ENHANCING THE RELIABILITY AND ROBUSTNESS OF ENGINEERED SLOPES

GEO REPORT No. 139

K.K.S. Ho, H.W. Sun & T.H.H. Hui

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PREFACE

In keeping with our policy of releasing information which may be of general interest to the geotechnical profession and the public, we make available selected internal reports in a series of publications termed the GEO Report series. The GEO Reports can be downloaded from the website of the Civil Engineering Department (http://www.ced.gov.hk) on the Internet. Printed copies are also available for some GEO Reports. For printed copies, a charge is made to cover the cost of printing.

The Geotechnical Engineering Office also publishes guidance documents as GEO Publications. These publications and the printed GEO Reports may be obtained from the Government's Information Services Department. Information on how to purchase these documents is given on the last page of this report.

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December 2003

FOREWORD

The systematic landslide investigation programme which commenced in 1997 has highlighted, inter alia, the need to further improve the slope safety system and enhance the slope engineering practice in order to reduce the failure rate of engineered slopes.

This report consolidates the key lessons learnt from studies of failures on engineered slopes, together with the findings of a preliminary review of the prevailing practice in respect of investigation and design of man-made slopes in Hong Kong. Recommendations on good practice are made with a view to further enhancing the reliability and robustness of engineered slopes. Areas that warrant further work are suggested.

The draft report was reviewed by Mr Y.C. Chan.

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ABSTRACT

One of the key findings of the systematic landslide investigation programme is that there is room for improvement in further reducing the failure rate of engineered slopes (i.e. slopes with geotechnical engineering input and geotechnical submissions processed by the slope safety system as conforming to the required design standards).

This report documents the key lessons learnt from studies of failures on engineered slopes and observations made from a preliminary review of the prevailing slope engineering practice in respect of man-made slopes in Hong Kong. Areas deserving attention in order to enhance the reliability and robustness of engineered slopes are identified and suggestions are made to improve the slope engineering practice.

CONTENTS

1. INTRODUCTION

Soils derived from tropically weathered rocks are typically heterogeneous and there are inherent variability and uncertainties in respect of the geological and hydrogeological conditions. Experience has shown that slope stability problems in tropical soils are usually not directly amenable to the principles of 'classical' soil mechanics, which must be combined with appropriate elements of geology, geomorphology and hydrology in far greater measure than is usually necessary for dealing with similar problems in sedimentary soils. Geotechnical engineering in residual soils therefore spans the narrowly separated fields of soil mechanics, rock mechanics and engineering geology, and the engineering geological approach is generally the most satisfactory one for these materials (Brand, 1985). Slope engineering in the weathered profiles in Hong Kong therefore calls for specialist geotechnical input and sound professional judgement based on experience and adequate ground investigation and testing, as well as attention to good detailing that accounts for the variability of the ground conditions.

The systematic landslide investigation (LI) programme was initiated by the Geotechnical Engineering Office (GEO) in 1997. The findings of landslide investigations have contributed to achieving an improved understanding of the mechanisms and causes of slope failures, as well as providing insight into the range of key factors causing failures of engineered slopes. One of the key findings of landslide investigations is that there is room for further improvement in reducing the failure rate of engineered slopes (i.e. slopes with geotechnical engineering input and submissions processed by the slope safety system as conforming to the required safety standards).

This report documents the key lessons learnt from studies of engineered slope failures together with observations made from a review of the practice for the investigation and design of man-made slopes. The principal factors involved in minor and major landslides are highlighted. Suggestions on improved slope engineering practice are made in order to further enhance the reliability and robustness of engineered slopes.

For the purposes of this report, reliability corresponds to the chance of satisfactory performance (i.e. one minus chance of failure). A more reliable slope design means a smaller chance of failure. Robustness reflects the sensitivity of a slope design to undetected or unforeseen adverse conditions (e.g. high transient groundwater condition, presence of weak clay seams, etc.). A more robust slope design means that the slope will be less vulnerable to undetected adverse conditions and hence has a smaller chance of failure.

2. FINDINGS OF LANDSLIDE INVESTIGATIONS

The common triggering events and notable generic factors leading to slope failures in Hong Kong are summarised in Tables 1 and 2 respectively.

The majority of the landslides in Hong Kong are shallow (<3 m deep) and of a small scale. On average, about 90% of the failures are less than 50 m^3 in volume and about 50% are less than 10 m^3 in volume. The findings of landslide investigations carried out by the GEO prior to the introduction of the systematic landslide investigation programme in 1997 are documented in selected GEO Reports and internal reports such as Special Project Reports

(Wong & Ho, 2000).

Between 1997 and 2001, more than 1,700 genuine landslide incidents were examined under the landslide investigation programme and 166 cases were selected for follow-up studies. A total of 96 landslides that occurred between 1997 and 2001 involved slopes with past engineering input and geotechnical submissions processed by the slope safety system (i.e. engineered slopes). Of these 96 landslides, 24 were major failures and 72 were minor failures, where major failure denotes a landslide of ≥50 m^3 in volume and a minor failure denotes a landslide of <50 m^3 in volume. These failures were studied to a sufficient detail to enable a meaningful diagnosis of the probable causes of failures. The findings of the annual diagnostic reviews of the landslide data are summarised by Wong & Ho (1999), Ho et al (2000 & 2001) and Lam et al (2002). The findings of landslide studies on the individual incidents are documented in a series of Landslide Study Reports (LSRs) which are lodged in the Civil Engineering Library. Summaries of significant landslides can be found in the Hong Kong Slope Safety Website. Apart from the above, a preliminary review of 13 major failures that occurred between 1993 and 1995 involving slopes with past engineering input and GEO's involvement was also carried out by the Special Projects Division.

A selection of major landslides on engineered slopes is illustrated in Plates 1 to 10.

Based on the landslide studies, the main contributory factors to failure of engineered slopes include the following:

(a) poor maintenance condition of the slope (e.g. blocked channels resulting in overflow, wedging action of unplanned vegetation, etc.),

(b) adverse geological weaknesses and adverse groundwater conditions not fully appreciated and accounted for in the design,

(c) inappropriate stability analyses that did not consider the critical cross-section or critical slip surface,

(d) inadequate slope detailing against erosion and washout,

(e) inadequate design review during construction,

(f) adverse changes in environmental conditions, and

(g) leakage or bursting of water-carrying services.

Some of the failures of engineered slopes are associated with inadequate workmanship and non-conformance with the specifications. This emphasizes the importance of ensuring adequate supervision of all the critical site activities and surprise audits during construction.

The findings of the systematic landslide investigation programme implemented since 1997 suggest that failures of engineered slopes are not due to inadequate slope safety standards in terms of the stipulated minimum factors of safety. However, the lessons learnt from slope failures highlight that there is room for improvement in the implementation of published guidance on good practice in the investigation, design, construction and maintenance of slopes. There is also a need to promulgate supplementary guidance on emerging good practice to the geotechnical profession. Observations made on the basis of landslide studies are, by nature, somewhat biased because the batch of slopes of concern has already suffered failures. Notwithstanding this, the lessons learnt from landslide investigations can provide an important feedback mechanism to the geotechnical profession to further improve slope engineering practice.

3. CAUSES OF MINOR FAILURES OF ENGINEERED SLOPES

3.1 Introduction

Given the dense urban development in Hong Kong, even minor failures close to buildings or busy roads can potentially result in significant damage or casualties (e.g. rock falls, detachment of soil mass from near the crest of a steep and high cut slope, etc.).

Based on a detailed review of failures of engineered slopes, the minor failures are mainly associated with the following problems:

(a) control of surface water,

(b) poor slope maintenance condition, and

(c) presence of local adverse geological features and adverse groundwater conditions.

These problems are discussed in the following sub-sections.

3.2 Surface Water and Insufficient Maintenance

Surface water plays an important role in triggering many of the landslides on engineered slopes, particularly shallow, small-scale failures. The failure mechanism mainly involves concentrated surface water runoff leading to erosion and/or water ingress during intense rain. This type of failure is exacerbated by an urban setting where roads and other paved surfaces upslope can act as conduits for rainwater runoff and are liable to flood and overwhelm the surface drainage provisions. There are two related root causes, viz. inadequate maintenance of the slope protection measures and drainage provisions, and inadequate attention to detailing of the surface drainage provisions.

Poor slope maintenance condition generally takes the form of blocked or cracked drainage channels, damaged hard surface cover or unplanned vegetation that can jack open joints in the soil or rock mass. Many of the landslides studied involved washout failure due to over-spilling from blocked or damaged crest drains, ingress of surface water into slopes through poorly protected slope surface or build-up of water pressure behind hard surface cover with blocked weepholes. Washout or water ingress leading to build-up of water pressure or internal erosion may also result from leakage or bursting of inadequately maintained water-carrying services.

Inadequate attention to proper detailing may include the use of drainage channels that are undersized or with inadequate fall for self-flushing, adoption of drainage layout and detailing that are prone to over-spilling, etc. Sometimes, surface water flow or ponding of water due to adverse site setting or topography is not adequately addressed in the slope design. For example, slopes below a low point of the crest platform or slopes below a road bend can be subject to concentrated surface water flow during intense rainfall leading to failure.

Recent landslide studies have shown that some failures occurred on slopes that had previously remained stable for a substantial period of time and apparently survived more severe rainstorms in the past. In this respect, these failures are a surprise. These landslides may reflect progressive deterioration of the slope condition which is promoted by inadequate maintenance, or adverse changes in the site setting (e.g. upslope developments altering the surface water catchment characteristics) which may lead to concentration of surface water flow.

3.3 Local Adverse Geological Features and Groundwater Conditions

Many of the smaller scale failures in soil cuts as well as in rock cuts are associated with the presence of local adverse geological features (e.g. weak geological materials with or without adversely orientated relict discontinuities) and adverse groundwater conditions (e.g. build-up of a local transient perched water table). For example, based on the LI data for the period 1997 to 2001, about 60% of the landslides on engineered soil cut slopes for the period 1997 to 2001 were small failures controlled mainly by local, adverse geological and/or groundwater conditions. These types of inherent geological and hydrogeological weaknesses in cut slopes are not easy to detect with the current ground investigation practice. In the case of rock cuts which typically do not have a surface cover in Hong Kong, the slopes can be especially susceptible to localised deterioration (e.g. progressive opening up of rock joints due to growth of tree roots or unplanned vegetation) or significant water ingress leading to the build-up of cleft water pressure. The same problem can occur in soil cuts, particularly where there is a pervasive set of subvertical relict discontinuities which could lead to toppling failure.

With the move towards increased use of vegetated cover for slopes, there could be a corresponding increased likelihood of small failures associated with possible local adverse ground conditions and groundwater conditions as compared to slopes with hard surface cover. It is important that such hazards of local shallow failures be mitigated against as far as possible, particularly in vegetated slopes (e.g. by judicious detailing and drainage provisions).

4. ENHANCED SLOPE ENGINEERING PRACTICE TO REDUCE MINOR FAILURES

The following measures are recommended to improve slope engineering practice for mitigating local, minor failures (Wong & Ho, 1999; Wong, 2001):

(a) Further to conventional slope stability assessment, an appraisal of the possible surface water environs and likely flow paths that may affect the slope should be carried out. This is intended to identify any surface water concentration problems so that suitable provision can be made in the surface drainage design. The identification of surface water flow paths requires the tracking of possible pathways outside the immediate site environs since the source of surface water concentration may be at some distance away from the site. Sufficient redundancy should be allowed for in the design of the surface water drainage provisions, with due regard to the actual site setting and environmental conditions, past performance in surface drainage and the uncertainties involved.

(b) Improved slope surface protection and drainage detailing should be adopted to reduce the potential for local failures and mitigate adverse effects from inadequate maintenance. GEO Report 56 (Wong et al, 1999) gives guidance on prescriptive modules of experience-based, standardised slope surface protection for both hard and vegetated cover and drainage measures. The Highway Slope Manual (GEO, 2000) gives guidance on the design of slope and road drainage and integrated surface water drainage assessment.

(c) The occurrence of minor failures arising from local adverse groundwater regimes and local weak geological materials, which may be exacerbated by progressive slope deterioration, is difficult to guard against confidently in design assessments. A pragmatic approach is to introduce improved protective and mitigation measures as an integral part of the slope design to cater for possible local detachments of failed material. Such measures might include slope protective meshing, use of debris traps, toe barriers or buffer zones, where space permits.

Efforts to promulgate the importance of regular slope maintenance and preventive maintenance works where appropriate should continue, particularly for private slopes. Some suggestions on possible enhancement of slope maintenance practice are given in Section 10.

5. CAUSES OF MAJOR FAILURES OF ENGINEERED SLOPES

5.1 Introduction

While insufficient control of surface water and the presence of local weaknesses can sometimes be contributory factors in sizeable landslides, the adoption of an inadequate geological or hydrogeological model in slope design is the most important factor for major failures on engineered slopes. The main problems are associated with the following:

(a) significant adverse geological features, and

(b) adverse groundwater conditions.

Some of the key factors involved in a number of major failures of engineered slopes include inadequate appreciation of the adverse site setting, inadequate ground investigation (GI) or inappropriate interpretation of the GI results and inadequate engineering geological input during the investigation, design and construction of slopes. In addition, some major failures have also occurred in areas with massive relict instability. These will be further discussed in the following sections.

It should be noted that in the majority of the major failures on engineered slopes, inadequate slope maintenance has played a relatively minor or insignificant role compared with the key factors noted above. This will be further discussed in Section 10.

5.2 Adverse Geological Features

A number of notable large-scale failures were controlled by significant adverse (in terms of low shear strength and unfavourable orientation) geological features (e.g. 1982 Junk Bay Road landslide, 1987 Cho Yiu Estate landslide and 1995 Fei Tsui Road landslide). These include persistent and weak relict discontinuities, such as sheeting joints, kaolin-rich seams, relict failure surfaces with evidence of past movement (e.g. slickensiding) and discontinuities with kaolin/manganese oxide/chloride infill. Adverse geological structures such as persistent clay-infilled discontinuity will also lead to adverse groundwater conditions. If such 'stability-critical' features are not detected and incorporated into the geological model, or where their strength parameters are not adequately characterised, the corresponding slope stability analysis is liable to be inappropriate. This will result in a misleading assessment of the actual safety margin of the slope.

Weak geological features can have a much lower shear strength than the matrix shear strength, with a ϕ' value of 15^o to 25^o and possibly lower. The conventional range of factors of safety of 1.2 to 1.4 as applied to the matrix strength with a ϕ' value of typically 33^o to 38^o will not be adequate to cater for such weak materials if these were not accounted for in the geological model. It is important that all significant geological features that can adversely affect slope stability should be identified and properly allowed for in the geological model and stability assessment.

5.3 Adverse Groundwater Conditions

Another problem associated with large landslides is adverse groundwater conditions that are not detected during the design and construction stages. Although surface water flow is an important factor in many of the shallow landslides in Hong Kong, the recent systematic landslide studies have highlighted the important role of subsurface water in causing landslides, particularly in the case of sizeable failures. This finding is derived from timely field observations which are made shortly after landslide occurrence and consideration of landslide data that have been appropriately classified. Field inspections have indicated that local transient seepage from landslide scars can dissipate very rapidly. Based on this, cognizance should be taken of the fact that seepage from slopes is liable to be missed if prompt and detailed field inspections are not undertaken in a timely manner following heavy rainstorms for slope assessments.

Groundwater regimes may vary through a mixture of mechanisms. For example, infiltration through the local catchment may give rise to relatively shallow groundwater response, whereas the deeper groundwater table is more affected by subsurface groundwater flow arising from infiltration at the uphill catchment.

5.3.1 Shallow Perching Phenomenon

One common cause of slope failure is the build-up of transient shallow perched groundwater. The 1993 Cheung Shan Estate landslide is a notable example. Shallow perching can arise where a thin surface layer of relatively permeable material overlies less permeable ground. A common setting involves shallow colluvium overlying saprolite, or younger colluvium overlying denser, older colluvium. The higher permeability of colluvium is usually the result of a relatively loose structure, especially if the coarse fraction percentage is high. Presence of erosion pipes could further increase the permeability contrast. The geometry of the failure scar due to shallow perching of groundwater is typically confined to the near-surface material of the slope face but it can extend uphill for some distance.

In the study of more than 250 failures of cut slopes along roads and catchwaters caused by the severe rainstorm on 5 November 1993, Wong & Ho (1995) reported that up to about 25% of the landslides (on non-engineered slopes) involved the build-up of a perched water table in the surface mantle of colluvium.

An unfavourable hydrogeological setting giving rise to shallow perching and presence of transient perched groundwater pressure can be difficult to detect in conventional ground investigation. Shallow perching is therefore not always considered in a routine slope stability analysis. Even if the thin colluvium mantle is identified during the design process, it may not be easy to design confidently against such failure given the uncertainties associated with a potentially rapid and large increase in the pore water pressure ratio (r_u) in the thin surface mantle.

5.3.2 Build-up of Groundwater Pressure in the Weathering Profile

Under-prediction of groundwater response has been a key problem in some of the large-scale failures. For example, the significant build-up of groundwater pressure above clay seams or infilled relict discontinuities not previously considered in slope design has contributed to deep-seated landslides (e.g. 1995 Fei Tsui Road landslide and the 1999 Shek Kip Mei landslide).

A notable number of large-scale failures involved significant rise in the deeper groundwater regime within the saprolitic soil profile following prolonged severe rainfall. This may be related to regional groundwater flow which could include concentrated subsurface groundwater flow along old drainage lines or major depressions in Grade III or better rock (i.e. the weathering front). The deeper groundwater flow regime may also be affected by infilled erosion pipes (e.g. 1997 Ching Cheung Road landslide). Groundwater pressure may also build up due to damming of water flow behind less permeable material such as dyke intrusions (e.g. the 1994 landslide at Castle Peak Road Milestone 14½).

In the case of groundwater flow through intensely fractured rock, there is the potential for upward flow of groundwater from the fractured rock to the overlying saprolite under confined conditions. Such conditions can lead to very high groundwater pressure near the base of the saprolite and there are piezometer data that lend support to this postulation (e.g. Jiao & Malone, 2000). Significant depressions in Grade III or better rock are liable to lead to concentrated subsurface water flow which can result in local high groundwater table with a significant storm response. The build-up of high groundwater pressure associated with the above adverse settings may lead to deep-seated failure.

A number of the major failures on engineered slopes occurred some time after cessation of intense rainfall (e.g. by one to two days or more in some instances). Such delayed groundwater response may, in part, be a function of antecedent wetness of the ground and subsurface seepage from a large catchment. Given the complexity and variability of the weathering profile, it is important to note that the subsurface catchment may not necessarily correspond to the surface catchment. This can give rise to surprises in terms of groundwater responses at a given site. Emphasis should be given to taking a more regional view of the site setting and available GI information in the assessment of the hydrogeological characteristics of the slope.

Groundwater flow through a saprolitic soil profile is likely to be highly non-uniform. The action of downward flow through a heterogeneous ground mass with decreasing permeability with depth, together with non-vertical seepage of infiltration down the slope, may result in seepage-induced groundwater pressure (Massey & Pang, 1988).

5.4 Reactivation of Relict Massive Landslide

Landslide studies have shown that sizeable landslides have occurred on slopes which have shown either active evidence of slope instability or geomorphological evidence of relict massive failures (Hart et al, 2001) on former natural terrain. It is possible that the history of failures is associated with some generic adverse geological and hydrogeological factors, or that the past instability has resulted in an over-steepened scarp or disturbance to the ground mass that did not fully detach from the slope. If such unfavourable site setting is not detected during API or ground investigation, the design will probably not cater for the potential adverse effects. For example, flattening the slope near the toe of a relict massive landslide could destabilise the relict instability as far as overall failure is concerned, although the factor of safety against local failure may be improved (e.g. the 1998 Fei Ngo Shan landslide).

Slickensided shear surfaces or shear zones have been observed in the saprolitic soil profile or along the contact between the colluvium and the underlying weathered rock. The slickensides could be a result of the geological process of weathering or a result of past slope instability. Relict slope instability is liable to be reactivated where adverse orientated relict features (particularly where the direction of past movement as reflected by the slickenside is also adverse with respect to the feature) are not recognized during slope design and construction.

5.5 Discussion

Many of the large-scale landslides on unsupported cuts that have been through the slope safety system were associated with undetected adverse geological features and/or under-prediction of adverse groundwater conditions. Similar findings were reported previously. For example, in the investigation of some major slope failures that occurred in 1982, Hencher (1983b) stated that "six of the eight cut slopes that failed had been investigated by drilling in recent years. In five of these cases, important aspects that controlled the failure were missed. In only one case were the true geological conditions recognized but even then the groundwater levels were underestimated considerably. In all cases where piezometric data were available and the groundwater level was known by other means, albeit approximately (e.g. observed seepage), the piezometric data did not reflect peak water pressure at the failure surface. This was principally due to failure to observe rapid transient rises and falls in water levels. A further problem was that many of the piezometers were installed at levels where they could not detect the critical perched water tables which developed". In one of the major failures studied by Hencher (op cit), significant corestones were mistaken as rockhead and hence the geological model assumed for design was not appropriate.

The above observations illustrate the difficulties and uncertainties involved in formulating a representative geological model and assessing appropriate design groundwater conditions, especially in sites with a complex geological and hydrogeological setting.

6. ENHANCED SLOPE ENGINEERING PRACTICE TO REDUCE MAJOR FAILURES

6.1 Early Recognition of Potentially Problematic Sites

Due attention should be paid to identifying potentially problematic sites at a sufficiently early stage so that adequate geotechnical and geological input, commensurate with the complexity of the slope, would be provided. The findings of landslide studies have pointed to a number of indicators of potentially problematic sites as follows:

(a) sites with relict massive failures,

(b) evidence of high groundwater or high level seepage associated with a drainage valley, subsurface drainage concentration (e.g. depression in the weathering profile), dyke or infilled persistent subvertical discontinuities,

(c) planar geological features (such as joints, faults, seams, bedding, foliation and planar soil-rock interfaces), especially where they are dipping out of the slope (e.g. as indicated by eutaxitic foliation in some fine ash tuff), laterally persistent, showing evidence of previous movement, associated with zones of weak materials such as kaolin, and affecting groundwater flow,

(d) evidence of progressive slope deterioration and movement,

(e) slopes with a history of failures,

(f) complex groundwater conditions with a significant storm response or delayed response,

(g) large cuttings in deep weathering profiles, and

(h) sites with heterogeneous bouldery colluvium which is conducive to rapid build-up of transient perched water table.

In addition to the above, particular attention should be given to sites with major water-carrying services or structures (e.g. service reservoirs) at or close to the slope crest.

For potentially problematic sites, the key is to ensure that there will be an appropriate level of geotechnical and engineering geological input during investigation and design as well as for construction reviews. The assessment of whether a given slope is potentially problematic or not should be continually reviewed as more information becomes available during the course of the project.

6.2 Improved Ground Investigation Practice and Enhanced Engineering Geological Input

As adverse geological and hydrogeological features that are not identified in the design geological model can be the principal cause of major failures, improved ground investigation practice with input from competent and suitably experienced contractors and adequate supervision by experienced personnel is called for.

Improved ground investigation practice entails better planning and project-specific input to help define the key questions to be addressed by the ground investigation. Areas that warrant attention include detailed logging by experienced and qualified personnel, flexible and staged investigation (in terms of locations of drillholes, sampling frequency, etc.) to make use of the interim findings in undertaking a suitably focused investigation, use of continuous Mazier sampling where appropriate to better appreciate the fabric characteristics and mass features and improved groundwater monitoring practice (see Section 7.7). Adequate engineering geological input during ground investigation and design verification at the construction stage is pertinent in order to identify possible presence of adverse geological materials and features and adverse groundwater conditions.

6.3 Adoption of Robust Design Solutions

Improved ground investigation practice and engineering geological input may still miss the more subtle adverse geological features given the heterogeneous ground conditions in tropically weathered rocks. Use of robust design solutions that are less sensitive to uncertainties associated with locally adverse geological and hydrogeological conditions is a pragmatic approach to minimise the chance of failure. Examples of robust design solutions include soil nails, reinforced concrete retaining wall together with backfilling (i.e. toe weighting), reinforced fill slopes and buttresses to unstable boulders which are also tied together using rock dowels.

Many of the soil cut slopes in Hong Kong engineered in the late 1970s and the 1980s involved cutting to an 'adequately safe' gradient based on theoretical stability analyses. The slope is unsupported otherwise (i.e. unreinforced) and is more vulnerable to the effect of unforeseen adverse geological and groundwater conditions as compared with the case where there is additional support provided to the slope. The chance of an unsupported cut being affected by unforeseen adverse features would be greater with increasing slope heights given the increased likelihood of encountering such adverse features in a more sizeable slope. Therefore, large cuttings with no reinforcement should be avoided as far as possible.

Since the late 1980s, the trend in Hong Kong has been towards the increasing use of soil nails as opposed to unsupported cuts in upgrading substandard slopes. To date, there have been no records of failure of any permanent soil nailed slopes in Hong Kong. The reinforcing action of soil nails allows stress redistribution where there are local weaknesses. The reinforced ground tends to behave as an integral mass through nail-soil interaction and spatial averaging effects. A soil nailed slope is therefore less sensitive to undetected isolated weak geological features and adverse groundwater regime and hence less likely to fail than an unsupported cut. The reinforcing effect of soil nails will also result in a more ductile failure mode with reduced debris mobility and prior warning of an impending failure through development of slope distress. Such ductile behaviour will also help to reduce the consequence of failure. Thus, the risk of a soil-nailed cut slope failure will be much lower than that for an unsupported cut because of the above factors.

To ensure that the reinforcing effect of soil nails can be mobilised, there should be sufficient confinement at the slope surface (e.g. suitably sized soil nail heads) to prevent disintegration of the ground mass within the active zone. Structural linkage of the soil nail heads by means of beams, grillages or, where necessary, shotcrete will also further enhance the robustness of soil nails because this will contribute to integral action of the reinforced ground mass.

Although a soil nailed slope may have the same calculated factor of safety as an unsupported cut in the same ground conditions based on conventional slope stability analysis, the soil nailed slope will actually be much more reliable (i.e. lower chance of failure) because of its robustness.

Soil nails increase the tolerance against uncertainties and reduce the consequence of failure. They provide a safety margin against local adverse conditions that are difficult to detect in practice. However, soil nailing cannot replace good understanding of the ground, especially if the slope is large and the consequence of failure is high.

Particular care needs to be exercised in the use of soil nails in sites with large relict landslides because the ground mass may have been significantly disturbed (e.g. through the opening up of subvertical relict joints). Where soil nails are adopted, it is important to ensure that the nails are bonded into competent ground beyond the disturbed mass. Other robust stabilisation schemes, such as providing toe weighting or toe support, should also be considered in such cases.

Special care is needed in the analysis of very long soil nails because the routine analysis methods may not adequately model the mechanism of nail-ground interaction. Also, extreme caution is needed where soil nails are inclined at a steep angle relative to the potential slip surface because such nails may not act primarily in tension as assumed in routine design methods commonly adopted in Hong Kong. The fact that these steeply inclined nails may not be as effective as that commonly assumed should be taken into consideration in the slope design.

A constraint associated with soil nails is that they may create a sterilised zone that could obstruct future development at the slope crest.

Martin (2000) cautioned that there are signs that the good performance record of permanent soil-nailed cut slopes could foster a false sense of security based on the following two beliefs:

> "(a) soil nailing can compensate for all adverse geological features and transient groundwater conditions, therefore geological assessment and detailed GI are considered to be of less importance, and
>
> (b) more or less any soil cut geometry is capable of being treated with soil nails, hence avoiding the need for even minor earthworks in design and so reaping the benefit of simpler construction and higher output."

The results of the above 'beliefs' could be a tendency to curtail the scope of ground investigation and possibly also engineering geological input in slope design where substantial input may be warranted for specific sites. Martin (op cit) noted that the above may lead to the routine stipulation of soil nails even for localised steep back scarps, overhangs, disturbed ground from relict instability, areas of possible stress concentration or areas of high erosion potential. In general, consideration should be given to suitable trimming of areas of weakened or potentially easily erodible ground prior to installation of soil nails. Trimming to a smoother and a less steep profile, where appropriate, can also facilitate effective installation of erosion control mat, safer and easier access for maintenance as well as reducing the chance of minor failures.

It is important that for potentially problematic sites, the scope of ground investigation and the level of geotechnical and engineering geological input should not be compromised by the adoption of more robust solutions. In general, each site has to be assessed on its own merits as to how much ground investigation is warranted.

6.4 Use of Prescriptive Drainage Measures

In view of the innate variability of the groundwater conditions, a pragmatic approach is to incorporate contingency drainage provisions in the design. This can be in the form of fairly generous use of prescriptive raking drains to combat the possible build-up of groundwater pressure. This may comprise a series of long and short drainage pipes to tap deep and shallow aquifers respectively where necessary. Counterfort drains may be considered where appropriate, e.g. for less steep cuts.

Prescriptive raking drains are mainly for lowering the base groundwater table. Where there are more permeable zones or preferential flow paths in the ground mass (e.g. erosion pipes), raking drains or counterfort drains will be effective to limit the transient rise in groundwater pressure if they intercept such permeable zones.

7. REVIEW OF SELECTED GEOTECHNICAL REPORTS

7.1 Introduction

The authors have examined more than 100 geotechnical documents most of which are slope design or assessment reports, and have taken part in many design review meetings. Observations and insight so gathered that are relevant to reliability of slope works are presented below. They are organized according to the stages of the investigation, design and construction.

It is emphasized that the review is of a preliminary nature and covers mainly government slopes. The relatively small sample size means that the review may not have fully reflected the prevailing practice, as the approach and rigour by different designers will inevitably vary from case to case, depending on the specific problems at hand. The slopes with geotechnical engineering input included in the present review were chosen randomly and none of them was selected because of landslides. Hence, the sample is not biased in this respect.

The principal objective of the review is to identify areas that warrant attention in order to reduce uncertainties through improved or more insightful routine engineering practice. It is neither the intention nor is it feasible to make any conclusions based on this review as to the extent of the issues highlighted as deserving attention. Whilst there may be isolated areas in respect of slope investigation, design and construction that do not fully comply with the recommended good practice given in GEO publications, this does not mean there is a problem because there may be compensating effects due to other design assumptions.

7.2 Desk Study and Site Reconnaissance

The review suggests that there is room for further improvement in the proper implementation of good practice as follows:

(a) detailed API should be undertaken by experienced and suitably qualified personnel, especially for potentially problematic sites by reference to all the available aerial photographs,

(b) detailed site reconnaissance should be conducted to look for and document signs of past instability and pre-existing tension cracks, including the surrounding area and in particular the ground within the catchment directly above the slope of concern,

(c) detailed review of the development history of the slope and overall site setting, details of past studies and works, slope failures and records of maintenance inspections and works should be carried out,

(d) detailed appraisal of the results of any past ground investigations in the vicinity of the slope should be carried out to better appreciate the overall geological and hydrogeological setting and establish the likely ground conditions to facilitate planning and design of the ground investigation, and

(e) detailed site reconnaissance should be carried out to look for indicative signs of the likely locations of water-carrying services.

A detailed desk study and API paying due attention to the history and nature of any past instability together with the overall site setting are essential and must be carried out as a basic requirement. Special attention should be paid to any geomorphological evidence of large-scale relict instability. It is important that the findings of the desk study and API must be duly taken into account in the design of ground investigation and slope upgrading works.

7.3 Review of Past Stability Assessments and Construction Works

Where there is a need to assess the adequacy of the past engineering input, the past geotechnical assessment or slope works should be reviewed with an open and critical mind. As a guide, past assessments of engineered slopes involving the following issues deserve attention:

Design Issues

(a) past stability assessment may have only considered part of the slope feature with respect to the slope boundary in the New Slope Catalogue,

(b) approach used and assumptions made previously no longer considered acceptable in the light of current knowledge/standards (e.g. reliance on suction),

(c) use of suspiciously high shear strength parameters (e.g. very high c' value for completely weathered granites/volcanic based on back analysis which may have implicitly included the effect of soil suction, especially if the site conditions have been altered subsequently such as through the replacement of a hard slope cover by a vegetated cover),

(d) past slope performance not fully taken into account in the previous stability assessment (e.g. large-scale relict failure or slopes with a history of repeated instabilities),

(e) stability analysis whereby the critical section or critical slip surface has not been considered,

(f) adverse hydrogeological setting (e.g. thin colluvium overlying saprolite, drainage valleys, depression in bedrock profile) not investigated in detail and design groundwater model not fully compatible with the setting and the available piezometer/seepage information, and

(g) stability assessment or design of landslide preventive works with no submissions made to the GEO for checking, or where GEO's District Division had questioned the basis of some of the key design assumptions.

Construction Issues

(a) works recommended following stability assessment not fully implemented,

(b) observations of adverse ground conditions during construction which invalidate design assumptions but may not have been attended to in sufficient detail,

(c) actual upgrading works substantially different from the design given in the geotechnical report that had been accepted by GEO's District Division, and

(d) assumptions on design groundwater condition not entirely consistent with the available data (e.g. seepage above the design groundwater table, high piezometer readings not considered, etc.), or a hydrogeological setting more adverse than the design assumptions (e.g. presence of subsurface drainage valleys revealed during construction).

Post-construction Issues

(a) failures (particularly genuine/sizeable landslides) since past assessment by old studies/slope works which may suggest that the previous geotechnical input needs to be reviewed,

(b) designed raking drains (or permanent anchors) not being monitored and maintained in accordance with the Maintenance Manual or recommended good practice,

(c) problems revealed by slope performance or condition (e.g. signs of distress suggesting potential instability problems),

(d) adverse changes in site setting or environmental conditions (e.g. unauthorized cultivation), and

(e) groundwater conditions more adverse than design assumptions (e.g. seepage observed at levels above the design groundwater table, high piezometer readings, etc.).

It is important that judicious judgement must be exercised regarding the relative significance of the various factors in the review. The performance and robustness of any past slope works should also be taken into consideration in the review. The findings of the review must be viewed in overall terms as to whether it requires further follow-up action.

In practice, one may need to classify which category of slopes would deserve a more detailed review such as that suggested above. In deciding on this, account should be taken of the history of slope performance as well as the nature of the past assessment (e.g. public or private development projects, discharge of DH Orders, default work, work under the LPM Programme), whether the design was submitted to the District Divisions for checking, the type of works carried out, if any (e.g. soil nails or cut back). For example, large unsupported cuts that were engineered in the early stages of setting up the slope safety system (i.e. late 1970s and early 1980s) would deserve a more critical review of the past assessment.

7.4 Ground Investigation

There should be adequate geological input in developing a preliminary geological model, with due regard to the findings of the desk study, API and site reconnaissance. The preliminary geological model should be used to plan and design the ground investigation (GI). The anticipated hazards and problems should be identified and the strategy and targets of the GI should be thoroughly understood by all members of the design team. It is important that the GI site supervisory team should also be fully briefed. In general, if the designer were not certain as to what should be identified, it is likely that he/she would not find the characteristic features even if they did exist.

It is important that the design of GI does not become routine and should not be handled by relatively inexperienced staff with inadequate review carried out by more experienced staff. For example, there have been cases with no insitu density tests or standard (i.e. Proctor) compaction tests specified in the investigation of old fill slopes and the assessment of the adequacy of fill slope is by reference to the calculated factors of safety as for a cut slope. Another example is inappropriate positioning of piezometers which could give misleading results in that high transient groundwater response may be missed.

Whilst the GI should ideally be staged to ensure that its rigour will generally be compatible with the degree of complexity of the ground conditions, in some instances this could be compromised to some extent by tight programming constraints (hence the importance of better project planning and more realistic programmes). Difficult and expensive access for GI works on some slopes, together with an unduly tight programme, may also have compromised the extent of investigation. This could result in unforeseen (but not necessarily unforeseeable) ground conditions being encountered during the construction stage leading to major delays and cost overruns.

It would appear that there is room for improvement in the review of information from past GI. The reliability of the available data, both from previous and current GI, should be assessed. The adequacy of existing drillhole logs and whether past test results are representative or not should be reviewed. Any significant discrepancies between previous GI and current GI should be examined.

Detailed site mapping and records of ground surface detail, including the local geomorphology, can be of great value and there appears to be much scope for improvement in this respect in current practice. At the design stage, inspection of exposures at the site, if any, is sometimes carried out in a cursory manner (Martin, 2000). A wider inspection of adjacent exposures in the area and an assessment of the extent of structural domains by an experienced engineering geologist will add to the insight. Adverse geological materials or structures that may control the geometry and mode of failure and affect the mass strength of the slope-forming material must be identified through systematic mapping of exposures.

The locations of some of water-carrying services are sometimes shown on plan as indicative only. Some designers have opted to leave the investigation of water-carrying services to the contractors during the works stage, partly because of programming constraints. There is merit for designers to pay special attention to this issue at a sufficiently early stage of the investigation where practicable in order to minimise surprises during slope works.

Information on transient seepage and surface water flow during heavy rainstorms is of great value to the designer. Signs of seepage in the adjoining slopes would also be of relevance. More attention needs to be given to these aspects during the investigation stage. Where practicable, site visits should be arranged during or shortly after heavy rainfall to better appreciate the groundwater and surface water conditions.

7.5 Engineering Geological Mapping and Formulation of Geological Model for Design

One of the lessons learnt from landslide investigations is that during the initial phase of ground investigation, emphasis should be directed to developing a representative geological and hydrogeological model rather than testing. In some cases, there is little discussion on data scatter and the credibility of the laboratory test results.

Mass characterisation of the materials is advisable despite its difficulties. Geoguide 3 (GCO, 1988) recommends the use of mass zonal classification schemes, such as the PW weathering classification system, augmented by mapping of surface exposures in an attempt to characterise mass features. In practice, the mass zonal classification scheme does not appear to be commonly used in developing geological models. Relict discontinuities are often not explicitly considered or commented on in design reports unless conspicuous adverse geological structures are apparent. The effect of relict discontinuities leading to a general weakening of the mass strength is sometimes not considered in routine slope design. Joint sets that are subvertical and orthogonal are fairly common which can act as release surfaces for slope instability but these are often not considered as water-filled tension cracks in routine slope design.

Martin (2000) observed that there has been an increasing tendency in recent years to over-rely on routine description of soils and rocks in Hong Kong. Decomposition grade and rock type alone (e.g. CDG) is frequently used as a shorthand descriptor in lieu of a full material description. This falls short of the recommendations in Geoguide 3 (GCO, 1988) and can lead to gross over-simplification of the ground conditions. This is especially of concern in geological models involving thick zones of undivided saprolite (usually based on summary decomposition grades from borehole logs alone), which may contain materials of significantly different grading with potential influence of relict discontinuities. The above practice may or may not be linked to the generalised geotechnical parameters quoted in the Second Edition of Geoguide 1 (GEO, 1993) which have been defined in terms of simple decomposition grades and rock types. Given the above "typical ranges of values of geotechnical parameters", some designers may tend to give less attention to mapping and proper laboratory testing in characterising the material properties and the potentially lower strengths along relict joints. The above practice is strongly discouraged.

Full engineering geological description of the materials is important but there is room for improvement in this respect. Where descriptors in strict adherence with Geoguide 3 terminology are used, it can sometimes stifle fuller description of unusual or important features where they are warranted. It can also result in ambiguous application of certain descriptors or classifiers, e.g. using the mass weathering zonal terms (based on rock/soil percentages) in rocks that weather uniformly without noticeable corestone development (Martin, 2000).

The person responsible for the design must ensure that there is adequate engineering geological input in the development of representative 2-dimensional geological models from the 3-dimensional reality (taking due account of relict discontinuities, corestones, weak infill, local groundwater complexities, etc.). In the absence of this, a meaningful slope stability analysis could not be possible. Thinking in terms of 3D in respect of suitably simplified geological and hydrogeological conditions is critical and must be borne in mind when cross sections are considered. The classification of adverse geological conditions and structures needs considerable judgement in assessing the degree of influence on slope stability. The uncertainty associated with geological modelling can be considerable and can have an overriding effect on an apparently complicated theoretical analysis. It is therefore important that there is adequate engineering geological input to the investigation and design of slopes as well as stability assessments. In practice, there may not be clear documentation as to whether appropriate engineering geological input has been provided by suitable personnel at the critical stages.

The notion of tending to err on the safe side may not always be as safe as that envisaged by the designer. For example, where a slope has a variable rockhead level, the adoption of a simplified geological model with the 'conservative' assumption that rockhead is at the lowest level mapped may not be adequate. This is because a high perched water table may actually develop at the section with high rockhead.

7.6 Assessment of Shear Strength Parameters

An area that deserves attention is the need to appreciate the uncertainties in the characterisation of shear strength and interpretation of test data. There is room for improvement in the specification of laboratory testing and interpretation of test results. For example, many of the test data at high mean effective stress levels are sometimes being used in routine determination of design shear strength for soil cuts in saprolites even for relatively shallow slip surfaces. Also, the potential influence of relict discontinuities on the test results are sometimes not considered explicitly.

In the case of loose fill slopes, triaxial tests are commonly carried out on so-called undisturbed samples but there are uncertainties as to whether the test conditions are representative in terms of replicating the relative compaction values in the field. In cases where volume changes due to saturation of the loose specimens or due to consolidation are not considered, the test specimens could have been densified in which case the results may not be representative of the field conditions (see GEO Technical Guidance Note TGN No. 7).

The characterisation of the geometry of sheeting joints (or other persistent discontinuities which are thought to be related to stress relief effects) and their operational strength is fraught with great difficulty. A potential difficulty is the presence of wavy sheeting joints that may have adverse local steepening of the discontinuity behind the slope face.

The characterisation of the shear strength of weak infill to relict discontinuities is difficult and must be done with extreme caution. Where direct shear box tests are carried out, aligning the relict discontinuity to ensure shearing through the discontinuity is difficult (but important) because it may be curvilinear with a variable infill thickness. Examination of the stress-displacement curves could help in judging whether the shearing was along the infilled discontinuity. Experience indicates that widely varying results can be obtained depending on the test preparation. In general, gypsum should be used to mount the specimen in a shear box when an infilled relict discontinuity is being tested.

Relict discontinuities are not just important in terms of their influence on the operational shear strength. Persistent discontinuities with clayey infill are conducive to local damming of water flow and possible build-up of localised water pressures. Furthermore, the relict joints can act as release surfaces in promoting stress concentration, with the potential of promoting progressive failure and possible local opening up of the relict-jointed soil mass. This in turn could increase the potential for direct infiltration and possible development of cleft water pressure.

Some designers attempt to build in more conservatism in the slope design by adopting pessimistic design parameters, e.g. lower bound of the generalised parameters given in Geoguide 1 (GEO, 1993). This approach may either be overly costly or may give rise to a false sense of security because the pessimistic assumptions may not be adequate to cover for undetected adverse geology where adequate engineering geological input is not provided. Assuming low strength parameters is not necessarily a robust approach, because the margin against geological and hydrogeological uncertainties is very low. The key is to aim to better understand the ground so as to reduce uncertainties, and to use robust measures to cater for inherent uncertainties and spatial variability of the ground mass.

In some of the cases, generalised strength parameters based on Geoguide 1 are adopted for preliminary design because of project programming constraints. The full ground investigation findings and laboratory test results that are subsequently available may be examined only with respect to whether the generalised parameters are adequate or not. Where the generalised parameters are shown to be conservative, the preliminary design becomes the final design and the site-specific testing data may not be directly taken into account.

It would appear some of the designers tend to think primarily in terms of factors of safety rather than examining the uncertainties involved. There is no one-to-one relationship between reliability of a slope (in terms of probability of failure) and the calculated factor of safety. A slope with a given factor of safety (based on best estimates of the shear strength in a slope with relatively homogeneous material) may actually have a lower probability of failure (i.e. more reliable) than a slope with comparable geometry that has a higher calculated factor of safety if the latter were subjected to greater uncertainties in terms of shear strength and groundwater conditions (e.g. more complex geology). This point is illustrated graphically in Figure 1.

7.7 <u>Groundwater</u>

7.7.1 <u>Groundwater Monitoring using Piezometers</u>

The review shows that in some of the cases where colluvium/fill/residual soil is present near the slope surface, the possibility of the build-up of a perched water table in the shallow thin colluvium was not investigated with the use of shallow piezometers. Some designers have argued that the possibility of build-up of a shallow perched water table is low based on consideration of relative permeabilities of the different layers (i.e. the strata are judged not to have sufficient permeability contrasts as to be conducive to the build-up of a perched water table). Such a judgmental approach without adequate support by site-specific test data should be treated with extreme caution.

Piezometers should be installed in the most appropriate locations both in plan and in elevation to avoid misleading results. Some of the piezometers have been installed several metres below the surface of Grade III or better rock. Piezometers are sometimes not installed in areas with potential drainage concentration, e.g. drainage lines or buried drainage valleys. It should be noted that the subsurface catchment that may give rise to subsurface drainage concentration may not necessarily correspond to the surface catchment. Adequate investigation should be carried out to try to define any significant depressions in the bedrock (notionally taken as Grade III or better rock) profile.

Halcrow buckets that can record the peak groundwater level are sometimes not installed. It is important that the buckets are placed at appropriate levels so that peak transient groundwater response will not be missed.

In some cases, the piezometer monitoring period only extended to 7 days which is obviously not adequate.

Design groundwater conditions are sometimes prescribed based on limited or inadequate piezometer data. The approach can be rather crude with no indication of consideration of the overall hydrogeological setting. Potential effects of geological features and catchment characteristics on the likely groundwater regimes and any delayed groundwater response where observed are not discussed explicitly.

There are also cases with no discussion of the severity of rainstorms in terms of return periods during the piezometer monitoring period when assessing the design groundwater condition. Whether short-duration rainfall or long-duration rainfall would be most critical for a given slope would depend on the actual site-specific hydrogeological conditions, site setting and the type of slope surface cover. The above considerations will provide a more reasonable basis for making a judgement on the projection of the available groundwater monitoring data to arrive at the design groundwater condition corresponding to the 10-year return period rainfall for different rainfall durations.

7.7.2 Assessment of Groundwater Conditions

As far as the assessment of design groundwater conditions is concerned, it would seem from the review that the general approach by practitioners has remained rather rudimentary. A common approach in many of the cases is based on the simplified assumption of the highest measured groundwater level during the monitoring period plus typically 2 m to 4 m to arrive at the design groundwater condition. Such a judgmental approach appears to have become the norm even for sizeable slopes with relatively complex hydrogeological conditions. The basis and rationale of assuming such a nominal rise in the groundwater level over the highest measured groundwater level may not be fully understood by some of the designers. The more rigorous and complicated methods suggested in the Geotechnical Manual for Slopes appear to be rarely used by practitioners.

The hydrogeological investigation carried out under the Mid-levels Study has provided much insight into the potential complex and heterogeneous groundwater regimes, including transient perched water tables, downward and downslope groundwater flow as well as upward hydraulic gradients from fractured bedrock, influence of erosion pipes and leakage from water-carrying services, etc., that may need to be considered in practice. Also, the effect of topography, in particular possible hollows or dishes present above the slope crest which could initiate convergence of groundwater seepage and development of locally high groundwater pressure, needs to be borne in mind. Care needs to be taken in the case where the permeability of the near-surface material (e.g. clay-rich material) on a slope is less than that of the underlying material because there is the potential for development of high groundwater pressure in the more permeable underlying layer through infiltration in the upslope area.

Perched water tables may form at the interface between colluvium or fill and the underlying soil, between zones of differing degrees of weathering, or within zones of weathering or colluvium (e.g. loose young colluvium overlying dense old colluvium). Perched water pressure is transient but its presence has been detected by piezometers or inferred from the geometry of failure scars.

The possibility of the development of a perched water table was not fully considered in some of the cases. It would be prudent to assume that perched water can develop where a permeable surface stratum overlies less permeable ground unless proven otherwise by adequate site-specific groundwater monitoring.

Sites with large cuts truncating sizeable catchment areas with the potential for concentration of groundwater along an infilled colluvial valley or old streamcourse are liable to have high groundwater conditions. However, sites located at the end of a spur could also have complex groundwater conditions. The complex groundwater conditions may be associated with the presence of persistent kaolin-infilled discontinuities and/or potential concentration of subsurface groundwater flow because of a rockhead depression and nearby faults. The possibility of through-flow along high permeability layers or soil pipes creating seepage pressures should be borne in mind. The build-up of a perched water table could be due to the reduction in permeability with depth because of a decreasing degree of saturation. Where there is upward seepage from more fractured and permeable rock below the surface of Grade III or better rock at the toe of a large catchment due to uphill recharge, significant groundwater pressure may build up.

Changes in the nature of surface cover and slope profile are liable to affect the groundwater regime significantly. New piezometer data would in principle be needed to derive or support design assumptions in such cases.

A careful examination of the slope surface for seepage signs, such as faint staining of chunam or shotcrete below weepholes, may give useful clues to the designers. However, there is no easy way of relating seepage to piezometric heads and any interpretation needs to be made with considerable judgement. The observed seepage may be a result of general perching or local perching mechanisms, preferential flow through erosion pipes or associated with the base groundwater level. Lack of signs of seepage is, in itself, not a reliable indicator that there will be no build-up of transient groundwater pressure during heavy rainfall.

7.8 Geotechnical Control

In principle, the District Divisions in GEO are to undertake a paper audit of a geotechnical report focusing on the major items and basing the comments more on engineering judgement and experience rather than on the calculations. This has generally worked well in identifying major deficiencies in design submissions. In practice, the calculated factors of safety are the essence of getting design submissions through the checking system. The validity of the calculated factor of safety is obviously contingent upon the formulation of a representative geological model and proper characterisation of material strength and groundwater conditions. In performing a paper audit of geotechnical submissions, it may not be easy for the checkers to identify potential shortcomings in the ground investigation or design geological models (e.g. adverse geological features not detected) as reflected by failure of some engineered slopes. Unlike the designers, the checkers may not have access to all the information and they have to rely largely on interpreted information provided by submitters. The checkers are also constrained by the amount of time that can be devoted to individual submissions.

In view of the above, the vetting of some of the major items (e.g. adequacy of engineering geological input to ground investigation and formulation of geological model, assessment of material strengths and groundwater conditions, verification of design assumptions during construction, etc.) is subject to constraint. For potentially problematic slopes with complex geology and a history of failures, it could be beneficial for the checker to visit the site (together with the designer as considered appropriate) to better appreciate the site setting, where time and resources permit. More emphasis on site audits during construction to complement the paper audit of the design submission will also be an appropriate way forward.

An average Category I qualified supervision personnel, whilst fulfilling the stipulated minimum professional qualification and experience requirements, may or may not have the requisite engineering geological skills and sufficient local experience to undertake design reviews during construction and revise the design geological model as appropriate, especially for geologically complex sites. Lessons learnt from landslide investigations highlight the need to review whether this aspect of geotechnical control in relation to ensuring adequate level of engineering geological input for particular slopes needs to be further tightened up or not.

8. SUGGESTIONS FOR IMPROVED PRACTICE TO ENHANCE THE RELIABILITY AND ROBUSTNESS OF ENGINEERED SLOPES

Systematic landslide studies have identified the need to further improve slope engineering practice in order to enhance the reliability and robustness of engineered slopes. Recommendations are made in Sections 4 and 6 for tackling generic factors leading to minor and major failures respectively. Based on the review referred to in Section 7, some suggestions are made below with a view to reducing uncertainties through improved or more insightful routine engineering practice:

Project Planning

(a) Better project planning with more realistic schedules and programming as far as possible so that sufficient time will be available for ground investigation, slope assessment and design of upgrading works and that the results of the site-specific GI findings including laboratory testing will be incorporated into the design.

(b) Programme rainfall sensitive work items (e.g. recompaction of loose fill slopes) for the dry season as far as possible. This will minimise potential problems during construction.

Desk Study

(a) A comprehensive aerial photography interpretation (API) report should be prepared by experienced personnel, with assistance provided by others with special expertise where necessary. The API should, inter alia, aim to establish the history and nature of any past failures that may affect the slope, and any geomorphological evidence of large-scale relict instability which may not be obvious because of the subsequent development.

(b) All available documentary information (including old topographical maps where appropriate) relating to the development history of the slope, past performance and previous studies and works should be examined in detail. The review of the available information should not necessarily be limited to the boundary of the man-made slope of concern.

Ground Investigation

(a) Designers should take due account of the findings of API and desk studies, together with their observations from detailed site reconnaissance, in designing the ground investigation.

(b) Designers should formulate preliminary geological models based on all existing information. This includes the geological and geomorphological setting, previous ground investigations, past assessment of the site and its vicinity, records of past failures. The models should form the basis for designing the ground investigation in terms of what questions are to be answered by what additional information. If the designer does not know what the ground investigation is targeted at, he/she is liable to miss the important features that may be present.

(c) Consideration should be given to specifying continuous Mazier sampling to supplement the alternate sampling and SPT where appropriate (e.g. for sizeable and potentially problematic slopes). All untested Mazier samples should be split for detailed examination by the designers as far as possible.

(d) Particular attention should be given to identifying any adverse geological features, such as unfavourably located fault zones or major joint sets, weak or relatively impermeable materials, pre-existing shear surfaces, preferential flow paths and internal erosion pipes, etc. Reference should be made to GEO TGN No. 4 for guidance on recognition of geological features hosting, and associated with, silt-rich or clay-rich layers in igneous rocks.

(e) Topographical survey of the slope should in general be carried out to provide more accurate information for stability analysis.

(f) Attention should be given to examining the specimens after laboratory testing and reviewing laboratory test reports in detail. Reference should be made to the sketches of the mode of failure in triaxial tests, stress paths, etc. and an assessment of whether the test results are reasonable and representative should be made. Any significant influence of relict discontinuities on the test results should be noted.

(g) Particular attention should be paid to significant core losses, sediment-infilled erosion pipes, etc. in the cores as well as to major structural geological features (e.g. faults, dykes, foliation) in formulating the geological model.

(h) Special attention should be paid to all buried water-carrying services that may affect the slope of concern at an early stage. The investigation should try to ascertain the locations of all buried water-carrying services (geophysical techniques such as electromagnetic induction survey have proved to be useful for this purpose in some cases). The condition of pipe runs should be examined (e.g. using CCTV) and records of maintenance inspections and works, results of leakage tests and past leakage history should be reviewed. The option of diverting the buried water-carrying services should be pursued at an early stage. If diversion is not feasible, alternative measures (such as ducting, etc.) to combat the potential adverse effects of water-carrying services should be considered.

<u>Hydrogeological Investigation</u>

(a) More frequent use of shallow standpipes and piezometers sited at potential perching horizons (e.g. colluvium/saprolite or fill/saprolite interface) is encouraged. Deeper piezometers should also be installed to measure the base groundwater level (installation of up to two piezometers in a drillhole should be acceptable provided there is adequate supervision).

(b) Sufficient piezometers should be installed at the appropriate locations (e.g. close to the surface of Grade III or better rock in areas of potential concentration of subsurface flow) and at suitable elevations where the ground conditions may be conducive to the build-up of local high groundwater pressure (e.g. old/young colluvium interface, persistent clay-infilled discontinuities in saprolite, locations of concentrated seepage, erosion pipes, etc.). The locations of piezometers should be continually reviewed during the course of ground investigation taking into account the findings of the preliminary geological logs from drillholes, trial pits and/or surface stripping.

(c) Halcrow buckets should be placed in all piezometers at the appropriate levels to measure the peak transient groundwater response. Automatic piezometer devices with an integrated pressure sensor and data logger to obtain real time transient responses should be considered, particularly for sites with high groundwater levels and complex hydrogeological conditions.

(d) The piezometers should be monitored for a sufficiently long period (preferably at least over one wet season if practicable) to minimise the uncertainties. Consideration should be given to installing additional piezometers as part of the slope works (as well as maintaining the pre-existing piezometers) to verify the design groundwater assumptions.

(e) More systematic consideration of the sources and pathways of water ingress into the slope should be made. As far as practicable, site visits should be paid during or shortly after severe rainfall, particularly for potentially problematic sites, to observe surface water flow, adequacy of existing drainage measures and any seepage from the slope.

(f) Where the design involves replacing a hard surface cover with a vegetated cover, this may bring about changes in rainfall infiltration characteristics and affect the groundwater response. Piezometric response measured during ground investigation before replacement of the hard cover may not be directly relevant and could even be misleading in some cases. It would be prudent to continue piezometric monitoring up to, and during, construction as far as possible in order to verify the design assumptions.

Design Considerations

(a) Designers should ensure adequate engineering geological input to the design process and assess the level and quality of such input in the investigation stage when using the GI information. Upon completion of design, designers should recommend the extent and level of geological input needed during construction, with due regard to the complexity of the site and robustness of the design solution.

(b) Designers should comprehend all the material facts and ensure that all the available information and specialist advice (e.g. detailed API and geological mapping) are fully taken into account and integrated in the geological models and design assessment to avoid overlooking any key information.

(c) In assessing the design option to be adopted, the reliability and robust nature of the design scheme should be considered, taking into account the sensitivity of the option to the uncertainties involved. In general, such an assessment may be done in a qualitative manner by experienced personnel having regard to site-specific information and examining what can go wrong.

(d) Designers should assess potential changes to the existing environmental conditions arising from the proposed works/development. Any adverse factors associated with the changes in environmental factors should be duly accounted for in the slope design.

(e) A holistic approach to stability assessment, accounting for past slope performance, overall site setting and geological/hydrogeological setting should be adopted.

(f) The back analysis approach can facilitate the assessment of design shear strength parameters, particularly where there are no significant changes to the environmental setting of the slope. Designers should however use the back analysis approach with care, see GEO TGN No. 6.

(g) Given the potential complexities of groundwater regimes in heterogeneous materials and uncertainties associated with the storm response, it would be good practice to undertake sensitivity analyses to examine the effects of different groundwater assumptions on stability. If the calculated factor of safety is found to be sensitive to changes in groundwater levels, a more cautious approach in terms of adopting a reasonably conservative estimate of the design groundwater condition would be warranted.

(h) The degree of perched water or seepage-induced water pressure (see Section 5.3.2) is not easy to determine, especially where the groundwater monitoring period is short or does not cover heavy rainstorms. In the case where the groundwater data from GI are inadequate, a possible approach to account for the uncertainty is to assume an appropriate minimum r_u value for stability assessment of slip surfaces at shallow depths, taking into account the surface cover and drainage provisions. What constitutes shallow depths has to be judged for each site but 5 m seems suitable for most geological setting. The above is intended to be applied flexibly with judicious judgement. For example, the approach may be a pragmatic way forward where considered judgement has concluded that seepage-induced pressure or perching is likely to develop during rainstorms, in the absence of specific piezometric data or seepage observations.

(i) A similar approach to item (h) above may also be adopted for the stability assessment of deeper potential slip surfaces where groundwater information is limited. The appropriate value of r_u to be used has to be judged with reference to the information in hand, site setting and experience from design of similar cases.

(j) The severity of the rainstorms in terms of return periods during the piezometer monitoring period should be examined to assist in the assessment of the design groundwater condition. This can be done by reference to the data from the principal raingauge of the Hong Kong Observatory at Tsim Sha Tsui given in Table 3 of the report by Lam & Leung (1994). Adjustments for the spatial variability of rainfall in different parts of Hong Kong in assessing the return periods may be made using the suggestion by Evans & Yu (2001), where considered appropriate, given that the rainfall at some locations may be more intense than that at Tsim Sha Tsui. Allowance for the spatial variability of rainfall may be useful for the design of surface drainage provisions and the evaluation of the return periods of specific rainstorms experienced during the piezometer monitoring period. It would be prudent to consider the effects of both short duration rainfall and long duration rainfall in assessing the design groundwater conditions by reference to the piezometer response and the severity of rainstorms during the monitoring period.

(k) For slopes with known significant storm response, robust groundwater assumptions should be made (i.e. with suitable allowance to cover for possible construction delay) for stability assessment of temporary cuts that are programmed to be carried out in the dry season.

(l) Having understood the ground with care, good quality information and geotechnical skill, uncertainties in the design groundwater condition could still be significant. Generous use of prescriptive subsurface drainage measures such as raking drains can improve robustness of the slope works. Apart from locations with evidence of seepage from the slope face, situations where prescriptive drains may be warranted include:

- interface of materials with significant permeability contrasts giving rise to the potential for perching
- sizeable catchment draining to the slope
- presence of stream course in the vicinity of the slope with the possibility of lateral drainage
- stability of the slope is especially sensitive to changes in design groundwater level
- contingency provisions against possible chronic leakage from nearby water-carrying services

- concern about possible damming effects of closely-spaced soil nails on groundwater flow

(m) In assessing the lengths and permeable zones of raking drains, designers should be mindful of the potential for long raking drains to bring water from deeper aquifers to the near-surface material thereby adversely affecting its stability. Extreme caution should be exercised in prescribing long raking drains in a slope with a large depth of fill (e.g. in a buried drainage line). For long raking drains, the pipes should not be perforated within the near-surface material and proper grouting should be carried out outside the solid pipes to minimise the risk of bringing water from the deeper aquifer to the near-surface material.

Slope Stability Assessment

(a) Ensure that the most critical sections are analysed, taking due account of slope height, angle, potential concentration of groundwater flow and locations of relict failure scars. In the stability analyses, the factors of safety should be assessed for the various critical sections such as steepest/highest section, section through failure scar and section with highest groundwater level.

(b) Emphasis should be given to the assessment of relict discontinuities in saprolites and their influence on the mass strength and the potential for leading to rapid build-up of transient perched water tables within the weathering profile.

(c) There should be systematic identification of all potential failure modes (e.g. washout, local detachments, stability of topsoil over rockfill and possible deep-seated failure). Apart from the calculated factors of safety, designers should amenable to theoretical calculations and are best dealt with through attention to good detailing and the adoption of prescriptive provisions.

(d) Calculations of factors of safety based on drained (i.e. effective stress) strength parameters, as used for cut slopes, are not appropriate for stability assessment of loose fill slopes. Water ingress leading to liquefaction corresponds to a drained-undrained failure, with potentially very low shear strength due to build-up of high excess pore water pressure associated with collapse of the metastable structure (HKIE, 1998).

(e) For an existing retaining wall, it is important to establish the geometry of the wall to facilitate meaningful stability analyses. Attention should be given to avoid placing undue reliance on old approved drawings without site-specific ground investigation including weephole probing (where possible), because such old drawings can be misleading. Thin masonry walls (e.g. a slenderness ratio of greater than 5) are liable to fail in a brittle manner without prior warning. For such thin masonry walls (or facing elements), stability assessment based on conventional methods of retaining wall analysis is not appropriate and they should be taken as substandard.

Slope Drainage Assessment

(a) Crest drains should normally be provided in engineered slopes. However, in the special case of small slopes well protected against erosion by hard cover and with minimal direct catchment uphill and no obvious depressions or low points to concentrate surface flow towards the features, crest drains may not be critical.

(b) A review should be carried out of the slope performance in terms of adequacy of the pre-existing surface drainage provisions, with due regard to the history of previous flooding or over-spilling of surface water and considering the overall site setting in the area.

(c) The outcome of possible blockage of critical sections of drains should be assessed and suitable redundancy should be allowed for in the design of surface drainage provisions. This can be done by adopting a suitably conservative 'n' value (e.g. channels assumed to be silted) to cater for possibility of partial blockage of the drainage channel with soil debris and vegetation.

(d) Consideration should be given to the need for sending details of the estimated flow volume and locations of discharge points to the pre-existing stormwater drainage pipes, together with the details of the proposed surface water drainage system for the slope, to the Drainage Services Department (or the Highways Department where the discharge point is to be connected to storm drains or cross road drains maintained by them) for comments. The above is suggested because the relevant authorities will have the information on the details and design capacity of their stormwater drainage systems. The objective is to avoid over-loading the existing facilities causing problems in the downstream area.

(e) The adequacy of the drainage channel can be sensitive to the size of the catchment area and the 'n' value assumed in the analysis. The drainage characteristics of the upslope area, as well as other details such as condition of drainage channels bounding the adjacent slopes, are relevant to the determination of the size of the catchment area.

(f) It would be prudent to take into account the effects of spatial variability of rainfall in the design of surface drainage measures for areas with more intense rainfall than that at Tsim Sha Tsui (e.g. see Evans & Yu, 2001).

(g) For roadside slopes, reference should be made to the Highway Slope Manual (GEO, 2000) for advice.

Peer Review and Independent Checking

(a) The quality assurance system should include adequate project design reviews at key stages (e.g. design of ground investigation, option assessment and detailed design), as well as independent checking by suitably experienced and qualified geotechnical professionals.

(b) For potentially problematic sites, the independent checking engineer should endeavour to visit the site in advance of doing an audit of the draft design report. Also, it would be good practice for the independent checking engineer to visit the site during construction in the case of rock slopes, particularly for large rock slopes. This is because the final design of rock slope stabilisation works is generally based on detailed face mapping carried out during construction (see GEO TGN No. 10).

Construction Control

(a) Ensure adequate site supervision of slope works having regard to the size and complexity of the site to improve construction control and prevent substandard works. Ensure that the supervisory staff will witness all the critical operations (such as insertion of steel reinforcement into the open bore and the subsequent grouting operation for soil nail construction).

(b) Carry out verification tests for critical activities, e.g. sand replacement tests and GCO probes in the case of fill slope recompaction.

(c) Consider carrying out site audits with no advance warning.

(d) Site supervisory staff to keep contemporary records of problems encountered during construction for the designers to review and assess their potential influence on the permanent works, e.g. hole collapse during drilling of soil nails and excessive grout loss.

(e) Site supervisory staff and qualified supervision personnel should keep abreast of lessons learnt from landslide investigations.

(f) Provide detailed briefing to site supervisory staff regarding the major uncertainties, key design assumptions and specific items to look out for.

Design Review During Construction

(a) Verification of design assumptions should be carried out by a qualified and experienced geotechnical professional who is familiar with the design during construction. Such design reviews during construction should be regarded as part of the design process. The exposed slope face should be mapped, with particular attention to any adverse geological features. In conducting the design review, the qualified supervision personnel should consider the need for further specialist advice or a second opinion from an experienced engineering geologist where appropriate, particularly for sites with complex ground conditions.

(b) The findings of verification of the design geological model during slope works should be incorporated into the as-built records. These, together with a schedule of key geotechnical design assumptions, should be included in the Maintenance Manual for future reference.

(c) Significant design revisions should be referred to the independent checking engineer and submitted to the GEO for checking.

Post-construction Review

(a) A post-construction review of the adequacy of design assumptions and slope performance by the designer during the Contract Maintenance Period (in addition to design reviews during critical stages of construction) is strongly recommended.

(b) Where there are significant uncertainties about the groundwater condition, designers should review the need for post-construction piezometric monitoring to verify the design assumptions. This may be needed to check for potential damming effects (e.g. due to closely spaced soil nails in a drainage line).

9. IMPROVEMENT IN DETAILING

Traditionally, the emphasis of the designers tends to be on the theoretical stability analysis and the calculated factors of safety. There is scope for improving the detailing of surface drainage provisions and slope surface protection measures.

Some examples of good practice in slope detailing are given below:

Surface Drainage

(a) Be fairly generous, where practicable, in the sizing of drainage channels and stipulating the fall of surface drainage channels to minimise the risk of blockage (Figure 2).

(b) Provide adequate stepped channels (say, a horizontal spacing of about 15 m to 20 m where practicable, incorporating suitable landscaping measures to minimise visual impact) to facilitate rapid discharge of runoff from crest and berm channels.

(c) Where there is a streamcourse or collection of surface runoff at a low point at the crest, it would be prudent to drain the flow through a stepped channel directly downslope. Large stepped channels may be camouflaged using aesthetically adapted covers.

(d) Avoid drainage measures that are more prone to blockage (e.g. open channels are generally less prone to blockage compared with buried channels. Also, open channels tend to be more likely to be cleared in routine maintenance.

(e) Avoid abrupt changes in flow directions as far as possible.

(f) Prescribe a crest channel of not less than 300 mm in size even where theoretical calculations suggest a smaller drain might be adequate.

(g) Provide upstands for drainage channels to minimise possible overshooting of surface water where considered appropriate (e.g. for slopes with a sizeable catchment).

(h) Ensure proper construction of drainage channels (e.g. concrete should be cast directly against the walls of the excavation) to avoid creation of gaps on the outside of crest drain which will promote water ingress.

(i) Consider the use of baffle walls in preference to catchpits at the junctions of drainage channels because catchpits tend to be susceptible to blockage by erosion debris or dead vegetation and are prone to cracking caused by minor slope movements.

(j) Consider the provision of a curved profile at the junction of berm channels and the down channels to improve the hydraulics of surface water flow, where practicable.

(k) Provide aprons of a sufficient size to the sides of stepped channels to minimise undue erosion caused by splashing

(l) Consider the capacity of the outlet of surface water drainage to avoid flooding the downstream area or overwhelming the highway drainage system due to improved drainage on the engineered slope.

(m) Provide multiple discharge points to mitigate flooding in vulnerable areas.

(n) Allow for adequate surface drainage provisions at the junction of an upper batter with a hard cover and a lower batter without a hard cover, or where there is a significant change in slope gradients between batters, in order to minimise additional water ingress into the lower batter due to concentrated surface runoff from the upper batter.

(o) Consider sloping berms for new cut slopes to enhance the self-flushing capacity of the drains, taking due care to ensure that splash erosion at berm-end junctions will not occur due to flow at relatively high velocity.

Subsurface Drainage

(a) Prescribe a drainage blanket behind the 3 m recompacted fill zone in the upgrading of old fill slopes, to extend as far up the excavated surface as possible. Such prescriptive drainage blankets must not be deleted even where no seepage is observed during construction.

(b) Attention should be given to ensuring proper alignment and connection of drain pipes of subsurface drainage provisions to avoid water leakage. Where drain pipe connections are located immediately behind stone facing panel to a retaining wall, there could be a risk of build-up of water pressure behind the facing panel in the event of leakage from the pipe connections. This should be taken into account in the design detailing.

Slope Surface Protection

(a) More extensive use of wire netting for rock slopes to prevent dislodgment of small rock blocks by ravelling failures as a result of progressive deterioration. The connections of the wire mesh netting at the slope toe should preferably be detachable to facilitate clearance of rock debris. A small tension may be applied to the wire mesh netting where considered necessary.

(b) More extensive use of wire mesh bolted onto soil nail heads in soil cut slopes (i.e. not necessarily restricted to very steep slopes).

(c) Ensure proper fixing of non-biodegradable mats erosion control mats to slope surface using adequate fixing pins and providing proper anchorage at the crest and toe of each slope batter to prevent the surface lifting off. Minor earthworks to trim slope surfaces to smoother profiles can facilitate proper fixing of erosion control mats.

(d) Examine the applicability of using bioengineering methods whilst vegetating slopes, with due regard to performance records and sustainability aspects and robustness.

(e) The provision of a new shotcrete cover over an existing hard cover, which is a construction expediency but denies the opportunity for face mapping, should only be done with extreme caution.

(f) The provision of a hard cover (e.g. stone pitching or shotcrete) together with subsurface drainage measures to local patches of weaker rock mass to limit infiltration and deterioration may be justified from slope stability point of view, subject to appropriate landscape treatment.

Other Considerations

(a) Vary the berm spacing to optimise the number of berms. Consideration may be given to omitting berms for slopes higher than 7.5 m where there is no major concern over erosion of the slope face and where adequate drainage is provided at the slope crest.

(b) Provide prescriptive (flexible or rigid) barriers above the crest of cut slopes to catch small boulders or rockfalls and minor slumps from natural hillside as a contingency measure where necessary. This needs to be done on a case-by-case basis exercising sound judgement.

(c) Pay due attention to the detailing of soil nail heads for temporary and permanent soil cuts to ensure the ability to develop adequate resistance near the slope face to prevent failure of the active zone (small-sized steel plates as sometimes adopted for temporary cuts, should be avoided).

In principle, the consideration of slope detailing should be tailored to the nature of the problem and the overall site setting. Engineering judgement needs to be exercised for specific problems. This may be illustrated by the example of potential over-spilling of surface water that may cause failure of the downhill slope. Where high velocity surface water flowing across a road above the slope crest is anticipated, the use of baffle walls can adequately prevent water overshooting the road edge. In the case where flooding is possible at a low point of the road, suitable road drainage measures, possibly in conjunction with baffle walls or upstands, would be preferred.

10. SLOPE MAINTENANCE

Based on systematic landslide studies, inadequate slope maintenance is found to be one of the most significant factors contributing to minor failures on soil cuts, rock cuts and fill slopes. In contrast, inadequate maintenance is generally not a significant contributory factor in many major failures on engineered slopes. However, lack of maintenance can sometimes contribute to major failures where the site setting is unfavourable (e.g. slopes with pre-existing tension cracks concealed below the hard surface cover which are sensitive to loss of suction arising from infiltration through local defects in the slope surface cover). It is also possible that an initial small failure caused by inadequate maintenance of the slope surface protective and/or drainage provisions could create an unfavourable setting and trigger a sequence of events that could lead to a large-scale failure.

There is now ample evidence from detailed landslide studies that some failures, especially large-scale failures of sizeable cut and fill slopes in Hong Kong, exhibit prolonged movement for a considerable period of time (several years or more) before final detachment of the unstable mass. This is reflected by open tension cracks infilled with debris and displacements across infilled discontinuities. Thus, slope deterioration (development of tension cracks and ground opening up thereby resulting in changes in hydrogeological conditions and increased water ingress) plays an important role in landslides. It is therefore important that any significant signs of slope movements be identified and reported during routine inspections for immediate follow-up inspections by geotechnical engineers. The periodic Engineer Inspection (EI) should also pay due attention to detecting signs of slope deterioration and distress and recommend the necessary works and follow-up studies to understand the causes of movement.

The requirements for EI under the slope maintenance system for government slopes mean that there will be continual geotechnical input to maintenance of stability of the engineered slopes after construction. The EI also provides a safety net in that any previous

Stability Assessment will be reviewed. The aim of this review is to identify whether the previous Stability Assessment contains any obvious deficiencies in engineering approach or invalid assumptions in the light of current local geotechnical engineering practice and safety standards and actual slope performance, and to judge whether the stability of the features would be affected by any visible changes (including landslides) identified during the site inspections.

Given the importance of EI, they should be carried out to a high standard. Landslide studies have revealed areas for improvement in the implementation of EI. For example, some EI did not detect or highlight potential problems such as unplanned vegetation on rock faces with root wedging, prolonged internal erosion, leakage of buried water-carrying services, past failures, inadequate follow-up actions taken in response to earlier recommendations, etc. (Ho et al, 2001).

It is important that the EI should take cognizance of the overall site setting and cover the area immediately outside the registered slope boundary, especially uphill, to check for possible signs of problems such as distress or tension cracks. In the case of rock slopes, recommendations from EI for removal of loose blocks and sealing up of open joints on rock slopes should be more specific in terms of the locations and the actual requirements. It would be advisable to allow for a follow-up inspection after the completion of the recommended works to review the adequacy of the works and ensure that no other loose blocks are exposed following scaling and that the sealing up of open joints has been done properly.

Technical audits of the quality of EI reports, which may include independent site visits, should be carried out in order to upkeep the standard of EI.

11. AREAS REQUIRING FURTHER WORK

It is recommended that the following areas deserve further work:

(a) Assess the feasibility of developing a classification system that takes into account key factors like slope size, consequence category, complexity of ground conditions, robustness of design scheme, etc. for assessing the level of geotechnical and engineering geological input expected during the investigation, design and construction stages.

(b) Review the usefulness and practicality of undertaking, on a periodic basis, systematic reviews and documentation of the key findings and lessons learnt through vetting of geotechnical submissions and site audits.

(c) Examine the need for, and practicality of, formalising an appropriate system to specifically audit the adequacy and quality of engineering geological input for selected slope works (e.g. large cuts).

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LIST OF TABLES

Table 1 - Typical Triggering Events for Slope Failures in Hong Kong

Triggering Event	Examples and Effects	Remarks
Rainfall	(a) Actions of running surface water, resulting in surface erosion of the slope, wetting of the slope-forming materials and subsurface water flow. (b) Wetting of the soil or rock mass and the materials in discontinuities, resulting in reduction of the unsaturated component of the soil shear strength. (c) Build-up of perched water level, resulting in increased water pressures in the soil or rock mass. (d) Development of pore water pressure in ground profile with gradual changes in permeability with depth (e.g. changes in the degree of weathering or varying degree of saturation), resulting in increased water pressures in the soil or rock mass. (e) Rise in base groundwater level, resulting in increased water pressures in the soil or rock mass.	Very common trigger; depends on the topography, effectiveness of the slope surface protection, hydrogeology, etc.
Bursting and leakage of water-carrying services	(a) Bursting of pressurised water main, resulting in concentrated water flow with significant erosive power. (b) Leakage from services, resulting in wetting of the soil or rock mass and increase in water pressures.	Not a common trigger, but a major contributing factor in some cases.
Adverse construction and other human activities	(a) Formation of an oversteep cut slope. (b) Excessive surcharging of the slope. (c) Excessive vibration, e.g. due to blasting. (d) Uncontrolled discharge of surface water.	Not a common trigger since introduction of territory-wide geotechnical control.
Deterioration or degradation due to exposure to weather and other natural effects	(a) Rock falls and overturning of trees associated with deterioration and erosion of slope surface materials, opening of relict discontinuities, deleterious effects of unplanned vegetation and wind effects, etc.	Occasional trigger for slopes with inadequate surface protection and maintenance.

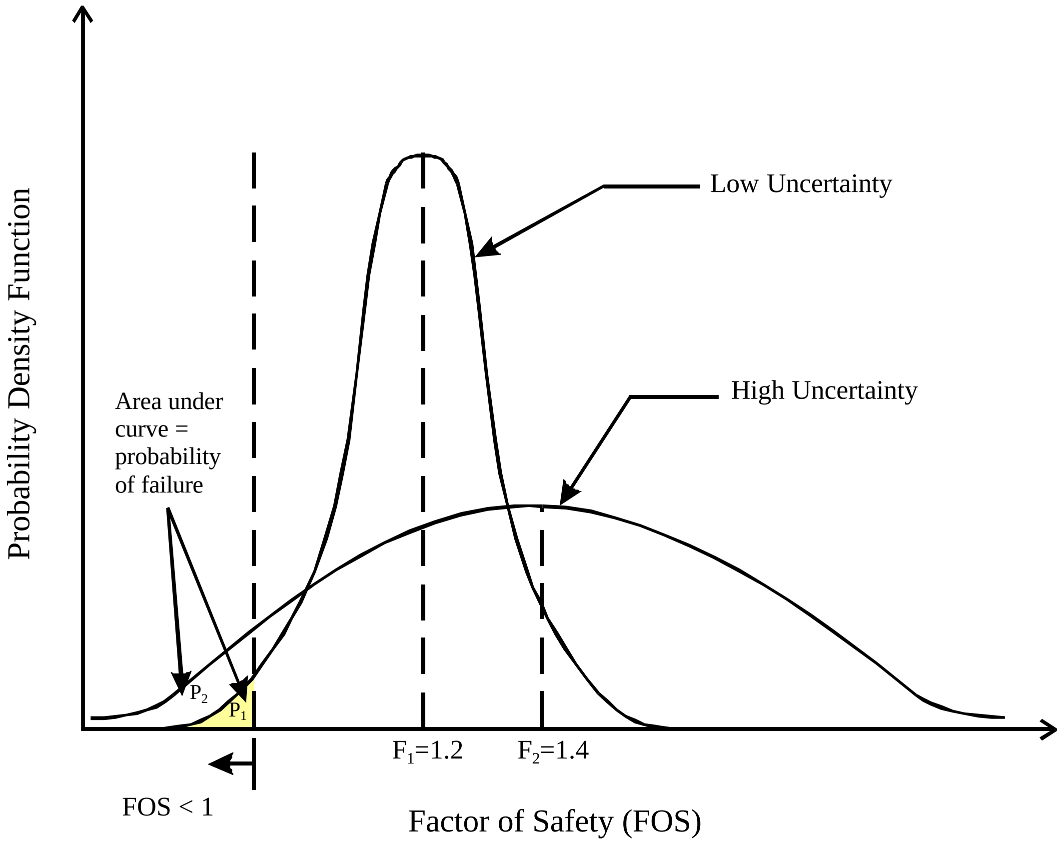
Table 2 - Some Notable Generic Factors Contributing to Failures of Engineered Slopes

Generic Factors		Relative Degree of Occurrence	Examples
Inherent adverse geological weaknesses and unfavourable hydrogeological regime	Adverse geological materials, e.g. intensively kaolinised granites and volcanics, weathered dykes, sedimentary layers within volcanic formations, etc.	NC	Hudson & Hencher (1984); Hencher & Martin (1984); Au (1986); Chan et al (1996)
	Adverse geological discontinuities, e.g. adversely-orientated, extensive and persistent, clay- or silt-infilled discontinuities, pre-existing shear surfaces or zones, well-developed discontinuities that are slickensided or heavily coated with minerals or kaolinite.	NC	Siu & Premchitt (1990); Hencher & McNicholl (1995); GEO (1996a)
	Hydrogeological setting favourable to development of perched water level, e.g. a surface layer of loose colluvium or fill overlying weathered rock	C	Pun & Li (1993); Hencher & McNicholl (1995); Wong & Ho (1995)
	Hydrogeological setting favourable to development of a high base groundwater table	NC	Wong & Ho (1995)
Inadequate design and construction practice	Inadequate slope surface drainage provisions and poor detailing, e.g. inadequately designed surface drainage channels that are vulnerable to overspill during heavy rain	VC	Au & Suen (1996)
	Inappropriate stability analyses that did not consider the critical cross-section or critical slip surface	NC	HAP (1998)
	Inadequate slope surface protection provisions and poor detailing, e.g. wetting of the slope and build-up of water pressures behind impermeable slope surface covers due to inadequate drainage	C	Wong & Ho (1995)
	Inadequate design review during construction	C	HAO (1998); FSWJV (1999)
Adverse topography	Topographical setting susceptible to concentrated discharge or ingress of surface water, e.g. slopes situated below a low point of a crest platform or below a road bend from which surface water may overflow	C	Au & Suen (1996)
Inadequate slope maintenance	Inadequate slope maintenance, e.g. unplanned-vegetation, cracked surface cover, blocked surface drainage channels and weepholes, etc.	VC	Malone & Chan (1996); Wong & Ho (1995)
Inadequate maintenance of water-carrying services	Leakage from poorly-sited or defective water-carrying services/reservoirs	C	GEO (1994); FMSWJV (2001); HCL (2002)
Adverse combination of circumstances	Knock-on effects	NC	Chan & Pun (1992); GEO (1996b); Wong & Ho (1995)

Legend:
VC Very common C Common NC Not common

Note: The examples given are those when the individual factors have played a part in the failure case histories. It should be noted that many failures are due to a combination of factors.

LIST OF FIGURES



Note: $P_2 > P_1$ although $F_2 > F_1$, i.e. a higher factor of safety does not necessarily correspond to a lower probability of failure because the failure probability will also be a function of the degree of uncertainty in the parameters.

Figure 1 - Relationship between Probability of Failure and Factor of Safety (after Lacasse & Nadim, 1998)

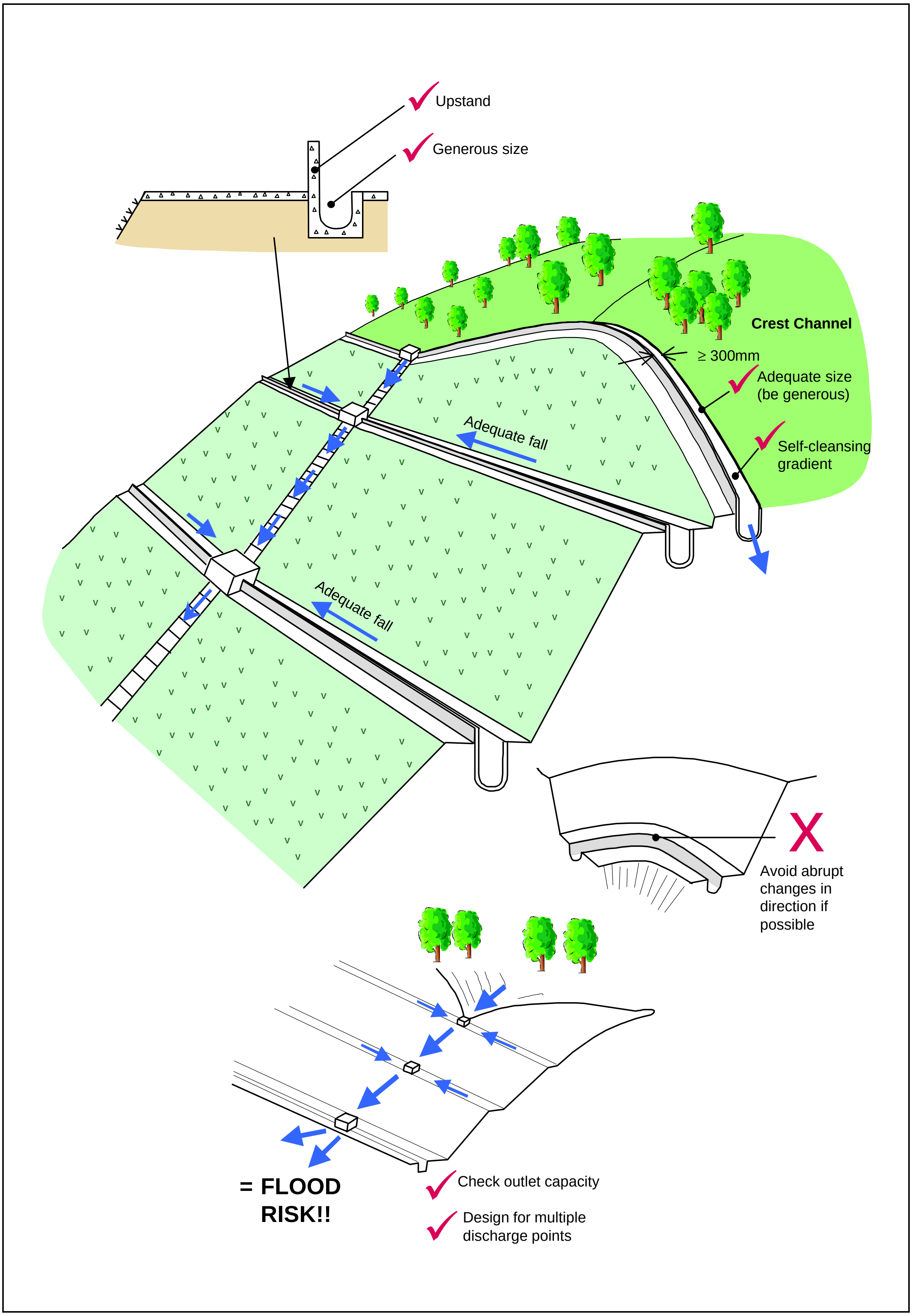


Figure 2 - Surface Drainage Detailing Considerations

LIST OF PLATES

Note: The landslide (200 m^3) affected a playground.

Plate 1 - The 26 September 1993 Landslide at Hiu Ming Street

Note: The landslide (500 m^3) completely blocked two traffic lanes of New Clear Water Bay Road.

Plate 2 - The 25 July 1994 Landslide at New Clear Water Bay Road

Note: The landslide (14 000 m^3) resulted in one fatality.

Plate 3 - The 13 August 1995 Landslide at Fei Tsui Road

Note: The landslide (80 m^3) affected a residential building.

Plate 4 - The 4 June 1997 Landslide at Chung Shan Terrace, Lai Chi Kok

Note: The landslide (30 m^3) affected the northbound KCRC railway track and caused disruption to the rail service.

Plate 5 - The 2 July 1997 Landslide at Kowloon-Canton Railway Corporation (KCRC) University Station

Note: The landslide (250 m^3) completely blocked a two-lane road.

Plate 6 - The 3 July 1997 Landslide at Hong Tsuen Road, Sai Kung

Note: The landslide (3 200 m^3) trapped a car and resulted in significant disruption to the traffic along Ching Cheung Road.

Plate 7 - The 3 August 1997 Landslide at Ching Cheung Road

Note: The landslide (200 m^3) blocked an access road to several private residential blocks.

Plate 8 - The 9 June 1998 Landslide at Sunny Villa, Castle Peak Road

Note: The landslide (1 000 m^3) completely blocked a two-lane road and trapped a taxi and a lorry.

Plate 9 - The 24 August 1999 Landslide at Tsing Yi Road, Tsing Yi Island

Note: The landslide (65 m^3) severed a footpath at the crest of the slope.

Plate 10 - The 24 August 1999 Landslide at Waterfall Bay Park, Wah Fu Estate