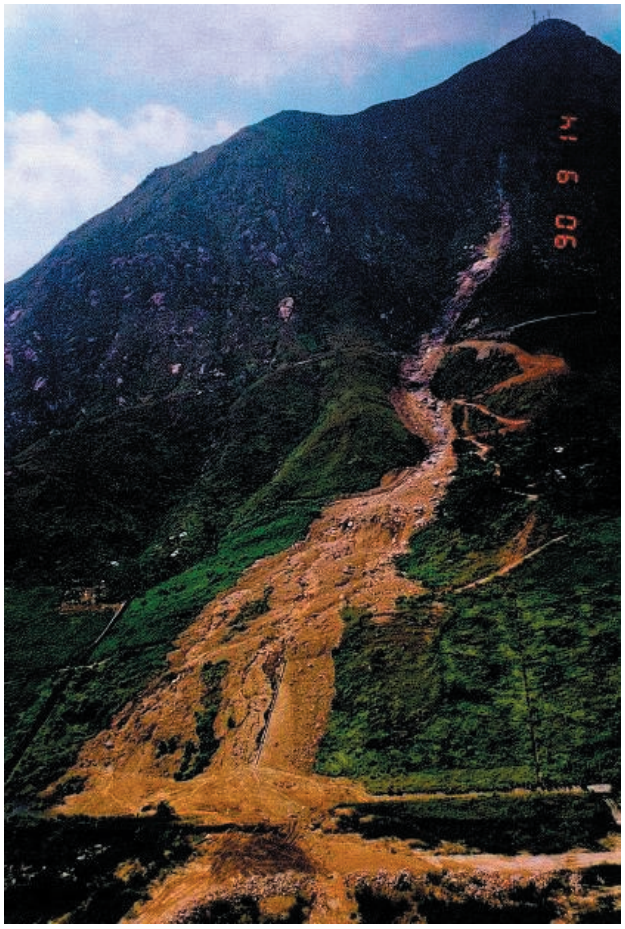

Fig. 3. The 11 September 1990 Tsing Shan debris flow

On 8 May 1992, a localized deluge occurred, bringing more than 350 mm of rain in a 9-hour period (with a return period of about 70 years) to the western coast of Hong Kong Island. Some 350 landslides were reported to the GEO, two of which resulted in a total of three fatalities. An overall review of the reported landslides was carried out by Evans (1992).

One of the fatal landslides occurred at Baguio Villas (1,500 m^3), which involved the collapse of a 9 m high, unregistered masonry wall retaining a fill platform in a temporary housing area (Figure 4). Fill material was released progressively into a 100 m high, 32º gully below. Subsequent concentrated runoff from intense short-duration rainfall flushed the debris down the 170 m long drainage gully, in the form of a major washout. The fluid mass struck the back of Block 44, which stands at the foot of the slope, punching a hole in the podium slab and piling up against the building. The debris entered the podium-level apartment through the windows and caused the death of a 7-year old boy inside. A government engineer, who was on inspection duty on the podium at the time, was also killed. The findings of the forensic investigation are documented by Chan et al (1996).

The 1992 landslides have highlighted two matters of concern: firstly, the 1977/78 Catalogue of Slopes was incomplete, with selected uncatalogued slopes posing a significant hazard to the community; secondly, many private owners were genuinely

Fig. 4. The 8 May 1992 Baguio Villas landslide

unaware of their obligations to inspect and maintain slopes under their responsibility. Subsequently, the GEO launched the Systematic Identification and Registration of Slopes in the Territory (SIRST) project to update the Slope Catalogue and the Systematic Identification of Maintenance Responsibility of Slopes in the Territory (SIMAR) project to delineate slope maintenance responsibility. The GEO also stepped up its public education and publicity campaigns on the importance of slope maintenance and the responsibility of private owners in relation to undertaking regular maintenance on their own slopes.

The 23 July 1994 Kwun Lung Lau landslide (Wong & Ho, 1997) involved the sudden collapse of a 100-year old masonry wall that was in good maintenance condition (Figure 5). The full height of the masonry wall together with the slope above failed and some 1,000 m^3 of debris was released, killing five people and seriously injuring three others on the busy footpath below. In addition, more than 3,900 residents were temporarily evacuated overnight for fear of collapse of the high-rise buildings they occupy. The Legislative Council subsequently voted to create a Select Committee to enquire into the circumstances of the landslide and related issues: this was only the second Select Committee in Hong Kong's history. Slope safety had not gained such public prominence in Hong Kong since the 1970s.

The masonry wall had a maximum height of 10.6 m and a base width of 0.8 m (which was about one-fifth of that shown in the approved drawing in 1965), i.e. a slenderness ratio of more than 13. This was exceptionally slender as compared with typical masonry walls of a similar construction in Hong Kong, which generally have a slenderness ratio of less than 4.

The geology of the landslide location comprised fill overlying partially weathered volcanics. The fill behind and above the wall was loose and generally very permeable. The wall had been inspected by a number of professional parties in the past and was found to be in fair or good condition with no signs of distress.

The technical investigation by the GEO (GEO 1994) established that the landslide involved buckling and brittle collapse of the thin masonry wall. The failure was triggered by subsurface infiltration from defective buried drainage systems, which saturated and weakened the soil mass. The state of knowledge at the time was that old masonry walls would fail in a ductile manner following deformation for some time. However, the Kwun Lung Lau masonry wall was in a good condition and yet it failed suddenly with little signs of deformation or distress. The failure mechanism was apparently different to the previous understanding and was therefore investigated in detail.

Fig. 5. The 23 July 1994 Kwun Lung Lau landslide

Advanced numerical analyses using the distinct element computer program UDEC, were carried out to assist in the diagnosis of the mechanism and causes of the failure. The analyses predicted that the masonry structure would fail in a complex mode. The masonry wall was found to bulge initially at about mid-height, accompanied by overturning of the portion of the masonry wall below this level. These deformation modes combined to lead to tensile failure and consequential sudden reduction of the shear strength of the affected mortar joints. The bulging and overturning action resulted in brittle fracture of the masonry wall at about mid-height, the ground behind lost support and slid forward. An important finding was that once tensile or shear failure of mortar joints was initiated, the wall would deform quickly, with instability developing in an uncontrolled manner. The failure mode of the thin masonry wall was brittle which, once initiated, would develop rapidly. Such a complex failure mechanism was not considered in conventional retaining wall analyses, which could be unconservative in the case of thin masonry walls.

Following the Kwun Lung Lau landslide, the local professional practice in respect of the assessment of stability of old masonry walls was rationalised. The assessment of the stability of old masonry walls is not straightforward because of their variable and non-monolithic construction. In general, the approach involved a combined assessment of the calculated margin of wall stability and the state of wall deformation based on visual inspection and this continues to be applicable to well-proportioned masonry walls. However, thin masonry walls (defined as slenderness ratio of greater than 5) are liable to fail in a brittle manner without prior warning and these should be taken as being substandard, irrespective of the apparent wall condition.

The Kwun Lung Lau landslide also highlighted the implications of leakage from underground water-carrying services on slope stability. Rainfall analyses indicated that the 48-hour rainfall preceding the landslide was the most severe (with a return period of about 28 years). The fatal landslide was a delayed failure in that it occurred several hours after cessation of intense rain.

Finite element seepage analyses were carried out to assess the contribution of the different sources of water in saturating the ground behind the masonry wall. Transient seepage analyses were undertaken to examine the 50-hour period starting from the beginning of the rainstorm through to the time of the landslide, taking into account the actual rainfall profile by the nearest automatic raingauge and the soil suction measurements.

The forensic investigation established that subsurface seepage flow took place through the permeable fill layer at an elevation higher than the landslide and that the principal source of water ingress was from defective sections of stormwater pipes located some 35 m behind the failed masonry wall. The seepage led to wetting up of the loose fill behind the masonry wall. The consequential settlement probably led to distress or rupture of a foulwater sewer running across the upper part of the landslide area, which had a rigid joint that was susceptible to ground deformation. This resulted in substantial saturation of the retained groundmass leading to the collapse.

The Kwun Lung Lau landslide emphasized the potential of adverse environmental changes involving leakage from defective water-carrying services (e.g. due to deterioration and poor maintenance) in destabilizing a slope. The fatal landslide led to the issue of a Code of Practice on Inspection and Maintenance of Water Carrying Services by the Government to upgrade professional practice in the investigation and maintenance of underground water-carrying services (ETWB 2006).

The Kwun Lung Lau landslide also highlighted the need to adopt a more integrated perspective in respect of slope stability studies (Morgenstern 1994). This led to the initiation of the systematic landslide investigation programme in 1997 by the GEO. Also, the Administration provided additional resources to the GEO to accelerate the Government's Landslip Preventive Measures (LPM) Programme.

Following the Kwun Lung Lau landslide, the GEO has pioneered the development and introduction of a more explicit risk-based approach and strategy for slope assessment and landslide risk management in Hong Kong (Wong & Ho, 2006), including the novel use of quantitative risk assessment techniques.

There have been some notable failures of unsupported or unreinforced cut slopes that were previously assessed and accepted under the Government's slope safety system in the 1990s as meeting the required geotechnical standards. Two examples are presented below.

The 13 August 1995 Fei Tsui Road landslide (Fig. 6), which resulted in one fatality and one injury, involved the massive failure (14,000 m^3) of an unsupported soil cut that was previously assessed to have an adequate factor of safety. The rain was heavy from the morning of 12 August 1995 to the time of the landslide in the early morning of 13 August 1995. Rainfall analysis indicated that the 31-day rainfall was the most severe, with a corresponding return period of about 95 years. The cutting was inclined towards the north at an average angle of about 60° to the horizontal with a maximum height of about 27 m. Rock was exposed at the lower part of the cut slope and the upper portion was covered with chunam. A 90 m long section of the 7.3 m wide Fei Tsui Road, with a pedestrian pavement of about 3.3 m wide along its northern side, was buried by the landslide debris. Some of the debris was deposited onto the playground across the road, and part of the debris piled up against the south-western corner of the Chai Wan Baptist Church to a maximum height of about 6 m. Prior to the fatal landslide, two previous failures, which occurred in 1987 and in 1993 with volume of about 50 m^3 and 30 m^3 respectively, were reported to the GEO, each with insignificant consequences. The forensic investigation (GEO, 1996a) established that the basal slip surface of the fatal landslide developed along a laterally extensive (>50 m) layer of kaolinite-rich altered tuff, which was about 15 m below the crest of the cut slope and dipping out of the slope at about 10° to 25°, whilst the back scarp of the landslide was defined by two steep joint sets. The approximately 0.5 m thick layer that controlled the basal failure plane was highly kaolinised and completely decomposed, with abundant kaolinite veins ranging from 2 mm to 20 mm in thickness. The landslide study highlighted the need for attention in the investigation and characterization of kaolinite-rich zones and the implications of such adverse geological features on the groundwater responses to severe rainstorms. Signs of strong seepage were observed above the clay-rich layer during the past engineering studies. However, given the relatively shallow inclination of the layer (up to 25°), it was considered in the past that the chance of a large-scale translational failure along this weak layer was remote. In the event, the failure occurred due to the build-up of a perched water table above the persistent kaolinite-rich layer following heavy and prolonged rainfall.

Fig. 6. The 13 August 1995 Fei Tsui Road landslide

Following the 1995 fatal landslides at Fei Tsui Road and Shum Wan Road, mineralogical studies of kaolin were carried out by the GEO to provide new insights into the mode and occurrence of kaolin infills. Also, a series of seven area studies of cut slopes and natural terrain was completed, covering granites of differing grain size in Kwai Chung, Homantin and Shatin, and coarse and fine ash tuffs in the Mount Davis, Aberdeen, Chai Wan and Tseung Kwan O areas. These studies have led to the production of a suite of 1:5,000-scale maps showing lithology and fabric, past failures, sites with perennial seepage, etc.

In July and August 1997, a series of three landslides occurred on a roadside slope above Ching Cheung Road, which is a major urban trunk road (Fig. 7). This roadside slope comprised an unsupported soil cut that was upgraded under the LPM Programme in 1992, during which the upper two slope batters were trimmed back for certain sections. Debris from the 1997 landslide (2,000 m^3) completely blocked a 50 m section of the road and trapped a vehicle. Ching Cheung Road was subsequently closed for about one month until completion of the temporary slope stabilization works. The geology of the ground was complex, comprising dominantly highly and completely decomposed granite, with medium to closely spaced joints. Some decomposed basalt dykes up to 1.3 m thick intruded into the granite with a notably lower permeability than that of the surrounding weathered granite. Extensive natural erosion pipes, up to about 250 mm in diameter, were present throughout the weathered profile. The pipes were generally infilled with quartz-rich fine silts and sands at the top, grading to fine gravels at the base. The site was used as a borrow area in the 1920s and it has a history of instability since the mid-1940s. Large-scale landslides were recorded on the slope in 1972 (7,500 m^3) and

Fig. 7. The 3 August 1997 Ching Cheung Road landslide

1982 (1,500 m^3) respectively. Following the completion of LPM works in 1992, a minor failure occurred in 1993.

The findings of the 1997 landslide investigation (HAP 1998c) highlighted the importance of giving due consideration to the slope performance history, the implications of complex and delayed groundwater response, and the potential adverse effects of high groundwater levels due to concentration of groundwater flow from sizeable upslope catchment areas and along old buried colluvial valley infills truncated by slope cutting, where the 1997 landslide occurred. The previous landslides could have resulted in local weakening of the weathered rock mass and contributed to the development of the natural pipe system in the slope.

The technical investigation established that the two landslides that preceded the major detachment on 3 August 1997 represented the progressive collapse of a large-scale landslide, which occurred on 7 July 1997. The 1997 landslide was probably triggered by the build-up of adverse transient groundwater conditions following severe rainfall in early July 1997 and the previous month, which has a return period of about 500 years. The 151 sixteen-metre long raking drains that had been installed in 1972 following a massive landslide at that time (but these raking drains had not been regularly maintained over the years mainly because of inadequate record-keeping) were unable to prevent the build-up of critical groundwater pressures.

The relatively high failure rate of engineered unsupported cuts highlights the need for enhanced engineering geological input. A more robust design scheme, less vulnerable to local adverse geological and groundwater conditions, such as use of soil nails as opposed to cutting back, is generally preferred in upgrading soil cut slopes in tropically weathered profiles (Wong & Ho, 2000a).

2.5 *Some notable landslides in the 2000s*

The 1 September 2001 channelised debris flow, with an estimated source volume of 250 m^3, occurred on the natural hillsides above Lei Pui Street, Shek Lei Estate in Kwai Chung (Fig. 8). The landslide occurred during heavy rainfall with a return period of about 14 years. The failure was probably triggered by direct infiltration and the development of critical groundwater pressures, including possible cleft water pressures within open joints in the granitic rock mass (MGSL 2002). The subject hillside catchment has a history of instability based on aerial photographs. The debris of the 2001 landslide entered into

Fig. 8. The 1 September 2001 Lei Pui Street debris flow

a drainage line below and further material was entrained, giving rise to a channelised debris flow with an active debris volume of up to about 800 m^3. Discounting the subsequent outwash, the debris travelled a horizontal distance of about 350 m, reaching an active construction site near Lei Pui Street, and some 50 m^3 of outwash material was deposited on Lei Pui Street and Shek Lei Estate below.

The channelised debris flow demolished two flimsy squatter structures. It was fortuitous that the four occupants had vacated their dwellings about two hours before the landslide because of concern about their personal safety. It is noteworthy that the consequences would have been much more serious had the large number of squatters previously located on the lower hillside area not been cleared under the NDC Programme in 1985 and 1995 respectively. In this regard, the 2001 debris flow was a near-miss incident. Given that there were no obvious changes in the environmental conditions of the landslide site, the fact that the landslide was triggered during a rainstorm event that was not as severe as those previously experienced by the landslide site, it may be inferred that the hillside had probably suffered progressive deterioration. After the landslide, a debris barrier, together with a retention basin, was constructed at the mouth of the streamcourse.

On 20 August 2005, a landslide (about 400 m^3) occurred on a hillside that has been disturbed locally by past human activities at Fu Yung Shan Tsuen and resulted in one fatality. Four registered squatter structures at the toe of the hillside were subsequently permanently evacuated. The landslide took place during a severe rainstorm with a return period of about 100 years. Human activities were evident on the hillside, which could be traced back to the late-1940s based on aerial photographs, but there are no records of any previous reported failures in the immediate vicinity of the 2005 landslide. The four affected

squatter structures had outstanding NDC recommendations which were made in 1992. However, the occupants refused to leave. The technical investigation (MGSL 2006) established that the landslide was probably caused by the loss of soil suction and transient build-up of adverse groundwater pressure in the near-surface materials following prolonged and intense rainfall. The presence of domestic rubbish on the subject hillside was considered to have provided an adverse hydraulic boundary condition, which promoted the build-up of high groundwater pressure over a large volume of the groundmass following heavy rainfall, and thus was a contributory factor to the failure.

The fatal landslide at Fu Yung Shan Tsuen highlighted the ineffectiveness of the implementation of NDC recommendations since the mid-1990s and the risk posed to squatters. The incident led to a review of the squatter policy by the concerned policy bureaux.

3 SYSTEMATIC LANDSLIDE INVESTIGATIONS

3.1 *Systematic landslide investigation programme*

The systematic landslide investigation (LI) programme was formulated by the GEO following the 1994 Kwun Lung Lau landslide based on the recommendations of Professor Morgenstern (1994). This new initiative was implemented in 1997 for a trial period of three years, with the assistance of consultants. Following the trial implementation, systematic landslide investigations became an integral part of the LPM Programme.

The main objectives of systematic landslide investigations are as follows:

1 identify slopes in need of early attention before the situation deteriorates to result in a more serious problem,
2 improve the understanding of the causes and mechanisms of landslides to formulate new ideas for technical development work and enhance the reliability of landslide preventive or upgrading works,
3 audit the performance of the Government's slope safety system and identify areas for improvement, and
4 provide evidence in forensic studies of serious landslides that may involve coroner's inquest, legal action or financial dispute.

Under the systematic LI programme, the information on all the reported landslides is examined. The reported landslides are screened by a panel of experienced geotechnical professionals and deserving cases are identified for follow-up inspection and detailed investigation. Typically, several hundred landslides are reported to the GEO every year and about 20 to 30 incidents are selected for detailed investigation.

The systematic LI programme has effectively served as an asset management tool and has contributed to enhancing public safety. Much of the enhanced slope engineering practice in recent years has originated from an improved understanding of landslides in Hong Kong through the systematic LI programme.

Areas with technical development arising from the systematic LI programme are presented by Wong & Ho (2000a). Examples of advances in technical knowledge include:

1 new insights into the different modes and mechanisms of landslides on man-made slopes and natural hillsides (Wong & Ho, 2000b) respectively,
2 progressive nature of some slope failures and the practical implications,
3 mobility of landslide debris in relation to failure mechanisms and influence of environmental factors,
4 effects of subsurface water flow and hydrogeological boundaries vis-à-vis uncontrolled surface water flow on slope stability,
5 influence of adverse geological materials and structures in slope failure,
6 the need to recognise and consider the implications of relict instability and past failures,
7 consideration of robustness in slope design, and
8 enhanced detailing of surface and subsurface drainage provisions.

In addition, the assessment of the annual failure rates of different classes of slopes (viz. old man-made slopes, engineered slopes with no structural support or reinforcement, engineered slopes with robust measures such as soil nails, etc.), based on the systematic landslide records and studies, has provided key data for the quantification of risk (Lo & Cheung, 2004).

3.2 *Lessons learnt from failures of engineered man-made slopes*

A diagnosis of the landslide data revealed that the annual failure rate of engineered slopes is only lower than that of non-engineered slopes by a factor of about 2 to 3. This emphasizes the need to further improve the design practice and slope detailing in order to enhance the reliability and robustness of engineered slopes.

Common problems associated with small-scale failures of engineered man-made slopes are related to the following:

1 uncontrolled surface runoff,
2 inadequate slope maintenance,
3 poor detailing of slope drainage provisions, and
4 local weaknesses in the groundmass.

In addition, there have also been cases of failure of a properly designed slope due to inadequate construction control and construction non-conformances.

The main problems associated with large-scale failures of engineered man-made slopes are as follows:

1 adoption of over-simplified ground model that does not adequately cater for safety-critical geological features in the groundmass,
2 use of inappropriate hydrogeological model that does not adequately account for adverse groundwater conditions, and;
3 insufficient attention given to the history of instability.

Systematic landslide studies have also revealed evidence of prolonged slope movement for a period of time (several years or more) before detachment of the failed mass, especially in sizeable landslides. This may be associated with progressive slope deterioration. The presence of distressed ground can pose a significant hazard as the eventual detachment may occur in an abrupt manner given rapid water ingress into a dilated groundmass during intense rainstorms.

Suggestions were made by Ho *et al.* (2002) and Ho (2004) on improved slope engineering practice with a view to further reducing the failure rate of engineered slopes.

4 DEEP EXCAVATIONS

In the past thirty years, there have been several examples of excessive displacement or collapse caused by deep excavations in Hong Kong. Many of these are catalogued in a GEO internal report by Man & Yip (1992), which led to increasing

requirements for design submissions and site supervision by the regulatory authorities including GEO, together with additional spot checks on site by their staff.

In 1998, the GEO appointed Ove Arup and Partners, Hong Kong, Ltd. (Arup), supported by ERM Hong Kong Ltd., to carry out a Quantitative Risk Assessment (QRA) of collapses and excessive displacements of deep excavations associated with private building sites. The main objective of the study was to identify the critical aspects of excavations on which the efforts of the GEO geotechnical control should concentrate. The risk was quantified in terms of risk to life to the public and also to workers. For the purposes of that study, deep excavation means a vertical or near vertical excavation deeper than 5m supported by a retaining wall system. The results of the study were reported in Arup (2002).

This part of this paper summarizes the principal findings and recommendations of the above study.

4.1 *Data sources*

The 1992 study by Man & Yip (op cit) examined 31 cases of collapse or excessive displacements arising from deep excavations from the late 1970s up until 1992. GEO provided one additional case that occurred in 1993 for the subsequent QRA by Arup. For many of the cases, the GEO supplied further information from their files. The data include the total planned depth of the excavation, the type of wall and number of planned strutting levels, details of the ground and groundwater conditions together with details of the failure including the year, depth and probable causes (if known). Other sources included a review of Arup's experience and a survey of other consultants in Hong Kong.

4.2 *Classification of failures*

For the purposes of the study, failure was defined as an event associated with the construction of a deep excavation that could potentially lead to loss of life. In general, they took the form of excessive displacement sufficient to lead to some secondary cause of a fatality or a collapse. A collapse could lead to damage or destruction of adjacent existing buildings and utilities, falling of pedestrians or vehicles. Another possible consequence is casualties of workers within the excavation site.

The failures were divided into categories as follows:

1 *excessive displacement,* where the movements are sufficient to cause unexpected disturbance to adjacent property, roads or services.
2 *small collapse*, represented by a plan area of up to 10 m^2.
3 *medium collapse*, represented by a plan area of 10 m^2 to 100 m^2.
4 *large collapse*, represented by a plan area in excess of 100 m^2.

The case histories were reviewed to classify the failure into one of the above categories and to determine the causes of failure. In all, there were 16 cases of excessive displacement and 9 collapses.

Of the 16 cases of excessive displacement 10 involved sheet pile walls of which eight were related to omitted or poorly detailed strutting. Four cases were associated with diaphragm walls, two of which resulted from problems during wall installation. There were four cases where the observed displacement did not exceed those predicted. In these cases, however, inadequate preparation was made to mitigate the effects of the predicted displacements.

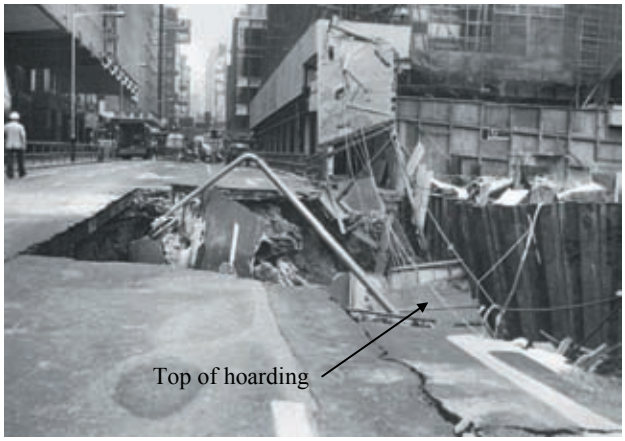

Fig. 9. Photograph of the 1981 collapse at Queen's Road

Two notable examples of the nine cases where collapse occurred were as follows:

– A large collapse that occurred in Queen's Road Central in 1981 (Figure 9). It can be seen that the failure was extensive with the top of the temporary hoarding dropping by about 9 metres. The failure is reported to have been caused by a combination of sheet piles that were not driven to the correct 'design' penetration together with the removal of temporary props to facilitate construction of the permanent works.
– A medium-sized collapse that occurred in Mau Lam Street in 1991. This collapse was caused by inadequate shoring between soldier piles (Figure 10). A full report of this collapse is given by Chan (1992).

Eight of the nine cases of collapse were associated with sheet pile walls and one with a soldier pile wall. Many of the sheet pile cases involved inadequate penetration and/or missing strutting. While it could be surmised that these were simply examples of the contractor taking opportunistic short cuts, often insufficient consideration of the robustness of the design was evident. A design showing sheet piles penetrating into very hard strata without adequate provision and cost allowance for pre-boring, for example, has a good chance of getting into difficulty during construction. Similarly, a design relying on many levels of struts, at a close vertical spacing of 3 m or less, is clearly very difficult to construct and there will be a strong temptation to over-excavate before installing the struts or even omitting some levels. The design also needs to consider the construction sequence of the permanent support works and the removal of the temporary struts.

4.3 *Observed number and distribution of failures*

The observed number of collapses or excessive displacements have been divided into 5-year periods and are plotted in Figure 11. As can be seen from the figure, the frequency of occurrence of excessive displacements is variable with no clear trend. However, the number of collapses has reduced dramatically since 1990. The number of failures against time will be expected to vary for a number of reasons. One reason may be variations in the economy, which has a significant effect on the total number of projects under construction at any one time.

Other factors include changes in legislation and geotechnical standards with regard to building control and checking regime. Special statutory and administrative measures were introduced

Fig. 10. Photograph of the 1991 collapse at Mau Lam Street

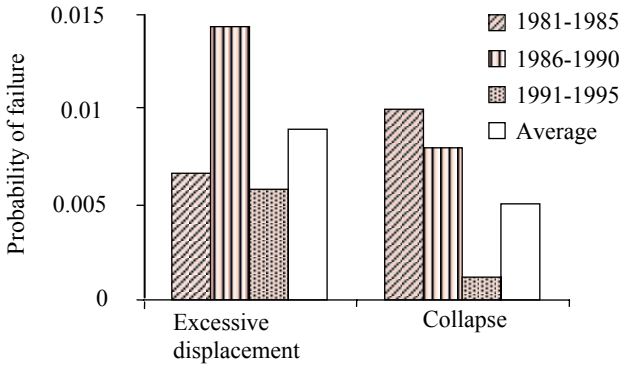

Fig. 11. Failure rates for different time periods

by the Government in the early 1990s to tighten geotechnical control of deep excavations in Hong Kong, as follows:

1 The Buildings Ordinance was amended to require the submission of excavation and lateral support plans for building developments. With this regulation, requirements for qualified supervision can be imposed to enhance site supervision of deep excavations.
2 A Practice Note for APs and RSEs was issued in 1991 to specify the requirements for excavation and lateral support plans.
3 Administrative measures were introduced by the GEO in 1992 to tighten geotechnical control of deep excavations.

The reduction in the occurrence of the failures since 1990 may imply the measures introduced by the GEO have had a significant effect on the quality of private excavation works especially with regard to construction control.

The observed failures have been correlated with other effects. Depth of excavation correlates reasonably with hazard level as shown below in Table 1. There is no clear correlation between hazard level and soil type however.

Table 1. Results from incident data for collapses

Depth	Observed number of collapses			
	Small	Medium	Large	Total
< 10 m	4	3	0	7
> 10 m	0	1	1	2

4.4 *Conclusions and recommendations of the study*

The quantitative risk assessment of fatalities arising from deep excavations in Hong Kong led to the following conclusions and recommendations:

1 The contribution to the risk is significantly higher for sheet pile walls than for other types of walls. This is mainly due to inadequate penetration due to obstructions and inadequate strutting. While this was largely due to opportunistic construction methods by the contractor, in many cases the design did not adequately consider the potential construction difficulties both during excavation and permanent works construction.
2 A dominant cause of the observed problems is poor site control. Occasionally, for cases of excessive displacement, the cause is due to inadequate planning for the predicted large displacements.
3 Government control of deep excavations has significantly improved since 1990.
4 Recommended measures to further reduce the risk include:
5 Improved site control by the contractor, the designer and additional random checking by the GEO.
6 Improved planning to ensure adequate allowance is made for construction difficulties and for the predicted settlements. It may be preferable to prepare for the movement rather than take extreme measures to attempt to reduce it.
7 Routine and close monitoring to give pre-warning of a potential medium or large-scale collapse. The potential for loss of life will be dramatically reduced if there are some prior warnings of impending collapses.

5 'SHORT PILES'

In the late 1990s and early 2000s, the engineering profession in Hong Kong was brought into disrepute by what became known as a series of 'short piling' scandals. These largely occurred at sites where the foundation system had been designed as large diameter cast in-situ concrete bored piles end bearing on rock. There was a series of examples discovered at both private and government construction sites and, in the worst case, the discoveries led to the demolition of two completed residential tower blocks (Chief Executive 2000).

While these cases are not a failure of a geotechnical material, they are certainly a failure of a geotechnical engineering process and deserve further consideration. In accordance with the conventional large diameter bored piling practice in Hong Kong, the piles in the cases of concern were designed to found directly on rock at presumed allowable bearing pressures of 5MPa or 7.5MPa depending on the rock quality.

5.1 *Previous conventional practice*

It was standard practice to carry out a pre-bore at each pile location and determine the minimum depth at which there will be at least 5 m of the required rock quality beneath that depth. The pile would then be excavated to that level, and the depth and cleanliness of the base checked by 'sounding' with a tape prior to installing the reinforcement cage and backfilling with tremie concrete. For private developments, the Buildings Department would finally choose a few piles for full length coring to demonstrate the quality of the concrete in the shaft, the depth of the pile toe and also the interface between the bottom of the concrete and the underlying bedrock.

5.2 *Fraudulent practices*

Superficially the above process appeared to be foolproof. It came to light however that the proof coring process was sometimes manipulated to produce a core sample of concrete and rock of the

desired dimensions and quality rather than that existing within the pile. This is clearly a fraudulent activity and was not anticipated. Further investigation of these projects where 'short piles' were discovered, revealed that other records and processes were also manipulated including false concrete delivery records, false sonic coring records, reinforcement cages with lengths missing and the use of measuring tapes with sections removed somewhere along their length. Unfortunately, it became apparent that once the act of deception was used, it could become widespread and pervasive.

5.3 *Motivation*

It is necessary to understand the background as to how the industry arrived at such a dismal position. In retrospect, it is apparent that the industry could become accustomed to accepting small 'deceptions' when proving the quality of the pile toe material. Often the surface of the rock material is not clearly defined and the excavation may have to extend several metres through quite good quality rock but not sufficiently good to classify as the 5 m thickness required. This is an expensive and time consuming process and some contractors found that a reasonable job could be achieved by constructing the piles onto rock but not necessarily to satisfy the 5 m requirement. Nevertheless, a satisfactory foundation was achieved in an economical and timely manner. Things clearly deteriorated as economic pressures increased and eventually foundations that could not perform adequately were constructed.

5.4 *Actions taken*

Clearly, site practices had to be modified when these events came to light. Resident site staff used their own tapes as a matter of routine in all cases and specifications were changed such that the contractor must include inspection points by the resident site staff at key stages during excavation and concreting. The Buildings Department was pro-active in addressing this major problem and changed the regulations such that it became necessary to carry out interface coring of every pile (Buildings Department 2004). This is achieved by casting a steel tube within the pile that terminates at about 0.5 m above the pile toe. After completion of the pile, a single core run of 1.5 m is taken through the base of this tube to demonstrate both the length of the pile and the quality of the concrete/rock interface.

The interface coring initially showed there were frequently problems at the base of the concrete with inclusions of soil or aggregate. Studies were carried out to determine what thickness of inferior material could be tolerated so as not to cause a problem to the subsequent superstructure (see, for example, the report prepared for the Hong Kong Construction Association by Arup (2001)). It was shown by Arup (op cit) that if good concrete to rock contact exists over a small area, then good performance would be assured. It follows that if any core hole shows a good interface, no further works are required. Calculations also showed that if an edge core (i.e. a core hole within 0.5 m of the edge of the pile) shows a thickness of less than 100 mm or a centre core (i.e. a core hole in the middle third of the pile) shows a thickness less than 50 mm, again the pile performance is considered to be satisfactory. Remedial grouting measures may be required if a greater thickness of poor material was encountered. It is noteworthy that the construction processes have improved in recent years however and interface problems are becoming less common.

6 CONCLUSIONS

There is a wealth of published information on failures in geotechnical engineering in Hong Kong. Some of the 'milestone' failures have had major bearings on the professional practice. It is incumbent that practitioners should seek to heed the lessons learnt in order to avoid recurrence of similar problems.

ACKNOWLEDGEMENTS

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Advances in Environmental Geotechnics in Hong Kong since the 1980s

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Abstract: This paper summarises the advances made in Hong Kong since the 1980s in three branches of environmental geotechnics; namely, solid waste landfills, greening of slopes and contaminated land.

1 INTRODUCTION

Hong Kong was preoccupied with basic infrastructure needs until the 1980's when the economy became sufficiently strong enough to be able to address environmental issues. In the field of environmental geotechnics there have been three main areas of advances since then: solid waste landfills, the greening of slopes and contaminated land.

2 LANDFILLS

The Environmental Protection Department (EPD) decided to upgrade Hong Kong's solid waste landfills to modern fully contained engineered facilities in the 1980's. This required Geotechnical Engineers to assist with the design and construction of liners to prevent the escape of polluting leachate from these landfills. The EPD then decided to rehabilitate the old disused landfills in the 1990's, and this required assistance with the design and construction of caps to prevent the escape of polluting gas. The steep slopes of the valley landfills, and soft ground beneath the coastal landfills, produced some interesting geotechnical engineering challenges.

Hong Kong has a population of nearly 7 million people, producing over 20,000 tonnes of solid waste each day. Whilst a proportion of the waste is incinerated, the majority is disposed into landfills. Located on the southern coast of China, Hong Kong has a terrain which is characterised by steeply sloping valleys in close vicinity to the sea. The solid waste landfills have been formed in valleys and on the coast. The locations of these landfills are shown in Fig. 1.

Currently, waste is being disposed into three specially designed modern landfills (termed strategic landfills), which are located in the Western New Territories (WENT), the Northern New Territories (NENT) and the South Eastern New Territories (SENT). Previously, waste was disposed into thirteen controlled tips, which have recently been restored to make them environmentally secure and to allow better use of the land.

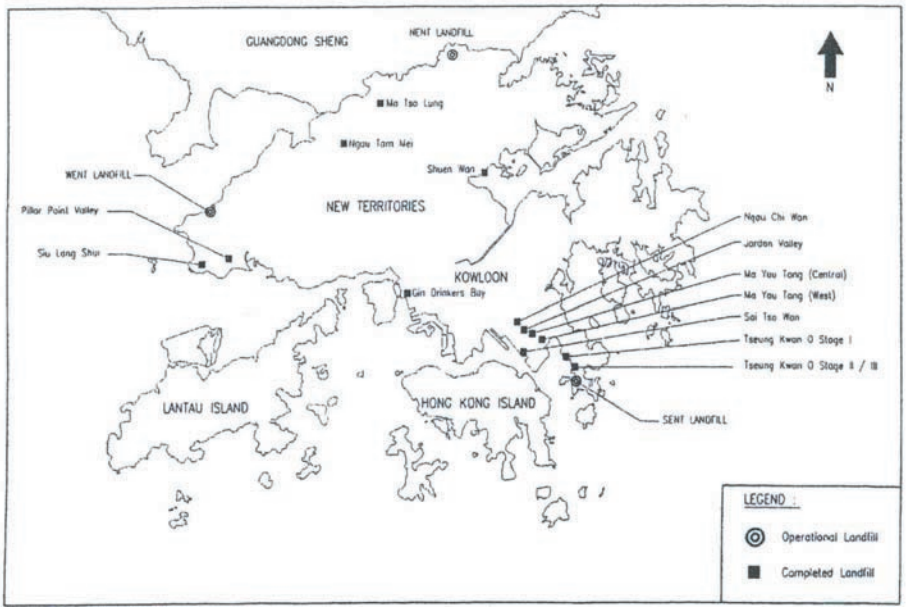

Fig. 1. Landfill Locations in Hong Kong

2.1 Valley and Coastal Landfills

Valleys in Hong Kong are characteristically steep sided with natural slopes comprising weathered rocks and colluvium at around 25° to 40°. In order to increase capacity before landfilling, and to provide daily cover material, soil and rock is excavated from the valley sides. Therefore, a large proportion of the total area of the lining and drainage systems needs to be placed on steep soil slopes and near vertical rock slopes. These valley landfills can accommodate depths of waste between 120 and 140 metres.

In contrast, the coastal landfills are formed on reclamation overlying softer material which limits the depth of waste to around 60 metres.

2.2 Lining and Drainage Systems

With an average annual rainfall of 2.3 metres per year, and a series of surface reservoirs, Hong Kong does not make use of groundwater for drinking water. However, any leakage of leachate from a landfill located on or close to the coast would pollute the sea, where there are a number of fish farms, and in some inland locations there is some use of groundwater for irrigation of crops.

The need for lining systems to contain leachate within the landfills was recognised in the early 1980s, and a series of materials have been used for this purpose. Initially, cement or shotcrete was used to seal the surface of steep slopes and a single geomembrane liner was placed on flatter areas. As the majority of the surfaces to be lined were sloping, a strong emphasis was placed on providing effective leachate drainage systems to conduct leachate away from potential contamination points. These were mainly formed using granular material derived from granite rock quarries.

Environmental assessments and outline designs for the strategic landfills commenced in the mid 1980s. These landfills are very large, with a capacity for 40 to 60 million cubic metres of waste. One aspect carefully considered in the studies for these landfills was the lining systems. It was concluded that a single composite liner could be used, with a continued strong emphasis on providing effective leachate drainage systems and the addition of a groundwater drain beneath the liner.

As Hong Kong does not use groundwater for drinking water, it was decided that a leachate detection zone, between a double liner, was not necessary. Instead, it was recognised that the drainage of leachate to a suitable treatment location, utilising the sloping ground, was just as important as leachate containment. In addition, with a sloping site it would always be possible to construct downstream remedial measures at a later date if any leachate was found to be leaking from the site. The provision of a single liner also had the advantage of focusing attention on the proper construction of that liner.

2.3 Liner Materials

A number of materials were considered for lining the strategic landfills. These included clay, bentonite amended soils, asphaltic concrete, shotcrete (on the very steep slopes) and geosynthetics.

The only clay available in Hong Kong is marine clay, deposited along the coast from the mouth of the Pearl River. Practical problems in drying out this marine clay, and potential slope stability problems when placed, precluded its use.

After a successful trial (Cowland and Leung, 1991), a bentonite amended insitu weathered soil was used for the liner for one landfill cell. However, it was found that a significant mixing and placing operation was needed, which was difficult to carry out and control during Hong Kong's frequent heavy rainstorms. In addition, there was some concern over possible dessication cracking if the finished liner became dry.

Subsequently, the use of geosynthetic clay liners (GCLs) solved the problem of providing the clay component of the single liner. GCLs are made in a factory, where quality is much easier to control than on site, and they can be deployed very quickly in between most major rainstorms.

Asphaltic concrete was studied in detail. Different mix designs were considered, and leachate compatibility tests were carried out (Overmann et al, 1993). Durability concerns and the practical aspects of construction in comparison with the ease of placing geosynthetics, particularly in the light of the experiences with the bentonite amended soil liner, influenced the decision not to use this material.

Shotcrete is very commonly used in Hong Kong for the protection and stabilisation of steep slopes. It was found to be a satisfactory material in the leachate compatibility tests, and different mix designs were considered. However, from an environmental viewpoint it is an unconventional material, and there was also some concern about cracking. Nevertheless, in order to maximise the capacity of the landfills, most of the slopes are steepened and shotcrete is applied to these slopes, where it is considered as an additional bonus component of the lining system.

2.4 Geomembranes

In common with landfill lining practice elsewhere, geomembranes were chosen to form the top layer of the single composite liner. Different polymer types were considered, including polyvinyl chloride (PVC) and reinforced chlorosulphonated polyethylene (CSPE), but in the light of economics and the leachate compatibility tests (Overmann et al, 1993) the now common choice of high density polyethylene (HDPE) was selected.

At the time, there was a concern over stress cracking of HDPE geomembranes, which was solved by selecting the correct resin, and welding defects, which was solved by the development of automatic twin track welding machines. It was also thought that there might be practical difficulties with placing geomembranes on very steep slopes, but resourceful installers solved these problems with ease (Cowland and Overmann, 1994).

2.5 Drainage Materials

Both granular and geosynthetic materials were considered for forming drainage layers in the strategic landfills. Crushed granite rock had been used for some time in the controlled tips, and was readily available from local quarries. It had the advantage of being relatively unaffected by any of the chemicals contained in leachate, and was strong enough to sustain high loads from very great depths of waste.

It is planned to place up to 140 metres depth of waste in the strategic landfills, and with a unit weight of about 1.4 tonnes/cubic metre (Cowland et al, 1993), this will exert a pressure of nearly 2 MPa on parts of the base of the landfills. After consideration of transmissivity under load, and long term

reliabilty, it was decided to use granular materials for drainage layers under the greatest depths of waste, and then under progressively smaller depths of waste to use three strand (triaxial) geonets, two strand (biaxial) geonets and then non-woven geotextiles with intermittent geonet strips.

This change to lighter geosynthetic materials under decreasing loads has been used more often for the groundwater drain than for the leachate drain. Geonets have been installed for drainage layers in areas where it is planned to place up to 100 metres depth of waste, and non-woven geotextiles with intermittent geonet strips have been installed in areas where it is planned to place up to 50 metres of waste.

2.6 Protection Materials

In order to protect the liner from the possibility of being punctured by granular drainage layers or the waste, protection layers have been installed on either side of the liner. Typically, these have been non-woven geotextiles with a weight of 600 g/m^2. Due to the very large depths of waste, deformation testing at unusually high pressures has been performed for these landfills (Frobel et al, 1998).

2.7 Current Lining and Drainage Systems

A typical arrangement for a slope lining and drainage system is shown in Fig. 2. In view of slope stability concerns, the geomembrane is usually textured on one side, which is placed in contact with the underlying geosynthetic clay liner. The design intention is that the upper smooth side of the geomembrane, in contact with the protection geotextile, will form a potential slip plane to allow movement to occur without affecting the integrity of the liner. In some instances, a geomembrane with smooth surfaces on both sides has been installed to allow more movement to take place.

On slopes steeper than 45°, especially the near vertical rock slopes, it is assumed that the preferential flow path of the leachate will be down the leachate drainage layer rather than sideways through the liner, and the geosynthetic clay liner has been omitted, leaving a single geomembrane layer. In the situation of a coastal landfill where the ground underlying the basal liner is softer than usual, a composite liner comprising a GCL encapsulated within two geomembranes has been adopted for extra containment protection.

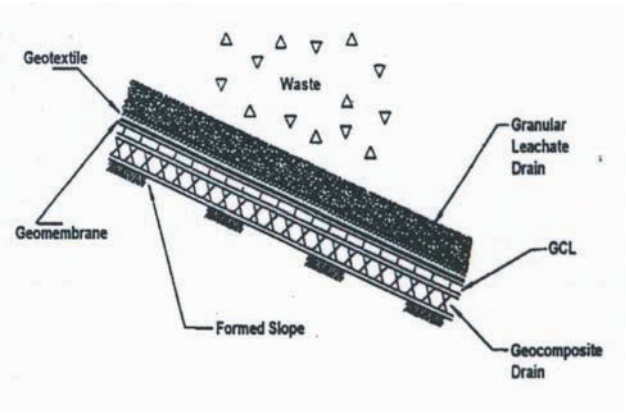

Fig. 2. Typical Hong Kong Lining System

2.8 Capping Systems

When they ceased to be used, the controlled tips were capped with a substantial soil layer of at least 1 metre thickness. Typically, the tips were formed into a flat topped dome, with side slopes of around 1:3. Studies and designs carried out for the restoration of these tips determined that geosynthetics should be used for rainfall and gas barriers and drainage layers, and for erosion control measures.

As active gas extraction systems were proposed, and for reasons of economy, it was decided that a single geosynthetic membrane layer would be sufficient to form a rainfall and gas barrier. After considering the characteristics of geomembranes and geosynthetic clay liners (GCLs), it was decided that this barrier layer should be formed with a very flexible polyethylene (VFPE) geomembrane. This type of geomembrane was required to accommodate the large amount of settlement, many metres, expected on the surface of the large depths of waste.

With the significant wetting and drying that occurs in Hong Kong's alternating wet and dry seasons, it was decided that there was a possibility of dessication cracking occurring in GCLs, which might allow landfill gas to escape. In addition, with heavy rainstorms in the wet season, it was possible that GCLs could become hydrated, which might lead to slope stability problems. It should be noted, however, that in the absence of economic constraints the ideal choice of barrier layer would have been a composite liner with a flexible geomembrane placed over a GCL.

With significant wet season rainstorms, the provision of adequate drainage measures for water infiltrating into the soil layer above the impermeable capping membrane is paramount. It was decided that a geocomposite drainage layer should be placed immediately above the geomembrane. It was also found that there should be frequent exit points to conduct the water out of this drainage layer to prevent water pressures from building up under the soil.

As it was planned to install active gas extraction systems, it was decided that it was not necessary to install a gas collection layer under the geomembrane. A landfill cap arrangement typical of those installed is shown in Fig. 3.

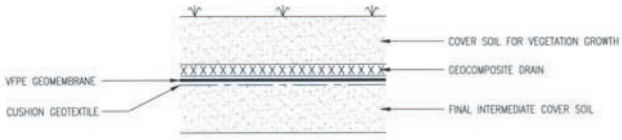

Fig. 3. Typical Hong Kong Capping System.

2.9 Erosion Control Measures

In order to grow vegetation, it was decided to place a soil layer over the geosynthetic capping materials. To avoid erosion of this soil layer in heavy rainstorms, a network of flexible surface water drainage measures are necessary. Erosion control mats, geopipes and geocellular materials have all been used successfully.

2.10 Stability Considerations for Geosynthetics on Landfill Slopes

Placing liners and drainage material on slopes of 40° and steeper, and capping material on slopes of around 20°, has given rise to some interesting design and testing considerations. With relatively low shear strengths expected through the bentonite layer of the geosynthetic clay liners and along the interfaces of different geosynthetic layers, it has been important to accurately determine the relevant shear strengths and to design adequate stability measures.

2.11 Shear Testing

A number of large scale shear testing programmes have been carried out for each range of materials and landfills. Testing has been carried out for individual interfaces, and GCLs, and also for the entire geosynthetic sandwich of lining and drainage materials. The effect of deformations, caused by the roughness of the surface being lined and the pressure applied by the waste, on the shear strength of interfaces has also been investigated (Frobel et al, 1998).

Considerable attention has been paid to determining the relevant shear strength through the mid-plane of geosynthetic clay liners. The GCLs used in Hong Kong's landfills have comprised a layer of bentonite powder, or pellets, encased inside two geotextile layers. If this bentonite were to become hydrated, then it could act as a layer of soft clay which could potentially form a slip plane on the slope. However, if the liner is carefully designed and constructed, the extent of hydration should be limited. In addition, the stitching, or needle punching, connecting the two geotextile layers together will help to strengthen the GCL, unless movement occurs which ruptures these bonds.

Testing programmes have been carried out to determine the effect of varying the degree of hydration, varying the applied pressures and varying the strain rates (Cowland, 1997 and Frobel et al, 1998).

2.12 Construction Details

Although it would appear that lining materials with a low interface shear strength would not be stable on a steep slope, Giroud and Beech (1989) have shown that the placing of waste in the landfill can be arranged in a manner that will buttress the slope to prevent failure from occurring (Fig. 4).

In the landfill cap, in order to prevent the soil overlying the geomembrane barrier from sliding due to saturation in a rainstorm, it is necessary to place a geocomposite drainage layer over the geomembrane. It is important to ensure that there is adequate exit drainage capacity for water to escape from this drainage layer, and also that there are no blockages in the drainage layer, so that water pressures do not build up under the soil.

2.13 Reinforced Fill for Increased Landfill Capacity

The most recent development in Hong Kong's landfills is to use geosynthetic reinforced fill to construct large bunds to retain waste around the landfill perimeter to substantially increase the capacity of the landfills. Using material excavated from inside the landfill, which increases the void space, and placing it in bunds reinforced with geogrids around the perimeter to increase the height of the sides can create considerably more capacity for waste deposition. The largest geogrid reinforced fill bund constructed for this purpose to date is 30 metres high and 300 metres long.

2.14 Leachate Ponds

A number of leachate retention and treatment ponds have been lined with HDPE geomembranes in Hong Kong. Careful detailing was found to be necessary for collars to pipes and joints to concrete surfaces. There is also a need to ensure that subsequent leachate treatment operations do not damage the liner, although, unlike the situation with a landfill liner, it is relatively easy to empty the pond and carry out repairs.

Geogrid reinforced fill bunds are also being used to construct the sides of new leachate ponds.

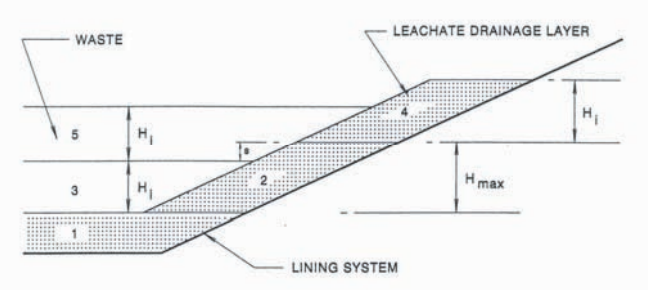

Fig. 4. Incremental Placing of Load (After Giroud and Beech)

3 GREEN SLOPES

The Geotechnical Engineering Office decided to provide a greener vegetated surface to Hong Kong's slopes in the early 2000's to improve the quality of the environment. Due to the steepness of these slopes and their proximity to buildings and roads, combined with Hong Kong's marked wet and dry seasons, this slope greening programme has also produced some interesting geotechnical challenges.

With 7 million people living in a hilly area of only 1,000 square kilometres, and heavy wet season rainfall, the need to mitigate slope hazards is very important in Hong Kong. Infiltration of rainwater is one of the main causes of slope failure, and this can be minimised with good surface protection.

A number of solutions are available to grow vegetation on steep slopes. These systems include erosion control mats, soil vegetation containers, gabions, geocells, coir geoblankets as well as reinforced soil and other hybrid systems. The use of these systems is still in its infancy. Challenges include ensuring stability on steep slopes and providing a system that will remain green in the dry season.

Erosion control mats were originally developed for relatively gentle slopes, and adapting them to prevent erosion on steep slopes at the same time as filling them with adequate soil for vegetation growth is proving to be a very interesting branch of environmental geotechnics. Testing procedures are being developed for the use of these systems on steep slopes, as well as their durability and their ability to encourage seed germination and plant growth.

4 CONTAMINATED LAND

The EPD decided to start to tackle the problem of contaminated land in the late 1980's. Recently this has become a more concentrated effort and advances are beginning to be made, again providing Geotechnical Engineers with some interesting challenges. The early adoption of pollution control criteria from the Netherlands, known as the Dutch 'B' levels, has recently been under study by the EPD and has led to involvement of stakeholders as part of the review. The aim is to develop a new set of criteria more closely linked to risk assessment and in-line with international practice.

4.1 Background

Whilst ground contamination has been an issue both in the developed and the underdeveloped world for decades, a concerted effort to 'clean-up' the environment has only been in progress reasonably recently. It is therefore perhaps not surprising that the definition of contaminated land is relatively new. Before embarking on a review of contaminated land in Hong Kong it is therefore appropriate to consider what is actually meant by contaminated land.

During the 1990's a concerted effort was being made in Europe (not least due to the European Directives) to get to grips with the liability of contaminated land that had been left from a century before during the industrial revolution. During the period that followed, several definitions were proposed and these are summarised in Table 1.

What becomes clear is that contaminated land is land that may be harmful (particularly to humans) as a result of human activity. This differs from derelict land which may have been subject to previous use but is not necessarily harmful to man – for example, a disused quarry.

4.2 Historical Events

In order to understand how contaminated land may have arisen in Hong Kong, it is worthwhile considering what was happening in the 1970's. Glaser et al (1991) identified the 1970's as being a period when large scale industrial, housing and infrastructure projects were advancing with the development of technology. This included the commencement of the MTRC projects in 1975. Reclamation from the sea by cut and fill was a major method of land development. In the 1970's the emphasis was on development on newly formed land with little emphasis on reuse of existing land.

During the 1980's a series of seemingly disconnected events resulted in significant changes in land use in Hong Kong, which kick-started the need to look at contaminated land. Following the frenetic pace of development in the 1970's and early 80's, the sudden downturn in the economy by the mid-80's resulted in a significant reduction in construction activity. Changes were also taking place in China and the move of Hong Kong's industrial base northwards was accelerating. At the same time, in 1989 it was decided to construct a new airport on the previously undeveloped Lantau Island.

With the commencement of the Airport Core Programme in earnest in the 90's, there was a shift of development away from the already congested areas of Hong Kong Island and the former industrial heartlands in East Kowloon and elsewhere.

One of the early industries to be affected by the move north was ship building, with industry finding a simple and cheaper alternative to production at readily accessible locations up the Pearl River. The Hong Kong locations of these older light industries were abandoned. Fig. 5 shows a typical light industrial site subject which was subject to a contamination survey with localised heavy metals and oil spills revealed.

As the economy regained strength, property development again took a foothold. This time around it was not confined to new reclamation sites and ribbon developments along the new infrastructure corridors. There were also the abandoned sites of shipyards which were beginning to look attractive in terms of their development potential and this in turn led to the development of EPD guidelines. There are a number of case studies of the clean-up of shipyard sites, both in Hong Kong and in the outlying islands (eg. Chiu, 2006).

Table 1. Comparison of Contaminated Land Definitions

Source	Definition of Contaminated Land
The UK Environment Act (1995)	*"Any land which appears to the local authority in whose area it is situated to be in such a condition, by reason of substances in, on or under the land that:* *(i) significant harm is being caused or there is a significant possibility of such harm being caused* *(ii) pollution of controlled water is being or is likely to be caused."*
EU Directive Article 2 (13), 2002	*"Land Contamination means the direct or indirect introduction as a result of human activity, of substances, preparations or radiations, into soil and subsoil, which may be harmful to human health or natural resources, including biodiversity"*
NATO in their Committee on Challenges to Modern Society	*"Land that contains substances which, when present in sufficient quantities or concentrations, are likely to cause harm, directly or indirectly, to man, to the environment, or on occasion to other targets."*
The Australian and New Zealand Environment and Conservation Council (ANZECC), 1992	"*A site at which hazardous substances occur in soil or groundwater above background levels and ... where assessment indicates it poses, or has the potential to pose, an unacceptable risk to human health or the environment*"

Fig. 5. Typical industrial unit subject to contamination survey

4.3 Guidelines on Contaminated Land

In 1977 the government created the Environmental Protection Unit to tackle pollution. This was subsequently replaced in 1981 by the Environmental Protection Agency, which in turn was replaced in 1986 by the Environmental Protection Department (EPD) to co-ordinate and carry out pollution prevention and control activities. Since April 2005, the EPD has taken on the additional role of Permanent Secretary for the Environment and Director of Environmental Protection. This significant development has placed the EPD in the position of both determining and implementing environmental policy.

In the 1980's the EPD commenced a programme of environmental review and assessment, and in 1986 they issued a Government Circular on the Environmental Review of major public sector development projects. Whilst key areas of concern were air, water, pollution and waste, project proponents were also grappling with the need to assess the old derelict sites.

The Professional Persons Environmental Consultative Committee (ProPECC) was set up in 1991 to provide a forum for the exchange of views on all environmental matters related to the duties and practice of architects, engineers, planners, surveyors and developers. There are currently 20 such notes covering a wide range of topics related to the environment.

In 1994, EPD published a ProPECC Practice Note PN 3/94 – Contaminated Land Assessment and Remediation. The Note sets out the framework for the assessment and remediation process. It also provides an indicative index for interpretation of assessment results. With the enactment of the Environmental Ordinance came further guidance. In 1999, EPD produced the Guidance Notes for Investigation and Remediation of Contaminated Sites of Petrol Filling Stations; Boatyards; and Car Repair/Dismantling Workshops.

Two key issues have remained high on the agenda in the redevelopment of contaminated sites. Whilst removal of contamination to landfill was seen at the time as a convenient method of clean-up, landfills were noted to be a scarce resource and the guidance emphasizes that this method of treating contaminated land should be a last resort and only used when the quantities involved are small. Secondly, the guidance note references the 'Dutch B-List' of the Netherlands Ministry of Housing, Planning and Environmental Soil and Groundwater Standards, which was the basis for cleaning contaminated land in the Netherlands and which had been widely adopted elsewhere, particularly in Europe. At the time it was the main reference used in Europe to determine whether a site is contaminated, although it was based on a rigorous assessment related to re-use of groundwater and therefore represents a stringent clean-up set of criteria.

The Dutch B-List (later replaced by the new list) has been recognized for some time to be out-dated and also less relevant to Hong Kong conditions where groundwater is not abstracted as potable water. The EPD has therefore been working on a strategy to use risk-based goals (RBG) for assessment of contaminated land. This study is currently subject to a period of public consultation. It includes a list of 54 chemicals of concern (CoC's) that will replace the former Dutch B-List (46 chemicals) to determine the need for remediation. As the existing contaminated land management practice in Hong Kong will remain unchanged, the RBG list will provide a framework for use in determining remediation methodology as a simple replacement of the Dutch B-List.

Other current legislation related to contamination that may affect contaminated land is as follows:

- Air Pollution Control Ordinance (Cap.311)
- Noise Control Ordinance (Cap.400)
- Waste Disposal Ordinance (Cap.354)
- Water Pollution Control Ordinance (Cap.358)
- Air Pollution Control Ordinance (Cap.311)
- Ozone Layer Protection Ordinance (Cap.403)
- Dumping at Sea Ordinance (Cap.466)
- Environmental Impact Assessment Ordinance (Cap.499)

4.4 Overview of Case Studies

The concept of 'polluter pays' is a principle that has driven development of brownfield sites throughout Europe. For example, the shortage of greenfield sites in Britain, combined with a significant housing shortfall, has led to many modern residential developments on brownfield sites. Unlike Europe, in Hong Kong the land belongs to the Government and is subject only to lease by the purchaser. Thus, the polluter pays principle may be difficult to implement in Hong Kong, leaving the Government with a potentially large liability. Nonetheless, it is worthwhile to illustrate how clean-up has progressed since the 1970's by reference to two selected case studies.

4.5 Shipyard Sites

The redevelopment of shipyard sites is driven by their commercial value, being on the waterfront and relatively close to the main urban conurbation. As a consequence, it was the redevelopment of the shipyards that became an important benchmark in contaminated land in Hong Kong and the 1990's saw several such developments. With a comprehensive set of guidelines, professionals were able to address the main areas of concern. Whilst, in many senses, shipyards were not highly polluting, they posed particular contamination potential that required careful scrutiny. Contaminated land site investigation typically involved historical desk studies followed by trial pit and borehole investigations, with sampling carried out in accordance with the ProPECC guidance notes, and with the preparation of a contamination assessment plan (CAP) followed by a remedial action plan (RAP).

By the late 1990's, EPD had promulgated a formal procedure of Environmental Impact Assessments (EIA) under the Environmental Protection Act. Whilst shipyards were generally not considered to be highly polluting, the assessment of contaminated land is particularly prone to discovery (ie. the more you look the more you will find). This was brought to light in a very public manner during the development of Hong Kong Disneyland.

At the time of the land procurement for Hong Kong Disneyland and the first reclamation contract, the Penny's Bay site had become one of the early major projects to be subject to the new environmental procedures under the EIAO. The shipyard at the head of the bay had been disused for many years (ship building having moved to China) and the site was in a derelict state. Fig. 6 is a picture of Penny's Bay looking North prior to commencement of decontamination of the shipyard.

There followed comprehensive surveys on existing and potentially harmful effects of contamination of the shipyard site. The results of the surveys are on public record and resulted in an expensive clean-up operation, which included off-site thermal desorption of dioxin soil. It is noted that other shipyard sites

have been remediated by a variety of methods including on-site treatment and disposal to landfill - Chiu et al. (2006).

Fig. 6. Choy Lee Shipyard prior to clean-up after commencement of reclamation

4.6 Kai Tak Airport Re-development

Following the opening of the new airport at Chek Lap Kok in 1998, the old airport at Kai Tak has been partially cleaned-up. The first phase of the remediation involved dealing with fuel spills below the runway pavements. Subsequently, remediation was undertaken including soil vapour extraction, air sparging and biopiles to remove oil and fuel spills, particularly in the north runway area.

The future development of Kai Tak was reviewed following legislation regarding the minimization of reclamation within Victoria harbour. A major study was awarded in 2002 to examine possible reclamation and remediation of the approach channel to the east of the former runways. The approach channel carried drainage run-off from the surrounding urban area that had resulted in decades of pollution. The quality of the harbour water had been dealt with by implementing a system to flush the nullahs that lead into the harbour. Nonetheless, there remained a build-up from the 1950's in the form of sediments accumulated in the base of the approach channel. The investigation (Fig. 7) in 2002 revealed that whilst the majority of the contamination is not significant to future development, some ground treatment may be necessary. One option was to dredge, treat and replace – an expensive option. A cheaper option is to use the geocontainers proposed for the Wan Chai reclamation scheme (Cowland and Lawson, 2006).

4.7 Rivers

Some rivers have been successfully cleaned up – eg. the Shing Mun river, which is a 7km long, 200m wide channel running through Shatin to the Tolo Harbour. This was remediated in 2002 including lining, dredging and bio-remediation (see web reference).

4.8 Petrol Stations

Another significant area of work on contaminated land, commencing in the 1990's, has been the clean-up of petrol stations. Leakage of stored fuel from underground tanks has long been a problem in petrol stations worldwide and this is currently managed through the EIA procedures (Chung et al., 2005).

Fig. 7. View looking South-east in Kai-Tak Approach Channel at time of ground investigation (jack-up rig in distance)

5 CONCLUSIONS

5.1 Landfills

Considerable advances have been made with the environmental geotechnics aspects of Hong Kong's solid waste landfills in the last 30 years. Modern containment liners and caps have been introduced to prevent the escape of environmentally harmful leachate and gas. Placing these liners and drainage systems on steep slopes presented some interesting engineering challenges, which have largely been solved.

The most recent development is to use geosynthetic reinforced fill to increase the capacity of the operating landfills.

5.2 Green Slopes

The quality of the environment is being improved by providing a green vegetated surface to Hong Kong's slopes. A number of solutions have been adopted and these are currently being evaluated. Challenges include ensuring stability on steep slopes and providing a system that will remain green in the dry season. Testing procedures are being developed for the use of these systems on steep slopes, as well as their durability and their ability to encourage seed germination and plant growth.

5.3 Contaminated Land

The liability of the future management of contaminated land in Hong Kong is a matter for legal clarification as it is linked to the wording in a particular land grant or lease. Irrespective of the liability, the future is likely to include redevelopment of 'old sites'. This work is already in motion under the Urban Renewal Authority (URA) who is responsible for improving the standard of housing and the built environment by undertaking, encouraging, promoting and facilitating urban renewal. Whilst the majority of renewal is concentrated on improving the standard of housing and layout of the built environment, it is likely that the

URA will need to address contaminated land issues once it moves into the previous industrial areas.

Hong Kong has seen a major shift in the assessment of contaminated land, particularly over the last decade. The enactment of the Environmental Impact Assessment Ordinance has prescribed that contamination studies are carried out systematically by the project proponent to satisfy the current standards with a view to providing a site clear of hazards. This is to be undertaken in a way that is least damaging to the overall environment so, for example, the dumping of contaminated material to landfill is an approach of last resort. The standards themselves are currently under review with an intention to replace the Dutch B-List with Risk Based Goals (RBG's) – the Dutch list being recognized as over-simplistic and conservative for Hong Kong's requirements. It is understood that the RBG's will be adopted as a set of rules for Hong Kong. The current procedures will remain.

Over the last thirty years, as the development of Hong Kong has matured, contamination has been revealed in some well known sites. However, Hong Kong's relatively recent development suggests it is unlikely that contamination will be as severe as some other parts of the world where land use has varied over many hundreds of years. Legislation in place under the Environmental Ordinance will assist with adopting appropriate technology for clean-up of those sites revealed to be contaminated.

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Use of Time Domain Reflectometry to check the quality of Steel Soil Nails with Pre-installed Wires

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Abstract: Since 2004, the Geotechnical Engineering Office has implemented a pilot quality assurance programme using time domain reflectometry (TDR) technique to check the quality of constructed steel soil nails with pre-installed wires at slopes upgraded under the Landslip Preventive Measures Programme. To date, more than 8,500 soil nails in about 650 sites have been tested using TDR to identify irregularities such that timely follow-up actions can be taken. This paper gives an overview of the use of TDR technique in assessing the quality of steel soil nails with pre-installed wires. It also describes cases where TDR has identified anomalies and the corresponding follow-up actions.

1 INTRODUCTION

Soil nails have been widely used in Hong Kong for slope improvement works since the mid-1990s. It is an insitu soil reinforcement technique by grouting steel bars in pre-drilled holes in full length. Similar to other buried works, it is difficult to verify the as-built length as well as the integrity of grout sleeve of an installed soil nail. To enhance the quality control of soil nailing works, the Geotechnical Engineering Office (GEO) of the Civil Engineering and Development Department (CEDD) strengthens site supervision and introduces an independent site audit on soil nail construction works, and in 2001 began to identify and develop non-destructive testing (NDT) methods that could be carried out on installed nails. The non-destructive tests are not to replace good supervision at the time of construction. They are to provide additional assurance and a deterrent against mal-practices. A number of techniques were tried and a few were found to have potential for field application (Cheung, 2003; Cheung & Lo, 2005). Among them, time domain reflectometry (TDR) technique was the simplest and least expensive. In mid 2004, the GEO introduced TDR to its soil nailing works under the Landslip Preventive Measures (LPM) Programme for pilot use during the independent site audit.

2 PRINCIPLE

TDR was first developed in electrical engineering in the 1950s for detection of faults in transmission lines. The technique has recently been extended to some geotechnical applications such as detection of slip plane and groundwater level in slopes, and determination of soil moisture content (O'Connor & Dowding, 1999; Siddiqui et al, 2000). In principle, TDR involves sending electrical pulses along a transmission line, which is in the form of coaxial or twin-conductor configuration, and receiving reflections or echoes induced by any discontinuities or mismatches in electrical properties in the transmission line. By determining the time for the pulses to travel from the pulse generator to the point of discontinuity or mismatch, the corresponding distance can be estimated using Equation (1) if the pulse propagation velocity, v_p, is known.

$$L = v_p t \quad \text{.....(1)}$$

where L is the distance between the pulse generator and the point of mismatch/discontinuity, and t is the respective pulse travel time.

The pulse propagation velocity, v_p, is related to the electrical properties of the material in the close proximity to the pair of conductors by the following expression (Topp et al, 1980):

$$v_p = \frac{v_c}{\sqrt{\varepsilon}} \quad \text{.....(2)}$$

where v_c is the speed of light in vacuum (3×10^8 m/s) and ε is the dielectric constant which measures how a material reacts under a steady-state electric field (for air $\varepsilon \approx 1$, and for cement grout $\varepsilon \approx 10$).

If a wire is pre-installed alongside a soil nail reinforcement, which is generally a steel bar, as shown in Fig. 1, the configuration becomes analogous to a twin-conductor transmission line and the end of the reinforcement-wire pair becomes a discontinuity or mismatch. This suggests that TDR can be used to determine the length of installed steel soil nails (Cheung, 2003).

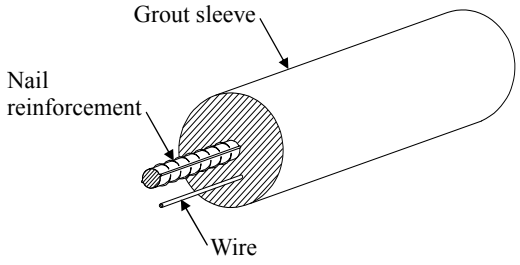


Fig. 1. Analogy of a nail with pre-installed wire as a twin-conductor transmission line

As indicated in Equation (1), the two key parameters that have to be known for the estimation of nail length are (i) the time for a pulse to travel from the nail reinforcement head to its end, t, and (ii) the pulse propagation velocity, v_p. Equation (2) further suggests that the pulse propagation velocity, v_p, along a reinforcement-wire pair surrounded by air will be much faster (2 to 3 times) than that along the pair embedded in cement grout. Hence, the pulse travel time along a nail with void in grout sleeve will be smaller than that for a fully grouted nail of the same length.

Apart from different pulse propagation velocities along the reinforcement-wire pair embedded in different materials, a reflection will be induced when an electrical pulse reaches the

location of discontinuity or mismatch in electrical impedance of the reinforcement-wire pair (e.g. a void or the end of a soil nail). The magnitude and polarity of the reflection depend on the amount of changes in electrical impedance at the location of discontinuity and it can be expressed in terms of the reflection coefficient, Γ(Hewlett Packard, 1998):

$$\Gamma = \frac{V_r}{V_i} = \frac{Z - Z_o}{Z + Z_o} \quad \text{..... (3)}$$

where V_r is the peak voltage of the reflected pulse
V_i is the peak voltage of the incident pulse
Z is the electrical impedance at the point of reflection
Z_o is the characteristic electrical impedance of the reinforcement-wire pair

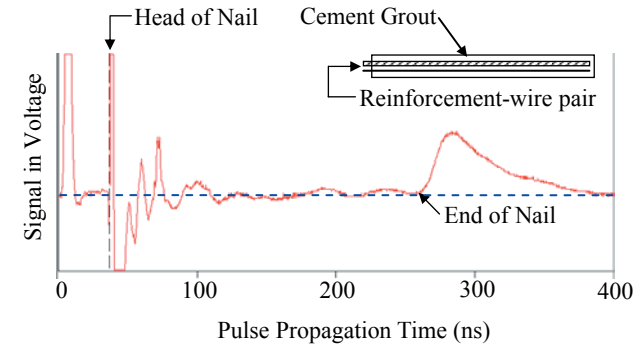
As indicated in Equation (3), when a pulse reaches the end of reinforcement-wire pair, a positive Γ (i.e. positive pulse reflection) will occur since Z_o(reinforcement-wire pair) < Z(nail end). However, if the wire is in electrical contact with the reinforcement at the end of a nail, Z(nail end) will tend to 0 and Γ becomes negative (i.e. negative pulse reflection). Fig. 2(a) shows a typical TDR waveform of a 12 m fully grouted reinforcement-wire pair. When the pair of the same length is enclosed by grout sleeve with void sections, as indicated in Fig. 2(b), there will be reflections at the location of the voids as well as the end of the pair. Moreover, the pulse travel time is smaller than that of the fully grouted pair.

3 OVERVIEW OF THE PILOT QUALITY ASSURANCE PROGRAMME

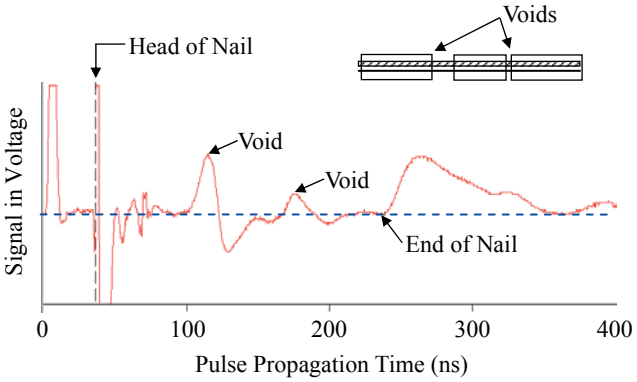
In 2004 GEO implemented a pilot quality assurance programme using TDR technique as part of the independent site audit to check the quality of soil nails with pre-installed wires in LPM sites. The objective of the programme is to experiment the use of TDR to supplement the site supervision in the quality control of soil nailing works. The use of TDR serves as a deterrent to sub-standard workmanship as well as a screening tool for nails showing sign of discrepancy to the norm. The trial also allows experience to be gained with application of the technique so that it could be further improved. Under the programme, 2% of the soil nails with a minimum of 5 are selected from each LPM site for audit on the length of installed soil nails. Apart from the TDR test on the calibration nails whose length needs to be known for the determination of the pulse propagation velocity, the tests on other nails are conducted and interpreted without prior knowledge of their length.

Guidelines on testing procedure and interpretation of test results using TDR have been standardized to fulfill the requirements of using NDT for quality control of geotechnical works as stipulated in GEO (2004) and to reduce the uncertainties of the test as far as possible. These guidelines can be accessed from the CEDD website at www.cedd.gov.hk.

Uncertainties associated with the TDR test method and the inherent natural variation of quality of soil nails have been identified and assessed (Cheung, 2006), to facilitate the determination of the precision limit of the test method (i.e. test-related uncertainty) and the effect of inherent normal variation of soil nails on the test results (i.e. test-unrelated uncertainty). Cheung (2006) shows that the 95% confidence level of the precision limit of a TDR test, which is not related to the natural variability in nail characteristics, is about ±5%.



(a) Fully grouted nail



(b) Nail with defects in grout sleeve

Fig. 2. TDR waveforms of a fully grouted nail and a nail with defects in grout sleeve

Cheung (2006) reported that the uncertainty due to reinforcement size (both diameter and length), the ground conditions, and the presence of couplers appears to be insignificant when compared with other sources of uncertainty. The 95% confidence level of the overall error in length estimation using TDR is estimated to be about ±9%. This increase in the uncertainty with respect to test-related uncertainty reflects indirectly the possible variability in the characteristics of soil nails within a site and between the sites that have been installed to the current construction practice (e.g. the method of nail installation, grouting, etc). Based on the consideration of the uncertainty of the TDR test method and to balance the number of defective nails to be detected against the number of false alarm, an 'alert limit' was devised at ±15% of the design length such that if the difference between the TDR-deduced length of a soil nail and its design length exceeds the alert limit, that soil nail is considered as anomalous and follow-up action will be initiated. A short TDR-deduced length could be due to either the as-built length of the reinforcement or the pre-installed wire or both is shorter than their corresponding design length and/or there are substantial defects in the grout sleeve. In addition to the deduced length, the TDR waveforms could also provide some telltale signs of certain anomalies. Thus an anomalous test result can be a short TDR-deduced length or a short TDR-deduced length coupled with an anomalous TDR waveform.

4 TEST RESULTS

Between July 2004 and December 2006, over 8,500 soil nails in about 650 LPM sites have been tested using TDR. In general, the percentage difference between the TDR-deduced length and design length of most of the tested soil nails, as shown in Fig. 3, does not exceed the alert limit. There are a small number of nails (less than 1%) with such difference exceeding the alert limit and further investigation was conducted.

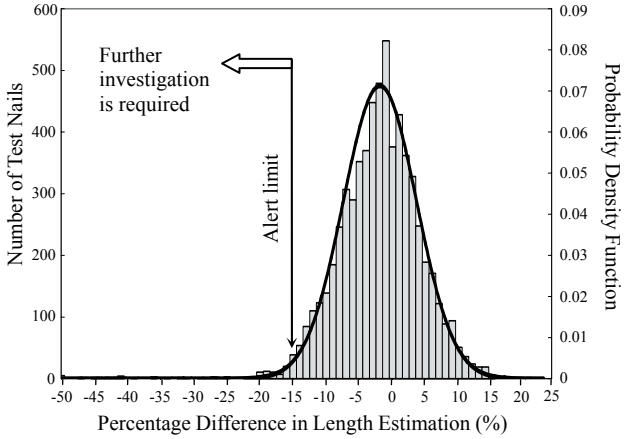


Fig. 3. TDR test results in the pilot quality assurance programme

Most of the soil nails with short TDR-deduced length are isolated cases, i.e. anomaly found in only one or two soil nails in a site and additional TDR tests on adjacent nails do not show any anomaly. The use of other NDT methods or exhumation of nails to confirm the exact cause may not be justified. Generally, a design review would be carried out assuming the anomalous nail to be not fully functional.

The following sections describe two cases where more detailed investigation was conducted to demonstrate how TDR help identify defective nails.

4.1 Case Study

4.1.1 Case A

The subject cut slope is about 10 m high and 115 m long with an average slope angle of 50°. The upgrading works comprised 95 soil nails (65 at the northern end and 30 at the southern end). All the soil nails were 7 m long and without couplers. During the independent site audit the TDR-deduced length of five out of ten test nails were found to be significantly shorter than their design length, and their TDR waveforms were anomalous. Further TDR testing of the remaining nails at the slope revealed two more nails with short TDR-deduced length and anomalous TDR waveforms. The seven nails with anomalous TDR test results, namely A13 to A17, B14 and C13, clustered at the northern end of the slope (see Fig. 4). The waveforms of these seven nails differ from those of the remaining 88 nails in two aspects (e.g. Fig. 5): (i) the presence of significant local reflections between the major reflection from the nail head and that from the nail end, and (ii) shorter pulse propagation time to the end of these nails.

The waveforms of the seven nails (e.g. Fig. 5(b)) bear some resemblance of the characteristics of that with grout defects in Fig. 2(b), suggesting the anomalies in these seven nails could be related to the presence of voids in the grout sleeves. A review of the site records also indicated that significant grout take was encountered at these locations during the construction of the nails.

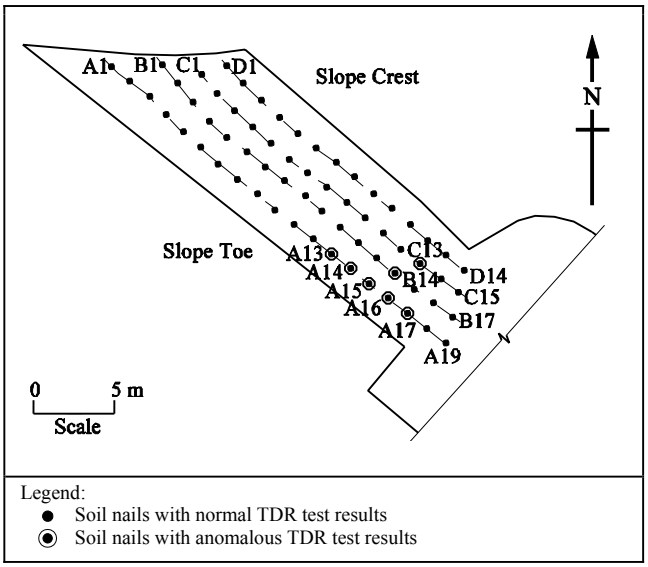


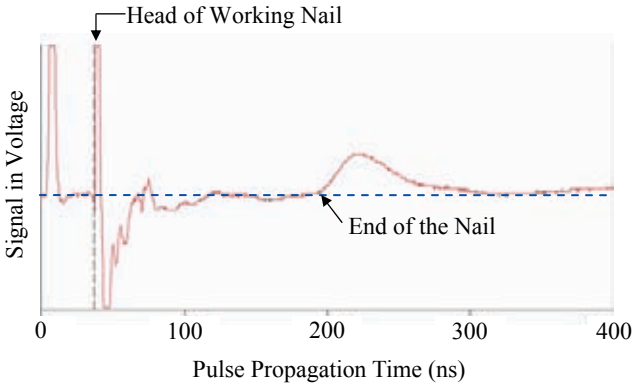
Fig. 4. Layout of nails at the northern portion in Case A

To supplement the TDR tests, another NDT technique, Electrical Resistance Method (ERM), was carried out. This method measures the electrical resistance between a soil nail and a remote electrode. It makes use of the spatial variation of the electrical resistance of soil nails as an indicator for checking the integrity of grout sleeve. If the measured electrical resistance at a nail is found to be significantly different from those of the adjacent nails of the same configuration, there is a high probability that the nail is anomalous. This method does not require measurement involving the pre-installed wire and hence potential anomalies associated with damaged wire can be ruled out. Details of the method can be found in Cheung & Lo (2005). Fig. 6 shows that the nails with high electrical resistance determined using ERM match with those with anomalous TDR results, lending support that the anomalies were related to existence of substantial voids in the seven nails. Due to the sensitivity limitation of the test, ERM is only capable of detecting significant grout defect in a soil nail (Cheung & Lo, 2005). This explains why A17 was not identified as anomalous by the ERM. It also highlights the importance of appreciating the merits and limitations of each NDT when they are used in a complementary manner.

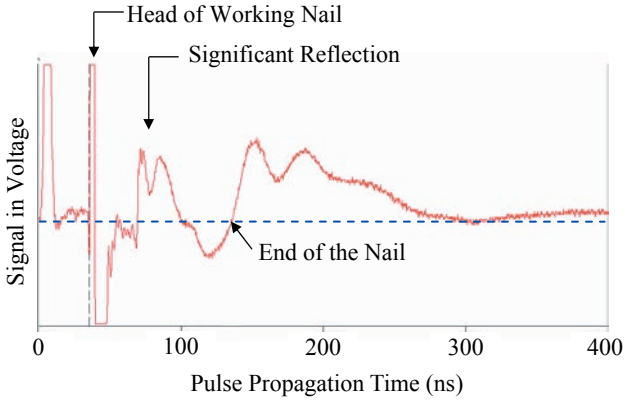
The sizes of voids in the anomalous nails were estimated to be in excess of 2 m. Given the sizes of the void were substantial, it was decided to remove the concrete nail heads for inspection. Upon removal of the concrete nail heads of the seven nails, void sections were noted. The seven nails were replaced and TDR was used to check the grout integrity of the replacement nails both during grouting operation and after installation.

4.1.2 Case B

Case B involves a 6 m high cut slope with an average slope angle of 60°. The upgrading works comprised 77 soil nails of either 5 m or 6 m long installed in three or four rows and 6 raking drains (see Fig. 7). The soil nails were constructed with corrugated plastic sheath. An independent site audit revealed that one of the five soil nails tested by TDR had a deduced length of about 3 m (c.f. design length of 5 m). Additional TDR tests revealed another soil nail, B4, with deduced length of about half the design length of 6 m.



(a) Typical TDR waveform



(b) Anomalous TDR waveform

Fig. 5. Typical and anomalous TDR waveforms in Case A

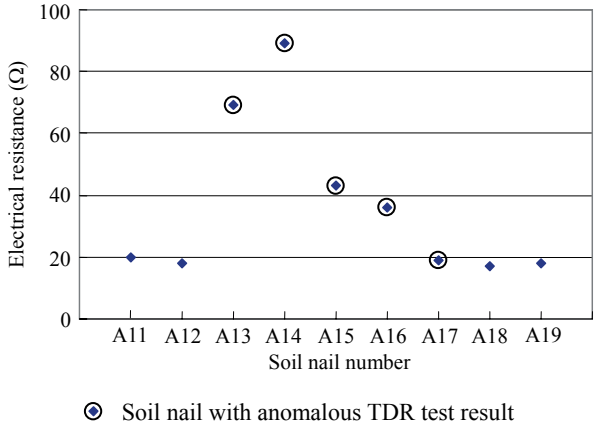


Fig. 6. Variation of electrical resistance along row A nails using ERM

No excessive grout take was noted from construction records. Review of the waveform did not disclose any evidence of grout defects apart from the short travel time of the pulse. The possibility of the anomalies being related to defects in grout integrity was considered small. The site constraints and the insulation effect of the corrugated plastic sheaths precluded the use of other NDTs, such as ERM, to supplement the TDR test; consequently the two nails were exhumed to ascertain the cause of anomalies (see Fig. 8).

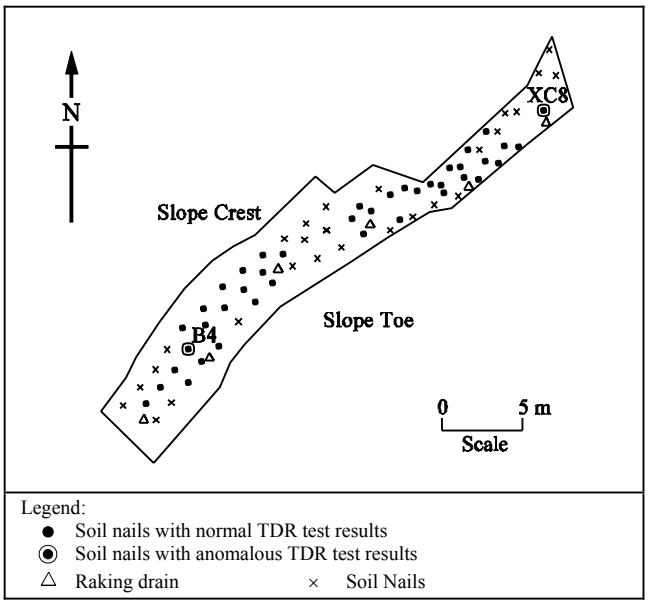


Fig. 7. Layout of nails in Case B

Stitch drilling around the nails concerned was carried out to minimise damage to the nails during drilling as far as possible. The grout sleeves of XC8 and B4 were noted to be intact when exhumed from the ground, except both were found damaged at a location about 3 m from the nail head. The damages, which appeared to be caused by mechanical processes, extended from the grout sleeve outside the corrugated plastic sheath to the reinforcement and the pre-installed TDR wires along the steel reinforcement were severed (see Fig. 9). Rust was noted on the surface of the reinforcement. Since the electrical pulse could only travel from the nail head to the point where the wire severed, the TDR-deduced length matched the distance between the two.

TDR tests were carried out before and after stitch drilling works and yielded same deduced length suggesting that the damage occurred before the stitch drilling works. The amount of rust covering the steel bar suggested that the damaged sections could have been exposed to the moisture in the ground for an appreciable period of time. A review of construction records and drawings (e.g. Fig. 10) indicated that a raking drain was constructed at about 1.5 m below each of the two soil nails XC8 and B4. The horizontal clearance between the soil nail and raking drain was only about 300 mm at both locations. The drilling works for the raking drains were carried out one month after construction of nails concerned and four months before the exhumation of the nails. The independent site audit was conducted immediately after the construction of the raking drains.

Judging from the close proximity and direction of the damaged sections relative to the raking drains as well as the construction sequence, it was concluded that soil nails XC8 and B4 were damaged by the drilling works for the raking drains located below them. The two nails were replaced. Guidelines on the setting out of raking drains were promulgated to avoid similar occurrence in the future.

(a) Drilling equipment used for stitch drilling

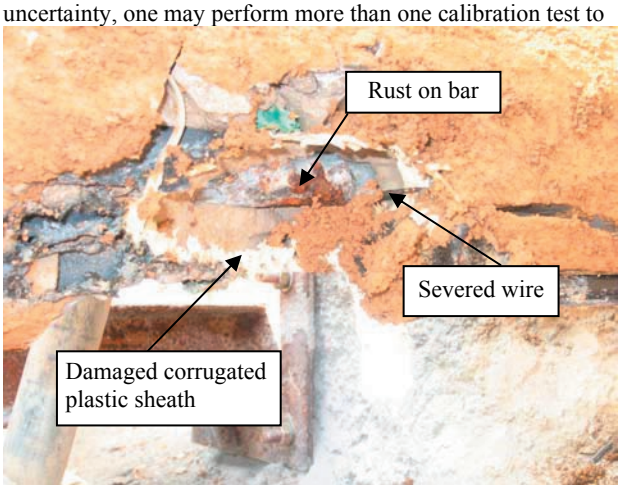
(b) Exhumation of soil nail

Fig. 8 Exhumation of soil nail using stitch-drilling technique

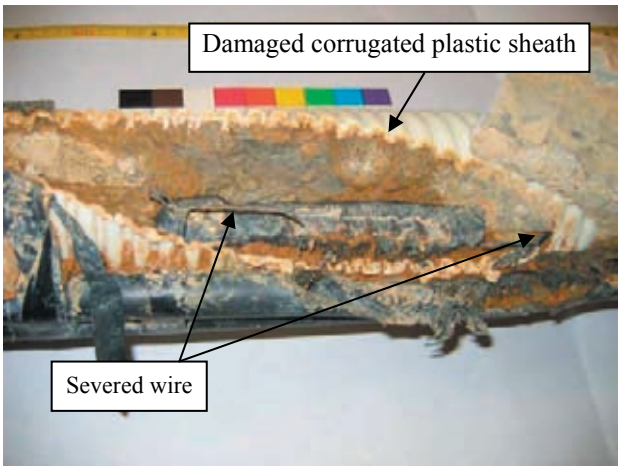
4.2 Observations

The large volume of TDR tests indicated that the general workmanship associated with the installation of wire did not have any significant effect on the test results. However, it was noted that some of the nails with short TDR-deduced length have a negative reflection in TDR waveforms. As discussed in Section 2, this indicates that the TDR wire is in direct electrical contact with the nail reinforcement. Consequently the TDR-deduced length only represents the distance between the nail head and the location of the electrical contact and is not evidence of "short" reinforcement. The GEO is currently examining the use of grout pipe with embedded TDR wires to minimise possible damage to wire and to facilitate installation (see Fig. 11).

The current practice requires the pulse propagation velocity to be determined from a nail of known length, which is commonly selected from a pool of working nails. However, the subject velocity depends very much on the variability of the grout sleeve of the nail selected for the calibration. To minimise this uncertainty, one may perform more than one calibration test to



(a) Soil nail XC8



(b) Soil nail B4

Fig. 9. Damaged section in Nails XC8 and B4

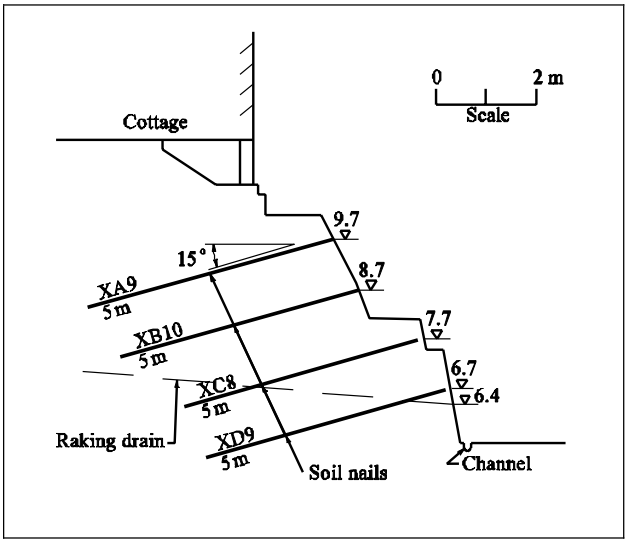


Fig. 10. Section showing soil nail XC8 and the raking drain below

determine a more representative pulse propagation velocity.

During the course of implementation of the pilot quality assurance programme, an experience-based chain of actions, as shown in Fig. 12, gradually evolved. If anomalies are identified during the independent site audit, an investigation on the extent and causes of the anomalies will be conducted. The exact course of the investigation depends on circumstances and each case will have to be determined on its merit. In general, it would include review of test results and construction records, conducting additional TDR tests in the vicinity of the nails with anomaly, use of other non-destructive techniques (e.g. Cheung, 2003; Cheung & Lo, 2005), and in some special circumstances exhumation of installed nails to confirm the nature of anomalies.

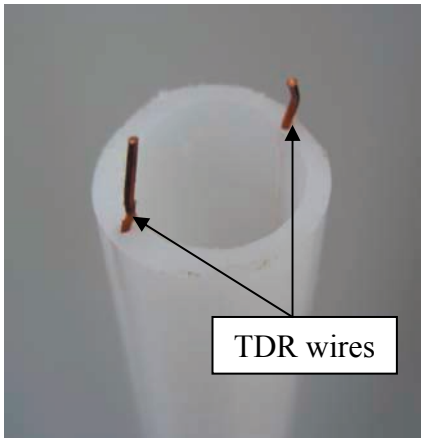


Fig. 11. Grout pipe with embedded TDR wires

5 CONCLUSIONS

In 2004 GEO initiated a pilot quality assurance programme on the use of TDR to assess the quality of soil nails with pre-installed wires at LPM sites. By the end of 2006, over 8,500 soil nails in about 650 LPM sites have been tested. Less than 1 % of the tested soil nails exhibit anomalous TDR results. It seems that these anomalous are commonly related to the integrity of grout sleeves which could have been adversely affected by the ground conditions or damages inflicted by construction activities.

The pilot quality assurance programme shows that TDR can be an effective tool to supplement site supervision in the quality control of soil nailing works, which cannot be checked easily after construction. The pilot programme allows insight and experience to be gained in applying the TDR method to soil nailing works and also enables refinement on the testing procedure, interpretation skills and establishment of follow-up actions to be made.

While TDR, like any other NDTs, does not give definitive answer to the cause of anomalies, it flags up nails that warrant further examination and coupled with appropriate NDTs, the validity of the TDR tests can be ascertained. The two case histories presented in this paper indicated that TDR can play a useful role in assuring construction quality and promoting improvements in installation techniques and quality control.

ACKNOWLEDGMENTS

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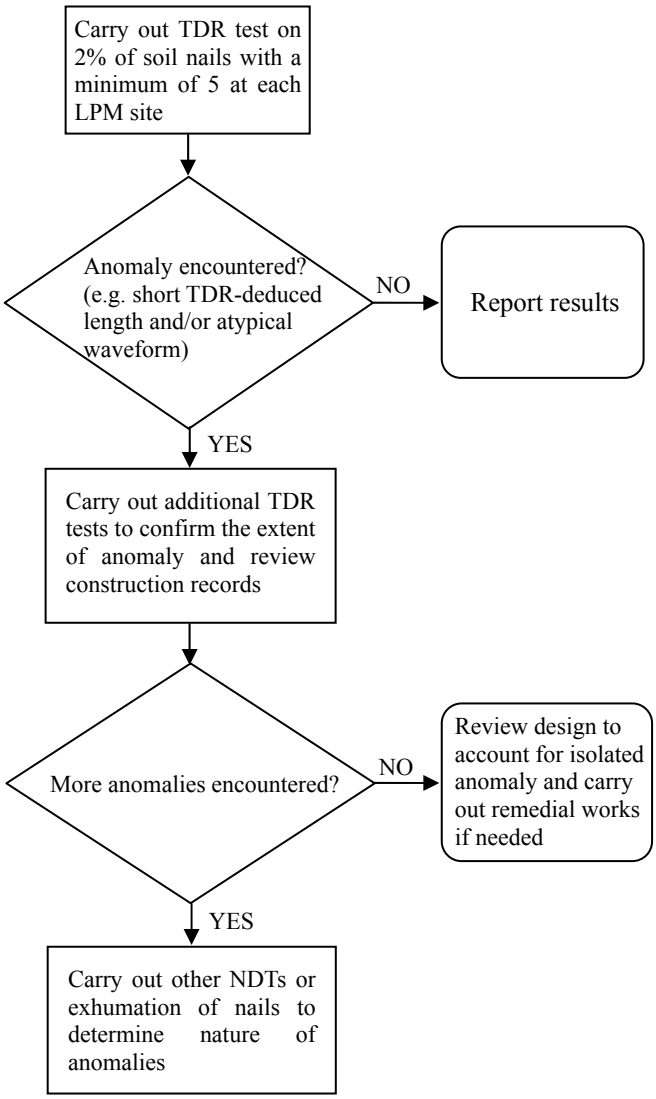

Fig. 12. General chain of actions evolved from the pilot quality assurance programme

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Development and Applications of Debris Mobility Modelling in Assessment of Natural Terrain Landslide Hazards

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Abstract: The subject of debris mobility has long been one of the key areas of development in slope engineering practice in Hong Kong. Back in 1962, Professor Lumb in an early publication acknowledged that consequences of slope failures can be devastating if debris avalanches down the slope reaching populated areas. Research in debris mobility has always been a key subject in landslide risk assessment and mitigation. In the 1990s, debris mobility of man-made slope failures has been studied using travel angles of historical landslides as the key debris mobility indicator. This empirical debris mobility model has proved to be useful in estimating runout distances of landslides on man-made slopes. Since early 1990s, there has been increasing concern on the potential hazards of natural terrain landslides to urban development amid its rapid expansion into steep natural hillsides. The travel angle approach, however, does not model adequately the runout behaviours of natural terrain landslides on sloping terrain, in particular, channelised flows and landslides of long runout distances. Over the years, it has been demonstrated that numerical modelling of landslide dynamics is an effective tool to not only estimating runout distances but also studying runout behaviours, such as the debris influence zone, runout velocity and flow depth. This paper reviews the technological advancement made in debris mobility modelling over the past decades, and presents the potential applications of the latest development of debris mobility modelling in assessing of natural terrain landslide hazards.

1 INTRODUCTION

Landslide is a common form of natural hazards in Hong Kong that can cause significant loss-of-life and socio-economical consequences. The landslide risk to the community is largely a combined result of three causal factors: the sub-tropical climate, the steep hilly terrain and the high density population. Since its formation in 1977, the Geotechnical Engineering Office (GEO) (named Geotechnical Control Office before 1991) has focused on landslides from man-made slopes amid the rapid urban development that entailed extensive site formation works for building and infrastructure developments. Urban development continued to expand in the past decade and gradually encroached on natural hillsides. The retreat of natural hillside boundaries for urban development has led to an increase in landslide risk from natural terrain. Since the early 2000s, reducing landslide risk from natural terrain has become one of the main issues under the GEO's landslide risk management framework and a landslide risk management strategy has since been developed to deal with natural terrain landslide hazards

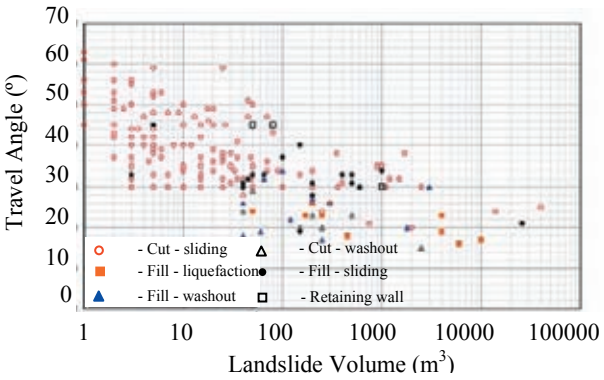

Fig. 1. Relationship between travel angle and landslide volume for selected man-made slope failures in Hong Kong

Irrespective of the nature of the landslide problem, the subject of debris mobility has been one of the key areas of development in slope engineering practice in Hong Kong. Back in 1962, Lumb (1980) acknowledged that consequences of slope failures can be devastating if debris avalanches down the slope reaching populated areas. Over the years, research in debris mobility has always been a key subject in landslide risk assessment and mitigation, and much development has been achieved in both the analytical skills and applications for dealing with landslide risk from natural hillsides.

2 PREVIOUS APPROACHES TO DEBRIS MOBILITY ASSESSMENT

The earliest approach to assessing debris mobility of natural terrain landslides has largely been based on the experience gained from the use of empirical method in estimating runout distances of landslides on man-made slopes.

In the 1990s, debris mobility of man-made slope failures has been studied using travel angles of historical landslides as the key debris mobility indicator. This empirical model was based on historical landslide data with due regard to the mechanisms of failures and modes of debris movement (Fig. 1). With the comprehensive database of landslides on man-made slopes, which contain quality landslide data obtained from field inspections and aerial photograph interpretation, the empirical approach has proved to be useful in estimating landslide runout distance.

Amid the growing concern on the potential hazards of natural terrain landslides in the 1990s, empirical methods, initially based on the concept of travel angle and later evolved into other enhanced formulations, have been developed and applied to assess debris mobility of natural terrain landslides.

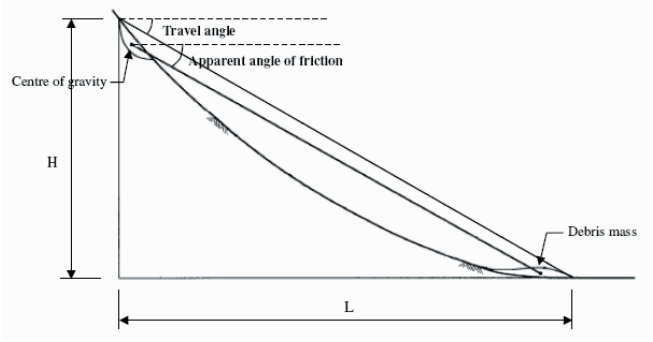

Fig. 2. Definition of travel angle.

2.1 *The travel angle method*

In this paper, the term "travel angle" (Cruden & Varnes, 1996) has the same meaning as the term "*Fahrböschung*" defined by Albert Heim in his book published in 1932 (Hsu 1978) and "reach angle" (Corominas 1996). This is the angle of the line connecting the head of the landslide source to the distal end of the displaced mass (Corominas 1996) (Fig. 2). The term is similar to other terms like "apparent angle of friction", "equivalent coefficient of friction" and "average coefficient of friction" but these terms are derived from the line linking the centres of gravity of the landslide source and the displaced material.

One of the first detailed studies on the travel angles of landslides in Hong Kong was described in Wong & Ho (1996). The study was carried out on landslides which occurred on the Lantau Island on 5 November 1993 at soil cut slopes alongside roads and catchwaters, and of which accurate data on the profile and travel distance of debris was available. Wong *et al.* (1998) then applied the same method on natural terrain landslides that occurred on the Lantau Island in the same rainstorm. It was observed that the travel angles of landslides tend to decrease (i.e. mobility of landslides increase) with an increase in debris volumes and that the travel angles of landslides are critically dependent on the failure mechanisms and modes of debris movement. Lo (2000) summarised the distribution of the travel angles for different types of landslides in Hong Kong (Fig. 3).

Lau & Woods (1997) mentioned a limitation of the travel angle method for natural terrain landslides. It is noted that the accuracy of estimating runout distance (d) by the travel angle method decreases rapidly when the slope angle at the final point of debris deposition approaches the angle of reach of the landslide. This limitation is particularly significant when the method is applied to steep natural terrain, given the same degree of change in the angles of reach (δ) (Fig. 4). Furthermore, the travel angle method does not take the effects of terrain characteristics on debris mobility into consideration. For example, different terrain profiles may lead to debris having different runout behaviours, and therefore different mobility, while their respective travel angles may stay the same. The complex terrain characteristics of typical natural hillsides in Hong Kong have therefore rendered the use of the travel angle method not quite appropriate.

2.2 The toe slope angle method

The toe slope angle is another parameter that has been studied in Hong Kong in the late 1990s for assessing debris mobility of natural terrain landslides. The toe slope angle is defined as the ground slope angle at the distal end of a landslide trail. Choi *et al.* (2003) examined the toe slope angle and the distance that landslide debris has travelled beyond 15° ground slope for more than 50 m of large (scar > 15 m wide) recent natural terrain landslides in the Natural Terrain Landslide Inventory (NTLI). The results show that only about 15% of the landslides in the NTLI (total = 560) reached ground with a slope angle less than 15° (or have toe slope angles smaller than 15°) and only two of

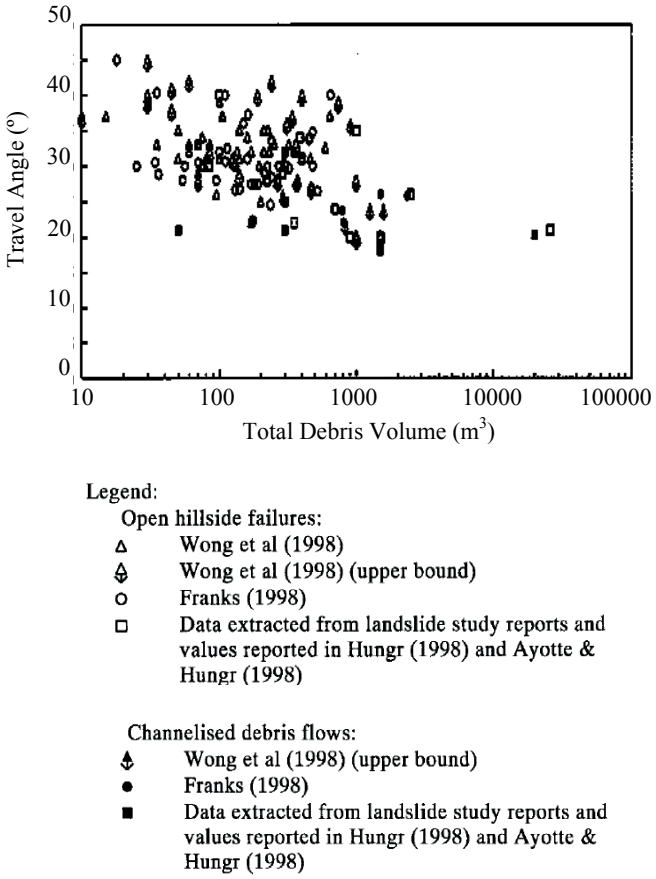

Fig. 3. Distribution of the travel angles for different types of landslides in Hong Kong.

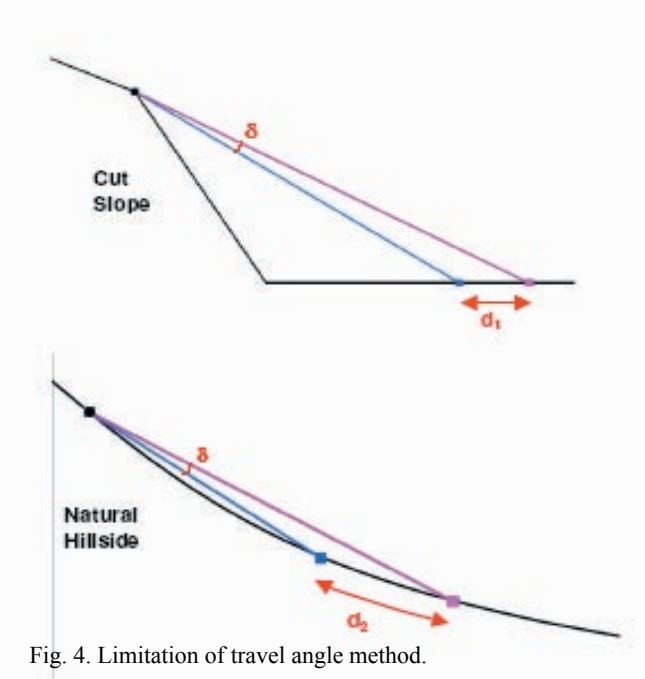

Fig. 4. Limitation of travel angle method.

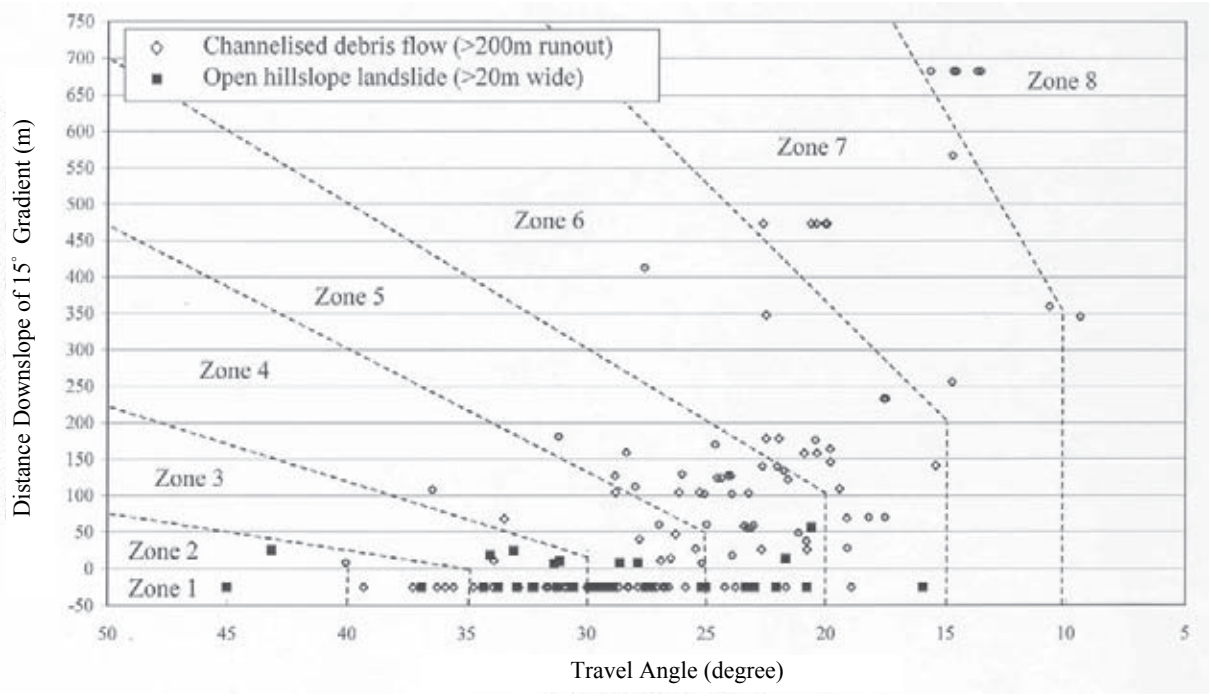

Fig. 5. Empirical damage zoning based on historical debris runout data.

them have travelled more than 50 m beyond 15° sloping ground. Based on this study, it can be observed that natural terrain landslides in Hong Kong do not usually travel beyond ground with a slope angle less than 15° and even fewer travel more than 50 m beyond 15° sloping ground. This suggests that the 50 m zone provides an adequate buffer area for debris deposition in the majority of the cases of natural terrain landslides in Hong Kong. The small portion of landslides with long runout distances is mostly associated with very mobile channelised debris flows. The findings serve as a rough appreciation of the typical landslide debris mobility in Hong Kong. The method has also been incorporated in the screening criteria currently adopted by the GEO for screening new development sites for natural terrain hazard studies.

2.3 *Hybrid method using travel angle and travel distance*

Because of the limitations of the travel angle method described above, an additional parameter, travel distance beyond 15° sloping ground, has been incorporated in the model to account for the effects of terrain characteristics along credible flow paths on debris mobility. An empirical classification of proximity zones of facility at risk of natural terrain landslide hazards based on the travel angles and the travel distances beyond 15° sloping ground of the historical landslides in the NTLI has been developed. This has been applied in the consequence model of the global quantitative risk assessment on natural terrain landslides in Hong Kong. (Wong *et al.,* 2004; Wong & Ho, 2006) (Fig. 5).

The development of the empirical approaches to assessing debris mobility of natural terrain landslides has provided geotechnical practitioners in Hong Kong with useful tools in estimating probable runout distances of natural terrain landslides and potential risk to population. There is however practical limitation of the empirical methods – they cannot provide information on the runout behaviour of landslide debris in motion. Runout behaviour may include debris velocity, debris thickness and lateral spread of debris, which are critical information for systematic study of landslide behaviours, determination of debris influence zone and engineering design of mitigation measures against landslide hazards.

3 LATEST DEVELOPMENT

In 1998, the GEO commissioned a pilot study to examine the applicability of numerical models for prediction of landslide runout distance. Under the pilot study, 20 natural terrain landslides in Hong Kong were back-analysed using the model DAN developed by Hungr (1995). DAN is a numerical model capable of simulating debris motions in a rectangular channel. Hungr adopted a Lagrangian finite difference scheme, which was first proposed by Savage & Hutter (1991), to develop the DAN model in which debris is divided into a series of vertical slices. Considering debris as an equivalent fluid, equations of motions were set up for each of the elements. With this, the Lagrangian finite difference scheme calculates the velocities of the element at every time step of the time-stepping calculating process and the elements advance to their new positions based on the calculated velocities. The model was demonstrated to provide satisfactory back-analysis results for the natural terrain landslides using Vollemy model to describe the debris rheology (Ayotte & Hungr, 1998).

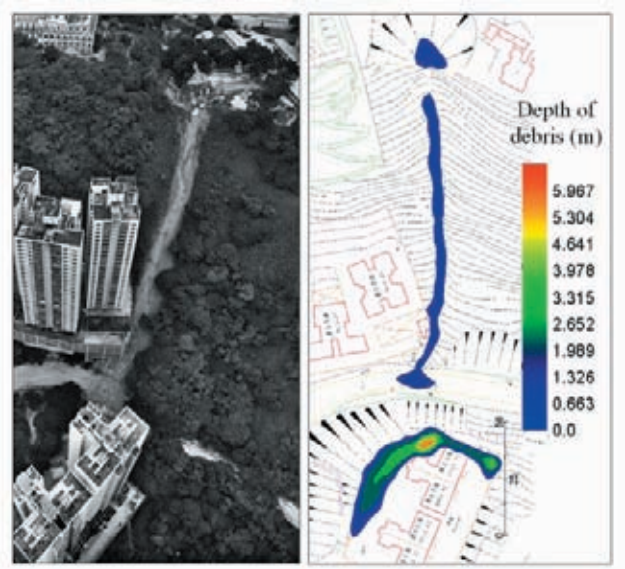

Fig. 6. 3-D debris runout modelling.

In the DAN model, the cross-section of a flow channel is assumed to be rectangular with frictionless sides. However, for natural terrain landslides which usually travel on complex morphology such as along topographic depressions or natural drainage lines, flow channels with trapezoidal shaped cross-section is considered to be a more reasonable assumption. The DAN model has subsequently been improved in this respect to better simulate the actual conditions of natural terrain landslides in motion. This enhancement has been incorporated in the Debris Mobility Model (DMM) by the GEO (Kwan & Sun, 2006). The DMM removes the assumption of rectangular channel and facilitates the numerical calculation based on trapezoid channel. It also calculates debris depth and top width based on an input of the geometry of a given trapezoid channel. The input entries involve heavily geo-informatics content, and because of this, the GEO has taken further initiative to embed the DMM onto the Geographic Information System platform in an effort to streamline the analysis process (Fig. 6).

Local geotechnical practitioners have also proactively contributed to the development of numerical prediction of debris mobility. For example, Manusell Geotechnical Services Ltd. (MGS) had developed the numerical package DebriFlo (MGS 2001). Similar to the DAN and DMM, DebriFlo simulates one-dimensional debris motion in a flow channel. It solves the "Leading Edge Equations" that describes the momentum balance of debris front as proposed by Takahashi & Yoshida (1979). DebriFlo also takes into account the effect of super-elevation, which is often encountered when the debris passes bends, in the analysis process.

Despite their successful applications in debris runout calculation, the one-dimensional models suffer some major limitations relating to (i) the requirement of a pre-defined channel alignment (i.e. the runout path) as an input for simulation and (ii) the incapability of predicting lateral spread of debris flows. Researchers in Hong Kong played an important role in the advancement of numerical modelling of debris mobility in this aspect. Chen (1999) developed a Lagrangian model for simulation of three-dimensional debris motions. In the model, debris is considered as composing an array of vertical columns. The spreading behaviours of the debris can be simulated by computing the motions of each of these debris columns. The model has been used to simulate a number of notable large-scale natural terrain landslides in Hong Kong, such as the Fei Tsui Road and the Shum Wan Landslides occurred in 1995 (Chen & Lee, 2000).

Chen's model, however, requires connectivity between debris columns during simulation. This connectivity requirement limits its application to single continuous debris trails only. Where there is complex terrain morphology resulting in bifurcated debris trails, such as the 2000 Tsing Shan Debris Flow (King 2001) (Fig. 7), a more versatile numerical technique is necessary. In this respect, the GEO in 2004 developed a robust three-dimensional numerical model for simulating debris motions over irregular and complex terrain profiles.

The three-dimensional debris mobility model, 3D-DMM, uses an alternative numerical scheme called Particle-In-Cell (PIC). PIC was first developed for computational fluid dynamics (Harlow 1988). The method was then adopted in numerical simulations of deformations of elastic-plastic materials (Sulsky *et al.*, 1995) and explosions under water (Liu & Liu, 2003). A simplified PIC scheme was used to study the formation of alluvial fan by Wang *et al.* (1997).

PIC adopts particle representation of the deformable materials concerned and calculates the properties of the materials, such as strain and stress, based on the Eulerian technique. The terrain on which the landslide initiates and flows is divided into an array of cells and the landslide debris is represented by a number of non-interacting particles. When the simulation starts, each cell on which the landslide debris initiates contains non-interacting particles, the number of which depends on the dimensions of the landslide. With the number of particles in the cells, the debris depths are determined and equations of motions are set up for calculation of debris velocities at these cells. Having calculated the velocities, the

Fig. 7. The 2000 Tsing Shan Debris Flow.

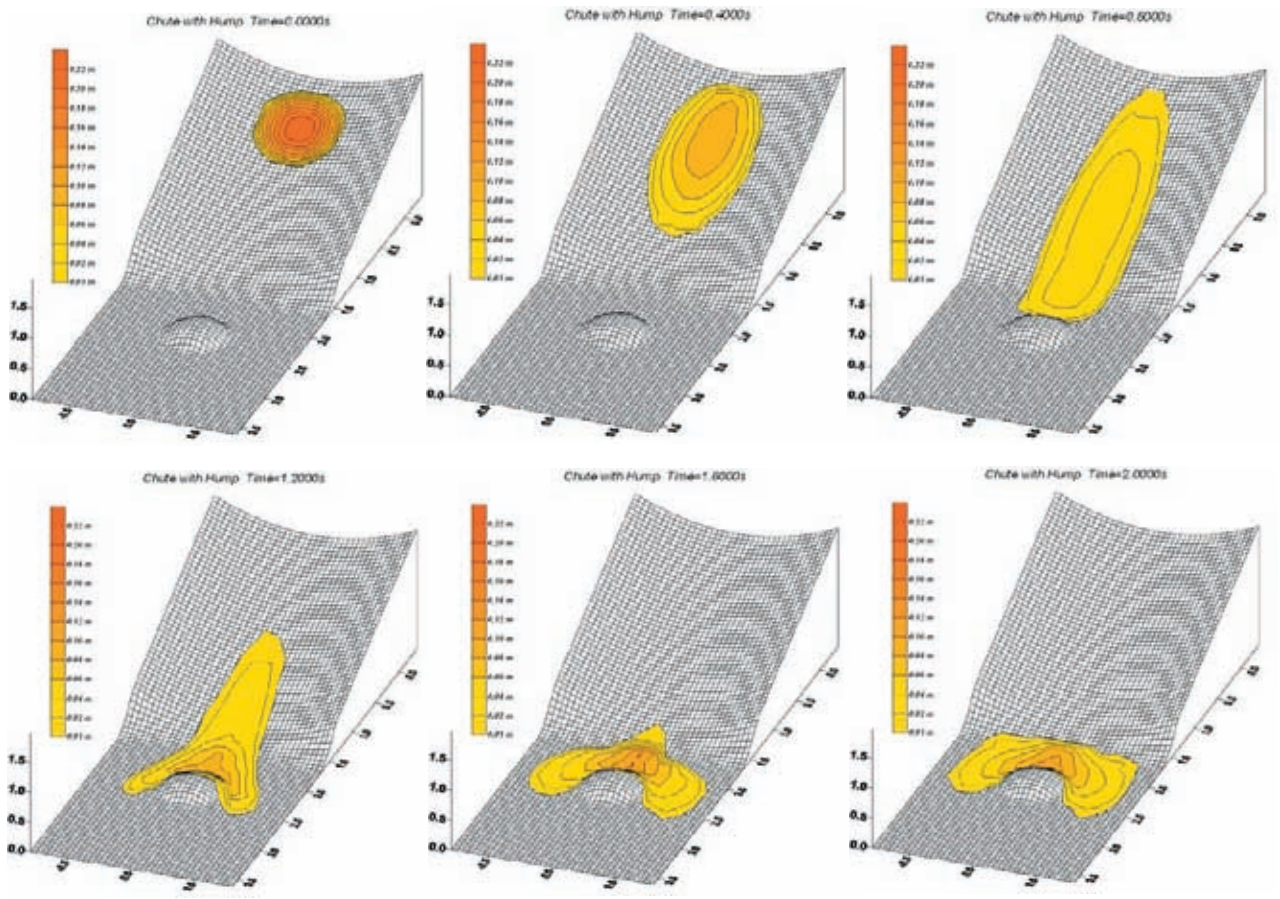

Fig. 8. Simulation of frictional materials flowing down a channel obstructed by a hump.

particles are advanced to new positions based on the topology of the terrain. With a time-stepping algorithm, the calculation steps are repeated and the debris motions simulated. The 3D-DMM has been used in the back-analysis of some notable landslide incidents in Hong Kong. The simulation for the Shum Wan Landslide that occurred in 1995 (GEO 1996) gives a reasonable representation of the actual situations (Ko & Kwan, 2006). Another simulation for debris flowing round a circular obstruction on a flat plane is shown in Fig. 8. It is observed that PIC inherits the advantage of particle method that permits simulations of material flows of large deformation, while the Eulerian technique allows fast calculations.

4 POTENTIAL APPLICATIONS

The DMM simulates debris motions along credible flow paths on hillsides and calculates at the same time flow indicators, such as runout velocity and flow depth. With the use of 3D-DMM, visualisation of flow motion is also possible for determination of debris influence zone. This choice of functions of the DMM allows for a variety of applications in assessing natural terrain landslide hazards. Two potential applications are given below to demonstrate the usefulness of the analytical skills.

4.1 *Site-specific application in vulnerability assessment*

The DMM can be applied to assess vulnerability factors that involve a combined use of numerical simulation of debris motions and the probabilistic material strength concept (Ko & Kwan, 2006).

Given a landslide of a particular volume hitting a facility, a numerical simulation of the debris motions takes terrain characteristics, debris mobility and spatial setting of the facility into consideration to determine the magnitude of the landslide impact on the facility. It predicts the runout behaviour of the landslide and the extent of the landslide influence zone. The attributes that form the vulnerability factor, like the proximity to the landslide source, the spatial setting of the facility and the runout characteristics of the landslide, are implicitly accounted for in the numerical simulation, the outcome of which would be reflected in the magnitude of the landslide impact on the facility.

The magnitude of the landslide impact on the facility adversely affects the structural stability of the facility and the extent of which depends on the facility type. The more robust the structure is, the higher the chance it could withstand the landslide debris, i.e. higher degree of protection to the occupants inside, and therefore, lower likelihood of loss-of-life. The probabilistic material strength concept applied to bending moment capacities of walls (Fig. 9) is therefore used at this stage to assess the likelihood of the facility that fails to resist the landslide impact, which is assumed to equate to the probability of loss-of-life, i.e. the vulnerability factor.

This analytical method has been applied to back-analyse two landslides, the Tong Fuk Landslide occurred in 1993 and the Sham Tseng San Tsuen Landslide in 1999. In both cases, the vulnerability factors obtained based on the method are consistent with the observations made on site after the mishaps.

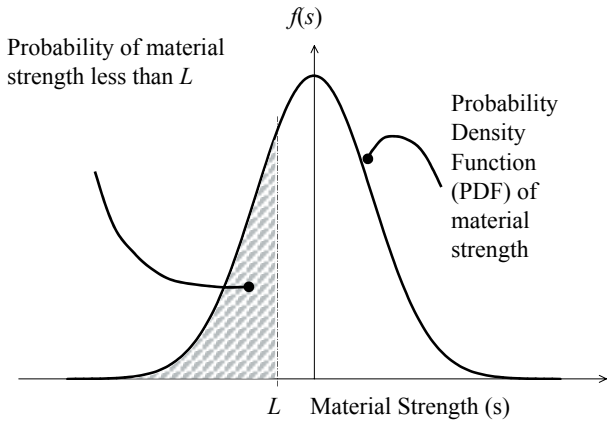

Fig. 9. Probability density function of material strength.

4.2 *Probabilistic approach in assessing debris mobility*

The DMM can also be applied to predict debris runout behaviours for hillside catchments in a probabilistic framework based on the analysis and use of probabilistic distribution of DMM's runout parameters, i.e. friction angle and turbulence coefficient. Such parameters may be correlated with attributes characterising landslides and morphology of hillside catchments, as runout behaviours of a landslide are controlled by, *inter alia*, the morphology of the hillside catchment within which the landslide occurs. It is a hybrid approach making use of both empirical and analytical methods. With the use of numerical modelling, about 150 natural terrain landslides with long runout distances are back-analysed to obtain their runout parameters. Based on the morphological parameters, i.e. catchment plan area, maximum height and average slope gradient, of the hillside catchments within which the landslides have occurred, the hillside catchments may be categorised into different sizes, e.g. "very large", "large", "medium" and "small" (Wong *et al.*, 2004). For each category of the hillside catchments, a probabilistic distribution of the runout parameters of the historical landslides that have occurred in that category of the hillside catchments can be developed. There would be, in the end, typical probabilistic distributions of runout parameters for different categories of hillside catchments. These probabilistic distributions may be applied to predict debris runout behaviours for other hillside catchments in Hong Kong.

This framework provides a means for prediction of runout behaviours of landslides for any given size of hillside catchment. With further calibration exercises and benchmarking tests, there is prospect that the framework can be developed into an effective tool for assessment of debris mobility and quantification of landslide risk.

5 DISCUSSIONS

The development and potential applications of debris mobility modelling discussed above demonstrate a possible scope for advancing the current practice in debris mobility assessment. From the experience gained over the years, the following key areas deserve further examination:

5.1 *Further calibration*

In order to obtain more realistic results, numerical models should be well tested and calibrated. Although the current development in numerical modelling seems to be promising, further study on the general application of the key runout parameters at different scales and types of landslides is considered necessary. This calls for a systematic calibration exercise.

5.2 *Flow-flexible barrier interaction*

It has been observed that there is an increasing use of flexible barriers to resist landslide debris. Nevertheless, there has been a lack of understanding on the inter-behaviours of debris flows and flexible barriers on impact. This has in turn limited the capability in evaluating the use of flexible barriers in resisting landslide debris/flows. With the advance of numerical modelling to simulate the flow scenario, it is recommended to conduct a systematic study on this subject to further enhance the understanding of the inter-behaviours of the flow-flexible barrier interaction.

5.3 *Trap effect*

The trap effect refers to the situation when a topographic depression (no matter of its size) is filled up by landslide debris travelling over resulting in accelerating the flow of the landslide debris. In most of the contemporary numerical algorithms for predicting debris runout behaviours, this trap effect, which is critical to a rigorous simulation of landslide motion, has not been considered. Because of its significance, it is prudent to incorporate the trap effect in future enhancement works as one of the improvement initiatives.

6 CONCLUSIONS

Natural terrain landslides can bring significant consequences to the public. Assessment of debris mobility is considered as one of the key elements to evaluate the degree of risk posing to the population. Over the years, much development has been made in the study methodology for dealing with the natural terrain landslide risk, owing to, *inter alia*, the fact that empirical method previously adopted for man-made slopes does not model adequately the runout behaviours of natural terrain landslides on sloping terrain. The latest development on the analytical approach using debris mobility modelling has proved to be an effective tool to assessing debris runout behaviours. Pending further calibration and benchmarking of the newly developed approach, continual technical development in debris mobility modelling for application in landslide risk assessment and mitigation is warranted. The development would, *inter alia*, further improve the quality of landslide risk assessment in Hong Kong.

ACKNOWLEDGEMENT

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Performance Evaluation of Electrical Strain Gauges and Optical Fiber Sensors in Field Soil Nail Pullout Tests

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Abstract: The pullout resistance of soil nails installed in slopes is affected by a number of factors. The measurement of the pullout resistance and the shear resistance of the soil nail is required. The existing technique of using electrical strain gauges monitoring pullout tests has limitations. This paper introduces two types of optical fibre sensors and associated devices for measurements of strain distribution in soil nails. The two types of sensors were calibrated first in the laboratory. Typical calibration results are presented. Afterwards, the two types of optical fibre sensors were used to monitor strains of two Fibre Reinforced Polymer (FRP) soil nails in Hong Kong together with electrical resistance-type strain gauges. The field test data were collected and analysed. In particular, the strains measured by the two types of optical fibre sensors are compared to those by electrical strain gauges and are found to be in good agreement. The advantages of optical fibre sensors are presented.

1 INTRODUCTION

Though many slopes have been stabilized in past decades, there are still thousands and thousands of slope features (slopes, retaining walls, etc) which require stabilisation in Hong Kong. More than 90 % of slope features have been or will be stabilized using soil nails. Soil nails are normally constructed by inserting a structural element such as a steel rebar in a pre-drilled hole and then grouting the hole with cement slurry. Most soil nails have a diameter of 100 mm and a length of 10 m on average. Soil nails are normally passive structures without any preloading. If a slope does not move, soil nails in the slope have no tension forces, otherwise, taking tension. The slope stabilisation is provided by soil nail pullout resistance in the resisting zone below (or beyond) the critical slip surface of the slope. The pullout resistance of a soil nail is an important parameter used in design and will affect slope safety.

The pullout resistance is affected by a number of factors, such as construction methods and process, properties of soil, cement grout, soil-grout interface, the geometry of the slope and drill hole, etc. (Berglund & Oden 1996; Chang & Milligan., 1996; Chu & Yin 2005a,b; Franzen 1998, Lee *et al.*, 2001; Milligan & Tei, 1998; Schlosser 1982; Schlosser & Guilloux, 1981; Shiu *et al.*, 1997). The pullout resistance is normally required to be verified (or checked) by field pullout tests. In a normal pullout test, only the pullout load and the displacement at the nail head are measured. To know the shear resistance at the interface between the cement grout and the surrounding soils, the axial strain of the nail has to be measured. The strain data are then used to calculate the interface shear resistance. The axial strains are normally measured using electrical resistance-type strain gauges. One limitation is that for each strain gauge, a cable with at least three wires bundled together is needed. If the number of strain gauges is large, many cables are needed. This will affect the quality of the grouted section. In addition, the existence of electromagnetic field will affect strain measurement. The resolution of electrical gauges is about 10^{-5} strain. Comparing with the electrical strain gauges, optical fibre sensors have a number of advantages, (i) reliable, (ii) high resolution of 10^{-6} strain, (iii) no electromagnetic influences; and (iv) robust and convenient to use.

This paper presents two types of optical fibre sensors: (a) bare Fibre Bragg Grating (FBG) point sensors connected in series (multi-points) along a soil nail for measuring point strains; and (b) average FGB sensors connected in series and enclosed in an aluminium tube to measure the average strain. The two types of sensors have been tested in the laboratory and the field with devices and protection for fitting a soil nail and data logger for field monitoring.

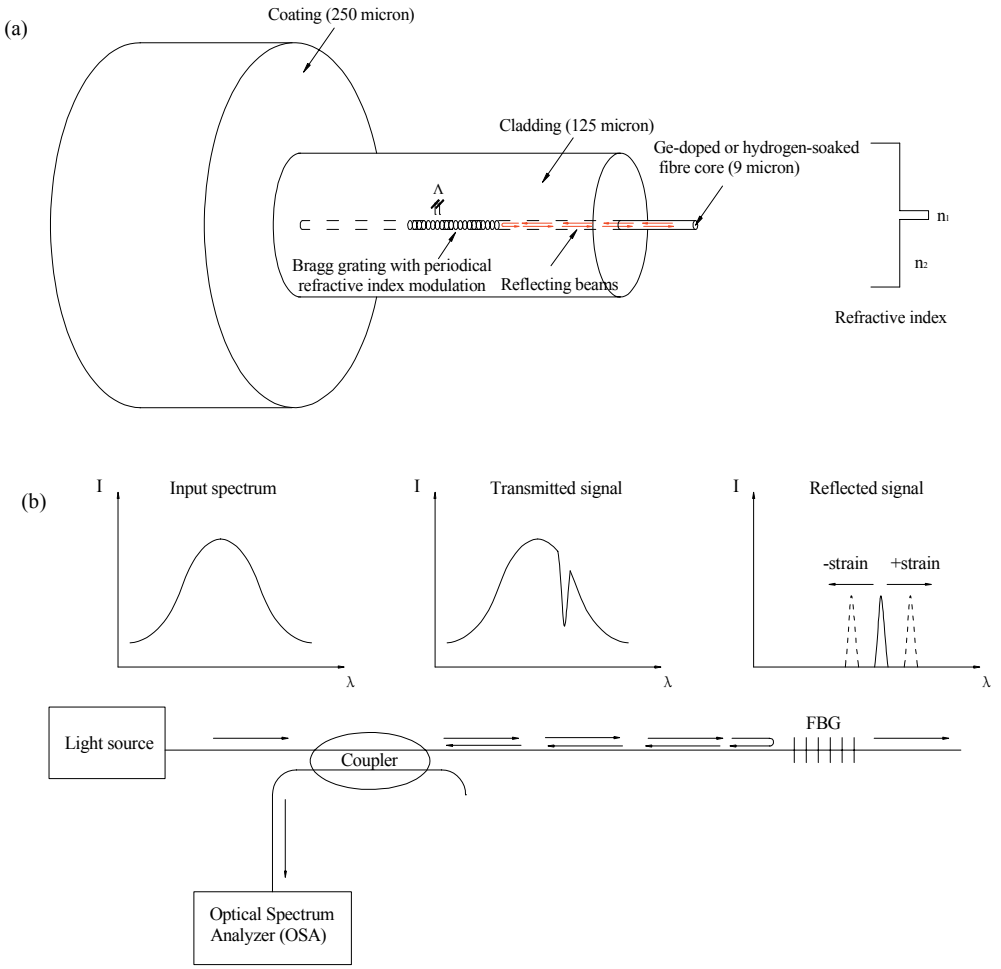

Fig. 1. Generic concept of Fiber Bragg Grating (FBG) strain sensing - (a) details of an FBG sensor and (b) functioning principle of an FBG

2 WORKING PRINCIPLE OF FIBRE BRAGG GRATING SENSORS

Fig. 1 shows the working principle of a Fibre Bragg Grating (FBG) strain sensor - (a) Bragg grating of an FBG sensor; and (b) functioning principle of an FBG. An FBG is written by a laser machine into a segment of Ge-doped single-mode fibre in which a periodic modulation of the core refractive index is formed by exposure to a spatial pattern of ultraviolet (UV) light using a phase mask (Hill *et al.*, 1993). According to Bragg's law, when a broadband source of light has been injected into the fibre, FBG reflects a narrow spectral part of light at certain wavelength, which is called the Bragg wavelength and dependent on the grating period and the refractive index of fibre (Morey *et al.*, 1989 and Meltz *et al.*, 1989)

$$\lambda_B = 2n\Lambda \tag{1}$$

where λ_B is the Bragg wavelength, typically 1510 to 1590 nm (1 nm = 10^{-9} m), n is the effective core index of refraction and Λ is the period of the index modulation.

The Bragg wavelength is strain-dependent through physical elongation or thermal change of the sensor and through the change in the fibre refractive index due to photo-elastic effect. Considering a standard single mode silica fibre, the relationship between Bragg wavelength and strain of the sensing fibre can be simplified as follow (Othonos & Kalli, 1999):

$$\frac{\Delta\lambda}{\lambda_0} = c\varepsilon \tag{2}$$

which can be re-written as

$$\varepsilon = \frac{\Delta\lambda}{c\lambda_0} \tag{3}$$

where c = a constant, for germanium-doped silica fibre, $c \approx 0.78$; λ_0 = the Bragg wavelength of the grating under strain-free condition; $\Delta\lambda$ = the wavelength change due to straining. Both λ_0 and $\Delta\lambda$ can be measured by an interrogator. Thus, the strain ε can be calculated using Eq. (3).

Comparing to electrical resistance strain gauge, this optical fibre sensing technology has these apparent advantages: immunity to electromagnetic interference and power fluctuation along the optical path, insensitivity to corrosion and fatigue, high precision, durability, absolute measurement, tiny size, reduced cable requirement and so on. By their multiplexible nature, hundreds of FBG sensors can be connected on a single fibre (with

an outer diameter of 250 μm) and easily integrated within a structural element without a serious negative influence on the mechanical properties.

In this study, the Micron Optics SI 425 Swept Laser Interrogator (Micron Optics 2006) is used to provide rapid and accurate measurements of the wavelength and changes of FBG sensors in real time. It can read the reflected wavelengths of the FBG gratings in an ascending order and interrogate up to 512 sensors simultaneously, with 1 pm (10^{-12} m) resolution and 10 pm repeatability.

3 TWO TYPES OF FBG SENSORS AND CALIBRATION TEST RESULTS

Two types of FBG sensors are used in the study. The first type is a series of bare FBGs normally adhered on the surface of a structure with covered with epoxy resin as shown in Fig. 2(a). This is the simplest protection method. Another type is a series of FBG sensors enclosed in an aluminum tube of 4 mm to 6 mm in outer diameter to provide adequate protection and measure the average strains of a segment with special set-up. The later is also called special average FBG strain sensor with logn gage length for measuring the average strain of a certain length, say, 0.25 m to 1 m. The aluminum pipe can protect the FBG sensor as shown in Fig. 2(b).

Both types of FBG sensors have been calibrated using a special device. The calibration test results are shown in Fig. 3(a) for a bare FBG sensor and Fig. 3(b) for four average FBG sensors.

Fig. 3(a) shows the relationship between the wavelength and the strain from a calibration on a bare FBG sensor. This relationship is linear with R^2 of 0.9914. Eq. (3) can be re-written as $\frac{\Delta\lambda}{\lambda_0} = c\varepsilon$, that is, $\frac{\lambda - \lambda_0}{\lambda_0} = c\varepsilon$, from which

$$\lambda = c\lambda_0\varepsilon + \lambda_0 \tag{4}$$

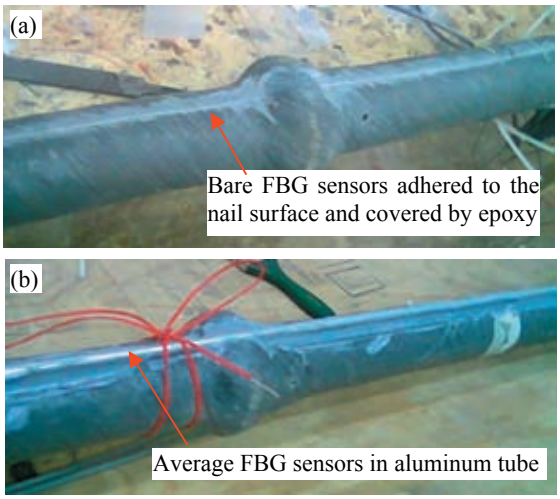

Fig. 2. Two types of FBG sensors on a soil nail – (a) bare FBG sensor (FBG-A) on the surface of soil nail for point strains and (b) the average FBG sensor (FBG-B) sensors in an aluminum pipe for continuous average strain strains.

Comparing $y = 1122x + 1539.7$ in Fig. 3(a) with Eq. (4), we have $c\lambda_0 = 1122$ and $\lambda_0 = 1539.7$. Thus, we have

$$c = \frac{1122}{1539.7} = 0.729 \tag{5}$$

This constant of 0.729 is slightly smaller than 0.78.

Fig. 3(b) shows the relationship between the wavelength and the strain from a calibration on an average FBG sensors Comparing $y = 533.27x + 1556.4$ in Fig. 3(b) with Equation(4), we have $c\lambda_0 = 533.27$ and $\lambda_0 = 1556.4$. Thus, we have

$$c = \frac{533.27}{1556.4} = 0.343$$

4 FIELD SETUP AND PULLOUT TESTING

The two types of FBG sensors were installed a slope in Lantau of Hong Kong. Four Fibre Reinforced Polymer (FRP) soil nails were installed into a man-made slope (Slope Registration No.10SW-C/C237) at this site for testing. The FRP nail was a pipe with outer diameter of 55 mm an inner diameter of 37 mm. The nail length was 3.6 m. The grouted nail had a diameter of 120 mm. Soil Nail 1, 2 and 4 were installed in the slope in depth 1 m to 2 m and grouted using pressure grout (about 1.5 MPa). Yeung *et al.* (2005, 2007) did pullout tests on FRP pipe soil nails at other sites.

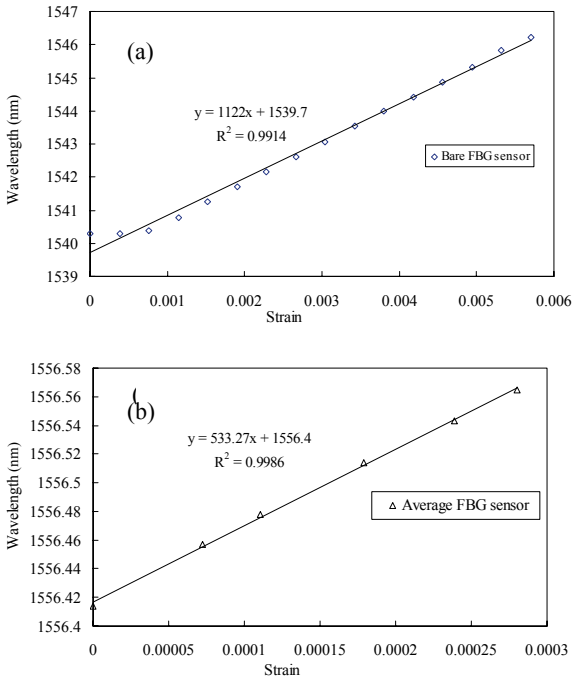

Fig. 3. Calibration results of (a) a bare FBG sensor (FBG-A) and (b) four average FBG sensors (FBG-B)

Fig. 4 shows (a) an FRP nail installed, (b) set-up of pullout device and testing ,and (c) an Interrogator for taking FBG sensor data. In Fig. 4(a), a steel rebar was inserted into the FRP pipe and was connected to the pullout hydraulic jack. A load cell was used to measure the pullout force and two LVDTs were used to measure the pullout displacements. The Interrogator was used for

taking FBG sensor data during the pullout testing. In fact, a total of four FRP nails were installed. But, only Soil Nail 3 and 4 were monitored using both electrical strain gauges and FBG sensors.

5 PULLOUT TEST RESULTS AND COMPARISSON

This section presents the main pullout test and monitoring results from Soil Nail 3 and 4. Fig. 5 shows two curves of pullout force vs displacement curves of (a) Soil Nail 3; and (b) Soil Nail 4 with two unloading/reloading cycles. It is observed that (a) the behavior is highly non-linear and plastic; and (b) the limit of the pullout resistance have been reached.

The strains along the soil nails were measured by electrical strain gauges and two types of FBG sensors during all stages of loading, including unloading and reloading stages. Fig. 6 (a) and (b) show a comparison of strains on 2 m pressurised grouted length from the end measured by two types of FBG sensors and electrical strain gauges on Soil Nail 3 under pullout loads of 100 kN and 140 kN. It is noted that the optical fibre cables of FBG-B were broken due to inappropriate installation on bearing plate when loading to 160 kN. Fig. 6 (c) and (d) show a comparison of strains by type FBG-A sensors and electrical strain gauges under pullout loads of 160 kN and 213 kN. It is observed from the comparison that strains from all types of sensors are in good agreement.

Fig. 7 (a) and (b) show a comparison of strains by two types of FBG sensors and electrical strain gauges on Soil Nail 4 under pullout loads of 100 kN and 140 kN. Fig. 7(c) and (d) show a comparison of strains by type FBG-A sensors and electrical strain gauges under pullout loads of 160 kN and 236 kN. It should be pointed out that the average FBG sensors (FBG-B) were damaged under load of 160 kN. One reason for this was that the aluminium tube with the FBG sensors was embedded in the cement grout and used to measure the strain of the cement grout. When the nail was subjected a large load, cracks might have occurred and caused very large strains. These large strains were probably beyond the upper limit of the average FBG sensors and caused damage to the sensors. It shall be noticed that a steel rebar of 32 mm diameter was inserted into the FRP nail pipe as an reinforcement and extension to be connected to the pullout device. Therefore, this segment has much higher stiffness. The strain in this segment was much smaller.

It is observed from the comparison that strains from all types of sensors are in good agreement.

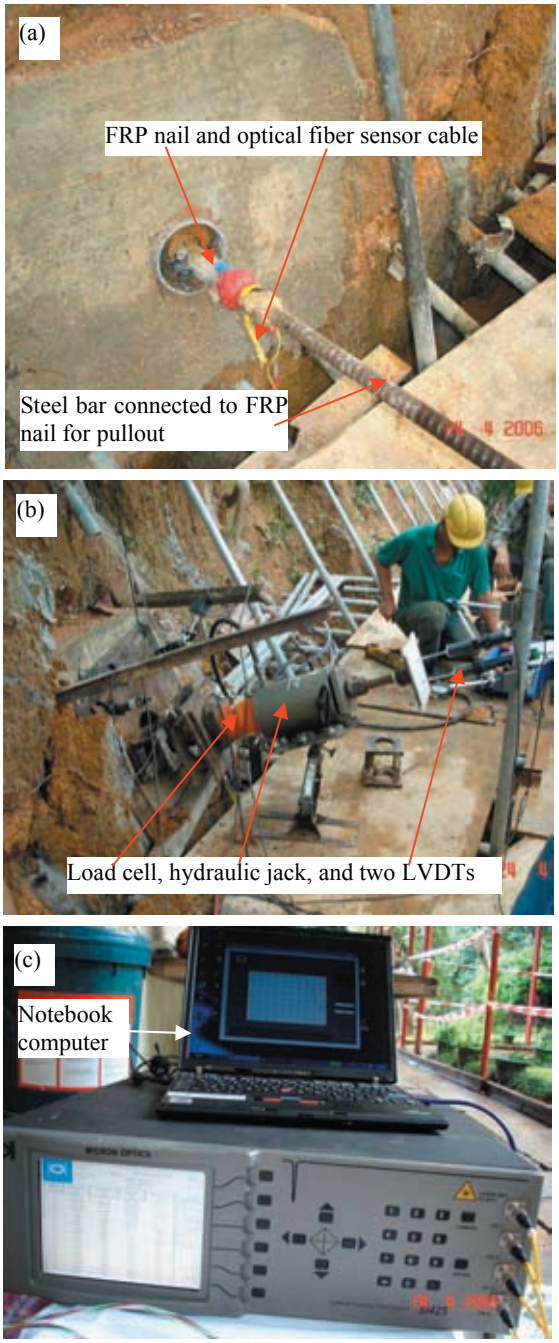

Fig. 4. (a) an FRP nail installed, (b) setup of pullout device and testing ,and (c) an Interrogator for taking FRB sensor data.

6 CONCLUSIONS

Based on this study, the following conclusions can be drawn:

1 The two types of optical fiber sensors have been used successfully for monitoring strain distribution along the soil nail during pull-out test. It is shown by comparison with

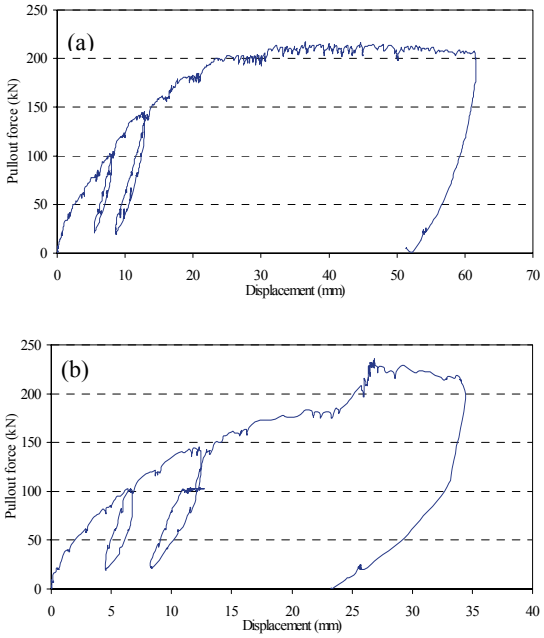

Fig. 5. Pullout force vs displacement curves – (a) Soil Nail 3 and (b) Soil Nail 4.

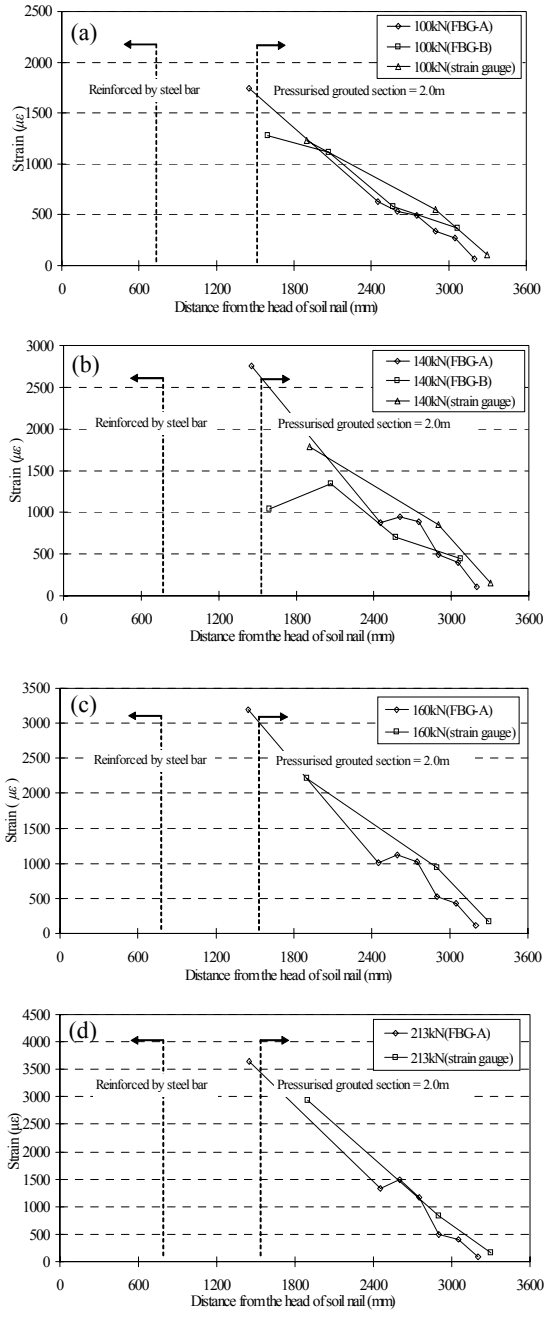

Fig. 6. Comparison of strains measured by FBG sensors and electrical strain gauges on soil nail No.3.

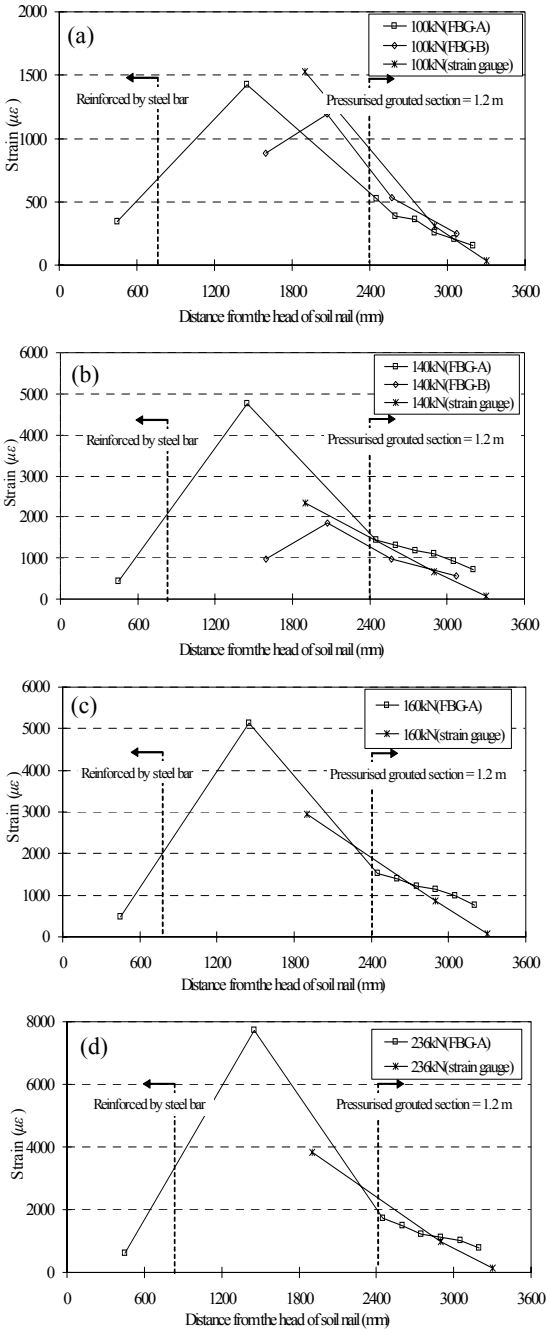

Fig. 7. Comparison of strains measured by FBG sensors and electrical strain gauges on soil nail No.4

electrical strain gauges that FBG sensors are reliable for strain monitoring.

2 In comparison to electrical strain gauges, the FBG sensors have the advantages of multiplexing and high resolution.

3 The two protection methods in this paper are proven to be effective in ensuring both fibre protection and efficient strain transfer in both laboratory tests and in-situ tests. However, the average FBG sensors were damaged under large loads. Special consideration and design shall be made to increase the upper limit of the average FBG sensors for measuring large strains.

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Technical Development in Enhancement of Appearance and Aesthetics of Man-made Slopes

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Abstract: The widespread urbanization on the hilly terrain of Hong Kong in the past resulted in the formation of a considerable number of slopes. Chunam plaster had been the conventional method for surface protection of slopes, whereas shotcrete finish gradually gained popularity in the mid-1990s. Whilst the hard covers can provide good protection to slopes against infiltration and erosion, they are visually intrusive. In response to the rising expectation from the public, a number of technical development initiatives have been undertaken since late 1990s for improving the appearance and aesthetics of man-made slopes so as to blend them with the surroundings, without compromising on safety. One of the initiatives is the preparation of the award-winning GEO Publication No. 1/2000, which provides comprehensive guidance on good practice for landscape treatment and bio-engineering for man-made slopes. Other initiatives include trying out new greening techniques on steep slopes and assessing their performance; and exploring the feasibility of using renewable energy for extracting water from stream courses for watering vegetation on slopes.

1 INTRODUCTION

Hong Kong's substantial economic expansion since 1970s has been accompanied by extensive civil engineering and building works on this hilly terrain, thus resulting in the formation of a considerable number of man-made slopes and retaining walls amid the dense urban environment. There are now approximately 57,000 registered man-made slopes in Hong Kong. About 70% of them are maintained by the government and the remainder by private owners. Many of them lie along two sides of roads and highways, or located close to residential buildings. They are highly visible and their appearance therefore can have significant impacts on the living environment.

2 FROM THE OLD DAYS TO THE PRESENT-DAY INITIATIVES

Since 1977, the Geotechnical Engineering Office (GEO) of the Civil Engineering and Engineering Department (CEDD) of the Government of the Hong Kong Special Administrative Region (SAR) has been carrying out landslip preventive works on man-made slopes to reduce landslide risk. Various government departments have also been undertaking slope maintenance works. Chunam plaster had been the conventional method for surface protection of slopes. In the 1990s, sprayed concrete gradually gained popularity as an alternative to chunam plaster. Upgrading and maintenance works in those days relied heavily on sprayed concrete to protect the slope surfaces.

Whilst the hard covers can provide good and immediate surface protection to slopes against rainwater infiltration and erosion of slope forming material, they are, however, visually unattractive as well as heat reflective. The widespread and increasing application of such visually intrusive slope covers, particularly for slope maintenance works, generated considerable public concerns in the late 1990s.

In response to the surging community interest in slope appearance, the Hong Kong SAR Government has taken many initiatives to improve the aesthetics of man-made slopes. The prime objective is to blend them with their surroundings, but without compromising on safety. One of the initiatives is producing comprehensive guidelines for the engineering profession on good practice for landscape treatment and bio-engineering for man-made slopes and retaining walls.

Another initiative is trying out new greening techniques on steep man-made slopes and assessing the performances of these techniques. The CEDD and other slope maintenance departments have been implementing trials of different new greening techniques on steep slopes in the recent years. The GEO carried out a study to assess the performance of these greening techniques.

In addition, the GEO took the initiative to explore the feasibility of using renewable energy for extracting water from underground or natural stream courses for watering vegetation on slopes.

3 PUBLICATIONS AND GUIDELINES

To address the public concern on slope appearance, the GEO in 1999 initiated a study entitled "Review of Effective Methods of Integrating Man-made Slopes and Retaining Walls (Particularly for Roadside Slopes) into Their Surroundings" (GEO Report No. 116) (Halcrow China Ltd., 2001). The study provided a review of prevailing landscape treatments used and practice adopted locally and internationally to reduce visual impact of slopes, and how effective they were in visually blending slope features into their surroundings. In addition, the review identified the need to improve the existing slope landscape guidelines and to incorporate landscape considerations in early stage of the slope design process.

Based on the results of the above review, the GEO produced a publication entitled "GEO Publication No. 1/2000 – Technical Guidelines on Landscape Treatment and Bio-engineering for Man-made Slopes and Retaining Walls" (GEO, 2000) in September 2000. The publication is intended to assist engineers in designing and implementing landscape and bio-engineering treatments for man-made slopes and retaining walls.

The publication can be divided into three sections. The first section primarily introduces landscape concepts and definitions. It also focuses on the approach of how landscape and geotechnical engineering should integrate within the design, implementation and maintenance processes. Major landscape design principles that designers should pay attention to include:

- Blending the slope feature into local topography and landscape to reduce visual contrast between the slope and the surrounding natural topographical pattern.
- Allowing for retention of existing vegetation, especially healthy mature trees.
- Maximizing planting opportunities, with the use of ecologically sustainable planting design.
- Minimizing visual impact of engineering elements, e.g. soil nail heads, drainage channels, accesses and signage plates.
- Minimizing long-term landscape maintenance commitments.

To illustrate the design principles, the publication presents 15 worked examples which provide landscaping solutions to typical scenarios of different types of slope and retaining wall features in Hong Kong. The examples are intended to illustrate how the relevant landscape and visual issues of a feature can be dealt with during the design process.

The publication also contains extensive technical appendices providing supporting technical information and photographic illustrations. Information provided includes:

- Order of capital costs and annual maintenance costs for some common landscape works
- Suggested plant species for slopes with illustrations
- Suggested planting mixes for slopes and retaining wall under different scenarios

The publication was presented the Grand Award in the "Outstanding Green Project Awards 2000" organized jointly by the Leisure and Cultural Services Department of the Hong Kong SAR Government, the Hong Kong Institute of Landscape Architects and the Society of Horticulture, Hong Kong.

To supplement the guidelines given in the above publication, the GEO, in 2002, produced a report entitled "Guidelines on Safe Access for Slope Maintenance" (GEO Report No. 136) (Lam *et al*., 2003). Slope maintenance, which is vital to the continued stability of slopes, often involves working at height. This report identifies a wide range of methods for access provision, some in the form of permanent access and others ad hoc measures. Selection of safe access methods is also discussed. As some components of the safe access may reduce the visual quality of the slope and its surroundings, the report also provides basic guidelines on making these access provisions as inconspicuous as possible.

4 TRIALS AND PERFORMANCE ASSESSMENT OF NEW GREENING TECHNIQUES

In order to enhance aesthetics of slopes, the GEO/CEDD and other slope maintenance departments have conducted trials of different new greening techniques on top of the hard surface cover of man-made slopes in the past few years. The GEO carried out a study to assess the performance of these greening techniques from two perspectives, namely, (i) the vegetation performance, i.e. the growth condition and sustainability of the vegetation, and (ii) the engineering performance of the techniques.

4.1 *Types of greening techniques*

The study covers 16 proprietary products that have been used in the trials. The products can be broadly classified into the following four types according to their general characteristics:

(1) Mulching System – There are eight products under this system. They generally involve one or more layers of mulch/soil mix on the slope, usually protected by a layer of steel wire mesh and/or one or more layers of erosion control mats (Fig. 1). The mulch/soil mixes vary in composition, type, thickness and additives among different products and in general have little tensile and shear strength.

(2) Cellular System – There are six products under this system. They generally contain compartments or panels of mulch or soil mix installed on the slope face (Fig. 2). The mulch/soil mix of one product is embedded into holes excavated on the slope face, exposing the top surface for planting.

(3) Reinforced Soil System – The one product under this category involves spraying soil mix and applying continuous synthetic fiber at the same time as reinforcement to the soil mix on the slope surface (Fig. 3).

(4) Planter Hole Method – The one product under this category involves direct planting of vegetation in planter holes formed on the hard slope cover (Fig. 4).

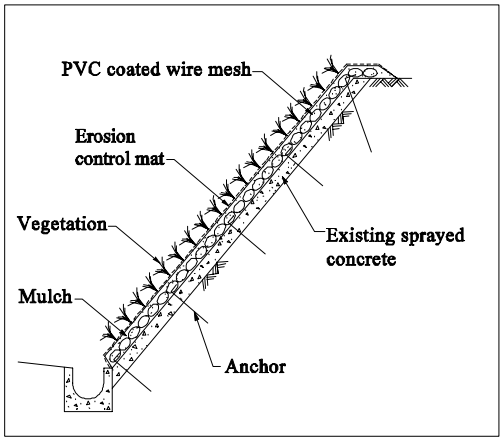

Fig. 1. Typical details of a product of the Mulching System.

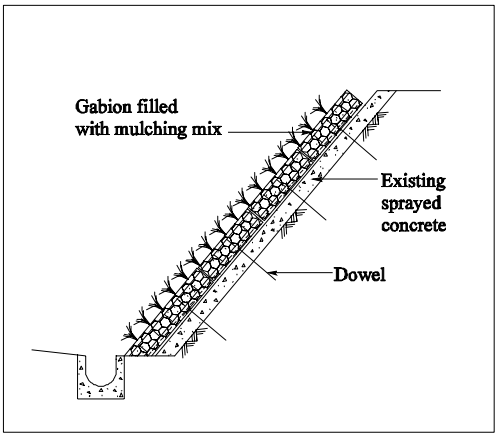

Fig. 2. Typical details of a product of the Cellular System.

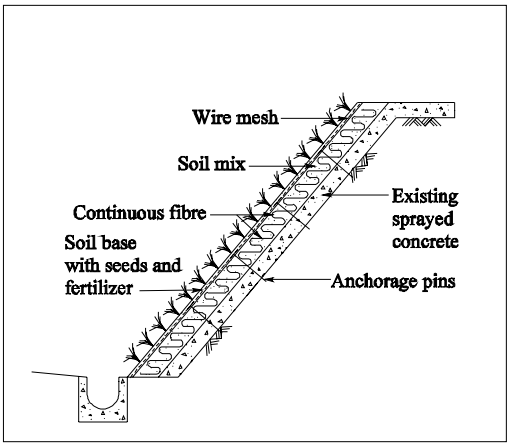

Fig. 3. Typical details of a product of the Reinforced Soil System.

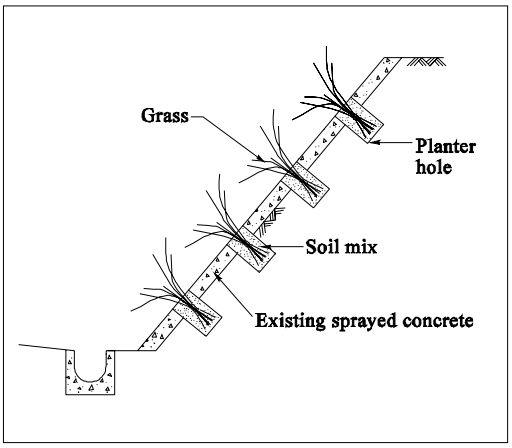

Fig. 4. Typical details of a product of the Planter Hole Method.

4.2 *Methodology for vegetation performance assessment*

The assessment with respect to vegetation establishment was based on the growth performance and sustainability of the vegetation planted or occurring naturally on the slopes. Three field inspections were performed in the autumn of 2002, the spring and autumn of 2003 respectively to evaluate the growth condition of the vegetation under different climatic conditions and the vegetation's ability to regenerate. The field inspections included identification of plant species; assessment of the percentage cover of dominant species and general health condition. Site characteristics including slope gradient and orientation, signs of seepage, distance of the slope from structures that would shade the slope, degree of isolation of the slopes from natural vegetation and degree of exposure of the slopes to road traffic were also noted in the field inspections.

In this study, species richness (**S**) is used for representing the number of woody native or exotic plant species that are believed to have established naturally on a slope since the greening works completed. Furthermore, the Shannon diversity index (**H**), which is a common quantitative index of species diversity, is used for describing the composition of plant communities that have been established naturally on a slope. The index **H** is calculated on the basis of **S** and is the sum of the proportion of individuals that each species contributes to the total in the sample. If it is given that two slopes, A and B, have the same **S** value, but on Slope A, one woody plant species dominates over other species and on Slope B, all plant species have equal distribution. Then index **H** of Slope B should be higher than that of Slope A, as a higher value of **H** means that the community has a higher evenness and a higher diversity of species.

4.3 *Methodology for engineering performance assessment*

The assessment included desk study of installation details of the products and inspection of the engineering elements of the products. The engineering performance of the products were assessed on the basis of the effectiveness of erosion control and structural integrity which includes the stability of the mulch or soil mix and the anchorage of the product to slope surface.

4.4 *Results of the assessment*

The vegetation and engineering performance varied widely among the proprietary products. Details of the results of the performance assessments are given in Lui & Shiu (2006). A summary of the results is presented below:

(1) Mulching System

Results of the assessment for vegetation growth show that, for three products under the mulching system, most slopes had consistently high percentage of healthy vegetation cover of over 70% and these slopes were planted with a climber species, *Wedelia trilobata*, as the main vegetation species. The coverage of the planted grasses on most slopes surveyed was found decreasing with time. For the other five products under this system, including two reinforced with cement in their mulch, they had low coverage of healthy vegetation cover. For diversity and richness of woody species naturally established, most slopes were found to have relatively low values of diversity index **H** and species richness **S**. This shows that slopes treated with such technique are not particularly good in supporting naturally established woody plant species.

From the engineering assessment, signs of deterioration of the mulch/soil mix were observed on slopes of a few products. The signs included surface cracks associated with desiccation, detachment (Fig. 5), bulges near slope toes, etc. These highlight the need for more structural support for the mulch/soil mix of some products. Majority of the products used temporary degradable mat for erosion protection irrespective of slope angle. The cracked mulch/soil mix could be washed away after the mat had decomposed, especially on steep slopes and at areas of high flow concentration. Edges of some products were found to be eroded, especially where adjacent to stepped channels.

Fig. 5. Detachment of mulch.

(2) Cellular System

Assessment results showed that growth performance of vegetation on products of this technique was variable. Grass did not grow well but the climber *Wedelia trilobata* grew well on some slopes. The relatively low **H** and **S** values recorded on most slopes suggested that very few woody species have naturally established. Isolated panels used in two products tended to dry up quickly probably owing to their large exposed surface areas, resulting in unhealthy plant growth with time. The mulch/soil mix, as a result, disintegrated and was easily eroded.

For one of the products, the mulch/soil mix was placed in holes excavated onto the slope surface. The top surface of the mix is exposed for planting whereas the mix can be in a direct contact with the material in the slope. The healthy vegetation coverage recorded on the slope with this product were consistently satisfactory in all inspections, probably owing to the supply of moisture and nutrients from the slope forming material to the vegetation through the holes. From the engineering perspective, there is not much concern about the stability problem of the mulch/soil mix. One product involved planting in holes spaced along PVC pipes anchored on slope surface, the performance of which is unsatisfactory in terms of vegetation growth and coverage.

(3) Reinforced Soil System

Vegetation assessment results showed that the slopes using this product had consistently satisfactory performance of healthy vegetation cover, especially with the planting of *Wedelia trilobata*. For the naturally established woody plants, relatively low **H** and **S** values were recorded on one slope whilst the other three had zero **H** and **S,** indicating that the number of woody plants established naturally were very low. The engineering condition of this product was satisfactory.

(4) Planter Hole Method

All slopes using this method were planted with Vetiver grass (Fig. 6) which was the sole vegetation cover of these slopes. The assessment results showed that all slopes with this product had consistently high or improving healthy vegetation cover between different inspections. However, old leaves of the Vetiver grass used in this product turned yellow and very few new leaves were able to emerge. The persistence of the old leaves owing to low rate of decomposition seems to inhibit new leaves from developing. In terms of engineering performance, no particular problem was identified.

Fig. 6. Planting of Vetiver grass.

4.5 *Discussion*

Different proprietary products use different materials and methods of construction. Based on the results of the vegetation and engineering assessments, the merits and limitations of different techniques are generally summarized in Table 1.

The overall performance of slopes treated with different greening techniques was in general satisfactory during the study period. Some products performed relatively better than the others under certain circumstances by having consistently satisfactory vegetation cover in the study period.

Table 1. Summary of merits and limitations of different greening techniques.

Techniques	Merits	Limitations
Mulching System	• Able to form smooth finished surface on uneven slope profile • Quick installation	• Mulch/soil mix is generally weak in strength and liable to crack and fall down • Mulch/soil mix layer thin, unable to support small woody species
Cellular System	• Some products allow vegetation to be established at nursery before installation of panels of soil mix on slopes. This can provide an instant greening effect. • One product allows vegetation to have direct contact with slope-forming soil.	• Small isolated panels of soil mix installed on slope surface may dry up, affecting plant growth and health • When without a dense vegetation cover, wire mesh of the panels of mulch/soil mix very visible and not aesthetically pleasing
Reinforced Soil System	• The use of fibre reinforcement permits a thicker soil mix which may support woody vegetation such as shrubs and small trees rather than grass species only.	• The thicker planting medium may limit the product's use on slopes with limited space in front of the slopes.
Planter Hole Method	• Direct planting of vegetation through planter holes into the soil allows more water and nutrients for vegetation than the other products which are applied on top of slope cover.	• Some species require regular cropping provide space for new growth. • Hard slope cover is exposed after cropping, leading to temporary visual impact. • Monotonus appearance due to the use of one single species

Slopes were dominated by the planting of climber, *Wedelia trilobata* and the grass species, *Cynodon dactylon* and *Paspalum notatum*. When *Wedelia trilobata* was planted together with grass species on some slopes, it generally grew better than the planted grass species and dominated the vegetation cover irrespective of the type of greening techniques, giving rise to a monotonous appearance. While this species is effective in greening slopes and can be used on slopes with a harsh environment where other species generally do not perform well, its use in wetland

Fig. 7. Details of the use of steel wire meshes in conjunction with non-degradable erosion control mats.

marshes, streams, canals or coastal areas should be avoided because it is known to be particularly invasive in such habitats.

Results of the vegetation assessment also showed that the diversity index **H** and species richness **S** values of most slopes are low, indicating that there was little diversity in plant species established naturally on the slopes using such techniques. This might have been due to the use of a thin layer of planting medium on top of a hard slope cover which cut off the supply of moisture and nutrients from the soil behind the slope cover to the vegetation. As such, planting directly on the soil surface of a slope is preferred to those techniques used on the shotcrete cover provided that there is no stability concern of the slope. This planting strategy can sustain a greater diversity of species, thus enhancing the ecological value of slopes. In addition, the use of steel wire meshes in conjunction with non-degradable erosion control mats on soil slope surface for planting (Figs. 7 and 8) is more effective in controlling surface erosion.

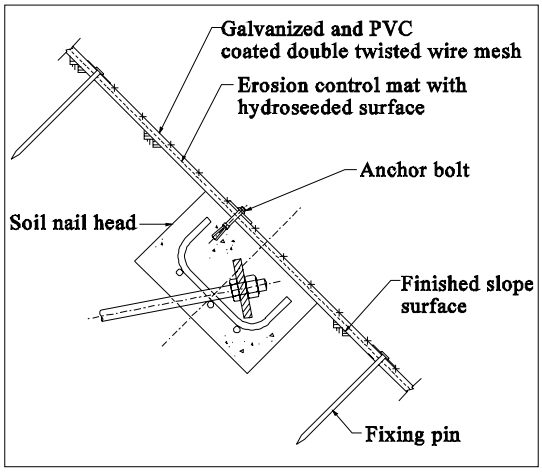

Fig. 8. Fixing details of steel wire mesh and erosion control mat on slope face with soil nail heads.

For almost 70% of the slopes in the study, the greening products had only been applied for less than three years at the time when the study was carried out. Furthermore, some products had been used on a few slopes only. As such, the long-term performance of the products in terms of their engineering performance and ability to sustain vegetation was still uncertain. In view of this, a further study is now in progress to review the long-term performance and sustainability of these greening techniques. Slopes with the greening products applied on for more than five years will be selected and re-examined.

5 AUTOMATIC SOLAR POWER IRRIGATION SYSTEM

5.1 *Background*

Vegetation on slope surface does not only add greenery to the environment, it can also enhance the local ecology. From slope engineering perspective, vegetation cover helps prevent soil erosion. The long-term stability of soil slopes relies on the establishment and healthy growth of vegetation.

It is difficult to establish and maintain vegetation on most man-made slopes without adequate watering. Grass and shrubs turn brown during dry seasons. Manual watering of vegetation can be costly, labour intensive and often not practical for high slopes or remote areas. In order to keep the vegetation on slopes healthy all the year round, the GEO has been exploring the feasibility of using the renewable solar energy for extracting water from underground or stream courses for watering vegetation on man-made slopes.

5.2 *Field trial*

In the field trial, an automatic solar power irrigation system has been installed on a reinforced fill slope located in Lantau Island. Fig. 9 shows an overview of the 120m-long slope which inclines at about 45° and has a maximum height of about 8m. Vegetation on the slope mainly comprises grass and shrubs, with a few trees. This slope is selected for the trial because it is located in a remote and open area. This would minimize human disturbances to the plants and the equipment and maximize the amount of sunlight for generation of electrical energy. Besides, there is a natural stream course close to the slope with water flow all year round. Water from this stream course is used for watering the vegetation on the slope.

5.3 *Details of the irrigation system*

Four photovoltaic (PV) panels, each measuring 1.5m x 0.8m on plan, are used for generating electrical energy which is stored in rechargeable batteries. The PV panels are oriented towards the south to receive more sunshine. The total maximum power output of the panels is 600W. The batteries provide power for a submersible pump and a booster pump as well as a control unit. The submersible pump is used for pumping water from the stream course for temporary storage in a water tank. Water in the tank is then drawn by the booster pump to a number of sprinklers installed along the slope toe for watering the vegetation on the slope.

The irrigation system is programmed in the control unit to run automatically every day. The whole slope to be irrigated is divided into three zones, each of which is served by two to three sprinklers. With the use of control valves, the system is programmed to irrigate the three zones alternately, each at a frequency of once every three days. This reduces the demand for water at any one time so as to avoid the use of a massive size of storage tank and a large number of batteries. About 2,000 litres of water are consumed in each irrigation. The operating time for each irrigation is limited to 30 to 45 minutes, corresponding to an average rainfall intensity of less than 10mm/hour. This is to avoid surface erosion on the slope resulting from strong water

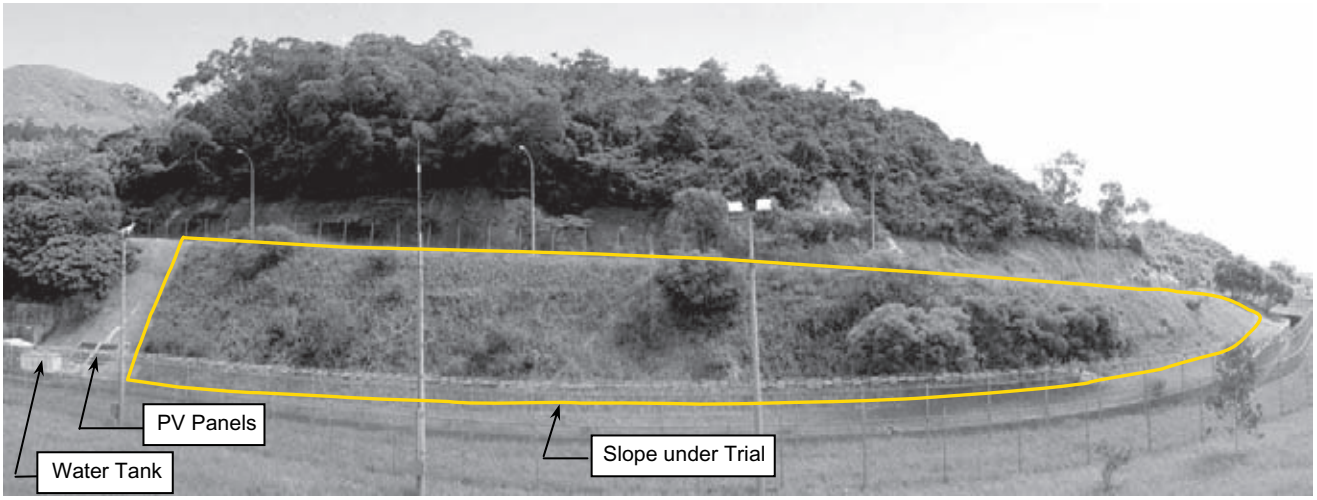

Fig. 9. Overall view of the slope under trial.

spray. After each irrigation, water is pumped automatically from the stream course to the water tank for the next irrigation cycle. Details of the system are given in Lui (2006).

A special feature of the system is that the rechargeable batteries can store power sufficient for the system's normal operation for three consecutive days. This enables the system to run on overcast days when supply of sunlight is limited. Another feature is that a rain detector unit has been installed so the irrigation function of the system can be skipped in rainy days to save water and energy.

5.4 *Benefits of the system*

The use of the automatic irrigation system powered by solar energy can greatly reduce the difficulties in manual watering of vegetation on slopes of difficult access, such as high slopes along expressways on which parking or slowing down of vehicles for watering is not allowed, and slopes in remote area where water and electricity supply is not readily available.

As the system utilizes groundwater or water from stream courses, it will avoid the need for tap water which is a precious resource in Hong Kong. Also the solar energy used for operating the irrigation system is renewable and freely available.

Results of the field trial have demonstrated the feasibility of the solar power irrigation system, although setting up the system is costly. If irrigation is considered necessary for a site, the use of an automatic irrigation system operated by solar power can be an option. With the knowledge and experience gained from the field trial, relevant guidelines on the use of such irrigation system have been prepared and are given in Lui (2006).

6 CONCLUSION

Since its establishment in 1977, the GEO has been exercising its expertise to enhance slope safety. The GEO has also been concerting efforts to provide good aesthetics and appearance of man-made slopes. From the chunam plasters in the old days of 1970s to the greenery in the new century, man-made slopes in the territory have been changing their appearance to enliven the environment and upgrade our living quality. The production of technical guidance documents such as the award-winning GEO Publication No. 1/2000, and development of greening techniques like the use of steel wire mesh and erosion control mats are significant technical developments in the enhancement of appearance and aesthetics of man-made slopes.

ACKNOWLEDGEMENTS

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Limit Equilibrium Methods for Soil Nail Design

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Abstract: This paper describes a comparative study among four different Limit Equilibrium Methods (LEM) of slices in stability analysis of soil-nailed slopes. Effects of different patterns of nail loads applied at different locations of a soil slope are investigated. Results of the study indicate that different LEM can yield different factors of safety (FoS) of nailed slopes. Also, the FoS values computed using some of the methods are sensitive to the assumed locations of nail forces, while the other methods give FoS that are insensitive to nail force locations. Depending on the nail patterns and locations, the Janbu's Simplified method may give a conservative or an unsafe solution. All the methods have problems of non-convergence, with some methods being more serious than the other. The paper presents and discusses the findings, and provides suggestions on some design aspects of soil nailing using the LEM of slices.

1 INTRODUCTION

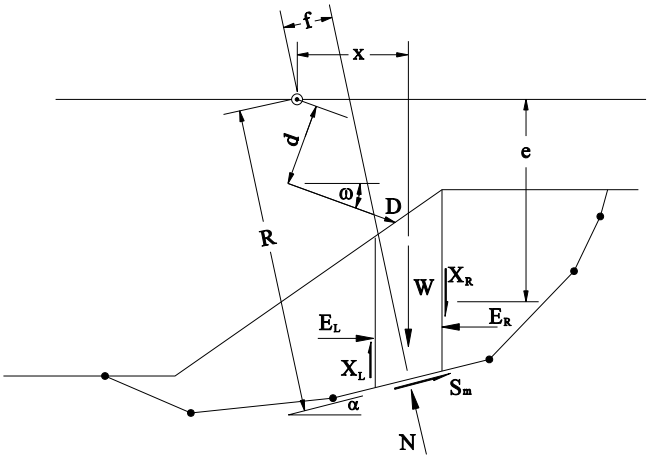
The technique of soil nailing has been applied for stabilizing existing and new slopes in Hong Kong for two decades. Because of their simplicity, Limit Equilibrium Methods (LEM) of slices are routinely used for designing soil nails There are however concerns over the use and limitations of the LEM. Some of these concerns are: (i) whether different methods would yield different design results; (ii) whether different assumptions in respect of the distribution and locations of nail forces would affect the design results; (iii) whether the nail forces should be applied at the slope surface or at the location where the line of action of the force intersects the slip surface; (iv) whether the nail forces should be applied in the same direction of the soil nails or just the horizontal component of the nail forces should be used; and (v) the problem of non-convergence is frequently encountered in analysis.

A study has recently been carried out to examine the above issues relating to the design of soil-nailed cut slopes.

2 LIMIT EQUILIBRIUM METHOD OF SLICES

This study reviews and compares the four two-dimensional (2-D) methods of slices, namely the Janbu's Simplified (JS) method (with and with no correction factor), the Janbu's Generalized (JG) method, the Bishop's Simplified (BS) method and the Morgenstern-Price (M-P) method in respect of soil nail design. These methods mainly differ in the equations of static equilibrium to be satisfied and the relationship between the inter-slice normal and shear forces.

Fig. 1 shows a typical slice in a potential sliding mass and the forces acting on the slice.



Soil Strength Parameters:

s	=	Shear Strength
c'	=	Effective Cohesion
ϕ'	=	Effective Angle of Internal Friction
u	=	Pore-water Pressure

Slice Parameters:

W	=	The total weight of a slice of width b and height h
N	=	The total normal force on the base of the slice
S_m	=	The shear force mobilized on the base of each slice
E	=	The horizontal interslice normal forces. Subscripts L and R designate the left and right sides of the slice, respectively
X	=	The vertical interslice shear forces. Subscripts L and R define the left and right sides of the slice, respectively
D	=	An external line load
R	=	The radius for a circular slip surface or the moment arm associated with the mobilized shear force, s_m for any shape of slip surface
σ_n	=	$\frac{N}{\beta}$ = average normal stress at the base of each slice
β	=	The base length of each slice
f	=	The perpendicular offset of the normal force from the center of rotation or from the center of moments
x	=	The horizontal distance from the centerline of each slice to the center of rotation or to the center of moments
e	=	The vertical distance from the centroid of each slice to the center of rotation or to the center moments
d	=	The perpendicular distance from a line load to the center of rotation or to the center of moments
ω	=	The angle of the line load from the horizontal
α	=	The angle between the tangent to the center of the base of each slice and the horizontal

Fig. 1. Forces acting on a slice through a sliding mass defined by a fully specified slip surface (after Fredlund *et al*., 1981).

Table 1 summarises the conditions of static equilibrium satisfied by the force limit equilibrium methods of slices. For soil nail design using these methods, it is often assumed that the nail forces do not modify the inter-slice forces. Details of these four methods are discussed in Abramson *et al.* (2002) and Shiu *et al.* (2006).

Table 1. Elements of statical equilibrium satisfied by various Limit Equilibrium Methods of slices.

Method	Force Equilibrium		Moment Equilibrium
	Vertical	Horizontal	
Bishop's Simplified	Yes	No	Yes
Janbu's Simplified	Yes	Yes	No
Janbu's Generalized	Yes	Yes	**
Morgenstern-Price	Yes	Yes	Yes

Legend:
** Moment equilibrium is used to calculate interslice shear forces

In addition to the four methods, particular mention is made to the general limit equilibrium (GLE) formulation which was developed by Fredlund in the 1970s (Fredlund & Krahn, 1977; Fredlund *et al.,* 1981). The GLE formulation is based on two factor of safety (FoS) equations: one for moment equilibrium (F_m) and one for horizontal force equilibrium (F_f) as shown in Eq. (1) and Eq. (2) respectively.

$$F_m = \frac{\sum\left[c'\beta R + \left(N - u\beta\right)R\tan\phi'\right]}{\sum Wx - \sum Nf \pm \sum Dd} \tag{1}$$

and:

$$F_f = \frac{\sum\left[c'\beta\cos\alpha + \left(N - u\beta\right)\tan\phi'\cos\alpha\right]}{\sum N\sin\alpha - \sum D\cos\omega} \tag{2}$$

where N is the normal force acting at the base of a slice surface. It is given by:

$$N = \frac{W + \left(X_R - X_L\right) - \dfrac{c'\beta\sin\alpha - u\beta\sin\alpha\tan\phi'}{F} + D\sin\omega}{\cos\alpha + \dfrac{\sin\alpha\tan\phi'}{F}} \tag{3}$$

The denominator of Eq. (3) is often referred to as m_α:

$$m_\alpha = \cos\alpha + \frac{\sin\alpha\tan\phi'}{F} \tag{4}$$

The definitions of the variables in Eqs. (1) to (3) are given in Fig. 1. The derivations of the equations can be found in Fredlund *et al.* (1981) or Krahn (2003). One special feature of the GLE formulation is that it can produce plots of factor of safety versus λ, where λ defines the relationship between inter-slice shear forces and inter-slice normal forces. The M-P method assumes that the inter-slice shear forces (X) are related to the inter-slice normal forces (E) as:

$$X = E\lambda f\left(x\right) \tag{5}$$

where $f(x)$ is a function, λ is the percentage of the function used.

Fig. 2 shows an example of a plot of FoS versus λ. Both the BS method and JS method ignore inter-slice shear forces. Having no inter-slice shear forces means that λ is zero. Since the BS method satisfies only moment equilibrium, the Bishop factor of safety falls on the moment curve in Fig. 2 where λ is zero. Similarly, as the JS method only satisfies overall horizontal force equilibrium, the Janbu factor of safety (without correction factor) falls on the force curve in Fig. 2 where λ is zero. The M-P factor of safety is determined at the point where the F_m and F_f curves intersect (see Fig. 2). The factor of safety satisfies both moment and force equilibrium at this point. If a constant inter-slice function $f(x)$ is used, the intersection point is the same as that computed using Spencer's method (Spencer 1967).

The λ plots as shown in Fig. 2 are useful for understanding the reasons for the differences in the assumptions in the inter-slice shear forces and the conditions of static equilibrium among the JS method, the BS method and the M-P method. They can also be useful to identify the location of FoS. The λ plots may also assist in overcoming some non-convergence problems as discussed later.

3 EFFECT OF DISTRIBUTION AND LOCATION OF APPLIED NAIL FORCES

3.1 *General*

To examine the effect of the location of applied nail forces, stability computations have been performed on a nailed slope. The stabilizing effect of soil nails is modelled as external forces on the slope.

To provide a set of nail forces as input in the methods of slices, reference is made to the nail forces derived from some previous analyses using the two-dimensional finite difference code, Fast Lagrangian Analysis of Continua (FLAC). Details of the FLAC analyses can be found in Shiu & Chang (2005).

In the M-P method, the inter-slice function $f(x)$ is assumed to be constant. Result of a sensitivity analysis has shown that the constant function yields FoS values almost the same as the half-sine function.

3.2 *Unreinforced model slope*

Before the effect of the location of applied nail forces is reviewed, analysis of the slope model without soil nails is first presented. Fig. 3 shows the geometry of the unreinforced model slope and the material parameters used for the present study. The slope is 20 m in height and standing at an angle of 55°. It has an up-slope angle of 10°. The shear strength parameters of the soil are assumed to be c' = 10 kPa and φ' = 43°. FoS values are determined by the four methods of slices. The computer software package SLOPE/W 2004 developed by GEO-Slope was used in the analysis. Ten non-circular slip surfaces (S1 to S10) through the unreinforced slope were considered. Results of the analysis are shown in Fig. 4. The minimum FoS values obtained are summarised in Table 2.

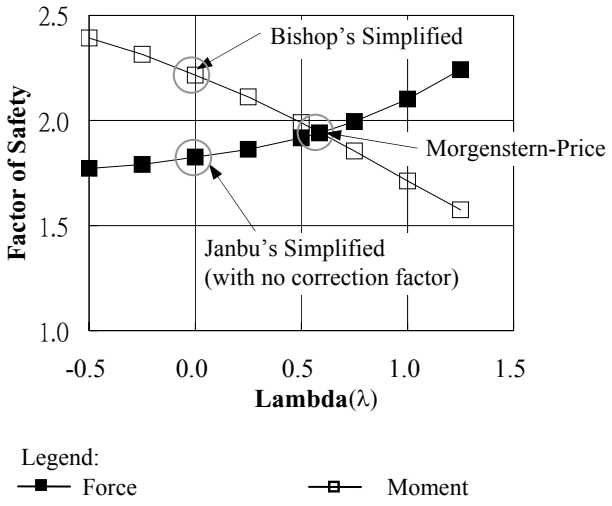


Fig. 2. Typical plot of factor of safety versus Lambda (λ).

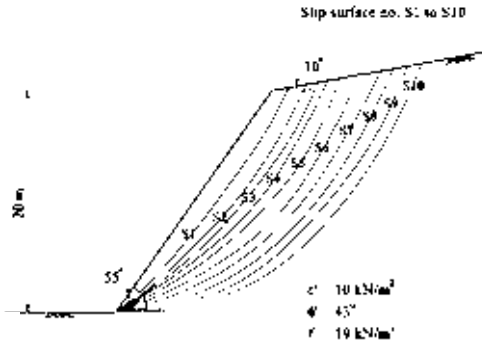


Fig. 3. Geometry and locations of slip surface for the unreinforced model slope.

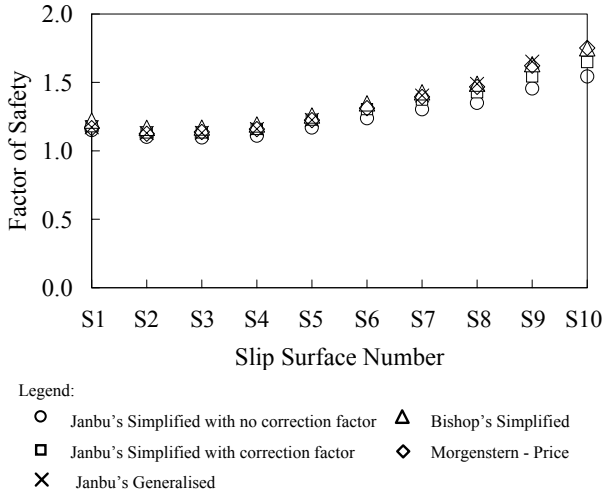


Fig. 4. Factors of safety of the unreinforced slope using different limit equilibrium methods.

3.3 *Model Nailed Slope*

3.3.1 Distribution of axial nail forces

Seven rows of soil nails are provided to the model nailed slope (Fig. 5) and this corresponds to a vertical nail spacing of 2.5 m. The horizontal spacing of the nails is taken to be 1.5 m. The nails are inclined at an angle 10° below the horizontal. Each soil nail is 20 m long with a 40 mm diameter steel bar in a 100 mm diameter grouted hole.

Fig. 6 shows the locations of nail forces and slip surfaces for the nailed slope model used in the limit equilibrium methods of slices.

Table 2. Minimum factor of safety (FoS) using different methods of analyses for the 20 m High model slope.

Methods in Slope/W	
Morgenstern - Price	1.13 (S2)
Bishop's Simplified	1.17 (S2)
Janbu's Simplified with Correction Factor	1.13 (S2)
Janbu's Simplified with No Correction Factor	1.10 (S2)
Janbu Generalized	1.13 (S2)

Note: ( ) Indicates the slip surface no. with the minimum FoS

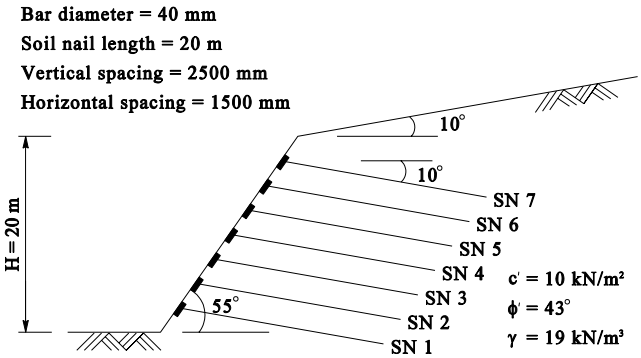


Fig. 5. Geometry and material parameters of model nailed slope.

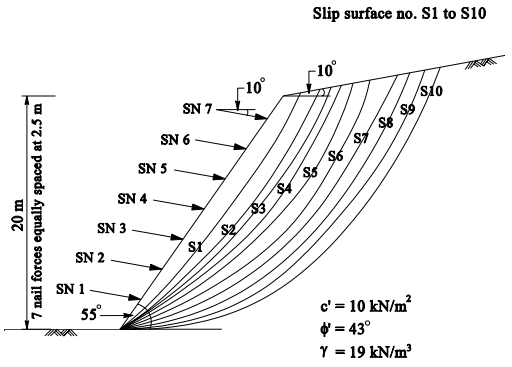
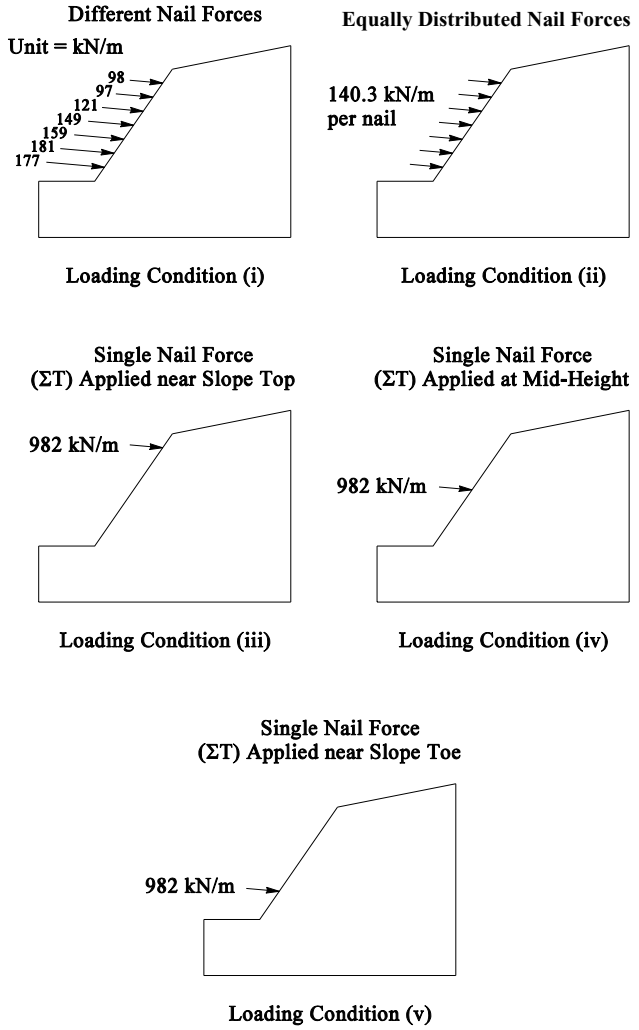


Fig. 6. Locations of nail forces and slip surfaces for the model nailed slope.

From the results of FLAC analysis, the distribution of the axial force developed in each of the seven rows of nails at limit equilibrium is given in Table 3.

Table 3. Distribution of axial nail forces.

Soil Nail Number	Axial Nail Force, T (kN/m)
SN1	177
SN2	181
SN3	159
SN4	149
SN5	121
SN6	97
SN7	98
	$\Sigma T = 982$



Note: Distribution of nail forces for loading condition (i) is obtained from FLAC analyses.

Fig. 7. Loading conditions for the nailed slope.

3.3.2 Loading conditions

To investigate the effects of the distribution and location of nail forces, five loading conditions (Fig. 7) are considered in the analysis:

1 different axial nail forces (T) are applied at individual nail locations;
2 the total of the axial nail forces (ΣT) is distributed equally among the seven rows of nails;
3 the single nail force (ΣT) is applied near the slope crest, i.e. at location of nail SN7;
4 the single nail force (ΣT) is applied at the mid-height of the slope, i.e. at location of nail SN4; and
5 the single nail force (ΣT) is applied near the slope toe, i.e. at location of nail SN2.

In the five loading conditions, the resultant nail forces are the same, i.e. 982 kN/m (ΣT). The nail forces are applied on the slope surface and in the same inclination as the soil nails.

3.4 Results of analysis for model nailed slope

3.4.1 Loading conditions (i): different forces (T) applied at individual nail locations, and (ii): total force distributed equally among all nails

The JG method is unable to produce solutions for any of the slips for the two loading conditions (i) and (ii) due to numerical problems. The problem of convergence encountered in this method is discussed in Section 7 of this Paper. The factors of safety computed by using the other methods are plotted in Fig. 8.

It can be observed that none of the methods can produce a converged solution for slip surface no. S1. Apart from shallow slips (e.g. S2 and S3), the BS, JS and M-P give similar FoS values at given slips for both loading conditions. The difference in FoS for shallow slips can be up to 14%, whereas that for deeper slips is smaller than 5%.

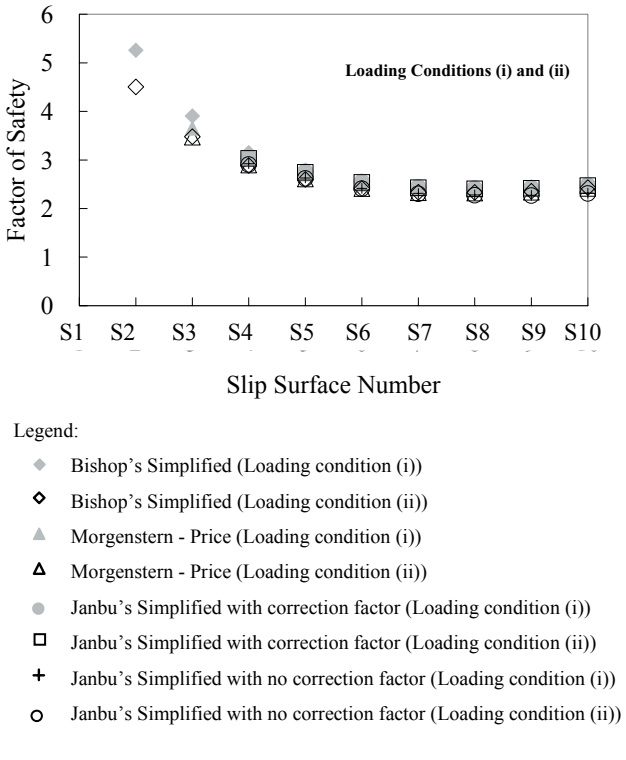


Fig. 8. Factor of safety versus slip surface number for loading conditions (i) and (ii).

Fig. 9(a) and (b) show the FoS versus λ plots for slip surface S8 for loading condition (i) and loading condition (ii) respectively. This illustrates the small difference in computed FoS using the different methods.

3.4.2 Loading condition (iii): single force (ΣT) applied at top of slope

Again, the JG method is unable to produce solutions for any of the slips due to numerical problems.

Fig. 10 shows the FoS obtained for the other three methods at different slip surfaces. The BS method produces converged solutions for all the ten slip surfaces. The JS and the M-P methods cannot compute converged solutions for some of the slip surfaces. The slip surface with the minimum FoS cannot be located for the M-P method due to numerical non-convergence for slips S1 to S5. However, the trend of the FoS curve shows that the minimum FoS is close to that of S6. For a given slip, the JS method gives the highest FoS and this is followed by the M-P method. The BS method usually gives the lowest FoS. The differences between the Bishop FoS and the Morgenstern-Price FoS are however small.

For illustrative purpose, the FoS versus λ plot for slip S8 is presented in Fig. 9(c).

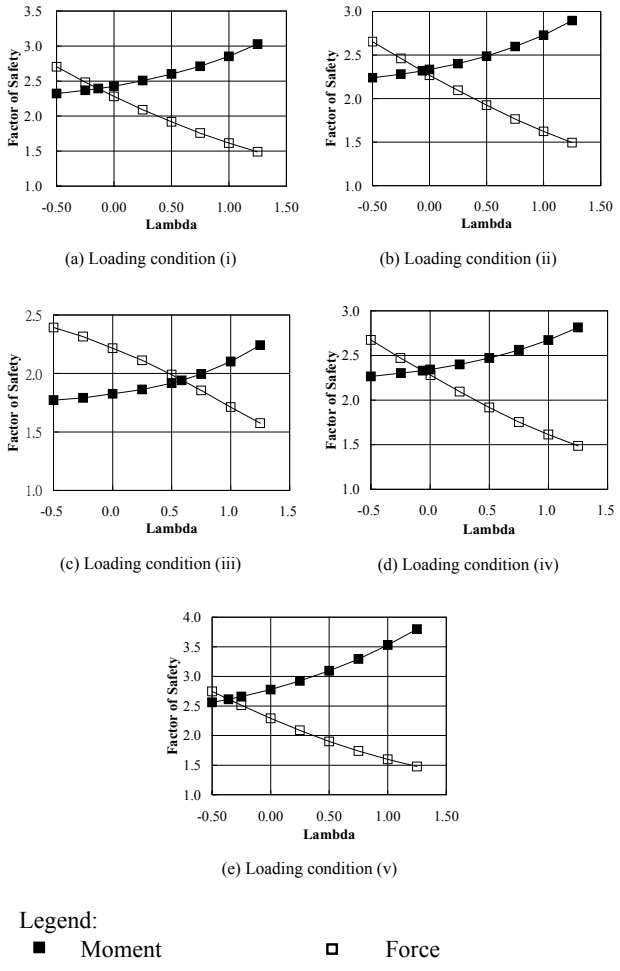


Fig. 9. Factor of safety (FoS) versus Lambda (λ) for slip surface S8 for (a) loading condition (i); (b) loading condition (ii); (c) loading condition (iii); (d) loading condition (iv); and (e) loading condition (v).

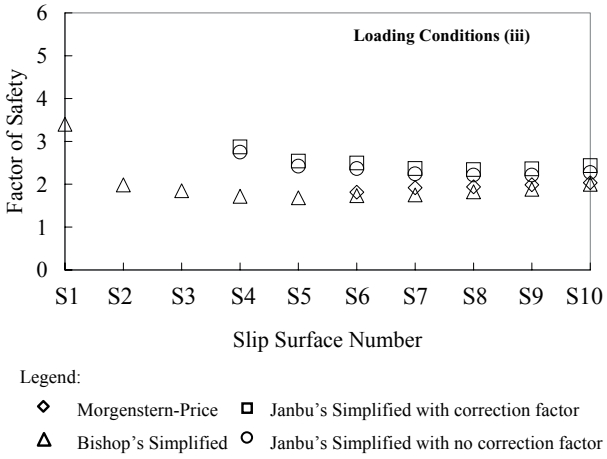


Fig. 10. Factor of safety versus slip surface number for loading condition (iii).

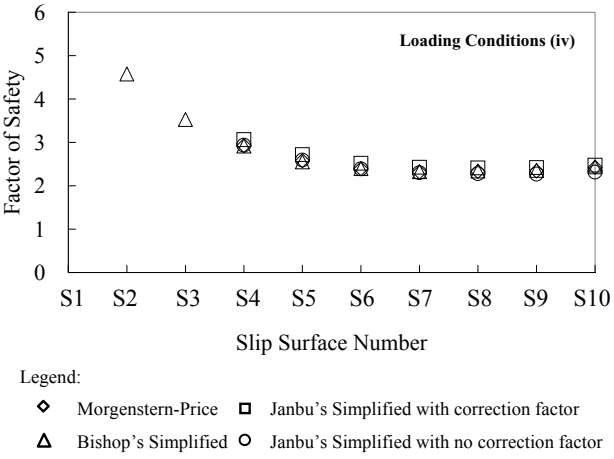


Fig. 11. Factor of safety versus slip surface number for loading condition (iv).

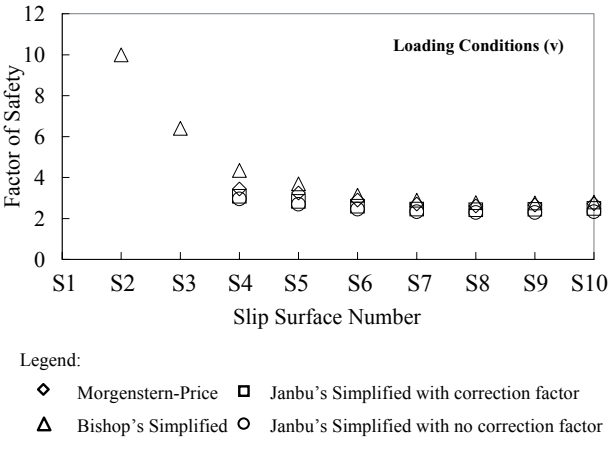


Fig. 12. Factor of safety versus slip surface number for loading condition (v).

3.4.3 Loading condition (iv): single force (ΣT) applied at mid-slope height

The JG method is unable to produce solutions for any of the slips due to numerical problems. Fig. 9 shows the FoS obtained for the other three methods at different slip surfaces. All the methods give similar values for the FoS. Both the M-P and the JS methods have convergence problems for the shallow and steep slips surfaces, S1 to S3. The BS method cannot give a converged solution for slip S1 only.

For illustrative purpose, the FoS versus λ plot for slip S8 is presented in Fig. 9(d).

3.4.4 Loading condition (v): single force (ΣT) applied near slope toe

The JG method is unable to produce solutions for any of the slips due to numerical problems. Fig. 12 shows the FoS obtained for the other three methods at different slip surfaces. All the three methods have problem of convergence for some of the shallow slips.

For illustrative purpose, the FoS versus λ plot for slip S8 is presented in Fig. 9(e).

3.4.5 Single nail force applied at different locations (loading conditions (iii), (iv) and (v))

The JG method did not yield any solution for any of the slips because of convergence problem. As such, no comparison of this method can be made.

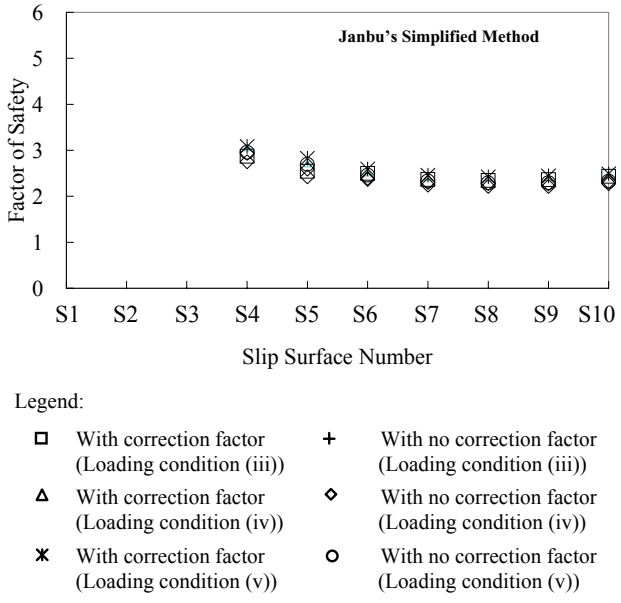


Fig. 13. Factor of safety values computed using Janbu's Simplified Method for loading conditions (iii), (iv) and (v).

Fig. 13 shows the factor of safety values computed using the JS method for the three cases of a single nail force applied at different locations, i.e. loading conditions (iii), (iv) and (v) respectively. The computed FoS are very similar for the three applied locations. This shows that the point of application of nail force has insignificant effect on the result. This is because the JS method does not consider moment equilibrium, and hence the point of application of external force, which affects the lever arm for moment computation, does not come into play.

The factors of safety computed using the BS method for the three loading conditions (iii), (iv) and (v) are plotted in Fig. 14.

For those converged slip surfaces, loading condition (v) gives the highest FoS, whereas loading condition (iii) gives the lowest FoS.

Fig. 15 shows the results for the M-P method. A similar finding of increasing computed FoS from loading condition (iii) to (v) is seen. That means applying the nail force at the slope toe gives a higher FoS than applying at the higher part of the slope. The reason for this is that the location of applied force controls the magnitude of resisting moment in the moment equilibrium methods. The lower the location of the applied nail force is, the larger the moment arm and the resisting moment are.

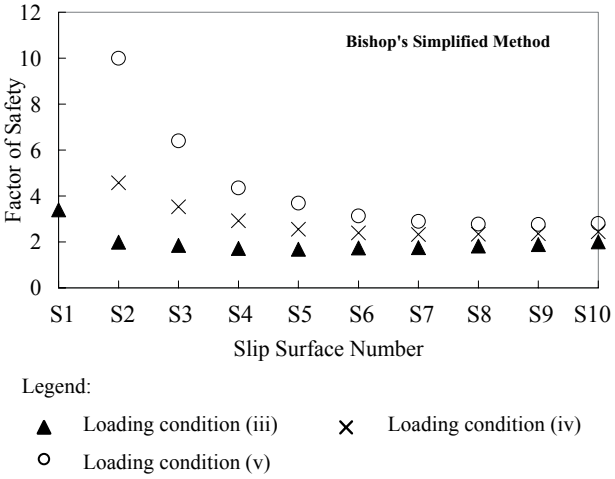


Fig. 14. Factor of safety values computed using Bishop's Simplified Method for loading conditions (iii), (iv) and (v).

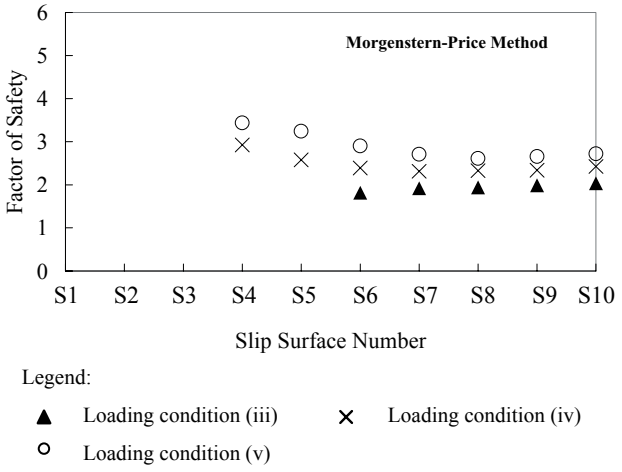


Fig. 15. Factor of safety values computed using Morgenstern-Price Method for loading conditions (iii), (iv) and (v).

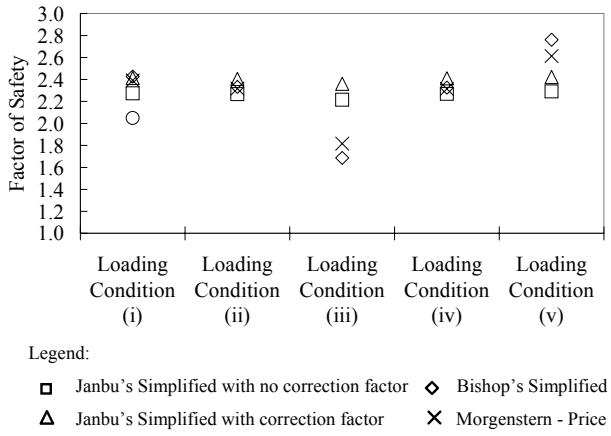


Fig. 16. Minimum factors of safety computed using different methods for loading conditions (i), (ii), (iii), (iv) and (v).

3.4.6 Single nail force versus distributed nail forces

The minimum FoS computed using different methods are shown in Fig. 16 and summarized in Table 4 for the five loading conditions.

Table 4. Minimum factors of safety computed using different methods of analyses.

Methods	Loading Condition (i)	Loading Condition (ii)	Loading Condition (iii)	Loading Condition (iv)	Loading Condition (v)
Morgenstern - Price	2.39 (S8)	2.32 (S8)	*1.82 (S6)	2.32 (S7)	2.61 (S8)
Bishop's Simplified	2.43 (S8)	2.33 (S8)	1.69 (S5)	2.33 (S7)	2.76 (S9)
Janbu's Simplified with Correction Factor	2.41 (S8)	2.41 (S8)	2.36 (S9)	2.41 (S8)	2.42 (S8)
Janbu's Simplified with No Correction Factor	2.27 (S9)	2.27 (S9)	2.22 (S9)	2.27 (S9)	2.29 (S8)
Janbu Generalized	NC	NC	NC	NC	NC

Notes: ( ) Indicates the slip surface no. with the minimum factor of safety
NC Non-convergent
* Minimum FoS cannot be searched due to numerical non-convergence for adjacent slip surface

Before application of soil nails, all five methods show that the slip surface with minimum FoS is shallow and at S2 as shown in Table 2. With the presence of soil nails, the slip surfaces with minimum FoS become more deep-seated. Their locations vary slightly from one to another among the five methods and also the loading conditions (see Table 4). In practice, the slip surface with minimum FoS is not necessarily the most critical one for the determination of soil nail forces.

There is a narrow disparity in the minimum FoS computed using the JS method amongst the five loading conditions. This means that as long as the total nail force is the same, the assumption of distributed forces or a single force does not affect much the results of the JS method.

For the BS and M-P methods, there are only small differences in the computed minimum FoS for loading condition (i), (ii) and (iv). This can be explained. In each of these loading conditions, the resultant nail force is applied either at or close to the mid-height of the slope. Therefore, the magnitude of the resisting moments due to the nail forces are similar. In fact the resultants of different systems of nail forces having the same magnitude and the same line of action on the slope should produce the same FoS.

The above can be looked at further by examining the FoS versus λ plots. Fig. 9 depicts such plots for slip S8 for the five loading conditions. For ease of illustration, they are combined into one plot in Fig. 17. As noted from the Figure, the FoS for moment equilibrium is sensitive to the location of the resultant nail force. It is because nail forces applied at different locations have different moment arms.

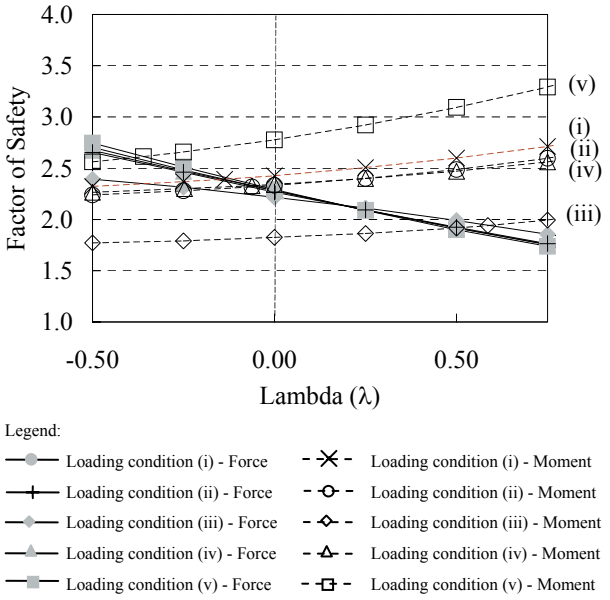


Fig. 17. Combined plot of FoS versus λ for slip surface S8 for loading conditions (i), (ii), (iii), (iv) & (v) for the nailed slope.

3.5 Discussion

As indicated in Fig. 16, under loading condition (iii), the minimum FoS given by the JS method (with no correction factor) is higher than that given by the M-P method. This is because the JS method only satisfies force equilibrium and is insensitive to the location of applied force. In the case like loading condition (iii), the solutions given by the JS method (with no correction factor) will be on the unsafe side. The JS method, whether a correction factor is applied or not, gives an approximate solution, which will be good enough for analysis that does not involve an externally applied force. In the case of soil-nailed slope, the error associated with the JS method due to ignoring the moment equilibrium may either err on the conservative or on the unsafe side, depending on the locations of the nail forces.

The M-P method satisfies both force and moment equilibrium of slices and gives a more exact solution. The present study confirms that the method is sensitive to the assumption on the locations of nail forces. The result of analysis will be true only if the locations of nail forces are correctly assumed. The lines of action of forces should correspond to the locations of soil nails.

4 EFFECTS OF NAIL LOADS APPLIED AT SLOPE FACE AND AT SLIP SURFACE

4.1 General

In the analysis using LEM of slices, the nail forces are usually applied on the face of slope. There is always a question as to whether the nail forces should be applied at locations where the line of action of the nail forces intersects the slip surface. This Section compares the effect of applying the nail forces at the slope face with that of applying the nail forces at the slip surface. The comparisons make use of the model of the nailed slope described before. Loading condition (i) is used in this comparative analysis.

4.2 Nailed slope

For the case when the nail forces are applied on the slope face, the factors of safety computed by using the various methods for slip surface S8 are summarised in Table 5. Fig. 18 shows the nail forces used and the slices. For illustrative purpose, free body diagram and force polygon showing the inter-slice forces for slice no. 10 are presented in Fig. 19; the middle nail force (SN4) is applied at this slice.

Table 5. Summary of FoS computed by various methods for slip surface S8 in the nailed slope.

Method	Slip Surface No.	FoS (Nail Forces Applied on Slope Face)	FoS (Nail Forces Applied on Slip Surface)
M-P	S8	2.39	2.35
BS		2.43	2.42
JS (with correction factor)		2.41	2.30
JS (with no correction factor)		2.28	2.18

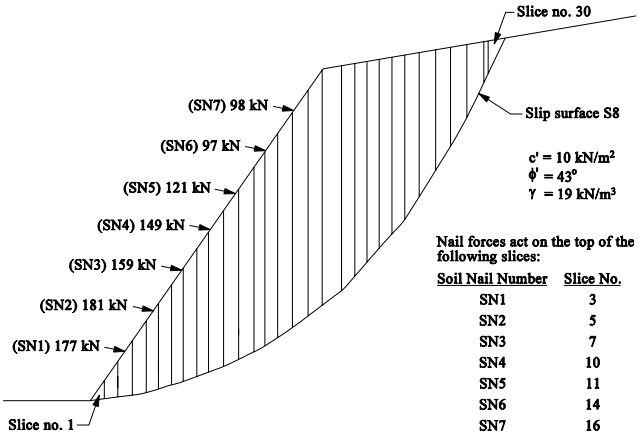
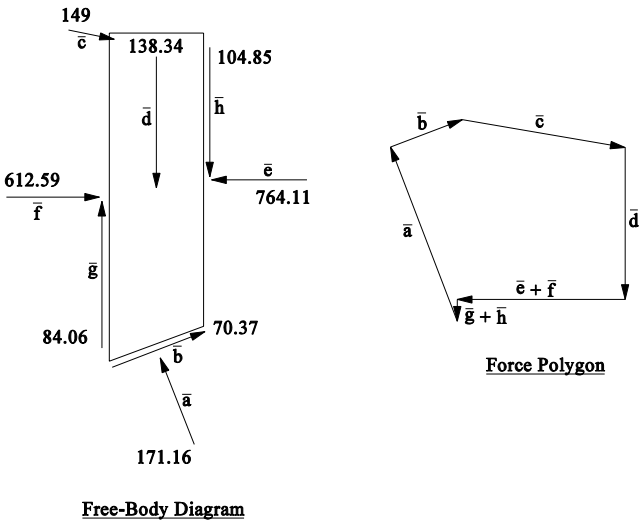


Fig. 18. Different nail forces and slices.



Note: Force unit in kN/m

Fig. 19. Free body diagram and force polygon for slice no. 10 when different nail forces act on the slope face.

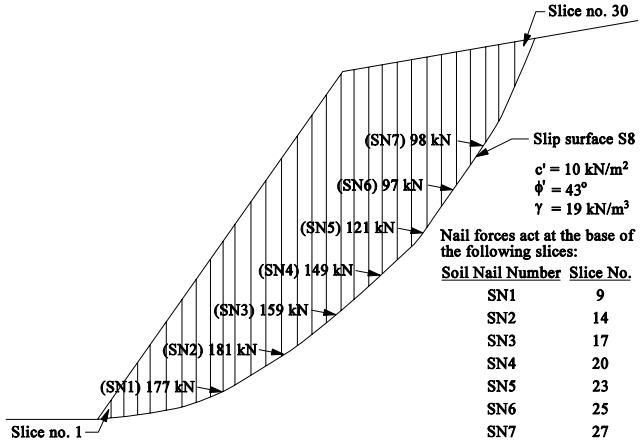


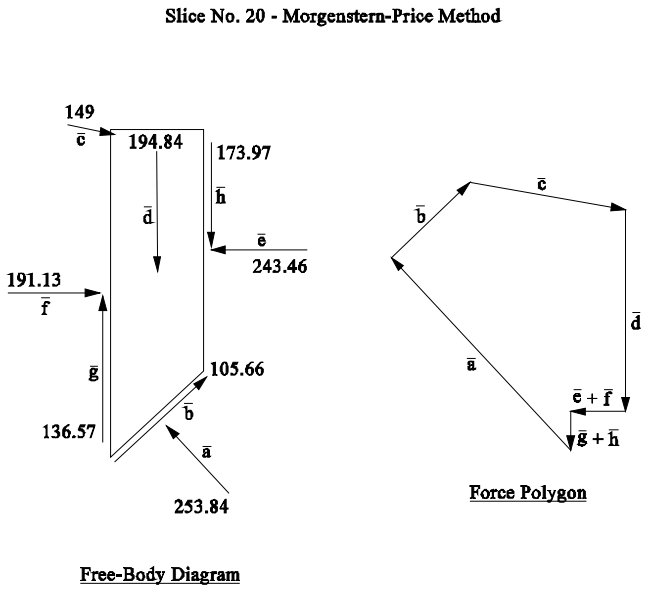
Fig. 20. Different nail forces act on the slip surface S8.

For the case when the nail forces are applied at where their lines of action intersect the slip surface S8, Fig. 20 shows the nail forces and the slices. Free body diagram and force polygon showing the inter-slice forces for slice no. 20 with the nail force of SN 4 are shown in Fig. 21.

It can be noted from Table 5 that the factor of safety values in these two cases are very similar. The maximum difference is 4.6% less in the latter case when using the JS method (with correction factor).

4.3 *Discussion*

The analytical results indicate that the application of loads on the slope face and that on the slip surface produce very similar values of factor of safety. In fact different distribution of nail forces having the same magnitude and the same line of action of resultant force should produce similar FoS results.



Note: Force unit in kN/m

Fig. 21. Free body diagram and force polygon for slice no. 20 when nail forces act on the slip surface S8.

5 EFFECT OF USING HORIZONTAL COMPONENT OF NAIL FORCE AND USING NAIL FORCE ALONG THE SAME INCLINATION OF SOIL NAIL

5.1 *General*

In reinforced fill slope construction, reinforcements are generally placed horizontally in compacted fill. The tension force provided by the reinforcement acts in the horizontal direction. However, soil nails are usually installed at an angle to the horizontal. Where the inclination of soil nails to the horizontal is small, neglecting the vertical component of the nail force should not give a much different FoS result. The difference gets larger when the inclination is increased. In current practice, some designers apply nail force in the same direction as the nail alignment in stability analysis while some just consider the horizontal component of the nail force.

5.2 *Analysis*

Stability analysis has been carried out to compare design using horizontal components of nail forces to that using the nail forces along the same inclination of soil nails. The Morgenstern-Price method is used in the stability analysis.

The slope model used in the analysis is 20 m high with an angle of 60°. It has an upslope angle of 20°. The slope is reinforced with three layers of soil nails. For the purpose of stability analysis, each nail is assumed to provide an axial force of 200 kN/m, which acts on the slope surface in the same inclination of the nail (ω). The relatively large soil nail forces are chosen so that the inclination effect of soil nails in the analysis may be seen more easily in this illustrative example. The geometry of the slope, locations of the soil nails and the assumed soil strength parameters are shown in Fig. 22.

Nine slip surfaces (1 to 9) through the toe of the slope have been considered. In the analysis the inclination (ω) of the nail forces is varied between 5^o and 30^o below the horizontal, at an interval of 5^o. For each inclination, the factors of safety of the slip surfaces are determined using two approaches: (a) nail forces are applied in the same inclination of the nails; and (b) horizontal forces, which are the same as the horizontal component of the inclined forces, are applied to the slope.

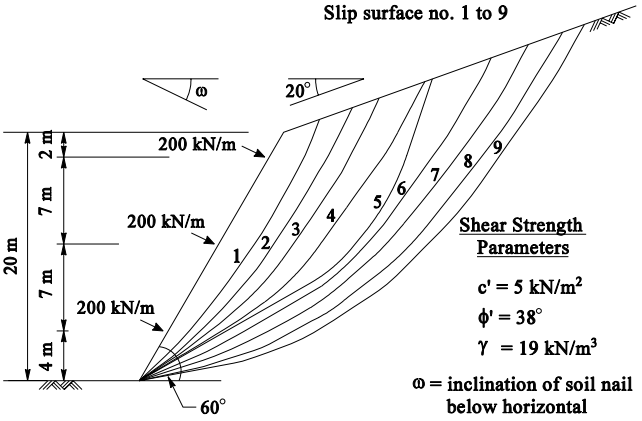


Fig. 22. Different nail forces act on the slip surface S8.

5.3 *Results of analysis*

The results of the comparative analysis are summarized in Table 6. It can be seen from the last column of the Table that the differences in the computed factors of safety between the two approaches generally increase as the nail inclination increases. For very shallow slip surface, the factor of safety computed using the horizontal component forces is greater than that using the inclined forces. The difference obtained from the present analysis is up to about 11%. The factor of safety computed using the horizontal components becomes closer to and eventually smaller than that computed using the inclined forces when the slip surface goes deeper. The difference for the deepest slip (slip surface no. 9) is not more than 2.4%. For the slip surface with the minimum FoS, the maximum difference is only 4.9%, when the inclination angle (ω) of the nail is 30°.

The approach of using the horizontal force component essentially ignores the vertical component of the nail force. The magnitude of the vertical force component and its effect increases with increase in the nail inclination.

As soil nails are normally installed at small inclinations to the horizontal, the differences in factor of safety computed using the two approaches are generally small. Nevertheless, it is preferable to apply the forces in the direction of the soil nails in order to obtain a more realistic model.

Table 6. Results of Comparative Analysis Using Inclined Nail Force and Horizontal Component of Nail Force.

Nail Force in Same Inclination of Soil Nail				Horizontal Component of Nail Force			Difference in FoS (%)
Inclination, (Degrees) (°)	Nail Force, F (kN/m)	Slip Surface No.	FoS	Nail Force, F_H (kN/m)	Slip Surface No.	FoS	
5	200	1	NC	199.2	1	NC	-
		2	1.48		2	1.51	2.3
		3	**1.34***		3	**1.36***	1.1
		4	1.36		4	1.37	0.2
		5	1.49		5	1.49	0.1
		6	1.44		6	1.44	-0.1
		7	1.45		7	1.45	-0.2
		8	1.51		8	1.51	-0.3
		9	1.59		9	1.58	-0.4
10	200	1	NC	197.0	1	NC	-
		2	1.43		2	1.50	5.1
		3	**1.32***		3	**1.35***	2.2
		4	1.36		4	1.36	0.4
		5	1.48		5	1.49	0.2
		6	1.44		6	1.43	-0.2
		7	1.45		7	1.45	-0.4
		8	1.52		8	1.51	-0.6
		9	1.59		9	1.58	-0.8
15	200	1	1.78	193.2	1	NC	-
		2	1.39		2	1.48	6.9
		3	**1.30***		3	**1.34***	3.1
		4	1.34		4	1.35	0.6
		5	1.47		5	1.48	0.3
		6	1.43		6	1.43	-0.3
		7	1.45		7	1.44	-0.6
		8	1.52		8	1.50	-0.9
		9	1.59		9	1.58	-1.2

Legend:
NC Non-convergence
* Minimum factor of safety among the nine values

Table 6 (Continued). Results of Comparative Analysis Using Inclined Nail Force and Horizontal Component of Nail Force.

Nail Force in Same Inclination of Soil Nail				Horizontal Component of Nail Force			Difference in FoS (%)
Inclination, (Degrees) (°)	Nail Force, F (kN/m)	Slip Surface No.	FoS	Nail Force, F_H (kN/m)	Slip Surface No.	FoS	
20	200	1	1.66	187.9	1	NC	
		2	1.34		2	1.45	8.6
		3	**1.27***		3	**1.32***	3.9
		4	1.33		4	1.34	0.7
		5	1.46		5	1.47	0.3
		6	1.42		6	1.41	-0.4
		7	1.44		7	1.43	-0.8
		8	1.51		8	1.49	-1.2
		9	1.59		9	1.57	-1.6
25	200	1	1.55	181.3	1	NC	-
		2	1.29		2	1.42	10.0
		3	**1.24***		3	**1.30***	4.5
		4	1.31		4	1.32	0.7
		5	1.44		5	1.45	0.4
		6	1.41		6	1.40	-0.5
		7	1.44		7	1.42	-1.0
		8	1.51		8	1.49	-1.4
		9	1.59		9	1.56	-2.0
30	200	1	1.44	173.5	1	NC	-
		2	1.24		2	1.38	11.1
		3	**1.21***		3	**1.27***	4.9
		4	1.29		4	1.30	0.7
		5	1.43		5	1.43	0.4
		6	1.40		6	1.39	-0.7
		7	1.43		7	1.41	-1.2
		8	1.50		8	1.47	-1.7
		9	1.59		9	1.55	-2.4

Legend:
NC Non-convergence
* Minimum factor of safety among the nine slips analysed

6 PROBLEMS AND LIMITATIONS OF METHODS OF SLICES

6.1 *Problems of convergence*

A common problem when analyzing the nailed slope is the difficulty of convergence. The situation for the problem is largely associated with (a) the steepness of the slip surface particularly near the crest, and (b) the application of concentrated line loads (nail forces) in the upper portions of the slope and excavation. Ching & Fredlund (1983) found that steep slips can cause conditions that give unreasonable m_α values (e.g. zero or negative m_α values) in the numerical procedure and result in convergence problem, where m_α is the denominator at the right hand side of Equation (3).

The convergence problems are particularly serious for Janbu's generalised method. This is mainly due to the assumption used in the method in respect of the distribution of stress on each slice. In this method, the line of thrust is often assumed to be located at one-third of the inter-slice height above the slip surface. The magnitudes of the inter-slice force are determined by taking moments about the base of each slice. As pointed out by Krahn (2003), this approach generally works well when the potential sliding mass has no significant stress concentrations. However, if stress concentrations exist that deviate significantly from the assumed stress distribution, convergence problems can occur. This is particularly true when features like anchors or soil nails are included in the analysis.

As suggested by Krahn (2004), the convergence problem could be resolved by relaxing the convergence tolerance and allowing the review of an additional grid point to see if the factors of safety change or whether the values are increasing or decreasing.

In SLOPE/W 2004, the lambda (λ) plot is obtained by calculating the FoS within the default range of λ from -1.25 to +1.25 for both the force equilibrium and the moment equilibrium equations. The intersecting point represents the λ value and the FoS for the M-P method where both the force and the moment equilibrium are satisfied. The default range of λ is so set because an intersecting point can be found within this range in most cases. However in some cases, even though an intersecting point is within the default range of λ, the computation may stop and no FoS can be found. It is because during the computation process when λ approaches -1.25 or +1.25 (i.e. when the interslice-forces are steeply inclined to the horizontal), a non-convergence problem may be encountered. In these cases Krahn (2004) suggested that the user can manually re-set λ to a narrower range to avoid the problem of non-convergence.

Cheng & Lansivaara (2005) reported the development of a software programme called "SLOPE 2000" for slope stability analysis. The programme makes use of a numerical approach called the "double QR method" to get close form solutions. This helps reduce the non-convergence problem.

7 CONCLUSIONS AND RECOMMENDATIONS

Stability computations have been performed on a nailed slope using force methods of slices including the Bishop's Simplified method, the Janbu's Simplified method, the Janbu's Generalised method and the Morgenstern-Price method. The following main observations are made from the computational results:

1 The FoS values computed using the Bishop's Simplified method and the Morgenstern-Price method are sensitive to the assumed locations of nail forces.
2 The FoS values computed using the the Janbu's Simplified method are insensitive to the assumed locations of nail forces. This is an inherent limitation of the method.
3 If the applied resultant nail force is located above the actual resultant nail force, the Janbu's Simplified method tends to give higher FoS values than those given by the Morgenstern-Price method. The Janbu's simplified method may give a conservative or an unsafe solution depending on the nail pattern and locations.
4 For the four methods reviewed, the FoS values derived based on the actual loading condition (i.e. condition (i)) are close to those based on the two conditions of evenly distributed nail forces and single nail force applied at the mid-height, as the locations of the resultant force of the three loading conditions are similar.
5 All the methods have encountered convergence problems. The problems are most serious for the Janbu's Generalised method which has not produced converged solutions for any of the slips analysed.
6 The approach of using the nail force in the same inclination of the soil nail and that of using the horizontal component of the nail force produce different FoS values. The latter ignores the vertical component of the nail force. The difference is not significant for small nail inclinations commonly used in design.
7 The limit equilibrium methods have some limitations. Such limitations should be borne in mind when interpreting the results of stability computations, especially when the stress-strain assumptions may not be valid (e.g. when compressive forces are mobilized in steeply inclined nails).

The following recommendations are made for the design of soil nails in cut slopes using limit equilibrium methods:

1 Methods which satisfy both force and moment equilibrium such as the Morgenstern-Price should be used for design.
2 Where problem of convergence is encountered, the convergence tolerance may be relaxed. This needs to be carried out with good engineering judgment. The accuracy of the limit equilibrium method used should be taken into account. The relaxation should only be applied to the non-converged slip surfaces. Plotting FoS values versus λ values helps identify and understand the convergence problem. In some cases the problem may be resolved by changing the default range of λ in SLOPE/W 2004. In addition, other numerical algorithms such as the Double QR method may help solve the non-convergence problem.
3 If the nail forces on the slope are modelled by using a single force, this force should be located near the expected line of action of the resultant of nail forces.
4 Nail forces should preferably be applied in the same inclination as the soil nails, though the approach of considering only the horizontal components of the nail forces is considered acceptable.
5 Each soil nail should be placed in an orientation with respect to the normal of the potential slip plane such that axial tensile force will be developed. If in doubt, numerical methods such as the finite element or the finite difference methods may be used to supplement the analysis.

ACKNOWLEDGEMENTS

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Evolution of Fill Slope Upgrading Methods in Public Housing Development

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Abstract: The Hong Kong Housing Authority has been upgrading existing loose fill slopes in their estates since the 1980s. The design approach and construction methods of them were evolving from initially providing an economical scheme to that with other enhancements such as greening and trees preservation. In this paper, the authors describe the evolution of loose fill slope upgrading works by referring to some public housing projects.

1 INTRODUCTION

In order to accommodate the rapidly growing post war population in Hong Kong, site formation works for public housing development, which typically involved cutting and filling natural terrains, were carried out over tight schedules. Soil compaction in the fill slopes was usually poorly controlled. They might be formed either by end-tipping fill over a sloping ground or by horizontal fill laying in great thickness with inadequate compaction. As a result, the fill slopes so formed became the loose fill slopes.

When the loose fill slope becomes unstable, the fill will deform at the onset of instability. These deformations will tend to produce a decrease in volume, i.e. contraction. If the fill is saturated, the contraction is resisted by the water in the soil pores. The pore water pressure will then be increased rapidly and causing a similar abrupt decrease in effective stress in the soil. When the effective stress falls below a threshold value, the soil will flow in a liquid like manner. The slope failure will then be presented as a flowslide with its failure mass moving at a high velocity down the slope.

Flowslide failure is quick and catastrophic. The devastating Sau Mau Ping failures in 1976 had caused many deaths and even more homeless people. To protect the residents living in the public housing estates, the Hong Kong Housing Authority have been upgrading existing loose fill slopes to the current safety standard in their estates since then.

2 RECOMPACTION METHOD

The slope upgrading works for Feature No. 11NE-D/FR6 in Sau Mau Ping Estate was among the earliest fill slope upgrading projects undertaken by the Housing Authority. It adopted the recompaction method recommended by the Independent Review Panel for Fill Slopes (Hong Kong Government, 1977). A typical section of the slopeworks is presented in Fig. 1.

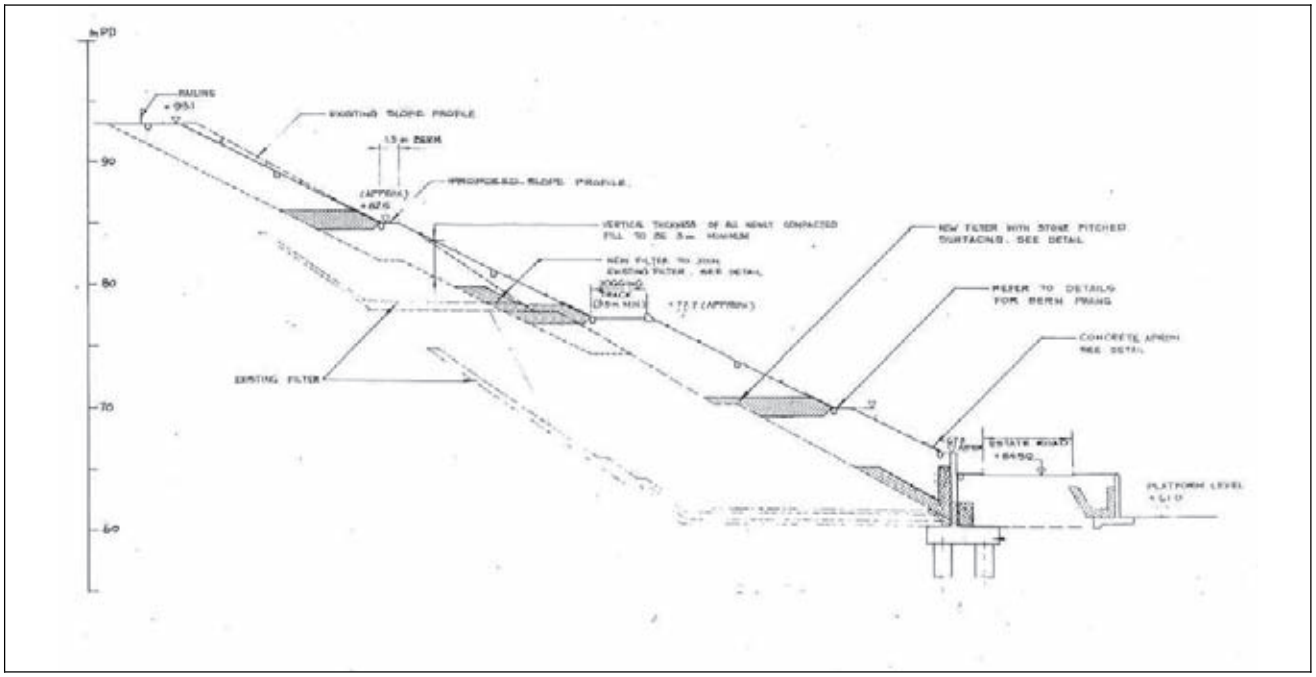


Fig. 1. Recompaction stabilization to loose fill slope.

The slope upgrading works comprised relocating the existing drains and utilities, removing the loose surface fill to a vertical depth of 3m, placing drainage blanket, backfilling the excavation by compacted fill and finally constructing drainage channels and vegetation protective covers to the slope surface.

Recompacting the top 3m loose fill did improve the fill slope stability in the following manners:

(a) Compaction increased the strength of the fill body;
(b) Potential structural collapse of the soil skeleton during shearing was eliminated;
(c) Rate of rainwater infiltration and the depth of wetting band during heavy rainstorm was reduced, and
(d) Hillside seepage along the inclined end-tipping fill layers down the slope was interrupted.

Fill materials of the following characteristics were selected for the recompaction:

(a) Soil particles smaller than 75mm in diameter overall and 55% less than 63μm, and
(b) Liquid limit below 45%, plasticity index less than 20% and a coefficient of uniformity smaller than 50 unless the fine portion (soil particles smaller than 63μm) was less than 30%.

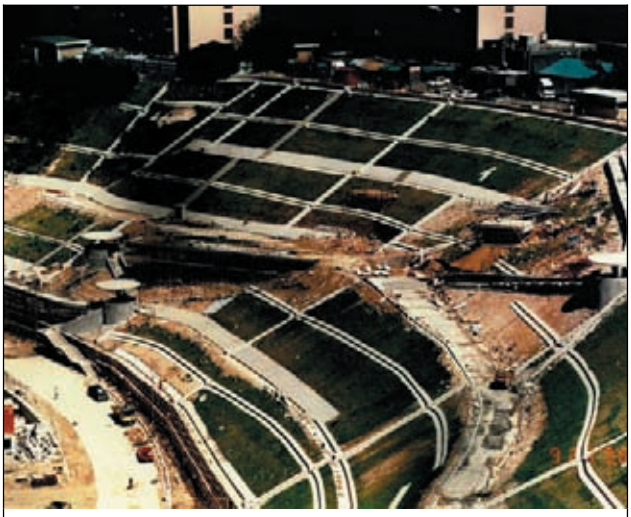
The fill was deposited in thin horizontal compaction layers of about 300mm thick. The compaction layers were placed slightly oversized and then trimmed back to ensure the fill near the final slope face was well compacted. The fill was compacted to at least 95% of the maximum dry density in general and 98% within 200mm of the final slope face. Drainage filter was provided at the interface between the existing loose and the recompacted fill layers. The filter was selected to comply with the filter rules for stability, permeability and segregation requirements. The groundwater collected by the filter was discharged into the area storm drainage system. During the re-construction of these fill slopes, temporary shoring works are usually required.

Table 1. Grass and tree seeds mix proportions in the hydroseeding spray.

Type	Species	Apply Rate (g/m^2)
Grass Species	Cynodon Dactylon (Bermyda Grass)	9
	Paspalum Notatum (Bahia Grass)	9
	Eremochloa Ophuroides (Centipede Grass)	2
Tree Species	Acacia Confuse Eucalyptus Citriodora Eucalyptus Robusta Trinstania Conferta Casuarine Equisetifolia	5

Vegetation protective cover was selected for the final slope surface in view of the prevailing greening policy from the government (Works Branch, 1993). A vegetation cover is better accepted by the public housing estate residents because it is green and visually comfortable. The slope stability is also benefited because the vegetation absorbs water from the soil by root intake and transpiration. In addition, the vegetation root systems reinforce the top soils against surface erosion.

The vegetation cover was designed to achieve natural sustainability. Broadleaf mixed woodlands are the dominant vegetations in Hong Kong natural slopes under the influence of its sub-tropical climate. This was used as the model for the vegetation design. It was also recognized that the broadleaf woodlands would need some time to establish. Therefore, a basic grass cover was provided as a short-term protection measure. The vegetation cover in Feature No. 11 NE-D/FR6 was applied by hydroseeding. The grass and tree seeds were mixed in the hydroseeding sprays. The seed mix proportions are presented in Table 1. Progress of vegetation establishment is shown in Fig .2.



Immediate after hydroseeding

Vegetation fully established

Fig. 2. Progress of vegetation establishment in Feature No.11NE-D/FR6.

3 FILL CAPPING METHOD

Feature No.11NW-B/F474 in Pak Tin Estate contains a loose fill slope formed in the early 1960s. The fill slope is abundant in mature trees of valuable species. The trees are well accepted by the Pak Tin Estate residents. The slope becomes a popular place for social gatherings. The objectives of the upgrading works to the loose fill slopes are to improve the slope stability and to retain

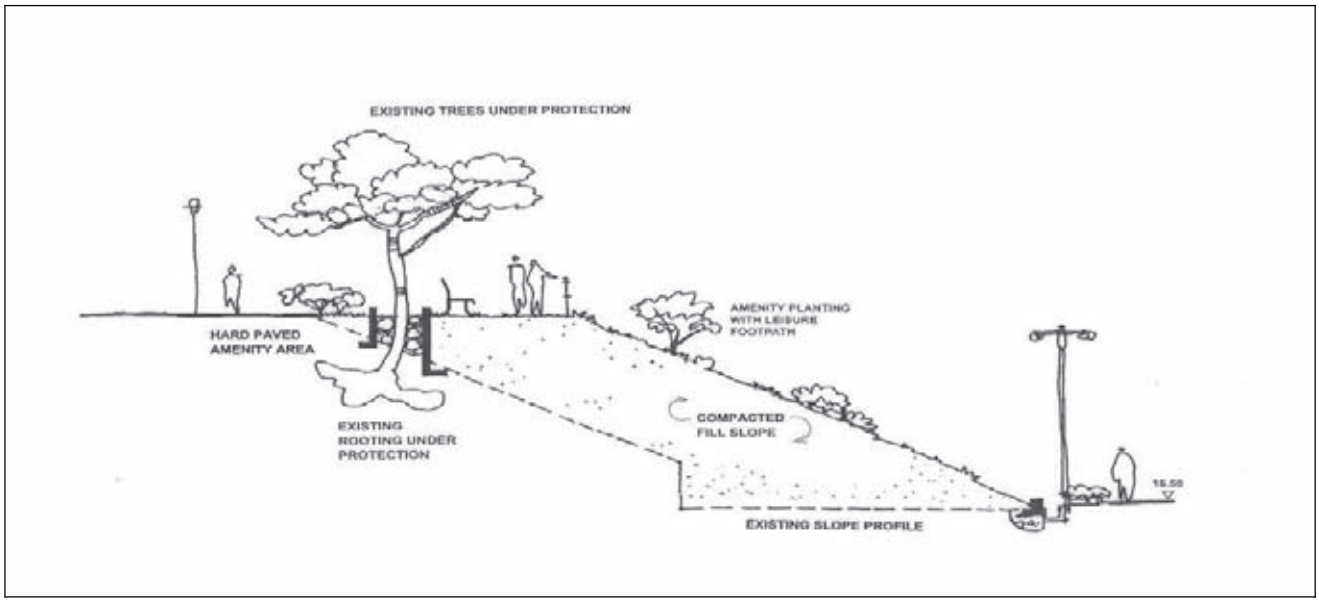


Fig. 3. Fill capping stabilization to loose fill slope.

the existing trees as far as possible. Additional open space and recreation facilities are also provided to enhance the slope into an amenity area.

Existing trees can either be retained in-situ or be transplanted to a temporary storage area during the construction works and then re-established in their original positions. Tree transplanting is only practical where they are growing on a flat surface or on gently sloping grounds. The steeply inclined rooting patterns of trees growing on Feature No.11NW-B/F474 greatly limit the chances of them being successfully survived in the transplanting process. It was therefore decided to retain them in-situ.

The additional open space allocated to the slope allowed the employment of the fill capping method for the slope upgrading works. Instead of excavating into the existing loose fill, a 3-m thick compacted fill cap was constructed on the existing slope surface to confine and strengthen the loose fill. The design principles and construction procedures of the fill capping method are similar to those for the recompaction method. Benches were formed on the existing slope surface. The filter layer was then placed followed by a 3-m thick compacted fill. The slope upgrading works were enhanced by covering the final slope surface with Chevron drains and vegetation layer. A typical section of the upgrading works is shown in Fig. 3.

Special tree rings were provided around each or group of existing trees prior to the earth filling. Firstly, the existing old tree rings were removed and a steel inner ring was rolled around the part of tree trunk that was to be buried. Then, an outer concrete ring was constructed. The tree was tied to the outer concrete ring by means of a steel wire and a trunk protection pad. The tree ring was then filled with aggregates. Depending on the size of the tree trunk to be protected, the filling levels at the inner steel ring is preferably lower than the outer ring. This is to reduce the imposed loads on tree trunk from the aggregates, hence the possibility of breaking them during the filling works. Finally, the steel wire and the trunk protection pad were removed. Details of the special tree ring are shown in Fig. 4. A view of the as-built tree ring is shown in Fig. 5.

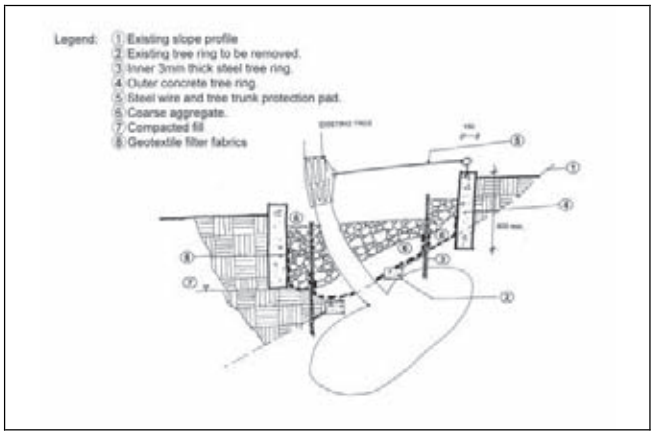


Fig. 4. Details of special tree ring in the fill cap.

Fig. 5. A view of the as-built tree ring.

When compared with the recompaction method, the fill capping method has the following advantages:

(a) Tree transplanting is not necessary;
(b) Excavation and the associated temporary shoring work is eliminated;
(c) Works period is shorter, and
(d) Construction cost is reduced.

4 SOIL NAILING METHOD

Conditions of Feature No.11NE-D/FR314 in Sau Mau Ping Estate are similar to Feature No.11NW-B/F474. It contains a loose fill slope with many mature trees of valuable species on its surface. However, the layout of the proposed redevelopment is more congested and cannot afford the required toe space for the placement of any fill capping.

In recent years, it has been established that sufficient resistance can be generated along soil nails installed in loose fill slope to reduce the risk of flowslide failure. Successful case histories have been reported by GEO (GEO, 2005) and Smith (Smith *et al*., 2005).

The existing fill slope in Feature No.11NE-D/FR314 is a Cat 1 slope of about 12m high. It is in good conditions with no sign of distress. No history of movement or abnormal heavy seepage or weak zones has been identified in the existing slope design and maintenance records. The average degree of compaction of the existing fill is about 80%. Based on GEO's interim guidelines on the use of soil nails to stabilize loose fill slopes (GEO, 2001), soil nails are considered to be suitable options for the upgrading works.

The soil nails are designed following the recommended method in the Hong Kong Institution of Engineers' final report on the preliminary study on soil nails in loose fill slopes (HKIE, 2003). Steady state undrained shear strength of the loose fill is used in the slope stability analyses. The soil nails are designed to provide sufficient axial resistance to increase the overall factor of safety for the slope to 1.1.

The proposed soil nails are arranged irregularly in some localized areas to avoid damaging the existing trees. In addition, their spacings are limited to 2m horizontally and 1.5m vertically. Concrete grillage is constructed to connect the nail heads on the slope surface. The concrete grillage is embedded 500mm into the slope surface to restrain potential downward movement of the loose fill. A typical section of soil nailing stabilization works is

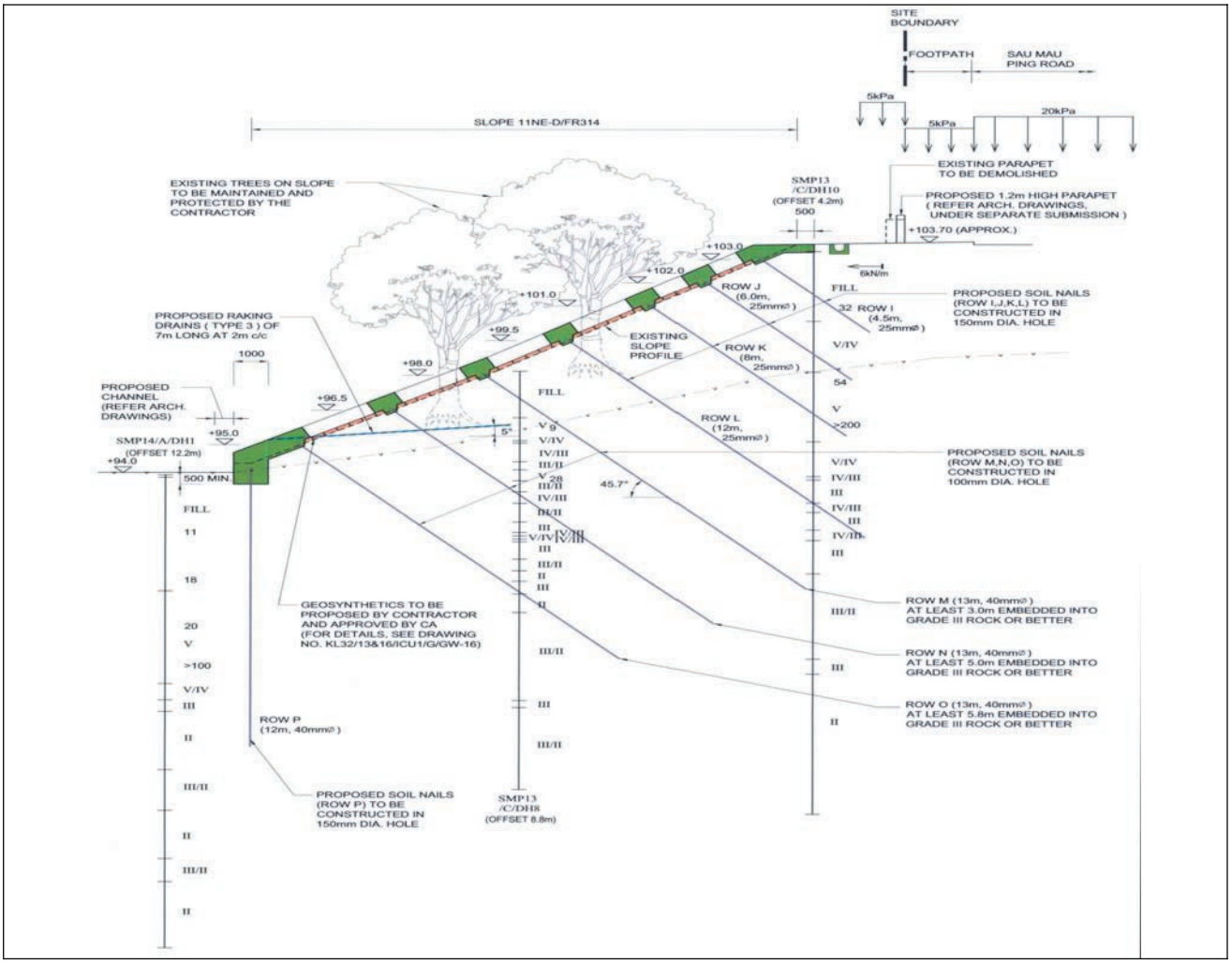


Fig. 6. Soil nailing stabilization to loose fill slope.

shown in Fig. 6.

The soil nailing upgrading method is very suitable for those loose slopes with abundant of mature trees but inadequate toe space for the fill capping method. Since no excavation and earth filling is required, the method is less sensitive to weather conditions and more robust in construction programming.

5 CONCLUSIONS

In this paper, three different measures for upgrading existing loose fill slopes in the public housing development have been briefly described, namely the recompaction method, fill capping and soil nailing methods. Among them, the recompaction method is a simple, economical and direct solution. Probably involving some temporary shoring works during the construction stage. Fill capping method can preserve mature existing trees on the slope while the soil nailing method can achieve the same purpose where extra space at the slope toe is not available.

ACKNOWLEDGEMENT

The Authors are grateful to the Director of Housing for his permission to publish this paper and to their colleagues of various disciplines and sectors who had contributed to the design and construction of the works cited.

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Starting from Scratch – A Personal Recollection of the Development of Geotechnical Engineering in the Housing Department

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Abstract: The paper draws on the Author's personal experience with the development of some of the strategies relating to the application of geotechnical engineering in the Housing Department since 1980. These include (a) an integrated strategy for the preventive maintenance of existing slopes that links up inspections, monitoring, studies, maintenance and improvement works; (b) a risk-based approach to the planning and implementation of major site formation works in foothill areas; and (c) a commitment from a geotechnical perspective to upholding the principles of sustainable development and environmental conservation.

1 INTRODUCTION

In August 1976, a tragic landslide hit Block 9 of Sau Mau Ping Estate. This resulted in, among other things, the establishment of a geotechnical control regime in Hong Kong and an awareness of the importance of geotechnical engineering in the Housing Department (HD).

The Author joined the HD in 1980 at the time when the Civil Engineering Section was being set up and the role of geotechnical engineering was starting to be recognized. He had since then participated in developing the strategies in the application of geotechnical engineering to the planning, design, construction, maintenance and improvement of public housing developments. Due to the limit on the length of the paper, only three of the strategic areas are presented.

It is hoped that this paper on personal recollection, together with reference to papers authored or co-authored by geotechnical colleagues of the HD, can help promote the sharing of experience in the geotechnical community.

2 PREVENTIVE MAINTENANCE OF SLOPES

The Sau Mau Ping landslide brought several tasks into action. Firstly, the HD had to ascertain which slopes within and around public housing estates it was to be responsible for their maintenance. There were interfaces to resolve, for example, in respect of the slopes between building platforms and public roads. Site visits were held with counterpart departments and decisions were made according to what subsequently developed into the "beneficiary maintains" principle. HD would take care of the maintenance of a cut slope above or a fill slope below a building platform on the presumption that the slope was formed to create the platform (see Fig. 1).

The second task that followed in 1980 was to prepare an inventory of slopes maintained by the HD with a basic data sheet for each slope. Binnie & Partners (B&P) were commissioned to carry out the task. Teams of geotechnical engineers (GEs) were dispatched to conduct slope inspections and to prepare a "Black File" of data sheets for each and every estate that had slopes. During the inspections, opportunity was also taken to draw up recommendations for slope maintenance, studies and/or other works. The inspection records and recommendations were held in a "Red File" for each estate. To ensure the teams adopt a consistent standard, B&P developed a detailed coding manual for recording observations with photographic illustrations of the different slope conditions that the codes represented.

Work of a routine nature such as clearing of drain channels and repair of disrupted surface protection was executed through

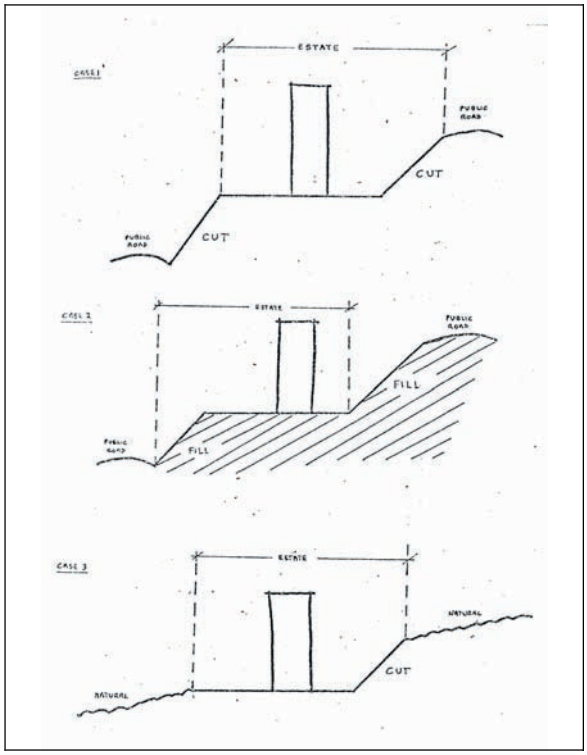


Fig. 1. The original sketch showing how HD determined which slopes should be included into the estate boundary.

maintenance term contracts, and work involving further investigations and studies was organized for individual slopes or groups of nearby slopes following their priority order in a ranking list (the List). The order of the List took account of the likelihood of instability of the individual slopes and the likely consequence in event of an instability occurring. The studies were carried out in a staged manner, with a Stage 2 study being a slope stability assessment and a Study 3 study being design proposals for the

upgrading works. With the staged process, the List was quickly screened through to sort out those slopes in need of upgrading.

The identified slope upgrading works were implemented with funds provided by the Housing Authority and by the Government. Nearby slopes were usually packaged into a single contract to facilitate administration. A term contract was also let to deal with prescriptive works for rock slopes. Where appropriate, some slopes were upgraded as part of the related estate redevelopment projects. A lot of the upgrading works to existing slopes were thus completed by the mid 80s.

Where a Stage 2 study revealed a serious concern on slope stability and a Stage 3 study was not yet complete, improvement measures such as installation of horizontal drains to control the rise of groundwater were put in place promptly sometimes together with a monitoring or surveillance plan (Insley & McNicholl, 1982; McNicholl & Cho, 1985). These measures helped reduce the risk of a potential landslide. Guidelines were also issued to facilitate prompt response to emergency incidents. These plans and guidelines proved to be effective and helpful to the handful of GEs who in the early 80s might have to stay overnight in an estate when groundwater rose close to the alert level or who might have to rush from place to place to deal with several reported incidents (Wong, Kwok & Leung, 2003).

In the early 80s, the HD encountered bitter experience with failures of ground anchors (Howells & Leung, 1988). This prompted an annual programme to monitor the performance of ground anchors installed on slopes that were handed over to the HD for maintenance (Leung & Chiu, 1987). Experience gained from the monitoring programme was recognized in the second edition of GEOSPEC 1 published in 1989. In the mid 80s, the monitoring programme was extended to cover performance of horizontal drains. GEO Report No. 42 reported on the performance of horizontal drains installed at a number of sites including several public housing estates.

Following the landslide at Kwun Lung Lau in 1994, a programme was initiated to detect leakages from buried water carrying services in the vicinity of slopes and retaining walls maintained by the HD. A pilot contract was first let for 5 estates to sort out the logistic arrangements before letting the main contract in 1996 that covered about 130 km of drainage pipes, 120 km of pressurized watermains, and 14000 manholes (Leung Barker & Kiew, 1997; Chan, 2003).

The number of slopes maintained by the HD has increased from around 700 in the early 80s to around 1200 in the mid 90s and around 1700 at present. Concerted efforts of various grades of staff contribute to an effective preventive maintenance system for slopes. There are three pillars in support of the slope inspection and maintenance cycle. First, estate management staff carry out routine housekeeping work such as clearing of surface channels and casual inspections in the course of their daily patrol. Second, technical staff conduct maintenance inspections before and during the wet season and arrange any necessary maintenance works to ensure the slope conditions remain satisfactory. Third, GEs conduct engineer inspections and respond to any reported anomalies concerning the stability conditions of the slopes.

Fig. 2 illustrates the HD's slope maintenance cycle that integrates various components. The maintenance cycle has been established and put in use since early 80s and refined over the years (Leung, 1996; Chim, 2003).

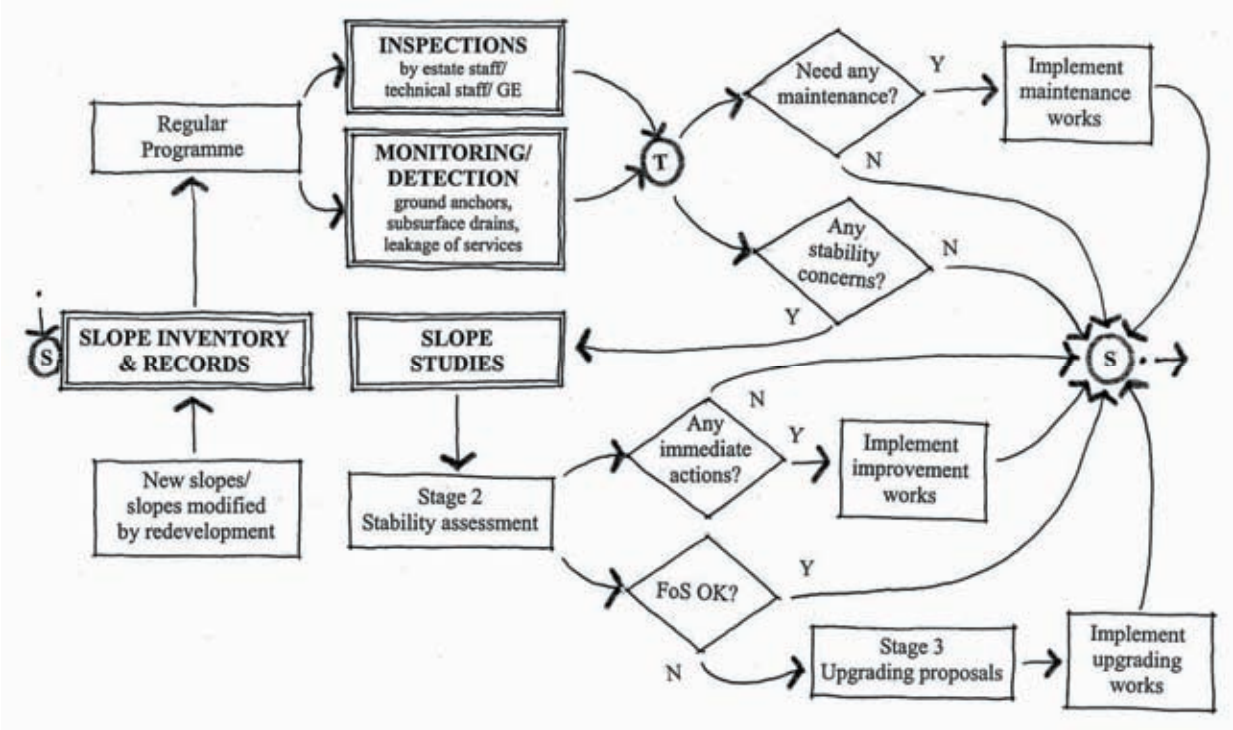


Fig. 2. The HD's integrated slope maintenance cycle.

3 MAJOR SITE FORMATION WORKS IN FOOTHILL AREAS

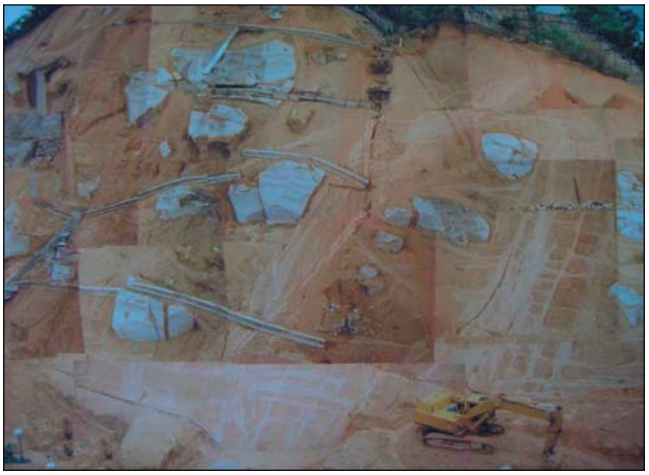
Although the HD relies on the Government to provide formed sites for its public housing developments, it has also carried out its own site formation works since the early 80s. Many of them involve substantial geotechnical works to create building platforms in hilly terrain which were originally occupied by squatters or cottage areas such as Ma Chai Hang, Chuk Yuen, Shaukeiwan foothills, Ma Hang and Tiu Keng Leng. In some other cases of redeveloping old estates such as Tsz Wan Shan and Sau Mau Ping, site formation works were also required to create larger building platforms.

Mitigation of geotechnical risks is one of the major considerations in implementing site formation works. It calls for the GE's attention as early as possible in the project planning stage (Leung, Yau & Roberds, 1997; Roberds et al, 1999). Some of the typical risks arise from the variability and uncertainty in the ground and groundwater conditions, threats from boulder falls and other natural terrain landslide hazards.

Plate 1. The Shaukeiwan foothill development utilizes access roads as buffer zones to separate the building platforms from the backslopes and natural terrain above. Surplus excavated materials from site formation were gainfully used to reclaim land in Aldrich Bay (bottom right), Tseung Kwan O and Sai Kung for future developments.

Cutting into foothill areas to create building platforms will invariably result in large peripheral slopes. Despite these slopes are designed to current slope safety standards using deterministic methods, it is advisable from a risk management point of view to reserve an area between the building blocks and the slopes as a buffer zone. This will help reduce the consequence of any unlikely event of slope instability. Plate 1 shows an example of such a layout planning concept.

Ground is variable. This is not only a result of geological and geomorphological processes, but can also be a result of human activities. Bodies of trash and loose fill, and potentially unstable rubble walls are usually found after clearance of squatters and cottage areas. Too often inadequate site investigation is given as an excuse for unforeseen ground conditions. In fact lack of insight and inadequate interpretation of ground information can also result in potential risks being overlooked.



(A) (B) (A) (B) (A)

Plate 2. Exposure on the face of a cut slope at Tung Tau showing the presence of corestones in weathered granite. Different sets of vertical boreholes, for example (A) and (B), may reveal different intensity of occurrence of corestones.

Plate 3. Exposure during construction of a substructure at Lok Fu showing weathered seams along the joints between granite rock slabs. A borehole sunk through the vertical seam will reveal a much deeper weathering profile which is different from those revealed by boreholes on either side.

To ensure an adequate site investigation and interpretation of the investigation results, a staged approach is adopted. This provides opportunities to evaluate the results of each stage of investigation and to plan for the next stage in step with the project planning and design process. Plates 2 and 3 show some of the features that may be present in the granitic rocks commonly found in Hong Kong.

Deterministic methods for assessing the stability of soil slopes are well established. However it is important to recognize the influence of weak seams (e.g. as shown in Plate 4) and give them due regard in the analyses (Hall, Yates & Leung, 1998).

Plate 4. Presence of kaolin seam in weathered volcanic tuff retrieved from a borehole sunk in Chai Wan.

The presence of joints and discontinuities can influence the stability of rock slopes. Exposed rock face is usually mapped and analyzed batter by batter as excavation level lowers. However it is important to perform an overall analysis of the whole slope. Persistent and critical joints may be few in number but extend over several batters to intersect at a lower batter (see Figure 3). The potentially unstable small rock wedge in this lower batter (X) in fact can be the tip of a much larger wedge (Y). Without an overall review for the entire slope, the larger potentially unstable wedge can easily be overlooked. Shear zones and intrusion dykes are other features deserving attention. Beggs & McNicholl (1986) described the investigation, design and construction of a high rock slope involving thorough mapping and analyses of the discontinuities.

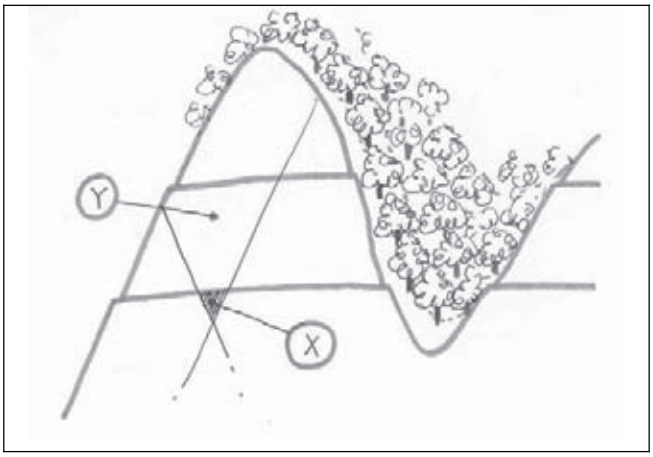


Fig. 3. The potentially adverse impact of a few persistent and critical joints may be easily overlooked unless an overall analysis for the whole rock face is carried out.

It is well-known that groundwater conditions can affect slope stability. Proper assessment of these conditions is therefore crucial for the satisfactory implementation of a major site formation. Pre-design monitoring of groundwater levels does not necessarily give a full picture. The designer has to take a wider hydrogeological perspective that covers areas beyond the site boundary, in particular the upper catchments which may have a greater effect on the groundwater conditions than the local impervious surface protection (Li, Au Yeung & Chan, 1995).

Plate 5. Natural erosion pipes encountered during site formation at Ngau Chi Wan foothill. (Top) Erosion pipe No.1 was about 2m wide with flow marks and deposits that indicated large transient flow. (Bottom) Erosion pipe No.4 was about 1m in diameter with continuous water flow.

In a foothill site involving extensive cutting into bouldery colluvium and partly into weathered granite, the measured groundwater level was low (Leung, 2003). Nonetheless the possible formation of perched water tables was recognized. In addition there were other groundwater uncertainties. Uphill of the site there was a large squatter settlement that was about to be cleared. The inhabitants had been tapping groundwater for their daily consumption. Clearance of the squatters meant that the canopy cover to reduce uphill infiltration would be lost and the artificial lowering of water table would no longer exist. The existence of natural erosion pipes added to the complex hydrogeology (see Plate 5). Further to the increased exposure to uphill infiltration, extensive excavation would alter the

topography which in turn would shorten the flow paths and lower the downhill water table. This could easily disturb the equilibrium state of the erosion pipes and activate their further development. Anderson et al (1983) presented a case study on the effect of topography on groundwater conditions.

In order to reduce groundwater risk, a considered approach to install subsurface drainage measures to control groundwater for major site formation works is recommended. The provision of subsurface drainage measures in an early construction stage also helps accelerate the process of groundwater lowering in a control manner and this permits subsequent excavation in the dry. McNicholl, Pump & Cho (1985) cited cases on the application of subsurface drainage measures.

In the 80s, focus on threats from natural terrain hazards was centred on boulder falls. Such threats were addressed by providing massive barriers to protect the housing development below. Barriers made of rock-filled polymer grid mattresses were used in one project (Threadgold & McNicholl, 1984) but soon gave way to barriers built with PVC-coated galvanized steel gabion baskets due to the concern on susceptibility to fire damage. And in the mid 90s, a more compact reinforced steel wire mesh barrier of a proprietary design was introduced and has become the preferred solution where space is very limited.

Threats from other types of natural terrain landslide hazards began to receive increasing attention since the mid 90s (Roberds, Ho & Leung, 1997). In one project, the HD responded to an imposed responsibility to look after a stretch of natural terrain by using a risk-based approach. A geomorphological field study and a Quantitative Risk Assessment were carried out and protective measures were implemented to safeguard the housing development (Li et al, 1998; Tse et al, 1999).

4 ENVIRONMENTAL CONSERVATION & SUSTAINABILITY

Extensive excavation into the foothills would generate large quantities of surplus materials. These materials were gainfully used for reclaiming more land, for reprovisioning breakwaters or for use as building materials (Beggs & McNicholl, 1986; Cheung et al, 1995; see also Plate 1). This arrangement echoes the concept of sustainability and is consistent with the policy on the use of natural resources.

Until the last decade, hard surfacing had been used extensively to protect slopes from rain infiltration and soil erosion. However there has been growing attention on slope greening and using geosynthetics to protect against soil erosion (Forth & Leung, 1989). There is also an emerging philosophy that recognizes slopes and retaining walls as part of the land and space for residents' enjoyment and use. A functional approach is adopted in the design and visual treatment of slopes and retaining walls that extends purely safety consideration into the realm of environmental conservation and sustainability. Leung (2005) presented cases to illustrate the considerations given to the design and treatment of slopes and retaining walls in layout planning and in the associated architectural and engineering studies. Slope design and treatment are conceived as a natural and sustainable component of the built environment (see Plate 6 for an example), and opportunities are exploited to create multi-purpose functions that can meet the residents' needs.

This paper begins with a reference to the tragic Sau Mau Ping landslide in 1976. The comprehensive redevelopment of Sau Mau Ping Estate provided an opportunity for revisiting the slope safety issue as well as an opportunity for improvements in the built environment. It was principally with these considerations that major improvement works to the failed slope and adjoining slopes were proposed and implemented between 1993 and 1997. The concept behind the slope improvement works was to create a central park that could integrate multi-purpose functions in addition to ensuring slope safety (see Plate 7).

The visual treatment of slopes and retaining walls calls for creativity from the GEs and collaboration with other disciplines such as landscape architects and horticulturists. It also demands a commitment to sustainable development and environmental conservation. Further examples of stabilizing loose fill slopes while preserving mature trees are given in Lai & Kwok (2007).

Plate 6. An attempt made in the mid-80s in a public housing development in Chai Wan to create an interesting bridge between the retaining wall with the toe planting and the lush vegetation on the backslope. This promotes a harmonious atmosphere of the built and natural environment.

5 CONCLUSIONS

The paper draws on the Author's personal experience as a GE working with the HD since 1980. It is hoped that this paper can help promote the sharing of experience among the geotechnical practitioners in respect of preventive maintenance of slopes, implementation of major site formation in foothill areas, and the commitment to sustainable development and environmental conservation.

ACKNOWLEDGEMENT

The Author is grateful to the Director of Housing for the permission to publish this paper and to his colleagues within the HD and partners in the geotechnical community who had contributed to the development and implementation of the strategies and projects cited.

Plate 7. Improvement works to the series of high slopes that divides Lower and Middle Sau Mau Ping have turned them into a central park. It provides recreational space for the residents, a "green" and "natural" environment, a scenic view corridor, a breeze path and foliage cover to regulate the micro-climate in the area. It also bears witness to a flourishing life that wipes away the reminiscence of the long gone tragedy.

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Instrumented Soil Nail Pull-Out Tests in Decomposed Tuff and Working Bond Study

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Abstract: A series of instrumented soil nail pull-out tests were conducted on three cut slopes in the northern New Territories under the Landslip Preventive Measures (LPM) Programme. A specially made inflatable packer and a real-time video monitoring system were developed to ensure the integrity and length of the grouted section. The relationship between ultimate pull-out capacity and effective vertical stress were investigated. A comparison of the measured ultimate and theoretical pull-out capacity is discussed and an empirical correlation between the pull-out capacity and SPT 'N' values has been established for the slopes in this study. The paper concludes that there is scope for a reduction in the safety factor on soil nail ultimate pull-out resistance and an alternative soil nail design approach based on SPT 'N' value of the soil stratum can be developed.

1 INTRODUCTION

1.1 *Background*

The current soil nail design methodology adopted in Hong Kong is generally based on the guidance set out in GEO (2005). In GEO (2005), the theoretical soil nail pull-out resistance provided by the soil/grout bond strength is a function of effective shear strength parameters of soil materials as given in Eq. (1).

$$T_l = Pc' + 2D\sigma_v'\mu^* \quad (1)$$

where T = Pull-out force (kN);
P = Perimeter of the reinforced grout column (m);
c' = Effective cohesion of the soil (kPa);
D = Nail drillhole hole diameter (m);
σ_v' = Effective vertical stress in the soil calculated at mid depth of the reinforcement in the resistant zone (kPa); and
μ^* = Coefficient of apparent friction of soil

The formula given in Eq. (1) was proposed by Schlosser & Guilloux (1981) and a Factor of Safety (FOS) of 2.0 was suggested by Watkins & Powell (1992) to determine the design pull-out resistance. In addition, Smith et. al., (2005) introduced the Working Bond Method which correlates the soil nail design pull-out resistance with the SPT 'N' values.

The critical slip surface is determined from slope stability computer programs by modelling the ground profile and, inputting the soil parameters and groundwater tables.

Internal guidelines from Geotechnical Engineering Office of Civil Engineering and Development Department, HKSAR conclude that most test nails are able to sustain the maximum pull-out test load and the observed pull-out resistance was generally much higher that the design bond strength. This implies that the theoretical pull-out resistance calculated from Eq. (1) may not be able to reflect the ultimate pull-out resistance accurately and further research into the subject is warranted.

A series of instrumented soil nail pull-out tests were carried out in order to evaluate the relationships between, the ultimate and theoretical pull-out resistance of test nails; and, the ultimate pull-out capacity and effective vertical stress. In total 34 tests were conducted on three soil cut slopes in the northern New Territories under the LPM Programme. The correlation between the pull-out capacity and Standard Penetration Test (SPT) 'N' values was also examined. All test nails were installed within the soil stratum with SPT 'N' value less than 100. In addition, the test nails were installed within 2.0m of the vertical alignment of the closest borehole.

2 SITES DESCRIPTION

2.1 *Works Sites*

Three soil cut slopes in the northern New Territories as shown on Fig. 1, namely Feature Nos. A (CC359), B (DC 79) and C (DC 214), were proposed for instrumented soil nail pull-out tests under the LPM Programme. The heights of the slopes vary from 21m to 30m and the slope angle varies from 35 ^{o} to 55^{o} to the horizontal.

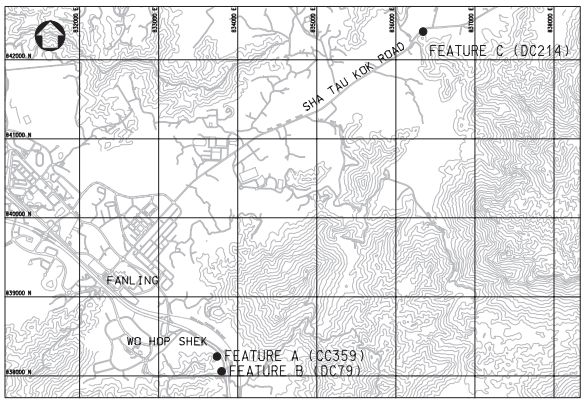

Fig 1. Location Plan

2.2 *Site Geology*

Ground investigation works were carried out prior to the installation of the test nails in order to establish the sub-surface conditions of each site. Boreholes were sunk at the critical sections on each slope with the maximum depth varying from 6.3m to 32.2m. The geology of the studied slopes generally comprises a thin layer of Residual Soil over Completely Decomposed Tuff (CDT). A typical section for Feature B (DC 79) is presented in Fig.2.

SPTs were carried out at 2m intervals in the boreholes and mazier samples were retrieved for laboratory testing to determine soil index properties and shear strength parameters. A plot of depth against SPT 'N' value and a p'- q' plot for the sites are given in Figs. 3 and 4.

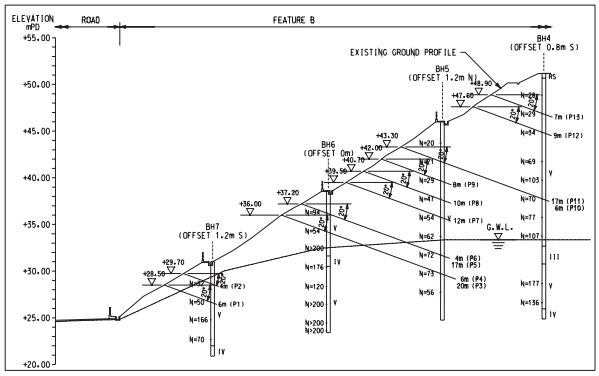

Fig. 2. Typical Section showing test nails and boreholes arrangement

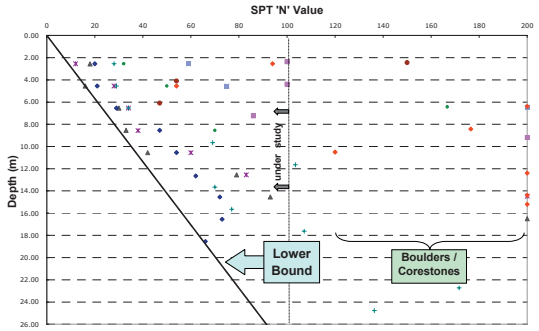

Fig 3. Plot of Depth against SPT 'N' Value

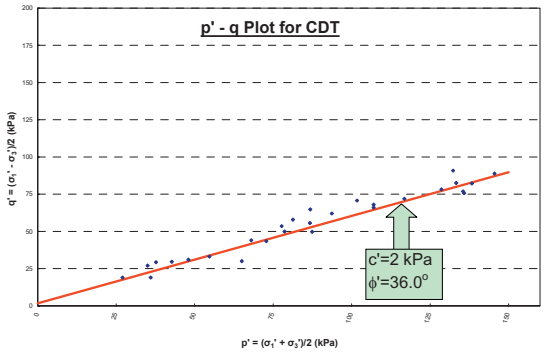

Fig 4. Plot of p'- q' of CDT from Three Sites

3 TEST SET UP

3.1 *Pull-Out Test Nails*

Test nail holes were drilled by a pneumatic drilling rig to form a hole with 150mm diameter at an angle 20° to the horizontal. The lengths of the test nails range from 5m to 20m. A 40mm high yield steel bar was used as the test nail bar.

To effectively isolate the 2-metre bond length for grouting, two types of specially made inflatable packers, namely "Single Packer" and "Double Packer", as shown in Figs. 5a and 5b were developed and used in this study. These specially made inflatable packers were fabricated from an inflatable rubber tube manufactured locally. The "Double Packer" has one additional inflatable rubber tube to cater for local variance of the drillhole wall, all the other details are the same as the "Single Packer". The packer was installed on the nail bar 2.0m from the fixed length end of the test nail and deflated during the insertion of the bar into the drillhole. The packer was then inflated with compressed air to about 2.0 bar pressure and maintained in an inflated state during the grouting operation. Grout was subsequently injected through the grout pipe in a continuous operation until excess grout emerged from the grout return pipe. After the grout reached final set, the packer was deflated. General views of the packers after inflation are presented in Figs. 6a and 6b.

In order to monitor the performance of the inflatable packer during the grouting operation, a real-time video monitoring and recording system was set up by installing a video camera inside the soil nail drill holes. This camera was placed in front of the packer on the nail bar as shown in Fig. 7. During the grouting operation, the inflatable packer was monitored by connecting the video camera to a LCD monitor set up on site.

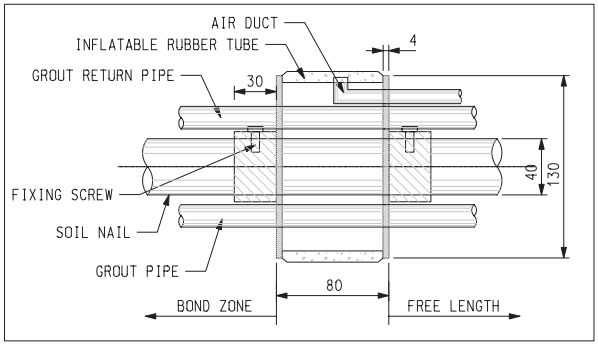

Fig. 5a. Specially Made Inflatable Packer - "Single Packer" Details

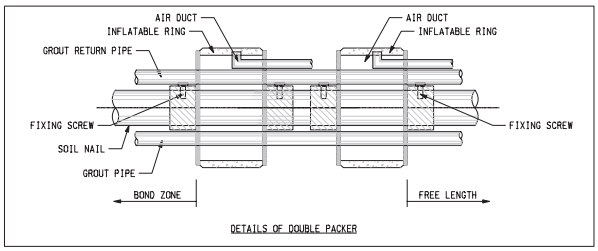

Fig. 5b. Specially Made Inflatable Packer Details - "Double Packer" Details

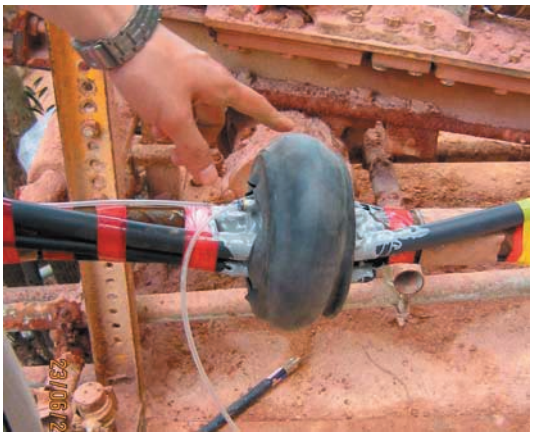

Fig. 6a. General View of the "Single Packer" after Inflation

Fig. 6b. General View of the "Double Packer" after Inflation

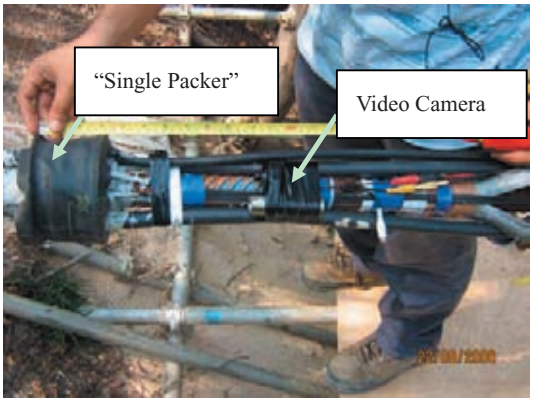

Fig. 7. General View of the Specially Made Inflatable Packer and Video Camera installed on the Nail Bar

3.2 *Instrumentation of the Pull-Out Test Nails*

Strain gauges were installed along the test nail bar including the bond zone and free length, to measure the strain variation during the pull-out test. These electrical-resistance strain gauges were installed in pairs, one at the top and one at the bottom of the nail bar, at intervals of about 0.5m and 1.0m to 1.5m in the fixed and free lengths respectively. Strain gauges were arranged in pairs to ensure accurate results were recorded. The strain gauges were connected to an automatic data acquisition system comprising a data logger and a laptop computer. Real-time continuous monitoring was performed throughout the pull-out test.

3.3 *Pull-Out Test Procedures*

The pull-out test principally follows the procedures as set out in HKSAR (2006). The test nails were loaded in stages: from the initial load (T_a) via two intermediate test loads (T_{DL1} and T_{DL2}) to the maximum test load (T_P). T_{DL1} is the design bond strength calculated from Eq. (1) with FOS of 2.0 and T_{DL2} is two times T_{DL1}. The maximum test load, T_P, is 90% of the yield load of the test nail bar.

During the first two loading cycles, the intermediate loads (T_{DL1} and T_{DL2}), were maintained for 60 minutes for deformation measurement. After completing the measurement at T_{DL1} and T_{DL2}, the load was reduced to T_a and the residual deformation was recorded. In the last cycle, the test load was increased gradually from T_a straight to maximum test load and then maintained for 60 minutes for deformation measurement. The test nail is considered to be able to sustain the test load if the difference in nail movements at 6 and 60 minutes does not exceed 2mm. Table 1 presents the loading schedule for the test nails for the three sites.

Table 1. Summary of the Loading Schedule

Test Nail No.	Length (m)	T_a (kN)	T_{DL1} (kN)	T_{DL2} (kN)	T_p (kN)
CC359-P1	16	10	56	112	520
CC359-P2	14	10	48	96	520
CC359-P3	12	10	40	80	520
CC359-P4	10	10	31	62	520
CC359-P5	7	10	21	42	520
CC359-P6	5	10	14	28	520
CC359-P7	19	10	57	114	520
CC359-P8	16	10	48	96	520
CC359-P9	16	10	40	80	520
CC359-P10	11	10	32	64	520
CC359-P11	8	10	23	46	520
CC359-P12	6	10	15	30	520
CC359-P12A	6	10	15	30	520
CC359-P13	7	10	23	46	520
CC359-P14	5	10	16	32	520
CC359-P15	10	10	35	70	520
DC79-P1	6	10	23	46	520
DC79-P2	4	10	30	60	520
DC79-P3	20	10	76	152	520
DC79-P4	6	10	23	46	520
DC79-P5	17	10	67	134	520
DC79-P6	4	10	30	60	520
DC79-P7	12	10	50	100	520
DC79-P8	10	10	41	82	520
DC79-P9	8	10	32	64	520
DC79-P10	6	10	23	46	520
DC79-P11	17	10	63	126	520
DC79-P12	9	10	32	64	520
DC79-P13	7	10	23	46	520
DC214-P1	6.5	10	24	48	520
DC214-P2	6.5	10	24	48	520
DC214-P3	8	10	33	66	520
DC214-P4	8	10	33	66	520
DC214-P5	8	10	33	66	520

4 RESULTS AND DISCUSSIONS

4.1 *Field Test Results*

Table 2 summarizes the ultimate pull-out capacity for each test nail respectively. From the test results, it is observed that 32 out of 34, i.e. 94%, of the test nails could sustain the load at T_{DL1}. Test Nail Nos. CC359-P10 and DC79-P5 failed prior to reaching T_{DL1}.

A 0.45m thick fractured quartz vein is located at the depth of the bond zone for Test Nail No. DC79-P5. The low pull out capacity for PC79-P5 may be explained by possible grout loss through the quartz vein.

It is known that the grout return pipe was placed at the bottom of the drillhole for CC359-P10. It is thought this may have allowed an 'air pocket' to form inside the bond zone, thus reducing the pull out capacity.

A typical plot of pull-out load against displacement is given in Fig. 8. The field test results generally demonstrate that the test nails were not over-grouted by comparison with the elastic extension of the free length and it was concluded that reliable results were obtained.

Table 2. Summary of the Ultimate Pull-out Capacity of Test Nails

Test Nail No.	T_{ult} or T_p (kN)	Test Nail No.	T_{ult} or T_p (kN)
CC359-P1	191	DC79-P2	80
CC359-P2	121	DC79-P3	187
CC359-P3	135	DC79-P4	191
CC359-P4	232	DC79-P5	51
CC359-P5	213	DC79-P6	275
CC359-P6	77	DC79-P7	177
CC359-P7	175	DC79-P8	229
CC359-P8	194	DC79-P9	107
CC359-P9	155	DC79-P10	98
CC359-P10	28	DC79-P11	266
CC359-P11	97	DC79-P12	289
CC359-P12	41	DC79-P13	80
CC359-P12A	65	DC214-P1 *	164
CC359-P13	317	DC214-P2 *	145
CC359-P14	471	DC214-P3 *	106
CC359-P15	520	DC214-P4 *	92
DC79-P1	65	DC214-P5 *	182

Note: * denotes Test Nails with "Double Packers"

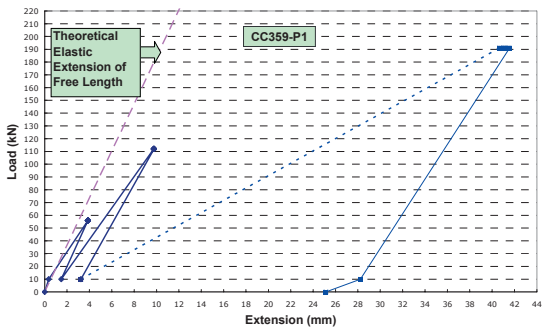

Fig. 8. Load against Displacement (CC359-P1)

4.2 *Interpretation and Discussion on Field Test Results*

4.2.1 Relationship between Pull-Out Capacity and Effective Vertical Stress

A plot of ultimate pull-out capacity of the test nails against effective vertical stress at mid-depth of the grouted section is presented in Fig. 9. The plot does not show a direct linear relationship between the ultimate pull-out capacity and effective vertical stress. A lower bound linear relationship between the ultimate pull-out capacity and the effective vertical stress at the mid-point of the bond zone is shown on Fig. 9.

The theoretical pull-out capacity is derived from Eq. (1) and a FOS of 2.0 is applied to determine the design pull-out capacity. Soil shear strength parameters of c'= 2kPa & ϕ'= 36^o have been adopted which are based on the consolidated undrained triaxial compression test results on the undisturbed samples. The field pull-out capacities are much higher than the theoretical and design pull-out capacities as shown on Fig. 9.

4.2.2 Relationship between Field and Theoretical Pull-out Capacities

Fig. 10 shows the relationship between the field and theoretical pull-out capacities of the test nails. Three lines, namely l_1, l_2 and l_3 are constructed in Fig. 10. l_1 denotes the upper bound of the design bond strength calculated from Eq. (1). l_2 and l_3 are the allowable bond strength with a FOS of 2.0 and 1.5 respectively applied to Eq. (1).

It can be concluded that the current soil nail design is reliable but tends to be conservative given most of the test data are above Line l_1. This suggests that a reduction in FOS against pull-out failure between soil and grout interface may be possible.

It may be noted that several of the tests were carried out with the soil nails located above the water table, possibly in partially saturated soil. The pull out resistance in a partially saturated soil where pore water pressures are negative would be expected to be greater than for a saturated soil at an equivalent depth.

4.2.3 Relationship between Pull-Out Capacity and SPT 'N' Values

A plot of the ultimate pull-out capacity against SPT 'N' values is given in Fig. 11. The results indicate that there is no direct linear relationship between ultimate pull-out capacity and the SPT 'N' values as shown in Fig. 11. A lower bound trend line (L_1) assuming a linear relationship is constructed to correlate the ultimate pull-out capacity (f_s) and the field SPT 'N' value is given in Eq. (2).

$$f_s = 1.4N \quad (2)$$

This empirical linear relationship given in Eq. (2) has discounted the test results for Test Nail Nos. CC359-P10 and DC79-P5.

The formula given in Eq. (2) is generally in line with GEO (2006) which suggested the ratio of the average mobilized shaft resistance to SPT 'N' value for large-diameter bored pile for saprolites in Hong Kong is between 0.8 and 1.4. In addition, a linear relationship with a ratio of f_s /N of 2.0 (L_2) is constructed in Fig. 11. The results show that over 95% of the ultimate pull-out capacity of test nails is above L_2. It indicates that the test results have similar findings with GEO (2006) which advised the relationship of f_s /N of 2.0 can be taken for the shaft resistance for small displacement piles such as steel H-piles.

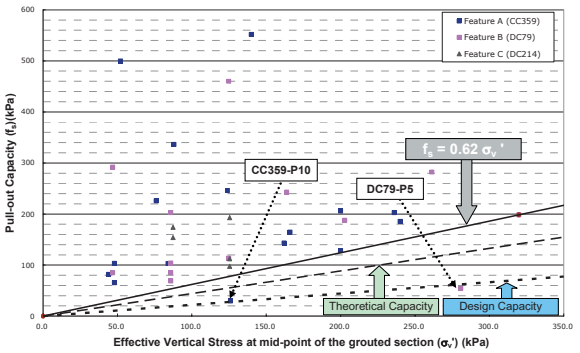

Fig. 9. A Plot of Ultimate Pull-Out Capacity against Effective Vertical Stress

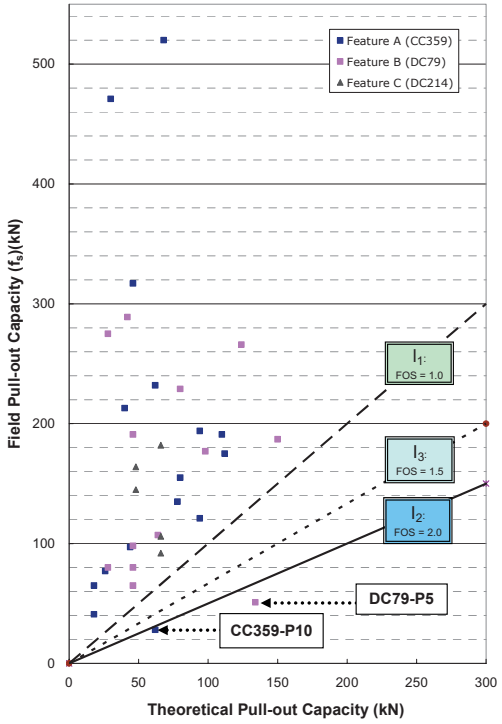

Fig. 10. Pull-out Capacities of Field against Theoretical

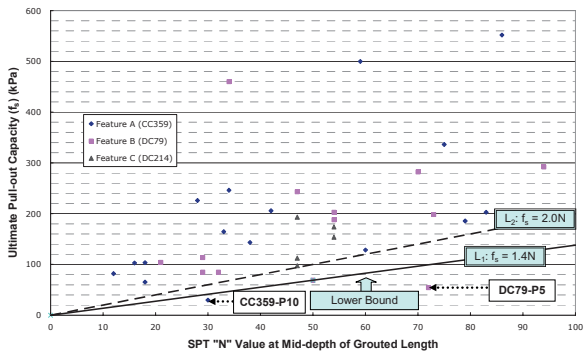

Fig. 11. A Plot of Ultimate Pull-Out Capacity against SPT 'N' Value

4.3 *Load Distribution along Test Nail*

A typical plot of induced force along the test nail is given in Fig. 12. The field test results indicate that the induced tensile force within the bond zone gradually increases with the applied test load. The pull-out resistance was mainly taken up by the front portion of the bond zone. It illustrates that the mobilization of the pull-out resistance rapidly diminishes when the induced pull-out resistance is transmitted into the bond zone of the bar. This result was reflected by the response of the installed strain gauges along the nail bar within the bond zone. In addition, a larger part of the bond zone progressively contributes to provide pull-out resistance when increasing the applied test load.

Assuming the tensile stress distributes linearly within the bond zone, the induced force (F_1) at the head of the bond zone can be interpreted as shown in Fig. 12. This induced force (F_1) is compared to the induced force (F_2) which was measured from the nearest strain gauges located closed to the front face of the packer. A small portion of friction loss (maximum 4kN, which is 4% of the total applied load) may be encountered due to there being some contact between the wall of the drillhole and the inflatable packer. This small portion of loss is considered to be negligible as it falls within the accuracy tolerance of the loading apparatus.

The tensile force within the free length of the test nail generally demonstrates elastic behaviour. A small portion of frictional loss was experienced at the location of the centralizer when the test loads were applied. This small portion of frictional loss is small when compared to the applied pull-out force.

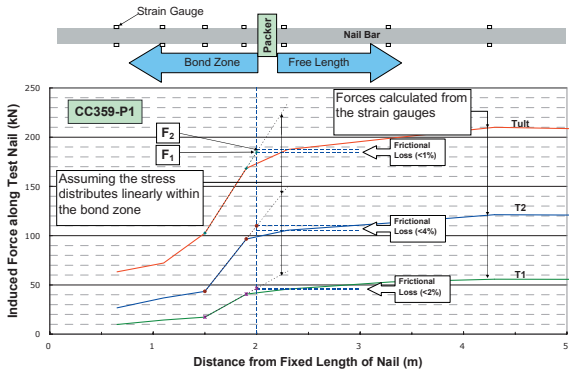

Fig. 12. A Plot of Load Distribution within the Fixed Length and along the Nail bar

4.4 *Performance of the Inflatable Packer*

The inflatable packers performed well and without incident during grouting operations. However, bursting of an inflatable packer was recorded on one occasion during inflation, following insertion into the drillhole. On this occasion the packer had been exposed to sunlight for extended periods after installing the packer onto the nail bar. It was concluded that strong sunlight may adversely affect the strength of the inflatable rubber tube. It is recommended to install the test nails immediately after packer fabrication.

4.5 *Soil Nail Design*

In order to compare the lengths of soil nail design determined by the Conventional and Working Bond Methods, one critical

section from Feature B (DC 79) was chosen to examine the allowable design pull-out resistance of soil nails.

The theoretical allowable pull-out resistances of Conventional Method and Working Bond Method are based on Eqs. (1) and (2) respectively. A design critical slip surface is determined from a slope stability programme by applying a point load on the slope boundary in order to derive a slip surface with FOS greater than 1.40. The design groundwater levels, shear strength parameters and soil properties are identical in both methods.

After determining the design critical slip surface, soil nails were modelled in the slope stability computer programme and the allowable pull-out resistance is calculated from Eqs. (1) and (2). A FOS of 2.0 is applied to determine the design pull-out resistance in order to guard against pull-out failure between soil and grout interface. Based on the slope stability analyses, a summary of soil nail length with different design methods is presented in Table 3. The results indicate that the average design bond length would reduce from 2.5m to 1.4m, or approximately 46% in total design bond length if Working Bond Method is adopted.

In addition, assuming a ratio of f_s /N of 2.0 in the Working Bond Method, the average design bond length would reduce from 2.5m to 1.1m which is approximately 58% of reduction in total design bond length as shown in Table 3. Given the free length of both methods are the same, there is a 7% and 10% reduction in total length of soil nails for f_s = 1.4N and f_s = 2.0N respectively.

Table 3. Summary of Soil Nail Lengths using Different Methods

	Nail Length (m)			Design Bond Length (m)		
	CM	WB		CM	WB	
Row (mPD)		f_s = 1.4N	f_s = 2.0N		f_s = 1.4N	f_s = 2.0N
A (+27.0)	16.0	13.8	13.2	4.3	2.1	1.5
B (+30.0)	16.0	15.8	15.2	2.3	2.1	1.5
C (+32.0)	17.0	17.2	16.2	2.3	2.5	1.5
D (+34.0)	17.0	17.8	16.3	1.7	2.5	1.0
E (+36.0)	17.0	17.3	16.3	1.7	2.0	1.0
F (+38.0)	17.0	15.5	16	2.0	0.5	1.0
G (+41.0)	16.0	13.8	14.3	2.7	0.5	1.0
H (+43.0)	15.0	12.8	13.3	2.7	0.5	1.0
J (+45.0)	14.0	11.5	11.5	3.0	0.5	0.5
K (48.0)	10.0	8	8	2.5	0.5	0.5
Total	155.0	143.5	140.3	25.2	13.7	10.5
Average	15.5	14.4	14.0	2.5	1.4	1.1

Note: *CM* and *WB* denote Conventional and Working Bond Methods respectively.

5 CONCLUSIONS AND RECOMMENDATIONS

The following conclusions and recommendations can be drawn from the above:

Specially made inflatable packers ("Single Packer" and "Double Packer") were developed and used in this study. It was found that the performance of the inflatable packers was reliable and practical to control the grouted length. A higher confidence level in terms of grout integrity and grouted length was possible, especially for the "Double Packer". These inflatable packers performed well compared with the traditional packer. Further development and wider use of this type of inflatable packer will provide a higher confidence level for pull-out test data.

The frictional loss due to the presence of packer and centralizers was minimal (maximum 4kN, which is 4% of the total applied load) and it falls within the accuracy tolerance of the loading apparatus.

Given grout loss and imperfections within the bond zone could occur after completing the grouting operation, it is recommended to allow sufficient volume of excessive grout to emerge from the grout return pipe and install this pipe at the top of the drillhole. Any local geological features should be identified prior to determining the location of the test nails. It is also recommended installing the test nails immediately after fabrication of the packer onto the nail bar.

The current design methodology based on Watkins & Powell (1992) was found to be reliable but tends to be conservative. Based on the data from these tests the FOS applied in determining the design allowable pull-out resistance can be reduced. With further testing a reduction in the FOS against pull-out failure between soil and grout interface may be possible.

An empirical correlation between the ultimate pull-out resistance and the SPT 'N' values was established. It was found that about half of the design bond length of soil nails could be reduced if the Working Bond Method is adopted compared with the Conventional Method. It is suggested to further investigate the feasibility to use SPT 'N' values to calculate the required bond length in order to provide a more cost-effective soil nail design in Hong Kong.

The Working Bond Method may become an alternative method for soil nail design in Hong Kong if sufficient reliable test data can be compiled.

ACKNOWLEDGEMENTS

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Analysis of Driven Piles with Load Transfer Method

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Abstract: The load transfer method using Winkler springs and hyperbolic relationship between the pile displacements and the shaft or the toe resistances is adopted herein for analyzing pile performance. The hyperbolic model is verified with 2 case histories on instrumented driven test piles. The interpreted shaft and base resistances are consistent with the experience reported in literature.

1 INTRODUCTION

The load transfer method proposed by Coyle & Reese (1966) has been one of the consistent frameworks for considering the load transfer mechanism of piles. However, in this load transfer method the soils were modelled by linear elastic-perfectly plastic springs. Since the stress-displacement curves for soils would be nonlinear, linear models are deemed inappropriate.

It is considered that the load-displacement relationship of the interface between the pile and the supporting ground could be expressed by the hyperbolic functions. The modified load-transfer method was adopted to analyze case histories on instrumented test piles. The agreement between the calculated and the observed values demonstrates that the hyperbolic load-displacement model is applicable for assessing the performance of the piles.

2 ANALYTICAL METHOD

2.1 Load Transfer Analysis

The load transfer method using Winkler springs, sometimes called the *t-z* curve method, that proposed by Coyle & Reese (1966) has been adopted for analyzing the performance of piles. Basically each pile is idealized as a series of elastic discrete elements supported by springs on its side and a spring at the base. These springs represent the soil-structure interaction. This method was adopted by Chang & Broms (1991) and Moh et al. (1995) for analyzing bored piles.

2.2 Hyperbolic Model

In the load transfer method proposed by Coyle & Reese (1966), the soils were modelled by linear elastic-perfectly plastic springs. Since the stress-displacement curves for soils would be nonlinear, linear models are deemed inappropriate. The pile shaft or pile base resistances versus displacement plots could be expressed by the hyperbolic curve approximation as illustrated in Fig. 1. The load-displacement curves for soil or rock could be represented by the hyperbolic function:

$$\tau = \tau_{max} / (1 + \delta_r / \delta) \quad (1)$$

where τ_{max} is the horizontal asymptote or the ultimate shaft resistance or base resistance, δ_r is the reference displacement and τ is the mobilized resistance at the pile displacement δ.

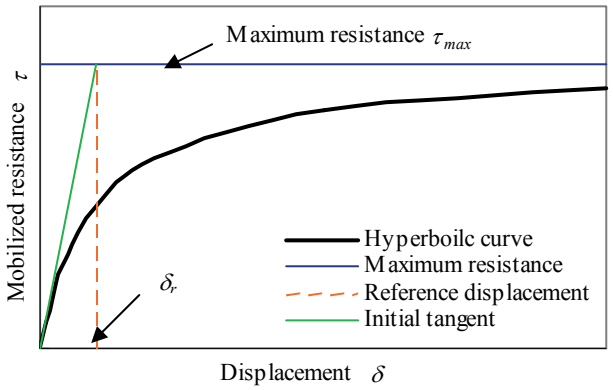

Fig. 1. Hyperbolic model for resistance versus displacement relationship.

Equation (1) shows that resistances of $0.5\tau_{max}$ and $0.9\tau_{max}$ would be mobilized at displacements of δ_r and $10\delta_r$, respectively. As defined in Fig. 1, the initial tangent of the curve intersects the horizontal asymptote at δ_r. Therefore the initial stiffness or the slope of the curve m is defined as:

$$m = \tau_{max} / \delta_r \quad (2)$$

Fleming (1992) adopted the hyperbolic load-displacement relationship to predict the performance of piles in clay, sand and weak rock materials. While most of the literature reported the use of the *t-z* curve method for analyzing bored piles, load transfer analysis on driven piles is rather limited probably due to less case histories on instrumented driven piles. However, there are results of tests on instrumented driven piles available in the literature. Premchitt & Gray (1987) reported the results of load tests on 3 driven precast concrete piles. Wang & Swann (2001) presented the results of instrumented driven steel tubular piles. These 2 case histories are critically reviewed herein and the hyperbolic parameters for various soil strata are interpreted from the pile load test results.

3 HYPERBOLIC PARAMETERS

3.1 Case 1 - Precast Concrete Piles

Premchitt & Gray (1987) presented a well-documented case history on 3 instrumented precast concrete piles located in the Kowloon Bay reclamation area, Hong Kong. As depicted in Fig. 2, the fill platform was underlain sequentially by the marine

Table 1. Summary of test piles for Case 1.

Pile no.	Diameter Outer mm	Diameter Inner mm	Pile length m	Maximum load MN	Head displacement Maximum mm	Head displacement Residual mm
58	500	250	41.5	5.4	19.0	2.0
118	500	250	41.0	5.4	21.9	2.7
26A	500	300	36.0	4.6	19.3	1.7

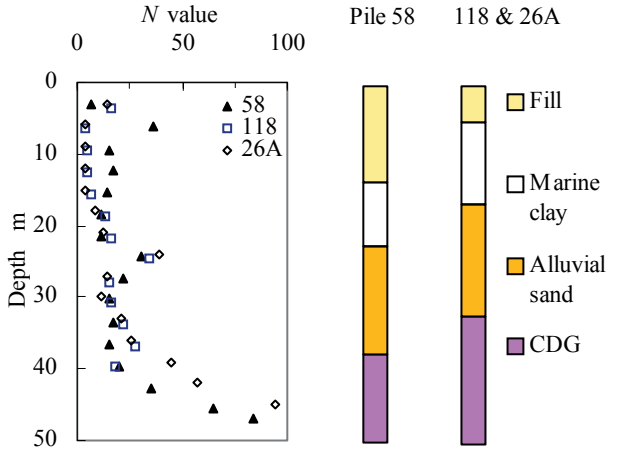

Fig. 2. Subsoil conditions for Case 1.

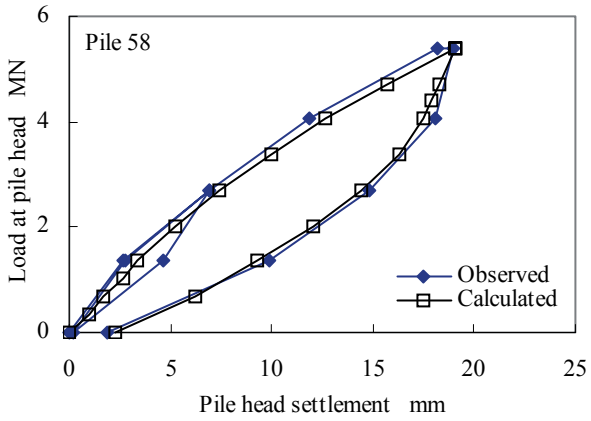

Fig. 3. Pile head displacements for Pile No. 58.

deposits, the alluvium and the completely decomposed granite (CDG). As reported by Fyte et al. (2000), the alluvium belongs to the Chek Lap Kok Formation of the Middle to the Late Pleistocene Age. The marine clay belongs to the Hang Hau Formation of the Holocene Age. The groundwater level was located at the depth of about 3 m.

The test piles were precast prestressed spun concrete close-ended tubular piles (Daido piles) of 500 mm in outer diameter and 36 m to 42 m in length. Cone-shape steel shoes were fixed to the pile tips. Table 1 summarizes the maximum displacements and the residual displacements observed at the pile heads. Most of the strain gauges in Test Pile 26A were not functional and only the pile head displacement data were reported.

The hyperbolic parameters for the pile soil interfaces along the shafts could be assessed directly from the *t-z* curves interpreted by Premchitt & Gray (1987). However, in order to take the variation of subsoil properties with depth into account and to assess the *t-z* curves for the pile bases, the author conducted numerical analyses by using the Winkler springs method based on the

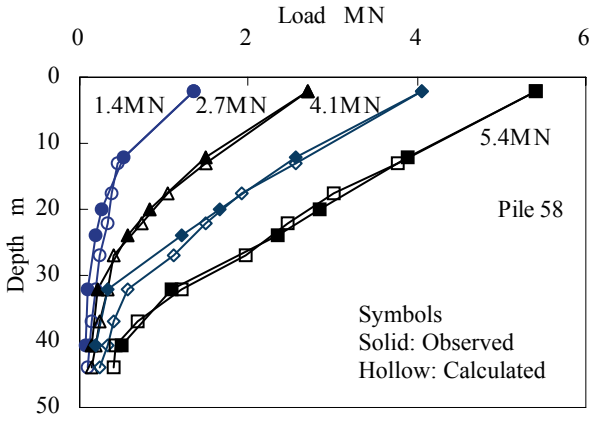

Fig. 4. Load distribution curves for Pile No. 58.

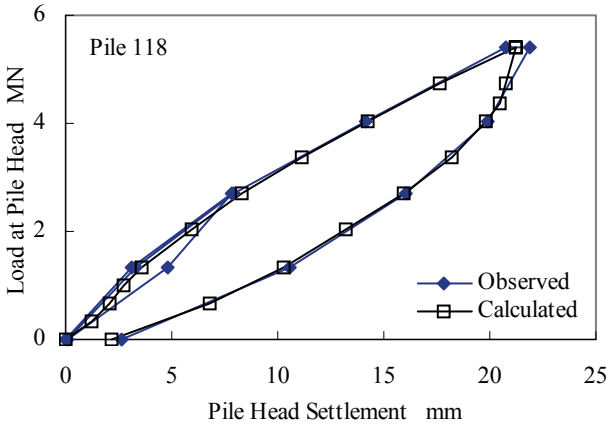

Fig. 5. Pile head displacements for Pile No. 118.

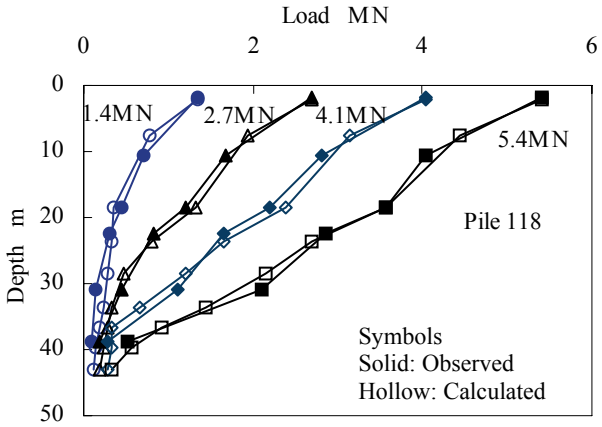

Fig. 6. Load distribution curves for Pile No. 118.

Standard Penetration Test *N* values, pile head displacements and the load distribution data. The sets of the hyperbolic parameters for describing the *t-z* curves along the shafts and at the bases of Test Piles 58 and 118 are back-calculated. Figs. 3 and 4 depict the results of calculated pile head displacements and load distribution curves for Test Pile 58, and Figs. 5 and 6 present those results for Test Pile 118. The hollow spaces inside the piles were grouted with concrete to prevent ingress of water. Young's modulii of 37.7 GPa and 11.5 GPa are adopted for the precast concrete shell and the grouted core respectively.

Table 2. Back-analyzed hyperbolic parameters for shaft resistance - Case 1.

Soil type	*N* value	Maximum shaft resistance τ_{s-max} kPa	τ_{s-max}/N kPa	Reference shaft displacement δ_{sr} mm
Fill	7 ~ 17	110 ~ 130	8	2
Marine clay	4 ~ 14	60 ~ 140	10	2
Alluvial sand	9 ~ 34	150 ~ 300	10	2
CDG	18 ~ 35	180 ~ 350	10	1.5

Table 3. Back-analyzed hyperbolic parameters for base resistances.

Case	Soil type	Average *N* value	Maximum base resistance τ_{b-max} MPa	τ_{b-max}/N kPa	Reference base displacement δ_{br} mm
1	CDG	49 ~ 75	6.4 ~ 9.8	130	3
2	CDG	58	1.9	32	3
	Granite bedrock	-	11.4	-	2

Table 4. Summary of compression test piles for Case 2.

Pile no.	Diameter mm	Pile length Gross m	Embedded m	Max. load MN	Head displacement Maximum mm	Residual mm
TP1	800	58.2	40.8	8.9	41.2	4.7
TP9	800	63.2	45.8	4.0	30.4	1.5

Table 5 Back-analyzed hyperbolic parameters for shaft resistance - Case 2.

Soil type	*N* value	Maximum shaft resistance τ_{s-max} kPa	τ_{s-max}/N kPa	Reference shaft displacement δ_{sr} mm
Marine clay	0 ~ 1	5 ~ 10	-	2
Alluvial sand	3	35	-	2
Alluvial clay	7 ~ 28	45 ~ 90	2 ~ 3.5	2
CDG	28 ~ 73	100 ~ 235	1.2 ~ 3.5	1.5

The back-calculated maximum shaft resistances and their corresponding reference shaft displacements (τ_{s-max} , δ_{sr}) and the maximum base resistances and their corresponding reference base displacements (τ_{b-max} , δ_{br}) are summarized in Tables 2 and 3 respectively. To allow for variability of the strata across the site and for comparison with other case histories, the τ_{s-max} and τ_{b-max} values are normalized with the *N* values. For the correlation with the base resistances, the weighted average of the *N* values within the depth of 10 pile diameters beneath the bases are used.

3.2 Case 2 - Steel Tubular Piles

Wang & Swann (2001) presented the case histories of load tests on open-ended steel tubular piles supporting the concrete deck of an off-shore development in Hong Kong. The instrumented piles were driven in marine environment with the seabed at a depth of 15 m below sea level. In a descending sequence, the subsoil comprised the marine clay, alluvial sand, alluvial clay and the completely to highly decomposed granite (CDG to HDG) strata. Table 4 summarizes the maximum displacements and the residual

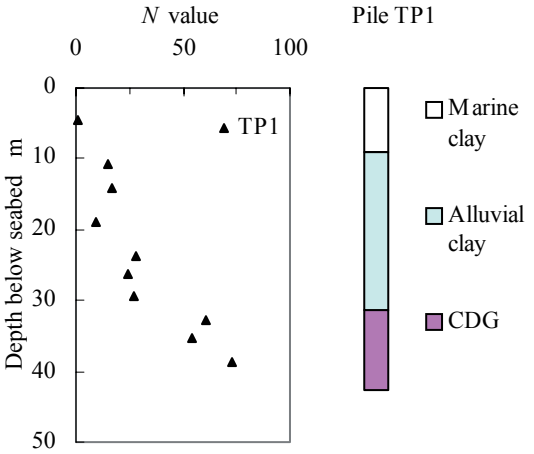

Fig. 7. Subsoil conditions for Case 2.

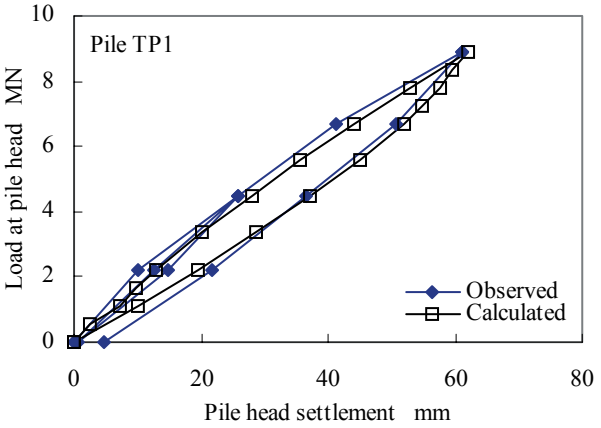

Fig. 8. Pile head displacements for Pile No. TP1.

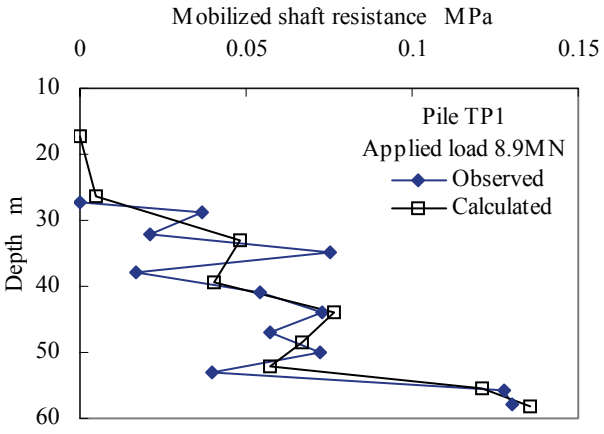

Fig. 9. Distribution of shaft resistance for Pile No. TP1.

displacements observed at the pile heads. Fig. 7 depicts the subsoil conditions in the vicinity of Test Pile TP1.

The hyperbolic parameters for the shafts and the bases were analyzed by matching the pile head displacements, the distribution of shaft resistance and the loads transmitted to the pile bases. Results of analysis are shown in Figs. 8 and 9, indicating a good agreement between the observed and the calculated data. The sets of back-calculated hyperbolic parameters are presented in Tables 3 and 5.

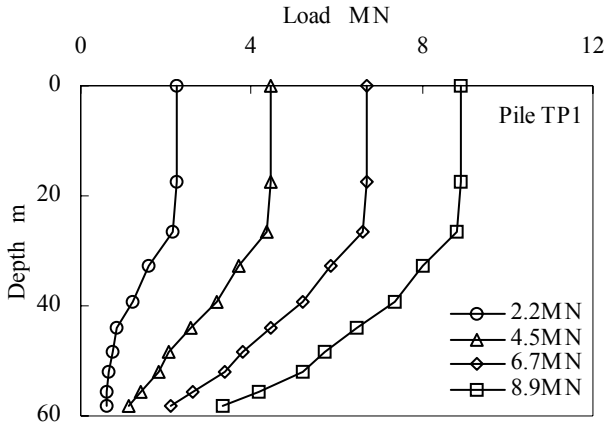

Fig. 10. Calculated load distribution for Pile No. TP1.

The calculated load distribution for Test Pile TP1 shown in Fig. 10 indicates that the pile base, driven to granite bedrock, contributed approximately 40 % of the pile resistance at the maximum applied load of 8.9 MN. Such a percentage of contribution of base load is in agreement with the measured results. It is noted that the seabed was located at the depth of 17.4 m below the pile head level.

4 INFERRED MAXIMUM RESISTANCES

4.1 Shaft Resistances

It should be noted that the precast concrete piles (Case 1) and the steel tubular piles (Case 2) reviewed in this study are close-ended large displacement and open-ended small displacement driven piles respectively.

Figs. 11 and 12 present the correlation of the τ_{s-max} values for the CDG, the marine clay and the alluvial deposits with the N values. For the large displacement piles in CDG, alluvial sand and marine clay, the τ_{s-max}/N ratios are fairly consistent and equal to 10 (kPa). For the small displacement piles in CDG, the τ_{s-max}/N ratios range from 1.2 to 3.5 (kPa), with an average of 2.7 (kPa). For the small displacement piles in alluvial clay, the τ_{s-max}/N ratios range from 2 to 3.5 (kPa), with an average of 3.0 (kPa).

The τ_{s-max}/N ratios inferred in this study are consistent with the experience on driven piles reported in literature. Ng (1989) proposed that the shaft resistance to N ratios ranging from 4 to 7 (kPa) might be taken for design of precast concrete piles in saprolite. Based on limited data, Geoetechnical Engineering Office (2006) reported that the shaft resistance to N ratios for small displacement piles could be taken as 1.5 to 2 (kPa).

Based on the data of 4 test piles driven in the weathered Singapore's Jurong Formation, Chang (2007) observed that the mobilized shaft resistance to N ratios for open-ended spun piles had an average of 2.3 (kPa). The Jurong Formation comprises sedimentary rocks including sandstone, mudstone and siltstone.

Shaft resistances mobilized along large displacement piles are significantly larger than those mobilized along small displacement piles. As shown in Fig. 11, the τ_{s-max}/N ratio for large displacement piles in CDG is 3.7 times of that for the small displacement piles. Similarly, Fig. 12 shows that the τ_{s-max}/N ratio for large displacement piles in clayey soil is 3.3 times of that for the small displacement piles.

The difference in shaft resistances that inferred from this study is consistent with the experience in Hong Kong. Based on limited

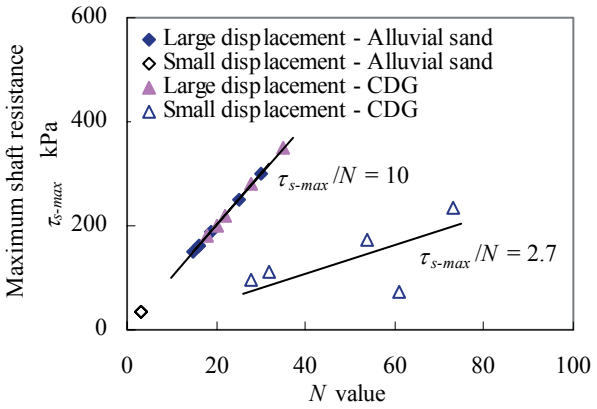

Fig. 11. Variation of maximum shaft resistance for CDG and sandy soil.

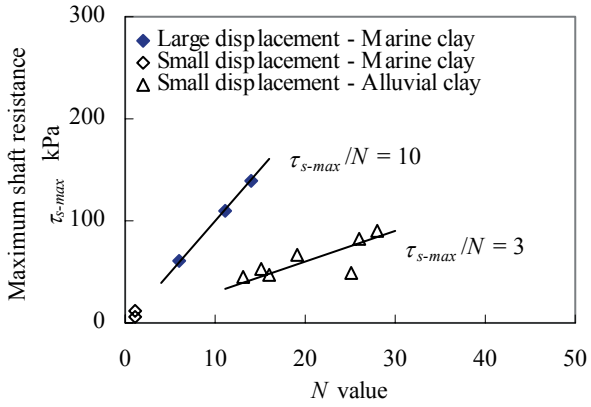

Fig. 12. Variation of maximum shaft resistance for clayey soil.

case histories on instrumented driven piles, Geotechnical Engineering Office (2006) proposed that the mobilized shaft frictions of the close-ended large displacement piles in saprolites be 3 to 4 times of those for the open-ended small displacement piles. It is believed that the large displacement driven piles would induce larger lateral stresses along the pile shafts than those along the small displacement driven piles.

4.2 Base Resistances

For pile bases rested on CDG, Tables 3 shows that the τ_{b-max}/N ratios for the close-ended (Case 1) and open-ended (Case 2) piles are 130 and 32 (kPa) respectively. The τ_{b-max}/N ratio of 32 (kPa) for the open-ended pile is consistent with the case history reported by Chang (2007), who observed that the mobilized base resistance to N ratio for the open-ended driven spun piles in the sedimentary rocks of the Jurong Formation was 35 (kPa). Such base resistances were developed at the maximum test loads ranging from 2.5 to 2.8 times the design loads. Based on the limited data presented in this study, it appears that the τ_{b-max} value for a close-ended pile can be 4 times of that for an open-ended pile.

For the open-ended steel tubular pile driven to bedrock (Pile TP1), the inferred τ_{b-max} value of 11.4 MPa is consistent with the experience in Hong Kong. McNicholl et al. (1989) stated that limited loading tests on driven piles suggested the base resistances could range from 16 MPa to 21 MPa.

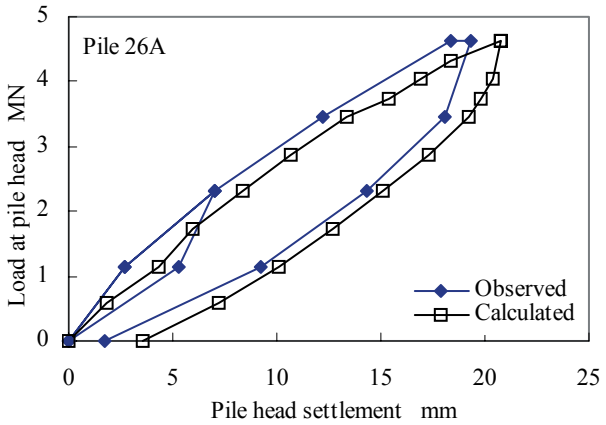

Fig. 13. Pile head displacements for Pile No. 26A.

5 PREDICTION OF PILE DISPLACEMENTS

The pile head displacement data of Test Pile 26A reported by Premchitt & Gray (1987) offered the opportunity for verifying the hyperbolic model. The sets of hyperbolic parameters assessed from the test piles of Case 1 and summarized in Tables 2 and 3 are adopted for the Class 2 prediction.

Fig. 13 shows that the calculated pile head displacements are in a close agreement with the observed values. At the maximum test load of 4.6 MN, the calculated and the observed pile head displacements differ by 1.4 mm only.

It is desirable to establish site-specific hyperbolic parameters to be adopted in designs based on the results of loading tests on instrumented piles because of the great variations in ground conditions and construction practice.

6 CONCLUSIONS

Based on the data reported in 2 case histories of instrumented driven piles, the load-displacement relationships of the pile-soil interfaces of various soil strata have been derived. The results of this study give the following concluding remarks:

(1) The load transfer with Winkler spring method with hyperbolic load versus displacement relationships for the pile-soil interfaces is applicable for analyzing as well as predicting performance of driven piles.

(2) The maximum shaft or base resistances for piles in completely decomposed granite derived from the 2 case histories are consistent with published data interpreted from instrumented piles.

(3) The maximum shaft resistance for close-ended large displacement piles is about 3.5 times of those for the open-ended small displacement piles.

(4) Based on limited data, it appears that the maximum base resistance for a close-ended large displacement pile in completely decomposed granite can be 4 times of that for an open-ended small displacement pile.

This study demonstrates the fact that the load-transfer method using Winkler springs is a useful analytical tool for performance-based design.

ACKNOWLEDGEMENTS

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Design and Construction of Shaft-Grouted Friction Barrette in Tung Chung Designated Area

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K. M. Chan
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Abstract: Designated Area of Northshore Lantau in Hong Kong is characterized by complex geological condition of karst-related deposits, metasedimentary rocks, soft or loose or weakly lithified superficial deposits and anomalously deep or steeply inclined rockhead prevailing over the site area. These features had caused difficulties in constructing foundations for the high-rise developments within part of the area. This paper aims at presenting the experience encountered during the design and construction of 103 nos. shaft-grouted friction barrettes with a maximum ultimate pile capacity of 45MN in supporting six towers of a residential development in the designated area. The pile capacity is enhanced through introducing post-construction grout into the interface between the perimeter of the pile and the surrounding soils. There is no precedent case for such pile type founded in the abovementioned geological formation. In order to verify the design assumptions, two instrumented trial barrettes were constructed and loaded to a maximum test load of 45MN.

1 INTRODUCTION

Six residential towers were proposed to be constructed within the site located at Wai Tung Road, Tung Chung of North Lantau Island. Fig. 1 shows the location of the proposed towers. Ove Arup and Partners Hong Kong Limited were commissioned by Bachy Soletanche Group to be the geotechnical consultant for the foundation contract. Shaft-grouted friction barrettes with a dimension of 2.8m x 0.8m on plan with an ultimate pile capacity of 45MN were designed to be the foundation system of the project. The pile compressive capacity is derived mainly from the shaft friction in alluvium, completely decomposed granite (CDG), completely decomposed rhyolite (CDR) and diamict deposit (DD) enhanced by shaft grouting, combined with a minor contribution from the end bearing resistance.

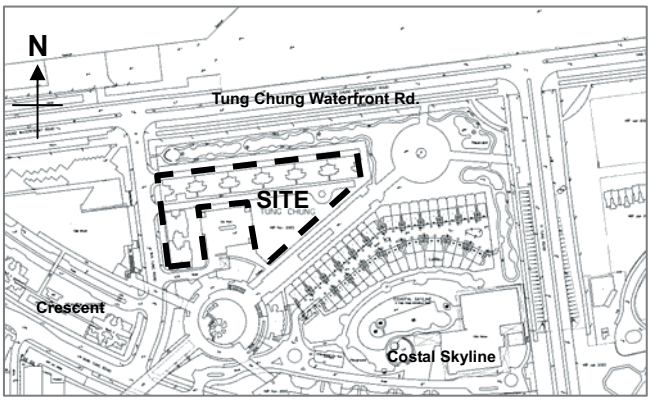

Fig. 1. Site location plan.

2 GEOLOGICAL CONDITION OF THE SITE

According to the Hong Kong Geological Survey Sheet Report No.6 (GEO, 2002) and Geological Map of Hong Kong - Millennium Edition (GEO, 2000), the Tung Chung New Town is underlain predominantly by the Medium Grain Lantau Granite intruded by granitoid dykes (feldsparphyric rhyolite to porphyritic microgranite) that form part of the Lantau Dyke Swarm.

Nigel and Lai (2006) reported that Tung Chung Formation contains collapse features created by extensive dissolution of marble xenoliths, which include laminated cavity infill deposits and soil pipes to depths of over 150m. The collapse features are associated with diamict deposit, which is a non-genetic term that describes poorly sorted, terrigenous, sediment composed of a wide range of rock fragments set in predominantly clay/silt matrix.

A total of 40 nos. of drillholes were sunk to a maximum depth of 200m below existing ground, which revealed that the site is underlain by complex ground conditions, comprising reclaimed fill with SPT N-values ranging from 8 to 44 overlying marine deposit with N-values varying from 2 to 12, which is then underlain by alluvial deposit with N-values ranging from 2 to greater than 200. Karst-related deposits comprising DD, CDG, CDR and decomposed metasedimentary rocks, silt, clay, sand and gravel deposits with N-values exceeding 15 to greater than 200 occur beneath the alluvial materials. The base of the karst deposits was found in excess of –195mPD without encountering solid bedrock. A typical geological profile across the site is shown in Fig. 2.

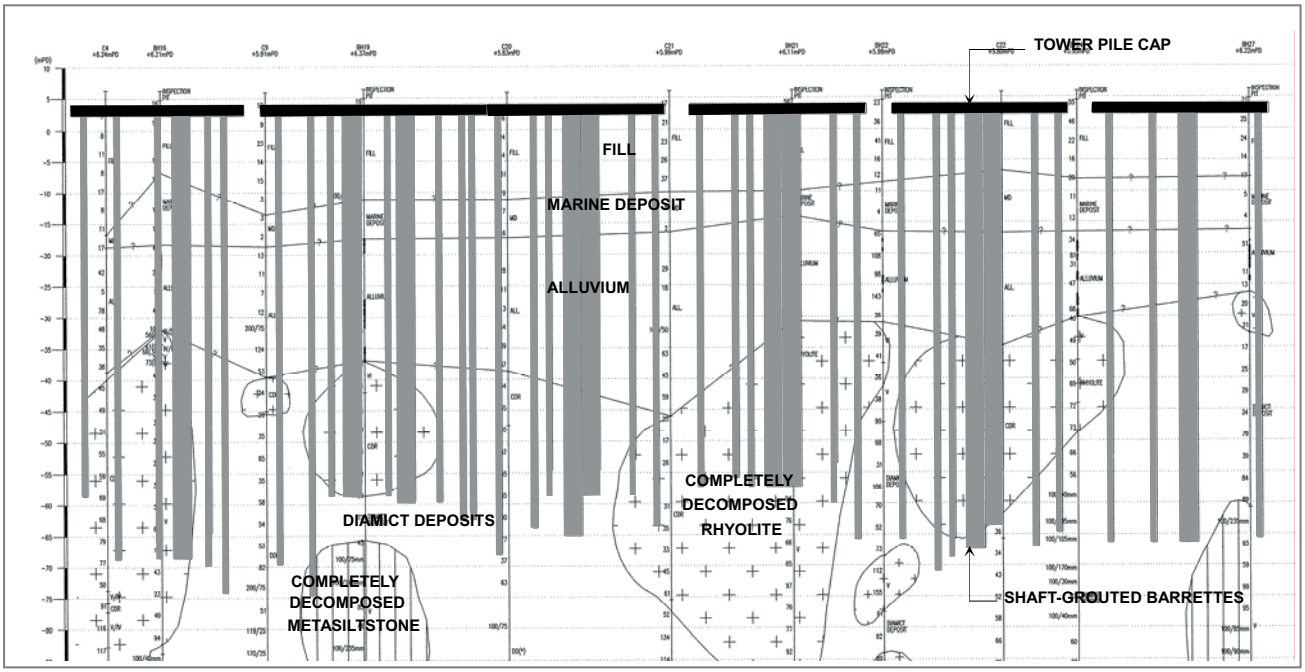

Fig. 2. Geological section across the site.

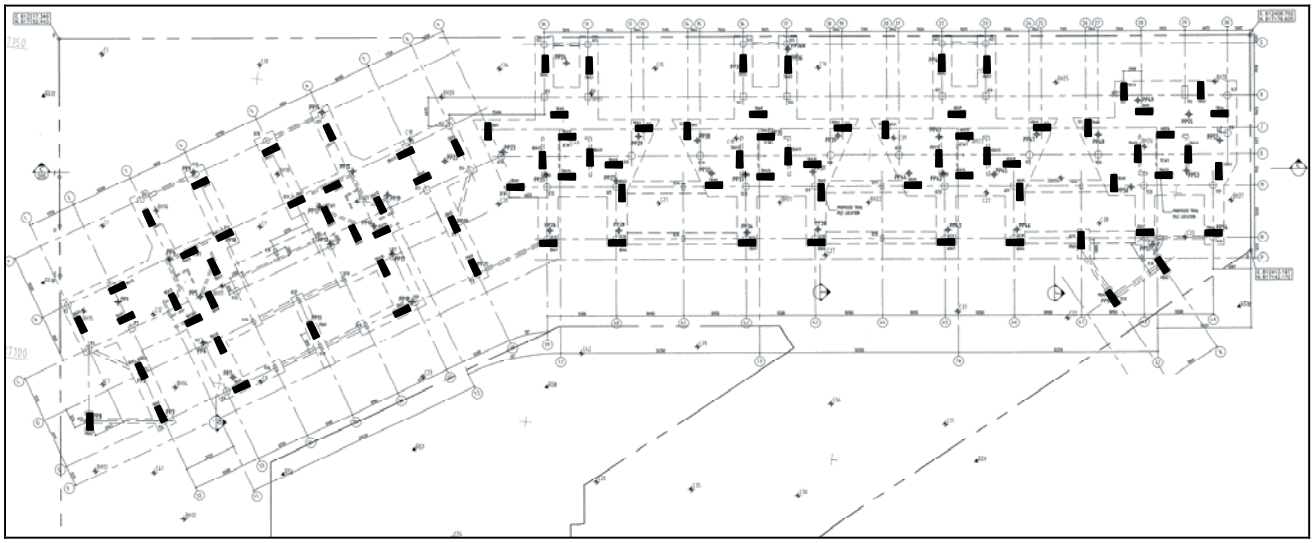

Fig. 3. Barrette layout plan.

3 FOUNDATION DESIGN APPROACH

3.1 *Use of shaft-grouted barrette*

The proposed development consists of six 18 storeys towers with a height of about 64m above existing ground and a single level of semi-basement. In the conforming design, over 600 nos. of steel H-piles were proposed to be driven to approximately 90m depth. However there were a few technical concerns associated with this piling option.

In view of the required substantial penetration depth, it would be difficult to obtain a true final set as some of the pile stresses are locked in along the pile. Besides, the residual load in the long driven pile may result in excessive pile settlement during the proof load test which may not fulfill the statutory requirement. In addition, the presence of a thick layer of gravel/cobble within the alluvial deposit would hinder the construction of the driven H-pile and would be difficult to control the verticality during pile driving resulting in high damaging rate. Lastly, as the piles were fully saturated within the footprint of the tower, replacement of damaged piles should be installed along the perimeter of the pile group resulting in larger pile cap.

Therefore, as an alternative to the conforming design, a total 90 nos. of 2.8m x 0.8m shaft-grouted friction barrettes were proposed with pile founding levels varying from –54mPD to –77mPD. The barrette layout plan is presented in Fig. 3.

There were a few reported cases of using shaft grouted barrette/bored pile in elsewhere of Hong Kong, such as the full-scale load testing programme for KCRC West Rail Phase I as reported by Plumbridge *et al* (2000) and MTRC Kowloon Station Mega Tower Development as reported by Lui *et al*. (2004). However, there was no precedent case of applying shaft grouting technique in this designated area.

3.2 *Typical arrangement to facilitate shaft grouting*

Mild steel shaft grouting pipes of 42mm internal diameter, with prefabricated manchettes at 1m intervals, were attached to the external face of the reinforcement cage, within the zone of concrete cover on the sides of the barrette. In total, eight shaft grouting pipes, including two spare pipes, were installed around the perimeter of the barrette; three on each long side and one on each short side. The arrangement of the shaft grouting pipes around the barrette is shown on Fig. 4.

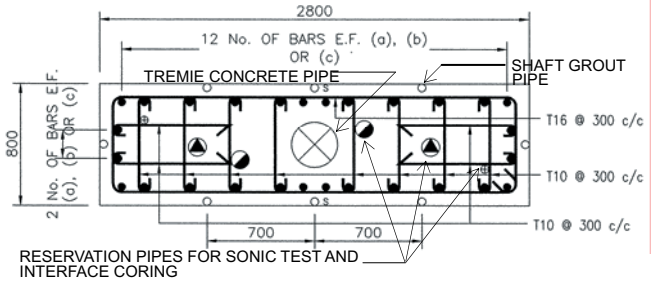

Fig. 4. Typical grout pipe arrangement of shaft-grouted barrette.

3.3 *Pile capacity*

The compressive capacity of the shaft-grouted barrette was derived mainly from the shaft friction in Alluvium, CDG, CDR and DD enhanced by shaft grouting, combined with a minor contribution from the end bearing resistance. The shaft frictions contributed by fill and marine deposit were conservatively ignored.

The shaft capacity for each barrette was estimated with direct correlation with SPT 'N' values, which were based on the averaged values from the adjacent boreholes for each soil type as given in Eq. (1).

$$\tau = f_s \times N \qquad (1)$$

where τ = ultimate skin friction (kPa), f_s = correlation factor with SPT N-value and N = mean blow count of SPT.

The ultimate skin friction was limited to 200kPa for alluvium based on previous experience elsewhere and 140kPa for saprolites with reference to limited data from instrumented piles elsewhere in Hong Kong. The adopted design parameters are summarized in Table 1. The ultimate end bearing resistance was taken to be 10x'N' (kPa) in accordance with Terzaghi & Peck (1967) and is considered to be conservative according to Plumbridge *et al*. (2000). A global factor of safety of 2 was applied to the sum of the ultimate shaft and base resistance to derive the allowable pile capacity.

Instead of conventional trial piles, two working barrettes TB1 and TB2 were selected to be load tested at early stage of the project to demonstrate that the barrettes had attained the required factor of safety as well as to verify the correlation factors with SPT N-values in various strata.

Table 1. Summary of design parameters.

Soil Stratum	Correlation factor with SPT N-value, f_s	Limit of ultimate shaft friction, τ (kPa)
Alluvium	5.0	200
Decomposed soils	2.4	140

4 BARRETTE CONSTRUCTION

4.1 *Barrette excavation & concreting*

The excavation of barrette was primarily carried out by hydrofraise, which breaks up the soil with two counter-rotating cutter drums mounted on a guide frame as illustrated in Fig. 5. It contains a built-in inclinometer which enables the real time monitoring of the verticality during excavation and the guide frame is able to be fine adjusted to control the verticality.

Fig. 5. Hydrofraise for barrette excavation.

To maintain the trench during excavation, the trench was topped up by bentonite slurry. The spoil suspended in the slurry was pumped to a desanding unit via the pump mounted above the cutter drums. Following desanding, the bentonite was recycled into the trench via a feeding pipe at the top of the trench. The properties of the bentonite were regularly checked to ensure that they complied with the requirements. The soft material that had accumulated at the base of the trench was removed using the mechanical grab prior to measurement of the final toe level.

Following the installation of the reinforcement cage, a tremie pipe was lowered down at the centre of the barrette and tremie concrete was then carried out. The bentonite slurry displaced by wet concrete was then pumped off for desanding before returned to the storage reservoirs.

4.2 *Water cracking and shaft grouting*

The shaft grouting tubes were flushed within 48 hours after concreting and the manchettes were cracked open with water prior to the concrete attaining too high strength. Water was pumped through the packer pipe to the manchette, where pressure building up inside the sealed section caused the manchette to open up. After completion of water cracking of one manchette, the packers were deflated and the packer pipe was raised to the next level.

Cement grout was then injected using a positive displacement pump starting with the lowest manchette in each grout tube. The

required volume of grout was injected as evenly distributed as possible, through a minimum of six manchettes at each level as shown in Fig. 4. It is worth noting that for shaft grouting, the primary criterion is in achieving the target grout volume instead of high grouting pressure as in usual grouting operation. The set up of packer pipes during shaft-grouting is shown in Fig. 6. The grouting operation was real-time monitored and controlled by an electronic system developed by Bachy Soletanche Group.

Fig. 6. Arrangement of packer pipes for shaft-grouting.

5 TRIAL BARRETTES TB1 AND TB2

5.1 *Load kentledge*

The reaction force for the load tests was provided by a 50MN steel kentledge. The kentledge comprised bundles of steel billets, sitting on top of a stacked lattice of steel Universal Sections supported on concrete blocks. The loads were applied to the temporary barrette cap by means of four hydraulic jacks to provide a maximum test load of 45MN. The applied loads were measured by four load cells directly placed above each hydraulic jack.

5.2 *Instrumentation*

Sister bar type vibrating wire strain gauges were installed at 17 levels, with each level consists of 4 gauges, inside the barrette to measure the strain along the barrette shaft under compression. The recorded changes in strain at the gauges can be interpreted to derive the load transfer down the barrette shaft.

The vertical displacement of the barrette head was measured directly using dial gauges and cross-checked by transducers. The dial gauges were attached to stand at two independent reference beams. The plungers of the dial gauges rested on plane glass glued to the top surface at the four corners of the pile cap.

5.3 *Testing procedure & acceptance criteria*

The trial barrettes were load tested to two times the design working capacity in four loading and unloading cycles with maximum test load of each cycle of 22.5MN, 33MN, 40MN and 45MN respectively. A 72-hour hold was carried out at cycles 2 to 4 in order to fulfill the statutory requirement in case higher load capacity cannot be attained. Based on the previous test performance of large diameter reinforced concrete piles in various projects, the following settlement criteria were adopted in this project.

(i) the maximum settlement at the head of the pile during the test cannot exceeds the value in

$$PL/AE + d/50 \quad (2)$$

where P = test load in kN, L = length of the pile in mm, A = cross-section area of the barrette in mm^2, E = Young's Modulus for the barrette in kN/mm^2, d = equivalent diameter for the non-circular barrette pile in mm.

(ii) the residual settlement at the head of the pile shall not exceeds d/50.

5.4 *Test results and interpretation of parameters*

The performance of the barrettes was found satisfactory. The settlement readings at peak test loads and after unloading are summarized in Table 2 and settlement curves are shown in Fig. 7.

Table 2. Settlement of trial barrettes.

Settlement	TB1		TB2	
	Allowable (mm)	Actual (mm)	Allowable (mm)	Actual (mm)
At Peak Test Load	87.6	23.6	92.9	22.0
Residual	33.8	7.0	33.8	6.7

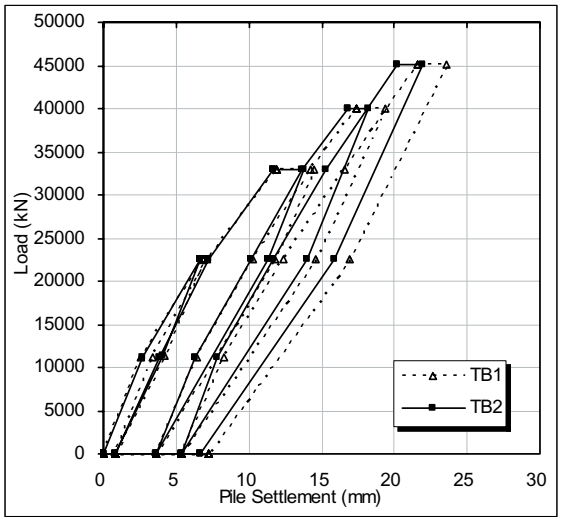

Fig. 7. Plot of load against pile head settlement.

The test results were analyzed to interpret the load distribution along the pile. The strain gauge readings at each level were averaged, and any isolated gauges giving anomalous results were excluded in the analysis. Corrections due to the effect of concrete creeping and reinforcement ratio along the pile were made during the load interpretation. The interpreted load against elevation for TB1 and TB2 are shown in Figs. 8 and 9 respectively. The plots of mobilized skin friction against displacement at centre of individual soil stratum are shown in Fig. 10.

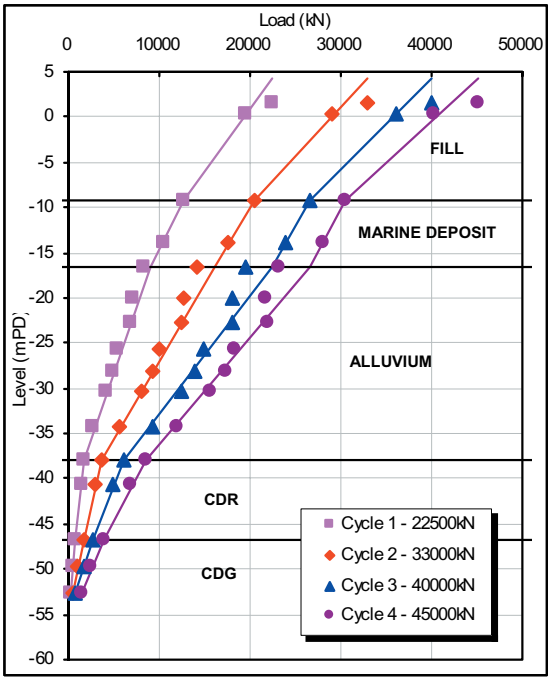

Fig. 8. Load versus level plot for trial barrette TB1.

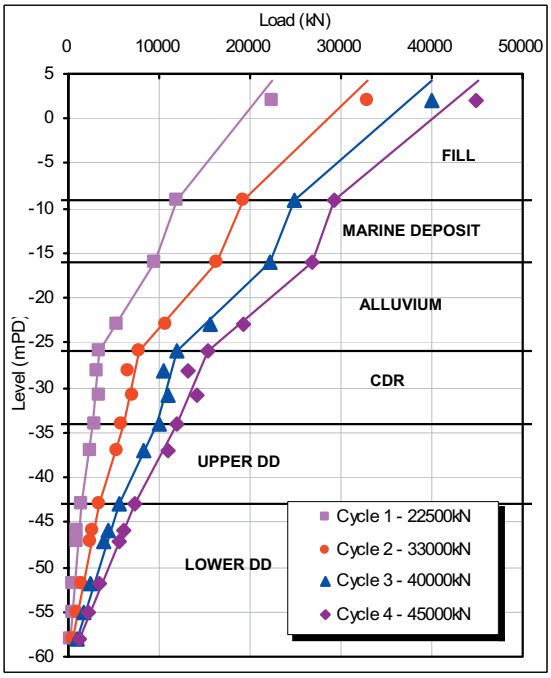

Fig. 9. Load versus level plot for trial barrette TB2.

The shaft friction in fill/marine deposit was conservatively ignored in the pile design. However, during the load test, it is observed that some loads are transferred to these layers and the ultimate shaft friction of the underlying soils cannot be fully mobilised at the peak test load. In order to estimate these ultimate values, extrapolation of the test results was applied

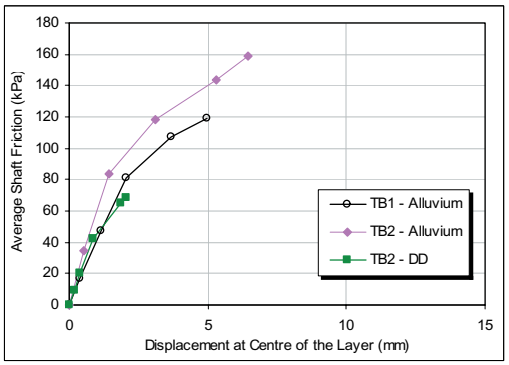

Fig.10. Shaft friction versus displacement plot.

Chin's (1970 and 1971) method was adopted to extrapolate the ultimate frictional capacities of individual soil stratum. This method assumes that the load-movement curve when the load approaches the failure load/ultimate pile capacity is of "hyperbolic" shape. The reason for using Chin's method over other methods such as Brinch-Hansen (1963) is that a linear relationship between pile head settlement/ultimate pile capacity (p/Q) and pile head settlement (p) can be established at early stage of the loading test without reaching or close to the failure load. The extrapolated design parameters for alluvium and diamict deposit and hence finally adopted parameters are presented in Table 3.

Table 3. Summary of finalized design parameters.

Soil Stratum	Correlation factor with SPT N-value, f_s	Limit of ultimate shaft friction, τ (kPa)
Alluvium	3.5	198
Diamict deposit	1.6	111

The results show that the ultimate shaft friction in alluvium is comparable with those proven by pervious projects as summarized in Table 1 except that the correction factor with SPT N-value is slightly lower. This is probably due to the fact that the mean SPT N-value of alluvium in this project is generally high resulting in a lower back-calculated correction factor. Nevertheless, this did not have significant implication to the overall shaft capacity derived from the alluvium layer. The results also suggest that the correction factor and the ultimate friction for diamict deposit, which has no previous data, are generally lower than that of other decomposed soils.

6 DISCUSSION

6.1 *Phasing of construction*

In order not to jeopardize the overall construction programme due to the trial barrette construction and load testing, an observational approach by dividing the barrette construction into two phases was adopted such that the Phase 1 working barrettes and the trial barrettes with load testing could be carried out concurrently. Two layouts were developed, namely Basic and Contingency Schemes, for the Phase 2 working barrettes, one based on the envisaged most promising and the other one based on the worst credible parameters respectively. The final layout was determined based on the trial pile results.

6.2 *Use of mean SPT N-value*

The benefit of using the mean SPT N-value for each soil stratum to derive the shaft frictional capacity is that the error due to individual SPT N-value can be averaged out. However, it is sometimes difficult to determine a reasonable mean SPT N-value within a soil stratum as the distribution of SPT N-value can be quite scattered in alluvium and DD in some of the boreholes with occasionally large SPT N-values, which in turn gives a relatively large mean SPT N-value. In this case, the back-calculated correlation factor with SPT N-value could be underestimated.

6.3 *Redundancy factor*

Conventionally, a redundancy factor is usually applied for driven steel H-piles with due consideration that there are chances that the driven piles can be affected by karst features beneath the pile toe or damaged during driving due to uneven karst surface. As the shaft-grouted friction barrettes were constructed using hydrofraise, the verticality of the pile was under full control and monitored during construction. Besides, most of the barrettes were founded above the marble bedrock, no redundancy factor was applied.

6.4 *Finalized barrette layout*

The design parameters finally adopted were lower than the initial design values adopted for the Basic Scheme but were better than those adopted in the Contingency Scheme. In order to further optimize the design, the layout of the Phase 2 barrettes was revised to minimize the additional barrettes required on top of the Basic Scheme. A total of 13 nos. of barrette was added based on the finalized design parameters. An overview of the site work during the pile cap construction is shown in Fig. 11. A proof load test was also carried out after completion of the working barrettes and the results were also found satisfactory.

Fig. 11. Pile cap construction.

7 CONCLUSION

It is demonstrated that shaft-grouted friction barrette with ultimate pile capacity up to 45MN is a feasible foundation solution for the development located in the complex ground condition of the Designated Area of Northshore Lantau.

Two instrumented working barrettes were constructed and tested, and the performances were found satisfactory in terms of pile head settlement at peak loads and the residual settlements.

From the load test results, it is observed that some loads were transferred to the fill/marine deposit and the ultimate shaft frictional capacity of the underlying soil strata could not be fully mobilized at the peak test loads. In this regard, extrapolation of the test results was carried out using Chin's method to predict the ultimate shaft friction in the soils.

It is concluded that the ultimate shaft friction in alluvium and diamict deposit can be up to 200kPa and 110kPa respectively. A correction factor of 1.6 with SPT N-value for diamict deposit is recommended for preliminary design for the similar piles.

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Development of Immersed Tube Tunnels in Hong Kong

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Maunsell/AECOM

ABSTRACT: Since the Cross Harbour Tunnel in Hong Kong was constructed using the immersed tube method in 1972, four other immersed tunnels have been successfully constructed to cross Victoria Harbour in Hong Kong. With the increase of the transportation demands across the Victoria Harbour to link Hong Kong Island, Kowloon Peninsula and China, more tunnels are planned by making use of the advantages of immersed tubes. This paper calls for the development of design theory and construction technology and summarises the historical development of immersed tunnels in Hong Kong and recent development of the construction techniques around the world.

1 INTRODUCTION

Hong Kong has established extensive immersed tube tunnel experience since the first steel immersed tube tunnel was placed across the Victoria Harbour in Hong Kong in 1972. There is four other vehicles/railway tunnel crossing the Harbour constructed by using immersed tube techniques. The layout plan of the five immersed tube tunnel across the Victoria Harbour is shown in Fig. 1. Except Cross Harbour tunnel and MTR harbour tunnel adopted binocular sections in 1970's, the other 3 harbour crossing tunnels used multi-cell box sections of reinforced or prestressed concrete. Table 1 summarises the details of immersed tube tunnels built in Hong Kong.

Fig. 1. Schematics of Harbour Crossing Tunnels in Hong Kong (1: Cross Harbour Tunnel; 2: Mass Transit Railway Tunnel; 3: Eastern Harbour Crossing Tunnel; 4: Airport Railway Western Immersed Tube Tunnel; 5: Western Harbour Crossing Tunnel).

2 DESCRIPTION OF IMMERSED TUNNEL IN HONG KONG

Each of the five immersed tube tunnels across the Victoria Harbour has its own special features and requirements to connect the access roads on both side of the harbour. The following sections will briefly describe the immersed tunnels in Hong Kong.

2.1 *Cross Harbour Tunnel (Road), 1972*

The Cross Harbour Tunnel was constructed using a single shell steel binocular section of two lanes each way and the total tube length is 1602m. The ballast concrete was placed in the midsection between tubes without side ballast pockets as shown in Figure 2. The continuous steel shell protected with concrete covering provided the waterproof of the tunnel. Tremie concrete method was adopted to join the tunnel units. The foundation of the tunnel was formed by a screeded bedding method.

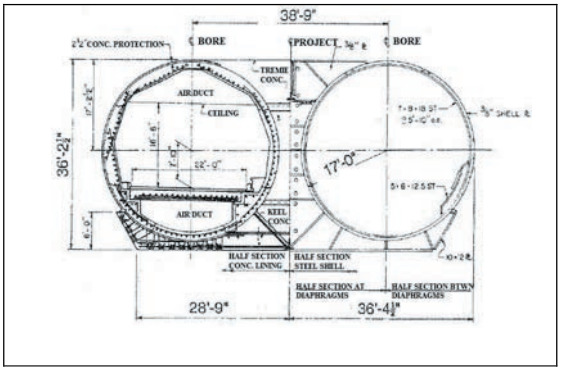

Fig. 2. Cross section of the Cross Harbour Tunnel.

Table 1. Summary of immersed tube tunnels in Hong Kong.

Tunnel	Year of Completion	Tunnel type	Length (m)	Width (m)	Height (m)	No. of Railway Track	No. of traffic lanes	Depth (m)
Cross Harbour Tunnel	1972	Single-shell steel binocular section	1602	22.16	11	-	2 x 2	28
MTR Harbour Crossing Tunnel	1979	Reinforced concrete double binocular section	1400	13.1	6.5	2	-	24.24
Eastern Harbour Crossing Tunnel	1989	Reinforced concrete box	1860	35.45	9.75	2	2 x 2	27
Airport Railway Immersed Tube	1996	Reinforced concrete Box	1260	12.4	7.7	2	-	28
Western Harbour Crossing	1997	Reinforced concrete Box	1360	33.4	8.57	-	2 x 3	25.3

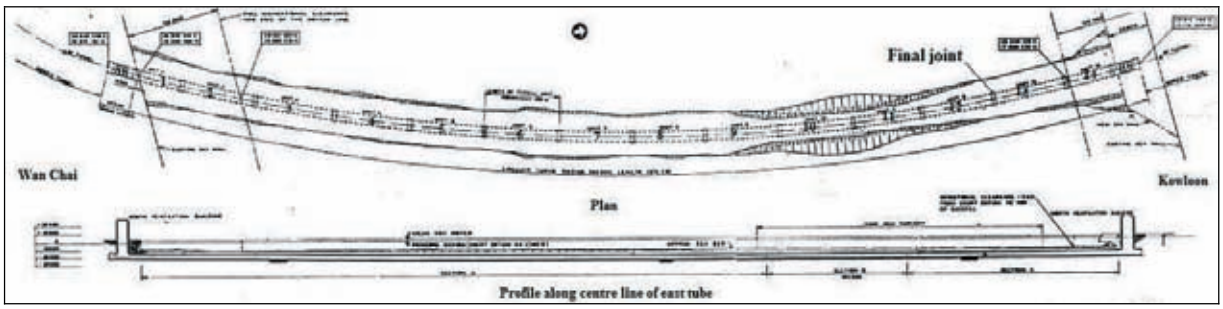

Fig. 3(a). Layout plan and longitudinal section of MTR Harbour Crossing Tunnel.

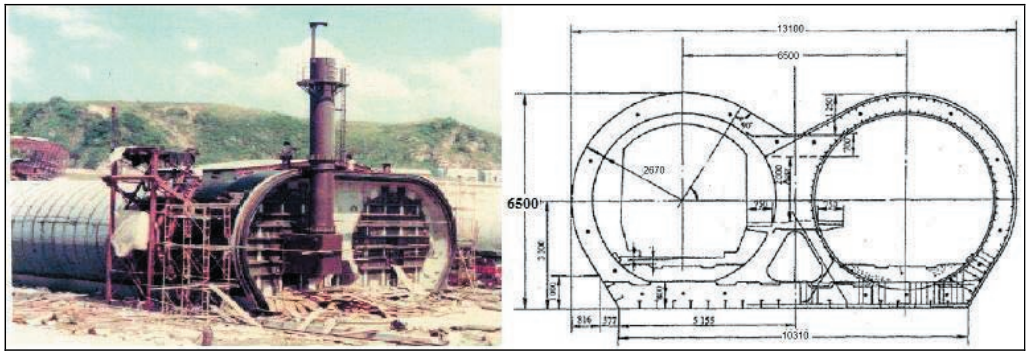

Fig. 3(b). Cross section of the Mass Transit Railway Tunnel.

2.2 *Mass Transit Railway Tunnel (Railway), 1979*

This is the second immersed tunnel in Hong Kong. It extended the Tsuen Wan MTR Line across the Victoria Harbour to interchange with the Island Line. A binocular section was constructed using concrete reinforced with longitudinal prestressing (Figs. 3a & 3b).

The total length of the tunnel is 1,400m comprising 14 units each 100m long. Twelve of the units are of standard design with the two end units specially designed to support the ventilation building and to facilitate the connection with bored tunnels on land. A flexible joint is provided below the seawall for compensation of differential settlement caused by the weight of the ventilation buildings and loads of backfill behind the seawalls. The tunnel units were sunk in place on a screeded gravel mattress in a dredged trench and subsequently connected with hydrostatic joints and locked into position relative to each other by means of a reinforced concrete shear ring in place in a recess between the units. The final closure joint between unit No. 12 and No. 13 was achieved underwater by a tremie concrete method (Hansen, 1979).

For provision of space for the shield for the bored tunnels, the end units were constructed with enlargement of a short section at the shore end. The bored tunnels were driven under compressed air pressure on both shores. The enlarged end sections of the tubes are equipped with a second temporary bulkhead to form an airlock compartment inside the end bulkhead of the tunnel unit in which air pressure could be equalized with the compressed air pressure in the tunnel drive and thereby create conditions under which the shield machine could safely be moved through the end bulkhead and facilitate completion of the junction with the immersed tunnel under compressed air pressure (Hansen, 1979). This became the first and only case up to now of driving the tunnel shield into immersed tunnels.

2.3 *Eastern Harbour Crossing Tunnel (Combined road and railway), 1989*

Eastern Harbour Crossing tunnel is a combined road and MTR rail link under Victoria Harbour between Quarry Bay in Hong Kong Island and Cha Kwo Ling in Kowloon. The road part of the tunnel connecting Island Eastern Corridor and Kwun Tong Bypass is a dual two lane tunnel similar in size to the Cross Harbour Tunnel. The rail part connects Quarry Bay and Yau Tong stations of the MTR Tseung Kwan O Line.

The total length of the immersed tube tunnel is 1,860m which consists of 15 precast reinforced concrete units, 35.45m wide, 9.75m high (Fig. 4) and with lengths up to 128m. Each unit contains five separate conduits in which two for the road, one for the road ventilation and the electrical and mechanical systems, and the remaining two for the railway. The units were cast in three batches of five in a dry dock of 5.5ha at Cha Kwo Ling (Matsumoto *et al*., 1989).

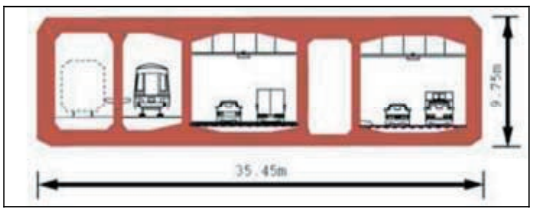

Fig. 4. Cross section of Eastern Harbour Crossing tunnel.

2.4 *Airport Railway Western Immersed Tube Tunnel (Railway), 1996*

Being located close to the Western Harbour Crossing Tunnel which was constructed concurrently, the Airport Railway Western Immersed Tube tunnel carries the new Airport Railway link under Victoria Harbour, which connects Central Station on Hong Kong Island, via Kowloon to the new airport at Chek Lap Kok. It has a total length of 1260m between landfalls and, because of the constrained alignment at the Hong Kong end, is curved in plan through most of its length to a radius of 850m. There are ten tunnel units of twin-cell section of prestressed concrete, each being 126m long by 12.4 m wide by 7.7m high (Fig. 5). The units were fabricated in three batches of 4, 2 and 4 units respectively, in the Shek O casting basin (Morris, 1997; Budge-Reid *et al.*, 1997; Enterkin *et al*., 1997).

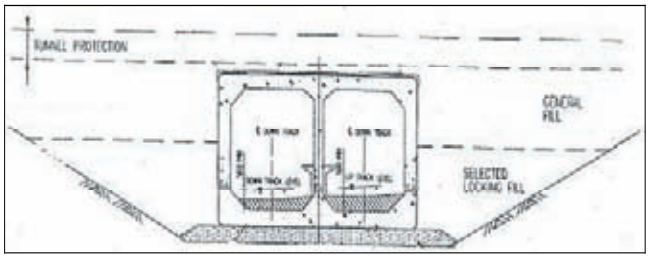

Fig. 5. Cross section of Airport Railway Western Immersed Tube Tunnel.

2.5 *Western Harbour Crossing Tunnel (Road), 1997*

The tunnel forms a straight alignment between the two ventilation buildings at West Kowloon Pennisula and Sai Ying Pun on each side of the Harbour.

The immersed tube section of 1360m was constructed as twelve tunnel units, each 113.5m long, 33.4m wide and 8.57m high (Fig. 6). The tunnel cross section is made up of four cells which accommodate dual three lane carriageways, carried in separate ducts, with two separate ventilation ducts on the outer sides which also carry the main tunnel services and control cabling. All units were cast in three batches in a dry-dock casting basin at Shek O on the southeast coast of Hong Kong Island. Four units were cast at a time and once completed were floated out to a temporary anchorage at Tseung Kwan O to be fitted for sinking. This procedure allowed the casting basin to be pumped out and made ready for the construction of the next batch of four units. Each batch required about 7-8 months to complete (Morris, 1997; Ogura *et al.*, 1997).

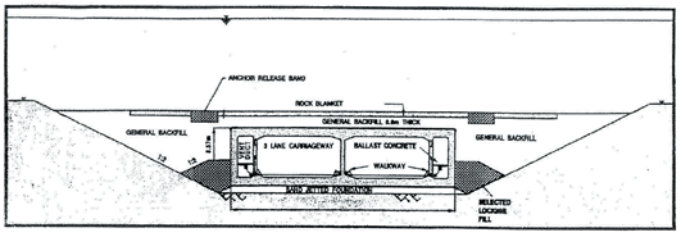

Fig. 6. Cross section of Western Harbour Crossing Tunnel.

3 DEVELOPMENT OF IMMERSED TUNNEL TECHNIQUES

Throughout 30 years of experiences in design and construction of immersed tube tunnels in Hong Kong, some special techniques have been developed both inside Hong Kong, China and other parts of the world. The following sections will summarize the development of immersed tunnel techniques in construction material and casting method, casting yard, tunnel joints and waterproofing aspects.

3.1 *Construction material and casting method*

Steel Immersed Tube Tunnel

The structure of steel shell consists of relatively thin-walled composite steel and concrete rings. The steel shell provides the water barrier. The ballast concrete is placed outside the shell in pockets formed between the structural diaphragms. The benefits of using the steel shell are that the watertightness is provided purely by the steel shell itself. Besides, it can be fabricated in a ship yard on land. Due to its light weight compared with reinforced concrete units, it can be lifted and launched sideways or longitudinally to the water. By virtue of the inherent ductility of the steel shell, they have a larger longitudinal strain capacity, and therefore less sensitive to foundation discontinuities and temperature deformation than concrete tunnels (Ahmet, 1997).

Concrete Immersed Tube Tunnel

The concrete immersed tube tunnel is most often used for double and multiple tube with rectangular shape. The box shape is best fit for the rectangular internal clearance required for motor traffic, with good conformity between resistance and weight. The box shape also permits practical concrete construction practice (Ahmet, 1997).

Traditional casting method was carried out by using movable steel shutter as formwork. The base slab was poured first and then vertical walls and last roof slab. All the immersed tube tunnel built in Hong Kong were using this method of casting inside the dry dock. The problem of this traditional casting sequence is the development of cracks due to heat of hydration generated from different stages of concrete casting. This occurs when casting walls on a base slab that was cast at an earlier stage. The heat of hydration causes substantial heating of the member. After some time the member will cool off to the ambient temperature. The result of cooling contraction of the wall connected to the rigid base slab will cause compression in the base slab and longitudinal tensile strain in the bottom part of the walls. To reduce heat of hydration, concrete with relatively low cement content and cooling pipes installed in the concrete adjacent to the construction joint were employed (Ahmet, 1997). These applications have been used successfully in Hong Kong immersed tube tunnel projects.

New Technique of Full Face Casting

The traditional casting method will cause risk of through-going cracks when casting fresh concrete against old concrete. With the high water pressure this should be avoid. In view of this problem, new technique of full face casting was first introduced in Oresund Link tunnel between Denmark and Sweden and this method was also employed for the on-going project of Busan-Geoje Fixed Link in South Korea.

In Busan-Geoje Fixed Link, the tunnel elements are constructed with segmental casting to avoid through-going cracks.

Fig. 7. Movable casting formwork in casting basin.

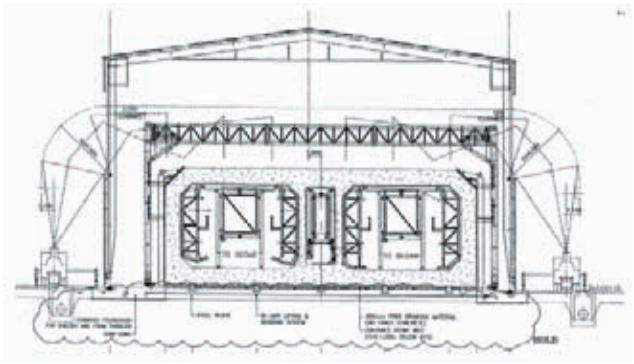

Fig. 8. Movable formwork used for full face casting.

22.5m long segment was fully cast without stops. One segment contains about 2,200m^3 of concrete. The sequence of casting employed for Busan-Geoje Fixed Link is: first the centre of the bottom slab followed by the casting of the bottom slab under the gallery and the corners under the outer walls. Hereafter follows casting of inner and outer walls and finally the top slab (Odgaard *et al*., 2006). Movable casting formwork as shown in Figs. 7 and 8 along the element length allows full section casting at various locations.

3.2 *Casting yard*

Generally immersed tube tunnel units are cast in a casting basin, when the construction of immersed tunnel units completed the casting basin is flooded with water. The tunnel units then float up and towed to the tunnel site. The formation of the casting basin involves extensive excavation and lateral support system. However the major problem is to find a suitable location for the construction of casting basin especially in the built-up areas. The previously used casting basin in Hong Kong is summarized in the following Table 2.

The casting basin used in Hong Kong making the advantages of existing quarry which is proved to be cost and time effective. The Shek O quarry is still available and it is ready for use for the next immersed tube tunnel in Hong Kong.

In additional to the traditional casting basin, an innovative construction method of using submersible barges for fabrication tunnel units is introduced in the Luntou-Shengwudao Immersed tunnel project currently under construction in Guangzhou. The 277m long immersed tube tunnel is connected by four tunnel units, each 23m wide and 8.7m high. The tunnel units of maximum 78m long are fabricated in submersible barges anchored at Xiaozhou shipyard located some 10km away from the site (Fig. 9.). The fabricated units will be transported by the submersible barge to the temporary mooring zone for temporary outfitting works and finally be towed to site and immersed for connection underwater (Fig. 10) (Lo et al., 2006). The innovative construction method is firstly used in China. By use of submersible barge for casting the tunnel units, the geotechnical aspects for formation of the casting basin are removed.

Fig. 9. Casting of tunnel unit on submersible barge.

Fig. 10. Towing of a submersible barge with a tunnel unit.

Table 2. Summary of casting basin previously used in Hong Kong.

Tunnel	Location of Casting Basin	Remarks
Cross Harbour Tunnel	Ship yard in Wanchai	Closed
MTR Harbour Crossing Tunnel	Area of reclaimed land in Chai Wan	Closed and become built-up area
Eastern Harbour Crossing Tunnel	Chao Kwo Ling Quarry	Closed
Airport Railway Immersed Tube	Shek O Quarry	Open
Western Harbour Crossing	Shek O Quarry	Open

3.3 *Foundation types*

Generally the requirement of bearing capacity of immersed tube tunnel is not significant as the tunnel is designed as float tunnel. The minimum factor of safety against flotation without consideration of backfill is 1.04. Therefore the soils under the tunnel units only suffer 4% of up-thrust force.

The major factor governing the foundation of immersed tube tunnel is the quality of sand foundation between the excavated trench and the tunnel bottom. Generally sand placing systems tend to be adopted to suit the conditions encountered at the site. The types of method used for immersed tube tunnel in Hong Kong are summarized in Table 3.

Some techniques have been used in China to improve the poor ground condition under the immersed tube tunnel. The Yongjiang tunnel and Changhong tunnel in Nibo, China constructed in 1995 and 2001 respectively have encountered poor ground condition. The riverbed consists of fluid-plastic gray silty clay. Besides, the siltation rate is very high. In Yongjinag Tunnel, a special silt removal device was developed for the trench dredging and the tunnel foundation was made up of a gravel bedding of 80-100cm thick. A cement grout foundation of about 40-50cm thick was formed by using pressurized injection method to fill the gap between the tunnel base and the bedding layer. While in the Changhong Tunnel, pile foundation was selected to support the tunnel units. The gap between the unit base and piles were filled using a grouting bag method.

For the Busan – Geoje Link Tunnel, Deep Cement Mixing (DCM) piles were employed to improve the underlying Marine clay deposit, which is the dominant type of founding soil along the tunnel alignment. By this method, cement is injected directly into the clay and in situ round columns of a clay/cement mixtures are made. The diameter of the columns depends on the equipment used. For offshore works normally 4 columns are made at the same time forming a square of 1.8m by 1.8m. This soil improvement method removes the risk of the subsoil settlement. Besides, DCM piles also reduce the difference in subsoil stiffness at the locations where the tunnel alignment changes from marine clay into the outcrops of bedrock at the both end of the tunnel alignment (Jensen *et al.*, 2006).

Table 3. Summary of foundation method for immersed tube tunnel in Hong Kong.

Tunnel	Foundation Type and Method
Cross Harbour Tunnel	Screeded bedding
MTR Harbour Crossing Tunnel	Screeded bedding, using jack-up rig. Conveyor belts take the stones to four vertical telescopic pipes through which they are fed into a horizontal steel box with an open bottom at the required level. The level of the horizontal box is controlled very accurately from a traveling gantry at the deck level of jack-up rig. From two pipe-beams fixed to the jack-up legs at the bottom of the dredged trench it is possible to exert horizontal forces dredging the screed box and by feeding stone into the box and keeping it full at all time to lay a well compacted and even mattress.
Eastern Harbour Crossing Tunnel	Sand jetting method. The method developed involved the pumping of sand water mixture through nozzles which moved transversely along the underside of the unit as the sand, jetted horizontally, packed progressively into the void between the unit and the trench bottom. Once a strip had been placed successfully across the full width then the system was moved forward along the axis of the tunnel unit.
Airport Railway Immersed Tube	Sand flowing method. A sand barge was positioned above the unit and flexible delivery pipes were connected onto the sand pipes cast into the walls of the unit. A sand/water mixture was then pumped under the unit using two adjacent pipes simultaneously. Echo sounders fixed in predetermined positions on the side of the unit and divers were used to confirm the sand had filled the gap under the unit. Each sand pipes was then grouted with non-shrink grout.
Western Harbour Crossing	Sand flowing method. Same technique adopted in the Airport Railway Immersed Tube Tunnel.

Table 4. Construction joints techniques.

Area	Construction Joints	Remarks
Hong Kong	A 200mm wide plain web section PVC waterbar was placed centrally in every construction joint. The laitance on each joint was completely removed to expose the coarse aggregate by green cutting with a high pressure water jet. Longitudinal reinforcements are continuously provided across the joints. Segments are cast adjacently.	The stiff monolithic tunnel unit is suitable for resisting seismic loading and control of displacement. However full-depth transverse concrete cracks can develop across the construction joints. In order to resist the shrinkage strain during concrete casting at different stages, sufficient reinforcement is provided. Besides, external waterproofing membrane is also provided.
China	A gap of about 800mm wide is left between adjacent segments during casting. Once the concrete of adjacent segments have gain sufficient strength and shrinkage, the laitance on each joint was completely removed to expose the coarse aggregate by green cutting with a high pressure water jet. Longitudinal reinforcements are continuously provided across the joints. Finally cast the gap with concrete.	The stiff monolithic tunnel unit is suitable for resisting seismic loading and control of displacement. Since a gap between adjacent segments is left to provide an additional degree of freedom for the segment to shrink. Therefore full-depth transverse concrete cracks are significant reduced. However the quality of joints depends largely on the workmanship and longer time is required for casting the full unit. External waterproofing membrane is usually provided.
Denmark	The vertical joint between two segments is basically an unreinforced cold joint provided with a cast-in flexible watrestop. A concrete shear key is provide to transfer the shear. Temporary longitudinal prestressing tendons over the full length of the tunnel elements are provided during transportation and installation.	In this way, the tunnel element can be subjected to flexural deformations without developing longitudinal tensile strain between the construction joints. However special care should be paid for immersed tube tunnel in earthquake zone.

3.4 *Tunnel joints*

Tunnel joints refer to construction joints for each immersed tube units, flexible joints between units and final closure joints. The development of tunnel joints will be described in the following sections.

Construction Joints

The length of immersed tunnel units is normally 80m-150m long, tunnel units are cast in the casting basin with segments of about 20m. Depending on the different construction practice in different area, there are several methods in dealing with the construct ion joints and the details are shown in Table 4.

Joints between units

The tunnel elements cast in the casting yard are provided with temporary bulkheads at both ends to ensure that the element is watertight and capable of floating. On one end of each tunnel element, an endless Gina gasket is mounted. When manufacturing of the tunnel elements is completed, the dock is flooded and the element floated. Each element is towed to its final position and then immersed. The immersed tunnel element is then pulled firmly up against the preceding immersed element with hydraulic jacks. The initial contact of the Gina should be accomplished using a low pulling force. When the Gina has full contact around the total circumference of the adjacent element, the water between the bulkheads is pumped out. Due to pressure difference between the bulkheads and the hydrostatic pressure on the outside of the tunnel, the Gina profile compressed and seals the joint. The Omega seal is then clamped across the joint on the inside of the tunnel (Trelleborge). This jointing method and specification have been used successfully and proved to be reliable for design life of 120 years.

For design of immersed tube tunnel in earthquake-prone areas, joint connection cables are installed across the flexible joints. The details adopted in Japan are shown in Fig. 11 (Inokuchi et al., 1994).

The joint consists of the following components:

1. Rubber gasket: holds back water at the joints and also serves as a spring in the longitudinal direction;
2. Joint connection cable: in conjunction with the rubber gasket, forms a composite spring resisting separation of joints.
3. Horizontal and vertical shear keys: prevent horizontal and vertical dislocation of the elements, and serve as spring by pinching the buffer rubber at points of engagement;
4. Secondary water stopping stopper rubber: gives enhanced safety against water inflows;
5. Rubber gasket stopper: steel component which keeps the compressive deformation of the rubber gasket at the joint within allowable limits;
6. Terminal steel shell: part of the element structure which adds reinforcement at locations such as where rubber gaskets are installed, and also maintains the flatness of the end surface.

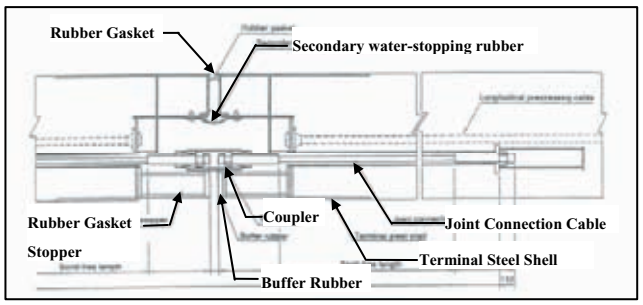

Fig. 11. Structure of flexible joint used in Japan.

Table 5. Summary of final closure joints in Hong Kong.

Tunnel	Final Closure Joint
Cross Harbour Tunnel	The final closure joint was carried out by a thimble unit under water
MTR Harbour Crossing Tunnel	The final closure joint was carried out between unit no.12 and no.13 under water. The final closure joint was achieved by placing an inner form fitting loosely in the gap between the units and an outer form enclosing the two ends of the units and thereafter filling the remaining space with concrete placed under water. Dewatering the final joint and removing the water pressure on the two ends the rubber at the opposite ends expand slightly and put the final joint in compression without losing more than a small fraction of the compression at the other ends.
Eastern Harbour Crossing Tunnel	The final closure joint was carried out between unit 15 and cut-and-cover tunnel in Chao Kwo Ling side. Tremie concrete infill was carried out between the base slab and rock shelf thrust block. Tremie concrete abutment was cast between the tunnel unit and the temporary wall before water was pumped out of the channel for cut and cover tunnel construction.
Airport Railway Immersed Tube	The final closure joint was within unit 9 with unit 8 carrying the precompressed flexible gasket joint and a short section of unit 9. The underside of each unit and 2m of the lower external walls were protected by a 9mm thick steel plate attached to the concrete by shear studs. After all units had been immersed, there was a residual gap of 2.5m left between the secondary end of unit 9 and the end of unit 8. Drivers placed struts in the residual gap between the free ends of the units. Once unit 8 and 9 were resting permanent sand foundation steel damplate closure panels with sealing gaskets were placed around the outside of the gap. The water inside the gap was pumped out and the bulkheads to unit 8 and 9 removed. Reinforcing steel was extended across the gap from cast-in couplers. Concreting was carried out in several stages to allow the temporary struts to be removed. After the roof had been concreted, grouting was carried out between the top of concrete and the steel closure panels.
Western Harbour Crossing	The final closure joint was within unit 11 and unit 12 with the same method of the Airport Railway Immersed Tube.

Closure Joints

Depending on the construction sequence of the unit immersion, the final closure joints can be carried out between immersed tube tunnel units under water or between tunnel unit and cut-and-cover tunnel on land. In contrast to the joints between units, different final close joints have been used in Hong Kong and these are summarized in Table 5.

3.5 *Water proofing membrane*

In order to prevent the water seep into the tunnel through the cracks that may develop, external waterproofing membrane is a prerequisite and plays an important part. The external waterproofing membrane can also protect the concrete against aggressive qualities of the surrounding environment such as seawater (Ahmet, 1997). From the experience of the Hong Kong immersed tube tunnel projects, different types of external waterproofing membranes are used and are summarized in Table 6.

Table 6. Summary of external waterproofing membranes used in Hong Kong.

Tunnel	External Waterproofing Membrane
Cross Harbour Tunnel	No used as the steel shell structure is watertight.
MTR Harbour Crossing Tunnel	6mm thick steel plate treated with corrosion protection provided at the tunnel bottom. The side wall and roof of tunnel was waterproofed with two-ply bituminous membrane.
Eastern Harbour Crossing Tunnel	6mm thick mid steel plate for the underside of the base of the units. The wall and roof were covered by 2mm thick epoxy-tar membrane.
Airport Railway Immersed Tube	The base was protected by a 9mm thick steel plate attached by headed studs. 2mm thick acrylic resin based waterproof membrane was applied to the walls and roof of the tunnel unit.
Western Harbour Crossing	The underside of each unit has a 1.5mm thick PVC membrane which was laid prior to steel fixing and is mechanically attached by PVC ribs on the internal face of the membrane which key into the concrete structure. On the upper walls and roof of each unit a minimum 2mm thick flexible waterproofing, acrylic resin based membrane was applied in 2 layers using airless spray techniques.

For the Dutch design philosophy, the expansion joint concept is used to avoid the development of transverse cracking of the concrete. Tunnel element of about 100m long is designed with expansion joints in the range of 20m apart. The expansion joint between two segments is unreinforced with a cast-in flexible waterstop. Besides, an external sealant or waterproofing membrane across the immediate vicinity of the joint is provided (Ahmet, 1997). This together with the full-face casting technique of the tunnel segment can ensure the tunnel units to be self-watertight and crack free. In this case, no external waterproofing membrane is provided. Special attention should be paid on the earthquake design, concrete mix design, production quality and workmanship.

4 PROJECTS AHEAD AND LIKELY FUTURE DEVELOPMENT IN CHINA

To meet the economic development of the Chinese 11th five-year plan, the transportation infrastructure needs continuous expansion and improvement across whole country. The tunnel options including immersed tubes would be widely adopted to avoid large-scale relocation of the existing facilities and to protect the scenery of the river banks. The projects ahead and likely future development in China will be briefly described below:

Tunnel Crossing Pearl River, Guangzhou

The first immersed tube tunnel in Mainland China was built in 1993 to cross the Pearl River. Recently in Guangzhou, more immersed tunnels have been under study, design or construction including three tunnels to connect a newly developed University City surrounding by rivers (Fig. 12). They are

- Zhujiang river tunnel (1993)
- Luntou tunnel (under construction)
- Bioisland tunnel (Under design)
- Zhoutouzui tunnel (Under design)
- Yuzhu tunnel (Under design)

Fig. 12. Immersed tunnels in Guangzhou.

Macau – Taipa Tunnel and Access Routes

The government of the Macau Special Administrative Region (MSAR) proposes to build a sub-sea tunnel between Macau and Taipa to cope with the significant growth of traffic and improve the accessibility of Taipa. The proposed tunnel will connect the Sun Yat-Sen Road on the Macau side and across on a new reclamation area aligned with the 1st Macau Taipa Bridge Nobre de Carvalho (MTBNC) on the Taipa side.

Hong Kong – Zhuhai – Macau Link Tunnel

The current proposal of an immersed tunnel as part of the 30 km Hong Kong – Zhuhai – Macau Bridge Project. This tunnel scheme would certainly challenge the technology in light of 5.5km long, 45m deep with dimension of 38m wide and 10.25m high immersed tube tunnel.

Hong Kong – South East Kowloon Development

The proposed South Kowloon Development involves the construction of approximately 2.5km long immersed tube tunnel connecting the existing roads to the proposed development. Fig. 13 shows the proposed alignment of the immersed tube tunnel.

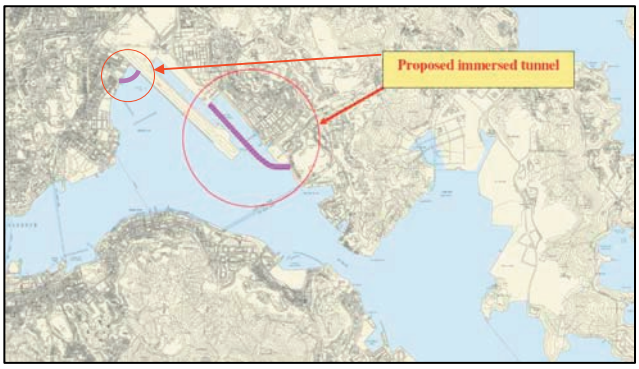

Fig. 13. Proposed immersed tube tunnels in South East Kowloon development.

5 CONCLUSION

Five immersed tube tunnels have been successfully constructed in Hong Kong from 1972 to 1997. With the consolidation of experience on developed immersed tunnel techniques from Hong Kong, China and other part of the world, it will be beneficial to further development of immersed tunnel engineering on long and deep tunnels with high quality and fast-track construction.

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