SAFE AND GREEN SLOPES
THE HOLISTIC HONG KONG APPROACH

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Abstract : Most landslides in Hong Kong are caused by heavy rainfall. In the 1970s, fatal landslide incidents brought huge loss of life and damage to property in Hong Kong. The catastrophes prompted the setting up of a Slope Safety System by the Government since 1977. The administrative framework has been continuously built up in the last 28 years. During this period of time, the Government in partnership with the profession and the community, progressively and systematically reduced the landslide risk. Increased knowledge on landslides, and application of geotechnical and information technology have brought improvement to the Slope Safety System. The improvement has taken place in many areas of slope engineering and in the administrative framework such as establishing standards, use of information technology, natural terrain landslide risk management, upgrading and maintenance of slopes. In parallel, public education campaigns have been launched to improve public awareness on slope safety and slope maintenance. In recent years, in addition to safety, great effort is devoted to enhance the appearance of slopes. The Government has pioneered vegetation trials and applied the findings in the slope works. The objective is to improve the aesthetics as well as ecology of the slopes and their surroundings. Standards and guidelines on landscape treatment and bio-engineering for man-made slopes have been prepared for the local practitioners in the design and selection of suitable vegetation species on slopes. Effort has also been devoted to encourage private slope owners to provide landscape treatment to their slopes. It is now generally recognised by the Hong Kong community that the appearance of the slopes has been greatly improved. Sustained effort by the Government and private slope owners needs to continue in reducing landslide risk and enhancing the aesthetics of slopes in Hong Kong.

INTRODUCTION

Hong Kong has a population of about 6.8 million within a land area of around 1,097 square kilometres. There is a great concentration of developments on steep hillsides, resulting in the formation of many man-made slopes and retaining walls. The seasonal rainfall is high, which often triggers landslides. The challenges Hong Kong engineers are facing with regard to slope safety are unique in the world.

In the 1970s, Hong Kong experienced a number of disastrous landslides (Figures 1 to 3), notable ones being the 1972 Po Shan Road landslide and the 1972 and 1976 Sau Mau Ping fill slope failures. In the 1980s, the landslide problems were mainly associated with un-engineered squatter developments on the hillside. In the 1990s, several serious fatal landslides aroused much public attention, e.g. the 1994 Kwun Lung Lau retaining wall collapse (Figure 4), the 1995 Fei Tsui Road landslide (Figure 5) and Shum Wan Road landslide (Figure 6) and the 1999 Sham Tseng San Tsuen debris flow (Figure 7) and Shek

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Kip Mei landslide (Figure 8). Although serious landslides still occurred from time to time, the scale and severity of the landslide problems have been decreasing. Landslide fatalities have been significantly reduced in recent years (Figure 9). This is attributed to the development of a comprehensive Slope Safety System (Chan, 2000) and advancement in geotechnology and practice (Chan, 2003).

Figure 1 - 1972 Po Shan Road Landslide

Figure 2 - 1972 Sau Mau Ping landslide

Figure 3 - 1976 Sau Mau Ping landslide

Figure 4 - 1994 Kwun Lung Lau landslide

Figure 5 – 1995 Fei Tsui Road landslide

Figure 6 - 1995 Shum Wan Road landslide

Figure 7 - 1999 Sham Tseng San Tsuen debris flow

Figure 8 - 1999 Shek Kip Mei landslide

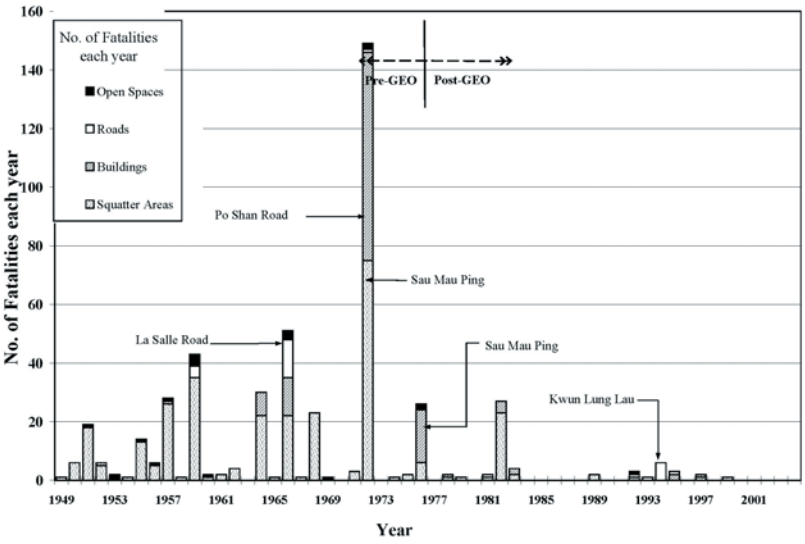

Figure 9 - Known landslide fatalities in Hong Kong

Over the years, the appearance of man-made slopes and retaining walls has been much improved through the concerted effort of the HKSAR Government and all the related professions. In the past, vegetation covers were provided only to the gentle slopes. Improvement in detailing of surface covers, involving the use of erosion control mats and steel wire mesh in soil nailed slopes, has allowed the provision of vegetation covers to slopes with steep gradients. Many more slopes can be vegetated. The proportion of vegetated surface covers on soil slopes upgraded under the Government's Landslip Preventive Measures (LPM) Programme has increased from around 50% in the past to over 70% now (Figure 10).

When the slopes cannot be vegetated for any reason and hard surface cover has to be used, hard treatment measures are provided to alleviate the visual impact of the surface protection cover. The Government has been applying findings of vegetation trials and

performance assessments to improve the ecology of slopes. Technical guidelines are promulgated to the professionals and private slope owners for their use in enhancing slope appearance.

Figure 10 – Examples of vegetated slopes and an upgraded masonry wall with wall trees preserved

HONG KONG SLOPE SAFETY SYSTEM

Many slope-safety related functions are implemented by various departments of the Government. Over the past decade, significant advances have taken place on system improvement through the formulation of a framework of the Slope Safety System by the Geotechnical Engineering Office (GEO) of Civil Engineering & Development Department (CEDD). The framework puts together the various functions in a systematic manner. It helps conveying clear messages to the administrators, the general public, geotechnical practitioners and other stakeholders what the functions are.

The Slope Safety System includes a number of key areas of work: formulating slope safety standards and regulatory framework, checking new geotechnical works, upgrading old substandard slopes, ensuring that slopes are regularly maintained, promoting public awareness and response in slope safety through public education, publicity, information services and public warnings, squatter inspection and clearance, and enhancing the appearance and aesthetics of engineered slopes.

Development of the Slope Safety System has been very rapid in recent years. A comprehensive new Slope Catalogue was completed in 1998. It now contains records of some 57,000 man-made slopes and retaining walls. The Slope Catalogue facilitates the planning of the slope upgrading and maintenance programmes by the provision of technical

data. The Lands Department has also established the Slope Maintenance Responsibility Information System (SMRIS) which provides the information on the parties responsible for the maintenance of each slope in the Slope Catalogue. The importance of promoting public awareness and response in slope safety and enhancing the appearance of engineered slopes is recognised, and these are now the key components of the Slope Safety System. The Slope Safety System is reviewed regularly and benchmarked internationally through a Slope Safety Technical Review Board. In order to continuously reduce landslide risk in Hong Kong, the tasks undertaken by the Government include the following:

I. Ensure That New Slopes Meet Safety Standards

(1) Improve slope safety standards, administrative and regulatory frameworks

Continuous improvement in technical standards and geotechnical control is needed to meet the changing environment and the changing public expectations on slope safety. The publication of the first edition of the Geotechnical Manual for Slopes (GCO, 1979) in 1979 and the second edition (GCO, 1984a) in 1984 marked the first step of Government to provide guidance for the standards of practice that should be adopted for the design, construction and maintenance of slopes and site formation works in Hong Kong. The Manual covers a wide range of geotechnical topics and each topic is later given full coverage in a series of Geoguides and GEO Publications, e.g. Geoguide 1 – Guide to Retaining Wall Design (GCO, 1982) and (GEO, 1993) and Geospec 1 – Model Specification for Prestressed Ground Anchors (GCO, 1984b & 1989). The main objective of publishing the Manual, Geoguides and Geospecs is to allow the profession to use a series of common, up-to-date and comprehensive geotechnical standards which are appropriate to Hong Kong conditions.

Other related technical publications include GEO Publications, GEO Reports on research and development work in the GEO, Geological Memoirs and reports on systematic investigation of major landslides in Hong Kong. Through these publications, the GEO is able to share experience, results of research and development and lessons learnt from landslides with the profession to enhance the level of geotechnical competence in Hong Kong (Chan, 1999).

Systematic investigation of landslides has brought improved understanding of the mechanism of landsliding. Areas for improvement have been identified. Enhancements have been introduced in many aspects of slope engineering and the administrative framework, including :

- Improved reliability of engineered slopes, through: adoption of more robust design measures (GEO, 2004a), improved slope detailing and drainage, and enhanced geotechnical and engineering geological input for slopes with complex ground conditions and hydrogeology.
- Improved slope safety risk management system, through: identification of slopes that warrant priority follow-up action, contribution to enhancement of slope maintenance standards via an improved Engineer Inspection process, identification of system improvement measures based on holistic reviews of slope problems, and contribution to development of natural terrain risk management strategy and approach to risk assessment and risk mitigation based on an improved understanding of natural terrain landslides.

- Enhanced public safety, through taking out-of-turn, integrated follow-up actions on about 100 slopes (and their adjoining areas as appropriate) based on the findings of landslide studies.

Geotechnical control on building and infrastructure developments also plays a key role in enhancing slope safety (Chan, 1997). The GEO has worked closely with the Buildings Department and the profession to regularly review the geotechnical control strategy and implement identified enhancement measures, such as the review of geotechnical control as part of the Slope Safety Review in 1995 (Works Branch, 1995) and the follow-up comprehensive review of statutory geotechnical controls in private slopes and developments completed at the end of 2000. Various amendments to the Buildings Ordinance have been made in the past two decades to enhance the geotechnical control of private building developments, with the key milestone of enacting the law in 2004 a register of Registered Geotechnical Engineer (RGE).

In the past, the Buildings Ordinance (BO) only required the appointment of an authorized person (AP) as the coordinator of building works and a registered structural engineer (RSE) for the structural elements of the building works. As such, the AP had to be held responsible under the law for geotechnical works although geotechnical engineers were usually engaged as sub-consultant to the AP for the design and supervision of the geotechnical works. In the last several years, the industry was consulted for the setting up of a statutory register of geotechnical engineers to undertake geotechnical engineering duties relating to private building works. As a result, the Legislative Council passed the law on 23 June 2004 to set up a register of RGE.

The Buildings (Amendment) Ordinance 2004 establishes a register of geotechnical engineers and introduces the requirement for the appointment of a RGE for the geotechnical elements of building works. The registration system has commenced operation since 31 December 2004, and the remaining provisions pertaining to RGE will commence operation 1 year later, i.e. on 31 December 2005. With effect from 31 December 2005, a RGE is required to be appointed for the geotechnical elements of building works as stipulated in section 4(1)(c) of the BO. The RGE will take up the statutory responsibility for the geotechnical elements. At that point of time, the respective responsibilities of AP, RSE and RGE in building works under the BO are mainly specified in Building (Administration) Regulation 12 with reference to the prescribed plans and documents they are required to prepare and sign.

The duties and functions of a RGE under the BO are classified into “executive role” and “advisory role” as appropriate in respect of the geotechnical elements of the building works (BD, 2004). The geotechnical tasks under the Buildings Ordinance and Regulations to be undertaken by the RGE are summarized in Table 1.

For the executive tasks, the RGE needs to prepare and sign the prescribed geotechnical plans and reports, and shall also be required to certify that the works have been carried out in accordance with the approved plans and that the works completed are geotechnically safe. For the advisory tasks, the RGE should prepare and sign the geotechnical report/supporting documentation. The signature is deemed to indicate the RGE’s acceptance of responsibility for the geotechnical report/supporting documentation.

Table 1: Geotechnical Tasks under the Buildings Ordinance and Regulations to be Undertaken by the RGE

Executive Role – Geotechnical Tasks
Ground Investigation in Scheduled Areas
Ground Investigation in Non-scheduled Areas
Site Formation
Groundwater Drainage Works in Scheduled Area 1
Water Supply and Wells
Remedial Works to Dangerous Hillsides
Long Term Monitoring (Post Occupation Permit)
Advisory Role – Geotechnical Tasks
Demolition Affecting Slopes and Retaining Walls
Geotechnical Assessment for General Building Plan where required
Foundation in Scheduled Areas Nos. 1, 2 & 4 and Designated Area of Northshore Lantau
Foundation Affecting Slope and Retaining Wall
Excavation and Lateral Support
Superstructure (Report on geotechnical design parameters)

(2) Check new slopes

With the intense building and infrastructure developments in Hong Kong, hundreds of new man-made slopes are formed each year. If uncontrolled, these new slopes could be substandard and would increase the landslide risk in Hong Kong. The GEO has been maintaining a dedicated team of professional geotechnical engineers with technical support staff to check the adequacy of all slope works, site formation works, earth retaining structures and deep excavations that are designed and constructed by the private sector, public authorities and government departments. The GEO has obtained certification to the international quality assurance standard of ISO 9001 and is maintaining this standard through regular internal and external audits and continuous quality improvements.

Various initiatives have been introduced to improve the performance of the quality-assured checking system. The initiatives include – increase the number of inspections of active construction sites to improve the standard of supervision of geotechnical works, and set up a comprehensive computerized district information system to enhance the effectiveness of checking of new slopes.

(3) Enhance land use planning

In addition to geotechnical checking, the GEO also provides input at the land use planning stage by advising government town planners, land administrators and project departments on land development proposals, land use zoning and special geotechnical conditions on land allocations. The purpose is to mitigate landslide risk and facilitating safe and economic developments at the earliest possible stage.

II. Enhance the Stability of Old Man-made Slopes

(1) Upgrade old man-made slopes

Before the GEO was established in 1977, there was very limited geotechnical control of

slope formation both in the private and public sectors. Hence, the safety standards of many old man-made slopes are in doubt. About 10,500 of the 39,000 old government man-made slopes affect developments and major roads. The Government has therefore been maintaining an on-going LPM Programme to systematically rectify substandard slopes so as to progressively reduce the risk from man-made slopes which affect the community directly (Figure 11).

Figure 11 – Recompaction of granular fill in upgrading a loose fill slope

Figure 12 illustrates the proportion of risk posed from old man-made slopes in terms of facility groups (Wong & Ho, 1998). About half of the risk derives from 10% of the slope population, i.e. from those affecting buildings and major roads (facility group 1). In order to ensure that the 'right' slopes are selected, a risk-based priority classification system (Wong, 1998) utilising QRA results has been developed. The Slope Catalogue provides the necessary information for the systematic selection of substandard slopes to be rectified. The annual provision of funds under the LPM Programme for rectifying substandard slopes has increased from $107 million in 1994-95 to some $895 million in 2005-06. The total capital expenditure on the programme in the five years starting from 1994-95 to 1999-2000 amounts to $2.9 billion. The funds committed in the extended 10-year programme from 2000 to 2010 will incur capital cost of about $9 billion at March 1999 price.

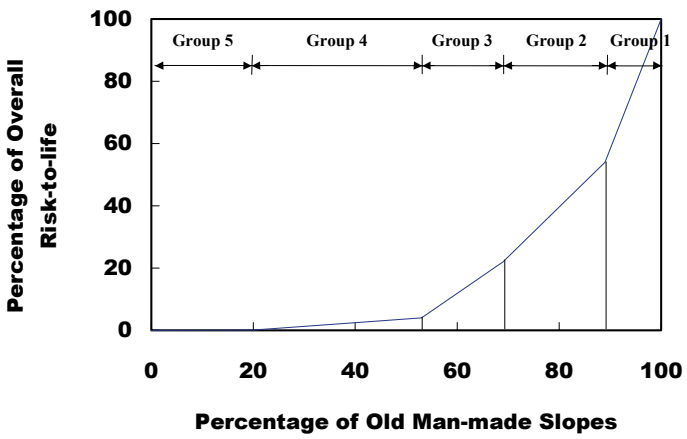

Figure 12 – Risk profile of old man-made slopes

For slopes which could affect or be affected by road and development projects, project departments will, as far as possible, adopt an integrated approach to upgrade these slopes to current safety standards as part of the implementation of the projects.

For those smaller slopes not covered by the LPM Programme, maintenance departments (e.g. Highways Department for roadside slopes, Housing Department for public housing estates, Water Supplies Department for catchwater and reservoir slopes, etc.) are taking an active role in reducing the landslide risk by improving these slopes through the use of prescriptive measures or enhanced maintenance.

(2) Promote slope maintenance

Regular maintenance is essential to the continued stability of all man-made slopes. To facilitate maintenance of man-made slopes, the GEO has published the standard of good practice for slope maintenance – Geoguide 5 in 1995, with the third edition published in 2003 (GEO, 2003). Geoguide 5 has been adopted as the minimum standard for maintenance of government slopes. The maintenance responsibility of man-made slopes in the Slope Catalogue has also been identified under the $73.6 million project – Systematic Identification of Maintenance Responsibility of Slopes (SIMAR) which was substantially complete in December 1999. There are now some 57,000 sizable man-made slopes in the Slope Catalogue. For each of the 39,000 government slopes in the Slope Catalogue, the departments responsible for maintenance are also identified. Maintenance departments have actively arranged Engineer Inspection of their slopes so that each slope has received an Engineer Inspection (EI) (comprehensive inspection by professional engineer) by September 2002. Currently some $650 million is spent on slope maintenance a year by the maintenance departments.

Government's policy is for slope owners to take care of their own slopes, i.e. Government maintains government slopes and private owners take care of private slopes. It is therefore important that private owners maintain private slopes and upgrade those which are substandard. Government, as a controlling authority on slope safety, also carries out safety-screening of private man-made slopes to establish prima facie evidence for serving Dangerous Hillside Orders (DHO) to private owners under the provisions of the Buildings Ordinance. Private owners are statutorily required to investigate and carry out necessary slope upgrading works for discharging the DHO. Similar statutory actions are also taken to require owners to repair their underground drains and water pipes which may affect the stability of adjacent slopes.

Under the present system, maintenance of private slopes requires voluntary actions from private owners. The GEO has completed a review of private slope maintenance and it has been recommended that the Government to encourage and facilitate owners to take up their responsibility for slope safety through public education, publicity and information services. Mandatory slope maintenance should be taken as a last resort.

(3) Provide information services

Although private owners are becoming more aware of their slope maintenance responsibilities, but as laymen, many of them may not possess the required information, knowledge and expertise in slope improvement or maintenance works. The GEO has

therefore provided the public with free comprehensive information on slopes through a computerized Slope Information System which contains information of some 57,000 man-made slopes in the Slope Catalogue. The information is also available on the Internet in the Hong Kong Slope Safety Website (http://hkss.cedd.gov.hk). Parties responsible for maintenance of each slope in the Slope Catalogue are also available from SMRIS in the Website of the Lands Department (http://www.slope.landsd.gov.hk/smris/).

III. Mitigate Natural Terrain Landslide Risks

About 60% of the land area of Hong Kong has not been significantly modified by human activities and can be referred to as 'natural' (Figure 13). Much of this natural terrain is steeply sloping and is covered by a mantle of weak saprolitic and residual soils derived from many thousands of years of weathering, as well as colluvial soils, which are the product of past landslides and erosion processes. So far, the present slope safety plan has focused primarily on substandard high-priority man-made slopes, which in the past have posed a relatively high landslide risk to the community in Hong Kong. As a result, there has been a significant reduction in landslide risk, as the high-priority man-made slopes are systematically dealt with as described above. Meanwhile, landslide risk from natural hillsides is on the rise due to encroachment of more developments closer to steep hillsides.

During the wet season, these slopes are subject to frequent intense rainstorms, so landsliding on natural terrain is a common occurrence (Figure 14). In the past, such landsliding has not been of great consequence, as it has occurred in relatively remote areas and hence has not posed a significant threat to life or property. Also, unlike man-made slopes, there have been no recorded fatalities in planned developments (i.e. excluding squatter areas) resulting from landsliding on natural terrain. In contrast, over the past 50 years, more than 470 people have died as a result of failures of man-made slopes and retaining walls. However, development has been continuing to extend into areas close to steep natural terrains, thereby increasing the risk (Figure 15). Indeed, there have been some 'near miss' cases involving landslides on natural terrain in which the consequences could have been very serious if the existing or planned developments below had been in use or occupied at the time when the landslides occurred (Chan, 1998).

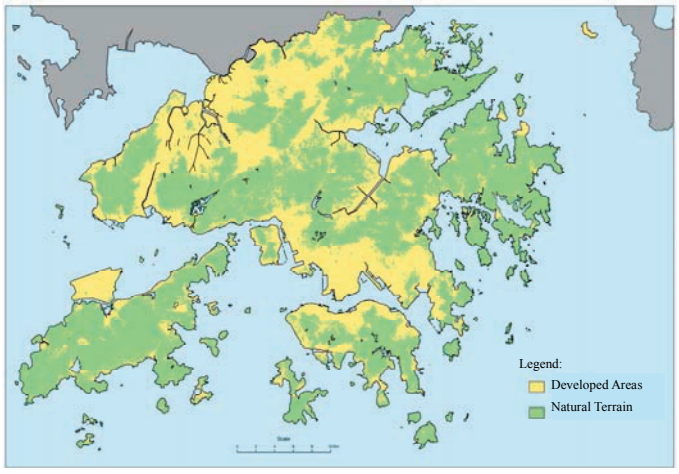

Figure 13 – About 60% of the land area in Hong Kong is "natural terrain"

Figure 14 – The Tsing Shan debris flow in 2000

Figure 15 – A 20 m^3 landslide in 1998 resulted in damage to property

To prepare for the potential risk posed by natural terrain, many research and development activities have been carried out in Hong Kong since early 1990s. Amongst these, detailed study of natural terrain landslides, such as the landslides on the natural terrain of Lantau Island due to the heavy rainstorm in November 1993 (Wong & Lam, 1997), greatly enhanced the understanding on the type, mechanism and mobility of natural terrain landslides. A comprehensive inventory of landslides on natural terrain has been set up. The inventory, which is called the Natural Terrain Landslide Inventory (NTLI), contains data on the location of the landslide crowns, the path of the resulting debris trail, the approximate date of the landslide, the width of the landslide scar and the vegetation cover. Up to the year 2000, the NTLI contains some 30,000 landslides, about 19,000 and 11,000 of which are considered recent and relict landslides respectively. Recent landslides in the NTLI refer to those that occurred within the period of comprehensive aerial photograph coverage, typically after 1950s.

The Government adopts the following principles in dealing with the natural terrain landslide hazards:

- For new developments, contain the increase in overall risk through studying the natural terrain hazards and undertaking any necessary mitigation actions where such hazards could occur. Avoid new developments on sites that are subject to severe natural terrain hazards.
- For existing developments, undertake mitigation actions urgently where there exists an immediate and obvious danger. Study natural terrain hazards where there is reason to believe that a dangerous situation could develop, such as persistent landslides affecting an existing development, and undertake mitigation actions when considered necessary.

As not all development sites are affected by natural terrain hazards, Ng et al (2003) established a set of simple guidelines on the basis of the runout distance of landslide debris and the likely consequence to assist planners, land administrators, project managers, etc. in identifying whether a site may be subject to natural terrain hazards. Where a site may be affected by natural terrain hazards, guidelines are also available for deciding whether it is more appropriate to consider relocation of the site or to undertake a natural terrain hazard study (NTHS) to identify any mitigation measures required (Chan, 2003). Ng et al (2003) also developed guidelines on how to conduct an NTHS. The guidelines highlights the

merits and limitations of and makes recommendations for three different approaches for the evaluation of natural terrain hazards, viz., factor of safety, QRA and design event (Chan, 2003).

Apart from hazard assessment, there has been substantial advancement in the design of mitigation measures. Lo (2000) reviewed typical mitigation measures applied in Hong Kong and established guidelines on the design of debris-resisting barrier. Various kinds of mitigation measures including insitu stabilisation, boulder fences, check dams, landslide debris barriers, etc (Figure 16 and 17) have recently been used in Hong Kong and the experience gained has provided much better understanding of the effectiveness and practicality of these measures (Tse et al, 2003). Other measures like bio-engineering are being tried (Campbell et al, 2005).

Over the years, significant advances have been made in information and related digital technologies. The technologies have become more readily accessible with improved capability and reduced cost (Wong et al, 2005). The GEO plays a key role in a number of areas of geo-informatics, including compilation and dissemination of geo-informatics datasets such as geological maps, Slope Catalogue, landslide data, ortho-rectified historical aerial photographs and digital elevation model; application of geo-informatics involving Geographic Information System (GIS), remote sensing and engineering analytical techniques; and development and promulgation of application of geo-informatics. In the last few years, development and applications of GIS expanded the capability of the GEO in natural terrain landslide management. The advances include advanced GIS search, browsing, editing and publication; GIS analysis; GIS modeling; mobile-location-based application; 3-D visualization and virtual reality application; and integration with remote sensing (Figure 18 and 19).

Figure 16 – Check dam built in Sham Tseng San Tsuen

Figure 17 – Boulder fence at Mid-levels

(a) Uploading GIS data to the mobile mapping system

(b) On-site navigation, data retrieval and mapping using the mobile mapping system

(c) Orthoimage, topographical map, ground features and GPS location displayed on a pocket computer

Figure 18 –Geotechnical mobile GIS mapping system

Figure 19 – 3-D virtual reality model of Tsing Shan Foothills in Hong Kong

With the above advances, it can be more systematic, consistent and rational in assessing and dealing with the rising landslide risk from natural hillsides.

IV. Minimize Damage Caused by Landslides

(1) Develop a Landslip Warning System

Most landslides in Hong Kong are caused by heavy rainfall. By using a combination of real-time rainfall data and rainfall forecasts from the Hong Kong Observatory (HKO), the Government is able to issue Landslip Warning through the media. The purpose of the Landslip Warning is to alert the public to reduce their exposure to possible danger from landslides. The issuing of the Landslip Warning also triggers an emergency system within government departments that mobilizes staff and resources to deal with the aftermaths of landslide incidents. The Landslip Warning is issued when it is predicted that numerous (more than about fifteen) landslides will occur.

The GEO operates an extensive network of automatic raingauges (Figure 20) providing real-time rainfall data to the Landslip Warning System. This network, which dates back to 1984, was upgraded in 1999 to improve coverage in the new development areas. The network now comprises 86 raingauges located throughout Hong Kong.

The data capture, control and processing system, which was also upgraded in 1999, receives data from the 86 GEO raingauges and from an additional 24 automatic raingauges operated by the HKO (Figure 21). The system is now being upgraded with General Packet Radio Services (GPRS) for data transmission, replacing the telephone lines for the raingauges. At the completion of the upgrade with GPRS, data transmission can be enhanced and can become more cost-effective.

Figure 20 – Automatic raingauge

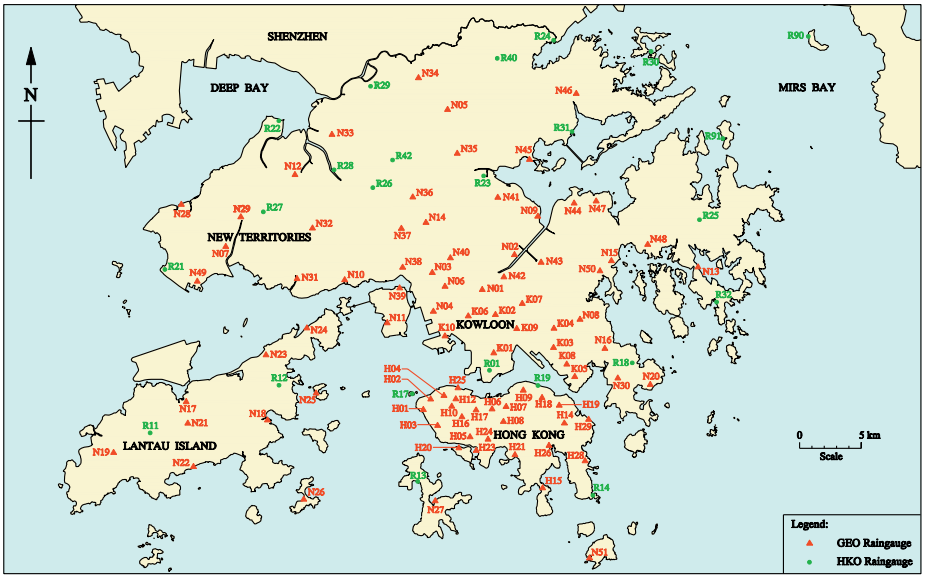

Figure 21 – Locations of the automatic raingauges in Hong Kong

The number of landslides in recent years has been low. The public perceives that as an indication of the slope safety problem in Hong Kong having been solved, when in fact the lower landslide activity could have partly been the result of less severe rainstorms in recent years. The GEO developed an index called the Landslide Potential (rainstorm) Index (LPI) in 2002 to measure the relative severity of a rainstorm with respect to its potential to cause landslides. The landslide potential in Hong Kong is a function of four factors namely: the intensity of rainfall; the size of the area affected by rainfall; the number of slopes in the rainfall area and whether the slopes affected can remain stable under such rainfall. The index can be used to communicate the severity of rainstorms in causing landslides to the public in order to raise their awareness. The concept of LPI can also be used to normalise the effect of rainfall in evaluating the performance of slopes. Evaluation of rainfall severity using this index was tried out in year 2003 and 2004 for internal reference (Figure 22).

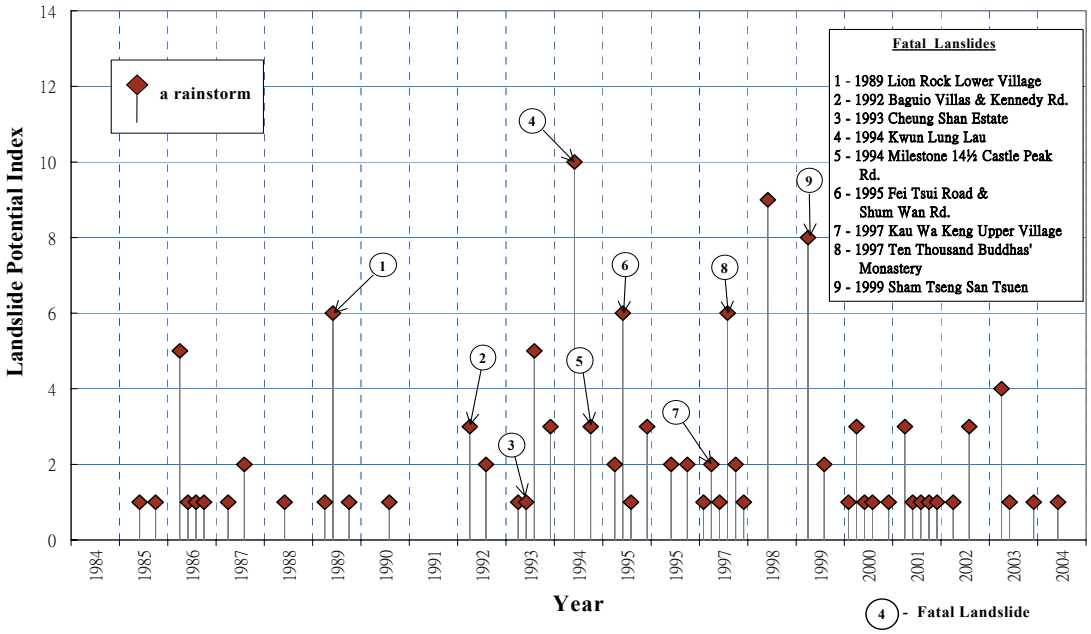

Figure 22 – Chart showing rainstorms since 1984 that resulted in the issue of Landslip Warning

(2) Provide emergency service

Most of the landslides in Hong Kong occur in the rainy season between May and September each year. There are 300 landslides on average reported to the GEO each year. The GEO maintains a 24-hour, year-round service to provide geotechnical advice to government departments on actions to be taken in case of danger arising from landslides. The service includes advice on closure of roads, evacuation of buildings and urgent slope repair works. The primary objective of the emergency service is to protect the general public from landslide hazards and to assist government departments to restore as soon as possible services to the public which have been disrupted by landslides. Where a high demand for emergency service is anticipated, such as when a Landslip Warning is in force and/or tropical cyclone signal number 8 or above is hoisted, the Emergency Control Centre (ECC) in the Civil Engineering and Development Building at Homantin is manned by 13 emergency teams on a rotational basis, working in approximately 8-hour shifts. The team controller of the ECC (a senior geotechnical engineer) receives landslide reports and dispatches geotechnical engineers promptly to landslide sites to provide necessary advice to other government departments. More senior GEO staff are available on call to give advice to geotechnical engineers in dealing with difficult or serious cases. A Crisis Management Planning Committee has been set up since 2004 to oversee the planning and preparedness for crises, including landslide crisis, and the improvements needed to the GEO Emergency System and Crisis Management System.

The GEO makes use of the advances in information technology and has enhanced the uploading of landslide information reported to the GEO in the Intranet. The system, called Integrated Landslip Information System (ILIS) has been successfully implemented. Staff can access details of each landslide reported to the GEO in the ILIS from their own computers.

(3) Improve the safety of squatters

A large proportion of landslide fatalities took place in squatters (Figure 23). Special non-development clearance (NDC) studies have been carried out by the GEO since 1982 to identify squatter structures which are especially vulnerable to landslide danger. Once squatter structures are identified as especially vulnerable to such danger, the Government will offer rehousing to the affected squatters. Since the mid 1980s, some 75,000 persons living in tolerated squatter structures on steep hillsides have been rehoused and the affected structures cleared. Some 58,000 of those cleared to date have been from the urban areas of Hong Kong and Kowloon. The GEO has also been carrying out inspection of tolerated squatter structures in the New Territories since the mid-1980s. Up to the end of 2003, inspections have been made of about 400 squatter areas and their nearby isolated structures, covering the more vulnerable parts of the squatter areas in the New Territories, and including dwellings adjacent to stream courses that might be subject to hazard from debris flows. This NDC inspection programme was completed in September 2003. From 1992 to September 2003, the GEO has recommended clearance for some 20,000 persons. Housing Department, Lands Department and Buildings Department are implementing the clearance recommendations. About 9,800 of these persons are yet to be rehoused. The GEO issues warning letters to squatters who have been recommended for clearance on slope safety grounds. Since 1998, the GEO has also been erecting warning signs in squatter areas where clearance recommendations are being processed. Squatters are advised to follow the safety advice given pending action for rehousing. When the Landslip Warning

is issued or during periods of heavy rain, temporary shelters are provided by District Offices to accommodate squatters or any members of the public in need.

Figure 23 – Landslides in August 1995 affecting the Lei Yue Mun Squatter Area

(4) Launch public education and publicity on slope safety

Since 1992, the GEO, with the assistance from Information Services Department, has been undertaking a systematic publicity campaign on slope maintenance to convey the following two key messages to the public, i.e. (a) inspection and maintenance are needed to keep slopes safe; and (b) action lies with the owners. To enhance and reinforce private owners' acceptance of their responsibility for slope safety, the GEO has set up a Community Advisory Unit since April 1999 to hold seminars/talks and meet-the-public sessions on slope safety and to meet with Owners' Corporations and individual private owners who have received Dangerous Hillside Orders. The feedbacks have been positive.

With the assistance of the universities and research organisations, the GEO has been conducting annual public opinion surveys to gauge the effectiveness of the campaign. The survey results in 2002 indicated that, 88% of the public were aware of the importance of slope maintenance and the owners' obligation to maintain their slopes. Recent results of the survey in 2003 indicated that there was a 10% drop to 78%, and the percentage rose slightly in 2004 to 81%. Regarding the public's concern on slope safety, the percentages dropped from 73% in 2002 to 63% in 2003, and further reduced to 59% in 2004. The reduction in awareness is likely due to an absence of serious landslides in recent years, leading to the general complacency on landslide risk.

The GEO commenced in 1996 an ongoing public education campaign on slope safety including personal precautionary measures to be taken in times of heavy rain. School education and student participation have been identified as important areas for effective promotion of slope safety. In conjunction with the Chinese University Press, the GEO prepared an education toolkit on slope safety for use by schools when teaching the subject on landslides and slope safety which has been included as part of the secondary school geography curriculum. A major exhibition was held in April 2004 at the Hong Kong Central Library on Hong Kong's history of landslide disasters to arouse the public awareness of landslide risk. To sustain the public education effort to raise awareness, the GEO in collaboration with HKO, Drainage Services Department, Security Bureau, Information

Services Department and Hong Kong Red Cross, launched a major event, "Safer Living – Reducing Natural Disasters" in March 2005. The objectives of the programme are to promote public understanding of the natural hazards, including landslides, facing the community and to raise the community's preparedness for and resilience to natural disasters, leading to safer living in Hong Kong. The whole event will last for about one year. The launching ceremony was held on 5 March 2005 (Figure 24), followed by an exhibition. A series of activities will then be arranged, leading up to a one-month major exhibition at the Hong Kong Science Museum in April 2006. The planned activities include exhibition, lectures, and seminars, slogan and bookmark competition, tropical cyclone name nomination contest, TV documentary series and rescue drill demonstration, etc.

Figure 24 – Launching ceremony by the group of officiating guests and the Campaign Ambassador aiming to raise public awareness on natural disasters

Risk assessment calculations indicate that the overall landslide risk arising from old substandard man-made slopes to the whole community of Hong Kong has been reduced to about 50% of the risk that existed in 1977. The Government is now sustaining its effort to further reduce the landslide risk from old man-made slopes to below 25% of the 1977 level by the year 2010.

ENHANCEMENT OF SLOPE APPEARANCE

Government Policy on Greening and Use of Shotcrete

The Government has committed to making man-made slopes look as natural as possible, blending them with the surroundings and minimising their visual impact. Guidance has been promulgated through technical circulars, e.g. WBTC No. 25/93 (Work Branch, 1993), and Policy Objective booklet on Slope Safety for All from 1998 to 2000 in the Policy Address of the HKSAR Government (Works Bureau, 2000). Wherever possible, vegetation is provided as slope surface cover. The GEO has the target of providing vegetation cover to at least 70% of the slopes upgraded under the LPM Programme. Over the last three years, this target has been achieved (GEO, 2004c). The use of chunam or shotcrete on slopes should only be considered as a last resort and only after other techniques have been explored and found not practical. WBTC No. 17/2000 (Works Bureau, 2000b) and CEDD TC No. 15/2004 (CEDD, 2004) stipulate that consideration may be given to introduce some graphic or wall painting design on slopes where the use of chunam or shotcrete is unavoidable to minimize the visual impact. The use of hard surface cover is controlled through the Departmental Vetting Committee on Slope Appearance for slope works under the LPM Programme.

For slope maintenance works, the maintenance departments aim to progressively replace any existing hard surface covers on Category 3 slopes, if geotechnically acceptable, with a vegetated surface covers. Aesthetic aspects of slope works are included in the scope of slope maintenance audits conducted by the GEO.

Engineers are now more knowledgeable in landscape design concept than before in blending such slopes with the surrounding environment. The commonly used methods are masonry block facing, ribbed or other patterned finishes, toe planters, colouring and planter holes, coupled with suitable retention of existing vegetation (Figure 25). More fancy techniques such as decorative artwork and artificial rock are occasionally used. Figure 26 shows the methods of providing vegetation and landscape treatment for slopes of different gradients.

Figure 25 – A slope with landscaped treatment implemented during upgrading

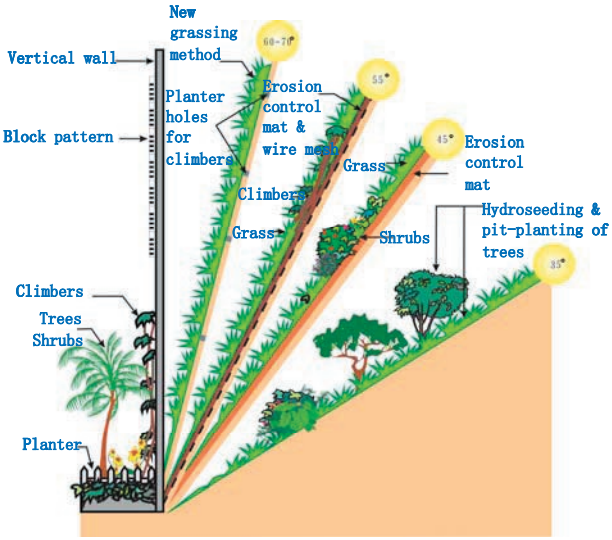

Figure 26 – Common methods of vegetating slopes

Technical Standards/Guidelines and Research

The role of vegetation in slope works was first promoted in 1984 when the technical guidelines on the use of grass, shrubs and trees for slope surface protection were published in the second edition of the Geotechnical Manual for Slopes. In 1999 a set of interim guidelines was stipulated in the revised GEO Report No. 56 (Wong et al, 1999) for prescriptive use of vegetation for soil cut slope up to 55°. In 2000, the GEO Publication No. 1/2000 – Technical Guidelines on Landscape Treatment and Bio-engineering for Man-made Slopes and Retaining Walls (GEO, 2000a) was promulgated, providing comprehensive technical guidance for use by the professionals in the planning, design, construction and maintenance of slope appearance enhancement measures. The publication was a pioneer document of the kind in the world. In recognition of its contribution to enhancement of the built environment, the publication received the Grand Award in the "Outstanding Green Project Awards 2000" organised by Leisure and Cultural Services Department in association with the Hong Kong Institute of Landscape Architects and the Society of Horticulture, Hong Kong. Another publication, Highway Slope Manual (GEO, 2000b) provides guidance on good practice on planning, investigation, design, construction and maintenance of highways slopes, covering both engineering and landscape aspects of slope design. GEO Report No. 116 (Halcrow China Ltd., 2001) presents a review of local

and international practice into reduction of visual impact of slopes and gives administrative and technical recommendations and proposals for further studies. Since 2002, a database called "Greening Methods on Slopes" was set up to consolidate updates of the information on the different new greening techniques that have been tried on government slopes.

On especially steep cut slopes, many new greening systems have been tried out. These systems are mostly proprietary products and generally involve the use of thick soil mulch, soil filled panels anchored onto the slope, planter-grillage system, or fibrous soil. In the past few years, more than twenty new greening systems have been applied to over 200 slopes in Hong Kong.

In the past few years the GEO in collaboration with the Kadoorie Farm and Botanic Garden has pioneered the research and study into the area of landscape treatment and bio-engineering on man-made slopes, including slopes with poor soil nutrition condition. Recently, the GEO has completed a planting trial of the use of native small tree and shrub species on steep slopes (Choi & Chau, 2004). The GEO has also completed a study (Lui & Shiu, 2004) in conjunction with the Department of Ecology & Biodiversity of the University of Hong Kong, on the review of the performance of different greening techniques and identification of vegetation that can successfully establish and self-sustain on steep man-made slopes. Based on the results of the planting trial and the study, guidelines on the selection of vegetation species for use on man-made slopes and suggested improvement to detailing and specification of the greening techniques were promulgated in 2004 under TGN No. 20 (GEO, 2004b). In this Annual Seminar, detailed findings of the planting trial at Yuen Tun (Hau et al, 2005), and the assessment of the growth condition of vegetation on slopes where the greening techniques have been applied, and evaluation of the engineering performance of the techniques (Lui et al, 2005) are described.

As part of the continuous improvement initiatives for achieving sustainability in slope greening works, the GEO is undertaking a further study on the application of various vegetation species on man-made slopes, aiming to expand the range of suitable vegetation species for landscape use in slope works. To promote the use of eco-friendly stabilization measures, the GEO is carrying out trial applications of bio-engineering measures for repairing natural terrain landslide scars, using predominantly native vegetation species. For preserving the appearance of old masonry walls and the wall trees, the GEO has embarked on a study to review the current practice and to identify improved measures in stabilization of old masonry walls.

Yuen Tun Vegetation Trial

To study the performance of vegetation establishment on steep man-made slopes, the GEO carried out a planting trial of eight native small tree and shrub species (Figure 27) at Yuen Tun, Tai Lam Country Park in collaboration with the Kadoorie Farm & Botanic Garden Corporation. Seedlings of 8 native species were planted on slopes with gradient of about 50° following hydroseeding. The trial commenced in July 2001 and monitoring of the vegetation growth has been carried out on a yearly basis.

According to the monitoring results, the survival rate of the plants is over 70% and the seedlings have grown over 100% from its original size. At places with more favourable soil and environmental conditions, some seedlings have become small trees. Among the eight species used in the trial, *Zanthoxylum avicennae* (Prickly Ash, 簕欓), *Schefflera*

heptaphylla (Ivy Tree, 鴨腳木) and *Gordonia axillaris* (Hong Kong Gordonia, 大頭茶) show better performance in terms of growth.

Figure 27 – Native vegetation species in Yuen Tun trial site

One observation of the vegetation trial at Yuen Tun is that the vegetation in the trial site is still mainly comprised of grass. The seedlings plants have not grown to create dense foliage cover to protect the slope surfaces. Although most of the seedlings have flowers and produce fruits, regeneration of the species from seed has not yet been observed. Other naturally recruited vegetations have invaded the trial panels to improve biodiversity of the vegetation system. A natural and sustainable vegetation system that requires minimum long-term maintenance can be anticipated. Based on the findings of the study, a number of recommendations for greening man-made slopes are being developed, with a view to establishing robust, cost-effective, and eco-friendly vegetation covers on man-made slopes.

Ecological Considerations

In addition to ensuring public safety, minimizing disturbance to the community during construction and enhancing slope appearance, the GEO is also very mindful of the ecological impact of slope works. In the past, the number of tree species commonly chosen for planting on man-made slopes is small. These include *Acacia confusa* (台灣相思), *Casuarina equisetifolia* (木麻黃), *Eucalyptus robusta* (大葉桉), *Eucalyptus citriodora* (檸檬桉), *Pinus elliottii* (愛氏松), *Tristania conferta* (紅膠木) and *Melaleuca leucadendron* (白千層). Very often, two to three of the above exotic species tend to dominate the slope cover. Dense canopy of exotic trees hinders the process of natural invasion of native species growing adjacent to the man-made slopes. Native plant species, which provide food and shelter are more suitable for local wildlife than exotic species. A biodiversified vegetation cover is ecologically stable and hence more sustainable. Establishing a sustainable vegetation cover on man-made slopes has been one of the long-term goals in slope greening. Ecological considerations play an important role in slope greening, as revealed by studies on the performance of vegetation establishment on man-made slopes.

The GEO is now making efforts to achieve ecological enhancement by adopting native species for restoring vegetation covers as far as possible, and protecting any rare plant species found on or in close proximity to the slopes to be upgraded under the LPM Programme. Ecological surveys are carried out where needed. GEO Information Note 16/2004 and GEO Information Sheet 17 (GEO, 2004d) promulgate the ecological enhancement in slope works.

The effort in preserving and protecting the native vegetation can be demonstrated in the following LPM projects in which rare plant species have been identified during ecological surveys.

- Project 1 – Slope upgrading works along South Lantau Road between Mui Wo and Pui O, Lantau: Wild orchids were identified on the slopes. Cages were provided to protect the orchids (Figure 28) during construction.

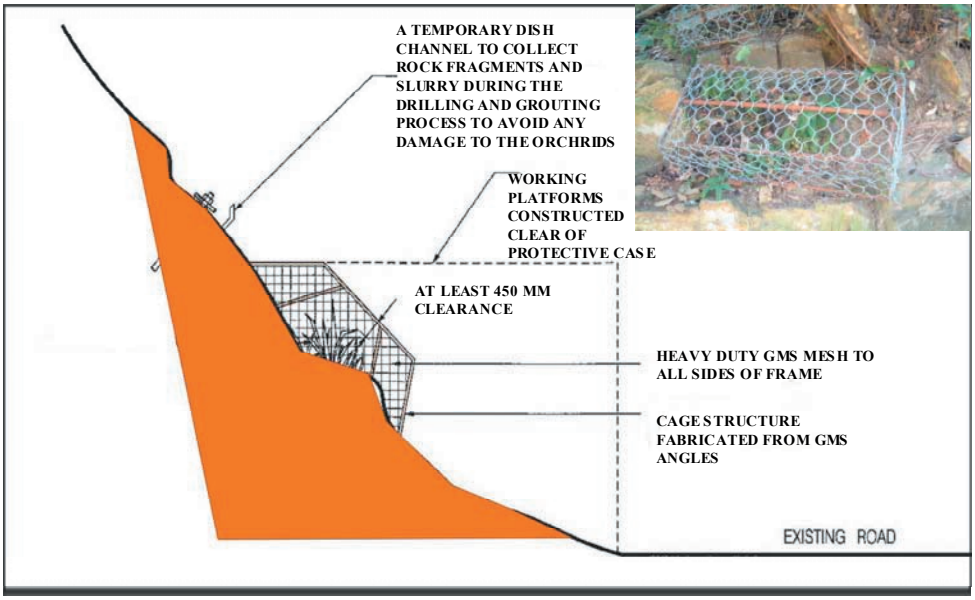

Figure 28 – Measures for protecting wild orchids in slope upgrading work sites

- Project 2 – Slope upgrading works along Keung Shan Road and South Lantau Road near Shek Pik Reservoir, Lantau: Chinese New Year flowers (Enkianthus quinqueflorus) (Figure 29) were identified. These flowers were classified as locally protected species found on a slope feature at Keung Shan. For those not affected by the works, the flowers were protected by chain link fence. If the flowers were affected by the works, the flowers were transplanted by a specialist landscape contractor.

Figure 29 – Chinese New Year flowers identified in a slope upgrading work site

Application of Bio-engineering Measures in Repairing Natural Terrain Landslide Scars

There are numerous natural terrain landslide scars in Hong Kong. In 2003, the GEO commenced a project to study and put on trial some soil bio-engineering measures to repair landslide scars on natural terrain (Campbell et al, 2005). The project was designed to identify measures with the potential to minimize natural slope deterioration, reinforce the soil mass, and improve resistance to debris movement. Five sites were identified to test the applicability and performance of the selected measures. Soil bio-engineering measures were designed and applied in various combinations at the Cloudy Hill North pilot site. Campbell et al (2005) describes these measures and presents some initial results. The findings are being documented to provide guidance on how to apply soil bio-engineering measures to other sites in Hong Kong.

Involve Private Slope Owners and Partnership with the Community

As one of the public education initiatives, the Layman's Guide to Landscape Treatment of Man-made Slopes and Retaining Walls was published in 2002 (GEO, 2002) for use by the general public. Following the publication of the Layman's Guide, a Slope Landscaping Kit was distributed to the Owners' Corporations, Mutual Aid Committees and Property Management Companies in the same year. To arouse the public's interest in enhancing slope appearance, the 'Best Landscaped Slope Awards' competition was organized jointly by the then Civil Engineering Department, the Professional Green Building Council, the Hong Kong Association of Property Management Companies, and the Hong Kong Institute of Landscape Architects in 2003. The response from the slope owners was overwhelming with 55 entries involving 90 slopes and retaining walls. The Awards Ceremony was held in February 2004 with the Champion slope awarded to Lung Tak Court, Stanley (Figure 30).

The enhancement in slope appearance has been made possible by promulgation of new technical guidelines, sharing of experience amongst practitioners and getting specialist advice from landscape experts where needed. Comprehensive technical guidelines on landscape treatment and bio-engineering for man-made slopes and retaining walls are now given in GEO Publication No 1/2000 (GEO, 2000a) described earlier.

Figure 30 – Best Landscaped Slopes Awards Prize Presentation Ceremony in February 2004 to promote enhancement of slope appearance and the champion slope at Lung Tak Court, Stanley

CONCLUSIONS

The overall landslide risk from old substandard man-made to the community of Hong Kong has been reduced significantly in the past two decades through the concerted effort of the HKSAR Government, the geotechnical profession and the private slope owners. A comprehensive Slope Safety System is in place to deal with the landslide problems. The scale and severity of the landslide problems in Hong Kong have been much reduced. Rapid development and advancement in technology will continue and that can open up more opportunity for its application in geotechnical engineering. Rising landslide risk from natural hillsides deserves increasing attention as more developments encroach closer to steep hillsides. On top of safety, the aesthetics of slopes are being enhanced, by integrating slope upgrading works and the provision of landscape treatment and bio-engineering at the design and construction stages. There is still much room for development and application. The Government has taken the lead in pioneering research and application of landscape treatment and bio-engineering in man-made slopes, and has started to extend its application to repair natural terrain landslide scars. Continuing joint efforts by the Government, the profession and the public need to be maintained so as to reduce the landslide risk, and to further enhance our surrounding environment – to make Hong Kong a safe and green place to live.

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ACKNOWLEGEMENTS

This paper is published with the permission of the Director of Civil Engineering & Development, Government of the Hong Kong Special Administrative Region.

SOIL BIOENGINEERING – AN INTEGRATED NATURAL APPROACH

Robbin B. Sotir[1]

Abstract: An integrated natural approach demands a holistic view of land and the watersheds that contain it. This requires comprehensive assessment, planning and management. Land represents integrated functions, and "function", is the bridge word that connects professionals, enabling them to develop and work with new integrated restoration strategies. Numerous areas of expertise are needed to provide the knowledge and awareness to achieve full function in the land for broad success. Soil bioengineering, a technology based on sound engineering practice and integrated ecological principles, views land and water as systems with integrated functions, rather than merely physically connected structures. Soil bioengineering, the focus of this discussion, represents one important component in defining these new restoration strategies for sustainable solutions. Case studies are presented in a balanced manner illustrating integrated restoration functions.

INTRODUCTION

Restoration is the process of repairing damage to the diversity and dynamics of ecosystems (Jackson et. al. 1984). The restoration process not only re-establishes the general structural foundation, function and dynamics, but also the self-sustaining behaviour of the ecosystem.

Restoration includes a wide range of procedures and measures designed to enable the watershed or interconnected basins to recover its functions, to a level consistent with established goals. The first, and most critical step in implementing restoration is to, wherever possible halt disturbance activities which cause degradation or preventing recovery of the ecosystem (Kauffman et. al. 1984). This may range from passive activities that involve removal or attenuation of chronic disturbance activities, to activities intervention and installation of major measures to repair damages to the structure of a stream or slope. The watershed is defined as the land that water flows across or under on its way to a stream, river, lake, bay or ocean (adapted from the National Watershed Coalition). Watersheds can be comprised of hillsides or be almost flat. They can have farmland, wetlands, small towns and big cities in them. Everyone and every living thing, lives in and makes up the watershed community. We influence what happens in the watershed by how we treat the natural resources – the soil, water, air, plants and animals.

Restoration technologies broadly include the reconstruction of some or all of a watershed's natural features, foundation and associated interrelated functions. These may include streams, rivers, upland cut and fill slopes and wetlands. In essence, restoration can include activities from single actions, such as vegetative plantings, to comprehensive solutions such as engineering and/or soil bioengineering, aimed both at the site itself and at the sources of degradation throughout the watershed. The specific goals of any particular restoration project should be defined within the context of the current and future conditions and disturbances in the watershed that will lead to rebuilding functionality in the system. Restoration typically

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will not involve returning a system to its pristine or pre-disturbance condition, but rather to establish self-sustaining foundations, within the context of the new altered regime.

The incorporation of soil bioengineering into the restoration further acts to ensure the integrity of the watershed landscapes through linkages with the physical, chemical, and biological processes that shape the living plant and animal communities.

HOLISTIC VIEW OF ISSUES AND SOLUTIONS

Human Activities in the Watershed

While some change, erosion and failure is acceptable and beneficial as part of the natural system dynamics, much created by human intervention is not. Human activities have contributed to changes and, in many cases, total loss in the dynamic equilibrium of the watershed. These activities centre on manipulating land and water systems for a wide variety of purposes, including domestic and industrial water supplies, irrigation, transportation, hydropower, waste disposal, mining, flood control, timber management, recreation, aesthetics and more recently, fish and wildlife habitat. Increases in human population and the associated industrial, commercial, and residential development, have also placed heavy demands on the watershed. These activities have led to major functionality losses in systems. Various options are available to slow down, and often completely prevent, further degradation. Additionally, follow-through into the construction phase and beyond is also critical to ensure the proper handling and disposal of surface and subsurface water and the protection of existing vegetation. These probably have a greater impact on both surficial and mass stability than any other factors, illustrating the importance of comprehensive planning.

Viewing land as systems with integrated functions, and understanding the importance of goals that rebuild functionality back into the watershed is the most effective long-term approach to solving these problems (Sotir and Christopher 2003).

Economics

Degraded systems negatively impact economics. Every damaged watershed is a waste of major economic potential that has the potential to be restored at a profit for the benefit of the community. Planning is an expensive but essential requirement for sound watershed projects. Economic factors to be considered include water supply; flood damage; residential, commercial and public land values; aesthetics; water quality; public image and tourism. Initially restoration of highly damaged systems is difficult and costly. However, when multi-objective restoration strategies bring more jobs and money into the community, restoration and protection accelerates (personal communication - Mike Freemont, Rivers Unlimited; 2004).

Products and Technology

Conditions in watersheds have dramatically changed due to development. The land is no longer able to accept the impacts and continue to successfully self-restore. Many segments of the population have recognized these changes and the need for restoration. During the past few decades many new manufactured products and technologies have been introduced to assist in protecting and rebuilding watersheds through restoration activities. The best have produced comprehensive systems that recognize the connections between land and water, thus increasing the options and enhancing the ability to restore damaged systems. Creating conditions for the reintroduction of vegetation into difficult conditions, such as highly steepened slopes is a good example (see Figure 1). In the larger context, however, advanced technologies may also pose a danger if we come to rely on one technology or product

exclusively. In some cases, practitioners now view a new product as a panacea losing sight of common sense procedures, such as site grading, the need to handle runoff, and the importance of observing basic erosion control principles. This, in some instances, has also led to the tendency of treating site-specific conditions that may only be symptoms of problems. In short, inappropriate use of products and/or technology and proceeding without understanding and responding to the big picture can lead to failure and discredit.

Integration of Technology

The engineering and environmental communities have worked independently to develop technological solutions. Practitioners of these technologies need to work more closely together, in order to produce the best possible effort in laying the foundation for restoring watershed function and dynamics.

Soil bioengineering integrates engineering and ecology, offering an important component in restoration. Soil bioengineering capitalizes on the benefits of vegetation for both the geological and hydrologic slope stability as well as erosion control. It incorporates woody vegetation to re-establish ecological, mechanical and hydraulic functions into damaged watersheds and serves to protect healthy systems. Additionally, it is most successful when both the engineering and the ecological benefits of vegetation are maximized (Sotir, 2000). This suggests opportunities for a more comprehensive and, in the long run, possibly a less expensive systems approach. Soil bioengineering represents only one important entity in the suite of tools and technologies available. As one entity it must be integrated with other technologies such as soil science, geotechnical engineering, hydraulics, hydrology, geology, habitat biology, geomorphology and landscape architecture.

Inert Systems

The reliance on inert systems alone such as concrete, typically offers singular limited functions for river, stream bank, slope and wetland stabilization with little understanding or regard for the complexity that exists within the watershed. In many cases they damage and lose a multiple of engineering, environmental and aesthetic self-sustaining functions and the associated benefits, all of which negatively impact the economics of the community. Inert materials are most often used because they are presumed to have predictable and invariant properties. In fact, inert materials such as steel, concrete, and synthetic polymers slowly degrade, decompose and/or decay and fail with time. Over time, these costs have become more apparent. Additionally, environmental and economic understanding and concerns have become major issues in all aspects of watershed development and management. Change is occurring, and those solutions that make use solely of inert structures and materials are being accepted less frequently because of their lack of environmental restoration benefits and because of there long-term maintenance costs (Sotir, 1998). Consequently, there is a great interest in designs that combine vegetation and inert materials into living systems that reinforce the soil and reduce erosion and failure while providing strong environmental benefits. These interconnected systems require responsible upland slope watershed management combined with river, stream bank and wetland protection. This combination is likely to address many of these concerns. Unlike conventional engineering or landscaping alone, soil bioengineering provides important and significant advantages in reaching the goals for rebuilding functionality, as it provides the initial foundation for the land to restore its self-repairing ecological and mechanical functions.

SOIL BIOENGINEERING SYSTEMS

Soil bioengineering is the combined application of engineering practices and ecological

principles to design and construct systems composed of living plant material, frequently in association with inert materials such as rock, wood, geosynthetics and geocomposites and other manufactured products to repair past damage and/or control soil erosion and shallow slope failures. While the analysis and planning stages typically take more time and expertise and are more expensive than conventional engineering, soil bioengineering may offer a long-term cost-effective and attractive approach for restoring stability in slopes, rivers, stream banks and shorelines, and offer protection against erosion and shallow mass movement. The value of vegetation in civil engineering and the role woody vegetation plays in the stabilization of slopes has gained considerable recognition in recent years (Greenway, 1987; Coppin and Richards, 1990; Shields 1993; Gray and Sotir 1996). Woody vegetation functions to improve the interrelated hydrology and mechanical stability of slopes through root reinforcement and surface protection. The biological and mechanical elements are analysed and designed to work together in an integrated and complementary manner to achieve the established goals. The new restoration strategies, assisted by the analysis and design of the stabilization systems, and are used to select and establish the appropriate vegetation for root reinforcement, functions and erosion control. These systems require knowledge of the mechanisms involving mass and surfical stability of slopes. Likewise when the vegetative aspects are appropriate to serve as reinforcements and drains, an understanding of the hydraulic and mechanical effects of slope vegetation is necessary (Sotir and Christopher, et al., 2000).

From the environmental restoration perspective a wide range of interrelated environmental functions are provided. Woody vegetation cools land, water and air temperatures and provides habitat for many aquatic, riparian and terrestrial species alike. Soil bioengineering assists in restoring living foundations to preserve and recover valuable and diverse habitat ecosystems for a variety of species. The survival of many species depends upon specific vegetative communities. Sites may have different combinations of features such as: high habitat diversity; high biological productivity; wildlife refuge/travel corridor function; and/or buffer functions that protect adjacent waters. Appropriately selected species for soil bioengineering system, set foundations in place that, over time, offer wildlife safe and productive areas for movement, nesting, raising young and feeding (see Figure 2).

Figure 1 Vegetated steep slope

Figure 2 Productive waterfowl area

Comparisons

While there are significant differences in soil bioengineering and conventional engineering restoration technologies, there are also many similarities in terms of functional requirements and solutions. Soil bioengineering, for instance, is often misunderstood to be a surface vegetation treatment alone. However, through the embedment of cuttings and rooted plants in the ground in various combinations, it is able to augment solutions to many

engineering issues (see Table 1). Live cuttings and stems are purposely imbedded and arranged in the ground or in earthen structures where they serve as soil reinforcements, hydraulic drains, barriers to earth movement, and hydraulic pumps or wicks. Techniques such as live fascines offer immediate surface protection and drainage similar to pole or french drains, capturing and redirecting seepage flows on the slope face. Brushlayers change a long slope into a series of shorter slopes. They provide immediate soil reinforcement much the same as conventional geogrids with the added ability to positively affect the direction of seepage, and physically binding the soil particles into a unitary mass.

At first glance, the soil bioengineering restoration construction methods may appear unworkable because of compatibility problems; that is, engineering requirements imposed for structural stability may clash with biological requirements of the vegetation. The backfill or crib fill behind or in a retaining structure, for example, should have certain specified mechanical and hydraulic properties if the structure is to perform satisfactorily. Ideally, the fill should be a coarse-grained, free-draining material. The presence of excessive amounts of clay, silt, and organic matter is not desirable. The requirements of free drainage so essential to the stability of earth-retaining structures, is also important to vegetation, which generally cannot tolerate waterlogged soil conditions. Establishment of vegetation, on the other hand, usually requires the presence of fines in the soil to provide some moisture and nutrient retention. In many instances, with a few small adjustments, these biological requirements can be satisfied without compromising engineering performance.

Table 1 Suitability of Different Soil Bioengineering Methods Based on Soil and Site Conditions.

		Soil Bioengineering Methods						
Factor or Failure Process	**Intensity or Type of Condition**	Live Staking	Live Fascine	Brush-layering	Branch-packing	Live Crib-Wall	Hedge-layer	Vegetated Reinforced Soil Slope
Slope gradient	*Steep*		✓		✓	✓		✓
	Moderate	✓	✓	✓	✓	✓	✓	✓
	Gentle	✓	✓					
Slope height	*High*	✓	✓	✓			✓	✓
	Low		✓	✓	✓	✓	✓	✓
Soil depth	*Deep*	✓	✓	✓	✓	n/a	✓	✓
	Shallow		✓			n/a		
Soil erodibility	*High*		✓			n/a		✓
	Moderate		✓	✓	✓	n/a	✓	✓
	Low	✓	✓	✓	✓	n/a	✓	✓
Soil strength	*Moderate*	✓	✓	✓	n/a	n/a	✓	n/a
	Low	n/a	✓	✓	n/a	n/a	✓	n/a
Slope type	*Cut*	✓	✓	✓	✓		✓	
	Fill		✓	✓		✓		✓
Surficial erosion		✓	✓		✓		✓	
Mass movement	*Shallow*	✓	✓	✓	✓	✓	✓	
	Moderate			✓				✓

Revised Sotir 2005; from Gray & Sotir 1996

n/a ≅ Not applicable

Vegetation

The loss or removal of slope vegetation can result in increased rates of erosion and higher frequencies of slope failure. Vegetation affects both the surficial and mass stability of slopes in significant and important ways. The stabilizing or protective benefits of vegetation depend on both the type of vegetation and type of slope degradation process. In the case of mass stability, the protective, rebuilding functionality benefits of woody vegetation range from mechanical reinforcement and restraint by the roots and stems to modification of slope hydrology as a result of soil moisture extraction via evapotranspiration.

The main benefits of employing woody vegetation for the mass stability of slopes and stream banks are root reinforcement, soil moisture depletion, buttressing and arching and surcharge. While there can be adverse effects, most of these such as wind throw and surcharge can largely be eliminated through the appropriate vegetation selection and slope design. Selective pruning on a three to ten year rotation may also mitigate these problems. These related benefits and impacts must be carefully considered and incorporated into the overall assessment, design and construction to reach and maintain the established goals.

The use of vegetation in soil bioengineered systems often provides a significantly more durable facing treatment, not only protecting the outward surface but also improving the internal drainage and stability of the system. Initially, the angle at which the vegetation is installed in the slope, 15º-45º sloping down from front to back, positively affects the direction of seepage. The zone immediately around the circumference of the branches tends to be more porous, and therefore acts as a horizontal drain, which diverts downward flow away from the face of the slope, thus improving the factor of safety (Sotir and Christopher et. al., 2002). The use of deeply-installed and deeply rooted woody plant materials, purposely arranged and embedded during slope construction offers:

- Immediate erosion control for slopes, stream and shoreline;
- Improved face stability through mechanical reinforcement by the roots;
- Reduced maintenance costs, with less need to return to revegetate;
- Modification of soil moisture regimes through improved drainage and depletion of soil moisture and increase of soil suction by root uptake and transpiration;
- Enhanced wildlife habitat and ecological diversity;
- Improved aesthetic quality and naturalization;
- Cleansing and filtering of pollutants carried by storm water runoff;
- Modification of temperature;
- Absorption of noise due to the soil mass;
- Reduction in quality and time of runoff improving storm water management; and
- Resistance to sliding or shear displacement with the development of roots.

The following represents the opportunities, challenges, and lessons learned from case histories where integrated technologies were applied successfully. These case histories illustrate soil bioengineering's usefulness in restoration to stabilize and protect a wide range of conditions to meet, and even exceed, the established multi-objective goals.

CASE STUDIES

Massachusetts Turnpike MM94.1

The construction of a new interchange between the Massachusetts Turnpike and Route 146 in Millbury, Massachusetts, required widening of the Turnpike at Mile 94.1 at an existing soil and rock slope 24m high. A section of the existing slope 274.5m long needed to be cut back up to 8.25m to accommodate the proposed on-ramp. It was necessary to develop a design

for stabilization and restoration of the proposed soil and rock cut. The project goals were to improve drainage, protect against slope failure, control soil erosion, revegetate and restore the soil slope aesthetics.

Rock Slope - The geology of the existing rock cut was studied to evaluate characteristics that would affect the stability of the proposed rock cut. The eastern 30.5m section of the rock cut was severely weathered. The remainder of the cut was moderately to slightly weathered.

Soil Slope - Before construction, the project area was a soil and rock cut 24m high, transitioning into a 2H:1V (H, horizontal: V, vertical) cut soil slope to the east of the project area. The soil at the site is dense glacial till consisting mostly of silty sand with gravel and ranging from 1.5 m to 15 m thick. A thin layer of topsoil partially covered the site. The northern slope orientation and active seepage areas result in very wet conditions on the slope. The soil temperatures are also lower than on slopes with different orientations, so the frozen soil thaws later in the spring. Conversely, in times of drought, this slope remains cooler and moister than others. Although this reduced solar exposure is not optimum for some types of vegetation, it was expected to be adequate for successful growth.

Rock Slope Design - Data obtained in the geologic reconnaissance were used to evaluate the stability of the proposed rock cut and to develop the final design slope of the rock cut. A bench 2m wide was created at the top of the rock cut to provide a buffer zone between the rock cut and soil cut (see Figure 3).

Rock Slope Construction - Perimeter control of the rock excavation was achieved with cushion blasting (see Figure 4). This control was necessary to minimize damage to the final rock face and to produce a planar cut. A boulder wall was constructed along the rock cut near the eastern end to stabilize severely weathered rock that failed during construction.

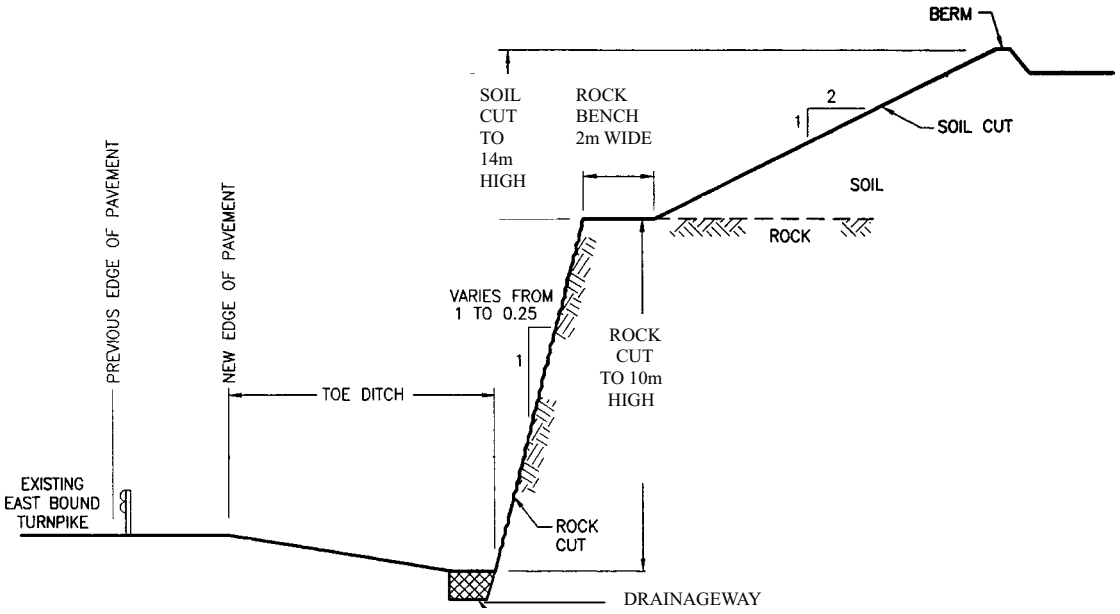

Figure 3 Typical rock and soil cut section

Soil Cut - A slope of 2H:1V was chosen for the soil cut, as indicted in Figure 3. The eastern end of the soil cut had a history of significant groundwater seepage problems, and a large slough was observed during site investigations. Drains constructed of pipe and wrapped crushed-stone were installed to collect the seepage and channel it away from the rock cut.

A soil berm was constructed at the crest of the soil cut to prevent surface water runoff from entering the cut. A large forested and wetland area slopes toward the top of the cut and provides a significant recharge area for groundwater and surface-water runoff. It is likely that this recharge area is the source of the groundwater observed seeping from the soil slope, particularly in the eastern section. The random locations of the seepage observed in the soil slope may also be from more pervious (sandy) lenses of soil in the glacial till that intersect the cut and drain more rapidly.

Soil Bioengineering Methods and Design Approach - The soil bioengineering system was designed to respond to the specific site conditions. Analysis revealed that some areas of the slope are especially wet, which increases the possibility of erosion and slope failure. For the wetter areas, a series of herringbone installations comprised of live fascines and brushlayer/live fascines were proposed (Figure 5, 6 and 7). These living systems are typically very effective in draining a slope by intercepting water (both subsurface and surface) and functioning as a drain. Drainage is also improved through evapotranspiration and internal drainage as the plants root and develop leaves. In the relatively drier areas of the site live fascine and brushlayer/live fascines were used at an angle without a central pole drain. The entire upper slope face was covered with coir erosion control matting.

Figure 4 Rock blasting operations during construction

The soil bioengineering system installed on this project provided the following immediate and long-term benefits: Mechanically, the stems and root system of the plant material protects the soil against erosion and shallow slope failure. Over time the roots will function as fibrous inclusions, reinforcing the soil mantle and increasing the resistance to sliding. Slope instability

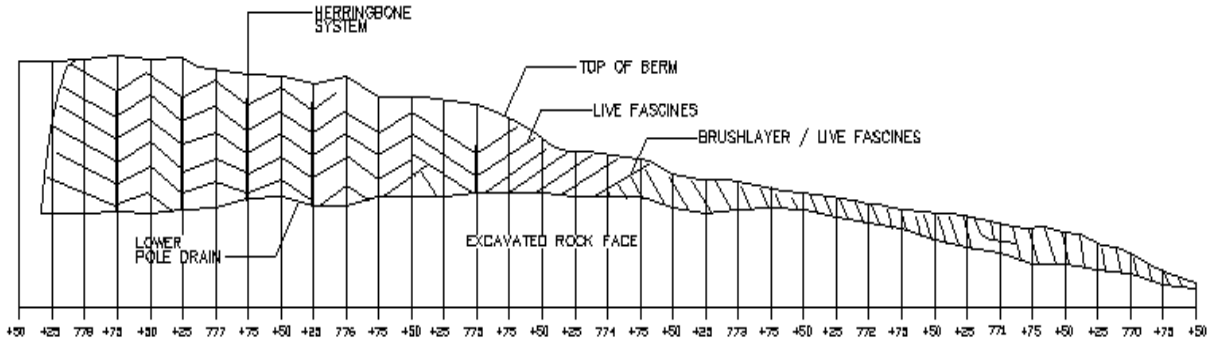

Figure 5 Face view of soil slope above rock cut

and erosion are reduced by transpiration of moisture and interception of rainfall. Hydrologically, the system immediately improved drainage and controlled seepage. The stems, leaves and surface litter add roughness, retards overland flow. Ecologically, the pioneer plants that were used are enhancing the wildlife habitat. Biodiversity has increased as vegetative invasion and natural succession occurs, creating self-sustaining native plant communities.

The key elements of success in this soil bioengineering-engineering combination include a stable rock cut with an adequate factor of safety against a deep-seated failure and collection and proper channelling of surface and groundwater seepage from the site. The project is working successfully in fulfilling the long-term mechanical, aesthetic and ecological goals of the project. Figures 8 and 9 illustrate the project immediately after construction and at the end of the first growing season.

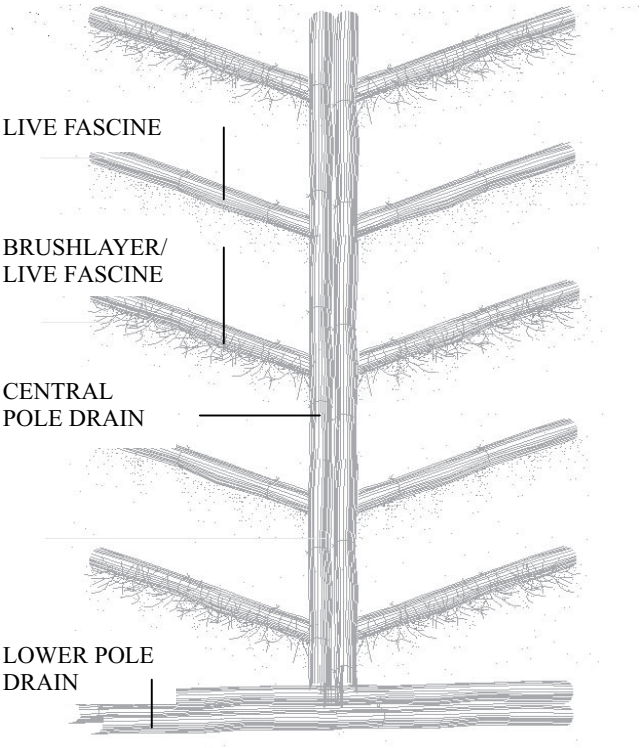

Figure 6 Herringbone system

Figure 7 Brushlayer/live fascine installation

Figure 8 Immediately after construction

Figure 9 First growing season

Lessons Learnt – The project was constructed during the winter and early spring. Constructing in the frozen soils was challenging, the thawing conditions made working in the excavations difficult and slowed production. On future projects of this nature the construction season should coincide with the fall and early winter, when melting and thawing conditions are not occurring.

Colrain Massachusetts, Greenfield Road – Slope

The project site is located on Greenfield Road, in northern Massachusetts at the entrance to the Town of Colrain. Improvements in road widening activities and encroachment on an adjacent, unstable hillside, resulted in slope failures. The road bank is a cut slope at 1.5H:1V (H-horizontal, V-vertical). Cut slope heights varied from 7.5m to 20m overall. Slope failures were characterized by slips and slumping along this 300m section of roadway (see Figure 10). Groundwater flowed out of the cut at numerous locations from both fractures in the underlying bedrock and through the exposed soil mantle face causing instability and associated failures.

Initial Conventional Design - The initial stabilization treatment proposed by the Massachusetts Department of Transportation was a crushed rock blanket, (see Figure 11) a treatment frequently employed by the agency. Due to the stark appearance of this solution, the town did not accept this option. Much of the economy in the Town of Colrain depends upon tourism, so it was important that the solutions blend into the natural scenic landscape. Rock blankets are also very difficult and expensive construction solutions.

Soil Bioengineering Designs - Three soil bioengineering designs were considered: (1) a crushed rock blanket with earthen brushlayer inclusions at periodic intervals. This design was considered to be too dry and lack nutrients for the vegetation to survive; (2) a drained fill brushlayer fill buttress. Concern was expressed about the possibility of a critical shear surface developing through the earthen fill adjacent and parallel to the brushlayers in the bottom, as well as sight distance; and (3) the soil bioengineering alternative ultimately recommended for the site was a combined drained brushlayer fill (see Figure 12) above a crushed rock toe. The top was finished using a rock drain and live fascines on each side (see Figure 13). This selected combination ensured that the shear stresses at the base of the slope would be handled with the required 1.5 factor of safety. Additionally, the vegetation would be installed in medium that would supply adequate moisture and nutrients. The combination produces a completely engineered, ecological and aesthetic slope.

Drainage - The earthen fill needed to remain in a drained condition - an important assumption in the stability analyses. A gravel filter course and a geotextile filter with in-plain drainage were installed against the back excavation, with the water discharging into the rock at the base.

The live cut branches from native woody vegetation was used in the soil bioengineering construction. Harvesting sites were within 50 Kilometres of the project. The construction period was from November of 1989 to the spring of 1990 (see Figure 14).

Figure 10 Existing failed conditions

Figure 11 Typical rock slope repair

Lessons Learnt - During the construction the MA DOT decided not to build the drainage channel above the newly constructed brushlayer installation. This resulted in water cascading over the slope creating rills in the soil face. In the fall of 1990, a brow berm was constructed to carry the water down the slope to a collection site. The project has developed well from a stability, aesthetic and ecological perspective (see Figure 15)

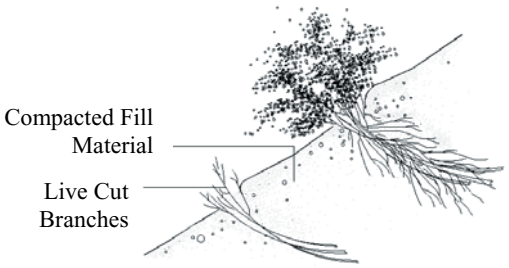

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Figure 12 Illustration of a brushlayer

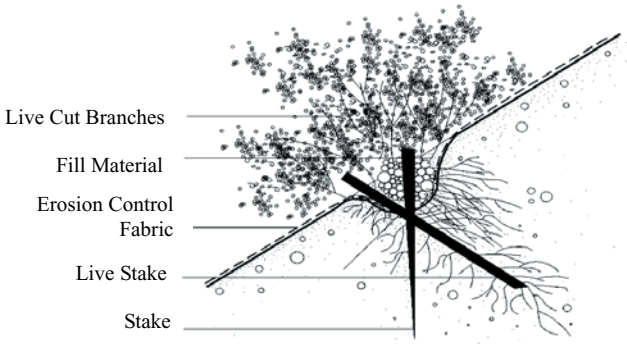

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Figure 13 Illustration of a live fascine

Figure 14 Immediately after construction

Figure 15 Growth in the fifth season

Long Leaf Stream - Flood Control

Long Leaf Creek is located within an urbanized watershed along coastal North Carolina. The specific stream reach addressed in this project is approximately six hundred meters (600m) in length and is located in a residential neighbourhood immediately downstream of a large shopping centre complex. It is a highly sensitive drainage improvement project with a variety of multi-objective goals specific to its location and function in the watershed. Development of the watershed caused the creek to deepen and widen - a result of increased storm water runoff and high concentration events (hurricanes). Properties along the stream were experiencing flooding, erosion and bank failures including reducing their property values (see Figure 16). Additionally, loss of aesthetic values; riparian corridor, water quality degradation; maintenance; and education needed to be addressed. The eroded stream banks ranged from 2m to 6m in height. The highly erosive soils, known as Baymeade, have a sandy loam texture.

The need for increased storm water capacity and hydraulic efficiency, combined with the residents' aesthetic and safety concerns and environmental requirements, prompted the City of

Wilmington to undertake a flood control improvement and stream restoration project.

Critical Issues - The interdisciplinary team developed a solution that would address the critical engineering, social, environmental and aesthetic issues. The critical issues and alternatives are shown on Table 2 below. The soil bioengineering vegetated reinforced soil slope (VRSS) system was selected as it best fit the multi-objective criteria.

While several soil bioengineering techniques were used, the main construction measure was the vegetated reinforced soil slope (VRSS), constructed over a rock foundation (see Figures 17 and 18).

Figure 16 Conditions prior to construction

Figure 17 Foundation construction

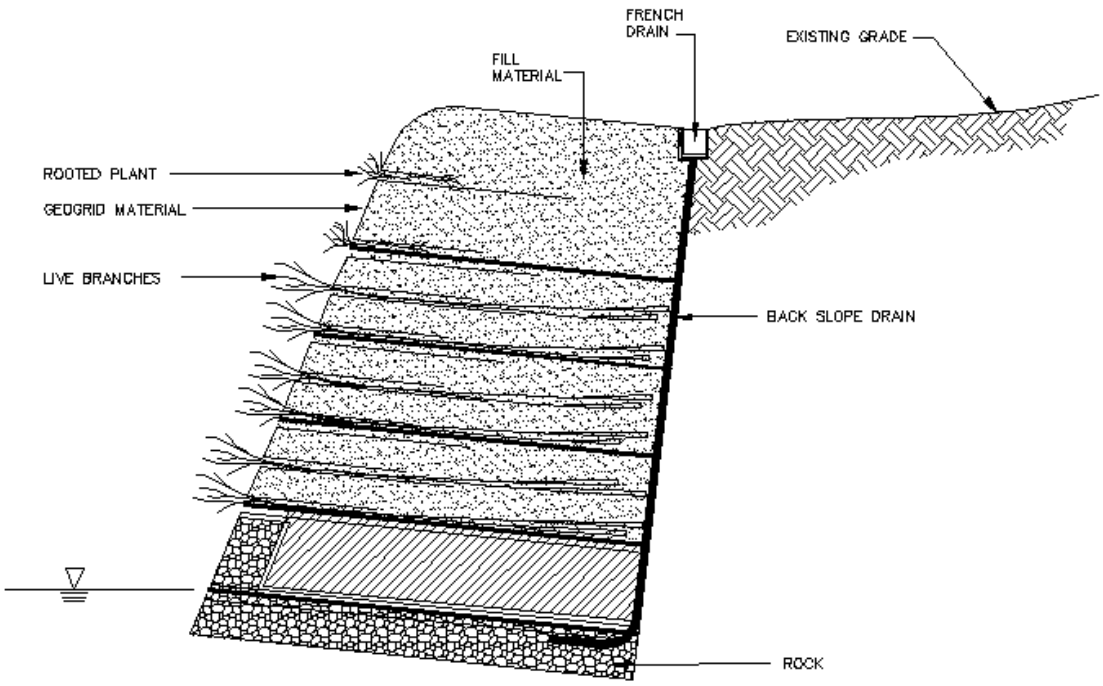

Figure 18 Illustration of VRSS structure design – note the live branch cuttings in the lower and middle layers and the rooted plants in the upper layers

Table 2 LONG LEAF STREAM - DRAINAGE IMPROVMENT ALTERNATIVES AND CRITICAL ISSUES

	ALTERNATIVES*				
Critical Issues	**1** **Intermediate Minimum Action[1]**	**2** **3:1 Side Slopes Grass Lining[2]**	**3** **2:1 Side Slopes Riprap Rock[3]**	**4** **2:1 Side Slopes Concrete Lined[4]**	**5** **Soil Bioengineering**
Stop Erosion & Stabilize Banks		•	●	●	●
Clean Out Trash & Debris	●	●	●	●	●
Safer & Healthier Area		●			●
Control Flooding		●	●	●	●
Environmental Improvement		•			●
Aesthetically Enhancing		•			●
Meets Hydraulic Efficiency		•	●	●	●
Permits Approval Probability	●				●
Minimize Property Loss	●				●
Low Maintenance			•	•	•

Sotir 2001

Legend: • Somewhat ● Fully

* A reinforced concrete box was also considered

[1] Does not address long-term needs.
[2] Does not address geotechnical issues of sandy bank material stability and major land loss concerns.
[3] Does not address safety concerns or reduction in property values.
[4] Unsafe during flood events and hostile to the environment.

Backfill Materials - The backfill used above the rock was tested and soils ameliorated to improve healthy plant establishment and growth. No organics or fines were added. Between each layer of wrapped fill material, a brushlayer was installed. These were 3m - 4m long and 25mm - 50mm in diameter consisting of common native woody plant species. The two upper layers were installed with bare root plants (hedgelayer) consisting of naturalized species and ornamentals to better incorporate the stream system into the property owners' garden settings (see Figures 19 and 20).

The construction of the project was completed in April of 1999 (see Figure 19). The project was designed for the 100-year event. During the first growing season (1999), hurricanes Dennis and Floyd produced an estimated 500-year event coupled with a long duration flow in the channel. While no flooding occurred, some damage was caused in the steepest upper reach of the channel at the toe. This was repaired in the spring of 2000 at a cost of approximately 7% of the original construction cost (see Figure 20).

Figure 19 Channel near completion

Figure 20 Channel 1 year after restoration

Lessons Learnt - While the 500 year event was much larger than the design event, the main reasons for failure can be attributed to reduced bed excavation depths, materials substitution and poor installation of the grade control structures during the construction phase. The project has performed well and the City is comfortable with the design and performance.

CONCLUSION

Soil bioengineering is a viable tool and offers a holistic and natural approach to restoration in the watershed. Success hinges on a wide range of variables from engineering (geotechnical, hydraulic/hydrology) to environmental water quality ecology, aesthetic expectations and timing that need to be considered during the assessment and design process. Sites can be assessed, designed and constructed to meet stability needs while offering enhanced visual aesthetic and environmental benefits with low long-term maintenance and economic feasibility.

Soil bioengineering provides impeccable, “invisible” engineering coupled with the highest possible level of ecological diversity and aesthetic function. (Sotir 2001)

There are many case studies to support the value of soil bioengineering in the watershed. The applications and integration of conventional engineering and ecological practices have grown and become more widely acceptable in the engineering community. However, much research is still needed to build higher levels of confidence in the engineering community. This is especially important since soil bioengineering is often used in highly altered urban watersheds for needs ranging from flood control to surface erosion. To further promote and

improve the integration of soil bioengineering in restoration of slopes and stream banks more systematic research is needed in the following areas:

1) Quantification and Documentation of Projects: more information is needed to quantify its use in different soil, site conditions, velocities and tractive force tolerances along stream and river environments. Testing is needed to evaluate different techniques and different species in a variety of conditions. Older projects need to be inspected and evaluated. It is important that the work of our mentors and former practitioners is not lost.
2) Costs/Benefits Analysis and Cost Comparisons: There is insufficient cost/benefit analysis of many projects and a need to compare soil bioengineering techniques with more traditional engineering alternatives. Economic analysis should take into account short and long-term maintenance practice, expected project life and overall economic benefit to the community. More economic information is needed to properly establish goals in soil bioengineering.
3) Ecological Baselines: More ecological baseline information needs to be collected prior to project installation. This will enable better information on habitat improvement measured by species diversity, ages, size, numbers and overall ecological health and functioning.

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ACKOWLEDGEMENTS

The paper review and comments from Robin Corathers, the Mill Creek Restoration Project, Alton Simms, Robbin B. Sotir & Associates, Inc., Elise Bellair, Cobb County, and Patrick A. Chao of Maunsell Geotechnical Services, Limited are gratefully acknowledged.

AN INNOVATIVE KOREAN SYSTEM OF PRESSURE-GROUTED SOIL NAILING AS A SLOPE STABILIZATION MEASURES

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ABSTRACT: Passive soil nailing is one of the most widely used slope stabilization measures in Hong Kong. The potential tensile capacities of soil nails are utilized to upgrade slopes to a factor of safety of 1.4 required by the Government. A delegation consisting of academics, consultants and contractors from Hong Kong was invited by a Korean soil engineering company to visit construction sites and fabrication workshops in South Korea to observe the fabrication and construction of an innovative soil reinforcement system. Observations and lessons learnt by the delegation are reported in this paper. Moreover, potential applications of the innovative soil reinforcement system with reference to local conditions are presented in this paper.

INTRODUCTION

The total area of the Hong Kong Special Administrative Region (HKSAR), China is approximately 1,103 km^2, accommodating a population of 6.8 million and one of the world's largest and most vibrant trading economies. The terrain of Hong Kong is hilly. Many slopes are thus man-made for land development to cope with the rapid development of Hong Kong. The stability of many man-made and natural slopes is of major concern to the Government and the public.

Passive soil nailing is one of the most widely used slope stabilization measures in Hong Kong over the last two decades. It is currently the predominant method used to upgrade the stability of existing soil cut slopes. The potential tensile capacities of soil nails are utilized to upgrade slopes to the required factor of safety of 1.4 promulgated by the Government. There has also been increasing use of the technique for forming new slopes.

Advancement in soil nailing technology may further enhance the safety of slopes in Hong Kong, improve the constructability of soil nails as stabilization measures, and thus reduce the construction and maintenance costs for these stabilization measures. A soil reinforcement system developed by Dae Won Soil Company Ltd of Korea may be adapted

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for application in Hong Kong. A delegation consisting of the first five authors of this paper visited the company in August 2004 to observe the fabrication and construction of the system. Their observations during the visit are reported in this paper. Concerns on application of the system in Hong Kong are identified. Moreover, ongoing research work to address these concerns and to investigate the technical feasibility of applying the system in Hong Kong is also presented.

CURRENT PRACTICE IN HONG KONG

High yield steel reinforcement bars are being used to construct soil nails in Hong Kong. The potential tensile capacity of a soil nail is governed by the minimum of: (1) tensile strength of the soil nail controlled by the yield stress of steel and the cross-sectional area of the steel bar; (2) bond resistance between the steel bar and the grout controlled by the bond strength of the nail-grout interface; and (3) bond resistance between the grout and soil controlled by the bond strength of the soil-grout interface.

Soil nails can be designed analytically using the engineering properties of soil, steel bar, and grout; geometry of the slope; and soil nail configuration (Watkins and Powell 1992). However, they can also be designed prescriptively. Guidelines on prescriptive design using soil nails as upgrading or preventive maintenance works for soil cut slopes and retaining walls are given in GEO (1999, 2004a), while those for concrete and masonry retaining walls are given in GEO (2004b). Technical guidance on good practice of design of steel soil nails for soil cut slopes is given in GEO (2005). Guidelines on the installation, testing, and acceptance of soil nailing works during construction to ensure quality of the works are given in GEO (2004c). Supervision, testing, and certification requirements at various stages of soil nailing works, and specification of the criteria for acceptance of the works are promulgated by BD (2003).

Corrosion protection is of paramount importance to the longevity of soil nails installed in slopes. Provision of sacrificial steel thickness is the most widely used method of corrosion protection of soil nails in Hong Kong. It allows for corrosion of steel by the provision of 2 mm sacrificial thickness on the radius of the steel bar. Products of corrosion that appear over time form a protective coating between the steel and soil. Whilst this coating offers no physical protection to the steel, it may slow down the rate of corrosion by changing the kinetics of the chemical reactions (Shiu and Cheung 2003). Moreover, the degree of protection is increased by providing a zinc coating (in the form of hot dip galvanization) to the steel bars.

Anyhow, there is a reduction of 4 mm in diameter for tensile capacity of the soil nail. When the required stabilizing force is large, soil nails of 40 mm diameter steel reinforcement are often installed at very close spacing. Such steel bars are heavy and thus difficult to maneuver on site. As a result, the zinc coating can be easily damaged. The length of each 40 mm diameter steel bar that can be handled on site is limited by its weight and individual site conditions and the typical length of each segment is approximately 3-5 m. Couplers are often used to connect bars to the required total length. Although couplers are also zinc coated, the screwing of the coupler onto a steel bar followed by the screwing of another steel bar onto the coupler can easily damage the zinc coating. Therefore, the coupler may be a weak link in the corrosion protection system. When soil nails are required in aggressive ground, a double corrosion system similar to that for prestressed ground anchors is required, resulting in significant increase in construction costs and time. The total cost of soil nail construction

including the steel nails, couplers, handling and transportation costs, drilling, and corrosion protection system in Hong Kong is much higher than that in many other countries.

Many engineers have reservation on the bond stress transfer of soil nails within loose fill slope. In particular, it is found that even though good compaction has been carried out to loose fill, the compacted dry density of the fill will decrease with time, possibly due to washout of fines by groundwater. In views of this concern, expensive and visually unpleasing concrete grillage is commonly used in Hong Kong.

THE KOREAN SYSTEM

An innovative system of soil reinforcement utilizing a combination of glass fiber reinforced polymer pipes and pressure grouting has been devised by Dae Won Soil Company, Limited of Korea. It has been used successfully as in-situ reinforcements to stabilize slopes, excavations, and tunnels in Korea. The system may be adopted in Hong Kong to improve the economy and constructability of soil nails.

GFRP is a material of lightweight, high corrosion resistance, and high strength. In fact, fiber reinforced polymer (FRP) rods are being used increasingly to replace conventional steel reinforcement bars and tendons in construction because of their high strength, lightweight, and superb corrosion resistance. The system utilizes glass fiber reinforced polymer (GFRP) pipes of 37 mm internal diameter and 5 mm thick. It is fabricated by a pultrusion process during which glass fibers are bundled together, drawn through a resin matrix, and then pulled through a shaping die. Centralizers are installed on the pipe in the workshop. Afterwards, two additional layers of glass fibers soaked with resin in cross directions are installed on the surface of the pipe at 45 degrees to the longitudinal axis of the pipe to improve its lateral load resistance and shear strength capacity. Holes are drilled through the pipe at a regular spacing. A glass fiber reinforced polymer pipe manufactured in the workshop and delivered to site is shown in Figure 1. The plastic tapes are used to temporarily cover the grout holes. Couplers are available for on-site coupling of pipes to facilitate the grouting process.

Holes of 105 mm diameter are drilled on the slope. The pipe is installed into the hole. The top of the hole is caulked with quick set cement. The annular space between the pipe and the hole is sealed with cement grout afterwards. A single packer system is inserted into the bore of the pipe approximately 4 hours to 8 hours later. The packer is first inflated to isolate the section to be pressure grouted. Pressured grout is then injected into the pipe, through the holes, to fracture the annular cement grout and permeate the surrounding soil.

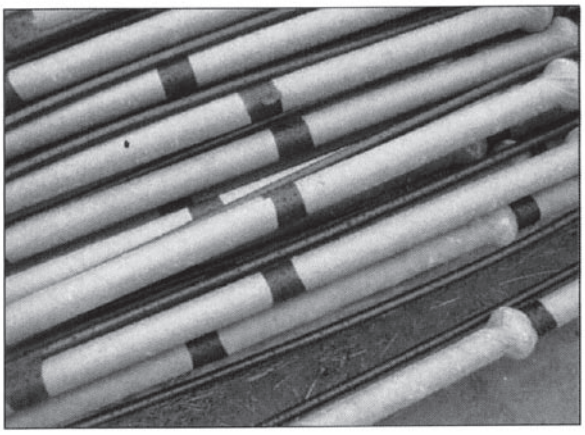

Figure 1 GFRP Pipe

The system is being used successfully in Korea to increase the shear strength of weak soil slopes; to reinforce cut slopes, tunnels, and excavations, temporarily or permanently; to construct seepage cut off to stabilize fault zones or other discontinuities in rock masses; etc.

The lightweight, high strength, and high corrosion resistance of the GFRP pipe would enhance the constructability of soil nails in Hong Kong. It is particularly suitable for congested sites where access is difficult. The high corrosion resistance can eliminate the need for provision of the 2-mm sacrificial thickness on the radius of the steel bar. The pressure grout may improve the bond strength of the soil-grout interface, resulting in an increase of the tensile capacity of soil nails if the bond strength of the soil-grout interface is the limiting factor controlling the overall capacity of the nail, and a corresponding reduction in the number of soil nails. Moreover, a hollow section can increase the bending capacity of the slender member, allowing for more tolerance on non-linearity of the installed nails. Therefore, there is a potential that the system can be adopted for soil nail construction in Hong Kong.

EVALUATION OF THE MATERIAL

The state-of-art review on the use of FRP as post-tensioning tendons by Reda Taha and Shrive (2003) reveals that unifiber FRP tendons exhibit a linear elastic response up to failure. The elastic modulus of GFRP and aramid fiber-reinforced polymer (AFRP) is usually in the range of 25% to 30% of that of steel, whereas carbon fiber-reinforced polymer (CFRP) rods have elastic moduli compatible to that of steel. The ultimate strains of FRPs are lower than that of steel, with GFRP exhibiting approximately twice the strain capacity of CFRP. The low failure strain values observed for most types of FRP tendons and the obvious lack of yield behavior become the key structural disadvantages of FRP materials, as there will be an inherent lack of ductility in the overall structural response. Moreover, FRP may have much lower shear and compressive strengths than steel bars, as the properties of FRP matrix dictate the lateral performance of the rod.

Creep rupture is the inability of a material to sustain a load lower than its ultimate capacity over a long period of time (Dowling 1999). The tendency for creep rupture of most FRP tendons prevents them from being used for long periods at much more than a defined ratio of their ultimate strength (Dolan 1993). Budelmann and Rostàsy (1993) recommended that the service stress level of FRP should not exceed 60% of the ultimate strength of FRP tendons.

Stress relaxation is a time-dependent phenomenon that represents the stress loss observed in materials held at a constant elongation. Stress relaxation depends on both the strain loss of the material and its stiffness. Hercules Aerospace (1995) has proposed that GFRP may have the highest stress relaxation based on its performance in both fatigue and creep tests. GFRP tendons have demonstrated poor fatigue performance, especially when tested at elevated stress levels. This poor performance was attributed to the loss of stiffness due to creep of the resin (Ellyin and Kujawski 1992; Walton and Yeung 1986). The results of Adimi and Benmokrane (1997) indicate GFRP bars can withstand the desired fatigue life of 4 million cycles when loaded to only 22% of ultimate capacity. Further information on durability of GFRP tendons is given in Tannous and Saadatmanesh (1998).

Although FRP rods may have their shortcomings in replacing steel reinforcement bars in reinforced concrete structures, it should be noted that FRPs have a definite technical advantage over steel for soil nail construction, as most disadvantages of FRPs would not realize in the construction of passive soil nail. There is also a concern that the polyester resin forming the matrix may react with the Portland cement grout in the long term. To avoid the problem, the polyester resin proposed for use as nails for the Hong Kong market will be replaced by urethane resin.

THE WAY FORWARD

A research program has been jointly developed by local academics and practitioners, and the Korean supplier to better understand the constructability and engineering performance of the Korean soil reinforcement system in Hong Kong. There are two major components in the research program: (1) a laboratory testing program to evaluate the material properties of the GFRP pipes; and (2) a site evaluation program to evaluate the constructability and in-situ engineering performance of the system. At the time of writing, test GFRP pipes have been manufactured in Korea and delivered to Hong Kong.

The first component will be conducted in the laboratories of the Department of Civil Engineering of The University of Hong Kong and of the Department of Civil & Structural Engineering of The Hong Kong Polytechnic University. The tensile strength, shear strength, bending behavior, Young's modulus, stress-strain characteristics, etc. of the GFRP pipe will be evaluated following different standards under a controlled environment. Different combinations of applied stresses simulating conditions in a slope will also be used in the laboratory testing program. The program is still ongoing at the time of writing.

Four full-scale 3-m long instrumented GFRP pipes have been installed in a slope stabilization work site in Hong Kong in March 2005. Holes of diameter 105 mm have been drilled and the drilling progress was monitored by the Drilling Progress Monitor developed by The University of Hong Kong as shown in Figure 2.

Six strain gauges were installed on each GFRP pipe in the laboratory as shown in Figure 3 and functionality checks on the installed gauges were also performed in the laboratory. The instrumented pipes were transported to site for installation afterwards. After the pipe had been inserted into the hole, the top of the annular space between the pipe and the hole were caulked by quick set cement as shown in Figure 4. The space was then grouted using cement grout as in conventional soil nail construction.

A few fours later, a packer as shown in Figure 5 was installed into the pipe bore as shown in Figure 6. The packer was inflated and the bottom one meter of the pipes were grouted at different pressures. The remaining space within the pipe bore was filled with cement grout after pressure grouting.

Although it was the very first time that this type of soil nail was installed in Hong Kong, no insurmountable installation difficulties were encountered on site.

Figure 2 Drilling of Soil Nail Hole

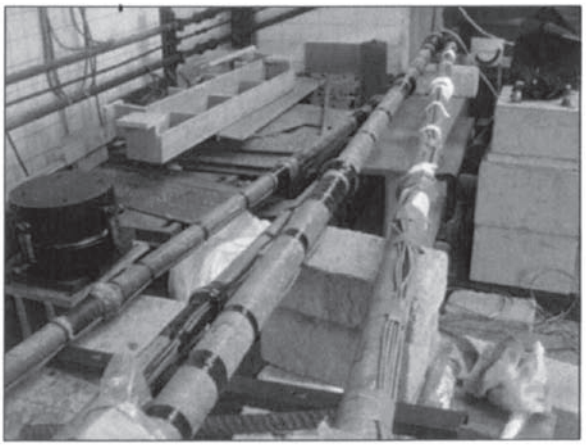

Figure 3 Instrumented GFRP Pipes

Figure 4 A Caulked Soil Nail

Figure 5 The Grout Packer

Figure 6 A Packer being Installed

The engineering performance of these soil nails will be evaluated by full-scale pullout tests. Special adaptors have been made to perform pullout tests of this new type of GFRP nails. The load applied on the pipe, elongation of the pipe, movement of the slope surface, and stress distribution along the length of the pipe will be monitored simultaneously.

CONCLUSIONS

The technical feasibility of applying a GFRP soil reinforcement system developed by a Korean Company in Hong Kong is being evaluated. Our site experience on the installation of the system indicates there is no insurmountable installation difficulties encountered on site. Four 3 m long instrumented GFRP nails were installed,and grouted at different pressures. The engineering performance of these installed nails will be evaluated by full-scale pullout tests. Material testing is still ongoing.

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SOME PRECAUTIONS IN SLOPE STABILITY ANALYSIS

Cheng Y M[1] and Lansivaara T[2]

Abstract: Slope stability analysis is well known to be a statically indeterminate problem and various assumptions on the internal force distribution have been used in different types of methods of analysis. For 'rigorous' methods based on Morgenstern-Price (with Spencer method as a special case) and Generalized Limit Equilibrium (GLE), many engineers have experienced the problem of convergence in analysis. Currently, most of the engineers in Hong Kong tackle this problem by modifying the prescribed failure surface until a converged result is achieved. In the present paper, the authors will discuss several major issues which are important for a proper analysis.

INTRODUCTION

Due to the rapid growth in population and commercial activities, stability of slope is a very important issue in Hong Kong and many other developed cities. For a typical slope stability analysis, the minimum factor of safety is required to be determined and most of the engineers are still relying on trial and error approach. In Hong Kong, about 50% of the slope stability analyses are carried out by Morgenstern-Price's method (1965) because Morgenstern-Price's method (M-P) appears to give a higher factor of safety in some cases and satisfies both moment and force equilibrium. Besides the Janbu's simplified method, Janbu (1973) (together with Lansivaara) have also developed the rigorous method and has put forward the correction factor concept by calibration with the Janbu's rigorous method for simple slope with no abrupt change in slope profile. If the correction factor by Janbu and Lansivaara (1973) is used, Janbu's simplified method will usually give factor of safety close to that by M-P method.

Slope stability analysis is well known to be a statically indeterminate problem and various assumptions on the internal force distribution have been used in different types of methods of analysis (see Abramson 2002 and Bromhead 1992). For simplified method based on Janbu or Bishop, convergence seldom occurs as the interslice shear force is neglected and complete equilibrium is not enforced. For 'rigorous' methods based on Morgenstern-Price (f(x)=1 is used in Hong Kong and M-P method is equivalent to Spencer's method 1967) and GLE, many engineers have experienced the problem of convergence in analysis. During location of critical failure surface by manual trial and error approach, most of the engineers in

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Hong Kong tackle the problem of 'failure to converge' by modifying the prescribed failure surface until a converged result is achieved. It appears that local engineers are not interested in overcoming the problem of 'convergence' but it has never been proved that a failure surface which fails to converge is not a critical failure surface !

Failure to converge is commonly experienced by many geotechnical engineers as iteration method is used in the analysis of the stability equation in many programs. Cheng (2003) has developed a new technique double QR method which is based on a complex matrix assemblage of all the slices. The special advantage of this method is that the factor of safety is determined directly from the matrix equation using a complex matrix manipulation. Based on the double QR method, Cheng (2003) has also proved that there will be N factors of safety for a problem with N slices. In this new approach, all the N factors of safety are determined from the matrix equation without using any iteration. The factors of safety from double QR method can be classified as:

1. imaginary number
2. negative number
3. positive number

If all the factors of safety are either imaginary or negative number, the problem under consideration has no physically acceptable answer by nature. Otherwise, the positive number (usually 1 to 2 positive numbers left) will be examined for the physically acceptability and the final answer is obtained. Under this new formulation, the fundamental nature of the problem is fully determined. If a physically acceptable answer exists, it will be determined by this double QR method. If no physically acceptable answer exists from double QR method, the problem under consideration has no answer by nature and the problem can be classified as 'failure to converge'. The authors have found that many problems which fail to converge with classical iteration method actually possess answers which cannot be evaluated by iteration method. The authors have also found that many failure surfaces which fail to converge possess normal shapes and should not be neglected in ordinary analysis and design.

In the present paper, the authors will discuss several major issues which are important for a proper analysis :

1. False convergence may be achieved by a large tolerance in analysis.
2. Classical iterative method of analysis may fail to converge even if the correct initial factor of safety is used in the iteration analysis.
3. Problem with the current practice in modifying the prescribed failure surface until convergence is obtained.

CASE STUDIES

Refer to Figs.1 and 2 for circular failure surfaces with 3 soil nails. For Fig.1, the soil nails loads are 60 kN, 80 kN and 100 kN and the order of the soil nail is from right to left as shown in the figure. For Fig.2, the soil nail loads are 30kN, 40 kN and 50 kN and the order of the soil nail is from left to right. The results of analysis are shown in Tables 1 and 2. In Tables 1 and 2, 'fail' means failure to converge by iteration method.

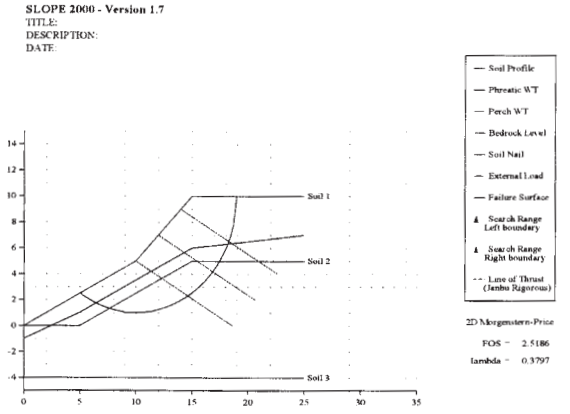

Fig.1 3 soil nails with water table

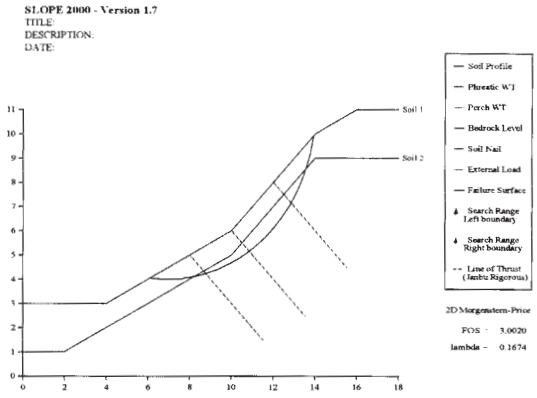

Fig.2 3 soil nails without water table

	Soil nail (60+80)	Soil nail (60+100)	Soil nail (80+100)	Soil nail (60+80+100)
Classical iteration method				
Bishop	2.083/2.047	2.328/2.291	2.482/2,466	2.567/2,518
Janbu	1.654/1.733	1.765/1.839	1.923/1.974	1.938/2.049
Morgenstern-Price	Fail/2.046	Fail	Fail	Fail
Double QR method				
Bishop	2.106	2.36	2.514	2.591
Janbu (f_o=1)	1.742	1.843	2.03	2.052
Morgenstern-Price	2.074	2.304	2.448	2.516

Table 1 Results of analysis for slope with soil nails as shown in Fig.1

	Soil nail 1 '30 kN'	Soil nail 2 '40 kN'	Soil nail 3 '30 kN'	No nail
Load applied at nail head				
Bishop	2.363/2.353/2.363	2.203/2.193/2.203	1.762/1.757/1.765	1.70/1.696/1.702
Janbu	1.95/1.959/1.97	1.956/1.971/1.973	1.683/1.701/1.702	1.495/1.494/1.511
Morgenstern-Price	2.307/2.298/2.312	2.17/2.163/2.172	1.756/1.749/1.786	1.675/1.675/1.682
Load applied at slip surface				
Bishop	2.377/2.366/2.375	2.211/2.203/2.21	1.762/1.757/1.765	
Janbu (f_o=1)	1.912/1.906/1.929	1.884/1.878/1.901	1.683/1.701/1.702	
Morgenstern-Price	fail/fail/2.32	fail/fail/2.168	1.767/fail/1.771*	

Table 2a

	Soil nail 1+2	Soil nail 1+3	Soil nail 2+3	Soil nail 1+2+3
Load applied at nail head				
Bishop	3.055/3.028/3.049	2.487/2.434/2.396	2.243/2.228/2.243	3.039/3.005/3.034
Janbu	2.527/2.57/2.558	2.059/1.995/2.183	2.157/2.204/2.182	2.762/2.85/2.793
Morgenstern-Price	2.98/2.967/2.984	fail/fail/2.369	2.231/2.226/2.235	3.005/2.983/3.002
Load applied at slip surface				
Bishop	3.087/3.06/3.077	2.396/2.379/2.439	2.283/2.275/2.279	3.114/3.09/3.10
Janbu (f_o=1)	2.374/2.362/2.4	2.157/2.179/2.033	1.972/1.965/2.003	2.441/2.428/2.478
Morgenstern-Price	fail/fail/2.986	2.363/2.355/2.388	fail/fail/2.24	fail/fail/3.016

Table 2b The first two FOS in each cell of Tables 2a and 2b are given by commercial programs using classical iteration method while the third FOS is given by Double QR method for problem as shown in Fig.2

As shown in Tables 1 and 2, Morgenstern-Price's method using iteration method suffers from failure to converge problem but answers actually exist for these problems (a precision of 0.0005 is used by the authors). Besides the case of soil nailed slope, a slope with no soil nail may also face the problem of 'failure to converge' which is shown in Fig.3. For the slope in Fig.3, the authors have tried different moment points and have found that convergence is obtained if the moment point is high above the failure surface while it will fail to converge if the moment point is close to or below the ground surface. For arbitrary problem, the region suitable for moment point (for convergence) has to be established by trial and error approach and there is no simple way to pre-determine this region. There are also problems under which the authors could not establish a suitable region for moment point calculation but answers exist by double QR method. The convergence problem is critical for M-P method but is rare for simplified Janbu's method. The authors have however constructed a problem as shown in Fig. 4 for Janbu's analysis where iteration method fails to work but the factor of safety can be determined by double QR method.

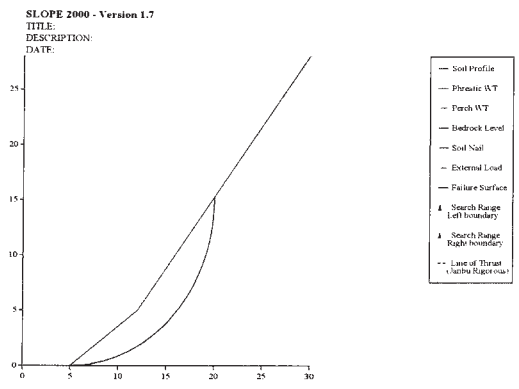

Fig.3 Failure to converge with M-P

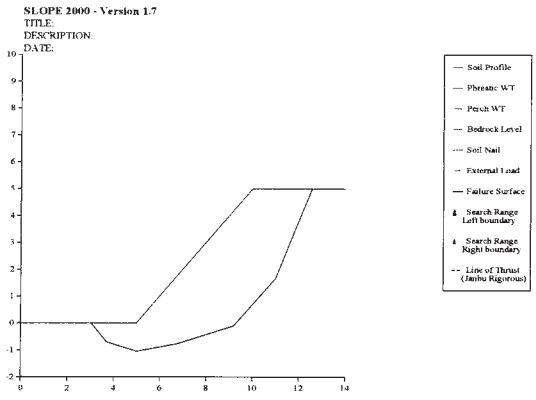

Fig.4 Failure to converge with Janbu's simplified

For the cases as shown in Figures 1 to 3, engineers should notice that even if the correct factor of safety is defined as the initial solution, iteration method may still fail to converge. The authors have found that changing the location of moment point by trial and error may give convergence in some cases, there are still some cases where a suitable moment point is difficult to be established. Engineers may be trapped easily by this problem easily when they are locating critical failure surface.

Besides the failure to converge problem, the authors have also discovered another interesting problem for M-P analysis. With reference to the problem as shown in Fig.5, if the tolerance for analysis is set to 0.01, the problem appears to converge for M-P analysis. The problem will however fail to converge if the tolerance if the tolerance is reduced to 0.001. Actually, the authors have found that F_f and F_m curve as shown in Fig.6 will never meet for this case and the convergence is not a true phenomenon. The authors have noticed that occasionally some local engineers set a relatively large tolerance to overcome convergence problem and this practice is not appropriate. It is interesting to note that even though this problem does not converge with a small tolerance, the converged answer for a larger tolerance appears to be reasonable as compared with other methods of analysis. It is not known whether this result is a general phenomenon or is just a special case and more research works are required.

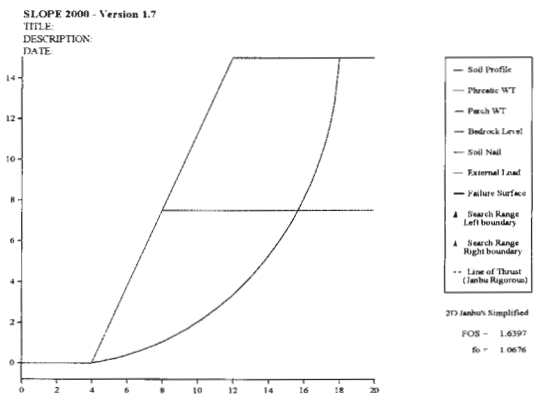

Fig.5 Effect of tolerance on M-P analysis

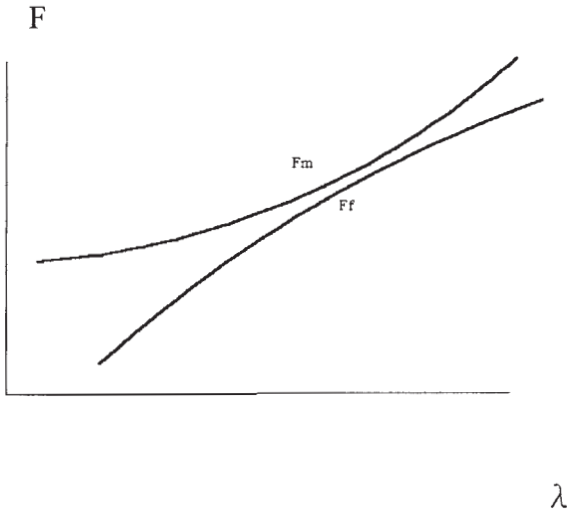

Fig.6 Effect of λ on F_f and F_m

REASONS FOR FAILURE TO CONVERGE

To investigate into the reason behind 'failure to converge', the equilibrium equations as shown below should be studied.

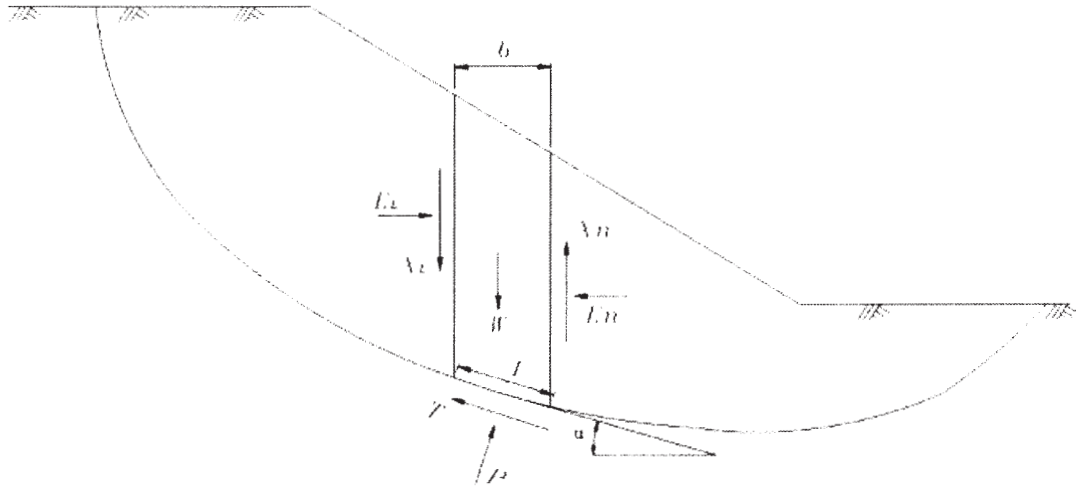

Fig.7 Forces acting on a failure mass

For slice as shown in Fig.7: at base – total normal stress= σ , shear stress= τ , pore pressure= u

Failure criterion: $s = c' + (\sigma - u)\tan\phi'$ (1)

Mobilized shear strength $\tau = s/F$ where F is factor of safety (2)

Now $P = \sigma l \quad T = \tau l$ so $T = \frac{1}{F}(c'l + (P - u\,l)\tan\phi')$ (3)

Resolve vertically : $P\cos\alpha + T\sin\alpha = w - (X_n - X_L)$ (4)

Rearranging and substituting for T gives

$$P = [W - (X_n - X_L) - \frac{1}{F}(c'l\sin\alpha - ul\tan\phi'\sin\alpha)]/m_\alpha \quad (5)$$

where $m_\alpha = \cos\alpha(1 + \tan\alpha\frac{\tan\phi'}{F})$

In solving the equilibrium equations to get the factor of safety F, the interslice shear force X_n and X_L have to be assumed to be 0 in the first step in iteration method or Newton-Rhapson method. Although this approach appears to be effective in many cases, it will lead to a wrong solution path in some cases. Double QR method avoids this problem by evaluating the factor of safety and all internal forces directly at the expense of complex formulation and computer time. For the case as shown in Fig.3, the authors have tried to pre-define the initial interslice shear force to be 50% of the correct value and convergence will be obtained at arbitrary moment point. That means, iteration or similar method will work if a suitable initial interslice shear force can be defined. Since it is impossible to pre-define the interslice shear force for a general problem, iteration method will leads to wrong solution path in the **first** step of solution in some cases. It appears that some problem will fail to converge by nature if iteration or Newton-Rhapson method is used as it is required to assume **X** to be 0 in the first step.

CONCLUSIONS and RECOMMENDATION

The authors have discovered many interesting problems in slope stability analysis which may affect engineering analysis and design. Factor of safety determination is equivalent to solution of non-linear equation. To the authors' knowledge, it has never been proved that iteration will be successful in solving non-linear equation under all cases which is clearly demonstrated in the present paper. It is true that iteration may be successful in many cases, but there are also many cases where answers exist but the answers cannot be determined by iteration method even if the correct answers are used as the initial trial. In response to the various problems for classical iteration method, Cheng (2003) has developed the Double QR method under which factors of safety can be determined directly from complex matrix manipulation. Actually, double QR method is the only method at present where iteration or convergence is not required.

Based on the results of in this paper, it is clear that slope stability analysis which appears to be a simple problem has various difficulties which require the attention of engineers. The double QR method that is developed by Cheng has attracted the attention of many engineers in other countries and this technique is now used in several countries in overcoming the convergence problem. Engineers should also be careful in performing the slope stability analysis as many problems which fail to converge actually possess acceptable answers.

For a proper slope stability analysis, convergence and acceptability of internal forces are required. Engineers should develop the necessary judgment that :

1. Answers may exist for problems which fail to converge with classical iteration method and this is clearly illustrated in the present paper. It is possible that the critical failure surface may not converge with iteration method so that engineers may miss the critical failure surface.
2. Any answer which is obtained through any method of analysis should be checked for acceptability of internal forces. Cheng and Zhu (2005) have constructed some cases where factors of safety may be obtained but the internal force distribution is not acceptable and the converged results should be rejected (this case is not common but is possible). Morgenstern and Price (1965), Abramson (2002) and Cheng and Zhu (2005) have suggested that acceptability of internal forces which appears to be missing in most of the commercial programs should be checked before acceptance of the results of analysis. Besides that, engineers are strongly advised to check the answer based on a particular method of analysis with other method of analysis. For example, engineers can check the results based on Morgenstern-Price's method with those by Sarma's method or similar.

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CASE REVIEW ON THE PERFORMANCE OF GEOFIBER SLOPES IN HONG KONG – A NATURAL SUCCESSION APPROACH

Albert N. L. Ho[1], Noriman S. C. Mak[2]

Abstract: The use of Geofiber (a continuous fiber-reinforced soil technique) on forming environmental slopes in Hong Kong recently has transformed the hillsides in the territory. An earthy dimensional fiber reinforced soil skin is replacing slopes that were plastered with unsightly shotcrete or chunam. This skin layer is strong enough to resist the severe bombardment of Hong Kong's tropical cyclone and provides a soft stable base essential to nurture a variety of plant growth that encourages natural succession of vegetation on slopes. With this special slope skin therapy, each face can be tailor-made to suit the environment and in particular to re-establish ecological equilibrium for long-term sustainability. The much-needed maintenance on monotonous single-species approach and limitations on grass-cover approach is now history with this new eco-engineering approach using natural succession principles on formed vegetated slopes. This paper outlines the eco-engineering approach to slope treatment with a natural succession model proposed. Observations were recorded and data were collected since 2001. An analysis is made to correlate vegetation conditions on slopes with parameters such as geometries, geology, locations and climatic factors in order to understand the succession behavior on slope faces. Case reviews two Geofiber slopes in Hong Kong on shotcreted or rock slopes are discussed

INTRODUCTION

The concept of eco-living and sustainable development has been practiced elsewhere in the world for some time. In New Zealand, all developments have to be scrutinized with assessment of possible impacts and influence on the environments and its biodiversity together with an analysis of the climate and soil factors. The approach tries to balance urban development with protection to the ecosystem of the environment. In Europe and North America, the eco-engineering approach has been adopted for past decades. It emphasizes on the use of indigenous vegetation in slope surface protection and enhancement of the soil strength with inert materials and relies on the self-sustainable vegetation on the slope for protection and stability. The recreation of ecological habitats promotes sustainability with natural preservation and maintenance.

The application of vegetation approach to slope protection in Hong Kong is still relatively young to date. The major obstacle in adopting this approach is to overcome Hong Kong's hilly terrain and its severe climatic and seasonal changes for sustainable and protective vegetation cover. Very often, a monotonous single-species approach using specified vegetation or grass cover is used without due considerations given to the actual environmental setting on the site surroundings. As a result, the vegetation cover installed will need periodic replacements that are not sustainable.

Fundamental to the restoration process on man-made slopes in Hong Kong using an

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Eco-engineering approach is the availability of an environmental skin that is strong enough to weather the severe Hong Kong climate and to be stable for nurturing natural succession and biodiversities of plant communities on the slope surfaces. It is attributed to a modern technology named Geofiber (Mak, 2003) - a Continuous Fiber Reinforced Soil method that a stable strong and permeable new soil skin can be formed to protect the surface and at the same time provides a natural ground for regeneration of vegetation growth on the slope face. This stable and permeable slope environment therefore favors natural succession of plant communities and work towards restoring the ecological equilibrium and in doing so, with very little maintenance required.

PROPOSED ECO-ENGINEERING MODEL

Adopting an Eco-engineering approach on slope treatment in Hong Kong will need to address the engineering aspect as well as the ecological aspect of the problem. The balanced philosophy will involve the appropriate selection of engineering methodologies on stabilization of slopes that is important towards providing a safe and stable slope. It also considers the ecological aspects aiming at restoring ecosystems for a particular slope area. The design habitat with appropriate selection of suitable native flora to blend with the surrounding environments for this slope surface is crucial for natural succession to take place. This balanced Eco-engineering model on slope treatment is summarized on Figure 1.

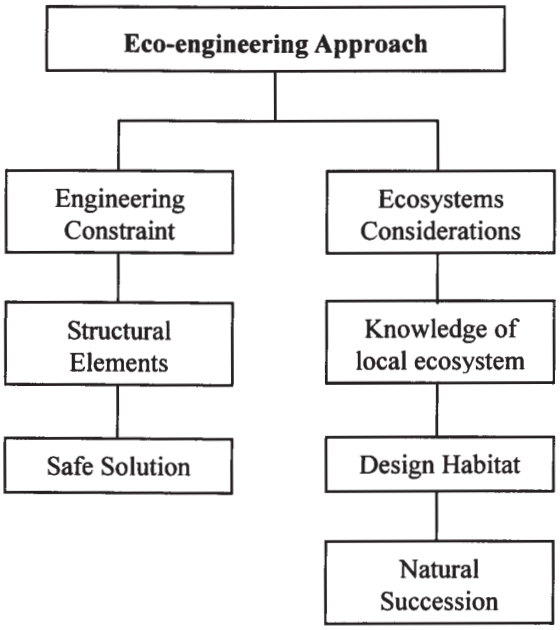

Figure 1 An Eco-engineering Approach to Slope Treatment in Hong Kong

Restoration of ecosystems on a particular slope takes time and depends on the surrounding natural environment and the inherent nature of the slope face. Natural succession is an important element in determining the success of restoration. It is a non-seasonal and progressive change in structure and species composition in a community (Dudgeon and Corlett, 1994).

The plant species that are able to establish on the slope depend on their growth rates, competitive abilities, shade tolerance and other environmental adaptation. The design habitat is a transient state to promote natural succession of plant communities.

A natural succession model is shown on Figure 2 (Mak et al. 2004). It simply identifies the effect of the initial design habitat on the constantly changing conditions of the slope face with regards to the plants societies under natural succession and without further human interference on its vegetation with time. It emphasizes on the long-term goal of ecological equilibrium and the transition that takes place for the long-term stable state to be reached.

With reference to the natural succession model proposed, a stable plant community would gradually develop into a mature stage under a stable physical environment in soil slope, with favorable biotic condition. At the beginning, the habitat is carefully designed and is made available for colonization by the pioneer plant species. It has limited biodiversity and the

pioneers will modify the slope through change in soil conditions and micro-climate.

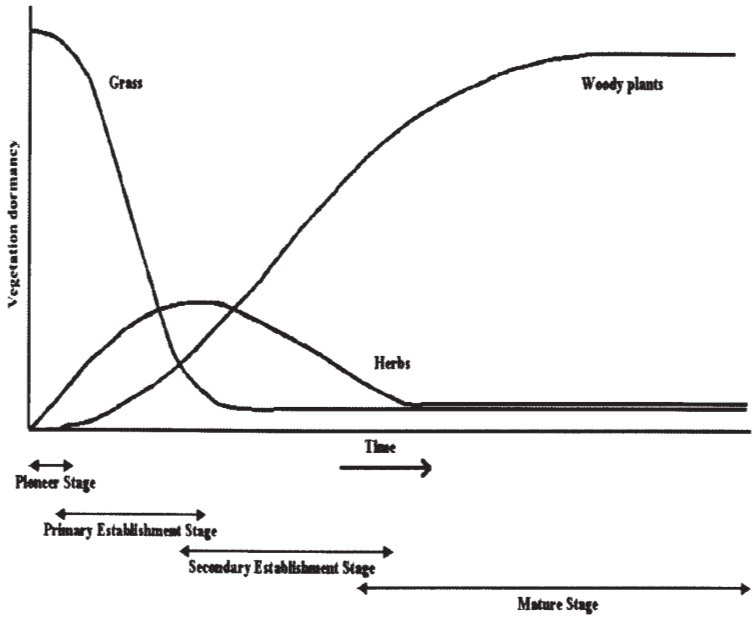

Figure 2 Natural Succession Model (Mak et al. 2004)

After undergoing several succession stages through transitional species, the slope would eventually be occupied by all the climax or subclimax species in a local floral environment. With Geofiber (continuous fiber reinforced soil) providing a stable soil skin of sufficient depth (200mm) for steep Hong Kong hillsides, the thick soil skin favors the establishment of shrub's rooting system and natural invaded plants with sustainable supply of water and nutrients (GEO, 2000). The implanted soil skin will be further stabilized and enhanced by the natural rooting systems to form an integrated matrix within the soil skin itself and also with the underlying in-situ material. It is obvious that succession is not simply a biological process, the soil condition, such as soil depth and fertility, may often be a limiting factor as tall, closed climax communities cannot develop on the shallowest soil (Dudgeon and Corlett, 1994).

METHODOLOGY

To provide a meaningful and useful basis for a design habitat favorable for natural succession, knowledge on the understanding of the characteristics of vegetation distribution in Hong Kong and the regional unique flora diversity around the slopes is essential. A review of the local plant communities and ecosystem of the surroundings is a good starting point.

A proposed methodology for an ecological assessment of a site is as shown on Figure 3. It involve a desktop study of all current available data, site reconnaissance of the considered areas, detailed designed of habitat with vegetation selection and site monitoring with time to collect more information on the process of natural succession. The proposed methodology is described in details in Mak et al. (2004)

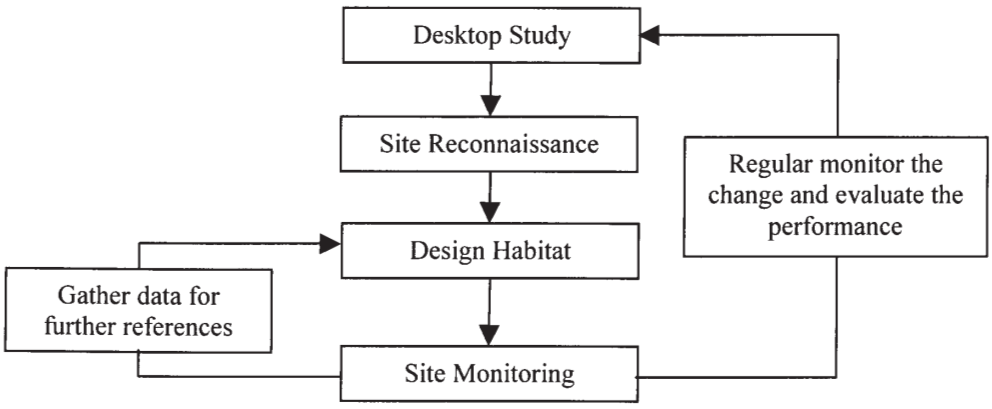

Figure 3 Methodology on Ecological Assessment of Slope (Mak et al. 2004)

CASE REVIEW

With the objective of achieving sustainability, the restoration process of slope is monitored and data are collected from over 40 slopes since 2001. Two cases are described here and the natural regeneration process is correlated to the site specific characteristics. The vegetation conditions on the two slopes are correlated with parameters such as geometries, geology, locations and climatic factors for our analysis; this further enables us a better understanding on the principal factors that influence the progress of natural succession for these sites.

CASE I - CUT SLOPE NO. 6, TSING YI NORTH COATAL ROAD

The Tsing Yi North Coastal Road completed in 2001 facilitates the direct connection between Northwest Tsing Yi and Tsing Tsuen Road. Cut Slope No. 6 is situated at Cheung Shue Tau and is approximately 130m long and with a maximum overall height of 20 meters.

Slope Characteristics

The lowest batter of the Cut Slope is a highly fractured rock slope, with shotcreted surface found at the eastern side of the slope. Rock mesh was placed at the highly fractured rock zones. With the application of Geofiber-CFRS, a stable soil zone is provided on the slope surfaces. Geofiber-CFRS possesses inherent shear strength and durability, which confers high erosion resistance. The thick planting medium provided by Geofiber-CFRS can support the growth of small shrubs (Lui and Shiu, 2004) and favor the multi-species approach for increasing diversity. The angle of inclination of Slope is about 65 degrees.

Location Characteristics

The Cut Slope No.6 at Tsing Yi North Coastal Road is northward facing and is expected to have less solar radiation and with lower temperature in Hong Kong and can affect vegetation growth especially in the dry season. The cooler temperature will reduce the evaporation of plant water and can sustain better plant growth as confirmed by the generally greater extent of woodland and tall shrubland on north-facing slopes in Hong Kong. (Dudgeon and Corlett, 1994). In addition, this slope situates near a coastal area and with local seepages ensures moisture of the plants within the slope. On the other hand, the slope is fairly exposed and close to road traffic. This combines with the age of vegetation are also influential factors on the health of vegetation cover (Lui and Shiu, 2004).

Vegetation Characteristics

This Slope is in close proximity to natural hillsides which provides abundant and diverse seed sources. Air borne seeds can be brought to this site easily and rich flora diversity can be found in Yau Kam Tau. This is exemplified by the presence of large-sized secondary forest, cultivated vegetations and artificial plantation nearby. The unlimited seed sources are significant for the potential regeneration of slope in natural succession (Table 1).

In order to enhance the process of seed dispersal, some flowering or conspicuous shrubs were planted on slope, including *Hibiscus Rosa-sinensis (單托大紅花), Gordonia Axillaries(大頭茶), Duranta Repens (金露花), Rhaphiolepsis Indica (車輪花), Tecoma Staus (黃鐘花), and Calliandra Haematocephala (紅絨球).* Selected grass seeds including Bermuda, Bahia and Manhattan Rye, have been planted for immediate groundcover.

Table 1 Plant composition in the surrounding hillside areas

	Scientific Name
Herbs	*Conyza sumatrensis (蘇門白酒草), Crassocephalum crepidioides (野茼蒿), Eupatorium catarium (假臭草), Euphorbia hirta(飛揚草), Pteris vittata (蜈蚣草), Mikania micrantha (薇甘菊)*
Ferns	*Cyclosorus parasiticus (華南毛蕨), Pityrogramma calomelanos (粉葉蕨)*
Shrubs	*Solanum torvum (水茄), Lantana camara (馬纓丹), Urena lobata(肖梵天花)*
Trees	*Sapium sebiferum , Ficus hispida(對葉榕), Macaranga tanarius(血桐)*

Climatic Factors

Monitoring work on the vegetation development and plant growth commenced since January 2002. In order to understand how the climatic factors (temperature and rainfall) affect the vegetation performance on the slope, meteorological data from the stations at or nearby Tsing Yi were collected, summarized and analyzed. A graphical representation of the relationship of the climatic factors and the overall vegetation performance (vegetation/green coverage) at the Cut Slope No.6 Tsing Yi North Coastal Road is shown on Figure 4.

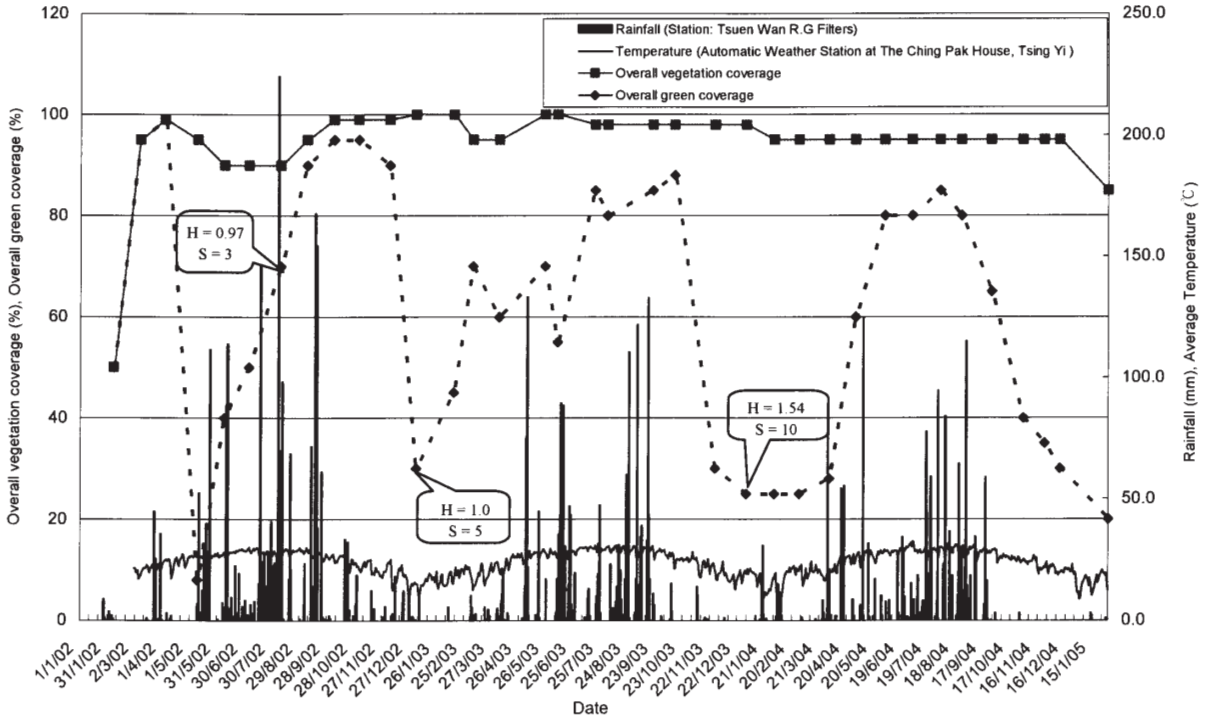

Figure 4 Relationship Graph Between climatic factors and overall vegetation cover / green coverage at Cut Slope No. 6 Tsing Yi North Coastal Road

Figure 4 shows the vegetation coverage (i.e. the area percentage of the slope covered by any types of plants) stays steadily over 90% from February 2002 to December 2004. This is consistent with findings on Geofiber-CFRS by Lui and Shiu(2004) on performance of the method. The green coverage of vegetation (i.e. the area percentage of the overall vegetation that was green) is more responsive to seasonal changes in line with the variation in temperature and rainfall (Figure 4).

In general, the grasses and shrubs remained green each summer and reached a green coverage of approximately 85% during this period. The vegetations usually changed from green to yellowish brown in the beginning of winter (around the end of September) and remained yellowish brown until March of next year. However, the increase in diversity of natural plant species enhanced the green coverage for successive dry winter period.

To gauge the natural succession process, two indicators are used. The Shannon Diversity index (H), an indicator on community composition and Species Richness (S), an indicator on total number of species are used. Based on the proposed natural succession model, the vegetation on the slope is in the pioneer stage. The increasing H and S values (Figure 4) for successive dry season shows that the vegetation cover has higher species diversity and is more advanced in natural succession to provide better green coverage over this dry period. The plant composition observed on Cut Slope No.6 to date originated from the surrounding hillside area is summarized in Table 2. The photographs for the natural vegetations are shown on Figures 5, 6 and 7.

Table 2 Natural Invaded Species of Cut Slope No. 6 Tsing Yi North Coastal Road

Time point	*Shannon Diversity Index (H)*	*Species Richness (S)*	*Observed Naturally invaded plants on cut slope no.6*
Aug 02	0.97	3	*Pteris vittata, Cyclosorus parasiticus, Pityrogramma calomelanos*
Jan 03	1.04	5	*Pteris vittata, Cyclosorus parasiticus, Pityrogramma calomelanos, Ficus hispida, Conyza sumatrensis*
Jan 04	1.54	10	*Pteris vittata, Cyclosorus parasiticus, Pityrogramma calomelanos, Ficus hispida, Conyza sumatrensis, Crasspcephalum crepidioides, Eupatorium catarium, Euphorbia hirta, Urena lobata, Parthenocissus dalzielii and Macaranga tanarius.*

Figure 5 Cyclosorus parasiticus

Figure 6 Ficus hispida

Figure 7 Lantana camara

CASE II – CHO YIU CHUEN – SLOPE FEATURE 11NW-A/C132

Slope 11NW-A/C132 is located east of Kai Kwong Lau in Cho Yiu and sited on a volcanic hillside. The slope is approximately 55m in length with a maximum height of 30m.

Slope Characteristics

The 55m long slope trends in a north-south direction, and with a westward facing. It has an averaged angle of 70 degrees with some areas up to 80 degrees for the lower part of the

slope, which comprises moderately weathered rock of about 7 meters high. The upper part of the slope is at an angle between 40 to 55 degrees with shotcrete. At the southern end, the slope has a maximum height of 30 meters.

Locations and Vegetation Characteristics

The slope faces west and is fairly exposed. The proximity of the building indicates that sunshine and wind exposures may be curtailed from the westerly direction. There is a natural hillside east of this cut slope. A car park situated just beside the slope with limited exposure to road traffic. The natural hillsides east of the slope provide diverse seed sources for ecological succession and composes of exotic plantation and native forest tree species as shown in Table 3.

Table 3 Plant composition in the surrounding hillside areas for Cho Yiu Chuen

	Scientific Name
Shrubs	*Psychotria asiatica (九節), Lantana camara (馬纓丹)*
Trees	*Acacia confuse (臺灣相思), Leucaena leucocephala (銀合歡), Macaranga tanarius (血桐), Machilus sp.(潤楠屬), Ficus variegata var. chlorocapa (青果榕), Cratoxylum cochinchinense (黃牛木), Mallotus paniculatus (白楸), Pinus massoniana (馬尾松)*

With this close proximity to the natural seed sources, only grasses were planted, including Bermuda, Bahia and Manhattan Rye. After nearly three years, root of grasses were found within the Geofiber skin, which believe to further reinforce the soil mass for protection of slope. The living plants can alter their color seasonally, but they possess the safety characteristic of further enhancing the protective cover by binding the soil together with nature roots development.

Climatic Factors

Monitoring work on the vegetation and plant growth and development commenced since May 2002. Similarly, to understand how the climatic factors (temperature and rainfall) influence the vegetation performance on this slope, meteorological data from nearby Tsuen Wan stations were collected, summarized and analyzed. A graphical representation of the relationship of the climatic factors and the overall vegetation performance (vegetation/green coverage) at the Cho Yiu Chuen is shown on Figure 8.

Figure 8 shows the vegetation coverage stays steadily over 90% from May 2002 to December 2004. As expected the green coverage of vegetation changes in response to rainfall and temperature changes in a seasonal cycle. As observed in the previous slope, the trend on green coverage of this slope behaves similarly with successive green coverage in winter(dry season) increased as a result contributed by natural invaded evergreen exotic or native vegetation. The rate of natural invasion is relatively fast for this slope. The Shannon Diversity

Index (H) and Species Richness (S) of the slope are shown in Table 4. The photographs of some of the natural invaded plants are shown on Figures 9, 10 and 11.

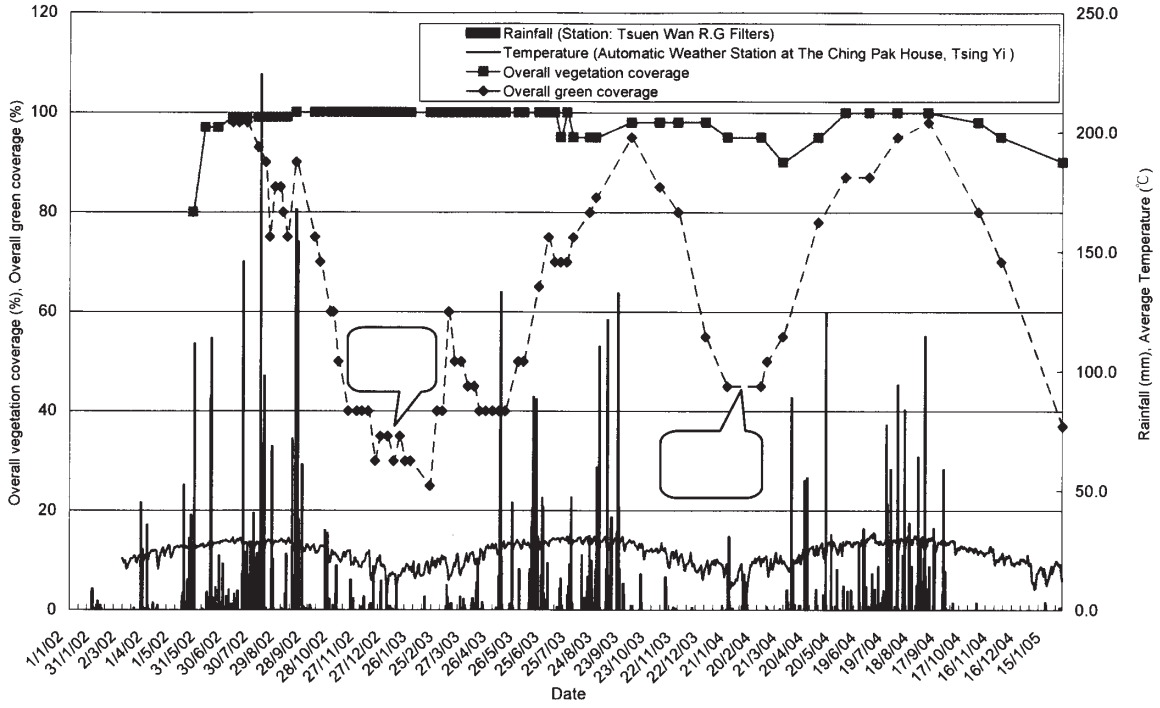

Figure 8 Relationship Graph Between climatic factors and overall vegetation cover / green coverage at Slope No. 11NW-A/C132

Table 4 Natural Invaded Species of Slope No. 11NW-A/C132 at Cho Yiu Chuen

Time Point	Shannon Diversity Index (H)	Species Richness (S)	Observed naturally invaded plants on Slope 11NW-A/C132
Dec 02	1.32	4	***Herbs***: *Eupatorium catarium, Eclipta prostrate, Pancium ssp., Melia azedarach*
Dec 03	2.05	14	***Herbs:*** *Eupatorium catarium, Eclipta prostrate, Pancium ssp., Melia azedarach, Lygodium sp.,Emilia sonchifolia, Solanum nigrum* ***Ferns:*** *Dicranopteris pedata, Colysis sp., Nephrolepis sp.* ***Shrubs:*** *Lantana Camara* ***Trees:*** *Macranga tanarius, Melia azedarach,Ficus hispida* ***Climbers:*** *Wedelia trilobata, Mikania micrantha*

Figure 9 Ficus hispida

Figure 10 Macranga tanarius

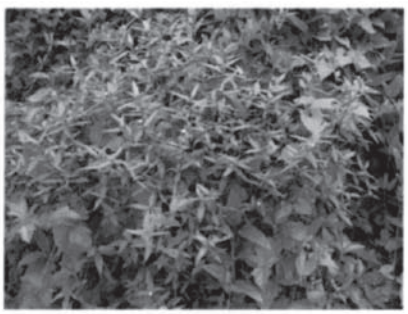

Figure 11 Eclipta prostrate

SUMMARY OF FINDINGS

Cases review for Geofiber slopes in Hong Kong suggests that species composition change due to natural invasion of native or exotic herbs, ferns, climbers, shrubs and seedling of woody plants with time. Through these baseline studies, the findings are :

1. They show that natural succession is taking place on the Geofiber-CFRS slopes. With favorable environmental factors and an ecologically sustainable and stable vegetation cover, natural succession can take place and will take place in time.
2. The green coverage is maintained in a seasonal manner. As with more natural invaded climbers, ferns, shrubs or woody plant seedlings that are evergreen in nature residing on the slope face with time, an expected trend of higher green coverage in the critical dry season is likely to be expected. The increase in species diversity (H) and species richness (S) contribute to the vegetation and green coverage for slopes. When this diverse community coexists and modifies the microclimate of on the slope surface within the Geofiber skin, natural succession will further be enhanced, and minimal maintenance is expected.

CONCLUSIONS

An Eco-engineering approach to slope treatment is proposed for the characteristic hilly terrains of Hong Kong. The proposed approach and the suggested model offer to extend the traditional engineering assessment of slopes during the planning and design stage to encompass the ecological setting of the environment for natural succession to occur. Cases on using Geofiber with proper design habitat can have satisfactory result on maintaining safe and sustainable green slopes. Studies on slopes with Geofiber skin have further exemplified that seasonal changes of vegetation color for the slope do maintain good integrity in the root system in the dry season for restoration works to begin in the next wet season.

These baseline studies on Geofiber slopes are useful indicators for slope behaviour and appearance for future design and planning expectations. A baseline reference is established on minimal interference for maintenance with corresponding expected seasonal green coverage on the slope faces. Nature will take its own course and will restore to ecologically balanced states given time with sustainable evergreen vegetation eventually reside to create the ecological habitat. Ongoing monitoring and data collection is of paramount importance for our human wisdom on knowing our earth better.

Concurrently, with the information obtained on this baseline approach, artificial means of promoting vegetation growth such as watering or addition nutrients can also be introduced during dry seasons or in periods of draught to enhance green performance on slope faces. Indeed, any artificial aids are useful but recurring actions are often not cost effective nor sustainable. Promoting natural restoration should take the first priority.

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AN ALTERNATIVE METHOD OF SOIL NAIL DESIGN IN HONG KONG

Martyn J. Smith [1], Ronald P.M. Li [1], Leslie H. Swann [1].

Abstract: In February 2003 the Geotechnical Engineering Office, (GEO) of the Civil Engineering and Development Department, (CEDD) ran a workshop on Cost-Effectiveness in Soil Nail Design. During that workshop, the authors broached the idea of an alternative method of soil nail design based on the 'Working Bond' theory, which is similar in its approach to the design of soil nails in rock and for rock bolts. In essence, the bond length of the soil nails for the same Factors of Safety (FOS) behind the same critical slip surface is reduced when using the working bond approach as opposed to the conventional approach, leading to significant costs savings. In June 2004, the authors gave a presentation to the Research and Development Committee of the HKIE on the Working Bond theory and how its application could save the Government of the order of 30-40% of the costs of LPM Works involving soil nails. Following the presentation, it was agreed that as $HK400M was spent annually on soil nailing, the industry should do more to ensure value for money, and further research into soil nail design as appropriate. This paper introduces the working bond method and provides worked examples comparing required lengths of soil nails using the conventional approach and the working bond approach, with areas of further research and suggested ways forward.

INTRODUCTION

The conventional method of soil nail designs in Hong Kong is based on the general formulae proposed by Schlosser and Guilloux (1981) and Carter and Gigan (1983), where the allowable pull-out resistance of the soil nail bar is determined by calculation of the overburden pressure at the mid point of the bond length portion of the soil nail, which is the length of soil nail behind the critical slip surface, which is obtained by slope standard stability software, SLOPEW, PCSTAB5, and the like, (Figure 1). The standard formula will be a variation of:

$$Ta = \{\pi d\, c' + 2\, D\, \sigma v \tan\emptyset'\}l$$

Where Ta = allowable pull-out resistance

πd = soil nail perimeter; c' = effective cohesion of the soil material

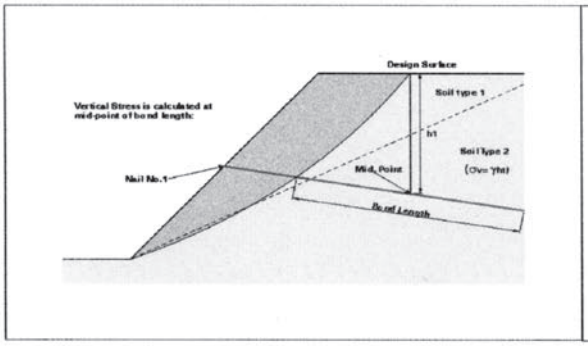
D = soil nail diameter; Tan $\emptyset'$ = coefficient of apparent friction of the soil

σv = effective vertical stress at the mid point of the bond length.

l = Bond length

However, there have been some previous studies on soil nails and ground anchors which have concluded that the pull-out capacity is independent of depth (Heymann et al, 1992; Schlosser 1982). The GEO is itself currently undertaking laboratory testing simulating soil nail pull-out tests at different vertical stress and soil moisture conditions with interesting results. (MGS, 2005).

[1]Jacobs Babtie, Babtie Asia Ltd.



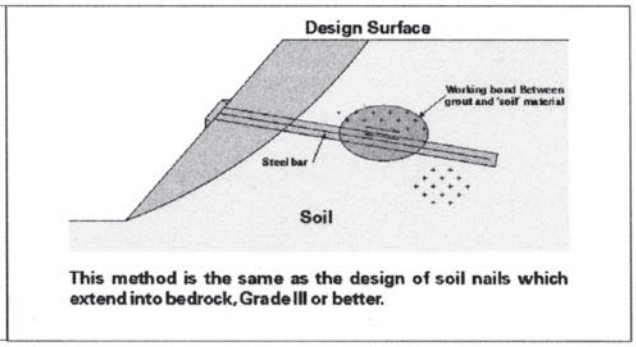


Figure 1: Conventional; design approach. Figure 2: Working Bond design approach.

This paper looks at the alternative method of using the actual working bond, which can be measured directly in the field by controlled pull-out tests, to design soil nails.

WORKING BOND

The working bond method of design is similar to the design of soil nails which extend into bedrock, defined as grade III or better rock. The concept considers the critical factor affecting the pull-out resistance to be the strength of the grouted anchored zone and the surrounding soil mass. This method will be relevant particularly to saprolite material which is derived from the in-situ weathering and decomposition of pre-existing igneous and volcanic rock, retaining the intrinsic structure of the parent rock, which bonds the mineral grains together. The method may also be suitable to transported soils, for example colluvium, although here the original rock structure will have been destroyed during formation. The reason why saprolites may be particularly suitable is that when drilled the soil nail drill hole becomes essentially free-standing and self supporting due to the arching effects and the presence of the saprolitic texture bonding the grains together. In this case, the vertical stresses at the soil nail are greatly reduced. The working bond method is used for soil nails extending into Grade III, II, or I rock, and so should be applicable to grade IV and V rock. The working bond will be dependent upon a number of factors, including parent rock type, original mineralogy, moisture content and pore water pressure.

Soil nails are currently accepted or rejected based on a pull-out test. It may be more appropriate, therefore, to complete pull-out test at an earlier date to provide actual pull-out resistance values for the design itself, or allow for changes of length in response to test results during the Works. It seems reasonable not only to justify the acceptance of the soil nail design on pull-out test results, but to base the actual design calculations on actual pull-out resistance also.

It must be noted that the working bond method looks at the design of the bond length of the soil nail only, (Figure 1 above). The free-length of the soil nails, which is that portion of nail within the 'unstable zone' in front of the potential critical design slip surface, is not affected. It is therefore clear that the 'working bond' method does not in any way change the FOS value nor the location of the critical slip surface. The working bond method simply looks at the optimization of the bond length of the soil nail needed behind the potential critical slip surface to provide the required horizontal force to resist sliding along that potential critical failure surface.

Determination of the Working Bond using Pull-out Test Results.

There are a number of methods available for the determination of the working bond for various soils. The obvious method, however, and probably the most accurate, will be by carrying actual pull-out tests on site, (possibly during the site investigation phase), at the specific locations, (height, spacing, depths) that the working soil nails will be installed. For multi-layered geological models, pull-out tests at various depths as related to different soil materials interpreted from the site investigation information and shown on the specific slope geological cross –sections should be carried out.

The draw-back with field pull-out tests to date has been the accuracy and control of the grouting for the pull-out test section. The length of the tested section is generally taken as the last 2.0m of the bond length of the soil nail. However, accurately ensuring the grouted length and thus the calculated working bond requires some further research. The choice of packers used and grouting methodology needs to be considered for accurate and repeatable results. It is understood that the GEO is currently looking at this and ways in which the pull-out tests accuracy can be improved.

An alternative to field trials is laboratory based pull-out tests which can be carefully controlled to simulate actual depths of soil nails and soil properties. However, this method is currently only possible for re-moulded or compacted materials and so the properties of the saprolitic soil bonding and intrinsic parent rock structure will be lost.

Worked example using field pull-out test results:

As an example, if we assume that the soil nail pull-out tests completed on a subject site are accurate, we can compare the length of soil nails and calculated pull-out resistance for an actual case, using the conventional approach and the working bond approach.

The formula for determining the pull-out resistance for actual tests is the same formula as designing soil nails which extend into rock, ie-

$$Twb = wb^* \pi Dl,$$

where Twb = allowable pull-out resistance
wb = working bond between grout and soil/rock, in N/mm2
πDl,= surface area of the nail bond length

In our recently completed LPM Agreement CE17/2000, one of the features comprised a relatively small, around 10.0m high feature in Shatin comprising completely decomposed granite with a slope angle of around 65°-70° with an intervening berm and chunam surface. Five rows of 32mm soil nails in 100mm diameter holes were designed with three pull-out tests. The pull-out tests were specified to be taken to 90% of the yield stress of the bar in order to try to get the nails to fail, ie, to induce shearing of the anchored zone.

The actual locations of each pull-out test are not in one cross-section, (see Figure 3), but are at different levels in different soil nail rows around the slope. In this way the slope forming materials at different levels and locations of the whole feature are tested. 'Movement' occurred on the three test nails at the following loads:

Row c = TN1 at 304kN (top test nail highlighted)
Row d = TN2 at 155kN (middle test nail highlighted)
Row e = TN3 at 205kN (lower test nail highlighted).

Test Nail TN1 was located in an area of the slope where large boulders were present at the surface, and the last 500mm of the test nail is recorded to have entered a grade III corestone within the slope. The other two tests were completely in CDG and all were grouted over a 2.0m test length. At these pull-out loads, the actual working bonds indicated are:

TN1 = 0.4838 N/mm2; TN2 = 0.2467 N/mm2; TN3 = 0.3263 N/mm2

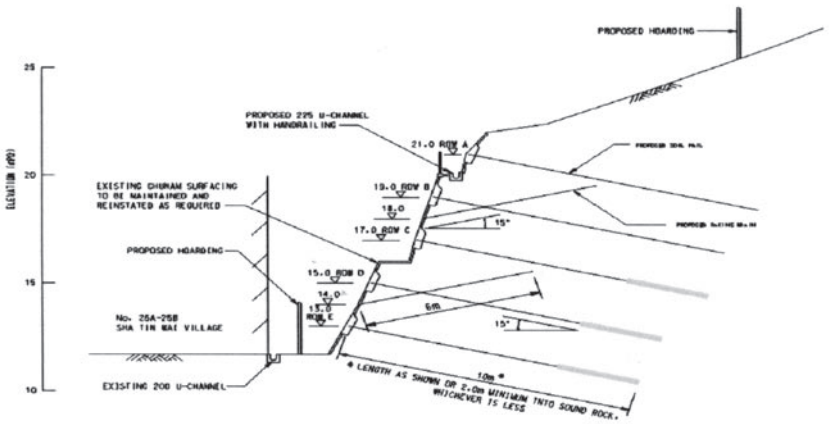


Figure 3: Case history for pull-out tests for a specific slope.

In addition, using the conventional approach, the maximum allowable pull-out resistance for the same 2.0m grouted section, without any FOS applied to the resistance (i.e. the ultimate pull-out resistance), would be:

Row c = TN1 = 50.07kN
Row d = TN2 = 55.84 kN
Row e = TN3 = 63.552 kN.

If the results of TN1 are discounted due to the 500mm of Grade III at the end, it is apparent from the results of TN2 and TN3, that the conventional approach in this case is very conservative, by a factor of 3. The length of the working nails in this feature was 10m long. However, if the working bond had been used in the design to determine the bond length behind the same critical slip surface, then the length could have been reduced to 6.0m. This would have resulted in a **40%** saving on the length, with the associated cost savings.

Determination of the Working Bond using the Standard Penetration Test, SPT.

There have been a number of published papers providing recommended rock / grout working bond values for design of anchors in the ground. A well known source is Littlejohn and Bruce (1977), who suggested a relationship between the working bond and SPT 'N' value. The relationship suggested is:

Working Bond = 0.01N. (N/mm2)

Other practitioners have also suggested relationships, Suzuki et al (1972, from Littlejohn), suggested:

WBult = 0.007N + 0.12 (N/mm2).

Various parties overseas are also using SPT's to determine 'design' pull-out strengths of soil nails from material properties, e.g the Eurocode Annex A, Draft 8: Annex A – 'Design of a soil nailed structure'.

The formula suggested in this paper, therefore, for determining working bond for a soil using SPT results is:

$$\text{Pull-Out Resistance} = \left(\frac{0.01 \times N}{2}\right) \times \pi \, D \times L$$

Here, a Factor of Safety of 2.0 has been applied to the SPT N value to allow for:

(a) Variations in the soil properties
(b) Variations in the SPT number, although either the average or lowest SPT per soil type tested could be used.
(c) It equates to the FOS= 2 which is applied to the 'conventional approach' formula for determining allowable pull-out resistance.
(d) It allows for any effects of high moisture content or high groundwater. Laboratory studies by Clouterre (1991) have shown that the maximum pull-out force can be divided by 2.0, or more, when the moisture content is increased from the optimum water content (proctor) to saturation moisture content.

Based on the above, for a range of given SPT numbers, design charts can be developed to show corresponding bond lengths of soil nails behind the critical slip surface:

Typical Soil	**Colluvium / RS**	**CDT**	**CDG**
End Force Required (kN)	**SPT = 10: Bond Length Required*:**	**SPT= 20: Bond Length Required*:**	**SPT = 30: Bond Length Required*:**
25	1.59m	0.80m	0.53m
50	3.18m	1.59m	1.06m
75	4.77m	2.39m	1.59m
100	6.37m	3.18m	2.12m
125	7.96m	3.98m	2.65m

*Bond length behind the critical design slip surface

When using this method for soil nails extending into bedrock, a minimum penetration is usually specified to be 2.0m, irrespective of the calculated length. For this method in soils, it may be prudent to apply a similar minimum bond length behind the critical slip surface of say 3.0m.

Worked examples using SPT 'N' value in soil nail design for Working Bond:

Using the same feature as an example, (Figure 3), the ground investigation boreholes indicated generally extremely weak, completely decomposed granite, recovered as clayey silty gravel. Using the above SPT tables, an SPT value of 30 could be adopted. Completely

decomposed granite is dense and so an N-value of 30 is conservative. The actual as-built design consisted of 5 rows of working nails:

Soil Nail Row	Required end force	Bond Length (conventional)	Total Length (conventional)	Bond length using SPT=30	Total length SPT=30	SAVING on bar length and costs
A	20 kN/m	4.6m	10m	0.42m use 3.0m	(10-4.6)+3=8.4m	16%
B	28kN/m	6.6m	10m	0.59m use 3.0m	6.4m	36%
C	48kN/m	8.6m	10m	1.018m use 3.0m	4.4m	56%
D	60kN/m	9.0m	10m	1.27m use 3.0m	4.0m	60%
E	71kN/m	9.2m	10m	1.51m use 3.0m	3.8m	62%

It is clear from the above that the working bond length would give an average saving of 46.0%, assuming the same number of soil nails per row. This is comparable with the 40% savings indicated by the actual pull-out resistance lengths indicated above.

Using the working bond it is important to make sure soil nails are long enough to intercept and additional potential slip surfaces behind the surface chosen to be the critical design surface, which may vary between existing FOS= 1.0 to 1.40, depending upon specific slope situations and practitioners. It is clear from the above tables, however, that the working bond method provides significant cost and programming savings over the conventional approach for the same FOS and potential design slip surface.

Effects of High Groundwater / Pore-Water Pressure / Moisture Content.

It is clear that the use of the actual working bond in soil nail design would be significantly improve the cost – efficiency of future designs. However, the effects of pore-water pressure and high groundwater need to be assessed. To illustrate this point, an example is provided below of a slope where the soil nails were pulled completely out of a slope, where a high ground water table was present. The example outlines the expected pull-out resistance using the conventional design approach and the actual pull-out resistance obtained by the pull-out tests on site.

The feature comprises a relatively gentle slope of around 35° with a maximum height of 14m, with an intervening berm. The desk study revealed records of previous small surface failures and evidence of seepage in the lower batter. The ground investigation revealed a thin layer of fill underlain by CDG, with pockets of colluvium in areas, associated with a pre-existing drainage valley. A large fill slope, comprising re-compacted fill approximately 20 years old, existed immediately adjacent to the feature. The design of the upgrading works included three rows of soil nails in the lower batter together with individual soil nails in boulders in the upper batter. Pull-out tests were completed in the lower batter and three of the tests failed. The table below gives the design and actual pull-out resistances. The conventional overburden method was used in the soil nail design. Comparisons with soil nail

length if the working bond had been used based on the actual pull-out test results are also provided:

Pull-out test	Parameters	Calculated Ta (kN) (allowable pull-out resistance)	Test Failure load (2m section) (kN)	Calculated Required Soil Nail Bond Length / Total length (m)	Working Bond pull-out results (N/mm2)	Required Bond Length (Working Bond) / total length
2	CDG 5kPa;35°	73.57	94.12	11.80 /15	0.150	1.55 / 4.75m
3	CDG 5kPa;35°	67.80	111	11.0 / 15	0.177	1.20 / 5.20m
4	CDG 5kPa;35°	75.60	110	10.50 /15	0.175	1.36 / 5.86m

It is clear from the above that while the high groundwater table did influence the pull-out tests, with three of the tests failing to reach the required test loads, due to the relatively low pull-out resistance required, the soil nails would have been significantly shorter using the Working Bond method. The long soil nails and bond lengths required in this particular case were due to the low surface angle of the feature and the subsequent low overburden height when calculating the vertical stress acting along the bond length.

DISCUSSION.

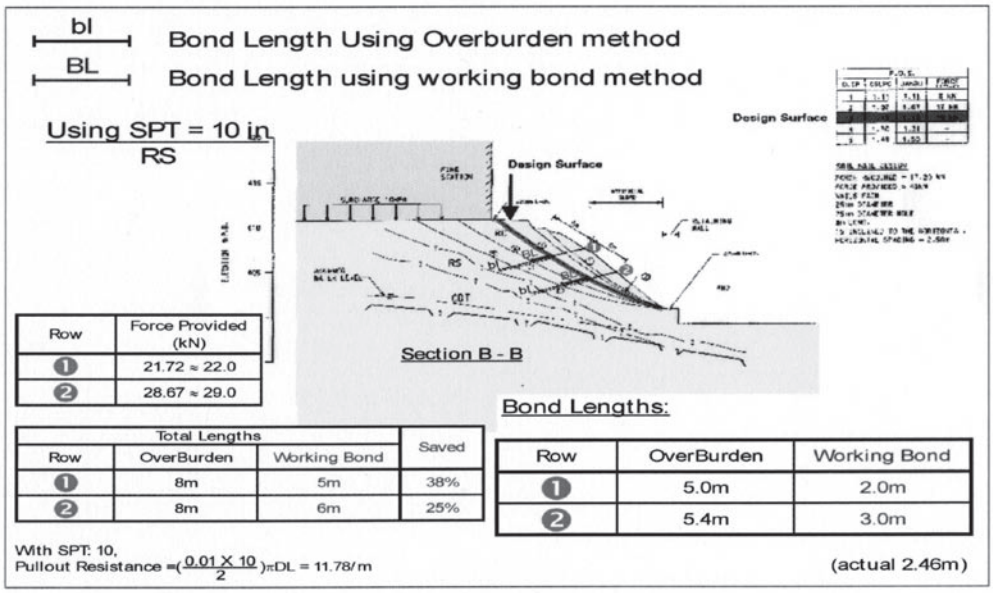
(a) Deep Seated Slope Failures - A number of practitioners have commented that shorter nails will not intercept the possible deep seated failures. However, the potential for a deep seated failure needs to be identified during the site investigation. A thorough desk study to determine the likely ground conditions, evidence of previous failures in the area, specific characteristics of the anticipated ground model, and detailed engineering geological and geomorphological site inspections should be undertaken. The ground investigation should be adequately planned and carried out, and the presence of relict joints, thick clay and kaolin seams, different soil contacts, piping where water may be present, perched water horizons and geological characteristics which could cause deeper seated failures should be identified. It is not appropriate to simply do a basic ground investigation and install excessively long soil nails in the hope of preventing deep seated failures from occurring, The nature of the deep seated failure is that even very long nails may not intercept the potential pre-existing failure surface. Research indicates that the vast majority of slope failures have taken place in Hong Kong within the upper 5-10m of a slope surface. Excessively long nails are, therefore, considered undesirable and unnecessary in the vast majority of cases.

(b) Groundwater levels, pore-water pressure and moisture conditions - It is clear that the pull-out resistance will be greatly affected by the moisture content of the slope forming materials. This needs to be investigated further with more actual pull-out tests completed on features with elevated ground water, together with SPT's in boreholes in the same features, to develop a better understanding and correlations between pull-out resistance, SPT values and soil type above and below the groundwater table. However, providing that it is possible to

carry out pull-out tests at the investigation stage, the effects of the elevated pore-water pressures / moisture contents should be able to be taken into account in the design.

(c) Pull-out tests inaccuracies – It is clear that the current pull-out test method may introduce inaccuracies into the working bond calculations. The type of the packer which is used to control the grouted section to 2.0m may not be successful, neither the full contact between grout and soil. This has been recognized for some time and the GEO are now undertaking several studies on soil nail design issues including the use of various packer systems, both inflatable types and the current 'manual' type.

(d) Existing foundations at the crest - One of the major advantages of using the working bond approach is the fact that many slopes upgraded by soil nails have structures at the crest. These structures have foundations to varying depths and varying degrees. In the example given below, use of the working bond method removes the soil nails outside of the foundation zone of the crest building. This can be very important for older buildings with shallow foundations.



Row	Force Provided (kN)
❶	21.72 ≈ 22.0
❷	28.67 ≈ 29.0

Total Lengths			Saved
Row	OverBurden	Working Bond	
❶	8m	5m	38%
❷	8m	6m	25%

Row	OverBurden	Working Bond
❶	5.0m	2.0m
❷	5.4m	3.0m

CONCLUSIONS

The following conclusions can be drawn from the above:

1. The use of the Working Bond between the anchored zone and the surrounding soil mass will provide significant cost savings for future soil nailed slopes, allowing programmes to be accelerated and possible impacting on Governments letting of more LPM Agreements.
2. The use of Working Bond may be a more appropriate and realistic method of designing soil nails than the currently adopted conventional overburden approach.
3. The design of the actual pull-out test needs to be improved, with better packer's and grouting methods to make sure the design length is correct for calculation purposes and that there are no voids in the grouted section. If pull-out tests are going to be used during the GI stage for design data, the test should be designed to ensure that the nails can actually be pulled out. In this situation a shorter bond

length may be more suitable, possibly fixed at 1.0m, using improved packers and grouting to ensure an accurate and repeatable bond length is produced.

4. The ground investigation stage of the slope assessment could include trial pull-out tests to provide design data. The GI pull-out tests could be designed to vary in both depth and location, possibly along 2-3 different cross-sections down a slope, to provide working bond information for different materials in the ground model.
5. Standard penetration tests should be carried out in ground investigation boreholes as a matter of course for future use for design purposes.
6. Correlations need to be developed between SPT 'N' numbers and actual pull-out resistance in different materials and hydro-geological regimes for added confidence.
7. More studies should be undertaken to confirm the applicability of the method for features with high groundwater, elevated pore-water pressures and high moisture contents. However, high groundwater and very moist conditions should be identified at the ground investigation stage and so actual pull-out tests can be carried out in these materials on site for design parameter.
8. The potential for deep seated failures should be identified at the ground investigation stage and appropriate designs completed. Long soil nails should not be installed as a matter of course in case the potential for deep seated failure exists. This is not an appropriate design method and is inefficient and carries significant cost implications. It is note that the ratio of slope height to soil nail length has gradually increased over the years between the late 1980's and the present day, from around 0.6 to over 1.0, with arguments for the possibility of deep seated failures included in the rational. This is considered to be an incorrect design approach.

ACKNOWLEDGEMENTS:

This paper is published with the kind permission of the Head of the Geotechnical Engineering Office, and the Director of the Civil Engineering and Development Department.

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INNOVATIVE STABILIZATION OF A FILL SLOPE UNDER THE LPM PROGRAMME INVOLVING STRUCTURAL SUPPORT WITH PASSIVE MITIGATION MEASURES

Martyn J Smith[1], Ronald P. M. Li[1], Leslie H Swann[1]

Abstract: The LPM Programme strives not only to improve the stability of slopes and retaining walls throughout Hong Kong, but also to improve their aesthetic appearance. To complete both requirements while maintaining the aesthetics of the local area for Country Park fill slopes can often stretch the skills of the designer. This paper outlines an example of an innovative design employing a combination of limited direct stabilization measures, in the form of soil nails and concrete grillage, in association with a passive system comprising a ring-net type debris flow catch fence along the toe to mitigate any debris flow from the remainder of the site. The site is located in a remote area adjacent to Lion Rock Country Park along the Wilson Trail in Shatin, the nearest vehicular access being around 5.7km away.

INTRODUCTION

Fill slope feature 7SW-D/F271 is located along the Wilson Trail adjacent to Lion Rock Country Park, (Figure 1). The feature was selected in the Landslip Preventive Measures (LPM) Programme which included investigation, design and construction supervision. However, as this feature was covered with trees and local vegetation, and is in constant view by the public using the Wilson Trail, an innovative design which did not rely on excavation was considered most appropriate. The paper outlines the design consideration process that was undertaken to rationalize the soil nail design, together with highlighting the construction issues of the proposed upgrading works in a remote area of Hong Kong.

SITE DESCRIPTION

The feature is located adjacent to Lion Rock Tunnel Road, (LRTR). The nearest vehicular access, however, is from Tai Po Road (Shatin Heights) some 5.7 km to the west, (Figure 1). A narrow winding stairway exists to the north through a small village which runs along a water course which flows down towards LRTR.

The slope is 60m across at the crest, which is marked by a catchwater, with a slope height of 30m and a surface slope angle of 35°-45°. A detailed ground investigation was completed comprising 23 trial pits together with a series of GCO probes. The location and lack of vehicular access, plus the need to minimize vegetation disturbance, precluded a borehole ground investigation. However, trial pits were taken to extended depths of 5.0m (Figure 2).

[1] Jacobs Babtie - Babtie Asia

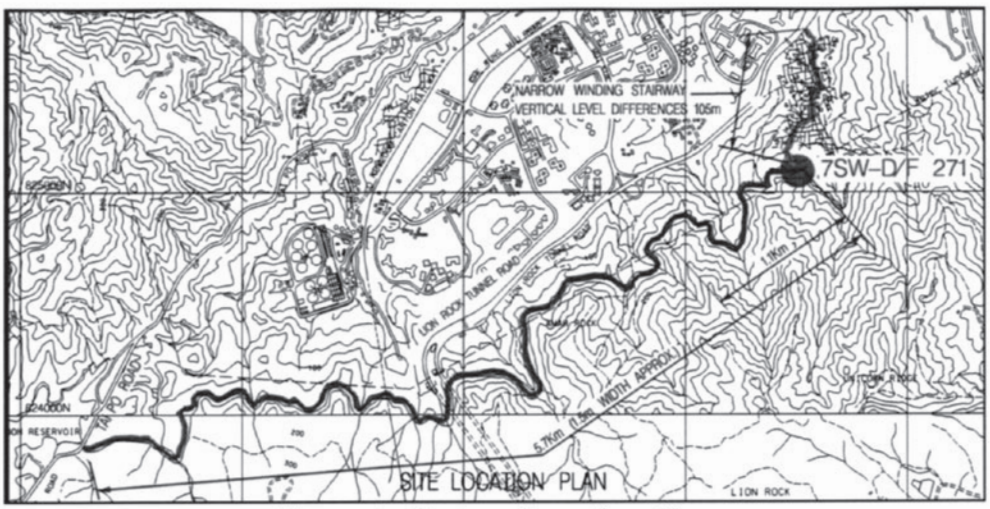


Figure 1: Feature Location Plan

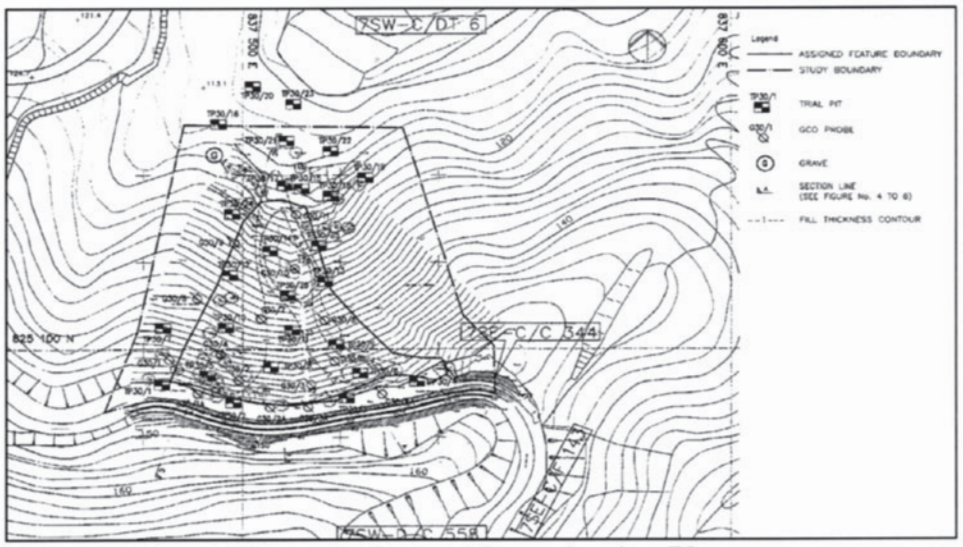


Figure 2: Ground Investigation Plan

The ground investigation revealed a general sequence of strata comprising silty, sandy gravelly fill of up to 2.80m in thickness, underlain by a thin layer of colluvium, followed by topsoil over the in-situ completely decomposed granite (CDG). The fill was found to be very loose (average relative density of 74%), and generally 1.0m or less in thickness over the majority of the site, with localized areas towards the centre of the feature where the fill reached its maximum thickness of 2.80m, (fill isopachytes are shown on Figure 2).

DESIGN

The design considerations for this feature were complicated by the fact that access was very limited. The designers had to provide a suitable means of upgrading a fill slope while at the same time limiting damage and disturbance to the vegetation on the site and the surrounding area. In addition, the Wilson Trail at the crest is used by members of the public for recreational uses and complaints during and after the Works needed to be minimized. As this feature was not large, cost considerations were not the main factor, while environmental considerations were considered to be a major issue.

The initial option assessment was held in March 2001 before the completion of ground

investigation (GI). At that time three options were proposed; excavation and replacement with compacted rockfill; construction of a debris flow barrier or retaining structure at the toe; excavation and re-compaction. While the excavation options were considered inappropriate due to the accessibility of the feature, the use of a debris flow barrier, i.e. a passive solution only, was considered to be insufficient.

Following completion of the GI and during a second option assessment meeting in August 2001, soil nailing, i.e. direct structural stabilization in the fill, was adopted.

At this point of time the GEO were developing their Interim Guidelines on the use of soil nails to stabilize loose fill slopes under the LPM Programme. Sensitivity analyses with various shear strength parameters for existing fill (as detailed on Table 1), together with variations of effective and total stress, hydro-geological conditions and factors of safety (FOS = 1.40, 1.10 & 1.0) were completed and the effects of these variables on the resulting design of the soil nails were compared.

Table 1: Fill's Shear Strength Parameter used in Sensitivity Analyses

Cohesion, c'/ c (kPa)	Friction Angle, Ø (°)	Remarks
0	35	Perched water infill (empirical)
0	21	(following HKIE 1998 suggestions)
0	11	DRY
0	11	fully Saturated
Su = 5(undrained),10	-	-
0	11	with water table

In the detailed soil nail design, the adopted shear strength parameters of existing fill of c' = 0kPa & ϕ = 11° representing saturated state and a minimum FOS = 1.40 to represent the worst condition were adopted. The design was completed in September 2001 and included up to 14m long soil nails at 1.50m spacing covering the whole area of the fill, together with a concrete grillage linking the soil nails, and an independent crest mass concrete wall to isolate the catchwater in case of slope failure. The extent of the soil nails and grillage are shown on Figure 3.

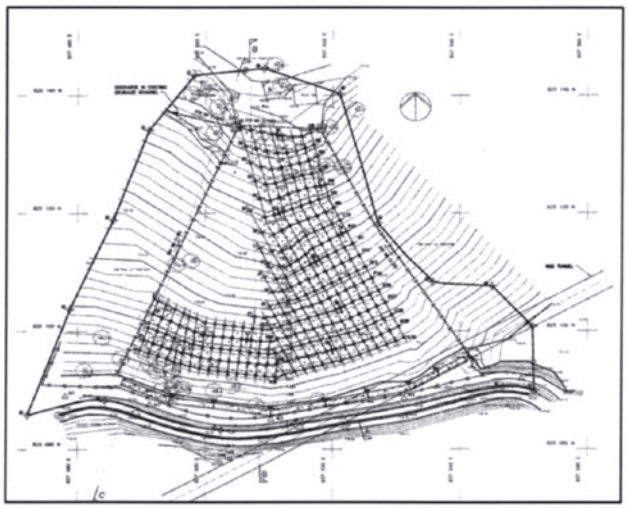


Figure 3: Soil Nail Design covering the whole fill area

In addition, the run-out distance of a potential liquefied fill mass / debris flow type

failure was also determined using aerial photograph interpretation, topographical maps and site inspections. It was considered that the farm buildings and the local village below the slope could be at risk from a fill slope failure.

However, when considering the numbers and extent of the required soil nails, and the remoteness of the site, it was decided to further review the scope of the detailed design. This review included the following:

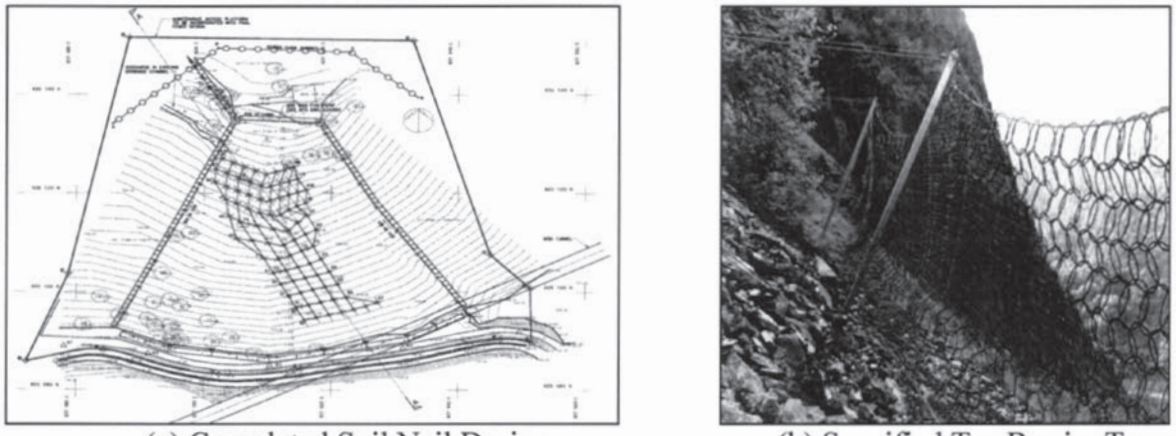
1. A possible reduction in soil nail numbers, to be installed into areas where the fill was >1.0m only.
2. A combined design using soil nails and a debris flow fence could be considered.
3. Additional ground investigation information to confirm the relative compaction (RC) of the existing fill.

In response to this, additional GI was undertaken with Sand Replacement tests and laboratory compaction tests. The additional GI confirmed a RC range of 64%-86%, with an average of 74%. The very loose nature of the fill was considered to be as a result of its formation, with cutting to form the catchwater resulting in material being randomly tipped downslope without any form of control nor compaction.

The use of a debris flow fence was also investigated and it was considered that a ring net type debris flow toe barrier could be used. This would be designed to accommodate 150% of the fill volume to be left in place, to allow for any delays in maintenance works.

After the detailed review, the final design comprises soil nails and concrete grillage in areas where the fill was >1.0m in thickness, with a debris flow catch fence being provided to cater for any minor failure in areas where the fill was <1.0m in thickness, (Figure 4). This allowed a review of the soil nail design, and soil nail lengths were designed at between 8.0m and 10.0m, while the soil nail spacing was increased to 2.0m where the fill was up to 2.0m thick, and 1.50m spacing where the thickest fill was >2.0m.thick. The design also included the mass concrete crest wall along the catchwater and surface drainage.

Following issue of the Interim Guidelines on the use of soil nails to stabilize loose fill slopes under the LPM Programme (GEO 2001), the minimum FOS for soil nails in fill was reduced to 1.10 when liquefied fill parameters of c = 0kPa, Ø=11° were used in the design. This FOS revision was used in the final soil nail design outlined above.



(a) Completed Soil Nail Design (b) Specified Toe Barrier Type

Figure 4: Completed soil nail design and specified toe barrier type

CONSTRUCTION

In light of the nature of the works and the remote location of the slope, there were a number of immediate concerns, as shown in Table 2, to be addressed by the Resident Site Staff (RSS) and the Contractor.

Table 2: Summary of Concerns and Issues

CONCERNS	ISSUES
Progress	Transportation of Plant and Materials
Quality	Concreting
Safety	Sanitation
Environmental Issues	Emergency Facilities
Complaints	Electricity and Water Supplies

Access

Vehicular access was not readily available for transportation of materials to the feature. However, in light of the wide-ranging scope of the upgrading works, vehicular access was desirable. In order to provide this access, the contractor was required to strengthen a number of existing platforms along the catchwater in order to widen the road for access by a small farm truck type vehicle. This was successfully completed and assisted with the delivery requirements, as shown below, in Figure 5.

Figure 5: Temporary Improvement Works along Wilson Trail

Construction of the Concrete Grillage

One of the major challenges to the successful completion of the upgrading works was the concreting works for the concrete grillage linking the soil nails together. A number of options were considered, including dry mix by shotcreting methods, hand mix by delivery of materials to the crest and pumping of concrete from a concrete delivery lorry from the village area at the toe. In light of the quantities that were needed, the quality of the works and environmental considerations, the latter option of using a concreting truck and pumping the concrete up into the concrete grillage was considered most appropriate, (Figures 6 to 9). However, this was not as straightforward as originally considered and 5 attempts were needed in order to ensure the required quality of concrete that could be effectively pumped uphill. A brief history of the concreting is summarized on Table 3.

Table 3: Brief History of Concreting

Attempt	Date	Comments
1	2/8/2004	Pipe burst.
2	19/8/2004	Concrete was too dry and the required workability is too low
3	20/8/2004	The slump exceeded the upper limit and additional cement was required.
4	20/8/2004	The contractor attempted to concrete the upper portion of the concrete grillage but the formwork collapsed due to the high workability of concrete, and all concrete was subsequently washed away.
5	19/9/2004	Successful concreting, to the required slump and strength, as confirmed by rebound hammer tests.

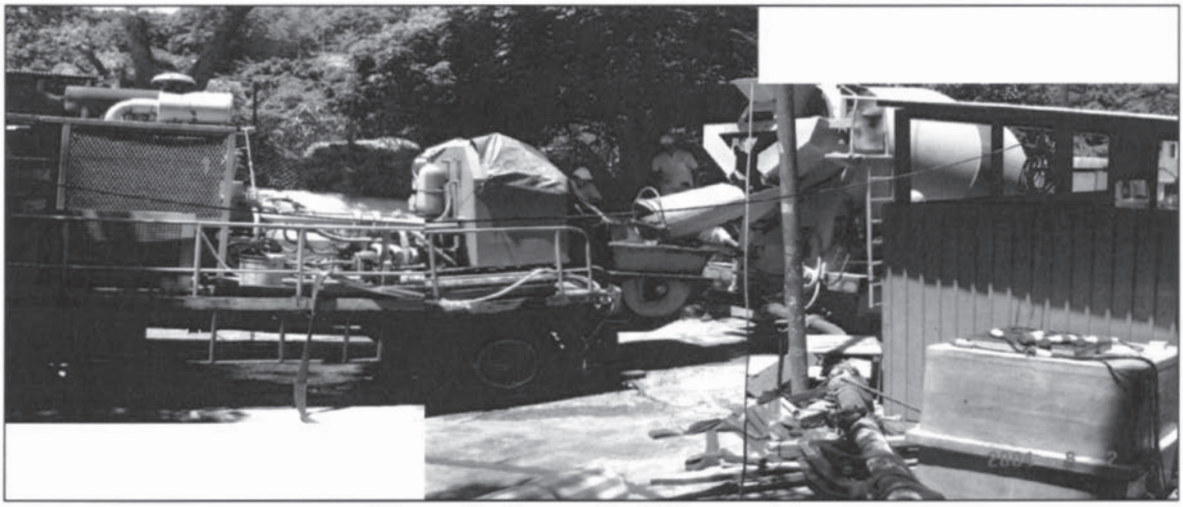


Figure 6: Concrete delivery at toe

Figure 7: Concreting pipeline through village along stream course.

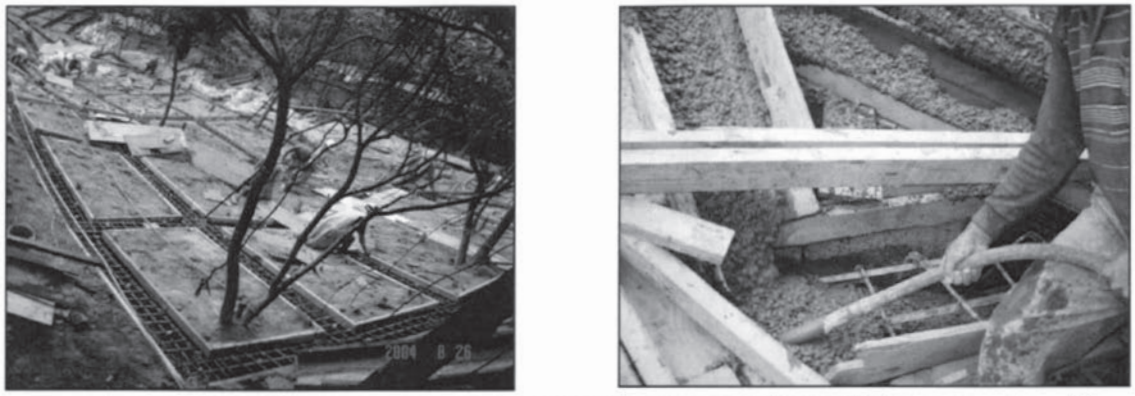


Figure 8: Fixing the steel reinforcement and concreting of the concrete grillage.

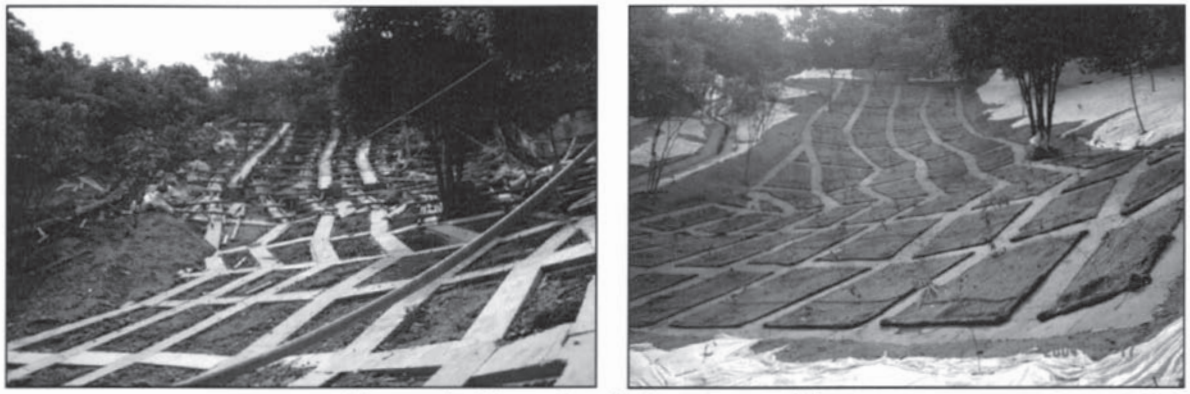


Figure 9: Completed Concrete grillage.

Debris Flow Barrier at the toe

The design also included construction of a ring net type debris flow barrier along the toe. The contractor used a specialist designer and supplier to provide the fence, which was fabricated in Switzerland and components were shipped to the site for erection. The detailed design was undertaken by the Specialist Contractor and checked in line with the methods outlined in GEO Report No. 104 (2000). The fence built is shown in Figure 10.

Figure 10: Completed Debris Flow Barrier along the toe.

Value Management through Partnering

In order to complete the slope upgrading works on time and with minimal complaint, and to overcome the site constraints and difficulties, a partnering approach between all

stakeholders, including the RSS, the contractor, the head office design team, GEO and the local residents and government departments, was considered essential. A Value Management Workshop was organized and attended by the site and head office staff and GEO representatives to identify the common objectives at the onset of the study, to determine options and solutions to the difficulties and follow through agreed courses of action.

With this philosophy in mind, which was actually used throughout the upgrading works to 30 different features, a very smooth running of the Contract resulted. On this particular project, the local residents and government departments, including DLO and WSD, were integral to successful completion and early and full liaison with their village representatives and departmental staff proved very effective. By partnering with the local villagers, security, sanitation facilities, electricity and potable water supply were provided locally.

CONCLUSIONS

The use of a combined design using direct in-slope support in the form of soil nails and concrete grillage in the thicker areas of fill, together with passive mitigation in the form of a debris flow catch fence in the remaining areas for this feature, has proven successful in minimizing disturbance to the local vegetation while at the same time ensuring increased safety to the public.

It must also be concluded that the use of debris flow barriers as upgrading works on loose fill slopes in Country Park areas is cost effective, as it mitigates the problems of delivery of materials, especially large volumes of concrete, to remote areas. It also avoids removal of vegetation and therefore provides environmental benefits.

ACKNOWLEDGEMENTS

This paper is published with kind permission of the Head of the Geotechnical Engineering Office and the Director of Civil Engineering and Development Department of the Hong Kong Special Administrative Region.

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DESIGN AND CONSTRUCTION OF A REINFORCED FILL RETAINING WALL IN THE LANDSLIP PREVENTIVE MEASURES PROGRAMME

Ronald P. M. Li[1], Martyn J Smith[1], Leslie H Swann[1]

Abstract: A reinforced fill (RF) retaining wall was constructed under the Landslip Preventive Measures (LPM) Programme to upgrade an existing fill slope which was formed as part of the embankment for Lion Rock Tunnel Road in Shatin. This paper briefly describes the design and construction considerations for the RF retaining wall to promote sustainable design and construction in LPM works. A fully vegetated rockfill slope, which partly used recycled aggregates, was also constructed on top of the RF retaining wall to replace the existing fill and enhance the final visual appearance.

INTRODUCTION

Stabilization of existing man-made slopes is one of the major construction activities in Hong Kong. As the understanding and techniques for soil nail design and construction are well established, soil nailing tends to be the most common method to upgrade the existing sub-standard slopes in the LPM Programme. However, soil nailing in certain circumstances is not feasible and other designs are required.

As an innovative design, in order to avoid the necessity of difficult and costly temporary works, a reinforced fill (RF) retaining wall was constructed under the LPM Programme to upgrade an existing previously engineered fill slope located immediately below Lion Rock Tunnel Road in Shatin. This paper briefly describes the design and construction considerations for the RF retaining wall which provided a cost-effective solution. A fully vegetated compacted rockfill slope was also constructed above the RF retaining wall to replace the upper layers of existing fill and enhance the visual appearance.

SITE DESCRIPTION

Feature No. 7SW-D/FR 445 is an existing re-compacted fill slope constructed some 20 years ago, located to the southeast of Fung Shing Court, Shatin, forming part of the embankment for Lion Rock Tunnel Road, as shown in Figure 1. The slope is approximately 210m in length, with a maximum height of around 18.5m. The original slope gradient varied from 30° to 35° and was covered by grass vegetation and scrub. The slope was formed from 1980 to 1981 and modified in 1993 in conjunction with a village house development at its northern
end.

[1] Jacobs Babtie - Babtie Asia

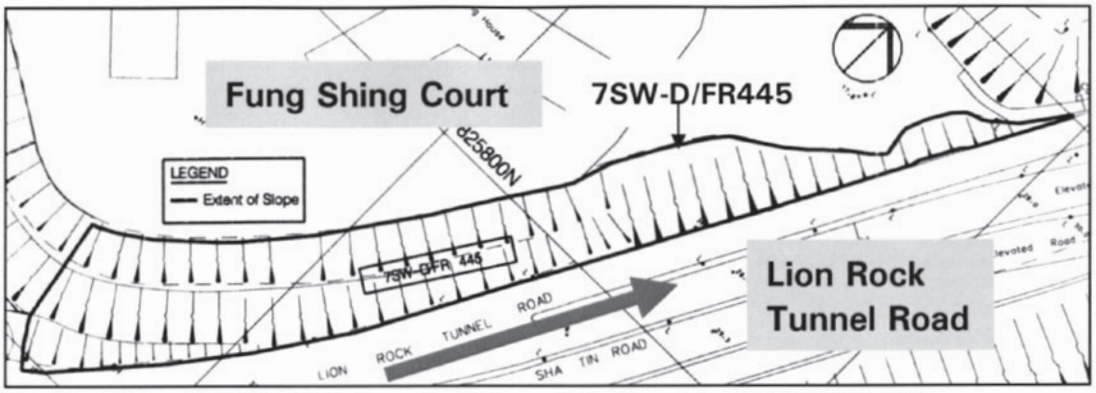


Figure 1 - Location Plan of Feature No. 7SW-D/FR 445

GEOTECHNICAL ASSESSMENT

Ground Model and Groundwater Condition

The ground investigation revealed a general sequence of strata comprising a varied thickness of fill material (Fill), maximum 11.0m, overlaying completely decomposed granite (CDG) with colluvium (Coll) and residual soil (RS) in some local areas. Continuous water flow was encountered in some trial pits during ground investigation. The design groundwater level was assumed 1.0m above the shallowest recorded water level within the fill layer. The typical ground model and groundwater conditions are detailed in Figure 2. Table 1 summarizes the design soil shear strength parameters.

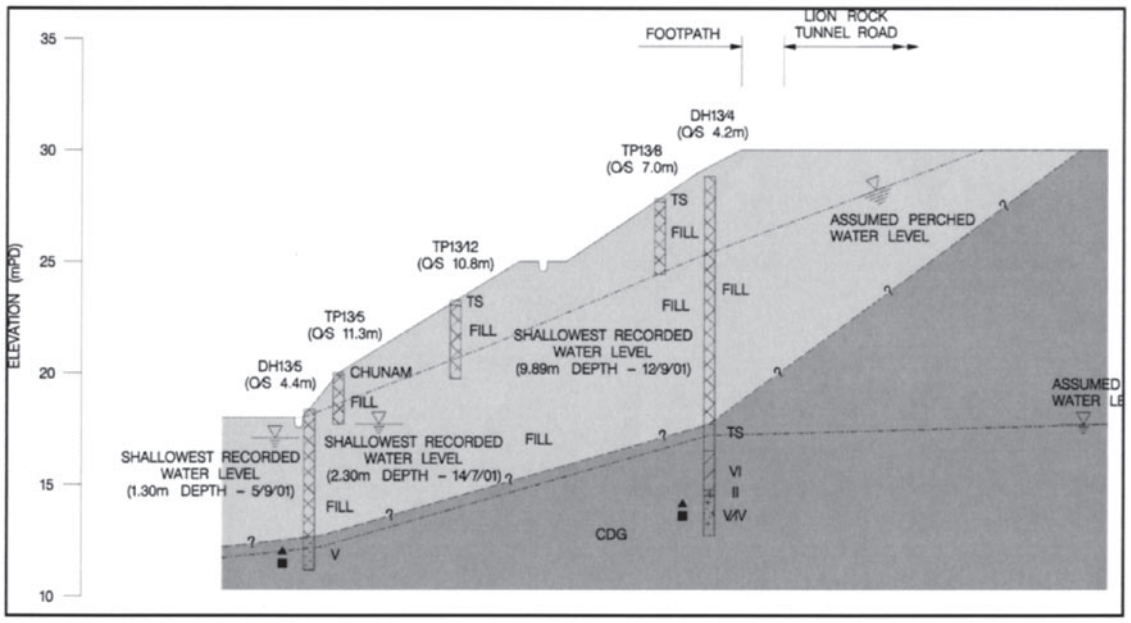


Figure 2- Typical Ground Model and Groundwater Conditions

Table 1 – Summary of Soil Shear Strength Parameters

Material Type	Effective Cohesion c' (kPa)	Effective Friction Angle ϕ' (^{o})	Bulk Unit Weight γ (kN/m^3)
Fill	0	35	18
Coll	3	33	18
RS	3	35	19
CDG	5	35	19

Stability Assessment

The stability assessment focused on the overall slope stability and the liquefaction potential of the fill body. The stability analyses indicated that the existing Factor of Safety (FOS) of the slope ranges from 1.08 to 1.17. As this feature represented an engineered, re-compacted fill slope over 20 years old, a detailed investigation of the state of the re-compaction across the feature was carried out. This included 19 nos. trial pits with sand replacement tests at 1.0m intervals, 19 nos. GCO Probes across the slope and standard penetration tests at 1.5m intervals in all 10 boreholes. These in-situ tests, and corresponding laboratory compaction tests, confirmed the spatial variations of relative compaction across the feature within the whole fill body. As shown in Table 2, a significant number of tests indicated that the fill did not meet the required nominal 85% RC or above to prevent liquefaction. In addition, there were two reported incidents that occurred in 1983 and 1993 which concluded "local slumping" and "landslip failure in fill". In view of the existing FOS below the current government standard, GEO (1984), the failures and the liquefaction potential of the fill, slope upgrading works were required.

Table 2 – Summary of Ground Investigation Results

Type of Test	Results (Top 4.0m Fill)	Loose Condition	Reference
SPT	4-12 (blows per 300mm)	SPT<10	GEO (1997)
GCO Probe	21% GCO Probe value <5	GCO<5	GEO (2000)
Relative Compaction (RC)	30% of RC < 85%	RC<85%	GEO (1999)

SCHEME SELECTION

Two feasible options were proposed in the scheme selection stage as follows:

Option 1: Re-compact the top 3.0m fill material and construct a reinforced concrete retaining wall at toe, including significant temporary works (piling or similar) to maintain Lion Rock Tunnel Road open at the crest.

Option 2: Remove the top 1.5m fill material with rockfill replacement and construct a reinforced fill retaining wall at toe

Option 2 was considered to be more appropriate, due to the limited excavation below the major highway, with less temporary works and minimal disruption to public. Other advantages included lower sensitivity to inclement weather and changing ground conditions, and lower generation of construction waste. In addition, the cost estimate indicated the RF wall would be cheaper by around 25% due mainly to the greatly reduced temporary works.

Soil nailing was not considered due to the thick nature of the fill body, which would require relatively long soil nails, as suggested by HKIE (2003).

DESIGN CONSIDERATIONS

Stability

The design of the RF retaining wall was based on Geospec 2, (GCO 1989), and Geoguide 1, (GEO 1993). In accordance with the published guidelines, the detailed design of the RF retaining wall included both external and internal stability checks. The external checks comprised sliding, overturning and bearing failure, and were completed using the limit state approach as advised in Geoguide 1. The overall slope stability was checked in accordance with the Geotechnical Manual for Slopes, (GCO 1984). The internal checks for the wall were based on an assumed width/height ratio of 0.70. In addition, as suggested in Geospec 2, (GCO 1989), the Contract Specification gave the contractor a choice in the selection of the reinforced fill product. The detailed internal design, therefore, was completed during the construction stage. The typical design details of the RF retaining wall are summarized in Figure 3.

Environment

A gentle, maximum 30°, compacted rock fill slope was constructed on top of the RF retaining wall for landscaping purposes, to provide a vegetated slope similar to the original appearance, as well as removing the existing upper fill layers. Recycled aggregates were used as part of the rock fill slope construction. The temporary works design, comprising soil nailing, was designed and installed to ensure the stability of the temporary slope under the highway. The typical compacted rockfill slope profile and temporary works details are summarized in Figure 3.

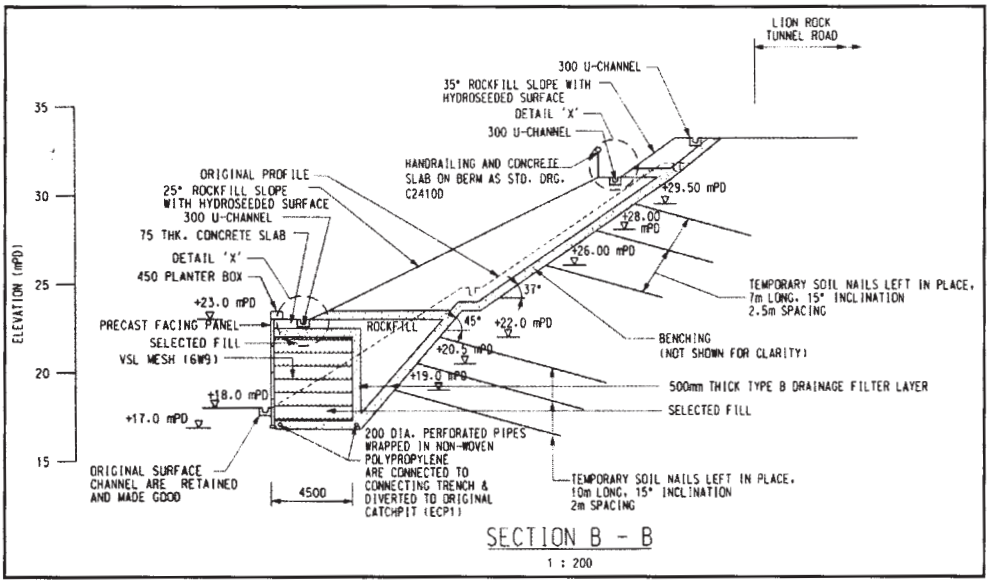
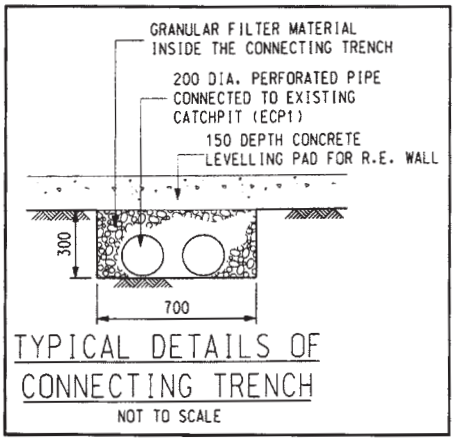


Figure 3 - Typical Design Details of the Slope Upgrading Works

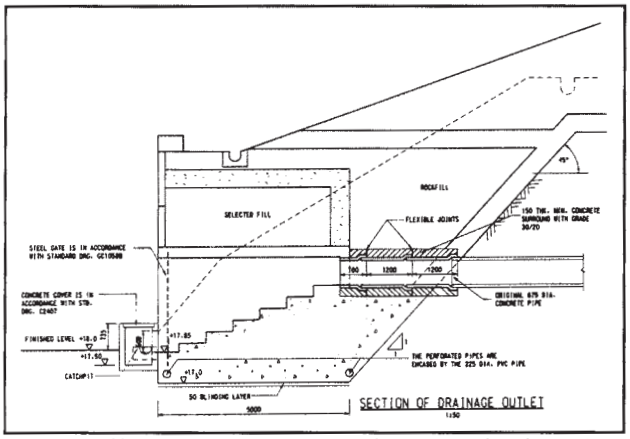
Sub-surface Drainage System

The sub-surface drainage system of the RF retaining wall comprises a layer of 500mm thick Type B granular filter materials along the base and the rear of the RF retaining wall. Two perforated pipes were placed at the base of the proposed wall to collect the sub-surface

water and discharge it into the existing manhole through a connecting trench. The details of the connecting trench are shown in Figure 4a. In addition, an existing buried pipe (675mmϕ), which collects the surface water from Lion Rock Tunnel Road, was located in the middle portion of the slope and intersected the proposed wall. As this pipe was required to be retained, a drainage outlet was designed to cater for the discharge of the pipe and the construction of the RF retaining wall. Details of the drainage outlet are presented in Figure 4b.



(a) Details of connecting trench



(b) Details of the drainage outlet at the buried pipe

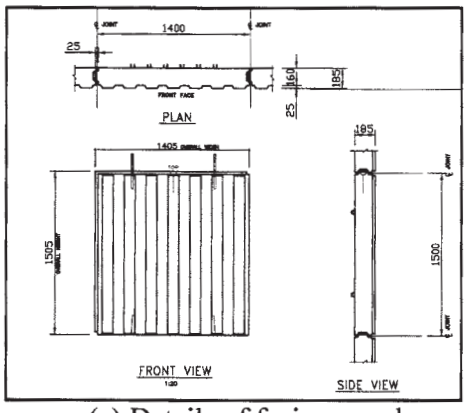
Figure 4 - Typical Details of the Sub-surface Drainage System

Public Liaison

As the construction site was located in close proximity to the residential housing block of Fung Shing Court, full and ongoing liaison with all the stakeholders, including local residents, management company representatives, the incorporated owners association and the district councilor, was sought in order to enhance public relations and ensure a smooth running of the construction works. As the construction vehicles were not allowed to queue or wait along the highway, access permission was required and obtained from the private parties.

Appearance

The facing panels of the RF retaining wall were pre-cast in the manufacturer's casting yard, which allowed higher flexibility in determining the appearance of the wall. An exposed aggregate type finish was adopted as the finishing of the panels to give a "texture effect". Details of the facing panels are presented in Figure 5.



(a) Details of facing panel

(b) Exposed aggregate finishing

Figure 5 - Typical Details of Facing Panel

CONSTRUCTION CONSIDERATIONS

Existing Obstructions

An existing buried box culvert, 2.5m x 2.5m, was found at the southern end of the feature. In order to retain and protect this box culvert, a modification of the base for the RF retaining wall was proposed as shown in Figure 6. This detail allowed the wall to sit on the box culvert without any diversion measures. In addition, the surcharge imposed by the wall and rock fill slope above the box culvert was critically reviewed to avoid any damage to the existing box culvert.

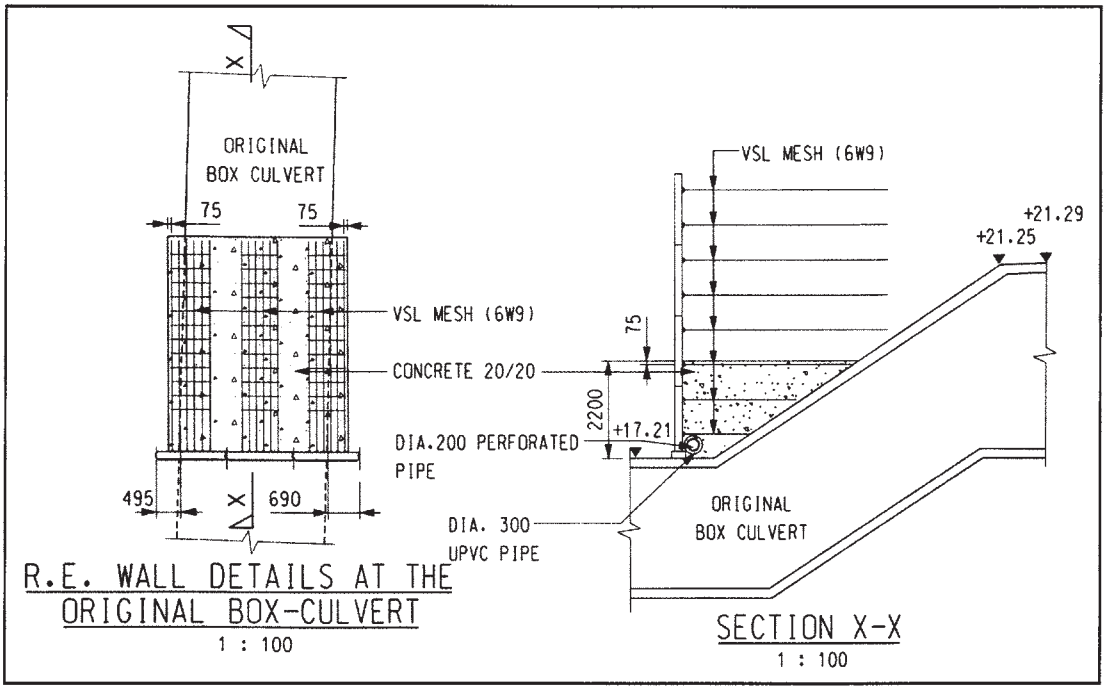


Figure 6 - Typical RF Retaining Wall Details at Existing Box Culvert

Contract Documents

As the construction specifications and method of measurement for the RF retaining wall were not available within the General Specification for Civil Engineering Works (1992) and Standard Method of Measurement for Civil Engineering Works (1992), detailed specifications, such as the requirements for the reinforced fill products and the selected fill materials, the testing frequency and the tolerance of the built-wall, were prepared. In addition, the method of measurement and item coverage were well-defined in the Contract Documents in order to minimize any financial issues during the construction stage.

Quality Control

The compliance of the selected fill material and reinforcing elements are critical for the long term stability of the wall as the RF retaining wall relies on the load transfer mechanism between the fill material and the reinforcing elements. The adopted selected fill materials, which were graded in accordance with Geospec 2, were imported for the construction of the reinforced fill construction. The selected fill materials were compacted to achieve 95% RC. In-situ density tests were carried out to verify the degree of compaction. In addition, the verticality of the wall was closely monitored. An experienced specialist contractor was required to be deployed in order to ensure that the wall fully complied with the drawings and specifications.

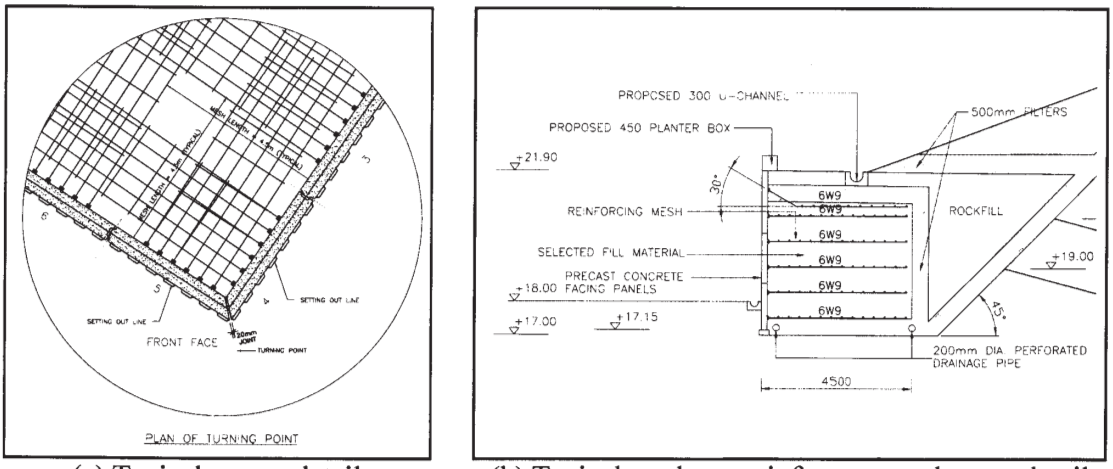
Safety and Ground Movement

The desk study revealed that utilities including gas mains, electricity cables and telecom cables were located at the crest of the slope. Precautions were taken to minimize the risks/hazards during construction. Additional trial pits and meetings with the utilities undertakers were arranged to confirm the exact locations and depths of the utilities. In particular, the angle of the top row of the temporary soil nails was adjusted to avoid any damage to deep utilities, including a telecom cable located 3.0m below the surface. In addition, a ground movement monitoring system was set up to monitor ground settlement, with no significant movement being recorded during the Works.

Construction Details

A wing wall was constructed at the southern end of the RF retaining wall. The corner detail of the wall, as shown in Figure 7a, was proposed. The advantage of this detail was to minimize the need for a cast in-situ reinforced concrete retaining wall or column at the corner of the RF retaining wall. This detail also maintained the monolithic characteristics of the RF retaining wall, minimized differential settlement and reduced construction cost.

Apart from the corner detail, a consideration of the top layer of the reinforcement mesh for the RF retaining wall was also required. As a coping beam or L-shape retaining wall above the reinforced block was not necessary, a detail as presented in Figure 7b was designed and built. This detail was proposed to allow the top layer of the reinforcement element to be encased inside the reinforcing block to ensure the stability of the topmost facing panel.



(a) Typical corner detail (b) Typical top layer reinforcement element detail

Figure 7- Typical Details at Wall Corner and Top layer of Reinforcement Element

CONCLUSIONS

The RF retaining wall provided an innovative design solution for this feature to allow upgrading works to be completed on a large fill slope while maintaining a major red route open and unaffected at the crest. Reinforced fill retaining walls are a cost-effective, environmentally friendly, quick to construct and aesthetically pleasing engineering solution to upgrade existing fill slopes and provide viable alternatives to the standard soil nailing or reinforced concrete walls. Reinforced fill structures such as this one are now being used in a

variety of locations on major projects, to replace standard bored pile walls and the like. The appearance of the completed slope upgrading works to this feature is shown on the photographs below.

(a) Completed Compacted Rockfill Slope (b) Completed RF Retaining Wall

General Views of the Completed Slope and RF Retaining Wall

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ACKNOWLEDGEMENTS

The paper is published with the kind permission of the Head of the Geotechnical Engineering Office and the Director of Civil Engineering and Development Department of the Hong Kong Special Administrative Region.

Enhancement of Slope Engineering Practice on Vegetated Slopes Based on Findings of Landslide Studies

T H H Hui[1] & H W Sun[1]

Abstract: It is policy of the Government of the Hong Kong SAR to make engineered slopes look as natural as possible. To pursue this policy, vegetation has been used as slope surface cover wherever possible in the formation of new slopes and in the upgrading of existing slopes under the Landslip Preventive Measures Programme. The findings of the systematic landslide investigation (LI) programme, which was initiated by the Government in 1997, have contributed to achieving an improved understanding of the mechanisms and causes of engineered slope failures, with a view to further improving the slope engineering practice and slope safety system in Hong Kong. This paper presents the salient findings from detailed investigation carried out under the LI programme on several notable landslides involving recently formed or upgraded vegetated man-made slopes, including slopes with a change of surface condition from hard surfacing to vegetated cover as part of the slope works. Areas deserving attention and improvement are also discussed.

INTRODUCTION

It is policy of the Government of the Hong Kong SAR in the recent years to make engineered slopes look as natural as possible and blend them with their surroundings. In the past, slope upgrading and maintenance works generally involved the use of sprayed concrete to form an impermeable hard surface for protection of slopes. There has been increasing demand from the public on the use of vegetated covers on slopes to improve the aesthetics and to create a sustainable ecology of our build environment.

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 as far as possible, particularly in vegetated slopes. Notwithstanding this, vegetated slopes with adverse geological/hydrogeological setting are liable to major or massive failures if these features are not detected and catered for during design and construction stages.

The implementation of the systematic landslide investigation (LI) programme in 1997 has contributed to achieving an improved understanding of the mechanisms and causes of engineered slope failures. This paper documents the key observations and lessons learnt from studies of engineered slope failures involving vegetated slope surface. Suggestions on improved slope engineering practice on vegetated slopes are made in order to further enhance the robustness of engineered slopes and to build a safe and green environment for Hong Kong.

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FINDINGS OF LANDSLIDE INVESTIGATIONS

Some recent landslide incidents on engineered slopes with vegetated surface cover have been reviewed/investigated under the LI programme. The salient findings are presented below.

The July 1994 Landslide at New Clear Water Bay Road

A massive landslide (about 800 m^3) occurred on an engineered vegetated cut slope (max. 20 m high) during heavy rainfall, about two years after completion of slope upgrading works (which comprised replacing the existing chunam cover with vegetation cover and provision of additional surface drainage channels). The landslide resulted in complete blockage of all the west-bound lanes of New Clear Water Bay Road (Plate 1).

Plate 1 – Massive Landslide at New Clear Water Bay Road

Several drainage lines intercepted the slope and water was observed issuing from raking drains in adjacent slopes, which suggest that the slope of concern might have an adverse hydrogeological setting with significant response of groundwater level during/following heavy rainstorms.

The failure was probably caused by excessive water ingress due to intense direct infiltration into the slope leading to rise of the main groundwater level (Tam and Lam 2005). In the design assessment of the slope upgrading works in the early 1990's, the groundwater condition for design was developed with reference to the results of groundwater monitoring, which was carried out in a relatively dry year and before the commencement of slope upgrading works (i.e. the entire slope surface was covered with chunam). The design groundwater level was set to be 1 m above the highest recorded water level. Based on the result of stability assessment, no soil nail reinforcement or cut-back was considered necessary.

This incident serves to highlight the possible adverse effect on the groundwater condition due to the removal of the hard surface cover as part of the slope upgrading works, in particular for slopes with adverse hydrogeological setting.

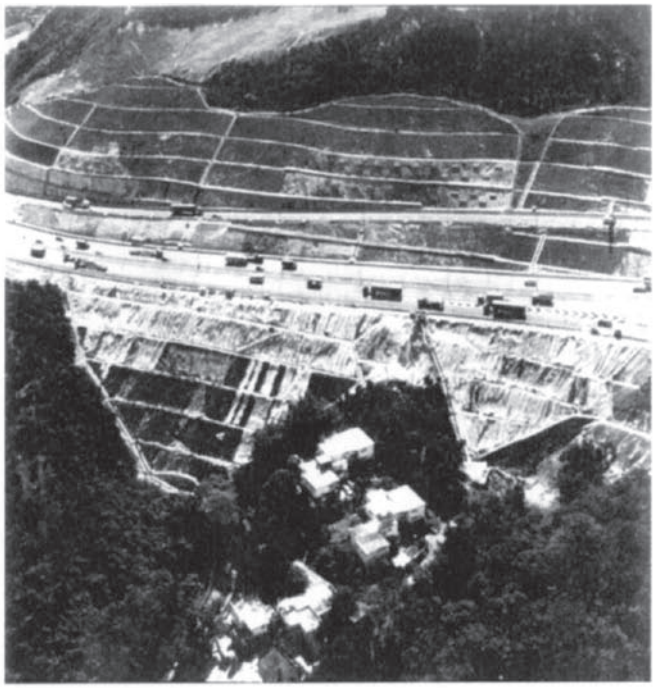
The May and June 1998 Landslides near Ting Kau Bridge

Several major landslides (total volume of about 1,300 m^3) occurred on an engineered vegetated fill slope between May and June 1998 (Plate 2). These landslides were triggered by moderate rainfall in the first wet season after the completion of slope upgrading works (which comprised construction of compacted rockfill with a soil fill cap of 300 mm to 500 mm thick).

The landslides involved shallow sliding failure of the soil fill cap along a surface of rupture located about 0.3 m to 0.4 m below and parallel to the slope face. The failures were probably caused by the development of transient elevated water pressure in the near-surface fill material following direct infiltration. Deficiency in detailing of the fill slope and

improper placement of fill material were identified as key contributory factors in these failures (FSWJV 1999).

The above incidents highlight the importance of adequate compaction control and proper surface protection and detailing to the soil fill cap overlying compacted rockfill.



The May and June 2003 Landslides at Fan Kam Road

Two minor landslides occurred on an engineered cut slope (max. 23 m high) in May 2003 and June 2003 respectively (Plates 3 and 4), about three years after completion of slope upgrading works (which comprised installation of soil nails except for the uppermost 7 m of the soil cut and replacement of the pre-existing hard cover with vegetation cover). The north bound lane of Fan Kam Road was closed temporarily as a result of the May 2003 landslide.

The 5 May 2003 landslide (20 m^3) occurred during intense short-duration rainfall and involved the failure of the surface mantle of colluvium in the upper unsupported cut portion. The failure was probably caused by the build-up of transient perched water level above the colluvium/saprolite interface (MGSL 2004). As part of the slope upgrading works, the existing shotcrete cover was removed and replaced with a vegetated surface. The removal of the existing hard cover might have promoted direct infiltration into the slope.

The June 2003 landslide (3 m^3) involved a shallow detachment of saprolite in between soil nail heads at mid-slope portion during a moderate rainfall. The landslide occurred on a locally over-steep (50° to 55°) portion of the cut slope. This local failure was probably caused by direct infiltration from concentrated surface runoff leading to the wetting up of the near-surface material and loss of suction (MGSL 2004).

The May 2003 incident serves as a reminder of the possible build-up of transient perched

water level within the surface colluvium, and call for the need of providing sufficient engineering input in stability analysis. The June 2003 incident highlights the importance of identifying local oversteepening on slope, which could lead to concentrated surface runoff and hence instabilities.

Minor Landslides at Route Twisk and Shing Mun Country Park

Four minor landslides (each about 2 to 3 m^3) occurred on three engineered soil cut slopes along Route Twisk in June 2001. These landslides were triggered by heavy rainfall in the first wet season following the completion of upgrading works on these slopes. Inadequate detailing of the slope protection cover (i.e. vegetated berm, see Plate 5) and surface drainage provisions (i.e. lack of drainage provisions and inadequate capacity of the existing drainage channel) were identified as key contributory factors in these incidents (Lam et al. 2002).

A minor landslide (about 3 m^3) occurred on an engineered soil cut slope at Shing Mun Country Park in September 2003, about six months after completion of slope upgrading works. The landslide was probably caused by the presence of a topographical depression leading to concentrated surface runoff towards the landslide scar (Plate 6) (Hui et al. 2004).

These incidents helps to highlight the importance of proper drainage detailing, together with slope surface trimming as part of slope upgrading works to remove local depression or over-steepening, so as to avoid localised failures due to concentrated surface runoff.

SUGGESTIONS FOR IMPROVED PRACTICE ON VEGETATED SLOPES

Observations and 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. Measures have been recommended to improve slope engineering practice for mitigation failures on engineering slopes (Wong and Ho 2000, Wong 2001, Ho et al. 2002). Some of these recommendations are particularly relevant for slopes with vegetated cover and are further discussed below.

Design and Construction

Where slope design involves replacing a hard surface cover with a vegetated cover, there could be changes in rainfall infiltration characteristics and influence to groundwater response. Piezometric response measured during ground investigation before removal of hard cover may not be directly relevant to the surface condition of the upgraded slope and could even be misleading in some cases. Given that the measurement of the seepage-induced water pressure is difficult, especially where the groundwater monitoring period is short or does not cover heavy rainstorms, Lumb's (1962, 1975) wetting band approach is sometimes used to give a rough estimate for the stability analysis and design of slopes in Hong Kong (GCO 1984). Also, it would be prudent to continue piezometric monitoring at selected sites during construction as far as possible so that the design assumptions can be verified.

The use of suspiciously high shear strength parameters (e.g. very high c' value for colluvium and completely weathered granites/volcanic), based on the results of back analysis or the adoption of generalised strength parameters as quoted in Geoguide 1 (GEO 1993), may have implicitly included the effect of soil suction, which may be altered subsequently such as through the replacement of a hard slope cover by a vegetated cover (GEO 2004a).

Given that improved GI practice and enhanced engineering geological input could still miss the more subtle adverse geological features, robust design solutions (e.g. soil nails, reinforced concrete retaining wall, etc.) that are less sensitive to local adverse geological and groundwater conditions (in comparison with solutions based on no positive slope reinforcement or support) are recommended (GEO 2004b). Where the slope upgrading works involves the replacement of existing hard cover with a vegetated cover, the upgrading works should be designed to meet the standards of a new slope. The detailed slope design should also take account of the average and local slope angles, the size of the upslope catchment and whether there is likely to be concentrated surface water flow onto the slope, signs of seepage from the slope surface, records of past failure of slopes of similar nature, etc. (GEO 2004d).

In view of the innate variability of the groundwater conditions, a pragmatic approach is to incorporate contingency drainage provisions in slope 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.

Locally weak geological material near the slope surface such as colluvium can be guarded against from failure by suction maintained where the slope is protected by hard surface cover. Upon replacement of hard cover by vegetation, direct infiltration into the slope may result in loss of suction and build-up a transient elevated water pressures above the interface between colluvium and the underlying relatively denser material, which could lead

to instability. The use of short soil nails coupled with enhanced erosion control measures such as steel wire mesh structurally connected to the soil nail heads is recommended to cope with possible local shallow failures.

The exposed slope face should be mapped after removal of hard cover, with particular attention to the presence of local weak geological material (i.e. colluvium) or local depression, which may lead to convergent of surface runoff. Undulating slope surface is suggested to be trimmed smooth as local depressions or protruding areas on the slope surface can result in ponding or concentrated surface runoff.

Slope Drainage and Protection Detailing

It appears to be a common practice in slope design that designers tend to emphasise on theoretical stability analyses and calculated factors of safety. It is important for designers to take a holistic view on possible hazards or failures during the service life of the slope. Based on the lessons learnt from failures of engineered slopes, the need for further improving detailing of surface drainage provisions and slope surface protection measures in slope design is also highlighted.

More systematic consideration of the sources and pathways of water ingress into the slope should be made, especially for slopes involving replacement of the existing hard surface cover with vegetation. Digital Terrain Model can be used to assist the design of surface drainage provisions, where appropriate, for slopes with sizeable catchment to provide more accurate information on possible pathways of surface water flow. As far as practicable, site visits should be arranged during or shortly after severe rainfall, particularly for potentially problematic sites that have adverse hydrogeological setting and are susceptible to surface erosion or concentrated water ingress, to observe surface water flow, adequacy of existing drainage measures and any seepage from the slope. It is also worthwhile of carrying out a review of the slope performance, in terms of the adequacy of the pre-existing surface drainage provisions with due regard to the history of previous flooding or over-spilling of surface water and the overall hydrogeological setting in the area.

Where potential over-spilling of surface water is possible, provision of baffle walls along critical locations of surface channels could be a solution to confine the surface runoff within the channels. Also, provision of a curved profile at the junction of berm channels and the down channels would help to improve the hydraulics of surface water flow.

Where rockfill is used, a thin layer of topsoil (generally 150 mm to 300 mm thick) may need to be provided at the surface of the rockfill for planting purposes. The potential instability of the soil-capping layer must be considered. Measures should also be taken to prevent loss of fines from the capping layer into the underlying rockfill (e.g. provision of a transition zone comprising granular material). Extreme care should be exercised in the specification of a geosynthetic filter fabric at the interface of the capping soil layer and rockfill because this would give rise to a smooth weak plane with potential instability problems. An erosion protection mat properly secured onto the slope using fixing pins should be provided on the surface of the capping layer (GEO 2004c).

CONCLUSIONS

The systematic LI investigation of the recent landslides, which included failures on slopes involving replacement of hard covers with vegetation covers, has contributed to the

identification of the possible causes of failures. The review has re-affirmed that there are rooms for improvement in our current slope engineering practice on vegetated slopes, and the need to pay due attention to change in groundwater condition of the slope as a result of replacement of hard surface cover with an vegetated cover. The review also highlights the importance of adequate slope drainage provisions and surface protection detailing.

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ACKNOWLEDGEMENTS

This paper is published with the permission of the Head of the Geotechnical Engineering Office and the Director of Civil Engineering and Development, the Government of the Hong Kong Special Administrative Region.

「雖由人作，宛自天開」[1]
CASES OF TREATING SLOPE STRUCTURES IN PUBLIC HOUSING DEVELOPMENTS

LEUNG King Wai [2]

Abstract: In Hong Kong, slopes and retaining walls are built to create leveled land for development. The prime concern to geotechnical engineers is to ensure their safety. Little attention had been given in the past to match them with the surrounding environment. In public housing development, there is an emerging philosophy that recognizes slopes and retaining walls as part of the land and space for residents' enjoyment and use. The paper presents several examples to illustrate how the philosophy is applied to integrate slopes and retaining walls with the planning and design of public housing developments.

INTRODUCTION

Hong Kong has a rugged terrain. Except for the coastal reclaimed areas, most of the development land is formed by cutting and filling along foothill areas that result in numerous slopes and retaining walls. The prime concern to geotechnical engineers is to ensure the safety and stability of these features.

Until the last decade, chunam or shotcrete surfacing had been used extensively to protect slopes from rainfall infiltration and soil erosion. Little attention had been given to slope greening and to how slopes and retaining walls match with the surrounding environment.

With this background, the Government issued a Technical Circular on "Control of Visual Impact of Slopes" (Works Branch, 1993) and supplemented it with another Technical Circular on "Improvement to the Appearance of Slopes" (Works Bureau, 2000). These circulars set out guidelines to minimize the adverse visual impact of slopes and retaining walls and to control the use of chunam and shotcrete. As far as practicable, vegetation cover for slopes should be used and existing mature trees preserved. Further details are given in the "Technical Guidelines on Landscape Treatment and Bio-engineering for Man-made Slopes and Retaining Walls" (GEO, 2000) and Technical Guidance Note TGN 20 (GEO, 2004).

Many authors have also reported on recent practices of slope greening and various greening techniques (Ling, 2001; Martin, 2001; Martin et al, 2001; Thomas et al, 2001; Kong, 2003; Choi et al, 2004; Lui et al, 2004).

In public housing development, there is an emerging philosophy that recognizes slopes and retaining walls as part of the land and space for residents' enjoyment and use. Several examples are presented in the paper to illustrate how the philosophy is applied to integrate these features with the planning and design of the developments having regard to the special circumstances of the projects. A functional approach is adopted in the design and visual treatment of slopes and retaining walls that extends beyond pure slope safety consideration.

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[1] Cited from 陳從周《說園》: "疊山理水要造成「雖由人作，宛自天開」的境界"

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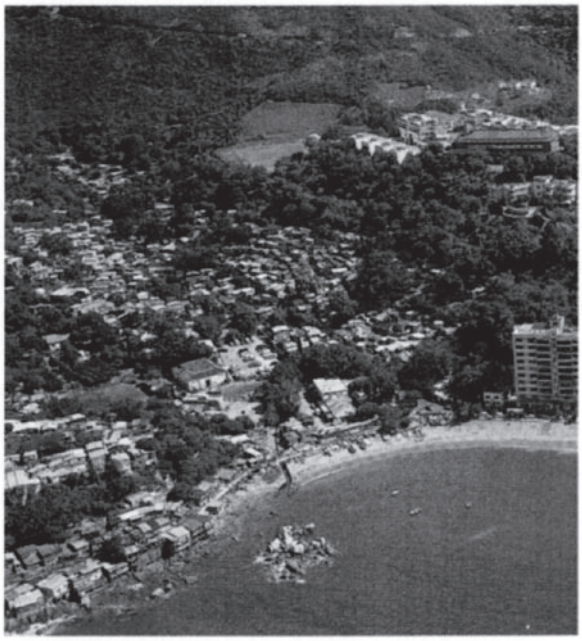
A GARDEN COURTYARD AT MA HANG

Ma Hang is located in a valley setting at Stanley. It was occupied by a village settlement of about 6000 people at the time of redevelopment. Cottage huts were built on terraces formed on steep, heavily vegetated slopes. In the late 1980s, the Government decided to redevelop the village for public housing and for in-situ re-housing of the affected villagers. It was also decided that the development should preserve the character of the area.



The basic philosophy adopted for the design of the development, including the site formation, slope and landscaping works, was to build a garden estate that would be compatible with the site and its surrounding environment. Four pillars were identified to underpin the slope design concept –

(a) conservation,
(b) integration,
(c) aesthetics, and
(d) safety.

Photo 1 (left) – A bird's view of the old Ma Hang Village before redevelopment.

Conservation

There were a lot of mature specimen trees in the steeply sloping valley. Their conservation presented a uniquely difficult problem for site formation. But the opportunity was taken to retain an existing knoll with mature trees and to transplant other trees affected by site formation to the new garden courtyard. The general terrain of the valley was preserved. The solution for the development layout, as well as slope and landscape design was looked at holistically in association with the site topography, the locations of the building blocks, the routing of pedestrian flow and underground services, recognizing the considerable constraints and opportunities.

Photo 2 (right) – The development layout was planned and designed holistically to integrate the natural and built environment in a harmonious manner. An existing knoll with mature trees was retained (background). It merges with the man-made slopes and the courtyard with a unified identity.

Integration

The slopes and retaining walls were built to integrate with the open space of the development so as to create a beautifully landscaped courtyard setting. The stream was recreated to flow down the valley under controlled conditions to provide a cohesive life-bringing element that would punctuate the building massing. The slopes were hydroseeded and planted with scrubs and trees. At chosen locations, they were also decorated with boulders, sculpture ducks and dragon to enrich the garden appearance.

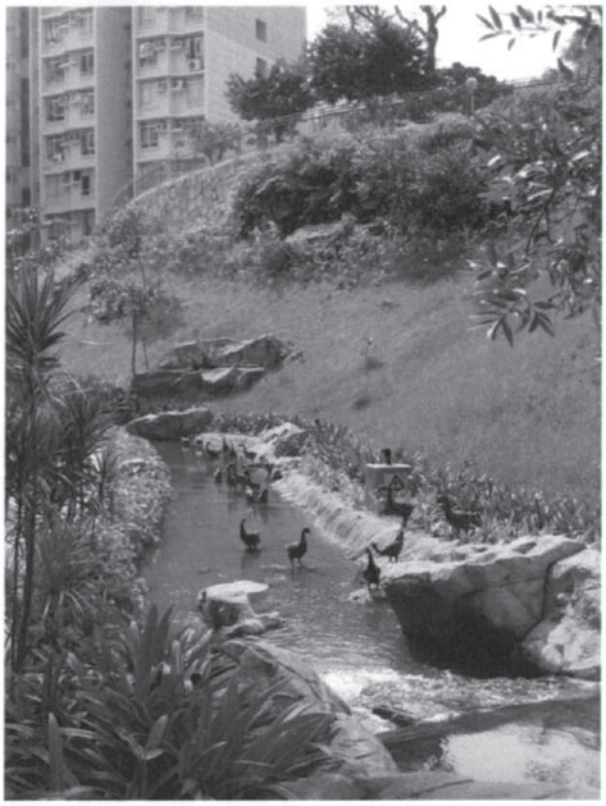


Photo 3 (right) – Slope No. 15NE-A/FR194. The vegetated slope and retaining wall are seen as an integral part of the garden courtyard and not as isolated features.

Photo 4 – Slope No. 15NE-A/FR193. The sculpture dragon is in fact a curved retaining wall built at mid-height of the slope.

Aesthetics

The landscaped courtyard was modeled by means of slopes with very limited use of retaining walls. On one of the slopes, a centre-piece retaining wall was built with a curved alignment at the mid-height and camouflaged as a dragon from whose mouth cascades the recreated stream. The wall performs an essential earth-retaining function as well as an aesthetic function in the serene garden ambience.

Safety

All the slopes and retaining walls were designed to the current safety standard. A network of drainage channels was provided to divert surface water into the main drainage system. The recreated stream was lined with a waterproof membrane to prevent infiltration into the toe of the slope. A slope maintenance manual was also drawn up to facilitate regular inspections and maintenance of the slopes and retaining walls.

TESTIMONY OF LIFE AT SAU MAU PING

Those who witnessed the development of geotechnical engineering in Hong Kong will not forget the tragic landslide that happened in 1976 at Sau Mau Ping. The failed loose fill slope was one of a series of around 30 m high cut and fill slopes that separated the lower and middle platforms of Sau Mau Ping. Although the slopes had been upgraded since then, over time the sprayed concrete surface deteriorated and acquired an unsightly darkish colour with huge spider web of cracks sealed with cement mortar.

Photo 5 – The failed loose fill slope in 1976.

Photo 6 – The unsightly recompacted slope in 1993.

The comprehensive redevelopment of Sau Mau Ping provided an opportunity to tackle the long-standing slope issue, not only in terms of slope safety but also for improvements in the built environment. It was principally with this consideration that major improvement works to this series of slopes were proposed and implemented between 1993 and 1997.

The philosophy behind the slope improvement works was to construct a central park that could integrate multi-purpose functions in addition to ensuring slope safety –

(a) provision of pedestrian linkages between Lower and Middle Sau Mau Ping,
(b) creation of usable space for the residents, and
(c) establishment of a "green" and "natural" environment.

Provision of Pedestrian Linkages

Before implementing the improvement works, the slopes were then a barren barrier baking hot in summers and chilly in winters. It was considered by many as a burden rather than an asset.

As part of the improvement works, ramps and steps were built on the slopes that integrate with the intermediate berms at convenient locations to provide pedestrian linkages between the toe and the crest of the slopes at Lower and Middle Sau Mau Ping. They could also be used as leisure trails for exercises. At a central location, a two-way all-weather escalator system was installed with mid-slope access to convey pedestrians.

Photo 7 (left) – Before redevelopment residents used long steep staircases to commute between Lower and Middle Sau Mau Ping.

Creation of Usable Space

The middle berm on the slopes was built with ample space to accommodate sitting out areas, fitness stations, and planter beds. The intention was to create a park to be used by residents for recreation, leisure, and interaction.

Nowadays it is not uncommon to find people strolling or jogging along the trails, doing exercises at the fitness stations, chattering in the arbours, or enjoying the light breeze under the woodland shade.

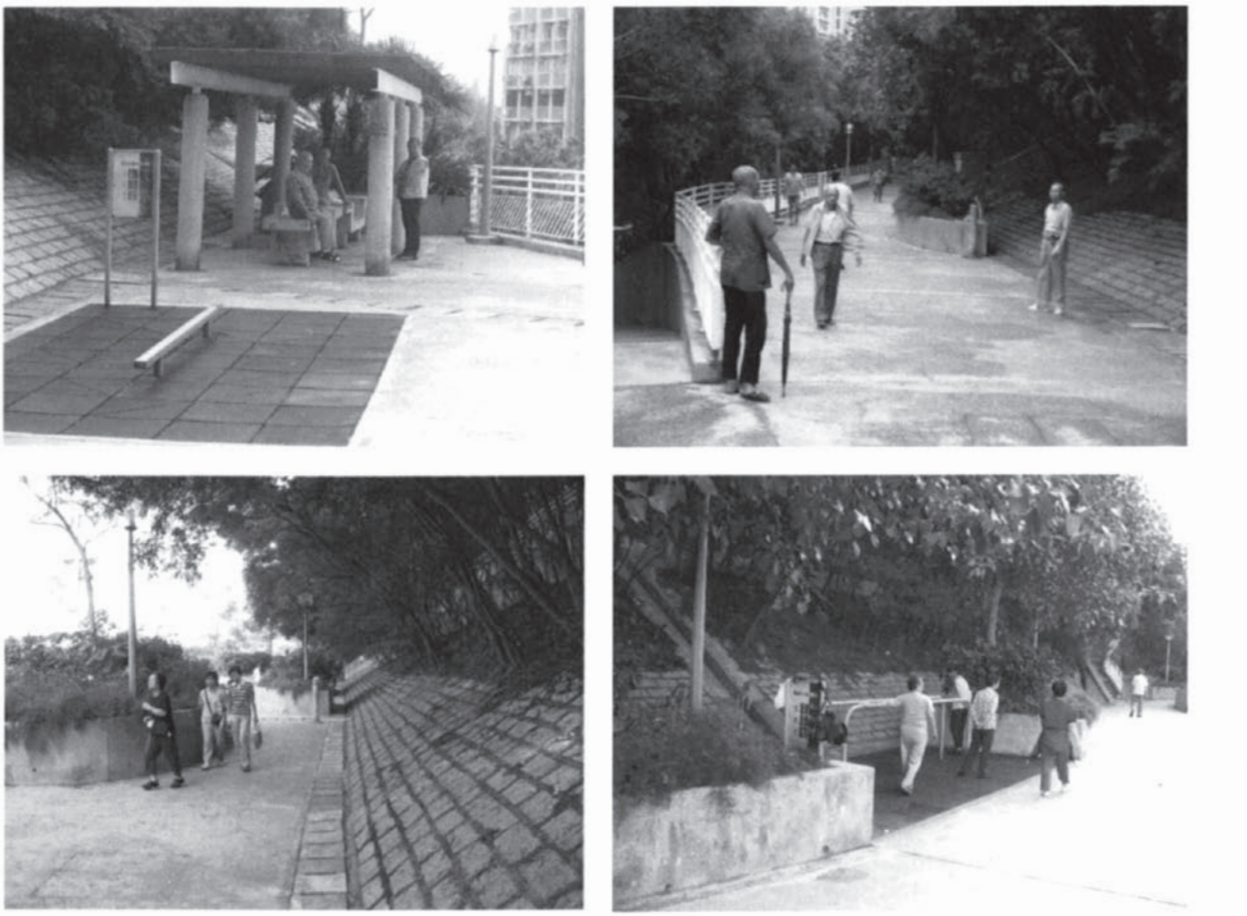


Photo 8 – The philosophy behind the slope improvement works is to create a park for use by the residents.

Establishment of a "Green" and "Natural" Environment

To realize the central park concept, the slope surface was hydroseeded with a mixture of grass seeds and tree seeds. The tree species used include Acacia confusa, Eucalyptus citriodora, Eucalytus robusta, Tristania conferta, and Casuarine equisetifolia. They were considered to be "pioneer species" that could quickly colonize bare soil and survive the relatively inhospitable conditions. Since then, natural vegetation succession has gradually resulted in a lush green woodland on the slopes.

The progressive transformation into a "natural" terrain brings a number of added benefits. Firstly the series of slopes forms a "green island" to break up the "concrete forest" of tall buildings. Secondly it provides a scenic view corridor for the residents. Thirdly it creates a breeze path and foliage cover to regulate the micro-climate in the area. Fourthly it bears witness to a flourishing life that wipes away the reminiscence of the long gone tragedy.

Photo 9 – The completed slopes in 1997. Hydroseeding was applied progressively from the far end with an obvious gradation in plant growth in the initial years.

Photo 10 – The completed slopes in 2003. Vegetation growth has resulted in a lush green woodland giving a "natural" terrain appearance.

OTHER EXAMPLES

Each site has its own setting. There is no universal treatment for slopes and retaining walls to suit different circumstances. It is incumbent on the geotechnical engineers to look beyond the confines of these features, to work closely with landscape architects, to be sensitive to the surrounding environment and to be creative.

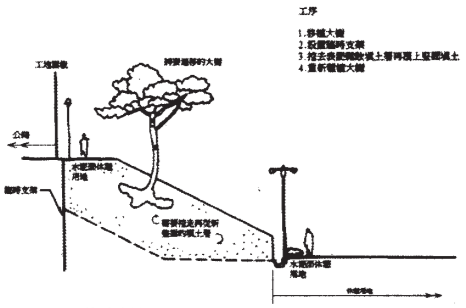
At Fung Wah Estate, a retaining wall was built by cutting into the natural hillside to create space for a children play area. The surface of the wall was embossed with artificial trees of a different texture treatment. It results in an interesting bridge between the man-made features with the toe planting and the lush vegetation on the backslope.

In comparison, a man-made cut slope was formed at Lee On Estate. With tree and scrub planting, it blends in well with the natural terrain higher up. Boulders were also used give a variety to the visual impact. A walking trail cuts across the slope to connect the amenity area at the toe with pergolas on the slope. The slope treatment complements the colourful play equipment and creates a spacious feeling by extending the amenity area onto the slope.

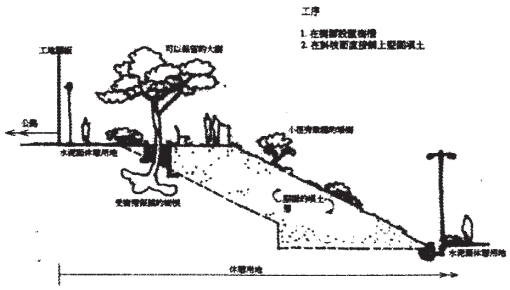
Photo 11 – The featured retaining wall at Fung Wah Estate.

Photo 12 - The featured slope at Lee On Estate.

At a recently completed phase of Pak Tin Redevelopment, an innovative approach was adopted to stabilize a loose fill slope in order to retain the mature trees near the slope crest.



(1) 傳統方法



(2) 嶄新方法

The original design was based on the conventional approach of replacing the top 3 m of loose fill with compacted fill. The proposed work would unavoidably require removal or transplanting of the trees. To ensure the adjoining road would not be undermined, temporary excavation and lateral support works would also be required.

The alternative approach was to provide a 3 m compacted fill capping to the existing slope to ensure the necessary safety standard could be achieved. Before laying the compacted fill, a ring pit was constructed around each tree or group of trees as protective measures. The loss of space at the slope toe was compensated by the gain in space at the crest (see diagram opposite). The mature trees could therefore be kept in place and construction time was much shorter.

Photo 13 – Construction of compacted fill capping.

Photo 14 – The completed slope works.

CONCLUSION

The first two reported cases are respectively the champion and first runner-up in the 2003-04 Best Landscaped Slope Awards Competition. Together with other examples cited, they illustrate the importance of an integrated approach that brings together the contributions of various building disciplines, in particular the geotechnical engineers and the landscape architects. The process of slope design in public housing developments is similar to that outlined in GEO Publication No. 1/2000 (GEO, 2000) but more complex. Treatment of slopes and retaining walls is conceived as an integral part of the development layout planning and design, as well as a natural and sustainable component of the built environment in the associated architectural and engineering studies. Opportunities should be maximized to achieve multi-purpose functions to meet the residents' needs. It is hoped that the sharing of experience in this paper can help promote a holistic and functional approach in the design of slopes and retaining walls that extends beyond pure slope safety consideration.

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ACKNOWLEDGEMENT

The Author is grateful to the Director of Housing for his permission to publish this paper, and to his colleagues of various disciplines and sectors who had contributed to the design and construction of the projects cited.

A SUSTAINABLE SKIN USING GEOFIBER FOR MAN-MADE SLOPES IN HONG KONG – A CASE REVIEW

Noriman S. C. Mak[1], Y. Alexander Ho[2]

Abstract: Hong Kong hillsides have been undergoing changes in the past decades: transforming concreted slope facings to more vegetated hillsides. Numerous techniques for sustainable vegetated slopes have been tried since 2000, and the use of continuous fiber-reinforced soil technique (Geofiber) as a protective skin therapy for man-made slopes in Hong Kong have provided valuable insights in creating sustainable hillsides. The unique skin of Geofiber is strong enough to resist the severe Hong Kong tropical cyclone conditions and provides a stable base essential for plant growth and natural succession of vegetation on slopes. This special slope skin therapy enables each man-made slope face to be tailor-made to suit the natural surrounding environment with the re-establishment of ecological equilibrium and long-term sustainability. This paper presents a case in Hong Kong that describes the characteristics of the Geofiber skin with time. Samples were collected and tests on the strength and permeability of the Geofiber skin were carried out initially and 3 years after application. The results on the Geofiber skin characteristics with time are promising and serve to provide a stable and sustainable protective skin to man-made slopes for re-establishing natural environment on slopes that needs very little maintenance.

INTRODUCTION

Concreted slope surfaces are gradually phasing out to vegetated surfaces on man-made slopes in Hong Kong. The system of impermeable membranes and weepholes for slope protection necessitated regular maintenance and periodic major re-surfacing for the slopes. In addition to the maintenance cost to upkeep the functional aspects of the protective elements, the unsightly visual appearance of the hard surface created on the slope is not environmental friendly nor sustainable. Over the past decades, a soft approach has been pursued in the slope industry all over the world for sustainable slopes with little maintenance.

For steeply inclined hillside surfaces of Hong Kong, the search for a technology, a system or a material to provide vegetated sustainable slopes have been particularly challenging. The ideal slope skin has to exhibit dual properties that satisfy the engineering and the ecological fundamentals of a slope face. In this respect, the material has to possess two unique characteristics:

1. An inherent skin strength that is strong enough to be formed on steeply inclined slopes and withstand the most severe weather conditions of Hong Kong.
2. A stable and sustainable skin and that promotes and allows rooting systems of natural vegetation or plants to be developed with multiple facets of ecological life forms re-established.

Numerous techniques and methods have been tried in Hong Kong with varying degrees

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of success to fulfill the two demanding criteria as described (Lui and Shiu 2004). A new approach in the slope skin therapy for steep slopes in Hong Kong using continuous fiber reinforced soil (Geofiber) has proved to be more versatile, environmental friendly, and truly sustainable.

DIRECTIONAL CONTINUOUS REINFORCED FIBER SOIL – GEOFIBER

The concept on the use of directional continuous reinforced fiber soil (Geofiber) is very simple. A stable skin (200mm thick) simulating natural weathered ground is provided with established directional fibers and earthy materials to provide the artificial soil matrix. This matrix therefore provides the foundation for favourable and healthy development of vegetation and natural regeneration of plants. With time, when the sophisticated plant societies gradually formed and their rooting systems well established within and beyond the installed skin layer, this skin layer is further stabilized and enhanced by the natural rooting systems to form an integrated matrix within the skin itself and also with the underlying in-situ material. Furthermore, the ecosystem of the environment is restored and a natural balanced habitat is once again re-established.

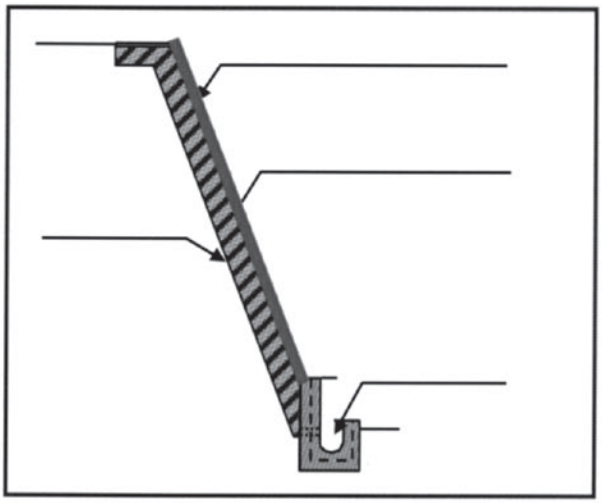


Figure 1 Geofiber-CFRS technique Typical Section

The method of forming the skin is vital in order to provide the necessary engineering and ecological characteristics for long term sustainability considerations. Directional continuous fibers are placed systematically and weaved into the earthy material to create the continuous fiber reinforced soil – Geofiber. This earthy matrix can therefore addresses the geotechnical issues of shear strength, deformation, permeability, durability, erosion resistance etc. on the slope without recourse to the use cement or other admixtures well known for enhancing the strength characteristics but detrimental to sustainable vegetation growth.

At the same time, the Geofiber skin utilizes earthy material which is a technique that uses artificial material that is closest to nature and can address the environmental issue in that area. In some cases, excavated material to form the slope can be recycled to form the Geofiber skin for restoration works. This is therefore a truly environmental friendly technique that can be used in the engineering and the construction industry.

GEOFIBER – CONTINUOUS REINFORCED SOIL TECHNOLOGY

With Geofiber, a continuous fiber reinforced soil technology, continuous polyester fibers are mixed with sand to form a three dimensional matrix to act as reinforcing materials against tension with enhanced shear strength characteristics (pseudo cohesion). In the conventional method of soil reinforcement using geotextiles, mat type of reinforcement materials, these are installed in soil in two dimensions only. Therefore Geofiber material, created by continuously jetting and mixing sandy soil and fibers at the same position with a special device via two transportation channels has a distinct advantage over the conventional reinforcement methods.

As Geofiber is very close to a natural soil, it is expected to exhibit similar engineering characteristics to that of natural soil.

STRENGTH STUDIES ON GEOFIBER

The three dimensional matrix of the Geofiber skin is a very good protective and stable substrate on nurturing roots development for sustainable vegetation growth. The threads itself are easily breakable on its own and hence it does not impede on individual root growth in a microscopic manner. On the other hand, the meshing of directional threads in a macroscopic manner (analogues to a trampoline) will provide the necessary strength and flexibility to accommodate the likely movements resulted from wetting and drying cycles. It is therefore of particular interest to study the strength behaviour of Geofiber with time for providing this sustainable skin layer.

The strength parameters of interest are the shear strength characteristics, density as well as permeability of the Geofiber skin with time. Observations were also made on root development with time to ascertain the healthy growth of the rooting systems within the Geofiber skin matrix and possible development within the underlying in-situ slope surface.

CASE REVIEW – CHO YIU CHUEN

Geofiber skin technology was adopted at a slope (Slope feature 11NW-A/C132) located east of Kai Kwong Lau in Cho Yiu as shown on Figure 2. The slope is approximately 55m in length and trends in a north-south direction. The lower part of the slope is at an averaged angle of 70 degrees with some areas as steep as 80 degrees. The upper part of the slope is at an angle between 35 degree and 55 degrees. The slope height ranges from approximately 7 meters at the northern end to 30meters at the southern end. The approximate surface areas are 850 m^2 as shown in Figure 3.

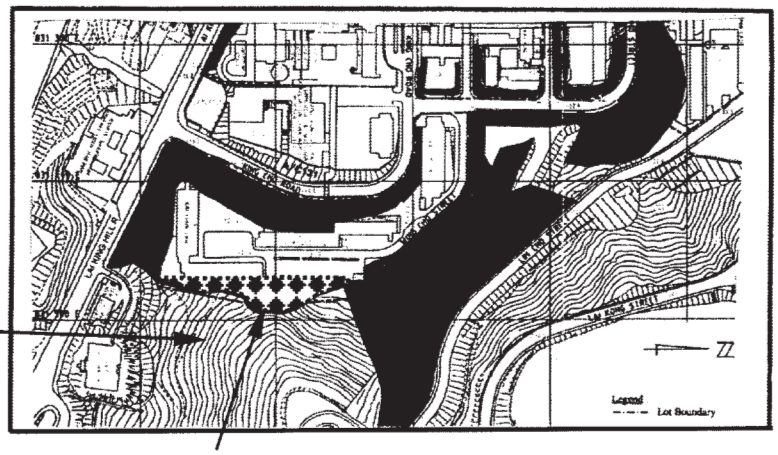


Figure 2 Location Plan of Feature No. 11NW-A/C132

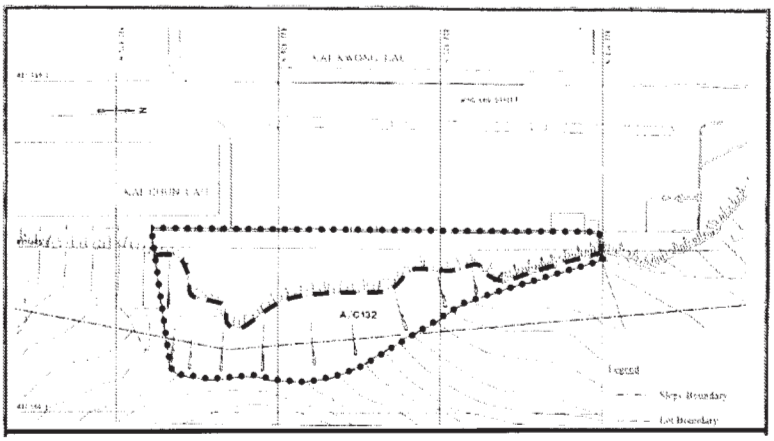


Figure 3 Layout Plan of Feature No. 11NW-A/C132

Prior to the application of Geofiber, the majority of the soil slope was covered with chunam. The lower portion of the slope in rock was left exposed (Figure 4).

Figure 4 Feature No. 11NW-A/C132 before treatment

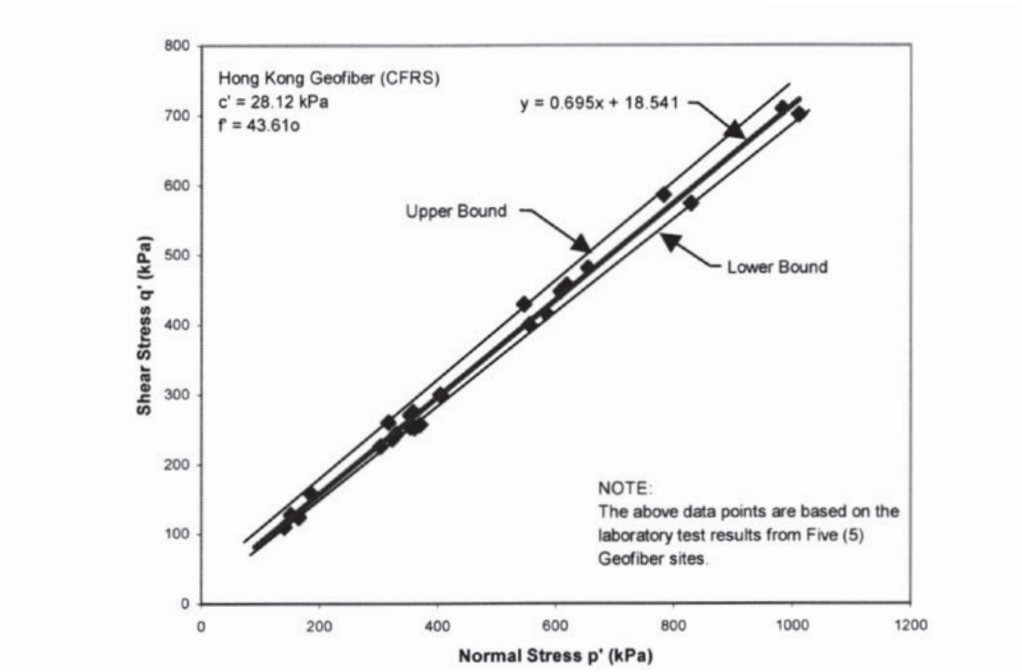
The forming of Geofiber as an environmental protective skin commenced in March 2002 for this slope. The Geofiber skin is of at least 200 mm thick for the upper portion of the slope. Openings were made in the chunamed surface prior to the application of the Geofiber skin to enable close contacts of the Geofiber skin with the underlying in-situ material for sustainable root development into the original ground. For the lower portion of the slope in rock, with fairly undulating surfaces on the rock face, the Geofiber skin is on averaged between 300mm to 500mm thick (Figures 5 and 6).

Figure 5 Geofiber-CFRS Application (1)

Figure 6 Geofiber-CFRS Application (2)

GEOFIBER SKIN IN 2002

To assess the engineering characteristics of Geofiber placed on the slope surface, sample panels were constructed to collect materials (similar to shotcrete panels) for testing in the laboratory in early 2002. This initial set of engineering parameters therefore serves as a benchmark to subsequent readings and data obtained. The initial set of information obtained in 2002 on the strength and permeability characteristic of Geofiber is shown on Figure 7. A comparison with selected Hong Kong soils is shown on Figure 8. It is of interest to note that similarities on the frictional angle of Geofiber with other soils common in Hong Kong and the distinct characteristic of a higher apparent cohesion resulted with weaving of the continuous fiber to form the directional fiber soil matrix in Geofiber compared to other soil types commonly found.



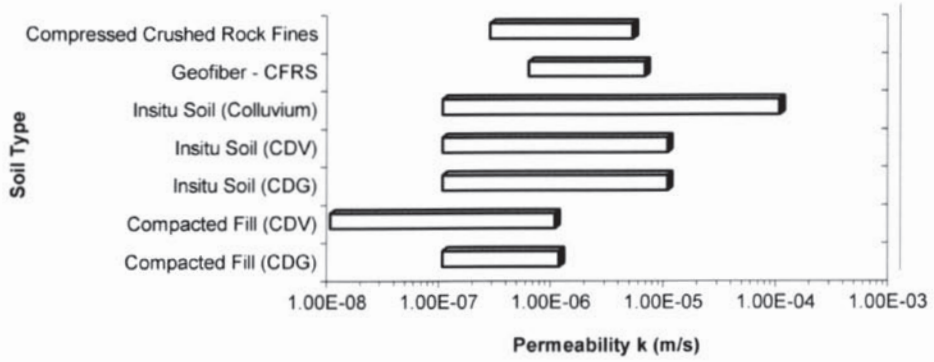


Table 1 Typical Ranges on Values of Geotechnical Parameters for Geofiber - CFRS and Selected Hong Kong Soils

Soil Type	Bulk Unit Weight (KN/m^3)	Dry Unit Weight (KN/m^3)	Shear Strength Parameters		Mass Permeability k (m/s)
			ø	c' (kPa)	
Geofiber – CFRS	18 - 20	15 - 20	40° - 43°	20 - 30	10^{-5} - 10^{-6}
Compacted Fill (CDG)	19 - 21	15 - 19	38° - 42°	0 - 5	10^{-6} - 10^{-7}
Compacted Fill (CDV)	18 - 21	15 - 19	35° - 38°	0 - 5	10^{-6} - 10^{-8}
Insitu Soil (CDG)	16 - 21	14 - 19	35° - 44°	5 - 15	10^{-5} - 10^{-7}
Insitu Soil (CDV)	16 - 21	14 - 19	32° - 38°	5 - 10	10^{-5} - 10^{-7}
Insitu Soil (Colluvium)	15 - 21	13 - 19	26° - 40°	0 - 10	10^{-4} - 10^{-7}

Note: Values of Geotechnical parameters for selected Hong Kong soils (refer Table 8 Geoguide 1)

GEOFIBER SKIN IN 2005

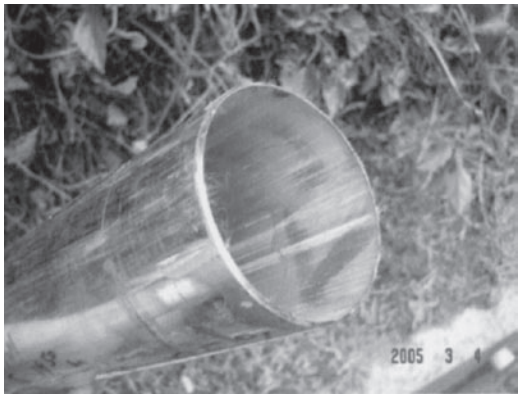


Figure 9 Coring of sample

To check the engineering characteristics of Geofiber placed in this slope since 2002, samples were cored to recover the Geofiber skin with minimal disturbance in March 2005 (Figure 9).

The samples collected were subsequently prepared for triaxial tests to establish the shear strength parameters as well as the permeability test similarly done in 2002. The shear strength result is shown on Figure 10 and the permeability test result is shown on Figure 11.

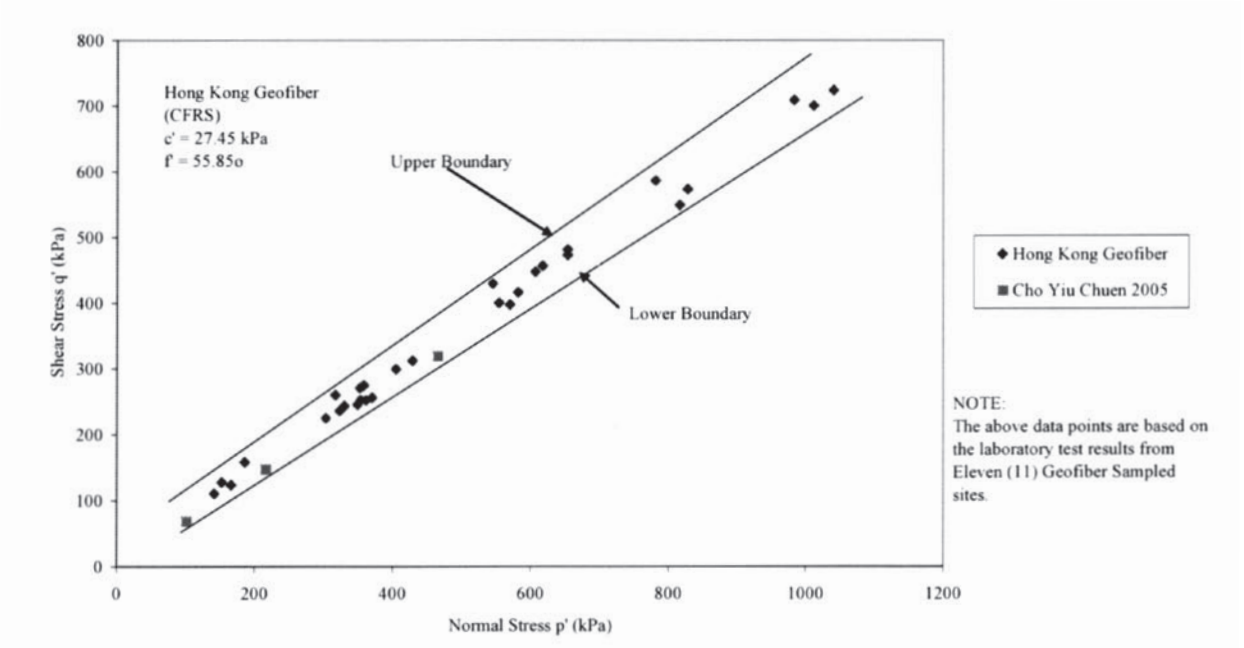


Figure 10 Geofiber - CFRS Strength Characteristics Triaxial Compression Test (p - q Plot)

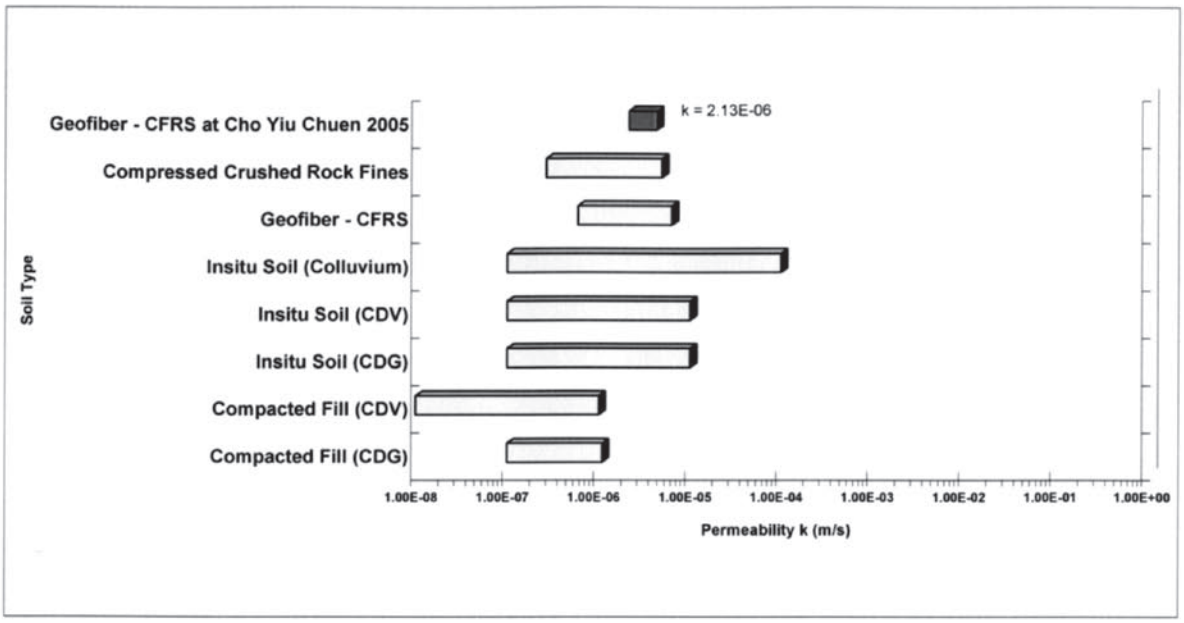


Figure 11 Permeability Characteristics of GEOFIBER – CFRS (Compared with SELECTED HONG KONG SOILS)

OBSERVATIONS ON ROOTING DEVELOPMENTS

Over the past 3 years, numerous significant events have bombarded this slope face. A detailed collection of the events and this slope's performance have been described in (Mak and Ho, 2004). It is of interest to check the integrity of the rooting system within the Geofiber skin, and the degree of growth over the 3 years. Bulk samples (Figure 12) were collected with particular care to minimize disturbance to the natural rooting arrangement. Figure 13 shows the rooting established since 2001 within the 200mm thick Geofiber skin.

Figure 12 Collection of samples

Figure 13 Root establishment

SUMMARY OF FINDINGS

The studies on the Geofiber strength carried out by comparing the engineering characteristics exhibited in 2005 to that of a baseline established in 2002 shows that the Geofiber skin possesses the same engineering characteristics over the past 3 years in terms of shear strength and permeability. The integrity of the Geofiber matrix and an environment for nurturing natural plant growth is further evidenced by the findings.

Further observations and studies on the rooting development with the extraction of samples on the slope face suggest that healthy roots are progressing in development within the Geofiber body for the past three years. The natural rooting systems intertwined with the fiber threads to provide a safe and durable skin for protection of the slope and at the same time nurtures future root growth.

CONCLUSIONS

A 3 year old slope with skin implanted using continuous fiber reinforced soil – Geofiber have been studied. The use of this therapy to provide a natural sustainable skin on man-made slopes in Hong Kong has been tested with encouraging and satisfactory result. This technique, Geofiber, utilizes natural material (often local excavated material) to provide safe and sustainable natural man-made slopes with directional continuous fibers. As natural ingredients are the major components of this technique, the Geofiber skin thus formed not only provide a durable and safety skin protection to the slope in consideration, it also cultivates plant growth of various origins by offering a very stable environment for the ecosystems to be re-established.

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Electrical Resistivity Imaging for Slope Investigation

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Chunlei Liu[2], Hui Fang[2]

Abstract: Significant number of landslides often occur in steep slopes in Hong Kong. The presence of underlying conduits, such as sinkholes, soil pipes, or cavities, may weaken the slopes. Most investigation results on slopes demonstrate that underground moisture plays an important role in slope stability. The information of groundwater infiltration/flow is thus critical in the understanding of the mechanism of slope failures.

Electrical Resistivity Imaging (ERI) could be used to trace the anomaly of high ground moisture. Firstly, underground moisture causes lower resistivity response related to that of surrounding materials. Secondly, ERI provides continuous underground electrical image with controllable resolution and surveying depth. Thirdly, ERI can be conducted in a period of 2-3 hours, providing a fast and effective non-destructive tool for slope investigation. Finally, ERI is suitable to steep slopes, without special requirement in topographic conditions. Underground structures can be identified based on the inversion results of the observed resisitivity data.

The present study was based on the electrical responses of the conduit erosion by ERI method. The trial test used automated 2-D Wenner configuration with 56 electrodes spaced 1m apart in data acquisition. The observed anomaly of low ERI results demonstrated the presence of soil pipes, and the pathway of groundwater flowing. The results were confirmed by saltwater testing in nearby boreholes.

1. INTRODUCTION

The steep slopes in Hong Kong have undergone strongly weathering process. Major slope failures happen not only inthe superficial deposits, but also in the underlain bedrock. The superficial landslide could result in serious damage in property and loss of human life. It is difficult to observe the conduit erosiondeveloping beneath the slope surface. The presence of soil pipes, sinkholes, or cover-collapsed cavities would significantly reduce slope stability. Monitoring and prevention of slope failures is one of the critical issues of the Hong Kong community. The ongoing Landslip Preventive Measures (LPM) Programme, is being implemented to rectify the safety of substandard existing slopes.

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Due to the limited land area of Hong Kong, a huge number of buried water-carrying services have been constructed within or in the vicinity of slopes. Water leakage of the buried water-carrying pipes may lead to slope failures (Morgenstern 1994, Zhang et al. 2004). Hong Kong government published Code of Practice on Inspection & Maintenance of Water Carrying Services Affecting Slopes in 1996. For those slopes with water-carrying services, allowance should be included in the slope stability analyses for possible leakage from buried services.

The observations of sinkholes or soil cavities beneath the slope surface are still a challenge, because underground erosion is closed to the groundwater activity in the bedrock full of fractures and joints. Recently, a great attention has been focused on the electrical resistivity imaging (ERI) method (Loke and Barker 1995). Firstly, the electrical signals measured are related to water moisture and therefore can be utilized in the description of the conditions of the soil/rock mass. Secondly, the ERI can obtain continuous image of the soil/rock mass, thereby overcoming the pitfall of one-point borehole survey. Thirdly, ERI method is flexible in data acquisition. The aim of this work is to present the electrical responses of the soil/rock mass of various state of weathering. In the selected trial test, ERI resistivity image provides a near-surface inspection tool for better understanding of groundwater behavior in jointed granite rocks. A specific band of low resistivity indicates physical erosion, which was proved by in-situ tracing of saltwater flows.

2. ELECTRICAL PROPERTY OF SOIL MASS

2.1 Soil Resistivity

Electrical Resistivity is an electrical parameter describing a mass material inhibiting electrical current flow, in terms of the voltage measured across a unit cube's length divided by the flowing current through the unit area. As shown in Fig.1, when a current I is injected into a uniform half space, the current density through a half sphere of radius r is:

$$J = \frac{I}{2\pi r^2} \tag{1}$$

Correspondingly, the electric field strength E can be determined as:

$$E = J\rho \tag{2}$$

where ρ is the electrical resistivity, and the potential V cross a distance r is:

$$V = \int_r^\infty E dr = \int_r^\infty J\rho dr = \int_r^\infty \frac{\rho I}{2\pi r^2} dr = \frac{\rho I}{2\pi r} \tag{3}$$

and the electrical resistivity becomes:

$$\rho = 2\pi r V / I \tag{4}$$

The electrical property is obtained by measuring the potential at a distance apart from an electrical source over the supplied electrical current, of physical unit Ohm-m or Ohm-cm.

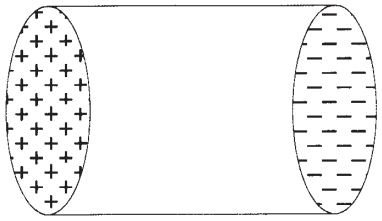

Fig. 1 Electrical Property of a material

Soil resistivity reflects the electrical property of the soil mass. Any material has an electric resistivity range different from that of surrounding materials. In general, the measured resistivity value in Hong Kong slopes varies from 10Ωm to 50000Ωm (IGGE(HK) 2003). Bedrocks have high resistivities (≥500Ωm), while boulders or soil pipes have relatively low resistivity values (<500Ωm roughly). It is difficult to determine the lithology based on a single resistivity value, because the electrical property may be affected by many factors, such as materials (soil/rock), water moisture, and temperature. In general, harder rocks have higher resistivity values. Table 1 reflects the resistivity changes with material property. For instance, water moisture may improve the conductivity of materials. In a material filled with pore water, temperature plays an additional role. When temperature increases, resisitivity values of underground rocks decrease accordingly. Soils under different conditions may have different electrical responses.

Table 1 The influence of material property on electric resistivity values

Material / property	Effect
Water	Decreases
Air	Increases
Salinity	Decreases
Clay	Decreases
Grain size	Increases
Porosity	Decreases
Fracture density	Decreases
Weathering degree	Decreases
Compaction	Increases
Lithification	Increases

The changes in electrical resisitivity can be utilized to reflect the anomalies in geological conditions. ERI has therefore been adopted for trial use ofunderground survey in slope investigation in Hong Kong (IGGE(HK), 2003).

2.2 Sinkhole Model

Sinkhole is a special geological structure, often occurring in limestone region. There are a lot of reports on the geo-hazards of karst conduits found in Hong Kong (Geological Society of Hong Kong, 1990). The presence of the sinkholes made the exploration and remediation of the slope difficult. Among the factors inducing slope failures, various water sources and conduits provide the critical conditions in

groundwater activity. Since the size of the conduits carrying water flow is so small that current non-destructive methods are unable to identify the defects in microscale. Several parameters are often used to predict the defects in massive scale, such as crack/fracture zones beneath the slope surface, ground deformation, and weathering zone/area. Groundwater activity resulting in resistivity alternation, thus, can be extracted using ERI signals.

Sinkholes are often observed in the deep rock mass. The development of sinkholes can be simplified in three stages, as shown in Fig.2. In stage I, the superficial cracks or fractures collect the groundwater or rainfall, flowing into the subsurface non-compact zone; the filled water will expand under high temperature, enlarging the fractural zones. In stage II, the acid water dissolves the surrounding rocks, washing off through deep conduits. In stage III, the sinkhole enlarges further, resulting in the upper roof of the sinkhole collapsed and forming over-collapsed cavity. During the different stages, the water-carrying conduits show signs of mechanical erosion.

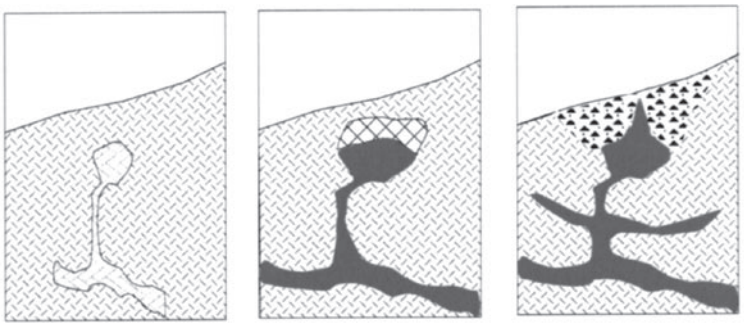

Fig. 2 Model of Sinkhole Development

2.3 Data Acquisition System

In order to obtain the signals of water flow, ERI can be designed in different geometries. Fig 3 demonstrates the geometry of Wenner configuration, in which the electrodes are equally spaced. In general, a bigger electrode interval has deeper penetration; however, the resolution may be reduced. When the major objective is to obtain the information of the deeper geology, wider electrode spacing is the best choice. On the contrary, shorter electrode spacing is the best configuration for shallower or smaller investigation areas. Therefore, it needs a careful balance between the penetration depth and spatial resolution in fieldwork design.

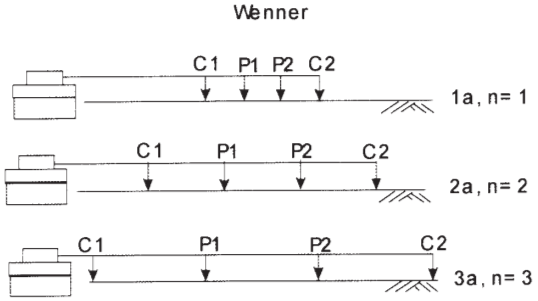

Fig.3 Wenner acquisition configuration. C1 and C2 are current electrodes, P1 and P2 are potential electrodes.

3. ERI TEST RESULTS

3.1 Site Investigation

A trial testing was carried out ina hillside, located on Hong Kong Island, shown in Fig.4. The underlain bedrock comprises granite rock, covered by a layer of weathered rock in combination of soil and boulders. A portion of the slope is covered by vegetation. The main part of the slope is sloping at angles of 20-40°. The maximum angle is 40°. Toward the toe of the slope, the angles flat off. The aim of the fieldwork is to investigate the conduits existed beneath the slope surface by using ERI method. Combining with saltwater testing, the underground conditions can be evaluated.

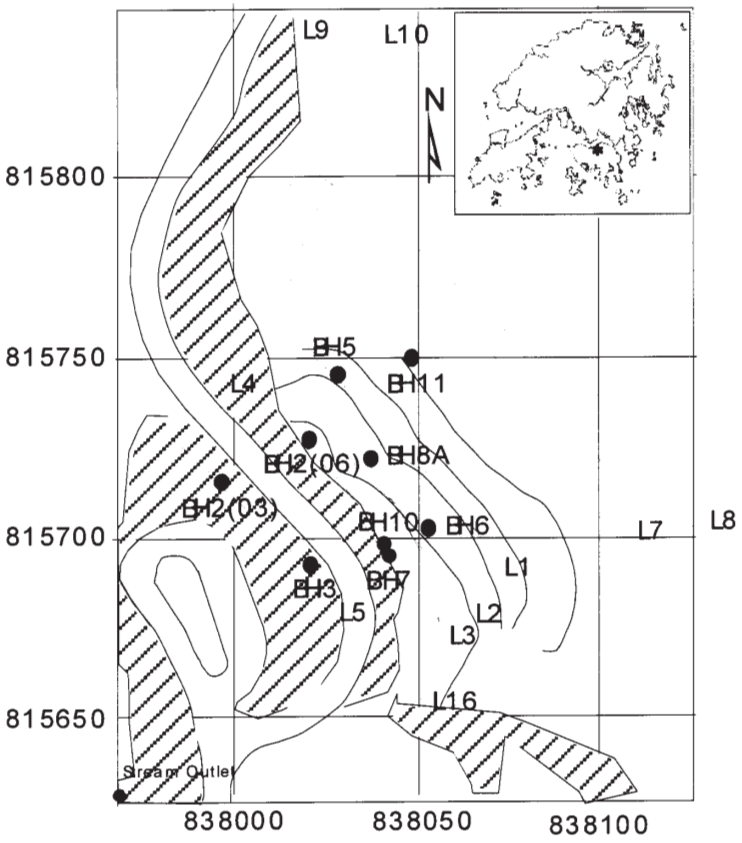

Fig. 4 Site near Yee King Road

3.2 Data Acquisition

As shown in Fig.4, several traverse sections are designed along the strike of the slope. Multi-electrode resistivity testing was conducted with a 64-channel Wenner system. The electrode spacing for these lines was selected at 3m apart. This setup provides a linear surface coverage over the maximum length of the slope features. The range of the setup allows a maximum investigation depth of about 12m. The coordinates of several points were measured as controlling points. The equipment parameters were listed in Table 2.

Table 2 Equipment parameters

Property	Quantity	Property	Quantity
Number of channels	64	Transmitter	lower than -80dB
Resolution	3μVpp	Output power	400 Vpp / 0.5 App
Maximum input signal	4 Vpp	Cycle type	Square wave
A/D converter resolution	16 bits	Pulse length	1s, 2s, 4s, 8s
Frequency Filtering	50/60Hz notching	Input impedance	20MΩ

3.3 Data Processing and Ressitivity Results

Self-calibration of equipment was carried out on site before the formal survey. The relative mean-square error of apparent resistivity is set at 1.7% for good quality raw data acquisition. Resistivity profiles were created through an inversion processing system RES2DINV (deGroot-Hedlin and Constable, 1990; Loke and Barker 1996). Extremely abrupt changes in resistivity raw data were smoothed during the inversion process. The resulting profiles of ERI were interpreted to represent subsurface features of interest.

Fig.5 showed the detailed subsurface information, especially over the water-saturated void. Several features were obvious. There was an extremely low resistivity zonemarked "A" at a depth of about 5m beneath the surface. The isolated low anomaly reflected a sinkhole in the initial stage of karst terrain. No obvious superficial low-resistivity zone, but significant anisotropy, occurred horizontally at a depth of less than 6m.

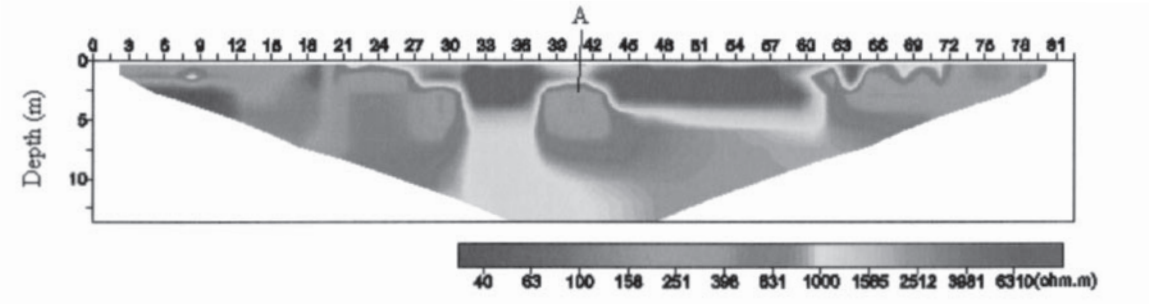

Fig. 5 Geoplogical structure inverted from resistivity results

3.4 Saltwater Testing

Five boreholes were selected for in-situ saltwater testing. As shown in Fig.4, the boreholes were located at different heights. When saltwater was injected in a borehole, sampled water conductivities were measured in other boreholes. The conductivity would increase with the increment of the sampled water salinity. Before the testing, all the boreholes were cleaned already with clear water for 24 hrs. The water was sampled in three depths: stream outlet (near ground surface), standpipe (5m below the surface), and piezometers (at 10m below the surface) from surface to deeper borehole. Details were listed in Table 3.

Table 3 Selected Locations for Saline Injection and Water Sampling

Test No	Saline Injection Point	Water Conductivity Sampling Point
1	BH 2(96)	BH 2(03), BH 3(s), BH 3(p) Stream Outlet
2	BH 11	BH 2(96), BH 3(s)*, BH 3(p)**, BH 5, BH 10, Stream outlet
3	BH 5	BH 2(96), BH 3(s), BH 3(p), BH10, Stream Outlet
4	BH 6	BH 3(s), BH 3(p), BH 7, BH 10, Stream Outlet
5	BH 8A	BH 2(96), BH 3(s), BH 3(p), BH 6, BH 7, BH 10, Stream Outlet

Note: * Water sampled at standpipe of the borehole; ** Water sampled at piezometer in the borehole

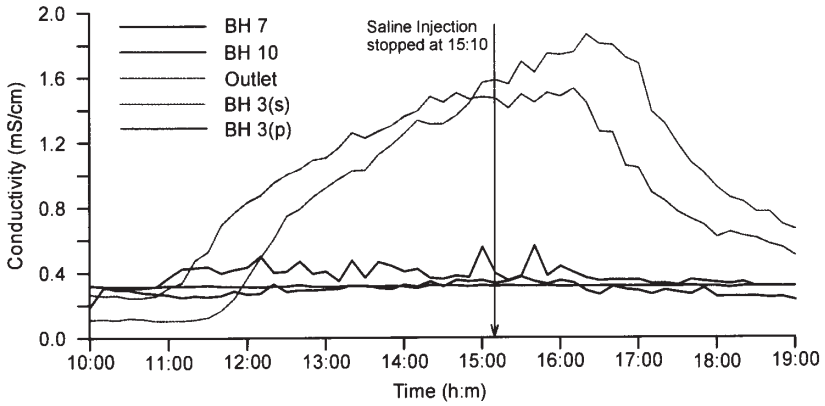

Fig.6 Curves of conductivity of water samples versus time

Saline water injected at location BH6, and water samples were collected at BH3(s), BH3(p), BH7, BH10 and the Stream Outlet during the time from 10:00:00 to 19:00:00 on Oct 9, 2003. The stream outlet has the longest distance from BH6. The results were showed in Fig.6. Obviously, borehole BH3 directly below BH6 had the strongest response when BH6 was injected with saltwater continuously. The water sampled at standpipe had the shortest period in response to the commencement of injection . The water head had an obvious response period of about 1hr (at 11:00am) after the start of injection (10:00 am). The water at stream outlet had a longer delay of 1.5hr, while the water sampled at piezometer had no obvious response in conductivity.

4. DISCUSSION AND CONCLUSION

Saltwater testing results confirmed the existence of soil pipes. Fig.6 indicates the flowing speed of the testing saltwater in the soil pipes or fractures is almost equal to the speed of the water flowing on the surface. Significant alternation of the conductivity of the water sampled at standpipes is also similar to the features measured on the surface. The results indicate a possibility that the soil pipes or fractures are extended to the surface. Hence, surface water and groundwater may interact somewhere in the slope.

Fig.5 shows the uneven zone of resistivity changes at a depth of about 5m below the ground surface. The zone probably reflects the most active zone of groundwater flow. As we know, the degree of weathering at a point decreases with depth. While the image shows higher resistivity values at near surface than deep below, groundwater activity may produce soil pipes or fractures in a variety of spatial geometries. The exploration of the flow paths is difficult using traditional surface checks, since a well-protected slope surface does not mean good ground conditions beneath the surface.

The measured data show a tendency of soil pipes development along the direction of the slope. This means soil pipes could be induced by the gravity of flowing water, rather than through natural fractures/joints. It should be noted that ERI method cannot replace some of the traditional inspection methods, but provides additional water related information in electrical property of the subsurface materials.

In conclusion, use of ERI characterizes the electrical property of slope features; the low anomaly points to the areas prone to sinkhole or void development. ERI relative anomalies allow one to deduce groundwater behavior in a small scale, as well as to observe the concealed slope failures. The erosion condition of the underground materials can possibly be evaluated based on the ERI results. The high quality of the measured data and saltwater testing demonstrate that ERI could be a non-destructive testing tool for slope survey under certain circumstances.

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STUDY ON THE USE OF FIBER SOIL IN SUSTAINABLE VEGETATION DESIGN ON MAN-MADE SLOPE

Ho Tai-Pui, Daniel [1]

Abstract: By using organic recycled material, Fiber Soil, to provide a sustainable vegetation and safe man-made slope, a study has been carried out to assess the performance of slope vegetation projects in Hong Kong for the past 6 years. The objectives are to reduce green waste material and demolished material from the slope stabilization site, shorten the construction period and re-establish the ecosystem on the man-made slopes. On fifteen man-made slopes, medium to high green coverage was found even after two dry years in 2003 and 2004. Concentrations of macronutrients in Fiber Soil would be stabilized in one year after completion of the system and thereafter. A mesocosm experiment and field trials on man-made slopes were conducted in mainland China and Hong Kong respectively, to assess the potential usage of local organic compost using Bio-Chemical Composting Treatment developed by the Southern China Agriculture University. In the experiments, the vegetation performance and nutrient contents on the slopes applied with different sources of Fiber Soil were investigated. Recommendations will be provided on how to use the Fiber Soil with associated ingredients according to different geometry, environmental and geology condition of slope in order to achieve an optimal performance of erosion control and high level of bio-diversity of vegetation on man-made slopes .

INTRODUCTION

Enhancing the aesthetics and ecological rehabilitation of man-made slopes has become a public concern and one of the key elements of Hong Kong government's policies since late 1990s. Under the Landslip Preventive Measures Programme, the appearances of 470 government-owned slopes have been improved by covering with green vegetation in the past three years (HKSAR, 2002). In order to optimize the land use and minimize the disturbance caused to the adjacent area, the traditional methods such as hydroseeding or hydroseeding with erosion control mat, has problems in establishing healthy and full vegetation covers on steeper slopes. For slope gradient greater than 60°, Geotechnical Engineering Office (GEO) of the Civil Engineering Department HKSAR suggested proprietary products to be used for slope greening (Chan, 2002).

Mulching system is one of the most common applications for establishing vegetation covers on non-soil surface slopes (Lui & Shiu, 2004). In general, the advantages of mulching system are quick installation and the ability to form smooth finished surface on uneven slopes (2004). In the GEO study for assessing the performance of eight proprietary products of the mulching systems, Lui & Shiu (2004) found that Toyo-Mulching System and Hong Kong Mulching System are two of proprietary products that have delivered satisfactory engineering and vegetation performance.

In Hong Kong, natural topsoil on man-made slopes are usually inadequate or their qualities of nutrient and humus contents are not suitable for root development of plant. Therefore, application of mulch/soil mix is necessary for establishing healthy vegetation covers. The major difference between these two mulching systems and the other six proprietary products is their qualities of soil/mulch mix. For both systems, organic

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Hydromulching technique is used to spray an organic mortar called Fiber Soil, which consists of organic compost, peatmoss, chemical fertilizers, planting seeds and tackifier. The Fiber Soil contains approximate 50% of imported organic compost by volume.

The main advantage of using Fiber Soil is to support self-sustainable vegetation on man-made slopes of soil or non-soil surface conditions by providing a growth medium for rooting anchoring and nutrients for plant. Decomposed material of organic matter includes nitrogen (N), phosphorus (P) and potassium (K) compounds, which are the macronutrients for plant development (Rieley & Page, 1999). In addition, it helps retaining soil moisture, reducing irrigation requirements, and enhancing soil structure over time (Caltrans, 2004). Even after adverse weathers like heavy rainstorms and tropical cyclones, the Fiber Soil is self-sustainable on steep slopes, and prevents in-situ earth materials from erosion.

In the performance review study conducted by the GEO and the Geotechnical Section, Campus Development Office of the Chinese University of Hong Kong, satisfactory vegetation coverings (84.1%±25.2% SD; 87.6%±14.4% SD) were observed on man-made slopes which have applied the organic Hydromulching technique (Lui & Shiu 2004 and Geotechnical Section 2002, respectively). As the vegetation surveys in both studies were conducted during the plant growing season, whether the vegetation can tolerate drought-stress after dry winter is not determined, especially in 2003 and 2004 when the records of annual total rainfall were 12.3% and 21.5% below average (HK Observatory, 2005). In addition, it is not certain if the Fiber Soil can provide sufficient nutrients to maintain establishment of plant in the long run.

In view of the above, the first part of study includes the performance review of Fiber Soil in March 2005. Fifteen man-made slopes was selected, where the organic Hydromulching technique was applied on hard surfaces such as shotcrete, rock-filled, chunam and bed rock, and the slope with the soil surface such as complete decomposed granite in different time periods, where examination on vegetation performance and nutrient concentration was conducted in March 2005.

In the second part of this study, we will examine the potential use of local recycled materials in the Fiber Soil. Organic compost in the Fiber Soil has been imported from foreign countries. Its major advantages include odorless and will not adversely influence the air quality around the job site. Recently, researchers of New Fertilizer Resource Laboratory of Southern China Agriculture University (SCAU) researchers has developed a rapid-composting method, called Bio-Chemical Composting Treatment. It greatly increases the decomposition rate of organic fiber of collected green waste, and the process of composting is reduced from 180 days to 15 days. SCAU organic compost is also odorless, and its initial nutrient quality is higher than that of the imported organic compost (Table 1).

Table 1. Initial nutrient concentrations of Fiber Soil Mix using SCAU or imported organic compost

Fiber Soil Mix using	Conductivity (100μS/cm)	Organic Matter (g/kg)	Total Nitrogen (%)	KCl - N (mg/kg)	Total Phosphorus (%)	Total Potassium (%)
SCAU Compost (Treatment)	20.05	41.01	1.77	682.24	1.14	1.44
Imported Compost (Control)	22.6	49.43	0.73	811.32	0.63	0.48

In Hong Kong, pressure on landfill usage has always been substantial. It is estimated that the landfill space will be completely exhausted in the next 10 to 15 years (Chan et.al., 2004). Therefore, government incentive and public attention of waste reduction and sorting have recently arisen. As a result, the cost of green waste collection and processing of organic compost will be greatly reduced. Using local organic compost on greening man-made slope greening will further relieve the pressure on the landfills in Hong Kong.

A mesocosm experiment in Guang Dong Province of China and three field trials in Hong Kong were conducted to examine the potential use of SCAU organic compost on slope

greening in Hong Kong. This study will compare vegetation coverage and short-term (less than 1.6 years) nutrient supplies in Fiber Soil using imported organic compost and SCAU organic compost.

METHODOLOGIES

Performance Review of Organic Hydromulching Technique

This study has selected fifteen man-made slopes, where the organic Hydromulching technique was applied by Toyo-Greenland Co., Ltd. and Hong Kong Mulching Co., Ltd. The major components of this technique were (1) installation of 3-dimension PVC coated wire mesh for reinforcing the Fiber Soil, (2) spraying of minimum of 50mm thick of Fiber Soil and (3). Installation of bio-degradable erosion control mat for protecting the finished surface of the Fiber Soil. The initial nutrient concentrations and properties of the Fiber Soil were listed on Table 1.

In addition, slow release fertilizer strip was installed for the some slope with gradient more than 45°. It was inserted horizontally into turf reinforcement mat at 500mm c/c. Seeds of 15gm/sq m of *Cynodon dactylon* (Bermuda 百慕達草) and 10gm/sq m of *Paspalum notatum* (Bahia 百喜草) or other recommended vegetation seed were mixed into the Fiber Soil. Climber species of *Wedelia* (蟛蜞菊) was planted by planter tube or plug planting at 1,000mm c/c on the surface according to the surface condition of slope. Eco-bag filled with Fiber Soil was installed for the pit planting work of woody shrub species according to the landscape design. Soil Creat was installed on the soil slope gradient more than 60° to provide the rigid cover of erosion control of earth material.

Green coverage on each slope was computed with site photo records taken on the same day. Healthy status of the plant (i.e. freshness of shoot tips, plant body and base) was examined in determining the overall green coverage on each slope. Naturally invaded plant species on each slope was counted. Thirty sub-sites were divided evenly on each slope, and ten of them were randomly selected for soil sampling. Top 5cm of Fiber Soil sample was collected twice at each sub-site using a 5.6cm diameter plastic sampling core. All samples on each slope were homogenized and sieved through a 60mm screen. Chemical analyses were conducted by SCAU within 48 hours. Fresh soil with DI water (1 to 5 dilution) was used to measure pH (pHs-25) and conductivity (DDS-307). The rest of soil samples were oven-dried at 50°C for 24 hours, and grinded through a 0.3mm sieve. Organic matter was determined by using Walkley-Black Method (Nelson & Sommers, 1982). Total nitrogen and phosphorus after digestion of Sulfuric acid and Hydrogen Peroxide solution (H_2SO_4:H_2O_2) and KCl-extractable nitrogren were determined by using Spectrophotometry method. Atomic Absorption Spectrophotometry was used to measure total concentration of potassium after H_2SO_4:H_2O_2 extraction. All the above analysis were conducted in duplicates.

Comparison Between SCAU Organic Compost and Imported Organic Compost

A mesocosm study was carried out in one trial site in Huang Jiang Hzen, Guang Dong Province of China and 3 trial sites in Hong Kong from May 2003 to April 2005. The organic Hydromulching technique was applied on four man-made slopes with hard surface cover. Their characteristics were summarized in Table 2.

In the mesocosm study in China, imported organic compost was used as the control result of performance of Fiber Soil. The SCAU organic compost was used instead of the imported organic compost for the application treatment. The initial nutrient concentrations and properties were listed in Table 1. Both of the control and treatment Fiber Soils were mixed with seed of Bermuda, Bahia, White Clover 白車軸草 and *Macroptilium atropurpureus* 紫花大翼豆 at the same rate 62.5g for each species per m^2, and applied to

70mm thick. 15cm long *Wedelia trilobata* shoots were plug planted 1,000mm c/c. During the first month after completion, watering was applied under no rainfall for 4 consecutive days. During the second and third months, water was applied if there was no rainfall for 10 consecutive days. Watering was not applied thereafter.

Percentage of the green cover on each slope was recorded every month, except in December 2003 and January 2004. Plant debris was excluded from the record. In the 3rd, 10th and 13th months after completion, soil sample was collected at the center and four midpoints between the center and corners on each slope. Soil chemical analysis were conducted as described in the above section.

For the trial site in Hong Kong, three man-made slopes were selected for comparing the performance of Imported and SCAU Organic Compost. Two sections were divided and applied with either control or treatment Fiber Soils. The characteristics of each section were described in Table 2.

Table 2. Characteristics of 3 slopes, selected for comparison between SCAU and imported organic compost

Site Location and (Label)	Date of Completion	Year After Completion	Slope Orientation	*Wedelia*	Surface Condition	Subsite	Slope Gradient (deg)	Max. Height of Slope (m)	Length (m)	% Tree Shade
Mesocosm study in Dong Guan District	2003/05/03	1.9	S	*Yes*	Shotcrete	NA	45	3	3	0
Chai Wan Road (1)	2004/09/28	0.5	SW	No	Shotcrete	Control	45	4.1	5.8	30
						Treatment	50	6.4	8.4	0
Yam O Tuk Fresh Water Service Reservoir (2)	2004/04/21	0.9	NW	Yes	Rock Filled	Control	45	24	34	0
						Treatment	45	8.5	12	0
Sai Tso Wan Landfill (3)	2003/08/15	1.6	NE	Yes	Shotcrete	Control	70	14.5	15.5	5
						Treatment	45	2	2.6	0

Three replicates of shoot biomass within a 30cmx30cm quadrant were harvested in each section. Fresh weight of each species was measured separately, and the percentage of green Bermuda grass and Bahia grass was visually estimated after harvesting. 30 numbers of sub-sites were evenly divided in each section, and 5 of them were randomly selected for soil sampling. Soil sample preparation and analysis were conducted in the above section.

Data Analysis

Un-paired Student T-test (StatView ver.5.0.1) was used to determine if there was a statistical ($\alpha = 0.05$) difference of mean green coverage and nutrient concentrations between the control and treatment Soil Fibers. One standard deviation will be reported after a plus/minus symbol (±).

RESULTS AND DISCUSSION

Performance of Organic Hydromulching Technique

Vegetation Coverage – After the dry winter in 2004, the mean percentage of green coverage of all slopes was 63.0% (±35.7% SD). Twelve out of fifteen slopes had over 45% green coverage, and their mean was 75.4% (±23.4% SD) (Fig. 1a).

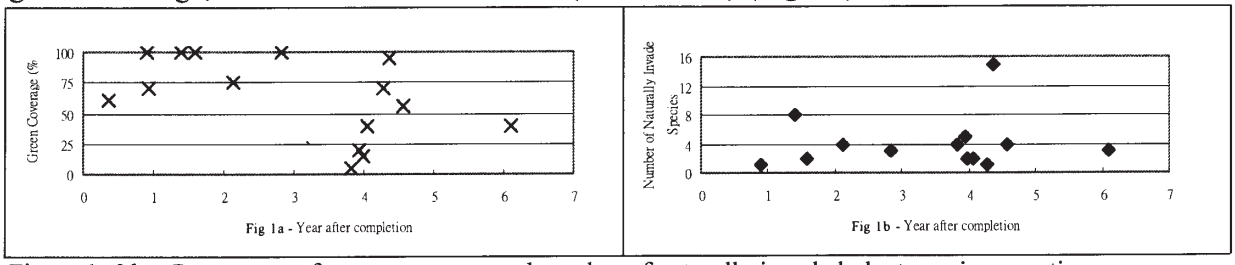

Figure 1a&b – Percentage of green coverage and number of naturally invaded plant species over time

The other three slopes with the lowest vegetation coverage at Tung Tau Wan Road, Tai Hang Road and Tiu King Ling with about 5%, 15% and 20% respectively. The first two sites might be related to heavily shade effects by trees and surrounding buildings. Plants on those shaded slopes might require a longer time to recover after the drought winter. The last site has value 15.0% vegetation coverage which was smaller than the GEO report (Lui & Shiu, 2004). *Wedelia* was the dominant species on all slopes, except for Tiu King Ling, where the dominant species was *Murdannia keisak* (水竹葉) and erosion of Fiber Soil was observed.

Nutrient Concentrations and Chemical Properties Over Time – The change of the quality of the Fiber Soil was measured from the Fig 2a to 2h.

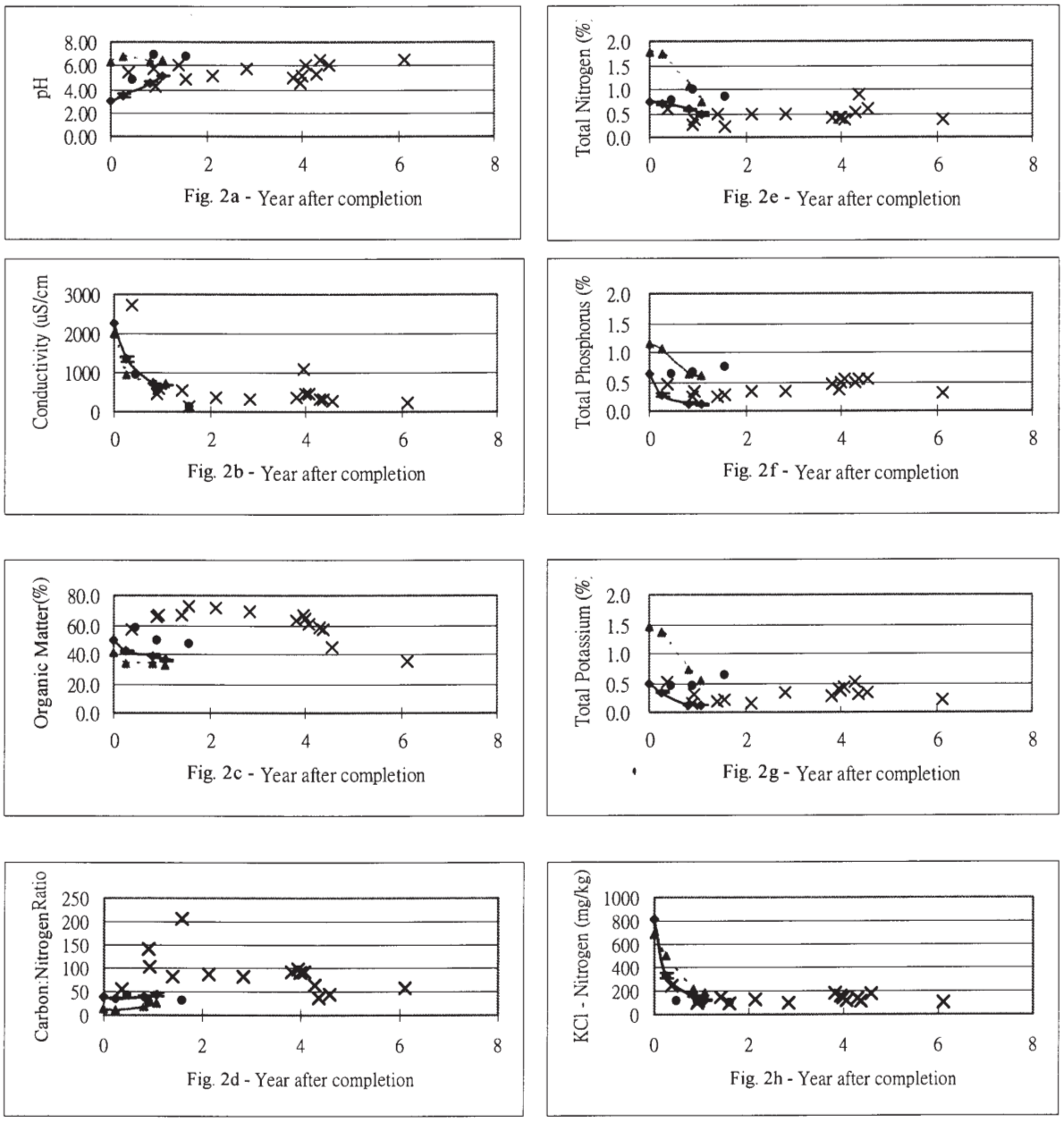

Figure 2a-h. Chemical properties (pH, Conductivity, Organic Matter and Carbon:Nitrogen Ratio) and Nutrient Concentrations (Total Nitrogen, Phosphorus, Potassium, KCl-Extractable Nitrogen) of Organic Fiber Soil. For Mesocosm study, control slopes using imported organic compost (—◆—), and treatment slopes using SCAU organic compost (--▲--); For field trials in Hong Kong, control slopes using imported organic compost (x), and treatment slopes using SCAU organic compost (•).

An increasing trend of pH value and a decreasing trend of conductivity were observed over time (Fig. 2a&b (x)). pH value becoming more neutral might help promoting plant growth and natural invasion of other plant species, which are tolerant to low pH in soil. During the early stage, high levels of conductivity might be resulted from the continuous supply of inorganic nutrients from fertilizer strips. Both trends indicated that Fiber Soil had become more suitable for vegetation over time.

The percentage of organic matter increased up to 70% in the first two years, and then gradually decreased over time due to the decomposition of Fiber Soil (Fig. 2c (x)). After six years, 29.0% of organic matter will be lost comparing with the initial level. Our finding supported their suggestion of mulch reapplication within 3-4 years (Caltrans, 2004). Dominant species changed from grass species (Bermuda & Bahia) during initial stage to *Wedelia* in the later season. The increase of organic matter in the first two years might be related to the decomposition of the pioneer grass species.

The mean total NPK within the first three years reduced 43.8%, 49.2%, and 45.8% from the initial values (Fig 2e-g (x)). The reduction of the nutrient concentrations might indicate the rapid vegetation growth during the first three years. After then, the total NPK concentrations stabilized and were similar to the concentrations in the first three years. Therefore, we might conclude that the total NPK availabilities would not be rapidly reduced in the long-run, after vegetation had established.

Similar to the result in the mesocosm study (Fig. 2h (—◆—)), rapid decrease of extractable nitrogen in the first year was possibly related to the rapid growth of plants and their uptake of ammonium and nitrate. In Figure 2h (x), the mean of KCl-Extractable Nitrogen was stable through out the six years (137.0±44.5 mg/kg). Therefore, it indicated that plant-available nitrogen could be sustained after vegetation establishment in the first year.

Comparison Between SCAU Organic Compost and Imported Organic Compost

Vegetation Performance - In the mesocosm study, vegetation covers on the slope using SCAU organic compost (Treatment) lasted for a longer period than the slopes using imported organic compost (Control) (Fig. 4). In the field trial in Hong Kong, the dominant grass species was generally Bermuda in the first year, and there was very little or no Bahia grass on all slopes. On Slope 1, total and green aboveground fresh weight of Bermuda in the Treatment site was statistically lower than that in the Control site (Table 4). However, on Slope 2 and 3 where *Wedelia* was planted which become the dominant plant species as a result. There was no statistical difference of Bermuda or *Wedelia* biomass between the Control and Treatment sub-sites (Table 4). In both studies, we did not find any adverse effect on the growth of *Wedelia* by using SCAU Organic Compost to substitute for Imported Organic Compost in Fiber Soil.

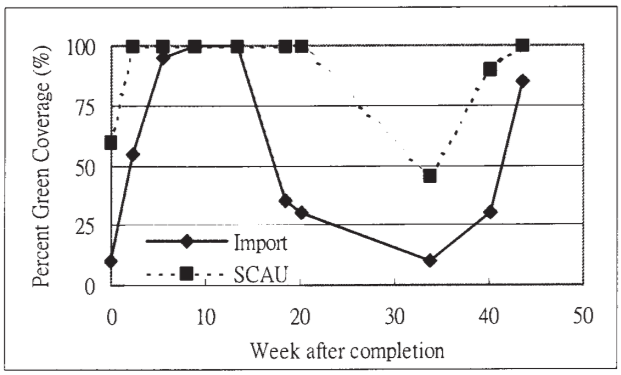

Figure 4. Percentage of green coverage of Control and Treatment Fiber soils, using imported and SCAU organic composts in the mesocosm study

Table 4. Mean and 1SD of Aboveground Fresh weight (FW) in three field trials in Hong Kong
([a] – Statistical Difference P<0.05)

		Fiber Soil using	
	Slope	SCAU Compost	Imported Compost
Bermuda Total FW (g)	1	30.3 ± 6.2[a]	48.8 ± 7.7
	2	22.1 ± 10.9	27.8 ± 9.0
	3	90.8 ± 21.4	72.9 ± 14.9
Bermuda Est. Green FW (g)	1	8.2 ± 2.4[a]	13.8 ± 2.9
	2	0.0 ± 0.0	0.0 ± 0.0
	3	9.0 ± 5.3	4.8 ± 2.0
Wedelia FW (g)	1	NA	NA
	2	75.3 ± 12.2	67.7 ± 18.5
	3	57.2 ± 32.4	23.0 ± 23.6

Nutrient Concentrations - In the mesocosm study, the initial nutrient concentrations of total nitrogen, phosphorus and potassium on the treatment slope were higher than that in the control slope, 58.8%, 44.7%, and 66.7%, respectively (Fig. 2e-g). Throughout the study, the differences of the final total phosphorus and potassium between the control and treatment Fiber Soils increased to 81.7% and 78.8%, respectively (Figure 2f-g). However, the difference of the final total nitrogen reduced to 36.0%. Since the concentration of KCl-extractable nitrogen remains constant in both treatment and control soil (Fig 2d), we suspected that much higher concentrations of Ammonium and Nitrate had been up-taken by plants or consumed up by microbial.

In the field trial, differences of total NPK concentrations between the treatment sites (•) and control sites (x) increased over time (Fig 2e-g), and there was no difference of KCl-extractable nitrogen between the two. Even though there were different results in the two studies, both results showed that nutrient levels of Total NPK and KCl-N in the Treatment Fiber Soils were comparable with those in the Control Fiber Soil.

Conductivity and pH value in the treatment and control Fiber Soils in the field trials follow the trends observed in the mesocosm study. (Fig. 2a-b) The initial levels of pH and conductivity in both soils became more suitable for plant growth over time. However, at the very beginning, the neutral pH and low salt content in the treatment Fiber Soil might help the initial vegetation establishment.

In both studies, percentage of organic matter in the Treatment Fiber Soil was relatively lower than that in the Control Fiber Soil (Fig. 2c). However, in the field trial, the quality of organic matter (C:N ratio) was more stable in the Treatment Fiber Soil, while the quality in the Control Fiber Soil decreased over time (Fig. 2d). Using SCAU organic compost as a substitution for imported compost might provide better quality of organic materials in the organic Hydromulching technique.

CONCLUSIONS

Although the vegetation survey was conducted before growing season and extending beyond two years with below average of rainfall records, the 12 man-made slopes applied with the organic Hydromulcing technique had medium to high vegetation coverage (75.4± 23.4%). We believed that the low coverage on three of these twelve slopes was mainly due to environmental factors, such as heavy shading effects by trees and surround building, rather than the moisture content and nutrient availability in the Soil Fiber. Such that the shade tolerance species may be designed to be planted in the similar site conditions.

This study showed that levels of macronutrients in the Fiber Soil were stabilized in the first year and would not reduce after then. We believed that the nutrient levels in the layer of Fiber Soil could be sustained over time, while supporting vigorous plant growth on man-made slopes. To further determine whether those nutrients in those slopes are adequate, we will conduct plant growth assessment (phytometer) in the future. Based on the result of this study, the nutrient concentrations as parameters of Fiber Soil for organic Hydromulching technique are recommended (Table 5).

Table 5. Long-term nutrient requirements of Fiber Soil for Organic Hydromulching Technique

pH	Conductivity (µS/cm)	Organic Matter (%)	C:N Ratio	Total Nitrogen (%)	KCl-N (g/kg)	Total Phosphorus (%)	Total Potassium (%)
6.0-7.5	400-3,000	> 40	25 - 150	0.1-1.5	0.1-0.2	0.28-0.55	0.2-0.5

In the past six years, there were 4 times of Black Rainstorm Warning Signal (greater than 100mm per day) and 11 times of Typhoon No. 8 or above (HK Observatory,

2005). Even after these adverse weather condition (over 200mm rainfall within 24 hours on the 27 June 2001), engineering materials and Fiber Soil could still sustain on all the slopes, except for Tiu King Ling, where dominant species was *Murdannia keisak* instead of *Wedelia*. As Lui and Shiu (2004) concluded in the GEO report, the Organic Hydromulching system could provide good and long lasting engineering performance. To further improve the versatility of the organic Hydromulching technique, different components (i.e. Fertilizer Strip, Eco-bag, Planter Tube, Soil Creat, permanent erosion control mat) can be applied on different site conditions regarding slope gradient, surface condition and the design of the planting species.

Extensive research conducted by SCAU and the field trials in Hong Kong demonstrated the possibility of using locally produced organic compost. Fiber Soil using SCAU organic compost had higher nutrient contents than that using imported organic composts. In addition, vegetation on man-made slopes, where SCAU organic compost was applied, could be established faster and last longer in dry winter season.

Using local organic compost on slope greening could both provide better greening result than imported organic compost, and also relieve the pressure on landfills in Hong Kong. Based on the current trend of slope greening, we estimated that about 200,000m^2 man-made slopes would be enhanced per year. Using the Fiber Soil with local organic compost will help reducing about 7,700 m^3 of green waste per year. We are currently conducting more studies for determining the proper proportion of SCAU organic compost in Fiber Soil that can promote the best plant growth and reduce green waste dumping to the landfill in the future.

ACKNOWLEDGEMENT

The author would like to express gratitude New Fertilizer Resource Laboratory of Southern China Agriculture University for their contribution to this study; Dr. JW Liao, Dr. XL Lu, Mr. SU Mao (SCAU) and Dr. LM Chu (CUHK) for providing useful comments on this manuscript, and Chuang Cheng-Yu Bula and Wailun Tan for conducting the field works.

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DOUBLE CORROSION PROTECTION OF LONG SOIL NAILS AT DEEP BAY LINK – A CASE HISTORY

Nick Koor[1] & Anson Cheung[2]

Abstract: Preliminary soil nails up to 30 m in length were installed at the Deep Bay Link project to test the effectiveness of grouting long nails. The nails incorporated a double corrosion protection system consisting of a corrugated HDPE sheath. Each preliminary nail was successfully tested to two times the design load and several were exhumed to inspect the grouted nail section. Grout was seen to have completely filled the internal and external void around the corrugated sheath with excellent contact between the grout and the surrounding saprolite. Permanent soil nails with double corrosion protection up to 28 m long are now being successfully installed at the site.

INTRODUCTION

Deep Bay Link (Figure 1) is a duel-three lane trunk road providing a strategic link between the Hong Kong Shenzhen Western Corridor and the existing Yuen Long Highway. Due to its strategic nature the HK$4.0B project has been fast tracked with a construction period of 28 months. Gammon Construction Ltd. are the Main Contractor for the northern section and construction is currently underway and due to be open to traffic by end-2005. The link traverses the broad low lying valley separating Yuen Long and Tuen Mun and is primarily supported by a reinforced concrete viaduct except where the road passes through the northern extremity of the Tsing Shan Range (Figure 1) in a 40 m deep cut. Soil nails support the cut which has up to 60° side slopes and nails up to 28 m long. It is the soil nailed slopes in the northern part of the project which are the subject of this paper.

THE SITE

Topography

The northern part of the site through which the road is in cut is characterised by a north south trending valley surrounded by rounded granitic hills. The valley bottom is at about +40mPD and the surrounding hill slopes have elevations ranging from about +60 to +80mPD which slope gently towards the valley at about 30°. Ephemeral streams drain into the valley from the surrounding hills. The natural terrain is generally sparsely vegetated with small trees and shrubs with some areas of gulley erosion where the vegetation is sparse.

Geological Setting

The Hong Kong Geological Survey Yuen Long Sheet 6 indicates that the site of the major cut is located in an area underlain by regionally metamorphosed fine grained granite. A major slip strike shear zone which trends north-east separates the fine grained granite from Tuffs and Tuff Breccias of the Tuen Mun Formation about 200 m to the south-east. Zones of intense shearing along this shear zone have produced mylonites within both the granite and volcanic rocks of the area. Quartz veins and minor quartzphyric rhyolite dykes are also

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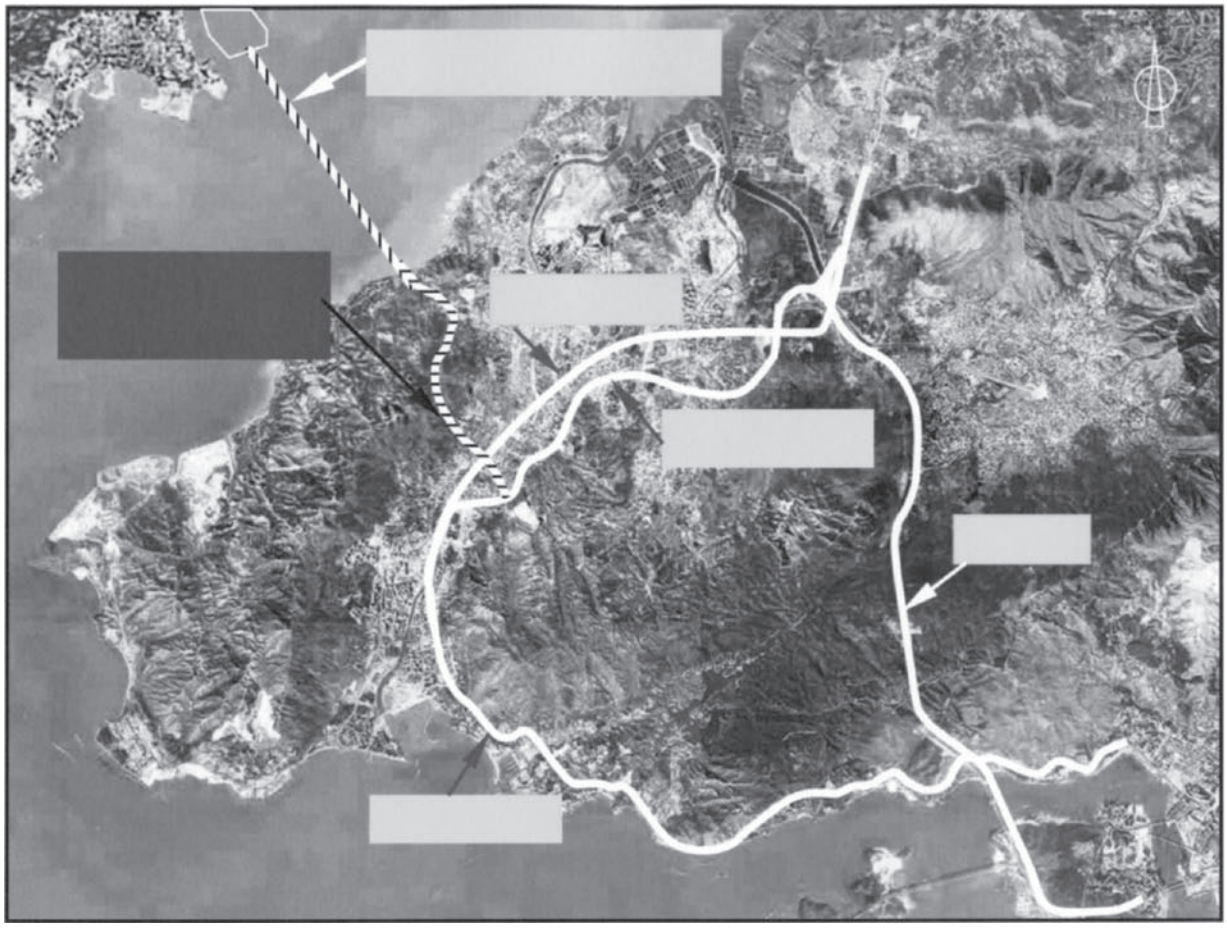

Figure 1 - Deep Bay Link & associated road transportation routes in Western New Territories – Hong Kong

present within the granite. Quaternary superficial deposits occur within the valley floors and are generally termed debris flow deposits on the geological sheet but they may be better named colluvium. Little colluvium is shown to exist on the granitic side-slopes.

CUT SLOPE DESIGN

Ground Conditions

Detailed ground investigation consisting of vertical drillholes, trial pits and laboratory testing confirmed that the site is underlain by metamorphosed fine grained granite with little or no colluvium being present. The granite is variably weathered with continuous Grade III rock as encountered in the drillholes at between 5 and 40m below existing ground levels, the shallower being in the base of the valley. Mylonite zones were also encountered in the drillholes and later observed in the cut slope face. Groundwater levels range between 0 and 20m below existing ground levels.

Slope Design

Limits set on land-take for the project predicted the adoption of either steep soil nailed slopes or some form of vertical retaining structure supporting a cut slope. Due to programme and cost considerations it was decided to adopt the steep soil nailed solution (Figure 2) as

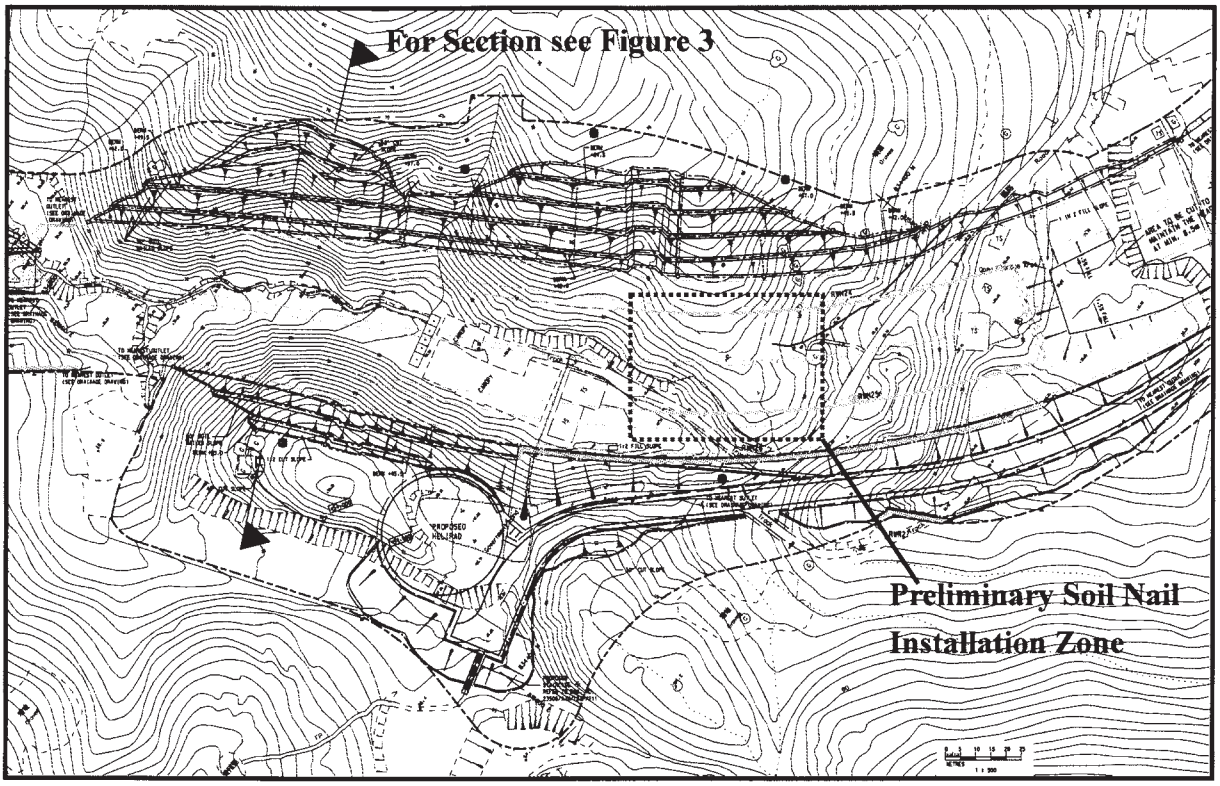

Figure 2 - Layout of cut slopes at northern end of Deep Bay Link

these would be quicker and cheaper to construct. Soil nailed slopes were designed with slope angles of between 50° and 60° incorporating over 2000 soil nails. Slopes up to 40 m high with up to five berms have been designed using conventional techniques used in Hong Kong (Figure 3). Due to the relatively steep side slopes and conservative design groundwater levels adopted, the nails supporting the lower two berms are up to 28 m long and have a design load of up to 460kN. The slope design life is 120 years.

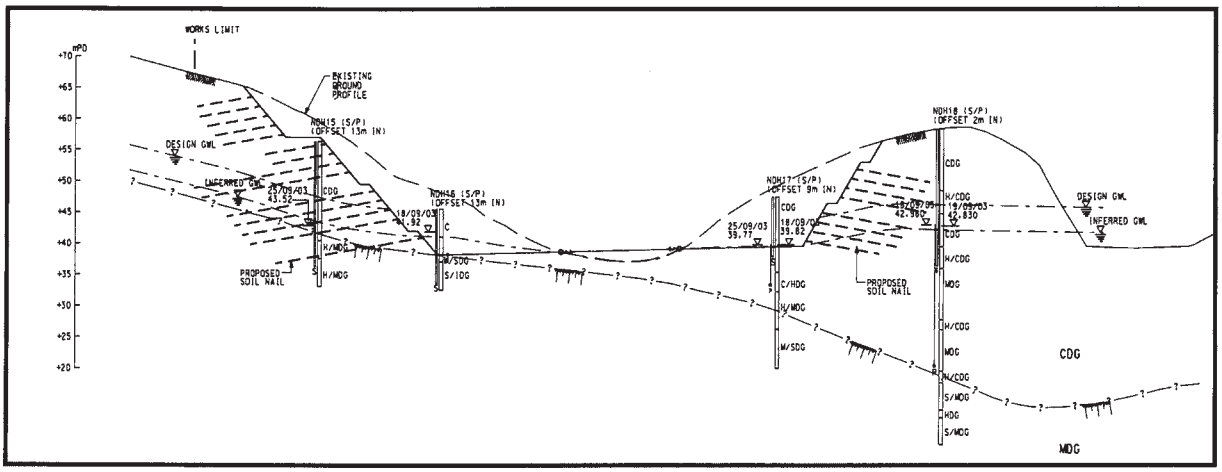

Figure 3 - Typical section through cut slopes at northern end of Deep Bay Link

Ground Aggressivity

In order to assess the potential corrosion of any steel elements installed in the slopes, a series of tests were made during the ground investigation to determine ground aggressivity. The testing and corrosivity assessment followed the recommendations made by Murray (1993). The results of the testing are summarised in Table 1. The results indicate a ranking mark of -6 which indicates that the soil condition is likely to be aggressive and therefore

corrosion protection is required. This result agrees with the study reported by Shiu & Cheung (2003) where they state that a significant portion of the local (Hong Kong) soils has a high corrosion potential.

Table 1 - Aggressivity table

Item	Measured Value	Marks
Soil Composition	Material containing not more than 75% and 10% of particles passing the 63 micron and 2 micron BS sieve sizes respectively. The material passing the 425 micron BS sieve, when tested in accordance with BS 1377, have a plasticity index less than 6	0
Groundwater level at buried position	Soil nail reinforcement will be submerged under design groundwater table	-4
Resistivity (ohm – cm)	10,000 or more	0
Moisture Content	Average 25.5%	-1
pH Value	Average 5.2	-2
Soluble Sulphate (ppm)	Average 105	0
Cinder and Coke or made ground	None	0
Redox potential	Average 417	+2
Presence of sulphate and hydrogen sulphide	0.09% (None)	0
Presence of carbonate	0.18% (Trace)	0
Chloride ion (ppm)	200	-1
	Total	**-6**

Buildabiltiy and Corrosion Protection

Several factors have to be taken into account when designing long soil nails in corrosive ground conditions. They are; the buildabiltiy of the nail; what corrosion protection system to use and; potential grouting difficulties. These three critical elements need to be considered holistically rather than in isolation.

The high nail design loads require the use of standard 50 mm deformed bars (F_y = 460N/mm^2) or smaller diameter (c.36mm) high yield bars such as that manufactured by Dywidag or Macalloy (F_y =1080N/mm^2). Other options such as carbon fiber strands were considered, but at the time of design such systems had not been used in Hong Kong and therefore the time constraints for Government approval ruled out their use. The pros and cons of using the two bar types are at Table 2. Due to the obvious practical difficulties in installing 28 m long 50 mm deformed bars the high strength bar option was investigated further as lead-in times for material ordering and shipment were available within the Contract period.

Table 2 - Pros & cons for different bar options

Attribute	Standard Bar (50 mm)	High Strength Bar (36 mm)
Availability	Easily available in Hong Kong	Limited number of manufacturers
Weight	Very heavy and difficult to man-handle	Much easier to man-handle
Testing	Most laboratories in Hong Kong set-up to test	Difficult to test with modifications required to test rigs
Corrosion Protection	Standard zinc protection	Zinc protection not recommended – use other methods such as plastic sheaths
Quality Control	Variable depending on source	Very good quality
Lead-in Time	Short	Long – three months or more

At the time of designing the nails there was no real guidance in Hong Kong on what corrosion protection measures are applicable to soil nails for different ground conditions. From the test results it was considered that the steel needed full corrosion protection. Concerns regarding the potential for hydrogen embrittlement of high strength steel elements

which have been zinc coated, Arup (2003), meant that an alternative to standard zinc coating was required. Options such as heat shrunk plastic or epoxy coatings to the bars were considered, but finally a double corrosion system was adopted consisting of a HDPE corrugated full length sheath which surrounds the bar and is filled with grout *in lieu* of the zinc coating. Manufactures of the high strength steel provide such double corrosion systems which compliment their range of bar sizes and include couplers, spacers and end cap with integral grouting tube. The Main Contractor opted for the Dywidag-Systems International double corrosion protection system (Plate 1). This consists of a 100 mm diameter sheath with end cap for the 36 mm diameter high strength bar to be installed into a 150 mm diameter hole.

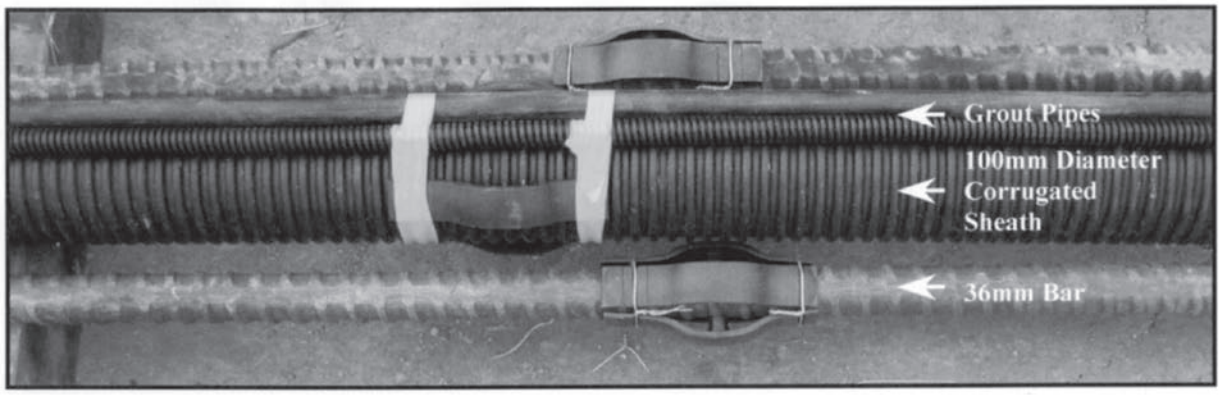

Plate 1 - Dywidag-Systems International soil nail double corrosion protection system

In order to address the constructability and grouting issues relevant to such long nails, eight preliminary soil nails up to 30 m long were installed at the site, load tested to two times the design load and exhumed for inspection. It is believed that at the time of writing that these soil nails are the longest installed with a double corrosion protection system in Hong Kong.

PRELIMINARY SOIL NAILS

Installation

The eight nails were installed at a location (Figure 2) which would later be within the bulk excavation for the cut slope formation so that there was; an opportunity to expose the nails after testing; inspect the grouting and the grout/soil contact; and take samples to be saw cut so that the full nail cross-section could be visually inspected. The preliminary nails also gave the Contractor an opportunity to test different installation sequences and systems. Preliminary nail details are summarised in Table 3. Initially the soil nail hole was designed to be 150 mm in diameter. However, due to difficulties in keeping the alignment of the hole straight for such long nails the sheath and steel bar could not be fully installed. The hole diameter was increased to 165 mm and installation become easy taking less than half and hour. Internal and external packers were used to isolate the test length and the nails were grouted using standard grouting equipment with the annulus between sheath and soil being grouted prior to the internal void. The sequence for the construction of the soil nails with double corrosion is at Figure 4.

Pull out Test Results

Each preliminary nail was tested to two times the design load using standard testing equipment. All the preliminary nails passed the pull-out tests, the results are at Table 3.

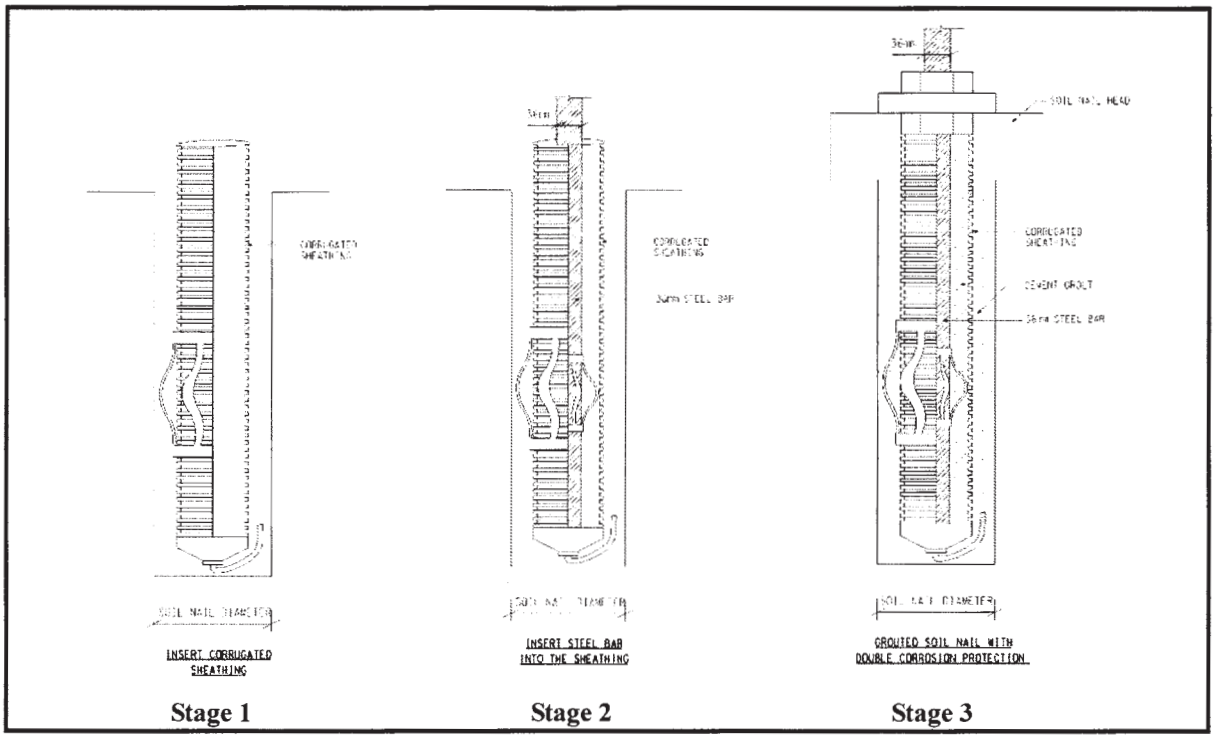

Figure 4 - Installation sequence of soil nail with double corrosion protection system

Table 3 - Preliminary nail details and results of pull-out tests

Soil Nail No.	Dywidag Bar Size (mm)	Nail Diameter (mm)	Bar Length (m)	Bond Length (m)	Effective *in situ* Vertical Stress (kN/m3)	Test Load (kN)	Max Shear – Soil/Grout (kN/m^2)	Measured Residual Movement (mm)	Measured Maximum Movement (mm)	Allowable Residual Movement (mm)
PT1	36	165	20	2.0	189.7	95	92	1.42	8.0	6.0
PT2	36	165	25	2.0	217.1	100	96	2.56	11.9	6.0
PT3	36	165	20	4.0	175.2	205	99	4.80	15.3	12.0
PT4	36	165	25	4.0	181.8	240	116	4.43	21.8	12.0
PT5	36	165	20	6.0	100.8	210	68	4.77	13.3	18.0
PT6	36	165	25	6.0	114.4	235	76	2.41	12.2	18.0
PT7	36	165	30	6.0	134.5	225	72	2.32	21.1	18.0
PT8	36	165	30	8.0	109.8	280	68	1.61	27.8	24.0

Nail Inspection

Once tested, three of the longest nails were carefully excavated (Plate 2) to allow close inspection of the nail and to facilitate sampling. Samples were also taken from the exposed grouted sections and saw cut (Plate 3). Several observations were made:

(1) The grout was continuous over the bond length (Plate 4) with no signs of voids apart from close to the packer.

(2) Some grout escaped past the external packer (up to 2 m) which generally formed a coating around the drillhole surface with a void between the grout and the sheath.

(3) Nail PT6 showed signs of crown collapse in the drillhole. The grout within the over-break contained inclusions of soil.

(4) The grouted diameter measured on nail PT6 ranged from 165 to 180 mm.

(5) The grout over the bonded length had excellent contact with the ground (Plate 4).

(6) The nails deviated from the design inclination of 10° to the horizontal by up to 10° for the 30 m long nail PT8.

(7) The saw cut sections revealed continuous sound grout in both the internal and external annuli (Plate 5).

(8) The corrugated sheath tended to float during the grouting of the outside annulus however the steel bar remained central (Plate 5).

The installation, testing and exhumation of the preliminary piles demonstrated that the corrugated sheath and steel bar could successfully be installed to up to 30 m and that the nails could be fully grouted to their distal end. This gave the designer and Contractor confidence to proceed with the permanent works installation.

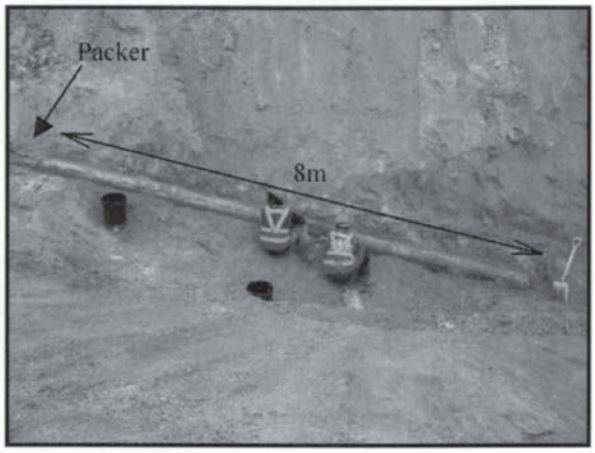

Plate 2 - Preliminary soil nail being exposed

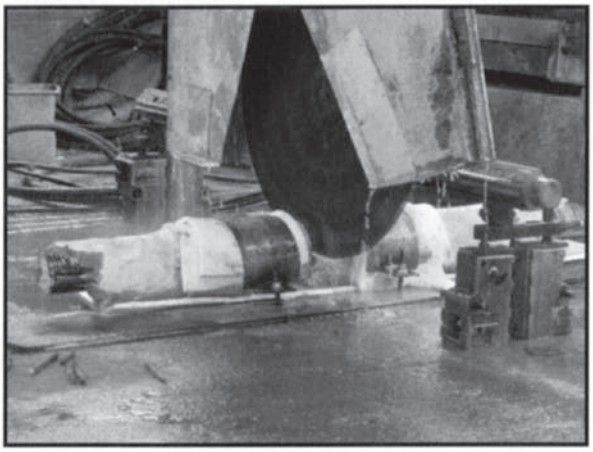

Plate 3 - Preliminary soil nail being saw cut

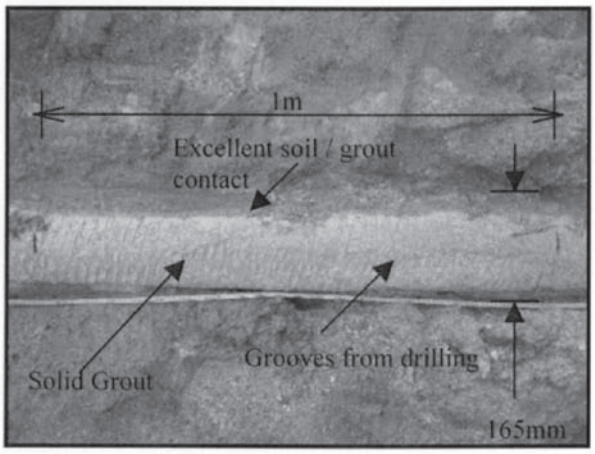

Plate 4 - Exposed preliminary soil nail

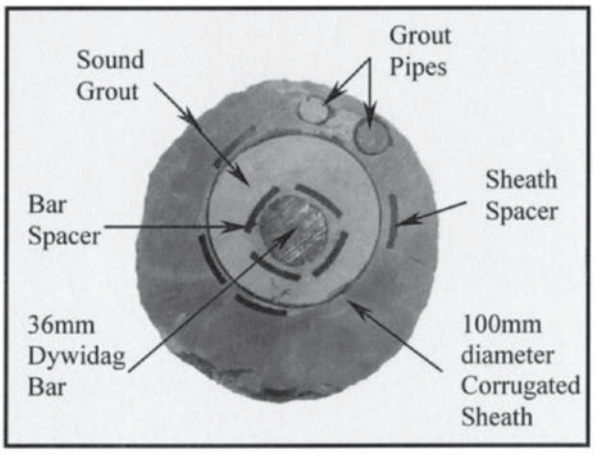

Plate 5 - Section through grouted preliminary soil nail

PERMANENT NAIL CONSTRUCTION

At the time of writing approximately 430 nails with double corrosion protection between 24 and 28 m long have been successfully installed at Deep Bay Link (Plate 6). Initial difficulties were experienced using standard Hong Kong soil nail grouting equipment. The Contractor was not able to fully grout these long nails and there was leakage from the outside annulus into the central annulus. The Contractor has modified his grouting procedure by using slightly larger diameter grout pipes (increased ID from 13 to 17mm), utilizing a pressure grout pump (HD Engineering Ltd GP60) and simultaneously grouting both annuli. On average each 28 m long nail takes about 3 hours to drill in saprolite, 15 to 20 minutes to install the corrugated sheath and steel bar and 15 minutes to grout.

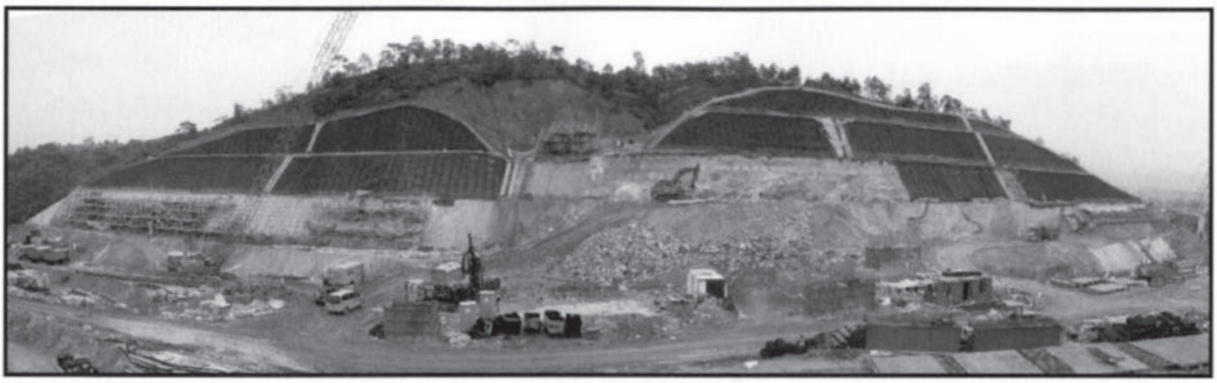

Plate 6 - General view of 40m high cut slope at northern end of Deep Bay Link during soil nail installation (March 2005)

DISCUSSION & RECOMMENDATIONS

The preliminary soil nail installation programme at Deep Bay Link demonstrates that nails up to 30 m long can be successfully installed and grouted using off-the-shelf double corrosion protection systems. As successful installation is very much contractor and ground condition dependent it is recommended that similar trials as these are made on projects where long nails are required. Zinc coating high strength steel is not recommended due to the possibility of hydrogen embrittlement. Grouting the annuli should be carried out simultaneously to avoid sheath floatation and possible grout leakage from the outer to inner annulus or *vice-versa*. For such long nails it is more efficient to use pressure grouting pumps with larger smooth bore grouting pipes than normal to ensure a continuous grouting process. Care should be taken when specifying nail diameters for long nails with double corrosion systems and designers should allow at least 30 to 40mm clearance between the sheath and the drillhole wall to allow for installation difficulties. The maximum drillhole deviation of 10° measured for the 30 m nail affects the horizontal nail capacity by less than 5% and is therefore considered not to be a concern bearing in mind the conservative design methodologies adopted in Hong Kong. Shiu and Cheung (2003) observation that many soils in Hong Kong are aggressive is supported and the authors recommend that the use of carbon fiber soil nail strands be further developed for Hong Kong conditions.

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ACKNOWLEDGEMENTS

The Authors wish to thank the Director of Highways Department of The Government of the Hong Kong SAR for permission to publish this paper.

HOLES ENCOUNTERED OF THE THIRD KIND

Keness K. L. Wong[1] and Terence C F Chan[2]

Abstract: Since mid-1980, soil nailing has been developed into a practical and cost effective means for stabilizing slopes in Hong Kong. However, installation of soil nails today is not free of problem, since they are installed below a mystery medium – ground, which may vary considerably within a small area. One of the common problems encountered in soil nailing works is the loss of grout through underground holes. Very often, underground holes are not easy to be identified by sinking limited number of boreholes at the site investigation stage of a project. The existence of these features may reduce the effectiveness of the soil nailing works and adversely affect the slope stability.

Why are there holes below ground? What actions can be taken if grout loss is encountered during soil nailing works? This paper records some cases identified with underground holes during implementation of slope upgrading works in Hong Kong, and explores the possible causes of formation of these holes. It also gives some case studies of the actions taken to overcome the problem of grout loss in soil nailing works.

INTRODUCTION

On 10 November 2002, a stray dog was found trapped in an underground hole (~1.5m × 3m × 6.5m deep) on the footpath at the crest of a soil-nailed cut slope overlooking Yee King Road, and was finally rescued by the Fire Services Department. This underground hole was exposed following the collapse of the footpath. Fortunately, this incident did not cause any casualty.

Detailed investigation of the incident revealed that the ground subsidence was due to underground erosion of the severely weathered and weak granite (Halcrow 2003) and some of the soil nails installed in the vicinity of the collapsed footpath had problem of grout loss. Underground stream systems and localized voids were found in the vicinity of the collapsed footpath. The presence of such underground stream systems and sizeable caverns is normally associated with karstic rocks, which are subject to dissolution, but is not very common in weathered profiles of relatively insoluble rocks such as igneous granitic and volcanic rocks.

In Hong Kong, various types of underground holes can be found in insoluble rocks. (Underground holes relating to karstic rocks are not discussed in this paper.) Some of them are formed naturally but some are formed by human activities. These holes may cause problem of grout loss and other construction difficulties in soil nailing works.

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The importance of grout loss in soil nailing works should not be underestimated. The grout of a soil nail serves the purpose of bonding the nail bar with the soil. A nail in a slope with improper grouting will lose its function partially or totally, and hence affect the slope stability. In addition, the leaked grout may block any subsurface drainage systems present and increase the risk of landslide due to pore water pressure building up during rainstorms. The grout may also fill up the nearby underground services and ducting. It is therefore advisable to conduct ample site investigation for detecting any potential grout loss problem before hand and make suitable allowance in the design and construction to avoid or mitigate the adverse effect on the slope works.

NATURAL UNDERGROUND HOLES

Erosion Pipes and Voids

The key process of formation of underground holes, such as gullies, pipes and localized voids, is erosion. According to Morgan (1995), soil erosion is a two-phase process consisting of the detachment of individual particles from the soil mass and their transport by erosion agents such as running water. Erosion can also cause the formation of underground stream e.g. the Yee King Road case recorded in Halcrow 2003.

Piping is a form of erosion, and is often associated with soils that tend to be dispersive and have a gap graded particle size distribution (Gray 1986). Piping is not an uncommon feature in Hong Kong; they exist in all the major soil types derived from insitu weathering (Figure 1 & 2) and transported soil mass (Figure 3). Eight cases of natural pipes/voids identified in Hong Kong between 1982 and 1986 were studied by Gray (1986). Among these cases, five of them were associated with pipes in weathered tuffs, two in weathered granite and one in bouldery colluvium. Natural soil pipes were also reported to be found at the backscarps of some recent landslide incidents such as the Lai Ping Road landslide in 1997 (Koor and Campbell 1998), Ching Cheung Road landslide in 1997 (Halcrow 1998), and natural terrain landslides on Lautau Island (Wong et al. 1996).

Figure 1. An underground pipe of ~0.5m dia. in moderately decomposed tuff (Jones 1986)

Figure 2. An underground soil pipe in saprolitic soil (Koor and Campbell 1998)

Figure 3. An underground void in bouldery colluvium (Jones 1986)

There are elements essential for the development of soil pipes: (a) there must be an outlet for drainage; (b) enough water must be available to fill the drainable cracks; and (c) sufficient hydraulic gradient must exist to move water and soil particles through a subsurface route (Jones 1981).

Holes Associated with Ground Movement

Slope movement or instability may create voids and gaps in the soil mass. They may appear on the surface as tension cracks near the slope crest, but in other instances, may be invisible underground. During implementation of the upgrading works for a man-made slope at Fan Kam Road, tension cracks were identified on the natural hillside above the slope (Figure 4). Ground investigation works including trial pits and geophysical survey were conducted subsequently, and found that there were also localized underground voids on a natural hillside (Figure 5).

Figure 4. Tension crests identified on hillside.

Figure 5. A localized underground void identified in a trial pit.

Other cases also suggest that in area of past ground movement such as faults, photo-geological lineament and landslide scars, underground holes and grout loss are more likely to be encountered (Ng 2004).

Other Natural Holes

Open joints in weathered rocks may develop into underground holes. According to Twidale (1982), weathering along the sheeting planes has in many places created gaping partings. At a few sites such weathering has proceeded to an extent that openings are sufficiently large to be called caves. Besides, animal burrows e.g. wormholes and root holes are also the causes of underground hole formation. They are usually found at shallow depth.

Holes may also be formed by a combination of the actions of erosion, ground movement, weathering and living organisms.

MAN-MADE UNDERGROUND HOLES

Underground tunnels, caverns and utility pipes are other form of underground holes. Tunnels and utility pipes installed in recent years are usually properly recorded, and their whereabouts can be traced based on the record of the undertakers. However, there are some unrecorded underground holes in Hong Kong such as disused tunnels and old caverns as well as abandoned utility pipes.

Disused Tunnels and Caverns

Extensive programmes to investigate and treat disused tunnels (Figure 6) have been undertaken by the Government. Up to 2004, 89 disused tunnel networks have been identified in Hong Kong, and the approximate locations of these tunnels are kept on drawings, which are available for public consultation in the Geotechnical Information Unit (GIU) of the Civil Engineering and Development Library (GEO 2004).

Figure 6. Entrance of a recorded disused tunnel in a slope at Kowloon Park.

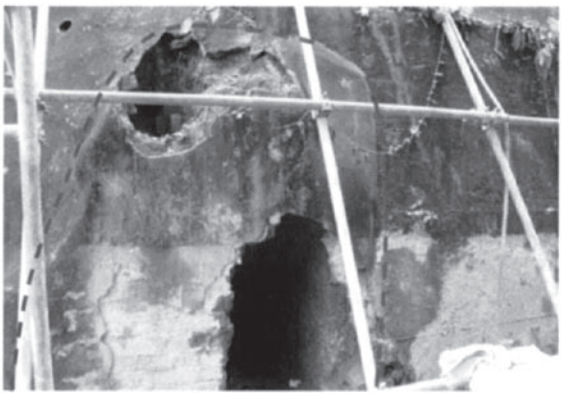

Figure 7. Entrance of an unrecorded cavern/ tunnel in a slope at Upper Albert Road.

However, discovery of unrecorded disused tunnels/caverns still happened from time to time in the past. During the soil nailing works on a cut slope feature along Upper Albert Road in late 2004 a disused tunnel was found (Figure7). This entrance to the tunnel was sealed by a brick wall and hidden behind sprayed concrete. No record of this tunnel could be located.

Two other cases of man-made underground holes were identified in slopes at the east of Cronin Garden and at Cape Collinson Road (Figures 8 and 9 respectively). The hole of the former case is located at ~6m below the surface of a slope to be upgraded, and was unveiled only during investigation of the causes of grout loss of the soil nails installed. CCTV survey and additional ground investigation (GI) works were conducted to reveal the hole condition. This hole was found to be formed in completely decomposed granite (CDG) of 2m × 3.5m × 1.8m high with brick walls on two sides (Figure 8). The hole at Cape Collinson Road is located close to the slope surface, and was discovered during drilling of soil nails. Grout loss was noted during installation of soil nails next to this hole and one of the soil nails actually hit this cavern. This cavern was exposed by enlarging the nail hole (Figure 9). This cavern was formed in moderately to highly decomposed tuff of 5.7m × 1.9m × 1.2m high, and appeared unlikely to be formed by erosion. Aerial photographs taken in 1963 indicated that there were a number of apparent adits in the vicinity of the slope. This cavern was therefore believed to be formed by human activities.

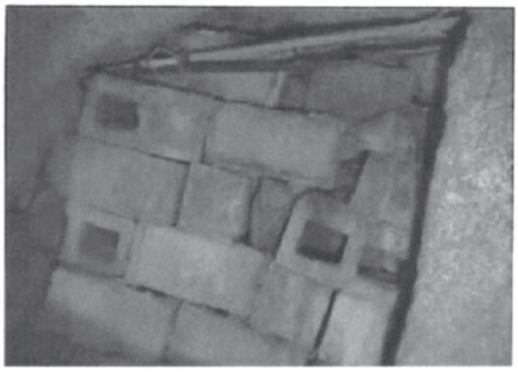

Figure 8. View of a sidewall of the underground hole

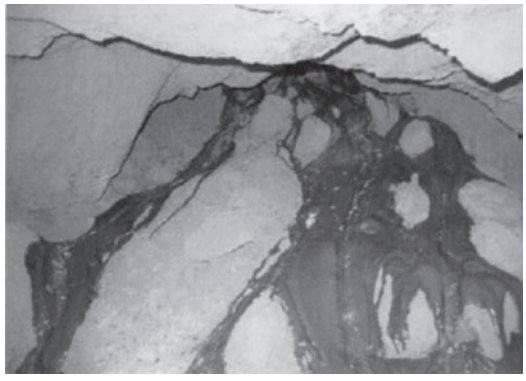

Figure 9. Grout observed inside the cavern.

Utility Pipes

Utility pipes will be another source leading to grout loss, if soil nails hit these pipes. To locate these services, referencing to utility plans is not entirely reliable. Trial pits and geophysics may be needed in some cases. This problem is particularly relevant in the case of abandoned utility pipes, which often do not appear on utility plans (Figure 10).

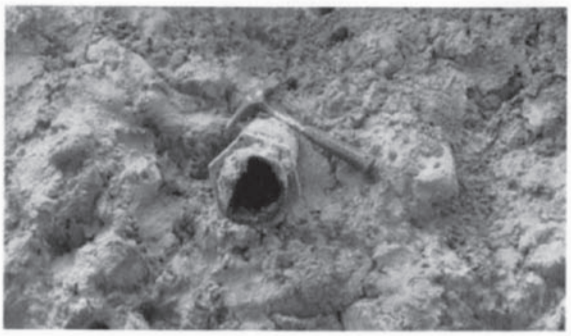

Figure 10. Abandon utility pipe on fill slope.

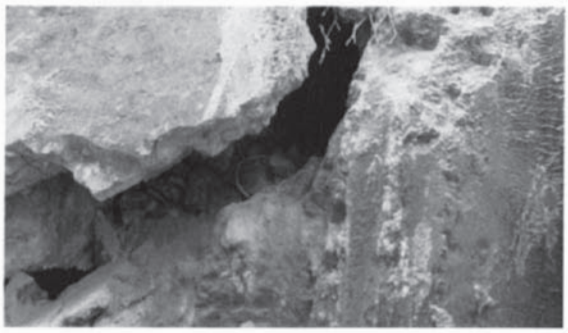

Figure 11. Void identified on fill slope.

Voids in Loose Fill and Masonry Retaining Walls

In recent years, soil nails have been used more often for upgrading old masonry walls and loose fill slopes. Experience shows that grout loss is a common problem encountered in soil nailing works on these two types of features. Since most of the old masonry walls are formed by dry packed methods albeit some with cement mortar bedding, voids exist between the masonry blocks. For many of the pre-1977 fill slopes formed by end-tipping without proper compaction, voids may exist and preferential drainage paths may be formed in the fill mass (Sun 1998).

CONTROL OF GROUT LOSS IN SOIL NAILING WORKS

As grout loss can adversely affect slope stability, the Geotechnical Engineering Office (GEO) has set up a control mechanism in June 2004 mainly for Landslip Preventive Measures (LPM) projects. In case grout loss (which is currently defined as a situation where one or more nails cannot be fully grouted after injecting a volume of grout equal to 5 times the calculated gross volume of the drillhole) is identified in a soil nailing site, the site staff are required to inform the Engineer's Representative of the Contract for onward request for advice from the designer. The designer would then review the design, identify the possible causes of excessive grout take and recommend rectification/remedial measures, if necessary. To ensure proper documentation of the cases, a database was set up in GEO. In 2004, 250 slopes were upgraded under the Government's LPM Programme; about 75% of these slopes were stabilized using soil nails and 16 cases were reported with the problem of grout loss. Two of these cases are discussed below.

CASE STUDIES

Case 1 - Slope at Plantation Road, the Peak

The slope is 5m high, 45m long and with an average angle of 60°. It is a cut slope covered with sprayed concrete. The slope is mainly composed of completely to highly decomposed tuff. A total of 145 soil nails were proposed for upgrading the slope. However, 37 (i.e. 25% of the total number) of the nails were reported to have grout loss problem during the soil nailing works. Grout was found leaked at various depths along the soil nails. To investigate the causes of grout loss, the designer reviewed the Aerial Photographs Interpretation (API) study and the GI data, conducted CCTV survey in the soil nail holes, and stripped and mapped openings on the sprayed concrete cover.

The designer noted from the API study that the nail having the most significant problem of grout loss coincided with a landslide scar. Through CCTV survey and mapping of the slope surface, the designer found that the slope contained highly fractured rock. Leakage of

grout was believed to occur through the fractured rock in saprolite. In order to prevent further grout loss from occurring in the remaining nails, the designer pre-set the sequence of soil nailing works so as to determine the extent of the problem area at the earliest time and adopted a combination of the following preventive/ remedial measures:

- Using grout with lower water to cement ratio (e.g. 0.38) for grouting the nail holes
- Installing casing for the initial length of the nails to isolate the nail holes from the voids

The designer also reviewed the soil nail design. As grout loss occurred only at the active zone of the nails, only minor changes on the soil nail heads were required to cater for the reduction in frictional resistance at the active zone.

Case 2 - Slope at St. Lawrence Catholic Church and School

The slope is 34m high, 60m long and at an angle of 28°. It was a fill slope covered with dense vegetation. In the original design, the site was assumed to comprise a layer of fill overlying CDG. About 770 soil nails were proposed to upgrade the slope. 75 (i.e. ~10% of the total number) of the nails were reported with grout loss problem at the early stage of soil nailing works. Leakage of grout was found at a depth of 7-10m measured along the nails. To avoid injecting excessive amount of grout into the slope, the designer required the contractor to stop the grouting operation if the hole cannot be filled by a volume of grout equal to 5 times of its theoretical volume during normal grouting operation.

The causes of grout loss were investigated through review of the API study, CCTV survey of the soil nail holes, and additional vertical drillholes in the problem area. The investigation revealed that there is a layer of bouldery colluvium between fill and CDG at the center portion of the slope with on plan area of 25m × 35m. Leakage of grout was believed to have predominantly occurred at the bouldery colluvium, which contains voids, and the loose soil fill especially near the interface between the bouldery colluvium and the overlying fill layer. In order to determine the extent of the problem area, it was agreed with the contractor to carry out soil nailing works starting from the perimeter of the slope and then working towards the problem area. In addition, different remedial measures were tried on site:

- Staged grouting – Grouting was carried out intermittently in 15-30 minutes interval,
- Pre-grouting – The nail hole was filled by quick-set silicate grout first and then redrilled for installation of the nail, and
- PVC/ steel casing – PVC casing was inserted into the hole manually after completion of the drilling of the hole. Alternatively, permanent steel casing was installed by Odex machine. PVC casing was used to prevent grout loss through the underground holes within 9 to 10m from the slope surface. Steel casing was inserted to a greater depth preventing hole collapse in the drilling process and grout loss at the same time.

Staged grouting was found unsuccessful to prevent grout loss, and pre-grouting was only successful in areas with small voids. Although the use of permanent steel casing was relatively expensive, it was more reliable to tackle the problem of grout loss and hole collapse. CCTV survey of soil nail holes was found very useful in determining the location and size of the voids, as well as the remedial measures to be used at different areas.

To incorporate the newly identified bouldery colluvium layer, the designer also refined the geological model and redesigned the soil nails. Soil nails near the center portion of the slope, where the problem of grout loss occurred, were increased in the numbers and lengthened to pass through the boundary colluvium and bond in the competent CDG stratum.

GROUND CONDITIONS AT THE GROUT LOSS LOCATION

The 16 reported cases indicate that grout loss were more likely occurred in ground with the following features: (1) bouldery colluvium, (2) fill which is loose or contains a significant proportion of boulders and cobbles, (3) landslide scar, (4) highly fractured rock, (5) masonry wall with joints/voids between masonry blocks, and (6) soil with degenerated root system of dead tree.

Apart from the above, cases recorded in Ng et al. (2004) indicate that sites with (7) old streamcourse/existing drainage line and (8) faults also experience grout loss in soil nailing works.

The presence of the above features is not the only and necessary condition for grout loss to occur during soil nailing works. However, under these situations, designers are recommended to conduct adequate ground investigation to identify the presence of holes in the ground and assess the likelihood of grout loss. They should assess the buildability of soil nails, incorporate in the design measures to prevent grout loss if necessary, and add provisions in the contract documents to record and control the quantity of grout injected into ground.

MEASURES TO PREVENT GROUT LOSS

Investigation

A simple way to detect underground holes is to monitor the drilling process of soil nailing works. Unreasonably high penetration rate of drilling and/or lack of air return in the drilling process may infer the presence of underground holes. With the aid of other investigation techniques such as CCTV survey, GI works and geophysical survey, designers can determine the nature, size, extent and location of the underground holes. CCTV survey is useful for viewing and recording the interior condition of the nail holes or drillhole. GI works such as trial pits can expose those underground holes at shallow depth. Sinking of drillholes can be used for detecting underground holes at greater depth, and collecting samples for detailed logging. Geophysical survey including resistivity and gravity survey has also been used successfully in determining the approximate location of underground voids (Halcrow 2003).

Preventive/ Remedial Measures

With reference to the 16 cases reported to GEO in 2004, and those recorded in Ng (2004), the following measures have been found useful and practicable to tackle the grout loss problem.

(1) Use of grout with lower water to cement ratio for filling the nail holes (e.g. 0.38 to 0.4)
(2) Staged grouting – Grouting is carried out intermittently to allow time for grout set.
(3) Repeated drilling and grouting – The nail hole is grouted and then re-drilled for installation of steel bar.
(4) Pre-grouting the area adjacent to soil nail holes
(5) Use of chemical grout or cement grout with expanding admixture – Quick-set admixtures can reduce the set time of grout to 20-30 seconds.
(6) Use of PVC or steel permanent casing.

In parallel with the above measures, designers should monitor closely the quantity of grout injected into the slope and assess the effects of leaked grout on the hydrogeology of the nearby area. Installation of prescribed raking drains into the slope subsequent to the soil nailing works may help prevent the rise in groundwater level due to damming up of the underground drainage feature.

If measure (6) is used, designers should assess the possible effect on the soil nail and nail head design. Sleeving the front part of the nail will significantly reduce the load transfer to the soil in the active zone, and increase the loading transferred to the nail head. The permanent casing, depending on how far it is extended into the nail, may also reduce the passive design bond length of the nails and therefore reduce the pullout resistance.

CONCLUSIONS

Underground holes are not uncommon in Hong Kong and can lead to the problem of grout loss in soil nailing works. Past experience indicated that grout loss and its effect can be prevented or mitigated successfully by making suitable allowance in the design and construction. The presence of these underground holes should be identified as early during the project as possible, preferably at design stage.

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ACKNOWLEDGEMENTS

The authors wish to thank Maunsell Geotechnical Services Ltd. and Ir. Chris Tang for providing information about the two LPM case studies. This paper is published with the permission of the Head of the Geotechnical Engineering Office and the Director of Civil Engineering and Development, the Government of the Hong Kong Special Administrative Region.

ANALYSIS FOR BEHAVIOUR OF NAILED STRUCTURES

Y. K. Shiu[1] and Garry W. K. Chang[1]

Abstract: The effects of soil nail inclination and nail length pattern on the behaviour of nailed slopes and excavations are analysed using finite difference code with strength reduction technique. The results are presented and discussed in this paper. Tensile or compressive axial forces can be developed in soil nails and this depends on the nail inclination. Tensile nail forces can improve stability whereas compressive forces can have opposite effect. Lateral deformations of nailed excavations are influenced by nail inclination as well as nail length patterns.

INTRODUCTION

Soil nails can be installed at various inclinations but different inclinations will produce different effects on the behaviour of nailed structures. Such effects are not readily analysed with limit equilibrium methods. In this paper, nail inclination, α, is the angle of a soil nail made with the horizontal; and nail orientation, θ, is the angle between a soil nail and the normal to the shearing surface of soil. Figure 1 illustrates the typical relationship between α and θ for a nailed slope.

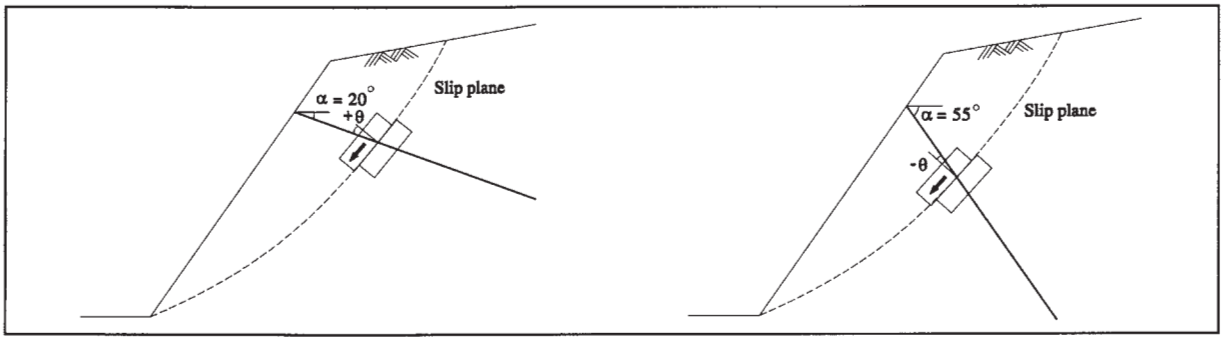

Figure 1. Relationship between Nail Inclination α and Nail Orientation θ

A number of experimental studies on the fundamental behaviour of reinforced soil mass were carried out in the past (e.g. Jewell 1980, Jewell and Wroth 1987; Marchal 1986; Hayashi et al 1988; Palmeira and Milligan 1989) and all the studies indicated the importance of the orientation of the reinforcements on the soil shear strength. In this study, numerical simulations have been conducted using the finite difference code, FLAC (Fast Lagrangian Analysis of Continua) to examine the effect of nail inclination on the strength and deformation of nailed slopes and excavations. In addition, the influence of different nail length patterns on deformation of nailed excavations has also been examined by means of the numerical simulations.

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EFFECT OF SOIL NAIL INCLINATION ON SLOPE STABILITY

Numerical Simulation

A simulated slope 20 m high with an upslope of 10° in gradient was adopted for the analysis. Figure 2 shows the geometry of the slope and the material parameters used in the numerical analysis. For the slope without soil nails, FLAC analysis gave a factor of safety (FoS) of 1.0, which compared well with a value of 1.1 using the Morgenstern & Price method. A Mohr Coulomb model was used for the soil and a cable element for the soil nail. The nail heads were modelled as elastic and infinitely long concrete beams of 400 mm in width and 250 mm in thickness.

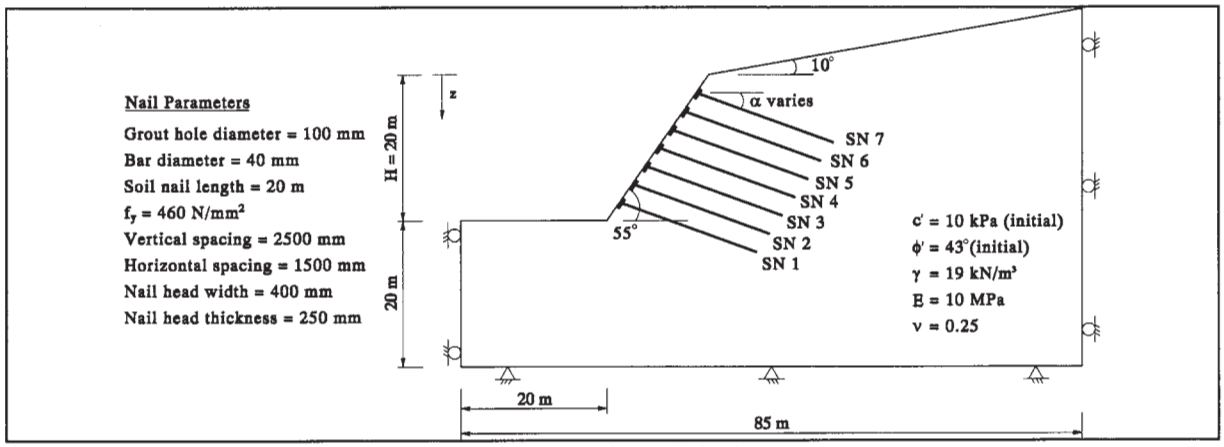

Figure 2. Geometry and Material Parameters of Model Slope

The effect of inclination of soil nails on stability of the slope model was examined by using the strength reduction approach. In this approach, FoS can be determined by reducing the shear strength of the soil until failure occurs. The nail inclinations α were varied between 0° and 65°. At a given nail inclination, the FoS of the nailed slope was calculated. Knowing that the FoS before provision of soil nails was 1, the increase in FoS (ΔFoS) due to the soil nails at that inclination angle could then be determined.

Results of Numerical Simulations

Figures 3(a) & 3(b) compare the distributions of axial nail forces between two nail inclinations of 20° and 55° at limit equilibrium condition. The maximum axial force at each nail is T_{max}. It is taken as positive for tension and negative for compression. For α = 20°, tensile forces are mobilized in all the soil nails; and the nail orientations (θ) are all positive. For α = 55°, compression forces are induced in the upper four rows of the nails whereas tension forces are developed in the lower three rows. The nail orientations are negative for the nails with compression forces and positive for nails with tensile forces. Figure 4 shows the variations of the maximum nail forces for α = 0° to 55° with relative to depth.

The magnitudes of the tensile forces in the upper nails decrease when the nail orientations (θ) change from positive to negative (Figure 1). At a certain range of negative θ, the nail forces change from tension to compression. This agrees with the test results reported by Jewell (1980) and Marchal (1986).

Increases in FoS (ΔFoS) due to the soil nails were calculated for different nail inclinations α. Figure 5(a) shows the relationship between the calculated ΔFoS and α for the model slope. The ΔFoS is close to 1.0 with little variations for the range of α between 0° and 20°. The ΔFoS decreases substantially as α increases beyond 20°, reflecting that the reinforcing action of the nails reduces rapidly with increasing nail inclinations. When α

equals 65°, the ΔFoS is close to 0. This indicates that soil nails at such large inclinations do not provide any appreciable reinforcing effect.

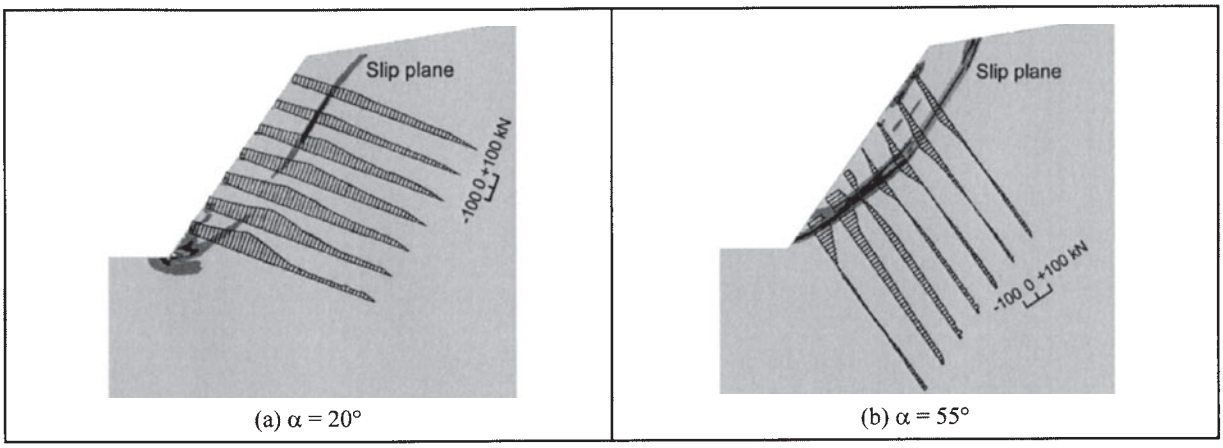

Figure 3. Axial Force Distribution in Nails for (a) α = 20° and (b) α = 55°

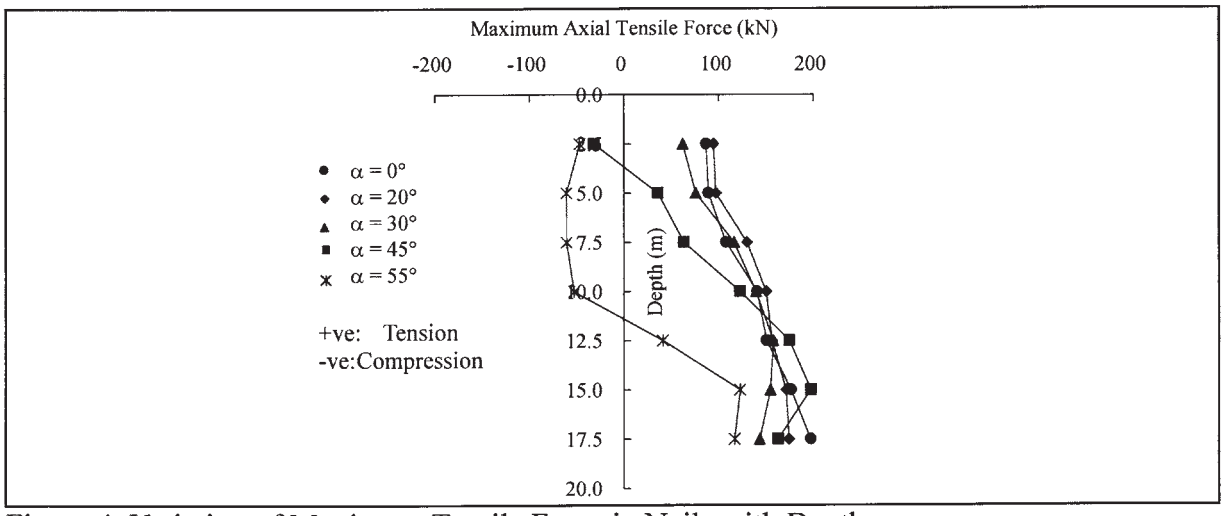

Figure 4. Variation of Maximum Tensile Force in Nails with Depth

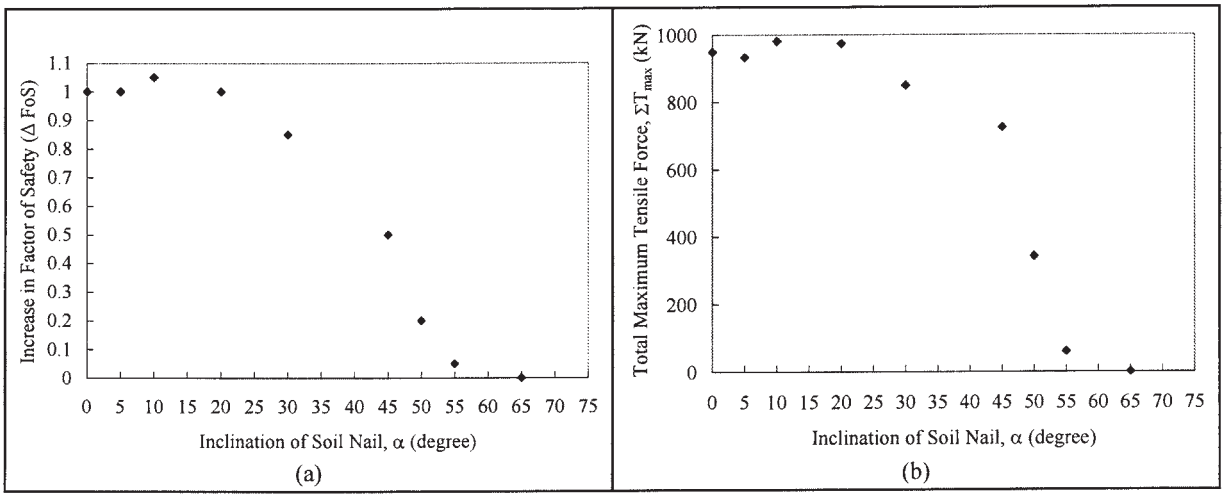

Figure 5. Variation of (a) ΔFoS with α and (b) Total Maximum Tensile Force ΣT_{max} with α

For a given α, the maximum nail forces (T_{max}) can be added together to produce a single

force ΣT_{max}. The variation of ΣT_{max} with α are plotted in Figure 5(b). It can be observed from the figure that ΣT_{max} decreases with increasing α. ΔFoS tends to be zero when ΣT_{max} is close to zero. The similarity between Figure 5(a) and Figure 5(b) indicates that ΔFoS is related to ΣT_{max}.

Discussion of Results

Conventional limit equilibrium approach using the method of slices is routinely adopted for soil nailing design. In these methods, required stabilizing forces are computed for different potential failure surfaces. The lengths of the nails projecting beyond the potential failure surfaces are considered to contribute to the stabilising forces. These nail forces are assumed to be tensile forces. This assumption is valid only if the nails are placed at small inclinations, i.e. close to the direction of principal tensile strain increment in soil (Jewell, 1980). It may be invalid for steeply inclined nails. As demonstrated from the simulations above, compressive forces rather than tensile forces can be induced in steeply inclined nails. In such cases, the limit equilibrium approach would over-estimate the reinforcing effect of the nails and the overall factor of safety of the slope. An illustration case is presented below.

ILLUSTRATION

The stability of a cut slope at Sham Shui Po was investigated in Year 2000 under the Landslip Preventive Measures (LPM) Program. The stability results showed that the slope portion under consideration had a minimum FoS of 0.72 which is below the required value of 1.2. Soil nailing was recommended to improve the stability of the slope. The soil nailing scheme called for 4 rows of 9m long soil nails with vertical and horizontal spacings of the nails at 1.5m and 2 m respectively. The nail comprised a 25mm diameter steel bar grouted in a 100mm diameter hole. The design was based on the conventional limit equilibrium approach using the Morgenstern and Price method. The required stabilizing force was calculated to be 60 kN/m and it was assumed to be distributed evenly between the nails. The inclination of the soil nails was 10° and the size of nail head was 400 mm x 400 mm x 250mm thick. Typical details of the recommended soil nailing works are shown in Figure 6(a).

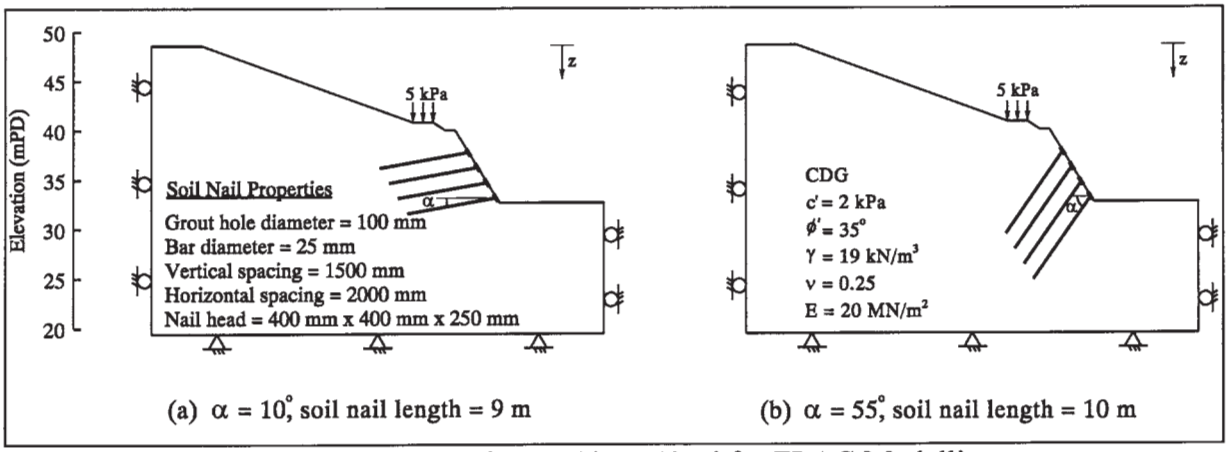

Figure 6. Simplified Slope Profiles of LPM Slope Used for FLAC Modelling

FLAC analyses have been carried out to recalculate the factors of safety of the slope before and after installation of soil nails. For the slope before nail installation, the FoS was found to be 0.72, the same as that determined from the Morgenstern and Price method. For the slope soil installed with nails, the calculated factor of safety was 1.6, which is higher than the designed FoS of 1.2. It is considered reasonable because the total nail length provided is

actually longer than that required for the purpose of maintaining constant nail length at different rows. Figure 7(a) shows the distributions of axial forces along the soil nails and the location of shear surface in the soil at limit equilibrium condition. Tension forces are developed in all the soil nails.

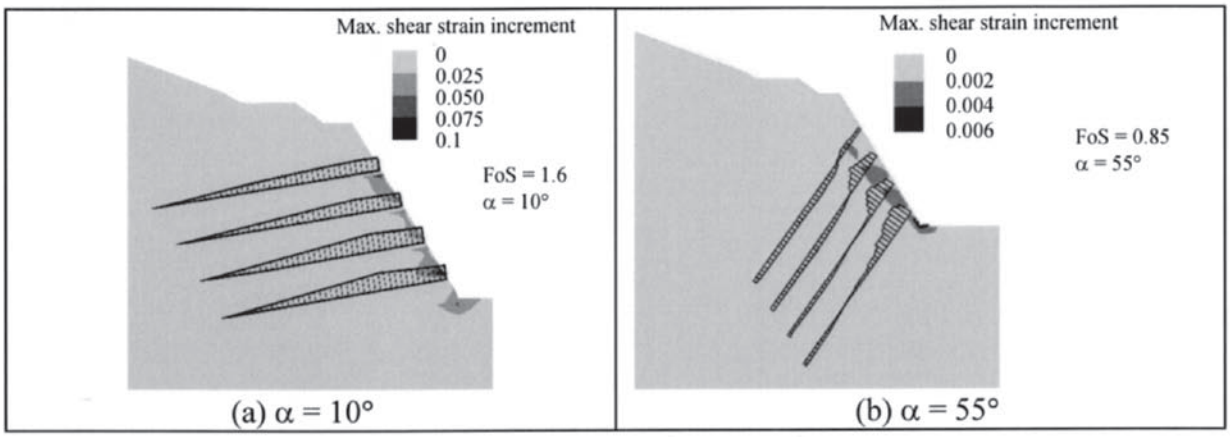

Figure 7. Axial Nail Forces and Soil Shear Strains for LPM Slope

In order to examine the effect of nail inclination, the inclination angle α was increased from 10° to 55° whilst the number and spacing of the nails were kept unchanged. The length of the soil nails was determined using the conventional limit equilibrium design approach in which tension forces were assumed to be developed in soil nails. For a minimum FoS of 1.2, the total nail force required for the hypothetical case of α of 55° is 156 kN/m. Using this total nail force, the required nail length is calculated to be 10m, 1 m longer than that required when α = 10°. Figure 6(b) shows the nail pattern.

Using the same nail pattern determined from the limit equilibrium approach (Figure 6(b)), the FLAC analysis gives a minimum factor of safety of 0.85 which is well below 1.2. Figure 7(b) shows the distributions of the axial forces developed in the nails. It is important to note that compression force is developed in the top row of nails whereas tension forces are mobilized in the lower three rows of nails. The variations of the maximum nail forces (α = 10° and α = 55°) with relative to depth are plotted in Figure 8. The total of all the maximum nail forces (ΣT_{max}) is about 170 kN/m for α = 10° and 27 kN/m for α = 55°. The results show that increased nail inclination results in substantial decrease of nail tension forces, leading to significant decrease of the reinforcing effect of the nails. This finding agrees with the analytical results of the simulated slope discussed above.

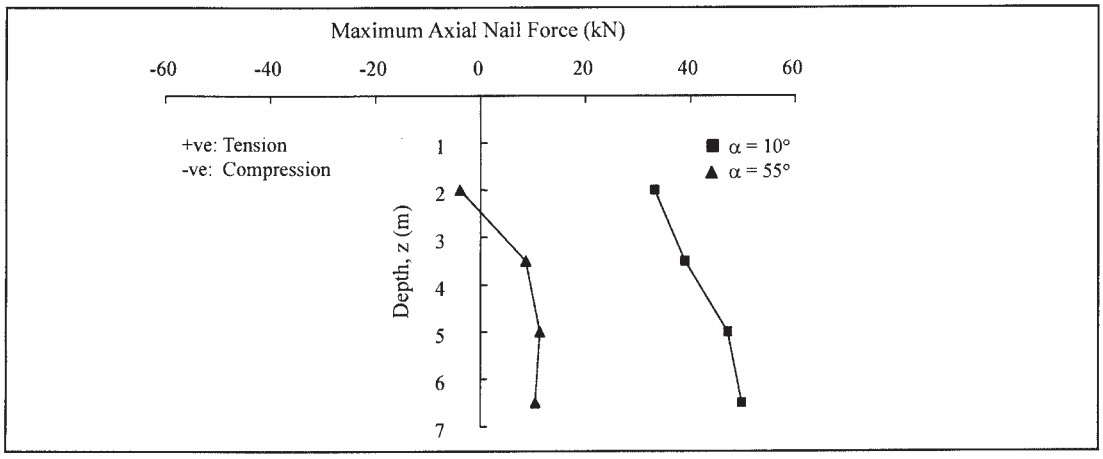

Figure 8. Variations of Maximum Axial Nail Forces with Depth

EFFECT OF NAIL INCLINATION ON DEFORMATION AND STABILITY OF NAILED EXCAVATIONS

To acquire a better understanding of the influence of nail inclination, numerical simulation using FLAC has also been conducted on nailed excavations (Figure 9). The vertical excavation was taken to be 6 m in height. Four rows of nails were assumed to be 10 m in length, with a 40 mm diameter bar in a 100 mm diameter grouted hole. Both the vertical and horizontal spacings were taken as 1.5 m. The soil parameters were c' = 10 kPa, ϕ' = 40°, unit weight = 19 kN/m^3 and elastic modulus = 20 MPa. The shotcrete facing used was 100 mm in thickness and was assumed to be connected to the soil nails.

The construction sequence of the nailed excavation was modelled in the analysis, see Figure 9. Construction progressed incrementally in a top down manner by repeating two steps of construction. The first step began with soil being excavated to a depth of 0.5m below the level of soil nail. Step 2 consisted of installing the soil nail and concrete facing. Steps 1 and 2 were repeated until the full excavation depth (6m) was attained. The effects of nail inclinations were studied by varying the nail inclinations between 0° and 35°. For α=35°, the excavation collapsed when the excavation reached a depth of 5.5m, before the bottom nail was installed. For α less than 35°, the excavations were stable throughout the whole construction process.

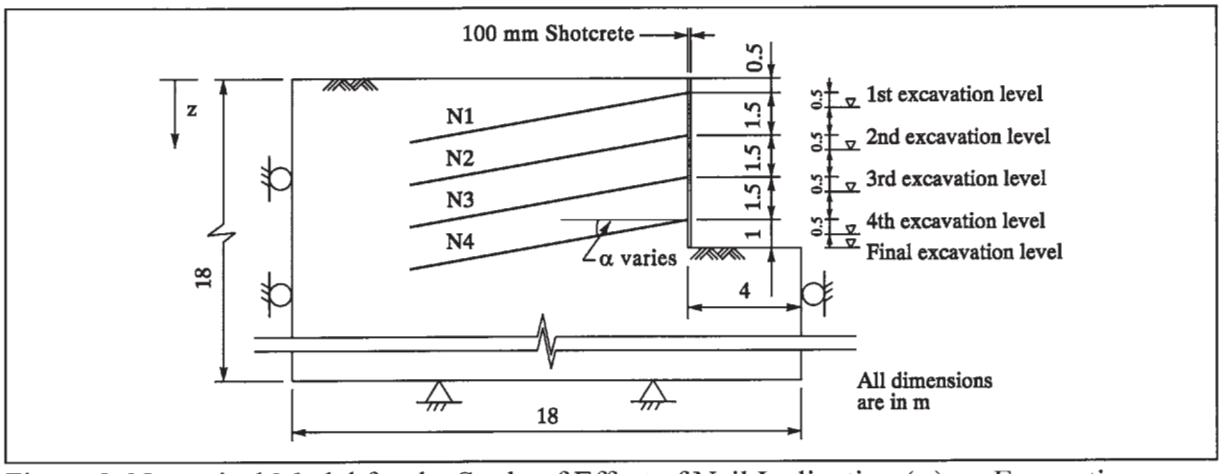

Figure 9. Numerical Model for the Study of Effect of Nail Inclination (α) on Excavation

Axial Forces Mobilised in the Soil Nails

Figure 10 shows the variations in maximum axial nail forces (T_{max}) with excavation depth (z) for different α values. The maximum forces mobilized in the nails are not constant and they vary with depth. Relatively large forces are mobilised in the first nail. This may probably be due to the cantilever effect before the second row of nails is placed. The forces increase from row 2 to row 3 of the nail, below which they start to decrease with depth. This variation in maximum nail forces is consistent with the field monitoring results observed by Plumelle et al (1990) and Shiu et al (1997). Except for row 1, the maximum tensile forces mobilised in individual nails generally decrease with increasing nail inclinations, indicating that the reinforcing action of the nails reduces when nail inclination increases.

Horizontal Deformation of the Excavation Face

Figure 11 shows the profiles of horizontal deformation of the excavation face as a function of nail inclinations. The deformation profiles are similar to those observed by Babu et al (2002). The horizontal deformations increase with increasing nail inclinations, and

there is a sharp increase in deformations when α increases from 25° to 30°. The analytical results show that the magnitudes of horizontal deformations are influenced by the nail inclination.

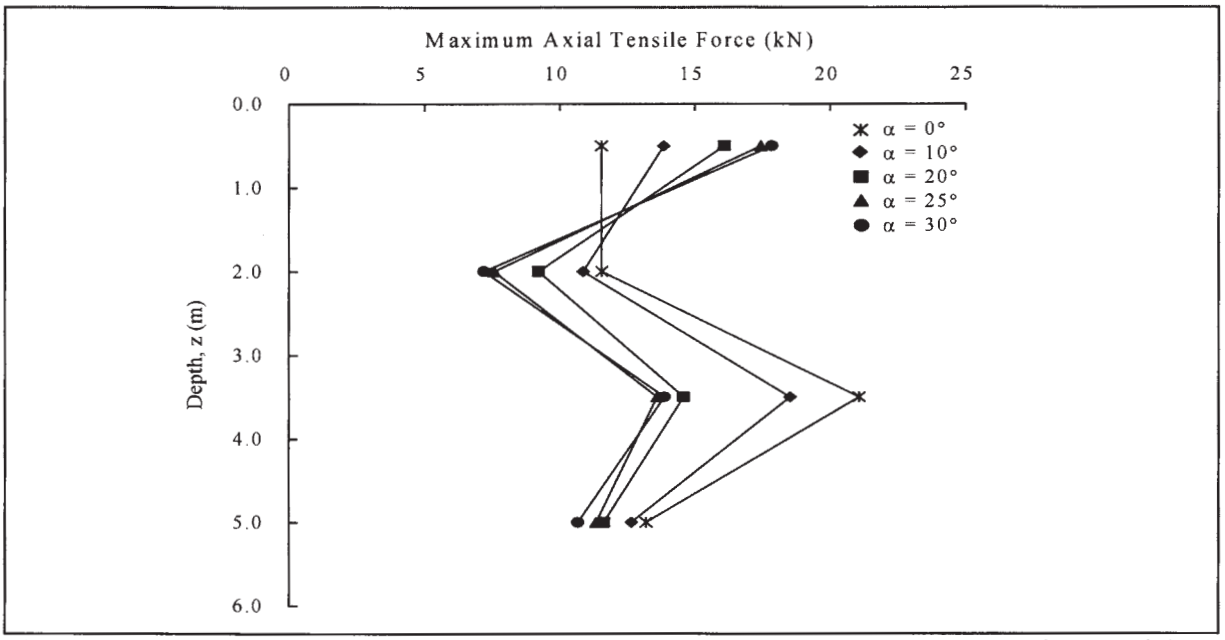

Figure 10. Variations of Maximum Axial Nail Forces with Depth for the Excavation Models

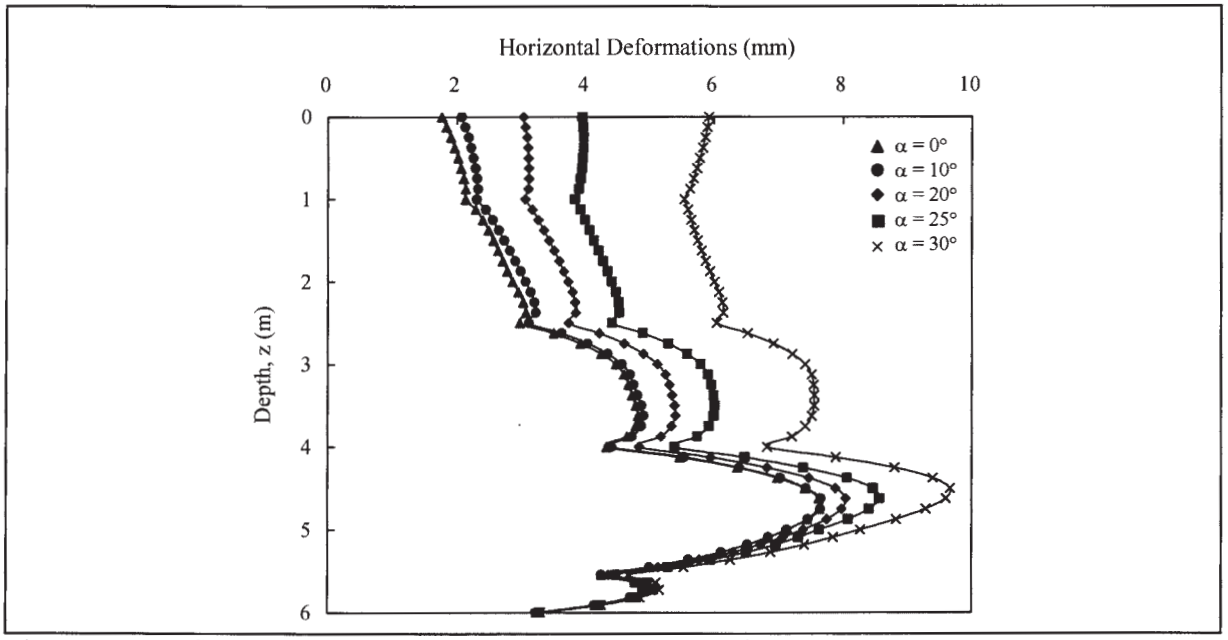

Figure 11. Profile of Horizontal Deformations of Excavation Face

EFFECT OF NAIL LENGTH PATTERN ON DEFORMATION

For limit equilibrium design methods, it is possible to define a wide variety of nail length patterns that satisfy stability requirements but that may not be satisfactory in terms of serviceability. It is especially the case for soil nailed excavations using the top-down method of construction.

FLAC analyses have been carried out to review the effect of different nail length patterns on deformation of excavations. The excavation is taken to be 6 m deep with four rows of

soil nails. The following three nail length patterns have been considered in the analysis: (1) nail length increasing with depth (Figure 12(a)); (2) constant nail length (Figure 12(b)); and (3) nail length decreasing with depth (Figure 12(c)). In each of the three patterns, the total length of soil nails is 32 m. The reinforcement is 40mm diameter steel bar grouted in 100mm hole. The nail inclination is 10°. Both the vertical spacing and the horizontal spacing are taken as 1.5 m. The soil strength parameters used are c'=5 kPa and ϕ'= 39°, with a unit weight of 19 kN/m^3 and an elastic modulus of 10 MPa. The shotcrete facing used is 100mm in thickness and is connected to the soil nails. The excavation sequence for the three cases is the same as that shown in Figure 9.

The FoS of the patterns (1), (2) and (3) after completion of construction are 1.8, 1.7 and 1.5 respectively. The profiles of horizontal deformations of the excavation faces obtained from the FLAC analysis for the three nail length patterns are plotted in Figure 13. Although nail length pattern (1) has the highest FoS, its horizontal deformations are the largest and this is followed by pattern (2). Nail length pattern (3), despite its lowest FoS, has the smallest deformation. The result shows that soil nails placed in the upper part of a nailed structure in a staged-excavation contributes more towards reducing the horizontal displacement. Those installed in the lower part are more effective in improving stability of the structure. This is because of their contribution in relation to the potential failure surfaces and their larger anchorage length beyond the potential failure surfaces.

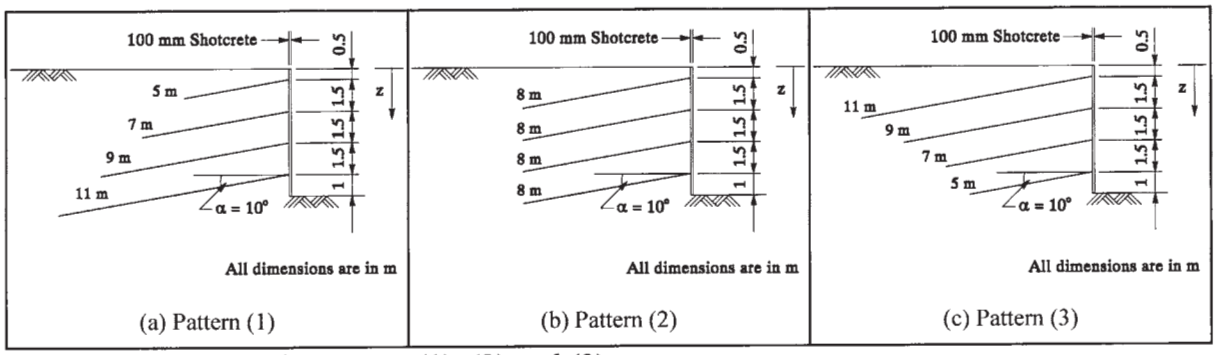

Figure 12. Nail Length Patterns (1), (2) and (3)

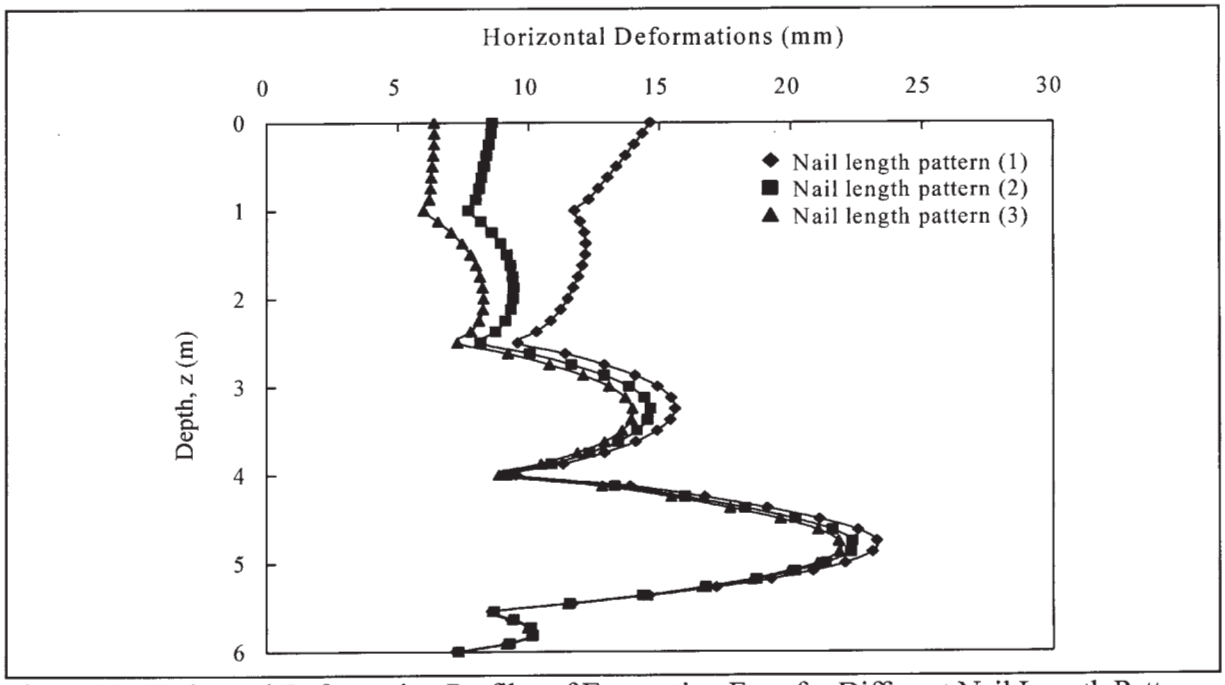

Figure 13. Horizontal Deformation Profiles of Excavation Face for Different Nail Length Patterns

CONCLUSION

The reinforcing action of soil nails decreases with increasing nail inclinations. For steeply inclined nails, compressive forces rather than tensile forces would be mobilized in the nails. The compressive forces can have adverse effect on the stability of nailed structures. Such effects cannot be readily analysed by means of limit equilibrium methods as these methods do not consider strains and displacements. Care is therefore required when using the limit equilibrium methods for the design of steeply inclined soil nails.

For nailed excavations, nail inclination and nail length patterns can affect the magnitude of horizontal displacements of the excavation. Nails in the upper part of the excavation are more effective than nails in the lower part of the excavation in controlling the deformations.

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ACKNOWLEDGEMENTS

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SOLAR POWER AUTOMATIC IRRIGATION SYSTEM

Becky L S Lui[1] and Y K Shiu[2]

Abstract: Planting of vegetation on man-made slopes can add greenery to streetscape and enhance the local ecology. Watering needs to be carried out in dry season in order to assist the establishment of the vegetation in the initial stage of planting and keep vegetated slopes green all the year round. A study comprising a desk study and a field trial has been conducted to explore the feasibility of using renewable solar energy for extracting water from underground or stream courses for irrigating vegetation on man-made slopes. In the field trial conducted at a slope on Lantau Island, an automatic irrigation system is installed and photovoltaic panels are used for providing the energy necessary for operation of the system. Water is pumped out from a natural stream course nearby, avoiding the need for tap water which is a precious resource. The performance of the system has been monitored since the installation in July 2003. This paper describes the design, installation and performance of the solar power automatic irrigation system. It also provides guidelines on the design and applications of the system.

INTRODUCTION

The Government of the Hong Kong Special Administrative Region has taken many initiatives in recent years to improve the appearance of man-made slopes and blend them with their surroundings. Among the various landscaping techniques, vegetating slope surface is the most commonly-used method. Planting of vegetation on slope surface does not only add greenery to streetscape and enliven the whole environment, it can also enhance the local ecology. From the slope engineering perspective, vegetation can help prevent soil erosion. The long-term stability of soil slopes relies on the establishment and healthy growth of vegetation.

Without adequate watering, it may be difficult to establish and maintain vegetation on most man-made slopes. Grass and shrubs turn brown or even wither during dry seasons. Manual watering of vegetation can be costly, labour intensive and often not practical for high slopes or remote areas where water is not readily available. In order to assist the establishment of the vegetation in the initial stage of planting and maintain healthy growth of vegetation during dry seasons, a study comprising a desk study and a field trial has been conducted. The objective of the study is to explore the feasibility of using renewable solar energy for extracting water from underground or stream courses for irrigating vegetation on man-made slopes.

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DESK STUDY

A feasibility study on the use of solar power irrigation system for watering vegetation on man-made slopes was undertaken in early 2002 (Leung, 2002). The study reviewed the practicability of utilizing solar energy and examined ways to extract groundwater. The findings of the study are summarized below.

Solar energy is rich in Hong Kong because of the geographical location. The mean total solar radiation on a horizontal surface is 1290 kWh/m^2. The potential of solar energy applications is therefore great. Local meteorological data indicate that the annual average daily sunshine is about 5.3 hours with a range of about 3 hours in March to 7.3 hours in July.

Solar energy is captured using photovoltaic (PV) technology. The PV technology uses the energy in sunlight to excite electrons in light sensitive materials to generate electricity. There are a number of factors which may affect the power generating capacity of photovoltaic cells other than daily and seasonal sunshine condition. For examples, air mass humidity and pollution, temperature and wind conditions, etc., may affect the effectiveness of a PV system. The PV cell should preferably be placed in a southerly direction and should not be in the shade of tall buildings and trees.

The PV technology is well developed. Solar power applications are common in many countries such as the USA and Japan. Most of the commercially available PV cells have efficiency slightly above 10% and their maximum output under ideal conditions is generally in the order of about 120 W/m^2.

Groundwater or water from stream courses may be used for irrigation. If groundwater is utilized, it is necessary to sink pump wells into the ground. The depths of the pump wells are dependent on the hydrogeological condition at and near the well location. Deep wells are required if the groundwater table is deep or rock head, which is usually less permeable than soil, is shallow.

THE FIELD TRIAL

The Site

The field trial has been conducted on a slope on Lantau Island. Plate 1 shows an overview of the 120m-long slope which inclines at about 45° and has a maximum height of about 8m. It is a reinforced fill slope formed in mid-1990s. Vegetation on the slope mainly comprises grass, creepers and a few trees. The whole slope surface to be irrigated is about 1,200m^2. This slope is selected for the trial because it is located in a remote and open area. This would minimize human disturbances to equipment and maximize the amount of sunlight for generation of electrical energy. Besides, there is a natural stream course close to the slope with persistent water flow all year round. Water from this stream course is used for irrigation.

The System

In the field trial, a solar power irrigation system is installed. The Electrical and Mechanical Services Department of the Hong Kong SAR Government has assisted in the system design and supervision of installation works.

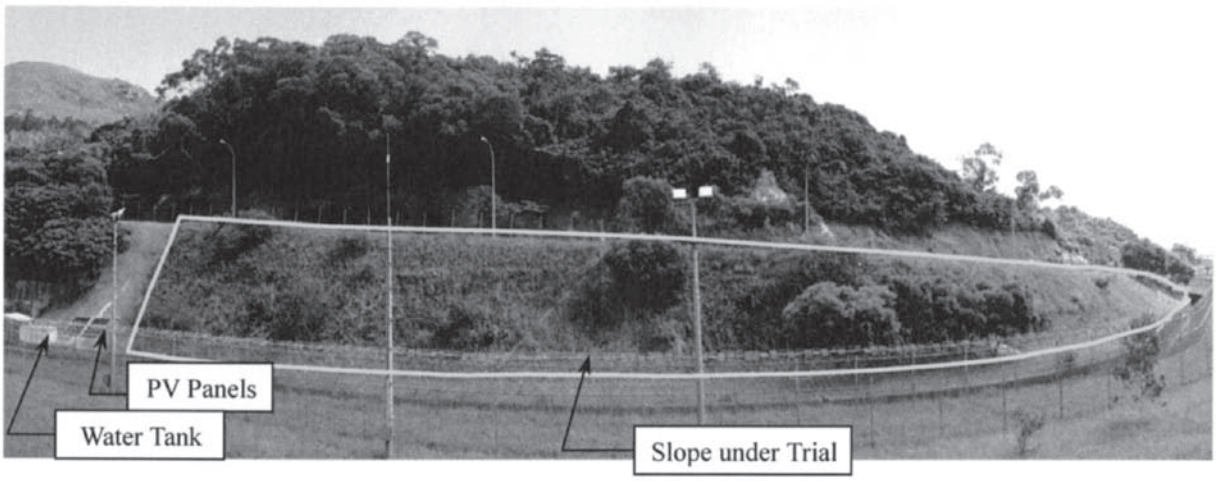

Plate 1 – Overall View of the Slope under Trial

Figure 1 illustrates a schematic layout of the solar power irrigation system. The PV panels generate electricity, which is then transferred to and stored in the rechargeable batteries. Four numbers of PV panels are used in the system, each measuring 1.575 m x 0.826 m and weighing 17 kg. The power output of each panel is 150 W and the efficiency is about 11%. In order to get maximum sunlight intensity, the PV panels are oriented in a southerly direction at an exposed area. Plate 2 shows the PV panels used in the system.

Eight rechargeable batteries, each having a capacity of 100 Ah, are used for providing power for the operation of other equipment in the system. The required capacity of the rechargeable batteries is determined on the basis of the daily power consumption and the minimum number of days for which the system can still be in effective operation under overcast weather. Plate 3 shows the rechargeable batteries.

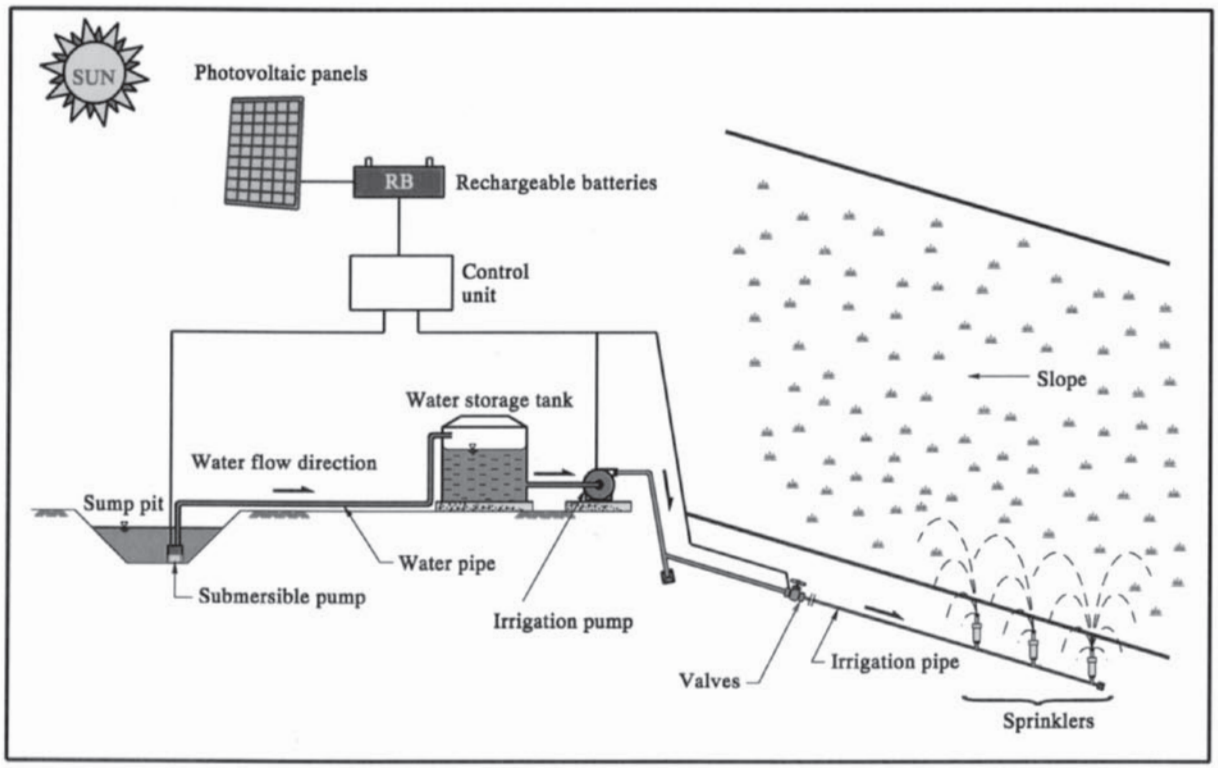

Figure 1 – Schematic Layout of the Solar Power Irrigation System

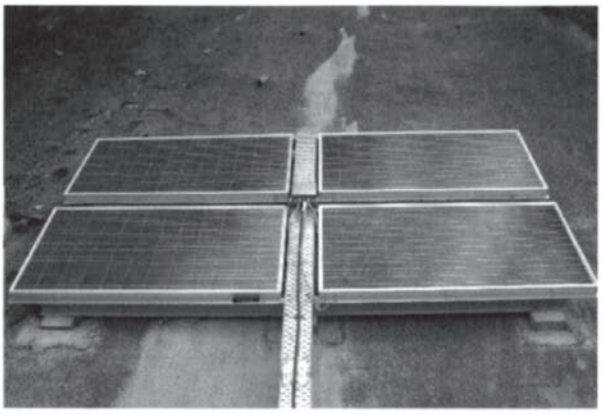

Plate 2 – Photovoltaic Panels

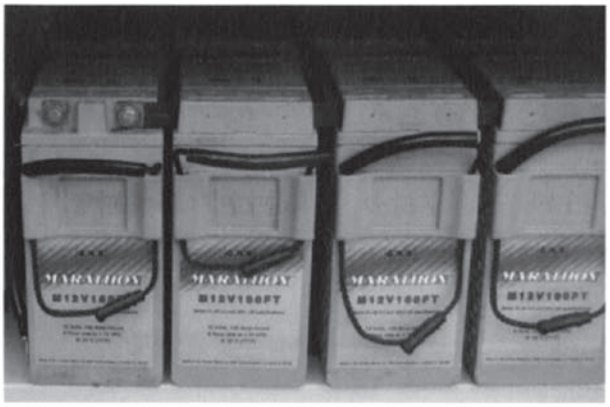

Plate 3 – Rechargeable Batteries

All the equipment of the solar power automatic irrigation system is controlled by a control unit which contains electrical timers, control relays, a charge controller and an inverter. Plate 4 shows the control unit. The timers and control relays control the period for the operation of the pumping and irrigation system. The charge controller controls the charging of the batteries by the PV panels. The inverter converts the direct current from the rechargeable batteries into alternate current for the operation of pumps.

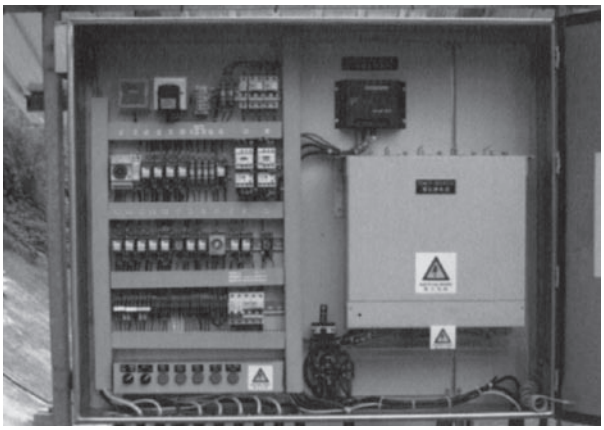

Plate 4 – Control Unit

In the site trial, components for the pumping and irrigation operation comprise a submersible pump, an irrigation pump, a water storage tank, sprinklers and the associated water pipes. The submersible pump, which is housed in a stainless steel case, is placed in a sump pit at the natural stream course. The submersible pump draws water from the stream course to the storage tank at a specified time period as set in the control unit. It has a designed power of 450W which represents a pumping capacity of 2,000 litres of water within an hour. This power capacity takes account of the elevation difference between the pump and the storage tank, and the pressure drop along the pipeline (Stryker, 2001).

The storage tank is made of glass fibre reinforced plastics. Plate 5 shows a view of the storage tank. At a specified time period, the water in the tank is drawn by the irrigation pump through a strainer and valves to the sprinklers installed along the slope toe for watering the vegetation. The power of the irrigation pump used is designed to be 1500W.

The slope to be irrigated is divided into three zones of approximately the same area, each of which is served by two to three sprinklers. With the use of control valves, the system is programmed to irrigate the three zones alternately, each at a frequency of once every three days. This reduces energy consumption and the demand for water at any one time so as to avoid the use of a massive size of storage tank. The watering need of the vegetation is designed to be 10 litres/m^2 every week. About 2,000 litres of water are consumed in each irrigating operation. The operating time for each irrigation is limited to 30 to 60 minutes, corresponding to an average rainfall intensity of less than 10 mm/hour. This is to avoid surface erosion on the slope resulting from strong water spray. The irrigation

pump has a capacity of pumping 2,000 litres of water to the sprinklers. After each irrigation, water is automatically pumped from the stream course to the water tank for the irrigation cycle on the next day. Plate 6 shows the water spraying operation.

The cost of the system installation is about $400,000. Details of the automatic solar power irrigation system are given in Lui (2005).

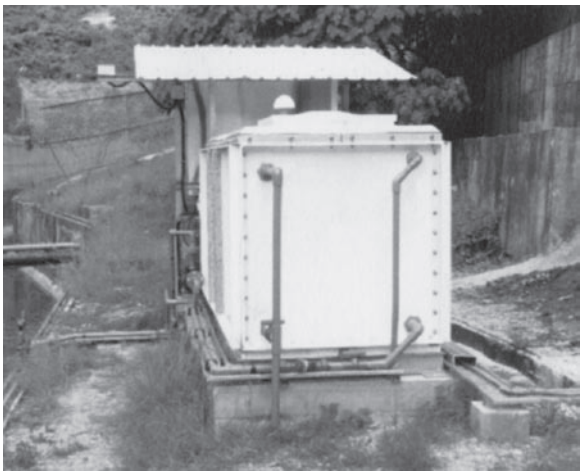

Plate 5 – Water Tank

Plate 6 – Spraying Operation

Special Features of the System

One of the special features of the system is that the rechargeable batteries can store power sufficient for the normal operation of the entire irrigation system for three consecutive days. This enables the system to run on overcast days when the supply of sunlight is limited. Another feature is that a rain detector unit has been installed and connected to the control unit. The irrigation function of the system can be skipped in rainy days in order to save water and energy.

System Maintenance

Since the installation of the system, maintenance work has been carried out on quarterly basis. This includes routine maintenance of the electrical and sprinkler systems, cleaning of pumps and water tanks and regular removal of debris and dust from the PV panels. The rechargeable batteries will have a life-span of about two to three years and cost a few thousand dollars to be replaced. Hence the maintenance cost for operating the automatic solar power irrigation system is small.

System Performance

The performance of the system has been monitored since the installation in July 2003. Two battery failures occurred in November 2003 and January 2004 respectively. The batteries were identified to be defective and were replaced afterwards. The irrigation system was out of operation for a few times in March and April 2004 owing to the prolonged overcast weather in these two months. In May 2004, some modifications were carried out to the system to reduce the power consumption of the system and increase the system efficiency. The overall performance of the system has been satisfactory since then.

DESIGN OF A SOLAR POWER AUTOMATIC IRRIGATION SYSTEM

The site trial has demonstrated that it is viable to use solar power to extract water from stream courses to water vegetation on man-made slopes. Such a system will be applicable to slopes which satisfy the following criteria:

(i) Slopes are located in remote area where water and electricity supply is not readily available in their vicinity or slopes are located along expressways where parking or slowing down of vehicles for watering is not allowed;
(ii) Slopes are located in an open area with adequate space for accommodating the PV panels, battery and water tank, and without being shaded from sunshine for the PV panels to work;
(iii) Water source is available such as presence of nearby natural stream course or shallow groundwater source; and
(iv) No excessive settlement of ground/structures nearby is envisaged as a result of groundwater drawdown in case groundwater is to be extracted.

A solar power automatic irrigation system should generally comprise the following three sub-systems:

(i) Solar Power Sub-system – It is responsible for generating electrical power from solar power and supplying electricity for the operation of the whole irrigation system;
(ii) Electrical Control Sub-system – It controls the charging of batteries and the timing for operation of pumps and sprinklers; and
(iii) Pumping and Irrigation Sub-system – It deals with the extraction of water, temporary storage of the water and irrigation of vegetation.

The major components of the sub-systems are shown diagrammatically in Figure 2.

If groundwater is to be used for irrigation, pump well(s) would be required for extracting the groundwater. The hydrogeological condition of the area where the pump well is to be installed should be determined. The size and depth of the pump well should be

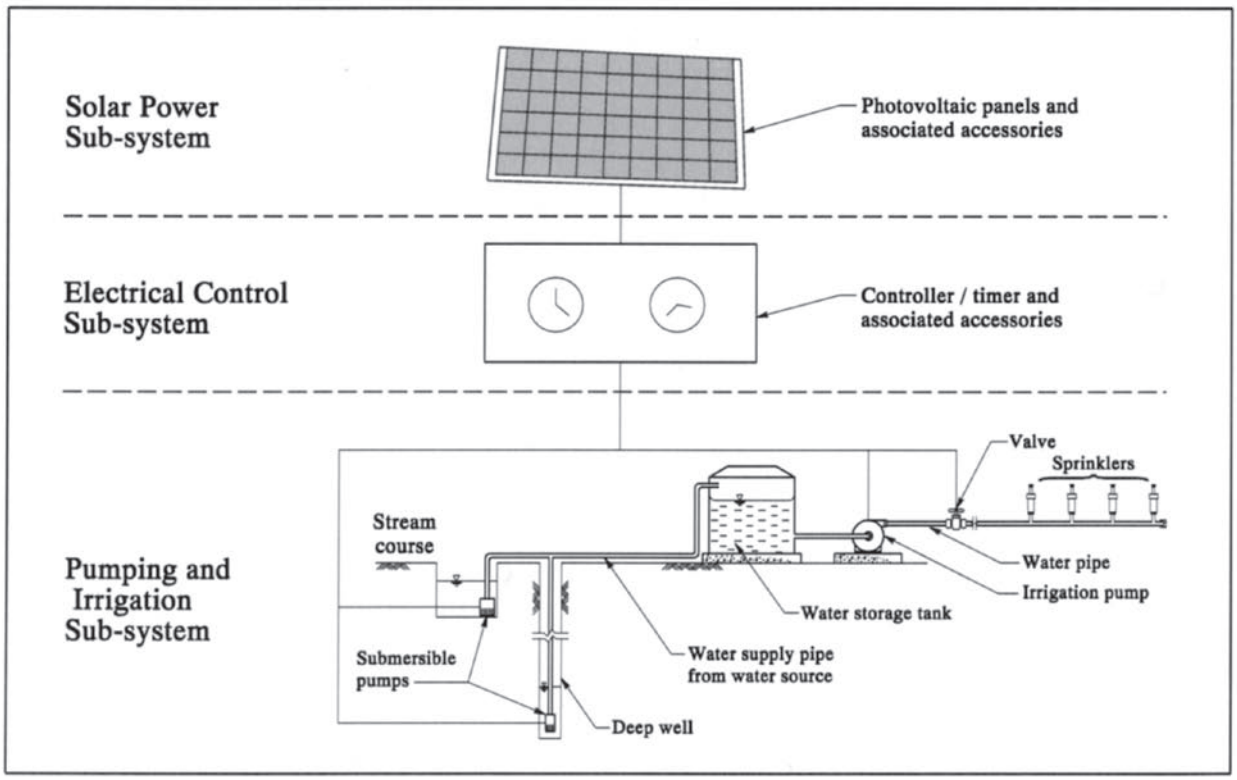

Figure 2 - Major components of a Solar Power Automatic Irrigation System

designed to ensure that the well can provide sufficient yield even in dry seasons when the water table is low. Assessment should also be carried out to estimate the groundwater drawdown, the associated settlement and the probable effects to nearby services or structures. A field test may also be required to determine the yield of the well.

CONCLUSIONS

A solar power automatic irrigation system has been installed on a trial basis. The performance of the system since its operation in July 2003 has been satisfactory. The results of the trial so far have shown that it is viable to use solar power to extract water from stream courses to water vegetation on man-made slopes. Such a system can reduce problems associated with manual watering of vegetation on high and remote slopes.

If irrigation is considered necessary for a site, the use of an automatic irrigation system operated by solar power could be an option in designing the irrigation system. Recommended guidelines for the design of such a system are provided for reference.

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ACKNOWLEDGEMENTS

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PERFORMANCE ASSESSMENT OF GREENING TECHNIQUES ON MAN-MADE SLOPES

Becky L S Lui[1], Y K Shiu[2] and Billy C H Hau[3]

Abstract: Many new greening techniques have been used or tried out on man-made slopes in recent years. A study has been carried out to assess the performance of the greening techniques applied on 100 slopes. The greening techniques evaluated in the study cover a wide variety of proprietary products. In the study, growth condition of vegetation on the slopes and engineering performance of the greening techniques were assessed. This paper describes the methodology adopted in the study. It also presents and discusses the result of the assessment together with the merits and limitations of different techniques. Recommendations on improvements to the application of the greening techniques are also provided.

INTRODUCTION

Before mid-1990s, sprayed concrete and chunam were commonly used as surface finishes to man-made slopes in Hong Kong. The widespread application of visually intrusive hard covers generated considerable public concerns in the late 1990s. In response to the surging expectations from the public in slope appearance, the government tried many new greening techniques on man-made slopes in recent years. A study has been carried out to assess the performance of the greening techniques applied on 100 slopes from two perspectives, namely, the growth condition and sustainability of the vegetation, whether planted or occurring naturally, and the engineering performance of the techniques.

THE GREENING TECHNIQUES

The greening techniques evaluated in the study cover a wide variety of proprietary products. The techniques can be broadly classified into the following two groups: Group 1 techniques are those products that are applied on slopes with a hard surface cover, and Group 2 techniques refer to those that are used directly on an exposed soil slope surface. Out of the 100 slopes assessed in this study, 79 were applied with Group1 techniques and 21 with Group 2 techniques.

Group 1 Techniques

There are 16 proprietary products under the Group 1 Techniques and they can be broadly classified into the following four types according to their general characteristics:

(1) Mulching System – The eight products under this system generally involve applying one or more layers of mulch or soil mix onto the slope surface, usually protected by a layer of steel wire mesh and/or one or more layers of erosion control mats. The mulch/soil

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mixes vary in type, composition and additives among different products and in general have little tensile, compressive and shear strength.

(2) Cellular System – The six products under this system generally contain compartments or panels of mulch or soil mix installed on the slope face. The soil mix/mulch of one of the products is embedded into the slope, exposing the top surface for planting.

(3) Reinforced Soil System – There is only one product under this category. The product involves spraying soil mix and applying continuous synthetic fiber at the same time as reinforcement to the soil mix on the slope face.

(4) Bio-engineering Method – Also there is only one product under this category. The method involves direct planting of vegetation, e.g. *Vetiveria zizanioides* (Vetiver Grass), in planter holes formed on the hard slope cover.

Group 2 Techniques

There are eight products under the Group 2 Techniques and they can be broadly classified into the following three types according to their general characteristics:

(1) Temporary degradable erosion control mats,

(2) Long-term non-degradable erosion control mats, and

(3) Long-term non-degradable erosion control mats in conjunction with steel wire mesh.

METHODOLOGY

In the study, the slopes were selected in such a way that slopes at a range of different gradients, orientations, degree of isolation and date of application were chosen. This would enable the evaluation of whether any of the factors have significant influence on the performance of the greening products. Slopes with difficult access for detailed investigation were not selected. Growth condition of vegetation on the slopes and engineering performance of the greening techniques were assessed in the study.

Assessment for Vegetation Performance

The assessment with respect to vegetation establishment was based on the growth performance and sustainability of the vegetation planted or occurring naturally on the slopes. Three field inspections were performed in the autumn of 2002, the spring and autumn of 2003 respectively to evaluate the growth condition of the vegetation under different climatic conditions and the vegetation's ability to regenerate. The field inspections included identification of plant species; assessment of the percentage cover of dominant species and general health condition. Site characteristics including slope gradient and orientation, signs of seepage, distance of the slopes from structures (such as tall buildings) that would possibly shade the slope face, degree of isolation of the slopes from natural vegetation and degree of exposure of the slopes to road traffic were also noted in the field inspections.

In this study, species richness (**S**) is used for representing the total number of woody native or exotic plant species that are believed to have established naturally on a slope since the completion of the greening works. Furthermore, the Shannon diversity index (**H**), which is a common quantitative index of species diversity, is used for describing the composition of the plant communities that have been established naturally on a slope. The index **H** is calculated on the basis of **S** and is the sum of the proportion of individuals that each species contributes to the total in the sample, i.e. the proportion of P_i for the i^{th} species by (Begon et al. 1996):

$$\mathbf{H} = -\sum_{i=1}^{S} P_i \ln P_i$$

where P_i = the number of plants of the i^{th} species in the sample / the total number of plants in the sample

An example is provided here to explain the idea of using **H** and **S** for describing the composition of plant communities on a slope. If it is given that two slopes, A and B, have the same **S** value, but on Slope A one woody plant species dominates over other species and on Slope B, all plant species have equal distribution. Then the index **H** of Slope B should be higher than that of Slope A, as a higher value of **H** means that the community has a higher evenness and thus a higher diversity of species. There is no bench mark figure for diversity indices to classify the level of diversity. They are usually used for comparison between different communities of similar nature.

Assessment for Engineering Performance

The assessment included desk study of installation details of the greening products and inspection of the engineering elements of the products. The engineering performance assessment was conducted after the first field inspection of the vegetation assessment. For each product, at least one slope with higher vegetation cover and one with lower vegetation cover were selected for the engineering performance assessment. This is to ensure that the engineering elements of the products on slopes with good and poor vegetation cover can be inspected. The engineering performance of the greening products were assessed on the basis of the effectiveness of erosion control and structural integrity which includes the stability of the mulch or soil mix and the anchorage of the product to slope surface.

RESULTS OF THE ASSESSMENT

Mulching System

Results of the assessment for vegetation growth show that, for three of the products under the mulching system, most slopes had consistently high percentage of healthy vegetation cover of about 70%. The majority of the slopes recorded with high percentage of healthy vegetation cover were planted with a climber species, *Wedelia trilobata*, as the main ground vegetation cover. The percentage cover of the planted grasses on most of the slopes surveyed was found to have decreased in the second inspection. On the other hand, the other five products of the mulching system, including two reinforced with cement in their soil mix/mulch, have low percentage of healthy vegetation cover. For the species diversity and richness of naturally established woody species, most slopes were found to have relatively low values of Shannon diversity index **H** and species richness **S.** This shows that slopes treated with this greening technique are not particularly good in supporting naturally established woody plant species.

Plate 1 – Detachment of mulch/soil mix

From the engineering assessment, signs of deterioration of the mulch/soil mixes were observed on slopes of a few products. The signs included

surface cracks associated with desiccation, detachment (Plate 1), bulges at the lower parts of the slopes, etc. These highlight the need for more structural support for the mulch/soil mix of some products. Majority of the products used temporary degradable mat for erosion protection irrespective of slope gradients. The cracked mulch/soil mix could be washed away after the mat had decomposed, especially on steep slopes and at areas of high flow concentration. Edges of some greening products were found to be eroded, especially where adjacent to stepped channels. Overall, the engineering performance of a few other products was found to be sound.

Cellular System

The percentages of healthy vegetation cover recorded on the slopes of two products were consistently satisfactory in all three inspections. Two other products under this system involved the use of panels of mulch/soil mix on the slope surface. Assessment results showed that growth performance of vegetation on these products was variable. The planted grass species did not grow well but the planted *Wedelia trilobata* was doing well on some slopes. The relatively low **H** and **S** values recorded on most slopes also suggest that few woody species have naturally established. These isolated panels tended to dry up quickly probably owing to their large exposed surface areas, resulting in unhealthy plant growth with time. The mulch/soil mix, as a result, disintegrated and was easily eroded (Plate 2).

For one of the products, the mulch/soil mix is placed in holes excavated on the slope surface. The top surface of the mix is exposed for planting. The percentages of healthy vegetation cover recorded on the slope with this product were consistently satisfactory in all inspections. From the engineering perspective, there is not much concern about the stability problem of the mulch/soil mix. Another product involved planting in holes spaced along PVC pipes anchored on slope surface. The performance of this product is unsatisfactory in terms of vegetation growth and coverage.

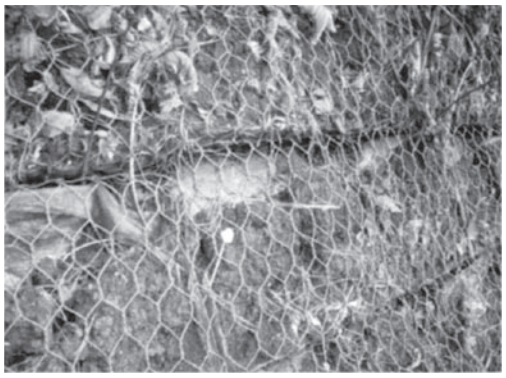

Plate 2 – Erosion of mulch/soil mix

Reinforced Soil System

The vegetation assessment results show that the slopes applied with this product had satisfactory performance of healthy vegetation cover, especially with the planting of *Wedelia trilobata*. On one slope planted with *Wedelia trilobata*, 100% of the vegetation cover was recorded as healthy in all inspections. For the naturally established woody plants, relatively low **H** and **S** values were recorded on one slope whilst the other three had zero **H** and **S,** indicating that the number of woody plants established naturally were low. The engineering condition of this product inspected is satisfactory except that, on one slope, a number of cracks were identified on the surface mulch layer of the product. The fiber-reinforced soil under the surface mulch, however, was still intact without any sign of deterioration.

Bio-engineering Method

All the slopes of this greening method were planted with Vetiver grass which is also the sole vegetation cover of these slopes. Results of the vegetation assessment show that all slopes with this product had consistently high or improving healthy vegetation covers between different inspections. However, old leaves of the Vetiver grass used in this product turned yellow and very few new leaves were able to emerge. The persistence of the old

leaves owing to low rate of decomposition seems to be inhibiting new leaves from developing. In terms of engineering performance, no particular problem was identified.

Group 2 Techniques

The proprietary products in this group are erosion control products. Results of the vegetation assessment show that most of the slopes inspected had satisfactory vegetation cover (i.e. ~70% or above). This implies that there was no significant difference in vegetation cover between slopes with different Group 2 techniques, and that the products performed more or less the same both in sustaining slope vegetation cover. The percentage covers of planted grass of most slopes were found to have decreased in the second inspection, whereas the total vegetation cover had increased. This indicates that the planted grass is being taken over by the natural vegetation established on the slopes. In addition, relatively higher values of Shannon diversity index **H** and species richness **S** were recorded on slopes using Group 2 techniques when compared to those recorded on their Group 1 counterparts.

From the engineering assessment, temporary degradable erosion control mats in some cases had decomposed before the vegetation was established to such a state that it could effectively protect the soil. This emphasizes that temporary mats must have service life sufficiently long to allow vegetation to establish and sustain. In one case, vegetation did not grow along the strips where two layers of the mat overlapped and erosion gullies were found under the mat along these areas, indicating that the thick layers of the mat at overlaps had suppressed plant growth. On some other slopes inspected, minor surface erosions were found.

Erosions of different degree of seriousness were apparent on some slopes with non-degradable erosion control mats. A common problem was that the mats were not pegged securely onto the slope surface and water got under the mats and caused soil erosion. For high and steep slopes, surface erosions were still found, indicating that erosion cannot be completely eliminated in such a condition.

The use of steel wire mesh in conjunction with non-degradable erosion control mat provides an effective means of controlling surface erosion. The steel mesh is usually stretched slightly and anchored securely onto concrete nail heads. This helps ensure that the erosion control mat is in good contact with the slope surface. Minor erosion was apparent on steep slopes. However, the eroded soil was trapped by the erosion control mat and the steel mesh (Plate 3). The trapped material acted like a protective layer to minimize further erosion.

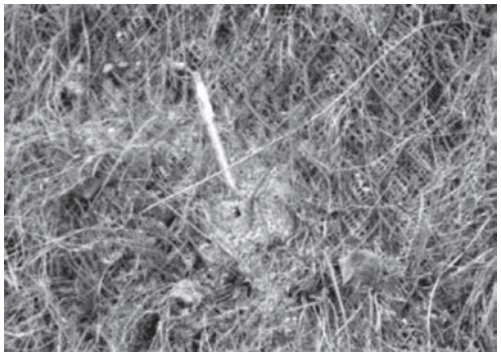

Plate 3 – Eroded soil trapped by erosion control mat and steel mesh

DISCUSSION

Group 1 Techniques

Different proprietary products use different material and methods of construction. Some products perform relatively better than others under certain circumstances. Based on the results of the vegetation and engineering assessments, the merits and limitations of different Group 1 Techniques are summarized in Table 1.

Table 1 – Summary of Merits and Limitations of Different Group 1 Techniques

Techniques	Merits	Limitations
Mulching System	• Able to form smooth finished surface on uneven slope profile, e.g. a cut slope with many exposed corestones • Quick installation	• Mulch/soil mix generally is weak in strength and liable to cracking and downward movement • Mulch/soil mix layer thin and unable to support small woody species
Cellular System	• Some products allow vegetation to be established at nursery areas before installation of panels of soil mix on slopes to provide instant greening effect. • One product allows vegetation to have direct contact with soil.	• Small isolated panels of soil mix installed on slope surface may dry up and result in unhealthy plant growth • When without a dense vegetation cover, the wire mesh of the panels holding mulch/soil mix very visible and aesthetically unpleasing
Reinforced Soil System	• The use of soil reinforcement permits a thicker layer of soil mix which can support growth of woody vegetation such as shrubs and small trees rather than grass species only.	• The thicker planting medium may limit the product's use on slopes with limited space in front of the slopes.
Bio-engineering Method	• Direct planting of vegetation into the soil allows more water and nutrients for vegetation than the other products which are applied on top of slope cover.	• Some vegetation species, e.g. *Vetiveria zizanioides*, require regular cropping in order to provide space for new growth. • The hard slope cover is exposed after each cropping, leading to temporary visual impact. • Monotonus appearance owing to the use of one single species

The greening products have been applied for less than four years on about 80% of all the slopes in the study. Furthermore, some products have been used on a few slopes only. As such, the long-term performance of the products in terms of their engineering performance and ability to sustain vegetation is still uncertain. Despite this, several products have been identified to have consistently satisfactory vegetation cover in the study period. It is considered that products which allow vegetation to have direct contact with soil underneath the concrete cover are able to sustain vegetation including small woody species.

From the vegetation assessment results on Group 1 techniques, the Shannon diversity index **H** and species richness **S** values of most slopes are low, indicating that there is little diversity in plant species established naturally on the slopes using Group 1 techniques.

Group 2 Techniques

Based on the results of the vegetation and engineering assessments, the merits and limitations of different Group 2 Techniques are summarized in Table 2.

Table 2 – Summary of Merits and Limitations of Different Group 2 Techniques

Techniques	Merits	Limitations
Temporary Degradable Erosion Control Mats	• Degradable	• Not suitable for steep slopes or slopes with concentrated surface runoff
Long-term Non-degradable Erosion Mats	• Strong, durable and ultra-violet stabilized • Mats are in 3-D matrices that help retain and entangle soil and plant roots together to form a "living" layer.	• Restriction of growth of woody plants and trees due to small hole sizes
Long-term Non-degradable Erosion Mats with Steel Wire Mesh	• Ditto • More effective in controlling surface erosion	• Ditto

Much higher **H** and **S** values were obtained for slopes with Group 2 techniques than those with Group 1 techniques, indicating that Group 2 techniques are more able to support a diversity of naturally occurring plant species. In addition, Group 2 techniques tend to have higher sustainability on the vegetation cover over Group 1 counterparts because vegetation can grow into the ground soil without the separation of a hard slope cover. Hence, provided that there is no stability concern of the slopes, planting directly on the soil surface of slopes is preferred as this can support more diversified vegetation cover, thus enhancing the ecological value of slopes.

Plant Species

On most of the slopes treated with Group 1 techniques, the **H** and **S** values are low. This shows that there is little diversity in plant species. Slopes with Group 1 techniques are dominated by the planting of *Wedelia trilobata* and the two grass species (*Cynodon dactylon* and *Paspalum notatum*). When *Wedelia trilobata* was planted together with grass species on some slopes, it generally grew better than the planted grass species and dominated the vegetation cover irrespective of the type of greening techniques, giving rise to a monotonous appearance. While this species is effective in greening slopes and can be used on slopes with a harsh environment where other species generally do not perform well, its use in agricultural areas and along wetland marshes, streams, canals, mangrove edges and coastal areas should be avoided, because it is known to be particularly invasive in such habitats (Lowe *et al.* 2000).

Results of this study show that planted species on slopes treated with Group 2 techniques, especially grasses, are gradually replaced by the naturally established native and exotic plant species. It seems that natural succession is progressing on slopes with exposed slope surface treated with erosion control mats, which may lead to a more diverse and sustainable vegetation cover without the need for recurrent maintenance. On the other hand, the high diversity of woody plants, such as trees, recorded on the 100 slopes has significant maintenance implications, especially on slopes with Group 1 techniques. Most Group 1 products have a thin soil mix (100 mm or less) adhered to the sprayed concrete surface. The thin planting medium is not able to support tree growth.

RECOMMENDED IMPROVEMENTS

Detailing

For the mulching, cellular and reinforced soil systems under the Group 1 techniques, the following improvements on detailing of the products are recommended to increase the system integrity:

(1) For products of the mulching system, support in the form of a layer of PVC coated steel wire mesh should be provided to the mulch/soil mix surface. For slopes with an uneven profile and where installation of the wire mesh is difficult, vertical and horizontal bracing in the form of small diameter steel bars can be used to enhance the stability of the planting medium.
(2) Long-term non-degradable mats should be used in order to provide more durable protection to the mulch/soil mix which is weak in strength.
(3) Edges of mulch/soil mix should be fully protected from surface erosion by erosion control mats wrapped around them.
(4) Concrete kerb or planter wall should be provided to support the toe of the mulch/soil mix layers.
(5) Openings should be formed in the existing concrete cover to enable direct contact between the mulch/soil mix and the soil underneath the concrete. This would allow more moisture and nutrients for the vegetation from the soil in the slope.

The following measures are recommended on the use of Group 2 techniques:

(1) Temporary degradable erosion control mats should have longevity of at least 12 months to allow sufficient time for vegetation growth and establishment on slopes.
(2) Manufacturer's recommended practice in respect of overlapping the edges of adjacent mats should be followed.
(3) For steep nailed cut slopes, long-term non-degradable erosion control mats should be used in conjunction with a layer of wire mesh.

Plant Species

A way to achieve sustainable vegetation cover for Group 1 techniques is to diversify the vegetation species on these slopes. As man-made slopes are usually relatively steep, the use of native species which are adaptable to such condition is recommended. Suitable native fern and herbaceous species such as *Blechnum orientale, Cyclosorus parasiticus and Dicranopteris pedata* are recommended. Some other native species which have proven to be suitable for planting on steep slopes can be found in Choi & Chau (2004).

CONCLUSIONS

There is a wide variation in the performance of different techniques and products. However, the vegetation growth and engineering performance of different greening techniques was quite satisfactory in general during the study period. Most of the products under the Group 1 techniques have only been applied on the slopes inspected for less than four years. The long-term performance of the techniques will require further assessment.

The study results show that the erosion control mats are generally effective in preventing surface erosion. They also indicate that there is no significant difference in vegetation covers between slopes treated with different Group 2 techniques. The products performed more or less the same both in sustaining slope vegetation cover and in supporting the diversity of naturally occurring plant species. Compared with Group 1 techniques, Group 2 ones can sustain a greater diversity of species, thus enhancing the ecological value of slopes. As such,

planting directly on the soil surface of a slope is preferred to those techniques used on hard surface cover provided that there is no stability concern of the slope. In addition, non-degradable erosion control mats are more effective in controlling surface erosion when steel meshes are used. Details of the findings of study are given in Shui and Lui (2004).

In selecting which products or techniques to be used, factors such as cost-effectiveness, long-term performance of the products, special characteristics of the slope, the surrounding environment and the future maintenance requirement should be considered.

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ACKNOWLEDGEMENTS

This paper is published with the permission of the Head of the Geotechnical Engineering Office and the Director of Civil Engineering and Development, Government of the Hong Kong Special Administrative Region.

USE OF CARBON FIBRE REINFORCED POLYMER REINFORCEMENT IN SOIL NAILING WORKS

W M Cheung and D O K Lo [1]

Abstract: Steel soil nails have been used extensively in slope improvement works in Hong Kong. In general, the adequacy of tensile strength and the durability of steel soil nails are the prime concerns in a soil-nailed slope design. Although steel nails can generally provide adequate tensile strength, under some circumstances twin-bar nails or other proprietary high-strength reinforcement have been used to provide higher nail capacity. In highly aggressive grounds, corrugated plastic sheathing is used as corrosion protection measure to steel reinforcement. Apart from higher construction cost associated with such measures, difficulties in installation and prolonged construction time are other areas of concern. With the advance of technology, there is an increasing trend of using composite materials made of synthetic fibres embedded in a polymeric resin, also known as fibre-reinforced polymers or fibre-reinforced plastics (FRP), as an alternative to steel reinforcement in civil engineering works. A study has recently been carried out by the Geotechnical Engineering Office to investigate the feasibility of using carbon FRP (CFRP) reinforcement in soil nailing works. This paper presents the findings of the study. Some guidance on the design and construction of soil nailed slopes using CFRP reinforcement under the Landslips Preventive Measures (LPM) Programme will also be discussed.

INTRODUCTION

Steel soil nails have been used extensively in slope improvement works in Hong Kong. In general, the adequacy of tensile strength and the durability of steel soil nails are the prime concerns in a soil-nailed slope design. Although steel nails can generally provide adequate tensile strength, under some circumstances twin-bar nails or proprietary high-strength reinforcement have been used to provide higher nail capacity. In highly aggressive grounds, corrugated plastic sheathing is used as a corrosion protection measure to steel reinforcement. Apart from higher construction cost associated with such measure, difficulties in installation and prolonged construction time are other areas of concern. With the advance of technology, there is an increasing trend of using composite materials made of fibres embedded in a polymeric resin, also known as fibre-reinforced polymers or fibre-reinforced plastics (FRP), as an alternative to steel reinforcement in civil engineering works. The main reason for using FRP as an alternative to steel reinforcement is their good corrosion resistance. The fibres commonly used in composites for civil engineering include carbon FRP (CFRP), glass FRP (GFRP), and aramid FRP (AFRP). Others that are not widely used include polyethylene, quartz, boron, ceramics and natural fibres. Shanmuganathan (2003) gave a state-of-the-art review of development and application of FRP composites in civil and building structures in various countries. CFRP reinforcement has been applied to slope stabilization works in some countries such as the USA, the UK, Spain, Greece, Japan and Korea. A study has recently been carried out by the Geotechnical Engineering Office to investigate the feasibility of using CFRP reinforcement in soil nailing works. This paper presents the findings of the study.

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COMPOSITION AND CHARACTERISTICS OF CFRP REINFORCEMENT

Composition

There is a variety of literature available describing the composition and properties of CFRP materials (e.g. Castro and Carino, 1998; IStructE, 1999; CIRIA, 2002; Uomoto et al, 2002; ACI, 2003; Malvar et al, 2003; Thiagarajan, 2003). As the name suggests, the fibres in CFRP composites are carbon in nature, while the matrix is a resin of thermoset or thermoplastic. The volume fraction of the fibres in the CFRP composites generally ranges from 50% to 65%. In general, the tensile strength of CFRP composites increases as the fibre content increases until it reaches about 70% where the tensile strength starts to decrease. It is because a further reduction in resin content will reduce the inter-laminar shear strength and the resin cannot hold the fibres together properly. CFRP reinforcement is typically manufactured by a process called pultrusion as shown in Figure 1. In this process, the fibres in the form of continuous rovings, are drawn off through a resin bath which impregnates the fibre bundle. They are then pulled through a die which consolidates the fibre-resin combination and forms the required shape.

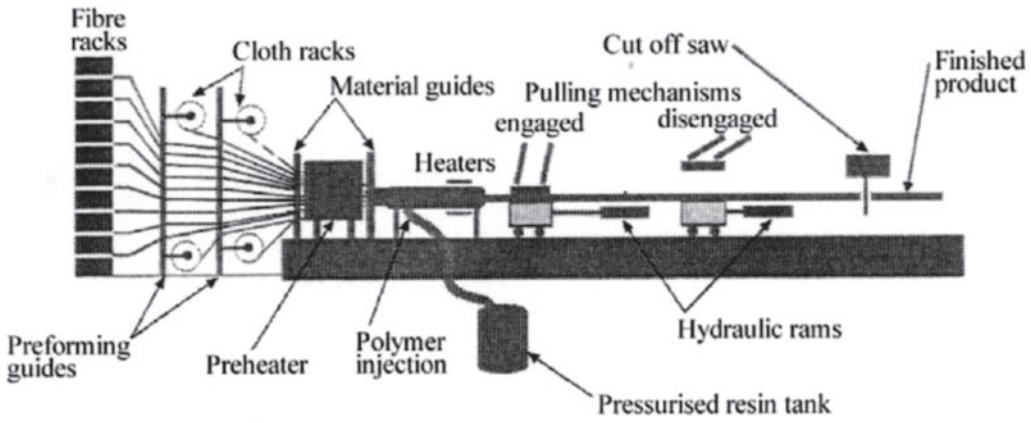

Figure 1 – The Pultrusion Process (CIRIA, 2002)

Physical and Mechanical Characteristics

General

CFRP reinforcement is anisotropic in nature and is characterized by high tensile strength in the direction of reinforcing fibres. It is highly corrosion resistant and has a much better strength-to-weight ratio than steel reinforcement. However, CFRP reinforcement does not exhibit plastic yield behaviour before failure as that in steel reinforcement upon loading. This necessitates consideration of its lack of ductility in geotechnical structures such as a nailed slope.

Density

The density of CFRP reinforcement ranges from 1.25 g/cm^3 to 2.10 g/cm^3, about one-sixth to one-fourth that of steel (c.f. 7.8 g/cm^3 for steel bars) (ACI, 2003). This offers an advantage of reduced mass when compared with the use of steel reinforcement.

Tensile Strength

The tensile strength of CFRP reinforcement depends on a number of factors. These

factors include the volume of fibre to the overall volume of the reinforcement, rate of curing, manufacturing process, and manufacturing quality control, etc. In general, the tensile strength of CFRP reinforcement is much higher than that of high yield steel. According to ACI (2003), the tensile strength of CFRP reinforcement ranges from 600 to 3,690 MPa (c.f. 460 MPa for high yield steel). Unlike steel, CFRP reinforcement does not exhibit plastic behaviour (yielding) before rupture. The tensile behaviour of CFRP reinforcement is characterized by a linearly elastic stress-strain relationship until failure. A schematic diagram showing the stress-strain behaviour of a CFRP reinforcement and a high yield steel reinforcement is given in Figure 2.

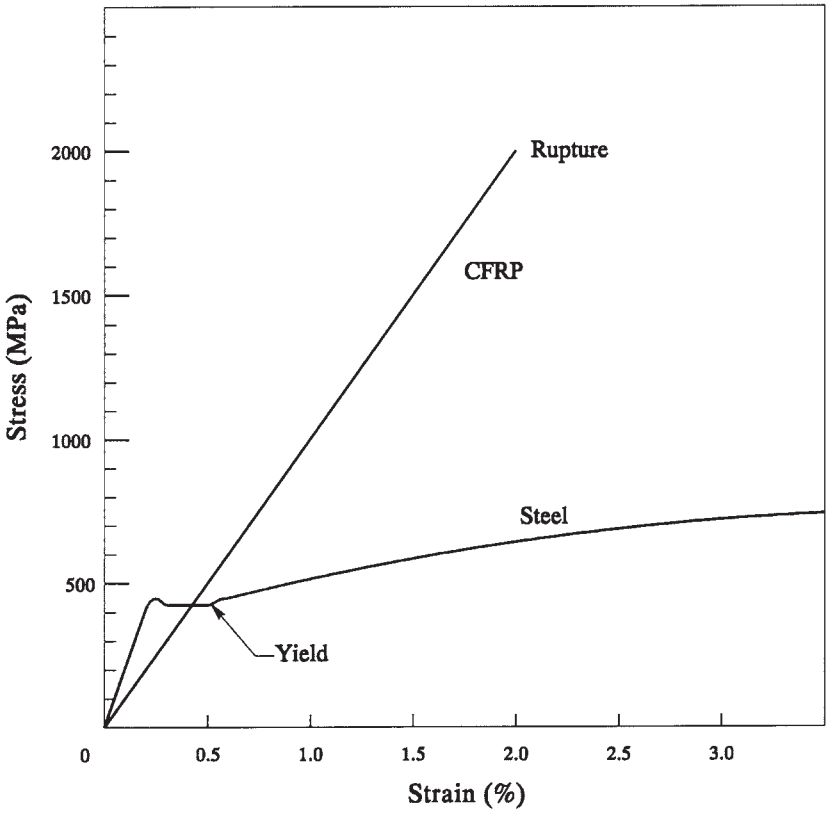

Figure 2 – Stress/Strain Curves of Typical High Yield Steel Bar and CFRP Bar

Tensile capacity of CFRP reinforcement, however, does not increase proportionally to the reinforcement size (e.g. diameter). This phenomenon is related to shear lag in which the surface bond stresses are transferred to the core of the reinforcement through shear stresses. The possibility for the occurrence of the inter-laminar shear failure increases as the reinforcement size increases.

Flexural Tensile Strength

When a straight CFRP reinforcement is bent and then subject to tension, the tensile strength will be reduced as compared with that of a straight reinforcement. The testing procedures described in JSCE (1997) or ACI (2004) can be used to determine the relationship between the bending diameter ratio, which is defined as the ratio of the external diameter of the deflector in contact with the CFRP specimen and the diameter of the CFRP specimen, and the tensile strength of the bent CFRP specimen. In general, the minimum bending diameter ratio without causing damage to the CFRP reinforcement is product-dependent.

Compressive Strength

The compressive strength of CFRP reinforcement is usually not high. ACI (2003) reported that the compressive strength of CFRP reinforcement is about 78% of their tensile strength according to some laboratory tests. The mode of compressive failure for CFRP reinforcement includes transverse tensile failure, fibre micro-buckling and shear failure. In general, compressive strength of CFRP reinforcement increases as the tensile strength increases. Yet, because of a comparatively high uncertainty of this characteristic, compressive strength of CFRP reinforcement is commonly ignored in structural design (e.g. IStructE, 1999; ACI, 2003).

Modulus of Elasticity

The modulus of elasticity of CFRP reinforcement is generally less than that of steel reinforcement, except in some CFRP products. Similar to the tensile strength, the modulus of elasticity of CFRP reinforcement is product-dependent. According to ACI (2003), the elastic modulus of CFRP reinforcement ranges from 120 to 580 GPa (c.f. 200 GPa for high yield steel).

Shear Behaviour

Shear failure in CFRP reinforcement can be divided into (i) inter-laminar shear between layers of un-reinforced resin between layers of fibres and (ii) transverse shear across the reinforcement. The inter-laminar shear strength is usually weak (ACI, 2003). This indirectly governs the tensile strength of CFRP reinforcement in which the surface bond stresses have to be transferred to the fibres of the reinforcement through shear stresses in order to mobilize the tensile stresses of the fibres. This suggests that an increase in reinforcement diameter does not necessarily increase its tensile strength proportionally. For the transverse shear strength of CFRP reinforcement, it is generally much lower than its tensile strength. According to Benmokrane et al. (1997), the transverse shear strength of some CFRP reinforcement obtained by laboratory tests is about 11% of their tensile strength, and that of steel reinforcement are about 47% of their tensile strength. As advised by one CFRP reinforcement supplier, the transverse shear strength of some of their products is about one-fifth of the tensile strength (Corba, 2004).

Bond Behaviour

A number of research studies have been conducted to investigate the bond between CFRP and concrete/grout (e.g. Zhang and Benmokrane, 2002; Malvar et al, 2003). According to the studies, the bond between CFRP reinforcement and concrete/grout depends on the design, manufacturing process, surface condition (e.g. spiral-wound or sand-coated) and mechanical properties of the reinforcement, and the environmental conditions. According to the IStructE (1999), there is no conclusive evidence to indicate that the bond strength is a function of the concrete strength as that in steel because the experimental results are contradictory. For example, ACI (2003) reported that the bond between CFRP reinforcement and concrete appears not to be significantly influenced by the concrete compressive strength. Yet, JSCE (1997) considered the bond strength to be related to the concrete strength and an empirical equation that related the bond strength to the concrete strength similar to that used for steel reinforcement had been developed. Nevertheless, both documents recommend that the design bond strength of a particular CFRP reinforcement should be established based on the laboratory test data provided by the manufacturer using standard testing procedures.

TRIAL INSTALLATION

Recognising the potential of using CFRP reinforcement in soil nailing works, trial installation and pull-out tests of two types of CFRP reinforcement (flat strips and round bars) were carried out at three LPM sites. The CFRP nails are not working nails and the purposes of the trial are to investigate the ease of nail installation and to identify the areas for improvement so as to cope with the current local construction practice.

Laboratory Tests

Before conducting the trial installation, a number of CFRP specimens were sent to a local university for determination of the tensile strength, elastic modulus, and the bond strength between the specimens and cement grout. The surface of the specimens was coated with sand grains. The procedures of the tests basically followed those of JSCE (1997). The tensile strength of the tested specimens (5 in numbers) varies from about 1,990 MPa to 2,550 MPa with an average of 2,280 MPa. The average strength is about 5 times that of high yield steel (c.f. 460 MPa). The average modulus of elasticity of the CFRP reinforcement is about 114 GPa, which is about 57% of the modulus of steel (c.f. 200 GPa). For the bond strength test, the design strength and mix of the cement grout are similar to that commonly used in soil nailing works. Instead of casting the specimens in a cement grout cube as stated in the standard test (JSCE, 1997), the specimens have been embedded in prisms in order to investigate the effect of grout bleeding on the bond strength under an exaggerated condition. The test results indicate that the bond strengths of the CFRP specimens embedded in the lower half of the grout prisms (average bond strength = 6.4 MPa) is generally higher than those of the specimens embedded in the upper half (average bond = 2.7 MPa). This suggests that grout bleeding could have significant effect on the bond strength if this factor is not controlled. Nonetheless, the case in which the CFRP specimens were embedded in the lower half of the grout prisms is believed to be more relevant to soil nailing works where the quality of grout is carefully controlled against bleeding. Moreover, it is expected that the ultimate bond strength of the former case would be higher than that obtained at present because the failure of all the tests is not caused by bond failure but the splitting of the grout prisms.

Mobilization, Handling and Assembling

The flat CFRP strips were delivered to the sites in rolls (about 1 m diameter), whereas the CFRP round bars were already cut to the required nail lengths before they were delivered to the site. Workers can carry the rolls of reinforcement easily uphill (a 30 mm x 4 mm CFRP flat strip weighs about 0.18 kg/m c.f. a 25 mm dia. steel bar weighs about 3.85 kg/m) and the flat strip was cut to the required nail length using a handsaw. No reinforcement connector (coupler) is required. The accessories of CFRP soil nails such as centralisers, plastic tube of grouting, etc. are similar to those used in steel soil nails. However, the accessories at the nail head is product-dependent, and is different from that of steel soil nails. For example, the accessories at the nail head of this CFRP product contain wedges slotted in a stainless steel conical head. For the case of double-flat-strip nail, the assembling of the two flat strips was carried out on site. Flat strips were separated by proprietary chairs. It took much time and more working space for the bundling up and fixing of flat strips.

Installation

The installation of CFRP round bars was easy and quick. Yet, because CFRP flat strips have relatively low stiffness when compared with steel reinforcement, the thrusting of the nails into boreholes to overcome minor blockages was difficult at times particularly when the nail was long. In general the installation of CFRP nails was easy.

Grouting

There is no difference between the grouting of CFRP and steel soil nails.

Pull-out Tests

Pull-out tests were carried out on all seven CFRP soil nails. In order to accommodate the conventional pull-out test equipment for steel reinforcement, a cylindrical adapter had been epoxy bonded to the CFRP reinforcement to be tested. All the seven head adapters were prefabricated in a factory. It is observed that some test nails manifested pre-mature rupture of CFRP reinforcement immediately behind the head adapter. All the pre-mature failures of CFRP reinforcement occurred at those of flat strip type, in particular the comparatively long ones. One possible cause for this pre-mature failure could be a damage of carbon fibres by stress concentration immediately behind the head adapter during transportation. Since the flat strip type reinforcement was delivered to sites in rolls, its flexibility (low rigidity) and the weight of the prefabricated head adapter (about 14 kg) made it susceptible to damage by the excessive bending stress induced at area behind the head adapter. The round CFRP reinforcement with its comparatively higher stiffness, on the other hand, was delivered to site as straight pieces and no such pre-mature rupture was observed. For the nail with double CFRP flat strips, it is found that individual reinforcement may not be tensioned simultaneously leading to possible pre-mature failure of the nail. Both cases illustrate that the tensile strength of CFRP reinforcement is highly sensitive to non-uniform or combined stress.

PROPOSED INTERIM DESIGN AND CONSTRUCTION GUIDELINES

CFRP reinforcement has been used in pre-stressed ground anchors in Japan since 1994 (Benmokrane et al, 1997). CFRP reinforcement has also been used as soil nail reinforcement to stabilise a railway embankment in Sunderland, UK (Unwin, 2001) and the wingwalls of a bridge abutment in London, UK (Ground Engineering, 2004). Moreover, there are reported cases where CFRP reinforcement has been used to stabilize a steep rock slope in Spain, a soil cut slope in Greece and some weathered rock slopes in Korea. However, no recognised design and construction standards for the use of CFRP reinforcement in geotechnical engineering works are available even though there are a number of national design guides on the use of CFRP reinforcement in concrete structures (viz. JMC, 1997; JSCE, 1997; IStructE, 1999; ACI, 2003).

The design of a soil-nailed slope using CFRP reinforcement is in principle similar to that of using steel reinforcement. The major difference is on establishing the appropriate material properties of the CFRP reinforcement for design. The proposed interim guidelines, to be used in design and construction of soil-nailed slopes under the LPM Programme, are based on a review of the four design guidance documents mentioned above taking due account of the experience gained in field applications and trials. Given that CFRP reinforcement is a relatively new material in geotechnical engineering applications, a reasonably conservative design approach has been adopted. The proposed guidelines are adapted from the current soil nailing design with suitable modifications in the following aspects to suit the material behaviour of CFRP reinforcements and to address concerns raised over their use.

Tensile Strength and Ductility

As mentioned earlier, CFRP reinforcement is anisotropic in nature and is characterized by high tensile strength in the direction of reinforcing fibres. However, CFRP reinforcement does not exhibit plastic yield behaviour before failure as that in steel reinforcement upon

loading. Some pilot research is being carried out in structural and material engineering aiming at improving the ductility of structural members. For example, Li and Wang (2002) reported that the ductility of a structural member reinforced by FRP materials could be improved if conventional cement mortar is replaced by steel fibre-reinforced mortar in the concrete matrix. Apart from changes in the concrete matrix, the ductility of the reinforcement itself can be improved if both fibres and steel strands are used together and bounded by resins (Harris et al., 1998; Lee, 2005). However, these approaches are still in development stage, and the practice currently adopted in building engineering to deal with the inherent brittleness of FRP materials is through design consideration. For example, a larger material safety factor for FRP reinforcement than that of steel or provision of redundant structural members are being used to improve the safety margin against the lack of ductility of FRP material (IStructE, 1999; ACI, 2003). In line with the international practice, a suitably conservative estimate of the design tensile strength using a relatively large material factor is proposed. The design tensile strength should not be more than 30% of the characteristic tensile strength of the CFRP reinforcement. The characteristic tensile strength is defined as the tensile strength value below which not more than 5% probability is expected to fall. At this design stress level, the tensile strain will be of about 0.2% to 0.5% (according to ACI (2003), the rupture strain of CFRP material varies from 0.5% to 1.7%). This not only caters for the uncertainty in material properties during production, the uncertainty of its long-term performance and site handling, but also to compensate for its lack of ductility. The relatively large material factor on tensile strength ensures that the design tensile strength can be mobilised at small tensile strain and thus there would be ample margin against brittle failure. Strain compatibility is also an important factor in the design. Shear strain is required to mobilise the peak shearing resistance and the soil strength will reduce as the soil shears toward the critical state. The proposed range of design tensile strain ensures that the required tensile strength can be built up rapidly and equilibrium occurs at a lower shear strain than that required to mobilise the peak shear strength of the soil. Nonetheless, given that the elastic modulus of CFRP reinforcement is product-dependent (according to ACI (2003), the elastic modulus of CFRP reinforcement varies from 120 to 580 GPa), it is recommended that a strain compatibility analysis should be carried out in the design. Apart from small design tensile strain, the inclination of the nails are confined to within 15° from the horizontal to optimise the reinforcing efficiency of the nails and also to limit the slope movement should the tensile capacity of the nails be mobilised (Clouterre, 1991; FHWA, 1998). If the combined effect of bending and tension is to be known, one can follow the standard testing procedure (e.g. JSCE, 1997; ACI, 2004) to determine the tensile strength of deflected CFRP reinforcement.

Shear Strength

Regarding the transverse shear strength, while some CFRP reinforcement may have low shear capacity, Benmokrane et al. (1997) reported that the transverse shear strength of some CFRP reinforcement can be about 11% of its tensile strength. For a CFRP reinforcement with typical tensile strength of 2,000 MPa, the shear strength is about 220 MPa, which is comparable to that for steel reinforcement. It is proposed that only CFRP reinforcement with transverse shear strength equal to or greater than 230 MPa should be used. As the shear strength of CFRP reinforcement is product-dependent, one can follow the standard testing procedure to determine the transverse shear strength by direct application of double shear (e.g. JSCE, 1997; ACI, 2004). Since CFRP reinforcement is a brittle material, care should be taken in the design to safeguard against the occurrence of sudden failure by providing a larger material safety factor and redundant nails as mentioned in the previous Section. One should carry out a numerical analysis to assess the shear deformation behaviour of the CFRP nailed

slope. Standard laboratory tests (e.g. ASTM, 1996; JSCE, 1997; ACI, 2004) should also be conducted to establish the failure envelope of the CFRP reinforcement under combined loadings of shear, bending and tension. In addition, non-standard tests can be carried out to assess the performance of CFRP reinforcement embedded in grout under the combined loadings (e.g. Park & Naaman, 1999).

Bond Strength between CFRP Reinforcement and Grout

Unlike steel reinforcement, there is no general agreed correlation between the bond strength between CFRP reinforcement and grout, and other relevant parameters. In general, the bond strength depends on the design, manufacturing process and mechanical properties of the reinforcement itself. Following international practices, it is proposed that the characteristic bond strength of a particular type of CFRP reinforcement should be determined from laboratory tests (e.g. JSCE, 1997; ACI, 2004) and a material safety factor of 1.4 as recommended by IStructE (1999) be adopted to derive the design bond strength.

Durability

While it has been reported that some CFRP reinforcement exhibit good durability under aggressive conditions, there is concern over the durability of the resin used in some CFRP reinforcement. In this connection, it is proposed that CFRP reinforcement with polyester resins should not be used because of their ease of degradation in highly alkaline environment. One should follow the procedure of the standard accelerated test for alkali resistance of the reinforcement (e.g. JSCE, 1997; ACI, 2004) to assess the performance of CFRP reinforcement under alkaline environment.

CONCLUSION

CFRP reinforcement, with its high tensile strength, durability, light weight, and flexibility, is a potential alternative to steel bars in soil nailing works. Its brittle behaviour, and low shear and bending capacity are concerns of using this material as soil nail reinforcement. It is not ready for general application. However, a conservative set of interim design and construction guidelines has been proposed for trial use in the LPM Programme to gain insight and experience. This would lead to better knowledge of the practicality and scope of application of CFRP material in slope reinforcement.

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ACKNOWLEDGEMENTS

The CFRP reinforcement used in the field trial and the respective laboratory test results were provided by Sireg S.p.A. Valuable assistance was given by Landslips Preventive Measures Division 1 of the Geotechnical Engineering Office, C M Wong and Associates Ltd, Halcrow

China Ltd, Barbican Construction Co. Ltd, Bachy Soletanche in arranging the site trial installation and pull-out tests. A number of colleagues and practitioners, in particular Mr. C M Wong and Mr. Chris Lee of C M Wong and Associates Ltd, Mr. Antonio Corba of Sireg S.p.A., Professor A.K.H. Kwan of the University of Hong Kong, and Dr. S.K.L. Lee of the Hong Kong University of Science and Technology, gave useful suggestions for improvement in the site trials and pull-out tests, and comments on the properties of CFRP materials. All contributions are gratefully acknowledged. This paper is published with the permission of the Head of the Geotechnical Engineering Office and the Director of Civil Engineering and Development, the Government of the Hong Kong Special Administrative Region.

STABILITY OF SLOPES WITH SOIL NAILS A RELIABILITY INDEX STUDY

J.M. Shen[1]

Abstract: This paper illustrates the use of standard methods of handling uncertainties to evaluate the reliability index (RI) of unreinforced and reinforced soil slopes under simplifying assumptions. The concept of random variables and standard methods of handling uncertainty is briefly explained and applied to the evaluation of RI. The results of a sensitivity study on the effect of partial soil factor and slope angle on the RI are presented for an example slope. They show that where the soil nail reinforcements contribute significantly to the resistance (for example, in steep slopes or where high partial soil factors are applied), the RI is high due to the large safety margin incorporated in the reinforcement design. On the other hand, the RI of flatter slopes may be low if partial soil factors are not applied. Comments on the use of partial soil factors are provided.

INTRODUCTION

It is commonly accepted that the factors of safety of different slopes do not reflect their probability of failure because the inherent variability of geology and soil properties is not taken into account (For example, see Figure 1 of Ho et al 2003). GEO (1987) mentions the use of the Reliability Index (RI), which is a more appropriate indicator of margin of safety, by treating the Factor of Safety (FOS) as a random variable. However, guidelines on its use are not given and this approach is seldom used in practice because of it involves the analysis of uncertainty. This paper first gives a brief introduction to the concept of random variables, and then examines the stability of unreinforeced slopes based on the RI approach. The concept is then extended to soil nailed slopes using the active horizontal force as the stability indicator. Resistance is provided by both the soil and the reinforcement, and they are combined as random variables to evaluate the RI. The results of a sensitivity study on the effect of partial soil factor and slope angle on the RI are presented and comments made.

RANDOM VARIABLES AND UNCERTAINTY

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Random Variables

Random variables are defined by their probability distribution functions (PDF). Common examples of PDF include the Normal distribution, the Poisson distribution, the t-distribution, the Uniform distribution and the Triangular distribution. Two important parameters of a PDF are the mean μ and the standard deviation σ. In this paper the notation "$\mu \pm \sigma$" is used to describe a random variable. If the random variables are Normally distributed, the sum of two or more random variables is also normally distributed with

$$\mu = (\mu_1 + \mu_2 + \mu_3 + ...\mu_n) \quad \text{Eq 1}$$

$$\sigma^2 = (\sigma_1^2 + \sigma_2^2 + \sigma_3^2 + ...\sigma_n^2) \quad \text{Eq 2}$$

Even if the random variables are not Normally distributed, the above equations are still approximately correct, based on the central limit theorem.

Function of Random Variables and First Order Approximate Analysis

Functions of random variables are themselves random variables in the sense that they possess a PDF, although evaluating their actual PDF may involve tedious and complicated processes such as numerical integration or Monte Carlo simulation. However, for a first order approximate analysis using propagation of error approach, simple differential calculus may suffice.

If the angle of internal friction ϕ' of a soil is a random variable, then both tanϕ' and coefficient of active pressure K_a (= $\tan^2(45°-\phi'/2)$) are functions of ϕ' and therefore themselves are random variables.

At $\phi' = 35°$, the relationship between small changes in ϕ', tan'ϕ and K_a is given by

$$\delta\tan\phi'/\delta\phi' = \sec^2\phi = 1.4903 \quad \text{Eq 3}$$

$$\delta K_a/\delta\phi' = -\tan(45° - \phi'/2) \times \sec^2(45° - \phi'/2) = -0.6616 \quad \text{Eq 4}$$

Thus for a first order approximate analysis, tanϕ' = 0.7002 ± 0.02334 (say) corresponds to $\phi' = 35 \pm 0.9°$ and $K_a = 0.2710 \pm 0.0104$.

ANALYSIS

Assumptions for Analysis

The following simplifying assumptions are made in the analysis to illustrate the RI approach.

1. Soil strength is measured by test specimens in the laboratory, and the results are represented by $\tan\phi' = 0.7002 \pm 0.0700$.
2. The strengths measured in the laboratory are unbiased estimates of the strength in situ. Systemic bias caused by various effects such as sample disturbance, degree of saturation, and plane strain etc are not considered in this paper.
3. The slope geometry, geology and loading (eg soil density and the once-in-10-years ground water table) are taken as known and deterministic. (The RI evaluated therefore represents the conditional failure probability for a given event and geology).
4. Soil nails are near horizontal and the total soil nail strength is controlled by the sum of tensile strengths of the individual reinforcement bars. Other modes of failure (eg bond failure) are not the controlling failure mode.
5. Grade 460 Y40 bars with tensile strength = 595.4 ± 29.8 kN are used in the soil nails and this represent the strength of each soil nail. Tensile strength of grout, effects of corrosion etc are ignored.
6. The design load for a grade 460 Y40 bar is 321 kN. [2]
7. The theoretical analysis (eg earth pressure theories used) is taken as exact, with neither bias nor uncertainty.

Therefore the only uncertainty in the analysis is the resistance, both of the soil and of the structural elements. This may be regarded as a "partial" reliability analysis related to resistance only.

Unreinforced Soil Slope Involving a Single Soil Stratum

The normal design practice is to provide the required factor of safety for a specified ground water loading. The current practice is to provide a FOS of 1.4 for a once-in-ten-years ground water level and 1.1 for a "worst credible" ground water condition. Various slip surfaces are chosen for stability analysis, and the minimum FOS obtained from the most critical slip surface.

[2] This corresponds to a FOS of 1.8 on 460 MPa, sacrificial thickness not provided. Structural design requirements vary between codes. For example, BA (1987) and GEO (2005) set a upper limit of 230 MPa for the allowable stress, while GEO (2002) requires a partial factor of only 1.5 for steel.

Consider the most critical slip surface to be of a size such that the total slip surface can be divided into k elements, each equivalent in size to a laboratory test specimen. The actual available total strength of the slip surface is governed by the mean resistance of these k "specimens". This is a random variable with $\tan\phi' = 0.7002 \pm 0.0700/\sqrt{k}$. Normally k is so large that the standard deviation approaches zero and the mean actual strength becomes deterministic with $\tan\phi' = 0.7002$.

The selected strength used for design is obtained, however, from testing n representative specimens in the laboratory. The selected strength will be a random variable with $\tan\phi' = 0.7002 \pm 0.0700/\sqrt{n}$. If the designed FOS is 1.4, the critical value of $\tan\phi'$ is $0.7002/1.4 = 0.5001$, and the RI is given by $(0.7002 - 0.5001) \div (0.0700/\sqrt{n}) = 2.86\sqrt{n}$. If $n = 9$, $RI = 8.58$. Similarly if a FOS of 1.1 is used for the worst credible ground water conditions, $RI = 0.91\sqrt{n} = 2.73$ for $n = 9$.

At this point readers are reminded that in actual design soil strength is not the only uncertainty. Assumption 3 above is highly unrealistic. In actual situation the geology and ground water conditions are the main uncertainties. Overall RI of the slope cannot be improved infinitely simply by testing more soils. In fact, there is little point in testing more than required to define the strength parameters.

Reinforced Slope

In a soil nailed slope or a retaining structure, both the soil and the structural component (retaining structure or soil nails) contribute to the resistance. Consider a soil with $\gamma = 20$ kN/m^3, $\phi' = 35 \pm 2.7°$ ($\tan\phi' = 0.7002 \pm 0.0700$) in the dry condition. If a vertical slope with a height of 10 m is to be supported by a system of soil nails (or a retaining wall), the soil nails will have to be designed for an active force of about 271 kN/m (the actual designed force will depend on the results of testing). If the soil loses its strength completely and behaves as a fluid, the wall will have to be designed for a force of 1000 kN/m. Thus it can be inferred that an implicit assumption in the design is that, the soil would contribute a stablising force of about 729 kN/m, and the outstanding 271 kN/m to be provided by soil nails.

Consider a feature with a width of 10m, and 9 soil specimens are tested to give the mean strength for design. The selected soil strength for design is therefore $\tan\phi' = 0.7002 \pm 0.0700/\sqrt{9}$, or 0.7002 ± 0.02334. This correspond to $K_a = 0.2710 \pm 0.0104$, and 2710 ± 104 kN are to be supported by the soil nails.

The design capacity of a Y40 nail is 321 kN (assumption 6 above), and to provide a design force of 2710 kN, 2710/321 or 8.46 (ie 9) bars are required. The actual resistance

provided by 9 soil nails would be (595.4 x 9) ± (29.8 x √9) = 5358.6 ± 89.4 kN. The soil resistance implicitly assumed in design = 7290 ± 104 kN. The total resistance is therefore 12649 ± 137 kN (Eqs 1 and 2) and the critical value of this resistance is 10000 kN. The RI of this design is therefore 2649/137 = 19.3. The calculations are shown in Table 1.

Table 1 - Component of Resistance and Calculation of Reliability Index for a Vertical Slope

Component	Numerical Value
Soil Resistance (kN)	7290 ± 104
Soil Nail Resistance (kN)	5359 ± 89
Total Resistance (kN)	12649 ± 137
Reliability Index	(12649 – 10000) ÷ 137 = 19.3

Effect of Partial Factor on Soil

Some codes require the use of partial factors on soil strength. For example, GEO (1993) and GEO (2002) both require a partial factor of 1.2 on tanϕ'. GEO (2005) implies the use of partial factors which is the same as the required FOS of the slope. The use of partial factors can be regarded as increasing the required soil nail force arbitrarily in the design, while the (implicitly assumed) soil resistance in fact remains the same. Similar calculations can be made to arrive at the RI.

Effect of Slope Angle

Unlike the use of partial factors, which is applied arbitrarily, the reduction in active pressure with slope angle is real. With decreasing slope angle, the required active supporting force is reduced and the soil resistance implicitly assumed in the design is increased. The total resistance required for equilibrium is still 10000 kN and the calculation of RI is the same as before except that a new expression for K_a is required. In this paper, it is assumed that K_a is reduced by the ratio tanϕ'/tanβ where β is the slope angle, ie,

$$K_{a\beta} = K_a (1-\tan\phi'/\tan\beta) \qquad \text{Eq 5}$$

The values of $K_{a\beta}$ as a function of slope angle and partial factor are shown in Table 2.

To evaluate the standard deviation of $K_{a\beta}$ as a random variable, we have

$$\delta K_{a\beta}/\delta\phi' = (\delta K_a/\delta\phi') \times (1-\tan\phi'/\tan\beta) + K_a\sec^2\phi'/\tan\beta \qquad \text{Eq 6}$$

$$= -0.6616 + .2814/\tan\beta \text{ (at } \phi' = 35°)$$

Table 2 – $K_{a\beta}$ as a function of slope angle and partial factor

Slope angle(°) / $K_{a\beta}$ / $\gamma_{\tan\phi'}$	90	80	70	60	50	40
1.0	0.2710	0.2375	0.2019	0.1614	0.1118	0.0449
1.1	0.3012	0.2674	0.2314	0.1905	0.1403	0.0727
1.2	0.3298	0.2959	0.2598	0.2187	0.1683	0.1005
1.3	0.3567	0.3228	0.2868	0.2458	0.1955	0.1277
1.4	0.3819	0.3482	0.3124	0.2716	0.2216	0.1543

Table 3 shows the values of $K_{a\beta}$ corresponding to $\tan\phi' = 0.7002 \pm 0.02334$.

Table 3 – Coefficient of Active Pressure $K_{a\beta}$ for $\tan\phi' = 0.7002 \pm 0.02334$.

Slope Angle β (°)	90	80	70	60	50	40
$K_{a\beta}$	0.2710 ±0.01040	0.2375 ±0.00962	0.2019 ±0.00879	0.1614 ±0.00785	0.1118 ±0.00669	0.0449 ±0.00513

Results and Interpretation

Calculations covering the range of partial soil factors $\gamma_{\tan\phi'}$ from 1.0 to 1.4 and slope angle β from 40° to 90° have been carried out and the results shown in Table 4. In reporting the results of calculation, fractional values of number of soil nails n required have been retained rather than rounding up to the next integer. Though this is physically impossible, it is used to avoid step functions and to give a smooth transition in the calculated results. Hence the vertical slope with a partial soil factor has a RI of 17.1 using n = 8.46 in the Table 4 instead of 19.3 using n = 9 reported in Table 1.

Table 4 – Results of Reliability Index Sensitivity Study on Example Slope

Slope angle(°) / Reliability Index / $\gamma_{\tan\phi}$	90	80	70	60	50	40
1.0	17.1	16.2	15.0	13.4	11.0	6.17
1.1	20.8	20.0	19.1	18.0	16.2	13.2
1.2	24.2	23.4	22.9	22.1	21.0	19.2
1.3	27.2	26.8	26.4	25.9	25.2	24.5
1.4	29.9	29.7	29.5	29.3	29.1	29.0

The unreinforced slope in the example of this paper has a RI of 8.58, which may be regarded as a "standard" for the purpose of comparison. From Table 4 it is seen that this value

is exceeded for all values in the table except one where the soil nail contribution is least. This is because of the relatively conservative design requirements for soil nails, which raised the RI of reinforced slopes substantially.

CONCLUSIONS AND RECOMMENDATIONS

In this paper, calculations based on standard methods for evaluation of uncertainties have been carried out to show the reliability index approach to evaluate the stability of soil nailed slopes, using horizontal force as the stability indicator. The results showed that reinforced slopes are much more reliable compared to unreinforced slopes. Ho et al (2003) reported that soil nail stabilized slopes are more robust than unreinforced slopes, and all of the 51 failures affecting engineered soil cut slopes reported between the years 1997 and 2001 involved unsupported soil cuts with no structural support. The results of this paper provide a quantitative explanation. On the other hand, one may wonder whether soil nail designs in Hong Kong are too conservative, especially if a partial soil factor of 1.4 is used. The use of the same soil partial factor as the overall FOS of the slope would remove inconsistencies in design concept present in various documents. For example, if an unreinforced slope with a FOS of 1.2 is to be upgraded to 1.4 with the provision of a retaining structure in accordance with GEO (1993), there will be no active force to be supported if a partial soil factor of 1.2 is used. The use of partial soil factor of 1.4 for slopes with low initial FOS, on the other hand, may be unduly conservative when compared to requirements of GEO (1993), which only requires a partial soil factor of 1.2. One way to overcome this difficulty is to specify the required partial soil factor as a function of the initial FOS without reinforcement, say $\gamma_{\tan\phi'}$ = (FOS + 0.1). Another way is to relax the allowable tensile load of the soil nails, and keep the requirement of $\gamma_{\tan\phi'}$ = 1.4. The large FOS for tensile force in reinforcement bars is based on applications in reinforced concrete, where two bars may control the failure of an important beam for the whole building with little additional reserve. Bars in a soil nailed slope, which is greater in number and with soil strengths as additional reserve, are likely to play a less important role and there may be room for relaxation.

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A TRIAL ON THE USE OF STEEL STRANDS FOR THE CONSTRUCTION OF SOIL NAILS

J.M. Shen & Dickson Kung[1]

Abstract: This paper describes a successful trial on the use of 15.2 mm diameter 7-wire strands with fusion bonded epoxy coating for the construction of soil nails in a roadside slope in Hong Kong. Subsequent pullout tests were conducted to confirm the performance of the nails. The steel strands are used mainly to replace the high yield tensile reinforcement bars in the traditional soil nail, with certain distinct advantages which are described in the paper. Three appendices are included in the paper to elaborate on aspects of design, corrosion protection and pullout test results of soil nails related to the trial.

INTRODUCTION

Soil nailing is the most popular method nowadays to stabilise slopes in Hong Kong. The design length of soil nails may vary from a few meters to (occasionally) over 20 meters. They are installed by first drilling a 100 mm diameter hole with percussion drilling, inserting a galvanised high yield steel bar fitted with centralizers (to maintain the bar's central position in the hole), grouting up the hole, and finally concreting the nail head. Traditionally galvanised grade 460 steel bars to CS2 with diameters of 25, 32 and 40 mm are used to provide the tensile resistance, while their flexural and shear resistance are ignored in design. To facilitate installation, the steel bars are normally cut to a maximum length of 6 m (or less if site constraints are high). If soil nails greater than 6 m are specified, couplers will have to be used to connect the bars. This has led to the problems of additional labour and quality control on the tightening of couplers on site, and problems of couplers not meeting the specification. The coupler location is also a potential weak spot in the corrosion protection system, due to various factors such as the cutting of threads after galvanisation; contact of metals from potentially different sources leading to galvanic corrosion; threads providing "hidden" places as source of concentration cell or crevice corrosion (the lower dissolved-oxygen content of water/moisture at these "hidden" places is sufficient to generate electric potentials which hasten the corrosion process); and reduced grout cover. Recently heat shrinkage plastic sleeves are required on couplers to provide additional corrosion protection, leading to further delays in the installation process.

With the problems of couplers mentioned above, it becomes attractive to use strands in soil nails to eliminate the use of couplers and all the associated problems. Strands are

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commonly used in anchors and pre-stressed concrete, but seldom used in soil nails. The immediate concern that comes to mind is whether the strands need to be anchored and pulled straight before grouting. If such a process is required, it would simply mean replacing one installation problem with another and the perceived saving in labour and time would be largely offset. Having examined the stiffness of the strand and the centralizers, and noting that trials on the use of the much more flexible carbon fibre reinforced polymer have been undertaken locally, it is considered that the strand can be installed in the conventional way without first anchoring, provided the centralizers are spaced closely enough, say at 1 m intervals. A trial is planned to study the feasibility of this proposal as well as to establish procedures on site.

PROPERTIES OF THE STRANDS USED

Mechanical Properties

The DSI 7-wire strand is adopted for the trial because it is the only type available with fused bonded epoxy coating fabricated in the factory. It has a diameter of 15.2 mm and an ultimate load of 260 kN. Table 1 shows the mechanical properties of the strand, in comparison with the conventional reinforcement steel bar.

Table 1 – Mechanical Properties of 7-wire Strand and Reinforcement Bar

	7-wire strand	Grade 460 Y25 bar	Grade 460 Y32 bar
Ultimate load (kN)	260	250 (176)$^{\#}$	410 (314)$^{\#}$
Yielding load (kN)	234	226 (160)$^{\#}$	370 (283)$^{\#}$
Ultimate bond strength in grade 30 grout (MPa)	2.75	2.50	2.50
Bond length for full strength in grade 30 grout (mm)	1 920	1 150	1 472
Mass per unit length (kg/m)	1.1	3.8	6.3

( )$^{\#}$ indicate values after deduction of 2 mm sacrificial thickness for corrosion protection.

Comprehensive guidelines for the design of soil nails using conventional reinforcement bars have been issued by GEO (2005). These need be modified only slightly to extend the application to strands, based on Table 1 above. The major issue probably concerns whether sacrificial thickness need to be applied for strands. The relevant suggested design modifications are given in Appendix A.

Corrosion Protection

The strand incorporates the latest fusion bonded epoxy coating technology, which is very common in the US and regarded as “the” standard corrosion protection in non-aggressive environments rather galvanising. Coating thickness over the crowns of the outside wires of

the strand is 0.6 to 1 mm. This is much thicker than the typical epoxy coating thickness for steel reinforcement which, unfortunately, did not perform well in some local projects due to the quality of coating. Thus there are locally some organisations which are reluctant to use them. Similar problems did occur with strands. For a period of time, no coating was applied to the centre wire or the inner surface of the outer wires, on the assumption that the ends of the strand would be properly sealed to prevent ingress of moisture. However, after some fatigue tests, failures induced by fretting corrosion on the uncoated inner surfaces were observed. This led to the development of a strand where the interstices are completely filled with epoxy resin during the same process as the exterior coating is applied. This is the type of strand that is used in the trial.

Unlike protection by galvanisation, epoxy coating is not a sacrificial layer and the coating thickness would not decrease with time. It is also an insulator and the electric current required for corrosion cannot pass through them. It should be a more effective coating compared to galvanisation and the effectiveness of the protection depends essentially on the quality and integrity of the coating. Additional information on corrosion protection is given in Appendix B.

Handling and Installation Details

Strands can be ordered in continuous lengths of up to 30m (in a rolled-up form) without the need for splicing. They need to be "straightened out" overnight before insertion into the drillhole. Their weight per unit length of 1.1 kg/m is only 30% of that of a Y25 bar or 20% of a Y32 bar. They can therefore be handled easily by two people. With no splicing to do, typically a strand can be inserted into the hole within a few minutes. Field cutting, if necessary, can be carried out easily with a wheel saw. A short plastic casing is used to protect the edge of the drillhole during insertion.

THE TRIAL

A proposal to carry out the trial was put forward to the Consultants of LPM Contract GE/2002/02 and approval and cooperation was soon obtained from GEO. The trial involved the installation of four soil nails with strands at feature 11SE-A/C430 at Tai Hang Road near Wang Fung Terrace, a slope that was being upgraded with conventional soil nails at the time. Details of the four soil nails are shown in Table 2. Pullout tests were planned and carried out on all four nails. Two of the trial nails were planned to have normal grouted length of 2m for the standard pullout test, while the other two were planned to have a short grouted length of 0.5m, with the expectation of achieving real pullout. The installation proceeded very smoothly. Insertion was completed within minutes, followed by grouting. The trial was

therefore considered a success. There were, however, some unexpected results from the pullout tests (Shen 2004), which are summarised in Appendix C.

Table 2 – Details of the Soil Nails and Dates of Pullout Tests

Nail Ref	Nominal Length (mm) A	Extension for Pullout Test (mm) B	Actual Grouted Length (mm) C	Stressed Free Length (mm) A+B-C	Inclination of Soil Nail (Deg)	Dates of drilling and grouting	Date of Pullout Test
ET1	12 000	1 050	2 450	10 600	45	2004/08/24 2004/08/26	2004/09/01
ET2	8 000	1 730	2 330	7 400	30	2004/08/24 2004/08/26	2004/09/01
ET3	12 000	1 250	800	12 900	30	2004/08/28 2004/09/01	2004/09/14
ET4	8 000	1 300	700	8.600	30	2004/08/28 2004/09/01	2004/09/17

SUMMARY AND CONCLUSIONS

A trial on the use of steel strands for the construction of soil nails is reported in this paper. The trial is considered successful and the use of strands for the construction of soil nails is regarded as feasible, with the following perceived benefits.

(a) eliminating the use of couplers and all the associated problems;
(b) easier handling and hence shorter construction time;
(c) requiring less working space;
(d) saving of material (due to both higher grade of the steel and less wastage);
(e) larger net nail hole area for easier grouting; and
(f) improved corrosion protection (fusion bonded epoxy coating vs galvanising).

It is therefore expected that it will become a popular method in the future.

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ACKNOWLEDGEMENT

This paper is published with the kind permission of the Head of the Geotechnical Engineering Office and Director of Civil Engineering and Development Department. The kind assistance from Fugro (HK) Ltd, and permission from GEO, to allow carrying out the trial in an LPM site, as well as the technical assistance of Mr Alex Ho of DYWIDAG-Systems International Far East Ltd, are gratefully acknowledged. The opinions expressed in this paper are that of the authors only and do not necessarily represent that of the Company.

APPENDIX A: SUGGESTED DESIGN PARAMETERS FOR STRANDS

GEO (2005) gives very comprehensive guidelines for the design of conventional soil nails using reinforcement bars. Since strands are very similar to bars and they are both made from steel with well known properties in construction, only minor modifications (based on Table 1) to GEO (2005) are required for extension of application to strands. These are proposed in this Appendix.

(a) Galvanisation and sacrificial thickness of steel are not required in view of the better corrosion protection offered by the improved fusion bonded epoxy coating.

(b) Allowable tensile load in the strand should be restricted to 55% of the characteristic strength of the strand or 130 kN.

(c) The ultimate bond stress between strand and Grade 30 grout can be taken as 1.75 MPa.

APPENDIX B: CORROSION PROTECTION METHODS

Shiu & Cheung (2003) give a very comprehensive report on the topic, with special emphasis on galvanisation, which is the current practice. The coverage on fusion-bonded epoxy coating and grout cover is, however, limited in the report. This Appendix provides some additional material on these two methods.

Fusion-bonded epoxy coating

The following is adopted from materials of the website "www.crsi.org" and readers are advised to visit the website for more information.

"Fusion-bonded epoxy coating principally protects against corrosion by serving as a barrier that isolates the steel from the oxygen, moisture, and chloride ions that are needed to cause corrosion. Epoxy coating also has a high electrical resistance, which blocks the flow of electrons that make up the electrochemical process of corrosion. In addition to serving as a circuit breaker, the coating protects in a way that is less obvious: coating reduces the size and number of potential cathode sites, which will limit the rate of any corrosion reaction that could occur. In order for macrocell corrosion to take place, a large area of steel surface is needed to serve as the cathode where oxygen reduction can occur.

Epoxy coatings have been used for almost 45 years in the U.S., starting with oil and gas pipelines. The technology was adapted for, and first used on, steel reinforcing bars on a bridge deck constructed in Philadelphia, Pennsylvania in 1973. Since that time, it is estimated that more than 4.5 million tons of reinforcing steel have been epoxy-coated in North America. This rapid growth is reflected by the widespread support of specifying agencies. A 1988 survey by Babaei and Hawkins reported that virtually every state department of transportation in a region of the U.S. with a corrosive environment has used epoxy coating as a corrosion protection strategy for reinforced concrete bridges.

Until recently, the use of epoxy-coated reinforcement was primarily limited to North America. It is now being used in most parts of the world, including Europe, Asia and the Middle East."

An internal study carried out by DSI also indicates that purposely damaged epoxy coating will lead to initiation of corrosion, but the corrosion does not extend to beyond a small area around the opening. Yeomans (2005) states "One such (corrosion protection) method is the use of coated reinforcement, the two most common coatings in use being fusion bonded epoxy coatings and hot dip galvanizing. The use of fusion bonded epoxy coated reinforcement has been widely reported in the literature and technical press and will not be dealt with further here." However, problems of epoxy coating are also widely reported and special care is needed for transportation and handling to avoid damage of the coating.

Grout Protection

Shiu & Cheung (2003) report that in some overseas practices the grout is assumed to offer no corrosion protection because it may become cracked. This assumption may be too conservative. The grout does not simply act as a barrier which will, of course, be affected by cracking. Most importantly it provides a suitably alkaline environment which passivates the

metal. This occurs as soon as fresh grout is in contact with zinc or steel, and this passivated condition will be maintained for a long period (depending on the hostility of the environment) if the grout remains intact. This passivated state will cease if the pH value reduces (eg due to carbonation), or the chloride content exceeds a threshold.

The following are some examples on the emphasis of the importance of grout and concrete cover on the corrosion protection of the reinforcement. Yeomans (2005) says "There can be little doubt that the most cost effective way to minimize the risk of corrosion in reinforced concrete is to ensure that the cover to the reinforcement is of adequate thickness and that the concrete itself is dense and impermeable". Herbst (1978) says "...it can be seen from the Pourbaix diagram that a steel surface which is in contact with a medium with pH-value between 9 and 13[2] will not be affected by corrosion regardless of what electric potential exists.....The diagram shows why cement grout with pH-values up to 12.6 has proven to be one of the best means of corrosion protection of steel. Fracture of steel rods within a cement grouted anchorage zone has never been reported." Thus the advantage gained with increased grout cover of the strand compared to the bar is considerable.

APPENDIX C: SUMMARY OF PULL OUT TEST RESULTS

The general arrangement of pullout tests is shown in Plate C1. Of the four soil nails, ET1 and ET2 have nominal grouted anchor lengths of 2m and the purpose is to verify their performance under contract specifications. Actual pullout is not expected. ET3 and ET4 have nominal grouted anchor lengths of 0.5m, and the intention is to achieve a true pullout failure. However, actual grout lengths achieved are slightly higher as shown in Table 2. In order to avoid bond failure at the strand/grout interface (estimated bond strength = 120 kN per m of grouted length, and hence would fail at 60 kN for a 0.5 m anchor length), 3 wedge connectors were added to the strand within the grouted length zone, in the hope that this will prevent failure to occur at this interface. The designated loading sequence in kN is 10, 30, 50, 70, 90, 110 (sustain and unload), 130, 150, 170, 190, 210 (sustain and unload). The pullout test results are summarised in Table C1.

ET2 was tested first, and had an unexpected failure of the cement-mortar pad shearing in a near horizontal plane (Plate C2). This is due to a combination of contributing factors, including inadequate age of mortar (less than 4 days), uneven bearing of steel plate onto the pad, reduced bearing area of pad on slope, insufficient extension of reinforcement to the top of the pad and voids in the mortar pad due to embedment of paper. The test stopped at a load

[2] Yeomans (2005), however, reports the passivating pH range to be 6 to 13 for zinc and 13+ for steel.

of 150 kN. ET1 was tested after ET2 on the same day, and the test was completed without any problem. ET3 was tested two weeks later and apparently failure of the wedge occurred. The authors believe that the first failure of the wedges in the anchor zone occurred at about 90+ kN, and the second failure at about 170+ kN. The failures were indicated by a loud "bang" and rapid movement of the dial gauge, indicating pullout. Attempts to achieve a true pullout did not realise and the failure mode cannot be confirmed. The amount extruded with the hydraulic jack, which is not quite suitable for the purpose, is only about 250 mm. On the other hand, ET4 was tested successfully to 190 kN without any signs of failure. Probably the load spread in the wedges of ET4 are more uniform and did not cause failure, while the load spread in the wedges of ET3 is more variable, leading to progressive failure of wedges one by one.

Table C1 – Summary of Pullout Test Results

Nail reference	Nominal Length (mm)	Maximum Load (kN)	Maximum Displacement (mm)	Permanent Displacement after test (mm)
ET1	12 000	210	86.0	13.0
ET2	8 000	150[a]	42.6	6.1
ET3	12 000	170[b]	70.3	~250[c]
ET4	8 000	190	40.4	12.0

Notes [a] Failure of cement-mortar pad [b] Probable failure of bond between strand and grout
[c] Distance pulled before attempt abandoned

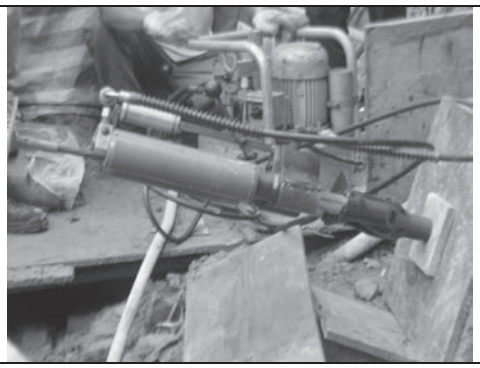

Plate C1– Arrangements for the pullout test

Plate C2 – Failed cement motor pad for ET2

The pullout tests indicate that the soil nails constructed with strands perform up to expectation. Despite the large free lengths, the load displacement curves do not show any sign of slacking, indicating that the construction and installation methods are satisfactory. Complete pullout, though intended, is not achieved even though the grout length is as short as 700 mm. With the benefit of hindsight, perhaps it is not a good idea to plan for pull out with strands. Pullout with a Y40 bar would probably be more appropriate. Even then, a mechanism for full extraction need to be worked out, as hand jacks are not practical for large displacements.

A CASE EXAMPLE OF ACHIEVING GREEN AND SAFETY UNDER THE LANDSLIP PREVENTIVE MEASURES PROJECT IN HONG KONG

C M Wong[1], Chris T L Lee[1] and P L Yip[2]

Abstract: A 10m high cut slope above Oaklands Avenue, between the junction with Lyttelton Road and a sitting-out area, was recently upgraded under the Landslip Preventive Measures Programme in 2003. This slope has a steep sloping angle of about 60 degrees. It is mainly composed of colluvium and has a long history of persistent groundwater seepage at about mid-slope height, as it is intersected by a buried streamcourse. Soil nails were installed as part of the slope upgrading works to strengthen the slope to the required safety standard. During the course of soil nail installation, numerous drillhole collapses and excessive grout loss were encountered. The design was revised by increasing the soil nail inclination angle, shortening the soil nail length and reducing the soil nail spacing. Temporary raking drains were also installed to relieve the ground water pressure prior to the installation of soil nails. An ODEX drill bit with continuous casing was used to prevent the drillhole from collapsing but these attempts were unsuccessful. Finally, the 'drill and grout' technique was adopted to overcome the construction difficulties.

Prior to the slope upgrading works, the subject slope was protected with shotcrete cover with many water stains on it due to persistent groundwater seepage. In order to improve the aesthetic appearance of the upgraded slope, a layer of about 250mm thick fibre-reinforced soil was applied on the slope surface as part of the slope upgrading works, thus allowing growth of vegetation on the steep slope. In addition, a toe planter was constructed along its toe.

INTRODUCTION

Slope No. 11SW-A/C92, located above Oaklands Avenue at the junction of Lyttelton Road, was selected for upgrading under the 10-year Extended Landslip Preventive Measures Programme in 2000. This feature was originally protected with shotcrete cover. Before the upgrading works, it was evident that a high groundwater table existed within the slope, as persistent seepage even with continuous water flow in wet season through existing weepholes was noticed.

The proposed upgrading works for the subject slope comprised the installation of soil nails and raking drains. During the course of the slope upgrading works, frequent collapse of drillholes and excessive grout loss were encountered during the construction of the soil nails in certain areas. In order to overcome the difficulties in the installation of the soil nails, numerous methods were adopted.

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To improve the aesthetics of the upgraded slope, where the existing shotcrete cover was marked with many water stains and unplanned vegetation, the designer decided to provide a 'green' cover to the slope as part of the slope upgrading works. However, owing to the steep slope angle (>55°), the provision of vegetation cover by hydroseeding would not be successful (GEO, 2000). To account for the actual site conditions, a new technique of applying Continuous Fibre Reinforced Soil (CFRS) on the slope surface was adopted.

DESCRIPTION OF SITE CONDITION

Site Description

Slope No. 11SW-A/C92 (see Figure 1 below) was located above Oaklands Avenue at the junction of Lyttelton Road, Mid-Levels. The slope is a soil cut slope of approximately 60m in length. It is standing at a gradient of about 60° with a maximum height of about 10m. Before the slope upgrading works in 2003, the entire slope surface was protected with shotcrete cover and a few large trees were present at crest and towards both ends of the slope. There were weepholes and raking drains in the slope. The weepholes were approximately 70mm in diameter and were staggered in an approximate 1.4m to 1.8m vertical and horizontal spacing pattern.

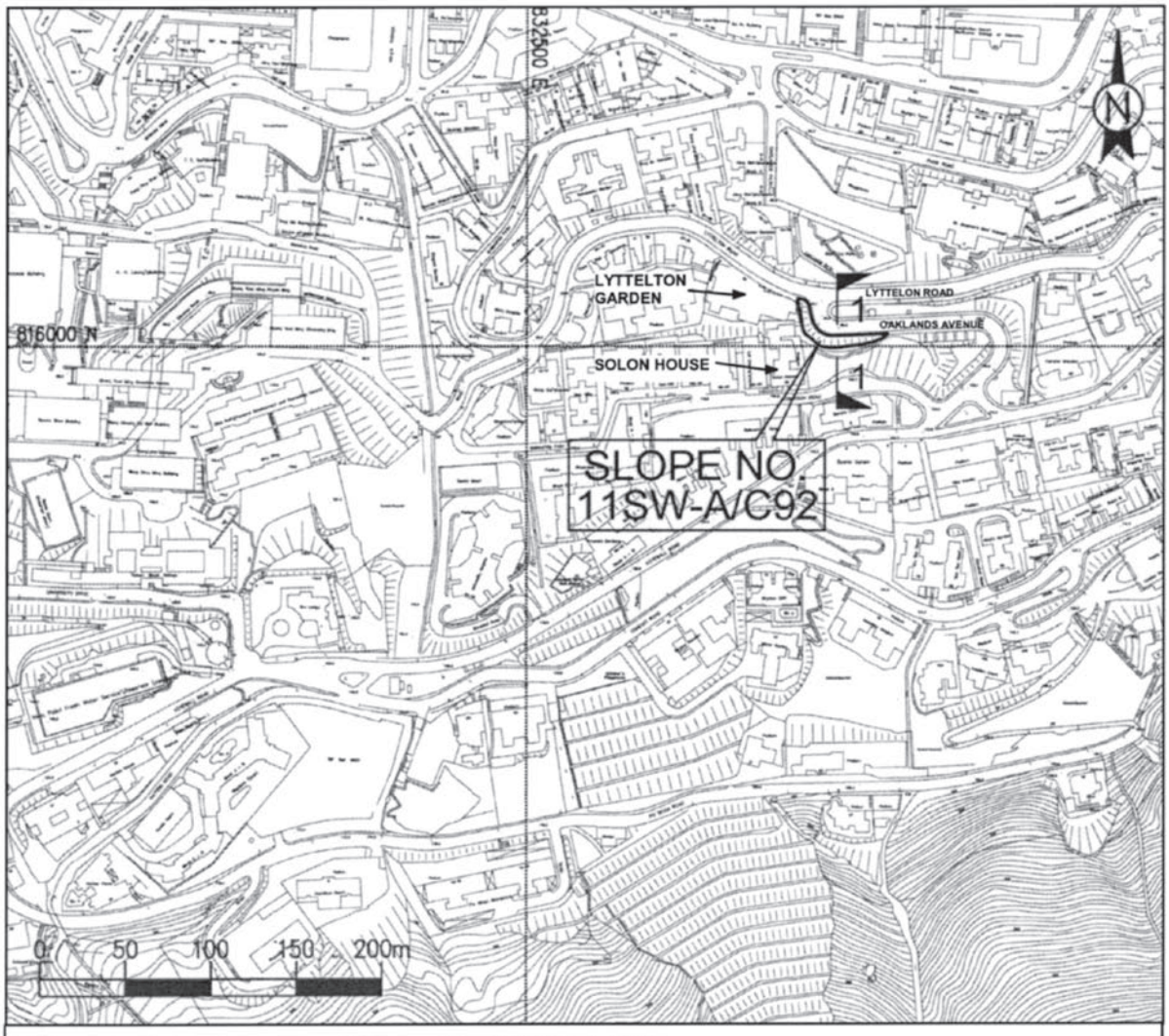

Figure 1: Location plan of Slope No. 11SW-A/C92

Oaklands Avenue runs along the toe of the slope. Lyttelton Garden (17 Lyttelton Road) and Solon House (88 Robinson Road) are high-rise residential buildings located on the western end and south-western crest of the feature respectively. Retaining Wall No. 11SW-A/R613 is located in between the two buildings. An open area currently designated as a sitting-out area with large trees and landscaped vegetation is located on the southern crest of the feature. Slope No. 11SW-A/FR12 is a fill slope located adjacent to the eastern boundary of the subject slope.

Geology

Ground investigation works were carried out in 2001 for the design of the slope upgrading works. The ground investigation works consisted of two vertical boreholes (DH1 and DH1A, one with piezometer), two surface strips (SS1 and SS2), two coreholes (CH1 and CH2) and two trial pits (TP1 and TP2). The ground investigation works revealed that the ground conditions comprise 0.7 - 1.2m thick fill overlying 6 - 7.3m thick colluvium, which in turn overlies thick deposits of Completely Decomposed Tuff (CDT) and Completely to Highly Decomposed Tuff (C/HDT). The locations of the ground investigation works are shown in Figure 2.

A review of the geological setting in the area shows that Slope No. 11SW-A/C92 lies to the west of a fault, Fault E as identified in the Mid-level Study (GCO, 1982) and to the south of a likely granite/volcanic (G/V) contact zone. The approximate locations of Fault E and the G/V contact zone are shown in Figure 2.

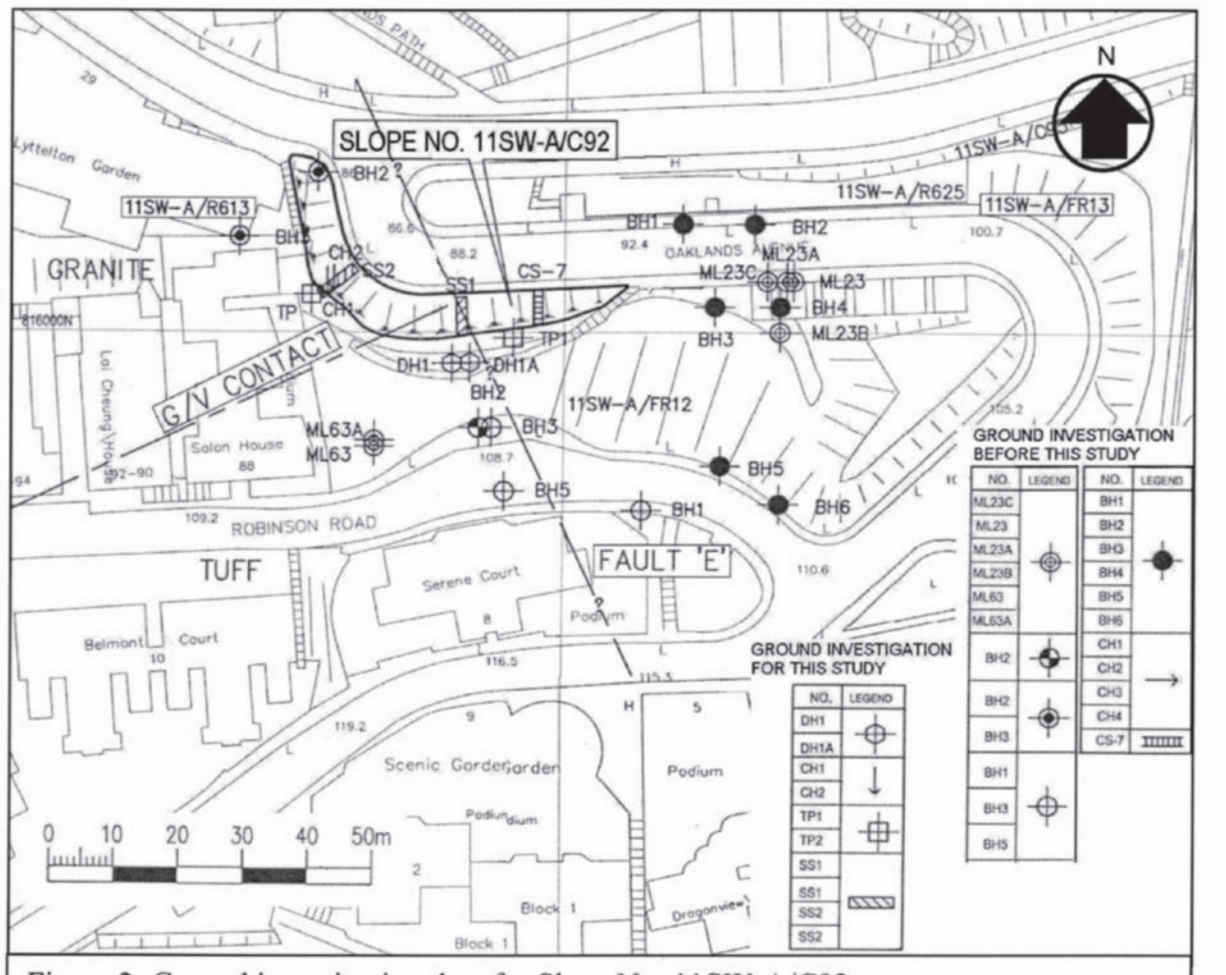

Figure 2: Ground investigation data for Slope No. 11SW-A/C92

Groundwater condition

Groundwater monitoring was carried out over the wet season in 2001 using the piezometer (with Halcrow buckets) installed in drillhole DH1 at the crest of the slope. The monitoring result showed a water level at about 4m above the slope toe. Groundwater monitoring was found from previous ground investigations in Mid-levels Study. A cluster of three piezometers from the Mid-levels Study (ML63A, ML63(U) and ML63(L)) was located in the uphill area south of the feature. The monitoring records of these piezometers showed that Halcrow type buckets were not provided and the measurement of water level was by dipmeter. Maximum piezometric level recorded within the colluvium stratum is in the range of +99 to +100 mPD. The approximate location of these piezometers and their recorded level are shown in Section 1-1 in Figures 1 and 3.

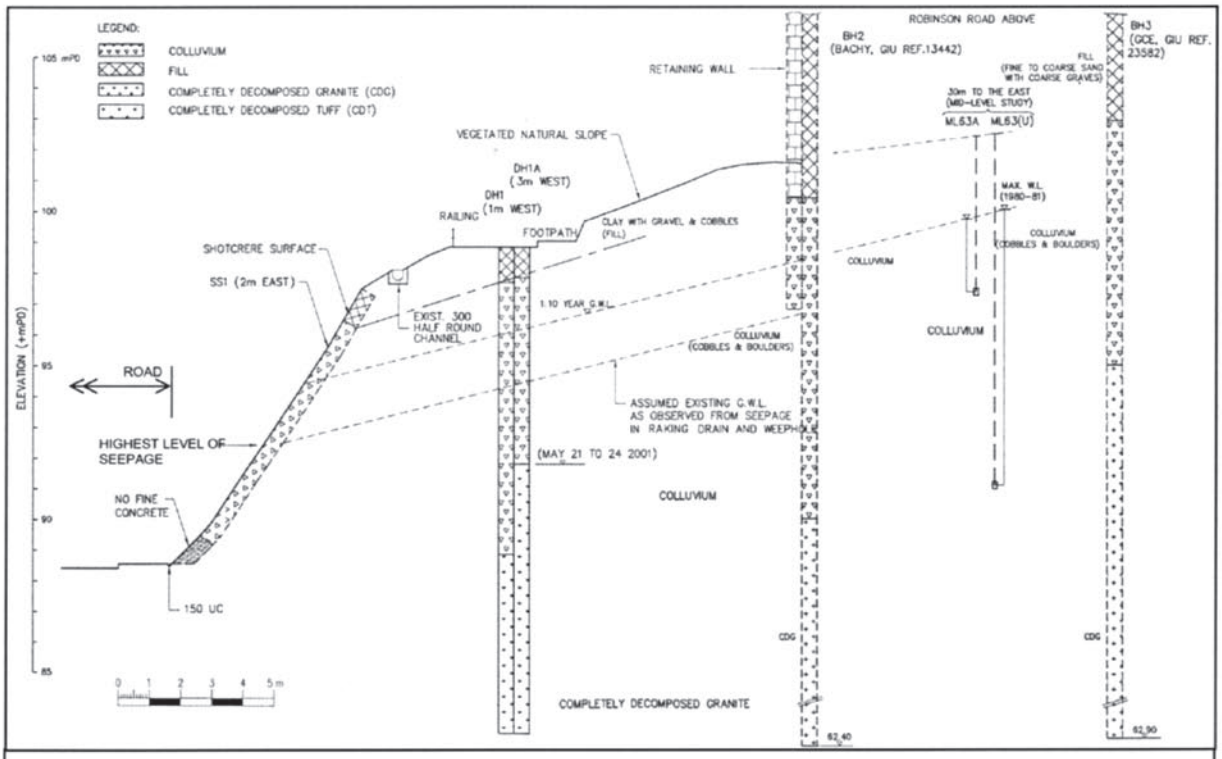

Figure 3: Section 1-1

The feature has a history of persistent seepage in the central portion. Based on site observations between 2001 and 2002, the highest seepage occurred at about mid-height of the slope in the central portion. No groundwater seepage was observed at the western portion of the slope.

Based on HIP (1979) and GCO (1986), the slope straddles across the path of old streams or drainage lines (see Figure 5). As discussed above, the slope lies to the west of a fault. The seepage could be a result of the presence of a zone of high transmissivity associated with the fault. Moreover, both Drainage Services Department (DSD) and Water Supplies Department (WSD) confirmed that there was no leakage of their water carrying services in the vicinity of the feature. The heavy water seepage from the

Figure 4: Water seepage at about mid-height of slope observed in 2002

slope face is therefore attributable to high groundwater level, but not the leakage of water carrying services.

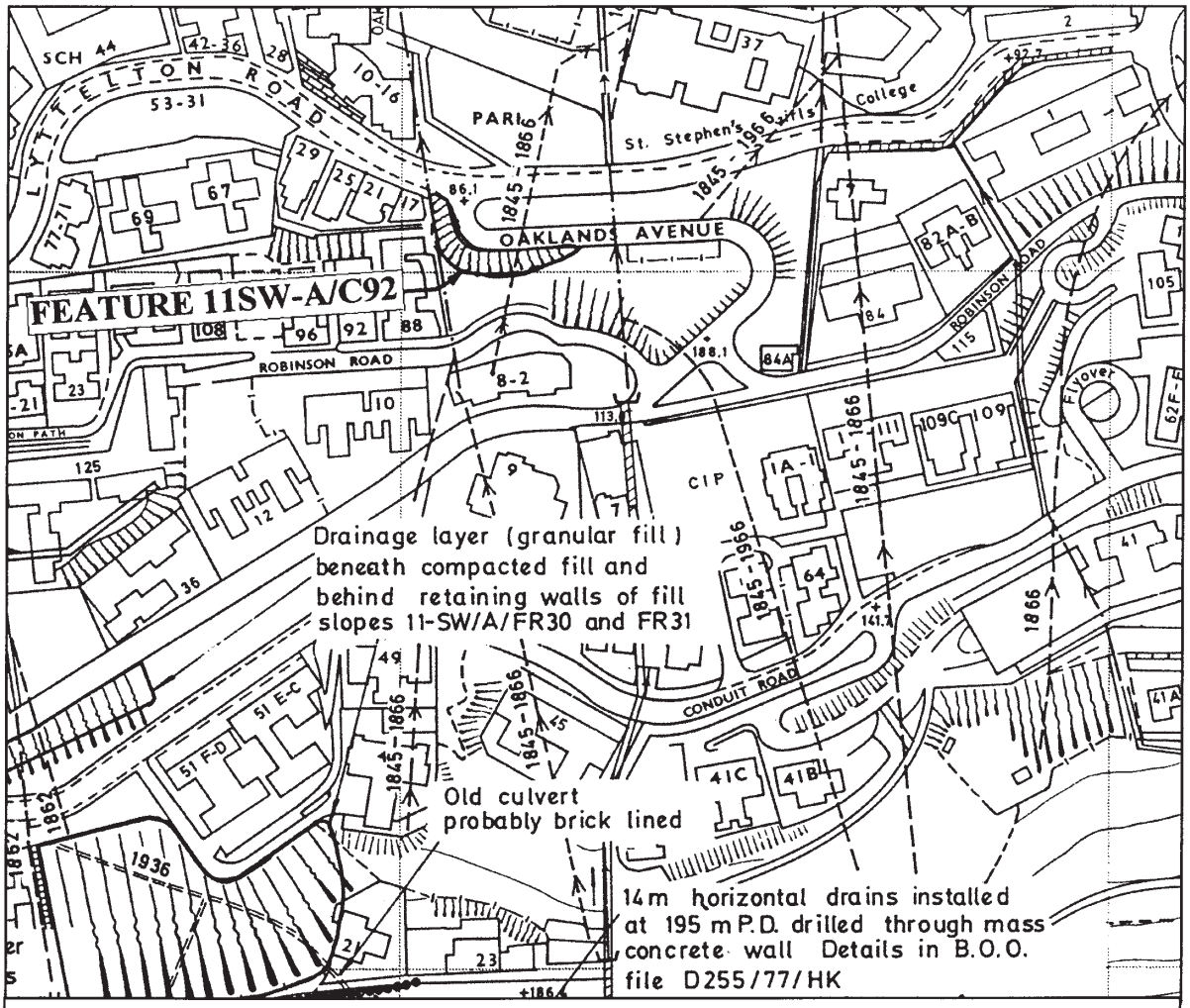

Figure 5: Layout plan of old stream courses extracted from Mid-Levels Study (GCO, 1986)

PROPOSED SLOPE UPGRADING WORKS

Design of upgrading works

The original proposed slope upgrading works comprised the installation of soil nails and raking drains, and construction of surface drainage channels. The original soil nail works involved 233 numbers of permanent nails. In order to facilitate the design of upgrading works for various slope heights, the feature was divided into five zones, Zones A1, A2, B, C and D (see Figure 6).

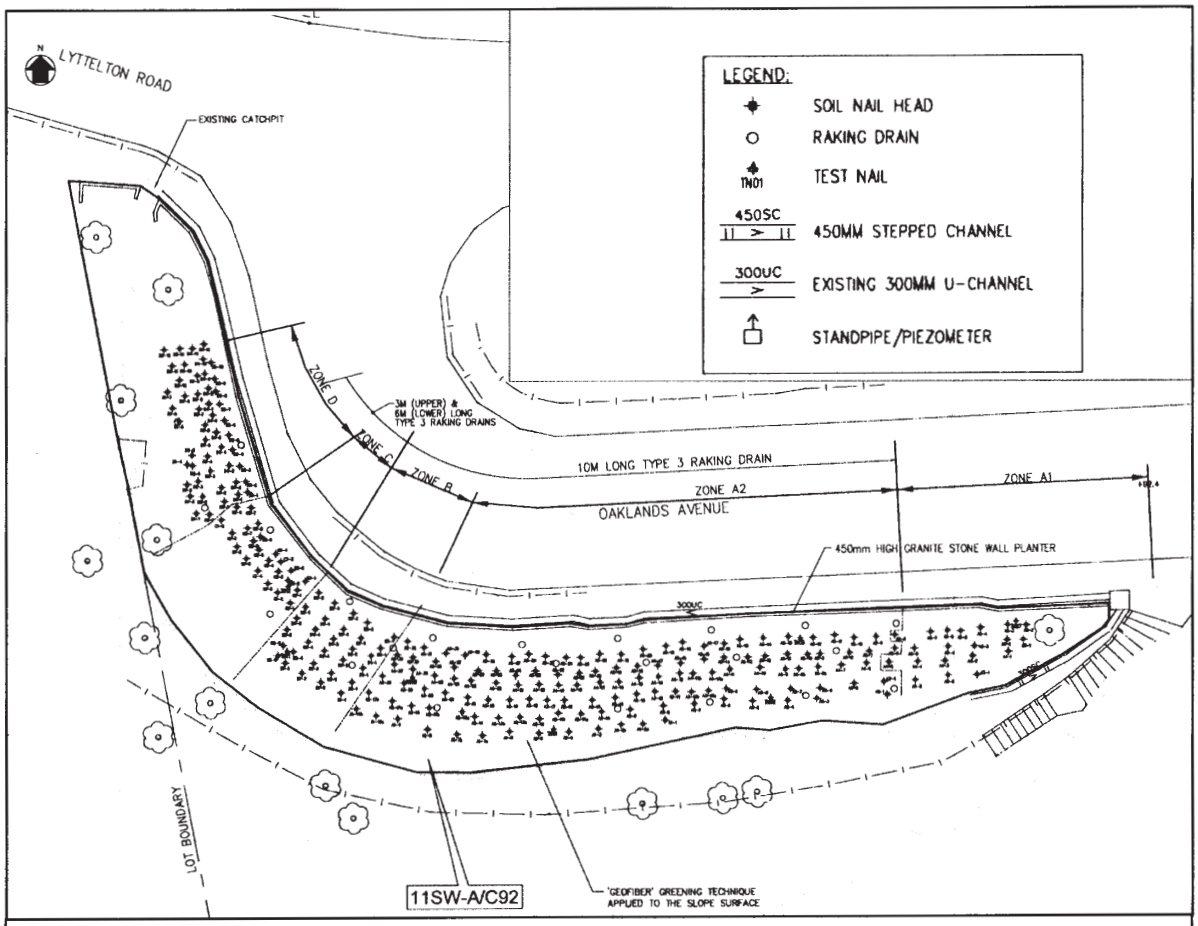

Figure 6: The layout of upgrading works for Slope No. 11SW-A/C92

Proposed Soil nails

Soil nails were installed as part of the slope upgrading works to improve the stability of the slope to the required safety standard. The original design of soil nails in Zones A1 and A2 consisted of 32 mm diameter steel bar in 100 mm diameter drillholes with lengths from 13m to 16m at 1.5m horizontal spacing. The soil nails were inclined at 10°.

The original design of soil nails in Zones B, C and D consisted of 40mm diameter steel bar in 150mm diameter drillholes with lengths from 5m to 12m at 1m horizontal spacing. Owing to the site constraint of the lot boundary of the adjoining lot, for the installation of soil nails in these zones, the nails were inclined downward at 40° to 50° to avoid encroaching into the private lots behind the slope crest.

Proposed Raking drains

In view of the presence of the high groundwater level behind the slope, new raking drains were proposed as prescriptive measures to drawdown the groundwater. Typical raking drains were designed to incline upward at 10° and at 4.5m horizontal and 2.5m vertical in staggered spacings. The existing raking drains on the slope were to be abandoned as grouting for the installation of a new soil nail might block the existing drains.

Proposed Drainage channels

The existing half-round 300mm drainage channel at the crest of the slope was to be retained and a new 225mm U-channel was proposed to replace the existing channel at the toe of the feature.

DIFFICULTIES ENCOUNTERED DURING SLOPE UPGRADING WORKS

During the course of soil nail installation, frequent drillhole collapses and excessive grout loss were reported. Heavy groundwater seepage was also reported and this led to difficulty in grouting with conventional equipment, as water flowed out continuously from the drillholes.

Identifying difficult area

By employing 2 drilling machines initially, the installation of soil nails was commenced in Zone A2 first. Numerous drillhole collapses and excessive grout loss were reported. One drilling rig was then moved to Zone A1 for the installation of soil nails. Occasional drillhole collapses were also reported in Zone A1, while further drillhole collapses were still being reported in Zone A2.

Temporary drains were installed to release the groundwater pressure, however the difficulties in Zone A2 still could not be solved. An additional drilling rig was employed and the installation of soil nails in Zones B, C and D was then commenced. Occasional drillhole collapses were encountered in these zones.

The installation of soil nails in Zones C, B, A1 and D was completed in order. The installation of soil nails in Zone A2 was the last to be completed and frequent drillhole collapses continued to be encountered throughout the installation works. Zone A2 was identified as the most difficult area (see Figure 7).

During the course of upgrading works, water was found to flow out continuously from some of the drillholes, especially from those in the central portion of the slope.

METHODS ADOPTED TO OVERCOME THE DIFFICULTIES

Two major difficulties encountered during the soil nail installation of this feature were:

(1) collapse of drillholes
(2) excessive grout loss due to high groundwater level

In order to overcome the difficulties, a design review was first carried out. Next, trials of various installation methods were made. These methods include provision of temporary raking drains, adjustment of construction sequence, use of casing, 'drill and grout' technique and staged grouting.

Design Review

Apart from the scheme of soil nails installation, alternative scheme of buttress construction was also considered. However, such construction would involve the permanent occupation of the existing footpath, this scheme was therefore decided to be not feasible. As such, the design review was focused on the modification of soil nail design.

The Contractor undertook trials on changing the inclination angle of the proposed soil nails in order to solve the difficulty of drillhole collapse. It showed that by applying the drillhole size and inclination of soil nails in Zone D to the most difficult Zone A2, i.e. changing the angle of drilling to 40° and the drillhole diameter to 150mm, the drillhole was less prone to collapse during the drilling works. This approach then allowed the Contractor to drill the holes up to a maximum depth of 11m. As such, apart from the installed soil nails, the inclination of the remaining nails was changed to 40° with a drillhole diameter of 150mm. The design of upgrading works was therefore revised accordingly. The number of soil nails was subsequently increased from 237 to 251.

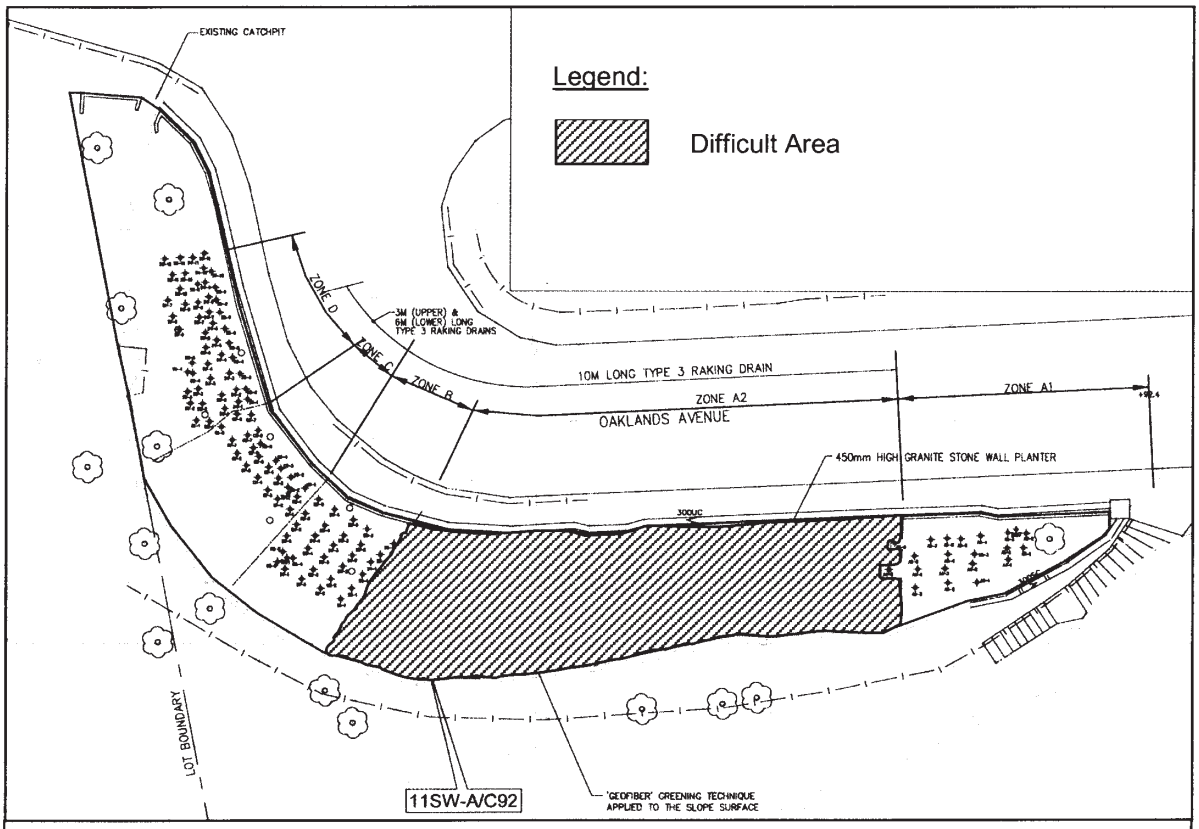

Figure 7: Indication of the most difficult area

Installation of temporary raking drains

One of the key contributing factors for the difficulties reported during the soil nailing works of this slope is the high groundwater level. The grout was drained away by the flowing groundwater. As such, at those locations where persistent seepage was encountered, temporary raking drains were installed. This method proved to be effective in minimising the grout loss during the soil nails installation.

Provision of casing to support the drillholes

After the revision of drillhole size and inclination, the situation was improved, however, collapses of drillholes were still encountered in numerous drillholes. In order to prevent the drillhole from collapsing, the method of providing casing to the drillhole was considered. The drill bits were changed to 'ODEX' type with continuous casing to be provided during drilling works. These attempts however were unsuccessful. As inspected on site, after drilling a few metres into the ground, the casing was found to be stuck in the hole.

The drilling was therefore terminated and the drilling rig required to be relocated. The jamming made the drilling works very difficult.

'Drill and grout' technique

Since the provision of casing was found to be not effective in preventing the drillhole from collapsing, 'drill and grout' technique was subsequently adopted. The details of this method are as follows:

(1) perform drilling works
(2) cease drilling works when drillhole collapse is encountered
(3) insert grout pipe into the collapsed hole and perform grouting works
(4) re-drill the grouted drill holes after 3 days
(5) repeat steps (2) to (4) until reaching the design length of soil nail

Peripheral Pre-grouting works

In some cases, the drillhole could not advance to the required depth even with the 'drill and grout' technique as described above. In these cases, pre-grouting at the periphery of the collapsed drillholes was adopted. Three 150mm diameter grout holes were drilled in the area surrounding the collapsed drillhole at distances of approximately 300mm. These grout holes were drilled using a percussive-rotary drill with air flushing system. The depth of hole was the same as the working nail or to a practicable drilling depth. The drillhole was grouted with normal grout under pressure (about 2 bars to 5 bars).

Staged grouting

Staged grouting was adopted to overcome the difficulty of excessive grout loss. As the existing geology, being colluvium, consisted of soil with much cobbles, the presence of voids or cracks led to excessive grout loss. Grout in voids or cracks would gradually thicken during repeated grouting and hence reduce grout loss. During grouting, when the grout level was detected to remain unchanged, the grouting was stopped for a few minutes. Grouting was then resumed after the grout in the voids or cracks has initially set. This procedure was repeated until the hole was fully grouted.

Performance Review

After the completion of slope upgrading works, continuous groundwater monitoring was carried out for a year to validate the design assumption and to review the effect of the upgrading works on the groundwater condition. Based on the monitoring records, there was no significant variation of groundwater throughout the whole year even after heavy rainfall. The measured highest groundwater table was about 4m above the slope toe, which was similar to the measured groundwater level prior to the commencement of works. The design assumption was therefore considered to be valid and the slope upgrading works was completed satisfactorily.

Summary of methods adopted for soil nailing works

For this site with frequent collapse of drillholes, excessive grout loss and high groundwater level, the installation of soil nails by conventional method was not applicable. Numerous methods as described above were tried. The soil nailing works for this feature were subsequently completed and the completion required a combination of these methods. The following presents a summary of the adopted methods:

(1) Carry out trial soil nail drilling works in the difficult area to choose the optimum drilling inclination and drillhole size such that the drillhole have the

least potential to collapse. Carry out design review to adopt the optimum drillhole inclination and size accordingly.

(2) Install temporary raking drains to release the groundwater prior to the soil nailing works.

(3) As the provision of casings to the drilling works was found to be ineffective, 'drill and grout' technique was adopted.

(4) Where the drillhole cannot advance any further by the 'drill and grout' technique, peripheral pre-grouting work was carried out.

(5) Where excessive grout loss was encountered, staged grouting technique was applied.

LANDSCAPING WORKS

The feature is a steep slope standing at a gradient of about 60°, which is not suitable to support vegetation growth by conventional hydroseeding method. In order to improve the aesthetic appearance of the upgraded slope, a layer of fibre reinforced soil was applied on the existing slope surface, as part of the slope upgrading works. This fibre reinforced soil allows the growth of vegetation on steep slopes. In addition, a toe planter was proposed to be constructed along the slope toe and raking drain outlets were extended down to the toe planter.

Toe wall planter and raking drain outlet

There was continuous water seepage from the raking drains. In order to reduce the water flow on the slope surface, UPVC downpipes were connected to the outlets of the raking drains, thus directing water flow down to the toe planter, which was made of granite blocks and constructed at the toe of the slope (see Figures 8 and 9). A toe drain was installed at the toe planter of the slope to discharge the groundwater to the existing drainage system.

Figure 8: Raking drain outlet

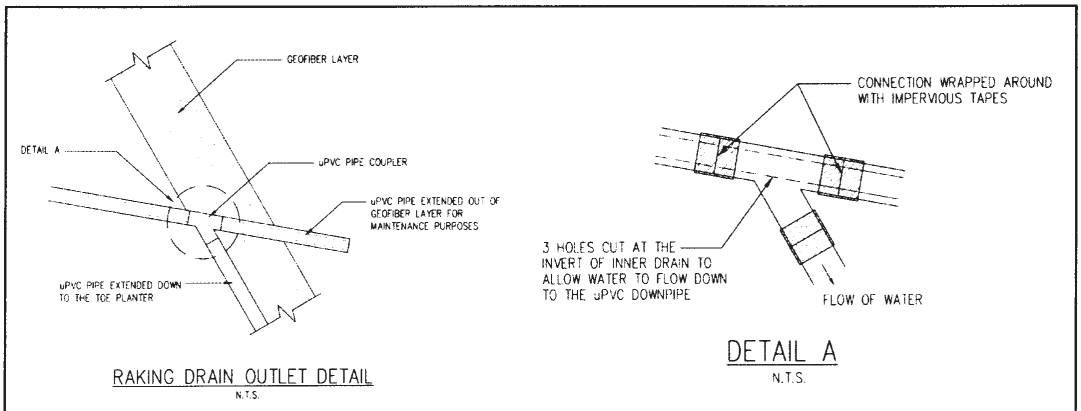

Figure 9: Details of the raking drain outlet

Continuous Fibre Reinforced Soil (CFRS)

CFRS was adopted as it is capable of enhancing sustainable vegetation growth to ultimately achieve the natural order. The inherent internal strength of the CFRS also enables very steep slopes to be more resistant to the effects of erosion. The CFRS was placed in layers and in predetermined directions by water jetting mixed with sand so that they could function as reinforcing materials against tension and so that the shear strength of the sand fibre matrix was increased. The CFRS comprised sandy material and continuous fibre only. No cement based material was used in the continuous reinforced fibre soil as this may hinder the growth of plants and ultimately the re-establishment of the ecological equilibrium of the environment.

In order to install CFRS, the slope surface was first cleaned and the drainage system of the slope was constructed. Holding down anchor bars were placed at 1.5m spacing for receiving the CFRS material. The depth of embedment of the anchor bars was approximately 450mm. Upon completion of the cleaning of the slope surface, drain mats were installed on slope surface. A drainpipe or granular material wrapped with nylon mesh was installed at the toe to collect all subsoil ground water, discharging it to the drainage system at the toe planter.

Prior to the installation of the CFRS material, fertilizer blocks were placed at spread spacings of 1.0m vertically and horizontally to provide nutrients for the CFRS body. The CFRS was then placed from the bottom upwards and hence the self-weight was supported from the base. The placement of the continuous fibre was under water pressure and was simultaneously placed at the same position as the sandy material that was applied under air pressure. The installed CFRS had a thickness of about 250mm (see Figures 10 and 11).

Figure 10: CFRS mixed with sand

Figure 11: The CFRS placed from bottom upwards

Planter holes were dug and plants (Wedelia trilobata) were installed. Wire mesh was placed on the top of CFRS surface and fixed using Geopins. When the mesh was secured, a vegetation base of at least 30mm thick with grass cover seeds and fertilizer was installed on the surface of CFRS (see Figures 12 and 13).

Figure 12: Installation of vegetation base

Figure 13: Feature 11SW-A/C92 after completion of CFRS installation

Appearance after works

The construction of landscape works started in August 2003 and was completed in September 2003. The slope surface was covered by grass in November 2003. Photographs of the slope before and after the construction of landscape works are shown in Figures 14 and 15.

Figure 14: Slope No. 11SW-A/C92 before the slope upgrading

Figure 15: Slope No. 11SW-A/C92 after the slope upgrading works in November 2003

CONCLUSION

The upgrading works of the slope comprised soil nails, raking drains and drainage channels. Collapse of drillholes and excessive grout loss occurred frequently during the installation of soil nails. Soil nail design and construction sequence were amended and adjusted to facilitate the drilling works. Temporary drains were provided to release groundwater pressure and hence the risk of collapsed drillholes. For drillholes in which collapse was still encountered, 'drill and grout' technique was adopted. 'Staged grouting' was adopted for overcoming the difficulty of excess grout loss.

CFRS and a toe planter were applied to improve the aesthetic appearance of the steep upgraded slope. Seepage on the slope surface was diverted to the drainage system at the toe planter. A green slope surfacing was achieved after the landscape works.

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ACKNOWLEDGEMENTS

This paper is published with the permission of the Head of the Geotechnical Engineering Office and the Director of Civil Engineering and Development, the Government of the Hong Kong Special Administrative Region. The authors acknowledge particularly the many helpful discussions, comments and suggestions they have received from Messrs. C C Mak and P C Wong.

A BALANCE BETWEEN SLOPE GREENING AND CARING FOR STAKEHOLDERS

P L Yip, P C Wong and C K Chow[1]

Abstract: There are increasing demands from the public for a green surfacing on our man-made slopes. The burning issue in the geotechnical engineering practice in Hong Kong in these days is to create green and safe slopes for the community. It is the policy of the Government of the Hong Kong Special Administrative Region (HKSAR) to have man-made slopes as natural as possible. A departmental Vetting Committee on Slope Appearance has been set up to review the need for hard slope surfacing as part of the slope upgrading works for Government maintained slopes. However, the seemingly most obvious option of having soft vegetation with attractive landscape treatment to the final slope surface sometimes does not have to be the inevitable choice. On several occasions, stakeholders objected to any vegetation on part or even whole of the slope surface, and prefer other eye-catching hard slope surfacing or without any planting because of some specific concerns such as hill fire, security, hygiene and mosquito breeding, etc.. Geotechnical engineers run an open door policy and are willing to align the expectation from the public at large with the stakeholders' or end-users' concern. The Paper will share some of our experience as to how to come up with a workable solution: striking a balance between providing an aesthetic appealing slope surface and meeting the expectation of the end-users. Two examples, namely slope upgrading works behind village houses and inside the areas under the control of Correctional Services Department (CSD), will be presented to illustrate how the designers, after thinking long and hard as well as applying cross-disciplinary knowledge, can improve the appearance of the man-made slopes and blend such slopes into their surrounding, notwithstanding that soft vegetation is not adopted.

INTRODUCTION

For slope upgrading works carried out by the Government of the Hong Kong Special Administrative Region (HKSAR), it is a policy that the man-made slopes have to be as natural as possible and a departmental Vetting Committee on Slope Appearance has been set up to review the need for hard slope surfacing as part of the slope upgrading works. The following illustrate two examples at which the designers of the Landslip Preventive Measures (LPM) Project work out the solution to strike a balance between providing an aesthetic appealing slope surface and meeting the expectation of the end-users. For private developments, the Authorized Persons and Registered Structural Engineers (APs/RSEs) are advised, through Practice Note for Authorized Persons and Registered Structural Engineers (PNAP) No. 270 (BD, 2004), that the use of shotcreting or other hard slope surfacing should only be considered as a last resort and only after other techniques have been explored and found not practicable, and if used, it should be avoided over the bottom 3m of a slope, as this

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is the area most visible to passers-by. It is also advised that, where the use of shotcreting or other hard slope surfacing is unavoidable, measures, such as toe wall planters, planter boxes, tree pits, creeper holes on the slope surface and colour/pigments added to the shotcrete, should be implemented to improve the visual appearance of the slope for blending in with the surroundings. Furthermore, it is suggested that tree rings of sufficient size should be provided on the shotcrete or other hard surfacing to cater for the continued healthy growth of trees present on the slope.

VETTING COMMITTEE ON SLOPE APPEARANCE FOR GOVERNMENT SLOPE FEATURES

A departmental Vetting Committee on Slope Appearance has been set up to regularly review and endorse the need for shotcreting or other hard slope surfacing to be carried out for slopes upgraded or maintained by the Government. Civil Engineering and Development Department (CEDD) Technical Circular No. 15/2004 (CEDD, 2004) provides guidelines for making submissions to this Vetting Committee. The Vetting Committee review and endorse the need for and details of any slope shotcreting or other hard surfacing to be carried out as part of maintenance or upgrading works to existing slopes or slopes formed in new projects, such as the LPM works delivered by the Geotechnical Engineering Office (GEO) of the CEDD. The scope of the review covers not only first-time shotcreting of or application of any other hard surfacing to an existing or new slope, but also as renewal of an existing hard surface cover on a slope. It is the policy of the Government to have man-made slopes as natural as possible. To justify the use of hard surfacing, the submissions of all hard surfacing proposals have to include, *inter alia*, detailed justifications together with a plan showing extent of the area to be hard-surfaced, the proposal for landscaping and mitigation measures to reduce visual impacts and to help blend the slope into its environment, information on the hard and soft landscape mitigation measures. Furthermore, if the justifications for hard surfacing are based on objection to soft landscape treatment from or preference to hard surfacing of the stakeholders, end-users or nearby residents, documentary evidence has to be submitted to the Vetting Committee.

SLOPE UPGRADING WORKS BEHIND VILLAGE HOUSES

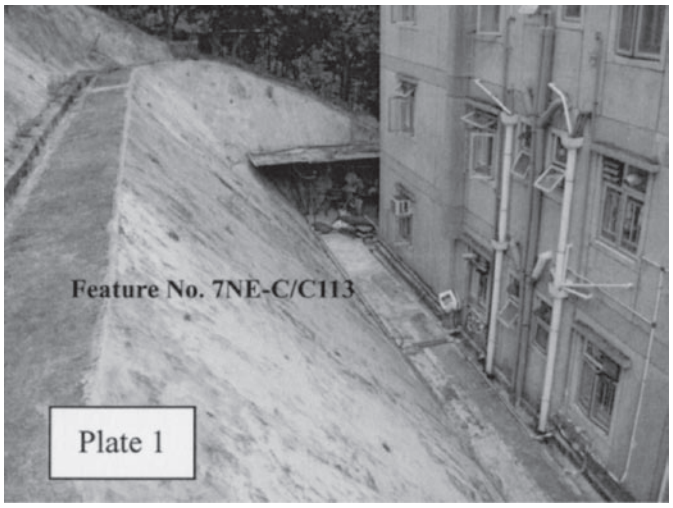
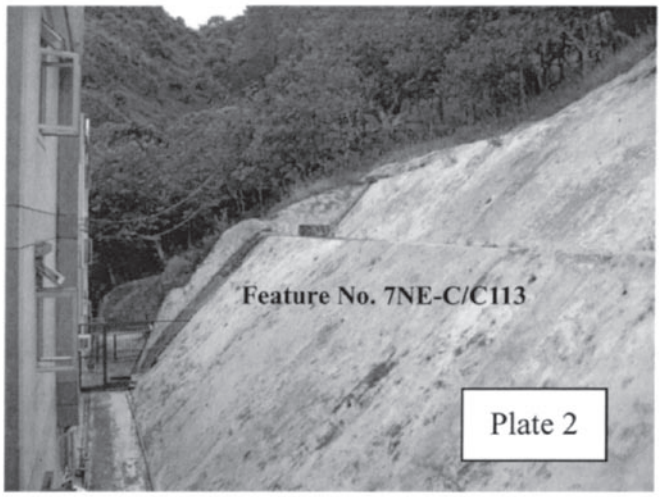


Plate 1

Notwithstanding the stringent criteria and restriction imposed in the adoption of hard surfacing for Government maintained slopes, experience is still obtained on several occasions when the nearby occupants of village houses objected strongly to any vegetation provided on part or even whole of the slope surface behind their home during the slope upgrading works.

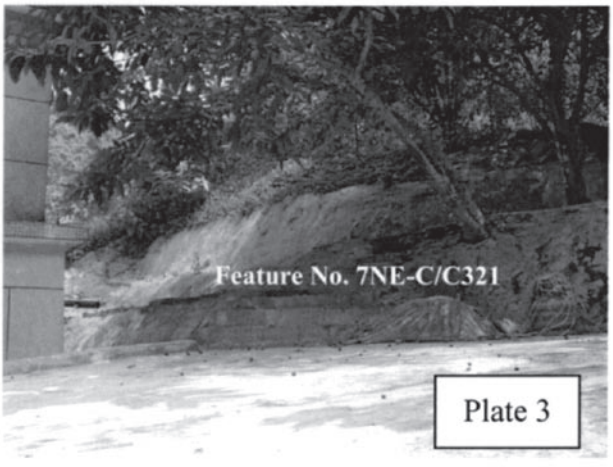
Both Feature Nos. 7NE-C/C113 (see Plates 1 and 2) and 7NE-C/C321 (see Plate 3) are cut slopes with hard surfacing before the carrying out of slope upgrading works and are located in Tai Po Mei Village, Tai Po. A number of village houses are situated at their toes.



Feature No. 7NE-C/C113 is an approximately 40m long cut slope with a concrete cover. The western portion (behind two village houses) of the subject slope is approximately 20m long with a maximum height of 9m inclined at 45° to the horizontal. A 1.4m wide berm (including an existing 220mm U-channel) is located at mid-height on this portion. The eastern portion (behind another village house) is about 20m long with a maximum height of approximately 7.5m, inclined at between 40° and 70° to the horizontal. The village houses are situated at its toe are three-storey residential buildings, whilst a vegetated natural hillside slope inclined at 20° to 30° to the horizontal is situated immediately behind its crest. There are some trees on the eastern end and beyond the crest of the subject slope.

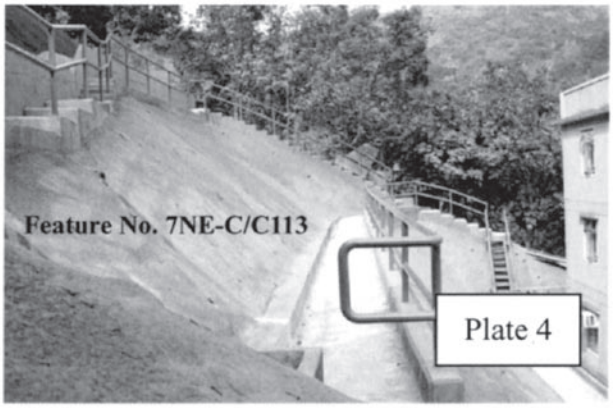
Feature No. 7NE-C/C321 is a soil cut slope with its lower part covered by shotcreting. It is about 15m long and with a maximum height of 5m. The lower batter of the subject slope is generally inclined at 60° to the horizontal, whilst the inclination of its upper batter is about 45° to the horizontal.

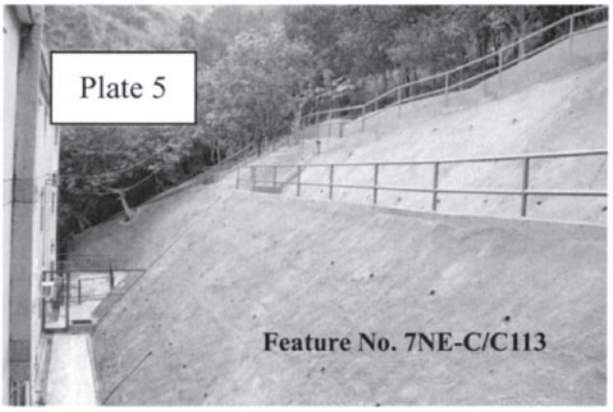
The ground conditions of Feature No. 7NE-C/C113 comprise colluvium overlying completely decomposed siltstone. The completely decomposed siltstone is described as firm to stiff slightly clayey, sandy SILT with occasional angular fine to coarse gravel. For Feature No. 7NE-C/C321, it comprises mainly of colluvium, which is described as firm, dark yellowish brown, mottled light yellow, dark pink and dark grey slightly sandy silty CLAY with some angular fine to coarse gravel and cobble of moderately weak to moderately strong, highly and moderately decomposed siltstone.



These slopes were intended to be made as natural as possible during the slope upgrading works. However, despite repeated and prolonged attempts to convince the nearby occupants of village houses to have soft vegetation options to the final slope surfaces upon the completion of the slope upgrading works, the Village Representative (VR) and, in particular, residents living in the village houses at the toe of these slopes objected strongly to any vegetation on any part of the slopes. Questionnaires written in Chinese, with sketches showing different types of surface protection including the greening proposal, were sent to the VR and residents living in the village houses at the toe of these slopes and they unanimously replied that they preferred hard surfacing such as shotcreting or stone pitching, rather than any kinds of soft landscape treatment. Various follow-up meetings were held so as to come up with a workable solution: striking a balance between providing an aesthetic appealing slope surface and meeting the expectation of the end-users.

After due consideration and lengthy consultation, the final slope upgrading works involve removing the existing concrete cover, installing soil nails and applying coloured shotcreting to the entire slope (in turtle green colour code 10B25 to BS5252F: 1976) were worked out.





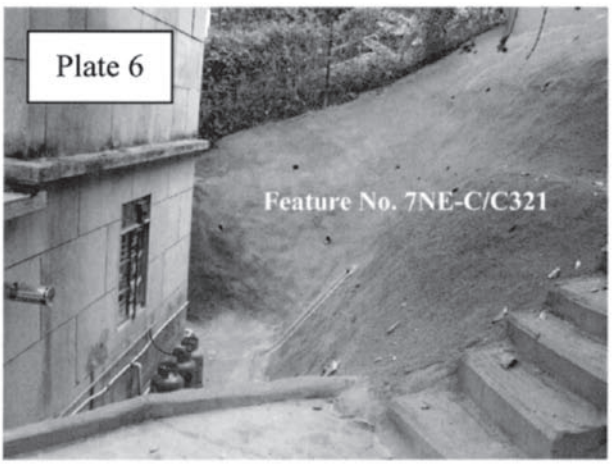
The villagers' concern and request, and the revised design together with the following justifications were made to the Vetting Committee for their assessment:

(a) there is a relatively low visibility from the public point of view as the slopes of concern are located immediately behind 3-storey high residential houses; and

(b) due to the close proximity of the residential houses at the toe of the slopes, hygiene and mosquito concerns expressed by the villagers will be effectively mitigated by the provision of a shotcreting cover to the slopes.

The above justified the choice in the surface cover. It is considered and strongly believed that the coloured shotcreting option would be one of the most effective measures to mitigate the appearance to some extent. We also believe that the views from the nearby occupants of village houses are not being abused, but on the contrary, we have tried to listen very carefully to their concerns, to pay respect to their objections and to come up with an acceptable and workable solution.

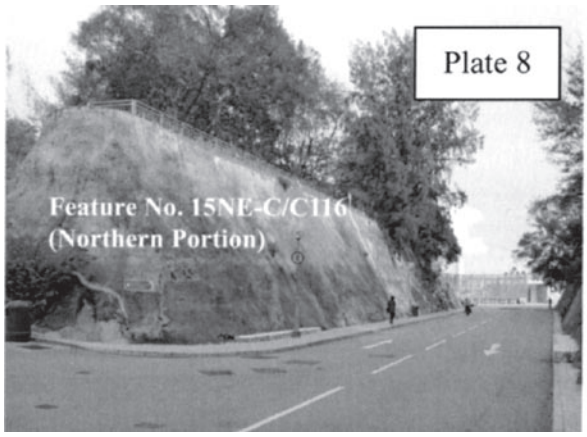
The completed works carried out at these slopes are shown in Plates 4, 5, 6 and 7.



SLOPE UPGRADING WORKS INSIDE AREAS UNDER CONTROL OF CORRECTIONAL SERVICES DEPARTMENT

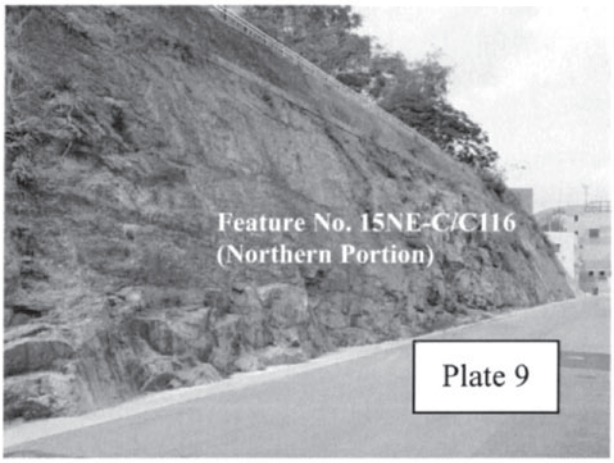
Slope upgrading works were carried out inside the area under the control of the CSD and they objected to any vegetation provided on the slope surface.

Feature Nos. 15NE-C/C116 and 15NE-C/C118 are soil and rock cut slopes located on the east side and west side of Tung Tau Wan Road at the entrance to Stanley Prison.



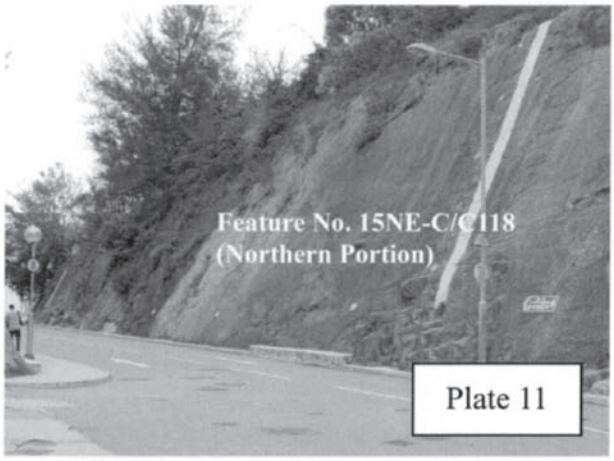
Feature No. 15NE-C/C116 (see Plates 8 and 9) is located above a footpath on the eastside of Tung Tau Wan Road and below a staff quarter of the CSD at Stanley Prison. The slope is divided into four portions, namely a northern portion (facing west) along the footpath of Tung Tau Wan Road, an L-shaped central portion above an open carpark and a gas regulating room, a southern portion (facing south) along Tung Tau Wan Road leading to Pak Sha Wan Correctional Institution and an eastern end portion above an electric substation.

The northern portion of the subject slope has a length of 80m and a height of 8m. The central portion has a length of 40m and a height of 9m. The southern portion has a length of 85m and a height of 9m. The eastern end of the subject slope has a length of about 18m and a maximum height of 6m. The gradient for the entire slope is 60° to 65° except the eastern end where its gradient is about 45°. At the crest of the slope is a vegetated open space in front of a two-storey staff quarter of Stanley Prison.

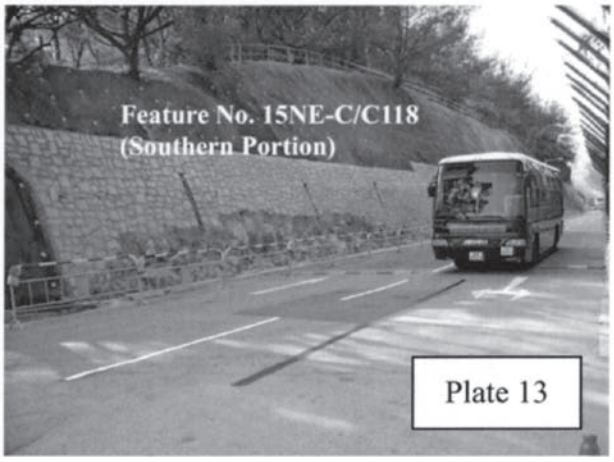


Feature No. 15NE-C/C118 (see Plates 10 and 11) is located along the west side of Tung Tau Wan Road at the entrance of Stanley Prison. The slope is divided into two portions, namely an east-facing portion and a south-facing portion. The east-facing portion of the subject slope has a length of about 120m, a maximum height of 13m and an average gradient of 65°. This portion has two areas of rock outcrops at the southern and northern ends. The south-facing of the subject slope has a length of about 30m, a height of 6m and a gradient of 30° to 35°. At the crest of this slope is a gently terrain with grasses and trees.

For Feature No. 15NE-C/C116, the cut slope (i.e. the majority) portion comprises 2m to



8m completely decomposed granite followed by completely to highly decomposed granite, overlying moderately decomposed granite. The minor fill slope portion at the eastern end comprises up to 6.5m of fill overlying completely to highly decomposed granite. For Feature No. 15NE-C/C118, the slope generally comprises a thin layer of residual soil (about 1.5m to 2m) followed by completely decomposed granite, and completely to highly decomposed granite overlying highly to moderately decomposed and moderately decomposed granite.



Installation of soil nailing has been recommended for upgrading both slopes. For landscaping amelioration, it was originally suggested to replace the existing hard surfacing with soft landscape treatment for both slopes. Both slopes are located at the main entrance of Stanley Prison and that the areas along their toes are easily accessible by inmates as well as visitors. Our stakeholder, the CSD, concern that soft landscape treatment at these slopes could be used to hide unauthorized materials such as small weapons, drugs, etc. and, therefore, requested the use of hard surfacing to the entire slope surface. To adhere to the Government's policy to green all slopes as far as practicable, various meetings with the CSD were then held to achieve a balanced proposal to only provide hard surfacing to the lower portion of the slopes which may be easily accessible by inmates and visitors, and soft landscape treatment be provided to the remaining upper portion of the slopes to provide a natural outlook. It was finally agreed that shotcreting with random pattern granite stone facing be provided to the lowest 4m of the slopes of concern and soft landscape treatment with hydroseeding be provided to the upper portions of slopes. A height of 4m is considered by the CSD as "easily accessible" taking into account of their normal operation. The approximate areas of the hard surfacing to the lowest 4m of Feature Nos. 15NE-C/C116 and 15NE-C/C118 are about 760m^2 and 550m^2 respectively and the approximate areas of the area with soft landscape treatment to upper parts of slopes are about 900m^2 and 1300m^2 respectively.

The upgrading works of Feature No. 15NE-C/C118 are nearing completion and are shown in Plates 12 and 13. The upgrading works at Feature No. 15NE-C/C116 are still under progress and is shown in Plate 14.

CONCLUSIONS

Great effort is taken in "greening" the slopes by all parties of concern. Special considerations have to be made for special cases. For example, for steep slopes at which the provision of hydroseeding being impracticable, special techniques such as mulching, fibre reinforced soil, etc. might be chosen by the designers as an alternative of soft landscape treatment. However, the seemingly most obvious option of greening of the final slope surface sometimes does not have to be the inevitable choice. Some of our experience as to how to come up with a workable solution of striking a balance between providing an aesthetic appealing slope surface and meeting the expectation of the end-users was shared. It turns out that the appearance of the man-made slopes could still blend into their surrounding; notwithstanding that soft landscape treatment could not be adopted for some reasons.

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BD (2004). *Practice Note for Authorized Persons and Registered Structural Engineers No. 270.* Buildings Department, the Government of the Hong Kong Special Administrative Region.

CEDD (2004). *Civil Engineering and Development Department Technical Circular No. 15/2004.* Civil Engineering and Development Department, the Government of the Hong Kong Special Administrative Region.

ACKNOWLEDGEMENTS

This paper is published with the permission of the Head of the Geotechnical Engineering Office and the Director of Civil Engineering and Development, the Government of the Hong Kong Special Administrative Region. The authors acknowledge particularly the many helpful discussions, comments and suggestions they have received from Messrs. Stellan Sum, Dale Tung, Felix Yau, T H Yu and W K Yuen.

BEHAVIOUR OF TWO SOIL NAILS UNDER PULL-OUT TEST

Tony C T Cheung[2], Terence C F Chan[1], S L Chiu[2] and Charles K L Tang[2]

Abstract: The behaviour of two soil nails under pull-out tests is reported. The set up and testing procedures for the tests followed the recommendations recently issued by GEO for use in LPM works. The steel bars of the test nails were instrumented with strain gauges for bond strength measurement. Ground investigation drillholes were sunk in close proximity to the testing locations. SPT and pressuremeter tests were performed at the bond section levels of the test nails.

Two different types of inflatable grout-packers, namely a proprietary product and a locally developed one were used in the tests with a view to ascertaining the effectiveness of the packers in forming the grouted bond sections.

The pull-out tests demonstrated that the recommended testing set-up is practical and easy to implement. With the aid of a suitable packer, the bond section of the test nail could be grouted to the required integrity and to the required length with little grout leakage to the free length.

The correlation between the measured bond strength and the SPT-N value / soil types was found comparable to those presented in the codes and standards of other places such as China, Japan, the United States and France.

INTRODUCTION

New set up and testing procedures for pull-out test in LPM works were proposed in the Design Technical Guideline No. 11 (DTG 11) issued by the LPM Division 2 of GEO in May 2004 (GEO 2004). The proposed set up was designed to minimise the friction loss in the load transfer at the nail head and the new testing procedures specified load increments relating to the design working load (DL), two times DL and the ultimate load of the soil nail. Two trial tests following the recommendations were carried out on an LPM works site at Fanling in August 2004 (MGS 2004). This paper presents the results of the pull-out tests on the two trial soil nails (Test Nail A and Test Nail B).

The Works Site

The testing site was an active LPM works site near Fan Kam Road, Fanling. The feature is a 55^{o} cut slope of about 7.5m in height and 140 m in length. The slope is composed of completely to highly decomposed tuff. Immediately above the feature is a natural slope of about 40^{o}.

The LPM works on the feature and part of the natural slope above it included installation of soil nails at 2m intervals in both vertical and horizontal directions and of lengths ranging from 6 to 18m.

1. LPM Division 2, GEO, CEDD
2. Maunsell Geotechnical Services Limited

Ground Investigation

To provide site-specific geological information, two drillholes were sunk close to the test nail locations. They were BH1 near the Test Nail A and BH2 near the Test Nail B (Figure 1).

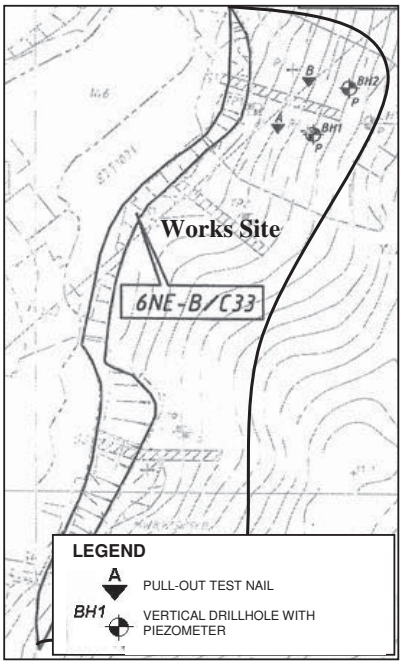

Fig. 1 G.I. and Test Nails Location

Both drillholes were sunk to a depth of about 20 m below ground, ended in completely decomposed tuff (CDT) with SPT-N values taken at 1.5m intervals. At the depth of the bond section of the test nail, in-situ pressuremeter test was carried out.

The ground investigation results showed that the SPT-N values increased with depth, ranging from 7 (BH1) and 22 (BH2) at 1m below ground to more than 200 at the end of the drillholes. At the level near the bond section of Test Nail A, the average SPT-N value was about 75, whilst it was about 40 for the Test Nail B.

In the pressuremeter test in BH1, the cavity pressure and the yield pressure were 4.52 and 16.49 bars respectively; and the test was ended with a pressure of 24 bars. The cavity and yield pressures looked unreasonably high indicating the possibility of encountering local hard material such as gravels. In the case of the test in BH2, the cavity pressure and the yield pressure were 2.48 and 4.47 bars respectively. A shear modulus (G) of 73.5MPa was estimated from the pressuremeter test results.

The SPT-N values and the shear modulus indicated that the soil at the bond section of both test nails was dense to very dense. Figures 2 and 3 summarize the general information about the test nails and the ground conditions.

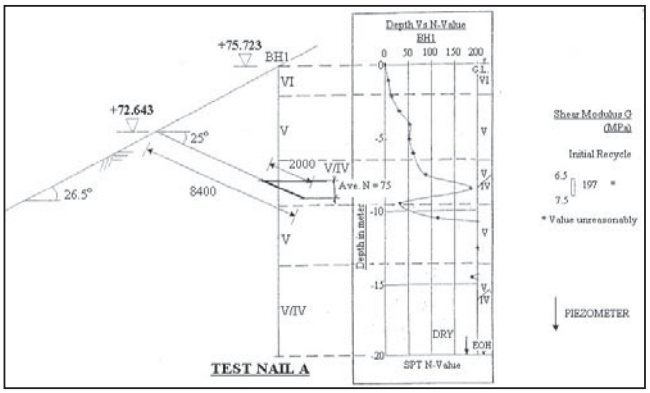

Fig. 2 Ground conditions at Test Nail A

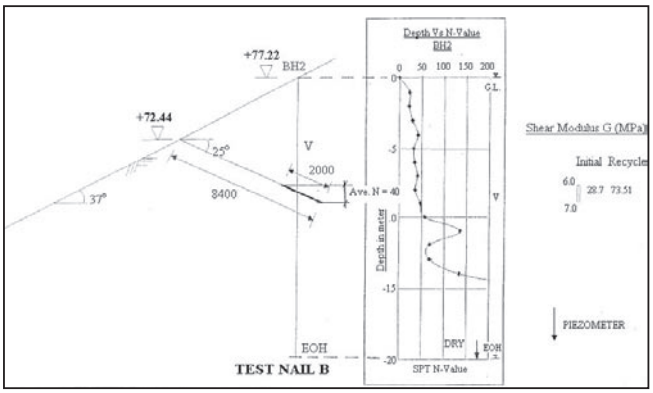

Fig. 3 Ground condition at Test Nail B

Installation of Test Nails

Nail holes were formed by a pneumatic drilling rig equipped with a drill bit of 105 mm in diameter. The nail holes formed were about 110 mm in diameter, 8.4 m in depth and at an angle of 25° to the horizontal. No casings were used in the drilling process. No hole collapse, voids or boulders were encountered in the course of drilling and cleaning of the nail holes.

A grouted length of 2 meters was formed for the bond section. Two different types of inflatable packers were used for this purpose. One is an air-filled packer designed and made locally for the present trial (HK Packer) and the other one is a grout-filled packer, a proprietary product imported from Germany.

Packer to Nail A

The HK packer was used in Test Nail A (Photo 1). The packer was inflated by compressed air to about 1 bar and remained inflated throughout the process of grouting. The bond section was grouted up by injection of grout (W/C ratio of 0.42, strength of 30Mpa) under a pressure of about 1 bar until excessive grout was seen emerging from the ventilation pipe. After the grout in the bond section had set, the packer was deflated and was free from contact with the nail hole wall.

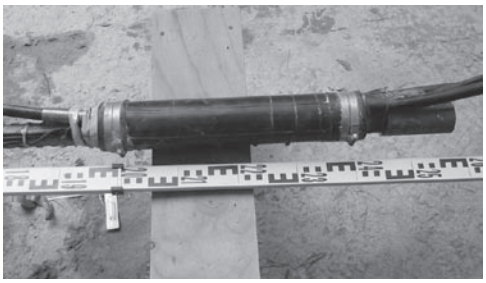

Photo. 1 HK Packer

Packer to Nail B

The German packer was used in Test Nail B (Photo 2). The packer was inflated by pumping grout into it at a grout pressure of about 1 bar (i.e., 100 kPa) and was left in place for about one day. Grouting to the bond section (W/C ratio of 0.42, strength of 30Mpa) was then carried out until the returned grout pressure of about 1 bar was reached (no grout return pipe was provided).

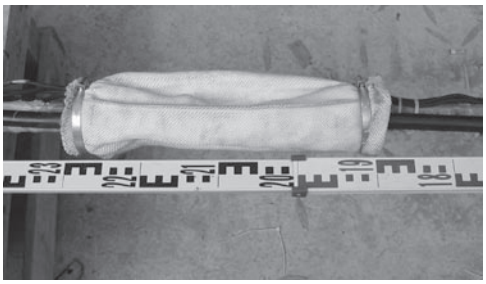

Photo. 2 German Packer

Instrumentation of Test Nails

On the steel bar of the test nail, strain gauges were installed in the bond section for measurement of the strain variation and thus the stress distribution along the bond section during testing. Four electrical resistant strain gauges were installed on Test Nail A at about 0.6 m intervals. As for Test Nail B, one additional strain gauge was installed at a location just preceding the German packer for measurement of the frictional resistance developed on the packer. This additional strain gauge on the Test nail B also served as a reference gauge for strain measurements obtained by other strain gauges on the nail bars under test loads. Layout of the strain gauges on the two test nails is shown in Figure 4.

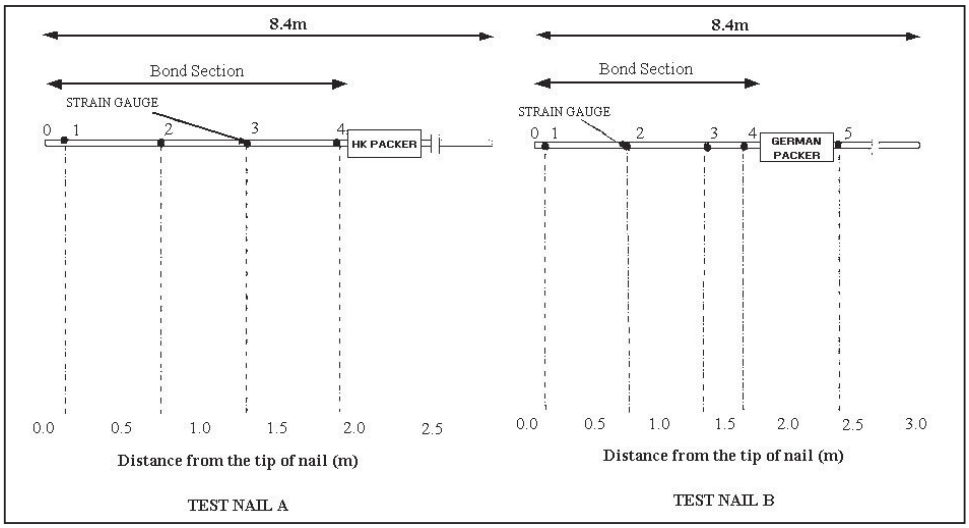

Fig. 4 Strain Gauge Arrangement

Test Set-up

The testing set up (Figure 5 and Photo 3) at the nail head principally followed the recommendations given in DTG11. The seating plate for the jack was fixed on a concrete pad resting on the trimmed slope surface for load spreading; and any deformation of the underlying soil would be more uniform because of the rigidity of the concrete pad. With a ball-bearing table supporting the jack and the test nail bar not in touch with the load transfer plate, the friction loss of the applied test loads could be minimised.

Test Procedure

Given the estimated overburden pressure at the test location, the maximum pull-out resistance was estimated to be 47.3 kN based on Mohr-Coulomb failure criterion (Watkins A. T. and Powell G. E., 1992). Given that a 40mm deformed bar is used in the test nail, the following load increments were adopted:

T_a = 5% T_p = 0.05 * 520 = 26 kN
T_{DL1}= 47.3 kN (corresponding to DL)
T_{DL2}= 94.6 kN (corresponding to 2 times the DL)
T_P=520 kN (corresponding to 90% of the yield load of the bar)

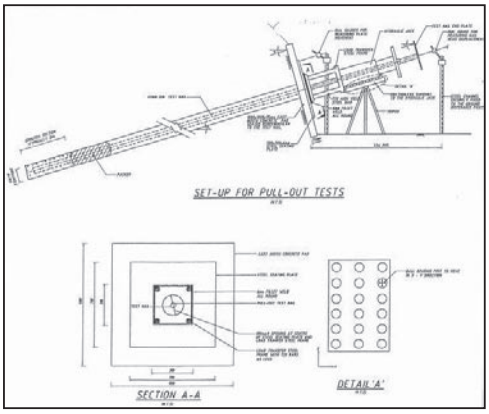

Fig. 5 The Testing Set-up

At each load increment, the test load was maintained for monitoring of nail head displacement for at least one hour. The test nail was considered to be able to sustain the test load if the difference of nail head movement recorded at 6 and 60 minutes was less than 2 mm (or 0.1% of the length of the bond section). In the course of testing, both test nails were able to sustain the test loads T_{DL1} and T_{DL2}. They were then loaded to their estimated ultimate pull-out resistance.

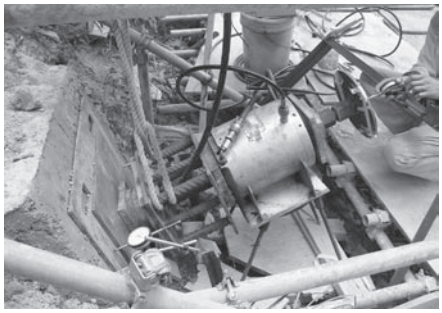

Photo. 3 The Testing Set-up

Test Results

Test Nail A was terminated at a maximum test load of 144.53 kN while the Test Nail B at a load of 175 kN where both of the nails experienced large movements: 40.48 mm for Test Nail A and 37.13 mm for Test Nail B. The plots of load/displacement of the nails are given in Figures 6 and 7.

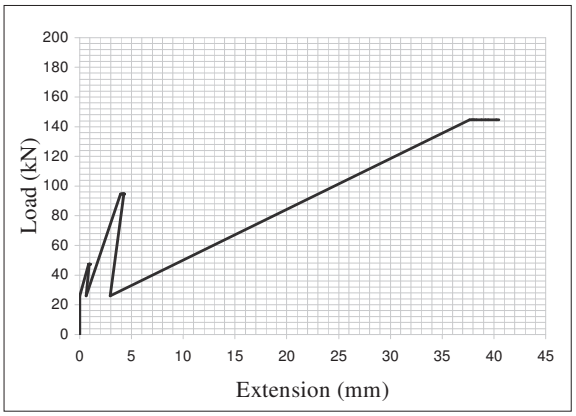

Fig. 6 Load-Displacement Plot for Test Nail A

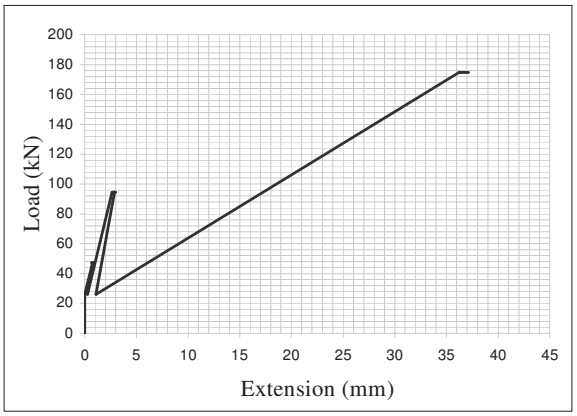

Fig. 7 Load-Displacement Plot for Test Nail B

The tensile forces prevailing in the test nail bars at different strain gauge locations are plotted against the test loads for Test Nail A and Test Nail B in Figures 8 and 9 respectively. The frictional resistances at the soil/grout interface over the sections between two consecutive strain gauges on Test Nails A and B under different test loads are given in Figures 10 and 11.

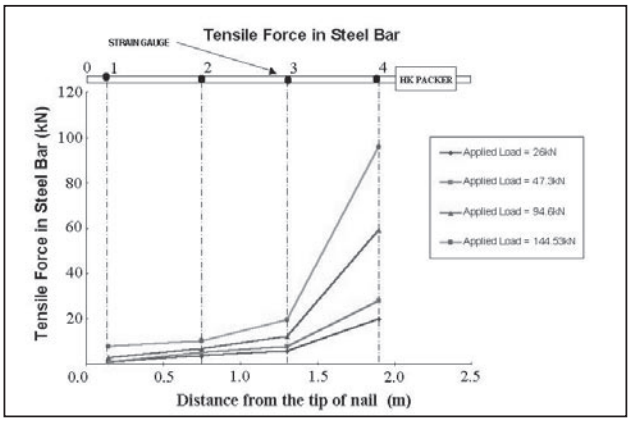

Fig. 8 Tensile Force in Test Nail A

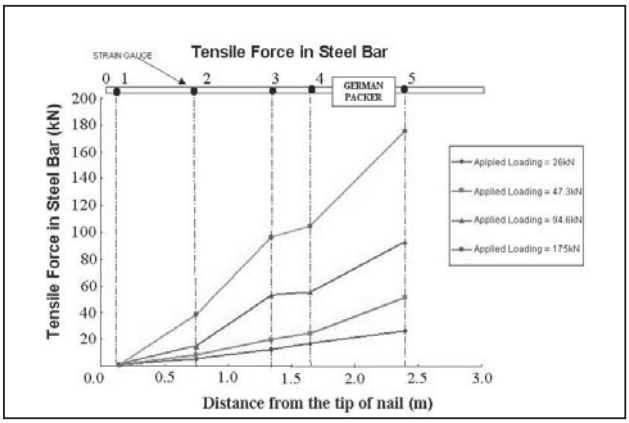

Fig. 9 Tensile Force in Test Nail B

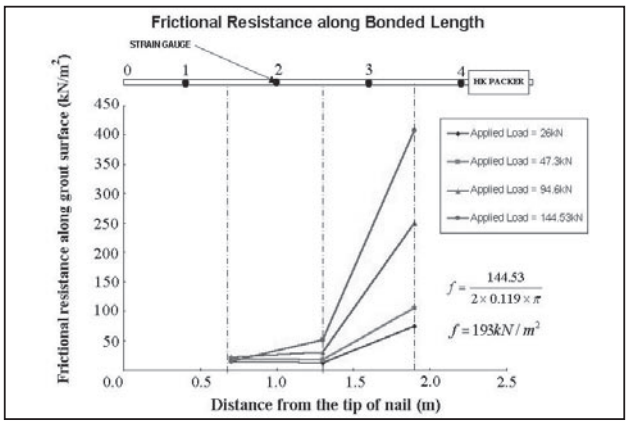

Fig. 10 Frictional Resistance Along Bonded Length of Test Nail A

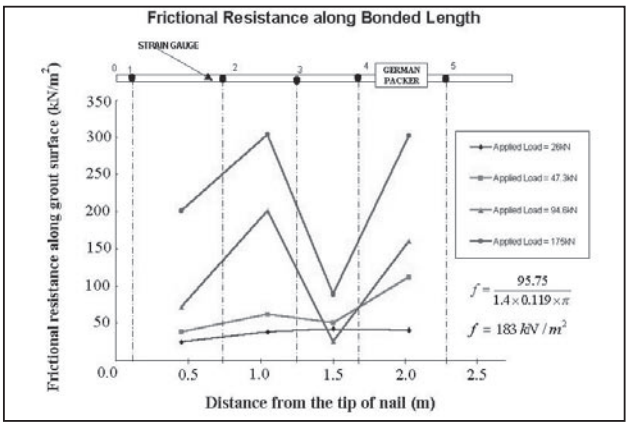

Fig. 11 Frictional Resistance Along Bonded Length of Test Nail B

Inspection after Tests

Following the tests, both test nails were pulled out for inspection. Pulling of the test nails was carried out by means of a hydraulic jack of capacity of 1000 kN. For Test Nail B, only the steel nail bar could be retrieved while the packer and the grout column were totally left behind in the nail hole. In contrast, Test Nail A was almost completely pulled out with the grout in the bonded section remained largely intact and attached onto the nail bar except for a missing portion of about 450 mm just behind the packer. The missing grout is believed to have been fractured and dislocated during the extraction under the high pull-out force. The diameter of the extracted grout cylinder of the bond section was found to vary from 116 mm to 122 mm (average 119mm).

DISCUSSION

The set up for the soil nail pull-out tests as recommended by the DTG11 is considered suitable and practical. The applied test load was found transmitted to the bond section with minimal friction loss as evidenced by the consistent results between the estimated and observed elongations of the free length of Test Nail B. In the case of Test Nail A, extrapolation of the strain profile from the strain gauge 4, which was about 100 mm away from the packer, indicated that the tensile load at the start of the bond section tied in with the applied test load at the nail head.

The test results revealed that the pull-out resistance of the test nail principally increased with the applied test load (Figures 10 and 11). The applied test load was mainly taken up by the front part of the bond section; and the mobilization of pull-out resistance diminished rapidly along the bond length toward the end of the test nail.

In the case of Test Nail B, the packer contributed significantly to pull-out resistance. Frictional resistance estimated from the strain gauges 4 and 5 indicated an average value of 301.95 kPa at the maximum test load of 175 kN. Given the packer measuring 400 mm in length and average size of the nail hole being 119mm in diameter, the frictional resistance contributed by the packer against the pull of 175 kN was estimated to be about 45 kN. Also worth-noting is the low frictional resistance between the strain gauges 3 and 4 suggesting that the grout between the packer and strain gauge 3, about 500 mm in length, was in poor condition. This may be due to the absence of a ventilation pipe with the German packer. During grouting of the bond section, air in the nail hole, having no outlet, might have been trapped in the area behind the packer resulting in a much weaker grout there.

The average bond strength of Test Nail A at the maximum test load of 145 kN was 193 kN/m^2. As for Test Nail B, with the correction for the frictional resistance of the packer and the weak air-trapped section, the average bond strength was estimated to be 183 kPa.

The calculated bond strength for the two test nails using the conventional method based on the Mohr-Columb failure criterion, was 75 kN/m^2. The ratio of the observed to the calculated bond strength for Test Nail A is 193/75 = 2.57; while for Test Nail B is 183 /75 = 2.44.

Attempt has been made to correlate the observed bond strengths of Test Nail A and Test Nail B with the SPT-N values and Shear Modulus of soil at the depths of the bond sections (Tables 1 and 2).

Identity	Average Bond Strength, f (kPa)	Average SPT-N value at the depth of the bond section, N (blows)	f/N
Test Nail A	193	75	2.5
Test Nail B	183	40	4.58

Table 1 Comparison of the average bond strength with average SPT-N of soil at test level

Identity	Average Bond Strength, f (kPa)	Limit Pressure Mpa
Test Nail A	193	>2.5
Test Nail B	183	>2.5

Table 2. Comparison of the average bond strength with pressuremeter test results

The ultimate bond strength at the soil/grout interface for the present tests were found generally in agreement with those recommended in the codes of practice for soil nails in various countries including China (CECS 96:97, 1997), Japan (JH, 1998), USA (FHWA, 1994 and 1998) and France (Clouterre, 1991) as shown in Table 3:

Countries	Soil Types	Recommended soil/grout bond strength in kPa	References
China	Dense Sandy Soil	160 to 200	The Chinese Standard CECS 96:97Soil Nailing support to Deep Excavation (CECS 1997)
Japan	Sandy soil of N-Value from 30 to 50	180 to 240	Japan Highway Public Corporation (JH 1998)
United States	Very dense silty sand and gravel	120 to 240	The Fedral Highway Authority (FHWA 1994 and 1998)
France	Sand of limit pressure at pressuremeter test ranging from 0.5 to 3 MPa	50 to 125	Clouterre (1991)

Table 3. Recommended values of bond strength at soil/grout interface

General Remarks

Some observations can be drawn from the results of the pull-out tests on the two test soil nails as follows:

(1) Reliable results have been obtained from the pull-out tests performed according to the recommendations given in DTG11. The recommended set up improved the load transfer of the applied load to the bond section.
(2) The recommended set up is practical and easy to be implemented.
(3) With the aid of a suitable packer, the bond section could be grouted to the required integrity and length.
(4) Deflated after the formation of the bond section, the HK packer is detached from the nail hole wall and thus does not affect the pull-out resistance of the test nail. On the other hand, the frictional resistance of the German packer needs to be accounted for in the determination of the pull-out resistance of the soil nail.
(5) The average bond strengths of Test Nail A and Test Nail B have been correlated with the SPT-N values as well as the pressuremeter test results. The results are found comparable to those published in the codes or standards in China and other overseas countries including Japan, France and the United States.

ACKNOWLEDGEMENT

This paper is published with the permission of the Head of the Geotechnical Engineering Office and the Director of Civil Engineering and Development, the Government of the Hong Kong Special Administrative Region.

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THE GREENING TECHNIQUES ON SLOPES AND THE COMPLEXITY BEHIND

Francis CH Wong, LM Chu, KC Chau[1]

Abstract: Safety on slope is more engineering oriented with clear and quantitative target to reduce the risk to life. On the contrary, greening the engineered slope is bio-scientific implementation taking into account people's perspectives to lengthen the harmonic effect to the natural environment. Whether safety and greening on Slopes could be an integrated works or a compromised combination, sometimes, really depend on the greening techniques. The government has done some R & D studies on the greening techniques and summarized the performance assessment in a special project report SPR6/2004 recently. From the view point of slope management and long term maintenance, apart from the construction cost, the implementation of the work and its long term reliability as well as greening performance will be the crucial consideration to decide on the adoption of such technique.

Different greening techniques in association with stabilization works have been implemented on slopes in the CUHK campus since 1998. Over the years, the greening performance in different slopes reveals the complexity of factors affecting the effectiveness of the techniques, including: (1) the slope condition such as soil and rock strata, geometry and hydrogeology; (2) the design concept and materials used in the techniques, its implementation and integration with stabilization works; (3) establishment of the vegetation and corresponding factors, e.g. local climate and ecology, species selection, bio-diversification.

In view of different applications, their merits and limitations in application to slopes in the campus have been reviewed. Concerns on the construction sequence, safety management and on-going maintenance as well as the future improvement on the environment subsequent to the greening works will be shared. The focus of studies in the campus on performance of several techniques will also be highlighted.

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INTRODUCTION

GEO Information Note 13/2001, (GEO, 2001) highlights the policy that makes Government slopes look as natural as possible to reduce their visual impact and improve the environment. It also describes considerations on the choice of surface protection and appearance of slopes after engineering works. In the design of slope works, choice of a surface protective cover to a cut slope is a critical consideration for stability, maintenance and visual impact. Similar concerns were cited in the CUHK Newsletters (蔡世彬, 2001 and Chan, 2002) that both safe and green slopes are important in the campus. In the Environment Report (CUHK, 2002), by applying the greening techniques, the campus environment will be benefited both from campus greening and slope protection.

A landscape treatment is used to enhance the appearance of a slope, either by hard or soft landscaping works, or by a combination of both. Hard landscaping refers to the use of "mineral based surface finishes or cladding such as stone pitching, sprayed concrete, concrete panels or even steel or plastic claddings" for stabilizing slopes (GEO, 2000a). Hard landscaping is used for most steep and all sub-vertical slopes, which require more surface protection.

Soft landscaping involves simpler greening techniques including hydro-seeding. It is often used for gentle slopes and certain steep slopes which required little or no surface protection. But these have been replaced or complemented by greening techniques that produce environmental friendly and aesthetically pleasing results, such as hydro-mulching (the spraying of a sticky reinforcing materials containing organic peat, then spraying grass seeds or planting shrubs on top of this materials). The resulting adhesion also provides protection to the slope surface. Such techniques have the dual effects of greening the slope and protecting surface soil. Some of them are defined as the use of living vegetation and/or plant materials to stabilize slopes and/or reduce erosion (Gray and Sotir, 1996).

Vegetation in the sense of green and natural is soft landscaping which is ecologically beneficial and less heat-reflective. Vegetation depletes water in the soil by root intake and transpirations, its roots bind soil particles together and can increase the near-surface soil strength through root reinforcement effects. However, vegetation takes time to establish and planting medium or substrate is critical to its sustainability.

FACTORS AFFECTING SLOPE LANDSCAPING

The decision to use soft or hard landscaping, or a combination of both, actually depends on many factors, which are normally inter-related and mutually influenced and can be categorized as follows:

Soil Factor. The rock forming minerals and nutrients derived from the parent materials by weathering process form the non-living skeleton of the soil. Organic material in soil is a mixture of living, dead and decomposing animal and vegetable materials. Humus forms from this process and helps to bind the mineral particles together to form a crumb structure. A sufficient volume of organic soil is important for root development and for sustainable supply of water and nutrients (GEO, 2000b). The main composition of nutrients including nitrogen (N), phosphorus (P) and potassium (K) determines growth and enhances plant physiological development, such as photosynthesis, flowering, fruiting, maturation, particularly lateral root development (Chiu, 2004). Soil systems develop and change through time. This is a dynamic process which responds to the prevailing environmental conditions. Within the soil profiles are bacteria, fungi and organisms and insects such as earthworms and beetles. These help to break down organic materials and enhance the soil structure.

Vegetation Factor. The various techniques for slope landscaping usually adopt a common species mix of Common Bermuda grass, Bahia grass, Perennial Ryegrass and several shrub species. Grass is the dominant species used due to its rapid establishment that provides instant protection to the surface soil or substrate (Greenway, 1989) and also achieve the contract instant green requirement under the contract. The distributing patterns of the species on the slopes and signs of natural regeneration such as flowering and/or fruiting reflect whether the species can self-regenerate or not.

Climatic Factor. All aspects of climate including sunlight, humidity, radiation and particularly temperature and precipitation, influence leaching, weathering and formation of humus in the soil. The presence of a distinct wet and dry season in Hong Kong with about 80% of rain falling between May and September makes moisture stress an important issue for vegetation growth on slopes. Regional climate such as typhoon in the wet season and sub-tropical zone characteristics are the catalyst of vegetation growth and soil derived.

Land Factor. For many soil types, the underlying rocks directly influence the soils above. Chemical composition of the rocks affects soil fertility. Solid rock, thickness of decomposed soil, slope orientation, height and its gradient, and particularly groundwater, are essential for both animal and plant life in the soil. Nutrients are carried in a solution which enables plants to obtain them via the root system. The supply of groundwater from slopes or dryness will affect the vegetation growth. Choi & Chau (2004) added the environmental data to study the performance assessment of vegetation growth, including the slope altitude, degree of shading by adjacent features, exposure to wind and presence of natural stream courses near the slope.

Engineering Factor. The stabilization works including shotcrete surface, raking or horizontal drains, rock fill, cement soil or re-compaction of backfill materials, etc will affect the growth of vegetation root. The sprayed concrete (or shotcrete) almost replaces the less durable soil-cement plaster, or chunam which was used extensively in the past. When properly constructed and maintained, a shotcrete cover is beneficial to slope stability because it virtually eliminates direct infiltration of water and provide reliable protection against surface erosion.

The relationship amongst these factors is summarized in the Figure 1.

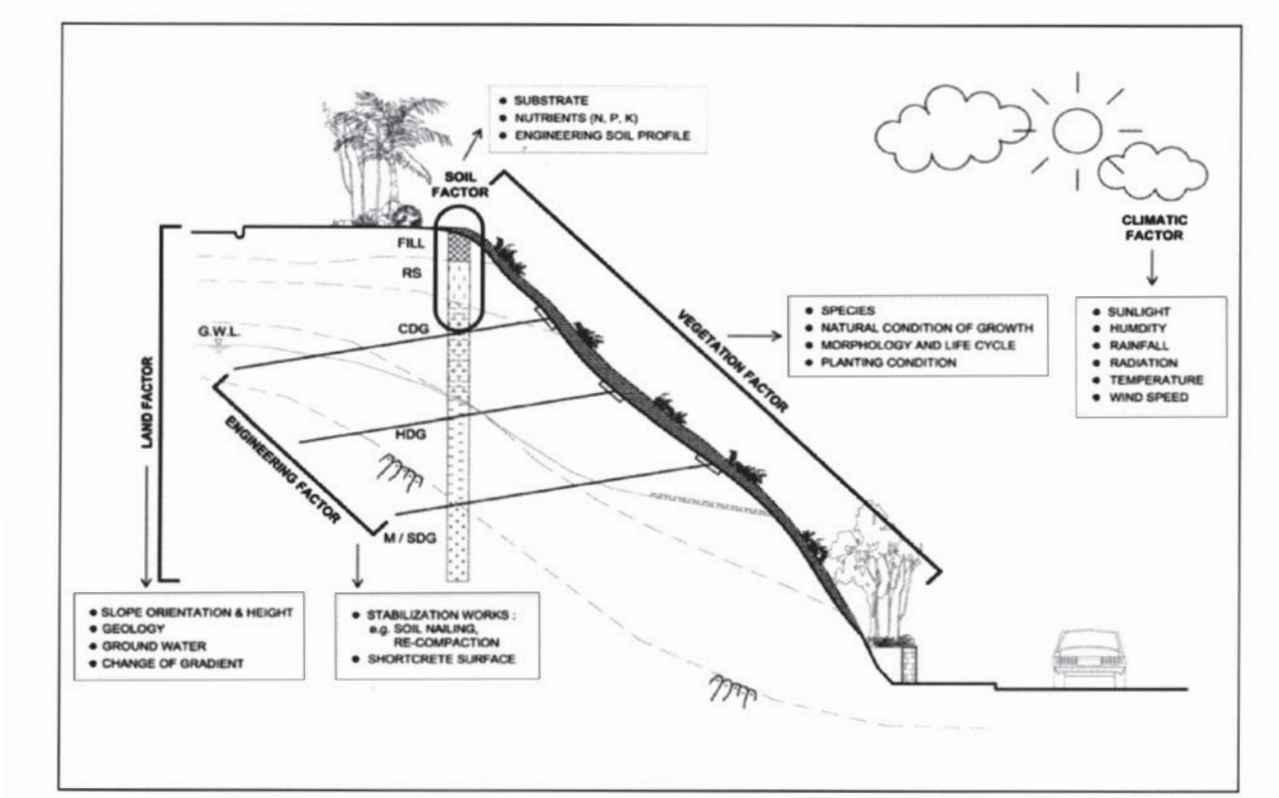


Figure 1: Factors Affecting the Greening Techniques

To cater for difficulties due to the various factors described above, proprietary products suppliers launch a number of greening and innovative techniques. Because of the complexities of factors and their mutual interrelated process, the performance of each technique in the past would be the key to its adoption. For this purpose, a systematic review was carried out by GEO and presented in a GEO report, Lui & Shiu (2004). The performance assessment is based on two aspects: (a) Growth of vegetation, including health conditions, coverage, dominance and diversity of vegetation; and (b) Engineering performance such as structural integrity and effectiveness of erosion control of the greening techniques. These assessments have reflected most of the users' concern and help to expedite the research and development study on greening techniques.

DEVELOPMENT AND ADOPTION OF GREENING TECHNIQUES

To develop a mature, safe, environmental, cost and effective greening technique, it will normally go through several stages: (1) Technology transfer; (2) Trial Panels for performance review and (3) Implementation. In Hong Kong, most of greening techniques have been developed from Japan's experience. To suit the local conditions and avoid the adverse impact on upgraded slope, the proprietary promoter is normally requested to provide sufficient job reference and technical data to satisfy the engineers' concerns, and sometimes, to comply with statutory requirements. Hence, they would approach different prospective users, including property owners, landscape architects, design engineers or main contractors to introduce their proprietary product. A conceptual implementation of a new technique could be summarized in Figure 2.

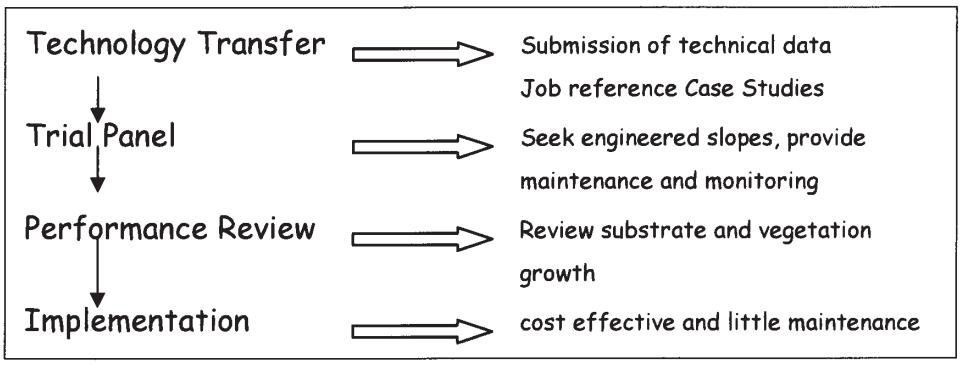


Figure 2: Conceptual Implementation of an Innovatory Technique

Similarly, there were several stages in the Chinese University campus making reference to the set-up of trial green panels as the watershed, in the form of pre-panels; panels and post-panels. The trial green panels were established from 1998 to 2003.

Pre-panels Stage (1996 - 1998) The greening techniques were implemented in association with the slope stabilization works since 1996. In respect of engineering requirements, all slopes have been covered by shotcrete. Planter holes or construction of planters at crest, toe or mid-berm of the slopes were the basic greening techniques at that moment. Hydro-seeding plus erosion control mat would be the top-up and fundamental greening implementation, which is still common nowadays and applied to those rocky soil cut slopes with highly fractured rock mass and/or high ground water table.

Panels Stage (1998 - 2002) As anticipated, the engineered slopes would bring about visual impact on the green campus. Since 1998 the University invited slope landscaping practitioners/designers to propose and install their unique landscaping work in the campus. As at 2002, there were 8 trial panels established and all placed on cut slopes near Residence Road and opposite to Science Building respectively. Under the same slope condition including physical geometry, ground materials, micro-environment and climates, etc, this would provide good performance reference for different techniques. A sketch with photo and brief sections is shown on Figure 3.

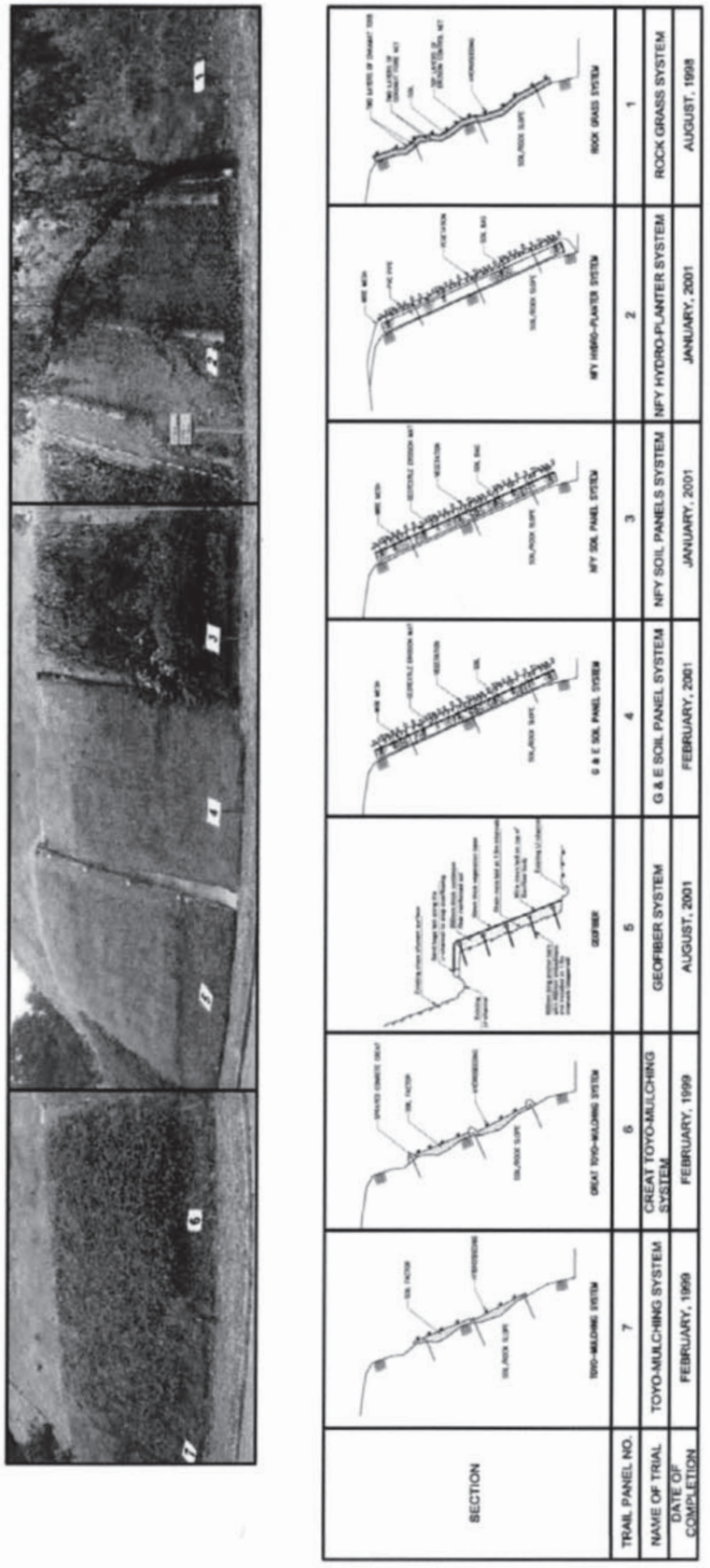


Figure 3: Trial Panels of different Greening Techniques

During these periods, various observations, concerns andincidents could help to enhance our understand on the greening techniques.

Trial Panels: The greening techniques in the trial panels included Rock-grass, Hydro-planter, Soil Panel, Geofibre, Toyo-mulching,, Biocrete, and Biodrain. The trial of some of them was the first in Hong Kong. The concept, rationale and installation details of these techniques are recorded or published in their brochure or papers, for example, the Geofibre and Greenwalls Biodrains System have been published in Construction News. Site observations on these techniques were focused on the thickness of substrate or planting medium, the nature and components of the substrate such as nutrients, presence of water absorbent, sticky adhesive nature, self-standing ground mass, etc. The thickness of the substrate from 100 to 150mm is most suitable for vegetation growth.

Regarding the effectiveness of these systems, Lui & Shiu (2004) have discussed in the report, "Performance Assessment of Greening Techniques on Slopes". From the maintenance or engineering point of view, concerns on the durability and sustainability of these systems, the extent and frequency of maintenance, the stability under adverse weather, etc. are the key to adoption on full implementation.

Registration System: To enhance innovative and effective maintenance, all trial panels were required to be registered. Under the requirements, All Greening Method and Robing Wall were included into the system and placed at another cut slope. The system provided a systematic review on the panels and also recorded the change of greening effects in the dry and wet season.

Performance Studies: Two performance studies were carried out in conjunction with two specialist contractors on their proprietary products. The studies were extended to about 30 slopes which have used the two proprietary products and located outside the campus. As the study period was comparative short, no significant findings except the maintenance concern were found. A primitive tool for performance assessment using photographical method incorporated photo-taking and digital calculation has been developed to calculate and distinguish the green and vegetation coverage (Figure 4). The calculation was developed from software Photoshop version 6.0 and AutoCAD R14.

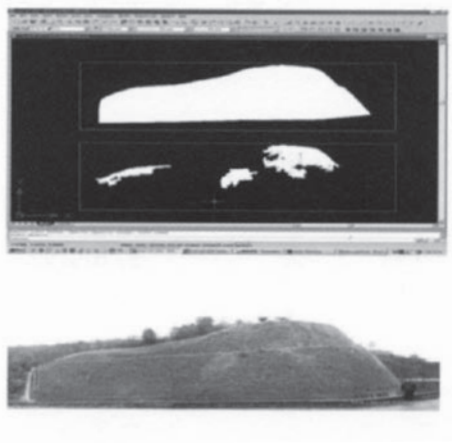


Figure 4: Photographical Method

Figure 5: Experimental Field of Toyo-mulching

Green Coverage. Green coverage is the common requirement to assess the performance of greening techniques and fulfillment of construction contract. However, vegetation coverage instead of the green coverage is the criteria adopted by the proprietary suppliers. The adoption of green or vegetation coverage is sometimes the different concept on whether the greening technique is successful or not. An inspection form is also developed to record the performance of scheduled grasses or shrubs. Throughout the study period, it is easy to distinguish the green and vegetation coverage by quantitative calculation which is also termed as time of establishment and mature growth, the latter implies the seasonal changes of vegetation including grasses or shrubs turn yellowish brown in the dry or winter season.

Experimental Field. One of the greening techniques, Toyo-mulching was developed from Japanese experience. An experimental field in Tsuyama (Figure 5), a small town north of Okayama, has systematic studies on various shrubs, groundcover, plants on different slope conditions including steep gradient, river bank, hard and soft surface on embankment, etc. The studies indicate heavy demand in maintenance.

Walk-over Study. The nature of walk-over study is similar but not identical to engineer inspection. It is a basic step for performance assessment. For each slope, information on the specific greening technique or the landscape treatment on engineered slopes, landscape considerations, assessment of greening technologies in terms of retention of existing vegetation, fit with surrounding landscape, naturalness of solution, visual simplicity of solution, mitigation of visual impact, contribution to environmental sustainability, adherence to aesthetic principles, relative overall order of costs, etc were recorded. Monitoring records were developed to list out all findings on site, such as cracks and soil erosion, grass lengths, colour, condition of tip, body and foot, coverage (bald, sparse, dense), wild shrubs/types of scheduled plants, micro local weather, and other observations related to monitoring. A sample of monitoring record is shown in Figure 6.

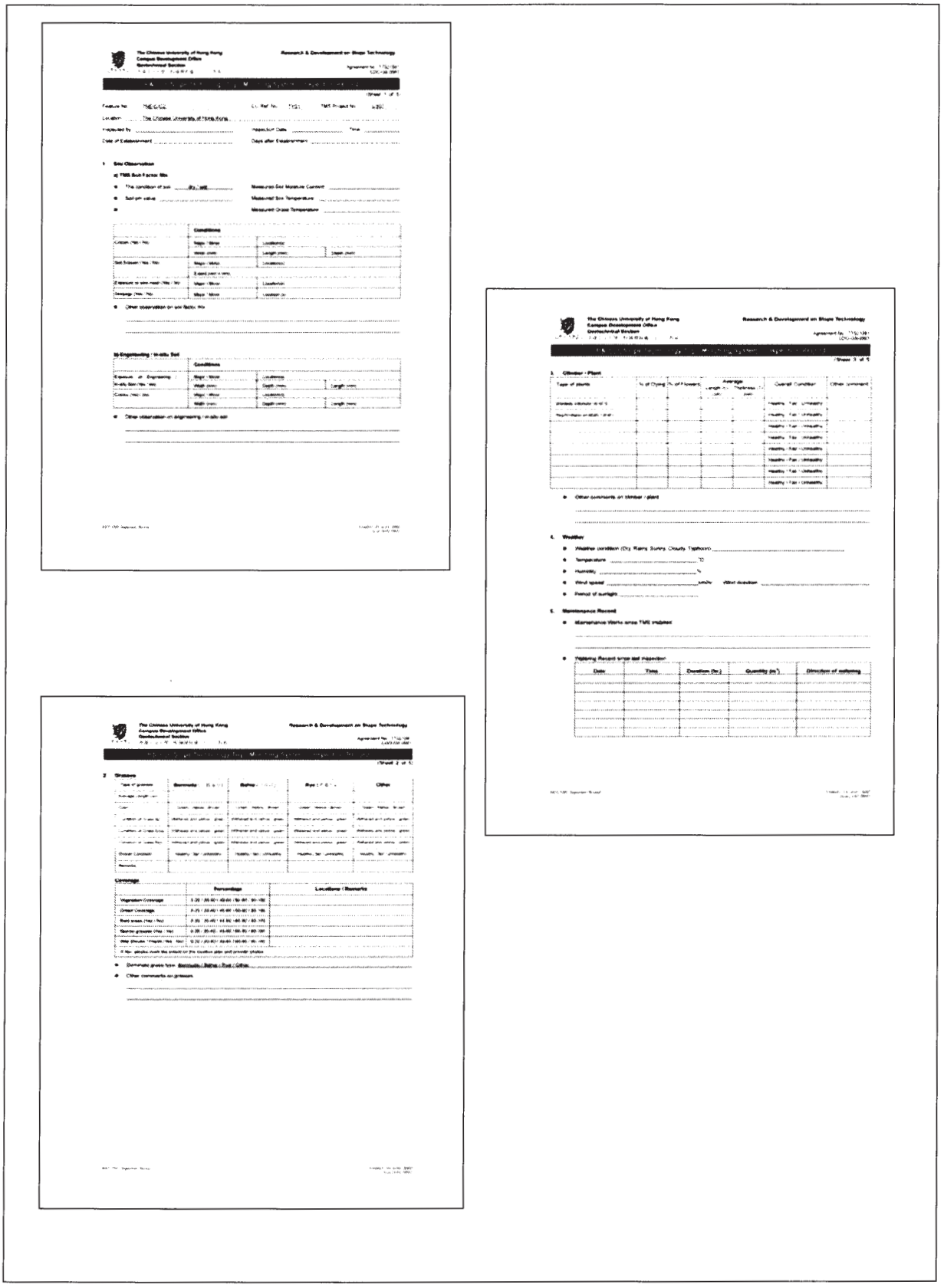


Figure 6 : Monitoring Record used in Performance Review

Post-panels Stage (2002 to present): There were two academic departments involved in the Research and Development on slope greening techniques. Several issues were identified.

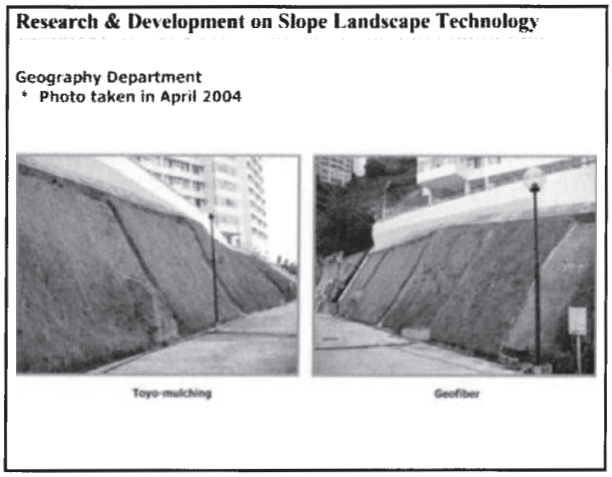
Substrate. In the Geography Department study of Toyo-mulching and Geofibre, it was noted that both substrates were highly organic and the peat moss component elevated the retention ability of water and nutrients. Chiu (2004) has highlighted the findings of the studies in her MPhil dissertation. The concept of the experimental plot is shown on Figure 7. The study would continue to focus on water monitoring and species selection.

#1	*#2*	*#3*	*#4*	*#5*	*#6*
Hydro-mulching					
Thickness 2"	Thickness 4"				
Bahiagrass ($10g/m^2$) and Bermudagrass ($15g/m^2$)	Bahiagrass ($10g/m^2$) and Bermudagrass ($15g/m^2$)	Bahiagrass ($10g/m^2$) and Bermudagrass ($15g/m^2$)	Bahiagrass ($10g/m^2$) and Bermudagrass ($15g/m^2$)	Bahiagrass ($10g/m^2$) and Bermudagrass ($15g/m^2$)	Bahiagrass ($10g/m^2$) and Bermudagrass ($15g/m^2$)

Figure 7: Experimental Plot to Study Substrate of Hydro-mulching

The substrate thickness affects the availability of water and nutrients to vegetation (HyD, 2003). Grass prefers 300 mm deep soil, as a normal understanding. The vegetation base of most of the greening techniques averaged 5 to 26cm. Rooting was further inhibited by the impermeable shotcrete lining or hard rock face underneath the substrates. Thicker substrates elevate the affinity for water and nutrients and improve rooting condition for plants. However, steep slope also increase the risks of substrate erosion and instability of the slope. In addition, the building up of a perched water table also adversely affect the stability of the thicker substrates.

Vegetation Growth. Biology Department designed 40 steel artificial slope panel trials to compare grass growth on Geofibre, Toyo-mulching and erosion control mat on sand at an inclination ranging from 50 to 60 degrees. Seeds germinated very well after one month but the swards wilted and died in the dry winter. The study would continue to focus on biomass production, surface runoff and soil erosion. The photos showing two studies are shown in Figure 8.



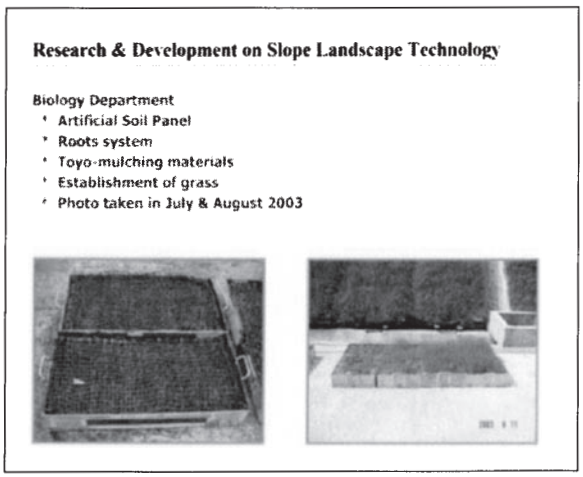
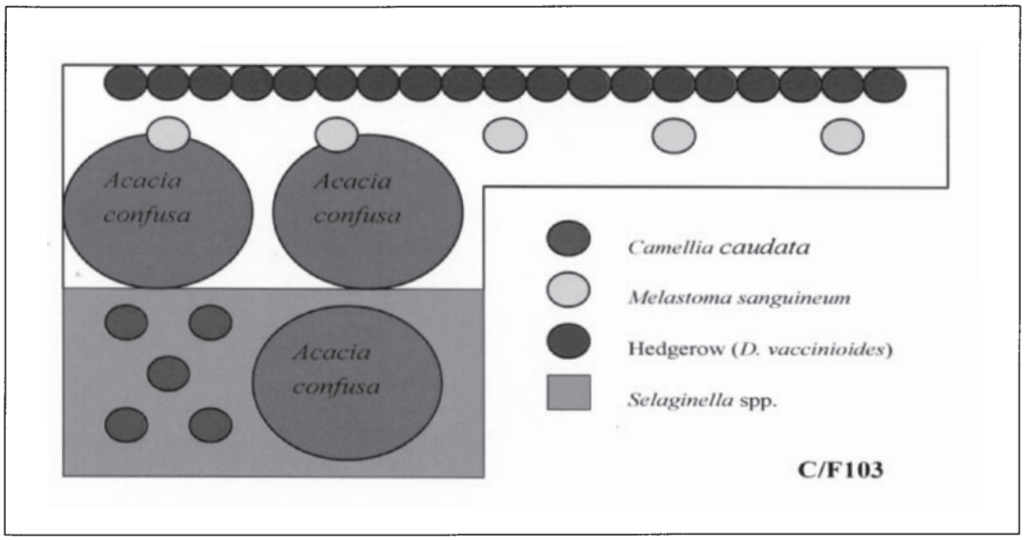


Figure 8: Academic Studies on Greening Technologies

Native Species and Bio-diversification. Grasses is a common vegetation adopted for slope greening. As it is widely spread, there are concerns on conservation of native species and bio-diversification. The exotic grasses have disadvantages of being unnatural, unsustainable and require relatively high maintenance; native species may be able to overcome these limitations. There were joint studies on selection of vegetation species between Flora Conservation Department of Kadoorie Farm and Botanic Garden and the University. After the slope stabilization works, Kadoorie recommended a planting plan consisting of ground cover, shrubs and trees on the slope. The arrangement incorporating designs based on the slope condition, micro-climate, ground condition is shown on Figure 9.



Recommended species	Chi. Name	Planting instructions	Remarks
Camellia caudate	尾葉茶	5 individuals, staggering arrangement, 1.5-2 m spacing	7 m tall, semi-shaded environment, attractive white tea flowers
Melastoma sanguineum	毛稔	5 individuals, 1 row, 1.5-2 m spacing	2-3 m tall, exposed environment, attractive pink flowers
Diospyros vaccinioides	小果柿	20 individuals, 1 row, 0.5-0.8 m spacing	1.5 m tall, exposed environment, tolerate to regular pruning
Selaginella spp.	翠雲草	Planting in rows, 0.2-0.3 m spacing between rows (the *Selaginella* will close the gap once grown up)	Fern, ground cover

Figure 9 Planting of Native Species in a cut slope

Irrigation and Maintenance: Grass is a strong survival plant and easily grows in a large extent. Under the principle of low maintenance, irrigation should still be applied during the establishment of the grasses and when vegetation starts to turn brown partly due to seasonal change. The amount of water applied should overcome the quantity lost in evapo-transpiration. Frequent and light irrigation is preferred to heavy and rapid irrigation that fails to wet the hardened substrate surface. In a large area, e.g. the campus, the increasing irrigation for those slopes which were greening, automatic irrigation system will be an alternative to avoid the heavy maintenance in future.

In summary, during the period, the proprietary products suppliers were frequently requested to clarify the following concerns frequently: safety of substrate; greening effect and vegetation cover, irrigation systems; and the percentage of Nitrogen, Phosphorous, Calcium which are important element for feeding vegetation. In addition, as Chiu (2004) highlighted the low ecological values and high maintenance costs of the established slopes, the goal of slope greening should shift from "artificial greening" to "naturalization" through the recruitment of native species. This can be achieved in two ways, namely natural recruitment by the exotic grasses and enrichment planting of shrubs, ground covers and low trees on permitted substrates. The alternative of using exotic grass species will be natural species that are drought-resistant and shallow-rooted.

ASPECTS FOR REVIEW OF GREENING TECHNIQUES

The adoption of a greening technique is different from engineering design which is normally to be screened by government departments in accordance with a series of standard reference. Total greening, i.e. 100% green coverage under the contract requirement is the common acceptance criteria although it sometimes involves the subjective perspective. Low monitoring is also a major requirement in term of the anticipated length of maintenance. The safety of the slope must be maintained when applying the greening techniques.

Concerns as shown in Figure 10 summarize all related issues that cannot be neglected. All these are inter-related and fall into the same management system. The adoption of greening techniques is likely the foci of the management system which is influenced by different perspectives of policy makers, technical disciplines/systems, statutory procedures, and habitat of decision making.

There are three strategies, as follows:

- The outer layer, Engineering Development and Planning (EDP) involves resource allocation, funding secure and priority setting.
- The middle layer, Environment Improvement and Monitoring (EIM) involves compromise of human perspectives, implementation of improvement philosophy and execution of environmental policy.
- The inner layer is Effective Maintenance and/or Engineering Upgrading (EM/EU), which is the routine maintenance and preventive/upgrading works execution.

To facilitate the selection of innovative greening techniques if all aspects above have been compromised, a checklist reflecting the practical concerns and minimize uncertain complexity behind, is recommended (Figure 11).

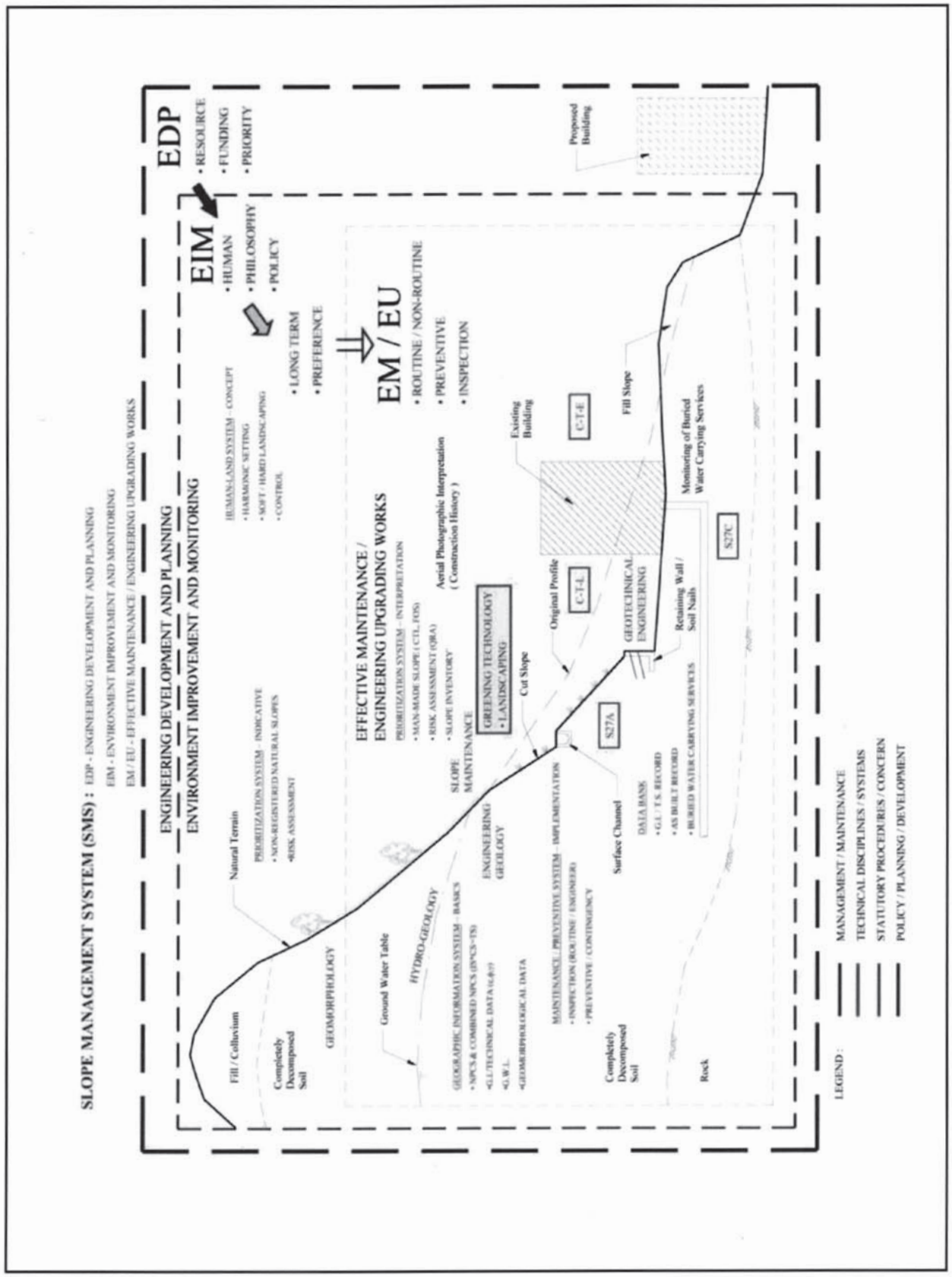


Figure 10 Slope Management System and Greening Techniques

The Designated Slope

1. What is the characteristic of the designated slope? Cut or fill slope? Rocky soil cut or soil cut slope?
2. How steep is the slope? What is the location of the slope, near a valley or at the hilltop? Facing the sunlight? Benefit the moisture from the South?
3. Is the slope in a convex or concave shape? Any sudden change of slope gradient or the presence of failure scar?
4. How thick are the soil strata, fill or soil cut? What is the soil originated from, volcanic or granitic rock; lack or rich in humus? Sandy or Clayey? Can the ground water or the rainfall be retained?
5. Should the hard surface protection such as shotcrete or stone pitching be needed?

Proprietary Products

1. What is the principle of the greening techniques? Where were it originated from, any technical reference and case in Hong Kong or other countries?
2. Is there any performance assessment available? Is the method or materials used sufficiently unique and already been patented? Any alternative of the materials?
3. What is the components/main composition of substrate used? Any typical section to show details/specifications as quality control?
4. Can the thickness of the materials be reduced or thickened for different vegetation planting? How is the sustainability and durability? Is it sensitive to temperature changes, i.e. contraction and expansion?
5. Any supplementary accessories like drain mat for water discharged or nutrients cells for installation?
6. Are there any particular issues for its installation? Is it sufficiently safe and comply with relevant Regulation and Ordinances? Would it disturb the stabilization works like soil nailing or placement of wire mesh?
7. How is the maintenance works, low or no maintenance? How long are the vegetation establishment, self stand? Any maintenance manual to follow?

Vegetation Growth

1. Can the vegetation be planted or diversified? Are they aggressive or harmonized with native species nearby naturally?
2. Does the vegetation self-regenerate? Are they sensitive to the shading of the slope by adjacent features?
3. How frequent to the grass trimming, watering, pesticide spraying?
4. Are the Nitrogen-Phosphorous –K(Potasium) in high demand by the vegetation.

Maintenance Concern

1. How frequent is the grass trimming, watering, pesticide spraying?
2. Can the slope routine maintenance works and engineer inspection be carried out easily?

Figure 11 Selected Questions as Checklist for Adoption of Greening Techniques

DISCUSSIONS AND RECOMMENDATIONS

Sharing the same conclusion in the GEO Report, Halcrow (2001), landscape issues have often been neglected in design consideration particularly in older slopes, but public/media pressure has resulted in increased attention to landscaping at the early design stages. Landscaping professionals including landscape architects, specialist contractors etc should be involved at every stage: planning, design, construction and maintenance.

The adoption of greening techniques is a combined consideration of different aspects. For the engineer, much concerns have been placed on safety aspects, however, for the trend is to look at the greening effect and its longevity as well. The compromise of different aspects and balance of the interest among various parties will be the first and important tasks for the slope dangerous.

The cost and related expenditure of greening is another important consideration. Normally, landscaping plus surface finishing for a slope is about 5 to 8 per cent of the construction cost, occasionally up to 10 per cent depending on the extent and scale of the surface protection and fractured rock mass. However, for the greening techniques in the market, their costs are almost in excess of the normal budget in the slope stabilization or upgrading work. This will discourage designers to adopt greening techniques. From the early 2000s, the proprietary products supplier has been looking for a cheaper replacement mainly on the materials used in the greening technique. In the meantime, the study on greening technique has also looked into the thickness and the sustainability of the substrate particularly under heavy rainfall. This will save the cost, however, the sustainability of the greening technique is becoming a concern.

In the mid 2000s, the study on greening techniques is likely to shift to the study of their vegetation base or substrates, which is the understanding of physical and chemical properties of growth substrates. Apart from cost saving, too thick substrate has stability and maintenance concern. The compaction on the substrate as required under the engineering design would adversely affect root penetration and water infiltration that, in turn, determines vegetation performance. Some greening techniques have selected long roots Vertiver as reinforcement.

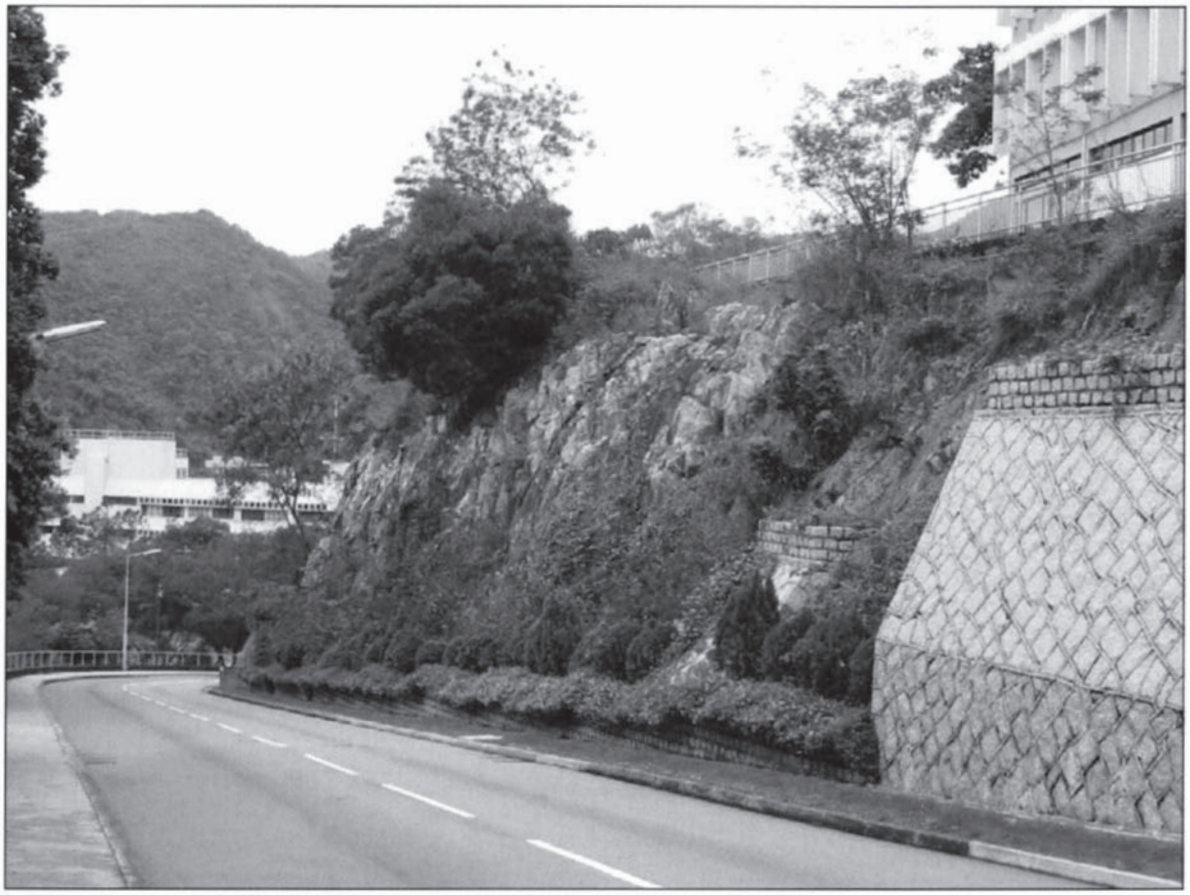
Exotic grasses because of their instant growth and coverage is easy to satisfy the construction contract requirement. Gradually, this may draw the public attention on bio-diversification and the conservation of native species. The selection of appropriate species other than the exotic grasses is crucial to continuous development of slope greening techniques. Identification of suitable vegetation species for use on man-made slopes by Choi & Chau (2004) is a good supplementary study on the greening techniques. It recommends preserving mature, native trees and topsoil, increasing the use of self-regenerating vegetation species, increasing the use of shrub and climber species on steep slopes, expanding the range of suitable vegetation species, etc. Native species are good to adopt because they are low nutrient-demanding, drought-resistant and preferably shallow-rooted. Selection of vegetation is sometimes affected by their availability in the market and also the mode of growth whether by spores or seeds.

Low or no maintenance of the greening technique or planting vegetation is also critical. Can an irrigation system be installed in association with the greening technique or the water retainability in the substrate is good enough to maintain a minimal or zero irrigation? Could the greening technique be sustainable and the vegetation is self-generated and continued to grow? Chiu (2004) has used four mixes of native species, ferns and groundcover to make comparison for their re-vegetation performance. The basic maintenance such as irrigation and trimming are required to keep the slope green throughout the year. The severity of these requirements is to be examined for a sustainable slope management strategy. Safety access to maintain is also a concern. Guidelines recommended by Lam, Siu & Chan (2002) for slope maintenance are also applicable.

Halcrow (2001) recommended further researches on the basis of their immediate practical use as follows: use of tree seeds, planting of native species, bio-engineering techniques, and proprietary products. Similar researches are also being carried out by the proprietary products providers, specialist contractors, academic departments and government departments. Also, conducting more planting trials and enhancing post-planting management practice in the form of effective maintenance are also recommended.

CONCLUSIONS

The adoption of greening techniques should include different technical discipline/systems to ensure the greening technique on slope is safe and complies with statutory requirements, EM/EU(Inner layer of Effective Maintenance and Engineering Upgrading). Meeting public concern, philosophy and policy on environment, relationship between the human being and the land are the focus of the middle layer, EIM (Environment Improvement and Monitoring). The EDP layer (Engineering Development and Planning) sometimes involve the resource, fund securing and prioritization. Without the support from the EDP layer, the comparable higher construction cost of the greening techniques will not have the opportunity to be adopted.



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SLOPE WORKS IN MAINLAND CHINA – A CASE STUDY OF SLOPE UPGRADING WORKS IN SHENZHEN

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Abstract: This paper presents some recent experience on slope design and construction works in Mainland China. The preparation work including information collection, ground investigation, and laboratory testing is discussed in detail. The design principle and specifications for slope upgrading works are introduced, and compared to Hong Kong's practice. The focus is placed on the assessment of slope stability and the design of soil nails as a means for upgrading a substandard slope. The construction materials involved and their testing for performance are also discussed.

INTRODUCTION

Prior to the signing of the "Ordinance of Prevention of Geological Disaster" in 2003 by the Prime Minister of China, only major structural work was managed by the Chinese Government. For some projects, slope work was considered as minor, and was therefore not systematically managed by the government. The new instruction, which became effective from March 2004, requires that any existing and future development shall consider the potential for geological disaster. Consequently, the design and construction work for slopes has to be rigorously managed by the government.

The increase in the involvement on construction projects of local consultants and contractors in Mainland China has made it necessary to understand the similarities and differences between standard practices in the Mainland and in Hong Kong. This paper presents the experience gained from a recent slope project carried out in Shenzhen. The detailed procedures from preparation of design works to construction are discussed and compared with common practice in Hong Kong.

PROJECT DESCRIPTION

The project involved slope upgrading works in Shenzhen. The ultimate objective was to ensure the safety of some high voltage transmission towers near a slope. The initial stability of the slope was assessed based on the available ground information. Improvement work was then proposed to enhance the stability of the slope in accordance with the code of practice used in Mainland China.

PREPARATION WORK FOR SLOPE UPGRADING WORKS

Government Submission in Mainland China

The submission procedures for a construction project may vary from one city to another in Mainland China. A general procedure for slope upgrading works in Shenzhen is described.

The first step is to submit the documents (general plan, bills of quantities and method statements) to the local construction authority for assessment and to apply for approval from the town planning and land use authorities. The local construction authority will then check the submitted information with the corresponding government departments for approval.

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Contractors (overseas contractors) must participate as a joint venture with a local company to apply for construction consent. The following information is required:

- Application form for construction consent
- Registered certificate for quality supervision
- Audit forms for site safety
- Certificate of the contractor qualification grade

All the mill certificates and sample test results shall be submitted to the quality control office of the local authority for checking. In addition, the "audit forms for site safety" shall be ready on site for checking by the Site Safety Control Office of government. Upon completion of the site works, all the as-built drawings shall be endorsed by the site supervisory engineer and submitted to the government for record.

The submission process described above is quite similar to that in Hong Kong. The only major difference is that the submission is sent to one government department in Hong Kong (such as the Building Department). It acts as a coordinator with other relevant departments to respond to the applicant. In Mainland China, the applicant will normally have to apply for approval from each department individually.

Ground Investigation

For general construction works, ground investigation (GI) shall be carried out in accordance with the following codes of practice and standards issued by the Ministry of Construction, People Republic of China (MCPRC):

- Code for Investigation of Geotechnical Engineering, GB50021-2001 (MCPRC 2001)
- Standard for Rock Grading, GB50218-1994 (MCPRC 1994)
- Standard for Soil Test Method, GB/T 50123-1999 (MCPRC 1999)

If the project is for highway, metro or municipal facility etc, reference should be made to the code for investigation of that particular division.

For drill holes, no special requirement is specified for the minimum number and spacing. The drill holes, however, have to penetrate into a depth of 2~5 m below the potential failure surface of the slope. Most of the equipment for GI is lightweight rigs with relatively low output power. Therefore, deep hole drilling may not always be feasible.

SLOPE STABILITY ASSESSMENT

The assessment of the stability of a slope is described in the Technical Code for Building Slope Engineering (MCPRC 2002). It employs the factor of safety approach. The required factor of safety is determined based on the category of the slope, which is a function of the importance of the slope and the neighboring structures. The minimum safety factor also depends on the method of stability analysis. For example, a minimum value of 1.3 is required if the stability of the slope is assessed by assuming a circular slip surface using limit equilibrium approach for a category 1 (i.e. most important) slope. If a planar failure surface is assumed in exactly the same slope, the required minimum factor of safety would be 1.35. This is presumably to take into account the simplification of the failure mode. Considering the popularity of commercial computer analysis programs, this conservatism regarding the over-simplification of the failure mechanism may be waived.

It is suggested in MCPRC (2002) that the highest historical rainfall intensity or the ten-year return period fall intensity should be considered when a stability analysis is conducted. The soil parameters should be determined from tests conducted on in situ samples. In general, the shear strength of the soil is characterised by the effective stress parameters c and ϕ. The concept of undrained shear strength can also be employed if the soil is relatively impermeable and saturated.

The guidance described in MCPRC (2002) is in principle identical to that recommended by the Geotechnical Engineering Office (GEO) in Hong Kong (GEO, 1984) as far as slope stability assessment is concerned. Although it appears that the required minimum factor of safety is lower for slopes in Mainland China, MCPRC (2002) suggests that the required factor of safety should increase accordingly in slopes with very complex geological formation, or if the structure is extremely important.

In this project, the original calculated safety factor was 1.104 which was regarded as insufficient. Improvement work was therefore proposed in an attempt to increase the overall stability of the slope. Soil-nailing was the chosen method of improvement. Considering the extreme importance of the power line towers, it was decided to upgrade the slope to achieve a minimum factor of safety of 1.4.

SOIL NAIL DESIGN

The design requirements for soil nails in Mainland China are also outlined in MCPRC (2002). The basic idea is to enhance the stability of the slope by applying tension force of the soil nails. Three major failure modes have to be checked regarding the performance of the nails. The first one is tensile failure of the steel bar. This is governed by the yield strength of the steel bar as shown in Eq. (1).

$$N_a \leq \frac{\xi_2 f_y A_s}{\gamma_0} \tag{1}$$

where N_a is the design axial force of the nail;
ξ_2 is a parameter for working condition of steel bar, and can be taken as 0.69 for long-term consideration;
f_y is the design yield strength of steel bar;
A_s is the area of steel bar, and
γ_0 is a parameter quantifying the importance of the slope project, which is equal to 1.1 for very important projects.

The second mode of failure considered is bond failure between the steel bar and the grout material. The limiting bond force is given by:

$$N_a \leq \frac{\xi_3 n \pi d f_b l_a}{\gamma_0} \tag{2}$$

where N_a is the design axial force of the nail;
ξ_3 is a parameter for working condition at the steel-grout interface, and can be taken as 0.6 for long-term consideration;
n is the number of steel bar;

d is the diameter of steel bar;
f_b is the design bond strength at the steel bar-grout interface, which is normally in the range of 2.1-3.4 MPa;
l_a is the length of the steel bar-grout interface, and
γ_0 is a parameter quantifying the importance of the structure, which is equal to 1.1 for very important structures.

The last checking criterion is shear failure along the soil-grout interface. The checking is performed by ensuring that:

$$N_{ak} \leq \xi_1 \pi D f_{rb} l_a \qquad (3)$$

where N_{ak} is the characteristic axial force of the nail;
ξ_1 is a parameter for working condition at the soil-grout interface, and can be taken as 1 for long-term consideration;
D is the outer diameter of the grout;
f_{rb} is the characteristic bond strength at the soil-grout interface, which is specified in a table depending on the soil type and soil conditions, and
l_a is the length of the soil-grout interface.

In Hong Kong, similar guidance is recommended by GEO in the GEO Technical Guidance Note No.23 (TGN 23) (GEO, 2005). This is treated as common practice for design of soil nails in cut slopes. All the three requirements mentioned above are covered in spite of the fact that the design equations are of different forms. For the first two failure modes (Eqs 1 and 2), the design philosophy is quite similar, except that the strength parameters are slightly different from each other. The consideration of the frictional resistance is also somewhat different.

In MCPRC (2002), the frictional resistance along the soil-grout interface is solely a function of the soil type and soil conditions. In other words, the vertical overburden pressure exerted on the nail will not affect the nail capacity. On the other hand, the equation suggested by Schlosser & Guilloux (1981) which is normally used in Hong Kong suggests that the frictional resistance of a nail increases linearly with the effective vertical stress in the soil acting at the mid-depth of the nail in the passive zone. This stress-dependent frictional resistance may result in an increase in the nail capacity, especially for slopes with great height.

Another major difference comes from the design of soil nail heads. MCPRC (2002) does not explicitly recommend the use of soil nail heads, which is one of the major requirements in soil nailing in Hong Kong practice. Notwithstanding these differences in the design methodology, the guideline or code of practice in both places emphasises the important role of in situ soil nail pullout tests, which will be discussed in the next section.

In this project, different configurations of soil nails were adopted depending on the corresponding slope profile. The resulting factor of safety of the entire slope is increased to above 1.4, which was considered adequate.

CONSTRUCTION METHODOLOGY

Construction of soil nails

The general procedure for the installation of soil nails in Mainland China is very similar to that adopted in Hong Kong (see Plate 1). All the works are supervised by an

independent site supervisory company. Hong Kong engineers if employed will act as client's representatives and technical advisors responsible for progress control and clarifications of technical matters.

Plate 1. The slope in Shenzhen prior to soil nail installation.

Soil nail pullout test

As stated in MCPRC (2002), soil nail pullout tests shall be carried out prior to the commencement of site work to verify the design assumptions. When all the soil nails are completely installed, proof tests will be carried out to check the workmanship of the finished product. This is also the case in Hong Kong practice, except for the following differences on the test requirements.

Pullout test:

- Maximum test load shall be lower than 90% of the ultimate design load of the soil nail
- Embedment length of test nail shall be 40%~60% of design length
- Total number of test nails shall not be less than 3
- Failure is defined at the state when the displacement of nails head does not convergence, or the nail is pulled out; or the total displacement is larger than the design requirement; or the displacement increment of one load cycle doubles the value of the pervious cycle
- The elastic displacement of test nails shall not be larger than 80% of the total displacement of the free length portion, and not less than half of the displacement of the length of the free length portion plus half of the bonded portion.
- The difference of ultimate load of individual test nails shall not be over 30%.

Proof test:

- Number of test nail shall not be less than 5% of the total number of nails or 5 whichever is less.
- The test load is set to be $1.1\xi_2 A_s f_y$ for permanent nail and $0.95\xi_2 A_s f_y$ for temporary nail; where A_s is the area of nail, f_y is the design strength of nail and ξ_2 is the working condition coefficient.
- The load increment of first three test cycles shall be 20% of the ultimate design load, while the following cycle shall be 10%. The test nail is accepted if the design load is reached and the displacement rate is zero. The elastic displacement of the test nail

shall not be larger than 80% of the total displacement of the free length portion, and not less than half of the displacement of the length of the free length portion plus half of the bonded portion.

- If nail failure occurs in any of the proof tests, an additional 30% of the total number of soil nails will need to be installed.

Table 1. Procedure for soil nail pullout test

Test Cycle	Percentage of ultimate load (%)												
	Load						Accu. load	Unload					
1	10	20	20				50				20	20	10
2	10	20	20	20			70			20	20	20	10
3	10	20	20	20	20		90		20	20	20	20	10
4	10	20	20	20	20	10	100	10	20	20	20	20	10
Observation Time (min)	5	5	5	5	5	5		5	5	5	5	5	5

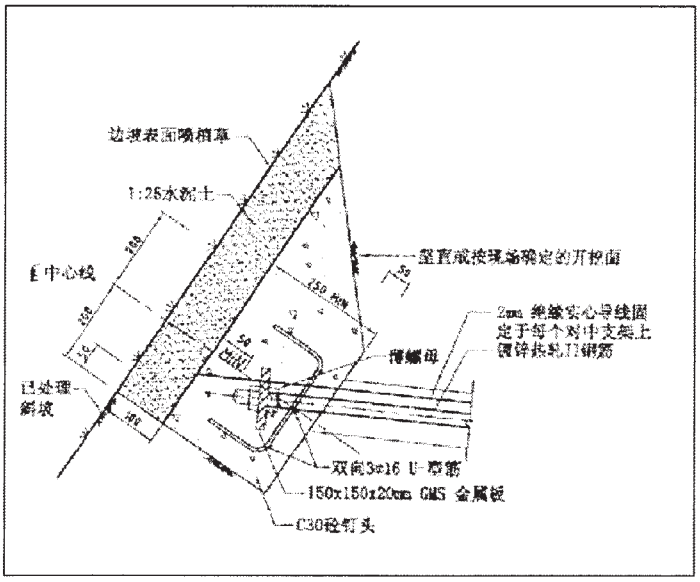

Figure1. Soil nail head design in Shenzhen.

Surface protection

As a global trend, vegetative cover on slope surfaces for erosion control is widely employed in Mainland China, which offers the following advantages:

- Match the surrounding natural topography
- Mitigate for visual impact
- Contribute to environmental sustainability
- Adhere to aesthetic principles

Common types of landscape treatment for slopes in Mainland China are hydroseeding with grass, hydroseeding with trees and shrubs, pit planting, climbing plants on slope surface,

or construction of a reinforced concrete grid structure over the slope surface. The voids in the grillage will be filled with soil and plants established on the filled soil.

In the present study, the site was considered to be far enough from roads and residences, and therefore it was not considered necessary to provide a grid structure over the slope surface. In the design stage, it was decided to sink the soil nail heads 100mm into the soil surface (see Figure 1), and cover them with cement soil. Finally, erosion control mats and hydroseeding were applied above, restoring the appearance to match with the natural environment (see Plate 2).

Plate 2. Erosion control mat covering slope surface before greening.

CONCLUSIONS

Engaging in a construction project in Mainland China may first appear to be challenging due to unfamiliar procedures and practice. This paper summarises some recent experience for a slope project carried out in Shenzhen. It demonstrates that the practice in the two places is, very similar, from general planning, submission to design methods and construction technology. This experience will enhance the confidence of local consultants and contractors in Hong Kong and encourage them to participate more in the construction activities in Mainland China, thus promoting technological exchange.

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ACKNOWLEDGEMENTS

The authors would like to thank Dr. Morgan W.W. Yang for his comments and effort in the preparation of this paper.

BLENDING OUR SLOPES WITH THE ENVIRONMENT - ACHIEVEMENTS, CONSTRAINTS AND CONTINUOUS IMPROVEMENTS

Rebecca Y.H. Chau[1] and Terence C.F. Chan[2]

Abstract: To meet the rising public need for a better living environment, the Government has adopted the policy of making man-made slopes look as natural as possible and blending them with the surroundings. In implementing the slope upgrading works under the Landslip Preventive Measures (LPM) Programme, the Geotechnical Engineering Office (GEO) of the Civil Engineering and Development Department pays attention not only to the stability of the upgraded slopes but also to their appearance. In pursuit of a naturalistic and environmentally conforming appearance for the upgraded slopes, the GEO strives to adopt vegetation as the slope surfacing cover and preserve existing vegetation in the LPM works. Significant efforts have been made in the past years in this respect, bringing about noticeable achievements in the enhancement of slope appearance. However, various constraints have been encountered in implementing the Government policy on slope appearance. To overcome the constraints and to attain solutions that will result in robust, cost-effective, ecologically friendly and self-supporting slope vegetation covers in the long run, the GEO is actively pursuing new slope greening initiatives. With the continued efforts, the GEO hopes to contribute to the creation of a safe, pleasant and sustainable living environment for our future. This paper summarizes the efforts made by the GEO in enhancing the appearance of man-made slopes, identifies the constraints encountered and discusses the new initiatives being pursued for further improvement.

INTRODUCTION

Visually pleasing landscape is one of the key elements for creating a livable environment. The hilly terrain of Hong Kong together with its intense urban development has resulted in a huge amount of slope construction over the past few decades. As man-made slopes exist almost everywhere in the urban area,,their appearance has a significant visual impact on our environment. To minimize the visual impact of man-made slopes and to blend them with their surroundings, the Government has adopted the policy of making man-made slopes look as natural as possible. Establishing vegetation covers on man-made slopes and preserving existing slope vegetation have been the primary aesthetic objectives in slope works.

ACHIEVEMENTS

To pursue the Government policy on slope appearance, the GEO has been making significant efforts in the past years, particularly in a systematic manner since the mid-1990s, to promote the use of vegetation as slope surfacing cover and preserve existing vegetation in

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the LPM works. Over the last three years, the GEO has achieved the target of providing vegetation cover to at least 70% of the upgraded slopes and the existing trees can generally be accommodated on the upgraded slopes (Figure 1).

Figure 1 – Greened slope with existing mature trees.

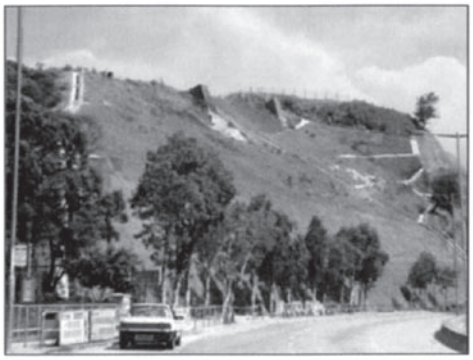

Figure 2 – Screen plantings at slope toe.

Use of Vegetation as Slope Surfacing Cover

The GEO uses vegetation for slope surface protection wherever possible in the LPM works. Shotcrete or other hard surface is used only after other techniques have been explored and found not practical or inadequate on safety grounds, such as at areas susceptible to high infiltration and surface erosion or in the case of emergency repairs (GEO 2004b). To prevent indiscriminate use of hard surface, a departmental vetting committee on slope appearance has been set up since 2001 to review and, where satisfied, endorse every hard surfacing proposal. Where the use of hard surface is unavoidable, landscape measures are provided to mitigate the visual impact of the hard surface. The mitigation measures are mainly in the form of screen plantings at slope toe (Figure 2), berm, crest or on slope surface, and hard surface treatments such as applying subdued colour (Figure 3), masonry facing, or decorative artwork to the shotcreted surface.

Hydroseeding with grass seeds is the basic step to provide an effective slope vegetation cover for erosion protection and has been widely used since the late-1970s. With the aid of erosion control mats and wire mesh, hydroseeding is generally successful on slopes of gradient up to 55°. To enhance aesthetic and biological diversity of the slope vegetation cover, vegetation types other than grass have been introduced into the vegetation mixes of slope greening works since the late-1980s and have become increasingly popular since the mid-1990s (Martin et al. 2001). Seeds of hardy and fast-growing tree species are incorporated into the hydroseed mix, or seedlings of trees, shrubs or climbers are pit-planted onto the hydroseeded slopes.

Attempting to provide reliable vegetation covers on the very steep slopes (slopes steeper than 55°), the GEO undertook a Continuous Improvement Project (CIP) between 1999 and 2001 to review incorporation of landscape and bioengineering practice into the LPM works (GEO 2000a). Initiatives included exploring and experimenting alternative greening techniques to conventional hydroseeding, new erosion control techniques, vegetation types more suitable for steep slope settings, and native vegetation species.

Under the CIP, a number of new greening techniques, including the proprietary systems, were tried on the very steep slopes in the LPM works (Figure 4). Ling et al. (2001) discusses some of these techniques. To provide a comprehensive basis for research, the GEO has set up since 2002 a common database called "Greening Methods on Slopes". This

database co-ordinates updates on the trials of the various new greening techniques on slopes that have been implemented by different government departments and serves as a reference for sharing knowledge and experience on the subject. To investigate the applicability and cost-effectiveness of the slope greening techniques recorded in the database, the GEO conducted a study in 2002 and 2003 in conjunction with the Department of Ecology & Biodiversity of The University of Hong Kong, on the performance review of these greening techniques (Lui and Shiu 2004). The study revealed that most of the techniques could provide a vegetation cover shortly after application, but their ability to sustain long-term vegetation growth was not yet certain.

Figure 3 – Subdued colour on shotcreted surface and tree rings for preserving existing trees.

Figure 4 – Planting on fibre-reinforced soil.

A variety of erosion control mats have been used in combination with hydroseeding since the mid-1980s, largely on a trial and error basis (Martin et al. 2001). The GEO commenced systematic studies of the use of different erosion control products on steep soil slopes since the late-1990s. Li et al. (2000) presents findings of field studies on some of the products. Ling et al. (2001) also indicates that a typical composite erosion control mat system first described in the GEO Report No. 56 (Wong et al. 1999) appears to be effective in establishing vegetation covers on steep slopes up to 60° based on some trials. Further completed studies on the subject include trial of organic biodegradable mats and synthetic non-biodegradable mats on steep cut slopes (Choi and Chau 2004) and the performance assessment of temporary degradable erosion control products and long-term non-degradable erosion control products (Lui and Shiu 2004). These studies indicated that some biodegradable products were not durable enough to perform their intended function, and problems of construction details for some of the products were observed.

For a better integration with the environment, the man-made slopes should blend both visually and ecologically with their surroundings. Enhancing biodiversity through the use of more vegetation types and wider use of native vegetation species are the means towards establishing a self-supporting slope vegetation cover in ecological balance with its surroundings and of minimal long-term maintenance requirement. However, the range of suitable vegetation species for use in greening steep slopes, particularly the native species, have long been limited. Hence, the GEO initiated in 2001 a planting trial of eight different native small tree and shrub species on steep soil cut slopes in collaboration with the Kadoorie Farm & Botanic Garden Corporation (Choi and Chau 2004). The survival rate of the plants under this trial was over 70%. The GEO also conducted a study in 2002 and 2003 in conjunction with the Department of Ecology & Biodiversity of The University of Hong Kong, on the identification of self-sustaining vegetation species on steep man-made slopes (Choi and Chau 2004). Amongst the species identified, over 70% were woody native species.

Over the years, a number of technical guidelines to promote the use of vegetation as slope surfacing cover have been developed. Guidelines on the use of grass, shrubs and trees for slope surface protection were first published in the 2nd edition of Geotechnical Manual for Slopes (GCO 1984). In 1999, prescriptive measures for using vegetation on soil cut slopes of different gradients up to 55° were given in the GEO Report No. 56 (Wong et al. 1999). In 2000, the GEO published a comprehensive guidance document, GEO Publication No. 1/2000, providing practical guidelines with focus on the use of vegetation, in the planning, design, construction and maintenance of works for enhancing the appearance of new and existing slopes (GEO 2000b). Amongst the other guidance documents promulgated, the most recent one is Technical Guidance Note (TGN) No. 20 (GEO 2004a), in which areas for improvement on the application of different greening techniques were identified and a list of native vegetation species that have been proven to be successful and reliable as planned vegetation was recommended for general use in greening slopes.

Preservation of existing vegetation

Existing vegetation, in particular mature trees, helps retain a mature-appearing landscape for slopes and blend the slopes with the surroundings (Figure 1). It also enhances biodiversity of the slope vegetation covers by attracting seed dispersers and increasing seed input for natural regeneration of slope vegetation.

To maximize preservation of existing vegetation in the LPM works, any slope stabilization schemes involving extensive removal of or disturbance to existing vegetation, especially the existing trees, are avoided in the LPM works wherever possible. These environmentally-unfriendly schemes largely include cutting-back of cut slopes, which necessitates excavation of slope surfaces and the natural terrain at the crest, and re-compaction of loose fill slopes, which entails total replacement of the top fill layer and the vegetation thereon. With the advance of slope engineering technology, soil nailing involving no earth removal and hence being more amenable to existing slope vegetation, has been adopted in the upgrading works for most cut slopes and in some cases loose fill slopes.

Where hard surface has to be used for slope surface protection, openings called "tree rings" are provided in the hard surface to accommodate the existing trees on the slope (Figure 3). Where the trees are located at the edge of the hard surface, the hard surface is set back to exclude the trees wherever possible to allow a better growth environment for the trees.

Trees growing on old masonry walls are valuable assets in Hong Kong in respect of both culture and environment. Efforts have been made to retain these wall trees. To minimize disturbance to the root systems of the trees, various methods that preclude wall demolition have been adopted in the upgrading works, including the use of soil nails or wall buttresses, and designing the stabilization works in suitable forms to accommodate the tree roots.

For the trees identified for retention in the upgrading works, various measures are taken to protect them from construction damage, including protecting the tree trunks with hessian wrapping and locating soil nails and other engineering features away from the tree trunks and tree roots as far as possible. Stringent provisions have already been adopted in the LPM contracts to enhance tree preservation and protection during construction.

CONSTRAINTS

Notwithstanding the substantial improvement achieved, the implementation of the Government policy in enhancing slope appearance through providing vegetation as slope surfacing cover and preserving existing vegetation is not free of constraints.

Use of Vegetation as Slope Surfacing Cover

A list of native vegetation species has been published in TGN No. 20 for general use in greening slopes. However, the current plant palette for slope greening works is still very small, limiting the creation of diverse and stable vegetation covers on man-made slopes. Although many species are found growing successfully on steep man-made slopes, their applicability as planned vegetation on slopes is uncertain and their commercial supply in the market is questionable. Working knowledge in this respect remains scarce, demanding further research to improve the current practice.

Although grass hydroseeding has generally been effective in providing a slope vegetation cover, the grass species used in the standard hydroseed mix normally do not perform well under shade. Very often, hydroseeded grass in the areas shaded by canopies of existing trees does not establish successfully (Figure 5). It is necessary to explore suitable shade-tolerant vegetation species for use in this situation.

Figure 5 – Poor performance of hydroseeding under shade and seedling plantings at lower slope portion.

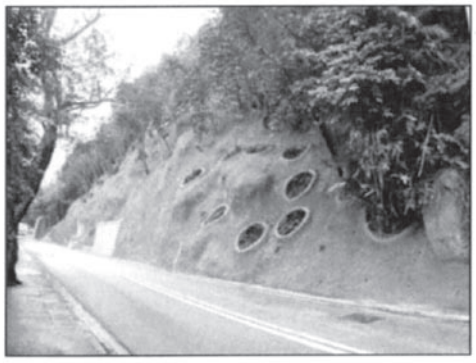

Figure 6 – Empty tree rings become eyesores.

Pit-planting of seedlings on slopes is a labour-intensive technique that is more prone to construction accidents than mechanical hydroseeding. The risk is particularly high on steep slopes and is not acceptable unless working platforms are provided for the workers. Unless the works sequence allows pit-planting to be carried out on the working platforms erected for the soil nailing works, which is not always possible, only the lower slope portions that are accessible from slope toes or slope berms are provided with seedling plantings (Figure 5).

While the public generally prefers vegetated slopes to hard surfaced slopes, some residents near the slopes, particularly those in village houses, hold an opposite view. In such cases, the lower slope batters have to be hard surfaced and only the upper slope portion can be vegetated at most.

Where screen plantings are provided to mitigate the visual impact of hard surface, only moderate success can be achieved in some cases. On many of the slopes provided with climbers in corehole planters on the hard surface, the climbers generally do not survive or they just grow poorly. Possible reasons for the poor vegetation performance in the corehole

planters are the scant planting medium, the exposed condition, and the heat radiant of the hard surface, which rapidly deplete the water available for healthy plant growth. Where toe planters or berm planters are provided for screen plantings, the compact space normally encountered at these locations rarely allows construction of planters of generous width and depth for the growth of large plants such as trees for more effective screening.

Preservation of existing vegetation

Although soil nailing can avoid earth removal on the slope surface and thus help preservation of existing trees on slopes, the drilling process and the soil nail head construction can still cause damage to tree roots. It is not uncommon to find a densely vegetated slope with existing trees growing closely and their roots densely knitted. In this situation, it is very difficult to position the soil nails so that damage to tree roots by the drilling process and the soil nail head construction can be totally avoided.

The use of soil nailing to stabilize a loose fill slope is a suitable alternative to recompaction option in respect of preservation of existing vegetation. However, this alternative option could be one or two times more costly than the conventional recompaction depending on site conditions. The designers would need to strike a good balance between vegetation preservation and cost.

In the case of existing trees growing on old masonry walls, root cutting or other disturbances to the trees to a certain extent are very often unavoidable during wall stabilization works, even if technique such as soil nailing or buttressing is adopted. As the physiology of these wall trees is largely unknown, it is difficult to determine to what extent the tree roots can be cut so that the root:crown balance and hence the stability of the trees are not adversely affected.

Where hard surface is used, tree rings are provided to help preserve the existing trees but the results are not guaranteed. The hard surface around the tree ring hinders soil aeration and water infiltration. The concrete forming the hard surface also changes the soil pH. This modified setting induces stress that hampers the long-term healthy survival of the trees, especially those sensitive species. To the extreme, the stressed trees may die a few years after the hard surface is formed and may even become hazards to the public. The tree stumps left within the tree rings also become eyesores on the slope (Figure 6).

CONTINUOUS IMPROVEMENT INITIATIVES

Although achievements have been made in enhancing the appearance of the upgraded slopes, the GEO sees the need for further improvement. To overcome some of the constraints discussed above and to attain solutions that will result in robust, cost-effective, ecologically friendly and self-supporting slope vegetation covers in the long run, the GEO is actively pursuing new slope greening initiatives to promote the use of vegetation as slope surfacing cover and preservation of existing vegetation.

Use of Vegetation as Slope Surfacing Cover

As follow-on to the previous slope vegetation study carried out in conjunction with The University of Hong Kong, the GEO has an on-going study on the application of various vegetation species for landscape use on man-made slopes. The key objectives of this study are to expand the range of suitable vegetation species for use in greening slopes, and to identify appropriate methods of application and maintenance requirements of the vegetation

species, with emphasis placed on self-regenerating species, in particular native species, and smaller species such as shrub species that are more suitable for use in steep slope settings. In the study, site trials of selected species using different application methods under different slope settings will be conducted. As a side target, the use of the potentially suitable shade-tolerant species will also be explored. It is intended to recommend, based on the study, a user-friendly, extended plant selection matrix giving clearer guidance on the use of appropriate vegetation species on man-made slopes under different site conditions and landscape design intents.

Preservation of existing vegetation

Acknowledging the importance of mature trees in enhancing slope landscape and ecology, the GEO continues to look into both engineering and arboricultural considerations to explore further measures that are more man-made slope specific, that minimize construction impact on existing trees and that enhance the tree resilience to construction damage.

Existing trees in poor condition on slopes are potential hazards to the public. To improve the current practice of assessing tree health, which involves visual assessment of tree defects in a somewhat subjective manner, the GEO strives to develop a tree health assessment methodology that identifies defective trees more objectively and accurately, thereby minimizing tree risk on man-made slopes. To pursue these objectives, the GEO has another on-going study aiming to improve the tree health assessment methodology so that hazardous trees can be justifiably removed or treated and to enhance preservation and protection of the healthy trees that remain.

For enhancing the preservation of the appearance of old masonry walls and the wall trees, the GEO has embarked on a study to review the current practice and to identify improved measures in stabilization of old masonry walls.

CONCLUSIONS

In response to public concern over the appearance of man-made slopes, the GEO has been taking a proactive stance to adopt vegetation as the slope surfacing cover and preserve existing vegetation in the LPM works. Considerable progress has been made in the past decade in this respect. Yet various constraints are encountered, and there is still significant scope for improvement. While a number of continuous improvement initiatives are being pursued, the future must see further advances in nursery production to ensure reliable commercial supply of the suitable vegetation species, in planting technology to enable safer planting on steep slopes, in planter details to enhance the effectiveness of screen plantings where hard surface is used, and in slope stabilization measures as well as construction technology to minimize damage to existing trees.

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ACKNOWLEDGEMENTS

This paper is published with the permission of the Head of the Geotechnical Engineering Office and the Director of Civil Engineering and Development, the Government of the Hong Kong Special Administrative Region.

SAFE AND GREEN SLOPES AT HEI LING CHAU ADDICTION TREATMENT CENTRE

Eddie K K Chan[1], and Paris C W Wong[2]

Abstract: There has been an increased awareness of the environmental aspects of slope upgrading works over the past decade. The surface protection involving either hydroseeding or natural stone facing for six features at Hei Ling Chau Addiction Treatment Centre were studied. These features were upgraded by soil nails prescribed in accordance with GEO Report No. 56 under Agreement No. CAAL014 by Property Services Branch, Architectural Services Department, HKSAR.

INTRODUCTION

Leigh & Orange Ltd was appointed in July 2002 by the Architectural Services Department as the architect on the Consultancy Agreement No.: CAAL014 – Term Consultancy for Minor Works to Government Properties. Ove Arup & Partners Hong Kong Ltd (Arup) is the engineering consultant for this Agreement. Under the Agreement, Arup has undertaken the investigation, design and construction including site supervision, for approximately 100 features in Hong Kong Island and Outlying Islands. Eight features in Hei Ling Chau Addiction Treatment Centre were assigned to be upgraded and six of them (Feature Nos. 10SW-D/C9, C24, C26, C30, C46 and C56) were completed in 2004. Two of them (Feature Nos. 10SW-D/C51 and C54) will be completed in May 2005. Only the six completed features are discussed in this paper.

Hei Ling Chau is a 'green' island which is located to the east of Lantau Island and north of Cheung Chau. Hei Ling Chau Addiction Treatment Centre is a restricted area under the control of Correctional Services Department (CSD). CSD places strong emphasis on its values of economy through optimizing the use of resources and stressing sustainability. In pursuit of its commitment to environmental protection, the CSD has adopted the following principles as set out in Chapter 7, CSD Annual Review 2003. Some of these principles are listed below:

(i) To fully comply with environmental legislation and regulations in the design and construction of new penal institutions, and to reduce their environmental impact to the minimum.
(ii) To meet, and where possible to exceed, the statutory environmental requirements in the operation of all penal institutions; in some institutions where old buildings and outdated facilities no longer meet with environmental standards, to redevelop or to upgrade their facilities accordingly.
(iii) To adopt green management principles in all activities and operations such as conservation of energy and natural resources, waste reduction, recycling and procurement and usage of environmental friendly products and raw materials.
(iv) To increase the environmental awareness of staff and put into practice green measures.

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Two aspects in providing long-term 'safe and green' slope are applied to the features at Hei Ling Chau. Firstly, the long-term structural support technique - soil nails are applied for the slope upgrading works in accordance with GEO Report No. 56 – Application of Prescriptive Measures to Slopes and Retaining Walls. The second aspect is to maintain the slopes in the long term with the vegetation type selected to ensure longevity of growth and the establishment of an ecological equilibrium.

Soil nailing is recommended since it is a cost effective slope upgrading method and it will not alter the existing environmental setting in the vicinity. However, during the soil nailing works, the vegetation or chunam on slope surface will be partly or completed removed or destroyed. In order to compensate and allow recovery of the green slopes, various methods are introduced. For the control of visual impact of new slopes, EWTB Technical Circular No. 25/93 stipulates that the use of chunam or shotcrete on slopes should only be considered as a last resort and only after other techniques have been explored and found impractical. Requirements on vegetation types, species and planting techniques generally followed the rules stated in the GEO Publication No. 1/2000. In particular, hydroseeding should be provided on slope with gradient less than 35°, hydroseeding and geosynthetic erosion control protection mat should be applicable to slope with gradient between 35° and 45°, hydroseeding and geosynthetic erosion control protection mat together with wire mesh should be applicable to slope with gradient between 45° and 55°, stone facing could be applicable to slope with gradient over 55°.

SITE LOCATIONS AND SITE DESCRIPTIONS

The six features located at Hei Ling Chau Addiction Treament Centre. The location of the features is shown in Figure 1. The relevant information of the features is summarized in Table 1 below:

Table 1. Summary of slope information

Feature No.	Height (m)	Length (m)	Average Gradient (max.)(°)	Crest Facility	Toe Facility	Original Slope Surface Protection
10SW-D/C9	4	20	50 (50)	Low usage footpath	Residential	Vegetated
10SW-D/C24	5	50	55 (55)	Undeveloped green belt	Dental clinic	Shotcrete
10SW-D/C26	4.5	45	40 (60)	Undeveloped green belt	Psychological unit	Vegetated
10SW-D/C30	3.8	40	45 (75)	Open space	Bungalow A	Vegetated
10SW-D/C46	3.6	40	40 (70)	Undeveloped green belt	Bungalow B	Shotcrete
10SW-D/C51	3.0	37	50 (60)	Undeveloped green belt	Workshop	Shotcrete
10SW-D/C54	4.0	20	45 (45)	Open space	Dormitory	Chunam
10SW-D/C56	10	25	40 (50)	Low usage footpath	Greenhouse	Shotcrete

Note: C51 and C54 will complete upgrading works in May 2005

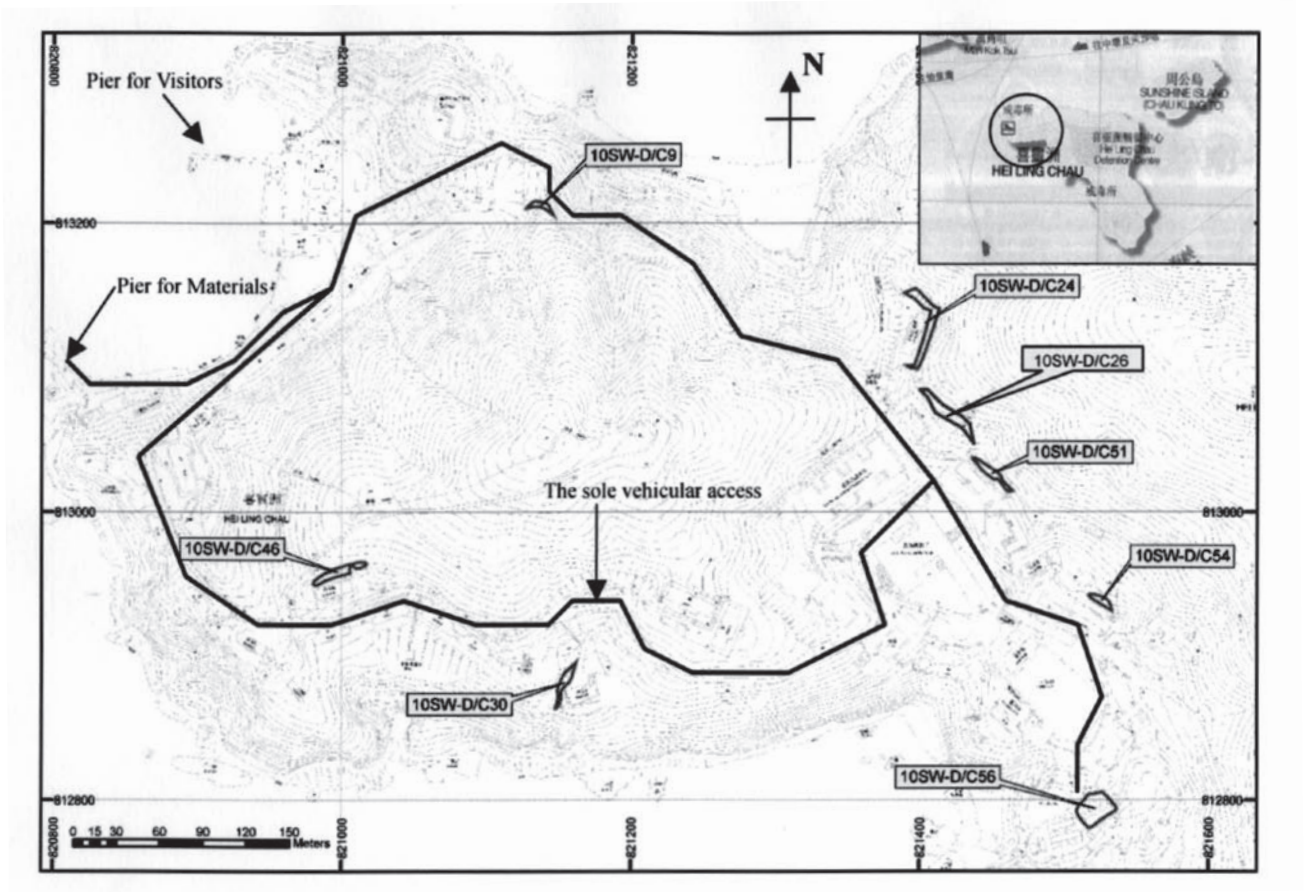

Figure 1. Site location plan.

GEOLOGY OF HEI LING CHAU

According to the Pre-Quaternary Geology of Hong Kong (2000), the feature is underlain by the East Lantau Rhyodacite. The East Lantau Rhyodacite consists of Feldsparphyric Rhyodacite to Porphyritic Granite dykes. The 1:20,000 Geological Map Sheet 10 (1991) suggests that the feature is underlain by Feldsparphyric Rhyolite (rf) of Jurassic to Cretaceous in age.

STRUCTURAL SUPPORT OF THE SLOPE TO ACHIEVE "SAFE" STANDARDS

Based on the observation, review and interpretation by a geotechnical engineer on information obtained from slope inspections, it can be concluded that:

(i) The consequence of failure and geometry of the slope are within the limits prescribed in GEO Report No.56 for prescriptive measures, that is for Group 1 facilities affected the slope, the height of the slope is ≤10m;
(ii) The slope-forming material is granitic material which complies with the requirements of GEO Report No. 56;
(iii) No adverse geological materials and adverse discontinuities were recorded;
(iv) No adverse groundwater conditions were recorded.

The qualifying criteria of GEO Report No. 56 have been satisfied, therefore, prescriptive upgrading works can be used to improve the condition of the features. No ground

investigation was proposed and carried out under the agreement. This results in less impact to the environment. After the slope upgrading works have been completed, the increase in factor of safety would be in the range of 0.3-0.5 and it meets the current geotechnical requirements.

The upgrading works and surface protection applied to the six features are presented in Figures 2-4 and summarized in Table 2 below:

Table 2. Summary of slope upgrading works

Feature No.	Structural upgrading works	Subsurface drainage	Surface drainage	Surface protection
10SW-D/C9	Installation of 49 nos. of 25mm diameter, 6m long soil nails	Installation of 5 nos. of 5m long prescriptive type 3 raking drains	Provision of crest and toe channels	100% provision of hydroseeding with erosion control mat and wire mesh
10SW-D/C24	Installation of 121 nos. of 25mm diameter, 8m long soil nails	Installation of 8 nos. of 5m long prescriptive type 3 raking drains	Provision of crest and toe channels	100% provision of hydroseeding with erosion control mat and wire mesh
10SW-D/C26	Installation of 77 nos. of 25mm diameter, 6m long soil nails	Installation of 10 nos. of 5m long prescriptive type 3 raking drains	Provision of crest and toe channels	60% provision of hydroseeding with erosion control mat and wire mesh; 40% provision of stone facing
10SW-D/C30	Installation of 62 nos. of 25mm diameter, 6m long soil nails	Installation of 6 nos. of 5m long prescriptive type 3 raking drains	Provision of crest and toe channels	50% provision of hydroseeding with erosion control mat and wire mesh; 50% provision of stone facing
10SW-D/C46	Installation of 44 nos. of 25mm diameter, 5m long soil nails	Installation of 8 nos. of 5m long prescriptive type 3 raking drains	Provision of crest and toe channels	50% provision of hydroseeding; 50% provision of stone facing
10SW-D/C56	Installation of 90 nos. of 32mm diameter, 11m long soil nails	Installation of 12 nos. of 7m long prescriptive type 3 raking drains	Provision of crest and toe channels	Provision of hydroseeding with erosion control mat and wire mesh

The general view of the features before and after slope upgrading works is illustrated in Plates 1a to 6a and 1b to 6b, respectively.

SITE CONSTRAINTS

Hei Ling Chau Addiction Treatment Centre is on an Outlying Island with no fixed link to the mainland Island. Access to the island is a significant problem. All of the construction materials such as cement, steel bars and plant must be transported to Hei Ling Chau by sea. The sole pier is located to the west of the island. All of the construction materials transported to the site must be via this pier with prior permission from CSD. The selected features are located at the mid level of the island as shown in Figure 1. The only vehicular access leading to the site is a 3m wide single lane road. The access must be kept clear all the times for emergency purposes. Only trucks of maximum laden load of 3 tonnes with CSD's permission, are allowed to be transported to the island for use as access vehicles.

Feature No. 10SW-D/C9 is located adjacent to the vehicular access. Transportation of construction materials to the site is not difficult. However, part of the feature is facing the access road, so drilling works and insertion of soil nails affect the access road of the island. Temporary traffic management is required with the permit from CSD. In addition, the working hours are restricted to 2-3 hours on Saturday. Weekdays working is not allowed at this portion of feature.

Feature Nos. 10SW-D/C24 and C26 are not close to the access road. The construction materials were delivered to the closest loading and unloading point and were then manually carried to the working site by labourers.

Sensitive buildings were found in front of the features. The dental clinic at the toe of Feature No. 10SW-D/C24 has a number of visitors each day. The psychological unit in front of Feature No. 10SW-D/C26 is very sensitive to noise. Bungalow B at the toe of Feature No. 10SW-D/C30 is a conference room for seminars. Bungalow A at the toe of Feature No. 10SW-D/C46 is the hostel of the CSD commander. The plants in a greenhouse at the toe of Feature No. 10SW-D/C56 are sensitive to soil nail works including nuisance caused by wastewater and dust. Therefore, the construction period is restricted to suit the daily operation of the sensitive buildings. A detailed construction programme is submitted to CSD for approval prior to the commencement of the works.

Water used for construction purposes such as cement grouting is another problem. Since water supply from Water Supplies Department to Hei Ling Chau is limited, the sole water source for purposes other than drinking and food is from a disused reservoir, which is located at the peak of Hei Ling Chau. The contractor is reminded to obtain adequate volume of water prior to the commencement of cement grouting. The volume of water to be used shall be kept to a minimum to prevent the production of wastewater and the wastewater must be properly desilted before discharging to the existing drainage system. All the construction wastes shall also be properly bagged and transported by barge for delivery to the nearest designated dumping area.

SURFACE PROTECTION

Two typical solutions to provide surface protection were adopted in this project, namely hydroseeding and stone facing with provision for planting as the first priority as far as possible.

The advantages of application of hydroseeding instead of a hard surface are numerous. It makes the slopes look green and natural. The vegetation can protect the soil surface from

being eroded by inclement weather. Moreover, by selection of appropriate species of plants, it can provide a better environment for wildlife. Colonization by native plant species will occur over time and therefore protect the ecological cycle in Hei Ling Chau.

Stone facing is another option for slope surface protection. It is normally applied to slope with gradient over 55°. The advantage of the application of stone facing is to provide a comparatively hard surface but aesthetically pleasing and economic protection to the slope surface. The erosion of bare slope surface due to inclement weather is minimal. In addition, stone facing can be constructed to different pattern to blend into existing environment and aspect. Climbers planted in specific planter holes and toe planters are additional measures to "green" stone facing.

UTILITIES

During the study of slope upgrading works, a lot of unrecorded utilities including water-carrying pipelines, LPG gas pipelines, electricity cables, and telephone cables were found in the vicinity of the slope as well as on the slope surface. These services ducts will affect the appearance of the slopes and the application of surface protection. The potential water leakage from the water-carrying pipelines may affect the stability of the slope. During the slope upgrading works, all these used and or disused utilities ducts are either realigned or removed from the slope surface or vicinity of slope. The damaged or buried water-carrying pipelines are replaced and reinstalled properly so that they will have minimal impact to the stability of the slope.

COMPARISON OF PRESCRIPTIVE DESIGN AND CONVENTIONAL ANALYTICAL DESIGN

The nail lengths of the six features based on prescriptive design are compared with those based on conventional design. Based on the typical soil strength parameters referred in Table 8 of GEOGUIDE 1 – Guide to Retaining Wall Design, soil strength parameters cohesion, c'=5kPa, and friction angle, ϕ'=35° and the bulk unit weights γ=19kN/m³ of the granitic materials are adopted. The groundwater table is assumed to be at one-third of slope height. It was found that the numbers of soil nails, vertical and horizontal spacing of both approaches are similar. Only the length of the soil nail was found to be reduced by 20% to 50%. The results are shown in Table 3 below:

Table 3. Comparison of soil nails by prescriptive and conventional design

Feature No.	Length of soil nail by prescriptive design (GEO Report No. 56) (m)	Length of soil nail by conventional design (m)	Prescriptive design vs conventional design
10SW-D/C9	6	4	1.50 : 1
10SW-D/C24	8	6	1.33 : 1
10SW-D/C26	6	5	1.20 : 1
10SW-D/C30	6	5	1.20 : 1
10SW-D/C46	5	4	1.25 : 1
10SW-D/C56	11	8	1.37 : 1

However, the time taken for carrying out ground investigation and laboratory testing is saved when prescriptive design is adopted. Figure 2 shows the comparison of the procedures for conventional analytical design and prescriptive design. Since no ground investigation work was carried out, the average construction period of each feature was about 60 days. Time saving from 1 month to up to 6 months is resulted for no ground investigation works.

CONCLUSION

Hei Ling Chau is identified as a 'green' island. Designers of construction works including those on man-made slopes are strongly encouraged to use natural materials as much as possible to improve the environment. Two aspects were emphasized during the slope upgrading works. Firstly, soil nailing was used as the long-term structural support technique due to its cost effectiveness and robustness, simplicity, and the least adverse visual impact to the existing environment. The second aspect is to maintain the slopes 'green' and natural. Hydroseeding with erosion control protection mat together with wire mesh was used for slopes with gradient lower than 55^{o}. Stone facing with planter hole and/or toe planter was applied to slopes with gradient over 55^{o}.

The construction cost and procedures of prescriptive design and conventional analytical design are also compared. Although the construction cost of soil nails designed by using prescriptive method is 20% to 50% more than that of conventional analytical method, prescriptive design was adopted to shorten the construction time and minimize the impact to the existing environment.

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ACKNOWLEDGEMENTS

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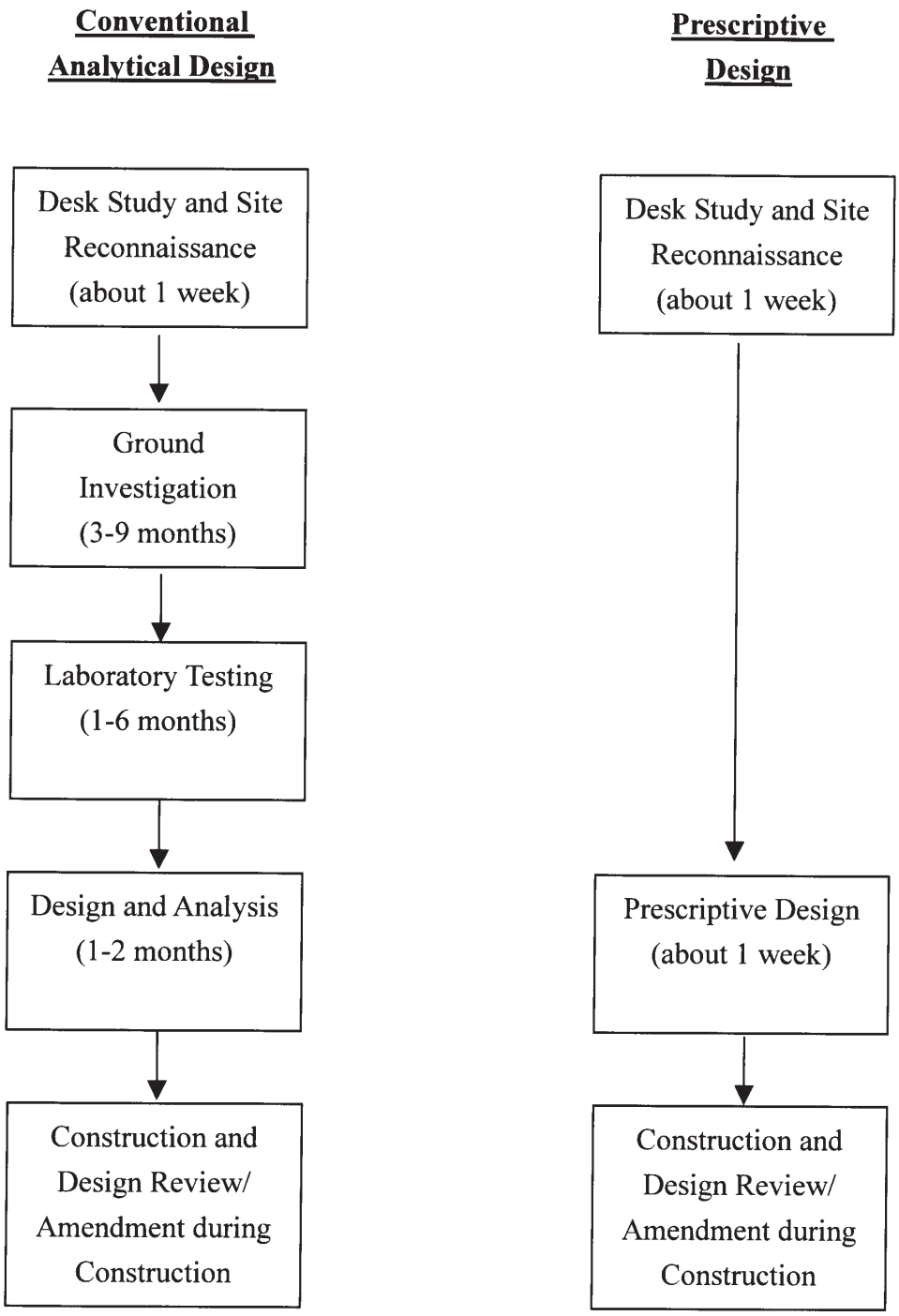
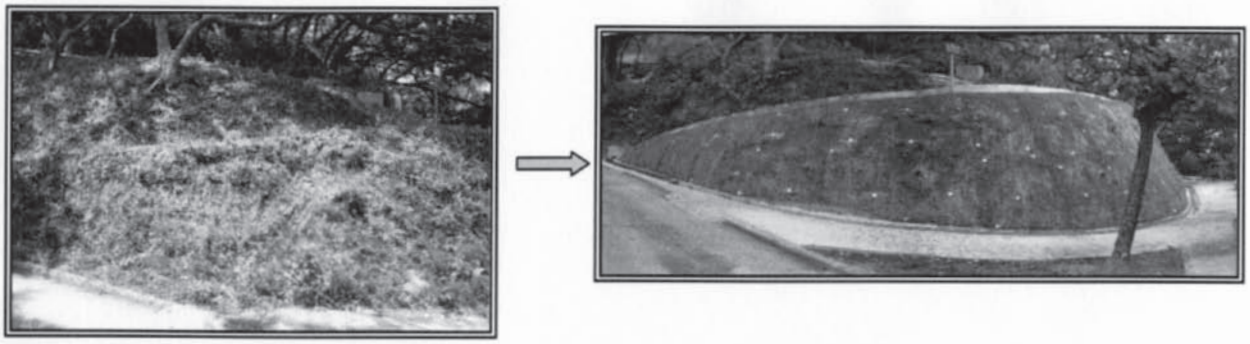
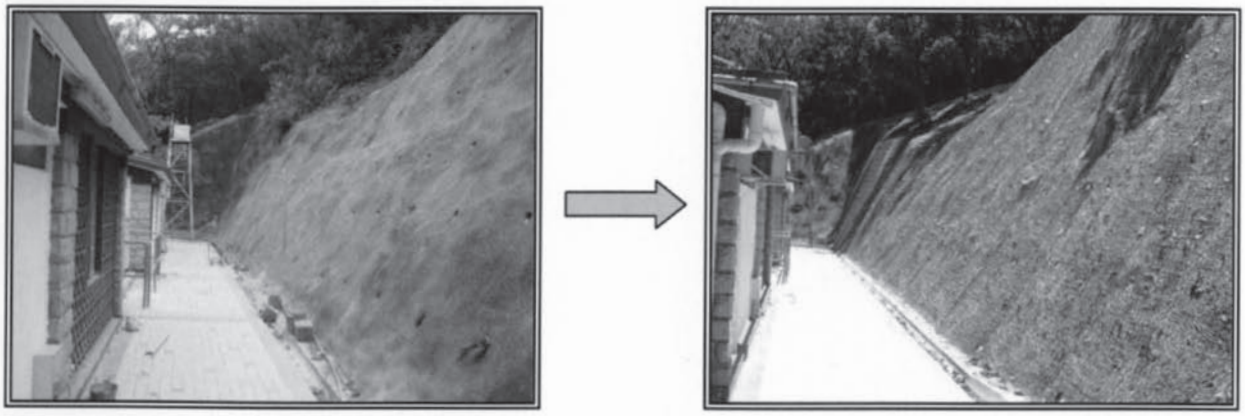


Figure 2. Comparison of the procedures for conventional analytical design and prescriptive design (extract from GEO Report No. 56)

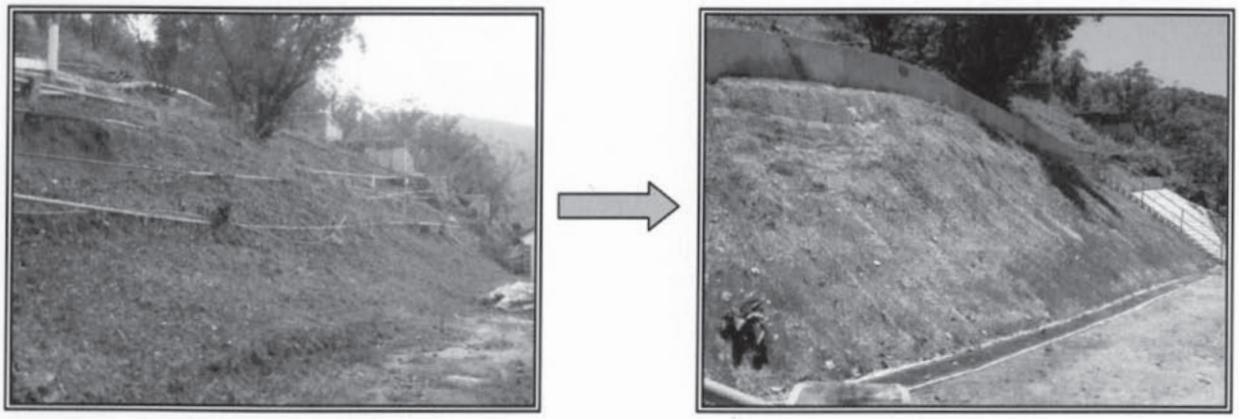
Before Slope Upgrading Works **After Slope Upgrading Works**



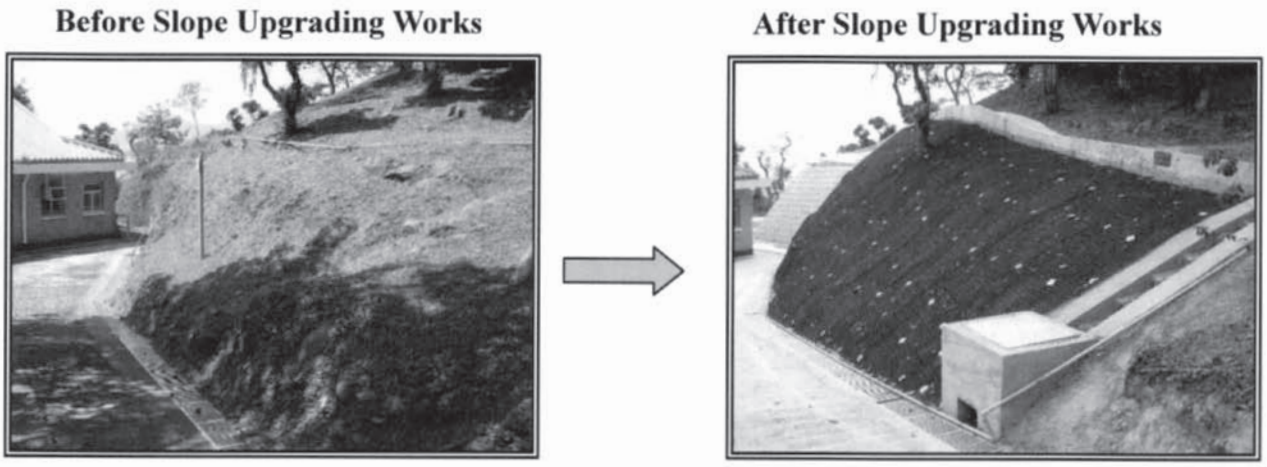
Plates 1a and 1b. Feature No. 10SW-D/C9



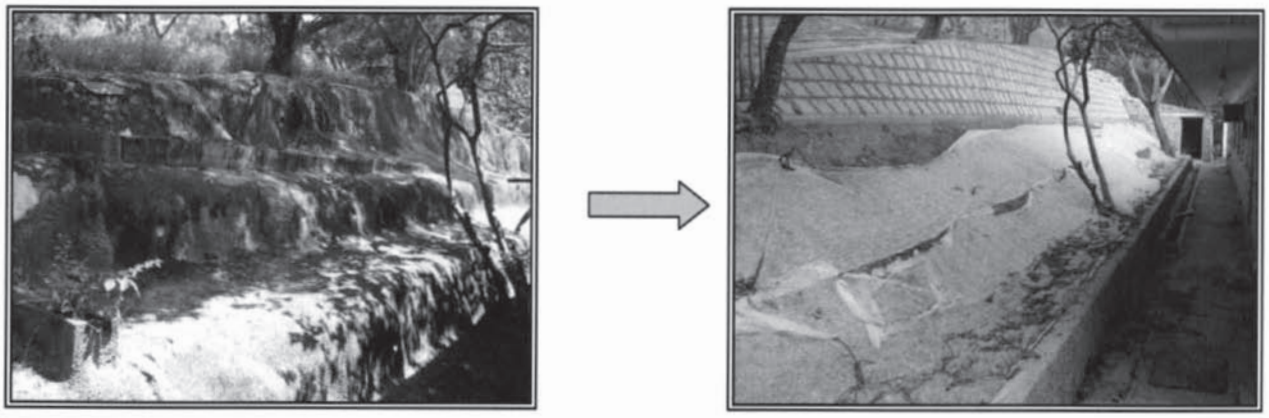
Plates 2a and 2b. Feature No. 10SW-D/C24



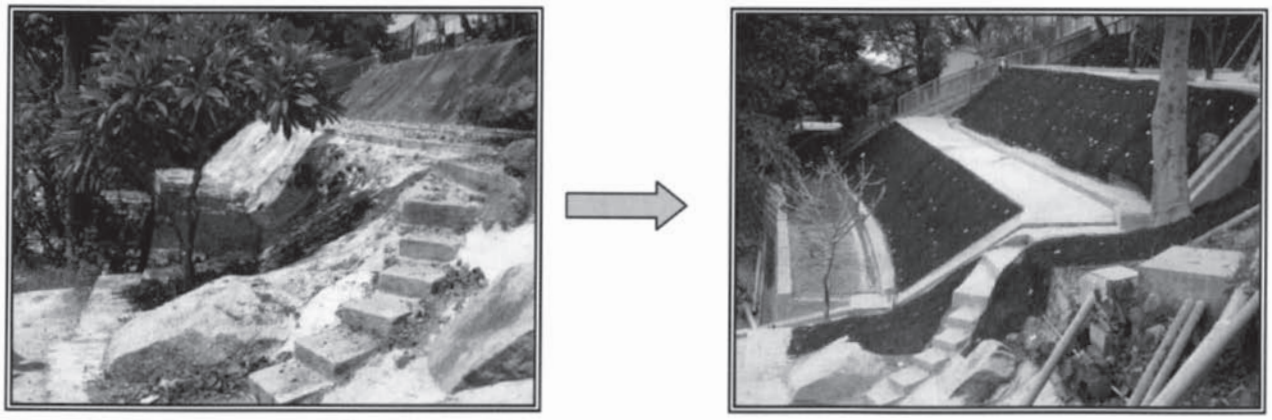
Plates 3a and 3b. Feature No. 10SW-D/C26



Plates 4a and 4b. Feature No. 10SW-D/C30



Plates 5a and 5b. Feature No. 10SW-D/C46



Plates 6a and 6b. Feature No. 10SW-D/C56

USING NATIVE TREE AND SHRUB SPECIES FOR ECOLOGICAL REHABILITATION OF MAN-MADE SLOPES IN HONG KONG

Billy C.H. Hau[1], Ken K.Y. So[2], K.C. Choi[3] and Rebecca Y.H. Chau[4]

Abstract: Apart from ensuring public safety, minimizing disturbance to local community during slope works and enhancing slope appearance, the SAR government is also mindful of the need for ecological rehabilitation. In theory, using native tree and shrub species on slopes will not only benefit local ecosystem, the eventual plant community established shall be self-sustaining and require minimum maintenance. In view of the lack of information on native plant species on man-made slopes in Hong Kong, a field trial using native small tree and shrub species at Yuen Tun, Tai Lam and surveys of naturally-occurring plant species on 20 man-made slopes in Hong Kong were conducted in the last few years by the government. On the other hand, there are comparatively more research works done on using native tree species in rehabilitating our degraded hillside slopes in Hong Kong. The results of these studies show that common hillside species, whether small trees or shrubs, are particularly adaptable on man-made slopes. Many species that are common in disturbed habitats in Hong Kong are also suitable. Many of the species that can establish naturally on man-made slopes produce fleshy fruits. This suggests that animal seed dispersal is important for ecological succession on man-made slopes. A list of native small tree and shrub species that is suitable for wider application or potentially useful on slope rehabilitation in Hong Kong is proposed.

INTRODUCTION

Since the end of the last decade, there has been stronger public demand for greened slopes. New guidelines were produced by the government to promote greened man-made slopes in recent years (e.g. Geotechnical Engineering Office; 1999; 2000; 2004a). Hydroseeding with a mixture of grass and tree seeds or planting tree seedlings on an already grassed slope were the conventional methods for slope greening in Hong Kong (Geotechnical Engineering Office; 2004b). There are many successful examples of establishing dense vegetation on man-made slopes with a gradient less than 45° with these

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methods. The current guidelines discourage planting trees on slopes steeper than 45° because when the trees mature, they add weight on the steep slope and are prone to strong wind which may cause damages to the slope (Geotechnical Engineering Office; 2004b). Exotic tree species are commonly used because their survival and growth rate are higher and their seeds and seedlings are commercially available.

Planting trees on man-made slopes is a management measure of land rehabilitation. In view of the many concepts and definitions on the topic of land rehabilitation, which are often confusing, Brown and Lugo (1994) have defined the various extent of land rehabilitation. Reclamation is the lowest level of land rehabilitation where only the productivity and biomass of the degraded land are restored (Figure 1). Thus, planting exotic trees on man-made slopes in Hong Kong without any longer-term measures to transform the eventual exotic plantation into a fully functional ecosystem is merely reclamation. Apart from ensuring public safety, minimizing disturbance to local community during slope works and enhancing slope appearance, the SAR government has pledged to integrate ecological enhancement in slope works in Hong Kong (Geotechnical Engineering Office; 2004b). As such, ecological rehabilitation should be attempted. To restore fully functional ecosystems on man-made slopes, the restored vegetation should be directed towards the formation of shrublands or young forests that are largely comprised of native or non-invasive naturalized exotic species. Given time, these shrublands and young forests will mature even without human intervention and achieve the ultimate level of land rehabilitation i.e. restoration. With such a vegetation cover, the system is fully functional and shall not require recurrent maintenance just like many of the natural vegetated hillside slopes in Hong Kong.

There are two general pathways of ecological rehabilitation on man-made slopes. Firstly, it could be initiated by first establishing an exotic or mixed native-exotic tree and shrub covers to ameliorate the poor condition for plant growth and establishment on the slope surfaces. Then, management measures such as thinning of the exotic trees and enrichment planting of native tree and shrub species should be introduced after the plantation canopy has been formed. This pathway is considered more appropriate for badly degraded sites that are far away from natural seed sources where natural succession will be slow or difficult to start without human intervention along the way. Alternatively, planting native shrub and tree species right away on man-made slopes that are not very exposed and are close to natural seed sources is a more direct approach of rehabilitation. However, whilst there are some research works done on using native tree species in rehabilitating our degraded hillside slopes in Hong Kong, information on native plant species on man-made slopes is generally lacking. This hinders the development of ecological rehabilitation of man-made slopes in Hong Kong. In view of this, the government initiated a field trial using native small tree and shrub species at Yuen Tun, Tai Lam in 2001 and a survey of naturally occurring plant species on 20 man-made slopes in Hong Kong in 2002 (see Choi and Chau

2004). The aim of this paper is to review the findings of these two studies and draw on the results of previous researches on ecological rehabilitation on natural hillsides to formulate a list of native small tree and shrub species that is potentially useful on slope rehabilitation in Hong Kong and identified species that are suitable for wider application.

THE YUEN TUN PLANTING TRIAL

The main objective of the trial is to study the growth and establishment of native small trees and shrubs on steep slopes. The planting trial was conducted on three soil cut slopes at Yuen Tun inside Tai Lam Country Park (Choi and Chau 2004). Prior to the planting trial, the slopes were upgraded under the LPM programme involving trimming back the slopes to a flatter angle of 50° mechanically by backhoe. In general, the slope forming material is described as light yellowish brown completely decomposed fine to medium grained granite. A few quartz veins are exposed on the trimmed slope surface. Classification tests and chemical analysis of soil samples trimmed from the slope showed that the soil is coarse gravely clayey sand with a trace of organic matter (Choi and Chau 2004). The average pH value of the soil is 5.1 indicating that it is slightly acidic. However, this is very typical of the soil on degraded hillside in Hong Kong (Dudgeon and Corlett 2004). Hydroseeding was applied and seedling shrubs and small trees were planted at approximately 0.8 m horizontal and 0.6 m vertical spacing after establishment of the grass cover on the hydroseeded slopes. A total of 1,241 seedlings comprising 8 native species were planted. All seedlings were produced in the Native Tree Nursery of the Kadoorie Farm and Botanic Garden and seeds were collected from Hong Kong. No maintenance work other than the removal of the weed *Mikania micrantha* from the panels was provided for the planting trial. Regular monitoring of the survival and growth of the plants were conducted.

The results of the planting trial show that the survival rates of the species planted were very high after three years (58-92 %; Table 1). The eight small tree and shrub species used have much higher survival than many other native tree species used on previous natural hillside planting trials in Hong Kong. Their growth rates were, however, comparable to these other native tree species. The comparatively poorer performance of *Rhaphiolepis indica* amongst the eight species may be attributable to the small seedlings used due to supply problem. The good performance of the species used at Yuen Tun suggests that shrub and small trees are more adaptable to severely degraded land such as man-made slopes. This fits in with the forest succession pathway whereas shrubs will establish prior to trees. Early studies in Hong Kong have already shown that in the absence of fire, degraded forest land with nearby seed sources will develop from grassland to shrubland in 5 to 10 years and to forest in another 20 to 30 years (Zhuang & Corlett, 1997). Shrubs may in fact be a better choice than tall trees for steep slopes as they will form a lighter vegetation cover than trees.

It should be noted that due to the death of some seedlings and the not very compact planting density, the canopy layer on the slope surface is not well formed. Replacement planting may be carried out 3 to 4 years after the original planting to fill in the gaps.

THE SURVEY OF NATURALLY OCCURRING PLANT SPECIES ON MAN-MADE SLOPES IN HONG KONG

Plant species, whether they are native or naturalized exotic, that can establish on man-made slopes naturally in Hong Kong would appear to be able to adapt to the harsh growing environment of such habitats. The primary aim of this survey was to identify naturally occurring plant species on relatively old soil slopes in Hong Kong. Twenty slopes that are inter-dispersed in Hong Kong were selected from the government slope register and surveyed in 2002/2003. All 20 slopes are cut slopes that are formed by trimming of the unstable surface and two have retaining walls at the toe. These were primarily soil slopes with ages ranging from three year to over 24 year old. All batters of all slopes were surveyed as far as is accessible. All plant species were counted, recorded into different height classes and identified on-site or collected for herbarium checking when in doubt.

The number of naturally occurring plants recorded on the 20 slopes includes 7,694 individuals of 52 native tree species and 35 native shrub species; 776 individuals of 30 native climber species and 761 individuals of 12 naturalized exotic species. Amongst the species recorded, 22 native species are considered common and widespread on the surveyed slopes (Table 2). Only two of these 22 species are typical forest species (*Psychotria asiatica* and *Ficus variegata*). 14 others are very common in hillside shrublands in Hong Kong and 6 others are common in disturbed habitats and wasteland. These results show that native woody plant species with strong pioneer characters in natural and disturbed forests/shrublands are establishing well on these man-made slopes. Young seedlings of eight of these 24 species were also very common on newly formed or upgraded slopes in Hong Kong (Lui and Siu, 2004). This suggests that these eight species, which are dispersed by either bats or birds in Hong Kong, are naturally dispersed to man-made slopes and are able to germinate, grow and establish well on such habitats. This also highlights the potential of using these eight native tree species on gentle soil slopes to promote natural succession. Of these eight species, *Mallotus paniculatus*, *Bridelia tomentosa*, *Litsea glutinosa* and *Sterculia lanceolata* have been tried in afforestation projects in Hong Kong with some success (Hau & So 2003). No published planting trial data are available for the other four species. Two well known invasive alien species (*Lantana camara* and *Mikania micrantha*) were common on the studied slopes and their establishment on slopes should be controlled.

THE USE OF NATIVE TREE SPECIES IN HILLSIDE REHABILITATION IN HONG KONG

Hau and So (2003) have reviewed seven planting trials for natural hillside rehabilitation in various parts of Hong Kong ranging from 70 m to 550 m above sea level by different local institutions. All trials were conducted on typical degraded hillside sites that were covered with either or both common shrubs (e.g. *Rhodomyrtus tomentosa*, *Melastoma sanguineum*, *Baeckea frustescens* and *Litsea rotundifolia*) and grasses (e.g. *Arundinella*, *Ischaemum*, *Eulalia*, *Eragrostis*, *Cymbopogon* and *Miscanthus* species). Container-grown seedlings of 30-50 cm in height were planted in these seven sites. Grass was cut in advance and grass leaves were left on site as mulching. The total number of seedlings planted at each site ranges from 300 to 1,000. A total of 49 native trees were used in these trials. Most of them bear fleshy fruits that are dispersed by birds in the winter dry season. The tree species used comprise most of the ecological characteristics of the tree flora in Hong Kong including early pioneers, common secondary forest dominant species and remnant forest species. Thirty species were used only in 1 site; 12 in 2 sites and 14 in 3 or more than 3 sites.

Of the 14 species that were planted in 3 or more than 3 sites, 5 species had consistently high survival (50-100 %) across all or most of the sites they were planted (Table 1). They are *Machilus breviflora* (5 sites), *Schefflera heptaphylla* (4 sites), *Choerospondias axillaris* (4 sites), *Cinnamomum camphora* (4 sites), *Schima superba* (4 sites) and *Mallotus paniculatus* (5 out of 6 sites were good). The rest of the 14 species had variable survival in the sites planted. On the other hand, 4 species had consistently high growth (RHI > 0.4) in all sites they were planted. They are *M. breviflora* (5 sites); *S. heptaphylla* (4 sites); *Cyclobalanopsis championii* (3 sites) and *Gordonia axillaris* (2 sites). The rest of the species had variable growth in the sites planted. Seven species used in these trials were found as common and widespread in the slope survey above (Table 1). Of these 7 species, the performance of *M. paniculatus*, *S. heptaphylla*, *B. tomentosa* (1 site), *Celtis sinensis* (1 site), were good. *Sterculia lanceolata* (5 sites) and *Cratoxylum cochinchinense* (2 sites) were variable. *Sapium discolor* (3 sites) was consistently bad.

DISCUSSION

To take an additional step in slope greening by improving the ecological values of future man-made slopes, those native tree and shrub species that are successful at the Yuen Tun planting trial are recommended for wider use. In addition, species that is found common and widespread in the 20 surveyed slopes, successful in at least one previous planting trial for natural hillside rehabilitation in Hong Kong in Hau and So (2003) and known to perform well on man-made or natural slopes in Hong Kong from other studies are also recommended for wider use on man-made slopes (Table 3). However, *Melastoma sanguineum*,

Rhodomyrtus tomentosa and *Microcos paniculata* are also recommended for wider use. The former 2 species are commonly planted on man-made slopes in Hong Kong and the latter is very successful in a planting trial in Aberdeen Country Park which is not reported in this paper (unpublished data).

On the other hand, those species on the second part of Table 3 are recommended for further field trials. They are either found on the 20 studied slopes or reported to be successful in at least one previous planting trial for natural hillside rehabilitation in Hong Kong by Hau and So (2003) or known to perform well on man-made or natural slopes in Hong Kong from other studies.

It should be borne in mind that no two sites are identical and the planting mix should be determined according to site characteristics and targets of the planting work whereas certain exotic species may still be more preferable than native ones particularly in badly degraded sites such as very exposed slopes and rock quarry fill slopes. In some cases, soil amelioration such as fertilization may be required to ensure better establishment of the planted vegetation cover. More native small tree and shrub species should be tried in future greening projects such that a longer list of usable native species on man-made slope could be made. Lastly, with respect to steep man-made slopes, allowing natural succession to proceed freely may not be always desirable. Any large tree species that can establish naturally on steep man-made slopes may threaten the stability of the slope. Longer term monitoring or management may be needed to control the establishment of large tree species if a sustainable vegetation cover comprising native small trees or shrubs are provided. Further studies are obviously required.

ACKNOWLEDGEMENTS

This paper is published with the permission of the Head of the Geotechnical Engineering Office and the Director of Civil Engineering and Development, the Government of the Hong Kong Special Administrative Region.

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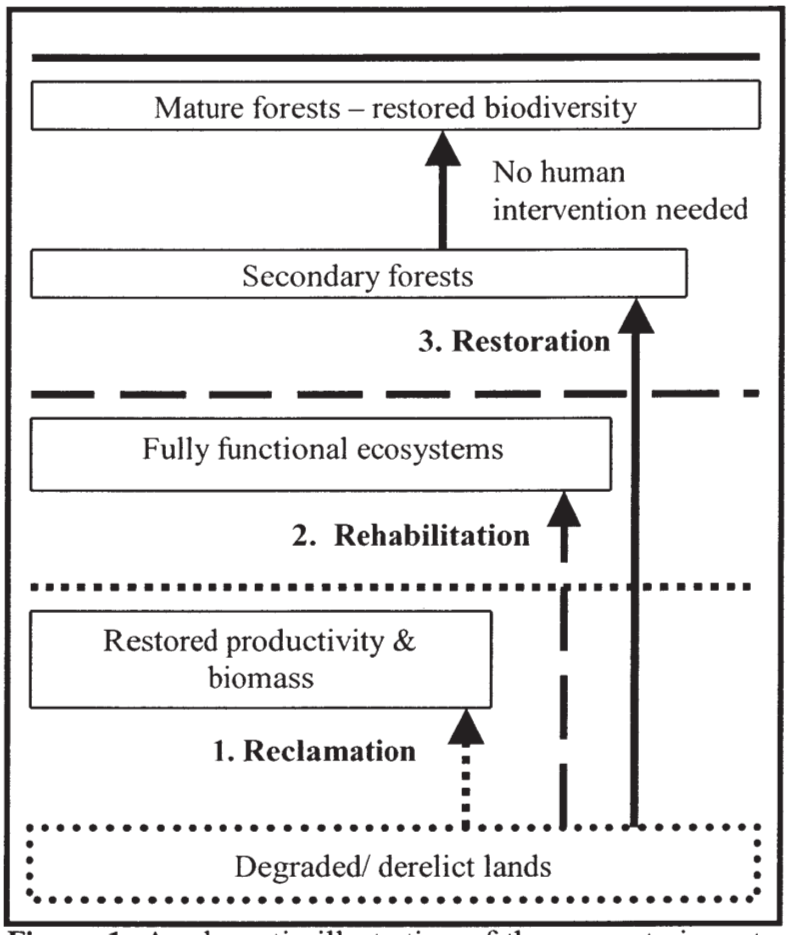

Figure 1. A schematic illustration of the concepts in restoration ecology on degraded land. Reclamation refers to the establishment of a tree stand, usually with fast growing exotic tree species, to restore productivity and biomass. Rehabilitation is the transformation of a degraded system into a fully functional ecosystem irrespective of its original state or desired state. Restoration refers to the return of a converted system into its presumed or relatively original indigenous state. (Adopted from Brown and Lugo 1994).

Table 1. Survival (% S) and growth (Relative height increment-RHI; N is the sample size) of each species at each site. Data from the first two years after planting were used in the computation except at Yuen Tun where the first three years' data were used. The relative height increment per year (RHI) was calculated using RHI = [ln (H_2) – ln (H_1)] / time in years, where H_1 and H_2 were the initial and final seedling heights respectively (Coomes & Grubb, 1998). "*" indicates species that were recorded as common and widespread on the 20 old soil slopes surveyed (Choi and Chau 2004).

Species	Yuen Tun			Kwun Yum Shan 1			Kwun Yam Shan 2			Tai Mo Shan 1			Tai Mo Shan 2			Ho Sheung Heung			Nam Shan			Hung Lung Hang		
	N	RHI	% S	N	RHI	% S	N	RHI	% S	N	RHI	% S	N	RHI	% S	N	RHI	% S	N	RHI	% S	N	RHI	% S
Antirhea chinensis																						6	0.08	16.67
Aporosa dioica																						100	0.18	73
Aquilaria sinensis																						10	0.1	50
Archidendron lucidum				20	0.79	89																		
Ardisia crenata	63	0.32	73.7																					
Bischofia javanica				20	0.83	100													9	0.81	22.22			
Bridelia tomentosa *				10	0.71	90																		
Castanopsis fissa				20	0.27	10	100	0.39	26	150	0.26	82				50	0.42	74	174	0.27	22.29			
Celtis sinensis *				20	0.69	95																		
Choerospondias axillaris							40	0.41	97.5	40	0.38	97.5				40	0.29	100				140	0.4	100
Cinnamomum camphora				19	0.82	100	40	0.19	100	40	0.13	95				40	0.12	97.5	30	0.18	69.7			
Cleistocalyx operculata				20	0.62	100													36	0.57	97.22			
Cordia dichotoma													14	0.07	57.14							25	0.31	76
Cratoxylum cochinchinense *				10	0.93	100													16	0.21	21.21			
Cryptocarya concinna																						100	0.11	58

Species	Yuen Tun			Kwun Yum Shan 1			Kwun Yam Shan 2			Tai Mo Shan 1			Tai Mo Shan 2			Ho Sheung Heung			Nam Shan			Hung Lung Hang		
	N	RHI	% S	N	RHI	% S	N	RHI	% S	N	RHI	% S	N	RHI	% S	N	RHI	% S	N	RHI	% S	N	RHI	% S
Cyclobalanopsis edithiae				10	0.4	82							20	0.77	85				6	0.27	14.29	200	0.23	47.5
Cyclobalanopsis myrisinifolia				20	0.66	27													16	0.36	29.41			
Cyclobalanopsis neglecta				20	0.66	43	40	0.67	80	150	0.64	97.5	18	0.58	90	40	0.65	90	25	0.52	15.39	82	0.27	48.78
Cyclonalanopsis championii				10	0.47	80							23	0.69	47.82				34	1.14	40			
Daphniphyllum calycinum				20	1.27	80																5	0.16	20
Diospyros morrisiana				20	1.17	95																80	0.25	22.5
Elaeocarpus chinensis																						100	0.3	31
Gordonia axillaris	199	0.41	84.7	19	0.69	45													18	0.65	73.68			
Ilex rotunda													10	0.06	60									
Liquidambar formosana				20	0.44	95													15	0.02	6.25			
Lithocarpus glaber				20	0.55	18													62	0.39	28.57			
Lithocarpus harlandii																						50	0.21	70
Litsea rotundifolia	210	0.34	88.7																					
Machilus breviflora				19	0.85	85	40	0.5	70	40	0.68	87.5				40	0.63	95	142	1.07	64.42			
Machilus chekiangensis																						80	0.33	67.5
Machilus chinensis				20	0.61	65																		
Machilus oreophila				20	0.31	80																		
Machilus velutina				20	0.92	65																		
Mallotus paniculatus *				20	0.39	100	40	0.29	42.5	40	0.3	100				40	0.26	87.5	202	0.45	73.4	220	0.48	80.45
Melicope pteleifolia	85	0.32	73.4																					
Myrsine seguinii																			7	0.5	62.5	15	0.16	73.3

Species	Yuen Tun			Kwun Yum Shan 1			Kwun Yam Shan 2			Tai Mo Shan 1			Tai Mo Shan 2			Ho Sheung Heung			Nam Shan			Hung Lung Hang		
	N	RHI	% S	N	RHI	% S	N	RHI	% S	N	RHI	% S	N	RHI	% S	N	RHI	% S	N	RHI	% S	N	RHI	% S
Ormosia emarginata				20	0.45	28																		
Pinus massoniana				20	0.77	58																		
Psychotria asiatica *	101	0.24	92.5																					
Pyguem topengii				23	0.63	60							11	0.45	1							100	0.18	35
Rauvolfia verticillata																			3	0.26	75			
Reevesia thyrsoidea				20	0.42	100																30	0.21	60
Rhaphiolepis indica *	63	0.15	57.6																					
Sapindus mukorossi				20	0.82	86																		
Sapium discolor *				20	0.35	55	100	N.A.	0	150	N.A.	24				50	N.A.	0						
Schefflera heptaphylla *	78	0.46	71.3	10	0.84	65	100	0.52	59	150	0.44	82				50	0.83	70						
Schima superba							100	0.19	78	150	0.15	96				50	0.23	86	3	0.47	83.33			
Sterculia lanceolata *				20	0.3	100	40	0.34	20	440	0.38	92.5				40	0.44	87.5	65	0.57	19.69			
Styrax suberifolius																						50	0.13	42
Syzygium cuminii																			65	0.56	40			
Syzygium hancei													101	0.5	97									
Trema tomentosa				20	0.44	100																		
Tutcheria championii													109	0.59	94							20	0.21	35
Viburnum odoratissimum																						24	0.3	79.16
Xylosma longifolium																						70	0.19	78.57
Zanthoxylum avicennae	95	0.3	89.9																					

Table 2. Common and widespread naturally-occurring native plant species recorded on the surveyed slopes

Category	Species	Chinese name
Native tree		
	Bridelia tomentosa	土蜜樹
	Celtis sinensis	朴樹
	Cratoxylum cochinchinense	黃牛木
	Ficus hispida	對葉榕
	Ficus variegate var. *chlorocarpa*	青果榕
	Ligustrum sinense	山指甲
	Litsea glutinosa	潺槁樹
	Macaranga tanarius	血桐
	Mallotus paniculatus	白楸
	Microcos nervosa (*M. paniculata*)	破布葉
	Rhus succedanea	野漆
	Sapium discolor	山烏桕
	Schefflera heptaphylla	鴨腳木
	Sterculia lanceolata	假蘋婆
Native shrub		
	Breynia fruticosa	黑面神
	Ficus hirta	粗葉榕
	Melastoma sanguineum	毛棯
	Psychotria asiatica	九節
	Rhaphiolepis indica	車輪梅
Native climber		
	Embelia laeta	酸藤子
	Dalbergia benthamii	兩廣黃檀
	Strophanthus divaricatus	羊角拗

Table 3. Native tree or shrub species for ecological rehabilitation on man-made slopes in Hong Kong. It should be noted that planting of medium or large tree species on slopes with gradient >45° is not recommended for safety reason.

Notes:
[1] Successful species in the planting trial at Yuen Tun, Tai Lam Country Park (Choi and Chau 2004).
[2] Common and widespread naturally-occurring species recorded on the 20 surveyed slopes (Choi and Chau 2004).
[3] Successful species in at least one planting trial for natural hillside rehabilitation reported by Hau and So (2003).
[4] Native species that are known to perform well on man-made or natural slopes in Hong Kong.
[5] Other naturally-occurring species recorded on the 20 surveyed man-made slopes (Choi and Chau 2004).
* Bears flower, fruits and/or seeds that are attractive to wildlife.
# High ornamental value due to colourful and attractive foliage, flowers or fruits.

Species	Chinese name	Growth Form	Note
Species recommended for wider uses on man-made slopes			
Ardisia crenata	硃砂根	Shrub	1, * #
Bridelia tomentosa	土蜜樹	Shrub or small tree	2, 3, 4 *
Gordonia axillaris	大頭茶	Shrub or small tree	1, 3 * #
Litsea rotundifolia var. *oblongifolia*	豺皮樟	Shrub	1 *
Mallotus paniculatus	白楸	Medium tree	2, 3, 4 *
Melastoma sanguineum	毛棯	Shrub	2, 4 * #
Melicope pteleifolia	三椏苦	Shrub or small tree	1 * #
Microcos nervosa (*M. paniculata*)	破布葉	Medium tree	2, 4 *
Psychotria asiatica	九節	Shrub or small tree	1, 2, 4 * #
Rhaphiolepis indica	車輪梅	Shrub or small tree	1, 2, 4 * #
Rhodomyrtus tomentosa	桃金娘	Shrub	4 * #
Schefflera heptaphylla	鴨腳木	Shrub or small tree	1, 2, 3 * #
Sterculia lanceolata	假蘋婆	Large tree	2, 3, 4 * #
Zanthoxylum avicennae	簕欓	Shrub or small tree	1, * #
Species recommended for field trials on man-made slopes			
Alangium chinense	八角楓	Medium tree	5 *
Aporusa dioica	銀柴	Medium tree	3, 5 *
Breynia fruticosa	黑面神	Shrub	2 *
Celtis sinensis	朴樹	Large tree	2, 3 * #
Cinnamomum parthenoxylon	黃樟	Large tree	5
Cratoxylum cochinchinense	黃牛木	Small tree	2, 3 * #
Cyclobalanopsis championii	嶺南青岡	Small tree	3 *
Cyclobalanopsis myrsinifolia	小葉青岡	Small tree	4 *
Cyclobalanopsis neglecta	竹葉青岡	Large tree	3 *
Desmodium heterocarpon	假地豆	Sub-shrub	4
Eurya nitida	細齒葉柃	Shrub	5 *
Ficus hirta	粗葉榕	Shrub	2 *
Ficus hispida	對葉榕	Small tree	2 *
Ficus superba var. *japonica*	筆管榕	Medium tree	5 *
Ligustrum sinense	山指甲	Small tree	2 * #
Litsea glutinosa	潺槁樹	Medium tree	2 * #

Species	Chinese name	Growth Form	Note
Macaranga tanarius	血桐	Medium tree	2 *
Machilus breviflora	短序潤楠	Medium tree	3 * #
Machilus chekiangensis	浙江潤楠	Large Tree	3, 5 * #
Melastoma candidum	野牡丹	Shrub	4, 5 * #
Myrsine seguinii	密花樹	Shrub or small tree	3 *
Reevesia thyrsoidea	梭欒樹	Small tree	3 * #
Rhus succedanea	野漆	Small tree	2 * #
Sapium sebiferum	烏桕	Medium tree	4 *
Schima superba	木荷	Large Tree	3 * #

The Use of Soil Bioengineering Measures to Repair Selected Natural Terrain Landslides in Hong Kong

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Abstract: In 2003, the GEO commenced an innovative project to study and trial soil bioengineering measures on natural terrain. The project was designed to identify measures with the potential to minimize natural slope deterioration, reinforce the soil mass, and improve resistance to debris movement. Soil bioengineering is perceived as a low-cost, environmentally compatible, and sustainable alternative to conventional engineering methods. Although not a "greening" project, the measures are compatible with greening initiatives. Potentially applicable measures for repairing failed/distressed natural hillsides were reviewed and the applicability and performance of selected measures is being tested at four sites. These were assessed using API, geomorphological mapping and vegetation surveys. In consultation with local experts, suitable plant species and species combinations were identified. These are mostly native, but some exotics were included for comparison. Soil bioengineering measures were designed and applied in various combinations at the selected sites. Planting was carried out using vegetative cuttings, material harvested around the sites, and rooted nursery seedlings. Planting in 2004 served as a trial for further sites in 2005, generating valuable baseline data on issues such as species tolerances, methodologies, and cost-effectiveness. Monitoring of performance continues, but initial results will be presented. Importantly, the findings are being documented to provide guidance for future application of soil bioengineering measures in Hong Kong.

Introduction

Currently in Hong Kong, the strategy for managing natural terrain hazards is directed towards keeping natural terrain landslide risk at levels that are as low as is practically achievable. For existing developments, the Government adopts a react-to-known-hazard principle in dealing with natural terrain hazards, *i.e.* to carry out studies and mitigation actions where significant hazards become evident. The strongly preferred technical approach is to avoid carrying out stabilization works over large areas of natural hillsides, because these would generally be both impractical and environmentally damaging. Instead, risk is mitigated by adjusting development layouts, incorporating buffer zones, and erecting defensive measures, such as debris-resisting barriers. Within this framework, soil bioengineering measures offer a potentially effective, low-cost, sustainable, and environmentally acceptable alternative to conventional slope works.

Experience with bioengineering in areas of natural terrain in Hong Kong has been largely restricted to direct planting, particularly in areas of severe erosion, most notably within the Country Parks (by the Agriculture, Fisheries and Conservation Department), and close to new towns (by the former Territory Development Department, now part of the Civil Engineering and Development Department). However, the soil bioengineering techniques described here have not

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previously been applied to natural terrain in Hong Kong, although they have been used successfully elsewhere. Therefore, in April 2003, the Geotechnical Engineering Office (GEO) began a pilot project to select, and to carry out field trials of, soil bioengineering measures for use on areas of natural terrain affected by recent, shallow landsliding and subsequent gully erosion. The project was designed to identify measures that are capable of minimizing the deterioration of natural slopes, primarily by reducing the potential for further erosion and failures, reinforcing the soil mass, and increasing the resistance of the slope to further debris movement and entrainment. Another important element of the project is to identify means of accelerating the natural revegetation of deteriorating slopes, thereby enhancing the local ecosystems. Significantly, although the works were not principally designed as a revegetation project, the types of measures being evaluated are entirely compatible with current greening initiatives. Thus, this project is a new approach to the remediation of natural terrain that has been disturbed by erosion and shallow mass movements, using previously untried soil bioengineering measures that are potentially not only cost-effective and self-sustaining, but that also provide a protective cover with acceptable mechanical and visual characteristics.

Soil Bioengineering

Bioengineering has been defined variously as 'the use of living vegetation, either alone or in conjunction with non-living plant material and civil engineering structures, to stabilise slopes and/or reduce erosion' and 'the use of any form of vegetation, whether a single plant or collection of plants, as an engineering material (*i.e.* one that has quantifiable characteristics and behaviour)' (Morgan & Rickson, 1995; quoted in GEO, 2000). The term soil bioengineering is considered by Gray & Sotir (1996) to be a specific term that refers to the use of live plants and plant parts, in which live cuttings and stems are placed in the ground, or in earthen structures, where they act as soil reinforcement, hydraulic drains, barriers to earth movement, and hydraulic pumps or wicks.

Soil Bioengineering Measures

Soil bioengineering techniques combine mechanical, biological, and ecological principles and practices to protect and enhance slopes, repair erosion gullies and remediate shallow mass movement scars. They integrate established engineering practices with ecological principles to provide a solution that provides broad mechanical, hydrological and environmental benefits. The effective application of these measures requires careful planning and designing, taking into consideration the specific characteristics of each site. These include parameters such as the geology, soils, slope angle, slope aspect, hydrology, existing vegetation cover, *etc.*, which should all be assessed before the appropriate soil bioengineering measures can be prescribed.

Several potential environmental benefits can be achieved by using soil bioengineering measures. Notably, their installation generally requires significantly reduced access for equipment, materials and workers, typically causing only minor site disturbance during installation. In sensitive locations, where preservation of scenery or wildlife habitats may be critical, soil bioengineering measures could offer environmentally acceptable solutions. Additionally, for sensitive or remote sites, such measures do not require long-term maintenance.

Available soil bioengineering measures can be described in terms of direct measures, applied to sites requiring repair or stabilisation, and indirect measures, applied to areas adjacent to deteriorated sites. The direct soil bioengineering measures, used here within selected landslide scars, comprise two distinct components, namely, 'living' (*i.e.* the planting of herbaceous and

woody species) and 'structural' or 'non-living' (*i.e.* erecting timber cribwalls and bender fences (Table 1 and Figure 1)). However, the two components are commonly combined (*e.g.* live cribwalls) and individual measures are commonly integrated into a comprehensive soil bioengineering strategy. The living approach includes conventional direct planting of grasses, shrubs or trees, and more specific soil bioengineering techniques that use stems or branch sections of living plants as the primary soil reinforcing and stabilising agents. The latter techniques include live stakes, live fascines, brushlayers, hedgelayers, branchpacking (Table 1), and also pole drains. After installation of the stems and branches, subsequent growth of foliage and roots, combined with establishment of invasive vegetation species, form the major structural components of the soil bioengineering system. Each soil bioengineering measure offers different immediate depths of effectiveness (prior to rooting), typically in the range of 200 to 1200mm for natural hillside applications.

Table 1 – Soil Bioengineering Measures Installed at Cloudy Hill North in 2004

Soil Bioengineering Measure	Definition	Landslide Scar		
		CHN 01	CHN 02	CHN 03
Live Stakes	Branches or plant stems, cut or pruned from living plants, that root readily from cuttings. Usually, all side branches are removed prior to installation.	*	*	*
Live Fascines	Bundles of live cut branches placed in shallow trenches constructed either along, or at an angle to, the contour. Provides surface protection and shallow slope stabilisation by mechanically reinforcing the soil mantle. Used as a drainage system when installed along a pole drain.	*	*	*
Brushlayer	Live cut branches, pruned from a designated harvesting area, installed along constructed benches, placed perpendicular to the slope face and with the growing tips directed out of the slope.	*	*	
Hedgelayer	Living plants with established roots, placed perpendicular to the slope on constructed benches with the growing tips directed out of the slope.	*	*	
Branchpacking	A method using poles and live cut branches and/or rooted plants installed perpendicular to the slope face with growing tips directed out of the slope. Commonly used to repair gullies or small slumps. The layers of live plant materials are alternated with layers of ameliorated soil.		*	
Live Cribwall	A structure comprising interlocking wooden beams, secured by rebar pins, and filled with soil and rock. Live cut branches and/or rooted plants installed between the frontal members.	*		
Bender Fence	A retaining structure comprising flexible boards woven between wooden poles. Installed along contour, and typically retaining ameliorated soil material, form a series of narrow terraces across the slope. Commonly planted with seedlings and/or live stakes.		*	*

Indirect measures also being installed comprise live barriers. These are belts of hardy species (trees and bamboo) that are strategically planted to restrict movement of landslide debris. They can be planted within drainage lines, below steep slopes where there is potential instability, to restrict the passage of channelised debris flows. They can also be planted on open hillslopes below recent landslide scars, to restrict movement of debris from open hillslope failures.

An important aspect of the project is the selection of suitable plant species and species combinations. The plants must tolerate the thin, well-drained soils, the steep slopes, and the exposed sites. Species selection was carried out in consultation with experts locally and elsewhere, following on-site vegetation surveys. The provisional selected species (Table 2) are predominantly native, mainly shrubs, and all are common or very common in Hong Kong. Therefore, they can be harvested from areas adjacent to, or within a reasonable haul distance of, sites and so are well suited to the local climate, soil conditions and available moisture. Importantly, they are relatively inexpensive. A few exotic species only were included to compare their performances with the native species. Selection of species took into consideration the plants' abilities to support natural colonization by the surrounding plant community, with the ultimate goal of producing a sustainable native or semi-native diverse plant cover. There is only limited data on rooting depths of native species in Hong Kong (Table 2) but further data will be collected, and the performance of the species will be reassessed.

The Limitations of Soil Bioengineering Measures

There are several important limitations to the application of soil bioengineering measures. Thus, they cannot be recommended where the landslide scars are in bedrock, there being no soil to act as a rooting medium for plants, and no soil to fail. Deep-seated failures with high back scars (with headwalls above about 2 metres) are beyond the capability of most rooted plants to successfully stabilise, and are also more prone to retrogressive failures. Steeper slopes in general (over about 35-50°) also present problems, being both difficult to access and expensive to repair. Although, the relative effectiveness of soil bioengineering measures on steeper slopes is being assessed during this project, shallower failure scars present the optimum opportunities for securing the soil mantle. Finally, some sites are subject to rapid natural recolonisation by vegetation. Bioengineering techniques appear unnecessary at these sites, largely because unwelcome vegetation clearance would be required. However, further data on such sites is required to establish criteria on how such sites should be assessed and handled.

Trial Site Selection and Planning

The project began with a Territory-wide review of potential sites, using aerial photographs to select a range of locations of varying geology and microclimate, recently affected by shallow mass movement. Initially, five sites were identified across Hong Kong, within which some 50 recent natural terrain landslides had occurred. These sites were considered to be suitable for testing the applicability and performance of the planned soil bioengineering measures.

The trial sites were closely examined using a combination of aerial photograph interpretation (API), field geomorphological mapping, and field vegetation surveys. Planting works were scheduled in two phases, to be carried out in 2004 and 2005. The first phase of planting in 2004 was limited to Cloudy Hill North, to the north of Tai Po, as it was intended to generate valuable baseline data on issues such as relative species tolerances, applicability of the different soil bioengineering measures, and comparison of their cost-effectiveness. These data were then used to plan the 2005 planting season. Collection of performance data is continuing.

Planting works are planned for 2005 at Cloudy Hill South, Por Lo Shan (west of Tuen Mun), and Sham Wat in North Lantau, and at further locations at Cloudy Hill North. However, some landslide scars identified during the initial survey proved, after more detailed evaluation, to be unsuitable for applying soil bioengineering measures and were excluded from the programme.

Table 2 - Provisional Plant Species List for Soil Bioengineering Measures

Species	Form	Status - Native (n) / Exotic (e)		Estimated Rooting Depth
Acacia confusa	Tree	Common	e	Deep
Ardisia crenata	Shrub	Common	n	Intermediate
Baeckea frutescens	Herb	Very common	n	Intermediate
Breynia fruticosa	Shrub	Very common	n	Intermediate
Bridelia tomentosa	Tree	Common	n	Uncertain
Canthium dicoccum	Shrub or tree	Common	n	Uncertain
Celtis sinensis	Tree	Very common	n	Uncertain
Cinnamomum camphora	Tree	Very common	n	Deep
Clerodendrum viscosum	Shrub	Common	n	Intermediate
Cratoxylum cochinchinensis	Tree	Common	n	Intermediate
Croton crassifolius	Shrub	Common	n	Intermediate
Duranta erecta	Shrub	Common	e	Shallow
Ficus microcarpa	Tree	Very Common	n	Deep
Gardenia jasminoides	Shrub	Common	n	Shallow
Glochidion eriocarpum	Shrub	Common	n	Intermediate
Glochidion lanceolarium	Shrub	Common	n	Intermediate
Glochidion zeylanicum	Shrub	Common	n	Intermediate
Gordonia axillaris	Tree	Common	n	Intermediate
Ilex pubescens	Shrub	Very common	n	Intermediate
Itea chinesis	Shrub/Tree	Common	n	Intermediate
Lespedeza formosa	Shrub	Common	n	Uncertain
Liquidambar formosana	Tree	Very common	n	Deep
Litsea cubeba	Tree	Common	n	Intermediate
Litsea glutinosa	Shrub	Very common	n	Uncertain
Litsea rotundifolia	Shrub	Very common	n	Intermediate
Macaranga tanarius	Tree	Very common	n	Intermediate
Machilus breviflora	Tree	Common	n	Intermediate
Machilus checkiangensis	Tree	Common	n	Intermediate
Melastoma candidum	Shrub	Common	n	Shallow
Melastoma sanguineum	Shrub	Common	n	Intermediate
Melia azedarach	Tree	Introduced, common	e	Deep
Melicope ptelefolia	Shrub	Common	n	Intermediate
Phyllanthus cochinchinensis	Shrub	Very common	n	Intermediate
Phyllanthus emblica	Shrub	Very common	n	Intermediate
Psychotria rubra	Shrub	Common	n	Intermediate
Raphiolepis indica	Shrub	Common	n	Intermediate
Rhamnus crenata	Shrub	Common	n	Intermediate
Rhodomyrtus tomentosa	Shrub	Very common	n	Intermediate
Rhus chinensis	Shrub	Very common	n	Intermediate
Rhus succedanea	Tree	Very common	n	Intermediate
Sapium discolor	Tree	Very common	n	Intermediate
Sarcandra glabra	Shrub	Very common	n	Shallow
Schefflera octophylla	Tree	Very common	n	Intermediate
Syzygium buxifolium	Shrub/Tree	Common	n	Intermediate
Syzygium jambos	Tree	Introduced, common	e	Deep
Zanthoxylum avicennae	Shrub	Common	n	Intermediate

Reasons for their unsuitability included their being too deep-seated, too steeply inclined, too little soil cover remaining, and natural revegetation already being substantially complete.

Implementation of the Soil Bioengineering Measures

During the first phase of implementation, in spring and summer of 2004, a comprehensive range of the major soil bioengineering measures were designed and installed in various combinations on three natural terrain landslide scars (CHN-01, CHN-02 and CHN-03) at the Cloudy Hill North pilot site. Seven main types of soil bioengineering measure were implemented (Table 1)(Figures 1 to 4). As the completed measures are intended to blend in with the surrounding ecology, both vegetative cuttings and rooted nursery seedlings were used. The former were harvested from designated areas around the landslide sites. Harvesting was limited to a maximum of 30% of the branches of individual plants, to avoid undue damage to the plant.

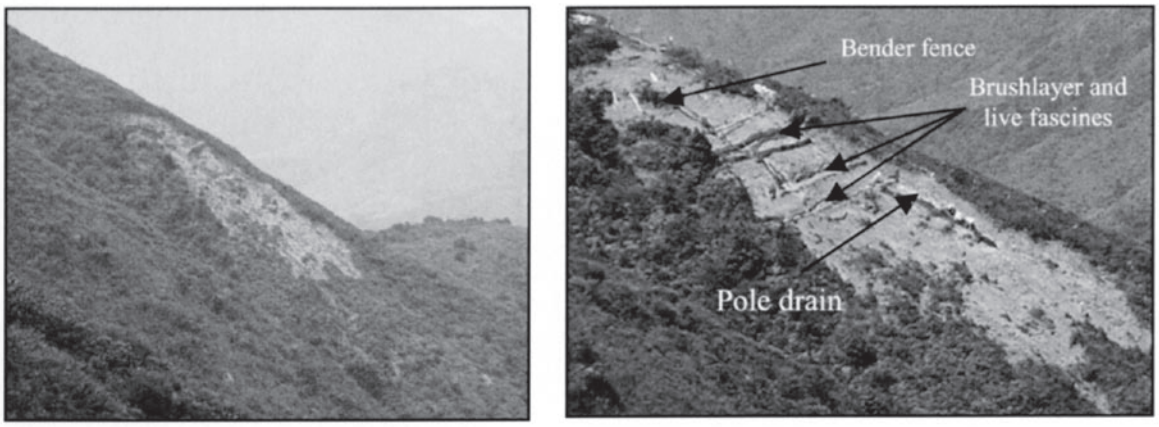

Figure 1 - Landslide scar CHN-02 (Cloudy Hill North), prior to (left) and during (right) installation of bioengineering measures

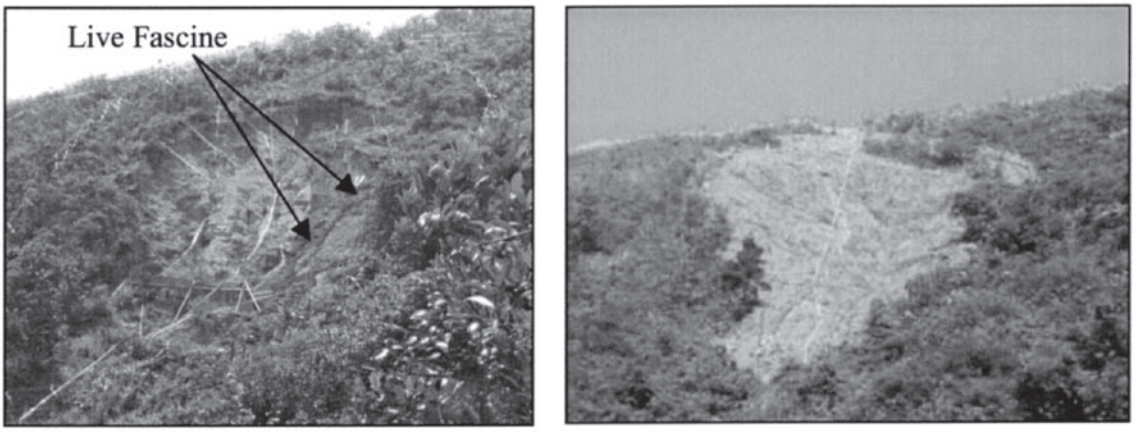

Figure 2 - Landslide scar CHN-03 (Cloudy Hill North), during (left) and after (right) installation of bioengineering measures

Performance of the Measures

Monitoring of the performance of the planted material is continuing, but available results indicate variable performances of the measures. As planting was undertaken mainly in July and August, later in the rainy season than would be preferred, trial watering was carried out during

the dry season on two of the landslide scars (CHN-01 and CHN-02). However, initial performance data suggest that the watering may not have had a significant effect on the relative performance of the plants.

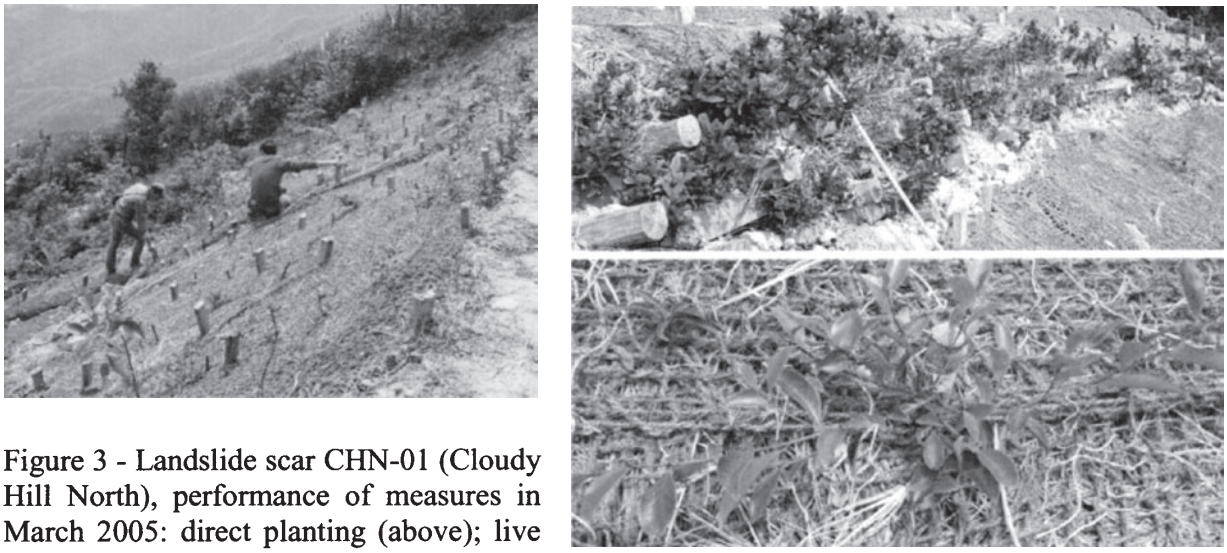

Figure 3 - Landslide scar CHN-01 (Cloudy Hill North), performance of measures in March 2005: direct planting (above); live cribwall - rooted and live cut material (top right); live stake (right)

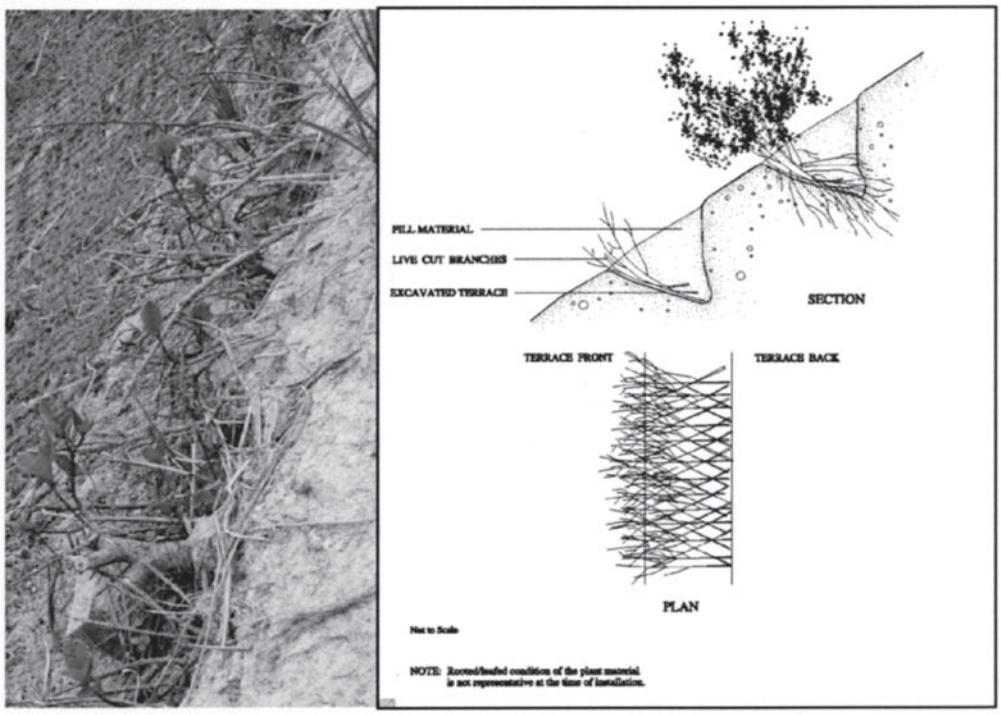

Figure 4 – Performance of typical brushlayer, Landslide scar CHN-01 (Cloudy Hill North) in March, 2005, compared with design configuration

Directly planted materials generally performed very well. By the end of the first dry season, nearly all of the plants had survived (typically 90-95%), and were showing signs of renewed growth with the impending onset of the next rainy season (Figure 3). Other rooted material had variable but generally reasonable rates of survival by the end of the first dry season, in, for example, cribwalls (c.90%) (Figure 3), bender fences (90-99%), hedgelayers (60%) and branchpacking (c.20%).

Live cut material used in live stakes, live fascines and pole drains appeared, by the end of the first dry season, to have generally low survival rates of 0-10%. However, the actual rate of survival may only become apparent with the onset of the rainy season and renewed growth that commonly follows a period of dry season dormancy. By contrast, live cut materials used in the brushlayers (Figure 4) and the cribwall (Figure 3) generally fared well. Survival rates by the end of the first dry season were, in both cases, up to 50% and 60% respectively. The reasons for the observed disparities in performance of these installations compared to other uses of live cut material are yet to be established.

Technical Guidelines

Importantly, the findings of the trial applications are being documented to provide guidance to non-specialists on how to apply soil bioengineering measures to other sites in Hong Kong. A set of preliminary guidelines was prepared for internal use in December 2004. It is planned that the guidelines will be refined to take account of the performance of both the 2004 and 2005 plantings, and published, after further consultation with practitioners, in mid-2007.

These Technical Guidelines are intended to assist in the development of, and ultimate adoption of, a consistent approach for the application of soil bioengineering measures to the remediation of natural terrain landslide sites in Hong Kong. The approach includes providing clear standards for the collection and assessment of desk study and site-specific data, guidance on the design and construction of specific soil bioengineering measures, the maintenance requirements during the establishment period of the measures, and the subsequent monitoring and evaluation procedures.

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Acknowledgements

This paper is published with the permission of the Director of Civil Engineering and Development and the Head of the Geotechnical Engineering Office. The authors gratefully acknowledge the valuable technical advice received from Dr Robbin B. Sotir (Robbin B. Sotir of Associates, Inc., Georgia 30064-2732, USA) throughout the project.

LABORATORY PULLOUT BOXES FOR SOIL NAIL PULLOUT SHEAR RESISTANCE STUDIES

Jian-Hua YIN[1], Li-Jun SU[2] and Lok-Man CHU[3]

Abstract: This paper first introduces a laboratory soil nail pullout box used before for studying the soil nail pullout resistance. Main results from the pullout tests using this pullout box are presented and discussed with a special attention to influences of overburden pressures and degrees of water saturation (or water content). The results from this type of pullout tests are for obtaining the elementary behavior of the soil/nail interaction shear strength. However, the installation of a soil nail and the performance of a soil nail of full length in a slope are boundary value problems. The authors discuss the definition and differences of the elementary behavior of a soil (or soil/nail interaction) and a boundary value problem of a whole geotechnical structure (*e.g.* a soil nailed slope). The paper presents a newly developed soil nail pullout box with full instrumentation for studying the nail pullout resistance under different controlled conditions. This new box overcomes limitations of the previous box and can be used to study the influences of (a) hole drilling and stress release, (b) soil saturation with water back pressure, (c) cement pressure grouting, (d) over-burden pressure, *etc*. The instrumentation package includes transducers/sensors for measurement of (a) positive and negative (suction) pore water pressures, (b) earth pressures, (c) axial force in the nail using strain gauges, (d) nail pullout force and displacement, (e) vertical pressure and displacement (or volume change) and (f) grouting pressure and control. Two such new soil nail pullout boxes have been made, setup and used for a series of soil nail pullout tests in The Hong Kong Polytechnic University.

INTRODUCTION

As a result of rapid economic growth in 1950's and 1960's and a large demand of land for use in residential, commercial and industrial, and infrastructural developments, there were about 25,000 cut and fill slopes formed in hilly terrains for the developments in Hong Kong. All these slopes were liable to failure in periods of intensive tropical rainfalls, and a number of disasters occurred in the early and mid 1970's. The Hong Kong Government has established a long-term public safety program, Landslip Preventive Measures (LPM) Program, to improve existing steeper cut or man-made fill slopes formed before 1977, bringing them to the current geotechnical standards. More and more new man-made soil slopes have been formed since 1977. For both economic and technical advantages, soil nailing has become the most popular solution to stabilization of both old slopes (before 1977) and new slopes (after 1977).

In Hong Kong, more than HK$0.8 billion is spent on slope upgrading each year. About 80% of slopes are stabilized using soil nails. For the design of a soil nail system, the interface shear strength between a soil nail and the surrounding soil is a key parameter for design and

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safety assessment of the soil nail stabilized slope. In current practice in Hong Kong, the interface shear strength (adhesion c_a and friction angle δ) is estimated by experience or is assumed to be the same as the shear strength (cohesion c and internal friction angle ϕ) of the soil. The interface shear strength is normally verified by a pull-out test on the soil nail in the construction stage (Powell and Watkins 1990). There are considerable uncertainties in selection and verification of the interface shear strength parameters. To enable a reliable and economic design of a soil nailing system, understanding of the fundamental interaction mechanism and shear strength between a cement grouted soil nail and the surrounding soils and direct measurement of the interface shear strength parameters are necessary.

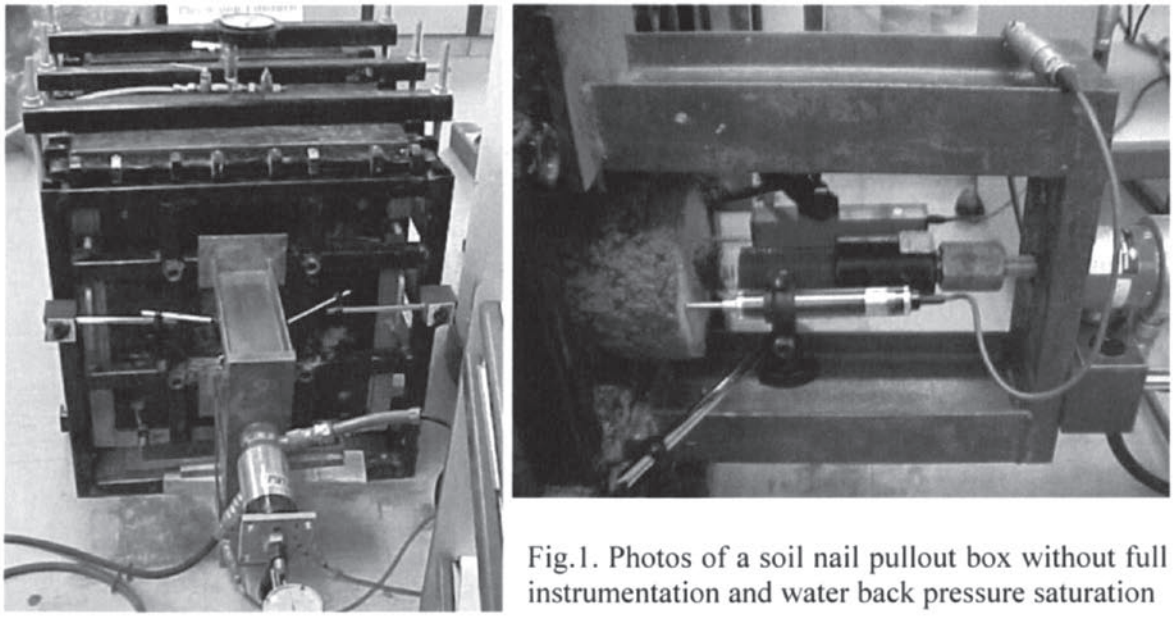

Fig.1. Photos of a soil nail pullout box without full instrumentation and water back pressure saturation

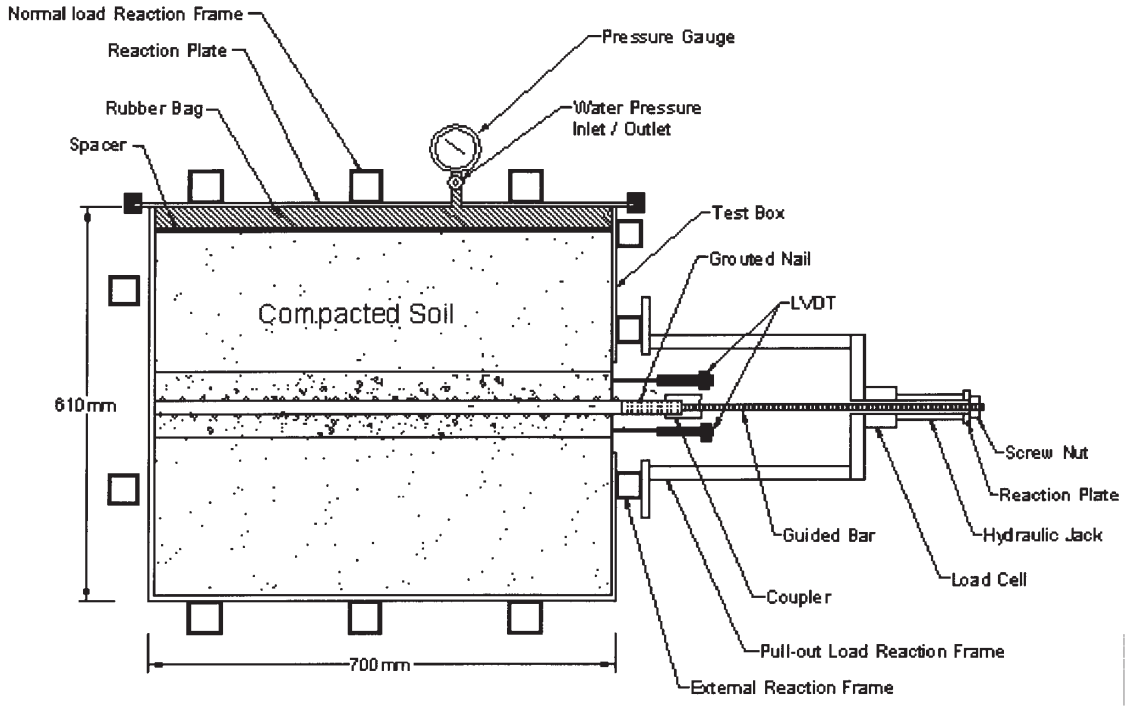

Fig.2. Cross-sectional view of a soil nail pull-out box and test setup (after Chu and Yin 2003a)

SOIL NAIL PULLOUT BOX WITHOUT FULL INSTRUMENTATION

A laboratory soil nail pullout box was designed and built by Chu and Yin (Chu 2003; Chu and Yin 2003a,b, 2004, 2005) (see Fig.1). This pullout test box had internal dimensions of 690mm-long by 560mm-wide by 605mm-high (see Fig.2). Dimensions of the box were chosen in order to compromise between the minimal effects of the boundaries and the difficulty of preparing a huge volume of soil sample. The pull-out test box was built up by 5mm rolled steel plates and 38mm x 38mm steel angle bars using welded and bolted connections. In addition, an external steel reaction frame was constructed outside the steel box in order to carry the soil compacting force, constrain the lateral soil pressure, and react the pull-out load. A steel plate with a 140mm diameter opening hole is attached to the front side of the box for the cement grouted nail to be pulled out. Friction between the soil and the side walls of the box was minimized by lubricating oil. The CDG soil was then placed in the six layers and compacted to achieve a relative density of 95%. A 100mm diameter hole was pre-drilled into the compacted soil by a concrete coring machine. The lengths of the soil nails in the tests are 500mm and 700mm. A 32mm diameter ribbed steel bar was centralized in the drillhole and cement slurry with a water-cement ratio of 0.45 was continuously grouted over the drillhole by gravity flow. The grouted nail was cured inside the soil for 28 days to achieve matured cement strength. After 28 days of curing, the desired normal pressure was applied through the pressurized water rubber diagram bag positioned between the top soil surface and the top reaction plate against normal load reaction frame. The rubber diagram bag was filled with water and was used to produce a uniformly distributed pressure on the top surface of the compacted CDG soil. The top flexible boundary is preferred for the purpose of measuring dilatancy of the soil-cement grout interface and slightly decreasing in friction coefficient. The pullout load was measured by a load cell located between the pull-out load reaction frame and the hydraulic jack. Pullout displacements were measured by two linear variable differential transducers (LVDTs) pointed at the nail head (see Fig.1 and Fig.2). These instruments were linked to a data logger that took and recorded data at desired time intervals.

The above soil nail pullout box had a number of limitations:

(a) There was no instrumentation inside the soil and on the nail.
(b) The soil stresses on the soil nail surface were not uniform as a cavity was created after the nail being pulled out.
(c) There was difficulty to make the soil close to full water saturation.
(d) The vertical stress was applied after the nail was installed. The influence of hole drilling and stress release on the soil nail pullout resistance was not investigated.
(e) It was difficult to carry out cement slurry pressure grouting.

INFLUENCES OF OVERBURDEN PRESSURE AND DEGREE OF WATER SATURATION ON SOIL NAIL PULLOUT RESISTANCE

A total of 18 soil nail pullout tests were carried out using the soil nail pullout box for different conditions. The following influences on the interface shear strength were investigated in the laboratory pull-out tests:

(a) Influence of different applied stress levels of 50, 100, and 300kPa.
(b) Influence of soil degree of saturation on pull-out shear strength.
(c) Influence of surface roughness of the soil nails with a regular or irregular nail surface.

All tests were carried out on a cement grouted soil nail installed in a completely decomposed granite (CDG) soil under a controlled condition using the previous simple pullout box without full instrumentation. The test results were used to investigate the influences of a number of factors such as the influence of water content, overburden pressure, relative density, and surface roughness of the nail at the soil and concrete grout interface.

This paper presents main results of pullout tests with different overburden pressures and different degree of saturation (or water content). It shall be pointed out that, in all these tests, the vertical pressure was applied after the hole was drilled and soil nail was installed. In this approach, the vertical stress on the soil nail was approximately equal to the vertical pressure applied on the top soil surface. The test results were useful to obtain the elementary shear strength at the interface between the soil and the nail surface.

Influences of Overburden Pressure

Fig. 3 shows soil nail pull-out test results under three vertical pressures of 55 kPa, 105 kPa and 305 kPa for a regular soil nail surface in a natural wet condition with the degree of saturation of the soil approximately 70% and water content of 19% (Chu and Yin 2005). It is seen that the shear strength and the volume compression increase with the vertical pressure.

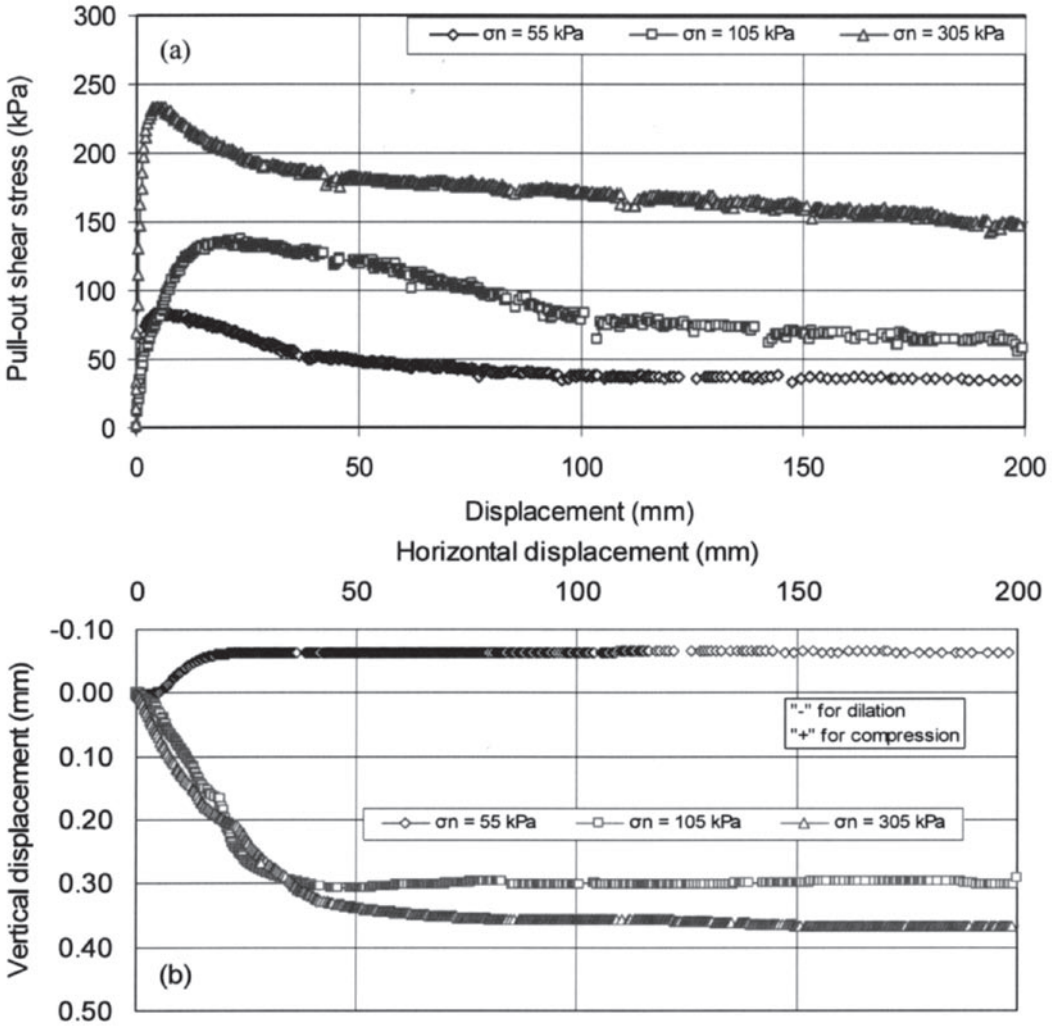

Fig.3. Pull-out test results of regular nails in natural wet condition – (a) average pull-out shear stress vs. displacement and (b) vertical displacement vs. horizontal displacement (after Chu and Yin 2005).

Since the vertical pressure was applied after the nail had been installed, the vertical pressure was almost all transferred to the soil nail surface. The stress conditions were very much similar to those in a direct shear box test. The differences between the pullout test and

the direct shear box test were (a) nail surface was circular compared to a plate surface in the later; (b) the acting lateral stress on the nail was different from that on the top compared to only the vertical stress required in the shear box test data interpretation.

Since the pullout test results are suitable for deriving the elementary interface shear strength, the Coulomb's shear strength criterion equation can be used or modified for describing the interface shear strength between the soil and the circular nail section. The Coulomb's criterion equation $\tau_f = c' + \sigma_n' \tan\phi'$, where τ_f, σ_n' are the shear stress and normal effective stress at failure and c', ϕ' are the internal effective cohesion and effective frictional angle of the soil. There are three types of equations for the interface shear strength between the soil and the circular nail section. The definitions and relationships are derived as follows.

The average pull-out shear stress τ_s (kPa) of a soil nail can be calculated from a measured pull-out load F (kN) divided by the active nail surface area A (m^2):

$$\tau_s = \frac{F}{A} = \frac{F}{(\pi D)L_s} = \frac{T}{\pi D} \tag{1}$$

where L_s is the embedded length of the nail, D is the diameter of the nail and $T = F / L_s$ is nail pull-out capacity per line meter (kN/m):

In the common design practice, the following equation is used for calculation of a soil nail pull-out capacity per line meter T (kN/m):

$$T = (\pi D c' + 2D\sigma_v' \tan\phi') \tag{2a}$$

$$\frac{T}{\pi D} = c' + \frac{2}{\pi}\sigma_v' \tan\phi' \tag{2b}$$

where c' is the soil cohesion and ϕ' is the internal angle of the soil friction. In Eq.(2), for the shear resistance due to cohesion c', the nail is considered to be circular with the perimeter length of πD; while for the shear resistance due to friction $\tan\phi'$, the nail is considered to be a plate with width D. The normal stress acting on the soil nail σ_n' is equivalent to the overburden pressure σ_v', which is the stress of the self-weight of the soil plus the applied stress on the soil sample. The main issue in Eq.(2) is that the internal cohesion and friction angle of the soil are used to represent (or to be the same as) the shear strength of the interface between the soil and cement grouted soil nail surface. This assumption might have some limitations.

Chu and Yin (2003a) proposed a similar equation, but used the interface shear strength parameters for the evaluation of the pull-out capacity per meter T (kN/m) of a soil nail:

$$T = (\pi D c_a' + 2D\sigma_v' \tan\delta'') \tag{3a}$$

$$\frac{T}{\pi D} = c_a' + \frac{2}{\pi}\sigma_v' \tan\delta'' \tag{3b}$$

where c_a' is the soil adhesion at the interface, and δ'' is the angle of the interface friction. It is seen that for the shear resistance due to cohesion c_a', the nail is considered to be circular; while for the shear resistance due to friction $\tan\delta''$, the nail is considered to be a plate.

From Eqs. (1) and (3), the average shear strength is

$$\tau_s = \frac{F}{A} = \frac{T}{\pi D} = c_a' + \sigma_v' \left(\frac{2}{\pi} \tan \delta''\right) = c_a' + \sigma_v' \tan \delta' \quad (4)$$

where

$$\tan \delta' = (2/\pi) \tan \delta'' \quad (5)$$

Eq. (4) is the third equation and is more fundamental as no dimension is involved. Eq.(4) can be used to fit the shear strength data points to find the parameters of c_a' and δ'. Use the relationship of $\tan \delta'' = (\pi/2) \tan \delta'$, δ'' can be evaluated. Using the data in Fig.3, the interface shear strength parameters were obtained and are listed in Table 1.

Table 1. The interface shear strength parameters from the pullout test results in Fig.3

Peak shear strength			Shear strength at displacement of 70mm		
c_a' (kPa)	δ' (degree)	δ'' (degree)	c_a' (kPa)	δ' (degree)	δ'' (degree)
63.5	29.5	41.6	32.4	26.2	37.7

Influences of Degree of Saturation

A number of pullout tests were done on the soil nail in soil with different degrees of saturation, that is, S_r, of 70%, 74%, 78% and 86% under the applied normal stress of 300kPa using the previous pullout box. The measured shear strengths at the interface between the soil and the nail are shown in Fig.4. It is seen very clearly that the pull-out shear strength decreases as the degree of saturation S_r increases.

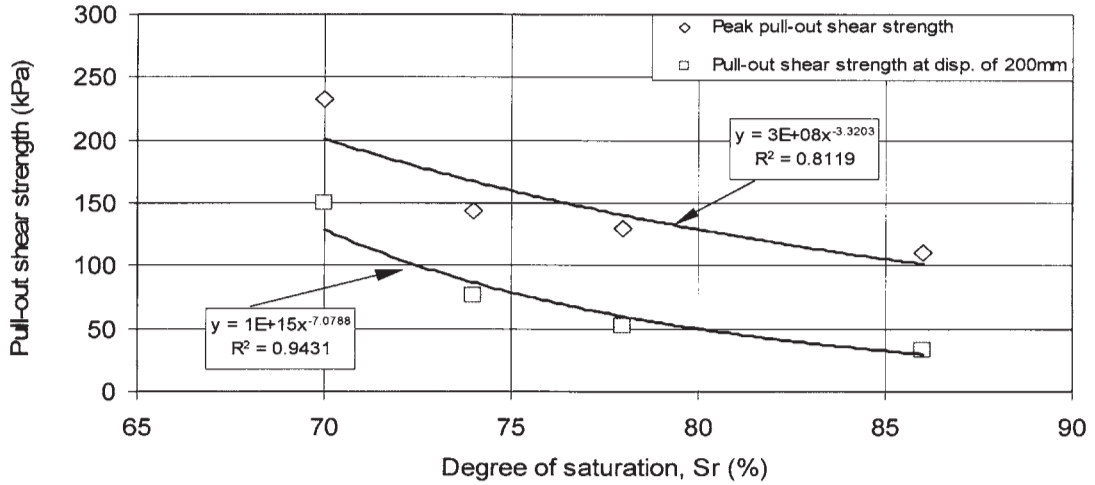

Fig.4. Relations between pull-out shear strengths and the degree of saturation, S_r of 70%, 74%, 78% and 86% under the normal stress of 300kPa (after Chu and Yin 2003a)

NEW SOIL NAIL PULLOUT BOXES WITH FULL INSTRUMENTATION AND INNOVATIVE DESIGNS

As mentioned earlier, the previous soil nail pullout box had five limitations. Based on the previous experiences and the demands for more comprehensive studies on the soil nail pullout resistance, two new soil nail pullout boxes have been designed, made, and setup in the Soil Mechanics Laboratory of The Hong Kong Polytechnic University. A sketch of the new soil nail pullout box is shown in Fig.5. A photo of the new box with a soil nail in test progress is shown in Fig.6.

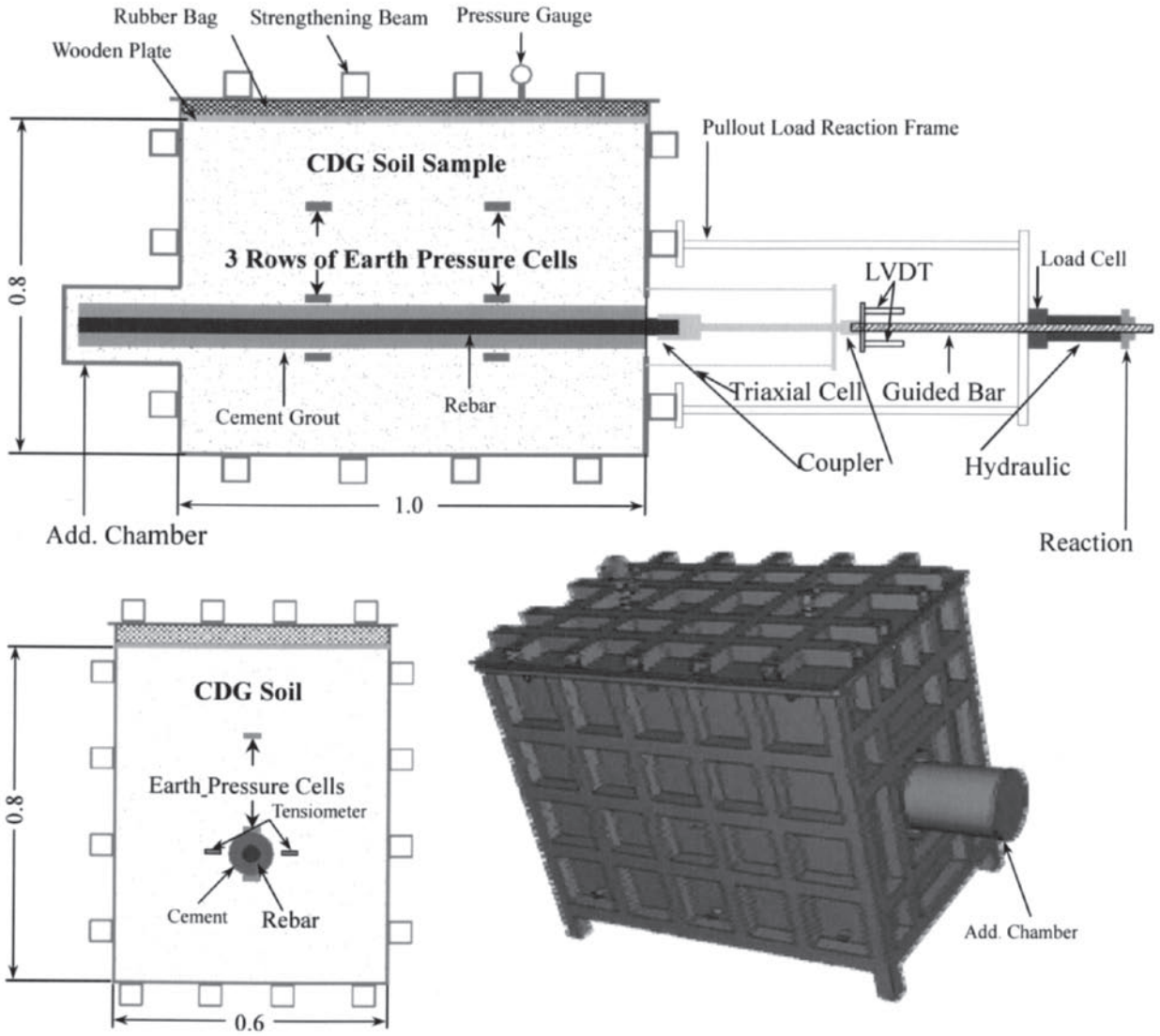

Fig.5. (a) Dimensions of a new pullout box and test set up – along soil nail (top), (b) dimensions of a new pullout box and test set up – cross-soil nail section (bottom left) and (c) a three-dimensional view of pullout box (bottom right)

The new pullout box in Fig.5 has the following advantages over the previous box in Fig.1 and Fig.2:

(a) There are comprehensive transducers inside the soil and on the nail – (i) six earth pressure cells for measuring total earth pressures, (ii) four miniature tensiometers for measuring suctions and small positive pore water pressures, (iii) two miniature pore water pressure transducers for positive pore water pressures, (iv) four strain gauges on the re-bar of the nail for measuring soil nail tension strain (axial forces), (v) pressure gauges and volume change meter in the top cover cap for water pressure control and vertical soil movement measurement, and (vi) two LVDTs and one load cell for measuring pullout displacement and load.

(b) An additional chamber is attached to the end of the soil nail. In this way, the soil stresses on the soil nail surface inside the box are uniform as no cavity is created after the nail being pulled out.

(c) A special triaxial cell is placed to cover the soil nail head. This triaxial cell is used to seal the nail and is subjected to back water pressure. In this way, saturation of the soil inside can be increased to close to 100%.
(d) The vertical stress is applied first in a test. The influence of hole drilling and stress release on the soil nail pullout resistance can be easily investigated.
(e) A special cement pressure grouting device is made to apply a constant grouting pressure. The influence of the cement grouting pressure can be easily investigated.

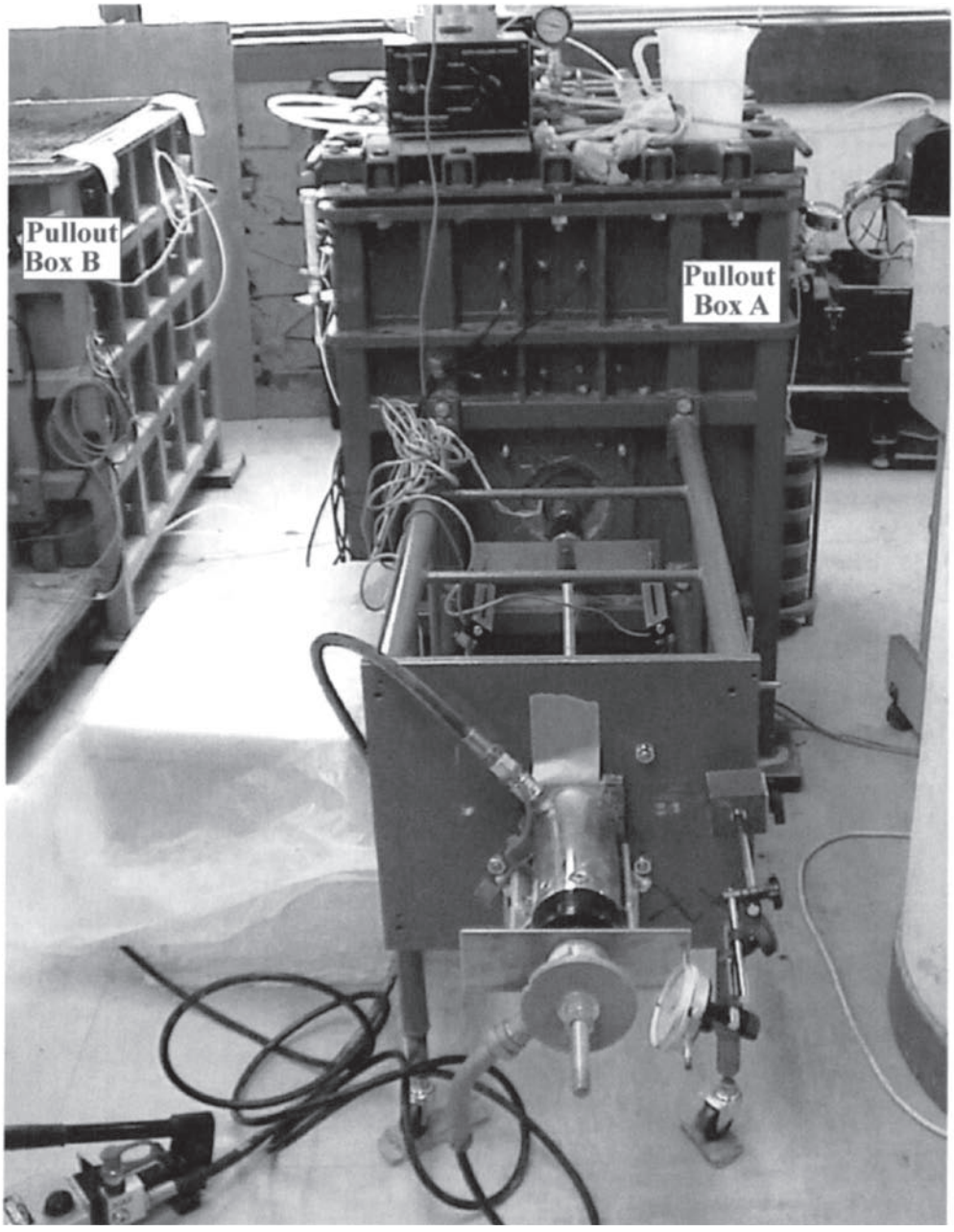

Fig.6. A photo of a soil nail pullout test in progress using the new box (right one) with setup of full instrumentation and loading system (no back water pressure application in this case)

Two new boxes have been made and setup in the laboratory as shown in Fig.6. The right one in Fig.6 is Pullout Box A and the left one with the top cover removed is Pullout Box B. The two boxes have been used to carry out a large number of soil nail pullout tests. The results of these tests will be reported in the near future.

ELEMENTARY BEHAVIOUR AND BOUNDARY VALUE PROBLEM – DIFFERENCE AND MODELLING USING THE PULLOUT BOX

To solve a boundary value problem (no time), for example, deformation and stability analysis of a soil nailed three-dimensional slope (see Fig.7), we need 3 types of equations:

(a) 3 stress equilibrium equations,
(b) 6 strain displacement compatibility equations, and
(c) 6 constitutive equations.

There are a total of 15 equations for solving 15 unknowns (6 stresses, 6 strains and 3 displacements) by considering proper boundary conditions. The 6 constitutive equations in fact describes the elementary stress-strain-strength behaviour of the soil (or interface). The simplest constitutive relationship (or equations) is an isotropic linear elastic relationship with only two constants, that is, Young's modulus and Poisson's ratio. The Mohr-Coulomb's failure criterion is another example of describing the elementary failure behaviour of a soil.

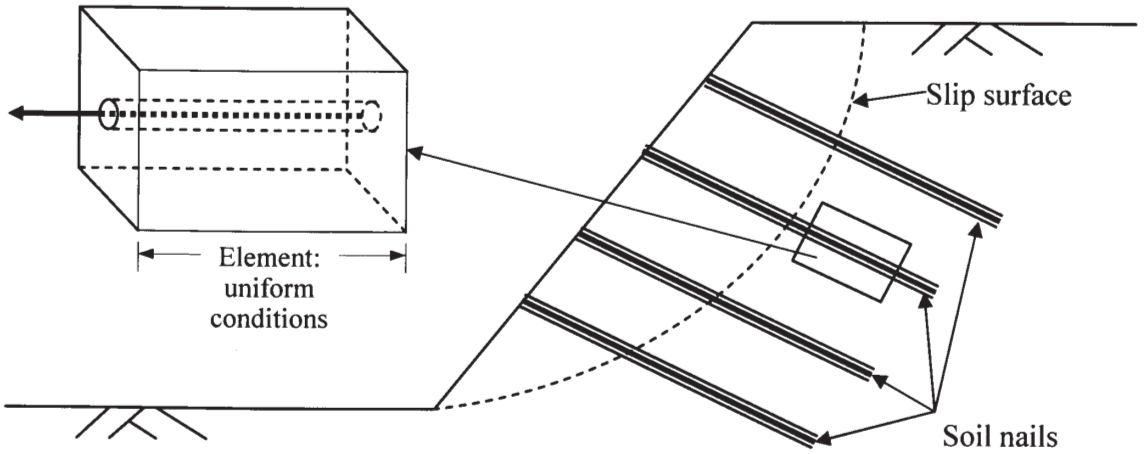

Fig..7. A boundary value problem and an element of a soil nailed slope: why and how to use the new soil nail pullout box to study the soil nail shear resistance and influence factors?

As shown in Fig.7, to study the performance of a full length of soil nail in the slope, we need a full-scale physical model (or a small-scale centrifuge model), since this is a boundary value problem for this nail. On the other hands, if our purpose is to study the influences of, for example, the installation procedures (hole drilling with stress release, cement slurry grouting) and overburden pressures on a soil nail pullout resistance, we need only a short length of the nail for this study as shown in Fig.7. This is because that a full length of a nail in a slope at the failure (the ultimate case for stability analysis) is actually subjected to a tension force. The interface shear stress is in the opposite direction toward the slip surface as shown Fig.7. The soil and stress conditions along this nail are not uniform. Thus, if a full length nail with non-uniform conditions is used for pullout testing, the influences of parameters such as overburden pressure and installation procedures cannot clearly identified and well studied.

In the new pullout box study, a short length of nail and soil is used (see Fig.7). The conditions of soil properties, stresses and loading in this length are all uniform. This length is considered to be an element. However, the stress and loading conditions in the cross-section are not uniform, for example, due to hole drilling and stress release, the radial stresses at the boundary of the hole are zero. The stresses and deformation in this cross-section are not uniform and affected by the boundaries and loading. The nail in the box is considered to be a

boundary value problem and but has uniform condition in the axis-direction (an element). The results obtained can be applied to a full length of nail in the slope subjected to the same conditions.

CONCLUSIONS

The previous laboratory box was used to carry out a number of soil nail pullout tests in which, the vertical pressure was applied after the nail had been installed. It was found that the soil nail interface shear strength was almost proportional to the overburden pressure, but decreased significantly with the increase of degree of water saturation. This old soil nail pullout box was not well instrumented and had a number of limitations. Two innovative soil nail pullout boxes have been designed, made, setup and used to study the mechanisms of soil nail pullout resistance and influence factors. The new boxes have the following new features and capabilities:

(a) There is comprehensive instrumentation of the soil and the nail in the box and application and control of loading.
(b) An additional chamber is attached to the end of the soil nail to make soil stresses on the soil nail surface inside the box uniform during pulling out.
(c) A special triaxial cell is placed to cover the soil nail head, seal the nail and apply back water pressure to achieve degree of saturation close to 100%.
(d) The vertical stress is applied first in a test so that the influence of hole drilling and stress release on the soil nail pullout resistance can be investigated.
(e) A special cement pressure grouting device is made to apply a constant grouting pressure so that the influence of the cement grouting pressure can be studied.

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ACKNOWLEDGEMENTS

The improvement, setup and use of the two new soil nail pullout boxes for soil nail pullout resistance studies have received valuable comments from Mr. Terence CF Chan, Mr. Herman YK Shiu, Dr. Raymond WM Cheung, Mr WK Pun, Mr. Tony Cheung, Dr. SL Chiu, Ms Carrie Leung, Mr. Charles KL Tang, and Mr Danny Fu. All these comments are gratefully acknowledged.

THE SURFACE WAVE - TIME DOMAIN REFLECTOMTERY (SW-TDR) TECHNIQUE FOR CHECKING THE LENGTH AND GROUT INTEGRITY/CONCRETE COVER OF INSTALLED UNDERGROUND STEEL REINFORCEMENT OF GROUND ANCHORS, SOIL NAILS, CONCRETED STEEL H-PILES AND BORED PILES

Terrence Tang [1] and K.P. Yim[2]

Abstract: A new measurement technique based on Surface Wave - Time Domain Reflectomtry (SW-TDR) has been applied in measuring the length and grout integrity/concrete cover of installed underground steel reinforcements. This paper presents the results of SW-TDR tests on installed underground reinforcement of ground anchors, soil nails, concreted steel H-piles and reinforced concrete piles (bored piles). In the tests, both potential and current profiles of the probing Surface Electromagnetic Waves were measured during the wave transmission and reflection process, from which the electrical impedance profiles along the steel reinforcement were calculated. From the impedance profile, the length, grouting/concrete distributions along the steel reinforcement were deduced. The deduced results were found correlating well with the actual length measurements, the construction record and known ground conditions.

INTRODUCTION

Ground anchors, soil nails, concreted steel H-piles and reinforced concrete piles (bored piles) are common geotechnical works installed in slopes or below ground for maintaining slope stability or transmitting building load into the ground. Quality control of length and grout integrity/concrete cover of the steel reinforcements in geotechnical works is of paramount importance for ensuring public safety. However, as these works are completely embedded inside the slope or below ground after installation, any quality monitoring or inspection of the steel reinforcements will be difficult, if not impossible. There is therefore a demand in the construction industry for the identification and development of a quick, accurate, reliable, non-destructive and in-situ measurement technique for checking the length and grout integrity/concrete cover of the installed underground steel reinforcements. This paper describes the principle of the Surface Wave – Time Domain Reflectometry (SW-TDR) technique and presents test results to demonstrate its applicability for this type of works monitoring.

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Principle of Surface Wave - Time Domain Reflectometry (SW-TDR)

This is based on Time Domain Reflectometry, using Surface Electromagnetic Waves as the probing waves. The method works on the same principle as radar. High frequency step-like Surface Electromagnetic Waves are excited and let transmitted onto the steel reinforcement of a soil nail, ground anchor, concreted H-pile, mini-pile or the reinforcement cage of a reinforced concrete pile (bored piles). The metal conductor acts as a waveguide for the Surface Electromagnetic Waves. The waves propagate along the metal conductor in the grouting/concrete material region. The velocity of the waves is determined by the dielectric properties of the surrounding materials. For general information on Surface Waves, readers may refer to papers by Sommerfeld [1], Zenneck [2], Goubau [3], Ling et al [6], Tang [7] or text books by Stratton [4] and Jordan et al [5]

When Surface Waves reach places where the local electrical impedance differs from the characteristic electrical impedance of the metal conductor, part or all of the wave energy will be reflected back. The polarity of returning waves will depend on the nature of impedance variation. The change of electrical impedance can be caused, for instance, upon the arrival of waves at the conductor end or the wave encountering of variation on cement grouting properties, (such as in the presence of voids or conductor in direct contact with surrounding soil etc.) SW-TDR records the potential and current magnitude of the probing waves during the entire wave transmission and reflection process. From the potential and current profile, the impedance profile for the metal structure is calculated. By analyzing these wave profiles, many parameters such as length, materials integrity and conditions around the conductors can be deduced. Propagation of Surface Electromagnetic Wave along a conductor is diagrammatically shown in Figure 1.

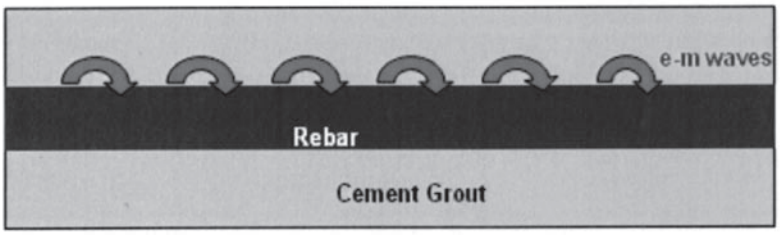


Figure 1 Surface Electromagnetic Waves propagating along a conductor

For cases such as soil nails and ground anchors etc., the electrical impedance, Z, relates to the inductance per unit length, L, and capacitance per unit length, C, of the conductor. The relationship can be approximated to:

$$Z \approx \sqrt{L/C}$$

The propagation velocity of the Surface Wave, v, is given by

$$v \approx 1/\sqrt{LC}$$

With the presence of void in cement/concrete mix around the metal structure or if there is any increment on grouting cover thickness, the local capacitance value will be decreased, leading to an increment of electrical impedance. When the probing electromagnetic waves reach places with

higher electrical impedance, part of the wave energy will be positively reflected and part will pass through. The coefficient of wave reflection, ρ, is given by:

$$\rho = \frac{Z_l - Z_0}{Z_l + Z_0}$$

where Z_l is the local electrical impedance value and Z_0 is the characteristic electrical impedance of the entire metal conductor.

On the contrary, when there is any decrement in cement grouting thickness, increment of dielectric values on the grouting or in situation where the steel reinforcement is in contact with soil, the local electrical capacitance value will be increased. This will lead to the lowering of electrical impedance for the region, which causes reverse (negative) wave reflection on the probing waves. Partial wave reflection and partial wave transmission will take place at every impedance mismatch region along the conductor. Upon the waves reaching the conductor ending, all the wave energy will be reflected back. Figure 2 shows the theoretical impedance profile for a soil nail with voided and soiled region along its grout core.

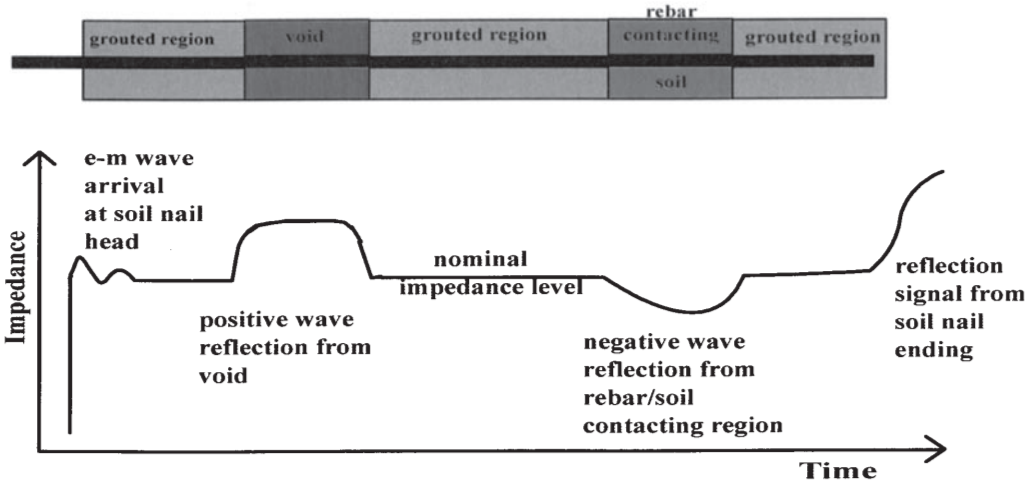


Figure 2: Soil nail with anomalies along rebar (above) and corresponding impedance profile (below)

INSTRUMENT USED

The instrument and the measurement technique is an in-house development project. The instrument has special circuitry for excitation and detection of Surface Electromagnetic Waves. The waves were excited and propagate on the tested metal object in a step-like manner, at a frequency of several hundred hertz. The maximum recording time resolution for the instrument was 500 picoseconds (1ps = 10^{-12} s.) In term of length, this corresponds to a distance resolution of about 1 cm for the normal wave velocities found on the common underground metal structures.

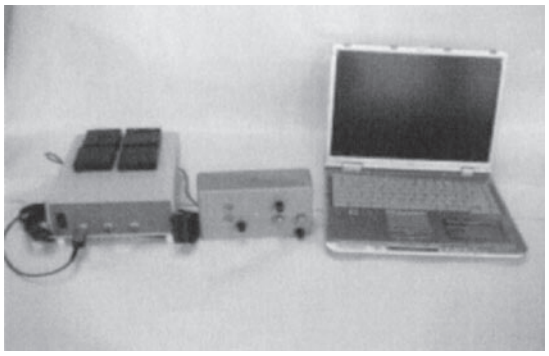


Figure 3 In-house developed Surface Wave Time Domain Reflectometer

CASES OF SITE MEASUREMENTS

The SW-TDR technique had been used in measuring steel reinforcements installed in slopes or below ground of a number of sites. Typical results of the SW-TDR tests carried on ground anchors, soil nails, concrete steel H-piles and reinforced concrete piles (bored piles) were shown in this paper.

In each measurement, 3 profiles along the steel reinforcement were shown: Potential, Current and Impedance. The profiles were placed one above the other for easy comparison. For full expression of information, the wave diagram included 2 X-axis and 2 Y-axis. The lower X axis was the wave propagation time. The time was in unit of nanosecond. The upper X-axis was the corresponding distance the wave had traveled during the time. The distance was in meter. The left Y-axis indicated the measured potential and current strength of the propagating waves. As the indicated current and wave potential profiles were deliberately placed one above the other for clear illustration and comparison purpose, their magnitudes were therefore in arbitrary unit. The right Y-axis was the calculated electrical impedance along the steel reinforcement. The impedance was in unit of ohm. The impedance profile was most important parameter to which time domain reflectometry was based on. In each wave diagram, there were markings on the time scale at which the waves entered the steel reinforcement and reflected from its ending. Brief comments on the observed integrity along the steel reinforcement were stated over the impedance profile.

The propagation velocity of the Surface Waves on each reinforcement type was calibrated at site before measurement with one or more similar metal reinforcement of known length, using following formula:

$$\text{Velocity} = \frac{2 \times \text{metal reinforcement length}}{\text{wave time span on the reinforcement}}$$

In the measurements, the distance to various locations along the steel reinforcement was calculated using formula:

$$\text{Length} = \frac{\text{wave velocity} \times \text{wave time span}}{2}$$

(A) SW-TDR test on ground anchors and soil nails

The ground anchors and soil nails under test were installed on retaining walls of 2 different features. They were originally installed in the 1970s. Because of age, there were no records found for these metal reinforcements, except for a few cases. The anchors/soil nails of known lengths and details were used for wave velocity calibration. The SW-TDR measurements took place in November, 2004, on a total number of 20 anchors/soil nails.

Prior to any tests on the site, several investigation holes were drilled at various locations on the wall/slope to obtain geological information on the slope. Pull-out load test were also performed on several anchors/soil nails, in addition to the SW-TDR test. In particular, on one soil nail, SW-TDR measurements were carried out before and after the pull-out load test. Typical results of the SW-TDR test measurements on anchors/soil nail were shown and discussed below:

Anchor A

The potential, current and impedance profile for this anchor were listed in figure 4.

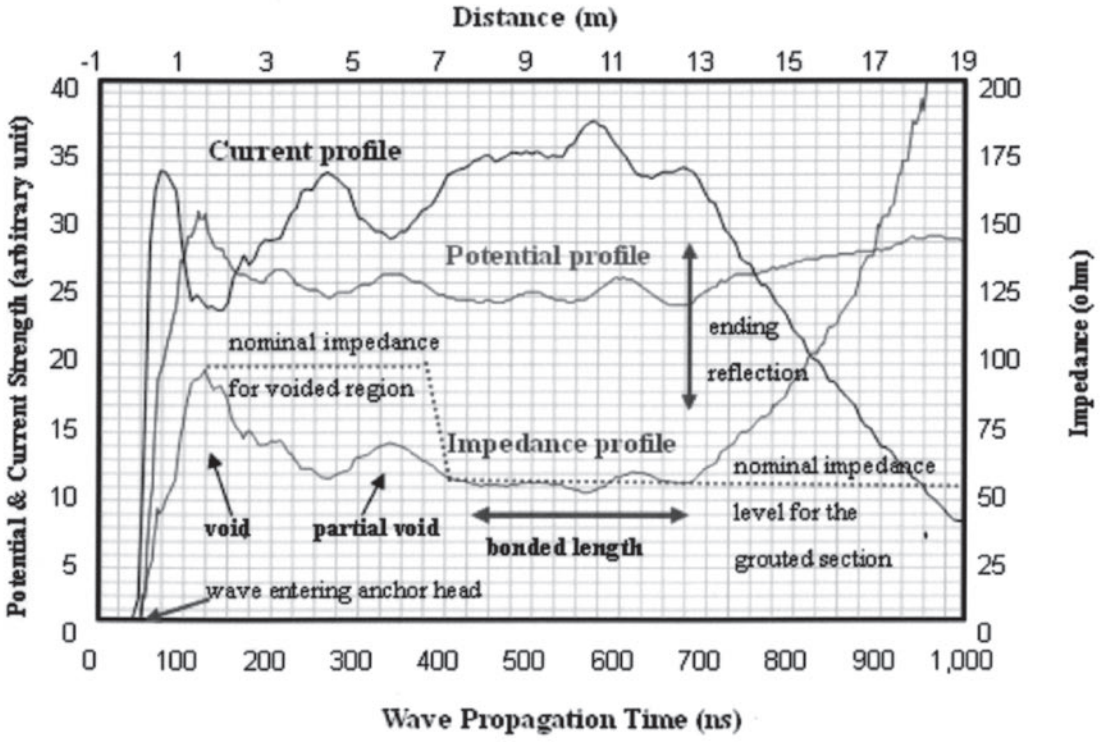


Figure 4: SW-TDR wave diagram for Anchor A

The wave time span for this anchor was measured to be 641 ns. At the site, the average wave velocity from 3 anchors of known lengths was measured to be 4.04 x 10^7 m/s. The 641 ns wave returning time would imply the length of the anchor to be 12.95 m.

The impedance profile showed the impedance of the steel reinforcement rose to 95 ohm level upon the waves entering the anchor head. This was the impedance level for the anchor over the voided (non-grouted) region. For the grouted region at the end of the rebar, the impedance was

measured to be 50 ohm as indicated. The dotted line indicated what should be the impedance profile for the ground anchor.

In comparing to the impedance level of 95 ohm at the voided region, the measured impedance of the steel reinforcement declined to some lower values in regions between the voided section at the beginning and the grouted section of the ground anchor at the end. The variation of impedance could be due to the presence of some other dielectric materials around the steel reinforcement. These materials caused the local lowering of impedance on the rebar, forming the partial voided region along the steel reinforcement. The dielectric material was likely the protective grease. From time about 400 ns onwards on the lower time scale, the steel reinforcement was found fully grouted. The bond length for the anchor was measured to be 5.56 m.

Anchor B

The potential, current and impedance profile for this anchor were listed in figure 5.

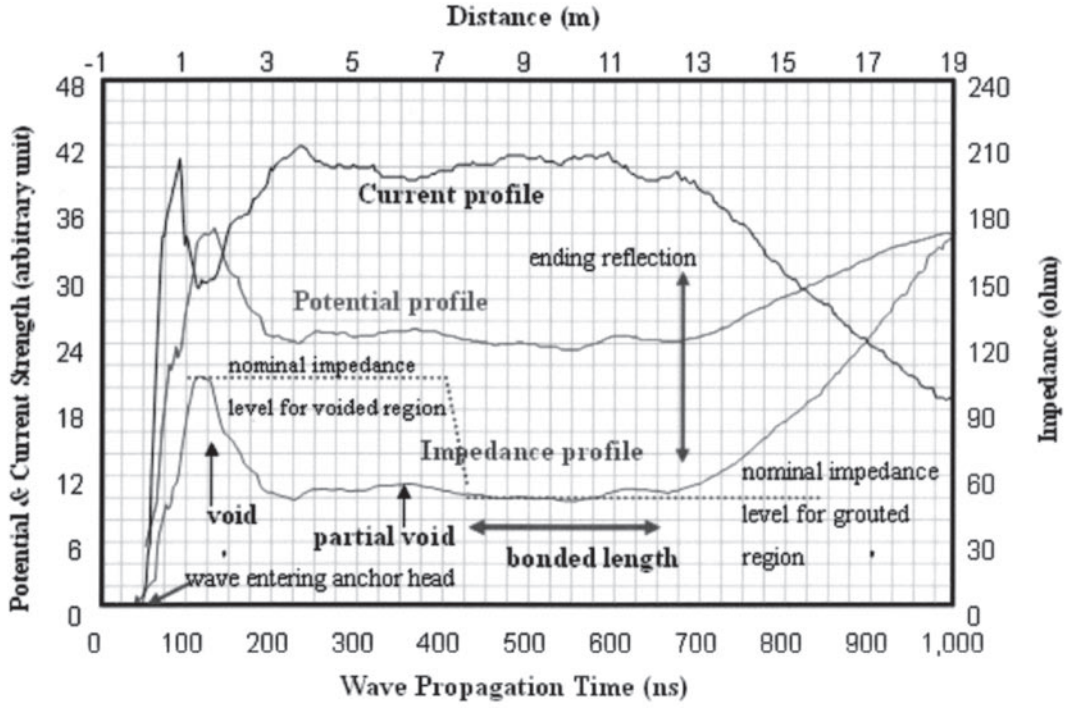


Figure 5 : SW-TDR wave diagram for Anchor B

The wave time span for this anchor was measured to be 620 ns. From the average calibrated velocity of 4.04 x 10^7 m/s, this would imply the length of the anchor to be 12.52 m.

The measured impedance profile showed situations similar to Anchor A above, with a voided (non-grout) region in the front part, followed by partial voided region and fully grouted section at the latter part of the anchor. In comparison to the impedance magnitudes at the partial voided region in Anchor A , the impedance at the partial voided region on this anchor was at lower values. This might be due to the thicker grease covering materials at the partial voided region on this anchor. The impedance profile indicated the bonded length for the anchor was 4.75 m.

Soil nail C

This was a soil nail on a different feature at the site. Pull out load test had also been carried out on this nail. SW-TDR measurements were conducted on this soil nail prior to and after the pull-out load test. The potential, current and impedance profile for the pre and post pull-out load test situation were listed in figure 6.

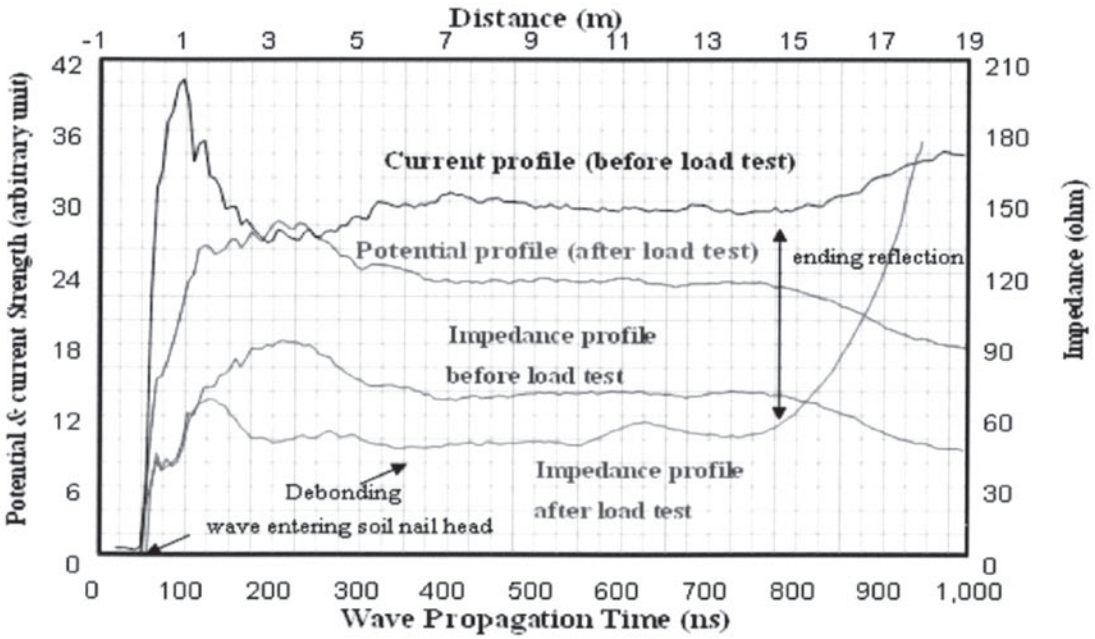


Figure 6 : SW-TDR wave diagram on soil nail C

The wave time span on the pre and post pull-out load test remained the same at 738 ns. Using the calibrated velocity of 40.40 x 10^6 m/s, this would imply the length of anchor to be 14.91 m.

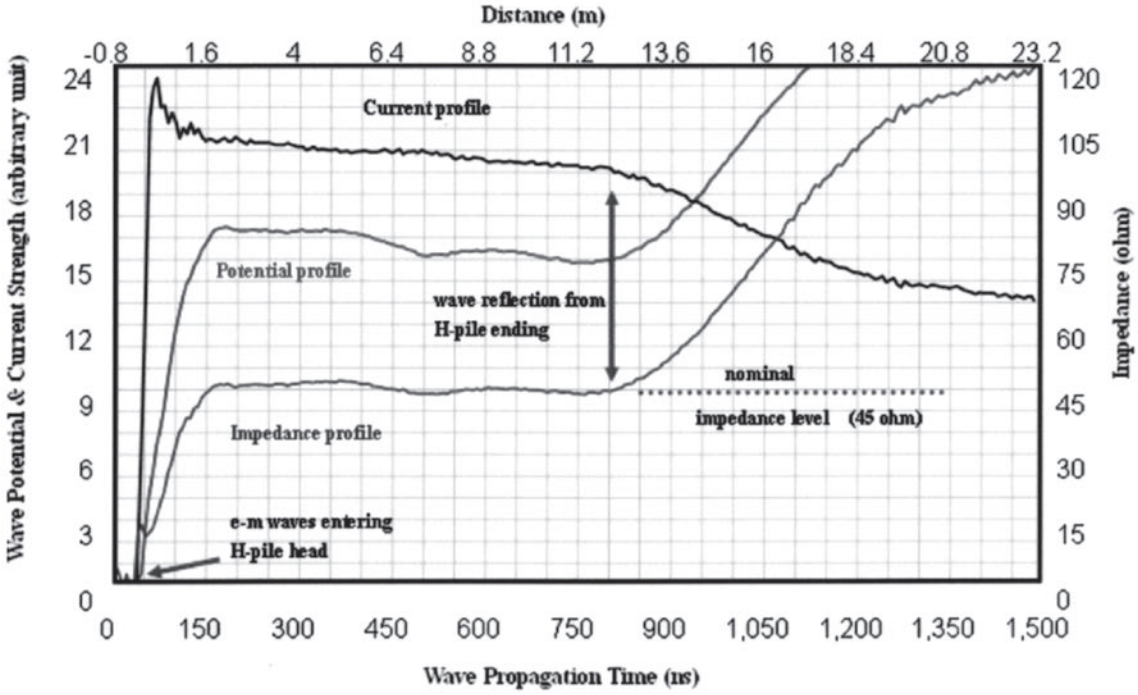
In comparing the 2 impedance profiles, there was a noticeable difference in magnitude and wave shape between the 2 cases. The impedance for the post pull-out load test case was generally lowered than before, from the previous 70 ohm to the latest 50 ohm. In addition, the polarity of impedance variation at the soil nail ending had been reversed; from negative change for the pre pull-out load test case to the positive change after the load test. This change of reflection polarity at end indicated the de-bonding and movement of the soil nail had probably occurred during the pull-out load testing. As shown by the decreasing impedance at its end, the soil nail was in contact with the surrounding soil at its end before the pull-out test. After the load test, the bonding between the grouting and soil could have been de-bonded and the steel reinforcement was no longer in contact with soil, resulting in higher electrical impedance at the end. This caused positive wave reflection for the Surface Waves. The general de-bonding of grouting to the surrounding soil could be noticed from the decreased anchor impedance value. The de-bonding situation started at 2.36 m after the soil nail head position and lasted till the end of the anchor.

(B) SW-TDR test on concrete steel H piles

The site was at a location very near the waterfront in Hong Kong Island. Owing to the close sea front proximity and reclaimed land nature of the site, there were concerns that pile shaft necking may occur during the construction of the in-situ concrete pile with H section insertion (concreted H-pile). SW-TDR technique was used to check the concrete cover integrity on the piles. Time of measurement was on January 2005, on a total 7 concreted H-piles.

Concreted H Pile D

The SW-TDR wave diagram for this concreted H pile was shown in Figure 7 below.



Firgure 7: SW-TDR wave diagram for H-pile D

This pile was used for the calibration of wave velocity. The wave time span on the concreted H pile was measured to be 750 ns. The length of the concreted H pile was given at 12.10m. The wave velocity was therefore calculated to be: 32.26 x 10^6 m/s. This deduced velocity was used as the calibrated wave velocity for the SW-TDR testing on the rest concreted H-piles at the site.

The impedance profile for this concreted H pile indicated that the overall concrete cover along the concreted H pile was very uniformly distributed, as seen from the smooth and almost constant impedance level along the entire H section. There was only a minor decrement of impedance at about middle part of the piling. The impedance variation was, however, very small and it could be neglected.

The constant impedance level was the nominal characteristic impedance for the concreted H pile. From the impedance profile, it was measured to be 45 ohm. This nominal level was marked next to the impedance curve.

Concreted H Pile E

The wave diagram for this concreted H pile was listed in Figure 8.

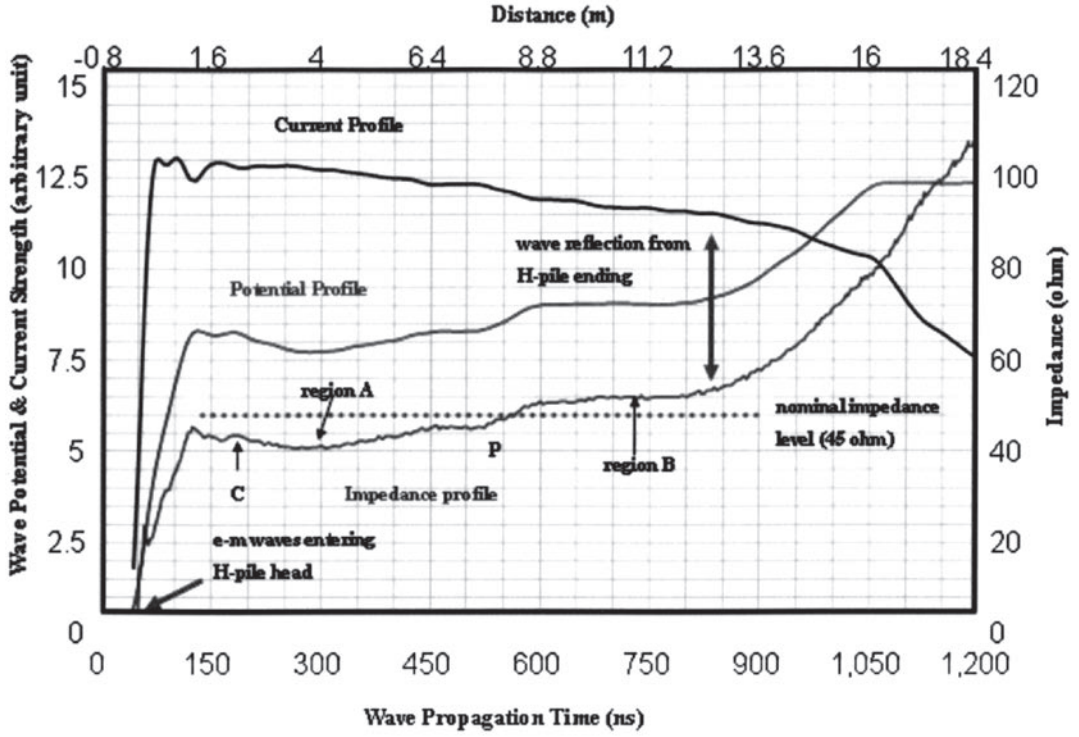


Figure 8: SW-TDR wave diagram for H-pile E

The time span from the arrival of Surface Electromagnetic Waves onto the pile head till the wave reflection from the ending was measured to be 776 ns. Using the calibrated velocity of 32.26 x 10^6 m/s, this would imply the length of concreted H pile to be 12.52 m.

In comparing to the 45 ohm nominal impedance, the general lower impedance in region A indicated that the grout cover may be thinner in this section of the concreted H pile. This could imply that the concreted H pile might have been placed off-centred. The off-centering situation in the present H pile was not a severe one, however, as indicated by the relative small deviation from the normal impedance level of 45 ohm.

At position C, the lowering of impedance was seemingly faster. This indicated the metal conductor had in contact with soil. The soil contact could have aroused from the metal rod spacers, which were welded onto the H section body at that location. Position C was measured to be 140ns wave time from the pile head. This distance would correspond to 2.26m.

From position P onwards, the impedance profile showed an increment in impedance values. The impedance was steady at 50 ohm till the end of the pile. This position P was measured to be 469 ns from the pile head position, corresponding to a distance of 7.56m.

From the discussion with the borehole drilling operator, it came to our understanding that there existed some layers of soft clay materials underneath the ground at the site. The clay layers were typically found at depths around 2-3 m and 6 – 7 meters or so below ground surface. The 7.56 m of position P corresponded well with second the lower layer of soft clay materials.

Soft clay materials are loose bounded materials with high degree of mobility and water content. During the removal of casing after grouting filling process, some soft clay materials might have intruded into the cement grouting at the clay layer level. Since the site was very close to sea and the soft clay was below the sea level, some water might even have penetrated into the grouting. Furthermore, it was understood that some soft clay materials had not been cleared from the bottom part of the drill hole after the drilling process. When cement grouting was poured in, the grouting would include and mix with the soft clay materials from the bottom. The net effect of these foreign inclusions, mixing or water penetration into grout materials would cause the dielectric value of the grouting materials to be higher and hence higher electrical impedance for the H section at the lower part of the pile.

There was no void or necking situation found in the piling. If existence of voids (no concrete cover) were the case, the measured impedance should be much higher at 127 ohm.

Concreted H Pile F

The measured potential, current and impedance profile for this pile were listed in Figure 9.

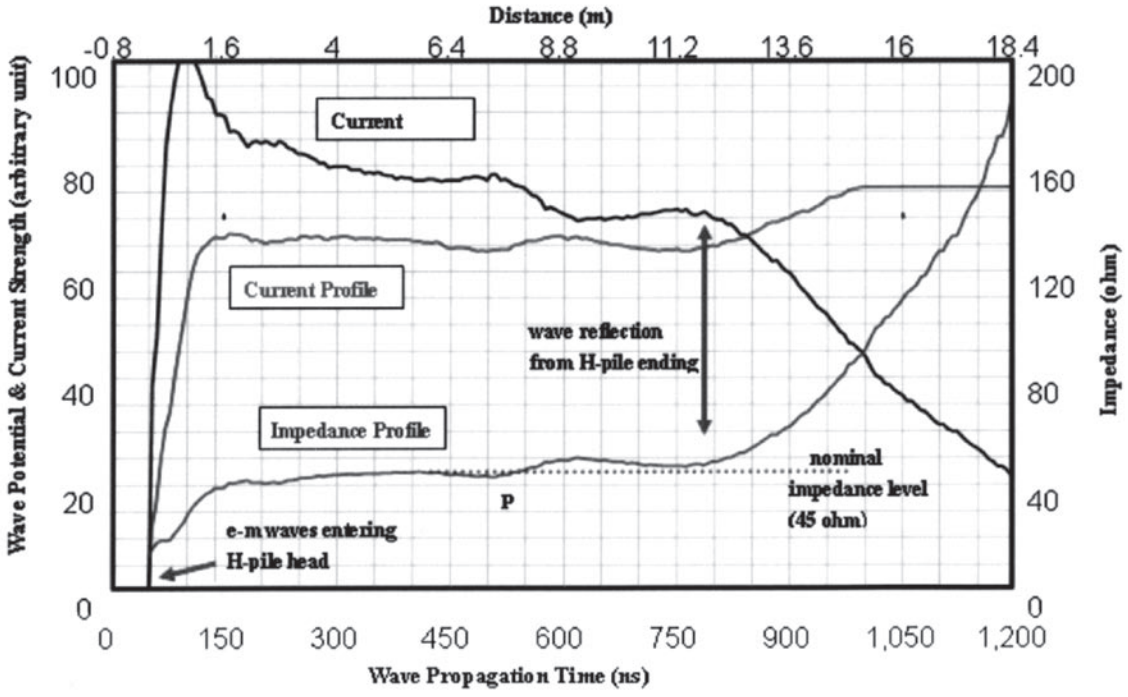


Figure 9: SW-TDRWave for H-pile F

The wave time span from e-m wave entering the concreted H pile head to that wave reflection from the ending of the pile was measured to be 767 ns. With wave velocity of 32.26 x 10^6 m/s, this could imply the length of the H section to be 12.37m.

The impedance profile for this concreted H pile showed a fairly constant level of 45 ohm, which was the nominal impedance for a "good" concreted H pile. There was, however, an increment of impedance starting from position P. The impedance value increased and fell again, forming a bulb shape impedance variation. Time to position P was measured to be 479 ns, corresponding to a distance of 7.73 m below the pile head.

As in previous case, the increment of impedance at P could be due to the encountering of clay layer at that level. Some soft clay materials should have intruded into grout core, resulting higher local impedance for the concreted H pile. However, the foreign inclusion of clay should not be much, as indicated by the small change of impedance from 45 ohm nominal impedance to 50 ohm highest. After the small impedance variation region, the cement grouting returned to the normal 45 ohm impedance. The maintaining of nominal impedance at the piling ending indicated there was no soft clay inclusion from the bottom part of the drill hole.

There was no voided region or necking situation found on this pile.

(C) SW-TDR test on Bored Piles

2 bored piles from 2 different sites were tested. The sole purpose of the testing was to verify the capability and accuracy of the SW-TDR results and its wave interpretations on the bored piles. All details of the steel reinforcement cage were carefully recorded prior to the measurements. These details were not released to us prior to our submission of the report. The SW-TDR testing on the bored piles were completed in July 2004.

(i) Testing on voided situation

The pile under test was a 2.2 m diameter bored pile. The steel reinforcement cage comprised 45 vertical rebar, each of size 40 mm in diameter. The bored pile formed part of a retaining wall abutting an uphill slope. During the construction of the pile, part of the cage was covered with polystyrene foam boards. The placement of these foam boards was to screen the steel reinforcement from the concrete mix during the casting stage. After the setting of the concrete, excavation would be carried out at the foam boards' side of the bored pile wall. Additional steel reinforcement would then be connected to the metal cage, for further construction of wall panels on the bored piles. The SW-TDR test was to investigate the ability of the technique in the detection of these 'voided' regions. The view plan for the steel reinforcement cage was shown in figure 10.

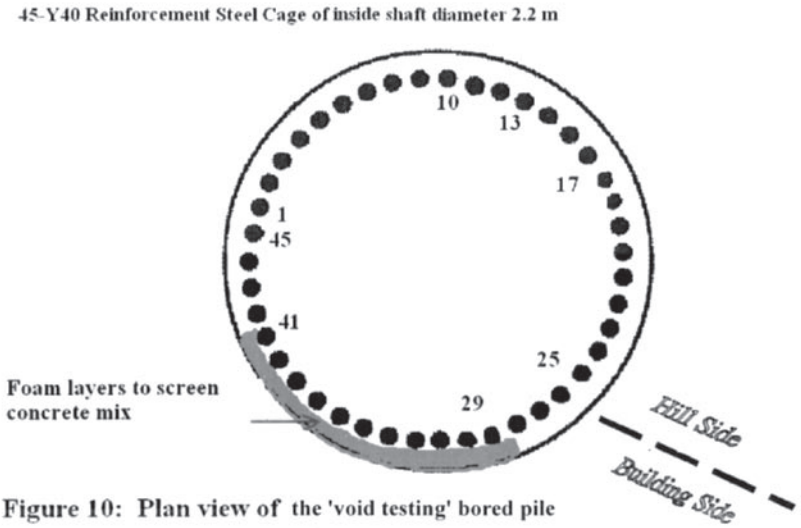


Figure 10: Plan view of the 'void testing' bored pile

For the testing of bored piles, the reinforcement cage acted as the waveguide for the Surface Electromagnetic Waves. The waves would always propagate on the shortest route to the cage (conductor) ending. The wave signals received at the steel cage head would compose all signals leading to the termination of the conductor system, including reflections from the overlap joints and anomalies along the rebar. The SW-TDR measurements on this bored pile were conducted on Rebar 13, 17, 29 and 41. Rebar 13 was used for the calibration of e-m waves.

Results on Rebar 13

There were no foam board placements along Rebar 13. The potential, current and impedance profile for this rebar were listed in figure 11.

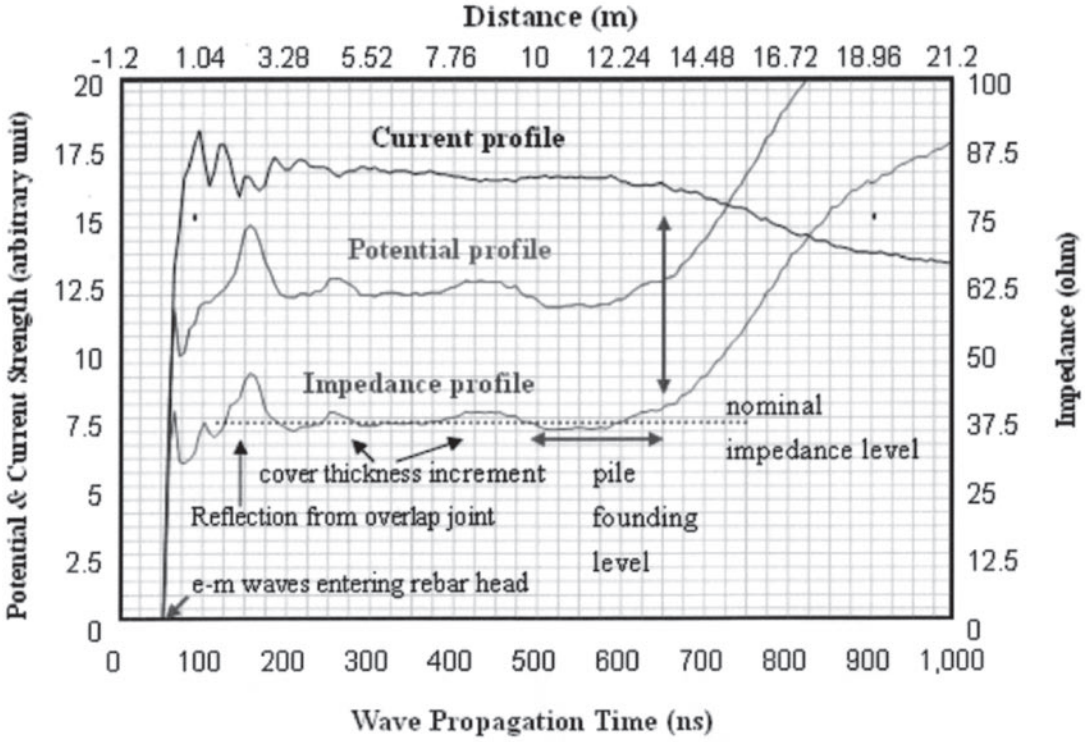


Figure 11: SW-TDR wave diagram on Rebar 13 of 'void testing' bored pile

The SW-TDR wave diagram recorded a wave reflection from the overlap joint at 1.5 m from the cage head, with 2 other smaller wave reflections due to the local increment of cover thickness. The decrement of impedance at the end of the pile indicated that the pile bedrock embedment length was 3.1m.

Total wave time span on the cage was measured to be 597 ns. The length of the case was 13.37m. The wave velocity was therefore calculated to be 4.479 x 10^7 m/s. This was the calibrated wave velocity for the rebar of the bored pile.

Results on Rebar 17 were very similar to Rebar 13 above and not reproduced herewith.

Results on rebar 41

Foam boards were regularly placed over the upper part of reinforcement cage on section from rebar 29 – 41. SW-TDR results on rebar 29 and 41 were very similar, and were typical shown in figure 12 below.

The impedance profile showed periodic impedance variation at the upper half of the piling. The periodically variation would be caused by the placement of the foam boards, which led to the local increment of impedance values. The pattern of the impedance variation corresponded to the arrangement of the foam board placements. The pile bedrock embedment length deduced from results on Rebar 29 and Rebar 41 was 4.01m, as compared to the 3.10m of embedment length on the hill facing side of the pile. This again agreed well with the actual ground condition.

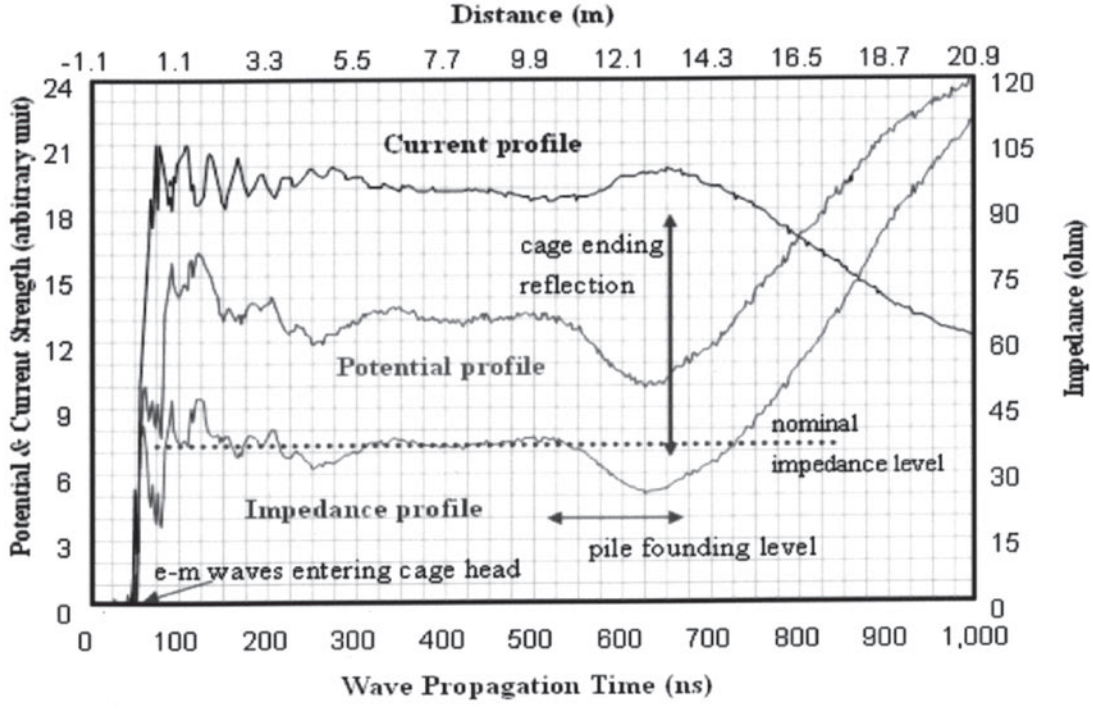


Figure 12: SW-TDR wave daigram on Rebar 41 of the 'void testing' bored pile

The position of the rebar overlap joint on the steel reinforcement cage, foam boards locations on the rebar and the pile founding level deduced from the wave diagrams showed good agreement with the contractor record for the tested pile.

(ii) Test for length measurement on bored pile

For further verification on the accuracy and wave interpretations on the SW-TDR measurements, a test had been carried out on a bored pile which had an additional dummy rebar added onto its steel reinforcement cage. This dummy rebar comprised 3 sections of rebar. The overlap joint between the first 2 rebar sections was constructed in the usual way and was conductive. The second overlap joint between the intermediate and lower rebar section was made non-conductive, by covering up the lower rebar head with PVC sleeve. Figure 13 showed the dummy rebar arrangement. The dummy rebar formed an integral part of the reinforcement cage.

Figure 13: Dummy bar arrangement on the steel reinforcement cage

The exact arrangement and positions of the dummy rebar sections were carefully measured and well recorded prior to the construction of the piling. The SW-TDR test was carried out on the dummy bar. From wave velocity measured on another bored pile at the site, the calibrated wave velocity on the bored pile for this site was measured to be 4.479×10^7 m/s.

During the measurement, the SW-TDR instrument was placed on the ground level, which was 1.5 m below the upper dummy rebar head. The measured potential, current and impedance profile of this dummy rebar were listed in figure 14. For easy understanding on the impedance wave shape, the actual dummy rebar arrangement was also listed directly underneath the measured impedance profile.

The impedance profile showed a positive spike-like increment at 12 m position from the top rebar head. This corresponded to the ending of the upper rebar section. The impedance over the 2m overlap section between the upper and intermediate rebar also showed an increment in value. When the Surface Waves arrived at the end of the intermediate rebar, it was strongly reflected. The reflection was due to the electrical disconnection at that overlap joint. The current magnitude showed a negative down-turn at the ending of the intermediate rebar, whilst the impedance profile

showed a strong increment in value. The time for this impedance reflection was measured to be 911ns, corresponding to a total length to the intermediate rebar ending from the ground level to be 20.34m. This reported figure agreed very well with the recorded value of 20.37m.

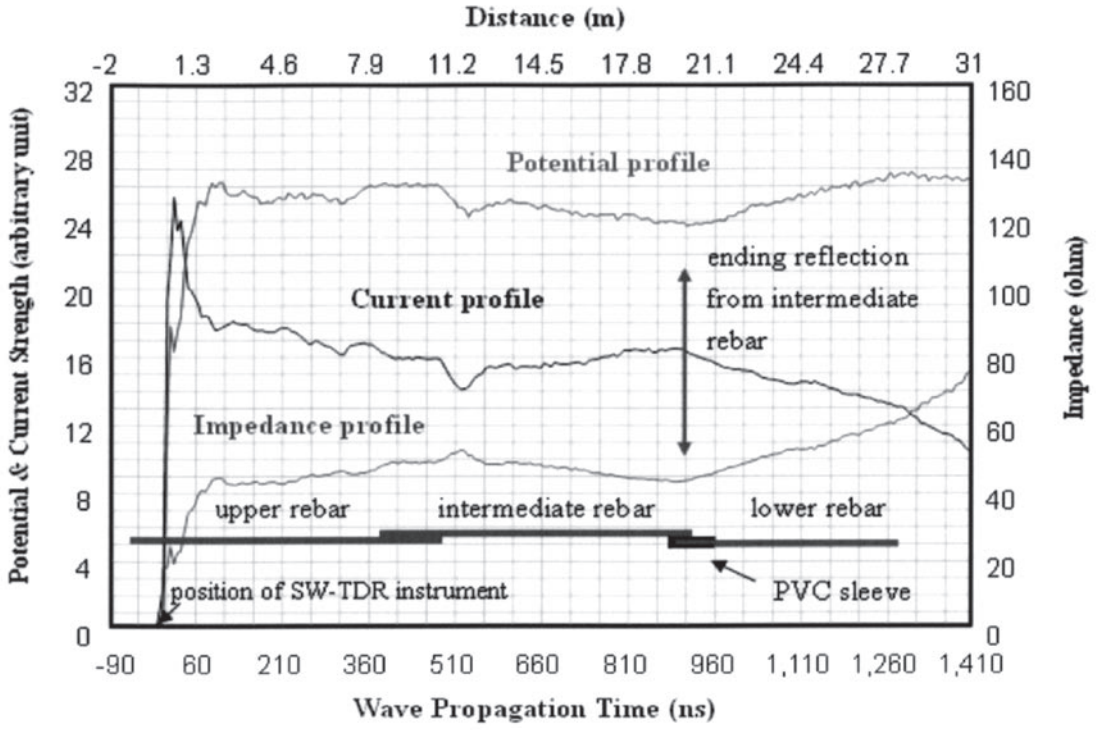


Figure 14: SW-TDR wave diagram on the dummy rebar

DISCUSSION

SW-TDR technique measures the impedance variation along the conductor. The steel reinforcement acts as the waveguide for the probing Surface Electromagnetic Waves. The wave velocity is a function of dielectric values of the surrounding grouting/concrete mix materials. The portion of wave transmission and reflection depends on the local impedance of the conductor, which, in turn, is related to the electrical inductance and capacitance value at the spot. The conductor can be the steel reinforcement in any underground geotechnical works. The above test cases clearly demonstrate that one can measure from the impedance profile the rebar length and deduce the cement grouting/concrete conditions and its thickness along the steel reinforcement, with a high degree of accuracy.

The velocity of Surface Electromagnetic Waves would be best calibrated at site with one or more similar steel reinforcement of known length. In situation when no steel reinforcement length is known for whatever reasons, it is still possible to calibrate the wave velocity by measuring the dielectric value on a small piece of grouting/concrete materials around the steel reinforcement and calculate the wave velocity afterwards. On the other hand, one could also use the average wave velocity found elsewhere on that type of steel reinforcement. The deviation from true value for

SW-TDR measurements without velocity calibration at site would normally be within 15%-20%, when comparing to the usual maximum 6% deviation in the calibrated case.

For full extraction of information from the SW-TDR measurements, wave interpretations on the wave diagram are, however, required. The interpretation will require trained personnel with analytical mind and adequate knowledge in the design and construction of the installed underground steel reinforcements. As the conditions and type of steel reinforcement varies from site to site, auto interpretation of waves by computer may not be materialized for some considerable time. To help wave recognition and calculation of anomalies for the less experienced personnel, a set of reference SW-TDR wave diagrams from known defective situation would be recommended. For defects of same nature, the wave profiles should be similar to that of reference guide. The sizes of anomalies can be calculated using the calibrated Surface Wave velocity at site.

The measured signals from the SW-TDR measurements on all steel reinforcements under test were large and clear. The length and integrity measurement agreed and correlated with known records and geological observations from nearby ground cores. SW-TDR technique can also be used for other geotechnical applications, such as slip plane monitoring, real time underground water table measurement and long term steel bar corrosion monitoring etc. Introduction on these geological applications can be found from another paper by Tang [7].

CONCLUSION

SW-TDR technique is identified as an in-situ and non-destructive test able to measure the length and grout integrity/concrete cover of installed underground steel reinforcements of geotechnical works. The results of the SW-TDR tests on installed underground steel reinforcement of ground anchors, soil nails, concreted steel H-piles and reinforced concrete piles (bored piles) clearly demonstrate that the method is reliable, accurate and informative. It is feasible and suitable for practical uses in the field for quality control of all types of installed underground steel reinforcements.

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ACKNOWLEDGEMENT

This paper is published with the permission of the Director of Architectural Services of the Hong Kong Special Administrative Region.

LABORATORY SIMULATIONS OF SOIL NAIL FAILURE IN THREE-DIMENSIONAL SLOPE

K.W. Tong[1] and K.T. Chau[2]

Abstract: This paper summarizes our recent laboratory simulations of failure mechanisms of three-dimensional (3-D) slope with soil nails, and their comparisons of the failure with the same 3-D slope without soil nails. The slope is 0.8m high with a scale of 1:3 and a slope angle of 65°. A sandy soil with water content of about 5% was compacted to form a three-dimensional slope by an electric vibrator in 8 equal layers each 100mm thick. The 3-D slope consists two 2-D slopes intersecting at 135° (note that when the intersecting angle becomes 180° the 3-D slope becomes a 2-D slope), with a soil relative density of 85%. The cohesion is 3 kPa whereas the friction angle is 51° (which were obtained by shear box tests on 3 undisturbed samples with a coefficient of correlation better than 0.999). For the case of soil-nailed slope, the soil nail is installed in-situ by drilling a hole of 20 mm into the soil, and mild steel bars of 6 mm diameter and length of 800 mm were placed in the drilled holes and cement-sand grout (with a 3-day strength of 18.7 MPa) were poured into hole. A total of eight soil nails were installed on two different levels on the two intersecting faces (i.e. each face has 4 soil nails). The inclination of the soil nails are at 20° with horizontal, pointing inward the slope face. The pullout bond strength of each nail is 16 kPa, and the coefficient of sub-grade reaction of the soil on the nails is about 68 ton/m^3. The slope is loaded to complete failure under a loading frame capable of applying 500 kN, and the location of the loading platen is selected such that soil nails are forced to fail. The loading capacities of the 3-D slope increases 25% if soil-nailed is installed. When peak load is attained, the vertical deformation at the top of the slope for 3-D slope increases by 100% if soil-nailed is installed. The 3-D slope failed in a catastrophic fashion whereas the soil-nailed slopes never failure catastrophically. The 3-D slip surface passes around most of the soil nails and only two soil nails actually failed completely. The left soil nail bended both downward and outward from the centerline but there is no pullout failure, whereas the right soil nail bended mainly downward and slightly outward but with pullout failure. The cement-grout was completely crushed at the bending points for both soil nail. The results of the present simulation provide the first ever failure mechanism of soil nail in 3-D slopes, and provide invaluable information for verifying 3-D mathematical models.

INTRODUCTION

In recent decades, soil nails have been used extensively to upgrade substandard and deteriorated soil slopes. A comprehensively design recommendations for soil nail are given by FHA (1993). There are, however, serious gaps between the practical use of this commonly employed method of slope remediation and the theoretical development of method of analysis for soil nailed slopes, especially for three-dimensional (3-D) cases.

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Simple theoretical analysis normally assumes that the bending stiffness of soil nails can be neglected and only the pullout strength is mobilized at failure. These pullout forces are then distributed as compressive pressure on the slope surface and the corresponding factor of safety of the slope is found by traditional method of slices such as the Morgenstern-Price method. More sophisticated approach models the soil nail as simple beam bending with soil pressure in the form of sub-grade reaction (which is more like pile deformation). But, frictional sliding or pullout resistance is highly nonlinear process that can hardly be modeled adequately. Most existing approaches also implicitly assume both the nail and the slope are two-dimensional (2-D). Since soil-nailed have never been loaded to failure in the field and is seldom loaded to failure in the laboratory. The failure mechanism of soil nails have never been documented comprehensively under real loading situation. More specifically, to the best of our knowledge, no study has been conducted investigating the failure mechanisms of soil nails in 3-D slopes both in the laboratory and in the field. This perhaps is the reason why the theoretical developments of soil nail analysis have not been fully matured (Smith and Su 1997).

The only related laboratory tests on soil nails are those by Kim et al. (1995), and Raju et al. (1997). More recently some laboratory tests on soil nails were also done at PolyU using large shear box (Yin et al., 2005). But none of these studies considered the soil nail in 3-D slopes. In terms of field tests, the only reported studies were those by The French National Project CLOUTERRE (FHA, 1993), Tang and Lee (2003), and Yin et al. (2003). But, again, none of these field tests consider the 3-D effect.

In order to fill this gap, we propose in this study a physical model of 3-D soil slope both with and without soil nails. Sandy soil will be used to build a 3-D slopes by using electric compactor. Soil nails are then installed similar to the procedure adopted in the field. The failure or slip surfaces of 3-D slopes with and without soil nails are compared. To the best of knowledge, the failure mechanism of soil nail in 3-D slopes is reported for the first time. The results of this paper should shed light on future laboratory, field, centrifuge, and theoretical studies.

PHYSICAL MODELS FOR 3-D SLOPES

Model set-up

A steel tank was built for making physical models of 3-D slopes. The dimension of the tank is shown in Fig. 1. The dimensions in plan view are given in mm.

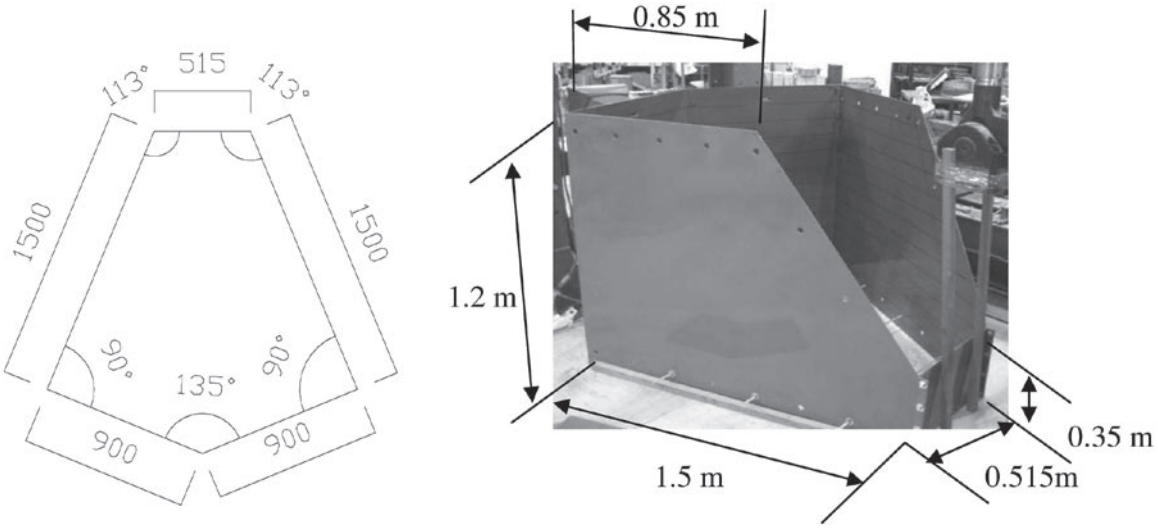

Figure 1. The layout of the steel tank used to build our 3-D slope models

The slope is 0.8 m high with a scale of about 1:3 and a slope angle of 65°. A sandy soil with water content of about 5% was compacted to form a three-dimensional slope by an electric vibrator in 8 equal layers each 100mm thick. The 3-D slope consists of two 2-D slopes intersecting at 135° (note that when the intersecting angle becomes 180° the 3-D slope becomes a 2-D slope), with a soil relative density of 85%. Figure 2(a) shows the 3-D slope without soil nail together with the locations of the LVD displacement transducers.

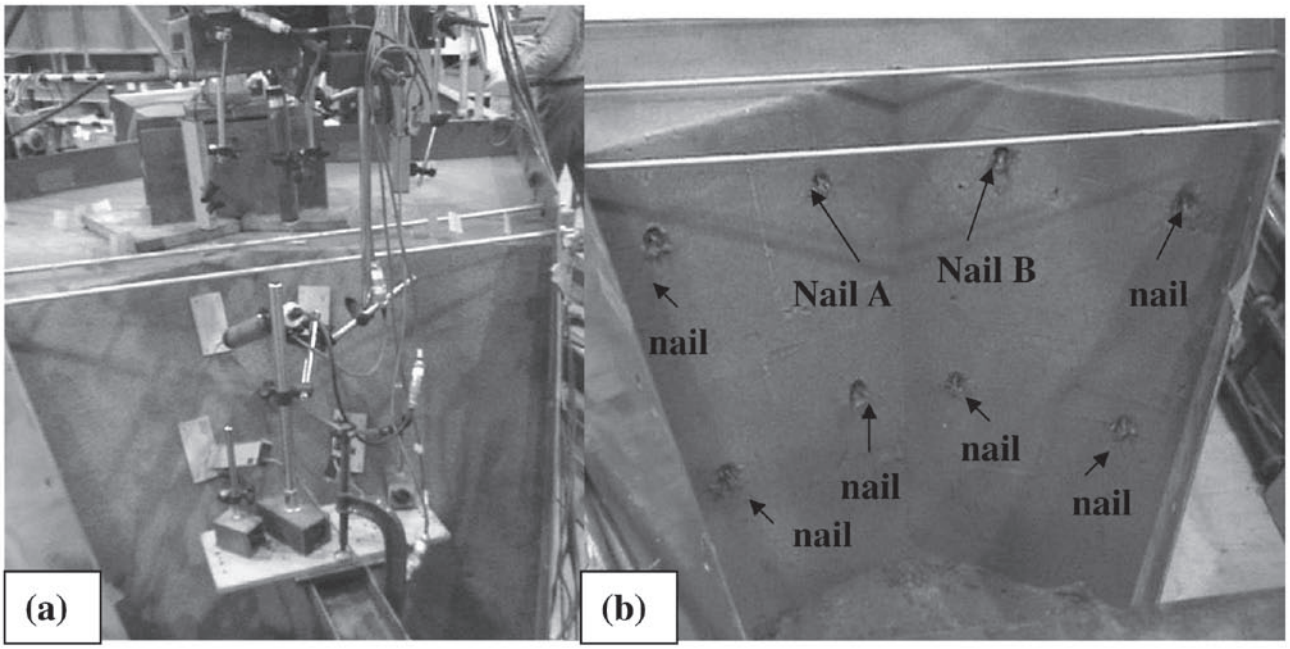

Figure 2. (a) 3-D soil slope without soil nail (LVD transducers were also shown); (b) 3-D soil slope with soil nail.

For our mechanically compacted sandy soil, the cohesion is 3 kPa whereas the friction angle is 51°. These shear strength parameters are obtained by conducting shear box tests on 3 undisturbed samples from our packed slopes. The applied normal stresses for the 3 samples are 5 kPa, 10 kPa and 15 kPa respectively. Although the friction angle is a bit high, the best fit Mohr-Coulomb failure model has a coefficient of correlation larger than 0.999 (i.e. it is very reliable). This high friction angle is probably resulted from compaction using mechanical vibrators in packing the soil. For the case of soil-nailed slope, the soil nailed is formed in-situ by drilling a hole of 20 mm into the soil, and mild steel bars of 6 mm diameter and length of 800 mm were placed in the drilled hole and cement-sand grout (with a 3-day strength of 18.7 MPa) were poured into the holes. The full details of the installation of the soil nails are referred to Tong (2005), and will not be given here due to space limitation. As shown in Fig 1(b), a total of eight soil nails were installed on two different levels on the two intersecting faces (i.e. each face has 4 soil nails). The inclination of the soil nails are at 20° with horizontal, pointing into the slope face. The pullout bond strength of each nail is 16 kPa, and the coefficient of sub-grade reaction of the soil on the nails is on average 68 ton/m^3. The full details of the pullout test are again referred to Tong (2005). The slope is loaded to complete failure using a loading frame capable of applying 500 kN, and the location of the loading platen is selected such that soil nails are forced to fail. Load cells and LVD displacement transducers are installed on the rigid loading platen so that the maximum load capacity can be monitored and the accelerating phase of slope failure can be cautioned during

the loading test. The load is applied through a hand-controlled hydraulic loading system. The loading process is actually displacement-controlled but with the vertical applied force being monitored at all times. All records were stored in a data acquisition system and transferred to a computer system. Typically, the slope is loaded to complete failure in two hours. Locations of crack initiations on the top and surface of the slope are monitored using flags of different colors as a function of the applied load.

Failure Pattern of the 3-D slope without Soil Nails

The 3-D slope without soil nail failed dynamically and catastrophically. The final failure pattern was shown in Fig. 3. Note that a very peculiar and spectacular shear layers were formed at the base or toe of the failed slope. The close-up views of these laminated formations were also given in Fig. 3. Each layer is about 2 mm thick or composes of a few grains of the sandy soil. This formation closely resembles what we normally observe in the folding layers found in mountains of sedimentary rocks when the rocks have been compressed significantly. The upper failure masses cracks into a number blocks which closely resembles active failure of blocks.

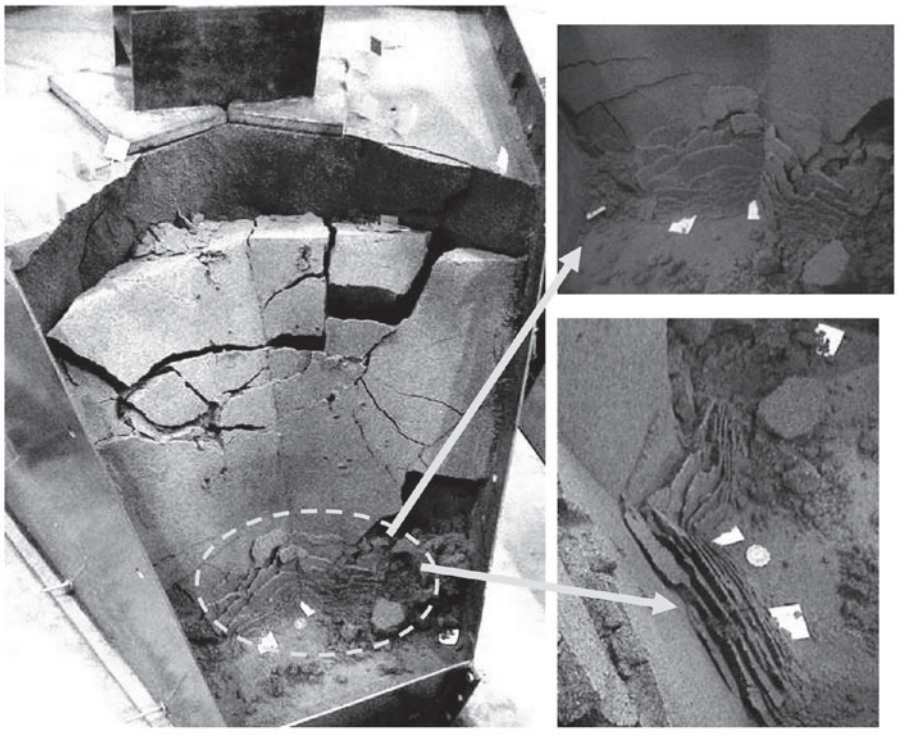

Figure 3. Failure pattern of the 3-D slope without soil nails. Close-up of laminated shear formation at the toe of the slope is also shown.

Failure Pattern of the 3-D slope with Soil Nails

The failure pattern as well as the mechanism of the 3-D slope with soil nails differs from those shown in Fig. 3. The process of failure is summarized in Fig. 4. As shown, although the deformation can be quite substantial but the failure process is incremental and there is no catastrophic failure as those observed in Fig. 3. The small colored flags stuck to the slope surface were used to show the positions of cracks observed during the loading process. The last picture in sequence shows the excavated surface of the failed slope. Nails A and B shown

in Fig. 2 were seriously bended and deformed. Nail A was bended both downward and sideway but there is no pullout failure, whereas Nail B bended mainly downward with some sideway bending with pullout failure (i.e. the remaining unexcavated nail can be pulled out easily by free hand). The cement grout near the bending points was severely crushed. All 6 other soil nails are basically intact, suggesting that the slip surface in soil optimizes itself in such a way that the number of soil nails involved in the failure process was kept to a minimum. Although both soil nails and displacement transducers were arranged in a symmetric pattern on the 2 slope faces, the observed deformations were found unsymmetric. Therefore, it seems that a slight imperfection on the 3-D slope will generate an unsymmetric failure pattern. As it was described earlier, the failure mechanism of Nails A and B are not the same and thus 3-D slope failure is a very complicated process and clearly an idealized 2-D is inadequate in predicting both the strength as well as the failure process of 3-D slopes.

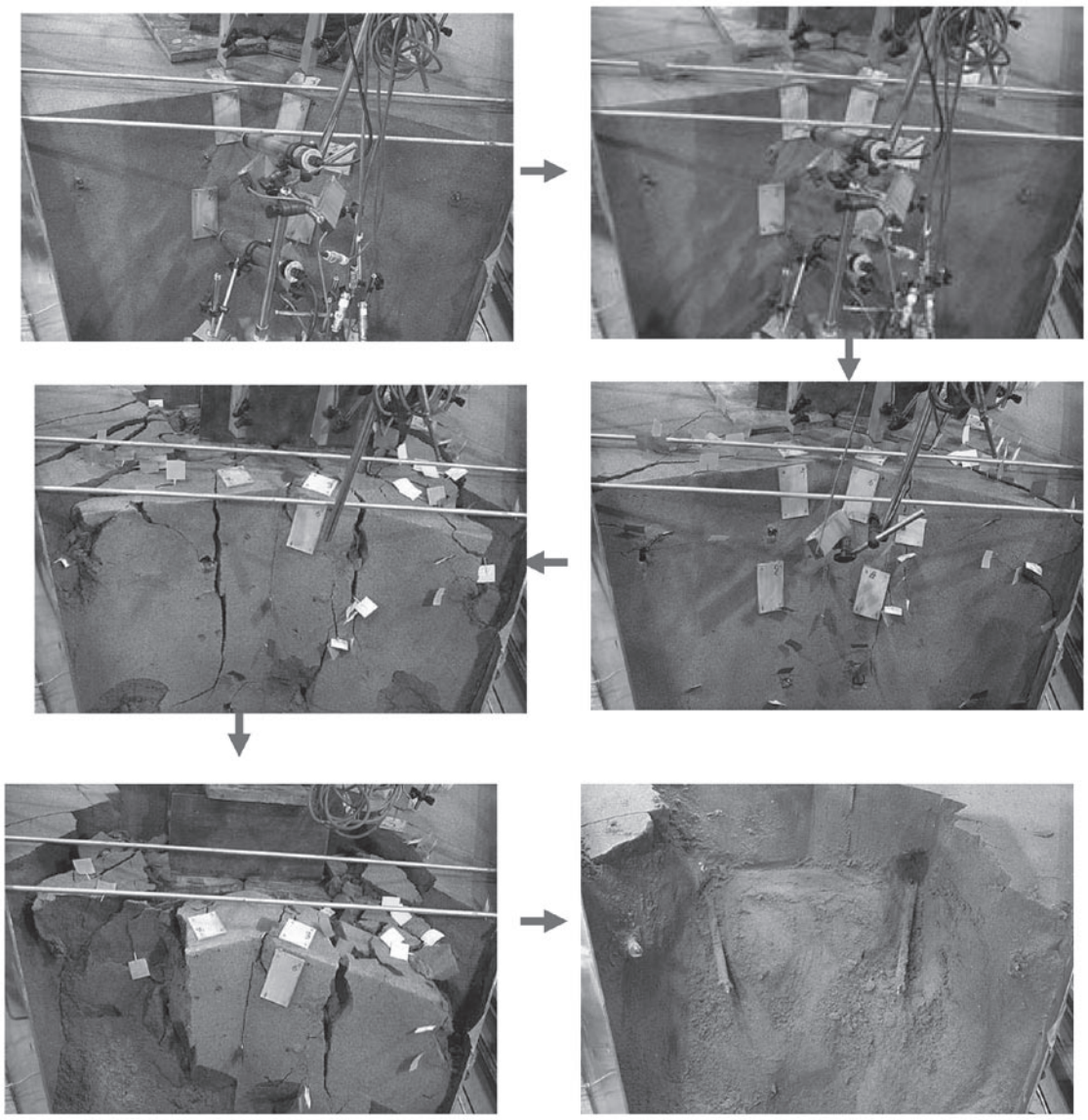

Figure 4. Failure sequence of the 3-D slope with soil nails. The last picture shows clearly the highly deformed nails A and B shown in Fig. 2.

RESULTS AND DISCUSSIONS

The main focus of this section is to report on the differences between the failures of

slopes with and without soil nails. The loading capacities of the 3-D slope increases 25% if soil-nailed is installed. When peak load is attained, the vertical deformation at the top of the slope for 3-D slope increases by 100% if soil nail is installed; and at ultimate failure, the vertical displacement increases by 9 times if soil nail is installed. The 3-D slope failed in a catastrophic fashion whereas the soil-nailed slopes never fail catastrophically. The 3-D slip surface passes around most of the soil nails and only two soil nails actually failed completely. The left soil Nail A bended both downward and outward from the centerline but there is no pullout failure, whereas the right soil Nail B bended mainly downward with some outward bending but there is pullout failure. The cement-grout was completely crushed at the bending points for both soil nails.

Figure 5 shows the highly deformed Nails A and B compared to one of the 6 other undamaged soil nails. As shown in the figure, the cement grout is crushed severely. This highly localized failure pattern probably suggests that we can propose a new arrangement of the soil nails such that the amount and location of soil nails can be optimized. This highly non-uniform failure pattern suggests that the present state-of-the-art method of theoretical prediction is quite primitive. Therefore, much more research and development is still needed to model such complicated nonlinear and non-uniform failure process.

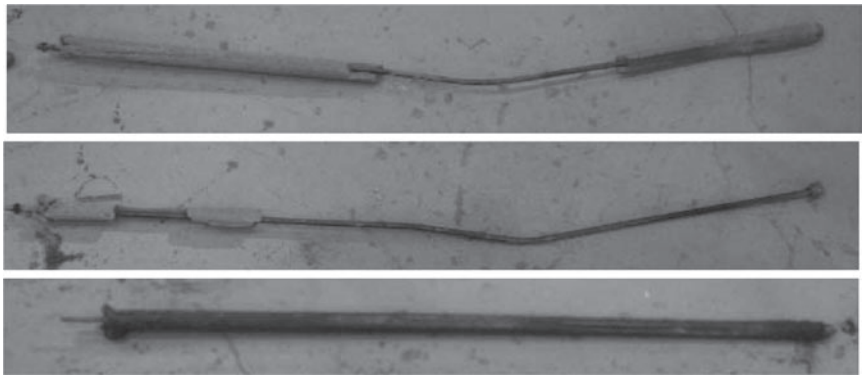

Figure 5. Highly damaged nails A and B comparing to one of the 6 other undamaged nails.

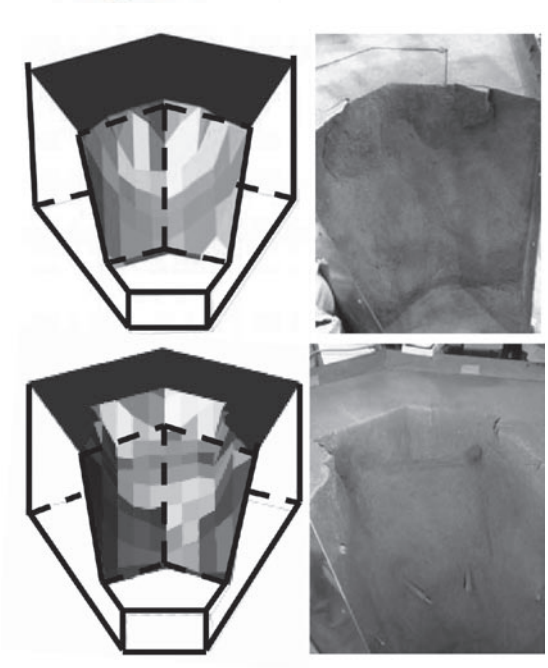

Figure 6. Three-dimensional plots of the failure surfaces for slope with (lower) and without (upper) soil nails.

To further illustrate the difference between the slip surfaces in slopes with and without soil nails, Fig. 6 shows the animated 3-D slip surfaces with the original photographs. It can be seen that the slip surface with soil nail is more deep-seated but is more localized in the middle, whereas the slip surface without soil nail is shallower but more widespread sideway. Clearly due to the presence of the soil nails, the slip surface changes and it evolves in a way that the resistance for failure will be a minimum. However, in the present case we force the upper slip surface passing through the nail by placing the loading platen closer to the edge of the slope surface, therefore soil nail failure becomes inevitable. Even with this procedure, only two soil nails actually contribute to the overall strength and stability of the soil slope.

To find the coefficient of sub-grade reaction of the soil, independent loading test has been conducted on the soil nail. The following formula can be derived for a horizontal head load applied at the top (Tong 2005):

$$K_L = \frac{6F(H_1 + H_2)}{y_1(3H_2L^2 - 2L^3)} \tag{1}$$

All parameters involved in the formula are given in Fig. 7. The derivation of eq. (1) should be referred to Tong (2005). The sub-grade reaction on the nails is about 68 ton/m^3.

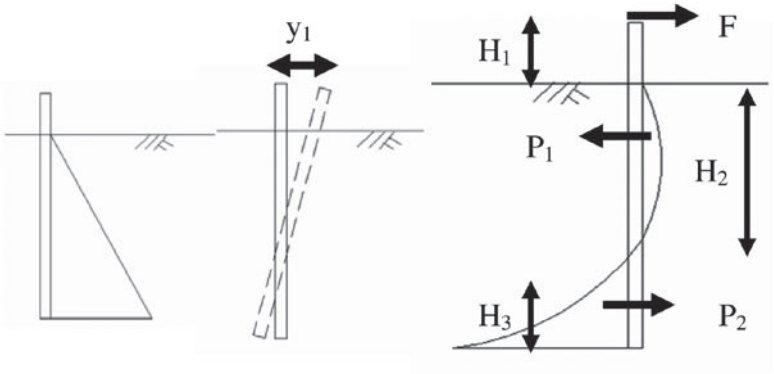

Figure 7. Parameters involved in calculating the sub-grade reaction on the soil nails

In order to ensure that the phenomena observed in our laboratory tests are comparable or similar to that observed in the field, dimensional analysis have been used to ensure mathematical similarity. Full details of the dimensional analysis are again referred to Tong (2005). To capture the main essence, the following equations have been assumed valid for both model and prototype:

$$EI\frac{d^4y}{dL^4} + K_s Dy = 0 \tag{2}$$

$$T_n = q_s \pi DL \tag{3}$$

$$\text{FOS} = \text{FOS}(c', \phi, q, \gamma, L, t, \sigma_{grout}) \tag{4}$$

where E -Young's modulus of the nail
I -second moment of inertia of nail
y -lateral displacement of the nail

L -length of the nail
k_s -coefficient of soil sub-grade reaction
D -diameter of the nail
T_n -normal tension force of the nail
q_s -bond strength between the soil and nail
c' - cohesion of the soil
q -dead load on the slope
ϕ -friction angle of the soil
γ -unit weight of the soil
t -time
σ_{grout} -strength of cement grout

Equation (2) is the governing equation for the response of soil nail, eq. (3) is the bond strength of soil nail, and finally eq. (4) is the factor of safety of the slope. We require that all these equations are simultaneously valid. Table 1 shows the chosen π factors and the corresponding targeted π factors.

Table 1. A summary of the π factors for the scale models and the corresponding prototype

Physical parameters	π factors	π-value	Values of parameters	
			prototype	model
Chosen π factors				
length of nail (m)	π_L	2.94	2.35	0.80
diameter of nail (mm)	π_D	5.00	100.00	20.00
pull out strength (kN/m^2)	π_{qs}	3.00	48.66	16.22
Required π factors				
sub-grade reaction (ton/ft^3)	π_{ks}	1.68	7.05	4.20
pullout strength (kN/m^2)	π_{qs}	3.00	48.66	16.22
pullout force (kN)	π_{Tn}	44.06	35.03	0.80
cohesion of soil (kPa)	π_c	2.94	9.05	3.08
height of slope (m)	π_h	2.94	2.35	0.80
strength of grout (MPa)	π_{grout}	2.94	54.93	18.70

Therefore, our model test resembles the behavior of a 2.35m slope with soil nail of 100 mm diameter, pullout strength of 49 kN/m^2, the sub-grade reaction of 7 ton/ft^3, (loose soil according to Poulos and Davis 1980), strength of grout of 55 MPa, and cohesion of soil is 9 kPa. All these parameters are within the probable ranges of real soil nails.

To study the 3-D effect, we also have conducted some numerical simulations on the loading capacity of the slope with and without soil nails using a numerical computer program called Slope 2000 (Version 1.7 Demo) developed by Dr. Y.M. Cheng of the Hong Kong Polytechnic University. The demo version is downloadable from YM Cheng's website. The computer program is 2-D, however our loading and slope are 3-D. As shown in Table 2, the peak load, that the slope can sustain, increases by 25% if soil nails are added. For the case of slope without nails, the error between the laboratory simulation and numerical result is 20%. When we do the 2-D numerical simulation, we have assumed the 3-D load being

spread along the length of the slope. Note that M1 In Table 2 corresponds to the first method of analysis using nails as shown in the upper row of Fig. 8 (assuming 2-D soil nail layers), whereas M2 corresponds to the equivalent surface load method as shown in the lower row of Fig. 8. We see that the M1 numerical simulation is comparable to the laboratory observation while the M2 (or equivalent load method) overestimates the loading capacity of the slope by 157%. Therefore, more detailed studies are required to verify the validity of the 2-D computer method for 3-D slope with soil nails. In addition, although the M1 result is very close to the observed value, the failure surface, however, differs significantly from the observed one. As shown in Fig. 6, our failure surface is in fact 3-D while the one for numerical simulation is 2-D. Therefore, we should not take the computed value as good prediction. In addition, clearly the numerical simulation cannot take into account the different failure modes observed for Nails A and B.

Table 2. Summary of results of loading capacity of slopes by computer program and from laboratory observations

Type of tests	Slope	Slope with soil nails	
Model Simulation (kN)	41	51	
Numerical Result (kN)	33	50 (M1)	131 (M2)
% error	20%	2%	−157%

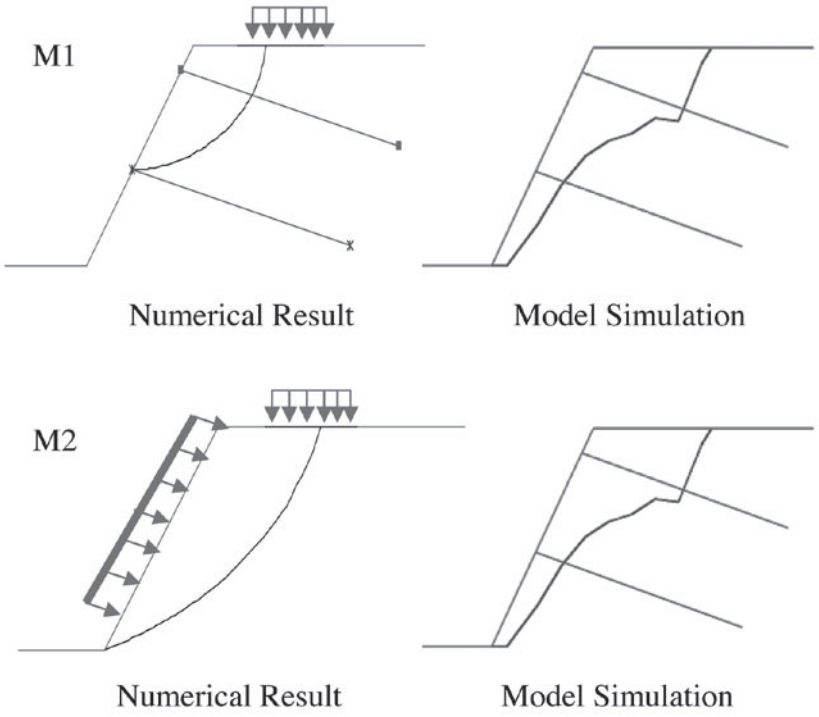

Figure 8. Comparison of failure surfaces by numerical simulation and laboratory tests

The results of the present simulation provide the first ever failure mechanism of soil nail in 3-D slopes, and provide invaluable information for verifying 3-D mathematical models and

for further experiments and field tests. Yet, there remain many issues to be addressed in future. For example, under what conditions will a soil nail fail under both bending and pullout, and under what condition will bending failure occur only. Under what situation will a 2-D analysis be good enough for 3-D slopes with soil nails. In future, various forms and shapes of 3-D slopes will be tested based on the present framework.

CONCLUSIONS

In this paper, we have summarized briefly some recent laboratory simulations of soil nail failures in 3-D slopes and its comparison with slopes without soil nails. This is the first ever well documented laboratory simulations for soil nail failure mechanism in 3-D slopes subject to 3-D loading. Full details of the test are referred to Tong (2005). Here, we have summarized the difference in failure mechanisms of three-dimensional (3-D) slopes with and without nails. The slope is 0.8 m high with a scale of 1:3 and a slope angle of 65°. A sandy soil with water content of about 5% was compacted to form a three-dimensional slop. The 3-D slope consists of two 2-D slopes intersecting at 135° (note that when the intersecting angle becomes 180° the 3-D slope becomes a 2-D slope), with a soil relative density of 85%. The cohesion is 3 kPa whereas the friction angle is 51°. They were obtained by conducting shear box tests on 3 undisturbed samples from our packed slopes. The applied normal stresses for the 3 samples are 5 kPa, 10 kPa and 15 kPa respectively. Although the friction angle is a bit high, the best fit Mohr-Coulomb failure model has a coefficient of correlation larger than 0.999 (i.e. it is very reliable). This high friction angle is probably resulted from compaction using mechanical vibrators in packing the soil.

For the case of soil-nailed slope, the soil nailed is formed in-situ by drilling a hole of 20 mm into the soil, and mild steel bars of 6 mm diameter and length of 800 mm were placed in the drilled hole and cement-sand grout (with a 3-day strength of 18.7 MPa) were poured into hole by gravity. A total of eight soil nails were installed on two different levels on the two intersecting faces (i.e. each face has 4 soil nails). The inclination of the soil nails is 20° with horizontal, pointing into the slope face. The pullout bond strength of each nail is 16 kPa, and the coefficient of sub-grade reaction of the soil on the nails is on average 68 ton/m^3. The slope is loaded to complete failure under a loading frame capable of applying 500 kN, and the location of the loading platen is selected such that soil nails are forced to fail. The loading capacities of the 3-D slope increases 25% if soil-nailed is installed. When peak load is attained, the vertical deformation at the top of the slope for 3-D slope increases by 100% if soil-nailed is installed; and when the slope fails, the vertical displacement increases by 9 times if soil nail is installed. The 3-D slope failed in a catastrophic fashion whereas the soil-nailed slopes never fail catastrophically. The 3-D slip surface passed around most of the soil nails and only two soil nails actually failed completely. The left soil nail bended both downward and outward from the centerline but there is no pullout failure, whereas the right soil nail bended mainly downward with some outside bending but there is pullout failure. Therefore, the failure mode is not symmetric and this is a strong and distinct indication of 3-D effect. All cement-grout were crushed at the bending points for both soil nails.

The results of the present experiment should stimulate further studies regarding the failure mechanism of soil nail in 3-D slopes, and also pose the question whether the 2-D approximation in designing 3-D slope is acceptable. Clearly, more studies need to be done.

ACKNOWLEDGEMENTS

The work described in the paper was fully supported by research funds from the Hong Kong Polytechnic University (ASD Project No. A226 and Infra-Faculty Project PE79 of the Faculty of Construction and Land Use). It constitutes part of the Final Year Project of the first author (KW Tong). This project was initiated after a visit to GEO with Ir. Raymond Chan and

his colleagues in 2004. Dr. Y.M. Cheng also advised on how to run his program. We are grateful to the constructive editorial comments made by Ir. Ringo Yu and Ir. S.H. Mak on the first draft.

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EXPERIENCE OF USING TIME DOMAIN REFLECTOMETRY (TDR) TECHNIQUE IN MAINLAND CHINA

K.L. Lo[1], K.F. Wong[2] and Joseph Y.C. Lo[3]

Abstract: The technique of Time Domain Reflectometry (TDR) was used to measure the lengths of some installed soil nails at the slopes adjacent to a high voltage electricity transmission line in Shenzhen. The measured results were analysed using simple statistical methods, and are presented in this paper. The results suggest that the TDR method offers an accuracy of ±1m, while the errors between the measurements and the true lengths of the nails can be modelled by a normal distribution curve. Merits and limitations of the use of TDR technique to determine the lengths of installed nails are also discussed.

BACKGROUND

A project was undertaken by Maunsell Geotechnical Services Ltd. to upgrade the slopes adjacent to the pylons of a high voltage electricity transmission line in Shenzhen. Most of the slopes that required upgrading were man-made cut slopes or disturbed natural terrain with a height ranging from 10m to 20m. Soil nails were used as a major part of the slope upgrading measures. Instability of these slopes would affect normal supply of electricity, and therefore the stability of the upgraded slopes and the quality of the completed works are the primary concerns of the client company. In addition to provision of full-time supervision of the works, the client also requested verification of the workmanship to enhance the degree of confidence in the quality of the completed works. In view of this, a large number of soil nails were tested after construction. Considering its reliability and ease of operation, Time Domain Reflectometry (TDR) technology was used to measure the lengths of the installed soil nails.

PRINCIPLE OF TDR TECHNIQUE

The technique of TDR was originally developed to identify faults along transmission cables in the 1950's which works on the same principle as radar. A pulse of energy in the form of an electrical signal is emitted from one end of a cable. When the pulse reaches the other end of the cable, or a fault along the cable, all or part of the pulse energy is reflected back to the receiver. The TDR equipment measures very accurately the time it takes for the signal to travel down and reflect back along the cable. The Velocity of Pulse (VOP) is unique for cables made from the same material, but varies from one material to another. If the VOP is known, the travel distance of the energy pulse can be determined. The insulating material separating a conductor is called a dielectric. When two conductors are placed together, they form a transmission line which has a characteristic impedance. The impedance of a cable is determined by the conductor diameter, spacing of the conductors and the type of dielectric or insulation used. A soil nail made of steel bar with a wire in parallel and surrounded by grout is analogue to a cable with a characteristic impedance. When an energy pulse travels along a soil nail, the pulse will be reflected when it encounters a change in impedance which could be due to presence of a coupler, a change in the size of grout, a change in the humidity or faults in the soil nail.

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Previous Experience in Using TDR for Measuring Lengths of Soil Nails

Geotechnical Engineering Office (GEO) of the HKSAR Government published the results of a study which aimed at investigating non-destructive methods for measuring the lengths of installed soil nails (Cheung 2003). It concluded that the TDR and the Electro-magnetic Induction methods were both reliable and sufficiently accurate for practical purposes. The TDR method has the merit of lower cost and a reasonable accuracy of about ±1m. The TDR technique has been adopted by the Government and local consultants as a quality control measure for soil nailing works.

Set-up and Instrumentation

Since some of the nails were planned to be tested by the TDR method, electric wires were installed along all steel soil nails during construction. The TDR equipment (model T631) manufactured by Bicotest Limited in UK was employed. The specifications of the TDR equipment are listed in Table 1. The software X600 TRACEabiliy developed by Bicotest was used for manipulation of the waveforms obtained from the measurements, which can be used to determine the traveling time of the reflected energy pulse.

Table 1. Specifications of TDR equipment

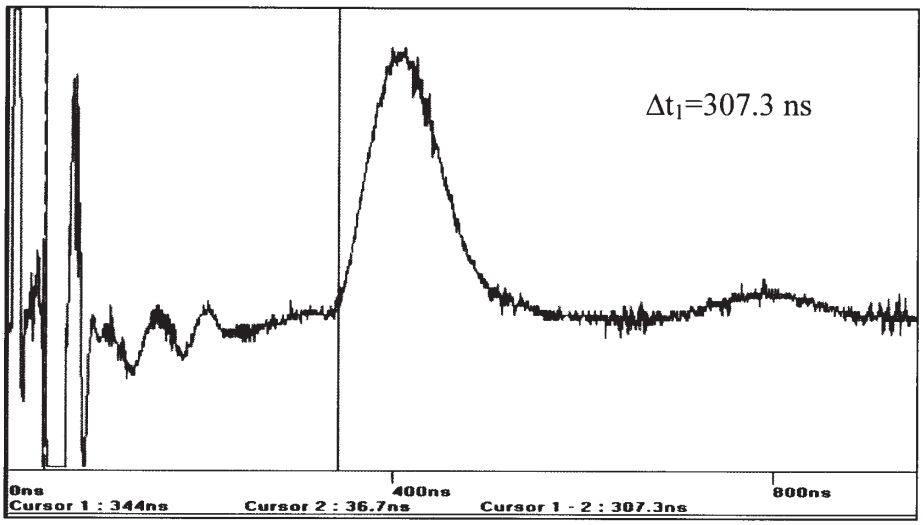
Accuracy	Typically <0.5% 48m range and above
Dielectric	PVF 0.300 to 0.999 V 90 – 300m/μs V 45 – 150m/μs
Maximum Displayed Range	3 – 12800 m
Cursors	2
Pulse Characteristics: *Waveform* *Amplitude* *Pulse Widths* *Source Impedance*	 • Sine Squared • Nominally 2.5V into selected impedance • 2, 10, 30, 100, 1200 ns half height nominal • 50, 75 or 93 Ω

Measurement Procedure

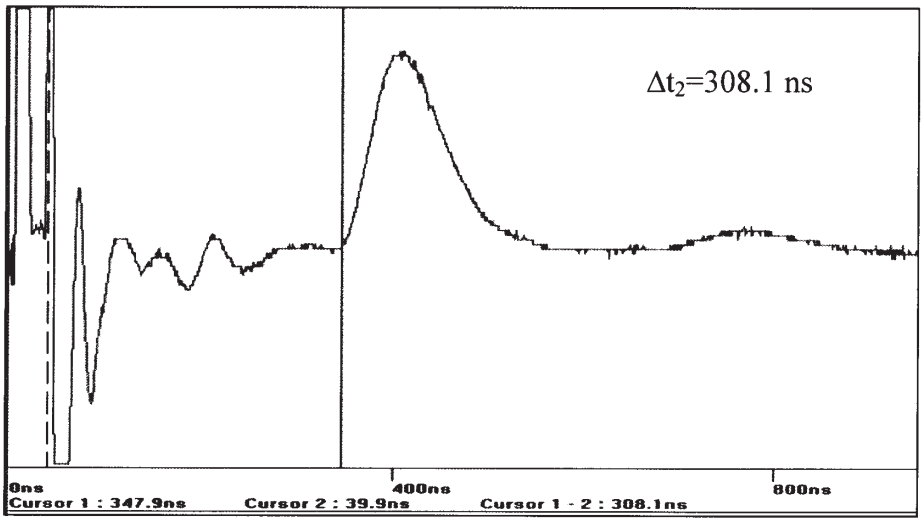
About 2000 soil nails were installed at 21 sites along the electricity transmission line. A total number of 65 soil nails, equivalent to 3% of the installed soil nails at each site, were selected for TDR testing. The tested soil nails had design lengths ranging from 6m to 17m. At each site, where TDR testing was carried out, the longest soil nail with a known length was used as a calibration nail to determine the VOP of the soil nails installed at that site. For each of the tested soil nails, measurements were taken using 3 different pulse widths of 2ns, 10ns and 30ns. During measurement, the TDR equipment detected the energy pulse reflected from the end of the installed soil nail and determined the time of travel, t, for a distance equivalent to twice the length of the soil nail under examination. The length of measured soil nail is determined from the formula below:

$$L = \frac{VOP}{2}\left(\frac{t_1 + t_2 + t_3}{3}\right) \quad (1)$$

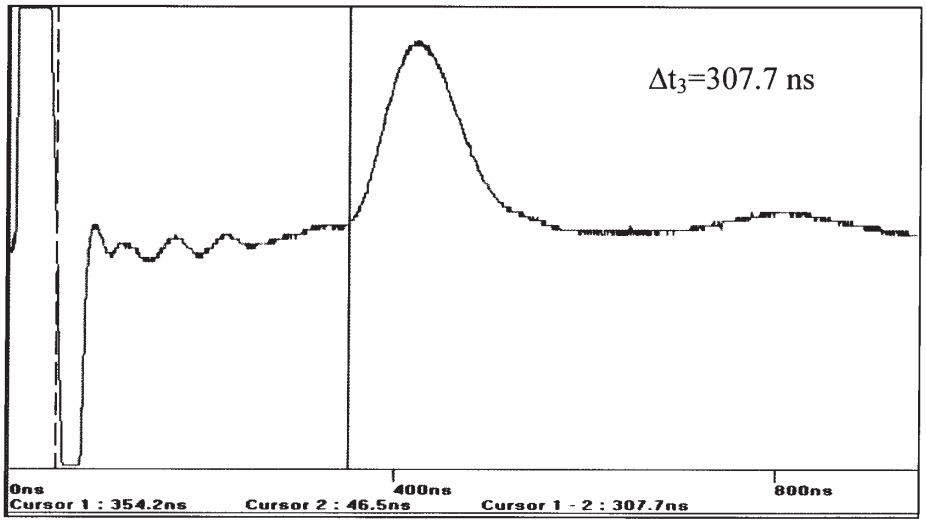
where L is the measured length of the soil nail;
VOP is the propagation velocity of the energy pulse in soil nail, and
t_1, t_2, and t_3 are the traveling time of the energy pulse with a pulse width of 2ns, 10ns and 30ns respectively, for a distance of 2L.



(a) Pulse width = 2 ns



(b) Pulse width = 10 ns



(c) Pulse width = 30 ns

Figure 1. Typical waveforms obtained from soil nail length measurement using TDR method

RESULTS AND DISCUSSION

The statistics on the differences between the measured and the as-built lengths are summarised in Table 2. It has been assumed that the as-built lengths of the installed nails are identical to the design lengths. This assumption is justified by the fact that the construction work was closely monitored and properly supervised on site. With this assumption, the accuracy of the TDR method can be examined. The results of all 65 tests are tabulated in Appendix 1.

Table 2. Statistics on the difference between the lengths measured by TDR method and the as-built lengths of installed soil nails.

As-built length of soil nail (m)	Number of nails tested	Maximum difference from the as-built length (m)	Minimum difference from the as-built length (m)	Mean difference from the as-built length (m)	Mean % difference between the measured and the as-built length
6	1	-0.04	-0.04	-0.04	-0.6%
8	2	0.05	0.03	0.04	0.48%
10	1	-0.66	-0.66	-0.66	-6.64%
12	17	0.40	-0.39	0.00	0.02%
13	7	0.00	-0.47	-0.17	-1.31%
14	14	0.92	-0.73	0.12	0.84%
15	12	0.50	-0.58	0.07	0.43%
16	8	0.42	-0.02	0.20	1.22%
17	3	0.88	-0.58	-0.09	-0.53%
Overall	65	0.92 m	-0.73 m	0.03 m	0.14%

Based on the results of the 65 tested soil nails, it can be concluded that the accuracy of the TDR method in this study is within $\pm$1m. This is in good agreement with previous reported values (Cheung 2003). The discrepancy could be attributed to the difference in the VOP between the calibration nail and the tested nails. Factors affecting VOP include moisture, temperature, and the diameter of grout. The influence of these factors is critical to the overall accuracy of the results obtained by the TDR method. In order to investigate the reliability of the TDR technique for soil nail length measurement, the data was further analysed using statistical methods. Firstly, the difference between the measured and as-built lengths of each soil nail was determined. Secondly, the mean value, μ, and the standard deviation, σ, of all the measurements are calculated using the following formulae.

$$\mu = \frac{\sum (L - L_{ab})}{N} = 0.0285m \tag{2}$$

$$\sigma = \sqrt{\frac{\sum (L - \mu)^2}{N - 1}} = 0.3422m \tag{3}$$

where μ is the mean value of the differences between the measured and as-built lengths of the 65 tested nails;

σ is the standard deviation of the differences between the measured and as-built lengths of the 65 tested nails;
L is the measured length of soil nail, and
L_{ab} is the as-built length of soil nail.

For this sample data of 65 soil nails measured by the same TDR apparatus, the arithmetic mean and standard deviation of error were 0.0285m and 0.3422m respectively. With these values available, a theoretical normal distribution curve for the sample data can be plotted using Eq. (4) as shown in the Figure 2. The theoretical cumulative normal distribution curve is also plotted and compared with the actual measurements in Figure 3. It can be seen from Figure 2 that the distribution of the discrepancies between the as-built and measured lengths of the tested nails can be modelled by a normal distribution. In theory, about 68% of the sample measurements are within one standard deviation of the mean value and about 95% of the measurements are bounded within two standard deviations of the mean value.

$$f(x) = \frac{1}{\sqrt{2\pi\sigma^2}} e^{-(x-\mu)^2/2\sigma^2} \tag{4}$$

where $f(x)$ is the normal distribution density function of value x

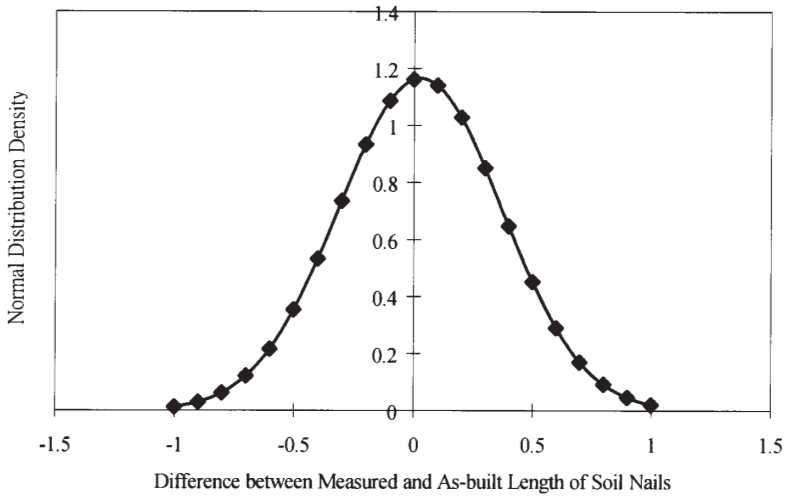


Figure 2. Theoretical normal distribution curve based on the calculated mean value and the standard deviation obtained from the measurements

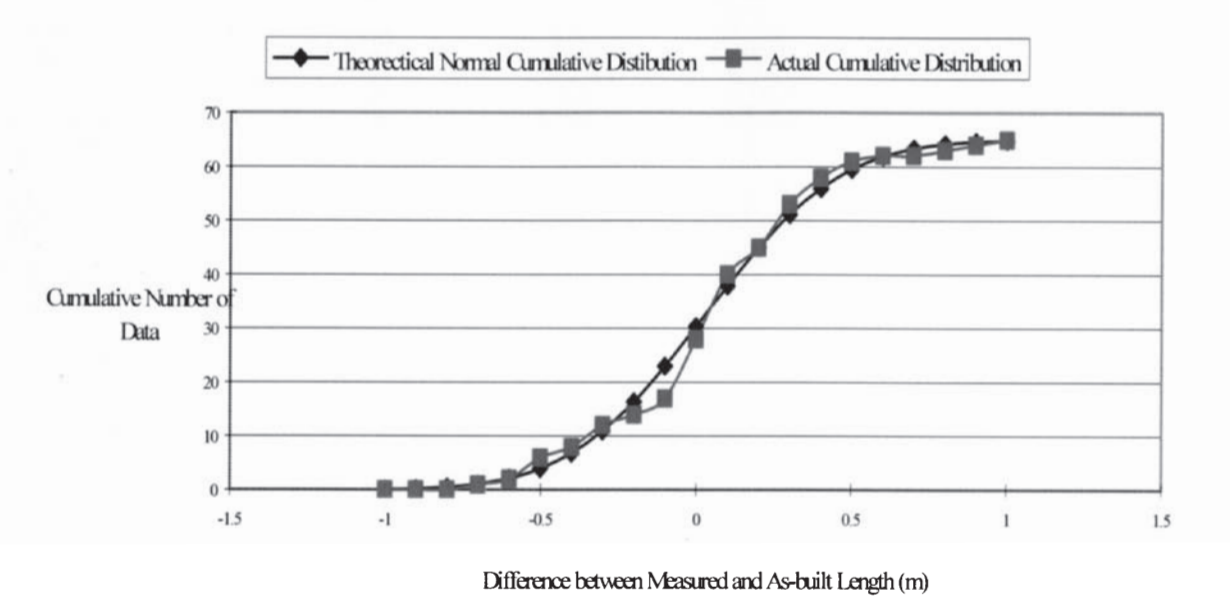


Figure 3. Comparison between the theoretical cumulative normal distribution curve and the actual measurements

MERITS AND LIMITATIONS OF USING TDR

Based on the experience gained from this study, it is suggested that the use of the TDR technique to determine the lengths of installed soil nails has the following advantages:

- Fast check for the lengths of soil nails with acceptable accuracy and reliable results;
- As complement to site supervision for quality control;
- Inexpensive to conduct the test;
- Site supervisory staff can be trained to operate the TDR machine;
- Measurements can be made at any time during or after construction, and
- Only small size machines are involved, suitable for remote sites.

On the other hand, the use of TDR technique has the following limitations:

- The machine only shows the traveling time of the pulse instead of the length directly. Some simple calculations are required to determine the length;
- Calibration is required for each combination of drillhole size and bar diameter, a soil nail with a known length is therefore required at each site;
- Difficult to identify the imperfection of grout quality, lack of research to investigate the response of reflected pulse to grout imperfections;
- Skills in the interpretation of the trace curve for determination of pulse traveling time is required. Large errors could be resulted due to incorrect interpretation;
- The location of the reflected pulse could be confused by other pulses due to imperfections or couplers. However, such disturbances would reduce when a shorter pulse width is used.

Some precautions for constructing soil nails to be tested with TDR technique are recommended to potential contractors. It is of great importance to make sure that the coating of the electric wire is not damaged during transportation and installation of soil nails. In the case of the TDR technique being carried out in nails with a nail head, the electric wire shall be extended to the head of steel bar. For the same batch of soil nails to be tested, the same type of electric wires must be used for all nails and the electric wire must be kept as straight as

possible along the soil nail. A computer with sufficient memory for data storage is recommended when a large number of soil nails are to be tested.

CONCLUSIONS

The TDR technique was used to determine the lengths of 65 installed soil nails at 21 sites along a high voltage electricity transmission line in Shenzhen. The accuracy of this technique was found to be within $\pm$1m. Moreover, it was found that distribution of errors between the measured and the true values can be closely modeled by a normal distribution. It was suggested that the accuracy of TDR was acceptable and reliable as a complement to quality site supervision of soil nailing works. However, precautions for proper installation of wires for TDR testing should be implemented during construction. Only trained personnel can undertake the TDR tests and interpret the results obtained.

REFERENCES

Cheung W.M. (2003). *Non-destructive tests for determining the lengths of installed steel soil nails*. GEO Report No.133. Geotechnical Engineering Office, HKSAR Government.

ACKNOWLEDGEMENTS

The authors would like to thank Dr C.Y. Cheuk for his comments during preparation of this paper.

APPENDIX 1. RESULTS OF TDR MEASUREMENTS FOR 65 SOIL NAILS

Nail no.	As-built length of installed soil nail (m)	Measured length of installed soil nail (m)	VOP (m/ns)	*t_1 (ns)	*t_2 (ns)	*t_3 (n2)
1	6	6.0	0.08823	133.6	141.5	130.5
2	8	8.1	0.09487	166.6	171.3	171.3
3	8	8.0	0.09487	169.7	172.9	165.0
4	10	9.3	0.08438	216.7	222.4	224.8
5	12	12.0	0.08596	278.2	278.2	278.9
6	12	11.8	0.08596	276.6	267.2	276.6
7	12	12.2	0.08596	284.5	282.9	284.5
8	12	12.0	0.08270	290.8	290.8	291.6
9	12	12.1	0.08270	292.3	293.9	289.2
10	12	12.0	0.09030	264.0	265.6	265.6
11	12	11.7	0.08677	270.3	270.3	267.2

Nail no.	As-built length of installed soil nail (m)	Measured length of installed soil nail (m)	VOP (m/ns)	*t_1 (ns)	*t_2 (ns)	*t_3 (n2)
12	12	12.3	0.07897	313.4	318.4	306.3
13	12	11.7	0.07897	294.2	299.2	292.2
14	12	12.4	0.07897	314.4	317.4	310.3
15	12	12.3	0.08696	281.1	287.2	280.1
16	12	11.9	0.08515	277.1	282.1	280.1
17	12	11.6	0.08515	272.0	272.0	274.0
18	12	11.9	0.08816	270.0	271.0	272.0
19	12	12.2	0.08816	274.1	281.1	278.1
20	12	12.0	0.08440	284.1	284.1	286.1
21	12	11.8	0.08440	280.1	282.1	280.1
22	13	12.8	0.08816	284.1	293.2	292.2
23	13	12.5	0.08615	284.1	292.2	296.2
24	13	13.0	0.07833	331.6	331.6	330.1
25	13	13.0	0.07833	331.6	334.8	328.5
26	13	12.7	0.08337	301.8	303.3	305.5
27	13	13.0	0.08337	309.0	311.9	314.9
28	13	12.9	0.08337	306.2	309.3	312.6
29	14	14.1	0.08677	322.2	325.3	325.3
30	14	14.1	0.08677	323.8	328.5	325.3
31	14	14.0	0.08457	331.6	330.1	328.5
32	14	13.9	0.08457	322.2	330.1	331.6
33	14	14.2	0.08457	334.8	337.9	336.3
34	14	14.0	0.08457	330.1	334.8	330.1
35	14	14.2	0.08696	326.4	327.5	324.4
36	14	14.4	0.08615	334.5	338.5	330.5
37	14	14.9	0.08412	350.5	352.7	360.7
38	14	14.7	0.08412	344.6	350.7	356.4
39	14	13.9	0.08412	327.0	329.4	336.3
40	14	14.0	0.08412	331.2	332.6	335.0
41	14	13.3	0.08412	312.9	315.2	318.7
42	14	13.9	0.08440	322.4	332.5	334.5
43	15	15.2	0.08823	348.9	341.1	341.1
44	15	15.1	0.08823	344.2	344.2	341.1

Nail no.	As-built length of installed soil nail (m)	Measured length of installed soil nail (m)	VOP (m/ns)	*t_1 (ns)	*t_2 (ns)	*t_3 (n2)
45	15	15.2	0.08823	344.2	344.2	345.8
46	15	15.2	0.08438	355.8	361.3	364.4
47	15	14.5	0.08438	337.7	345.0	348.1
48	15	15.5	0.08438	361.5	367.4	372.1
49	15	14.4	0.08438	342.0	342.6	340.8
50	15	15.0	0.08438	355.4	352.5	360.5
51	15	15.5	0.08252	373.7	376.4	377.2
52	15	15.2	0.08252	365.6	370.3	372.7
53	15	14.5	0.08252	352.2	351.8	353.6
54	15	15.3	0.08252	369.1	372.3	374.1
55	16	16.0	0.08507	372.5	381.9	377.2
56	16	16.2	0.08507	377.2	381.9	381.9
57	16	16.4	0.08507	386.6	386.6	381.9
58	16	16.3	0.08615	376.8	380.9	374.8
59	16	16.0	0.08161	391.3	392.9	392.9
60	16	16.4	0.08161	397.6	403.9	405.5
61	16	16.3	0.08337	389.4	390.0	393.7
62	16	16.0	0.08337	382.9	383.1	383.7
63	17	16.4	0.07866	416.5	419.6	416.5
64	17	16.4	0.07866	418.1	418.9	416.5
65	17	17.9	0.07866	447.2	459.7	456.6

* - t_1, t_2, t_3 denote the traveling time of the energy pulse with a pulse width of 2ns, 10ns and 30ns respectively, for a distance of twice the length of soil nail measured.

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