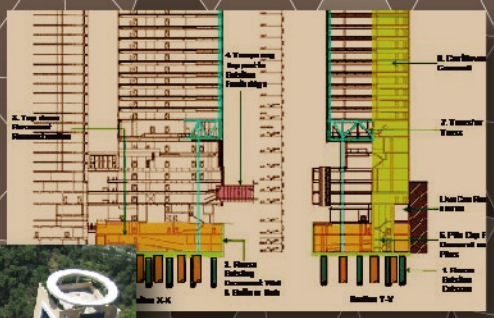
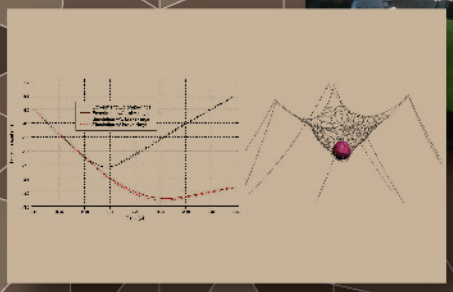
THE HKIE GEOTECHNICAL DIVISION 33RD ANNUAL SEMINAR, 2013

GEOTECHNICAL ASPECTS OF HOUSING SUPPLY AND DEVELOPMENT





31 MAY 2013
HONG KONG

Jointly organised by

THE HONG KONG INSTITUTION OF ENGINEERS 香港工程師學會
Geotechnical Division 岩土分部

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Since 2003

Proceedings of the 33rd Annual Seminar
Geotechnical Division, The Hong Kong Institution of Engineers

Geotechnical Aspects of Housing Supply and Development

31 May 2013
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Jointly organized by:
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Hong Kong Geotechnical Society

Captions of Figures on the Front Cover

Top-left: Controlled Surface Blasting Works for Land Formation for Development at Anderson Road (By courtesy of Ove Arup & Partners Hong Kong Ltd)
Bottom-left: Advanced 3D Modelling for Interactions of Landslide Debris and Flexible Barriers (By courtesy of Ove Arup & Partners Hong Kong Ltd)
Central: Reconstruction of a Fill Slope at a Congested Site in Kwai Chung (By courtesy of the Housing Department)
Top-right: Re-use of Existing Basement and Foundation in Redevelopment Projects in Urban Districts (By courtesy of AECOM)
Bottom-right: Excavation for Construction of an Adit Connecting a Housing Development to MTR Station (By courtesy of the Housing Department)

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FOREWORD

This proceeding contains the paper for the 33rd Annual Seminar organised by the Geotechnical Division of The Hong Kong Institution of Engineers. Over the years, the Geotechnical Division has organized seminars annually on variety of engineering subjects. These annual seminars aim to provide a platform for local practitioners and researchers to consolidate their engineering experience and geotechnical expertise on various topics.

This year, the Geotechnical Division has chosen the subject of "Geotechnical Aspects of Housing Supply and Development" as the theme of the 33rd Annual Seminar. Hong Kong's steep terrain poses severe constraint to availability of land and challenge to housing supply for meeting the demand from its increasing population and to fuel its economic growth. Over the past decades, the geotechnical profession has made significant engineering contribution to facilitate the housing development in Hong Kong. The papers included in this proceeding present a lot of valuable geotechnical engineering experience and insight gained in overcoming challenges in housing supply and development in areas of natural terrain hazards, slope upgrading works, excavation and lateral support, foundation, land reclamation as well as slope maintenance.

On behalf of the Geotechnical Division, I would like to thank the Hong Kong Geotechnical Society for jointly organizing this seminar. In particular, I am grateful to our Guest-of Honour, Prof Anthony B L Cheung, GBS, Secretary for Transport and Housing of the Government of the HKSAR; the Invited Speakers - Mr K K Ling, Dr Sam Ng, Ir K W Leung and Ir Calvin Lam; the speakers and the authors of the papers for their contribution to this seminar. Lastly, I must thank the Organising Committee, under the leadership of Ir Rupert Leung, for their hard work in making this seminar possible.

Ir Tony Cheung
Chairman, Geotechnical Division
Hong Kong Institution of Engineers (2012/2013 Session)

May 2013

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Learning from Successful Cases: Workable Recipes for Underground Development

K.K. Ling and Margaret H.Y. Chan
Planning Department, Government of the Hong Kong SAR

ABSTRACT

In land-strapped Hong Kong, land is our strategic resources to sustain our economy and society. By conceiving "land" as "space" with multiple dimensions (at-grade, above-ground, underground and inter-spatial), the scope of our developable space could be much greater than what is currently known to us. Compared to surface development, the use of our underground space is still very limited. How could we better harness the hidden potential of our underground space?

By making reference to some notable cases, both overseas and local, the workable recipes for underground development are to be unveiled. The key ingredients could be broadly categorised into three categories: (1) planning ingredients (e.g. innovative mindset, appropriate siting, complementary inter-spatial relationship, and good coverage, connectivity and mix of uses); (2) design ingredients (e.g. careful design of access points and easily navigable underground channels); and (3) implementation ingredients (e.g. good management, enabling government provisions, private initiatives and community support). Nonetheless, having the right ingredients does not suffice. We also need to have the right recipes to create a signature dish from the ingredients. The acronym of such workable recipes is "2S2C", viz. Sustainability, Strategic, Coordination and Collaboration, which will be detailed in the paper.

1 INTRODUCTION

In land-strapped Hong Kong, land is our strategic resources to sustain our economy and society. So, how much land do we have? A simple answer is that Hong Kong has a total land area of about 1104 km^2, which represents a two-dimensional measurement of the land mass in Hong Kong. Yet, the answer would be quite different if we conceive "land" as "space" with multiple dimensions, viz. at-grade, above-ground, underground and inter-spatial. Theoretically, the amount of developable space could virtually proliferate if the potential of different levels of space could be better harnessed.

Hong Kong is a renowned vertical city. The city is characterised by compact development at the surface and upper levels, but development at the subterranean level is rather limited. This suggests that one probable solution to alleviate the land shortage issue is to better utilise the underground space. So, why not?

With reference to both overseas and local cases, this Paper attempts to unveil the workable recipes for underground development with a view to shedding light on the pursuit of underground development in Hong Kong.

2 IS HONG KONG RIPE FOR UNDERGROUND DEVELOPMENT?

2.1 Right time

In recent years, our Government has put in place an explicit policy support for underground development in Hong Kong. The 2009 Policy Address put forward an initiative to launch strategic planning and technical studies to facilitate planned development of underground space aiming at promoting the enhanced use of rock caverns as part of the pursuit of sustainability development. It is further highlighted in paragraph 82 of the 2013 Policy Address that, "reclamation aside, rock caverns and underground space are viable sources of land supply. We are studying the feasibility of relocating the Sha Tin Sewage Treatment Works to a cavern to make way for housing or other uses. We are also conducting a study on the long-term strategy of rock cavern development with a view to preparing rock cavern master plans and formulating policy guidelines. Moreover, we will further explore the

potential of developing underground space in the urban areas. This includes studying the possibility of linking up the underground space of existing or planned structures in the urban areas."

From a policy perspective, it seems to be the right time for underground development in Hong Kong. The scope is no longer confined to rock cavern development, but also underground development in the urban areas.

2.2 Right place

While the hilly topography of Hong Kong is constraining our surface development, the underlying geology is actually offering a good opportunity for cavern development. According to the "Enhanced Use of Underground Space in Hong Kong: Feasibility Study" commissioned by the Civil Engineering and Development Department (CEDD) with the support of the Planning Department (PlanD), about 64% of the land area in Hong Kong has high to medium suitability for cavern development. This land area is largely underlain by strong granitic and volcanic rocks, and some 55% of it is located within statutory protected areas such as country parks whereby development underneath would be subject to approval by relevant authorities.

In terms of geological suitability, Hong Kong is the right place to pursue development at caverns, including the underground space.

2.3 What else is needed?

Being at the right time and right place to pursue underground development, what else is missing in the Hong Kong context?

Engineers and planners alike have long been studying the subject. A brief account is as follows:

- The Study of the Potential Use of Underground Space (SPUN) (1988): confirmed the viability of underground development as an alternative to conventional development above-ground and one which could offer significant environmental benefits;

- The Metroplan Study (1991): investigated potentially viable uses for large man-made underground space in rock caverns and advocated pursuing the opportunity of underground development as a response to the shortage of land in Hong Kong;

- Cavern Project Studies (CAPRO) (1991): ground investigations of two underground development schemes, viz. Refuse Transfer Station in Mount Davis and Government Supplies Department Warehouse in Chai Wan;

- Preliminary Engineering Geology Studies (PEGS) (1990 & 1993): assessed the potential for relocating various existing and planned facilities in rock caverns;

- Cavern Area Studies (CAS) (1992 & 1998): classified the suitability of land for rock cavern development based largely on general engineering geological information;

- Enhanced Use of Underground Space in Hong Kong - Feasibility Study (2011): concluded that Hong Kong, with its natural setting and strong rocks particularly suitable for rock cavern development, could create an environment in which cavern development forms part of the mainstream development process;

- Increasing Land Supply by Reclamation and Rock Cavern Development cum Public Engagement - Feasibility Study (commissioned in 2011, ongoing): aimed to explore the feasibility of increasing land supply by reclamation and rock cavern development; and

- Long-term Strategy for Cavern Development (commissioned in 2012, ongoing): aimed to formulate policy guidelines to facilitate cavern development for both the public and private sectors, reserve strategic cavern areas for cavern development and select suitable government facilities for systematic relocation to caverns.

As early as the 1980s, the viability of underground development has been established. In the 1990s, the Metroplan Study has advocated the underground opportunity as a planning strategy for Hong Kong, but the study

focus then was mainly on cavern developments on a project-by-project basis. Utilisation of the caverns was mainly to house purpose-built government facilities such as the Island West Refuse Transfer Station, Stanley Sewage Treatment Works and Kau Shat Wan Explosives Depot. The 2010s could be seen as a turning point whereby cavern development has been increasingly conceptualised as part of the planning strategy in pursuit of sustainable development. More recently, greater attention is also given to exploring underground development in the urban areas. All these suggest that underground and cavern development has been transcending from a purely engineering pursuit to a planning strategy for sustainable development, encompassing not only public projects but also the wider urban areas which would also involve the collaborative initiatives of the private sector and the community. This growing awareness of the need for a holistic underground planning strategy is gathering momentum in Hong Kong.

3 INGREDIENTS AND RECIPES TO SUCCESS

3.1 Literature review

The use of underground space is not new. Mankind has used it for defensive purposes as well as for mining, transportation (subways, tunnels, underground railways, etc) and other fields such as the development of hydroelectric facilities. To fully appreciate the merits of going underground, perhaps, we should first understand the reasons for doing so in light of literature review. A summary of the common push and pull factors for underground development are as follows:

- The push factors:
  - lack of alternative surface space;
  - high real estate prices for surface development; and
  - community objection to the physical presence of certain "not-in-my-backyard" (NIMBY) surface facilities (e.g. storage of less desirable materials, refuse transfer stations, etc).
- The pull factors:
  - the opacity and containment of underground space could protect the surface from the nuisances and dangers associated with the underground facilities;
  - resilient from the risks and disturbances on the surface environment (e.g. severe weather such as typhoons and extreme temperature or structural damage due to floodwaters);
  - allowing grade separation of various transportation corridors which helps promote traffic safety and ease traffic congestion;
  - allowing the siting of facilities near the existing ones, thereby offering better services to the surrounding community;
  - placing facilities underground could release the surface space for other beneficial use; and
  - environmental preservation (e.g. less visual impact and less damage to natural vegetation as compared to a surface structure).

3.2 Learning from empirical cases

The use of underground space has been intensifying with the world trend of urbanisation. There are successful examples of underground development around the world. Taking a closer look at some selected cases is one of the best ways to obtain some clues on the workable recipes for underground development.

(a) Seasoned cases with longer history of underground development

(i) Helsinki , Finland

Since the 1960s, the City of Helsinki has been adept at widely utilising the opportunities for underground construction. Underground resources play a central role in the development of the city structure. On average, under each 100 m^2 of surface area, there is 1 m^2 of underground space. It is envisaged that there is still much room for developing the underground resources for future needs.

The popularity in the use of underground space and the need to coordinate different projects has led to the preparation of the Helsinki Underground Master Plan (UMP), which is often cited as a masterpiece of a planning-led and coordinated comprehensive underground master plan at a city-wide level. The drafting of the underground city plan in Helsinki can be traced back to the 1950s in association with the development of Helsinki Metro. A series of studies have been conducted, and a space reservation system was implemented in 1984 to coordinate the underground plans of various municipal authorities and private underground construction projects. The UMP was introduced in 2009 to reserve underground space for long-term development. Instead of merely showing the current use of underground space, it reserves underground space for the long-term development of public and private facilities.

The UMP shows the location of more than 400 existing underground spaces as well as the reserve areas of more than 200 underground spaces with potential for future development. The existing and planned underground facilities are shown in grey and blue respectively, and orange for possible future uses. Taking into consideration factors such as location, space allocation, accessibility, present and planned ground-level uses, traffic connections, land ownership, and possible recreational, landscape and environmental protection values, the future development areas are grouped into five categories: (1) community technical systems; (2) traffic and parking; (3) maintenance and storage; (4) services and administration; and (5) unnamed rock resources to connect present use, planned use and possible future use.

The UMP is administered by the Helsinki City Planning Department. It is a legally binding plan and enjoys the same status as the Helsinki Master Plan. To prevent surface developments from sterilising potential underground space, the Building Code in Finland usually restricts the depth that a landowner can utilise to one or two basement storeys. Moreover, regulations for underground development in the city require that the top of a cavern/underground development to be at least 6 m below ground to avoid direct conflict with the landowners' basement development.

Key ingredients and recipes are as follows:

- Underground master planning is regarded as an integral and significant part of the land use planning process;
- A strategic and long-term vision for the use of underground resources;
- A unified mechanism, including planning, development and legal provisions, to coordinate both private and public initiatives to underground uses;
- A supporting community for underground use;
- Many cavern facilities are partly funded by the National Civil Defense Department that provides incentives for facilities to be developed as dual use facilities (i.e. as defense shelters and other purposes during normal times); and
- The legal status of the UMP reinforces the systematic nature and quality of underground construction and the exchange of information related to it.

(ii) Montreal, Canada

The Montreal case is a good example of a concerted approach to building an underground city through a set of city-enabled, privately-developed underground connections. In Montreal, studies point to four reasons for the development of its underground city: harsh winters, a sufficiently dense population, high land prices on the surface, and the development of underground transportation systems.

Montreal's Underground City, also known as the indoor city or officially as RÉSO (or "La Ville Souterraine", a French word meaning network), is a pool of interconnected complexes in and around Downtown Montreal in Canada. It is renowned as one of the largest underground complexes in the world with over 32 km of tunnels encompassing about 4 million m^2 of connected areas. There are about 200 entrance points to the underground city, connecting about 80% of office space and 35% of commercial space in downtown Montreal. It is estimated that some 500,000 people make their way through the subterranean city each day.

The first link of the underground city could be dated back to 1962, with the construction of Montreal's first modern skyscraper, the Place Ville-Marie, an underground shopping mall to cover an unsightly pit of railway tracks north of the Central Station, and a tunnel to link it to the Central Station and Queen Elizabeth Hotel. The underground links expanded with the advent of the Montreal Metro in 1966, forming the core of the underground city. The expansion continued to link up metro stations, pedestrian corridors and underground shopping malls, and connecting to the landmark office complexes, hotels, condominiums, parks, banks, museums, entertainment venues, cathedral, universities, government buildings, exhibition halls, commuter train stations and bus terminals in the city.

Key ingredients and recipes are as follows:

- Providing a warm and safe sheltered pedestrian network around Montreal, especially during the winter months;
- Covering a wide portion of the city, with convenient access points to and from the surface across its entire underground complex;
- The entire network is interwoven with subways, major metro stations and transport facilities, various landmarks developments and a wide variety of land uses and amenities in the city;
- The functions of the underground city are as diverse as and complement with those of the city above;
- Different underground segments has its unique character, reflecting the city diversity and making it easily recognisable and navigable from within;
- The policy of the Montreal Urban Community Transit Commission of offering the aerial rights above metro station entrances for construction through emphyteutic leases, i.e. a type of real estate contracts specifying that the lessee must improve the property with construction; and
- Proactive private initiatives, e.g. office buildings use their connection to the underground city as a selling point for their office space.

(b) Upcoming cases with shorter history of underground development

(i) Mainland China

In Mainland China, the rapid rate of urbanisation in recent decades has catalysed the exploration of the planning and use of underground space. Many mega cities including Beijing, Shanghai, Guangzhou, Shenzhen and Hangzhou are stepping up their planning initiatives for urban underground space, including the preparation of underground development strategies and plans. Beijing, for example, is developing an integrated underground plan to link mostly basement type complexes; Guangzhou has integrated underground planning in designing the new town centre, viz. Zhujiang New Town; and Shenzhen has successfully launched the nation's first underground sewage treatment plant at Phuket, with parks and recreational facilities atop for public enjoyment. The early awareness of the need for an underground development strategy for the urbanised space will help promote more rational spatial planning in the long run.

(ii) Singapore

Notwithstanding that Singapore is a relatively late comer to underground development, the Singaporean Government is vigorously promoting the development of underground space as a pivotal solution to addressing the land availability issues. In 2007, the Government set up an inter-agency Underground Master Planning Task Force under the Ministry of National Development to map out the long-term development of the underground, bringing the underground space to a strategic level. The Government is working to put in place enablers for underground development such as in developing an underground master plan and an underground land bank, establishing a National Geology Office, and developing a subterranean land rights and valuation framework.

The Singaporean Government has also commissioned a detailed feasibility study of an underground science city and concept studies for applications such as an underground landfill, the preliminary findings of which were presented at the 13th World Conference of Associated Research Centres for Urban Underground Space held in November 2012. The former proposed 40 linked caverns for accommodating data centres as well as research and development (R&D) facilities, while the latter could contain about 40 years' worth of garbage.

In the urban context, in August 2012, the Urban Redevelopment Authority (URA) of Singapore released a circular, viz. "Central Area Underground Master Plan – Revisions to the Cash Grant Incentives Scheme for Underground Pedestrian Links", which sought to improve pedestrian connectivity in the city centre by providing cash grant incentives to the private sector for underground pedestrian links in designated urban centres. Developers could claim back a certain amount of money per m^2 when developing the desired pedestrian links, or given gross floor area exemptions for these links. While the effectiveness of these "carrots" is yet to be ascertained, some commentators remarked on the limited achievement of those incentives back in the 2000s (e.g. offering business owners grants for consultancy fees to build underground links, and partially reimbursing the tunneling costs for the underground links for Orchard Road). They suggested that instead of government directives, it was the escalating land prices that forced developers to consider the underground options, as this has narrowed the cost difference between building underground and on the surface.

(c) The local cases

(i) Underground Admiralty (Plate 1)

Admiralty is a prime business and transport node in Hong Kong. Underneath the busy trunk roads and the premier offices, commercial and hospitality facilities on the surface is a network of interconnected underground chambers and valves (e.g. MTR lines and stations, pedestrian links and public carparks) that are complementing the functions of the city above, helping to resolve urban problems (e.g. traffic safety and congestion, lack of carparking spaces), and contributing to revitalising the neighbouring Wanchai district (e.g. the Star Street precinct). The government-built and privately operating underground carpark at Admiralty, for example, has served the dual purposes of providing public carparking spaces and leaving the surface above for green park facilities, viz. Harcourt Garden. Private initiatives such as concerned developers and MTRC have also been playing an active role in designing an integrated building mass with underground links. For example, the 280-metre Three Pacific Place Link is equipped with travelators, providing the public with a safe, direct and all-weather access to the Pacific Place complex, MTR Station and the Star Street precinct. This showcases how underground space could evolve in a more organic manner in urban Hong Kong to promote compact development and urban regeneration in Hong Kong. In redeveloping the existing buildings in Admiralty, the Government would also endeavour to harness the opportunities for optimising the inter-spatial planning and design potential of future development. As in the case of Queensway Plaza which PlanD will soon commission the "Planning and Design Study on the Redevelopment of Queensway Plaza, Admiralty - Feasibility Study", the study objective includes improving the pedestrian connectivity with developments in the vicinity, including underground links, and to upgrade the overall existing public realm in the process of investigating the planning, architectural and engineering feasibility in redeveloping the site for commercial uses.

Plate 1: Compact development at Admiralty with integrated underground pedestrian links, transport and parking facilities

Key ingredients and recipes are as follows:

- An enabling planning mechanism to facilitate the utilisation of underground space for pedestrian links and underground railway (e.g. pedestrian subways and underground railway structures are always permitted in all zones on the statutory town plans);
- Location factor: Admiralty is not only a premier business district but also a nodal public transport interchange, including the MTR interchange station connecting the Tsuen Wan Line with Island Line as well as the future South Island Line (East) with a future passenger platform sited under Harcourt Garden. Public and private investment in the underground infrastructure is justified by the large business and passenger thresholds.
- The underground infrastructure could provide a grade separated transport solution to the busy traffic, particularly along Queensway;
- The comprehensive network of underground pedestrian links provides direct and convenient access to the nodal MTR Admiralty Station as well as the city centre and its adjoining areas;
- Proactive private initiatives in shaping the underground space in the Admiralty and the neighbouring Wanchai district; and
- The underground links help revitalise the neighouring older areas, notably Star Street, which in turn complement the office tenants with a cosy and stylish hangout destination filled with cafes, bars, lifestyle boutiques and galleries.

(ii) Tsim Sha Tsui harbourfront (Plate 2)

Tsim Sha Tsui harbourfront is a renowned place of attraction in Hong Kong. Three out of the ten "Top Attractions in Hong Kong" as gathered by the Hong Kong Tourism Board are sited at the harbourfront, including the Avenue of Stars, the Clock Tower and Tsim Sha Tsui Promenade. The harbourfront, however, is dissected by Salisbury Road – a major trunk road running parallel to the Tsim Sha Tsui coast and characterised by busy

vehicular traffic. The hotel and shopping clusters are primarily abutting the other side of Salisbury Road, which are accessible by a web of underground pedestrian subways. These subways are developed incrementally without comprehensive planning at the outset, though they eventually mingled well to provide the public with a reasonably seamless underground pedestrian network connecting to the East Tsim Sha Tsui Station and to access points at various parts of Tsim Sha Tsui (e.g. Middle Road, Peking Road, Kowloon Park, Canton Road, Hanoi Road, Nathan Road, Mody Road, Tsim Sha Tsui East, etc). The East Tsim Sha Tsui Station links the West Rail Line and the Tsuen Wan Line by subways. In addition, the underground link is connected to the very first wholly underground department store of the territory, thereby minimising structural obstruction of the harbour view for public enjoyment. This case illustrates how smart use of underground space has been incrementally developed at the reclaimed land through the collaborative efforts of various parties to facilitate pedestrian circulation as well as preservation of the harbour view.

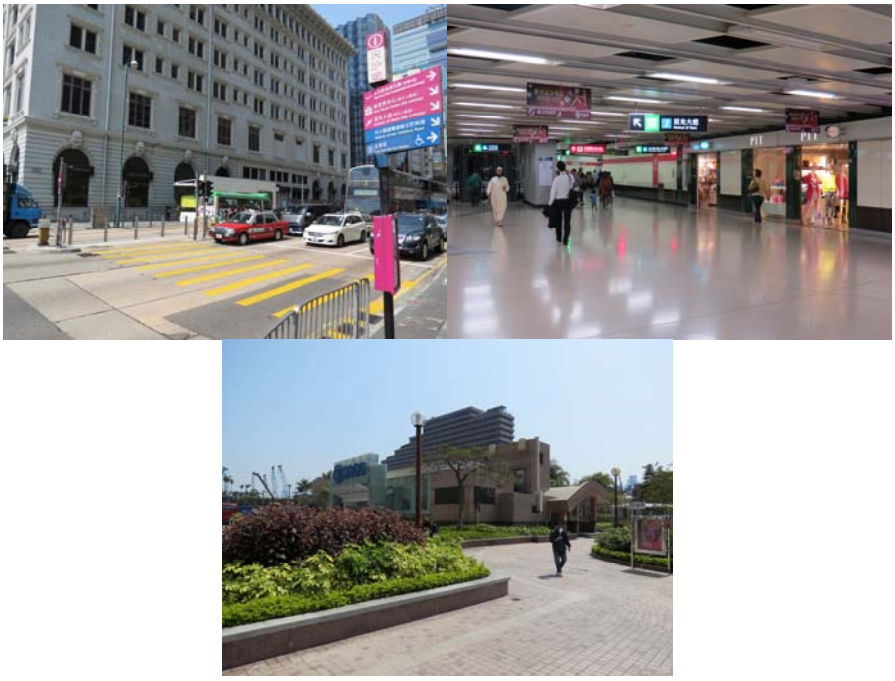

Plate 2: Smart use of underground space in Tsim Sha Tsui to facilitate pedestrian circulation and preservation of harbour view

Key ingredients and recipes are as follows:

- Similar to the Admiralty case, the enabling planning mechanism, collaborative private initiatives, locational factor, traffic and pedestrian circulation benefits provided by the underground links, etc are all attributing to the success of the Tsim Sha Tsui case; and

- The careful siting of the access points to the surface has promoted the usage of the underground links, which provide the public with more direct, accessible, safe and all-weather pedestrian links as compared to the surface crossings. Some parts of the underground links also contain shops, eateries and other ancillary facilities, serving the diversified needs of the users apart from pedestrian circulation purposes.

(iii) HKU Centennial Campus and related public projects (Plates 3 and 4)

Today, the campus of the University of Hong Kong (HKU) is encompassing the former fresh water service reservoir and associated waterworks facilities of the Water Supplies Department (WSD). The new campus, known as the Centennial Campus, is to celebrate HKU's centenary in 2011 as well as to cater for additional intake with the implementation of the four-year undergraduate degree curriculum in 2012. The development has been carefully planned to utilise the available space in a holistic manner. The site is chosen amidst the developed

and land-scarce Central & Western district, as close as possible to the existing HKU campus. The development has unprecedently reprovisioned the salt water service reservoirs in caverns. It would also adaptively re-use the affected historic buildings at the site and integrate them into the new development: the former Western Senior Staff Quarters will be converted into the office of the HKU Press and exhibition and reception areas of the University; the former Western Staff Quarters into an information centre of HKU; and the deck above the fresh water service reservoirs on both sides of the Elliot Treatment Works Building into landscaped gardens of the campus. Soon, the University will be serviced by the MTR West Island Line, with station entrances connecting to the MTR via both footbridges and underground walkways. Like mastering a Rubik's cube, this case demonstrates a skillful optimisation of the inter-spatial relationship of different uses while maintaining the integrity of the various functions in a three-dimensional (3-D) manner.

Plate 3: HKU Centennial Campus and the reprovisioned salt water service reservoirs in cavern

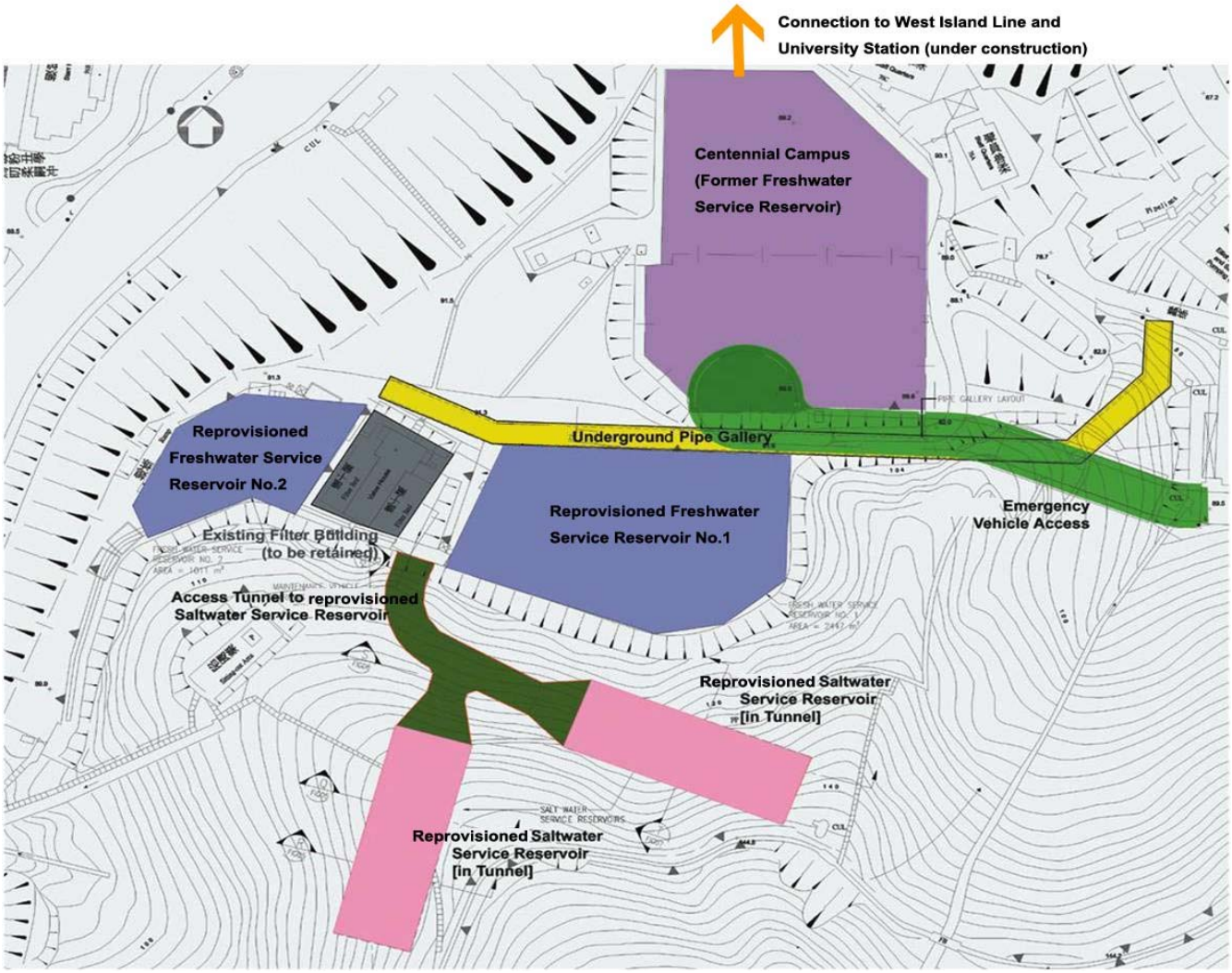

Plate 4: Layout plan of the HKU Centennial Campus (existing service reservoir site) and the reprovisioning of the affected service reservoirs
(Source: http://www4.hku.hk/cecampus/upload_files/newsletter_content/ori_pic/5/issue1_02_img01_lrg.jpg, modified)

Key ingredients and recipes are as follows:

- Appreciation of the inter-spatial relationship of the concerned uses and innovative mindset: the development has given due consideration to the efficient use of space and effective integration of surface and underground uses at the planning and design stage, e.g. innovative use of the caverns for reprovisioning the salt water service reservoirs, and adaptive re-use of the historic buildings and structures on the surface of the site to blend in with the new campus;

- Siting: the new campus is sited near the existing one to minimise logistical issues and to foster a sense of unity for students and staff of the University. It is easily accessible to the university student dormitories in the Pok Fu Lam area and the future public transport node provided by the MTR HKU Station;

- A consciousness of environmental, ecological, sustainability and heritage issues:

  - no massive excavation and extensive tree felling, thereby preserving the woodland habitat of the natural hillside;

- the proposed finger-like layout of the cavern avoids encroachment onto the country park above while providing adequate width for tunnel access at the cavern portal for operation and maintenance purposes;

- compared with an alternative open-cut scheme which would result in the formation of large and steep cut slopes on the hillside, the cavern option would minimise the adverse visual impact to the neighbouring residents, country park visitors and users of the new campus;

- the cavern scheme involves soil and rock excavation of less than 40,000 m^3, while an open-cut site formation scheme would have required about 280,000 m^3 of excavation; and

- the cavern scheme would minimise the dust and noise impacts during construction;

- Time and cost savings: the duration of construction of the cavern scheme for the re-provisioning of waterworks facilities was about 30 months, which was much shorter than the open-cut site formation scheme;

- Collaborative initiatives: various actors, including HKU, professional consultants and various government agencies have worked together to facilitate the development of the project from scratch to fruition. These include cautious choice of location for university expansion and reprovisioning of affected facilities, careful design of the facilities to eliminate interface issues, proper sequencing of works, etc; and

- Early and continuing dialogues in the planning process: according to our records, discussion between HKU and PlanD on the preliminary proposal for the university expansion could be dated back to 1998. As the proposed site for university expansion was predominantly zoned "Government, Institution or Community" ("G/IC") where educational uses were permitted, PlanD's roles were more of a facilitator and advisor instead of a regulator. PlanD had tendered planning advice on the zoning matters, siting and proposed design, and advised HKU to consult WSD and other relevant departments on the proposed reprovisioning of the waterworks facilities. In addition, PlanD had also bridged between HKU and MTRC in the process of devising the overall linkages of different parts of the campus with a view to improving their connectivity with the proposed HKU Station of the West Island Line. Such dialogues are vital to ironing out concerns and enhancing the planning merits of the scheme.

3.3 Essential ingredients and recipes

In light of the above cases, the key ingredients for successful underground developments could be broadly categorised as follows:

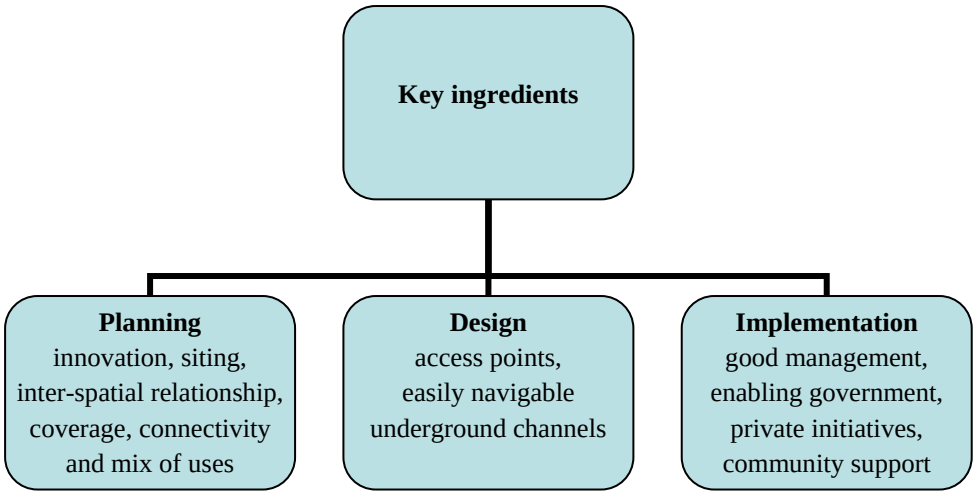

Plate 5: Key ingredients to successful underground development

- Planning ingredients:
  - innovative mindset;
  - appropriate siting of underground uses;
  - complementary inter-spatial relationship between underground and surface uses;
  - good coverage, connectivity and mix of uses;
- Design ingredients:
  - careful design of access points;
  - sensible design of easily navigable underground channels;
- Implementation ingredients:
  - good management;
  - enabling government provisions (including government incentives);
  - private initiatives; and
  - community support.

Nonetheless, having the right ingredients does not suffice. We also need to have the right recipes to create a signature dish. The acronym of such workable recipes is "2S2C", viz. Sustainability, Strategic, Coordination and Collaboration, as explained below:

Plate 6: The 2S2C Recipes

- Sustainability as the overarching consideration:
  - "Sustainability" should be the overarching consideration in substrata planning. Compared to the above-ground development which could be easily demolished, development in the substrata layers of the land could not be reinstated and prior use would often constrain the future use of the land. Unplanned development of underground space based on the "first-come-first-serve" approach would have an adverse impact on the future development of a city.

- To ensure sustainable use of the underground space, one must cherish it as a spatial asset and appreciate it in light of the 3-D relationships between the built environment and supporting infrastructure. Conflicts with prior uses and unappreciated impacts on other underground resources often make the overall use of underground space suboptimal. A way to promote sustainable use of the underground resources is through the preparation of underground master plan which provides a strategic and long-term vision for the use of underground resources, as exemplified in the Helsinki case. Singapore and many cities in Mainland China with a rapid rate of urbanisation are also stepping up their underground planning initiatives.

- As sustainable development is the overarching planning principle for Hong Kong, similar consideration for underground development should also be an integral part of the overall planning consideration in the local context.

- Strategic and integrated planning approach to underground development:

  - To better utilise and rationalise the use of underground space vis-à-vis surface space, planning considerations should be given greater weight and priority. The strategic benefits of utilising underground space in a wider territorial context, as compared to a surface option, should be taken into account at the early planning stage. As the unofficial motto of the Finnish Tunnelling Association goes, "always examine the underground alternative".

  - In planning for urban growth, Hong Kong has long adopted an integrated land use-transport-environment planning approach, which is instrumental to promoting sustainable development in Hong Kong. We give priority to the use of railway as the backbone of our public transport network as it is a more environmentally friendly and more efficient mass carrier than road-based transport modes. We have complementary land use and zoning regulations to promote high density developments near rail stations to capitalise development opportunities and to reduce reliance on road-based travel. We also encourage the design of major activity nodes to be within 500m walking distance from the railway stations or public transport interchanges, and good connectivity between the nodes and facilities with well-planned pedestrian walkway networks.

  - As a step forward, such an integrated planning approach to development could be refined to include underground planning. As a start, the following could be considered:

    (i) as shown in the Helsinki and Montreal cases, conception of the use of underground space is often associated with metro development. In planning for our future railway network, due consideration could be given to optimising the use of the connecting underground chambers and valves, taking into account the inter-spatial relationship with the surface land uses to promote a compact and sustainable urban development; and

    (ii) a stock taking exercise undertaken by the "Enhanced Use of Underground Space in Hong Kong" indicated that over 400 existing government facilities have the potential for relocation to rock caverns in line with the potential land uses listed in the Hong Kong Planning Standards and Guidelines (HKPSG). In planning for such facilities in future, early consideration should be taken to navigate development underground, which could be explicitly spelt out in the HKPSG or other administrative guidelines. In addition, a long-term strategy should be developed to systematically relocate existing government facilities underground.

- Coordinated planning, regulatory and monitoring mechanism for underground use:

  - In addition to an appropriate strategy, a coordinated planning, regulatory and monitoring mechanism for underground development is also needed. As shown in the Helsinki case, a unified mechanism including planning, development and legal provisions is instrumental to coordinating both private and public initiatives to underground uses.

  - The current statutory planning framework in Hong Kong is primarily intended to govern surface development, and unless otherwise prescribed, relevant planning controls applicable to surface development would also apply to the underground space. This allows flexibility in a way that appropriate cavern developments can be facilitated by the planning application or zoning amendment mechanisms based on individual merits. On the other hand, this may not be clear or systematic enough to cope with a more widespread use of underground space anticipated in future. More explicit guidelines and regulatory mechanisms, covering not only town planning but also other related aspects, would be needed.

- Collaborative initiatives of various actors:

  - The cited cases show that government directives alone would not be sufficient to foster sustainable underground development. Often, the government would also need to provide an enabling environment to promote collaborative private initiatives. In addition, the private sector has to be convinced of the merits, in particular financial benefits, in investing in the underground infrastructure. Soliciting community support for underground development is also important.

  - Even within the public sector, a collaborative multi-disciplinary approach is also needed for building an intelligible underground database to facilitate underground planning. As an attempt to explore the technical feasibility of establishing a 3-D underground geographic information database to facilitate underground planning, PlanD has initiated an in-house "Technical Feasibility Study for Establishing a 3-D Underground Geographic Information System (GIS) Model for Causeway Bay". The pilot study reveals that underground information is currently scattered among various government departments and public utility companies, and strenuous effort and multi-disciplinary inputs from engineers, planners, IT experts etc would be needed to gather, consolidate and interpret the relevant data. As such, any further initiatives to construct 3-D underground GIS database on a territory basis would require multi-agency and multi-disciplinary collaboration.

4 FOOD FOR THOUGHT

In relation to underground planning, some food for thought is also worth considering in the context of Hong Kong:

4.1 The urban opportunities

In terms of cavern development, the Government has identified five Strategic Cavern Development Areas in the "Enhanced Use of Underground Space in Hong Kong - Feasibility Study" (viz. Mount Davis, Lion Rock, Shek Mun, Siu Ho Wan and Lam Tei), which are primarily sited in the less urbanised areas. In view of the genuine need for "space" in meeting the sustainable development of Hong Kong, the potential of utilising the underground space in the urban areas should not be underplayed. The Government, therefore, is currently looking at the cavern development opportunities in some of the more urbanised areas. For example, in the recently announced Pilot Schemes of Rock Cavern Development under the Stage 2 Public Engagement Digest of "Enhancing Land Supply Strategy: Reclamation outside Victoria Harbour and Rock Cavern Development", Diamond Hill Fresh Water and Salt Water Service Reservoirs are included as one of the pilot schemes, in addition to the Sai Kung Sewage Treatment Works and Sham Tseng Sewage Treatment Works.

Back in 1992, PlanD commissioned the "Planning Study on Underground Development of Commercial Facilities extending beyond Private Land". The following key locational criteria were identified as essential to the consideration of potential underground commercial/carparking projects: concentrations of both residents and working population, recognised focus of commercial activity providing facilities to support the population, high level of pedestrian flows, and good public transportation facilities including proximity to MTR stations. Areas designated "Open Space", "Road" and "G/IC" on the statutory town plans were identified as the primary development opportunities for underground development projects extending beyond private lot boundaries. As part of the government efforts in enhancing the pedestrian environment, PlanD further commissioned the "Study for Planning for Pedestrians" in March 2001, culminating to the preparation of pedestrian plans for areas such as

Causeway Bay in consultation with the public. Such plans included proposals for underground pedestrian-cum-retail link to facilitating accessibility and alleviating congested crossing in the area. In 2009, the Transport Department has further commissioned a study to improve the pedestrian environment in Causeway Bay, including proposals for a pedestrian subway system connecting Victoria Park and the junction of Leighton Road/Wong Nai Chung Road with consideration for extending the system to the northern fringes of Happy Valley. In 2011, the Highways Department commissioned a feasibility study to take forward the proposed Causeway Bay subway system, and a series of public consultation activities have been conducted, including the public forum held in July 2012, to solicit public views. Throughout these years, the Government has been persistently pursuing the implementation of the underground scheme in Causeway Bay. Compared to the case of Tsim Sha Shui, underground pedestrian and retail links in Causeway Bay are rather limited. Notwithstanding this, the Government is still striving to provide a robust enabling framework to promote the building of underground links connecting to different buildings through carrying out relevant studies and public consultation activities to solicit views on the underground proposals. Incentives such as bonus plot ratio for underground public passages are available to private developers to encourage them to retrofit their properties to provide the underground access and links. The building of a successful underground walking city would require not only careful planning and design inputs, but also the collaborative efforts of the concerned parties.

While recognising possible constraints and difficulties in underground development directly beneath the urban areas, such as the common use of deep foundations and land ownership issue, Hong Kong does have some successful underground development in its prime business and tourist districts in Admiralty and Tsim Sha Tsui respectively. It is worth seriously considering the potential of retrofitting the underground space in urban Hong Kong to provide a sustainable solution to some of our municipal issues such as over-congestion, food waste or even columbarium demand.

- Urban congestion: developing a comprehensive network of underground pedestrian links within the urban centres, connecting MTR stations and various residential, commercial or institutional developments, exploring the potential of developing underground shopping arcades, parking spaces, common utility trenches, etc. This can also be regarded as a resilient solution to facilitating pedestrian flow during inclement weather such as typhoons.

- Food waste: in Hong Kong, the Government has initiated the Kowloon Bay Pilot Composting Plant to collect food waste and reproduce compost from it. The facility is built on ground but in view of its operation nature, there could be possibility of utilising the underground space for similar purpose and integrating them underneath buildings as part of the solution to part of the municipal waste issues.

Plate 7: Kowloon Bay Pilot Composting Plant

- Columbarium demand: possibility of placing columbarium chambers underneath the public open space in various districts. Underground columbarium in a park setting could be found and well accepted in some parts of the world, e.g. Church of St. Mary of the Angels in Singapore.

In addition, relocating other NIMBY facilities (e.g. refuse transfer facilities, sewage treatment plants, etc) underground could free up plots for other beneficial uses and help resolve interface issues. Innovative mindset and community support for such initiatives would be needed.

4.2 An appropriate accounting equation

Project-based feasibility assessment often does not adequately reflect the overall benefits of a scheme to the community. As a result, potential underground options often appear to be more costly than the above-ground options, especially if land savings and the alternative beneficial uses for releasing the surface land are not counted. To properly reflect the true benefits of underground development as compared to the surface alternative, some alternative accounting methods could be considered:

- Cost-benefit analysis: quantifying non-monetary and intangible benefits (e.g. aesthetics, safety, less traffic congestion, etc) and opportunity costs (e.g. releasing the surface space for alternative uses) and comparing above-ground and underground development (e.g. land purchase versus space creation, solar heat load versus longer ventilation; weather proofing versus drainage of seepage water, etc); and

- Life-cycle costs: taking into account land cost, construction cost, savings in special design features, energy savings, maintenance costs, replacement costs, and indirect benefits of underground structures (community valuation of drawbacks of above-ground structures in terms of environmental degradation), etc.

To fully capitalise the merits of underground development, it should not simply be taken as an individual engineering project but must be perceived as a key land use and urban design element within an integrated planning process. As such, any planning and engineering feasibility study for new developments should also explore the possibility of underground development option at the initial planning stage using the appropriate accounting equation.

4.3 Other Possibilities

With technological advancement in underground exploration and the growing popularity of underground usage elsewhere in the world, Hong Kong could actively pursue more innovative uses of its underground space. For example, in the near future, almost all the universities in Hong Kong would be connected with or located in close proximity to a web of underground railway links. Possibility of building an underground university and R&D city in Hong Kong might not be a dream.

In addition, noting that the remaining local quarry at Anderson Road would soon be closing down and the one at Lam Tei would be closing down within ten years, development of new quarry sites would be of strategic importance to cater for the needs of the local construction industry. To "kill two birds with a stone", cavern quarrying could be considered a viable option. Instead of relying on import, cavern quarrying could help provide aggregates locally. It also helps create additional space in caverns for housing suitable facilities.

5 CONCLUSION

Notwithstanding that Hong Kong is only at a teething stage in formulating its planning strategy for underground development, it is hoped that in light of the case studies above, we could apply the right ingredients and recipes in mapping out and building an underground labyrinth to foster the sustainable development of Hong Kong. Different professional disciplines including engineers and planners, the Government, the private sector and the community would all be members of the chef team to turn Hong Kong into a signature underground city.

ACKNOWLEDGEMENTS

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Rock Caverns – "Unlimited" Space for Future Development

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ABSTRACT

The hilly terrain and underlying geology of Hong Kong offer an excellent opportunity for placing urban facilities underground. About two-thirds of Hong Kong's land is found to be suitable for rock cavern development. Conservative estimates suggest that some 3,500 ha of land could be made available by cavern development at the urban fringes. Given the potential for multi-layer cavern development, a substantially greater usable area could be created.

In September 2012, the Civil Engineering and Development Department instigated a study on "Long-term Strategy for Cavern Development", which is to develop a holistic approach in planning and implementing cavern development, so as to render it a sustainable means for expanding land resources. The study also places emphasis on private sector participation because many private sector facilities, such as storage, warehousing and data centres, can benefit from rock caverns' stable and secure setting.

The formulation and implementation of a long-term strategy for cavern development could provide a sustainable approach in easing the pressure of land shortage. Developing a systematic relocation programme for suitable Government facilities could release surface sites for other uses including housing. Also, placing NIMBY facilities in caverns could remove incompatible land uses. Furthermore, reserving rock cavern space could accommodate future public and private sector facilities underground, thereby reducing the land take.

1 INTRODUCTION

Hong Kong's topographical setting with steep natural hillsides pose significant constraints to development (Figure 1). Land, particularly in the urban areas, is a scarce resource. To support social and economic development, there is a pressing need to increase the supply of land and optimise the usage by sustainable and innovative approaches. One possible way is through rock cavern development for suitable types of land use.

In October 2009, the Chief Executive of the Government of the Hong Kong Special Administrative Region (HKSAR) presented the 2009-10 Policy Address on "Breaking New Ground Together". The Development Bureau put forward a new initiative under the Policy Agenda of 'Developing the Infrastructure for Economic Growth' to launch strategic planning and technical studies to facilitate planned development of underground space, which are aimed at promoting the enhanced use of rock caverns as part of Hong Kong's pursuit of sustainable development. In the 2013 Policy Address on "Seek Change, Maintain Stability - Serve the People with Pragmatism", the Chief Executive highlighted that rock cavern development is a viable source of long-term land supply and stressed the need to conduct a study on the long-term strategy for cavern development with a view to preparing rock cavern master plans and formulating policy guidelines.

This paper examines the potential of rock caverns as a valuable land resource for Hong Kong, in light of the policy support for its development, and the work in progress in order to realise this potential. The paper (i) summarises the findings of a recently completed cavern study, (ii) establishes the hidden land resource from cavern development, (iii) examines the background and constraints to cavern development, and (iv) outlines the scope and expected outcomes of the current study on long-term strategy for cavern development.

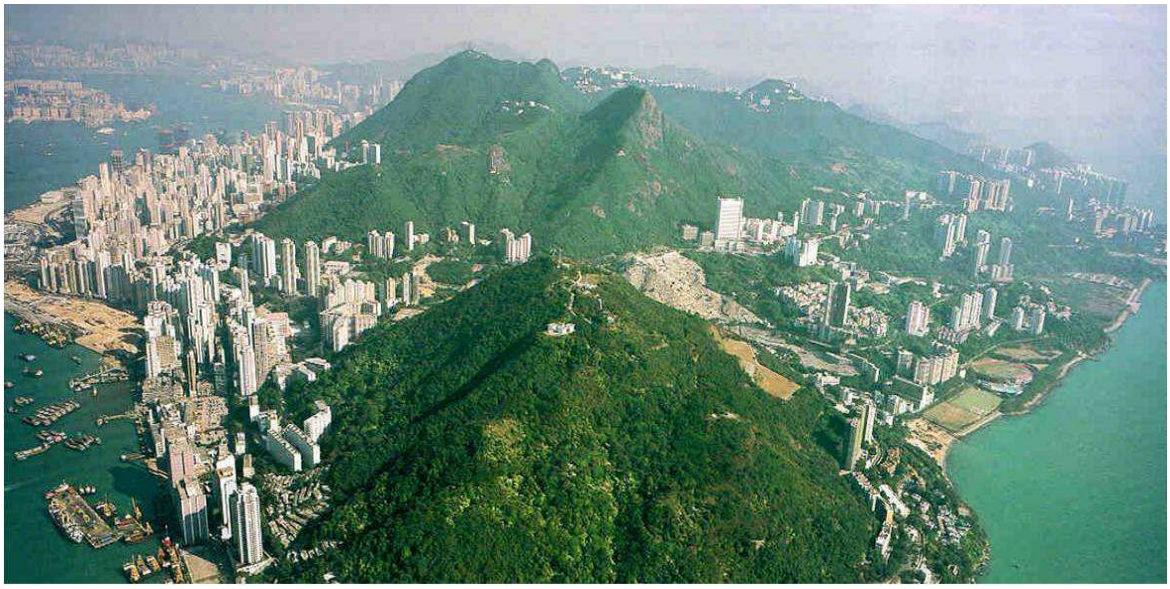

Figure 1: Aerial view of Hong Kong Island

2 STUDY ON ENHANCED USE OF UNDERGROUND SPACE IN HONG KONG

A strategic planning and technical study on "Enhanced Use of Underground Space in Hong Kong", hereafter referred to as the "Underground Space Study", was commissioned by the Geotechnical Engineering Office of the Civil Engineering and Development Department (CEDD) in March 2010 to take forward the policy initiative of rock cavern development as laid out in the 2009-10 Policy Agenda. The study, which was completed in March 2011, explored the opportunities to enhance the effective use of land resources in Hong Kong from a new perspective through the planned development of underground space. The study is summarised by Chan (2011) and the Executive Summary of the study has been made publicly available (Arup 2011). The key findings were as follows:

- Hong Kong is particularly suitable for developing rock caverns from a geological perspective. The hilly areas in the urban fringes of Hong Kong with strong rocks and convenient access are particularly suitable for cavern development.
- By reprovisioning suitable Government facilities inside caverns and releasing the original land as well as any adjacent sterilized land for housing and other uses, cavern development is a viable option to increase land supply.
- Cavern development could also accommodate new infrastructure facilities which would otherwise occupy surface land. Reserving underground space could cater for future projects and expansion of underground facilities.
- For those NIMBY (Not-in-My-Backyard) facilities like sewage treatment works and refuse transfer stations, the cavern option would help reduce adverse impacts on the local environment, remove incompatible land uses and alleviate the NIMBY sentiment.

3 HONG KONG'S HIDDEN LAND

The Underground Space Study has identified that two-thirds of Hong Kong's land is of medium to high suitability for cavern development (Figure 2). Comparatively, those areas closest to the urban fringes have higher potential due to the relatively easier access from existing transport networks and lower excavation and support costs for the relatively shorter access tunnels to proposed cavern developments. These urban fringe areas are also generally not compromised by private land ownership issues.

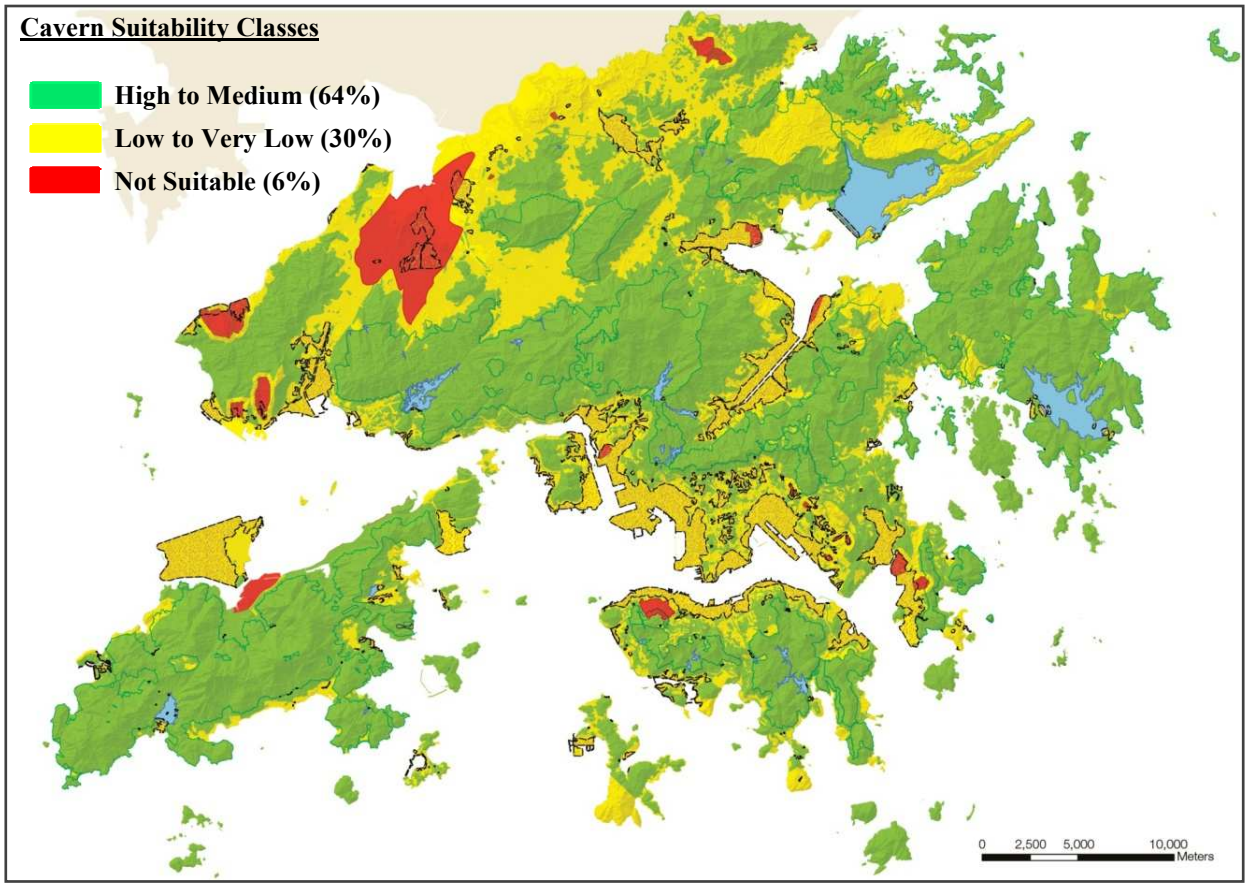

Figure 2: Distribution of cavern suitability classes

Here we make a simple estimation. If two-thirds of Hong Kong's land area (1,104 km^2) is considered suitable for cavern development, this would amount to some 700 km^2 (70,000 ha). Conservatively assuming that only 10% of this area (7,000 ha) is readily accessible within the urban fringes due to access or land ownership issues, and allowing a further 50% reduction to cater for intervening rock pillars for support, this could still provide some 3,500 ha of developable land in plan area (equivalent to about 180 no. of Victoria Park). If we think three-dimensionally, a multiplier effect may also be realised by vertically stacking caverns at different elevations and also by forming multiple floors within a single cavern to increase gross floor areas. This is illustrated in Figure 3.

As the land area occupied by existing Government facilities considered suitable for cavern development is around 400 ha, relocating those facilities in caverns could release the land for other uses. Relocation of NIMBY facilities can also enhance the value of neighbouring land. For existing Government facilities, the released sites are usually not very large (say 1-2 ha) but are still attractive as they are generally close to urban areas with good infrastructure support and hence can be readily turned to developable land without much works required. However, it is recognised that housing certain large Government facilities in caverns will still require resolving some specific technical issues (e.g. fire safety for sports facilities, ventilation for water treatment works), and therefore it will take a longer lead time to realize their potential.

Given that some 3,500 ha of land may be formed from cavern development, and that the existing Government facilities considered suitable for cavern development only occupy around 400 ha, there is considerable scope for developing new public and private sector facilities in caverns, thereby reducing the future land take. The Underground Space Study reviewed overseas practice and recommended other potential land uses for cavern development for inclusion in the Hong Kong Planning Standards and Guidelines (PlanD 2008), as shown in Table 1. It is considered that some cavern facilities may have opportunities for private sector participation, if appropriate, and therefore suitable enabling or facilitating mechanisms should also be established.

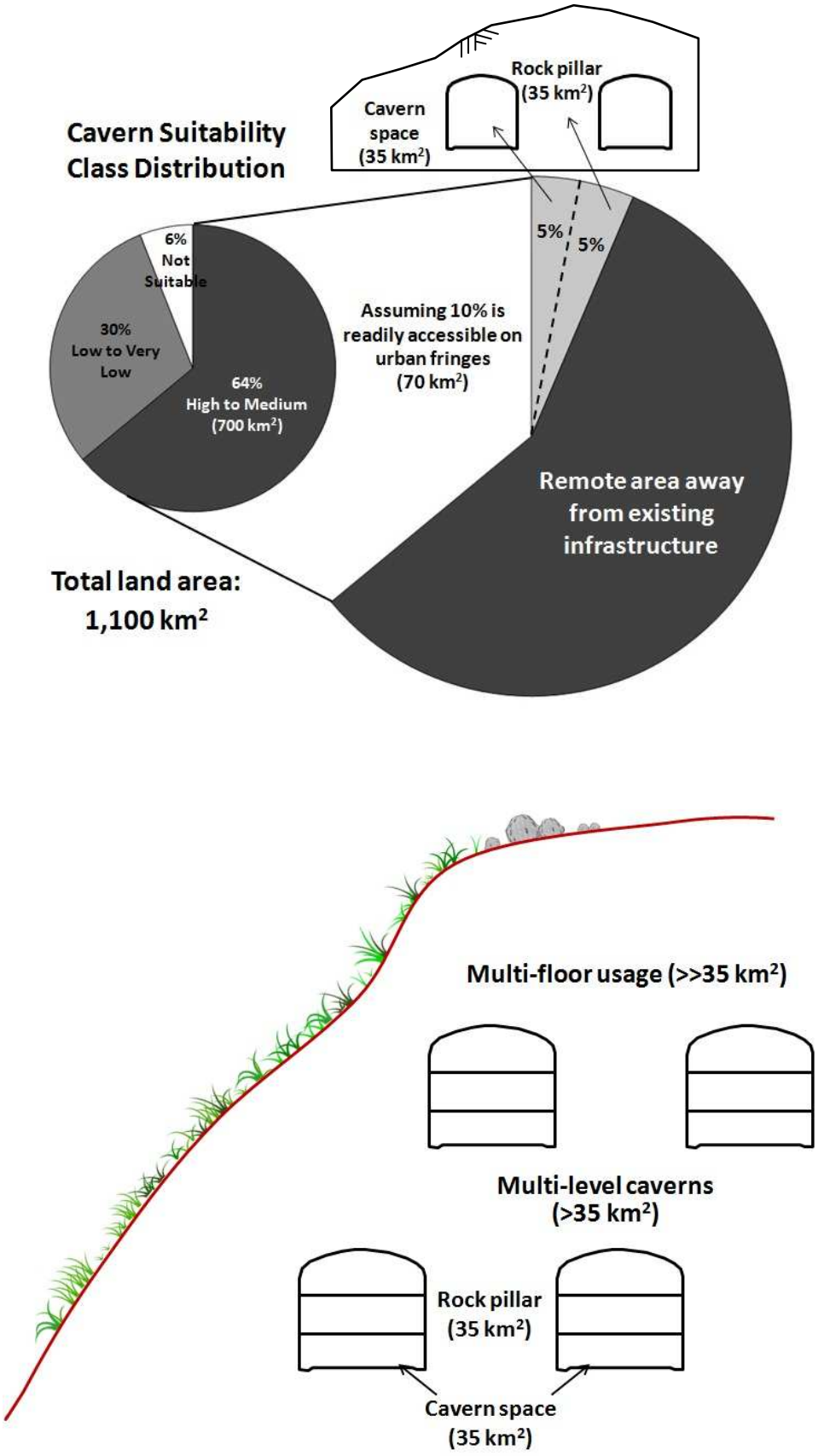

Figure 3: Hong Kong's hidden land

Table 1: Potential land uses for cavern development

Land Use Category	HK Planning Standards & Guidelines (2008)	Additional Potential Land Use (Arup 2011)
Commercial	Retail	Food/Wine storage Warehousing
Industrial	Industry Storage/Warehousing Oil bulk storage LPG bulk storage	Dangerous goods Data centre Research laboratories Science park
Government / Institution / Community (GIC)	Civic centre Indoor games/Sports Hall Incinerator Refuse transfer facility Service reservoir Sewage/Water treatment plant Slaughterhouse Wholesale market Transportation connections & networks Columbarium/mausoleum/mortuary	Archives Bicycle park-and-ride Car/Vehicle parking Crematorium Refuse collection point Maintenance depot, e.g. rail and bus Underground quarry
Public Utilities	Power station	Substation

4 WHERE ARE WE NOW IN TERMS OF CAVERN DEVELOPMENT?

Hong Kong has a rich legacy of underground tunnelling. During its development, some 430 km of tunnels have been constructed and some 180 km are planned up to 2020 (Pang & Woodrow 2009). These underground structures cater for water supply, mass transportation (such as railways and roads), drainage, conveyance of sewage and electrical cables. Hong Kong also has a diversity of deep basement-type excavations formed by cut and cover methods for MTR station concourses, car parks, retail and commercial space and more recently storage tanks for flood control.

Cavern construction is an established technology that has shown continual improvement in its application (NRC 2013). Since the 1980s, the Government of the HKSAR has already carried out the necessary technical and preparation work for cavern development in Hong Kong. Technical standards on cavern engineering and fire safety design, as well as planning guidelines to establish the ground work for rock cavern development, have been published. The details have been outlined by Chan and Ng (2006).

A few purpose-built rock caverns were constructed in the mid-1990s to accommodate public facilities to meet the needs of the community (Figure 4), namely the Island West Refuse Transfer Station (1997), Stanley Sewage Treatment Works (1995) and Kau Shat Wan Explosives Depot (1997). Also in 2009, the University of Hong Kong reprovisioned the Western Salt Water Service Reservoirs in rock caverns to release 2 ha of land, occupied by the service reservoirs as well as the adjacent land, for the Centennial Campus development, which includes three buildings for the Faculty of Arts, Faculty of Law and Faculty of Social Sciences.

Yet, these facilities, which are a testament to their success, remain relatively unknown to the general public. They are predominantly NIMBY facilities, which are now effectively hidden underground (Figure 5), with minimal visual and environmental impact and attract very few complaints. As an example, relocating the Western Salt Water Service Reservoirs to rock caverns provided a sustainable and environmentally friendly solution as compared to the original open-cut scheme. The amount of excavation in soil and rock was significantly reduced to one seventh of the original scheme and substantial areas of woodland were saved, in addition to preserving three graded buildings.

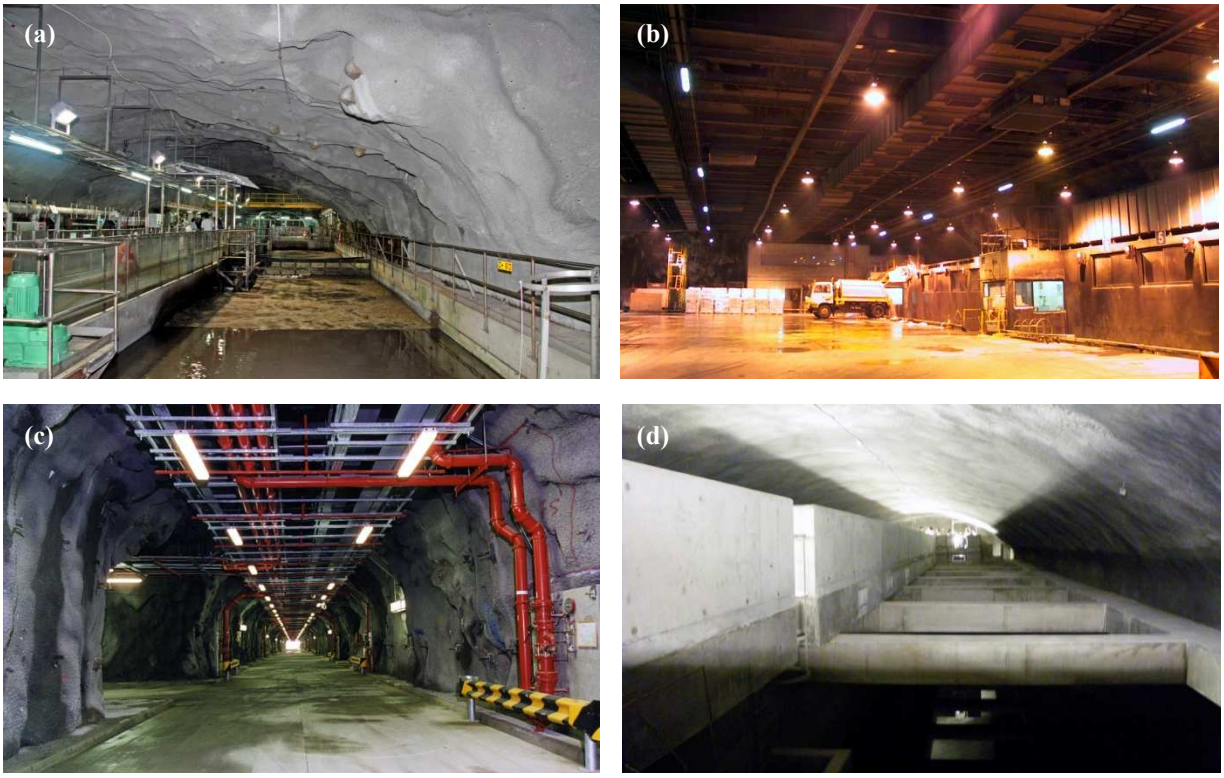

Figure 4: Examples of purpose-built caverns in Hong Kong:
(a) Stanley Sewage Treatment Works, (b) Island West Refuse Transfer Station,
(c) Kau Shat Wan Explosives Depot and (d) Western Salt Water Service Reservoirs

Figure 5: Cavern portals of (a) Stanley Sewage Treatment Works and (b) Island West Refuse Transfer Station

It is evident that rock caverns can be cost-effective, through releasing valuable surface land for other beneficial uses, and in some cases, yield additional environmental, safety and security benefits. Nevertheless, the exploitation of cavern schemes to house suitable facilities in Hong Kong has generally been limited. To date, rock cavern development is either out of necessity or done as a last resort in circumstances where suitable surface land is not available.

The feedback of Government consultative bodies and the public response to the initiative of enhancing the use of rock caverns as part of Hong Kong's pursuit of sustainable development was generally positive (CEDD 2013). However, there remains a general feeling that this type of development is more suited for NIMBY

facilities, which is likely to be due to perceived psychological impacts of going into a dingy, unpleasant underground space.

5 WHY ARE CAVERN ENVIRONMENTS CONSIDERED UNPLEASANT BY SOME PEOPLE?

Goel et al (2012) described some of the psychological and physiological considerations of working or living underground, including fear of darkness, disorientation, entrapment, negative associations to burial and death, lack of natural light and ventilation, etc. Despite the above possible concerns, the fact is that a significant proportion of the general public use underground space on a daily basis, but may be unaware of this due to the pleasant and well maintained surroundings.

Take Mass Transit Railway (MTR) as an example. Ling (2011) reported that the pedestrian subways between the Central Station and Hong Kong Station have a daily pedestrian flow of 120,000, while the subway system connecting the Tsim Sha Tsui Station and Tsim Sha Tsui East Station handles some 170,000 pedestrians daily. Around 200,000 passengers use the Tai Koo Station and Sai Wan Ho Station daily, which are wholly formed within rock caverns constructed in the 1980s. The new Admiralty and Lei Tung Stations of the South Island Line as well as the Hong Kong University and Sai Ying Pun Stations of the West Island Line, which are now under construction, will also be housed in rock caverns with a combined estimated daily pedestrian flow of over 200,000 by 2031.

In addition, Hong Kong has many notable basement developments within the urban areas that contain retail developments, vehicle parking etc., that are widely used by the general population. Goel et al (2012) noted that the windowless nature of a department store does not seem to bother people as much as with other functions due to a continual contact with people as well as constant activity. There are in fact many other facility types that are well suited to and therefore designed as a windowless environment, including museums, theatres, libraries, gymnasiums and laboratories.

Based on overseas experience, it is considered that there is significantly more potential to utilise rock cavern development in Hong Kong, for a much wider range of facilities than are currently in use. This will require efforts to alleviate the negative psychological and physiological concerns through appropriate design strategies and techniques, many of which have already been successfully employed in various underground buildings in Hong Kong. Some pragmatic measures are provision of an above-grade building or notable entrance to form a smooth and pleasant transition from the surface to underground space, use of a higher ceiling than typical one-storey height to give a sense of space, adoption of beamed daylight system or full-spectrum artificial lighting, and creation of a positive environment using warm, bright colours, plants, water features and artwork.

6 WHY IS CAVERN DEVELOPMENT NOT TAKING OFF?

The Underground Space Study has benchmarked Hong Kong's practice against worldwide practice, and has raised the issue of whether we can do more and how the Government of the HKSAR can take forward the initiative. A number of key constraints were identified for cavern development in Hong Kong (Roberts & Ng 2012) and these can be broadly grouped under two main categories, unfavourable regulatory mechanism and unfavourable financial assessment, as listed in Table 2.

Despite the promulgation of planning and technical guidelines on rock cavern development in Hong Kong in the early 1990s, there has been no systematic assessment of the potential and the benefits of rock cavern development in the land use planning process and the development of public works projects, as demonstrated by the few public facilities that have been housed in rock caverns. The Hong Kong Planning Standards and Guidelines (PlanD 2008) with regard to rock cavern development, first published in 1991, were revised in 2008 to highlight the need to identify the opportunities at the early stage of the land use planning process and to assess the cavern option at the initial project planning stage for new project proposals, as well as to set out some of the implementation issues affecting rock caverns. Despite the promulgation of the revised guidelines in 2008, there is still slow progress in the planned use of rock cavern developments.

Table 2: Constraints on rock cavern development

Unfavourable regulatory mechanism	Unfavourable financial assessment
• Policy guidelines • Planning & zoning policies • Land ownership framework • Mechanism for valuation of cavern space • Project benefits may not reflect community benefits	• Cost of land formation • Value of the land & adjacent sterilised land • Land value enhancement by relocating NIMBY facilities • Intangible cost (e.g. less complaint handling)

In Hong Kong, the owner of the surface land also owns to the sky and to the depths and has the right to develop its underground space. Underground development for public purposes can be developed underneath private lots pursuant to the relevant Ordinances (e.g. Lands Acquisition Ordinance, Land Resumption Ordinance); however, this is subject to compensation to the land owner. Conversely, privately orchestrated underground development cannot be developed beneath private lots under the ownership of others.

The development model for many of Hong Kong's projects is of a mixed-use type and the ownership of the land is often amongst multiple parties. From an administrative viewpoint, it would be difficult to organize all of the owners to consent to the underground development beneath these land plots. To avoid complicated and potentially contentious land ownership issues, it is preferable to develop caverns to house Government facilities beneath areas where there is no private ownership. In this respect, the hillsides in the urban fringes are particularly suited for cavern development.

To facilitate optimal underground development, a revised regulatory framework would be required. Components of these amendments would include the ability to create separate ownership between surface and underground developments. However, this would only be an option in case of newly disposed land and legislative amendments may be required for the interfacing between joint land owners.

It is recognised that each individual public works project by its very nature must have a public demand and public support. Whether a particular facility should be located within caverns is largely left to the individual facility providers to decide (Ling 2011). One of the key concerns amongst potential end users is the general perception that caverns are more expensive in terms of both capital and operating/maintenance costs. It is likely that in some cases operating/maintenance costs can be expected to be higher, which is mainly a result of the additional energy costs associated with lighting and ventilation for an underground facility, depending on their nature. However, most buildings in Hong Kong also require lighting and ventilation. Furthermore, temperatures within caverns generally remain stable in the range of 20°C to 25°C all year round as compared to the mean surface temperatures varying between 15°C and 31°C. Therefore, a significant reduction in heating and cooling cost can be expected for a facility placed in caverns if a uniform temperature environment is required for efficient operation.

When a new MTR station is publicly announced, this has a generally positive effect, which is often reflected by a sudden jump in property prices in the immediate area. Can it therefore be assumed that locating or removing a NIMBY facility such as a sewage treatment works or refuse transfer station to a more remote area away from the urban area would also have a positive effect on the surrounding developments? Is this something that can be quantified and included in an economic assessment as an overall community benefit that could far outweigh the higher project costs by adopting cavern development?

The present economic analysis for assessing the cost of a specific Government facility has assumed that the land is provided at no extra cost to the facility provider. This has resulted in any cost comparison between the surface and cavern option being skewed in favour of the surface option. For instance, there is a new Government facility planned for development in an area of reclaimed land adjacent to steep hillsides where the cavern option may also be considered. In comparing the costs, as cavern construction is an integral part of the works, it is normally included in the overall cost. However, the previous cost of formation of the reclaimed area prior to the project implementation is invariably not considered as part of the overall cost in the surface option resulting in an inequitable comparison.

The analysis has also not taken into account the opportunity cost of any alternative use of that specific site or the cost of any adverse impact or sterilisation on the potential use of the neighbouring sites (Ling 2011), i.e. in building the facility at that location, Hong Kong has forgone the opportunity to build other facilities to meet the community's needs, or to improve the local environment by freeing up land for other community uses.

Many of the benefits of cavern development to the community are intangible, e.g. additional open space and reduced congestion or likely reduction in complaints. For NIMBY facilities which are needed to support the needs of the community, they can be located away from the public eye if they are placed underground. Examples are the Stanley Sewage Treatment Works and Island West Refuse Transfer Station, where the daily operations have been effectively shielded from the public view to the extent that many of the local population are unaware of their existence (Figure 5). There is currently no framework to take intangible benefits into account to offset the additional costs associated with cavern development, and the overall benefit to the community is often overshadowed by the immediate public needs of a project.

Only by overcoming the constraints can the full benefits of rock cavern development be realised. Some of the benefits are itemised in Table 3. These can be related to the broader issues of land supply, land use compatibility and the environment, or be specific to the type of facility.

Table 3: Benefits of going underground

Benefits	Examples
• Enhancing land supply	• Release surface land for other uses through relocation of existing facilities and placing of new facilities
• Protection of environment	• House NIMBY facilities (e.g. sewage treatment works) away from public view
• Stable and secure conditions	• Favour goods storage, data centres, archives, etc
• Flexibility in layout and geometry	• Favour warehousing and logistics, etc
• Flexibility for future expansion	• 3-D environment can be reserved (e.g. vertically and laterally)
• Removal of incompatible land uses	• Locate NIMBY facilities away from developed areas (e.g. refuse transfer station, columbarium)
• Source of rock products	• Re-use of excavated rock materials

7 WHERE ARE WE HEADING?

To take forward the cavern initiative as a sustainable means for increasing land supply, the CEDD commenced a study on the "Long-term Strategy for Cavern Development" in September 2012 for completion by late 2015. The formulation and implementation of a long-term strategy for cavern development would provide a systematic and sustainable approach in easing the pressure of land shortage in Hong Kong, particularly in the urban areas.

The prime objective of the study is to develop a holistic approach in planning and implementing rock cavern development. The key items of the strategic study and the expected outcomes are summarised below.

7.1 Formulation of policy guidelines

Policy guidelines will be formulated to facilitate cavern development for both public and private sectors. The study will aim to develop a proactive policy for cavern development to mandate Government departments to consider the cavern option at the initial project planning stage for suitable facilities, and to facilitate private sector participation in cavern development.

An important element is to develop an appropriate method of economic analysis for comparing cavern and surface site options on an equitable basis, taking account of factors such as the value of the surface land, cost of land formation, value of excavated materials for re-use, value of sterilised land nearby, enhancement of

land value in the vicinity, opportunity cost of the surface site, cost impact on maintenance and operation, and other intangible benefits.

7.2 Preparation of Cavern Master Plans

A territory-wide Cavern Master Plan will be prepared to delineate strategic cavern areas and outline the specific areas within the strategic cavern areas for compatible land uses for future Government facilities and suitable private sector land uses. Mechanisms will also be developed for reserving these areas such that their potential for cavern development will not be compromised by future surface or underground development projects.

The plans will include cavern locations for re-housing suitable Government facilities and the potential portal locations in association with the existing transport links, corresponding protection zone, any known and planned surface and underground facilities or development projects above and within the strategic cavern areas, and suitable private sector land uses that could be considered for locating within the strategic cavern areas.

7.3 Formulation of a systematic relocation programme

A systematic programme will be developed for relocation of suitable Government facilities (25 no.) to caverns in a planned manner, thereby gradually releasing the surface land for other uses. This will take account of their status, e.g. plans for expansion or re-provisioning, so as to develop a workable schedule that meets the needs of Government departments and community expectations. Broad planning and technical assessments will establish the preliminary technical feasibility, identify key issues that need to be further considered in future detailed feasibility studies, and recommend suitable time frame for implementation.

7.4 Development of mechanisms for private sector participation

This aims to explore the demand for use of rock caverns by undertaking consultation and survey with relevant stakeholders for suitable private sector land uses, and to facilitate private sector participation. The study will place emphasis on private sector involvement as an integral part of the cavern initiative, because many private facilities, such as storage, warehousing and data centres, can benefit from a stable and secure setting offered by rock caverns. If the private sector embraces this initiative, this could significantly reduce the land take for these facilities in the future.

Apart from facilitating policy, strategy and procedures, enabling schemes adopted in some overseas countries, including direct Government funding, low land premiums, cash grant or tax incentives, as well as Public Private Partnerships, may be considered. The suitability of these enabling schemes for Hong Kong's situation will be examined through outline feasibility studies for housing a warehouse/logistics facility, data centre, columbarium, vehicle parking, bus depot and a shopping arcade in rock cavern.

7.5 Formulation of a framework for cavern ownership

The opportunity for separate ownership for surface and cavern developments for future land disposals will be explored, so as to facilitate optimal underground development. This aims to review the overseas practice and arrangements currently being used in respect of separate ownership for surface and cavern developments, review the land ownership issues with respect to underground development including relevant Ordinances, and recommend a suitable framework for Hong Kong conditions.

7.6 Review of technical issues

Several key issues that are crucial for cavern development will be considered under the study, which include:

(a) updating of Geoguide 4: Guide to Cavern Engineering which was published in 1982,
(b) reviewing the limits on blasting vibrations in Hong Kong,
(c) preparing conceptual fire safety designs and fire safety management plans for high population density facilities (columbarium and indoor sports hall in caverns),

(d) reviewing the key technical issues of housing water treatment works and service reservoirs in caverns,
(e) recommending appropriate contract forms for implementing cavern development, and
(f) carrying out a Strategic Environmental Assessment for cavern development.

Resolving these issues will help maximize the opportunities to use rock caverns for housing a wider range of facilities, some of which can take up a large area of surface land.

7.7 Public engagement and consultation with stakeholders

These will be undertaken to gauge public/stakeholders' views and foster understanding and acceptability of the policy guidelines to facilitate cavern development, the Cavern Master Plans, the systematic relocation programme for re-provisioning suitable Government facilities to rock caverns, mechanisms for private sector participation and specific issues as identified during the course of the study, e.g. housing columbaria in rock caverns. The public engagement process will include three stages, namely envisioning, public engagement and consensus building.

8 WHAT ARE THE OPPORTUNITIES?

Hong Kong's planning and engineering professions have continually risen to the challenge of the changing future development needs of Hong Kong. We have the appropriate ground conditions and certainly the required technical skill sets, as demonstrated by the wealth of underground development in Hong Kong. However, systematic consideration of the cavern option will require a change in mindset, not only by the Government but also all stakeholders. It will require the willingness to evaluate and embrace the latest technologies so as to reduce the facility footprint whilst maintaining and ideally upgrading and improving the existing services.

We need to create an environment in which the cavern development option is not overlooked or treated as unconventional. The planning and execution of cavern development need to become part of the mainstream development process to help create the optimal living environment. Apart from NIMBY facilities, opportunities for housing a wide variety of suitable Government and private sector facilities in rock caverns are plentiful and evident elsewhere (Figure 6). Rock caverns could become a part of Hong Kong's sustainable planning and development strategy.

9 CONCLUSIONS

In 2011, the study on "Enhanced Use of Underground Space in Hong Kong" concluded that the geology and topographical setting of Hong Kong are very favourable for cavern development. Members of the public are generally supportive of the initiative of enhancing the use of rock caverns as part of Hong Kong's pursuit of sustainable development.

By reprovisioning suitable Government facilities inside caverns and releasing the occupied land as well as any adjacent sterilized land for housing and other uses, cavern development is a viable option to expand land resources. Cavern development could also be a good source of land supply for accommodating new Government and public sector facilities which will otherwise occupy surface land. Reserving underground space could accommodate future projects and expansion of existing underground facilities. For those NIMBY facilities like sewage treatment works and refuse transfer stations, the cavern option may help to reduce adverse impacts on the local environment, remove incompatible land uses and alleviate the NIMBY sentiment.

Conservative estimates suggest that some 3,500 ha of land could be made available by cavern development at the urban fringes. Given the potential for multi-layer cavern development, a substantially greater usable area could be created in underground space.

The study on "Long-term Strategy for Cavern Development", commenced in September 2012 for completion by late 2015, will formulate policy guidelines to facilitate cavern development for both private and public sectors. By adopting a holistic approach in planning and implementation, this should allow rock cavern development to become a sustainable means for enhancing land supply.

Hong Kong has a wealth of such "hidden land" that can provide almost "unlimited" space for future development. We should treasure this precious asset and exploit it in a coordinated and proficient manner in order to sustain the growth of our city. Let us team up together to work diligently for a better future for Hong Kong.

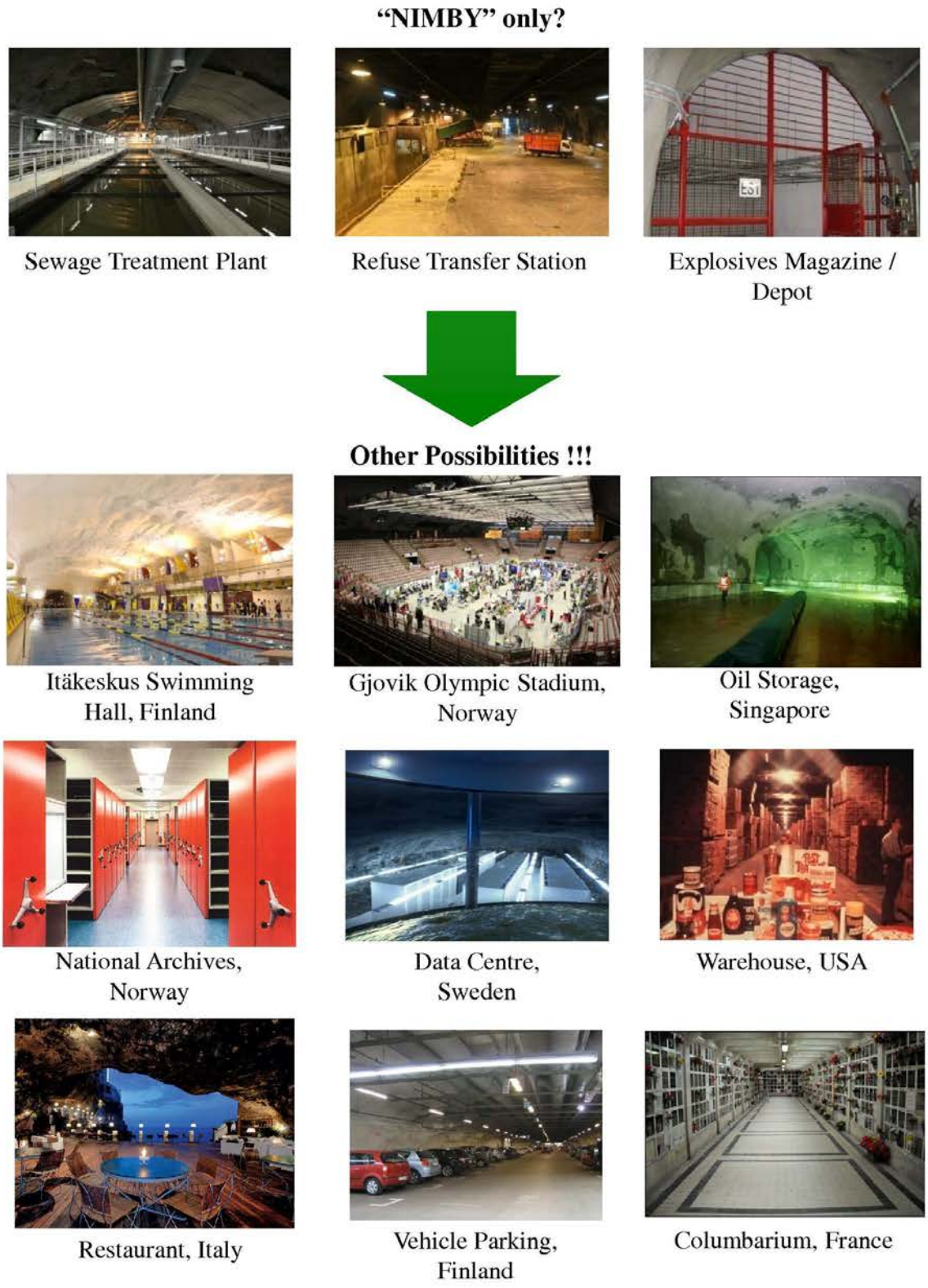

Figure 6: Opportunities for rock cavern development

ACKNOWLEDGEMENTS

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Geotechnical Aspects of Public Housing Development

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ABSTRACT

This paper briefly reviews the supply of land to the Hong Kong Housing Authority (HA) for public housing development and the input provided by geotechnical engineers in the Housing Department (HD), HA's executive arm, at various stages of the development programme. Drawing on past experience, the author considers the greatest contributions that geotechnical engineers can make in the delivery of public housing come from taking appropriate measures to mitigate geotechnical risks and creating opportunities for sustainable development.

1 INTRODUCTION

安得廣廈千萬間
大庇天下寒士俱歡顏
風雨不動安如山
杜甫（公元712-770年）《茅屋為秋風所破歌》

These verses quoted from a Chinese poem vividly portrait a puzzle that bothers many people throughout history, a dream for all ages:

How can we build houses in tens of thousands
Pleasant shelters for the poor in numbers by the millions
Unshaken by storms and remain stable as mountains
Dù Fǔ (712-770 AD) "Lament for a shanty damaged in the autumn wind"

1.1 1954-1972

Decent living space is at a premium. To provide public housing to the needy, the necessary land must be made available first and serviced land is a scarce resource in Hong Kong. In the immediate years after World War II, massive immigration from mainland China had resulted in severe overcrowding in private tenements and widespread squatting on the city outskirts mostly in foothill areas.

The first period in the development of public housing began shortly after a disastrous fire broke out in Shek Kip Mei on Christmas Day 1953 that left some 53 000 people homeless. Clearance and resettlement of squatters to release land for development was the main thrust behind the provision of public housing. In

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addition the Government subsidized the Hong Kong Housing Society, a voluntary organization, to develop rental estates and set up the former semi-independent Housing Authority in 1954 to develop low-cost housing. But these efforts were uncoordinated. By 1973 when the resettlement programme was terminated, a total of 234 059 units had been built to accommodate over one million inhabitants (Yeung 2003).

1.2 1973-1982

In 1972, the then Governor of Hong Kong, Sir Murray MacLehose announced the Ten-year Housing Programme to provide decent housing for 1.8 million people between 1973 and 1982. Associated with this initiative, a new Housing Authority (HA) was formed by way of the Housing Ordinance (Chapter 283, 1973) and the New Town Development Programme was launched. An adequate supply of serviced land with supporting infrastructure is of paramount importance in developing new towns. And so, excavation into hillsides, reclamation from the sea, and filling of low-lying areas were employed in forming new land for development. (Chow 1997).

Tsuen Wan, Sha Tin and Tuen Mun were the first three areas chosen for development of new towns. In the late 1970s and early 1980s, public housing estates like Fuk Loi, Lek Yuen, Wo Che, Tai Hing and Yau Oi were built. According to the ambitious Housing Programme, it would entail an annual production of 35 000 to 40 000 units. However for various reasons, the target was not met and only 220 527 units were built to house only one million people (Yeung 2003).

1.3 1983 and after

When the Ten-year Housing Programme concluded in 1982, there was no public announcement of new housing targets or new objectives. But the new towns continued to develop. In the 1980s, Tai Po, Yuen Long, Fanling and Sheung Shui emerged as the second-generation new towns. In the late 1980s and early 1990s, three more new towns started to take shape, namely Tin Shui Wai, Tseung Kwan O and Tung Chung. Stepping into the new millennium, 48.7% of the overall population in Hong Kong was accommodated in public rental units or subsidized sale flats. On a territory basis, the then public housing population was 62.8% in the New Towns and 37.3% in the urban area (Yeh 2003).

In 1987 the Government released the Long Term Housing Strategy (LTHS). The plan to year 2001 called for 960 000 new housing units to be built. At this level of demand, the public and private sectors would have to produce respectively 40 000 and 30 000 units every year. HA (1989) commented that: “At the end of March 1989 the projected shortfall in flat supply to 2001 was over 90 000 which, assuming an average building density of 600 flats per hectare, will require 150 hectares of additional land…”

In response to public demand, the Government published a consultative document in January 1997 and a White Paper the year after to review and update the LTHS (HB 1998). Among other initiatives, the White Paper put forth the following three targets:

- to build not less than 85 000 public and private housing units a year
- to achieve a home-ownership rate of 70% by 2007
- to reduce the average waiting time for public rental housing to 3 years by 2005

However, due to the unforeseen decline in Hong Kong’s economy following the Asian financial crisis, the Government subsequently gave up the first two targets. Indeed since then, there has been a drop in the percentage of Hong Kong’s population which is accommodated in public rental units and subsidized sale flats (Table 1). This indicates that the rate of public housing production, and by inference the supply of land, does not match with the growth in population.

Table 1: Public Housing Population from 1976 to 2011

Year	1976	1981	1986	1991	1996	2001	2006	2011
HK’s Total Population (millions)	4.40	5.11	5.50	5.67	6.41	6.71	6.86	7.07
Public Housing Population (%)	39.6	39.1	45.8	46.7	49.9	48.7	48.8	46.6

Sources of data: Yeh (2003), CSD (2007, 2012a,b)

Under the circumstances, one could ask: Is the dream of poet Dù Fǔ so remote to be without hope? Indeed the quotes from his poem deserve the whole community to seriously ponder.

1.4 Site formation by the Housing Department

It has been a long standing arrangement that the Government will provide HA with serviced land ready for HD to immediately start building works. However, the amount of serviced land allocated is not steady and this caused fluctuations in public housing production.

In a number of cases, HD had to carry out site formation works to turn unserviced land into sites with infrastructure suitable for housing development. That included prior actions on rezoning and clearance. Some of the notable examples are the site formation works at Ap Lei Chau in the early 1980s (Plate 1), Shaukeiwan Foothill East and West Sites in late 1980s (Plate 2), and Tiu Keng Leng in late 1990s (Plate 3). These site formation works involved extensive excavation in mixed ground conditions. The excavated materials were gainfully used to reclaim land elsewhere or reserved for specific use.

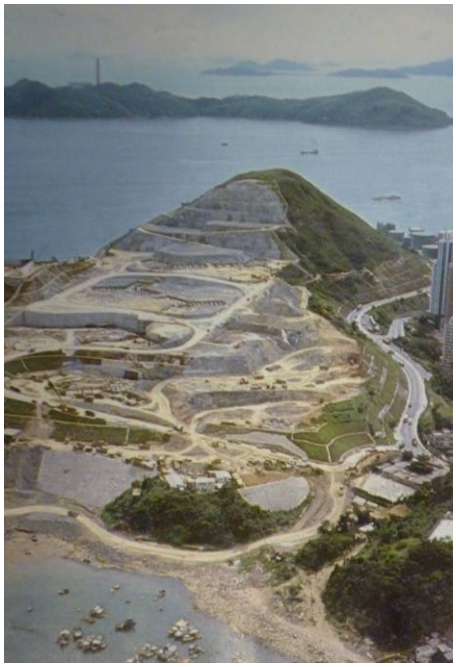
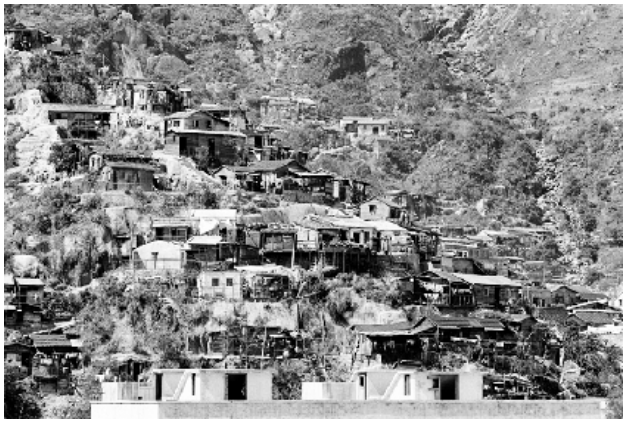


Plate 1: Ap Lei Chau Site Formation. Lei Tung Estate and Yue On Court are now located on this site. The photograph was taken around 1982. Beggs & McNicholl (1986) presented an account of the formation of high rock slope.



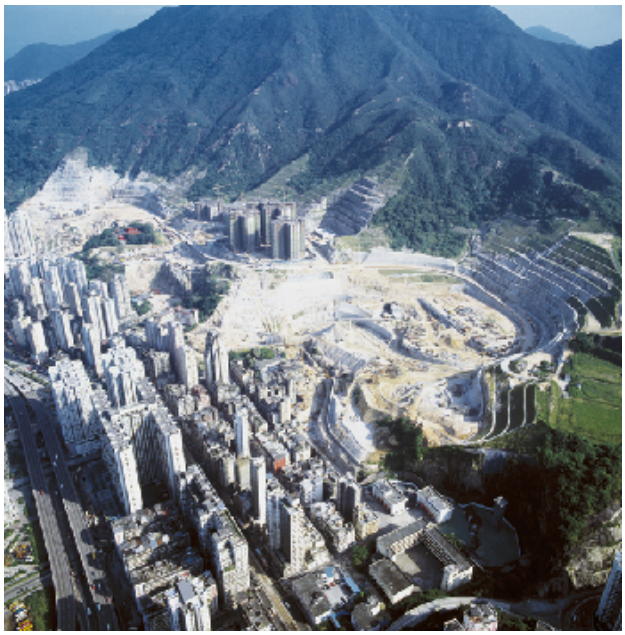


Plate 2: Shaukeiwan Foothill Development. (*Left*) Large extent of squatters inhabited the hillside prior to clearance. (*Bottom Left*) Site formation works in progress. New blocks were being built on the East site on the far side. (*Bottom Right*) Yiu Tung Estate and Tung Chun Court on the East site had been completed. The photographs were taken respectively around 1962, 1992 and 1994. Cheung et al (1995) discussed aspects of the site formation contracts.

Plate 3: Tiu Keng Leng Development. (*Top*) Cottages spotted over the hillside on the left while Junk Bay was being reclaimed. (*Bottom left*) Tiu Keng Leng Cottage Area was cleared and site formation was in progress. (*Bottom right*) View of Kin Ming Estate built over the formed land of Tiu Keng Leng. The photographs were respectively taken around 1995, 1999, 2004.

1.5 Geotechnical and building control

The three projects cited above involved the formation of large slopes. Under an administrative arrangement between HD and the Geotechnical Engineering Office (GEO), design of slopes and related matters would be submitted to GEO for geotechnical checking. This has become a standard practice since 1980, if not earlier, for all projects involving formation of new slopes or review of the stability and upgrading of existing slopes.

In 2000, HD established an Independent Checking Unit (ICU) to exercise building control on building works. There is no exception to works handled by geotechnical engineers in HD. The control system parallels that exercised by the Buildings Department (BD) on private building development. A similar system for approval of plans, consent to commence works, compliance with site supervision requirements and certification of completion is implemented. In terms of in-house projects, HD's Chief Architect, Chief Structural Engineer and Chief Geotechnical Engineer respectively discharge roles and responsibilities similar to those of the Authorized Person (AP), Registered Structural Engineer (RSE) and Registered Geotechnical Engineer (RGE). The division of such roles and responsibilities is set out in BD's Practice Note for AP, RSE & RGE APP-141. Geotechnical engineers have executive role in ground investigation, site formation and remedial works to dangerous hillsides, and have advisory role in other types of building works that require geotechnical input to support submissions by the architects or structural engineers.

2 PLANNING, DESIGN AND DELIVERY OF PUBLIC HOUSING

It is a long and challenging process to turn a vacant piece of serviced land into homes for the residents. It takes at least six years from searching for developable land to delivering habitable units. The estates, after their completion, will be looked after with life-long management and maintenance (HA 2011).

The public housing development process comprises the following stages –

- Site inception and acceptance
- Feasibility study and conceptual layout
- Design and tender
- Construction
- Post-construction maintenance

By looking at the content of each stage, one would appreciate that geotechnical engineers are not only required to be technically competent, but also have to possess a concern for mitigating geotechnical risks and maximizing opportunities for sustainable developments.

2.1 Site inception and acceptance

When a site is considered for possible public housing development, a site potential study is carried out. This involves participation of various professional disciplines as a team. They will identify the constraints so as to ensure there is no insurmountable problem that hinders public housing development.

To the geotechnical engineers, this is a stage for collating available geotechnical information from different sources and conducting desk study and site reconnaissance. Sometimes information is very limited and geotechnical engineers will have to take calculated risks and consider possible mitigation measures when making site selection decisions.

At this end of this stage when it is satisfied that the site has the potential, the site will be earmarked for public housing development and included into the development programme.

2.2 Feasibility study and conceptual layout

Functional and technical feasibility studies are conducted in this stage. Apart from site constraints, development opportunities are also reviewed. This will help establish a set of development parameters for preparation of a conceptual layout and a project cost ceiling. For complicated cases, value management studies and community engagement workshops will be arranged to determine the optimal development option.

Where possible, geotechnical engineers will arrange a preliminary ground investigation with the intention of obtaining a general picture of the geology and groundwater conditions of the site. Sometimes access could be restricted by existing land use. Chui (2003) gave an account of the difficulties of conducting ground investigation in terrain covered by closely-packed cottages. Geotechnical engineers must plan well in order to maximize the retrieval of valuable information from the preliminary ground investigation.

2.3 Design and tender

The conceptual layout is then developed into a scheme design proposal with themes that meet the design objectives and fulfill client requirements. After approval of the scheme design by HA's Building Committee, the General Building Plan is prepared together with the supporting geotechnical assessment for submission to the ICU. The scheme design is also further developed into a detailed design.

Geotechnical engineers will conduct detailed ground investigation for the design of site formation, foundation and other building works with significant geotechnical content. Where they have an executive role, they will ensure sufficient contingency is built into the design and contract provision for risk mitigation and environmental enhancements.

2.4 Construction

Once the contractor is procured, a contract team is set up to administer the contract and supervise the works. Geotechnical engineers have a role to play as Technically Competent Persons under the RGE stream. One of

their major duties is to review the as-encountered ground conditions and field monitoring data. Where necessary, geotechnical engineers will initiate appropriate actions such as modifying the design.

2.5 Post-construction maintenance

Due to the hilly terrain of Hong Kong, often there are formed slopes in public housing estates. In the past, slopes are seen as a burden. However in the past two decades, a design philosophy has emerged within HD that takes slopes as part of the estate open space and part of the built environment for enjoyment by the residents. Where opportunity arises, for example when an old estate is due for redevelopment, the site will be modified following this philosophy.

3 MITIGATION OF GEOTECHNICAL RISKS

Mitigation of geotechnical risks is one of the major considerations in implementing site formation works. It calls for geotechnical engineers' attention as early as possible in the project planning stage (Leung et al 1997; Roberds et al 1999). Some of the typical risks arise from the variability and uncertainty in the ground and groundwater conditions, threats of boulder falls and other natural terrain landslide hazards.

The Shaukeiwan Foothill Development cited earlier presents a good example. At the feasibility study stage, the author and his project geotechnical engineer were tasked with proposing a site formation plan. It was intended that the plan should form the basis of securing land and planning approval from the Government, as well as the conceptual layout for subsequent preparation of the scheme design.

3.1 Look far and wide

In order to perceive the possible risks, we have to look far and wide. Plate 4 shows a surface geology and land instability map prepared at the time by an engineering geologist in the team. The need to prepare such a map at the outset seems obvious nowadays, but in those days, it required a lot of persuasion to convince non-geotechnical personnel to adopt a broader view instead of just focusing on the site itself. The map highlighted the surface hazards that might affect this foothill site. As a result of such an assessment, several gabion barriers were constructed intercepting the possible paths of boulder falls and debris flows.

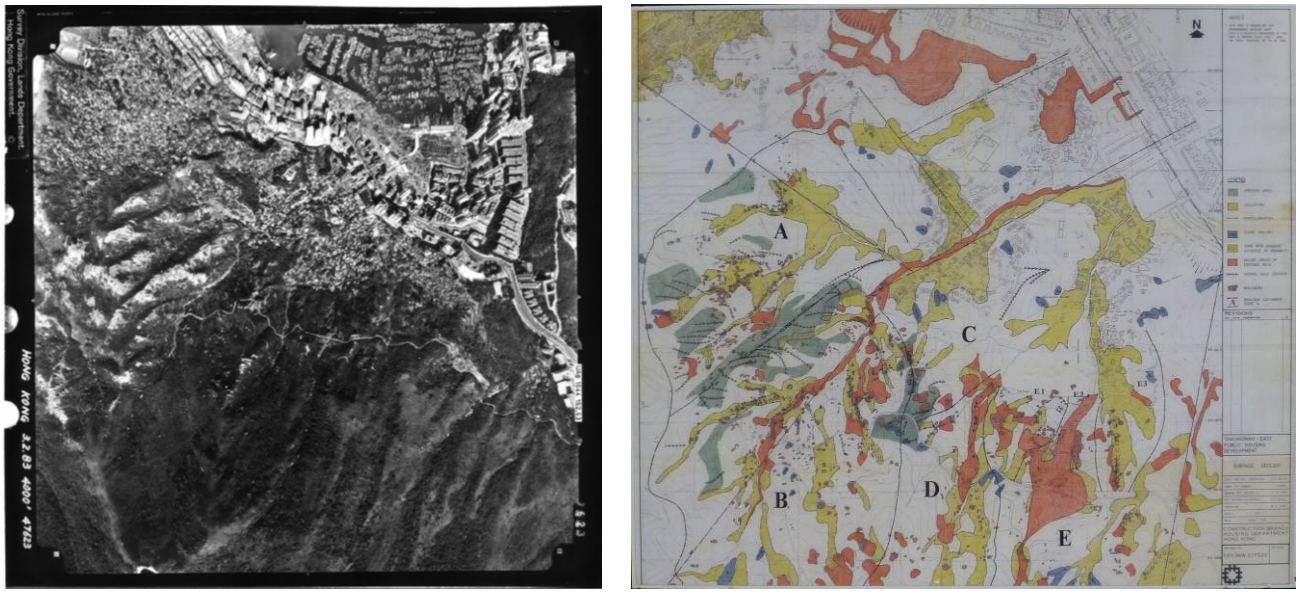


Plate 4: Shaukeiwan Foothill Development. Aerial photographs (*left*) and surface geology and land instability map (*right*) that covered a broad area had assisted in the planning of the public housing development and the associated site formation works.

In the study, a catchwater running above the site was identified. It would collect rain water from a series of catchments. Any overflow from a blocked catchwater might discharge a lot more water than that would be

expected from a single catchment. There were also risks associated with the abandoned quarries. All such risks were taken into account in the design of the site formation works.

The catchwater above Shaukeiwan Foothill Development brings to mind a sad incident that occurred at Mei Chung Court in Shatin on July 2, 1997, the day after Hong Kong was returned to China. Severe flooding occurred at a semi-sunken car park with serious erosion on the peripheral slopes (Plate 5). Heavy downpour since the end of June had caused a substantial rise in water level in a stream which belonged to another water catchment area. The large volume of water brought with it cobbles and boulders, leaves and tree branches. The materials blocked the stream course under a footbridge about a hundred metres away from the estate and were trapped by a wire fence along the footbridge. This formed a barrier to divert the entire stream onto a footpath at the back of the estate causing the damage.

Plate 5: Flood damage at Mei Chung Court on 2 July 1997. Large volume of water in a nearby stream was blocked by debris at a footbridge and was forced to divert to flow along a footpath (*left*) and then down a slope causing serious erosion (*centre and right*).

3.2 Prepare for the least expected

The flood damage at Mei Chung Court was something least expected, if not unexpected. In hindsight, if a concrete upstand of sufficient height had been provided when the boundary fence was built, the flood damage might have been avoided. No doubt, this would require some sort of imagination and foresight. Geotechnical engineers should always consider possible risks and build in contingencies.

Cutting into foothill areas to create building platforms will invariably result in large peripheral slopes. Despite these slopes are designed to current slope safety standards using deterministic methods, it is advisable from a risk management point of view to reserve an area between the building blocks and the slopes as buffer zone. This will help reduce the consequence of any unlikely event of slope instability. Such a layout planning concept has evolved since the early 1980s when the first batch of geotechnical engineers were appointed by HD. Plate 6 shows an example using this concept.

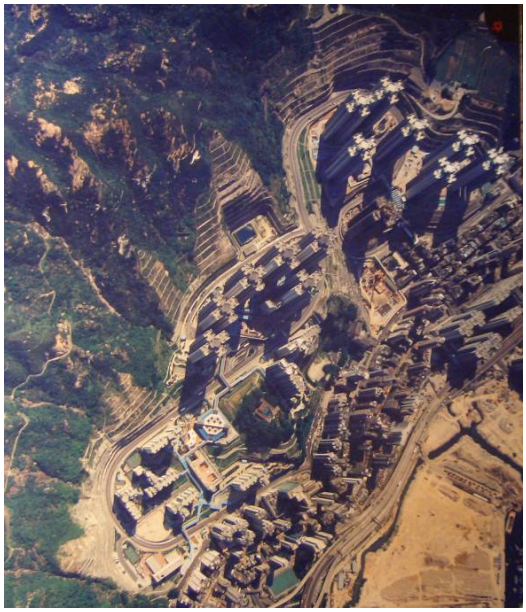


Plate 6: (*left*) Shaukeiwan Foothill Development. The road system forms a buffer between the slopes and the blocks.

Plate 7: (*right*) Lam Tin South Development. A raised footpath on walls forms a drainage ditch cum debris barrier.

In Lam Tin South, where Tak Tin and Kwong Tin Estates are located, a drainage ditch cum debris barrier was adopted instead. The platform above the barrier is used as a footpath (Plate 7). Such an arrangement of combining the functions of a drainage ditch and a boulder/debris trap has been used in some other projects, such as the recently completed Chai Wan Estate.

Ground is variable. This is a result of geological and weathering processes. For example, corestones, differential weathering and soft seams are features well known to geotechnical engineers. But variable ground conditions can also be caused by human activities. Indeed the disturbance can be beyond imagination. Large body of trash can raise a concern no less than loose fill (Plate 8). Too often inadequate site investigation is given as an excuse for unforeseen ground conditions. In fact lack of insight and inadequate interpretation of ground information can also result in potential risks being overlooked.

Plate 8: (*Left*) Trash fill at Tiu Keng Leng Cottage Area found during site formation. (*Right*) An interesting mix of fill material, including abandoned domestic furniture and equipment, were used to form a squatter platform.

Geotechnical engineers understand that the presence of joints and discontinuities can influence the stability of rock slopes. Stereoplots are used as a technique to analyse whether the prominent joint sets can produce kinematically admissible modes of failure. An analysis concluding that a rock slope is stable may provide a false sense of comfort because impact of external disturbance, for example effects of blasting, is not taken into account (Plate 9). In one incident, the effect of the blast was so great that it caused failure of a large wedge. It therefore requires foresight and preparedness to manage the risks of the least expected events.

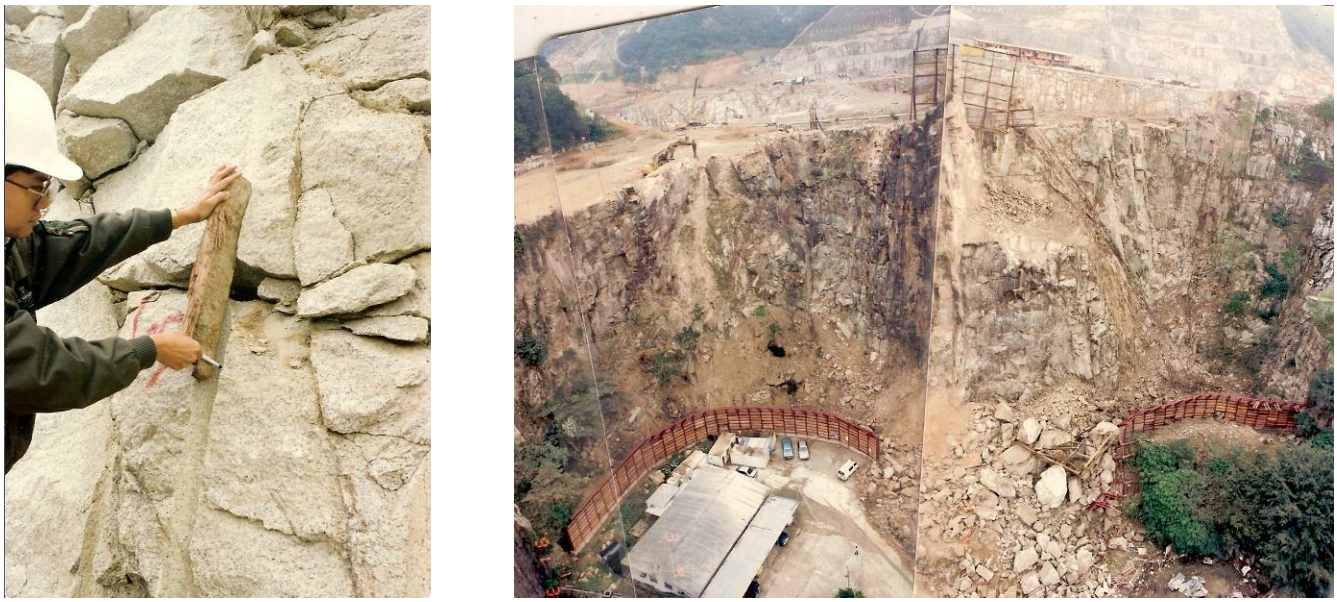


Plate 9: Blasting effect can cause displacement of blocks of rock (*left*) and even failure of rock wedge (*right*).

3.3 Observe and review

Rock slope stability assessment is a typical example requiring observation and review. Another example is groundwater regime.

It is well-known that groundwater conditions can affect slope stability. Proper assessment of these conditions is therefore crucial for the satisfactory implementation of a major site formation. Insley et al (1982), and McNicholl et al (1985) have reported HD's experience in surveillance of pore water conditions in large urban slopes. However pre-design monitoring of groundwater levels does not necessarily give a full picture. The designer has to take a wider geohydrological perspective that covers areas beyond the site. Instrumented field trials of rainstorm simulations indicated that the upper catchments might have a greater effect on the groundwater conditions than the local impervious surface protection (Li et al 1995, Plate 10).

Plate 10: Rainstorm simulation trial on the northern slopes of Tsz Wan Shan

Observational method has been advocated by many geotechnical practitioners. It involves an estimation of the worst scenario and provision for contingency measures in addition to designing for the most probable situation. The method is also suitable for dealing with geotechnical works affected by groundwater conditions. Leung (2003, 2007) reported an encounter with natural erosion pipes during the excavation into cobblery colluvium at the foothill of Kowloon Peak. Uphill of the site there was a large squatter settlement.

At the design stage, the measured groundwater level was low. Nonetheless the possible formation of perched water tables was recognized by the designer and contingency provisions were made in the contract for a liberal application of horizontal drains and erosion protection mats. There was also a provision to construct masonry protection to the slope surface in case it was required by the actual ground conditions encountered during construction. The as-encountered conditions turned out to be worse and the contingency measures were barely inadequate (Plate 11).

Plate 11: Ngau Chi Wan Street Site Formation. Extensive seepage and discharge of large amount of groundwater by horizontal drains

A review was promptly carried out. The large squatter settlement uphill was about to be cleared. The inhabitants had been tapping groundwater for their daily consumption. Clearance of the squatters meant that the canopy cover to reduce uphill infiltration would be lost and the artificial lowering of groundwater table would no longer exist. The discovery of several natural erosion pipes added complexity to the problem (Plate 12). With much effort and a combination of subsurface drainage measures, the problem was finally resolved. Sixteen months after completion, the slope and the drainage measures stood the test of a severe rainstorm with a return period of 1 in 5 years. For over 15 years, regular inspections of the slope have been carried out. Groundwater levels and flows in the horizontal drains are also monitored. No further problems have been encountered since then. The sub-surface drainage measures prove to be performing satisfactorily over the years.

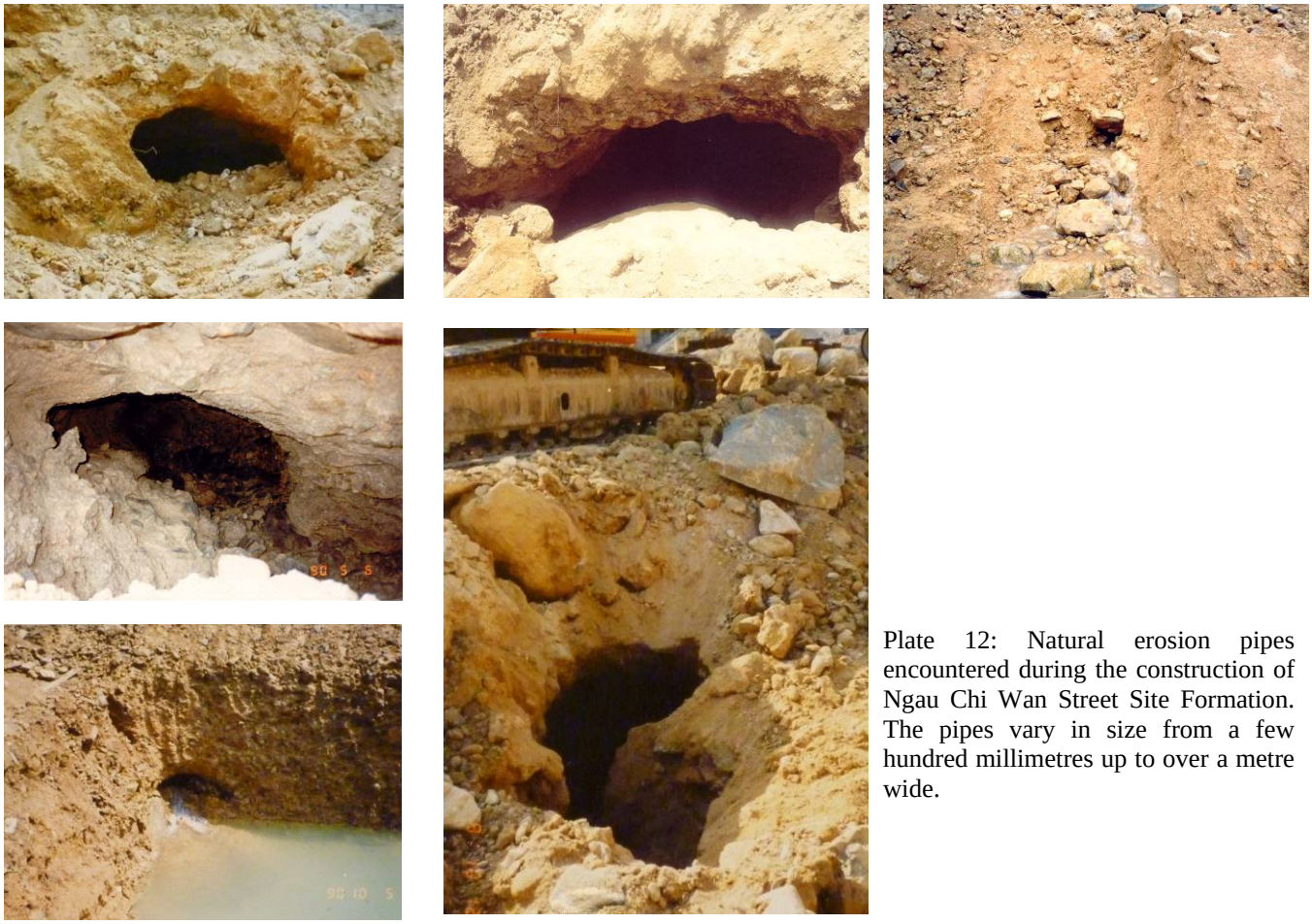


Plate 12: Natural erosion pipes encountered during the construction of Ngau Chi Wan Street Site Formation. The pipes vary in size from a few hundred millimetres up to over a metre wide.

4 OPPORTUNITIES FOR ENVIRONMENTAL ENHANCEMENTS

Apart from forming new land, HA also has a Comprehensive Redevelopment Programme (CRP) to redevelop old public housing estates such that land can be put into more effective use in a socially, economically and environmentally sustainable way. Many of these old estates contain slopes. The redevelopment provides an opportunity to upgrade sub-standard slopes as well as to improve the estate physical and social environment. On top of technical considerations, this would call for a caring and people-oriented attitude from geotechnical engineers in engineering the redevelopment works.

4.1 Think big

Slopes take up useful space. In public housing developments, such space is not permitted to count towards the plot ratio calculations. Therefore a philosophy has emerged among geotechnical engineers in HD that recognizes slopes should become part of the amenity space for use by the residents, physically or visually. Treatment of such amenity space is conceived as a natural and sustainable component of the built environment. Opportunities have been taken to create multi-purpose functions for these spaces that meet the residents' needs. Leung (2005, 2010) has presented cases of how slopes have been incorporated into the estate setting for the enjoyment of the residents.

Four pillars underpin the current concept of upgrading or modifying existing slopes during redevelopment –

- slope safety
- aesthetics of the landscape treatment
- integration with the estate layout
- conservation of the environment

The first two pillars are obvious to all geotechnical engineers and GEO have provided standards for good practice for reference. However the other two pillars are less obvious but is getting mature through practice in the last 10 to 15 years within HD. They would require geotechnical engineers to think big and collaborate with their counterparts as a team. Two award-winning cases are presented below as an illustration (HKHA 2011).

4.2 Upper Ngau Tau Kok Estate

The old Upper Ngau Tau Kok Estate was built as a low-cost housing estate in 1967/68. HA embarked on the Phase 1 redevelopment of the estate east of On Tak Road in 1997 with completion in 2002 as the reception blocks for the remaining part of the estate. The remaining phases of the redevelopment were merged together for commencement in 2002 and completion in 2009 as a reception estate for the redevelopment of Lower Ngau Kok which is the last project in the CRP. Therefore HD had a very clear idea of the future tenants of this final part of Upper Ngau Tau Kok Redevelopment.

The site is bounded by Ngau Tau Kok Road to the west, On Tak Road to the east and On Shin Road to the south. The original development comprised linear blocks located on a series of terraces (Plate 13). Staged ground investigations were carried out around the blocks before they were vacated. Obviously the terraces would not be large enough to accommodate the standard block types normally used by HD without substantial modifications.

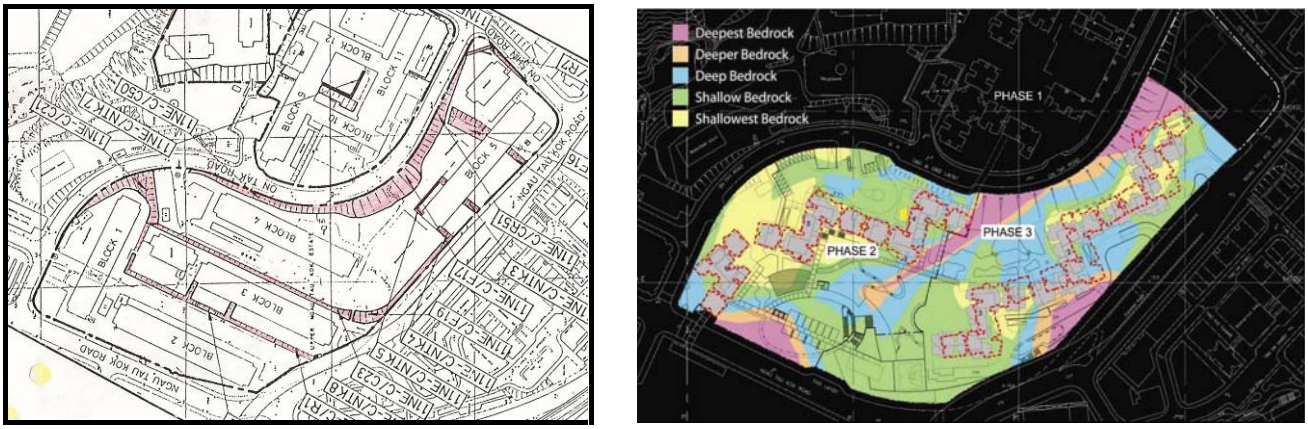


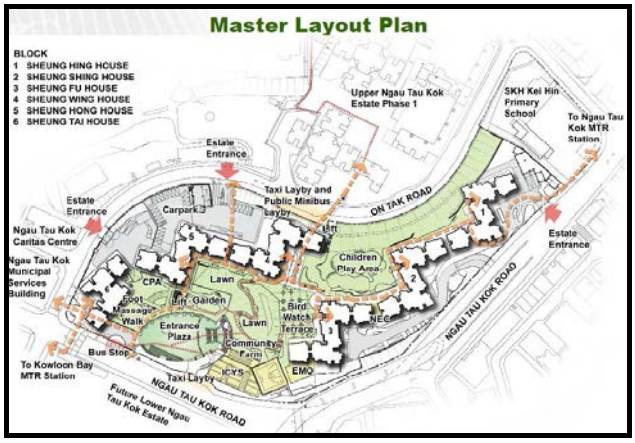
Figure 13: (*Left*) Layout of the old Upper Ngau Tau Kok Estate. (*Right*) The redevelopment layout showing the relative depths of founding stratum. The blocks are located in areas with shallow bedrock.

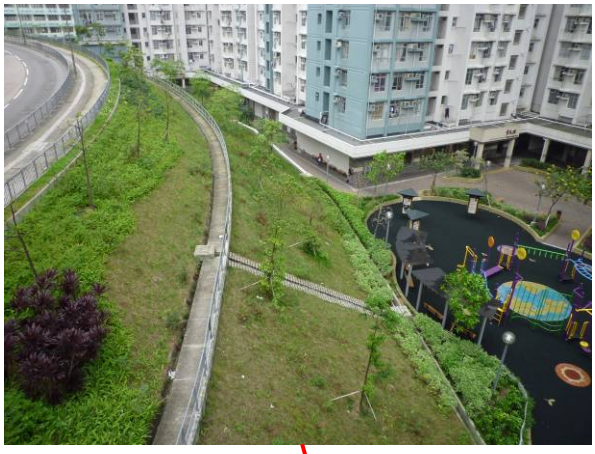
During the feasibility stage, the project team had conducted various studies including micro-climate studies. On the basis of these technical studies, a Z-shaped layout utilizing non-standard block design was proposed that would –

- locate the blocks on shallow bedrock thereby reducing the amount of excavation and spoil disposal
- maximize natural light in common areas and hence reduce the intensity of artificial lighting
- maximize the use of natural shade and tree planting to create a cooler environment
- improve wind flow and ventilation and reduce solar heat gain.

On the human side, a people-oriented approach was adopted. Since the target tenant group and stakeholders had already been identified, a series of community engagement activities were conducted. These included opinion gathering, briefing sessions and workshops and the feedbacks received were incorporated in the design. Special provisions are made to meet the needs of the disabled and elderly. Indeed 21% of the estate population is elderly with an average age over 65.

As far as the existing slopes were concerned, the backslopes were retained, improved and vegetated. The inter-terrace slopes were completely regraded into a sloping courtyard with greeneries, footpaths, exercise stations and sitting out areas. It forms a gently rising centrepiece of the whole estate that connects the entrance plaza to the upper platform and leads to the backslopes. It is also a wind corridor and a view corridor. The site area is 3.5 hectares with 30% overall greenery ratio, half of which is green slopes (Plate 14).





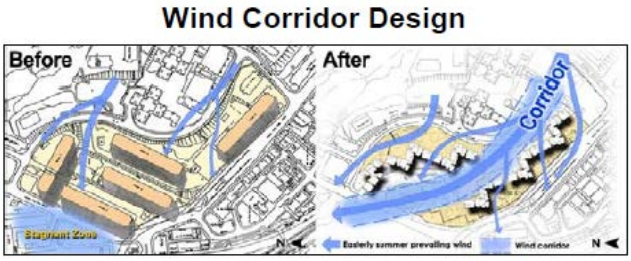


Plate 14: Upper Ngau Tau Kok Estate. From the entrance plaza, the site gradually rises to the upper platform and leads to the backslopes. It is a wind corridor as well as a view corridor.

4.3 Sau Mau Ping South Estate

Sau Mau Ping South Estate is also completed in 2009. The site is 4.9 hectares in area with a single building platform bounded on four sides by slopes. Compared with other public housing estates, it has the highest greenery ratio of 43%. Half of the greenery is at-grade or on roof-top, the remaining half comes from greening of slopes and, to a much lesser extent, vertical noise barriers.

From the geotechnical assessment carried out before redevelopment at the feasibility study stage, all the slopes were found to be in need of upgrading works in order to comply with current safety standard. The highest 17m high loose fill slope below Sau Mau Ping Road was covered by sprayed concrete while other

loose fill slopes were covered with mature trees. An early decision was therefore made to preserve the trees and replaced the hard surfacing with vegetation.

For the loose fill slopes with trees, the solution was less straightforward. While guidance document is available for stabilizing loose fill by soil nailing, it was difficult not to cut some of the trees given that a concrete grillage covering 50% of the slope surface had to be provided. A survey of the tree locations was conducted, layout of the grillage beams was carefully planned, standard construction details were modified, and eventually it was able to preserve 114 trees and transplant another 2 trees out of a total of 208 at a preservation rate of 58% (Plate 15). This compares favourably with the trees grown on the building platform – 15 trees were preserved and 40 trees transplanted out of a total of 83 trees at a preservation rate of 66%.

Plate 15: Sau Mau Ping South Estate. (*Left*) Condition of loose fill slope with trees before upgrading works. (*Centre*) Slope upgraded with soil nails and concrete grillage. (*Right*) Slope condition about a year after.

For the 17 m high fill slope with an area around 7 000 m^2, a conventional solution was adopted – to build a retaining wall at the toe and recompact the fill to a flatter angle and higher density. However to think big, the project team had to address not only the first two pillars of slope upgrading mentioned earlier but also the other two pillars. In response, the project team worked out an ecological master plan with a view to regenerating a natural habitat that would blend in with the environment. A total of some 4 100 indigenous trees and shrubs of 36 different species were planted on the slope. The biodiversity provides a sustainable greenery that would engender good habitats for insects, birds and other kinds of animal life (Plate 16). It provides a long term vision that public housing is no longer simply the provision of shelter and home to people but also establishing a balanced ecosystem that links human settlement to the environment (Plate 17).

Plate 16: Sau Mau Ping South Estate. (*Left*) Original condition of the 17 m high fill slope before upgrading works. (*Right*) The slope was reconstructed with the surface turfed and covered with newly planted shrubs and trees.

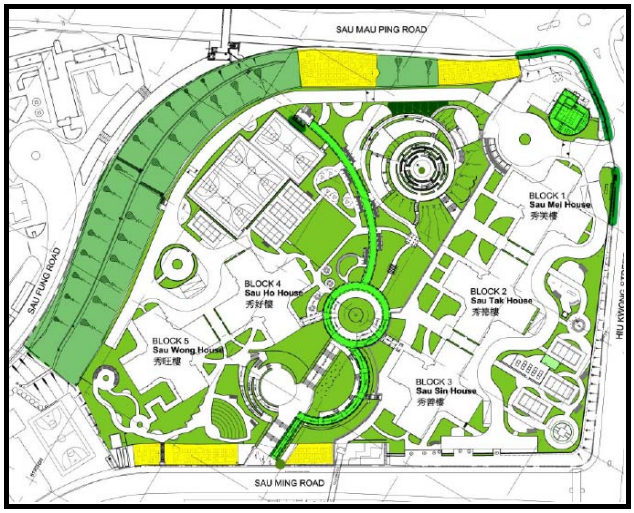


Plate 17: Sau Mau Ping South Estate. (*Top Left*) The master greening plan. (*Top Right*) The 17 m high fill slope has become an eco-garden. (*Bottom left*) The estate by night.

5 CONCLUSIONS

Going back to the quotes from Dù Fǔ at the beginning of the paper, one may ask how much have been achieved in the past 60 years of public housing development.

As regards quantity, some 3.3 millions of Hong Kong's population are accommodated in public rental units or subsidized sale flats. The demand for public housing is still very high. There is still a long waiting list of applicants hoping for being allocated a public housing unit. New ways have to be found in order to form or release land for public housing.

As regards quality, HD has moved a long way to provide a pleasant environment to the residents. In this, the geotechnical engineers have a part to play. It is the author's view that the greatest contribution geotechnical engineers can make is in taking appropriate measures to mitigate geotechnical risks and creating opportunities for environment enhancement when dealing with slopes.

ACKNOWLEDGEMENTS

The author is grateful to the Deputy Director of Housing/ Development & Construction for her permission to publish this paper and to his colleagues in HD who had worked with him over the years. Some of them, like him, may have already retired and others are continuing to devote themselves in public housing development. The author has learnt much from their work and experience.

Special tribute is given to his colleagues in the Geotechnical Engineering Section (GES) particularly in this year they will be celebrating the 20th anniversary of the Section.

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Redevelopment in Dense Urban Areas – Experience and Projects by Urban Renewal Authority

Calvin C.L. LAM
Urban Renewal Authority, Government of the Hong Kong SAR

ABSTRACT

Hong Kong has the densest population in the world. Whilst small in area, most Hong Kong people prefer to live close to the central business districts in the urban areas– Hong Kong north shore (Central, Wanchai, Causeway Bay) and Kowloon peninsular (Yau Tsim Mong, Kowloon City, Shamshuipo and Kowloon East). Demand for housing in these areas is ever increasing. As all these areas have been densely developed long ago, the only way to create new housing supply there would be by redevelopment or reclamation. The Urban Renewal Authority is the statutory organization tasked to improve the living conditions in such densely populated and old urban areas by redevelopment and rehabilitation. Redevelopment in such densely developed areas with adjacent buildings (often old and dilapidated) and existing buzzing business which must be kept in operation with minimum disturbance, creates many geotechnical challenges. The URA and its consultants have formulated some innovative and practical solutions to tackle these issues. In my presentation I would use URA projects (completed or being implemented) as examples to illustrate the solutions chosen for basement, foundation and other geotechnical aspects.

Controlled Surface Blasting Works for Land Formation for Development at Anderson Road

James Rickard, David Rule & Michael Wright
Ove Arup and Partners Hong Kong Limited

ABSTRACT

The Development at Anderson Road - Site formation and Associated Infrastructure Works project is a large scale multi-disciplinary project commissioned by the Civil Engineering and Development Department (CEDD) for forming platforms and construction of the associated infrastructure works to meet the long-term public housing demand in Hong Kong. Ove Arup and Partners Hong Kong Limited (Arup) were appointed by CEDD to carry out the detailed design and supervision of the construction works. The construction contract commenced in January 2008. To match with the public housing development programme, land platforms are to be formed in stages, from 2011 to 2013, for handing over to the Housing Department for building works. The associated infrastructure works will be substantially completed by end-2014. The ultimate population intake is planned to be approximately 48,000. The project mainly comprises formation of about 20 hectares of platforms and related slopes and retaining walls; roads, bridges, a storm water drainage system, including an underground storm water storage tank, sewerage system and water mains and landscape works.

The project site is an existing hillside bounded by roads at the edge of the Kwun Tong urban area. Extensive excavation and slope improvement works are required for forming the platforms. Approximately 2.6 Million cubic metres of rock are to be excavated, mainly by blasting. Blasting under careful control has proven to be less disturbing and more acceptable to the nearby residents than conventional rock excavation methods. The prime objective is to ensure safety during blasting works. Stringent blasting specifications and tight supervision requirements are provided in the works contract to ensure public safety and prevent damage to surrounding existing slopes, utilities etc. Particular precautionary measures include: ensuring that the blasting directions will always be away from the residential areas; specifying no-blast and reduced blast zones along the slope edges above the public areas; a temporary high strength rockfall fence (flexible barrier); the use of blasting cages covered by thick rubber blasting mats to further reduce the air-overpressure and dust generated from the blasting works and temporary closure of public roads during blasting times. Several measures were also implemented following liaison with local stakeholders such as restricting the number of blasting days per week and blasting times permitted.

1 INTRODUCTION

The Development at Anderson Road (DAR) is a site formation project to construct platforms in the hillside between Anderson Road and Sau Mau Ping Road above Kwun Tong in East Kowloon. The platforms are to be used for public housing, public amenities and associated infrastructure. The project was first proposed in the 1990s as a means of providing some of the promised public housing. A Consultancy Agreement was ultimately awarded in 2006 to Arup. The construction contract was awarded in early 2008 to China State Construction Engineering (Hong Kong) Ltd.

The site is bounded to the east by Anderson Road, to the south by Po Lam Road, to the west by Sau Mau Ping Road, Shun On Road and Lee On Road and to the north by Clearwater Bay Road. Several platforms were proposed to cut into the crest of numerous existing cut slopes, disturbed terrain and natural hillside above Sau Mau Ping Road. At the eastern side of the platforms a series of substantial cut slopes were to be formed in soil

and rock, see Figure 1. Further information on the geotechnical aspects of this site formation can be found in Wright et al. 2013.

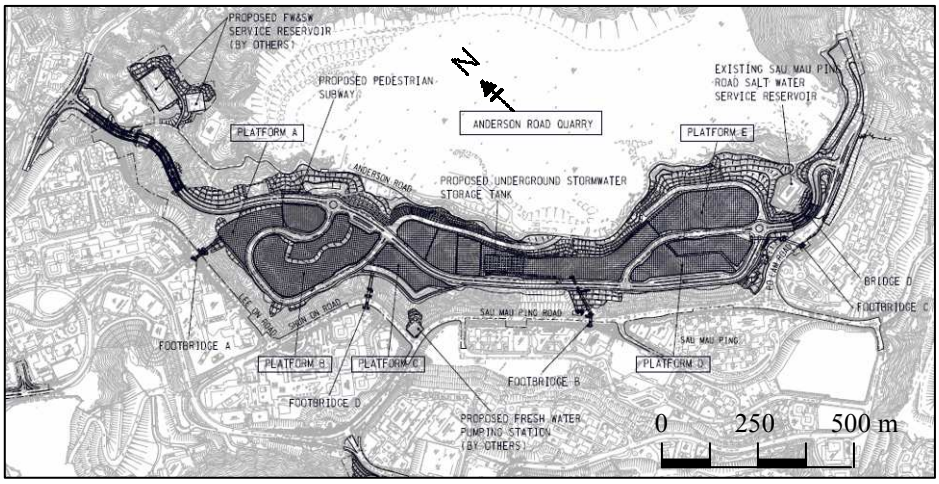

Figure 1. Site Plan

A total of approximately 2.7 million m^3 of soil and 2.6 million m^3 of rock were to be excavated. The rock was to be excavated principally by blasting.

2 THE SITE

The site generally slopes downwards from east to west with elevations of +220mPD near Anderson Road and +100mPD at Sau Mau Ping Road. Two typical terrain types were identified, with rounded lower lying hills with thicker soil deposits in the north and south of the site, and a steeper more planar area in the centre of the site with rockhead close to surface. A major fault is shown on the 1:20,000 Geological Map Sheet 11 extending from Shun On Estate in the north to Sau Mau Ping Estate in the south-west. A large valley cuts through the northern part of the site associated with a minor south-west to north-east trending fault, see Figure 2.

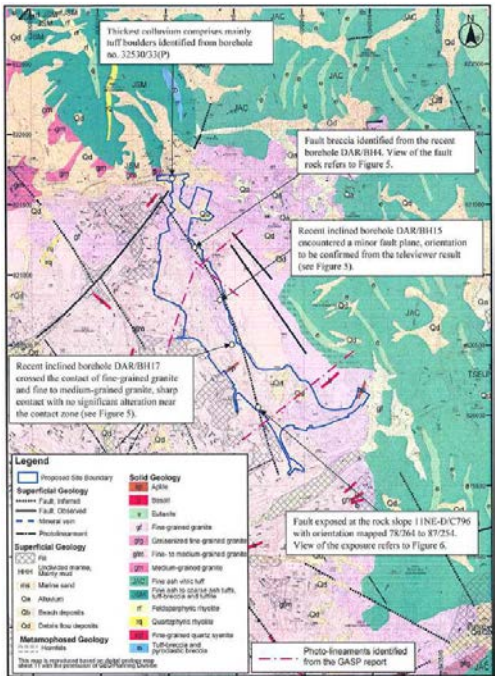

Figure 2. Geological Map

The site is underlain by fine to medium grained granite. A series of fill slopes and retaining walls were present supporting Anderson Road along the eastern boundary of the site. The natural hillside was partially covered with colluvial deposits over a variable thickness of CDG. Significant corestone development was encountered in the central areas of the site, with individual corestones and boulders up to 400m^3 in size.

There are three orthogonal joint sets within the main rock mass. In addition there are very well developed sheeting joints above Sau Mau Ping Road. The sheeting joints have a spacing of 2 to 6m, and aperture of 200-300mm and persistence of over 100m. The sheeting joints dip at 20-30° on the upper slopes and at 30-40° on the lower slopes. Sheeting joints are not well developed in the northern portion of the site.

3 THE HISTORY OF THE AREA

Several blasting related incidents have occurred in earlier projects in the area which have led to the enhanced precautionary measures adopted to ensure safe blasting works on the DAR project.

In the 1980s there was significant instability in the rock slopes above Sau Mau Ping Road which has been attributed to a failure along the sheeting joints which are found in the area.

In 1997 a mass of rock blocks were dislodged during blasting which resulted in Sau Mau Ping Road being completely blocked (GEO 1999).

In 2003 surface blasting work was carried out at Jordan Valley near Choi Hung MTR Station. There were two incidents of fly rock during this work, resulting in damage to property and injury to nine members of the public (Tse 2008).

4 BLASTING CONSTRAINTS

The constraints on blasting at DAR can be broadly divided into two groups; the technical issues and the public perception.

The sensitive receivers that affect blasting within the site include many slopes and retaining walls adjacent to the surrounding roads, see Figure 3. A number of Category 3 fill slopes, some with retaining walls, were found downslope from Anderson Road. Above Po Lam Road, Sau Mau Ping Road, Shun On Road and Lee On Road there were a large number of Category 1 cut slopes in soil and rock, some of these had previously been upgraded by LPM work. Also there were several slopes within the site, mainly disturbed terrain features or small Category 3 features. All of these slopes were assessed and an allowable Peak Particle Velocity (PPV) was calculated. This is the greatest ground movement that can occur safely at each slope as calculated in accordance with GEO Report 15 (GEO 1992). The maximum instantaneous charge weight for each blast can be calculated using the distance to a sensitive receiver and the allowable PPV using site specific constants.

Before blasting could commence, upgrading work was required on several of the features to ensure that they would be stable under blasting vibrations. Work ranging from placement of rock fill to extensive buttressing of rock blocks and boulders was carried out at early stages of the works.

Various utilities are located under the roads which surround the site. An electrical sub-station which serves Anderson Road Quarry had an allowable PPV of 13mm/s and provided the greatest constraint on charge weights in the central part of the site. Relocation of an overhead power line that ran through the site was required prior to the start of blasting. Three WSD reservoirs were located to the south and north of DAR.

At the commencement of blasting there were a number of old temples within the site that had not been constructed in accordance with any building regulations. The temples were generally wooden construction with some rusted steel frames and occasional concrete columns. These temples were calculated to have an allowable PPV of 5.7mm/s, but were demolished after the construction of replacement temporary temples on Po Lam Road.

An on-site explosives magazine store was not permitted due to the location of DAR and the proximity of the public, and no suitable location could be found within a reasonable distance. Consequently all explosives delivery had to be carried out by Mines Division of CEDD from Kau Shat Wan, Lantau on a daily basis. This provided a constraint on the number of blasting windows per day and the timing of the blasts.

The surrounding high-rise residential buildings, some of which are less than 100m from the blasting area, and other structures such as schools and shopping centres, provide the greatest sensitivity to air-overpressure (AOP). Numerous public consultation meetings were held with local residents and representatives to determine what arrangements would be acceptable to the public.

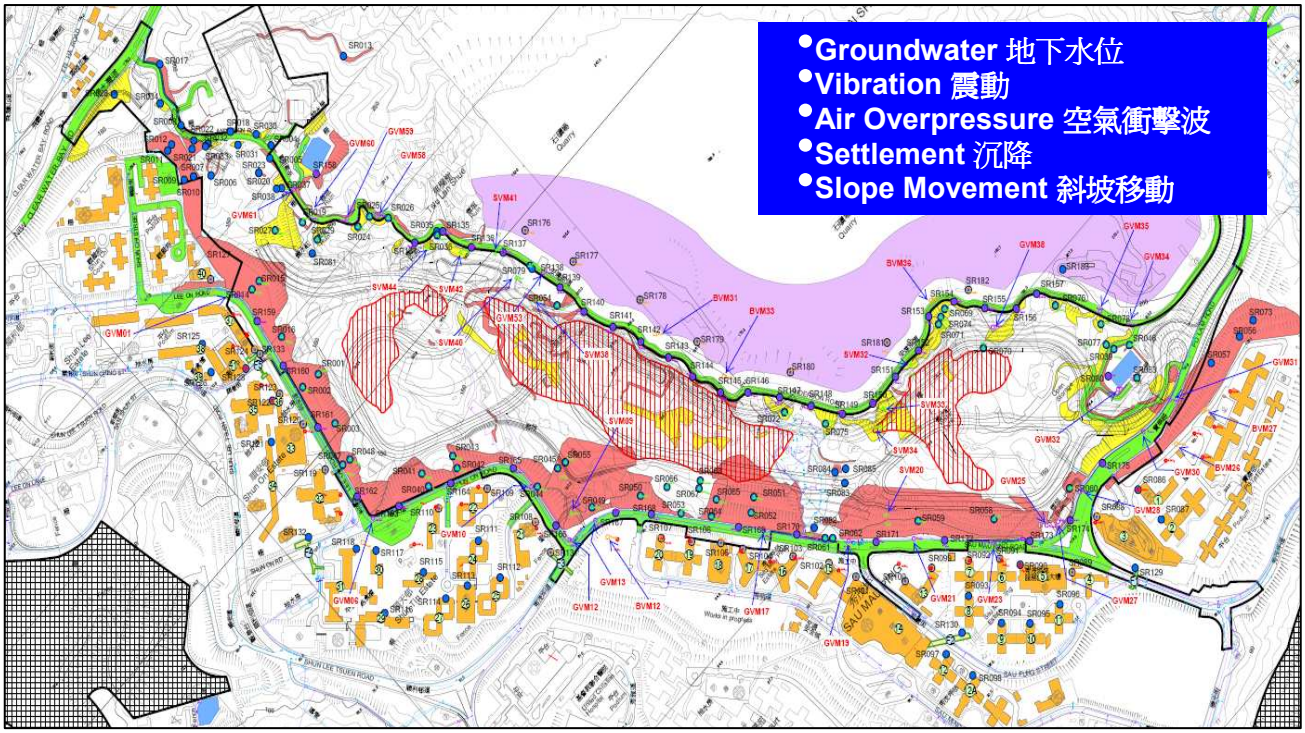

Figure 3. Plan of Sensitive Receivers with Monitoring Points

Blasting commenced In November 2008 with four blasting days permitted per week, limited to weekdays, with blasting windows of 2pm to 3pm and 4pm to 5pm. On a few occasions three separate blasts per day were carried out, however for the majority of the blasting works, two blasts were initiated per day. Following positive feedback from the residents and extensive consultation, blasting on 5 days per week, with blasting on a Saturday allowed if a Public Holiday occurred during that week, was permitted.

Continuous liaison with resident's groups and regional representatives has ensured that there has been a very positive reception to blasting works from the local population. No complaints directly linked to blasting works have been received and requests to increase blasting in order to reduce the use of hydraulic breakers have been put forward. Many other measures have been implemented to minimise the impact on the public such as the use of silenced drill rigs for blast hole drilling and control of the timing, routes and number of truck movements through the surrounding urban area.

5 BLASTING METHODOLOGY

Blast volumes have varied from 145m^3 to 4,860m^3. The quantity of explosives used per blast has ranged from 30kg to 2,095kg. 76mm dia blast holes have generally been used with cartridge emulsion and bulk emulsion explosives. Generally 50mm dia cartridge emulsion has been used for the smaller blasts and bulk emulsion has been used for the larger blasts. The maximum powder factor for bench blasts has been 0.46kg/m^3. The bulk emulsion was stored on site in a Cat 7 Store and was mixed as it was pumped into the blast holes using a pump-truck which was registered with Mines Division as a Mobile Manufacturing Unit (MMU), see Figure 4.

A combination of cartridge and bulk emulsion in different parts of the same blast has been used to good effect. Permanent rock slopes have been formed by pre-splitting techniques using 40g per metre detonating cord with a 1kg cartridge at the base of the blast hole.

Figure 4. Detonators, Cartridge and Bulk Emulsion MMU

A non-electric detonation system has been used, with a combination of down-the-hole detonators and surface delay connectors being used to produce a phased blasting sequence. Double-decked blasts have been employed using two different down-the-hole detonator delays to double the maximum instantaneous charge weight in one hole, whilst ensuring that the predicted PPV are within the allowable limits.

Adjacent to the crest of the slopes on the downslope side of the site a no-blast zone was designated to minimise the possibility of dislodging any rocks or boulders from the slope. The no-blast zone is 8m horizontal distance from the crest of the original slope surface. Between 8 and 24m, a reduced blast zone is implemented where blast holes are limited to 50mm diameter using 32mm dia cartridges and the blast whole depth must not exceed 5.5m. Bulk emulsion cannot be used in the reduced blast zone in order to ensure exact control of the charge weight and also to ensure that no liquid bulk emulsion migrates into any open sheeting joints that may be present. Anderson Road is closed for every blast and Sau Mau Ping Road is closed during blasting in the reduced blast zone.

The throw direction for the blasting works cannot be towards the public areas. Generally this means that blast throw can be to the north, east or south, but cannot be towards the west. In some locations this has meant that trench blasting must be employed to create a free face for excavation of the next level of blasting. Trench blasting was also used for the first blast of utility trenches and sand traps which are below formation level.

6 PROTECTIVE MEASURES

Extensive stabilisation works of existing boulders and rock slopes were carried out prior to the start of blasting in order to minimise the risk of slope failures during blasting works. A temporary rock fall fence was constructed along the toe of the slope adjacent to the public footpath.

A temporary flexible rock fall barrier was constructed at the mid-slope of the areas with fall lines which lead to the public areas. The flexible rock fall barrier was constructed on existing slopes below the final formation level, see Figure 5.

Figure 5. Flexible Rock Fall Barrier

Protective measures are required for every blast, see Figure 6. All blast holes are covered by a blast cage consisting of robust steel members, covered with strong wire mesh. When more than one blast cage is being used for a blast, the cages must be held together using chain locks to prevent the cages moving apart and top screens to close any gaps between cages preventing a path for fly rock to open up. Also the blast cages must be stabilised with two concrete blocks weighing 4 tonnes each. Rubber tyre mats must be placed over the top of the blast cages, and draped over the sides of the cages facing the public, to provide additional stability, to prevent fly rock, to reduce air-overpressure and to contain any dust that has been produced.

Figure 6. Blasts with Protective Measures Completed

On the top of the blasting surface a layer of gunny sacks and wire mesh weighed down with sand bags must be placed to control the displacement of the blasted rock. A layer of hanging mesh weighed at the bottom by sand bags must be placed over the free face to minimise the movement of rock from the free face and reduce the effect of any blow outs.

The blast cages must be surrounded by blast screens, constructed of robust steel members covered with wire mesh. These screens must be placed to provide protection up to 45° above the blasting surface. In addition to these physical protective measures, evacuation of all non-essential personnel from the blast area and road closures was carried out.

7 INSTRUMENTATION AND MONITORING

Vibrographs are placed at the five sensitive receivers that were calculated to receive the greatest percentage of their allowable PPV. The results of these vibrographs are assessed and any exceedance of the Alert, Alarm and Action levels would trigger a series of reviews and inspections to ensure that the receivers had not been damaged and to assess the maximum charge weight for future blasts in that area.

Air-overpressure monitoring is also carried out at the three closest structures. These are frequently the residential blocks in Sau Mau Ping Estate.

The slopes around the blasting area are also visually monitored by a team of engineering geologists to detect any instability caused by the blasting works and assess any necessary actions to safeguard the public.

8 BLASTING PERSONNEL

The Blasting Assessment Report sets out the requirements for the Contractor's and Consultant's staff for blasting works.

The Contractor's staff includes a Blasting Engineer, an Engineering Geologist who will carry out geological mapping of the blast area, a Chief Shotfirer to mark out the blasts and carry out charging and initiation of the blasts, an Earthworks Superintendent to manage the protective measures and site evacuation and a Technician to be responsible for blast monitoring.

The Consultant's staff includes a Blasting Specialist and an Engineering Geologist who will advise the Project Director and carry out site visits. The Resident Site Staff includes a Senior Resident Engineer, a Blasting Supervisor and an Engineering Geologist to supervise the blasting on a day to day basis.

9 DETAILED BLAST MONITORING

Temporary cut slopes and fill slopes were constructed in the vicinity of several blasts, in some cases as close as 8m, to gain information on the effects of blasting. These slopes were heavily instrumented with vibrographs at the crest and toe of the slopes and fill slopes were also provided with vibrographs at the mid slope and underneath the fill body. The results of these monitored slope behaviours were provided to GEO and, partially as a result of this work, TGN 28 (GEO 2010) was issued providing a less onerous method of allocating allowable PPV levels for selected slopes.

The results of the detailed Blast Monitoring will be the subject of a separate paper.

10 CONCLUSION

The blasting works at the DAR site are in a sensitive urban location with recorded slope instability and several blasting related incidents having occurred previously in the area. The blasting works have been carried out under strict supervision with robust protective measures and suitable methodology to provide safe blasting which will cause minimum inconvenience to the public. Blasting is nearing completion at the time of writing with about 1650 blasts completed without any incidence of flyrock or destabilisation of the existing slopes.

ACKOWLEDGEMENT

This paper is published with the kind permission of the Civil Engineering and Development Department, of the Government of Hong Kong Special Administrative Region.

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Geotechnical Considerations in the Design and Construction of Development at Anderson Road

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Ove Arup and Partners Hong Kong Ltd

ABSTRACT

The Development at Anderson Road Project is the largest urban site formation project in Hong Kong currently under construction. This Project is to provide 20 hectares of land for housing, educational, government, institution and community uses. Over 150 slopes and retaining wall features are to be formed on the mostly undeveloped but disturbed steep terrain and around 40 existing registered slope features in the vicinity are to be upgraded for the formation of level platforms, new approach roads and bridge abutments. The hilly but disturbed topography of the site area, with well-documented history of slope failures, presence of persistent sheeting joints and localised boulders on steep terrain with difficult access, poses a great challenge during the design and construction stages. The experience gained and lessons learnt from this project will be useful in any future site formation projects of this scale.

1 INTRODUCTION

The Site is located at the north-eastern part of Kwun Tong, Kowloon, Hong Kong, see Figure 1. This Project is to form about 20 hectares of land platforms in an area of 58 hectares of hilly terrain together with the following infrastructure and landscaping works:

- Road works of about 4,700 m in length;
- 3 road bridges, 4 footbridges, 2 pedestrian subways and relocation of noise barrier;
- Underground storm water storage tank, drainage, watermains and sewerage works; and
- Landscaping works.

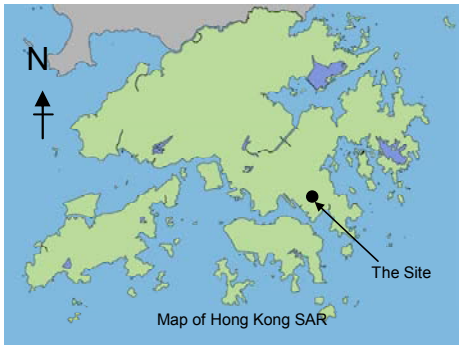

Figure 1: The Site

The Site slopes generally downwards from east to west with elevations ranging from about +220 mPD near Anderson Road to +100 mPD near Sau Mau Ping Road. For the formation of the development platforms, excavation of approximately 5.5 million m^3 of soil and rock is required. About 2.1 million m^3 of rock is to be excavated using blasting methods. Retaining walls and slopes are required to create level platforms with significant level difference between platforms. Reinforced earth walls, reinforced concrete retaining walls, bored pile walls as well as cut and fill slopes are proposed for retaining the level difference.

The proposed development is shown in Figure 2. The geotechnical works include foundations, slope and retaining walls, geotechnical instrumentation, and open blasting. In this paper, the design background, the site condition, the difficulties, and the lessons learnt will be discussed.

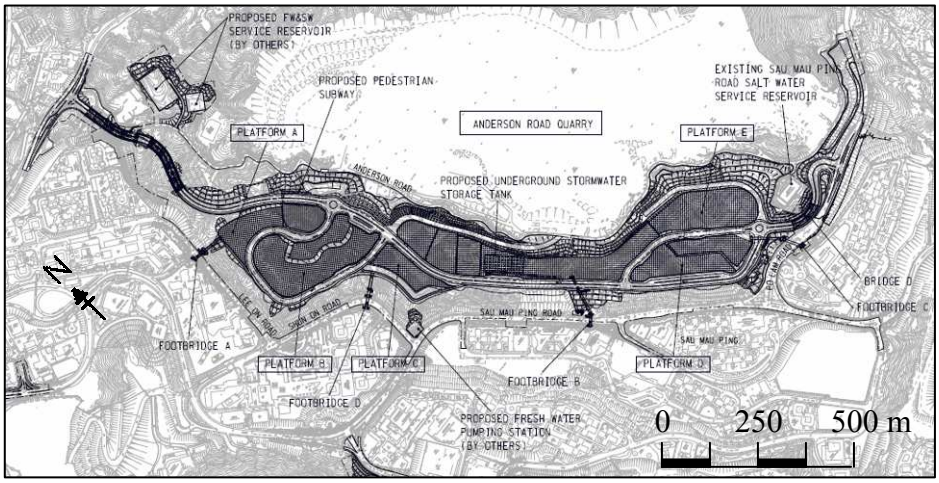

Figure 2: The Proposed Development at Anderson Road

2 SITE HISTORY

The Site was predominately natural terrain before the 1960's with some disturbance due to small scale quarrying, formation of terraced farmlands and building platforms for some single storey buildings. Many squatter structures for domestic and light industrial uses were built within the site from the 1960's up to the mid 1980's. Many of the small cut and fill slopes and disturbed terrain were formed during this period. Uncontrolled dumping was also carried out particularly adjacent to Anderson Road. There are a total of 151 existing registered man-made features that fall within the site boundary, which were identified during the design stage. There are 51 existing slopes to be removed and 52 existing slopes to be largely modified due to the development. 151 new formed slopes will be formed after the completion of the development, see Table 1.

Table 1: Summary of New Formed Slopes and Existing Slopes under this Project

Category	Description	No. of Features
1	Existing features reviewed for the site formation works. 39 nos. required upgrading, 9 nos. no works required	48
2A	Existing features to be removed and replaced by new formed feature due to the site formation works	51
2B	Existing features to be largely modified due to the site formation works	52
1+2A+2B	Total numbers of existing registered man-made features	151
3	New formed features (including slopes and retaining walls) to be formed by the site formation works	151

There are a total of 160 previous landslide incidents, which are mainly boulder falls, washouts and failure on man-made slopes or disturbed terrain. The formation of the slopes and retaining walls within this disturbed terrain is one of the major difficulties in this Project.

3 GEOLOGY

Based on the 1:20,000 scale Geological Map Sheet 11, see Figure 2, the Site is underlain by fine- to medium-grained granite with the presence of minor aplite dykes. The depth of weathering varies from a few metres to exceeding 30 metres. A major fault is shown to extend from Shun Lee Estate in the north towards Sau Mau Ping Estate to the southwest.

Superficial deposits including Colluvium and Fill cover part of the Site. Colluvium is usually present along stream courses and the material is generally described as unsorted sands, gravels, cobbles and boulders in silt/clay matrix according to the published geological maps. Both the engineering geological map of the Geotechnical Area Studies Programme (GASP) (GCO, 1987) and the aerial photography indicate fill slopes along the south-western side of Anderson Road.

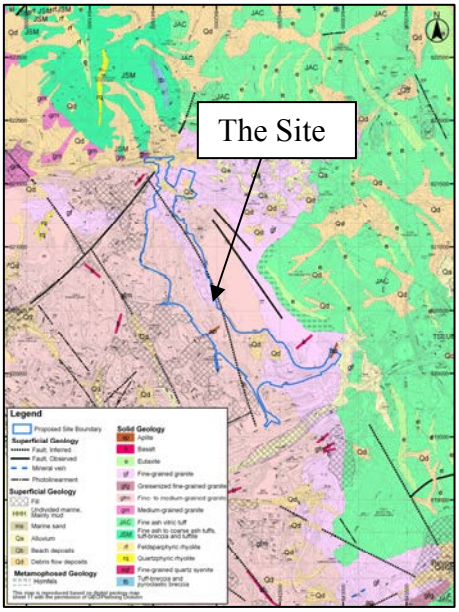

Figure 2: Geological Map Sheet 11 (Part Print)

3.1 Sheeting Joints

Sheeting joints can be seen over much of the existing slopes particularly above Sau Mau Ping Road and are well developed in massive granite. There are several records of instability of the slopes along Sau Mau Ping Road including a large rockfall event occurred in 1997 which was attributed to failure along sheeting joints, during nearby blasting operations (GEO, 1999). Instability of the rock slope along the sheeting joints could occur if the slope was fully saturated and subjected to blasting vibration.

For this project, detailed geological mapping and assessment of the slopes have been carried out on site as excavation proceeds. It was observed that the spacing of the sheeting joints is typically in the order of 2-6m. The aperture of the joints can be up to 200-300mm. They are generally very persistent, over 10-20m, dipping at 20-30 degrees at the upper portion, to 30-40 degrees at the lower portion of the sheeting joint and dip generally in the direction towards downhill side of the slope (i.e. Sau Mau Ping Road), see Figure 3. Several measures were included in the works contract to minimize the effects of blasting operations on existing slopes. Where excavation of existing slopes was required, an 8 metre-wide non-blasting zone and 8 to 24 metres wide restricted blasting zone, measured from the slope edge, were implemented. Extensive rock slope stabilization works were completed before blasting was allowed to commence.

Figure 3: Typical Inclined and Infilled Sheeting Joints identified on Site

3.2 Boulders

It is observed that sheet and gully erosion are common on the hillside within the Site especially along the ridgelines. Bouldery colluvium is noted on the incised natural stream course. Rock outcrops and boulders are observed over the northern portion of the Site suggesting shallow rockhead at that area (Figures 4 and 5).

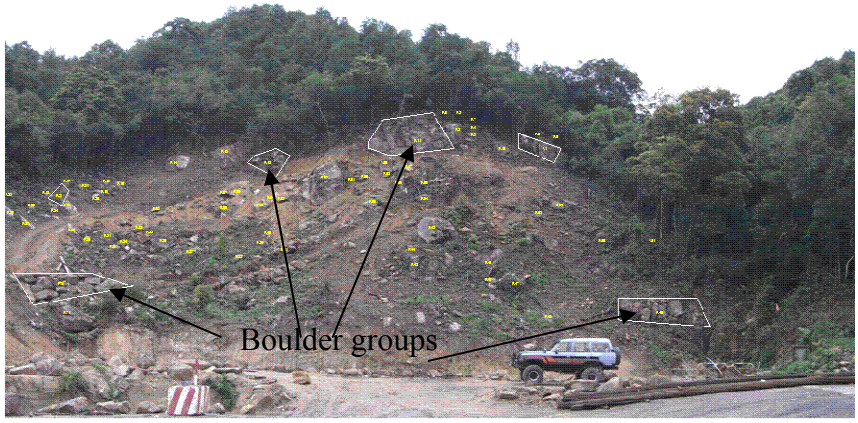

Figure 4: Boulder survey above the development platform

Figure 5: Very large unstable boulder was removed before blasting

The following measures have been taken on site.
For boulders along stream courses:

- Carry out run-out analysis for the boulders mapped to assess the potential impact of the boulders to the downhill facilities;
- If unstable boulder identified, flexible catch fencing could be considered within the stream course and at the toe of the stream course depending on local morphology.

For boulders lying on the sloping ground:

- Detailed boulder survey to be carried out during construction to suit the site formation works. The removal of the boulders as preventive measures is adopted in this Project (Figure 6);
- Stabilize/remove individual unstable boulders before the blasting works could commence, where necessary.
- Temporary rock fall fence (high strength flexible barrier) installed at the platform perimeter above public areas, see Figure 7.

Figure 6: Boulder removal by hydraulic rock splitters

Figure 7: Rock fall fence installed before blasting commenced

4 GEOTECHNICAL DESIGN PARAMETERS

There have been several phases of ground investigation on the site with 172 boreholes, 44 trial pits and 18 surface strippings. The geotechnical design parameters that are to be used have been based upon the results of the all the recent and past ground investigations findings and laboratory testing results. Figure 8 shows the s'-t plot result for the soils encountered in this Project and the adopted design parameters are shown in Table 1.

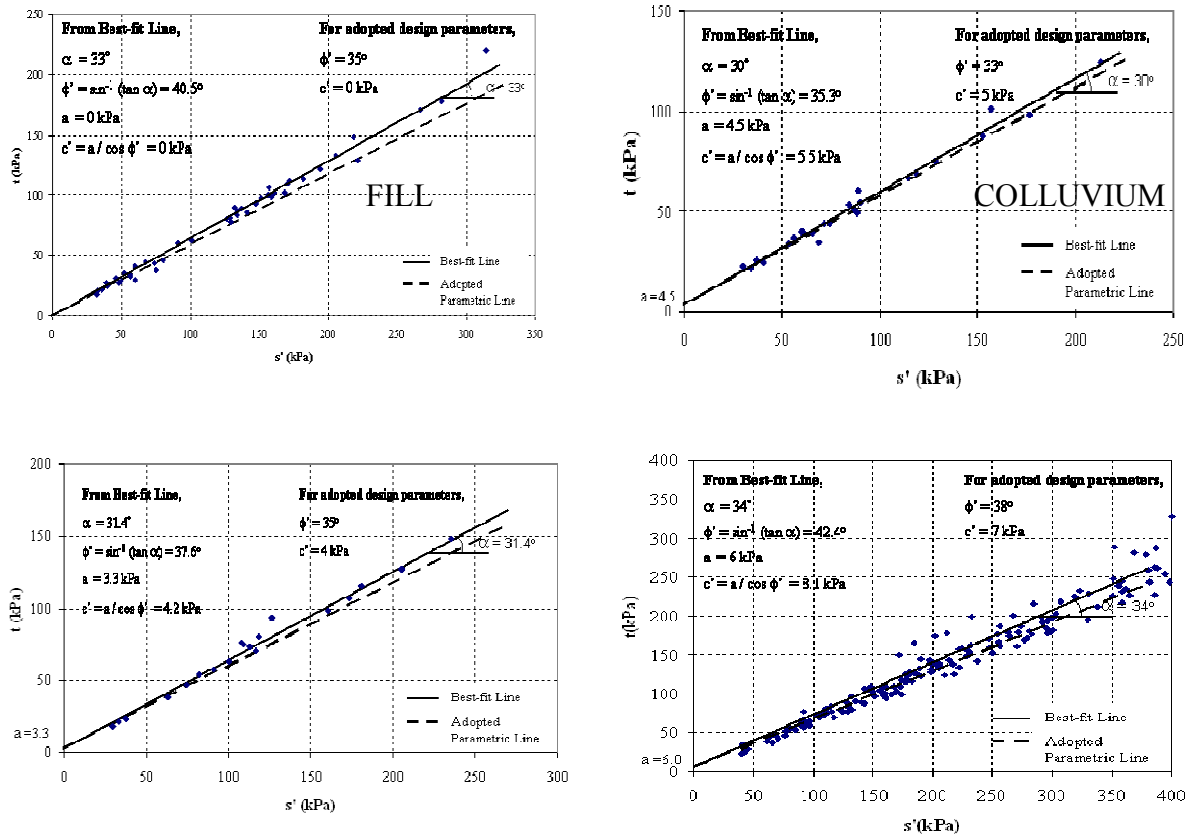

Figure 8: s'-t Plot for the soil material

Table 1 Adopted Design Parameters

Material	Bulk Density (Mg/m3)	Cohesion (kPa)	Friction Angle φ (°)
Fill (General)	18	0	35
Fill (Compacted)	18	0	38
Colluvium	18	5	33
Residual Soil (RS)	19	4	35
Completely Decomposed Granite (CDG)	19	7	38

5 DESIGN PHILOSOPHY

The design of the site formation is using cut and fill principle to maximize the useable land and minimize the disposal of surplus material. The selection of the proposed platform level is subject to the following:

- Site topography
- Rock head level
- Cut and Fill balance
- Required platform area

A typical section of the proposed platform is shown in Figure 9 below

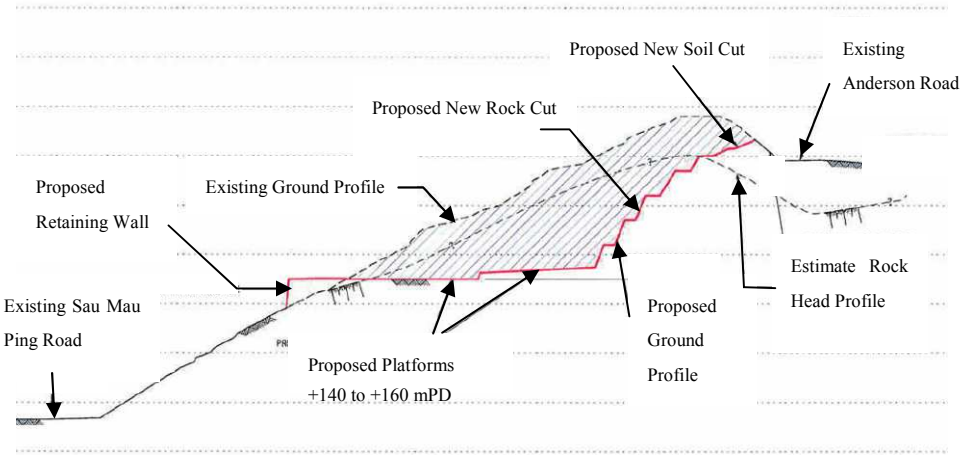

Figure 9: Typical section showing the site formation

The design of slopes is in accordance with Geotechnical Manual for Slopes (GEO, 2000). Soil cut slopes are designed at an angle of 35° with 7 m high benches considered to be appropriate for the slopes within completely to highly decomposed granite which comprises mainly silty fine to coarse sand and no significant kaolinisation or other adverse geological conditions. Where soil slopes are required to be cut at steeper angles, soil nails are added. The design slope angle with soil nails is up to 60° whilst providing a vegetated slope surface utilizing erosion control mat and wire mesh. As the site formation involves slope cuts up to 50 to 60 m high, berm widths of 2 m are considered to be appropriate for future maintenance and inspection works.

Typical rock slope angle is 70°, with 10 m high benches and 2 – 3 m wide berms used with drainage and planters. This will give an overall slope angle of 57° - 60°. Blasting is the major method for the formation of the rock slopes and pre-split blasting techniques have been employed to improve the stability of the rock cut slopes to reduce the effects of blast damage on the slope face (Rickard et al. 2013).

Different options of retaining system were reviewed to provide the required platform area for the development. In early designs for the development, large-diameter bored pile walls were generally adopted for retaining heights greater than l0 m. However, this type of wall requires the use of heavy equipment, which is particularly difficult to use on sloping ground such as at Anderson Road. Therefore, different options were proposed subject to the existing topographic and site conditions. Figure 10 shows the scheming of the site formation development during early design stage.

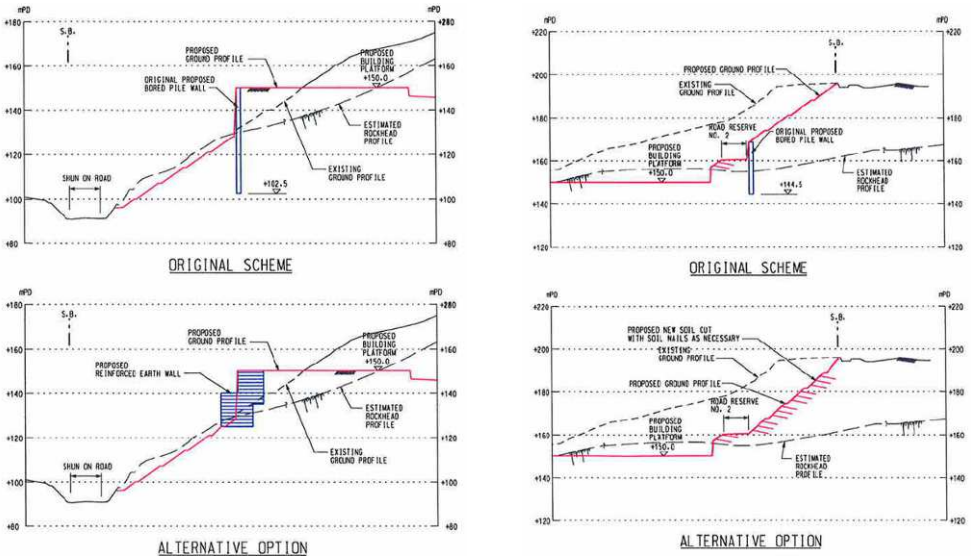

Figure 10: Scheming for the proposed site formation development

Except for one area where space constraints are extremely onerous, adjacent to an existing service reservoir, reinforced earth walls are preferred as alternatives to bored pile walls for the support of retaining height higher than 10 m. The geometry of the reinforced earth retaining walls will comprise rectangular single and multi-tiered structures. The structures are formed using frictional fill material reinforced with hot dip galvanized steel strips and contained by reinforced concrete panels. The internal stability of the walls is designed by a specialist contractor, and the wall and backfill material above the base are assumed to act as a single gravity block.

In this project, a relatively large number of reinforced earth walls for the supporting system were introduced. The use of reinforced earth wall can enable re-cycling of materials excavated within the site, thus reducing import and export of materials through the surrounding residential urban area. The walls themselves uses far less concrete than typical RC retaining structures, particularly for higher retaining structures and there is also potential to recycle the components at the end of service life. In this project, there is about 20% to 50% saving in terms of material and cost by using reinforced earth walls.

5.1 Cantilever Bored Pile Wall Design – Using C580 Design Approach

In general, as a traditional approach, the design of a cantilever bored pile wall is carried out in accordance with the recommendation given in Section 11.2.3 and Figure 50(c) of Geoguide 1 (Second Edition) (GEO, 1993) where a simplified model was adopted for the determination of the embedment depth of the pile, (Pang et al 2005). However, in this project, the design approach of CIRIA Report C580 has been adopted for the design of a permanent cantilever large diameter bored pile wall for the support of sloping ground and retaining around 20 m level difference for the construction of a new road in front of the wall, see Figure 11. This is the first recorded Hong Kong project adopting the C580 design approach for a permanent application with the approval from the Geotechnical Engineering Office. By using the C580 approach, an average pile length reduction of 3.5% was achieved resulting in about 10% saving of total bored pile cost (Ho, et al, 2013).

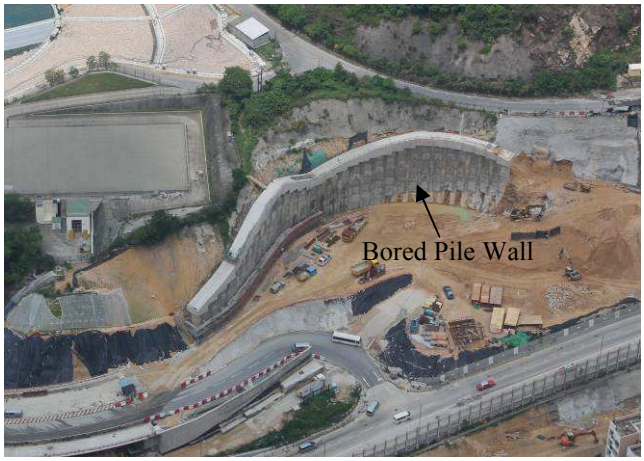

Figure 11: Permanent cantilever bored pile wall designed to C580 under construction

6 GEOTECHNICAL MONITORING SYSTEM

The objectives of the instrumentation and monitoring are as follows:

1. To verify that the ground behaviour is consistent with the design assumptions.
2. To confirm adequacy of ground-support systems.
3. To monitor the behaviour of existing adjacent structures that may be affected by construction-related activities especially during blasting.

A comprehensive monitoring scheme has been implemented in this project. The types and numbers of the geotechnical instrumentation that have been installed is listed in the Table 2 below and the locations of the geotechnical instrumentation for slope formation works is shown in Figure 12.

Table 2: Geotechnical Instrumentation

Type of Instrument	Unit	Monitoring Commencement Date	Type	No. of Instrument Installed	Remarks
SITE FORMATION AND GEOTECHNICAL WORKS					
Ground Settlement Markers	nr	14/7/2008	GS	67	For the monitoring of adjacent structures and newly constructed slopes
Building Settlement Markers	nr	23/7/2008	BS	8	For the monitoring of adjacent structures
Proposed Piezometers and Standpipes (Include AGMD)	nr	19/6/2008	PZ+SP	108	For the review of slope design
Vertical Inclinometer	nr	9/10/2008	VI	44	For the monitoring of adjacent structures and newly constructed platforms
Real Time Ground Water Monitoring Device (AGMD)	nr	19/7/2008	AGMD	30	For the review of slope design
Automatic Slope Movement Monitoring System	m	15/8/2008	ASMS	4158	For the monitoring of slope movement during blasting and other major excavation works
RC & RE WALL WORKS					
Building Settlement and Movement Markers	nr	8/9/2010	PRISM	62	For the monitoring of adjacent structures and newly constructed walls
Ground Settlement Markers	nr	14/7/2008	GS	8	For the monitoring of existing road and platforms
Building Settlement Markers	nr	23/7/2008	BS	10	For the monitoring of adjacent structures and newly constructed walls
Vertical Inclinometer	nr	24/6/2011	VI	3	For the monitoring of adjacent structures and newly constructed walls
Optical Fiber Sensor & Strain Gauge	nr	24/6/2011	BOTD	6	For the review of bored pile wall design
Vibrating Wire Strain Gauge	nr	22/7/2011	VWSG	32	For the review of bored pile wall design

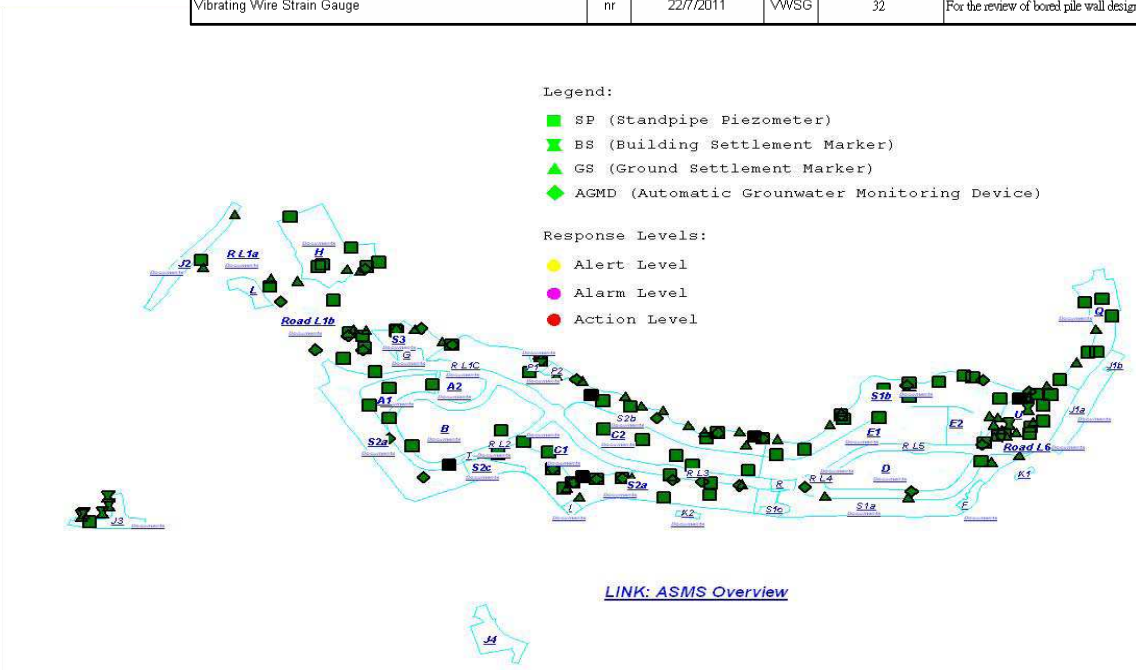

Figure 12: Locations of geotechnical instruments for slope formation works

Tight controls of the geotechnical monitoring have been implemented by introducing three trigger levels, Alert, Alarm, and Action (AAA). For detailed monitoring for the surface blasting see Rickard, et al 2013.

6.1 Automatic Groundwater Monitoring Device (AGMD)

In this project, there have been 30 numbers of AGMD installed around the site for the monitoring of the ground water level during the slope formation works. The AGMD records the ground water at 5 minute intervals and the data is plotted with rainfall intensity taken from the Hong Kong Observatory automatic rain gauge at Tseung Kwan O (TKO) as shown in Figure 13 below.

The data shows that the ground water level drops while the excavation was in progress and there is about a one week delay on response of ground water level after heavy rainfall.

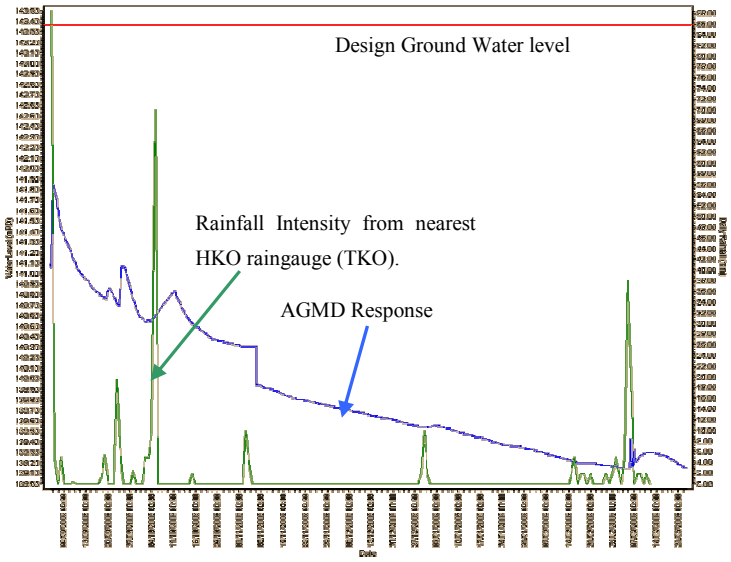

Figure 13: Typical AGMD data obtained on site

7 DIFFICULTY AND LESSONS LEARNT

7.1 Site Difficulty

The site runs along the downhill side of the Anderson Road and comprises a series of minor sub-catchments that have valleys and depressions that lie within the hillside forming the site. These sub-catchments issue at various points onto the lower existing slopes and roadside edges of Po Lam, Sau Mau Ping, Lee On and Shun On Roads. The site can be split into two typical terrain types with rounded low lying hills with gullies and stream valleys in the southern and northern sections. The northern section has more incised valley areas while the southern area is less incised. Surface run-off from Anderson Road Quarry discharges through the site.

Due to the past history with the presence of squatter and tire repairing works, the sizable valleys running through the stream course are mostly disturbed and subject to past illegal dumping, for example, asphalt and discarded tires (see Figure 14). Besides, some of the existing stream course outlets have been inadequately considered during the previous developments (Figure 15). The site formation works will have a significant impact on the adjacent residents and therefore, the excavation works carried out in the wet season is a major concern.

Figure 14: Condition of existing steam course

Figure 15: Existing steam course outlet condition
(Photo taken one day after Typhoon signal no.8 with red rainstorm in June 2008)

Localized parts of the site were found to have concentrated development of corestones above rockhead. As corestones were often large and composed of strong to very strong granite, delays to excavation to rockhead in order to commence blasting works or for founding of structures were encountered. .

7.2 Lessons Learnt

Drainage is an important factor for the site formation work especially for the construction of reinforced fill walls and thus the performance of the wall is affected if the drainage (temporary or permanent) is not properly considered and constructed. Typically, the design of the reinforced earth wall body assumes a dry condition with water pressure acting externally at one third of the retaining height.

During construction of reinforced earth walls, there were incidents of the dislodgment of the wall panels and loss of fill body, see Figure 16. Investigations were carried out and concluded that the following are the causes:

- the blockage of the surface drainage system and sub-surface drainage outlets,
- uncontrolled surface water ponded above the reinforced earth wall,
- inadequate temporary drainage,
- the concentrated storm water flow exceeded the design capacity of the granular filter layer.

Figure 16: Dislodged panels during the construction of reinforced fill wall

8 CONCLUSIONS

This is the largest urban site formation works currently carried out in Hong Kong to provide useable land mainly for housing development and associated community facilities.

The site is subject to the following conditions resulting in various challenges in the site formation design and construction:

- highly disturbed and steep terrain
- past history of slope failures
- development next to urban area
- presence of adverse geological conditions such as persistent sheeting joints
- localized boulders
- existing hydrological conditions

The use of a comprehensive geotechnical instrumentation monitoring system, strict blasting procedures and new design approach, such as CIRIA C580, has contributed to the safe and economic delivery of the Project.

ACKNOWLEDGEMENTS

This paper is published with the kind permission of the Civil Engineering and Development Department, the Government of the Hong Kong Special Administrative Region. Grateful thanks is given to Mr. Haydn Chan of Arup who provided valuable comments on the geological review on the site formation.

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Case Study of Rockfall Hazard on Natural Terrain Adjacent to Urban Development

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AECOM

ABSTRACT

Hong Kong has a high population density and significant portion of hilly terrain. A challenge facing sustainable urban development is the close proximity to steep natural terrain hillsides susceptible to hazards from landsliding. Rockfall is one type of hazard and this paper outlines a case study of a rockfall hazard assessment of a natural terrain hillside area in Hong Kong Island. Due to the difficult access of the site, an approach involving detailed aerial photo interpretation, LiDAR survey data and direct access via abseil ropes was adopted to determine potential hazard location and likelihood of occurrence, in order to select appropriate cost-effective mitigation measures that enhance public safety and sustainable development.

A natural terrain hazard study was undertaken to identify the nature, location and magnitude of the rockfall hazard posed to the development area below. Subsequently any necessary mitigation measures to mitigate the hazards were proposed. It was concluded that the hazard could be mitigated by the use of different types of mitigation measures including flexible barriers along the toe area of the natural terrain, and targeted in-situ stabilization within the vicinity of the identified potential source areas.

1 INTRODUCTION

As part of the ongoing Landslip Prevention and Mitigation Programme (LPMitP), AECOM were commissioned by the Geotechnical Engineering Office (GEO) of Civil Engineering and Development Department (CEDD) to undertake the investigation, design and construction of natural terrain mitigation works for five study areas located on Hong Kong Island.

Although potential landslide hazards were identified in the hillsides under study, one of the study areas included an extensive area of rock outcrop (Seymour Cliffs) above the urban area of Mid-Levels and potential rockfall/boulder fall formed a major part of the hazard assessment. This paper presents the methodology adopted for the rockfall hazard assessment and the findings.

2 THE STUDY AREA

The study area is located on a northeast-facing natural hillside above Conduit Road and Po Shan Road, and immediately downslope from Victoria Peak on Hong Kong Island (Figure 1). Lugard Road traverses approximately the mid-slope portion of the study area. Seymour Cliffs with a total height of 100 m is located below Lugard Road and falls within Pok Fu Lam Country Park area. Dense urban development, comprising mainly residential buildings and major roads, are located at the toe area of the study area hillside. An existing boulder fence (constructed in 1985) is located along the lower part of the hillside.

The elevation of the study area rises from 160 mPD at the residential development area, to a level of about 538 mPD at the Peak. The study area consists of densely vegetated natural terrain and is moderately steep to very steep with an average gradient of about 40° and increasing locally to over 80° typically at the Seymour Cliffs. The cliff topography is irregular with some very steep/sub-vertical slabs and some prominent sub-horizontal ‘ledges’ within the feature.

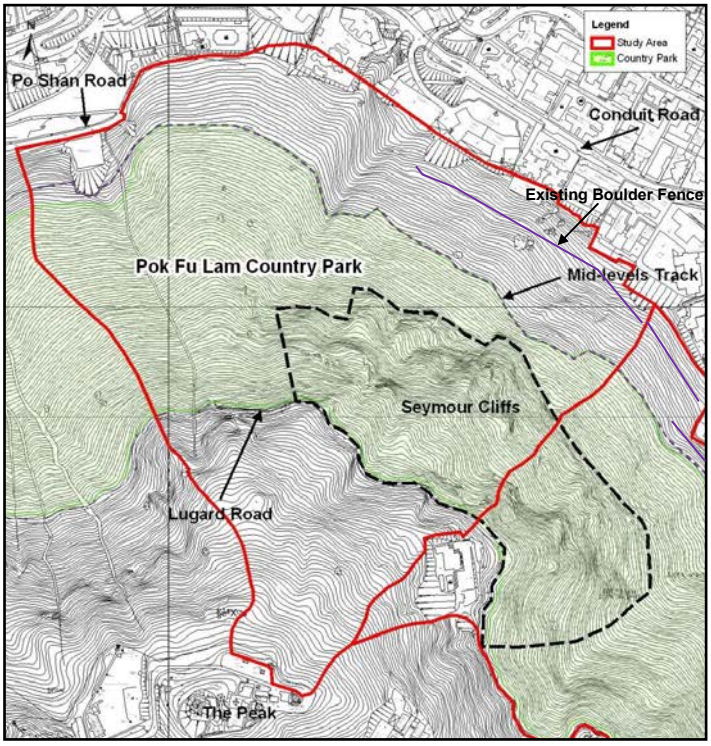

Figure 1: Study Area Location Plan

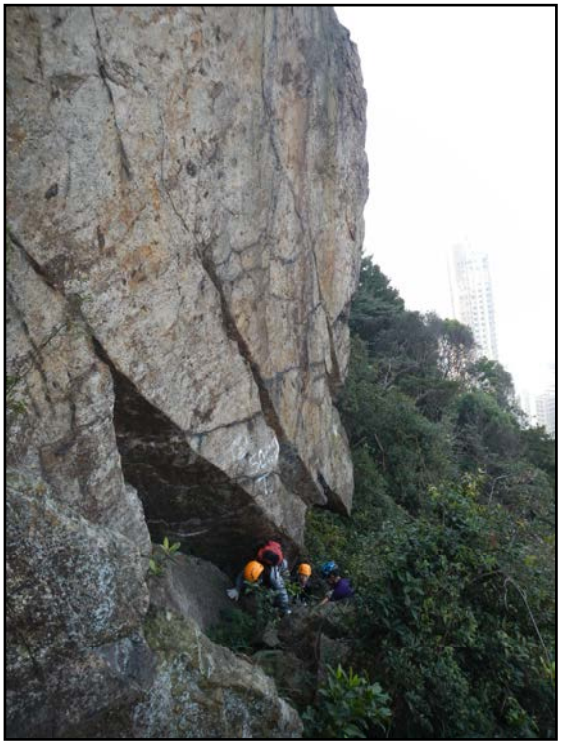

Plate 1: Abseiling on Seymour Cliffs

3 ASSESSMENT METHODOLOGY

The Seymour Cliffs area is accessible along certain footpaths along the base but mostly it is inaccessible due to the steep terrain and dense vegetation surrounding and intermittently covering the outcrop. Thus, the assessment would need to rely initially on indirect assessment from previous studies, detailed aerial photograph interpretation (API) and field reconnaissance from observation points (footpaths and adjacent building roof tops). The main objective was to prepare an initial engineering geological model, incorporating the above information, to identify potentially unstable blocks, slabs and wedges within the cliff area. Following this initial stage, limited direct access via abseil technique (Plate 1) was planned to focus on those areas and assess the stability with respect to the planning of mitigation works.

3.1 Previous Studies

Mid-Levels Study: Main Report and Report on Geology, Hydrology and Soil Properties (GCO, 1981 & GCO, 1982)

This report indicated that the topography of Seymour Cliffs has been developed by "massive rock-falls followed by the slow disintegration of volcanic rocks and the associated falls of rock fragments". However, it was considered that the frequency of such a major rock landslide event would be on the "geological time scale and need not be considered as relevant to the present stability study". It was recommended that "regular inspections of the area should be made and any rock fragments which could cause an immediate and obvious danger situation should be secured".

This report included detailed field notes and discontinuity orientation information which were reviewed under this study. The data was found to be accurate when located on the site-specific plans using the latest survey data, with boundaries of identified features accurate to within a few metres when overlaid onto Light Detection and Ranging (LiDAR) topography and current field observations.

Mid-Levels Seymour Cliffs Improvement Works (GEO, 1996)

A photogrammetry study to identify potential hazardous rock outcrops at Seymour Cliffs was undertaken by GEO to identify boulders/rock outcrops that warranted closer inspection. Photogrammetric measurements of the inclination of rock surfaces were carried out. The resulting data was processed under this study assuming that the steepest inclination between the set of data points for each surface will be close to the overall dip of

the surface. Comparison with other mapping data indicates that dips processed from the data fit well with the structural pattern of mapped joint orientations from other sources (Figure 2). The locations of data points also fit well with the LiDAR topography and outcrops shown on the orthorectified photograph.

3.2 Aerial Photograph Interpretation (API)

Due to the steep terrain, the low accessibility of the study area, its large size and the dense vegetation cover, a detailed API was carried out, mainly using the 1963 aerial photos and orthophotographs, and the 1988 oblique photos of Seymour Cliffs, which yielded considerable detail.

The API was aided by preparation and use of orthophotographs created from digital image pairs formed from the 1963 aerial photographs and corrected using ground control points. The aerial photograph image was corrected to true scale and the orthophotographs were referenced and managed using a Geographic Information System (GIS). In addition, LiDAR data of the study area shows the highest density of LiDAR returns in areas of rock outcrop. LiDAR was also valuable in determining structural features, such as major joints and faults.

The configuration of hazardous boulders/rock outcrops on the Seymour Cliffs as identified under the previous study was compared. The locations of some of the features were slightly adjusted to fit better with the LiDAR topography and orthorectified photograph. The API study was instrumental in the initial assessment of the rockfall hazard within the Seymour Cliffs area. Additional features of interest were identified and other geologically controlled features, such as photolineaments and major joints, were also detected (Figure 2). No evidence of recent rockfalls or observable signs of deterioration/instability were apparent from the API.

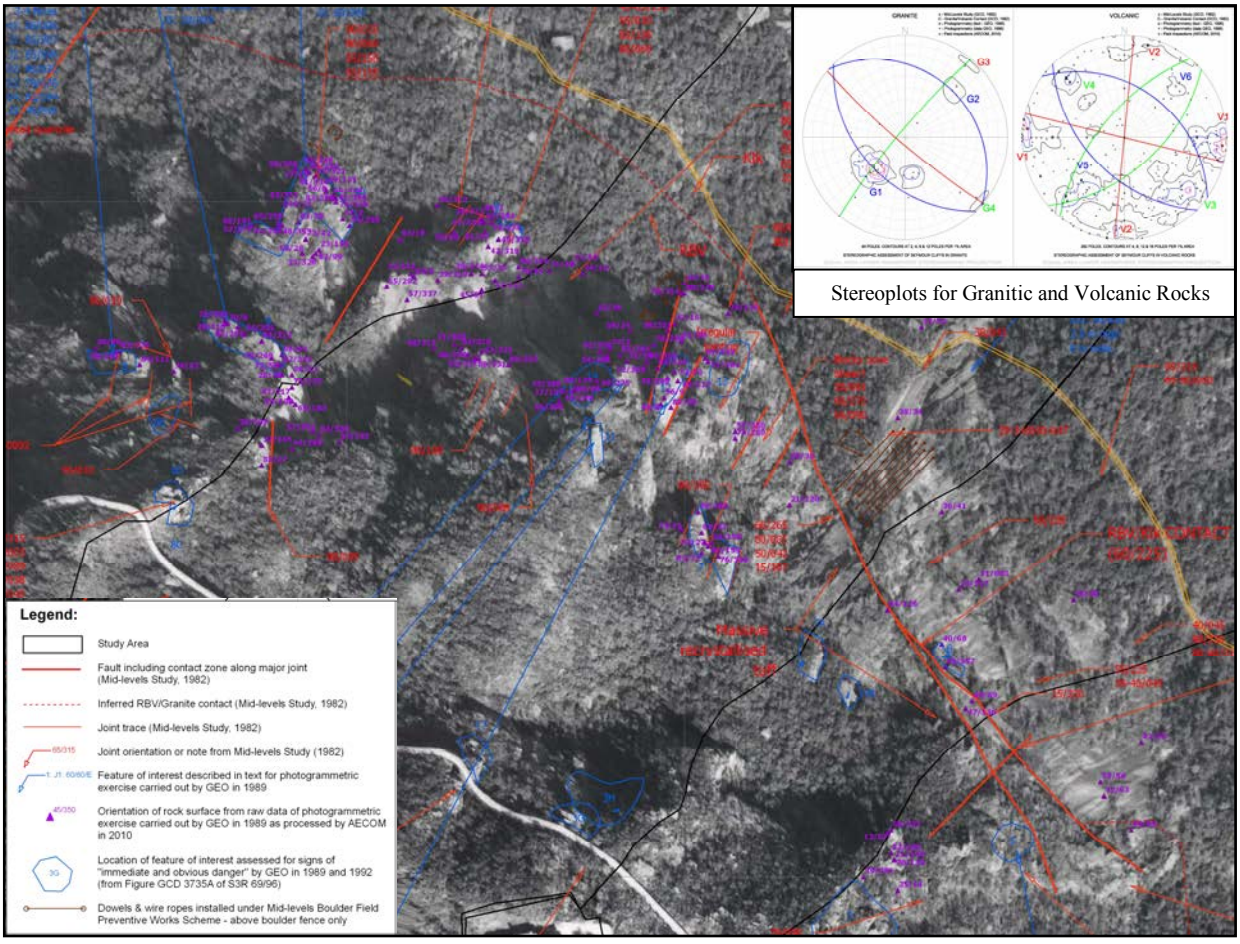

Figure 2: Desk Study Data Map of Seymour Cliffs (using a rectified 1963 aerial photo as a base)

3.3 Engineering Geological Mapping

The initial Engineering Geological plan was constructed from indirect information due to the steep terrain and dense vegetation cover of the study area, making access difficult to the cliffs. As Seymour Cliffs fall within the country park area, the erection of scaffolding is not a practical way to facilitate inspection. The initial Engineering Geological plan allowed the targeting of access routes using abseiling/ rock climbing techniques to directly assess the cliff area to inspect previous improvement works and newly identified features of interest.

Field mapping in the assessment stage was limited to ground truthing the API and visiting particular areas of interest. Climbing rope access was set out in the selected areas of the Seymour Cliffs to target the hazardous features identified during the Seymour Cliffs Study and specific features identified from recent API. No significant signs of distress or deterioration were observed in the previously carried out improvement works and the hazardous boulders. The rock discontinuity measurements taken from the field closely match with the readings based on LiDAR contours. The rockfall hazards from the specific features and the need for mitigation were assessed first hand. Field locations were accurately measured with the help of the surveyors from the contractor.

4 DESCRIPTION OF FINDINGS

4.1 Geology and Geomorphology

The 1:20,000 geological map shows that the Seymour Cliffs are underlain by metamorphosed fine ash vitric tuff in the upper and middle portions, and medium-grained granite in the lower portion. The granite has intruded the volcanic tuff and the contact is shown to traverse the site in a roughly northwest-southeast direction. The intrusive contact between tuff and granite dips towards the southwest at a medium angle into the slope (GCO, 1982). Talus deposits were mapped immediately below Seymour Cliffs at the lower eastern portion of the study area.

Seymour Cliffs are a northeast-facing hillside that descends from Victoria Peak which marks a relatively sharp convex break-in-slope and is relatively steep from 45° to over 80°, overall angle of about 50°. Downslope from the rock cliff, the hillside comprises moderately inclined terrain (25° to 40°), which becomes less steeply inclined along a concave break-in-slope below Seymour Cliffs.

Bare rock exposures were identified by API and were clearly evident in the field. Some of these rock exposures have a cliff form and were evidently the scarps of major failures in the past. The lower terrain below the Seymour Cliffs is generally hummocky in profile and appears to mainly consist of talus deposits strewn with surficial boulders.

4.2 Identification of the Rockfall Hazard

4.2.1 Individual Loose Rock Blocks

Specific rock outcrop or blocks within Seymour Cliffs which were identified as posing danger to the toe facilities were stabilised in the late 1980's. No obvious movements or signs of deterioration of rock cliffs were apparent from comparison of aerial photographs over the 30 year record, or direct observations.

This study confirmed that a rockfall hazard is present from the downslope movement of rock blocks, either from the array of blocks loosely arranged on the hillside or newly initiated from slowly degrading rock cliffs. Generic kinematic analysis of large outcrops using overall stereographic projection has proven that failures could occur. The main mechanisms for initiation are sliding and toppling failure, as evidenced from field inspection and measurement and plotted in stereoplots (Figure 2).

It is considered that the potential unstable rock blocks to reach the facilities is high as this is borne out by the fact that fallen rock blocks are noted to have been generally arrested on the lower slope where concave hillside starts to flatten.

A large plane, possible major joint or low-angle fault, identified from API was subsequent inspected on site. It is a persistent (about 80 m plan distance), rough stepped and undulating joint plane. This planar surface appears to be the probable basal failure surface for many relict rock failures as numerous small size boulders (<1 m^3) were observed on the downslope side. Some very large angular boulders (up to 5 m across) were

located at about 80 m to 140 m downslope from the large plane which may have resulted from long-term regression of the cliffs.

The potential rock failure mechanism involves sections of the steep cliffs resting on the plane which might topple or slide onto the large planar surface which would then deflect the debris downslope. Dislodgements of blocks immediately above the plane have created occasional overhangs at the toe of the steep cliffs along the interface, which over time would allow the gradual regression of the steep cliff face above the plane. The interface of the steep cliff and the prominent plane is considered to be a relatively active (over geological time scale) zone of attrition along which the cliffs are slowly regressing.

4.2.2 Large-scale Instability

Large-scale instability in the granite is not a major issue as most of the slope surfaces are formed along pre-existing stress-relief joints which have an average dip of about 35° and there are no steep breaks in slope where daylighting of the joint set could occur.

The eastern and main portions of the Seymour Cliffs are mainly comprised of relatively smooth-surfaced massive recrystalline tuff which is flanked on its western edge by a series of jagged cliffs, defined by relatively persistent joints. These jagged, western cliffs comprise a series of steep rock outcrops that run diagonally up the hillside. The eastern and central portion massive rocks are kinematically stable which tends to buttress the jagged western flank (Plate 2).

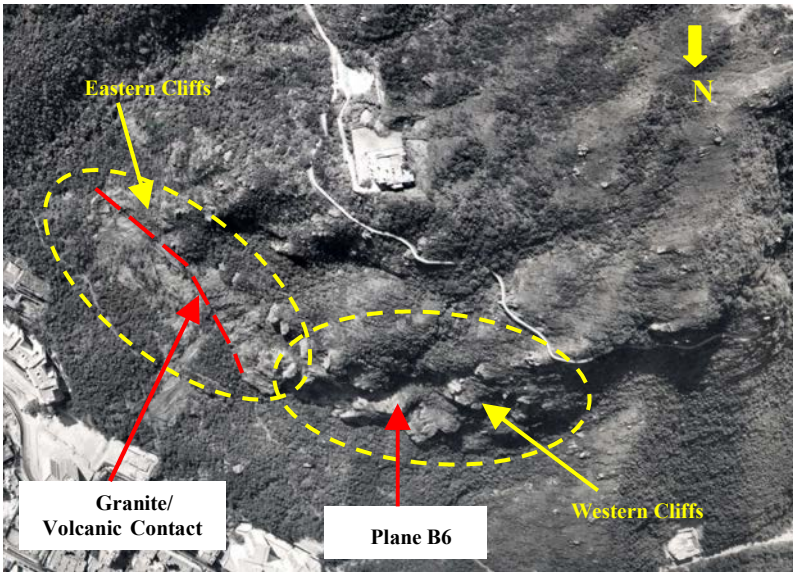

Plate 2: Aerial photo showing the western cliffs edges characterised by sharp wedge-shaped break-in-slope, whereas the eastern cliffs show a smoothed surfaced massive rockmass

There is no evidence for potential large-scale failure of the cliffs as the granite/volcanic contact dips into the slope and no overall failure mechanism is evident in the relatively massive eastern and main portion of the cliffs.

The main joint set that is most critically orientated is Set V5 which dips 40-80° northeast (Figure 2). In the Mid-levels Study Report, this set is reported to have a low persistence "a few metres continuity". This is confirmed from site observations where it generally appears to have a persistence of about 5 m. Thus the chance of large-scale sliding failure along this joint set is very low.

The above-mentioned singular, persistent plane is buttressed by the large volume of massive rocks which lie to the east (eastern and main portion of the cliffs). Consequently, the plane can only act as a basal release surface to isolated volumes of rock which overlook the edge of this feature but it is unlikely to give rise to large-scale failure. Measures to reduce the overall, long-term deterioration of the cliffs such as the proposed

stabilisation of individual blocks lying above this plane and other similar features should significantly enhance large-scale stability.

5 MITIGATION MEASURES

With the extensive urban development at the toe area of the hillside below the cliffs, it is not practical to adopt passive mitigation strategy. Active mitigation strategies including preventive and defensive mitigation measures are adopted.

The existing boulder fences were designed to absorb an impact energy of up to 100 kN-m and may not be robust enough to mitigate potentially unstable rock blocks (about 4.5 m^3) from the upper hillside. The proposed mitigation approach is in the form of an additional flexible barrier below Seymour Cliffs (i.e. on the mid-slope between the toe of the cliffs and existing boulder fence) to mitigate minor rockfall from the periphery of Seymour Cliffs. This also provides protection against residual boulder fall and minor open hillslope failure from the mid-slope portion.

The remaining hazard is from occasional individual large boulders/blocks above 4.5 m^3. It is probably best tackled by inspections and assessed on a case-by-case basis during the construction stage according to actual discontinuities present. It is considered cost-effective to apply individual mitigation measures, such as buttressing, dentition and dowels, to prevent their initiation of downslope movement.

6 CONCLUSIONS

i) API can be aided by the use of digital orthophotographs. A good quality orthophotograph of the study area and LiDAR contours, if available, could help locate and measure features accurately before proceeding to the field mapping stage.
ii) The existing data and observations from previous studies should be reviewed and subsequently verified. This is a valuable data set available at the early stage of the study.
iii) Abseiling/Rock climbing is a useful technique to aid mapping and direct assessment, in particular for a cliff or a steep hillside.
iv) The rockfall hazard identified can be economically mitigated by the combination of a flexible barrier and in-situ stabilization of individual boulders deemed to pose a significant hazard in the current setting.

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.

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Reconstruction of a Fill Slope at a Congested Site in Kwai Chung

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ABSTRACT

Rainstorms in August 2005 hit Hong Kong and triggered over 200 landslides. Among which, a major landslide with debris volume of 100m^3 took place at the lowest batter of a newly upgraded 24m high fill slope below a Public Rental Housing redevelopment in Kwai Chung. Subsequent investigation revealed that the upgraded fill slope possessed inadequate relative degree of compaction of 95% with the presence of unsuitable material in the fill body. Subsequent reconstruction of the subject feature involved re-compaction of the top 3m of general fill of about 5,000 m ^{3} and construction of the associated drainage works. While planning for the slope reconstruction, a 60m long footbridge was to be built spanning over this slope. The footbridge abutment and a 35m high lift tower were both encroached into the slope with mini-pile foundations at mid berm as temporary support for footbridge construction. The left-in mini-piles and the associated temporary works posed severe constraints to the slope reconstruction works in this already congested site.

During slope reconstruction, foreign materials were unveiled in vast area of the slope and previous underground drainage system was also exposed within the fill body. Design amendments were therefore made to accommodate these findings.

This paper presents the development history of the slope**,** highlights site constraints and conditions encountered during design and construction stages and methods to overcome them. Measures to enhance quality control of the construction works are presented and lessons learnt from the project are also discussed.

1 INTRODUCTION

1.1 Development history of the slope

Site formation works in Kwai Chung District was started in 1960s. The concerned slope was formed in 1964 between two sites for the construction of re-settlement estates at its crest and toe. Overall height of the slope was about 24m with an average slope angle of about 33 degrees. The slope was trapezoidal in shape of length of approximately 90m and 10m at slope crest and toe respectively. A 10 degrees platform was formed between slope toe and the re-settlement estate, and a latrine was later constructed on this platform Recompaction of the slope was carried out in late 70s and hard surfacing was later applied. The slope layout remained unchanged until 2000.

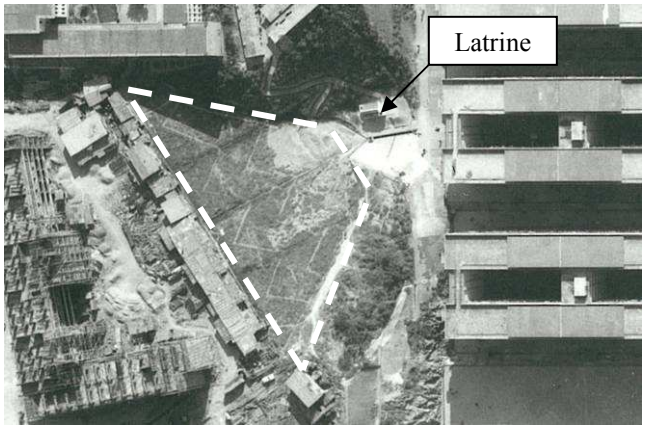

Plate 1 : Slope layout in 1964

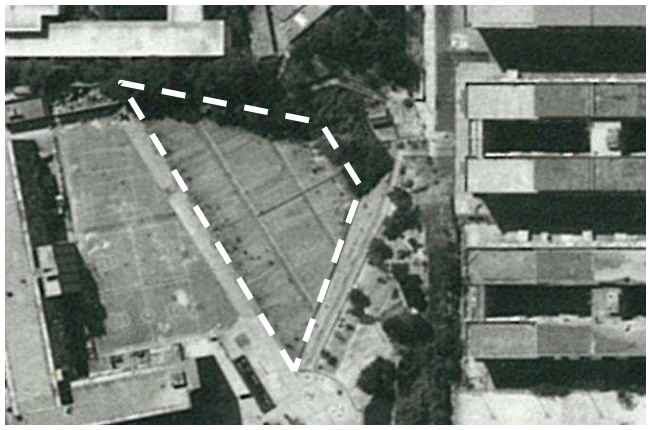

Plate 2 : Slope layout in 1984

In 2000, the re-settlement estate above the slope was considered for redevelopment and a geotechnical feasibility study was conducted. Ground investigation (GI) works, including drillholes and trial pits, carried out for the redevelopment revealed that the slope forming FILL material was composed of loose to firm, sandy to clayey silt with occasional cobble sized granodiorite and rhyolite fragments of thickness up to 5m overlaying localized medium dense to dense fine to coarse SAND of completely to highly decomposed granodiorite of thickness up to 11m. Bedrock of medium strong, moderately to slightly decomposed medium grained granodiorite varies from 5m to 15m below slope surface. Groundwater monitoring revealed that the groundwater table was located below rockhead.

Field tests results also revealed that the in-situ dry density of fill was below 90% of the maximum dry density (MDD). In addition, stability analysis at the critical section of the slope indicated that the slope possessed a Factor of Safety below the current safety standard and upgrading works was required.

1.2 Slope upgrading works in year 2000

In 2000, slope upgrading and preventive works included removing the existing hard surface for slope greening and recompacting the top 3m of the existing fill to achieve a relative degree of compaction of 95% of the MDD; making use of the toe platform and re-grading the existing 3-batter slope to a 4-batter slope of 30 degrees gradient; and providing a toe retaining wall at the new slope to accommodate the level difference at slope toe were proposed and implemented. Other typical measures included placing a granular drainage filters at the base of the new fill body and provision of surface drainages to the slope and green initiative of using vegetated slope surface were adopted.

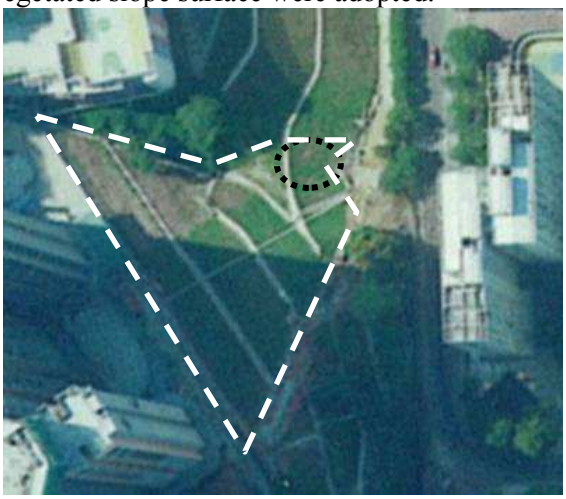

Plate 3 : Slope layout in Oct 2004
(Landslide location in Aug 2005 circled)

Plate 4 : Landslide at lowest slope batter (Aug 2005)

In August 2005, about six months after the completion of slope works, a landslide occurred at the lowest batter of the slope during which time Amber Rainstorm Warning and a Landslide Warning were in force. The landslide formed a scar of approximately 18m wide, 12m long and 0.8m deep. Post-landslide investigation, comprising three trial pits, seventeen GCO probings and fifteen insitu density tests by sand replacement method, were carried out in vicinity to the landslide area and the results revealed unsuitable materials and inadequate degree of compaction. Further field tests, including twenty-seven trial pits, seventy-eight GCO probings and fifty-three insitu density tests, at the entire general fill portion of the slope indicated that majority area of the fill was under-compacted. In addition, concrete fragments and other unsuitable materials were also found inside the fill body. To ensure the fill slope conforms with the current safety standard, the whole slope, involving re-compaction of 5,000m^3 of fill was considered necessary.

2 DESIGN CONSIDERATION TO ACCOMMODATE FOOTBRIDGE CONSTRUCTION

While planning for slope reconstruction after landslide, a lift tower and footbridge were proposed to be constructed over the fill slope to improve pedestrian access between the estate and a nearby Mass Transit Railway station. The footbridge, linking the crest platform and lift tower at toe, has a clear span of about 60m and is located at 35m above ground level. The redevelopment estate at slope crest had been completed and residents intake was in progress. The Emergency Vehicular Access (EVA) which runs along the slope crest has to be maintained in service at all time. The abutment of the proposed footbridge was therefore needed to be founded within the slope at crest. Similarly, the pedestrian footway had rendered the lift tower to situate at the small platform adjoining the slope toe. The lift tower equipped with three elevators had to be built encroaching partly into the slope.

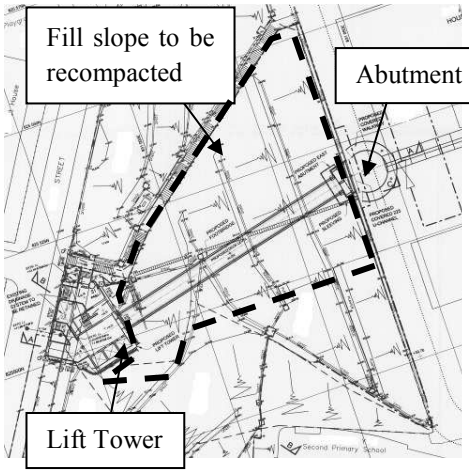

Figure 1 : Proposed footbridge and lift tower

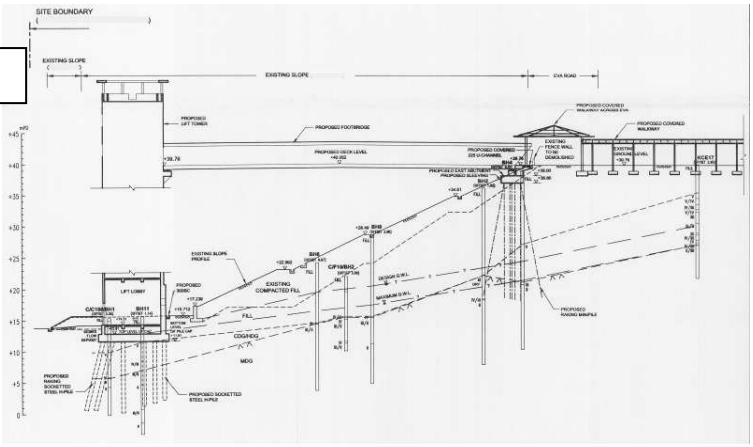

Figure 2 : Sectional view of the proposed footbridge and lift tower

Footbridge founding at slope crest would pose slope stability problem. Guided sliding bearings was provided to the abutment so that horizontal loads would only be taken up by the lift tower at slope toe. Perimeter structural wall was designed as a screen wall to minimize encroachment of the lift tower into the slope and reduce the extent of excavation and lateral support. Lateral earth pressure derived from 'At-rest' ground condition was adopted for structural design of the screen walls supporting the uphill batters.

The abutment at the highest slope batter had limited working space. Mini-pile foundation was adopted to reduce the required working area and to minimize the slope disturbance. The lift tower at slope toe adopted socketed Steel H-pile foundation to support the heavy structure. Raking piles were used in both foundations to enhance the lateral resistance of the structure against footbridge loadings. Compressible sleeving of 50mm thick polystyrene surrounding the pile cap at slope crest was provided to prevent any load transfer to the slope.

A new covered walkway connecting the footbridge and the domestic blocks at 30m from slope crest was proposed. The new covered walkway had to provide 5m of head room and 7m of clear span over the EVA

adjoining to slope crest. Relative large loading from this section of the covered walkway would pose slope stability concern thus the structure was designed to be supported on the footbridge abutment. For other footings that were located at least 7m beyond slope crest, assessment indicated that loading effect of shallow footings on slope stability was negligible.

3 SLOPE RECONSTRUCTION WORKS WITHIN A CONGESTED SITE

The slope reconstruction works in 2009 were aimed at removing all unsuitable materials in the existing fill body and recompacting top 3m of the slope to the original design profile of the upgrading works in year 2000. There was restricted access to the slope reconstruction works as the crest platform had already been occupied since 2005 and the lift tower had been built at the immediate slope toe. Another issue being that there was no area available for stockpiling of excavated soil materials and accommodating necessary site facilities.

The only available access to the site was via the adjacent existing slope. It was a 30 degrees cut slope upgraded together with the redevelopment in 2000 and its first berm was just 2m above the public road. A temporary haul road at about 16 degrees was therefore formed by cutting down one side of this existing slope to match with the road level. To further facilitate maneuver of heavy trucks at the site entrance, temporary traffic arrangement (TTA) proposal with closure of one lane of the dual lane public road during the reconstruction works period was prepared.

Site entrance is fronting the EVA of the estate located below the slope, which runs perpendicularly with the public road. Closure of one lane of the public road may pose maneuver difficulties for long fire engine to turn into the EVA. Efforts were made to liaise with relevant government authorities including Highways Department, Transport Department, Hong Kong Police Force and Fire Services Department to resolve their comments so that a practicable arrangement could be worked out. Approval to TTA proposal was successfully sought at the early stage of the construction period.

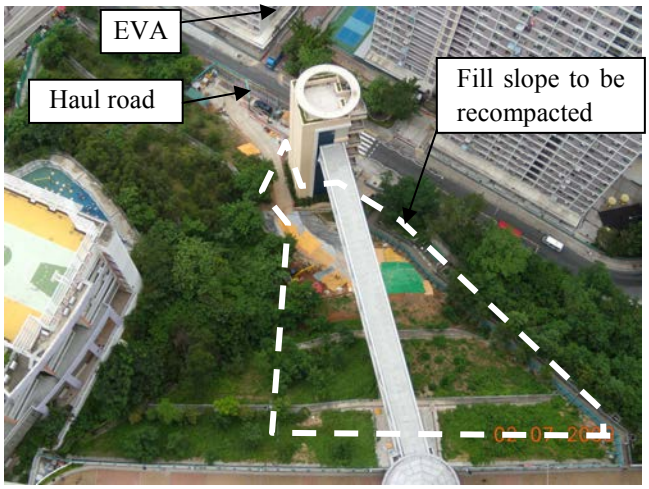

Plate 5 : Temporary haul road on adjoining slope

Plate 6 : Suitable fill excavated from upper slope batter used in recompacting the lower slope batter

Stockpiling of materials on site was not feasible. Off-site stocking area was sourced but no nearby vacant site was available. A works sequence was therefore devised to utilize the existing soil materials excavated from the immediate upper batter. Unsuitable debris or over-sized concrete fragments was screened at the spot of excavation and disposed off site. Screened soil fill was then transported to one slope batter below the location of excavation at where slope recompaction was performed. Recompaction therefore started from the lowest batter and worked its way upward to the slope crest. However, during excavation at mid slope, constraints from a number of left-in mini-piles which was installed to provide temporary support of the falsework system for bridge construction were also exposed and had to be removed to allow recompaction of the fill. Fill for recompacting the highest batter had to be imported from other source. The imported fill was transported to the slope via the EVA at slope crest. Particular care was taken for transport and placing of fill

at this location to ensure safety of estate residents and minimize nuisance. With good planning of works sequence and timely arrangement of lorries for disposal of unsuitable materials off site, the recompaction works were carried out efficiently within this congested site.

The site is bounded by public housing estates at slope crest and toe hence nuisance generated by the works could affect the residents in close vicinity. At the commencement of the site works, briefing on works details was given to estate management offices so that the estate residents could be informed of the upcoming construction works. Communication channels were also maintained with the management offices of the estates throughout the contract to ensure that any complaints could be effectively addressed with prompt actions.

4 DESIGN AMENDMENT DURING SLOPE RECONSTRUCTION

During excavation for recompaction at the lowest slope batter, a suspected manhole with two drainage pipes, a 300mm dia PVC pipe and a 250mm dia cast iron pipe, connected to its upstream end were found. The drainage pipes and the manhole appeared to be part of a dis-used culvert. Further excavation at the downstream end of the culvert revealed a brick wall fitted with a 200mm perforated pipe connecting to the newly constructed retaining wall further downhill. A disused u-channel blocked with concrete was connected to the sidewall of the culvert. The culvert was filled with debris. Further investigation revealed that the PVC drainage pipe was blocked by concrete. Results of Closed Circuit Television (CCTV) Survey indicated that the cast iron pipe was intruded by existing mini-piles of the temporary support for footbridge construction. No water outflow from these drainage pipes was observed after a typhoon during the slope works construction period in 2009.

Excavation also revealed an underground concrete structure filled with no-fines concrete. However, the voids between the aggregates had already been filled up by the rich cement paste of the no-fines concrete. The concrete structure was suspected to be a septic tank of the former latrine left-in place by a previous contractor. Other unexpected materials included a concrete platform, debris composed of plastic sheets and bottles and a concrete-filled U-channel were exposed in the fill adjacent to the culvert. Granular filter material that might be the filter layer placed by previous contractor was also exposed. However no sub-soil drainage pipes was provided to this filter layer to discharge the infiltration intercepted.

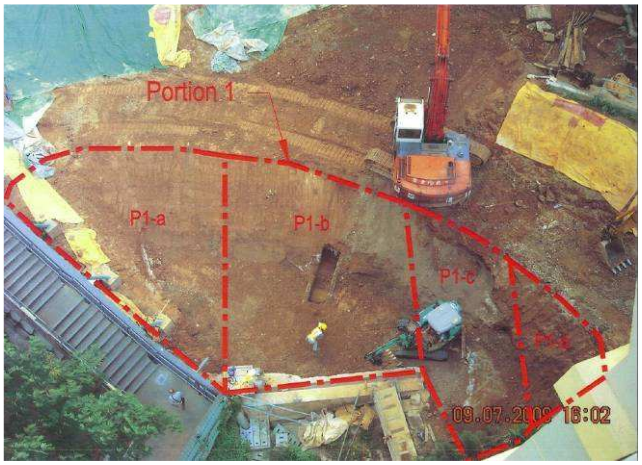

Plate 7 : Exposed culvert during reconstruction

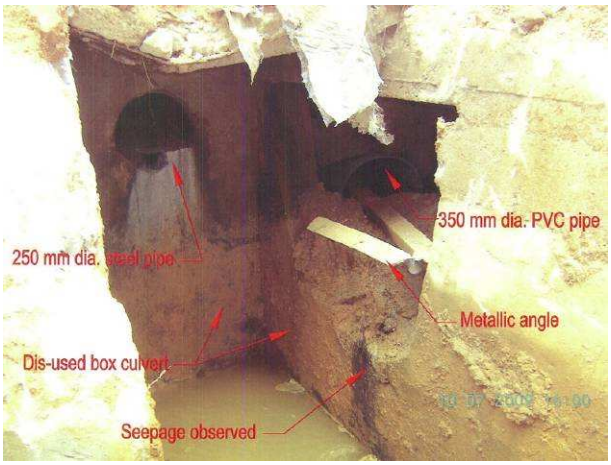

Plate 8 : Drainage pipes inside culvert

The dis-used culvert, concrete platform and impermeable debris materials may prohibit rainwater from infiltrating into the ground. Filter layer without drainage outlet would accelerate perched water table buildup and create a local spot of high water pressures hence posing stability concern to the slope. The disused culvert was therefore demolished and a new catchpit was constructed to account for possible water inflow from the abandoned PVC and cast-iron pipes. The new catchpit was filled up with porous no-fines concrete to provide sufficient resistance to overburden soil load and at the same time maintaining its drainage function. All

unsuitable debris material and the concrete platform were removed and a new filter layer was provided at the base of the fill body and above the underground catchpit. To further safeguard the slope, particular the lowest slope batter, from ground water rise due to shallow rockhead, a series of 10m long prescriptive raking drains were installed into the old fill below the granular filter as an additional precautionary measure.

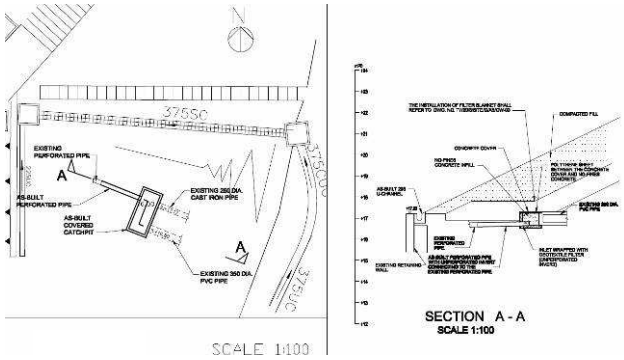

Figure 3 : Details of new underground catchpit

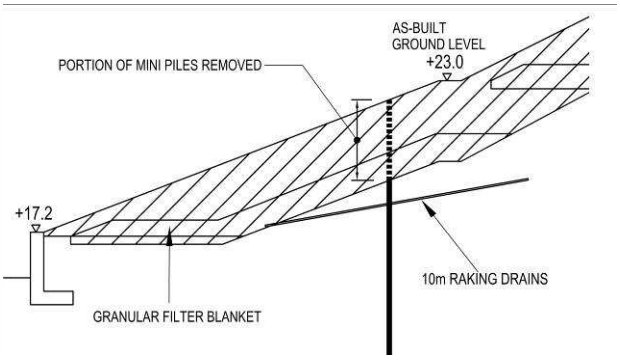

Figure 4 : Raking drains below granular filter layer

These findings of unexpected old underground drainage pipes and structure were revealed after the slope reconstruction had commenced, hence any delay in providing solution would attract contractual claims for extension of time and additional cost from the contractor. The design amendment including seeking approval from checking authority was therefore completed within a very short period of time in order not to delay the contractor's progress and induce contractual claims.

5 ENHANCEMENT TO SITE SUPERVISION AND QUALITY CONTROL ON SITE WORKS

With the deficiencies identified from the findings on the quality of upgrading works executed in Year 2000, quality of fill may vary substantially between batches of soil materials. The following measures were therefore implemented to enhance quality control of the compactions works so as to achieve satisfactory compaction results:-

- Conduct trial compaction to verify plant capability before carrying out the actual soil compaction works.
- Conduct policing test by Direct Testing Laboratories appointed by Housing Department and Public Works Laboratory, in addition to compaction tests carried out by the contractor's laboratories;
- Conduct Proctor Tests for each layer of fill compacted and whenever there was variation in fill materials;
- Carry out sand replacement tests at top, middle and bottom of each fill layer compacted to ensure the required standard is achieved throughout each layer;
- Carry out over-filling can ensure achieving of 95% relative degree of compaction near final finished slope profile;
- Deploy site staff with appropriate supervision experiences on slope works as full time Category III site supervisor and Category I site supervisor.

With all the measures implemented, reconstruction of the slope in a congested site, involving recompaction of 5,000 m^3 of fill, was completed in fourteen months. No contractual claims, dispute nor accident had been reported throughout the contract period. The completed slope was also finally given the Certificate for Design and Completion for Slopes and Retaining Walls works issued by Geotechnical Engineering Office.

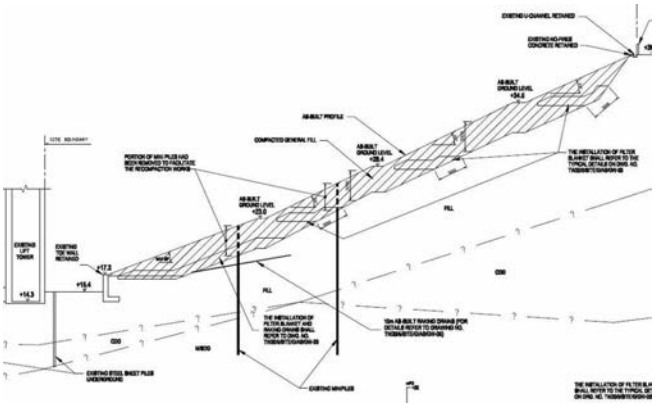

Figure 5 : Sectional view of the works after reconstruction

Plate 9 : General view after reconstruction (May 2010)

6 CONCLUSION

Reconstruction works had been carried out for a slope of which the post-landslide investigation indicated that the quality of previous works might not be up to the required standard. The design and reconstruction works had overcome multiple constraints. Enhancement to site supervision and quality control were implemented and the works were completed successfully. Lessons learnt from this project works are summarized below:

- Detailed desk study and site investigation can provide essential information to redevelopment sites. Proper keeping, retrieving and studying of previous maintenance and alteration records can greatly reduce the need to make urgent design amendments during construction stage.
- Quick design review and close liaison with checking authority during construction can help maintain site progress and forestall contractual claims.
- Structural screen wall, instead of self-standing retaining wall, can be an effective form of ground support in congested sloping site.
- Arrangement of adequate tests and deployment of experienced site staff to carry out works supervision can effectively monitor works quality hence reduce the chance of having unsatisfactory works not being discovered.
- Good planning of working sequence for congested site can eliminate the need to find suitable stockpiling area, reduce the need of transportation of soil, and minimize import fill from external sources, thus reducing carbon emissions from such activities.
- Maintaining of close liaison with estate management office can enhance public understanding on the slope reconstruction works and can ensure complaints are effectively addressed with prompt actions.

ACKNOWLEDGEMENTS

The Authors are grateful to the Director of Housing for his permission to publish this paper and to his colleagues of various discipline and sectors who had contributed to the design and construction of the project.

Excavation for Construction of an Adit Connecting a Housing Development to MTR Station

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ABSTRACT

To facilitate pedestrian movement with improved accessibility between an existing Mass Transit Railway (MTR) station and a public housing estate under redevelopment, an adit connecting the MTR concourse and the Ancillary Facilities Block (AFB) within the estate was constructed. Aligned parallel to the MTR station with a clear distance of approximately 2.2m, the lift shaft and adit structure was constructed with great difficulties particularly in the temporary Excavation and Lateral Support (ELS) works. Challenges include: close proximity to MTR structure posing a risk of damaging existing structure during installation of sheet piles; risk of ingress of groundwater causing flooding into the MTR station; high groundwater level so that groundwater drawdown may result in risk of settlement at MTR structure and adjacent features; weak subsurface geological condition; tight project programme etc. This paper presents the site constraints, the corresponding design considerations and the construction measures and monitoring for the ELS works. The difficulties encountered during the construction works and project experiences are discussed.

1 INTRODUCTION

In a public housing redevelopment in 2011, an adit connecting an existing MTR station and a proposed AFB was constructed. The size of the adit structure is about 19m x 13m with connection to the AFB by a lift shaft. The other end of the adit was connected to the MTR concourse by breakthrough of the MTR station wall. In order to construct the underground adit, a 13m deep cut-and-cover ELS system comprising sheet pile walls with horizontal steel struts and walings was adopted. Layout plan of the adit structure and its associated connections with AFB and MTR station are shown in Figure 1. An illustrative ELS plan, cross-section and a general view are presented in Figure 2, Figure 3 and Plate 1 respectively. This paper outlines solutions implemented to tackle the construction difficulties arising from various site constraints.

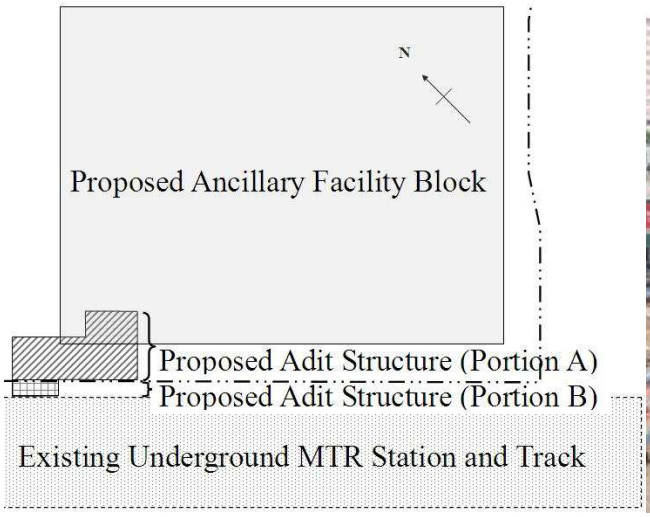

Figure 1: Layout plan of the adit, AFB and MTR Station

Plate 1: General view of ELS works for the adit

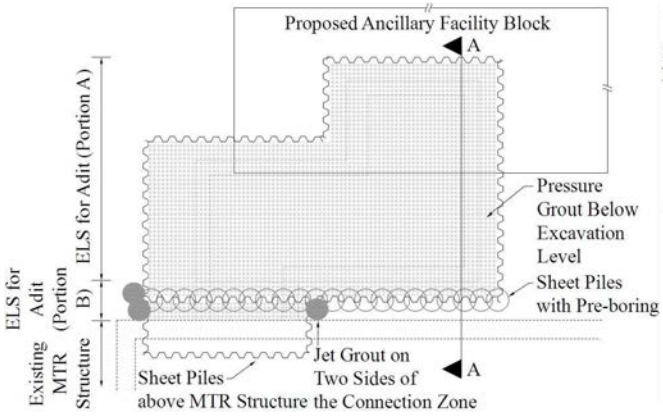

Figure 2: Illustrative ELS Plan for the Adit

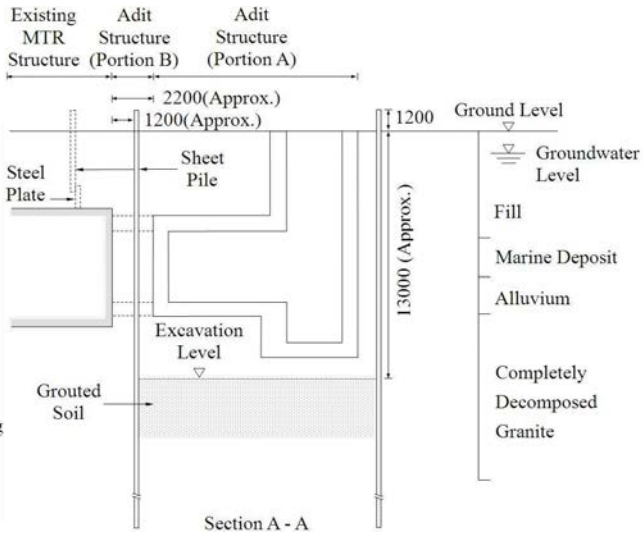

Figure 3: Section A-A of the ELS Works
(Note: Lateral supports are not shown for clarity)

2 BACKGROUND AND SITE CONSTRAINTS

The site was situated in an old reclamation area in Kowloon. Former factory blocks occupying the site were demolished to make way for the redevelopment. Geological condition of the site is underlain by fill, localized marine deposit, alluvium, completely decomposed granite and bedrock of granite.

Constraints for the construction of the adit are summarized as follows:

- While allowing working space for adit construction, the remaining clearance between the MTR station and the sheet piles of the ELS system for Portion A was merely about 1.2m. In the process of installing sheet piles, if obstructions such as localized corestones and/or remains of old concrete piles were encountered, this would pose a high risk of damaging existing MTR structure. Some sheet piles were installed on top of the MTR structure. They were even closer to the MTR structure.
- The groundwater table is high and very close to the ground surface. This increased the difficulty in design and construction of the ELS works. Prevention of water ingress into the MTR station throughout the construction was also one of the major concerns.
- Owing to the high groundwater table, settlement of the MTR structure and adjacent features due to groundwater drawdown was carefully considered and prevented.
- The site was overlain by weak superficial deposit of fill, marine deposit and alluvium, hence challenges were posed on wall deflection control.
- A tight construction programme with many critical activities for the adit construction had to be followed in order to meet the planned housing production.
- Beside the adit, other concurrent constructions of the project were also affecting the MTR structures. As MTR Corporation Limited (MTRCL) has limits on allowable impacts on MTR structures for the entire project (PNAP-APP24), the remaining limits solely allocated for adit construction were very stringent. The values of these limits are presented in Table 1.

Table 1. Limit allowed for ELS works of the adit

	Limit stated in PNAP APP-24	Impact due to other constructions	Limit for ELS works of the adit
Pressure change	20kPa	8kPa	12kPa
Total movement	20mm	8mm	12mm

3 DESIGN AND CONSTRUCTION CONSIDERATION

3.1 Consideration for construction sequence

The construction sequence was arranged to minimize adverse effects on the MTR structures. Construction of the adit was divided into two portions, namely A & B, as shown in Figure 1. Excavation and construction for Portion A, being the first portion, were carried out concurrently with the construction of the south-western part of the AFB raft foundation. This was then followed by the superstructure of the AFB. Lastly, the excavation and construction of Portion B, which is the connection between the adit and the MTR station, was constructed. This sequential arrangement allowed immediate settlement of the AFB to take place before completion of adit connection so that differential settlement and lock-in stress at the connection could be minimized. Moreover, the risk of water ingress into MTR station was reduced by deploying the two-stage excavation, because breakthrough at Portion B was conducted only in the final stage. Construction sequence of the adit was designed at the planning stage and specified as a contractual requirement such that the scheme was followed by the contractor accordingly.

3.2 Consideration for sheet piles installation

Sheet piles were mainly installed by vibratory hammer except for several areas subjected to special treatments as presented below.

Concern over verticality and vibration during installation of sheet pile parallel to the MTR structure was keen, as the verticality of sheet piles was susceptible to obstruction of boulder and remains of old concrete piles. Moreover, clearance between sheet piles and existing MTR structure was extremely narrow. In view of this, pre-boring with sand backfilling for sheet piles along MTR alignment was incorporated. Augering method was employed for pre-boring to minimize vibration effects. Down-the-hole hammer would be used if obstruction was encountered. However, vibration and ground loss due to the use of down-the-hole hammer were areas of concern. A trial test of boring by down-the-hole hammer was carried out for addressing boring parameters such as advance drilling rate and pressure, in order to further assure the impact to adjacent ground was kept minimal.

To further protect the MTR structure, press-in piling method was adopted for installation of sheet piles along the pre-boring alignment. As shown in Plate 2, the machine makes use of hydraulic rams to provide the force necessary to jack a sheet pile into the ground based upon skin friction and adhesion of the adjacent driven sheet piles (Finlay et al. 2001). Comparing with the vibratory hammer, press-in piling method could significantly reduce vibration of the nearby sensitive structures. In addition to the aforementioned measures, verticality of the sheet piles and pre-boring holes were checked by inclinometer during installation to ensure adequate protection for the MTR structure. A guided steel frame was also used to ensure the installation alignment of sheet piles.

Plate 2: Installation of sheet piles by press-in piling method

Aside from challenges with sheet piles installation along MTR alignment, some sheet piles for Portion B were also planned to be installed above the MTR station. Damages in any form of the MTR station could not be tolerated. By means of two hand-dug inspection pits prior to sheet pile installation, level of the station roof was verified. Steel rods installed on the sheet piles would prevent over-penetration of sheet piles. The sheet piles were installed to 1m above the logged level to minimize risk of damaging the structure, followed by hand-dug excavation with steel plates installed below the toe to serve water cut-off purpose.

3.3 Consideration for hydraulic cutoff

Since the MTR structure is founded directly on soil, its movement is susceptible to settlement of ground due to groundwater drawdown during the excavation works. Another consideration was that ingress of water into MTR station was highly undesirable. In terms of groundwater control, stringent hydraulic cutoff measures throughout every stage of construction were incorporated. Major considerations are presented below.

3.3.1 Use of grouting inside the excavation area

In order to enhance water cutoff of the ELS system, grouting comprising cement grout and chemical grout by tube-a-manchette (TAM) were applied. Grouting was carried out inside the ELS system below the excavation base, instead of along the gap between sheet piles and the MTR structure, to minimize adverse effects on the MTR structure during the grouting operation. The TAM grouting inside the excavation not only reduced groundwater seepage from the excavation base, but also enhanced soil stiffness for the control of wall movement during the course of excavation. Prior to grouting, some trials of grouting were performed to determine the suitable grout composition to meet the hydraulic cutoff requirement.

3.3.2 Use of jet grouting at the connection/breakthrough area

Stringent hydraulic cutoff control in Portion B of the ELS system was required to prevent water ingress to MTR station after breakthrough of the MTR station. The clearance in Portion B was too narrow for sheet pile installation as it may damage the water proofing of the existing MTR structure. Jet grouting was preferred over TAM grouting for this sensitive zone, as it results in more certain water cut-off efficiency.

Jet grouting can be defined as a method based on the introduction of hydraulic (sometimes combined with pneumatic) energy in order to erode soil and mix or replace the eroded material with an engineered grout to form a solidified in-situ element. (Brill et al. 2003)

Jet grout columns at the two sides of connection were formed to provide both water cutoff and temporary retaining functions. A trial of jet grout column was conducted on site to check the effective grout column diameter by retrieval of grouted soil samples for inspection. A core sample of the trial jet grout column is shown in Plate 3. The core sample showed that a rigid and impermeable cemented soil was formed within the designed effective zone. Grouting parameters such as withdrawal speed, rotation rate, grout and air pressure, grout injection rate and grout mix proportion were reviewed to suit the actual site condition.

Plate 3: Core samples of jet grouted soil

3.3.3 Prevention of rainfall and surface water

The amount of water entering the excavation area had to be minimized as much as possible. Some precautionary measures in this regard were deployed to guard against water from sources other than ground water. The top of sheet pile wall was installed 1.2m higher than the ground level to prevent flooding of surface water from entering the excavation area. In addition, a retractable rainfall cover was provided to protect the excavation area from rainstorms.

3.4 Monitoring

Owing to constraints brought by nearby sensitive structures, monitoring works were crucial during construction. Settlement markers along MTR tracks, inside station and on adjacent ground were set up for monitoring. Vibration markers and piezometers were also used to monitor the impacts brought by the ELS works. Automatic groundwater alarms were installed to alert on site personnel once the limit of groundwater level was reached.

4 CONSTRUCTION DIFFICULTIES

Several construction challenges were faced during the project: minor ones such as the realignment of sheet piles due to encountering old concrete piles, as well as the narrow working space inside the ELS system. Two major difficulties are outlined below.

4.1 Incompletion of pumping test

A pumping test was conducted to verify effectiveness of the hydraulic cutoff system. Dewatering wells and observation wells were installed on site. During the test, dewatering was conducted to lower the groundwater level inside the sheet pile wall to the excavation level. The observation wells inside the ELS system showed that the rate of groundwater drawdown was exceptionally slow during dewatering. With this exceptionally slow rate, time for completing the pumping test would pose a delay to the construction programme. The slow dewatering rate might be due to uncontrolled penetration of grout above the intended grout zone level, which has reduced the permeability of soil in test zone.

In response to that, the dewatering was terminated after 72 hours of operation followed by 48 hours of monitoring of observation wells. Recovery of groundwater level inside excavation area was found to be slow which partially justified the good quality of water cut-off system. Notwithstanding, several actions were taken to reduce the risk of excavation without a complete pumping test. First, the permeability of soil to be excavated was back analyzed based on the available data of the pumping test. The result was then put into a computer model to refine the prediction of groundwater drawdown at every stage of excavation. At each stage of excavation, the actual geological and groundwater condition was reviewed in comparison with the computer predicted condition to check the design assumption and ensure that the groundwater drawdown was properly controlled. Additional measures including provision of recharging well and increasing monitoring frequency were set up.

4.2 Difficulty of sheet pile installation

In the course of sheet pile installation in the zone without pre-boring, it was difficult for some of the sheet piles to penetrate to the targeted depth. Removing the sheet piles followed by pre-boring would have profound time implications to the project. An alternative was explored, as to whether an enhancement in the soil strength due to grouting inside the excavation would allow a reduction in the penetration depth requirement of sheet piles. The grout zone was treated as a “grouted soil slab” to reduce the sheet pile wall movement and ground settlement. As the performance of the grouted soil was uncertain, nine verification drillholes were sunk. Standard Penetration Tests (SPT) were carried out and triaxial compression tests were conducted on Maizer samples obtained from the grout zone. Test results for SPT and triaxial compression test for the in-situ and grouted soil are plotted in Figure 4 and Figure 5 respectively. The tests revealed that the soil was improved with significant increase in SPT “N” values and shear strength of the soil in the grout zone.

Eventually, stability of the as-built ELS system taking into account the enhanced strength of grout zone soil was checked to be satisfactory.

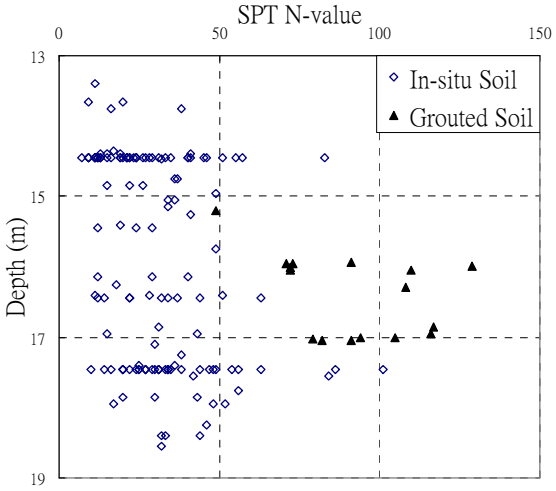

Figure 4: Comparison of SPT results

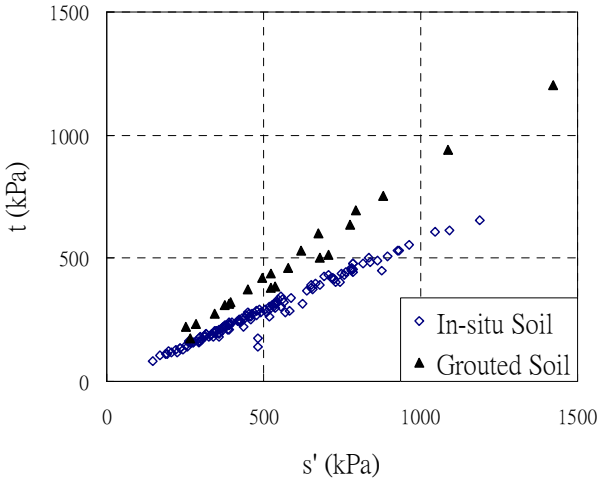

Figure 5: Comparison of s'-t plot

5 CONCLUSION

Despite many challenging constraints, the project was completed on time with satisfactory performance. No undue settlement on the adjacent ground and MTR structures were observed. The monitoring records showed that impacts on surrounding ground and structures were small and existing site conditions were found in agreement with the design assumptions.

Lessons learnt from this project are summarized below:

- Effective communication between different parties, including Housing Department, the contractor and her consultant, MTRCL etc. determines the success of the project.
- Use of alternative technologies such as press-in method for sheet pile installation and jet grouting could effectively resolve actual site problems.
- Advanced planning of construction sequence and specifying it in the contract is an effective mean to address construction and design difficulties.
- Trial tests before commencement of actual works further alleviate the risk of actual construction particularly for works next to sensitive structures.
- Enhanced strength of grouted soil could be utilized for stability analysis of the ELS system given that sufficient control and verification tests are conducted.
- Proper survey and record of old foundations during building demolition could facilitate the construction in redevelopment projects.

ACKNOWLEDGEMENTS

The authors gratefully acknowledge Director of Housing, the Government of the Hong Kong Special Administrative Region for permission to publish this paper. Special thanks to all parties involved for their efforts throughout the design and construction stage. The authors wish to thank Mr. Charles C.L. Chan for support in the preparation of this paper.

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The Use of Ground Improvement Techniques at the Nam Fung Portal, South Island Line (East), Hong Kong

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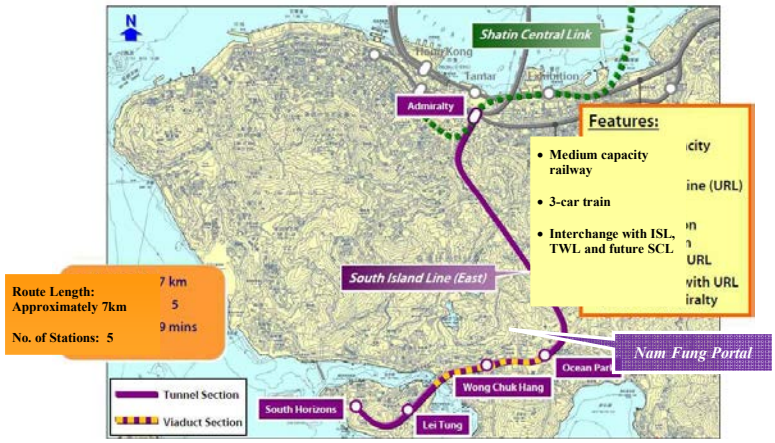
ABSTRACT

The South Island Line (East), (SIL(E)), will extend the existing Mass Transit Railway Corporation Limited (MTRCL) network from Admiralty to the Southern District of Hong Kong Island. The SIL(E) is divided into several works contracts, of which Contract 902 was awarded to Nishimatsu Construction Company Limited (NCC) in July 2011. Major components of the Contract 902 works involve construction of ventilation buildings at Nam Fung Portal (NFP) and at Hong Kong Park, connected by 3.2km drill & blast tunnel excavation. The NFP, constructed in sloping terrain, comprises a piled shaft and transition box structure with lateral supports, through which the tunnel portal was excavated. The initial tunnel drive runs directly below Nam Fung Road. To mitigate the potential risk of settlement of Nam Fung Road and other sensitive receivers above and in the vicinity of the portal, pre-excavation canopy grouting was carried out from the surface down to tunnel level to form a 3m grout canopy perimeter around the tunnel before tunnel excavation. In addition prior to tunnel portal excavation "All Ground Fastening" (AGF) fore-poling was adopted. This combination provided a successful and effective groundwater cut-off against fluctuating groundwater levels and enhanced support thereby effectively reducing displacement. Many of today's site formations in Hong Kong, whether for temporary site formations such as the NFP or more permanent site formations are carried out in areas of limited land availability. These sites will encounter sensitive receivers which require protection against displacement. This paper provides a brief background on the construction of the NFP and details the innovative ground improvement methods used to mitigate displacement of the Nam Fung Road situated above the portal which could be applicable for other site formations currently being carried out with similar site constraints.

1 INTRODUCTION

A significant portion of the forthcoming MTR SIL(E) construction runs underground through a tunnel excavated beneath Mount Cameron which rises to approximately +325mPD on Hong Kong Island. The tunnel, constructed as part of Contract 902 extends between Admiralty Station to the north and the Nam Fung Ventilation Building to the south. The SIL(E) line extends from the NFP to new stations at Ocean Park, Wong Chuk Hang, Lei Tung and terminating at the South Horizons station in Ap Lei Chau (Figure 1).



.Figure 1: MTR South Island Line (East) Alignment (MTRCL, 2009)

During construction of the Aberdeen Tunnel Portal in 1977 and more recently the Hong Kong Electric Cable Tunnel Portal at Nam Fung Road in 2004 excavated in similar geological & hydrogeological conditions found at NFP, tunnel and portal slope instability in soft ground and high groundwater inflows were encountered on these projects (Cochrane, R.A. 1984, & Bevan, O 1984). Therefore a robust approach was taken for construction of the NFP to mitigate risks from potential adverse ground and groundwater conditions.

As part of this construction the tunnel portal and shaft, referred to as the NFP has been formed from a pipe pile excavation and lateral support structure and transition box, refer to Figures 2 & 3. The NFP was formed in an area of relatively complex ground and groundwater conditions including variable bedrock levels and partially weathered rock, saprolite with corestones and superficial deposits with variable boulder to silt sized constituents (referred to as mixed ground) and a fluctuating groundwater table including perched groundwater conditions. As the portal is surrounded by sensitive receivers, including Nam Fung Road and existing utilities, located above the portal crest and St. Paul's College to the East, intensive monitoring to check ground displacement is required and innovative ground improvement techniques and robust construction methods were adopted to mitigate risks against such displacement.

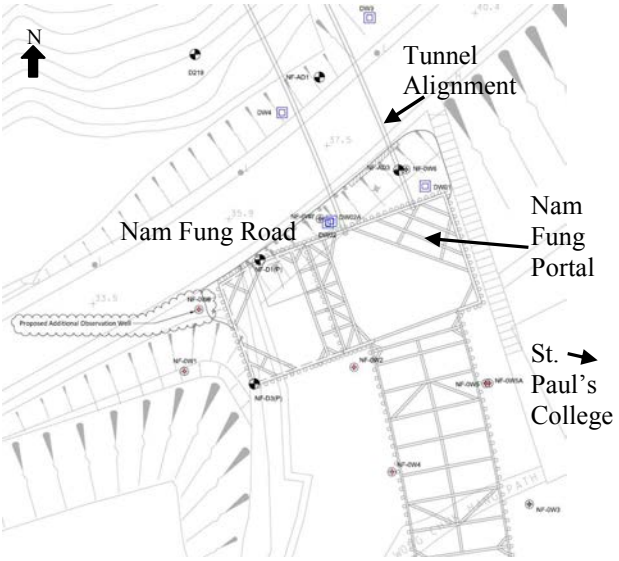


Figure 2: Nam Fung Portal Location

Figure 3: Nam Fung Portal, Aberdeen

Pre-excavation canopy grouting comprising a two stage injection of a Bentonite Cement (BC) followed by Silicarizer chemical grout injection through a Double Tube Double Packer Grouting (DDG) system within each grout hole was provided by specialist grouting subcontractor Raito Kogyo Company Limited (Raito). In addition, prior to tunnel portal excavation "All Ground Fastening" (AGF) fore-poling was adopted and pre-excavation canopy grouting using BC and chemical grouts was provided by Raito. This combination provided an effective and successful groundwater cut-off against fluctuating groundwater levels in addition to enhancing ground conditions thereby reducing displacement.

2 GROUND CONDITIONS & HYDROGEOLOGY

Based on the findings of the site investigation (SI) a summary of the main ground units in the vicinity of the NFP excavation are summarized below. The geology and ground conditions encountered are presented in Figures 4 to 6 and summarized below:

- Fine Ash Vitric Tuff / eutaxite (Ap Lei Chau formation), comprises welded fine ash vitric tuff with inter-layered eutaxite (bedded tuff) bands (Sewell et al, 2000). The tuff has a maximum Unconfined Compressive Strength (UCS) of up to 400MPa, typically has closely spaced (60 to 200mm spaced) joints and is blocky, angular and welded (GEO, 2007). The rock weathers to clay / silt saprolite, up to say 15m depth, and rarely forms corestones. The eutaxite is typically welded, therefore has a reduced

weathering depth however has a greater joint intensity and strength anisotropy parallel and perpendicular to the bedding (GEO, 2007).

- Quartz Monzonite (Tei Tong Tsui formation), comprises east north east to west south west trending dykes up to 50m thick, South HK island. Crystal constituents range from sizes of porphyritic, fine to medium grained (Sewell et al, 2000). It has a similar UCS to granite, maximum 250MPa, a lower free silica (quartz) and greater feldspar content, and as a result, is more prone to disintegration and decomposition weathering effects. In its completely decomposed state it is clay rich with frequent kaolin bands of up to 50mm thick infilling relict joints.
- Superficial deposits (Fanling formation). The deposits include alluvium and colluvium (debris flow deposits). The alluvium comprises well sorted yellowish brown, clay, silt, sand and gravel; the debris flow deposits typically infill valleys and comprise poorly sorted boulders and cobbles in a gravelly matrix. (Fyfe et al, 2000). The superficial deposits were located above the tunnel horizon (Figure 6).

The SI also revealed that photo-lineaments trending north-west-north to south-east-south, approximately perpendicular to the Aberdeen Fault, were located both east and west of the NFP. The mapping information, carried out as the tunnel excavation advanced north, revealed that partial weathered (PW), zones (comprising both soil and rock), were present with rock to soil ratios (PW grades) ranging from PW30/70 to PW50/90 (GEO, 1992). These zones were potentially present immediately above the tunnel invert and in the vicinity of the NFP.

Typically the permeability of saprolites, weathered from the parent rock anticipated in the NFP, would range from 10^{-6}m/s for granite (monzonite) and 10^{-7}m/s for fine grained tuff (GEO, 2000). The results of 8 packer tests carried out in the vicinity of the tunnel excavation, revealed a greater range, varying from 10^{-5}m/s to 10^{-8}m/s, influenced by the variable geological and fluctuating groundwater effects.

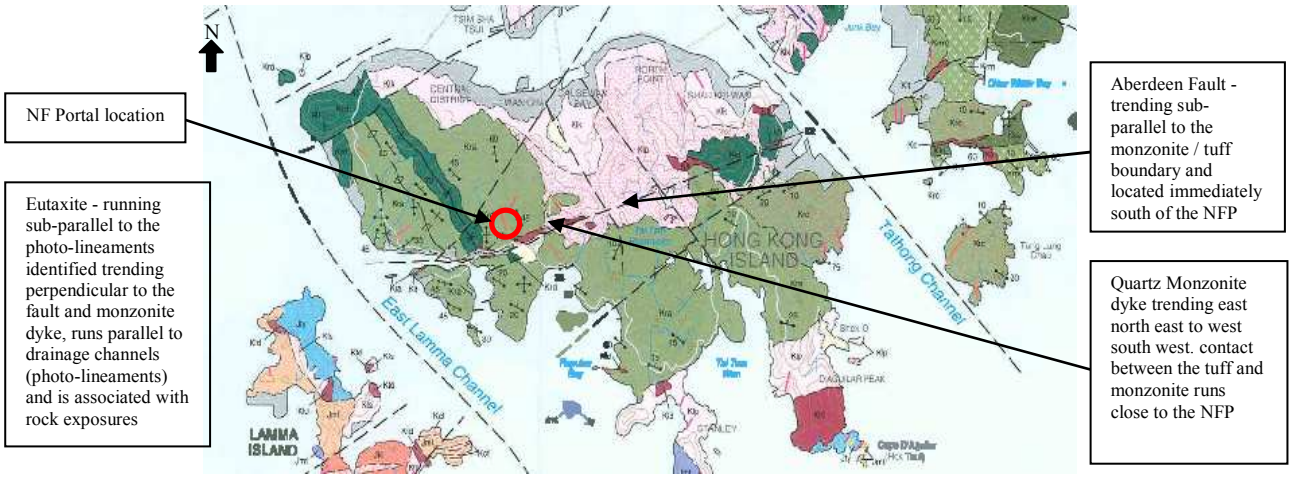


Figure 4. Geological Map, Sewell et al (2000)

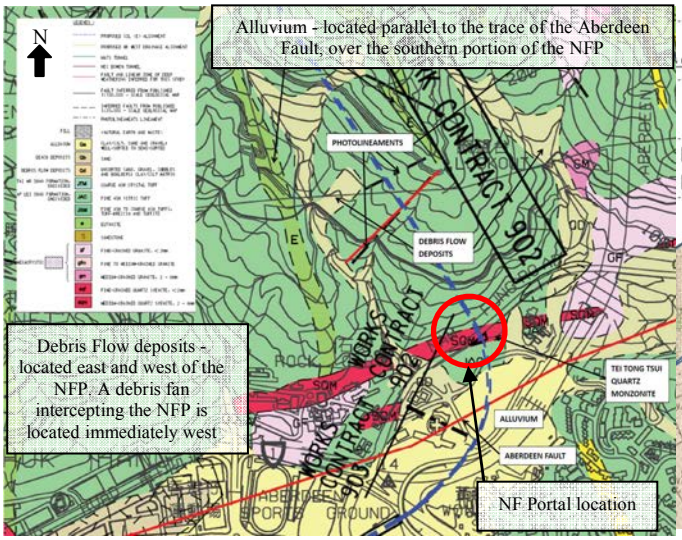


Figure 5. Geological Map, 1:20,000, HKGS (1996)

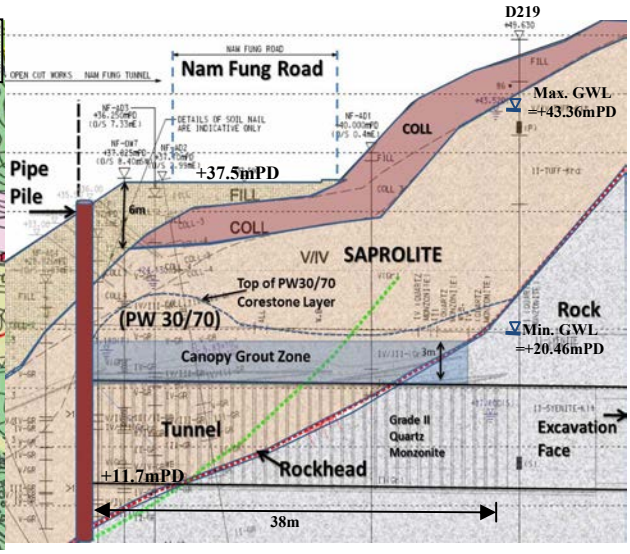


Figure 6. Geological Cross Section through portal

Groundwater records taken from piezometers installed in the vicinity of the portal also revealed rapid fluctuations during periods of heavy rainfall. This was further substantiated from long raking drain installations installed above the canopy grout zone (Figure 7) which recorded significant water inflows immediately after periods of heavy rainfall.

3 TUNNEL PORTAL PRE-SUPPORT AND CONSTRUCTION SEQUENCE

The tunnel portal pre-support is presented in Figures 7 and 8 below and comprised:

1. An initial pre-excavation canopy grouting drilled from surface (3m zone) prior to tunnel excavation to provide a groundwater cut-off and improve the ground by increasing shear strength;
2. Installation of 6 long raking drains (30m to 45m) long above the canopy grout zone (with inclination of 3°-4°) to facilitate groundwater drainage surrounding the tunnel excavation zone.
3. Horizontal steel pipe pile (114mm dia.) canopy roof was installed to provide structural support during excavation;
4. Injection of cementitious grout through pipe pile perforations; and
5. Split top heading excavation was adopted and installation of steel arch ribs at 1m center spacing during tunnel excavation together with application of 250mm thick shotcrete reinforced lagging.

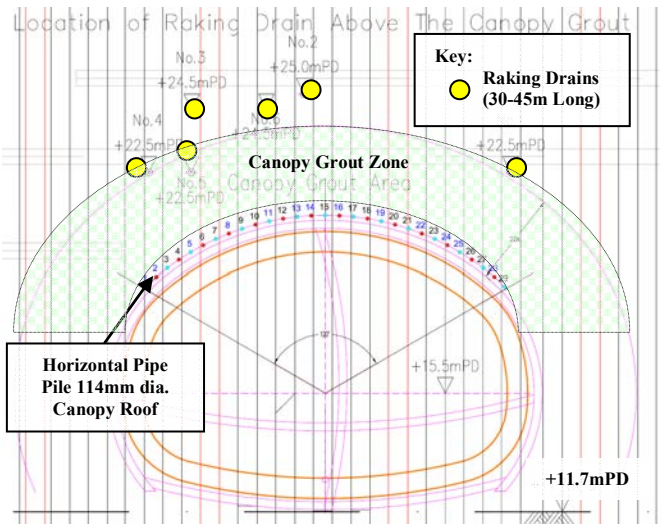


Figure 7: NFP Pre-support / Pre-grout Zone Configuration

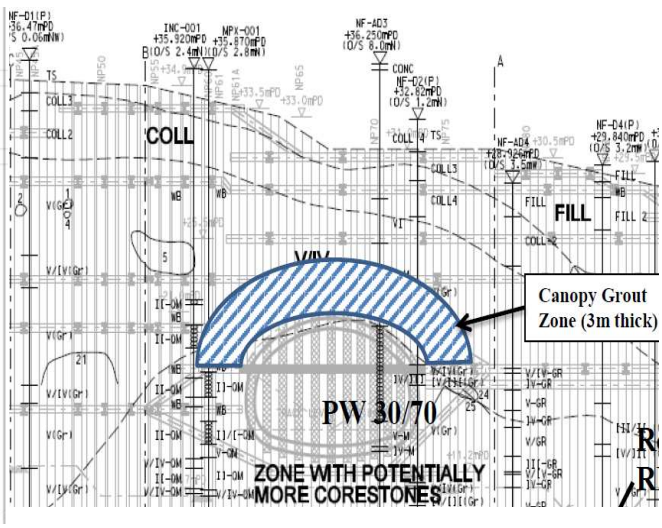


Figure 8: Canopy Grout Zone

4 THE PRE-EXCAVATION GROUTING SYSTEM AND TARGET

The pre-excavation grout canopy installation involved injection through a Double Tube Double Packer Grouting (DDG) system, with target reduction in permeability to 10^{-8}m/s. The grouting technique involved an initial injection using Bentonite Cement (BC) followed by Silicarizer chemical grout, refer Figures 9 & 10. The maximum grout hole spacing was approximately 1400mm between the base of each grout hole (Figure 11), with grout arrays drilled along 14 individual gridlines (references A to N, Figure 12). The spacing was based on the estimated grout flow properties and anticipated void ratio / permeability of the ground into which the grout be injected; the grout canopy extended to a maximum 3m thickness around the tunnel excavation perimeter. The injection was in stages through the DDG.

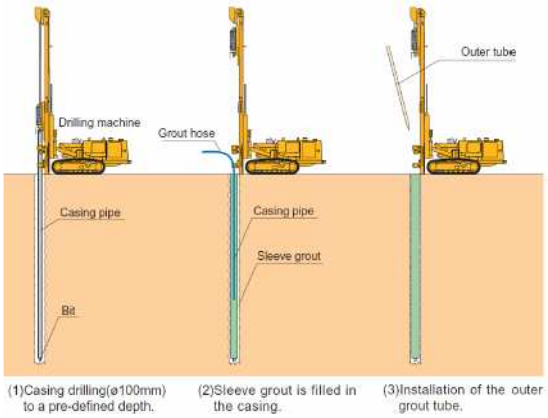


Figure 9. DDG Grout Injection - Preparation

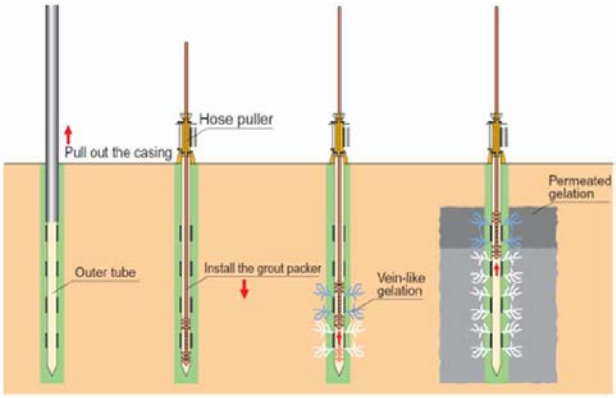


Figure 10. DDG Grout Injection - Sequence

Given the range of ground conditions anticipated, BC grouting including chemical grouting using Silicarizer was adopted. This was carried out as part of Nam Fung Tunnel canopy grout installation, in predominantly soft/mixed ground to facilitate safe tunnel portal formation and soft ground tunnel excavation works in "dry" conditions. Refer to Figures 9 to 12 for the grout array set up, method and control.

The contract specification for groundwater inflow criteria specifies allowable inflows up to 10L/min for individual source or 50L/min for 50m tunnel length, or 0.5L/m/min for single probe or 1L/m/min for more than 1 probe hole. Grouting termination criteria is mainly by volume, and secondly by pressure. Grout volume is calculated based on the estimated void ratio and volume of ground targeted, generally subject to grout hole spacing. Each grout injection phase is 0.33m before the packer is raised to the next subsequent higher level. Secondly if the grout intake volume is not achieved, the grouting pressure is increased to 10 bar. If the grout pressure exceeds this then the grouting is stopped and next grout phase is commenced.

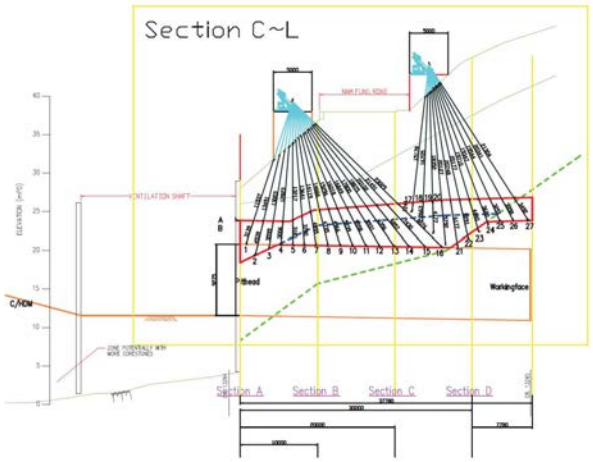


Figure 11. Canopy Grout Array – Section View

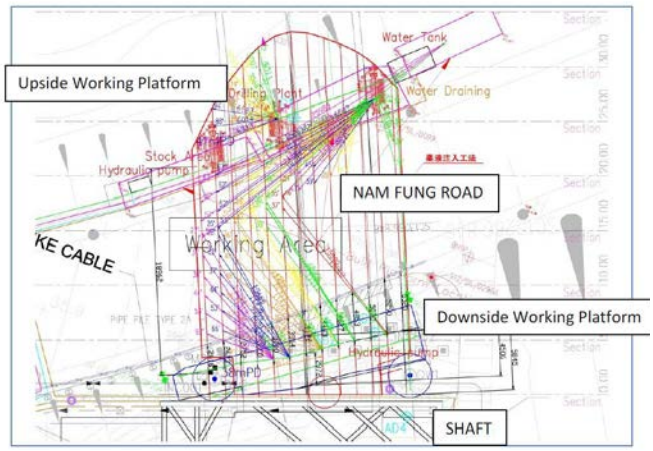


Figure 12. Canopy Grout Array – Plan View

5 GROUTING MATERIALS

The grout used in the DDG method of injection typically comprises:

1. Cement Bentonite to infill any voids, improving the ground and allowing removal of the casing for the subsequent grouting.
2. Silicariser (chemical grout) to allow injection into the finer ground through the initial BC injection after the BC mix has set. The gel time can be varied to suit the objective, generally 3 to 4 minutes was used.

The typical grout mix for both grout stages is presented in Table 1 below.

Table 1: Typical Grout Mixes for BC Grout and Silicariser per 1000L ($1m^3$) Grout Injection

Cement	Bentonite	Water	Sodium Silicate	Sulfuric Acid	Water	Total
250kg	60kg	895L	250L	40 to 60L	690 to 710L	1000L

Figure 13: DDG Canopy Grouting – Site Set Up

Figure 14: DDG Canopy Grouting – Control Center

6 IMPROVEMENT IN PERMEABILITY

The success of the post canopy grouting was verified by permeability testing in drillholes carried out within the vicinity of the grout injection on both sides on Nam Fung Road. Permeability testing was carried out before and after grouting in order to verify the improvement / effectiveness by canopy grouting. The results of the permeability testing, summarised in Table 2, indicated a significant reduction in the permeability of the groundmass was achieved by the canopy grouting.

Table 2: Permeability Test Results Carried Out Pre and Post Grouting

Hole	Depth	Before Grout		After Grout		Hole	Depth	Before Grout		After Grout	
	(mPD)	Date	(m/sec)	Date	(mPD)		(mPD)	Date	(m/sec)	Date	(m/sec)
1	20.4-20.9	12 Mar	$1.03x10^{-4}$	14 Apr	$2.86x10^{-7}$	6	18.4-18.9	12 Mar	$7.66x10^{-8}$	13 Apr	$8.35x10^{-8}$
2	12.5-13.0	13 Mar	$2.74x10^{-5}$	14 Apr	$3.09x10^{-7}$	7	20.4-20.9	12-Apr	$1.16E^{-06}$	8-May	$6.98E^{-08}$
3	23.5-23.0	12 Mar	$1.06x10^{-5}$	13 Apr	$5.07x10^{-7}$	8	18.4-18.9	12-Apr	$2.58E^{-06}$	8-May	$9.87E^{-08}$
4	20.8-21.3	12 Mar	$1.63x10^{-5}$	13 Apr	$8.26x10^{-8}$	9	22.8-23.3	12-Apr	$1.12E^{-05}$	8-May	$2.44E^{-07}$
5	21.9-21.4	12 Mar	$9.19x10^{-8}$	13 Apr	$5.04x10^{-7}$	10	20.3-20.8	12-Apr	$6.44E^{-06}$	8-May	$9.16E^{-08}$

7 CONCLUSIONS

To date the tunnel excavation for the tunnel running beneath HK Island has progressed a considerable distance from the portal area (>1km). During this period the pre-excavation grouting and portal excavation support has continued to provide a successful and effective groundwater cut-off, despite groundwater records

showing that there has been rapid and significant rises in the groundwater level above the tunnel crown. Refer to Figures 15 & 16 demonstrating dry conditions for the initial tunnel drive.

Figure 15: Initial Tunnel Excavation Through Interlocking Corestones (PW50/90) in Dry Conditions

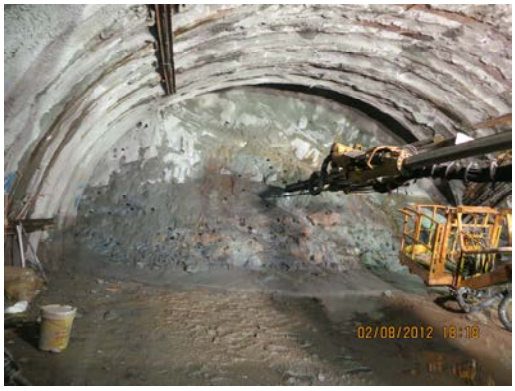


Figure 16: Top Heading Excavation in Dry Conditions

Observations following canopy grouting show:-

- The use of BC Grout successfully in-filled voids in addition to enhancing strength/stiffness of the surrounding groundmass.
- The use of silicarizer for canopy grouting successfully reduced the permeability of the ground surrounding the tunnel crown from about 10^{-6}m/s to about 10^{-8}m/s. This was verified by post-grouting permeability testing;
- The geological probing and tunnel geological mapping records indicate "dry" conditions within the tunnel excavation (Figures 15 & 16) and canopy grout zone above the tunnel crown.
- Geological mapping records indicate a grade V and IV saprolite with interlocking Grade II and III corestones of Quartz Monzonite.

The various grout injection phases at NFP successfully provided a ground water cut-off in addition to minimizing ground displacement.

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Pumping Test For The Construction of the West Kowloon Terminus of the Express Rail Link

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ABSTRACT

The hydraulic cut-off of an excavation and lateral support system is normally verified by a full-scale pumping test and partial pumping tests may be considered if the site is large. However, problems may still arise when the project is very large and complicated like the West Kowloon Terminus of the Express Rail Link. This site is located on a land with complex history of reclamation and use. Its geology comprises reclamation fill overlying marine deposits, alluvium and residual soils. Perimeter diaphragm walls were constructed to serve as temporary support for bulk excavation and as permanent basement wall when in service. Micro-fine grout was injected into the rock fissures at shallow wall panels to minimize groundwater seepage through the wall toe during bulk excavation. Because of the size of the site, construction constraints and hence phasing of works, partial pumping tests were carried out zone by zone with each zone in two stages along the perimeter. The test results were interpreted and the effectiveness in hydraulic cut-off of the walls was inferred. As the success of lowering the ground water level for the whole site within the walls was still in doubt, multi-stage dewatering assessment was required during bulk excavation.

1 INTRODUCTION

For private building developments in Hong Kong, the procedures for dewatering activities and precautionary measures against impairing stability and causing undue settlement of the adjoining buildings, streets and lands are given in the PNAP APP-22 (BD 1994), and referenced to the General Specification for Civil Engineering Works (HKG, 2006) and BS8004 (BSI, 1986). Pumping test is normally required by the Buildings Department (BD) to verify the effectiveness in the hydraulic cut-off of the excavation and lateral support system prior to the granting of the excavation consent. Traditionally, a full-scale pumping test is to be carried out and partial pumping tests may be considered if the site is large. However, problems may still arise when the project is very large and complicated like the West Kowloon Terminus (WKT) of the Express Rail Link (XRL) constructed by the MTR Corporation Limited (see Plate 1).

Plate 1 – The WKT Design Impression

In this paper, the partial pumping tests along the perimeter diaphragm walls at the WKT site are presented. The problems due to the size of the site, construction constraints, phasing of the works, geology and inclement weather are explained. The test results are interpreted and the effectiveness in the hydraulic cut-off of the diaphragm walls is inferred. As the success of lowering the ground water level for the whole site within the perimeter diaphragm walls is still in doubt, multi-stage dewatering assessment is required during the course of the bulk excavation.

2 THE SITE AND GEOLOGY

The XRL provides an efficient means of transport between Hong Kong and Mainland China. The Hong Kong section of the XRL comprises an underground terminus station (the WKT), an approach tunnel and about 26 km of tunnel. The WKT site is located on a reclaimed land surrounded by the Austin Station on the east, Victoria Harbour on the south, Kowloon Station and the above-station development on the west and Jordan Road on the north. Figure 1 shows the ground geology of the WKT site.

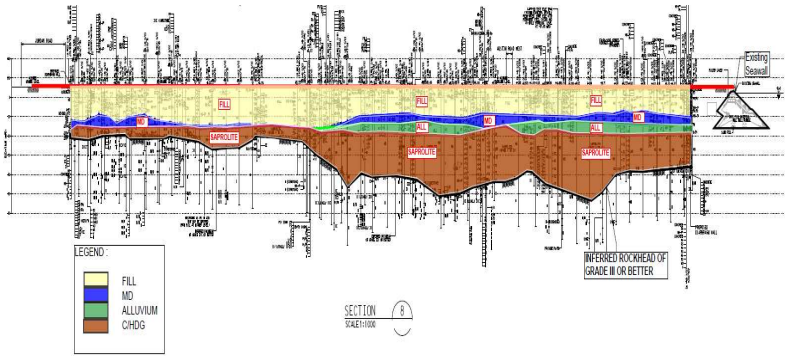
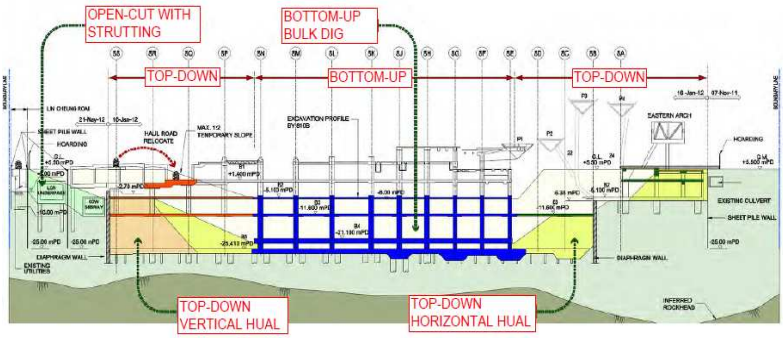


Figure 1 – Geology across the Site (N-S Direction)

The ground is generally flat with an existing ground level at about +4mPD to +5.5mPD and groundwater level at about +0.8mPD to +1.8mPD. As the site is close to the Victoria Harbour, the groundwater table at the south is influenced by the tidal fluctuations. Its geology comprises reclamation fill overlying the marine deposits, alluvium and residual soils (completely and highly decomposed granite). The marine deposits are very soft to firm silty clay with inter-bedded deposits of marine sand. The alluvium is firm or dense but locally soft or very soft. The interface between marine deposits and underlying alluvium is irregular. The site has a complex history of reclamation and use so that at relatively shallow depths beneath the ground surface there is a variety of filling materials and artifacts. The reclamation has given rise to disturbance of the marine deposits in the form of mud waves so that some very soft spots are found. In some locations, the marine deposits have been treated by surcharging, geotextiles and wick drains. The undrained shear strength and thicknesses of the marine deposits are therefore very variable. Despite large amount of ground investigation data are taken, it could still be unsafe to take average strengths for a given locality because of the variability over even quite short distances.

3 DESIGN AND CONSTRUCTION OF THE STATION STRUCTURE

Figure 2 shows the cross section and schematic construction of the station structure.



\Figure 2 – Cross Section (W-E Direction) and Schematic Construction of the Station Structure

It is about 550m long, 220m wide and 30m deep. It involves the installation of about 1.3km run of 1.5m thick perimeter diaphragm walls which are to serve as the temporary support for the bulk excavation of about 6 million m^3 of soils and rocks, and as the permanent basement walls when they are in service. These diaphragm walls also act together with about 380 bored piles and 3,300 socketted H-piles as the piled foundation for supporting the station structure and the future residential development. The perimeter diaphragm walls are designed and installed to "toe-in" into Grade III or better rock by 300mm. The wall panels are 1.5m thick and typically 6m long. Their depths vary from about 20m to 55m depending on the designed rockhead level. At the areas of high rockhead, the base of the wall panels are provided with shear pins to guard against the kick-out instability, and fissure grouting is carried out to form a hydraulic cut-off.

The overall construction scheme is extremely complex and therefore a broad and simplistic overview is given. Partial top-down construction at the central and full top-down at two ends of the station is adopted. After installation of the diaphragm wall, groundwater lowing takes place within the area of the station such that the water level is to be maintained at about 1m below the final excavation level of -24.5mPD in general and -32mPD at a narrow strip. The difference in water head retained by the diaphragm wall is therefore about 25m and may be as high as 27.5m generally. Bulk excavation then takes place with a cut slope supporting the diaphragm wall with a slope of about 26^o or 1 in 2. As the stability of the cut slope in certain areas is marginal and the factor of safety is even less than one in places, jet grout columns are installed to stabilize the marginal lengths of the cut slope. Once the final excavation level is reached, construction of the central portion of the WKT structure takes place by the bottom-up method from the basement level 4 slab up to the basement level 1 slab. Progressive top-down construction of the WKT structure then takes place with the floor slabs providing lateral support to the diaphragm wall. The floor slabs are supported on steel stanchions installed through the cut-slopes.

4 PROBLEMS ENCOUNTERED IN THE PUMPING TESTS

As seen in Plate 2, the site is surrounded by the Austin Station, Victoria Harbour and some high-rise buildings, and was dissected by heavy trafficked roads.

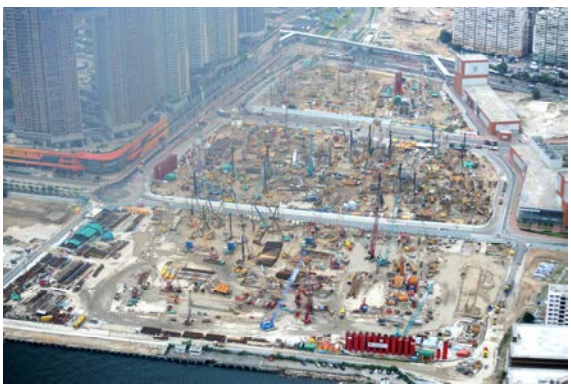


Plate 2 – WKT during Foundation Construction

Because of the size of the site, the scale and complexity of the project, the foundation works were sub-divided into 4 contract packages, and the civil works into 2 contract packages for construction. The construction sequence and methods were therefore largely affected by the interfacing of each contract. In this connection, a number of pedestrian and temporary traffic management schemes involving demolition and provision of a footbridge and construction of 2 temporary vehicular steel decks had to be implemented. Furthermore, the groundwater retained in the diaphragm wall cofferdam was estimated to be over 1,000,000m^3. Hydraulic analysis indicated that it would take more than 6 months to lower the ground water inside the excavation to 1m below the formation level if a full-scale pumping test was adopted. Partial pumping tests were therefore carried out upon phase completion of the diaphragm walls and fissure grout underneath the wall toe so that the bulk excavation for the station structure could proceed progressively. Large wall deflections and ground settlements were expected due to substantial hydraulic head difference. Excessive ground water drawdowns and settlements would be encountered at locations where the perimeter walls were not enclosed. Test zones were therefore designed with the necessary contingency measures to safeguard these effects; e.g. the affected utilities were supported prior to the pumping tests, and the pavements and carriageways were resurfaced during and after the pumping tests. There were also interpretative problems associated with the geology and inclement weather such as the difficulties in determining the initial readings, sudden rises in groundwater levels, large differences between standpipe and piezometer levels at steady stage, impermeable marine deposits, localized corestone zones and high rockhead areas. They are to be explained as the test results are to be interpreted.

5 THE PUMPING TEST

5.1 General Arrangement of the Test and the Testing Procedure

Figure 3 shows the general arrangement and summary results of the partial pumping tests. A total of 115 dewatering wells and 32 observation wells (with both standpipes and piezometers) inside the cofferdam, 52 standpipes, 47 recharge wells and over 100 ground and utility settlement markers outside the cofferdam were installed. 12 zones and each zone in 2 stages to 2 target levels of -7mPD/-9.0mPD and -25.5mPD along the periphery diaphragm walls were carried out. The initial readings of the standpipes were taken as the measured lowest ground water table within 7 days from the commencement of the pumping test to cater for the seasonal and/or tidal effect. However, the initial readings were difficult to determine sometimes due to the influence of the construction and/or dewatering works in adjacent. Partial pumping tests were started with all the pumps in that zone being switched on and the control ON/OFF switches levels were at the designed levels. Steady states were considered to have reached when the flow rates of the groundwater drawdown inside the cofferdam were less than 0.1m per hour, and there were no significant changes in the discharge volumes. The pumping was then maintained for at least 72 hours. Two days of recovery period was finally carried out. However, sudden rises in groundwater levels or spikes in the water level time graph during the steady state and recovery periods were observed sometimes due to the heavy rainfall, groundwater suddenly running in from the adjacent ground, malfunction and/or blockage of the dewatering wells. For each of the partial pumping tests, the test results were interpreted and reported to BD/GEO for phase acceptance and consent so that the bulk excavation of the station structure could proceed progressively. When all the partial pumping tests were completed, the test results were interpreted collectively and reported to BD/GEO as a comprehensive record.

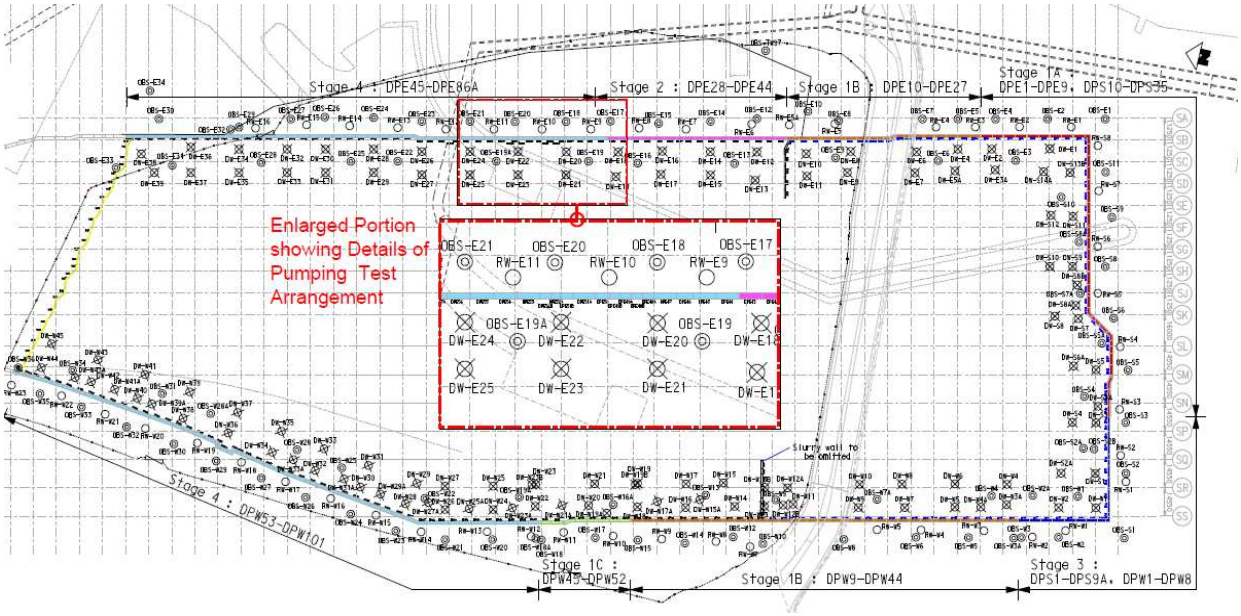


Figure 3 - General Arrangement for the Partial Pumping Tests

5.2 Ground Water Response Inside the cofferdam

Geological sections (Figure 1) show that there is a thick layer of marine deposits at about -5mPD to -15mPD. The water levels inside the cofferdam measured in the standpipes might be trapped and stayed at higher levels than those in the piezometers. Large head loss due to water seepage through the wall toe could be ruled out as there was no corresponding large water drawdown outside the cofferdam as measured by the standpipes and piezometers outside the cofferdam. In view of this existence of the impermeable layer of marine deposits that could affect the readings of the standpipes, the readings of the piezometers were used as the principal interpretation for the water drawdown inside the cofferdam. However, in the high rockhead areas, the water levels inside the cofferdam measured in the standpipes were observed to be lower than those in the piezometers sometimes. This was likely due to the slow response to the hydraulic head because the piezometer tips were installed into the rock. In all partial pumping tests, the water levels inside the cofferdam measured by the dewatering wells and the piezometers could achieve the first target level (i.e. -9mPD) and maintained steady for at least 72 hours before switching off the pumps. However, in most cases, the water levels inside the cofferdam could not achieve the second target level (i.e. -25.5mPD). These are likely due to the complicated geology with localized high rock and corestones area and impermeable layer above the rock head which dictate and limit the water drawdown outside the wells. The water levels were still maintained steady for at least 72 hours before the pumps were switched off. Despite the water levels in many of the observation wells could not reach the second target levels of -25.5mPD, the final stage of all pumping tests could be inferred as practically achieved as evidenced by: i) all water drawdown levels in the dewatering wells were below –25.5mPD, ii) long duration of steady state periods (over 72 hours) were maintained with flow rate of groundwater drawdown rate < 0.1m/hr generally, and iii) discharge rates in the dewatering wells were low, generally less than $2m^3$/hour.

5.3 Ground Water Response Outside the cofferdam

In all pumping tests, the water level drawdown outside the cofferdam was less than the allowable limit of 1m at steady state condition and without activating any of the recharge wells inside the testing zones. Although the ground water levels in some localized areas (such as high rock head area, corestones area and other impermeable area) could only be drawn to levels higher than the target levels inside the cofferdam in some test zones, the ground water levels immediately within the cofferdam had been drawn to -15mPD or lower in general. This indicates that there was at least 20m water head difference between the outside and inside of the cofferdam. Should there be any substantial seepage, under such large hydraulic difference, there would be

significant water drawdown and hence ground settlement outside the cofferdam. Furthermore, for each of the observation well outside the cofferdam, there is a corresponding dewatering well inside the cofferdam with dewatering level of at least –25.5mPD or below. This proved that the water level inside the cofferdam had actually been dewatered down to –25.5mPD or below.

5.4 Settlement Response Outside the Cofferdam

For all stages of the pumping tests, the allowable maximum settlements for the external sensitive receivers are determined as given in Table 1.

Settlement Markers	Allowable Settlement/Distortion
Ground	< 25mm
Water mains, cooling mains, gas Storm water drains/sewer	< 1:1000 < 1:500
Inside KSL Tunnel	< 4mm

Table 1 – Allowable Maximum Settlement/Distortion

An assessment report for the predicted ground settlement due to dewatering works together with the drawings showing the predicted settlement contour plans was submitted to BD/GEO. Despite there were some ongoing construction activities adjacent to the test zones carried out by other contractors, the settlement and utility settlement markers outside the cofferdam of all test zones were within the allowable limits. In view that the ground water drawdown out the cofferdam was well within the testing acceptance criteria, all pumping tests to the second or final target level are therefore considered satisfactory.

5.5 Recovery after Pumping Stopped and Final Groundwater Levels Achieved

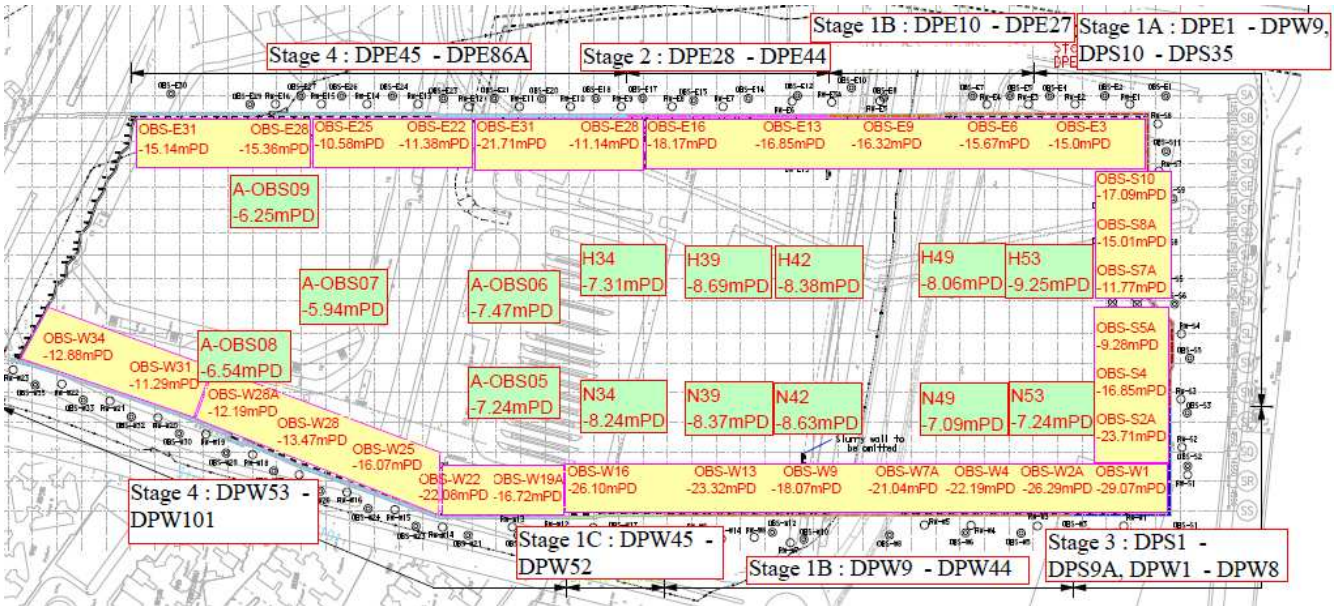


Figure 4 – Final Achieved Levels in Partial Pumping Tests

When the steady states were completed, the pumps were stopped for at least 2 days for recovery period. The water levels inside the cofferdam were recorded and the recovery rates based on the standpipe measurements were about 0.1-0.3m/hr. The slow recovery rates also on evidence of the effectiveness of the hydraulic cut-off of the cofferdam. The final achieved levels at different zones of pumping tests are summarized in Figure 4. In general, the achieved levels along the periphery of the cofferdam were at about -9.28mPD to -29.07mPD. These levels were about 10m to 30m below the existing ground water level (+1.0mPD). The high water levels in some localized areas were likely due to the underlying layer of marine deposits, high rockhead and

corestones, which would form perched water tables during the tests. As the water drawdowns and the ground settlements outside the diaphragm wall panels were within the acceptable limits and the discharge rates of the associated dewatering wells were about $2m^3$/hr or even much less, the hydraulic cut off of the diaphragm wall panels was proved to be acceptable.

6 MULTI-STAGE DEWATERING ASSESSMENT DURING BULK EXCAVATION

However, additional observation wells installed inside the WKT site showed that the water drawdown ranged from -6.25mPD to -9.25mPD. Likewise, this is likely due to the underlying layer of marine deposits, high rockhead and corestones which lead to form a perched water table inside the site. As the pumping tests were carried out in stages and along the perimeter only, the success of lowering the ground water level for the whole site within the cofferdam could be in doubt. Multi-stage dewatering assessment was therefore required during the bulk excavation to prove the success of lowering the ground water level for the whole site within the cofferdam to 1m below the designed excavation level. A dewatering plan showing the layout of the pump wells, positions of the pumps in the pump wells, capacity of the pumps and justification with design calculation for the sufficient dewatering capacity was submitted to BD/GEO. Prior to each stage of the excavation, the ground water inside the cofferdam had to be dewatered to at least 1m below the intended excavation level or the designed level according to the ELS design, whichever is the lower.

60 additional deep dewatering wells and 5 sump pumps in shallow pits were therefore installed at evenly space in the central excavation area. The water levels in the dewatering wells and observation wells, the ground and utility settlement markers and the inclinometers in diaphragm wall, slope, JGC and ground were monitored daily. Measured readings showed that the ground water level could have achieved the target level in the central excavation area only, but not at the sloping ground near the diaphragm walls. Further assessment had to be carried out to demonstrate that the higher water level near the diaphragm walls would not affect the stability of the wall/slope/JGC system and the general lowering of the ground water level for the whole site within the cofferdam had no adverse effect to the adjacent facilities.

As statutory required, the monitoring results were interpreted and reported to BD/GEO monthly in the RGE's T5 monthly reports. Response actions would be taken according to the approved monitoring plans if there is any exceedance of AAA values during the course of dewatering. When the excavation has reached the final formation level, the monitoring results and their interpretations due to dewatering were summarized and reported in a summary report to BD/GEO as a proof of successful lowering of the ground water level for the whole site within the cofferdam to 1m below the designed excavation level.

7 CONCLUSIONS

Because of the size of the site, construction constraints and hence phasing of the works, partial pumping tests had to be carried out zone by zone and each zone in two stages along the periphery. Due to the complexity of the geology like mud waves, high rockhead and corestones, high water levels in form of perched water tables were observed in some localized areas during the tests. Despite these have caused difficulty in the interpretation of the test data, the hydraulic cut off of the diaphragm wall panels was proved to be acceptable based on the groundwater drawdowns, ground settlements and discharge rates. Notwithstanding this, the success of lowering the ground water level for the whole site within the walls was still in doubt. Multi-stage dewatering assessment was required during bulk excavation as a proof of successful lowering of the ground water level for the whole site within the walls without any adverse effects to the adjacent facilities.

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Design and Construction of a Landfill Containment Bund cum Seawall Supported on Stone Columns Installed in Very Soft Marine Mud in Cotai, Macau

SURAJ DE SILVA & L.T.T. FONG
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ABSTRACT

The Cotai Landfill is the main receiving facility in Macau for building construction waste. As the dumping site is underlain with a thick layer of very soft to soft marine clay deposits, the uncontrolled end-tipped material has generated mud waves and they were encroaching the piles supporting the Macau International Airport taxiway nearby. In order to prevent future potential damage to the taxiway, the Macau Government engaged AECOM Asia Co. Ltd. (AACL), to design a containment bund adjacent to the taxiway to retain the waste and to prevent further generation of mudwaves that would affect the taxiway. The sustainable design prepared by AACL comprised the installation of vibrocompacted stone columns installed in over 20 m thick, very soft to firm, moderately sensitive marine clay and alluvial clay, as the foundation to the waste retention bund, thereby avoiding the dredging and off-site disposal of a significant volume of dredged sediments. This paper presents the design approach and construction of the stone columns and the behaviour of the completed bund/seawall.

KEYWORDS: Soft Clay, Ground Treatment, Stone Column, Mudwaves, Seawall.

1 INTRODUCTION

In 2000s, Macau experiencing a major construction boom, the site formation and building works for major casinos, roads and infrastructure on reclaimed land as well as demolition of existing structures generated a large volume of construction waste. It comprised predominantly of spoil materials from basement excavations and piling works.

Cotai Landfill is the main receiving facility in Macau for building construction waste materials. Due to the rapid development activities in late 2000s, the landfill material was being generated at a much faster rate than initially anticipated. In 2008, it was observed that dumping of the construction waste have spread out as well as caused the underlying soft marine mud to displace in the form of mudwaves towards the Macau International Airport (MIA) Southern Taxiway Bridge (STB) and the cooling water intake at the Coloane power plant located in the vicinity of the Landfill (Figure 1).

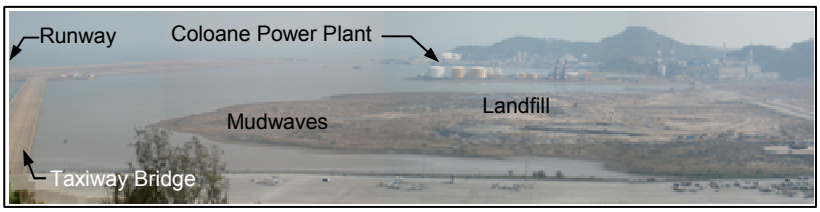

Figure 1. View of the Landfill Site and Mudwaves

This paper present the work for the initial phase of the protection measures against the mudwaves along the STB of MIA. The purpose of the proposed works is to achieve the following objectives:

a) prevent the propagating of mudwaves toward the MIA and hence protect the STB;
b) maintain the operation of the landfill site, as well as maximise the capacity of the landfill site, and;
c) to facilitate the future development of the MIA extension project.

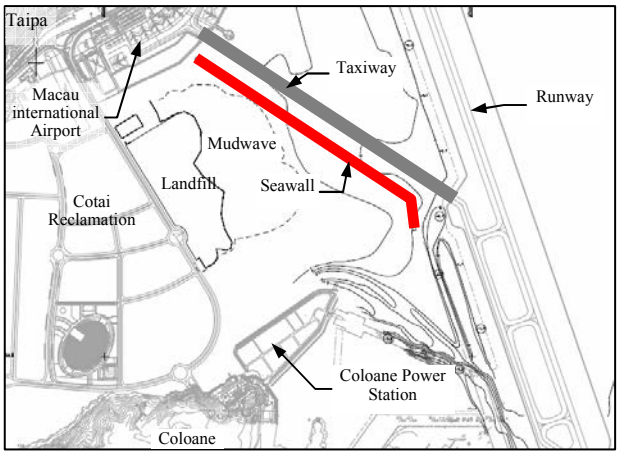

Figure 2. Site Layout Plan

2 SITE GEOLOGY

The superficial deposits within the study area comprise marine deposit (MD) of the Holocene age overlying a layer of alluvium of the Pleistocene age. Underlying the alluvium is the saprolitic soil consisting of completely decomposed granite (CDG). The solid geology comprises coarse-grained granite of Jurassic-Cretaceous age. Fill material has been subsequently placed over the marine soft clay deposit by the landfilling activities. Figure 3 shows the typical geological section of the site. Table 1 shows the typical thickness and constituentcy of the strata.

Table 1. Summary of Geological Strata

Strata	Thickness (m)	Constituent
Fill	4 to 7	C&D waste comprised disturbed mud, silty clayey sand, concrete, bricks, wood, steel
Marine Mud	13 to 28	Very soft to soft, dark grey, clay to silty clay with occasional shell fragments.
Alluvium	0 to 58 average 30	Soft to stiff, mottled yellowish brown light grey to brown, silty CLAY, CLAY/SILT Medium dense to very dense, yellowish brown to yellowish grey, silty fine to coarse SAND.
CDG	0 to 10	Sandy silty to silty fine to coarse SAND
Bedrock	-	Moderately strong to strong, moderately to slightly decomposed granite.

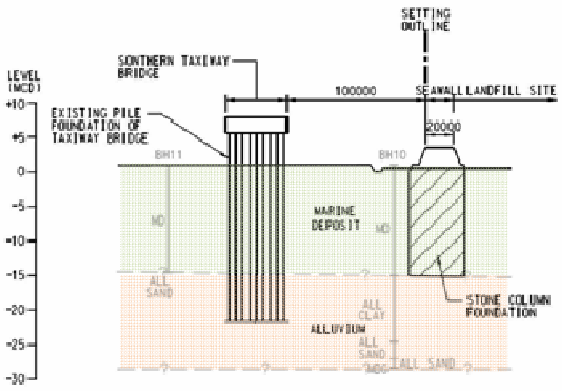

Figure 3. Typical Geological Section

3 DESIGN OF THE BUND CUM SEAWALL

The purpose of the seawall is to contain the dumped surfical clayey and other materials from spreading towards the STB, to prevent the further generation of the mudwaves that would impact the STB and to provide a stable and secure edge to the landfill. Since the founding material of the seawall is very soft to soft marine clay, ground treatment, by means of stone columns, are required to strengthen the foundation of the seawall in order to ensure its stability. It also improved the shear strength and stiffness of the composite soil mass to minimise the influence of the lateral load induced by the landfill soil mass that could possibly cause disturbance to the STB.

3.1 Principle of Ground Improvement by Stone Columns

Stone column construction involves the partial replacement of the very soft subsurface soils with compacted, vertical columns of stone that completely penetrates the weak strata. The stronger and stiffer material will attract more stresses (i.e. the stone columns) and therefore the composite ground comprising stone columns and soft clay (Barksdale, R.D. & Bachus R.C. 1983) will be stronger and stiffer and capable of carrying a larger load originating from the landfill behind, preventing the formation of mudwaves. The stone columns will also act as vertical drains within the soft clay facilitating the rapid dissipation of the excess pore pressures allowing it to quickly consolidate and gain in strength, thus further increasing the stiffness of the composite soil mass over time.

The strength of the composite ground depends on the percentage of soil replaced by the stone columns, i.e. the replacement ratio, a_s, as defined by (1) and illustrated in Figure 4.

$$a_s = C_1\left(\frac{D}{s}\right)^2 \tag{1}$$

D - Diameter of the compacted stone column
s - Centre to centre spacing of stone columns
C_1 - Constant depending on stone column configuration pattern

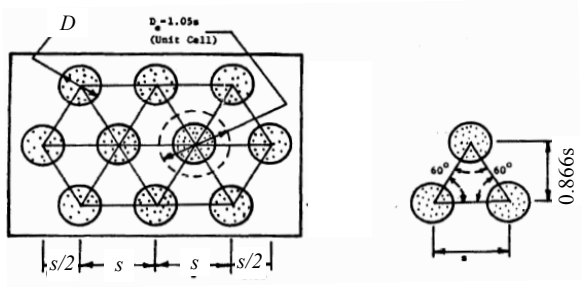

Figure 4. Replacement Ratio of Stone Columns

For this project, a 1.2 m diameter stone column at 2.5 m c/c spacing in a triangular pattern was adopted. The replacement ratio a_s is about 21%. Since the stone aggregate columns are much stiffer than the soft clay, the stresses will concentrate at the stone columns. The distribution of the stresses can be expressed by the stress concentrated factor n, defined as:

$$n = \frac{\sigma_s}{\sigma_c} \tag{2}$$

σ_s = stress in the stone column
σ_c = stress in the surrounding cohesive soil

The stresses over the soft soil and the stone column in term of a_s from (1) and the average overall stress on the ground surface, σ, is illustrated in a unit cell concept in Figure 5.

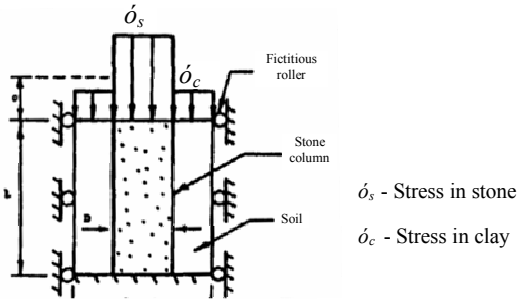

Figure 5. Ideal Unit Cell of Stone Column

$$\sigma_s = n\,\sigma / [1+(n-1)\,a_s] = \mu_s\,\sigma \tag{3}$$
$$\sigma_c = \sigma / [1+(n-1)\,a_s] = \mu_c\,\sigma \tag{4}$$

μ_s = Ratio of stress in stone in relation to σ
μ_c = Ratio of stress in cohesive soil in relation to σ
Using the expression of μ_s in (3), the stone column foundation can be modeled as a composite material with the average shear resistance expressed as:

$$c_{avg} = (1 - a_s)\,c \tag{5}$$
$$\phi_{avg} = \tan^{-1}(\mu_s\,a_s\,\tan\phi_s) \tag{6}$$

3.2 Stability Analysis
The stability analyses of the stone column seawall were carried out using the computer software "Slope/W" with automatic circular failure mode and sliding block failure modes. The stone column foundation was modelled as a

composite material with the embankment of landfill in place behind the seawall (Figure 6). With a_s of about 21%, the average cohesion of the composite material immediately after placing the stone columns will range from 2.8 kPa to 12.7 kPa; and the average friction angle of the composite material is about 20.4°.

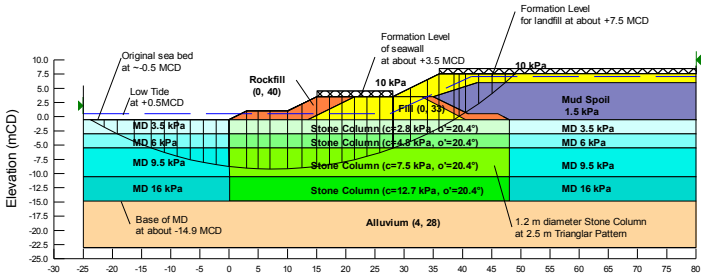

Figure 6. Geological Model for Slope Stability Analyses

To maintain the seawall stability, a 50m wide stone column treatment zone was required. The seawall revetment and the landfill side embankment slope profiles were proposed at a gradient of 1V:2H. A toe bund is provided as counterweight to stabilise the rockfill revetment. A typical section is shown in Figure 7.

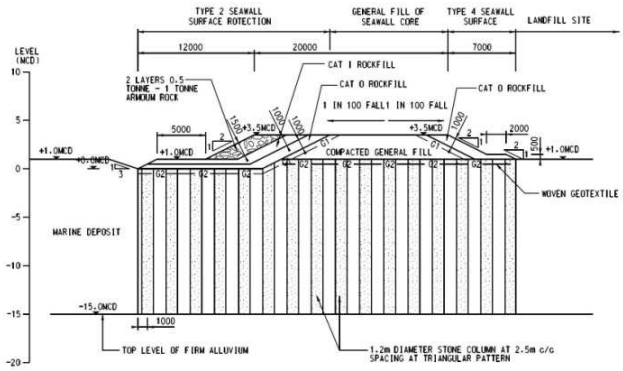

Figure 7. Typical Cross Section of the Seawall

The stone columns will also act as vertical drains to provide drainage paths for the excess pore water pressures arising from the vertical load of the embankment. With a typical 15 m thick marine deposit and an equivalent rectangle embankment width of about 30 m, the average increase in the effective stress of Marine Mud is only 25 kPa. Based on the radial consolidation theory and settlement reduction due to stone columns, 0.6 m settlement was estimated to occur within a year. Due to uncertainty in the drainage performance of the stone columns, the increase in the strength of the marine clay as a result of consolidation was not taken into account in stability analyses.

3.3 Assessment of Impact on Taxiway Pile Foundation

Since the STB is a critical facility to the MIA any damage to the STB will significantly affect the operation of the MIA. In order to control the additional load imposed from landfill site to the STB, numerical modelling was carried out to assess the impact on to the STB during installation of the stone columns, construction of the seawall and when filling behind the seawall, as part of the landfilling operation.

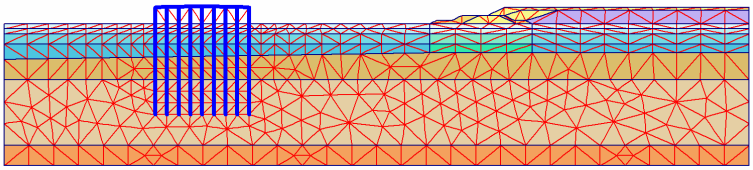

Figure 8. Finite Element Model

A Finite element model was developed (see Figure 8) using PLAXIS. The model adopted a geological profile with the deepest seabed level and thickest alluvium of the entire length of the seawall. The analyses were carried out with the marine clay behaving as an undrained material, which is considered to be an appropriate approach to model the actual behaviour of the soft clay. The structural elements included the beams and the PHC piles of the STB were modelled as a continuous beam/wall element in the PLAXIS model with an influence zone of 3 times the diameter being adopted (LECM 2008b). The model simulates the full history of the site including the original ground conditions, formation of the STB, installation of the stone columns, formation of the seawall and landfilling operation behind it.

The results from numerical modeling show that the maximum additional bending moment due to stone column installation, seawall construction and landfilling activities is only 6 kNm near the top of the STB piles and the total load of the pile is still within the acceptable limit of the original design. The additional shear force is considered to be not significant. The maximum predicted movement of the nearest piles to the seawall is about 8 mm (Figure 9). This predicted lateral movement is likely to span across a few spans of the taxiway and the actual magnitude of relative movement between each span of taxiway structure is unlikely to be a concern as movement joints have been provided between the taxiway spans and it should be capable of withstanding this relative movement.

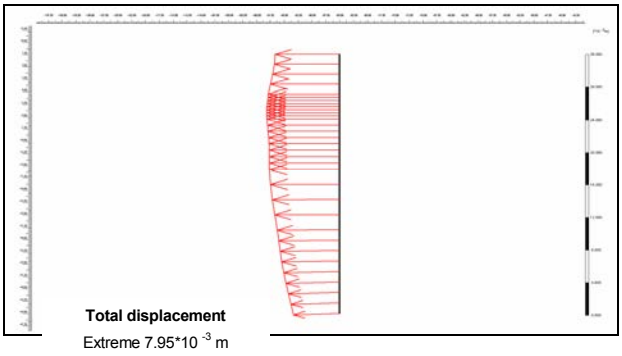

Figure 9. Predicted lateral Displacement of the Taxiway Foundation

3.4 Construction

Since the site is situated close to the MIA, there are certain physical constrains imposed by the Civil Aviation Authority of Macau on construction activities. The entire site is within the navigation restriction zone of the MIA (Figure 10). Marine access to the site is restricted and no mooring of vessels was allowed within the navigation restriction zone around the Airport. To avoid any disturbances on the movement of aircraft along the STB, all works were required to be carried out outside a zone of 57.5m from the centreline of the STB. Based on the information from the Civil Aviation Authority of Macau (AACM), the height restriction in the vicinity of the MIA along the runway is stringent and the use of high cranes was restricted. As a result, dredged seawall was adopted near the eastern end of the seawall.

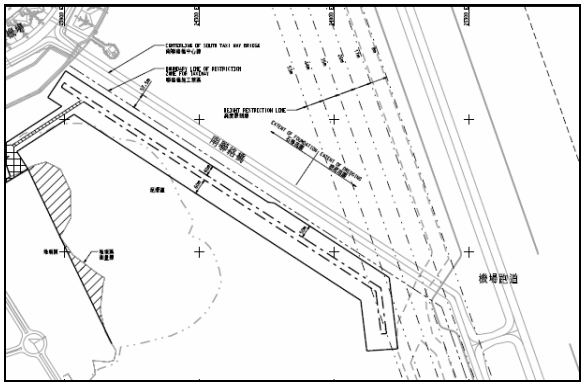

Figure 10. Height Restriction of the Site near the Runway of MIA

There was no readily available land access to the site as it is restricted by the landfill site and the MIA. Marine access was also limited due to the shallow draft and the contractor had to carry out additional dredging for navigation. The original design proposed a temporary platform for land based stone column construction. During the construction stage, the contractor changed to marine based method for installation of the stone columns.

To ensure the mud will not be mixed with the stone aggregates due to collapse of hole or necking, wet bottom feed method was adopted (Figure 11). In this method a long steel feeder tube with a vibroflot attached to the side of it, with the inside filled with stone aggregate, is inserted into the ground through the soft clay. Once the tube reaches the base of the stone column the tube is gradually withdrawn releasing the stones and compacting them with the vibrating action, causing the the mud to squeeze outwards. The repeated up-down motion of the vibrating tube, while the tube is withdrawn helps form a compacted stone column of the required diameter in the mud. Acceptance of the stone column was based on the depth vs SC diameter plot, and the degree of compaction (volume of aggregate, and energy consumed) from the installation records. A site trial was carried out to determine the optimum arrangement on apparatus, setup, minimum required compaction time, amount of aggregate consumption per metre length, and maximum energy consumption (Figure 12).

Figure 11. Stone Column Plant for Wet Bottom Feed installation

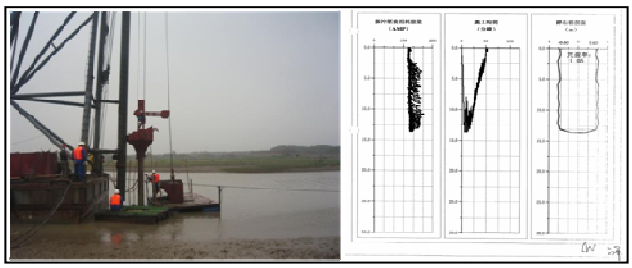

Figure 12. Site Trial Establishing the Acceptance Criteria

3.5 Monitoring of Taxiway and Taxiway Piles

The movement of the taxiway pile foundation and the MIA seawall were closely monitored during the seawall construction phase. No significant movement of the taxiway, the piles and the seawall was measured.

Monitoring will continue during the future landfilling operation stage when the loading is maximized in the landfill.

4 SUMMARY AND CONCLUSIONS

Construction waste was dumped over very soft marine mud at the Cotai Landfill of Macau. This uncontrolled dumping pushed the very soft mud generating mudwaves that were then encroaching onto the piles supporting the taxiway bridges of the Macau International Airport.

Due to the seriousness of the problem, government of Macau commissioned AECOM Asia Co. Ltd., to develop a robust solution to contain the construction waste being dumped in the area and to protect the taxiway bridge of the Macau Airport. The design solution developed was to construct a containment bund/seawall that is founded on a stiffened and strengthened soil block by improving the soft clay with stone columns. This paper presented the design approach of the containment bund including the limited equilibrium stability analysis and the numerical analyses carried out that demonstrated that the solution is appropriate as the bund will contain the landfill with minimal impact on the taxiway bridge piles. The analyses also demonstrated that the impacts during construction are also negligible.

During construction, the taxiway and seawall were monitored by independent parties and survey results indicated the installation of the stone columns and construction of the bund had minimal impact on the taxiways foundation piles. The seawall has been completed in November 2011 (Figure 13).

Figure 13. The Completed Stone Column Seawall

ACKNOWLEDGEMENTS

This paper is published with the permission of the Head of Gabinete para o Desenvolvimento de Infra-estruturas of the Macau SAR Government.

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Mega-scale Land Formation by Reclamation in the Middle East - Al Raha Beach Development, Abu Dhabi, UAE.

Suraj De Silva & Henry Wang
AECOM

ABSTRACT

Al Raha Beach Development, a 650 (10 km x 0.8 km) hectare upmarket residential and commercial development was undertaken by the Abu Dhabi developer ALDAR to house a resident population of 120,000. The reclamation was built over very soft contaminated silts and clays which had deposited in the channel since it was dredged in the 1970s. Extreme care was therefore necessary in forming the reclamation and the seawalls. The vertical seawalls in the Western Precincts were founded on marine stone columns. The Seawalls in the Eastern Precincts were formed with diaphragm walls installed from land, after reclamation. The very soft mud layers within the reclamation area were treated by consolidation under the reclamation fill. The reclamation sandfill comprised carbonate sand dredged from nearby islands. It was densified to prevent liquefaction and to achieve the design strength by deep vibrocompaction. The reclamation formation was compacted with high impact rollers to achieve an allowable bearing capacity of 150 kPa. This paper describes the reclamation design approach, ground treatment techniques used, the testing & verification of treatment and the behaviour of the reclamation upon completion. It presents the results achieved from these carbonate sands used as reclamation fill.

1 INTRODUCTION

In 2005, Maunsell Middle East(ME) (now AECOM ME) was engaged by, ALDAR, the largest developer in Abu Dhabi as the designer and project manager of this major high end residential and commercial development, located about 40 km from Abu Dhabi City centre. The development is earmarked to accommodate a resident population of 120,000. The brief included the design and project management of the site formation by reclamation and infrastructure works which included the interchanges connecting the development to the main road network in the area. The proposed reclamation extent and the locations of interchanges are shown in Figure 1.

The reclamation area was large at 650 hectares extending to a length of about 10 km and a width of about 800 m. The reclamation was carried out within a channel that was dredged in the 1970s to win fill materials to raise the ground elevation at the Abu Dhabi International Airport site and other nearby areas. The channel was dredged down until the sedimentary rocks were encountered at a depth of about 10 to 12 m, with deeper trenches and pits at some isolated locations. Over the past 30 to 35 years the dredged channel has experienced sedimentation where very soft silts, clays and very loose sands have deposited. The thickness of these deposits was greater within the trenches and the pits. The original reclamation approach was to dredge and dispose these very soft sediments offshore before reclamation. However, environmental tests indicated that these sediments were contaminated, particularly with heavy metals, which necessitated that the sediments be left in placed and capped before reclamation.

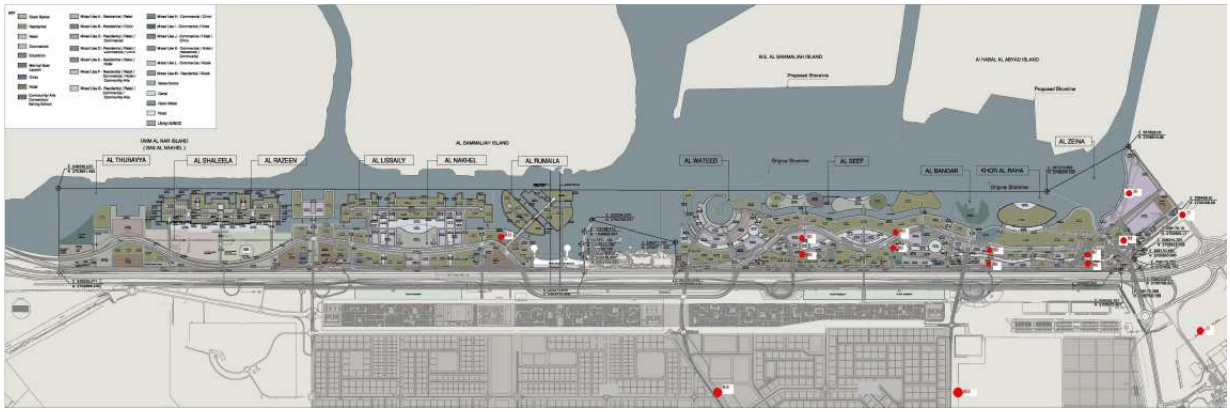
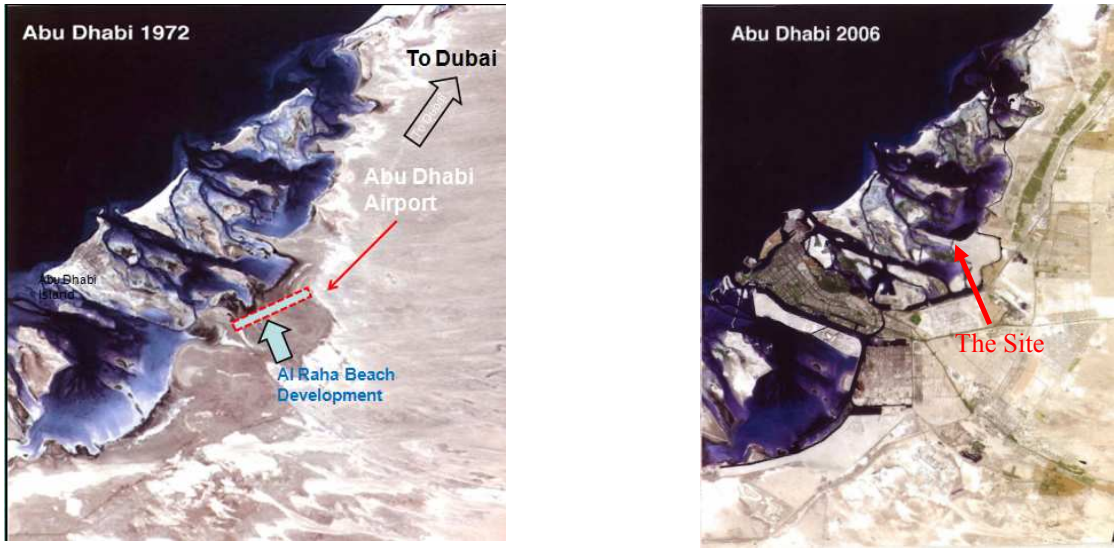


Figure 1 : Al Raha Beach Development – Site Layout Plan

2 SITE LOCATION AND SITE HISTORY

Abu Dhabi City is located on the Abu Dhabi peninsular. The Al Raha Beach development site is located about 40 km from the Abu Dhabi City centre, near the Abu Dhabi International Airport, immediately to the east of the main Abu Dhabi – Dubai Expressway. Prior to any development, the virgin land comprised low lying sabhka, and intertidal areas with mangroves. However, since the 1970s these areas were raised by filling mainly from winning materials from dredging channels in the area. This included the current site called the Airport Channel, as most of the fill material for the construction of the Airport was won from dredging this channel. Aerial views of the site taken in 1972 and in 2006 are shown in Figure 2 below.



(a) (b)

Figure 2: (a) Aerial Photo dated 1972 Showing Site Location (b) Aerial Photo dated 2006 Showing the Airport Channel

3 REGIONAL GEOLOGICAL SETTING

Abu Dhabi Emirate has a hyper arid climate, characterised by a prolonged dry summer of high temperatures between April to November. The winter period is characterised by mild temperatures and slight rainfall. The wettest month is February. Mean annual rainfall is less than 100mm/year. The rainfall available for runoff and contribution to groundwater recharge is very variable. Actual evaporation in the region is higher than rainfall (approximately 100 mm/year), whilst potential evaporation is more than 2000 mm/year.

High evaporation means that effective rainfall recharge to groundwater is very limited. The site is located in the coastal marine zone, characterised by tidal flats, paleodunes and sabkha. The costal marine zone extends approximately 25 kilometres inland. Further inland are internal dunes and around approximately 100km from the coast are piedmont plains.

Superficial deposits are underlain by Lower Fars formation, which is a thick aquiclude (up to 650m), comprising mudstones and marly dolomites, intercalated with evaporites (gypsum and anhydrite) deposited in a shallow marine environment. The site (the former Airport Channel) was formed in the 1970's by dredging. Some very recent sediments were deposited on the seabed during this time and these deposits comprise very soft, medium to fine grained silty sands, sandy silts and sandy clays.

4 GROUND INVESTIGATION

The ground investigation was directed mainly to identify the extent and the thickness of the very soft silt, clay and loose sand present at the bottom of the channel, and to obtain the design parameters of this material in order to undertake the reclamation design. Therefore, in addition to conventional boreholes, CPTs and vibrocores, a large number of special CPT probing tests were carried out using a smaller cone with an cone area of only 2 cm^2 that is pushed into the very soft seabed from a set-up that sits on the seabed. This is called GEMs CPT device. In addition, field vane shear tests were carried out from jack-up platforms to obtain the undrained shear strength of the soft silts and clays.

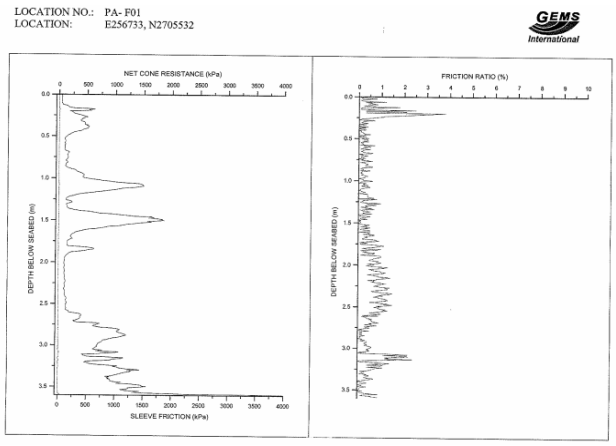
4.1 GEMS Seabed CPT Probing Device

This is a quick way of probing the seabed. A CPT frame is lowered down to the seabed and a small CPT cone (2 cm^2) is pushed into the ground using hydraulic motors and the cone penetration resistance and the sleeve friction are measured (see Figure 3). This CPT device is much quicker and easier to set-up and position, test, remove, and move to next position. These results enabled the thickness of the very soft to very loose seabed sediments to be determined and to be clasified(see Figure 3c). Probings were on a 50 m x 50 m grid to cover the reclamation and seawall areas. The thickness and distribution of soft sediments were also assessed using Seismic Reflection Surveys undertaken by EGS, standard Piezocone Penetration Testing (PCPT), Vibrocoring and Field Vane Shear Tests. The soft sediments were also sampled using piston samplers, Shelby tubes, vibrocores and grab sampling. These samples were then tested to obtain the index and consolidation properties.

(a)

(b)

(c)



(d)

Figure 3: (a) The GEMS CPT Probing device that sits on the seabed (b) The pontoon used to carry out the GEMs CPT probes from (c) Sample of sediments retrieved from the seabed (d) Typical Set of results from the GEMS CPT Probing device

5 SITE GEOLOGY AND GROUND CONDITIONS

The seabed sediments, on which the reclamation fill will be placed, would affect the short term and long term settlement of the development, the stability of the reclamation during the reclamation works, after completion of reclamation, and the behaviour under earthquake conditions. The characteristics of the sediments would also dictate the reclamation and ground treatment required. Itwould also have an impact on the cost of reclamation and the time required for construction. The characteristics of these sediments will also dictate the foundation requirement for the seawalls, particularly the sloping and the vertical blockwork seawalls. Dredging of these unsuitable materials and replacement with suitable materials ortreatment of the very soft to very loose seabed sediments will be required before such seawalls can be founded.

The ground investigation revealed that the seabed is underlain with a layer of very recent sediments deposited in the last 30 years which comprise very soft silts, clays and very loose sands. This layer thickness ranged from 0 m at some locations to as thick as 6 m in isolated dredged pits and trenches; but generally around 0.5 m to 2.3 m thick in most part of the site. This recent sediments generally overlie the sedimentary bedrock encounetered. But at some locations, dense silty sand was encountered, possibly originating from weathering of the sandstone bedrock. The bedrock comprises calcareous mudstone, siltstone, sandstone and calcarenite and crystalline gypsum (see Figure 4).

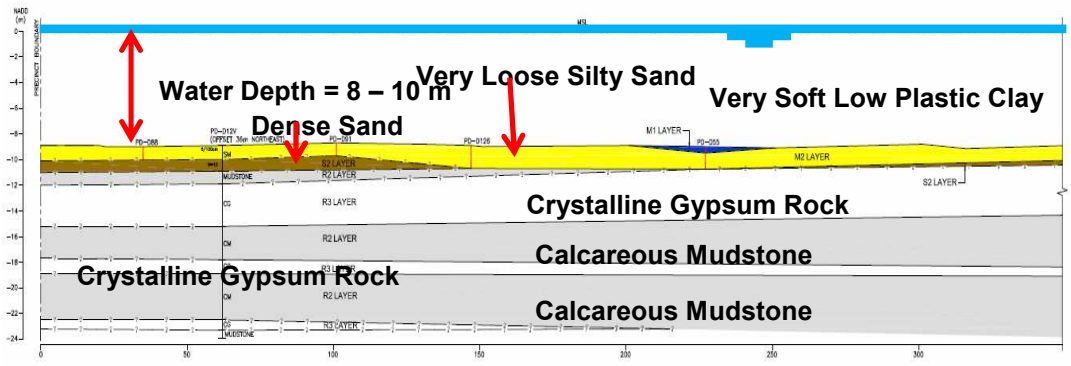


Figure 4: Section indicating the Typical Ground Conditions at the Site

6 SITE SEISMICITY

6.1 Seismic Setting of UAE

The geological features of the United Arab Emirates follow that of the Arabian Platform. The rocks in the Arabian Platform accumulated on the stable marine-to-fluviatile shelf. Tectonically the UAE is situated in the South-eastern part of the Arabian Plate, which is one of the youngest to make up the surface of the earth. The plate comprises a crystalline basement of Precambrian continental crust about 40 - 50 km thick, an overlaying basement sequence of younger Phaneroic sedimentary rocks that range in thickness from 0 to 10 km, in addition to basalt and the oceanic basin. The separation and splitting of the Arabian plate from the African Plate along the Red Sea and the Gulf of Aden axes followed by drift of the Arabian Plate to the north and northeast, lead ultimately to a collision with the Eurasian plate that resulted in the Zagros fold-belt and thrust belt. Zagros fold belt is the major source of earthquakes in the eastern border of the Arabian Plate. The Zagros Fold, the Thrust Belt and Makran Subduction Zones are the only two fault systems that have direct effect on the seismicity of the UAE (see Figure 5).

6.2 Site Specific Probabilistic Seismic Hazard Assessment

A site specific seismic hazard assessment study was carried out for the Al Raha Beach development study. The study was undertaken by Prof Azm S Al Homoud of the American University in Sharjah, UAE. In this study, the seismic sources were identified and separated into various regions and the Abrahamson and Silva (1997) attenuation relationships were used in assessing the development of Peak Ground Acceleration (PGA) and the spectral ordinates for distant earthquake events. The Campbell and Bozorgnia (2003) acceleration relationships were used for near source ground motions (less than 100 km away). The PGA was evaluated at the site following the procedure based on the method given by Cornell (1968) and incorporated in the computer Program EQRISK, the program adopted in this study. The PGA values obtained for the bedrock at the site from this assessment are given in Table 1.

Table1: Site Specific Peak Ground Accelerations obtained from Site Specific Seismic Hazard Assessment Study

Earthquake Return Period in Years	Peak Ground Acceleration, PGA, in cm/sec^2 at Rockhead	PGA at Rockhead in terms of ground acceleration, g, where g = 9.81 m/sec^2	PGA at the ground surface/Seabed in terms of ground acceleration, g, where g = 9.81 m/sec^2
50	58	0.059	0.080
75	67	0.068	0.092
100	70	0.071	0.096
200	83	0.085	0.115
475	104	0.106	0.143

Since at the time of the seismic study a limited number of post vibrocompacted CPT data were already available, this information was used in determining the ground type after reclamation and ground improvement at the site. The ground at the site comprised very dense soil and soft rock, which corresponds to soil profile S_c according to the UBC97 (Unified Building Code, 1997, of the USA). The Peak Ground Acceleration Amplification Factor was assessed based on the world wide database on site-dependent seismic response and acceleration data from actual recorded data during earthquakes (Seed and Bray, 1997). According to this database the site is classified as AB2 and the applicable amplification factor is 1.35. The ground surface accelerations for the various return periods are also shown in Table 1 above. The amplification was considered to increase linearly from the top of the bedrock to the ground surface.

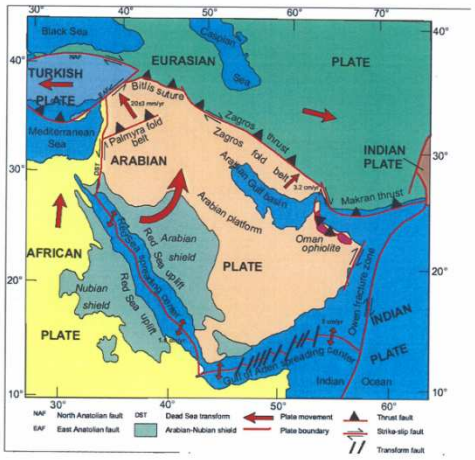


Figure 5: Tectonic Setting of the Arabian Tectonic Plate (After Johnson, 1998).

Figure 6: Capping Sand Layer being placed using a Spreader Barge

7 RECLAMATION AND SEAWALL DESIGN

7.1 Reclamation Approach

As mentioned before, due to the presence of a base layer of very soft silt and clay layer, with some very loose silty or clayey sand in some isolated locations, which was contaminated, dredging it and dumping it offshore was ruled out; thus imposing a major constraint on the method of reclamation and on the reclamation programme. Hence this layer had to be left-in place and the reclamation had to be carried out over this layer of very soft silt and clay.

The reclamation approach was hence changed from a dredged to a non-dredged reclamation. The placement of the fill over the very soft clay had to be carried out with great care in order to prevent failures of the placed fill, with the fill sliding on the very soft clays and silts. Therefore, initially a capping layer of thickness of at least 2.5m had to be placed over the soft clay and silts, before less careful and controlled filling can be placed over it. This 2.5m capping layer had to be placed in two initial lifts of 0.6 m which was then followed by a 1.3 m second lift with leading edge slope angles shallower than 1V:15H. This operation was slow as the sand had to be sprinkled using spreader barges. Figure 6 presents the placement of the sand from a spreader barge. The spreader head operator was assisted by a computer display on board the spreader barge which displayed the bathymetric depths along the set run lines.. Daily multi-beam bathymetric surveys provided the information to check on the thickness on the capping layer placed and on mud-waves, if any had formed. For the ease of planning and reporting work the seabed was sub-divided into 250 m x 250 m square parcels. It took one week to cap one parcel with a 2.5 m thick capping layer. The pumping distances for the capping materials in the Eastern Precincts ranged from 500 m to 2.4 km.

After placing the sand capping layer in small lifts, the capping layer was surveyed with (1) side scan sonar (2) echo sounding and (3) diver inspections to check if any failures had occurred in the seabed. Figure 7 presents an underwater view of the capping layer on the seabed. Capping layer was placed in both the Eastern and the Western Precincts, and the total volume of sand placed in the capping layer was 13 million cubic metres.

After placing the capping layer, the reclamation in the Eastern Precincts was carried out by placing further fill slightly more liberally; however, maintaining leading edge slopes of at least 1V:10H at all times by hydraulically pumping fill using floating pipelines (see Figure 8) from the 15,270 hp Cutter Suction Dredger 'Al Mahar, placing material from an elevation of about -4.5 mNADD to +2.1 mNADD over pumping distances of 750 m to 4.3 km (from the dredger to the discharge point).

The reclamation filling above the capping layer in the western precincts was carried out by placing land based fill material borrowed from a site located about 40 km away and transported in 10-wheel trucks and end-tipped onto site (see Figure 9). In the Western Precincts the reclamation fill was tipped into areas enclosed with vertical L-shaped seawalls.

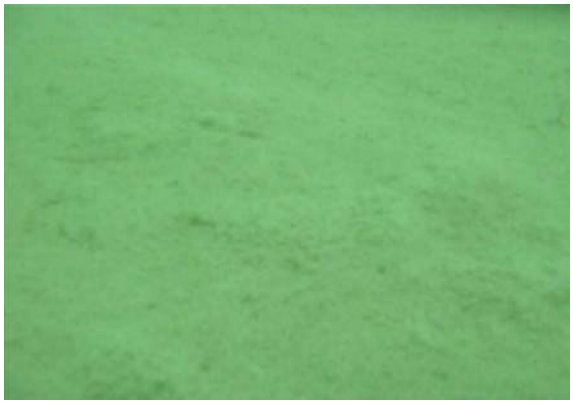


Figure 7: View of Capping Layer

Figure 8: Pumping Ashore of Dredged Sandfill using Floating Pipelines

7.2 Reclamation Fill Sources and Method of Placement

A large proportion of the reclamation fill was dredged from the islands located to the North of the site from the Al Sammaliah and Habl Al Abiyadh Islands (see Figure 1). The material won from these islands is predominantly calcareous silty sands. Figure 10 shows a vertical cut face of the islands that were dredged. The material won from the land based borrow areas also provided silty sands that were suitable for reclamation. In addition the spoil generated from a deep building basement excavation (e.g. the Central Market Building) in Abu Dhabi City was also placed in the Western Precincts. This material comprised silty sand and soft sedimentary rock fragments originating from calcareous sandstone, siltstone, crystalline gypsum and some mudstone.

Figure 9: Land based Reclamation fill being placed on Site

Figure 10: Vertical Face of Marine Borrow Area

8 SEAWALLS

Two types of seawalls were adopted at the site. In the Eastern Precincts they were diaphragm walls. In the Western Precincts they were L-shaped Precast concrete units placed over soft ground improved with stone columns.

8.1 Eastern Precincts

Initially it was proposed that concrete blockwork seawalls be used. However, in that case it was necessary to construct the seawalls before reclamation could commence in any given section. However, given the constraints of the presence of the very soft silts and clays in the seabed that could not be dredged, the extremely tight programme to get the infrastructure in place, and due to the fact that the dredgers were already on site placing reclamation fill, it was not possible to to construct the blockwork seawalls. Moreover, with the need to speed up the work due to the delays experienced in having to place a capping layer carefully, it was decided to reclaim the land first and then form the seawalls using diaphragm walls (after the diaphram walls are constructed the soil in the seaward side is dredged away leaving a berm in front of the D-wall), mobilising a large number of diaphragm walling grabs and hydrofraises to site. Figure 11 shows a schematic diagram, and Figure 12 shows the installation of the diaphragm walls on the recent reclamation.

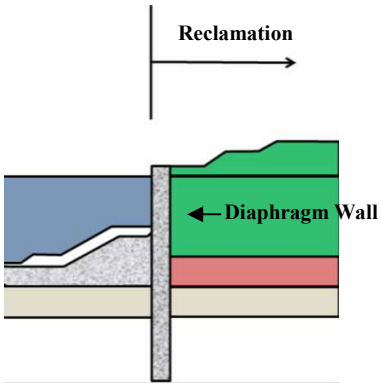


Figure 11: Schematic View of Diaphragm Wall Seawall Constructed at the Eastern Precincts

Figure 12 : Construction of Diaphragm Wall Seawalls in the Eastern Precincts

8.2 Western Precincts

Particularly due to the angular shapes of the various reclamation island formations, and the need to expedite the construction of the reclamation (though, the programme was not as tight as for the Eastern Precincts), the contractor proposed RC Precast L-shaped seawalls in the Western Precinct Area. The schematic section of the seawall is shown in Figure 13 below. Figures 14 and 15 show the precast units in the casting yard and the placement of them in the reclamation. The partially complete angular shaped reclamation island is shown in Figure 16.

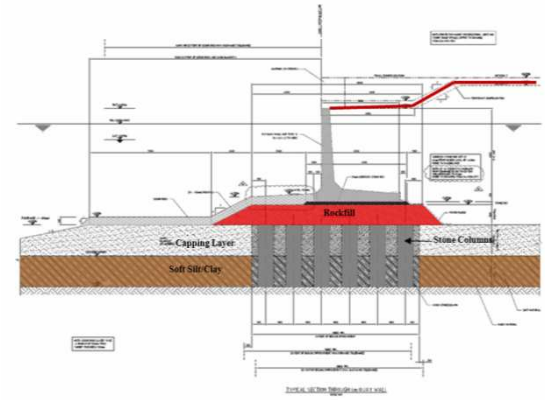


Figure 13: Sectional View of Precast L-Shaped Seawall Construction

Figure 14: L-Shaped Seawall Unit Casting Yard

Figure 15: Installation of L-Shaped Seawall Units

Figure 16: Aerial View of Site Construction

9 GROUND IMPROVEMENT

9.1 Design Requirements

Upon completing the reclamations it was required to undertake ground improvement works on the reclamation in order to achieve the following design requirements; (1) Prevent liquefaction of the reclamation fill under the design earthquake events and to achieve factors of safety of 1.5 and 1.1 under the OLE and CLE earthquake events respectively (see Table 2); (2) Achieve a minimum relative density of the reclamation fill of 60%. (3) Limit the settlements of the reclamation to the following values; Short term (2 years) < 15mm; Medium term (5 years) < 25mm; Long term (20 years) < 50mm. (4) The angular distortion in the reclamation surface to be limited to 1V:500H. (5) The allowable bearing capacity of the reclamation ground at an elevation of 2.1 m NADD to be at least 150 kPa for a 3m by 3m pad footing (the settlement of the footing limited to 25mm under an applied stress of 150 kPa).

Table 2: Design Earthquake Events and Factors of Safety Achieved

Earthquake Level	Earthquake Type	Return Period (years)	Acceleration at Bedrock (g)	Acceleration at Ground Level (g)	Damage Criteria	Factor of Safety
Level 1	Operation Level Earthquake (OLE)	100	0.071	0.096	No damage, fully functional	1.5
Level 2	Contingency Level Earthquake (CLE)	475	0.104	0.143	Damage can be repaired to restore full operational functioning of the structure	1.1

9.2 Ground Improvement Works

Initial CPT data carried out on the newly reclaimed land indicated that the relative density of the sand is inadequate to achieve the above stipulated design requirements. Therefore, it was proposed to carry out the following ground improvement works; (1) Densification of the ground using the deep vibro-compaction technique (2) Installing sand compaction piles in the underlying capped very soft silt/clay layer to minimise on-going settlements (3) Carrying out impact compaction of the formation surface at 2.1 m NADD to achieve the bearing capacity of 150 kPa.

Liquefaction analyses and ground improvement works design indicated that the CPT q_c versus depth profile shown in Figure 17 need to be achieved, with the q_c values corrected to account for the crushability/compressibility of the calcareous sands encountered at the site. Figures 18 and 19 show the plant

used in ground improvement. Figure 20 shows the actual CPT q_c data from a typical location where the criterion has been met.

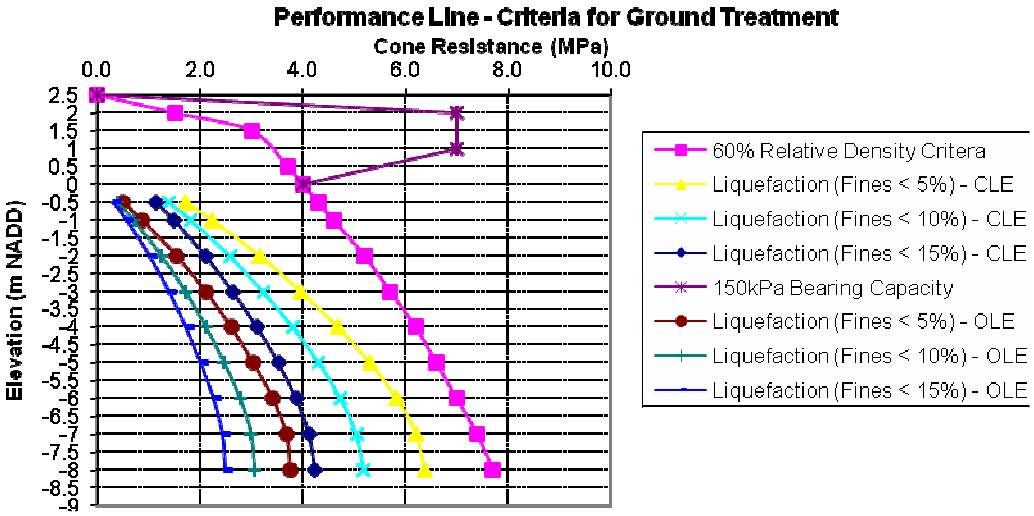


Figure 17: CPT qc Criteria to be Achieved to Satisfy the Design Requirements

The vibrocompaction work was carried out at various spacing as shown in the Table 3 below.

Table 3: Ground Improvement Works Carried out on Site

	East Vibrofloation (SBF-NSCC JV)	West Vibrofloation (AST-M&R JV)	West Stone Columns (AST-M&R JV)
Grid	Triangular	Triangular	Square
Spacing	4.0m increased to 4.3m	3.75m	1.5m
Foundation Design	Schmertmann (Targetted ZLTs to confirm Es/qc relationship)	Assume E=16,000kN/m³ (Denser grid of ZLTs to confirm)	3D FEM calibrated by On Site ZLTs.

10 INSTRUMENTATION

After completing the reclamation and carrying out the ground improvement works, instrumentation including vibrating wire piezometers, observation wells, spider magnet extensometers and settlement markers were installed in instrumentation clusters across the site. Piezometer monitoring results and settlement monitoring results are shown in Figures 21 and 22. An instrumentation cluster location is shown in Figure 23. They indicate that there is no excess pore pressure and that there is no on-going settlement.

Figure 18: Landpac Impact Roller used to compact the formation level at +2.1 m NADD

Figure 19: Deep Vibrocompaction in Progress

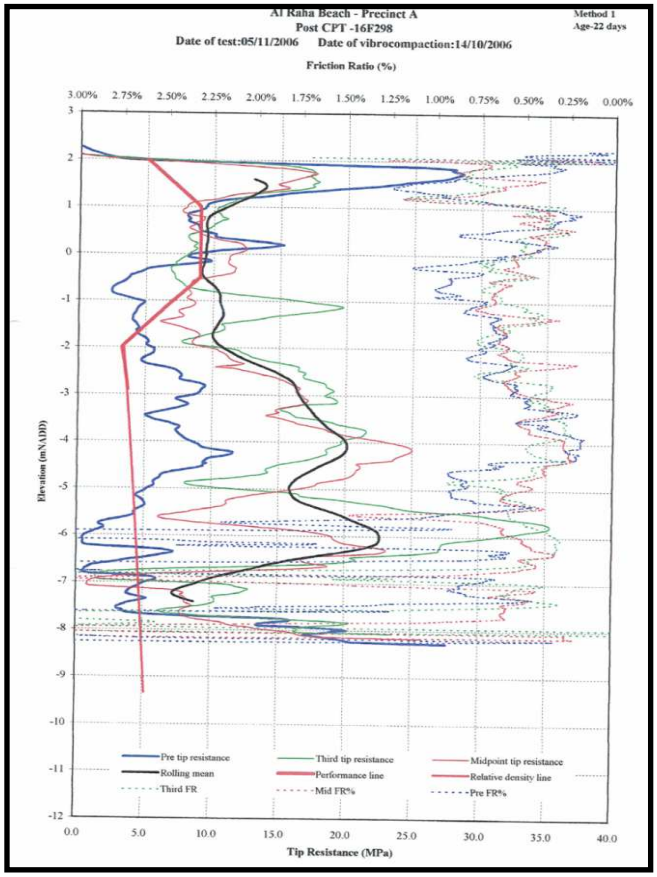


Figure 20: Typical CPT qc Profiles after Treatment to compared with the qc criterion.

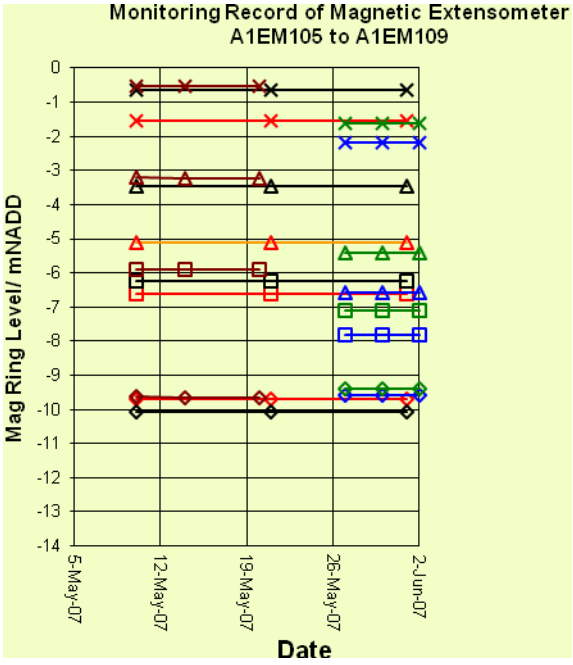


Figure 21 : Settlement Monitoring Results from Spider Magnet Extensometers

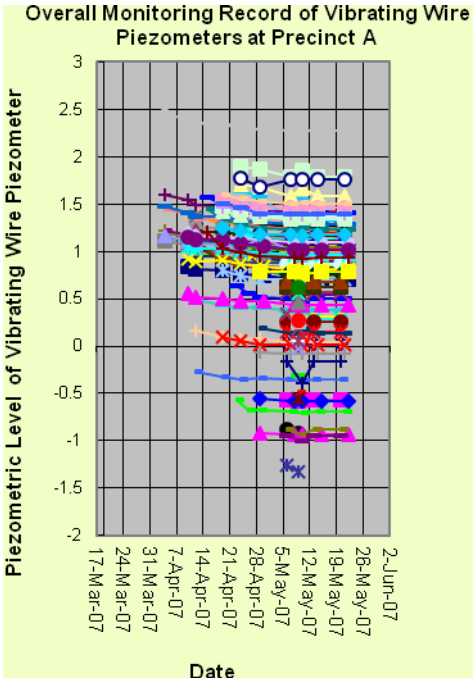


Figure 22: Piezometric Head Monitoring Data from Vibrating Wire Piezometers

11 DEVELOPMENT ON THE NEW RECLAMATION

Due to the need for fast track development of the site, as soon as the newly reclaimed land was ready, following ground improvement works, construction of infrastructure and building works commenced. They included a long canal feature along the length of the Eastern Precinct where the side walls are supported by diaphragm walls, the construction of many iconic buildings and other residential developments and marinas. However, the development of the Western Precincts has been postponed following the financial down turn experienced in 2008.One of the many iconic buildings proposed on the Site, the ALDAR Headquarters has been completed and is currently in occupation, as seen in Figure 24.

Figure 23 : An Instrumentation Cluster Location

Figure 24: Iconic ALDAR Headquarters Building Constructed on Al Raha Beach Eastern Precincts.

12 SUMMARY AND CONCLUSIONS

1. A major reclamation was carried out without incident, by leaving contaminated very soft silts and clay in place on the seabed and placing reclamation fill over it. The reclamation approach was to carefully place 3 layers of a capping layer, 0.6 m, 0.6 m and 1.3 m in thickness, using spreader barges, and then placing the rest of the reclamation fill with greater liberty, hydraulically, using floating pipelines.
2. The reclamation of the Eastern Precincts was completed in one year, between March 2006 and March 2007. The placement of the capping layer in the Western Precincts was completed between November 2006 and January 2008.
3. Two types of seawalls were used; diaphragm walls in the Eastern Precincts and L-shaped precast RC concrete walls placed on seabed treated with stone columns in the Western Precincts. The diaphragm walls were adopted in order to catch up with the programme, as the reclamation could proceed in advance of the seawall construction, and due to the fact that the dredgers were already mobilised to site. After installing the diaphragm walls the soil outside the diaphragm walls were dredged and removed to form the islands. In the Western Precincts, the seawalls were formed first and land sourced fill was placed within.
4. Ground improvement of the fill was carried out using vibrocompaction in order to prevent liquefaction and achieve the required factors of safety against liquefaction, to achieve the minimum relative density of the sandfill of 60 % to limit future ground settlements, and to achieve an allowable bearing capacity of at least 150 kPa.
5. Instrumentation and zone load tests indicated no excess pore pressures, and that there was no on-going ground settlements in the new reclamation.
6. The newly reclaimed areas were assigned for development immediately after completing ground improvement works.

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Some useful schemes for retaining wall design for forming building platforms

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ABSTRACT

The creation of large level platforms for building developments often call for construction of retaining walls for supporting cut slopes or retaining soil backfill. Pile walls (such as bored pile walls) or conventional reinforced concrete retaining walls are commonly used for this purpose. Such design schemes are often costly because the mobilization of equipments for construction of large-diameter pile wall on sloping ground is difficult if not infeasible. A large temporary cutting or substantial shoring works may also be required for construction of a conventional retaining wall. This paper discusses some cost effective design schemes of retaining walls. Case studies of some recently completed retaining walls designed using such concepts are discussed.

1 INTRODUCTION

Due to scarcity of land and the presence of hilly terrain in Hong Kong, retaining walls are often required for supporting natural or man-made for creating flatter land for building and infrastructure developments. Cantilever walls or L-shaped reinforced concrete retaining wall are commonly adopted for this purpose.

In the past, hand-dug caisson was a cost-effective way for forming cantilever retaining wall in hilly terrains with difficult construction access. This construction method has practically been banned in Hong Kong since 1995 for health and safety reasons. Since then, large diameter bored pile walls have been adopted to replace hand-dug caissons. Bored piling works involve heavy equipments. One or more steel temporary platforms may be necessary for supporting the construction plants and crawler cranes for bored piling works on a slope. Such temporary steel platforms are costly to construct and dismantle.

Some efficient and effective retaining wall designs as described in Figure 1 have been available for some time although they are not common or have not yet been used in Hong Kong. They include retaining walls with a sloping wall back (Figure 1a), the so-called wall-on-wall system (Figure 1b) and wall-slab system (Figure 1c) (天津大學, 1980; 路基設計手冊編寫組, 1984; Li, 1990). A design concept of cantilever wall with stabilizing base has been introduced by Carder et al. (1999) and Powrie et al. (1999) (Figure 1d). For the wall-on-wall system in Figure 1b, a structural slab is provided behind the wall back of a gravity wall to support the soils above it, producing two beneficial effects. First, the self-weight of the soil above the slab produces a resisting moment to the wall. Second, the overburden pressure directly underneath the structural slab is small (see Figure 1e) as the slab is supporting the self-weight of the soils above it. This leads to a local reduction in earth pressure. As the shielding effect diminishes with depth, the reduction in earth pressure can only occur over a certain depth below the structural slab as indicated in Figure 1b. The wall-slab system in Figure 1c is an extension of the concept of a wall-on-wall system. The structural slab is extended to beyond the failure plane of the active wedge. The structural slab completely supports the soils above the slab, leading to a significant reduction in earth pressure all the way below the structural slab.

In the following, more detailed discussions will be presented for the two retaining wall schemes in Figure 1a and Figure 1d as the other two systems in Figure 1b & c have not yet been used in Hong Kong.

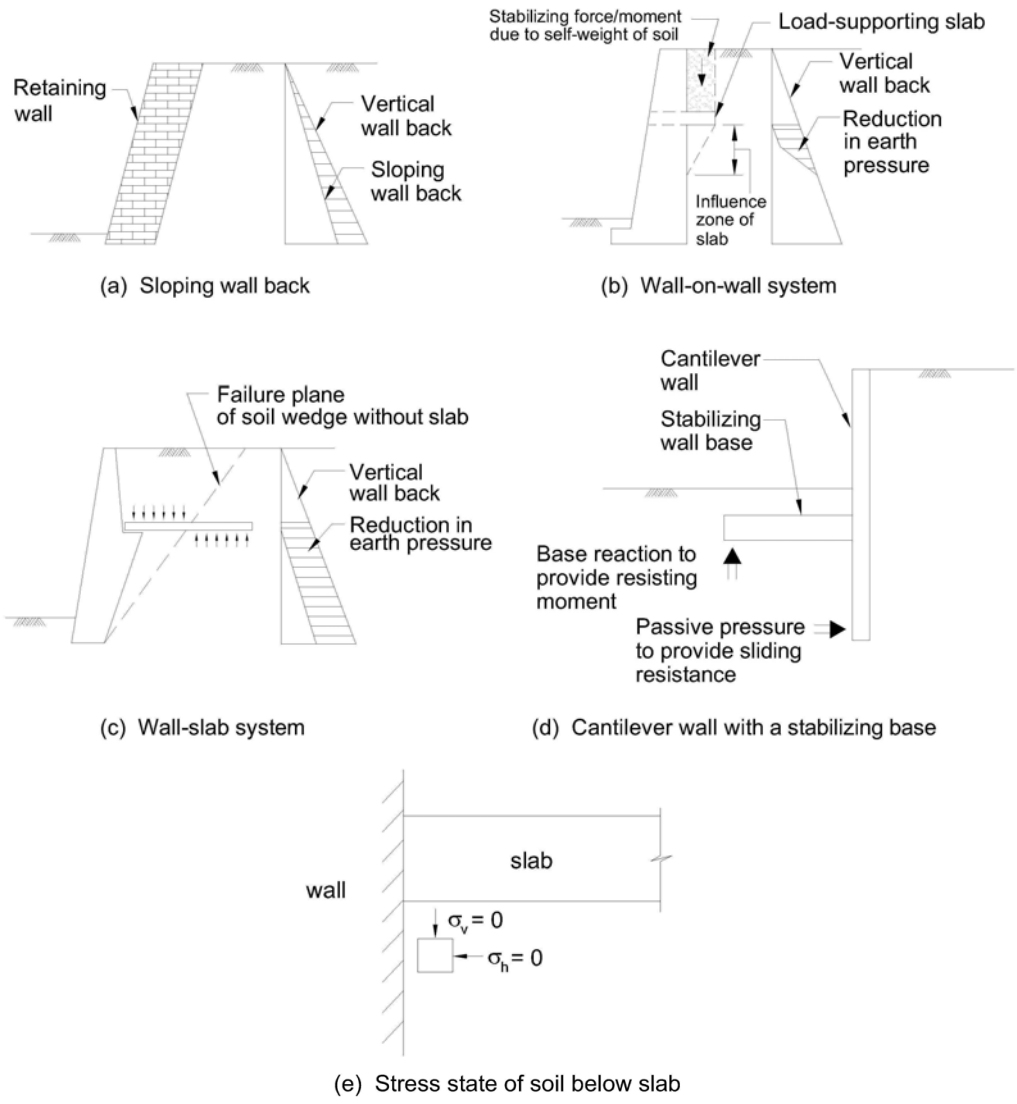

Figure 1: Some efficient retaining wall design

2 RETAINING WALL WITH A SLOPING WALL BACK

Figure 2 shows the value of K_a for a retaining wall supporting a soil with angle of shearing resistance $\phi = 35^o$ and wall friction angle $\delta = 0$. The inclination α is defined as positive when the wall back leans backward. The results in Figure 2 are obtained using the formula for K_a presented in Geoguide 1 (GEO, 1993) based on the Coulomb's method. It can be observed that the reduction of K_a with α is significant, with K_a reducing from 0.25 for a vertical wall back to 0.126 for a steep wall back with a relatively small inclination of 25^o.

The findings, as indicated by Figure 2, have been known for a long time and discussed in many geotechnical textbooks. In fact, geotechnical engineers have made use of such a concept for decades for the design of crib walls or perhaps for centuries for the design of slanted gravity walls (Figure 1a) for retaining fill platforms or protection of a steep cut slopes.

Sometimes, a vertical or near-vertical wall face may be preferred to a slanted retaining wall to maximize the usable space above the wall top. This is often the case for road widening works where additional traffic lanes are to be provided above and in front of the wall. A gravity wall with a constant inclination of wall back as indicated in Figure 3a may not be economical due to the large cross sectional area and hence material cost. Earth pressure increases with overburden pressure and depth. The total reduction in earth pressure will be more significant when the reduced K_a occurs at a lower depth. For this reason, the wall geometry in Figure 3b is more efficient as it reduces the cross sectional area of the retaining wall. A sloping back leaning backward is

provided at the lower portion of the wall to take advantage of a larger reduction in earth pressure. A vertical or forward leaning wall back is adopted for the top part of the wall to reduce the cross sectional area even if it will slightly increase the earth pressure.

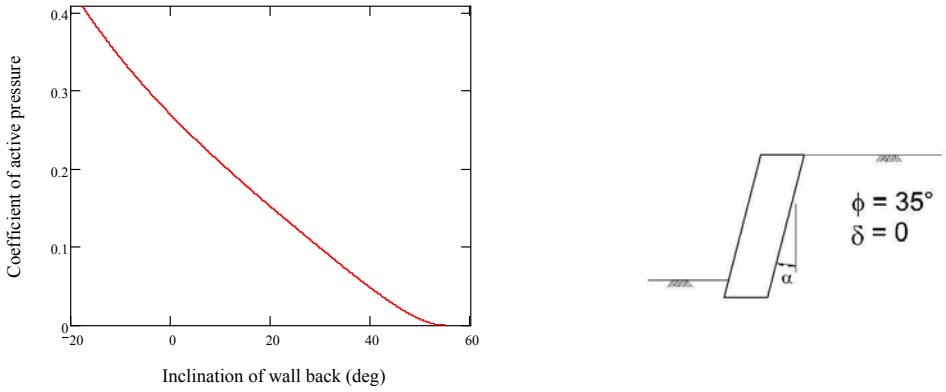

Figure 2: Variation of K_a with inclination of wall back α

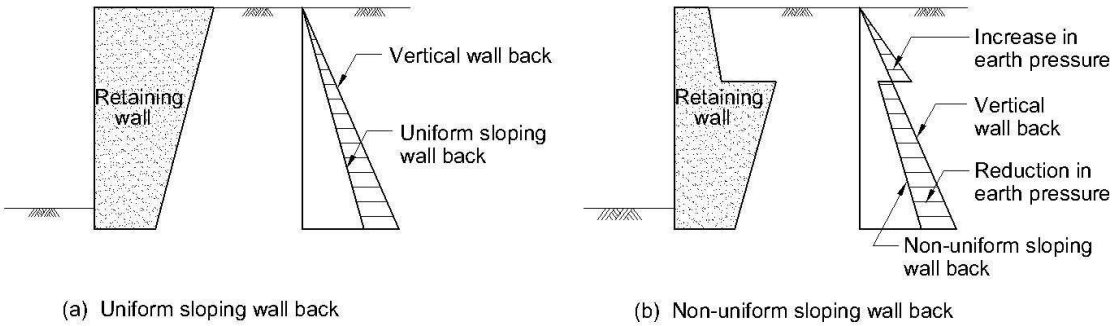

Figure 3: Retaining wall with non-uniform sloping wall back

To further reduce the volume of concrete, the base width can be reduced as shown in Figure 4a. In doing so, the bearing stress will increase and the factor of safety against bearing failure may not be adequate. An extended reinforced concrete toe may need to be provided to enhance the bearing capacity. To reduce the material cost even further, reinforced concrete wall base and wall back can be used as shown in Figure 4b. A cross wall can be provided at suitable spacing to reduce the bending moment on the wall stem, wall base and wall back, but it will make the backfilling process more difficult. The wall geometry as shown in Figure 4 looks like a goose and such a retaining wall has often been dubbed the "goose shaped wall".

Figure 5 shows some examples completed goose shaped walls that have been used in several road widening projects in Hong Kong. All the walls in Figure 5 need to support a sizeable noise barrier above the wall stem. Therefore, those retaining walls need to resist wind moment in addition to soil pressure. Figure 6a shows a photograph of a goose shaped wall under construction and Figure 6b the completed wall. The slope behind the wall in Figure 6a is a soil nailed cutting.

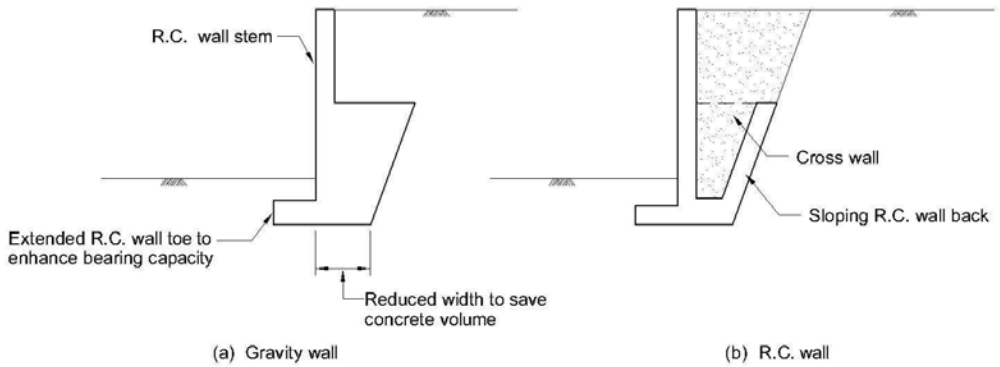

Figure 4: Variation of retaining wall geometry with sloping wall back

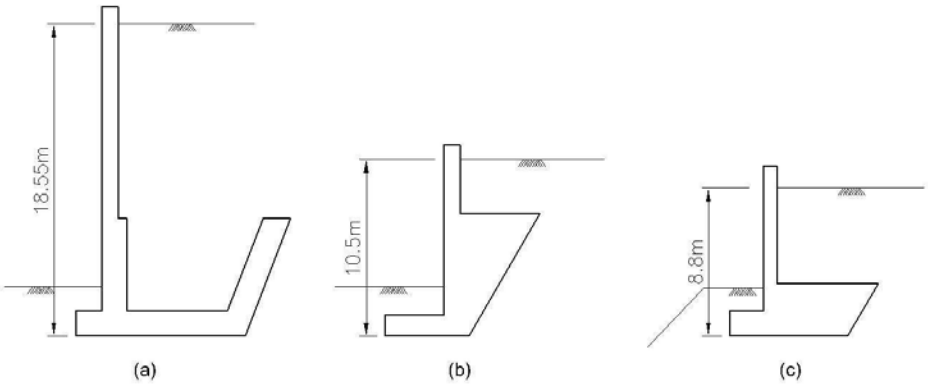

Figure 5: Examples of goose shaped wall used in Hong Kong

(a) under construction (b) completed wall

Figure 6: Photographs of a goose shaped wall

It may be argued that a goose shaped wall is not an efficient wall system in terms of the total volume of concrete used. The greatest benefit of a goose shaped wall often comes from the reduced cost of shoring works and not the material costs, especially when a retaining wall is needed for supporting an existing slope or when there is a restriction on the extent of temporary slope cutting. A goose shaped wall has a narrower base in comparison. It can often be constructed much more economically using a soil-nailed cutting (Figure 7b). For an L-shaped retaining wall, the extent of a soil nailed cutting will be larger due to its wider base. If there is a restriction on the extent of slope cutting, one may have to resort to a strutted or tied-back embedded wall for constructing the L-shaped wall (see Figure 7a). In Hong Kong, only passive soil nails are allowed for permanent works. Although soil nails are often used for supporting the temporary cutting for construction of goose shaped walls, such active soil nails cannot be relied upon for enhancing the wall stability. An alternative is to install additional soil nails after the cutting has been formed. The additional soil nails can be integrated with the retaining wall as indicated in Figure 7c. As long as the goose shaped wall is designed to factors of safety larger than one against sliding and overturning, the additional soil nails can remain passive during their working life. If this design concept can be accepted, the size of the goose shaped wall and also the extent of soil cutting can be substantially reduced.

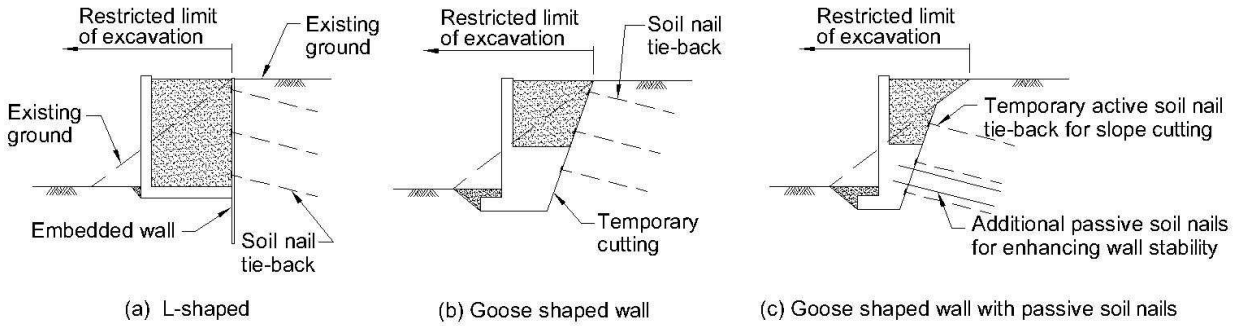

Figure 7: Construction of retaining wall

3 CANTILEVER RETAINING WALL WITH STABILIZING BASE

As the height of the retaining wall increases, the goose shaped retaining wall may no longer be efficient or practicable, unless the concept of passive soil nails as described in Figure 7c can be accepted. It is because the goose shaped wall may become too massive and the required extent of temporary slope cutting too large to be economical. Under this situation, designers often resort to a cantilever wall formed by large diameter bored piles because the design work for a cantilever wall is extremely simple. Such retaining walls are, however, nightmares for the contractor if there are to be constructed on an existing slope.

The concept of a cantilever wall with a stabilizing wall base as discussed in Figure 1d offers an alternative to purely cantilever bored pile walls as the size of the cantilever wall can be reduced with the help of a stabilizing base. In this system, the soil reaction from the stabilizing base will provide a resisting moment to enhance the overturning stability of the cantilever wall and the passive soil resistance on the cantilever wall contributes to increasing the sliding stability of the wall. While the concept is attractive, the difficulties lie in developing a suitable method for temporary support of the cantilever wall before constructing the stabilizing base and designing suitable details for connecting the stabilizing base to the cantilever wall. If large diameter bored piles are still used for the cantilever wall, the advantage of this concept will be lost.

Li (2012) has recently suggested a modification to the concept of the cantilever retaining wall with a stabilizing base as illustrated in Figure 8.

A steel soldier pile wall can be constructed initially for supporting the excavation. The soldier piles can be supported by raking struts or tie-backs as indicated in Figure 8a. The installation of steel soldier piles requires much lighter or smaller equipment and a steel temporary platform may not be necessary for constructing the soldier piles on sloping ground. Once the excavation in front of the steel soldier piles is completed, a reinforced concrete reversed L-shaped retaining wall can be built to integrate with the soldier pile wall to form a composite cantilever retaining wall with a stabilizing base (Figure 8b). To ensure that the soldier pile and the reinforced concrete retaining wall can work effectively as a whole, shear studs or reinforcement need to be provided between the wall stem and the soldier piles to resist the interface shear forces and tension. The concept of integrating passive soil nails with the retaining wall as discussed earlier for the goose shaped wall should also be useful for reducing the size of the reversed L-shaped retaining wall.

The design concept illustrated in Figure 8 has recently been adopted to replace a 3m diameter bored pile wall for supporting a major slope with a retaining height exceeding 20m. No steel platform was necessary for constructing the composite wall. Figure 9a shows a close-up view of the tied-back soldier piles with shear studs installed. Figure 9b is a panoramic view of the completed excavation in front of the soldier piles and Figure 9c shows the completed cantilever wall with the stabilizing base buried in the soils.

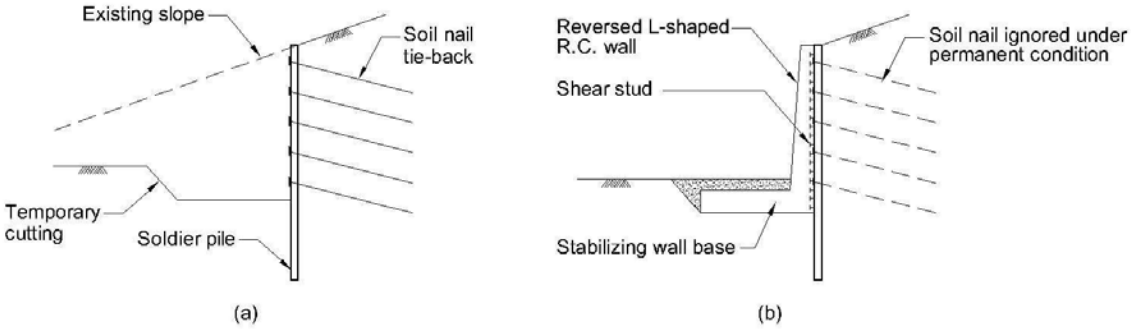

Figure 8: A modified concept of cantilever retaining wall with stabilizing base

(a) (b) (c)

Figure 9: Photographs of a retaining wall with stabilizing base

4 DISCUSSIONS

The retaining wall systems discussed in this paper are not new. They have been described in geotechnical textbooks published in the mainland China in the early 1980s or perhaps even earlier. The concept and case history of a cantilever retaining wall with stabilizing wall base were published 14 years ago by Carder et al. (1999) and Powrie et al. (1999). As discussed by Li & Lo (2008), engineers are usually conservative and prefer to be followers rather than pioneers. By showing that some of the more efficient retaining wall systems described in Figure 1 have actually been used successfully in civil engineering projects in Hong Kong, it is hoped that the concepts discussed in this paper can be adopted more widely for economical design of retaining wall for infrastructure and building developments.

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Reappraisal of the Acceptance Criteria of the Static Load Test for Housing Projects in Hong Kong

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ABSTRACT

The current acceptance criteria for the test pile in the static load test generally adopted for private and public housing projects in Hong Kong are specified in Section 8.4 of the *Code of Practice for Foundations* published by the Buildings Department of the Hong Kong SAR Government (Buildings Department 2004). These criteria were developed on the basis of the maximum settlement of the test pile induced by twice the design pile capacity exerted on the pile for at least 72 hours, and the residual settlement of the pile after the maximum load exerted on the pile has been removed. Basically they are partially based on the research results of Davisson (1972). The acceptance criteria adopted by the Architectural Services Department of the Hong Kong SAR Government for pile foundations of public buildings are basically identical (Architectural Services Department 2012). It is timely to reappraise the applicability and validity of these acceptance criteria for pile foundations of Hong Kong housing projects so as to avoid excessive conservatism in the design and construction of pile foundations, or to mitigate the risk of foundation failure.

1 INTRODUCTION

1.1 Background

The total area of the Hong Kong Special Administrative Region (HKSAR), China is 2,755 km^2, including 1,104 km^2 of land and 1,651 km^2 of sea. Hong Kong is accommodating a population of approximately 7.07 million as of mid-2011 (http://www.yearbook.gov.hk/2011/en/pdf/Facts.pdf) and one of the most important financial and trading centers in the world. The average population density of Hong Kong is 6,540 persons/km^2. However, most of the population is being housed in 215 km^2 of urban development because of steep natural terrain and stringent planning controls. Over 400 km^2 have been designated as protected areas including country parks, special areas, and conservation zonings. As a result, the most densely populated District Council district is Kwun Tong with a population density of 55,200 persons/km^2. The concentration of population and economic activities in such a small area exert an intense demand for land to sustain the rapid economic development of Hong Kong.

By the end of 2011, approximately 2.1 million people have been living in public rental housing estates of the Hong Kong Housing Authority and the Hong Kong Housing Society. The total number of flats was 761,200. Due to the relatively small land supply for housing developments in Hong Kong, it is inevitable to construct high-rise residential buildings to cope with the rapid growth of the Hong Kong population and the ever-improving living environment. These high-rise buildings have to be supported by pile foundations due to the demand for the extremely heavy load-carrying capacity foundations. The most widely adopted pile types in Hong Kong for high-rise residential buildings are large-diameter bored piles, driven steel H-piles, socketed steel H-piles, and driven precast prestressed high-strength spun concrete piles.

Testing of foundations is often required for one or more of these purposes (Buildings Department 2004): (a) to ascertain the performance of the foundation under load; (b) to establish or justify the design parameters; and (c) to verify the structural integrity of the foundation elements. The types of tests chosen and the numbers of tests required should be appropriate to the type of foundation and suitable for the purposes required. With the exception of large-diameter bored piles, the engineering performance of most types of piles is evaluated by

maintained static pile load tests. Moreover, static pile load tests are routinely required by construction contracts as quality control and assurance measures for pile foundations.

1.2 Current practice in Hong Kong

The current acceptance criteria for test piles in static load tests generally adopted for private and public housing developments in Hong Kong are stipulated in Section 8.4 of the *Code of Practice for Foundations* published by the Buildings Department of the HKSAR Government (Buildings Department 2004). These criteria were developed on the basis of the maximum settlement induced by twice the design pile capacity exerted on the test pile for at least 72 hours, and the residual settlement of the test pile after the maximum load exerted on the pile has been removed. The acceptance criteria adopted for foundations of other public buildings by the Architectural Services Department of the HKSAR Government are basically identical as given in the *General Specification for Building. 2012 Edition* (Architectural Services Department 2012). However, it should be noted that the Civil Engineering and Development Department of the HKSAR Government adopts the Brinch-Hansen 90% criterion (Brinch-Hansen 1963) for acceptance of piles, which is different from that adopted for pile foundations of housing or building projects in Hong Kong.

Regardless whether it has been explicitly stated in various government publications, these acceptance criteria are basically developed partially on the basis of the research results of Davisson (1972). In fact, many practicing geotechnical engineers have cited the reference routinely and indiscriminately without a thorough study or understanding of the original publication. It should be noted that Davisson (1972) proposed only one settlement criterion for the estimation of ultimate pile capacity, but there are two acceptance criteria for test piles in static load tests performed in Hong Kong.

The original intentions of Davisson (1972) in the development of the settlement criterion would be fundamentally scrutinized in this paper so as to answer the question, "Have we misinterpreted the intentions of Davisson (1972) in adopting the criterion?" The limits of applicability of these acceptance criteria will be identified to answer the question, "Was the pile database used by Davisson (1972) to develop the criterion similar to ours?" If the nature of field data in these two databases is different, are we over-extrapolating the results of Davisson (1972)? It is thus timely to reappraise the applicability and validity of these acceptance criteria in Hong Kong housing development projects to avoid excessive conservatism in the design and construction of pile foundations, and to mitigate the risk of foundation failure.

2 THE PILE ACCEPTANCE CRITERIA

2.1 Current acceptance criteria

The acceptance criteria being used in private and public housing development projects in Hong Kong for piles with a diameter or the least lateral dimension not exceeding 750 mm are (Buildings Department 2004):

(i) the maximum settlement measured at the pile head under the maintained load of twice the design pile capacity for at least 72 hours before removal should not exceed

$$\frac{2WL}{AE}+\frac{D}{120}+4\ \text{mm} \tag{1}$$

where W = design pile capacity (kN); L = length of the pile (mm) (for piles with rock sockets, L is measured to the center of the rock socket; and for piles without rock sockets, L may generally be taken as the entire length of the pile); A = cross-sectional area of the pile (mm^2); E = Young's modulus of the pile material (kN/mm^2); and D = the least lateral dimension of the pile (mm); and

(ii) when the rate of recovery after the removal of the maximum test load is less than 0.1 mm/hour observed in a period of not less than 15 minutes, the residual settlement at the pile head should not exceed the greater of

(a) $\frac{D}{120}+4\ \text{mm}$; and (2a)

(b) 25% of the maximum pile head settlement during the test. (2b)

2.2 Original intentions of Davisson (1972)

The original intentions of Davisson (1972) were to answer two of the most important questions with respect to high capacity driven piles based on the use of wave equation analyses of pile driving: (1) the pile cross-sectional area required to be driven to a certain load capacity; and (2) the hammer energy required to achieve that capacity (Davisson 1972). Piles of capacities exceeding the range of 80 to 100 tons were considered as high capacity piles in his discussion. Interpretation of load test results and final driving resistance were reviewed as subsidiary topics. The method of interpreting pile load test results was developed to give a consistent comparison of the results of wave equation analyses with those of static load tests. The method has the advantage of being consistent with the results of wave equation analyses of pile driving which incorporate the physical properties of the pile and soil system in the analyses. Moreover, the method can be adopted in technical specifications for construction contracts.

It is evident that the criterion was not originally developed as one of the pile acceptance criteria. It was intended as a settlement criterion in the static load test so as to obtain similar ultimate pile capacity obtained by wave equation analyses. Whether the ultimate pile capacity obtained by wave equation analyses is a good indication for the engineering performance of the pile is worthy of further discussion but beyond the scope of this paper.

2.3 Davisson criterion

A pile of length L, cross-sectional area A, and Young's modulus E is considered as a fixed-base, free-standing column for the purpose of calculating its elastic deflection under exerted load. The elastic deflection line is then plotted as shown in Figure 1 to serve as an index for the interpretation of static load test results. An offset criterion similar to that used for some metals was adopted to determine the ultimate pile capacity. The offset criterion was developed for the tip resistance of the pile by considering the pile penetration required to cause yielding of the soil at the pile tip Q_{tip}. The penetration is known as soil quake which is approximately 0.10 in. for most soils for piles of normal dimensions of approximately 1 ft wide for the case histories presented in Davisson (1972). It was determined by Davisson (1972) that when the pile settlement is 0.15 in. plus quake below the elastic deflection index line, the ultimate pile capacity can be presumed to have been reached. The settlement criterion can also be applied to the side resistance pile. In addition to maintained static pile load tests, the method can also be applied to constant-rate-of-penetration pile load tests. Details of these different types of pile load tests are given in Tomlinson (2001).

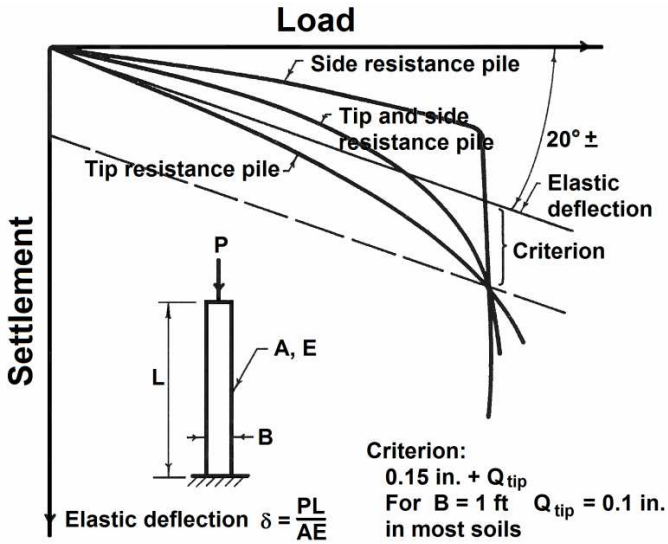

Figure 1: Interpretation of pile load test (modified from Davisson 1972)

2.4 The generalizations

The quake of 0.10 in. for piles of width of approximately 1 ft adopted by Davisson (1972) was later generalized to D/120 in Hong Kong, assuming the quake is proportional to pile width without any justification or investigation. The settlement of 0.15 in. was later converted to 4 mm. These generalizations are included in Eqs. (1) and (2) as pile acceptance criteria in Hong Kong housing projects. Moreover, the criterion on residual settlement after removal of the maximum test load, i.e., Eq. (2), does not exist in Davisson (1972).

3 DISCUSSION

The development of the Davisson criterion (Davisson 1972) was not originally intended as one of the acceptance criteria for piles in static load tests. It was developed so that the ultimate pile capacity obtained from the static load test is similar to that obtained by wave equation analyses. The criterion has been adopted as one of the acceptance criteria for piles of up to 750 mm in diameter in static load tests in Hong Kong housing and building projects without a similar comprehensive study for its applicability. Pertinent points are discussed as follows:

3.1 Number of loading cycles

The test procedure in Hong Kong requires the test load be applied in 2 equal increments up to the design pile capacity, then released and re-applied in 4 equal increments up to twice the design pile capacity and maintained for at least 72 hours before removal. The load-settlement characteristics of the 2nd loading cycle are used in the pile acceptance analysis in Hong Kong. However, the loading procedures used to obtain the static load test results presented in Davisson (1972) were not given in his paper. It is thus assumed the load increments were applied in a single loading cycle. The 1st and 2nd load increments in the 2nd loading cycle of static load tests in Hong Kong are basically re-loading the pile and the stiffness of the pile is thus increased. However, as cumulative settlement is used in the assessment, the stiffening effect of re-loading is thus nullified. Therefore, the use of the maximum settlement criterion is comparable to that of the Davisson criterion in this regard.

3.2 Duration of maximum loading

There is an important qualification for the Davisson interpretation method for static load tests (Davisson 1972), i.e., pile settlements due to long-term creep or consolidation are excluded in the analysis. The load-settlement data presented in the paper were obtained when the increment of load was held for not more than 15 minutes to 1 hour. The test pile may experience a considerable amount of settlement at twice the design pile capacity and produce an offset in the load-settlement curve under a prolonged test load. However, as the creep or consolidation phenomenon is not related to ultimate pile capacity, such curves cannot be interpreted directly by the Davisson criterion (Davisson 1972).

However, it is required in static load tests performed in Hong Kong that the maximum load is maintained for at least 72 hours before removal. Therefore, creep or consolidation settlement may have occurred during the test and the offset from the elastic deflection index line so measured is larger than that used in the Davisson criterion. As a result, the ultimate pile capacity so estimated is smaller than that by the Davisson criterion, resulting in unexpected conservatism or unnecessary failing of an acceptable pile.

3.3 Applicability of parameters in the Davisson criterion

It should be noted that the quake of 0.10 in. was assumed for piles of normal width of approximately 1 ft, i.e., 300 mm, by the experience of Davisson (1972) in the United States, in particular, for piles driven in the Chicago, New Orleans, and Milwaukee areas. The linearity between quake and pile width up to 300 mm has yet to be determined, not to mention extrapolating the linearity to pile width of 750 mm. Nonetheless, the quake was later generalized in Hong Kong from (0.10 in./1 ft) to D/120 in Eqs. (1) and (2), assuming the quake is linearly proportional to the diameter or the least dimension of the pile and the domain of linearity can be extended to piles of widths up to 750 mm. The quake is the settlement at the pile tip when the founding soil yields and the parameter can be measured. In fact, Davisson (1972) recommended that the quake should be

measured directly by a tell-tale during a static load test. Even if measurement is not made, a procedure to estimate the quake from the static load test data is given in Davisson (1972). The appropriate value of quake for the founding soil should then be used to estimate the ultimate pile capacity. However, the value of quake is a fixed function of pile width in Hong Kong practice without any engineering consideration or judgment on the geologic conditions at the pile tip. The approach violates the original intention of Davisson (1972).

It was determined by Davisson (1972) that when the pile settlement is 0.15 in. plus quake below the elastic deflection index line, the ultimate pile capacity can be presumed to have been reached. The settlement of 0.15 in. was later converted to 4 mm in Eqs. (1) and (2), regardless of soil type, pile type, or pile width. The settlement of 0.15 in. is evidently an empirical value from Davisson's experience in soils of the United States, in particular the Chicago area, and driven piles of normal widths, i.e. approximately 300 mm. Its validity for use in Hong Kong soils, piles installed by methods other than driving, and piles of larger widths is yet to be established.

3.4 Applicability of the Davisson criterion to different pile types

The Davisson criterion has been widely used in Hong Kong to determine the ultimate pile capacities of piles of different types of up to 750 mm in width in static load tests as indicated in Eq. (1). It should be noted that the static load test data used by Davisson (1972) to evaluate the validity of the proposed criterion were obtained from driven piles mostly of width of approximately 300 mm or smaller. Considerable extrapolation has been performed to apply the criterion to different types of piles of width larger than 300 mm. In Hong Kong, the criterion has been applied to most types of piles being constructed in Hong Kong for housing projects. As the engineering behavior of driven piles is not necessarily similar to those of the other types of piles, the validity of the extrapolation is uncertain. The criterion has even been extended to large-diameter bored piles not founded on rock in some railway projects in Hong Kong. It has been demonstrated by Kulhawy & Hirany (2009) that the use of the Davisson criterion in estimating the ultimate capacity of large-diameter bored piles would underestimate their ultimate capacities. The indiscriminate use of the Davisson criterion to determine ultimate pile capacities of piles of different types is obviously inappropriate.

3.5 Comparison with other acceptance criteria

The Davisson criterion is considerably more stringent than provisions of various building codes in the United States at the time of publication of the paper. For example, a pile of approximately 300 mm in width can usually be considered to fail at the offset settlement of 6.5 mm below the elastic deflection index line using the Davisson criterion. However, the New York City Code then allowed an acceptable offset settlement of 19.05 mm (0.75 in.). Other building codes in the United States allowed up to 0.28 mm/tonne (0.01 in./ton) of test load. For a 200 tonne test load, this would result in an acceptable offset settlement of 56 mm (2.2 in.) below the elastic deflection index line as opposed to 6.5 mm in the Davisson criterion. It should be obvious that building code methods of interpreting static load tests are not applicable to high capacity driven piles and should not be applied to them (Davisson 1972).

3.6 Residual settlement

It should be noted that the criterion on residual settlement does not exist in Davisson (1972), as it was not the intention of Davisson (1972) to develop acceptance criteria for test piles in static load tests. The residual settlement is measured after the pile has been subjected to the maximum load, i.e., twice the design pile capacity. The pile is loaded to twice the design pile capacity to evaluate its engineering performance at the ultimate limit state. It is inappropriate to use the residual settlement as an acceptance criterion at the ultimate limit state, as settlement is a serviceability limit state criterion. This approach is an illogical combination of the ultimate limit state and the serviceability limit state. Moreover, the satisfaction of the criterion stipulated in Eq. (2) requires full recovery of the elastic deflection of the test pile after the test load equivalent to twice the design pile capacity has been removed. However, there may still be negative side resistance acting on the test pile when the test load has been removed, preventing full recovery of the elastic deflection. Therefore, the criterion may be technically incompatible. A test pile failing to satisfy the residual settlement criterion does not imply the pile cannot perform its intended engineering function. The contractor normally drives the pile unnecessarily to a geologic stratum stronger than required so as to reduce the maximum settlement and to

satisfy the residual settlement criterion, leading to unnecessary conservatism, and unnecessary increase in construction costs and time.

If it is required to evaluate the performance of the test pile at the serviceability limit state, it should be evaluated when the test load exerted on the test pile is equivalent to the design pile capacity. The residual settlement should be measured during the 1st loading cycle after the test load equivalent to the design pile capacity has been removed. The approach is consistent with the concepts of the serviceability limit state.

4 CONCLUSIONS

The following conclusions can be drawn from this study:

(1) The original intentions and the domain of validity of the Davisson settlement criterion to estimate the ultimate pile capacity are reviewed in detail (Davisson 1972).
(2) Many aspects of the Davisson criterion have been misused when it was adopted as one of the acceptance criteria for test piles in static load tests in Hong Kong.
(3) The domain of validity of the Davisson criterion has been extrapolated for Hong Kong soils, piles installed by methods other than driving, and piles of widths greater than 300 mm without any justification or investigation for its proper application in Hong Kong.
(4) The residual settlement criterion adopted in static load tests in Hong Kong was not proposed by Davisson (1972). The criterion is inappropriate as it is an illogical combination of the ultimate limit state and the serviceability limit state of the engineering performance of the pile.
(5) The inappropriate uses of the Davisson criterion and the residual settlement criterion lead to unnecessary conservatism, and unnecessary increase in construction costs and time in the installation of piles in Hong Kong.

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Interpretation of Inclinometer Readings for Deep Excavation

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ABSTRACT

Deep excavations carried out in urban areas are increasingly more complex. The design scheme has to cope with many site constraints with literally no compromise on the construction time. These complex deep excavation designs are normally carried out with the aid of numerical modelling techniques. Due to the assumptions and simplifications made in the numerical models, the actual performance of the deep excavation may deviate from the predictions. Field monitoring therefore plays an important role in the evaluation of the actual performance of deep excavations. The measurements allow designers to verify the assumed ground conditions as well as the assumptions made in the design models. This paper discusses the use of field monitoring data to evaluate performance of the deep excavation at the West Kowloon Terminus of the MTR Express Rail Link, where the average depth of excavation is around 30 m. A comprehensive instrumentation and monitoring scheme has been implemented to assess ground movement and wall deflection during excavation. The focus has been placed on the reinforced concrete diaphragm wall deflection registered from a series of inclinometers installed in the wall panels. A number of interpretation methods are described and their merits and limitations are discussed.

1 INTRODUCTION

The complexity of deep excavations in urban areas and their close proximity to nearby sensitive structures have led to increasingly important roles for field instrumentation. Preset Alert-Action-Alarm (AAA) limits, determined based on numerical analyses during the design stage, ensures that performance of the retaining structure are reviewed during the construction phase in a timely manner. On the other hand, field monitoring data provides valuable information to the designers, who can review the appropriateness of parameters and assumptions adopted in the design phase.

Inclinometers installed in reinforced concrete diaphragm walls provide an effective means to monitor deflections of the wall and impacts of nearby construction activities to the retaining structure. Although deflections of diaphragm walls are routinely monitored, it is a less common practice to deduce the corresponding bending moments induced in the diaphragm wall, which is another crucial indicator of its performance with regards to the limit states. This may be attributed to the fact that no widely-accepted standard procedures exists for the determination of bending moments induced in a diaphragm wall panel based on measurements of lateral movements. Researchers such as Ooi & Ramsey (2003) and Liew & Choo (2004) suggested approaches to deduce the bending moments which are based on a similar concept, but they adopted different procedural details that may lead to significant discrepancies on the bending moment estimates. These discrepancies can affect the judgments on the performance and integrity of the diaphragm wall. Based on experience gained from the extensive instrumentation programme at the West Kowloon Terminus (WKT) of the MTR Express Rail Link, this paper presents the various approaches that may be adopted to estimate bending moments based on deflections measured by inclinometers. The merits and limitations of the approaches will be discussed, together with the necessary precautions when applying these techniques.

2 ESTIMATION OF BENDING MOMENTS BY NUMERICAL BACK-ANALYSES

In general, the bending moments (M) induced in the diaphragm wall is related to its curvature based on the following equation:

$$M = E_c I_e \kappa \tag{1}$$

where E_c = Young's modulus of concrete, I_e = effective moment of inertia and κ = curvature of the wall panel, which may be approximated by using different approaches. In this section, κ is approximated indirectly by the use of numerical back-analyses.

The use of finite element analysis software for engineering analyses has become increasingly popular in Hong Kong. When estimating the bending moments induced in the diaphragm wall, it may be tempting to conduct numerical back-analyses, e.g. using PLAXIS, to match the measured wall deflection, either by fine-tuning the soil stiffness around the retaining wall or through other numerical techniques, such as imposing volumetric contractions or even prescribed displacements near the wall.

As the goal of such back-analyses is to match the 'estimated' wall movement with the inclinometer readings, the estimated curvature (κ) of the wall will be very similar, if not identical, to the profile from field monitoring data. It should be noted, however, the bending moments calculated by the software are directly proportional to E_c and I_e, and their actual values for the constructed retaining wall often deviate significantly from their respective design values. As will be discussed in later sections, I_e often changes along the depth of the wall. Therefore, the bending moment estimates obtained from such back-analyses should be interpreted with caution, especially when design values of E_c and I_e are adopted as parameters in the analyses.

3 ESTIMATION OF BENDING MOMENTS FROM DEFLECTION MEASUREMENTS

Another common approach for the estimation of bending moment is through direct analysis of the inclinometer measurements (e.g. Ooi & Ramsey 2003; Liew & Choo 2004). Equation 1 still applies for this approach, but instead of assuming a certain value for moment of inertia, the value of I_e is evaluated with consideration of the wall deflection.

The moment of inertia (I) of a reinforced concrete section varies with the extent of cracking within the section. Once the concrete is cracked, contribution of the steel reinforcement should be taken into account in the evaluation of I. Therefore, the effective moment of inertia, I_e, depends on the geometry of the section, properties of the concrete and steel reinforcements, and the applied moment, M. I_e can be evaluated by the following equation:

$$I_e = \left(\frac{M_{cr}}{M}\right)^m I_g + \left[1 - \left(\frac{M_{cr}}{M}\right)^m\right] I_{cr} \le I_g \tag{2}$$

where I_g = gross (uncracked) moment of inertia, and I_{cr} = moment of inertia of a cracked transformed section. There are various recommendations on the value of m (Kalkan 2010), but the adopted value is usually around 3 (Ooi & Ramsey 2003; Liew & Choo 2004). M_{cr} is the cracking moment of the section and corresponds to the modulus of rupture of concrete (f_r) as follows:

$$M_{cr} = \frac{f_y I_g}{y_t} \tag{3}$$

where y_t = distance from centroid axis of cross-section to the extreme fibre, and f_r is evaluated based on the design strength of concrete.

According to Equations 1 and 2, the bending moment (M) and effective moment of inertia (I_e) are mutually dependent, and usually an iteration procedure is used to determine their values. For example, in the first iteration, the uncracked I_g can be adopted to obtain the corresponding bending moment profile. This profile is then used to determine I_e (Equation 2). The updated I_e values are then used to update the bending moment

estimates for the cracked section (Equation 1), and the process is iterated until a set of M and I_e values satisfying both Equations 1 and 2 are obtained.

A reasonable assumption of E_c is still required in this approach, but the estimated M is not as sensitive to E_c as in the first approach using numerical back-analyses. This is because changes in E_c will be counterbalanced, to some degree, by changes in I_e during the iteration process. For example, an increase in E_c will lead to increase in M during the first iteration (Equation 1), but increase in M will then cause reduction in I_e (Equation 2) in the subsequent iterations. Therefore, using this approach, the estimated M is no longer directly proportional to the assumed value of E_c.

3.1 Evaluating curvature from inclinometer measurements

Contrary to the back-analysis approach, the curvature (κ) induced in the retaining wall can be directly approximated by taking the second derivative of the measured lateral wall deflection with respect to the depth (Ooi & Ramsey 2003). Figure 1 shows an example where piecewise cubic curve-fitting is conducted for a window of 11 data points from real inclinometer readings obtained at the WKT construction site. For a typical reading interval of 0.5 m this represents a window of 5 m. The second derivative of the best-fit cubic curve represents the curvature for this portion of the readings. This window is moved along the depth of the measurements to obtain κ along the entire diaphragm wall panel. The diaphragm wall is 1.5 m thick, with Grade 45 concrete concreted by the tremie method. The corresponding parameters in Equation 3 can then be evaluated, and based on the reinforcement details at different depths of the wall (that can be used to evaluated I_{cr}), the bending moments can be estimated following the procedures represented by Equations 1 and 2.

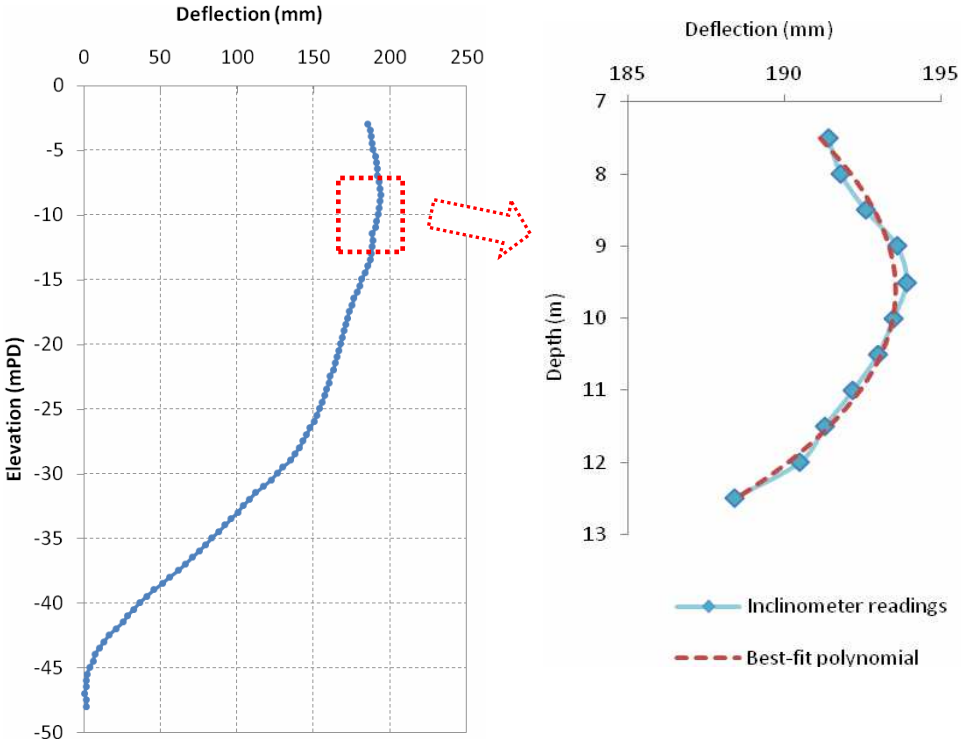

Figure 1: Curve-fitting of deflection profile obtained from inclinometer

Similar approaches have been suggested by researchers such as Ooi & Ramsey (2003) and Liew & Choo (2004). However, there have been significant discrepancies among different researchers regarding the 'optimum resolution' of the curve-fitting exercise, indicated by size of the window (from 2 m to 5 m) and the order of the best-fit polynomial (from quadratic to 6th order polynomials). As shown in Figure 1, the 'best-fit' cubic polynomial does not exactly match every data point from the inclinometer readings. Adopting a smaller

window and/or higher order polynomial will enhance the fitting. However, this tends to capture measurement errors that may arise from different sources such as the instrument errors, defects in the casing or human errors during the measurement process. These errors would lead to unrealistic sharp changes in the profile of curvature or bending moment.

Figure 2a shows the bending moments estimated by using 3rd order and 6th order curve-fitting of the wall deflection shown in Figure 1 (assuming E_c = 18.5 MPa). In this case, the general profile is similar below the elevation of -15 mPD, but the discrepancies are significant above this elevation. Adopting a higher resolution for the curve-fitting (6th order polynomial) leads to abrupt changes in the bending moments above -15 mPD. It should be noted that in this case no slabs or steel struts had been installed to support the wall when the inclinometer readings were taken. The sharp changes of bending moments within the diaphragm wall are considered unlikely, and the results obtained by the 6th order curve-fitting may be manifestations of instrumentation errors at those elevations.

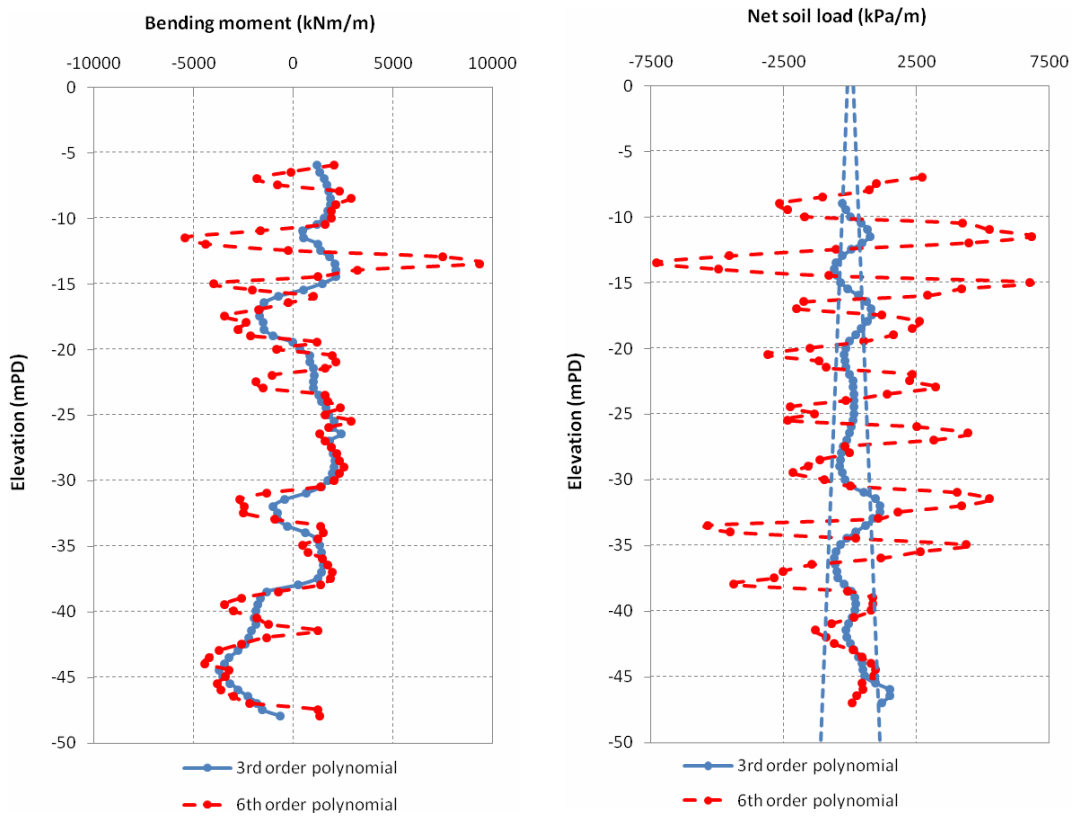

Figure 2a (left): Bending moment estimates by 3rd and 6th order curve-fitting;
Figure 2b (right): Net pressures acting on the wall deduced by twice differentiating bending moment profiles

As mentioned earlier, Ooi & Ramsey (2003) and Liew & Choo (2004) suggested different sizes for the window and orders of best-fit polynomial, and their recommendations may lead to significantly different bending moment estimates. The 'optimum resolution' for the curve-fitting may vary with project conditions, and the level of skills of the individual undertaking the instrumentation work. To determine the appropriateness of the curve-fitting settings, the bending moment profile can be differentiated twice with respect to the depth, and the resulting profile will be the net soil and water loads acting on the diaphragm wall along its depth.

For example, Figure 2b shows the net loads along the depth of the wall obtained by differentiating the bending moment profile in Figure 2a. With the 3rd order polynomial fitting, the loads are generally within the probable limits of soil and water loads, represented by the two straight lines in Figure 2b. In this case, the limits of soil and water loads are determined using in-situ groundwater information and assuming at-rest lateral earth pressure coefficient. In the absence of struts or slabs to provide propping force, it is improbable

that the net loads should exceed these limits. Depending on the project conditions, it may be possible to use the active or passive earth pressure coefficients to determine the limits. It should be noted that with 6th order polynomial fitting, there is substantial fluctuation in the net load profile (dashed line), which exceeds the probable limits of soil and water loads. In other words, the load profile in Figure 2b also suggests that 6th order polynomial likely captures measurement errors from the inclinometer readings and the corresponding bending moment profile may not be representative of the bending moments experienced by the wall panel.

4 ESTIMATION OF TENSILE STRAINS IN REINFORCEMENT BARS

The approaches described in Sections 2 and 3 involved assumptions on the Young's modulus of the reinforced concrete (E_c), and variability regarding E_c are inevitable. Meanwhile, Equation 2 is an empirical formula of I_e and its applicability depends on factors such as the reinforcement ratio of the reinforced concrete. To avoid the uncertainties regarding concrete properties, another approach is proposed herein, which involves estimating the tensile strain induced in the steel reinforcement bar (ε_{st}). Comparisons between ε_{st} with the yield strain of steel ε_y (f_y/E_s) gives an indication of level of strength mobilization of the tensile reinforcement. ε_{st} can be evaluated based on the curvature of the diaphragm wall panel, according to the following relationship:

$$\varepsilon_{st} = (d - x)\kappa \tag{4}$$

where d = effective depth and x = depth of neutral axis.

ε_{st} may be compared with the yield strain of the steel reinforcement (Figure 3). Alternatively, material factors and/or load factors can be considered in the comparison. For example, ε_{st} may be compared with $\varepsilon_y/\gamma_m\gamma_f$, where γ_m and γ_f are the adopted material factors and load factors.

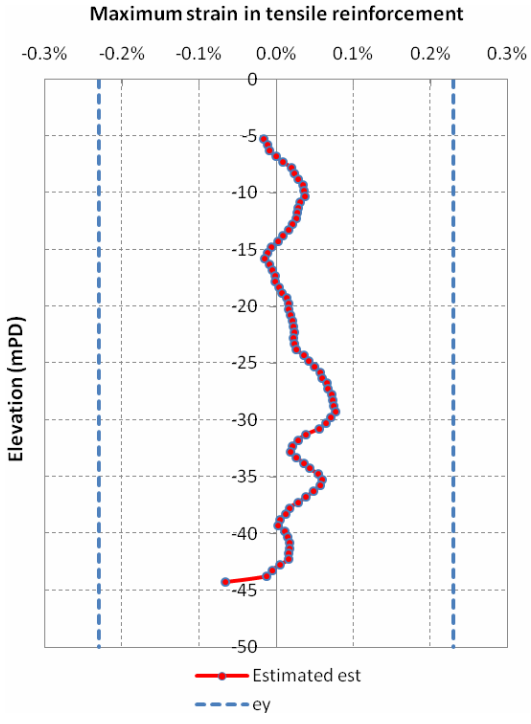

Figure 3: Comparison between tensile strains in steel reinforcement and yield strain of steel

In Equation (4), the depth of neutral axis (x) depends on the amount of steel reinforcements, the applied bending moment and degree of cracking in the concrete section. In theory, a higher value of $d - x$ results in a more conservative (higher) estimate of induced tensile strain. In general, assuming x to be the depth of neutral axis for a fully-cracked concrete section leads to a conservative assumption, unless when the section is loaded

almost to its yield strain, which is unusual in normal design and construction scenarios. Tensile strains in steel reinforcements obtained by this approach may be reviewed together with the bending moments estimated by previous approaches, to ensure that uncertainties in E_c and I_e do not lead to unrealistic estimates.

5 CONCLUSIONS

This paper discusses several approaches to evaluate bending moments induced in diaphragm walls based on inclinometer measurements of wall deflections. Although the curve-fitting approach is commonly used in the derivation of curvature in the retaining wall, the choice of the polynomial and size of window for curve-fitting can have significant impacts to the bending moment estimates. Inappropriate choice of these parameters may lead to unrealistic results and false alarms regarding the performance of the retaining structure.

Two approaches are introduced in this paper, which provide 'reality check' to the bending moment estimates. The first approach considers the limiting pressure conditions acting on the wall compared with net pressures deduced from the inclinometer readings, and may be used to confirm the appropriateness of curve-fitting parameters. The second approach considers tensile strains induced in the tensile steel reinforcements, which eliminates the needs to assume E_c and I_e parameters in evaluating bending moments. These two approaches may provide more comprehensive interpretation of the inclinometer data, resulting in more complete assessment of the performance of the retaining wall.

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Back-analyzed Elastic Moduli for Sand and Saprolite

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ABSTRACT

Numerical analyses have been conducted on an excavation supported by diaphragm walls embedded in various types of soil. In this case study, elastic moduli for sand fill, alluvium and saprolites were back-analyzed and related to cone tip resistances and to blow-counts of standard penetration tests.

1 INTRODUCTION

Cut-and-cover excavation supported by the diaphragm walls has been one of the popular construction methods for basement of buildings, rail tunnels, metro stations and expressway tunnels. Coastal reclamation would be one of the source of land supply for housing developments. Ground movements and lateral wall deflections are often the major concern especially where the development is carried out in urban areas. In order to assess the wall and ground movements due to excavation, numerical analysis are commonly conducted for design and for performance review. One of the key input parameters for analysis is the Young's moduli for soil, which are often obtained by back-analyzing case histories of deep excavations. This paper presents an excavation case history with the diaphragm wall embedded in various soil strata. Numerical analyses have been conducted and the back-analyzed Young's moduli for various soil strata are obtained. The elastic moduli are related to the in-situ tests results obtained from cone penetration and standard penetration tests. The relationships are compared with those back-analyzed from other case histories.

2 CASE STUDIED

The performance of the diaphragm walls observed during excavation for a cut-and-cover tunnel reported by Wong (2012) is studied herein. Figure 1 shows the configuration of the tunnel and the location of instruments relevant to this study. The typical subsoil profile is depicted in Figure 2. In the descending order, the ground is composed of fill of about 20 m in thickness, the alluvial deposits of about 2 m in thickness, saprolites of

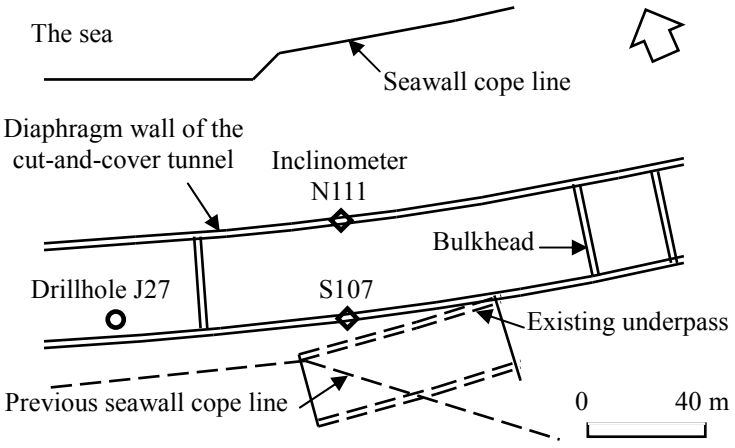

Figure 1: Layout of instruments for section crossing S107-N111

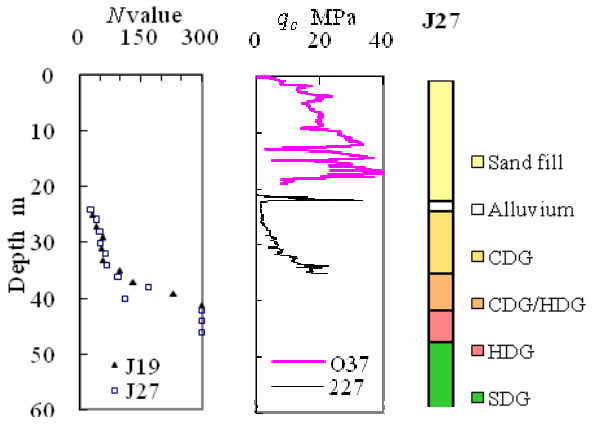

Figure 2: Typical subsoil conditions

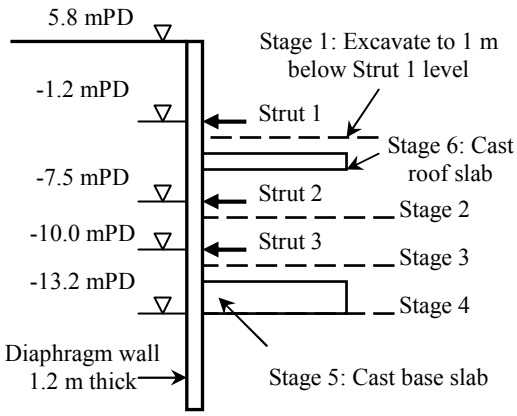

Figure 3: Construction stages

completely decomposed granite (CDG) to highly decomposed granite (HDG) and the bedrock of slightly decomposed granite (SDG). The intermediate stratum CDG/HDG is a transition zone between CDG and HDG. As depicted in Figure 2, the standard penetration test *N* values for CDG and CDG/HDG range from 15 to 80 and from 80 to 200 respectively. For HDG, the *N* values range from 200 to over 300. The alluvium belongs to Chek Lap Kok Formation that deposited in the Middle to the Late Pleitocene epoch.

The ground level, for example, at drillhole no. J27 that located on the south side of the project site was 4.0 mPD. The mean sea level is at 1.2 mPD. The fill mainly comprises hydraulically placed sand fill. Wong (2009) reported the properties of sand for this case of interest. It comprises sub-rounded to sub-angular, well-graded fine to coarse sand. The fine contents generally less than 6 % with the mean grain size ranging from 0.4 mm to 1.2 mm and an average uniformity coefficient of 4.3. The gradation of sand for this case of interest is similar to that for Chek Lap Kok (CLK) sand, the characteristics of which have been reported by Shen et al. (1997). After densified with vibrocompaction, a layer CDG fill compacted to 95 % of the maximum dry density was laid on top of the sand fill up to the elevation around 5.5 mPD.

Based on results of calibration chamber tests conducted on 5 types of sand in Hong Kong, Shen et al. (1997) reported that the relative density, D_r , of sand can be related to the cone tip resistance, q_c , determined from cone penetration tests and to the in-situ mean normal effective stress. Adopting the empirical coefficients established for CLK sand, Wong (2009) assessed the D_r values for sand fill of this case of interest. Based on the post-compaction q_c profiles, it is assessed that the sand fill is densified to the D_r values of 85 % or larger. Although Figure 2 shows that silt seams are encountered at the depths of 12 m and 15 m, adjacent q_c profiles confirm that they are localized features.

The cut-and-cover tunnel of this case of interest has the lowest excavation level of -13.2 mPD. The diaphragm wall panels forming the integral part of the tunnel box structure are typically 1.2 m in thickness, 31.8 m in width and 11 m in height. The wall panels are embedded at least 300 mm into SDG. The bottom-up method was adopted for construction. As depicted in Figure 3, excavation was conducted in Stage 1 to Stage 4. The lower two levels of struts were removed in Stage 5 after casting the base slab and the uppermost struts were dismantled in Stage 6 after casting the roof slab.

3 NUMERICAL ANALYSIS

An instrumented section crossing inclinometers no. S107 and N111, which were installed in the south and the north walls respectively, has been selected for numerical analyses. Back-analyses have been conducted to

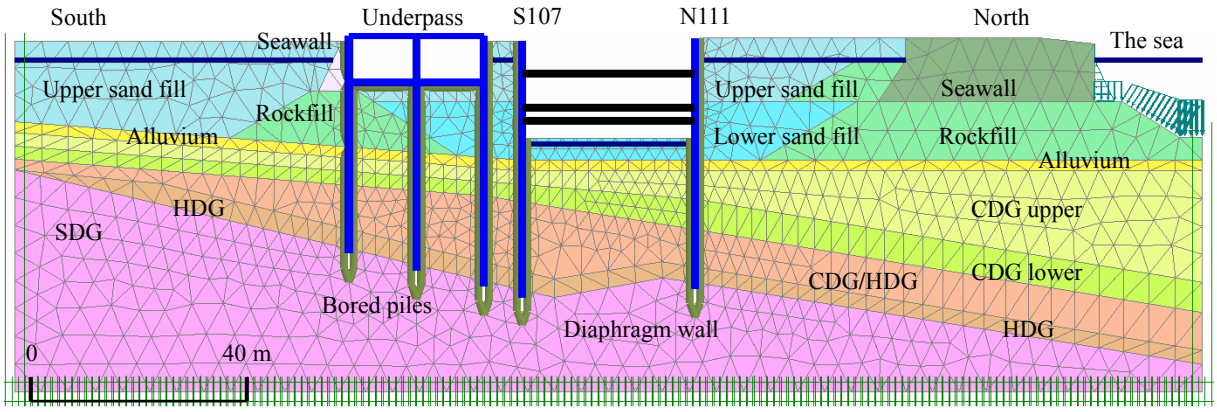

Figure 4: Finite element mesh adopted in PLAXIS analysis

calibrate the soil parameters. This is achieved by comparing the results obtained in back-analyses with the observed performance of walls. For the case of interest, analyses have been performed by using the finite element program PLAXIS (PLAXIS BV, 2011) developed at Delft University and made available commercially by PLAXIS BV of the Netherlands. Figure 4 shows the finite element mesh adopted in the analyses. Since SDG is a competent stratum and ground movements therein are expected to be small, the base of the finite element mesh is placed at a depth of 5 m to 40 m below the SDG surface.

The ground strata are modeled by 15-node elements. The Mohr-Coulomb model is adopted for simulating the behavior of soils in all the subsoils. The fill, the alluvium and the saprolites are modeled as the drained type materials. The soil parameters adopted in the analyses are summarized in Table 1. It is noted that the drained Young's moduli, E', for various soil given in Table 1 are the back-analyzed values. The sand fill behind the southern seawall was built in an earlier phase. In order to simplify modeling, the soil parameters for this sand fill and the superficial CDG fill follow those for the upper sand fill. It is noted that the analysis would become unstable in the final stage of excavation if the cohesion intercept, c' , of 5 kPa or less is adopted for sand fill. The set of effective shear strength parameters for sand fill presented in Table 1 is determined by back-analysis. The diaphragm walls are simulated by plate elements and the Young's modulus for concrete, the E_c value, of 25,000 MPa is adopted with the characteristic strength of 42 MPa. As suggested by CIRIA Report C580 (Gaba et al. 2003), the stiffnesses of the diaphragm wall, the $E_c I$ and the $E_c A_c$ values, where I and A_c are the moment of inertia and the sectional-area of concrete respectively, are reduced by 30 %, giving a value of 2,520 MN-m for the former and 21,000 MN/m for the latter to account for the influence of tremieing and degradation of concrete during excavation.

Table 1: Soil parameters adopted in numerical analysis

Soil type		Unit weight kN/m^3	Effective shear strength c' kPa	Effective shear strength ϕ' degree	Average N value	Average q_c MPa	Drained Young's modulus E', MPa	Poisson's ratio ν'
Sand fill	Upper	19	8	38	-	10 ~ 18	18	0.30
	Lower					28 ~ 32	55	
Rock fill		20	0	48	75	-	82	0.30
Alluvium		19	2	38	-	18	35	0.30
CDG	Upper	19	5	39	20	-	24	0.35
	Lower				50	-	66	
CDG/HDG		19	5	42	120	-	160	0.35
HDG		20	5	43	240	-	360	0.35
SDG		22	10	50	-	-	1,000	0.35

Table 2: Stiffness of struts adopted in numerical analysis

Strut level	Level mPD	Member	Sectional-area A_s cm^2	Spacing of struts, m	Strut stiffness $E_s A_s$ MN/m
1	-1.2	2 x UB 610 x 305 x 238 kg/m	607.6	5.5	2224
2	-7.5	2 x UB 914 x 305 x 289 kg/m	737.6	5.5	2700
3	-10.0	2 x UB 914 x 305 x 289 kg/m	737.6	5.5	2700

Table 3: Stiffness values for the underpass adopted in numerical analysis

Stiffness	Outer piles @ 6.7 m c/c	Inner piles @ 5.4 m c/c	Outer wall 1.0 m thick	Central wall & roof 0.8 m thick	Base slab 1.8 m thick
$E_c A_c$ MN/m	6,594	8,181	25,000	20,000	45,000
$E_c I$ MN-m^2/m	927	1,150	2,083	1,067	12,150

The stiffness values for the struts and for the underpass are summarized in Table 2 and Table 3. The struts are simulated by node-to-node anchors and a Young's modulus for steel, the E_s value, of 205,000 MPa is adopted. The underpass located on the south side of excavation is supported on reinforced concrete bored piles of 1.5 m in diameter. An existing seawall founding on a rock fill mound is located further south to the underpass. The north and the south seawalls are reinforced concrete and masonry structures, which are modeled as non-porous materials with the shear strengths of 5 MPa and 10 MPa respectively and with the Young's moduli of 2,900 MPa and 5,000 MPa respectively.

As reported by Wong (2012), prior to excavation the average piezometric levels in fill and in HDG were located at 1.0 mPD and 1.4 mPD respectively. Pumping inside the trench was conducted to lower the groundwater level to 1 m below the excavation level at each stage of construction. When the final excavation level was reached, the piezometric levels outside the diaphragm wall in fill and in HDG were drawndown to -0.5 mPD and 0 mPD respectively. To simplify modeling, a piezometric level of 1.2 mPD outside the wall in various strata has been adopted in the analysis. Since CDG is considered as a relatively impermeable stratum, the piezometric levels in the HDG and SDG strata inside the trench remaining at 1.2 mPD throughout excavation are adopted. The piezometric pressures in CDG and CDG/HDG inside the trench are interpolated between those in the alluvium and in the HDG strata. Figure 5 depicts the water pressures adopted in the analysis.

The wall deflections obtained from PLAXIS analyses are presented in Figure 6. The computed maximum deflection for the south and the north wall are 31.6 mm and 60.9 mm respectively in Stage 4, which closely match with the deflections of 28.8 mm and 61.2 mm observed from inclinometers no. S107 and N111 respectively. On the other hand, the analysis over-predicts the deflections for the north wall in the early stages,

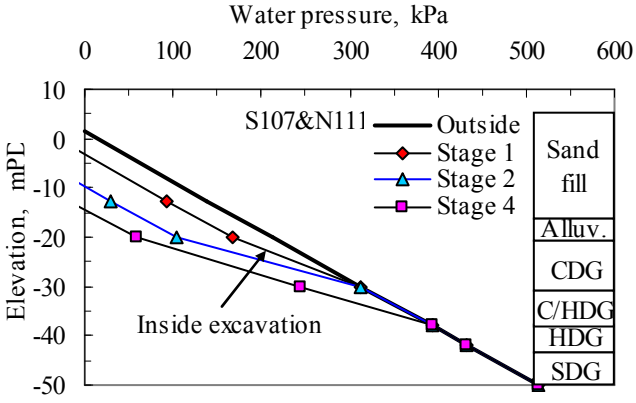

Figure 5: Water pressures adopted in the analysis

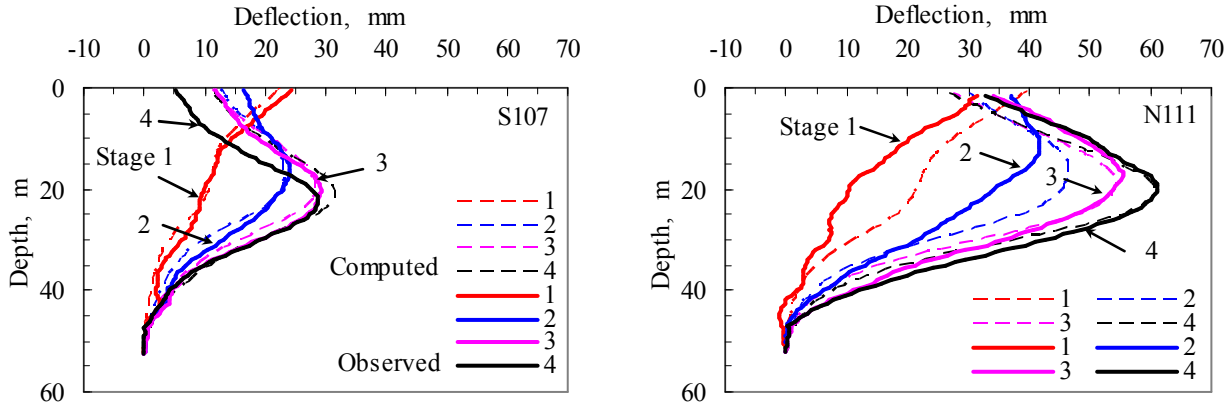

Figure 6: Comparison of computed results with observed wall deflections

for instance N111, by 9.4 mm. In Stage 1 the observed and the computed maximum deflection for the north wall are 31.6 mm and 40.0 mm respectively. The discrepancy between the observed and the computed profiles in the early stages is attributed to adoption of the constant drained Young's modulus, E', for soil in the Mohr-Coulomb model, while in reality, the E' value is stress and strain dependent.

4 CORRELATION OF YOUNG'S MODULI WITH IN-SITU TESTS

4.1 Fill and alluvium

The back-calculated E' values for sand fill and alluvium are related to the q_c values. The E'/q_c ratios depend on the axial strains, relative densities (D_r) and the over-consolidation ratios of sand. As summarized in Table 4, the E'/q_c ratios range from 1.3 to 1.9. The calculated principal strains, ε_1, outside the north wall in sand fill and in alluvium, have the maximum values varying from 0.0017 to 0.0046. It is noted that the sand fill is densified to the D_r values of 85% or larger.

Jamiolkowski et al. (1988) presented the E'/q_c ratios of Ticinio and Hokksund sand. The E' values were obtained from the triaxial CK_oD compression tests performed on pluvially deposited specimens. The q_c values of sand are determined from calibration chamber tests. Being moderately crushable silica sand, the mean grain size for Ticinio and Hokksund sand are 0.53 mm and 0.39 mm respectively, with the corresponding uniformity coefficient of 1.6 and 2.0. As summarized in Table 4, for normally consolidated Ticino sand with the D_r value of 85 % and at the average axial strains, ε_a, varying from 0.00113 to 0.00694, the E'/q_c ratios range from 2.9 to 1.4. For Hokksund sand with the D_r value of 90 % and at the average ε_a values varying from 0.00152 to

Table 4: Drained Young's moduli versus cone resistances in deposited sand

Jamiolkowski et al. (1988)				This study			
Sand type	D_r value %	Average axial strain, ε_a	E'/q_c	Sand type	D_r value %	Maximum principal strain outside excavation, ε_1	E'/q_c
Ticino	85	0.00113	2.9	Upper sand	85	0.0046	1.3
		0.00232	2.3				
		0.00694	1.4	Lower sand	85	0.0017	1.8
Hokksund	90	0.00152	1.5				
		0.00316	1.1	Alluvium	-	0.0022	1.9
		0.00939	0.7				

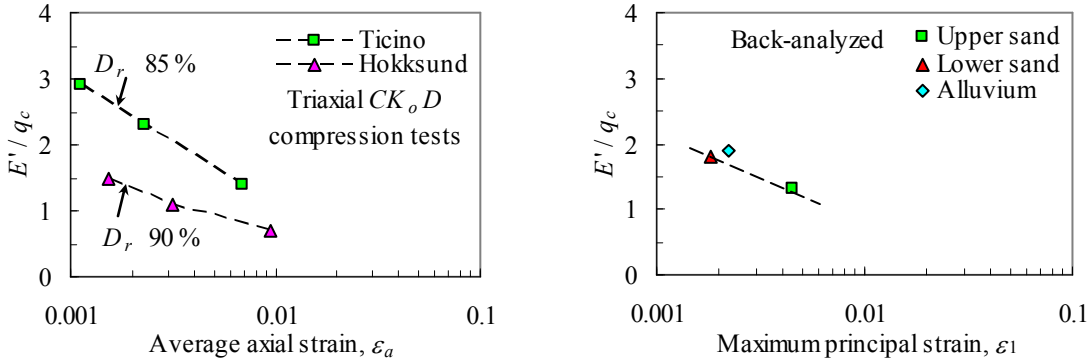

Figure 7: Drained Young's moduli for various types of sand

0.00939, the E'/q_c ratios range from 1.5 to 0.7. Figure 7 shows that the E'/q_c ratios for Ticino and Hokksund sand decrease with increasing axial strains and depend on the D_r values.

Due to finer in grain size and relatively more uniform in gradation, the E'/q_c ratios for Ticino and Hokkund sand are not readily comparable to those of sand fill for this case of interest. However, as presented in Figure 7, the E'/q_c ratios for the upper and the lower sand fill have the trend of degradation of moduli with strains. At the maximum principal strains outside the excavation ranging from 0.0015 to 0.005, the back-analyzed E'/q_c ratios for sand fill vary from 1.3 to 1.8. While the alluvium is considered as age deposits, the limited data obtained from this study would suggest the E'/q_c ratio of 1.9.

4.2 Saprolites

The E' values for saprolites obtained from this case of interest are compared with those reported in literature. As summarized in Table 5, the E' values for CDG and CDG/HDG back-analyzed from this study range from 24 MPa to 160 MPa. These back-calculated E' values are consistent with those reported by Yiu & Ko (2010) and Yau & Sum (2010), who have adopted the E' values ranging from 18 MPa to 200 MPa in the PLAXIS analyses. It is noted that the wall for ELS Case 2 was embedded in completely decomposed volcanic (CDV).

The back-analyzed E' values for saprolites are related to the N values. As presented in Table 5, the E'/N ratios (in MPa) for CDG, CDG/HDG and HDG are 1.2, 1.3 and 1.5 respectively. Variation in E'/N ratios with various saprolites in this case of interest is likely due to different strains mobilized in field. The calculated maximum shear strains immediately next to the inner side of the north wall in CDG, CDG/HDG and HDG in Stage 4 excavation are 0.009, 0.005 and 0.0008 respectively. Due to degradation of moduli with strains, the secant moduli decrease with increasing strains.

The E'/N ratios for CDG and CDG/HDG ranging from 1.2 to 1.3 (in MPa) established from this case of

Table 5: Drained Young's moduli of saprolites obtained from various case histories

Case history	Saprolite	E'	E'/N	Depth of	Maximum deflection, mm		Remarks
		MPa	MPa	excavation, m	Observed	Computed	
Nathan Road	CDG~C/HDG	18~200	1.5	17.5	22	27	Yau &
West Kowloon	CDG~C/HDG	30~200	1.5~2.0	18.6	53	68	Sum
	CDG/HDG	200	2.0				(2010)
ELS Case 1	CDG	22.5~67.5	-	8.0	-	41.6	Yiu & Ko
ELS Case 2	CDV	18~56	-	15.5	-	70.8	(2010)
This study	CDG	24~66	1.2	18.3	61.2	60.9	North wall
	CDG/HDG	160	1.3		28.8	31.6	South wall
	HDG	360	1.5				

interest are 70 % to 80 % of those reported by Yau & Sum (2010), who have adopted the E'/N ratios ranging from 1.5 to 2.0 (in MPa) in the analyses. The discrepancy may be caused by different assumptions on other parameters, for instance, groundwater pressures. More case histories on excavation shall be collected for studies in the future to verify the relationships between the E' and the N values for saprolites.

5 CONCLUSIONS

Numerical analyses have been conducted on an excavation supported with diaphragm walls. The result of the study has observed the following:

(1) The drained Young's moduli for sand fill are related to the cone tip resistances. For the sand fill at this site with the relative density of 85 % or larger at the maximum principal strains ranging from 0.0015 to 0.005, the E'/q_c ratios range from 1.3 to 1.8.

(2) An empirical relation between the drained Young's moduli and the N values are observed for granitic saprolite. For this case of interest, the E'/N ratios vary from 1.2 to 1.5 (in MPa).

Since elastic moduli are highly stress and strain dependent and affected by the state of soil, the empirical relationships with in-situ test results obtained from this study may be a useful reference for numerical analysis on excavation using the Mohr-Coulomb soil model with similar relative densities under similar magnitude of wall deflections.

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Re-use of Existing Basement and Foundation in Redevelopment Projects in Urban Districts

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ABSTRACT

Seeking new land for housing supply and development has always been an important and challenging issue in Hong Kong. Redevelopment of existing buildings provides an effective option to ease the demands for land supply. However, the structural elements of existing buildings, such as their basement and foundations, can become site constraints to redevelopment projects. Re-using them can be a cost-effective way to turn potential site constraints into useful purposes for the redevelopment. This paper presents one project, namely York House (former Landmark East, east wing of Landmark Atrium) in Central, one of the Hong Kong's busiest business districts, where the major geotechnical challenges for the redevelopment include difficulties encountered during site investigation, verification and assessment of the existing conditions of the foundations, design challenges in incorporating the new works into the existing foundations with the modification of basement structures, difficulties in constructing the new foundation system in existing basement structures and the concerns and mitigation measures during demolition of the superstructure. The experiences presented in this paper can be useful reference to developers and engineers in their process of developing cost-effective solutions for future redevelopment projects.

1 INTRODUCTION

Redevelopment in urban area is always a challenge to the engineers as the existing structures, i.e. foundation, superstructure and basement etc, may become the site constraints to the redevelopment. The engineers need to consider the impact to the surroundings due to the development, the technical difficulties, the time and cost to overcome the constraints due to the presence of existing structures. An effective way to overcome some of these issues is the re-use of existing structures as part of the redevelopment. Redevelopment of Landmark East, former east wing of Landmark Atrium (current York House in Central) is one of the examples.

York House (hereinafter called YH) was a 23 levels commercial building with 6 levels of retail podium and 3 levels of basement redeveloped from the original 7 levels building, namely Landmark East, east wing of Landmark Atrium (hereinafter called "LME"). It was located at Central and built during the 80's. The site was confined by existing buildings and busy traffic (Figure.1). The podium was linked to Landmark retail area on the west side and to another premise by a footbridge on the east side. A car ramp which was situated inside the existing basement traffic through the site to the adjoining building had to be maintained throughout the construction, This became one of the site constraints of the redevelopment. An innovative basement and foundation design, by combining the new and old structures, was adopted so that the redevelopment was completed in a environmental friendly, safely and cost-effectively manner. The whole process of development is described in this paper.

1.1 Site investigation

Systematic approaches were taken to review the existing conditions of the structures. As part of the desk top study, all existing site investigation records, as-built records were examined. A subsequent site inspection and site investigation were carried out to review if any condition had been changed or deviated from what were revealed in the desk study process. Existing caisson foundations, with diameters ranging from 1.8m to 2.2m and founded on bedrock were used to support the building including the basement structure (Figure.2).

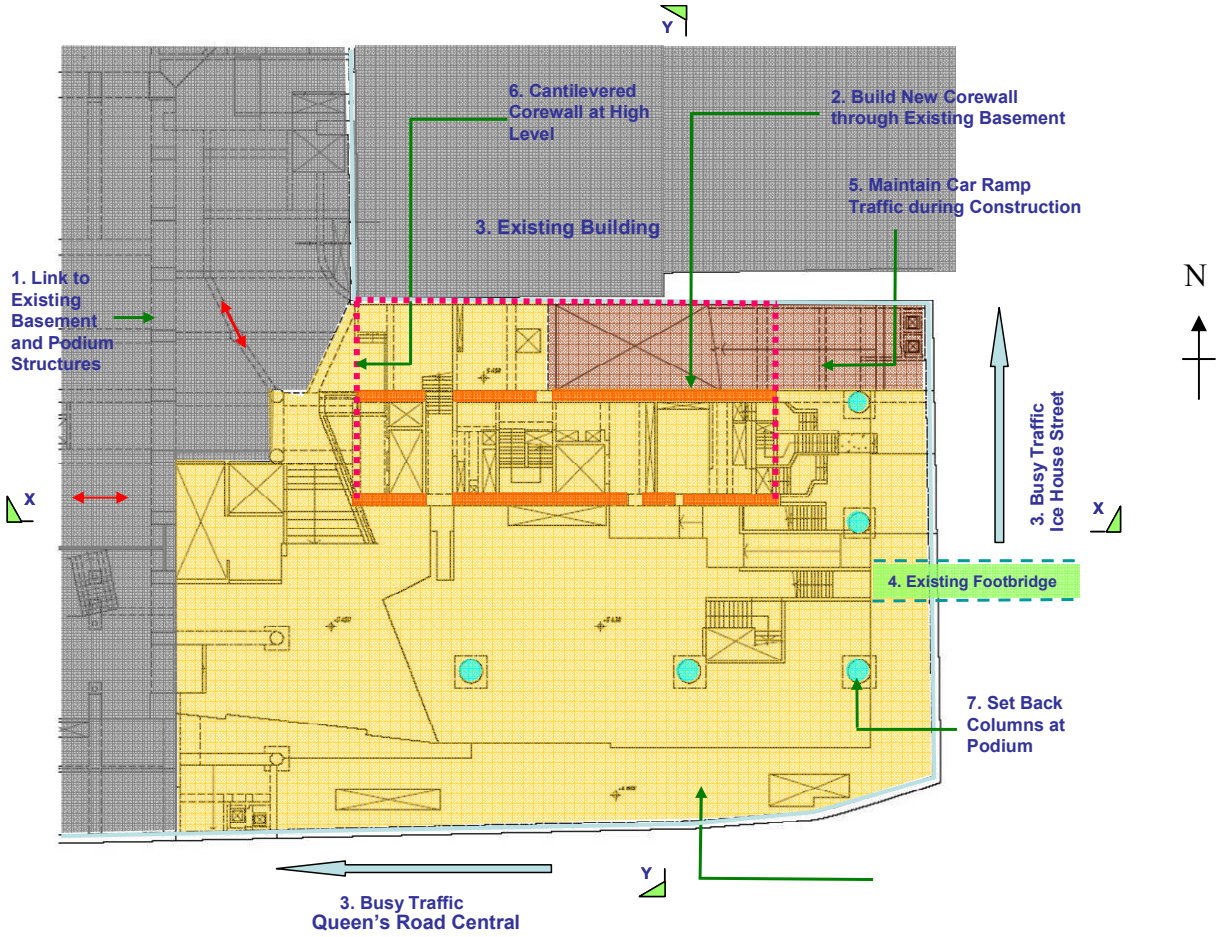

Figure 1: Layout of Proposed YH

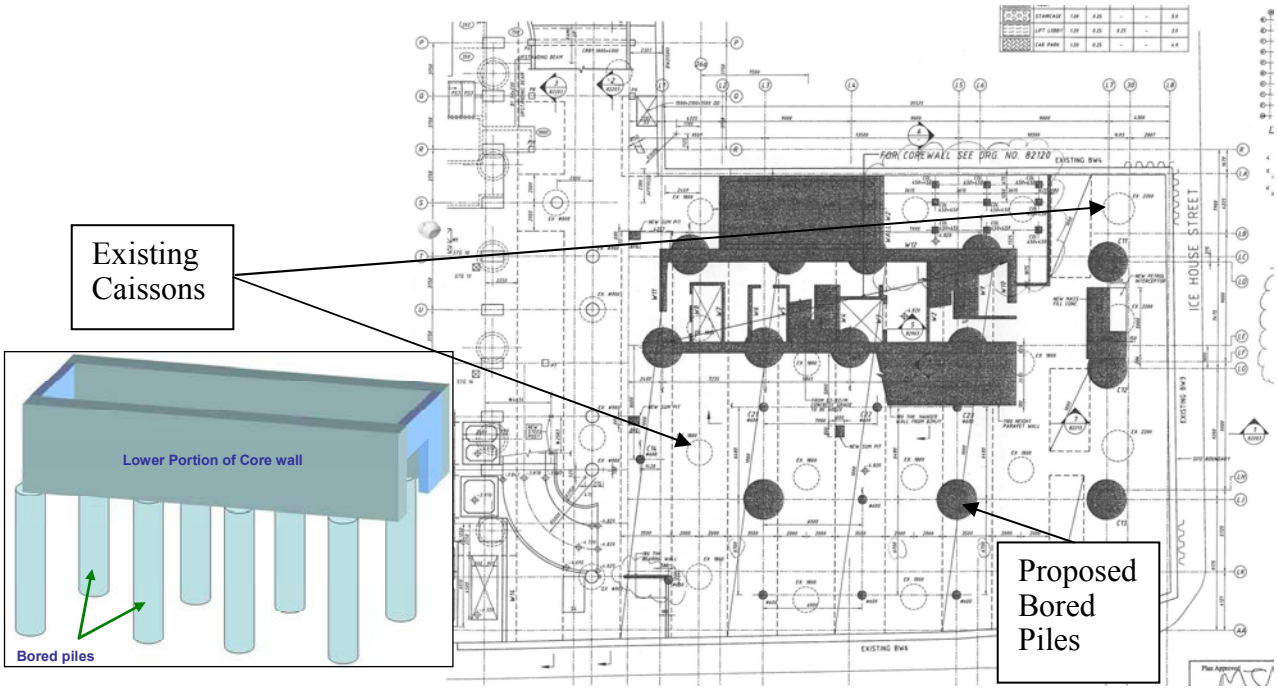

Figure 2: Layout of Existing Caissons and Proposed Bored Piles

In order to verify the conditions of foundation of the LME, full coring into existing caissons down into the founding bedrock with minimum depth of 5m or 3 times the diameters of the caissons, whichever was deeper, was carried out. The purpose of the full coring served three purposes: to verify the concrete condition, the founding level and founding conditions. Assessment was carried out at each full coring. The results indicated that the concrete strength and founding levels were consistent with the as-built records. There were few local weathered materials within the bedrock underneath the caissons. Bearing capacity was reviewed and the apparent stiffness of the pile against the loading was assessed, taking into account the interface conditions, the rock conditions and the load spread below the caisson at different stratums of rock. That information was then taken into consideration in the planning of the layout, design of structural element and the new foundation and formed the principle factor affecting the load path in the structural design.

Besides full coring, additional boreholes were also sunk down to further verify the soil conditions and to check if the condition of the groundwater had been changed. The challenges of sinking boreholes were that the basement car park was required to be maintained in operation throughout the site investigation period and the headroom of the car park was very limited. The lowest headroom was only about 1.7m which was carried out underneath a staircase, therefore, shorter casings for the boreholes were used to reduce the length of each core run to accommodate this site constraint. The basement was about 10m deep below ground, the average groundwater table was anticipated at about 2m below ground i.e. water head underneath the basement slab could be 9 m. Drilling through existing basement slab might cause significant water inflow together with lost of fine material which could cause significant settlement outside the basement. Therefore, gate valve head with removable gate valve was installed at the each drillhole location of basement concrete slab. Double guard barrier surrounding each drillhole location was provided to avoid any flooding prior to drilling and it was also used to collect the water coming out during drilling works, if any. The collected water would then be pumped to de-silting tank before discharging to the public discharge system. Additional boreholes were also sunk down outside the basement to reveal the soil conditions around the site. Piezometers and standpipes were also installed to investigate the ground water conditions. The site investigation revealed that the basement structure was surrounded by fill material and the water table was confirmed at about 2m below existing ground surface. The highest design groundwater table was then set to 1m below ground level. The findings were consistent with the findings from desk study.

1.2 Re-use of existing foundation and design of new foundation

Since YH was a 23-storey building, which was much higher than the original LME, new foundation was required. In view of the nature and rigidity of the existing caissons, bored pile foundation was proposed as the foundation together with the existing caissons to support the proposed YH and existing basement structure which took into account the findings of site investigation. Innovative pile-cap-free structural arrangement was adopted. Corewalls of YH was designed as a vertical diaphragm to transfer the tower loads directly to the bored piles so that the need for pile caps were eliminated and the new building above existing basement could be solely supported by the new foundation. The existing foundation was only required to support the existing basement structure i.e. there was no significant increase in loading for the existing foundation (Figure. 3). Tube-In-Tube installation method (Figure. 4) was established to overcome the underground water pressure and eliminate dewatering during installation of the new bored piles through the existing basement. No excavation and dewatering were required and the existing basement could be maintained. A larger casing filled with water with a head equivalent to water head underneath basement slab was constructed, sealed and fixed on top of the existing slab. The water head inside external casing was maintained at constant level throughout the construction of bored pile. Then inner casing, which was used to maintain the ground stability during construction of bored pile underneath the basement slab, was inserted. Breaking through of the basement slab was then carried out and the concrete fragment and the steel reinforcement of basement slab were then removed. Subsequent processes, with the temporary lateral support of inner steel casing, were the same as typical bored pile installation. Under this method, the water head was maintained as the same as surroundings, as there was no significant water flow into the excavation shaft of the bored pile throughout the construction, hence dewatering works was not required. The lateral load transfer paths of the existing basement floors were reassessed to allow for the temporary floor openings that were required to facilitate the bored piles installation.

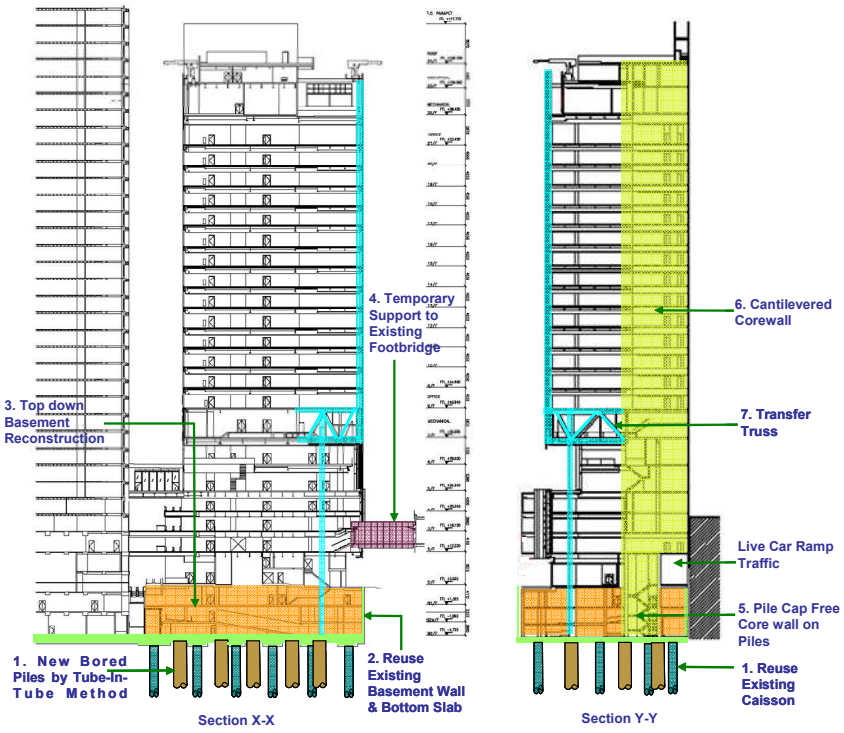

Figure 3: Sections of Proposed YH (section mark refer to Figure.1)

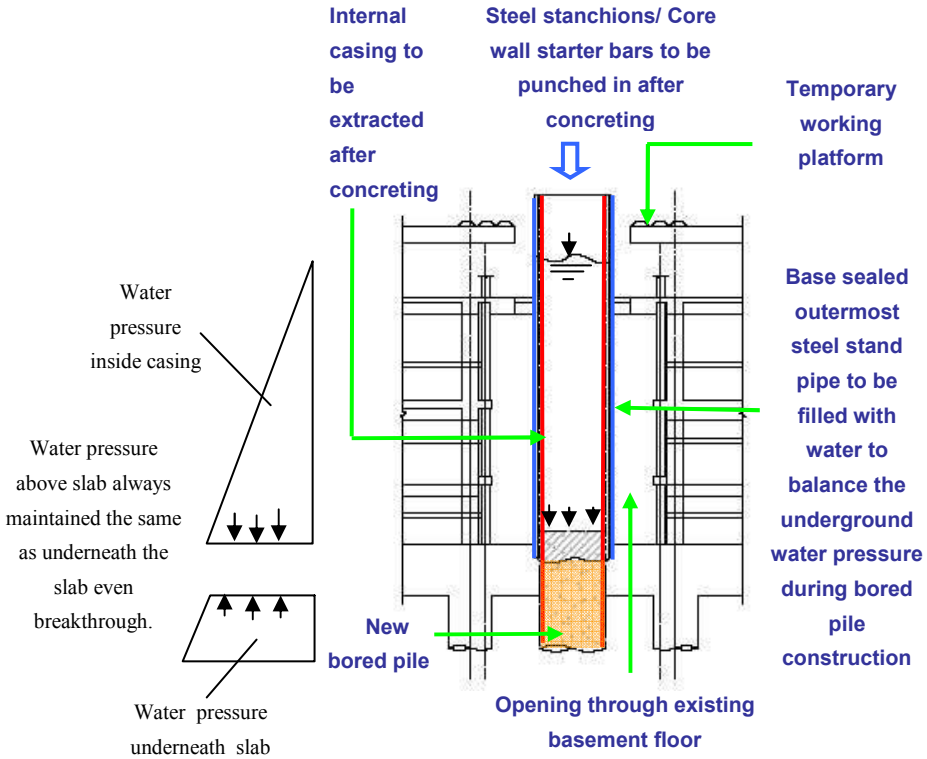

Figure 4: Tube-in-Tube bored pile installation method

Plate 1: Carry out bored construction next to the live car ramp & footbridge support

1.3 Demolition of existing superstructure above basement

There was about 9m water pressure underneath existing basement slab. The demolition of superstructure reduced the weight on the basement structure, and as a result uplifting was a major concern. Since LME was a part of the Landmark Atrium and the original pile cap extent was larger than the LME, hence the uplifting pressure could be resisted by the weight of remaining part of the Landmark Atrium. In addition, existing caissons also served as uplifting piles to hold down the basement structure. The uplifting capacity of caissons was estimated by considering the lesser of frictional force between the caissons, and the weight of soil being mobilized taking into account the overlapping of pull out cones. The checking confirmed that the superstructure above ground could be demolished without causing any uplifting stability problem. Otherwise, temporary measures, such as providing temporary counter balance weight, temporarily dewatering, additional uplifting piles, would be required. Nevertheless, it was one of the key concerns in any demolition of superstructure with basement. Review of construction sequence of existing building could also help to find out the most effective way to resolve the uplifting issue.

1.4 Breakthrough of pile cap to form the new lift shaft

New lift shafts were required in the redevelopment which needed to break through the existing basement slab locally to form the shafts with soffit level about 2m lower than the existing basement slab. Innovative approach by using grouting to eliminate any excavation and lateral support was adopted. The objectives of grouting were to strengthen the soil against water ingress, to maintain the stability of the pit against hydraulic failure and to maintain the overall and lateral stability even without any lateral support. The opening of each shaft was about 3m x 3m. Grouting composed of Bentonite:Cement:Water was carried out at first for sleeve grouting purpose and then followed up with soil improvement grouting process. The latter was sub-divided into two stages: (i) Bentonite/Cement grout; (ii) Quick set chemical grout-LW. Tube-a-Manchette method was adopted in the grouting process. Same as the borehole drilling through the basement slab, precautionary measures and control valve were provided prior to drilling through the slabs. The grouting, i.e. grout strength and the extent, was designed to resist three key potential failure mechanisms: hydraulic failures, collapse of lateral soil and uplifting failure of the grouted soil mass with huge water pressure underneath it. Trial mixing and in-situ trial grouting were carried out to verify if the performance of the grouting achieved the design requirements prior to working grouting carried out. The actual performance of the grouting achieved the design requirement and the works were completed satisfactorily.

1.5 Excavation and Lateral Support (ELS)

The original design assumptions, design approaches, construction sequence for the construction of LME were reviewed in desk study process. They were taken into account in the new ELS design so that the impact to the basement wall due to redevelopment could be assessed i.e. lock in loading etc.

Top-down basement reconstruction (Figure. 5) was adopted to make use of existing rigid basement floors and walls as earth retaining structures. Since no excavation around the basement and no dewatering works within basement were required, as a result, this minimized the ground movements and no significantly change to the conditions of soil and ground water both inside and outside the basement wall occurred. This also

allowed early commencement of the superstructure works and thus reduced the cost construction and programme. Corewalls was built by bottom up method within the existing basement to improve the overall integrity and stability during construction of the high level cantilevered core. The sequence and shoring arrangement were considered comprehensively to cater for the stability and load transfer of the top-down plus bottom-up basement reconstruction in associate with the site constraints.

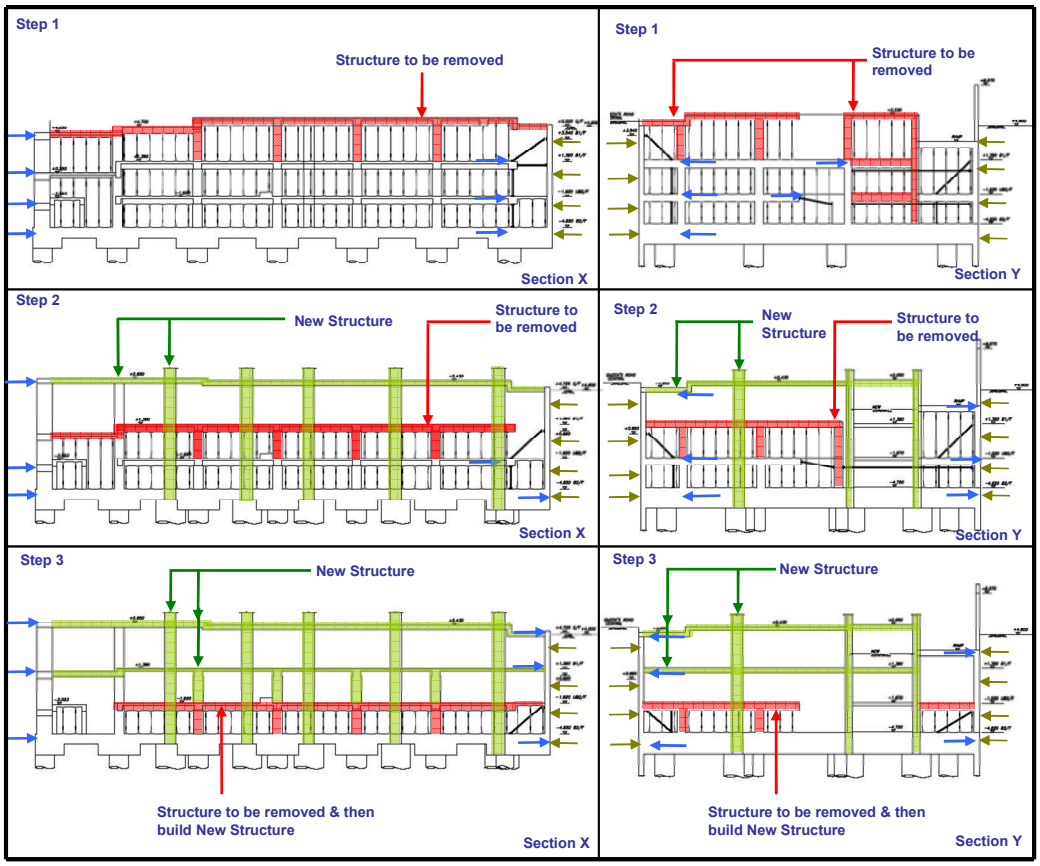

Figure 5: Top Down Construction

3 CONCLUSIONS

Re-use of structural elements of existing building can be a cost-effective, time saving and environmental friendly solution in urban redevelopment. Comprehensive studies about the as-built details of the existing structures together with site constraints and history are required prior to determine how to re-use the existing structural elements. Rigorous approaches during design and construction are required to merge the new structure with the existing building elements. Redevelopment of Landmark East, east wing of Landmark Atrium which combined the new building with existing structural elements proved successful. The keys to success of this project were: comprehensive desk study, systematic investigation approaches, innovative design and construction ideas such as Tube-in-Tube bored pile installation method, improvement to soil underneath base slab by TAM grouting to eliminate the ELS works etc., appropriate construction sequences to maintain a stable and safe environment throughout the demolition of existing structure and construction of new structure.

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The Use of Glass Fibre Reinforced Polymer Bars as Soil Nails to Permit Future Housing Development, Hong Kong Special Administrative Region

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ABSTRACT

The Mass Transit Railway Corporation Ltd (MTRCL) Kwun Tong Line Extension (KTE) project is a 2.5 km long underground extension of the existing Kwun Tong Line (KTL) running from the current terminus at Yau Ma Tei (YMT) station to a new station terminus in the Whampoa District. An intermediate station at Ho Man Tin (HOM) will serve as an interchange station with the future Shatin to Central Link (SCL).

The proposed HOM station is being constructed in an open cut excavation with a maximum depth of excavation in soil / weathered rock of approximately 48m, and 57m in competent rock at sections where the KTE caverns span across the station. Excavation of the proposed HOM station box requires formation of a number of temporary soil cuts, with a maximum slope angle of 75°. As there is future development proposed to the North of the station box, and given that piling works will be required for this development, glass fibre reinforced polymer (GFRP) bars were proposed as a substitute for high yield steel deformed bars in the soil nailing system.

1 INTRODUCTION

1.1 Project Background and Description

The KTE is an extension of the existing MTR Kwun Tong Line from Yau Ma Tei Station to Whampoa, with two new stations at Ho Man Tin and Whampoa. Passengers can interchange at the proposed Ho Man Tin Station for the future Shatin to Central Link. It will provide convenient, fast and reliable means of public transport between Yau Ma Tei and Whampoa, and will enable residents in Ho Man Tin, Hung Hom and Whampoa area to have direct access to MTR service, saving time for interchange from road transport to the railway network. The alignment of Kwun Tong Line Extension will extend from the existing overrun tunnel of Yau Ma Tei Station, run along Gascoigne Road cross Wylie Road and reach Ho Man Tin Station at the site of the ex-Valley Road Estate. It will then run through Chatham Road North via Wuhu Street and Tak Man Street, and extend to Whampoa Station at Tak On Street.

1.2 Site Location and Topography

Ho Man Tin Station is located at the site of former Valley Road Estate. This platform was created by backfilling the existing valley following the contours in a SE direction. The station comprises a large box that will contain the KTE station cavern (KTE platforms) and the SCL station box, which is located above the KTE station. The station is to be constructed beneath the fill platform occupied by the Hong Kong Housing Authority's Valley Road Estate and is bounded by Fat Kwong Street (to the North), Chung Hau Street (to the

West) and Yan Fung Street (to the East). The site is also allocated for a MTRCL property development including two level semi-basement parking and 10 numbers of 25-storey residential towers.

The dominating feature for the northern HOM station box is a NW-SE trending fault zone, which has resulted in the formation of a deep valley with extensive weathering and shearing of the underlying rock mass. It is noted that in the deepest part of the valley weathered rock extends to an approximate depth of -35mPD. This zone is at least 50m wide and extends through Chatham Road to the Wuhu Street playground area to the SE. The valley has previously been backfilled for the construction of the Valley Estate and there is evidence of small amounts of alluvium and colluvium along the valley floor.

2 DESIGN BASIS

Soldier pile walls with preloaded struts and/or soil nails/ground anchors were originally proposed in the conforming design. At the northern part of the site, soldier pile walls were envisaged to be installed concurrently with the bulk excavation works. Temporary horizontal struts were also envisaged to be installed by stages during the course of excavation to facilitate the excavation down to formation level: Figure 1(left) depicts a section view of the conforming scheme across the northern part.

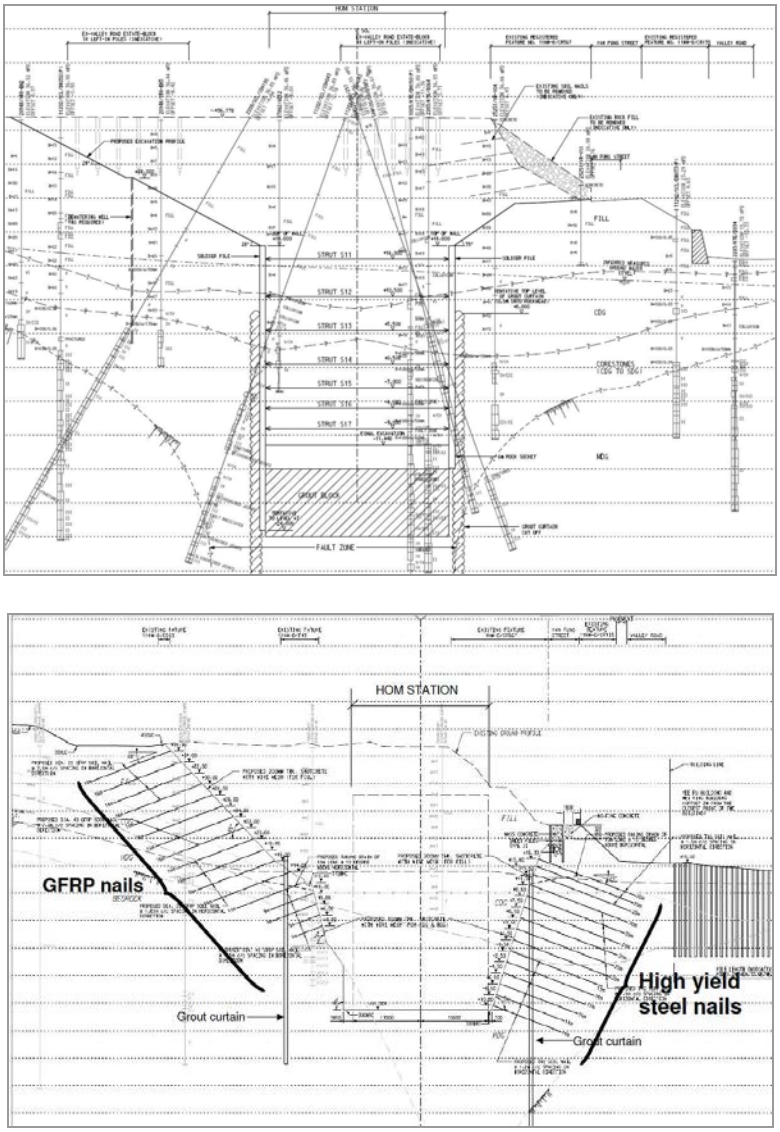

Figure 1: Section view of the conforming ELS scheme (above) versus the Contractor's alternative open excavation scheme (below) in the northern part of the site

In view of the geographical limitation, the Contractor adopted an alternatvie scheme where the upper soft ground near the surface would be excavated by an open cut method. In areas where space constraints severely limit slope cutting, pipe pile walls were constructed i.e. along Chung Hau Street and Fat Kwong Street. The station box's excavation outline is formed by phased cutting of slopes in fill, completely decomposed granite (CDG) and highly decomposed granite (HDG). The cutting and strengthening works are being carried out in stages. The steeper slopes are strengthened with soil nails as the excavation proceeds down to the projected final excavation level at about -11.3mPD. Steeper slopes ranging from 45° to 75° are formed and soil nails will be used to reinforce these slopes and achieve acceptable factors of safety (Figure 1 right).

In the implementation of slope strengthening works, conventional high yield deformed steel bars and glass fibre reinforced polymers (GFRP) bars were used as soil nails. The slopes that require strengthening are in fill with a maximum inclination of 75° with fill thickness varying between 10m to 20m, and in weathered rock (CDG/HDG) varying from 45° to 75° with varying heights between 10m to 20m. All proposed cut slopes in fill and saprolites are shotcreted. The highest soil cut slope is located in the northern portion of the station box where deep rock head is encountered due to the presence of the inferred fault zone. The existing road elevation is +36.9mPD and the site formation level for the northern part is -11.3mPD, which results in an overall total excavation height of about 48m.

Although GFRP (Glass Fibre Reinforced Polymer) tie-back nails were proposed in the original scheme to be installed for the soldier pile wall at the South end (interfacing area with SCL approaching tunnels), their use was not envisaged to be as extensive as currently implemented in the open excavation scheme. In effect, there is also future housing and commercial development proposed to the North-West and North-East of the station box as indicated in Figure 2.

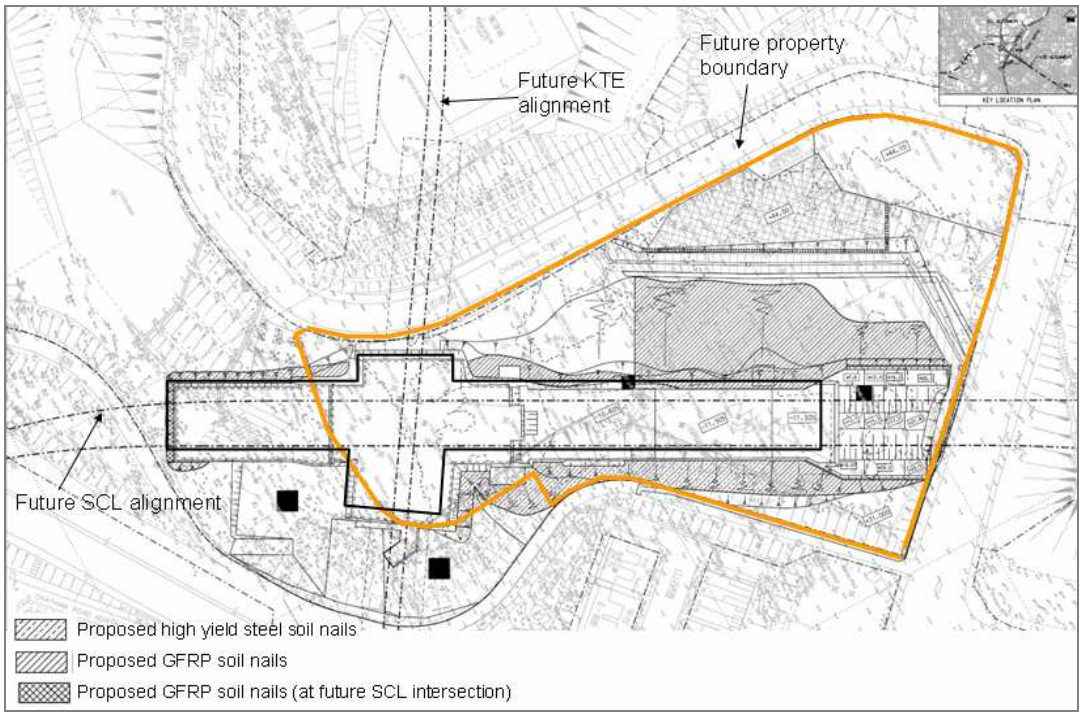

Figure 2 : Schematic layout of areas of soil nails for future development

Given the SCL interface shoring works would need to pass through the soil nails to the South and piling works may be required for the future development to the North, the use of GFRP bars is warranted as a substitute for conventional high yield deformed steel bars in the soil nail system so that they can be broken out later: GFRP material can be cut with working tools like saws, piling/drilling equipment and TBM tools. This avoids damage to cutter heads and does not delay work progress as piling or cutting through GFRP bars is unproblematic as the fibre bars can be split into small pieces.

2.1 Approval Process

In accordance with the Instrument of Exemption (IoE) issued under Section 54(2) of the Mass Transit Railway Ordinance (Chapter 556) for the KTE project, design submissions are to be processed under the consultation procedure with Buildings Department.

However, given the proposed use of GFRP bars does not comply with the requirements as stipulated under Building (Construction) Regulations 54, to the effect that "reinforcement for concrete shall be hot rolled steel bars, cold reduced steel wire or steel fabric of suitable composition, manufacture, and chemical and physical properties", a submission of a dossier to BD's Technical Section panel was required, presenting case notes of the recommended use of GFRP bars as the alternative reinforcement material of the temporary soil nails.

The use of crimping connector and connection stud was also subject to examination in the technical dossier since soil nails up to 24m long are proposed, hence a proprietary connector is necessary to splice GFRP bars at the ends.

2.2 Technical Specifications

2.2.1 Composition of the soil nail system

The soil cut slopes have a maximum slope angle of 75°. GFRP soil nail bars of 25mm and 40mm diameter are installed at a spacing of approximately 1m to 2m centre-to-centre. The soil nails are envisaged to improve the overall stability of cut slopes through the mobilization of tension in the soil nails. The tensile forces are developed in the soil nails primarily through the frictional interaction between the soil nails and the ground. The resistance against pullout failure of the soil nails is provided by the soil nail being embedded into the ground.

The GFRP bars are composite materials made of glass fibers embedded in a polymeric resin matrix. The physical and mechanical properties of GFRP bars are listed in Table 1, and those of steel are also presented for ease of comparison.

Table 1: Physical and Mechanical Properties of GFRP Bars and High Yield Steel Bars

Properties	For GFRP bar		For High Yield Steel bar
Nominal Bar Diameter	25mm	40mm	25mm/40mm
Specific Gravity	2.0	2.0	7.82
Modulus of Elasticity	40,800 N/mm^2	40,800 N/mm^2	210,000 N/mm^2
Manufacturer's Guaranteed/Ultimate Tensile Strength	550 N/mm^2	460 N/mm^2	460 N/mm^2
Manufacturer's Guaranteed Transverse Shear Strength	137 N/mm^2	115 N/mm^2	-
Guaranteed/Ultimate Tensile Strain	0.0135	0.0113	0.10

A clear advantage of the GFRP bars is their light weight: a GFRP bar weighs only a fourth of its steel counterpart, having the same dimensions. Other advantages of GFRP bars are their high tensile strength, high fatigue endurance, high corrosion resistance (not dependent on a coating); they have low thermal and electric conductivity, and formation of microcracks is minimal at the GFRP/grout interface during cyclic temperature variations (Table 1.1 of ACI 440.1R-06 reported by ACI Committee 440).

2.2.2 Design considerations

There is no specific code or standard addressing the design of soil nails using GFRP bars, though Geoguide 7 Section 5.12.4 allows for the design of soil nails using alternative reinforcement materials. In the absence of a recognized guideline, the design of GFRP soil nails was carried out in accordance with Geoguide 7 Equation 5.2 to determine the allowable pullout resistance provided by the bonded length of the GFRP soil nail.

The design bond strength between a GFRP bar and grout followed the American guide ACI 440.1R-06 "Guide for the Design and Construction of Concrete Reinforced with GFRP Bars" published by ACI Committee 440. This ACI guide is based on the knowledge gained from worldwide experimental research, analytical works and field applications of GFRP bars. The bond strength between a GFRP bar and grout can be estimated based on Equation 11-3 from ACI 440.1R-06

$$f_{fe} = \frac{0.083\sqrt{f_c'}}{\alpha}\left(13.6\frac{\ell_e}{d_b} + \frac{C}{d_b}\frac{\ell_e}{d_b} + 340\right) \quad (1)$$

where f_{fe} = bar stress that can be developed for embedment length ℓ_e, f_c' = specified compressive strength of concrete, α = bar location modification factor and C = the lesser of the cover to the center of the bar or one-half of the center-on-center spacing of the bars being developed.

Considering the GFRP soil nails would be adequately protected by grouting and their temporary use for a maximum period of 2 years, the environmental effects on GFRP bars are considered not significant in this case and therefore no environmental reduction factor is used on the guaranteed tensile and shear strength of the GFRP bars. However, due to the lack of ductility of fibre reinforced polymer materials, a factor of safety of 2.0 is adopted in deriving the allowable tensile and shear capacity of GFRP bars against tension and shear failure instead of 1.5, as recommended for high yield deformed reinforcement bars loaded against tension failure according to Table 5.6 of Geoguide 7.

2.2.3 GFRP Bar Crimping Connector and Connection Stud

GFRP bars are manufactured to the proper shape during fabrication and for use in soil nails and in-situ bending is not required. Where connectors are required for soil nails longer than 12m, they are factory supplied with the bars as they are crimped on to the bars rather than threaded to ensure the connections have at least the same tensile strength as the bars. The bars and crimping connectors act as a system, as shown in Plate 1 and Plate 2.

Plate 1: GFRP bars (40mm dia.) spliced with crimping connectors and connection stud

Plate 2: Connection stud M42×4.5-8g (Grade 6.8) measuring 120mm long for a 40mm dia. GFRP soil nail

The crimping connector is attached to the GFRP bar by mechanical means, i.e. a concentric force is applied onto the connector in sections to secure the connection. Each GFRP bar is manufactured and delivered to site with a steel crimping connector attached to either one end or both ends of the bar. The free end of the steel connector has internal threading. The connection between two GFRP bars is facilitated with an intermediate short threaded steel stud connecting the two steel connectors. The steel stud and the connectors are fixed by using torque wrench to ensure the steel stud fix in the connectors as the manufacturer's requirement. In

addition, a center mark by permanent marker is marked on the steel stud and the distance between the connectors is measured to control the position and embedment depth of the steel stud.

2.3 Testing Requirements

During the approval process, the GFRP bars were tested in a local HOKLAS accredited laboratory to ensure the bars' guaranteed ultimate tensile strength and shear strength exceed 550N/mm^2 and 137N/mm^2 for 25mm diameter bar and 460N/mm^2 and 115N/mm^2 respectively for 40mm diameter bar. Out of the 25 nos. samples of each bar diameter tested in ultimate strength, all achieved the guaranteed values and exceeded these, or average by 20%. Then the GFRP bars together with their crimping connector and connection stud underwent two other test setups to justify the respective material strength of each component of the spliced GFRP soil nails and to establish that the connections achieve "bar-break" failure, which requires failure occurring in the GFRP bars away from the connectors.

During construction, the GFRP bars and their connectors/connection studs are subjected to a stringent and comprehensive quality assurance and quality control programme before shipment and after delivery to site. For the workshop production of GFRP bars, the supplier must attest its system has acquired Standard ISO 9001 certification for the manufacture of GFRP bars from the certifying body *Bureau VERITAS limited*. Manufacturer's test certificates on the raw material, the GFRP bars, the crimping connectors and connection studs must be produced. The mechanical properties of all bars (cross-sectional properties, modulus of elasticity, tensile strength, guaranteed ultimate strain, shear strength, glass fiber content, dye wicking and Barcol hardness) are required to comply with the relevant ACI and ASTM test standards.

At least 5 nos. samples of GFRP bars from every batch delivery to site are selected for testing of tensile strength, ultimate strain, modulus of elasticity, shear strength and pullout strength upon delivery, according to standard test methods ACI 440.3R-04/B.2 and B.4. As for the crimping connectors and connection studs, tests are carried out on their cross sectional properties, ultimate tensile strength and elongation, where applicable to the relevant standards (GB/T 700 for steel grade Q235, GB3098.1 for grade 6.8) before delivery to site. In addition to the tests described above, at least 6 nos. or 2% whichever is the greater from every batch of the GFRP bars spliced with connector system are selected for testing of the ultimate tensile and elongation and verify the "bar-break" failure mode upon its delivery to site.

2.4 Instrumentation and Monitoring Scheme

An essential part of the construction method is the extensive use of instrumentation and monitoring. Approximately 5% of the installed soil nails are selected for monitoring of their movement and tensile forces of the soil nail system by setting AAA response levels based on design calculations. Surveying tools such as prism targets on soil nail heads and on the slope's shotcreted surface are being used. The motivation for this comprehensive instrumentation scheme is to adhere to an observational method to monitor and validate the performance of the soil nail system as the excavation progresses.

To date, a total of 35 soil nails (both high yield steel and GFRP) have been strain gauged and have surface prisms installed at the proximal ends. Twenty inclinometers are installed and numerous ground settlement markers cover the whole site, targeting the crest of the formed slopes. Utility monitoring points are used to monitor any nearby sensitive receivers. The instrumentation is used to assess the state of deformation of the soil cut slopes and the buildup of loads in the soil nails.

A detailed system of in situ testing and monitoring to validate the soil nail system is being implemented during construction. Test nails equivalent to at least 2% of the soil nails are installed to demonstrate that the design pullout capacity is achieved. More than 40 nos. of nails have been successfully tested, using a hydraulic jack which is attached on the crimping connector at the proximal end of the bar (Plate 3), applying the proof load to the test GFRP nail.

Plate 3: GFRP test soil nails subject to in situ pull-out test

3 CONCLUSION

The main advantage of using GFRP bars as soil nails to stabilize the proposed soil cut slopes is to allow for the future proposed Ho Man Tin station to be constructed in an open cut without restrictions or limitations. Since shoring works with interfacing MTR contract would need to pass through the soil nails and piling works may be required for the future development to the North of the site, the use of GFRP bars is a suitable alternative to conventional steel soil nails as they can be drilled through later.

In the long term, it may be worthwhile to examine a wider use of this material for temporary works since it allows construction beyond site boundaries, which gives more flexibility to excavations which are carried out in close proximity with other existing sites or buildings. Secondly, studies should be carried out for their use in permanent works under Hong Kong's landslip prevention and mitigation programme. The characteristic of high corrosion resistance is especially valuable for sites with aggressive ground conditions. Also, the cuttability and flexibility of the bars as well as their light weight are appealing for confined working spaces where the bars can be more easily installed, or where in constrained sites (for instance behind village cottages), the support of lifting equipment is not readily available. Where remote sites call for easy transportation and handling of materials, GFRP bars may also be a viable option.

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GIS System Applications for Ground Model and Geotechnical Data Management for Foundation and Building Projects

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ABSTRACT

The increased complexity of the built environment has promoted an increased use of digital databases and 3D ground modelling to aid the assessment of foundation designs. In Hong Kong there are good sets of geotechnical data that can be used to provide early as well as detailed indications of the ground and geotechnical conditions for a particular site. Within the urban areas there are numerous existing boreholes as well as known foundation information that can be used to characterize and build the geological and geotechnical model for the site. Project-specific geotechnical data and foundation data can be used to further develop initial models and can be stored and retrieved through GIS applications. These models can aid the current project design and can be stored to aid any future redevelopment or changes in the future.

With an increased emphasis on complete data models for projects and buildings this paper presents some of the latest examples of how GIS has been implemented on building projects in Hong Kong and elsewhere in Asia.

1 INTRODUCTION

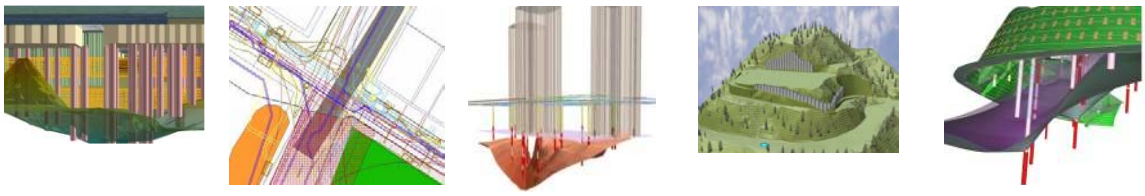
In Hong Kong there are various sources of geotechnical and ground related data that are readily available to the public as well as Consultants working on Government Projects. The Geotechnical Engineering Office (GEO) of Civil Engineering and Development Department (CEDD) Hong Kong has various sets of digital data such as borehole, landslide, slope and geological information. There are other sources of textual data as well as scanned data such as building and existing foundation records. The GEO has developed and released a GInfo gateway that allows the public access to over 200,000 drillhole records, slopes, planning, tunnels and various other geotechnical data through online search. In addition to the existing data sources there are many project-specific data acquired / analysed and generated for a particular project. The ground investigation will generate AGS borehole data as well as AGS laboratory data as long as your project specifies the digital submission requirements of the ground investigation data.

These records are becoming more frequently employed in more complex ways to analyse, visualize and display the ground and geological data of the project and the surrounding area. In particular obstructions can be shown and verified as well as analysed surfaces and layers displayed to show what interfaces will be encountered during excavation. On foundation and MTR projects it is common to provide videos showing 3D spatial elements of known data that have an impact on the development to allow better decisions to be made about risk allocation and to consider appropriate construction planning and risk avoidance measures. Indeed it is apparent that there is a growing need to capture, keep and share ground and foundation related information from a project to ensure that as-built information is accurately logged and saved for future generations.

The spatial geotechnical data for foundation and building projects usually relate to:

1. Existing obstructions such as old foundations, pipes, sewers, cables (electric and telecoms), gas, etc This data can be sourced from the respective BD databases as well as utility companies. This data should be gathered and input to the model to ensure that claims are minimized from the construction phase of the project.

2. Existing geological and geotechnical data from boreholes, trial pits and various other forms of ground investigation. This is supplemented with new borehole information as the project proceeds.
3. Analysed information such as soil layering, rockhead, groundwater table contours and other information based upon the geotechnical investigation.
   Ground monitoring data from current and adjacent sites.



Existing Foundations | Utilities | Combined Ground Models | Ground Profiles | Surfaces, Rockhead etc

Figure 1: Examples of Typical 3D GIS Data on Building Projects

Arup have been using 3D GIS based tools for over 15 years. With the use of some customized tool, engineers can create 3D ground model in GIS environment from raw borehole data in minimal time. This paper presents some tools customized to enhance the process of borehole data and some examples of how GIS had been applied on building projects.

2 SPATIAL DATA AND GIS MODELS

Digital basemaps are used to build and develop GIS models at the feasibility and concept design stages of a project. As design development continues more accurate digital basemaps with detailed site survey is required to better define the GIS project. For buildings and small scale projects accurate site survey is crucial to accurately define the various elements of the elements of the physical features around and within the project site. Figure 2 shows some of the elements that can be built into a GIS project and data model.

Data can be imported into GIS projects from various sources including satellite images, digital ortho images as well as CAD files, textual data, spreadsheets as long as it has spatial coordinates and a linkages to map related information.

Usually a GIS project with defined data structure including 3D model is initiated for each project with all relevant ground information and desk study data compiled and included in the model.

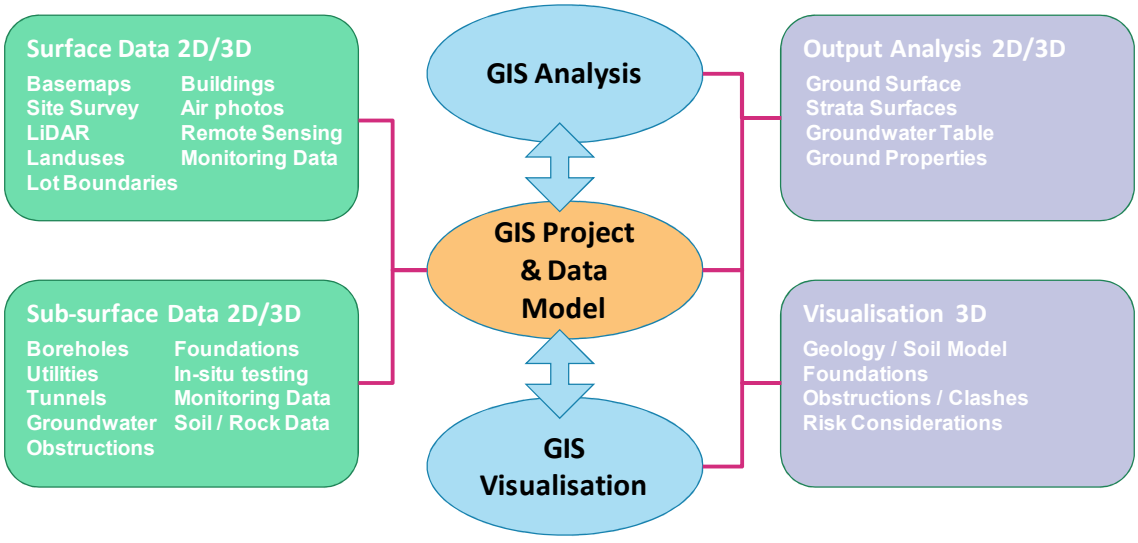


Figure 2: Generalised GIS Data Model, Process and Output

Ground models and 3D site data is needed to develop the baseline surfaces to characterize the site formation as well as below ground GIS surfaces. Ground levels can be extracted from the B1000 basemaps in Hong Kong available from the LIC but within urban areas and within rapidly changing hillside areas the ground model can have significant errors.

Recently LiDAR survey data is available for most of Hong Kong's areas. This data can be used to build up more accurate ground surface models as well as confirm building structure data. In built up areas ground surfaces can be more easily modeled where existing 3D basemap data is not available or very limited. An example of a point cloud data model versus the B1000 LIC basemap data is shown in Figure 3.

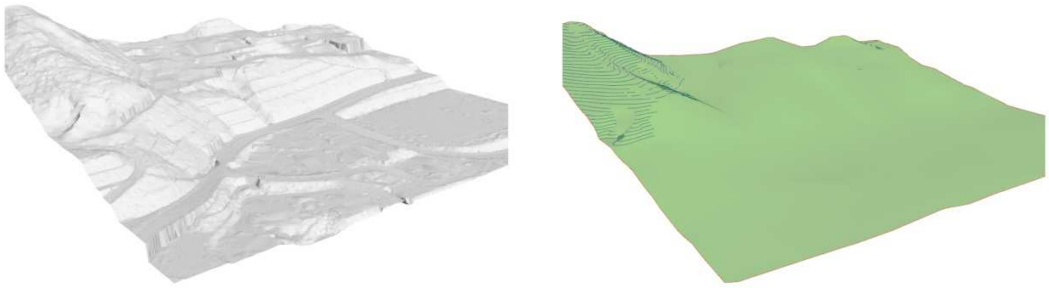


Figure 3: LiDAR Point Cloud Surface Model (Grey) and TIN Model for B1000 basemap (Green).

Other spatial data needs to be included in the model such as boreholes, strata surfaces/interfaces, slopes, foundations, utilities etc to show what influence the data has on the project. The data usually forms a cascading set of information that reflects ever deeper portions of the site and the subsurface conditions that could be encountered.

GIS technology allows specific solutions to be developed to solve problems and allow data to be extracted, manipulated and analysed. The quality of data, its accuracy and ownership should always be reviewed and checked when developing and building a GIS project and database.

3 GIS DATA APPLICATIONS

Arup have developed various tools within the ArcGIS environment that allow geotechnical and geological data to be extracted from various digital formats such as AGS borehole information and then displayed or analysed within 3D GIS projects.

3.1 gINT Data Reclassification

Borehole logs and data acquired from GEO database as well as old AGS data may have different data descriptions such as CDG equivalent to Grade V weathered granite etc hence the descriptions for the same geological type are often different. These variations in wording make grouping and classification of geology type difficult and require lots of manual input.

A simple Macro was written in MS Excel to reclassify different description for the same geology type into a standard name or into a major group. A library of geology description keywords was built for lookup reference. The macro also cross checks the input gINT file with the library keyword. The remaining unclassified geology description can be added into the library to make the lookup tables and the macro more efficient. This macro acts as a data cleaning tool for any gINT AGS data gathered from a project site or

acquired from GEO. It minimizes the manual work required, whether it is for the production of geological cross sections or 3D ground models.

3.2 3D Borehole Generator

With the increased use of 3D ground models in foundation and underground structure design, it became essential to be able to create the model from raw data promptly. A tool was developed in ESRI ArcGIS ArcScene environment to automatically generate 3D borehole sticks from gINT database (Figure 3a). The tool can directly input gINT gpj or any table format to create 3D point, 3D polyline or multipatch to represent borehole stick. Both vertical or inclined boreholes can be generated and there are pre-defined schema to create borehole stick for different representation of data such as SPT, geological strata, core recovery, RQD etc. Users can apply predefined symbology to illustrate different soil type by different color.

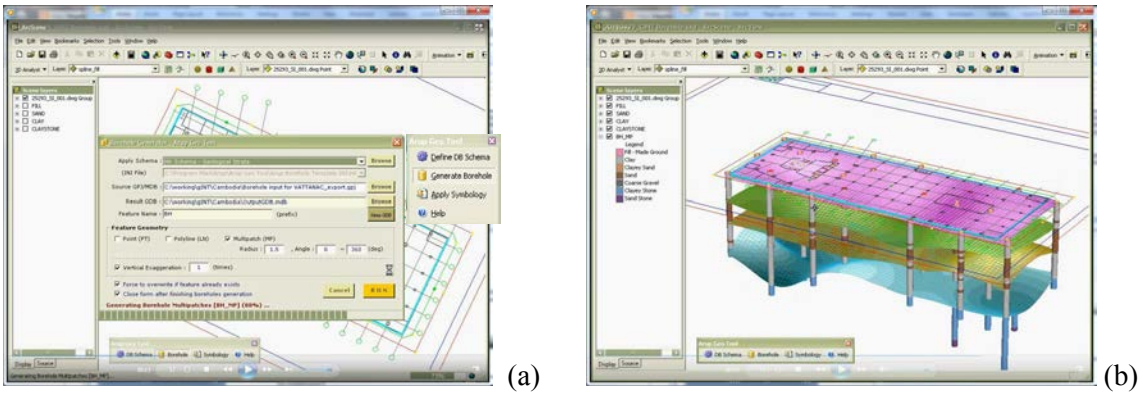


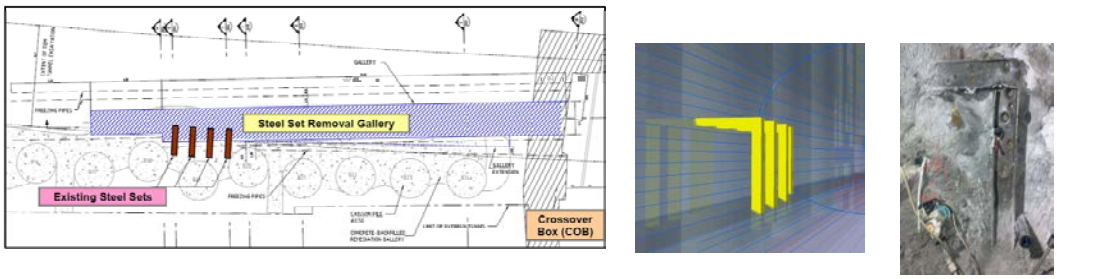
Figure 3: Borehole Generator Tool in ArcGIS

The tool was developed with an easy user-friendly interface that can be used by an Engineer or Geologist. Together with the reclassification macro discussed in previous section, a 3D ground model with borehole stick and surfaces showing the major soil type can be instantly generated without the help of GIS specialist, see Figure 3b.

4 PROJECT EXAMPLES

4.1 As Built Foundations and Temporary Works

An example of how as-built foundation data for an adjacent old caisson pile wall was built into a 3D GIS model and used to visualise its impact on a tunnel is shown in Figure 4. The model allowed the Client and Contractor to identify the risks, visualise the problem and develop a solution for the removal of the steel set obstructions. The excavation identified the steel sets exactly to identify the problem for nearby tunneling. A 3D GIS for the proposed tunnel extension to show the constraints of the obstructions and how it was finally removed.



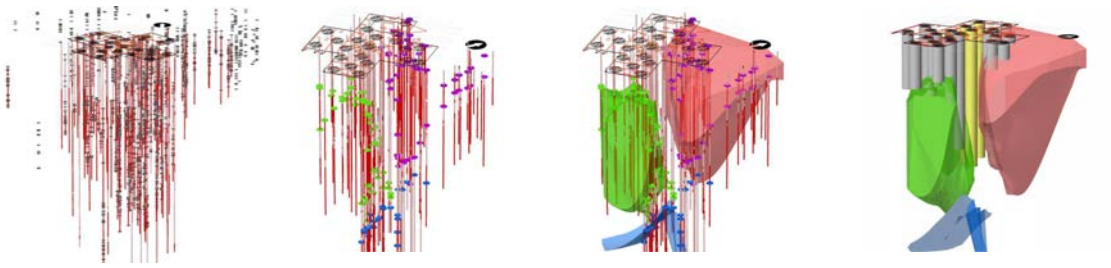
Existing Foundations Temporary Works 3D GIS Modelling Actual Steel Sets

Figure 4: Existing Foundation Steel Set temporary works 3D GIS Visualisation

4.2 Foundations with Deep Weathering Profile

For this project a 3D ground model was developed to review and confirm the foundation design system for one of the buildings that encountered a deeply weathered fault zone. A detailed closely-spaced ground investigation was carried out and a 3D geological model was developed. The model was built from borehole AGS data and then the boreholes were classified into two types: granitic rock (i.e. material with weathering grade of Grade III or better) and saprolite (i.e. material with weathering Grade IV and/or V), as shown in Figure 5a.

Due to the complexity of the geology rather than using the auto-generated 3D model an experienced geologist manually selected the points to define the top and bottom of a similar rock groups in ArcScene. After detailed examination of the model, the points were categorized into three rock group, as shown in Figure 5b. A solid multipatch was created using the points of each rock group and clearly illustrates the sub-vertical deep weathering zone, as shown in Figure 5c. The engineer can see what influence the feature has on the foundation, as illustrated in Figure 5d.



a) Soil & Rock Model | b) Rock Group Zones | c) Solid Rock Zone Model Developed | d) Foundations and Rock Zone Model

Figure 5: GIS 3D Geological Model for assessment of Foundations

GIS allow user to view the geological and ground model to help the Engineer and Client understand the ground condition and complications of the geology and the ground risks more clearly.

4.3 Foundations Founded on Soil

In this example the problem was to define in a 3D context how the ground strength and friction characteristics of the ground could relate to a friction pile / barrette foundation solution. In Figure 6 borehole sticks have cylinders to represent the SPT-N values and the surface represents the soil strata with SPT-N values exceeding 100 blows.

The 3D model was able to show to the Client and the Authorities how consistent and uniform the soil strength would be to demonstrate that the foundations for this very tall building would behave in a consistent manner.

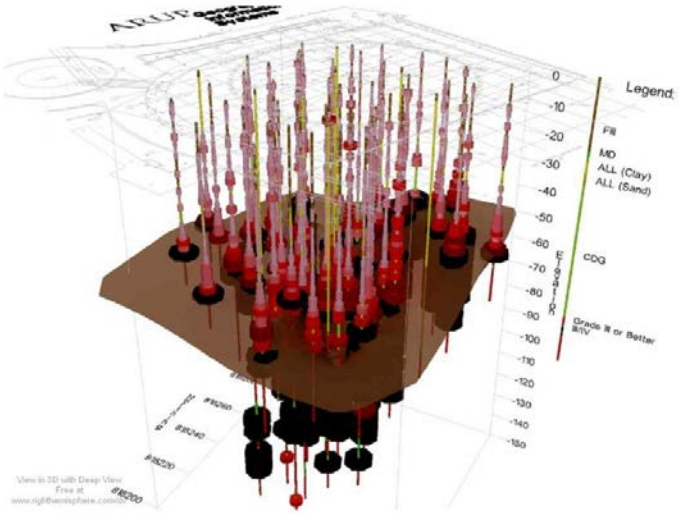


Figure 6: GIS 3D Model for Soil Strength of Founding Soil

4.4 Foundations Founded on Rock

This example is a deep basement project in South Korea where it was important to display the different discontinuity information (see Figure 7) to explore the local variations in orientation for the basement wall toe stability as well as the rock stability below the wall.

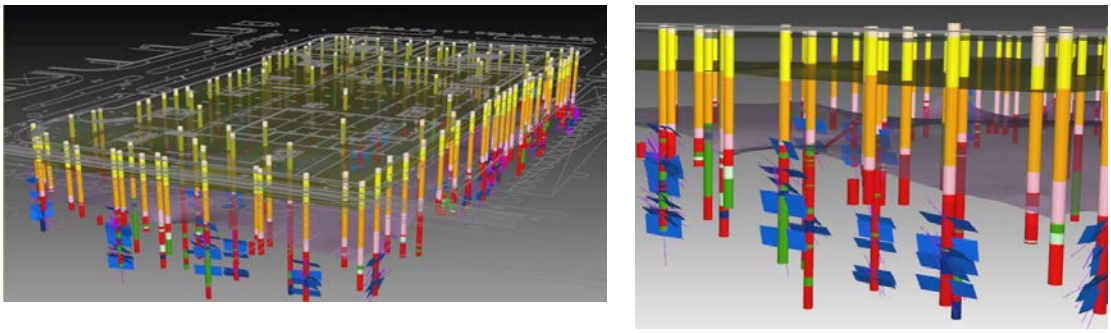


Figure 7: GIS 3D Model for Discontinuity Orientations of Basement Wall Toe Stability

5 CONCLUSIONS

There are multiple sets of data that can be incorporated into GIS project models that can be used to display and analyse various aspects of building foundations and building projects. GIS tools can be developed within the GIS environment to extract and build models to provide visual as well as quantifiable models to allow better decisions to be made and data to be stored.

Monitoring Ground Instability in Wide Areas and Single-building Cases by Means of Satellite A-DInSAR

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ABSTRACT

Satellite Differential SAR Interferometry (DInSAR) represents one of most innovative ground and structures displacement monitoring solutions in urban areas. At present several Advanced DInSAR techniques (A-DInSAR) based on multi-image data processing are available to achieve information about trends and time series of displacement through the analysed period with high accuracy.

In the context of geotechnical issues, A-DInSAR can be a useful tool in several situations, both for diagnostic and monitoring purposes. As a matter of fact, A-DInSAR is one of the few techniques available for detection and measurement of ground displacements occurred in the past, thanks to satellite SAR data archived by several Space Agencies starting from 1992. Hence, it is possible to obtain information on ground stability conditions (e.g. landslides, subsidence) when no preliminary monitoring activities had been performed before on specific areas. Moreover, A-DInSAR is a valuable tool for monitoring of ground deformation during construction (*post-operam),* thanks to new satellite images acquisitions planning.

Given its features and new high-resolution SAR satellites, A-DInSAR represents a powerful solution for investigation of ground and structures displacements both for large areas and defined buildings or structures.

1 INTRODUCTION

Monitoring of displacement is today a well-accepted control tool, nevertheless, it can be very often considered as a source of information for investigation and diagnostic purposes (Mazzanti 2012). In particular, when no information about past displacements are available for some areas or when very large areas have to be monitored, satellite SAR interferometry (DInSAR), especially with new Advanced DInSAR (A-DInSAR) techniques, can represent a unique resource (Bozzano & Rocca 2012). Other techniques, in fact, (e.g. aerial photos records) may provide such information, but generally, accuracy of detected displacements is much reduced when compared with the results of A-DInSAR.

In what follows, this paper presents some results of A-DInSAR applications to detect and quantitatively derive past or unknown deformation processes caused by several phenomena in various contexts. In the first one, a 20 square kilometres river basin in central Italy, severely affected by landslides, has been analysed by detecting and evaluating displacements occurred on the slopes during the period 1992-2010. Thanks to A-DInSAR results, the state of activity of many landslides, has been evaluated and mapping of landslide areas has been refined. Displacements in terms of time series and dynamics have been properly defined. A-DInSAR data have been merged with more traditional methods (aerial photos, field surveys, geotechnical monitoring) to find a useful integration of information both in spatial and temporal terms (Rocca et al. 2013).

Furthermore some details about A-DInSAR applied to some single buildings are discussed to show the great opportunity to analyse displacement distribution at different portions of the structures.

A-DInSAR could become a new standard in monitoring and geological survey for both the site planning and natural terrain hazards comprehension and design of mitigation measures. Several sensors and satellites with various characteristics can be used in terms of resolutions (both spatial and temporal), accuracy, sensitivity and frame dimensions, making A-DInSAR able to afford several situations with different characteristics (Bozzano & Rocca 2012).

2 MONITORING OF DISPLACEMENT BY A-DINSAR

Starting from the first 1990s, satellite DInSAR has gradually become a common tool used to detect and measure ground displacements, especially for the scientific community dealing with geology, geophysics and geomatics. (Massonnet et al. 1993; Peltzer & Rosen, 1995; Rott et al. 1999). Today, thanks to the developments of new data processing approaches (Ferretti et al. 2001; Berardino et al. 2002; Kampes 2006; Perissin & Wang 2012), Advanced DInSAR can be considered as a well-established technique able to provide information of ground displacements with high accuracy.

The basic principles of the technique are the following: SAR satellites fly along predetermined orbits paths acquiring radar images of earth's surface. They review the same area with a frequency that depends on the satellite itself (once every 35 days for older satellites and with higher frequencies for new generation satellites). The analysis of images stacks acquired is performed comparing the phase of the microwave signal sent to the ground and backscattered to the sensor. The signal phase differences, specifically processed, are related with displacements occurred during the time interval between images acquisition. In this way it is possible to derive the velocity trends of objects with millimetre accuracy.

A-DInSAR is characterized by many advantages: for example, it operate in a full remote way (no artificial reflectors need to be installed); it is possible to perform analyses at different scales, in wide areas (more than thousand square kilometres) as well as at single-building scale (the latter, in particular, thanks to new high-resolution sensors); widespread results over analysed areas can overcome the problem of limited information, punctually distributed, often achievable using standard geotechnical monitoring methods. Moreover, probably the most important feature of A-DInSAR is its capability to investigate in quantitative way displacements occurred in the past, analysing SAR data archived from 1992 by Space Agencies in charge of satellites management.

On the other hand, some limitations must be considered: only some pixels of the images are characterized by stable and strong enough signal to be processed. Commonly, they are called “Persistent Scatterers” (PS) (Ferretti et al. 2001) and they are related to highly reflective objects on the scene (e.g. buildings, concrete or metal structures, exposed rocks etc.). For this reason, areas totally covered with vegetation are not analysable. Moreover, the amount of detectable displacement is related to a portion of the microwave signal wavelength, which is commonly in the order of few millimetres. For this reason, very fast displacements are not detectable.

Finally, as for other remote sensing monitoring technique, displacements are detectable only along the satellite ‘Line of Sight’ (LOS). Nevertheless, because the same area can be observed from different points of view, while the satellite flies along different orbit paths (the so-called ascending path, when the orbit is from South to North and descending when it is from North to South), it is possible, combining these different information, to decompose displacement direction in vertical and W-E component, while the N-S is in any case, not detectable.

3 INVESTIGATION BY A-DINSAR

3.1 Landslides

This section presents a study on a small basin (about 20 km^2), crossed by a stream, located in a low-hilly area in the central Italy. By field surveys and multi-temporal analysis of aerial photos from 1954 to 2002 more than 90 landslides were identified and mapped. The landslides affect more than 75% of this portion of the basin and were classified in translational sliding (38%), earth-flows (31%) and complex mass movements (31%). Landslides areas are found to be ranging from very few hundreds m^2 to some km^2. Landslide deposits are related to the mobilization of pelitic and psammitic lithologies, especially in the middle and lower portions of the slopes. In the mainly pelitic areas, geomorphological elements related to the largest landslide phenomena, such as niches, scarps and counterslopes, are highly altered and degraded, and often modified by the strong anthropic activity in the area.

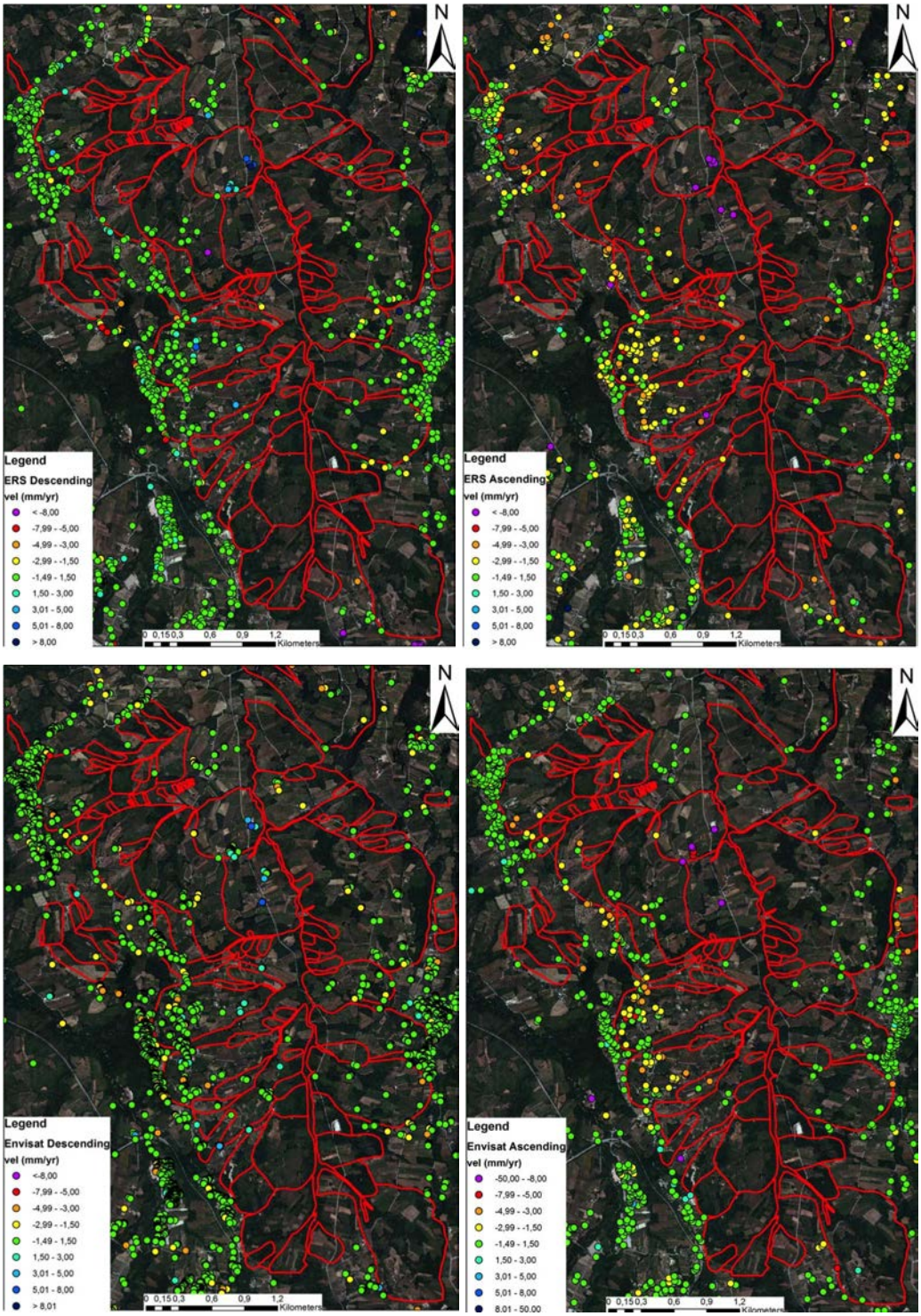

Figure 1. – ADInSAR results: red polygons identify the mapped landslides; PS are represented as coloured points. It is clear the evidence of instability processes during the analysed period (ERS: 1992-2001; Envisat: 2003-2010)

In order to refine landslide mapping and to derive state of activity and the evolution rate, A-DInSAR investigation have been performed. This study have been carried out, by using images derived from the European Space Agency (ESA) archives, available in the frame of the CAT-1 project "Landslides forecasting analysis by displacements time series derived from Satellite and Terrestrial InSAR data" (ID: 9099). Four different datasets have been used to characterize the landslides historical displacements; specifically, ERS1-

ERS2 and Envisat satellite data both in ascending and descending orbit acquisition geometry have been selected for the period 1992-2010 (Rocca et al. 2013).

The SAR data have been processed through SARPROZ, a software tool specifically developed for multi-image InSAR analyses such as PS (Ferretti et al, 2001) and QuasiPS (QPS) (Perissin & Wang, 2012). For this case study, two different approaches have been used: i) standard PS analysis where velocities are evaluated applying a linear trend model (Ferretti et al, 2001); ii) local analysis of small areas, performed for most interesting zones, which allows to achieve higher accuracy in displacements estimation, and, especially, to detect non liner trends. It is worth noting that non-linear movements are considered crucial for a suitable investigation of landslide processes affecting the investigated area.

A-DInSAR results (Fig. 1) allowed the investigators to derive useful information for more than 30% of the mapped landslides affecting the basin (more than 55% in terms of area), especially for largest ones and those affecting urbanized areas.

Thanks to the availability of the long-term data record (1992-2010), the displacement information allowed the refinement of landsides mapping and in deriving the state of activity. Furthermore, some coalescent landslides previously mapped were recognized as a unique event and some landslides were internally distinguished thanks to the identification of differential deformations. Furthermore, some areas of particular interest have been analysed by a local analysis, with *ad hoc* data processing performed to investigate localized processes.

Considering the size of the studied area, a detailed presentation of the results on observed landslides is not possible for this paper. By way of example, results and interpretations related to one of the most interesting processes are shown. In this specific area (Fig. 2), located at the foot of a wide complex landslide with many coalescent bodies, displacements during the overall period 1992–2010 were detected. Thanks to the ascending and descending geometry combination it was possible to infer the movement direction that is mainly horizontal. This particular dynamics produces, in fact a singular result of PSs, which show opposite direction of displacement (positive in descending geometry and negative in the ascending one; see blue circle in Fig. 2). If only one image stack related to a single orbital geometry had been used, the comprehension of the process would be strongly compromised. The upper part shows lower displacements starting from 2003, while in the lower part displacements are evident since 1992, with a higher rate and a more horizontal component. Furthermore, the overall movement trend is strongly not linear with several acceleration stages recognized. Displacement rates are high (>15 mm/yr) (Fig. 3).

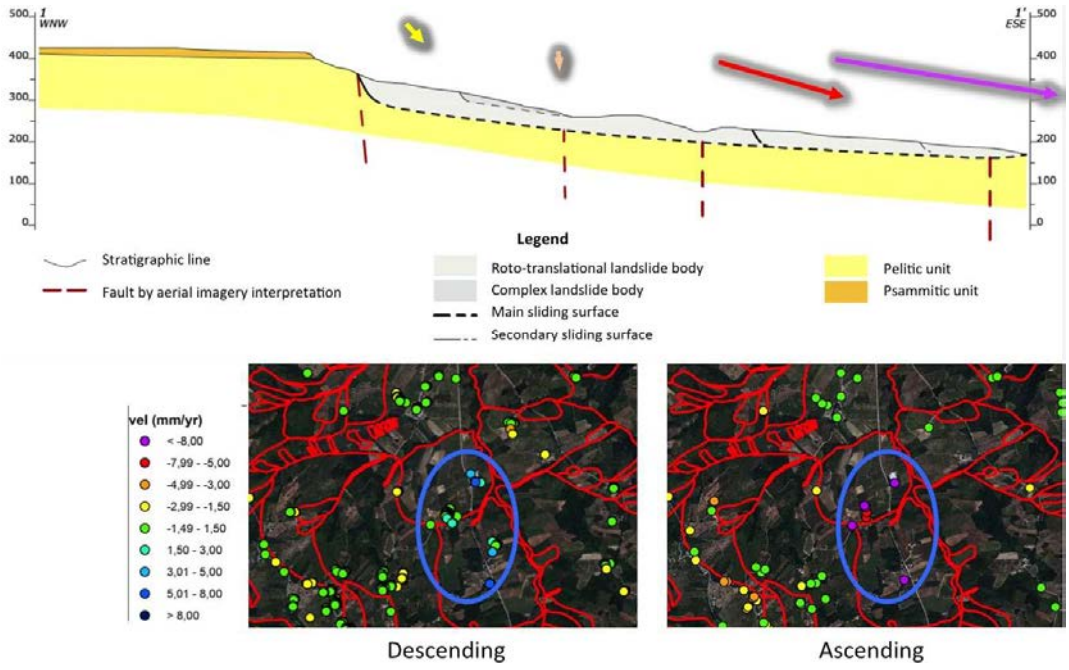

Figure 2. – Single landslide process dynamics derived combining ascending and descending A-DInSAR information. Vertical and horizontal displacement components have been inferred, so deriving the real displacement direction for all the landslide body portions.

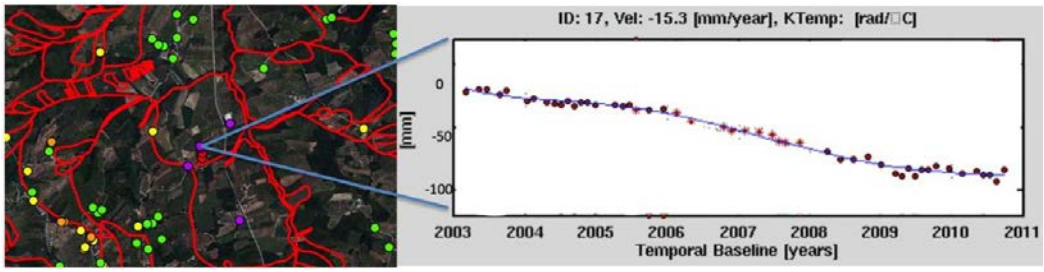

Figure 3. – Time series of displacement related to a PS detected on the discussed landslide

3.2 Buildings

In addition to landslides, A-DInSAR is a valuable tool for investigation of other deformation processes that induce displacements of terrain or structures. For example, the following results are relative to an A-DInSAR analysis performed over an area near the city of Rome, Italy. This area, during the past decades, was affected by ground subsidence caused by dewatering linked to quarrying activities (Brunetti et al. 2013). The process affected a wide area (several sq. km), but in this work the authors wish to focus on some details. Because of a pronounced geological heterogeneity, the effect of subsidence depends on the thickness of the compressible layers and is more evident exactly where these are thicker. As a consequence, some large buildings with foundations built partially on thick compressible layers and partially on less thick compressible layers were affected by differential settlement processes. This effect was well recognized also by A-DInSAR results. In this case, the combination of descending and ascending data has been very useful to detect this dynamics thanks to the difference between the two LOS displacement magnitudes (Fig. 4).

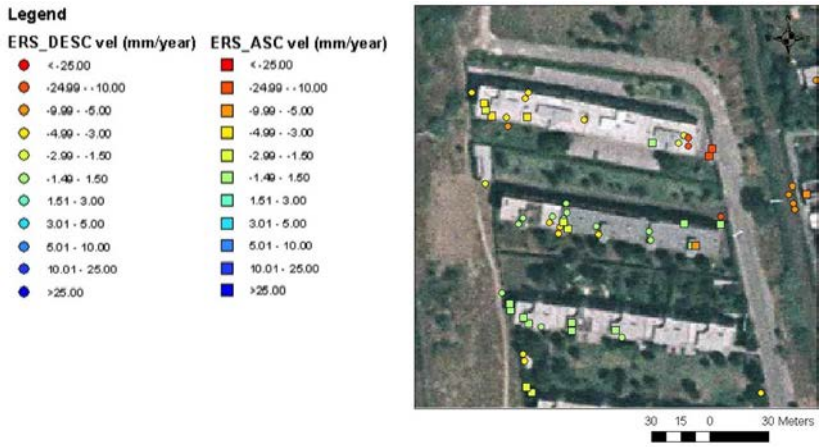

Figure 4. – Ascending and descending PS related to a couple of buildings (higher part of the picture) affected by differential settlement process (period: 1992-2001).

Another brief example, useful to understand A-DInSAR potential in building investigation, is relative to a high rise building in Hong Kong. In this case, thanks to the building height (more than 90 m) and the high-resolution data available, a single orbital geometry is enough to detect a hypothetical and very small tilting process possibly affecting the building. This hypothesis is related to a sort of gradient of displacement, which increases with increasing height (Fig. 5). To perform the analysis TerraSAR-X data relative to a four years period (2008-2012) have been used.

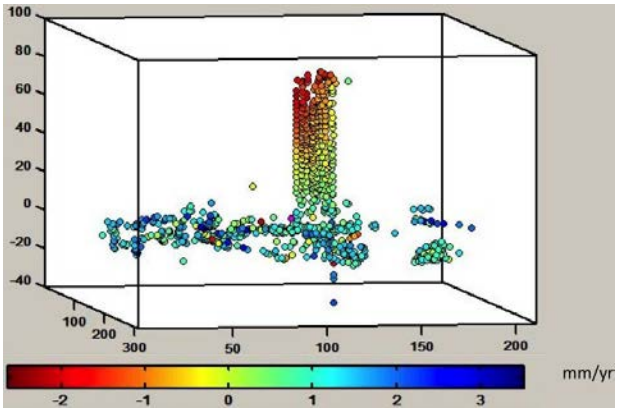

Figure 4. PS related to a high-rise building with a clear gradient of displacement along the structure.

4 CONCLUSIONS

In this paper, we have showed some results related to A-DInSAR analyses performed in different contexts for investigation of different deformational processes. Large area analyses as well as local area investigations are achievable in order to improve knowledge about the observed processes. Both landslides and subsidence and local deformation processes have been observed and described with very good detail concerning the dynamics. In this perspective, A-DInSAR can represent a diagnostic tool especially for investigation of past events when no other monitoring data are available. This solution can substantially change the approach related to projects and building works in areas with unknown ground instability problems. On the other hand, it can also be an effective tool for planned monitoring of large areas (e.g. during building activities in urban areas) if new satellite images acquisition is expressly planned.

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Monitoring Natural Slopes and Man-made Structures by TInSAR: Understanding Behavior and Forecasting Method

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ABSTRACT

Remote sensing techniques for the monitoring of displacements are opening new opportunities in the field of geotechnical engineering and geology. Terrestrial SAR interferometry (TInSAR) is one of the most innovative techniques and it promises to be a very effective solution, which will be extensively used in the near future. TInSAR is characterized by several interesting features such as: (i) high density of information; (ii) fully remote capability; (iii) long range capability; (iv) panoramic perspective; (v) spatially continuous efficacy and (vi) high accuracy. Thanks to these features, TInSAR has been used for investigation and diagnostic purposes (viz. landslide and structural movement monitoring,), and provided very useful data.

1 INTRODUCTION

Over the past decades, displacement/movement monitoring has become an important tool for assessment of stability conditions for the control of many geological, geotechnical and structural problems. Hence, it is now a common requirement to have various monitoring plans covering different aspects in large construction/infrastructure projects (e.g. tunnels, dams, highways). In other words, displacement data obtained through monitoring is considered one of the most useful information for understanding the behavior of the ground/structures that we are interacting with and, therefore. Monitoring of displacements is in fact now widely accepted and, quite often, is also imposed by authorities in charge of workers safety and by insurance companies.

By showing some real examples based on application of Terrestrial SAR Interferometry we aim to promote better appreciation of the use of displacement monitoring as a control tool and as an investigation and diagnostic tool. In other words, under certain conditions, the monitoring of displacement allows to derive information on the behavior of both natural and man-made structures that may be useful for both decision-making and design purposes.

2 REMOTE METHODS FOR THE MONITORING OF DISPLACEMENTS

Recent development in methods of remote monitoring are changing the traditional view for the following reasons (Mazzanti 2012):

1) they are characterized by a “panoramic perspective”, this means that they provide the opportunity for looking at the subject area with a wider perspective than contact systems, even if, quite often, they are less precise in term of positioning;
2) their use is negatively affected by the presence of obstacles along the line of sight (vegetation, working machines etc.);
3) they do not require direct interaction with the monitored object/area;
4) they may only look at the superficial effect of the ground/structure deformation;
5) their accuracy is strongly site-specific (depending on distances, weather conditions etc);
6) in some cases (e.g. satellite InSAR) they are able to derive information about historical displacements.

These features imply both advantages and limitations with respect to contact monitoring systems, but it is not the intention of the authors to discuss about this point. For the aim of this paper it is enough to outline the differences and the different risks and opportunities that may have led to.
Remote sensing methods may be classified in two main categories, viz. (1) partially remote techniques and (2) fully remote techniques (Mazzanti 2012). Only the second category is characterized by all the features described above and, especially, by the fully contactless approach, since they do not require the installation of targets or sensors on the ground/structure.

2.1 Terrestrial SAR Interferometry

Terrestrial Synthetic Aperture Radar Interferometry (TInSAR, also referred as GBInSAR) is a ground based radar technique for the remote monitoring of displacements (Antonello et al. 2004; Luzzi 2010; Mazzanti 2011). TInSAR is applied by using equipment made of a linear rail and a radar sensor (Fig.1). The sensor, moving along the rail during data acquisition, derives 2D SAR images. Then, by the interferometric technique (i.e. by comparing the phase difference of each pixel between two or more images collected at different times) displacements along the instrument line of sight (LOS) are derived. Hence, both colored images and time series of displacement of each pixel can be achieved (Fig.2). Pixel resolution ranges from less than one meter to few meters, depending on the rail length and the sensing distance. The accuracy in displacement measurement may range from few decimal mm under ideal conditions (short term and short distance monitoring), to some mm in more complex conditions.

Figure 1. Picture of the TInSAR system IBIS-L by IDS S.p.A. installed on a QUIB basement by NHAZCA S.r.l.

Depending on the equipment used, the temporal resolution (i.e. the data sampling rate) of SAR images may range from a few minutes to a few seconds. Furthermore, by using microwaves signals, TInSAR is able to collect data under any weather and lighting conditions.

TInSAR monitoring can be performed by installing the equipment in a stable location (up to 4 km away) with a panoramic view of the monitored area, and it does not require the installation of contact sensors or reflectors in the monitored area.

3 INVESTIGATION BY TERRESTRIAL SAR INTERFEROMETRY

Over the past 9 years the authors have been following the development of the TInSAR technique from its early stage, viz. the trial use of the first developed prototypes, to extensive and long-term applications by using industrial equipments. This long-term involvement allows the authors to build up experience in its use and to understand its main advantages and limitations of TInSAR and its efficacy in different applications. Specifically, the authors have been directly involved in projects concerning volcanoes, architectural heritages, civil buildings, dams, bridges, viaducts, tunnels, pipelines and several slope instability processes ranging from earth-flows, deep-seated rotational landslides, small translation landslides and rock cliffs instabilities (Bozzano et al, 2008, 2010, 2011; Mazzanti et al, 2011a; Mazzanti & Cipriani 2011).

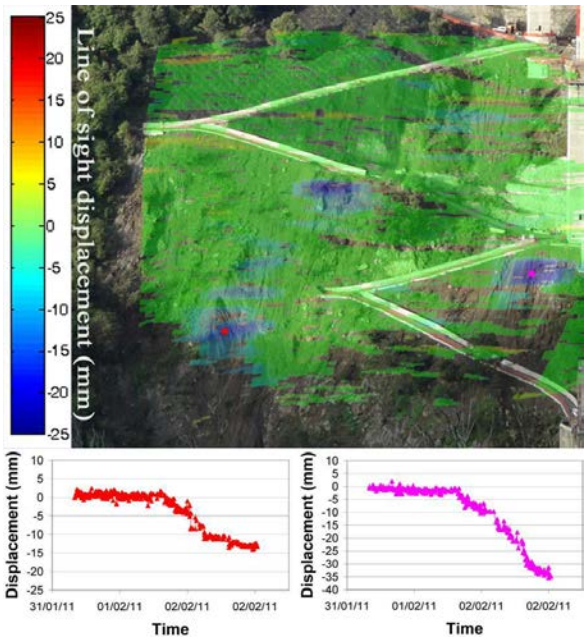

Figure 2. Displacement map overlaid to a slope picture and time series of displacement of two pixels.

In the early stage development of TInSAR technique, it was used mainly as an efficient safety control measures for continuous monitoring of instability processes which may generate risk to life and human activities (Casagli et al, 2003; Casagli et al, 2010; Bozzano et al, 2011). For this application the following main features are required: (i) fully remote capability; (ii) accuracy in displacement measurement; (iii) high sampling rate; (iv) effectiveness under all weather and lighting conditions.

In recent years, TInSAR has demonstrated to be a powerful tool in the hands of geologists and geotechnical and structural engineers for investigation of ground and structural instability processes, thanks to same features described above, but also to: (i) the 'panoramic view' capability; (ii) the continuous spatial information; (iii) the high information density and long sensing distance.

Selected case examples of TInSAR monitoring for investigation purposes are described below.

3.1 Landslides

Landslides are probably the geological/geotechnical process that analyzed most extensively using Terrestrial SAR Interferometry (e.g. Antonello 2004 et al; Luzi 2010; and Mazzanti 2011b).

In 2009, a slope affected by an earth flow, in the Lazio Region (Italy), was monitored continuously for one year mainly as an emergency measures. A small portion of the slope, failed in the late 2008 following an intense rainfall. Due to the geological nature of the materials involved, e.g. alluvial sand and silt deposits of the River Tiber, and due to heavy water ingress, the landslide debris at the lower part of the slope moved forward as an earth flow, hitting some houses and a major pipeline (Fig.3).

Due to the unstable conditions of the area and to the dynamic of the landslide, conventional investigation methods (geological surveys, geophysics, borehole etc.) were not feasible, and only a visual inspection could be performed.

The continuous monitoring using TInSAR, with a sampling rate of around 5 minutes, provided useful information on the slope behavior. Displacements collected on first day showed movements up to 40 mm along the 'Line of Sight' (LOS) in the deposit zone, while no movements were detected in the source area of the slope (Fig.4). Hence, the hazards were found to be mainly related to the flowing deposit, and no imminent retrogressive processes were going on.

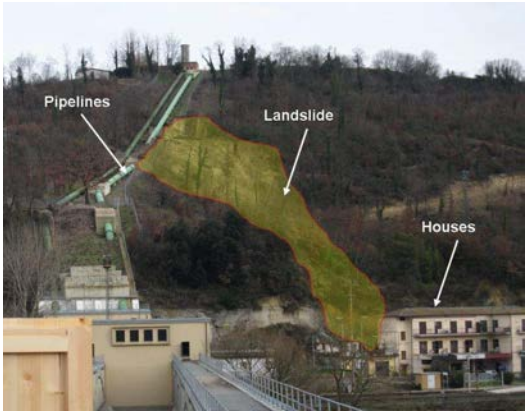

Figure 3. Picture of the slope and identification of the 2008 earth flow (in yellow) and affected houses and pipelines.

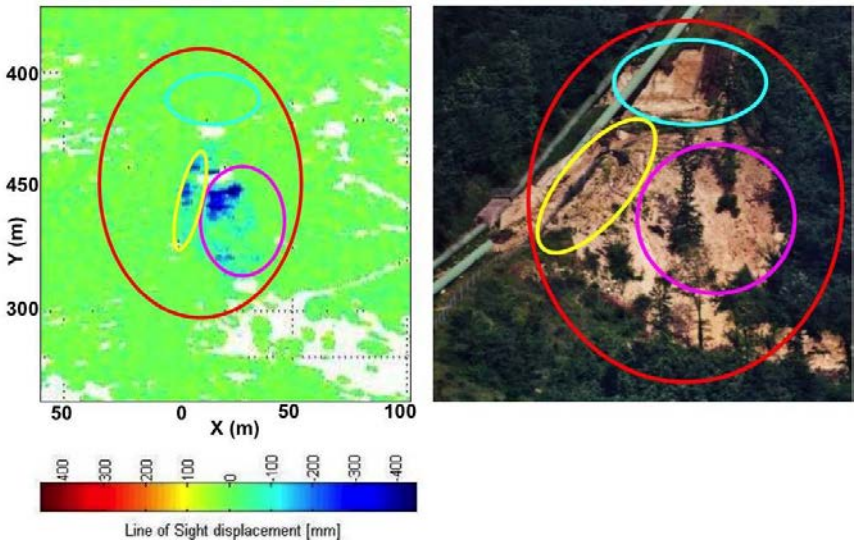

Figure 4. Picture of the landslide (on the right) and 24 hours displacement map of 23-01-2009 (on the left). Colored ellipses identify corresponding sectors.

Data collected during following days confirmed the stability of the up slope scarp and the movement of debris. Regarding the deposit, it was possible to depict a displacement behavior characterized by heterogeneous movements with accelerations and decelerations localized to small areas and a rapid response (i.e acceleration) to rainfalls. Such a behavior was interpreted as the result of a superficial movement of the mass, which implies a low hazard to infrastructures.

In the authors'experience, one of the most interesting applications of TInSAR is the monitoring of a complex deep-seated landslide affecting the entrance of a tunnel during construction in the southern part of Italy (Bozzano et al, 2008, Bozzano et al, 2011; Bozzano et al, 2012). Following the collapse of the tunnel portal during construction, a detailed investigation of the slope were carried out (Bozzano et al, 2011), thus allowing to achieve an engineering geology model of the slope and to design countermeasures for its stabilization. The continuous monitoring by TInSAR allowed detection of different displacement patterns of the slope and its reaction with the stabilization measures implemented (viz. anchored bulkheads, gabions, shotcrete, etc.) during different phases such as excavation, construction of the structures and tunneling (Bozzano et al, 2011). Furthermore, by the combination of TInSAR data with other collected information, a detailed zoning of the whole slope was undertaken. However, the main contribution of TInSAR monitoring was the understanding of the overall transient behavior of the slope and its reaction to rainfalls and excavation activities. Specifically, by the back-analysis of more than 10 landslides (from 10 to 10,000 m3), occurred during the monitoring time, the main behavior of the slope, before failure, was derived (Mazzanti et al,

2011b). Suitable models for the prediction of landslides were also developed and calibrated (Bozzano et al, 2012), thus allowing the prediction of future events and the displacement to rainfall correlation to be derived. A similar investigation has been performed also for the anchored bulkheads, thus allowing an improved definition of stability thresholds with respect to the one derived from geotechnical data (Bozzano et al, 2012).

3.2 Rock cliffs

Over the past years, some authors have been trying to apply remote sensing methods for the investigation of natural rock cliffs and man-made rock cut slopes (among the others Lim et al, 2005; Mazzanti et al, 2011a). At this regard, recent studies have demonstrated that TInSAR may represent a useful tool to investigate stability condition of natural cliffs and that it is still more effective if combined with other remote and traditional techniques (Mazzanti et al, 2011a).

By monitoring vertical cliffs it is possible to identify sectors affected by permanent micro-movements (sometimes half a millimeter or less), which may indicate a fairly stable condition. Furthermore, by comparing TInSAR time series with temperature and rainfalls data it is possible to identify blocks affected by cyclic movements which can be assumed as the most susceptible to collapse.

This information may be very useful if combined with conventional investigation systems based on geomechanical analysis. As a matter of fact, conventional geomechanical methods look only to preconditioning factors like joint features (orientation, spacing, aperture etc.), rock stiffness etc. On the other hand, information derived by TInSAR may provide useful indicators of the state of activity of a block by looking to its deformational behavior. Successful applications have been carried out by the authors on different cases in Italy over the past years (Mazzanti et al, 2011b).

3.3 Man-made structures

Remote monitoring of displacement of man-made structures by TInSAR may be very useful to complement conventional solutions. In what follows two examples are briefly presented.

The precise mapping of sectors of a building affected by displacement may be derived from TInSAR monitoring thanks to its widespread coverage and spatially continuous capability. An interesting example refers to a civil building in the city of Rome, which was affected by displacements during the underground activities for the construction of the third Metro Line in Rome (Fig. 5) (Mazzanti & Cipriani, 2011).

The rapid mapping of deformation pattern of a concrete or earth dam in response to increasing and decreasing of the water level may be easily derived by TInSAR monitoring. Also in this case the following features are particularly relevant: i) widespread view, ii) spatial continuity of information and iii) high data sampling rate. Furthermore, especially in the case of concrete dams with a sensing distance lower than 100 m, high accuracy (on the order of 0.1 mm) in the displacement measurement can be achieved.

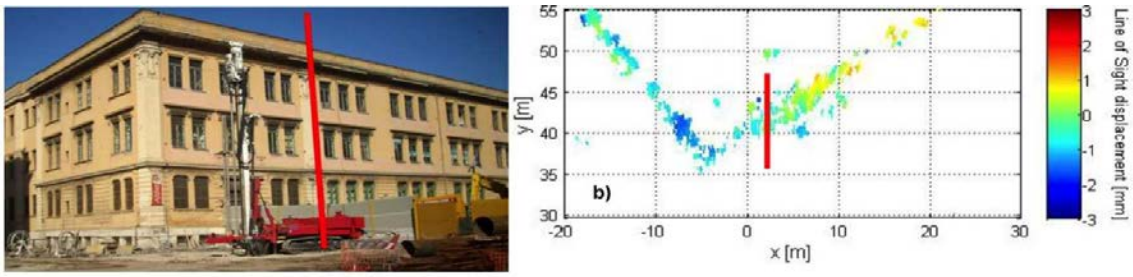

Figure 5. Picture of the building and displacements map derived by TInSAR. The red line identifies the same sector.

4 CONCLUSIONS

In this paper several case histories where the observational method was successful applied are presented. The focus has been placed on unconventional applications, i.e. the investigation and diagnosis of geological,

geotechnical and structural instability problems. In the past years, several new techniques have been developed in the field of displacement monitoring, thus increasing the opportunities offered by the observational approach, like the investigation purpose. Terrestrial SAR Interferometry (TInSAR) is probably one of the most interesting one since it combines several useful features like: i) fully remote efficacy; ii) widespread view; iii) spatially continuous information (i.e. maps of displacement instead of single points); iv) long range attitude (up to some km); v) effectiveness under any lighting and weather conditions and vi) high accuracy in displacement measurement.

The huge amount of information provided by TInSAR, has been demonstrated to be very effective for investigation purposes. Various types of information can be gained, e.g. geological, geotechnical or structural conditions using suitable monitoring by TInSAR such as: i) precise mapping of active slope and structural instabilities; ii) identification of stable vs. unstable zones (i.e. moving and not moving zones); iii) analysis of displacement correlation with triggering factors (e.g. rainfalls, excavations etc.). All these data, if suitably combined with existing information, may help to calibrate different models for describing the behavior of slopes or structures.

To conclude, thanks to the recent developments and available techniques, the monitoring of displacement is now a new weapon for geologists and engineers in the investigation of their professional challenges.

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Applications of Geotechnical Baseline on Deep Excavation and Foundation

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ABSTRACT

Geotechnical Baseline Reports (GBRs) have been adopted as a tool to balance geological risk and cost of construction projects between the client and the contractor. Over the past five years, application of GBR in Hong Kong construction projects was initiated with tunnelling works. However, recently, GBRs are being adopted in other construction works, such as deep excavation and foundation projects, to define what the expected ground conditions are and as risk allocation tools.

This paper discusses the applications of GBRs on three different deep excavation and foundation projects in Hong Kong. These include case studies on (1) bulk excavation and slope stability of a deep excavation project; (2) diaphragm wall construction of an underground structure; and (3) construction of a bridge. The baselines approaches for key geotechnical issues which may induce significant impacts to the works of the aforesaid projects are discussed.

1 INTRODUCTION

The demand of housing supply in Hong Kong has been rising continuously in the past few years. It is foreseeable that more buildings and associated facilities such as transportation systems will have to be built to satisfy the need of the society. Constructions of buildings and associated facilities commonly involve deep excavation and foundation installation. In many cases, the risks of encountering adverse ground conditions went to the contractors and resulted with high contract prices and/or arguments on claims.

Geotechnical Baseline Reports (GBRs) have been adopted as a tool to balance geological risk and cost of construction projects between the client and the contractor. Over the past five years, application of GBRs in Hong Kong construction projects was mainly associated with tunnelling works. However, recently, GBRs have also been more commonly adopted in other construction works, such as deep excavation and foundation projects, to define what the expected ground conditions are and as risk allocation tools.

The purpose of a GBR is to define ground conditions and that are to be included in the contract as contractual statements (i.e. baselines). Risks related to ground conditions consistent with or less adverse than the baselines are allocated to the contractors, and those significantly more adverse than the baselines are to be accepted by the clients.

The GBR should provide baselines for ground conditions that are contemplated by the proposed works. As the key parts of the GBR, baselined items should be related to those ground characteristics, ground conditions, geotechnical parameters and values that would affect the cost and time for the construction. This should take into account the ground conditions assumed in the design and in the selection of construction methods and plants. The baseline items should be quantifiable and, for the purposes of good definition of the baselined items, there should preferably be sufficient data to form representative database for assessing the baseline ranges or values (although in the extreme case one can baseline an item for which there is no data and the contract is based on the assumed values. If the ground is found to be outside the baseline then the change has to be evaluated). The baselined items should be determinable on site during construction.

Geotechnical Interpretative Report (GIR) or the geotechnical factual data are sources of data for preparing the GBR. The data that is relevant to the works is then baselined according to the balance of geological risks that the client is prepared to accept and that the Tenderers are required to price. It is necessary that the GBRs are to be drafted initially between the geotechnical interpreter, the designer, and the quantity surveyor, and then discussed with the client to ensure that the client understands the associated risk and the levels of the baselines and the associated risk should be adjusted to the client's requirements.

As the site critical ground conditions and geotechnical properties, required products, construction types and methods, quality and quantity of available ground investigation (GI) information and constraints for different constructions may be different, the baselines approaches for key geotechnical issues may vary site by site. This paper discusses the applications of different baseline approaches on different critical ground conditions for three different deep excavation and foundation projects in Hong Kong. These include case studies on (1) bulk excavation and slope stability of a deep excavation project; (2) diaphragm wall construction of an underground structure; and (3) construction of a bridge.

Although GBRs may involve various degrees of geological interpretations in many scenarios, GBRs should not be considered as high class interpretative report. A GBR is better thought of as an extension to the item description attached to the Bill of Quantities.

The GBR in the tender represents what is assumed will be encountered for the purpose of defining the indications of the contract and also to provide a platform for sharing of geological risks between the client and the contractor.

2 CASE 1 – BULK EXCAVATION AND SLOPE STABILITY FOR A DEEP EXCAVATION PROJECT

Case 1 involves the formation of a large scale deep excavation at reclaimed land. The geotechnical engineering works required in the contract of Case 1 involves, a) Bulk excavation down to the required formation levels; b) Formation of temporary open cut slopes (to laterally support the diaphragm walls before installations of lateral support system) and carrying out Excavation and Lateral Support (ELS) works.

The ground conditions, which are considered that would significantly affecting the time and cost of the construction of the above engineering works, and the baseline approaches for these ground conditions are stated below.

2.1 Excavation difficulties in rock and AHM –

GI identified that the surface of bedrock (i.e. rockhead) is located above the formation level at parts of the site and identified the presence of large size (over 1m3) artificial hard materials (AHM) and corestones above the formation level. Compared to excavation in soil, these materials will impose potential excavation difficulties and may require non-conventional tools to excavate.

The baseline approach for such ground conditions is to quantify the volume of materials which will require non-conventional tools for excavation based on information from the available boreholes and then baseline the proportions of excavation in rock and in AHM in terms of percentages volume for the excavation of the whole contract site.

One of the challenges is that it is difficult to identify the size of corestones and AHM from boreholes. Also although many of the geologists or even geotechnical engineers would have an idea on what is a corestone when seeing it on field, it is not always straight forward to distinguish whether a rock core in a borehole is actually a corestone or a zone of rock which cannot meet the engineering criteria of bedrock. There is no commonly agreed length of soil zone to objectively define whether the rock core above it is corestone or part of the geological bedrock. For this reason, the approach of baselining the volume for corestones, as distinct from volume for bulk excavation is not considered appropriated. Instead the approach is to baseline the overall volume of rock for excavation, which includes the defined engineering bedrock, the zones of rock which do not meeting the criteria of the engineering bedrock and the corestones above the formation level.

For the AHM, it is considered that the previous reclamations were constructed in accordance with the standard engineering reclamation requirement and the main sources of large size isolated AHM would be the dumped materials from the demolished old breakwater and old seawalls at the site. The volume of these dumped materials, which was estimated based on information from the available archival records for the

demolished old breakwater and old seawalls at the site, is taken as the basis for quantifying the volume of large size AHM excavation for baseline purpose.

2.2 Potential Stability Issues for Temporary Cut Slopes Related to the Presence of Soft Marine Mud –

Based on the available GI information, it is considered that a significant part of the soft materials which may affect the stability of the temporary cut slope is within the marine deposit. On the other hand, it is considered that the top levels and base levels of the marine deposit would affect the stability of the temporary slopes. Using the profiles of the estimated top and base levels of the marine deposit and the profiles of the temporary slopes, one may also be able to estimate the volume of marine deposit within the mass of the slopes. This will be useful information for estimating the amount of ground treatment works (if required) at the marine deposit.

Considering the above, the baseline approach is to give a highest top levels profile and a lowest base levels profile to the marine deposit within the mass of the defined temporary slopes, based on the inferred top levels profile and base levels profile of marine deposit determined from the available boreholes information.

2.3 Potential Disposal Issues for the Excavated Marine Deposit –

It is considered that contaminations are present at the marine deposit. The contaminations induce potential disposal issue of the excavated marine deposit, which could cause impact to the cost and time of the construction. The baseline approach for this matter is to have the volume of marine deposits as determined from the top and base profiles of marine deposits being increased by 10% to be the baseline volume of marine deposits with respect to disposal of marine deposits off site.

3 CASE 2 – DIAPHRAGM WALL CONSTRUCTION FOR AN UNDERGROUND STRUCTURE

The main geotechnical works of Case 2 consist of construction of permanent diaphragm wall for an underground structure, ELS as well as site formation works for various elements of the project. Available GI and published geological information suggest that the site is likely be intersected by several faults and the rockhead of the site is anticipated to be undulating. The proposed diaphragm walls were designed to be founded on rocks of different allowable bearing pressure depending on the nature of the walls and the loading to be sustained. Accordingly two definitions of rockheads were given in accordance with the design requirements and relevant codes.

3.1 Founding level

The rockhead profiles and depths will affect the panel depths and hence the cost and time for the construction. With consideration to the arrangement of diaphragm walls of different required bearing capacities and the variation of rockhead across the site, the baseline approach is to review panel by panel and demarcate zones with baseline on ranges of elevation of the two inferred rockheads.

3.2 Excavation within diaphragm walls and underground structure

The GI information indicates that natural boulders, corestones and zones of rocks which do not meet the relevant criteria of bedrock for founding levels are present above the relevant rockheads and formation level at parts of the site. These materials will impose potential excavation difficulties and may require non-conventional tools to excavate. The baseline volumes of rock excavation for the diaphragm walls and within the proposed underground structure have been given separately in according to the preamble and method of measurement of the tender.

For rock excavation within diaphragm walls, the baseline approach is to give a maximum value to the volume (in percentage to the total length of whole diaphragm wall) of rock for excavation above the relevant inferred rockhead within the diaphragm walls. This value of rock excavation includes boulders within the natural superficial deposits, rock in grades better than IV and corestones within insitu materials located above the relevant inferred rockhead or founding levels of the diaphragm wall panels.

The baseline volume of rock for bulk excavation within the deep excavation site includes boulders within the natural superficial deposits, rock in grades better than IV and corestones within insitu soil located above the final excavation levels.

The contents of boulders within natural superficial deposit, corestones within the insitu materials and rock in grades better than IV above the inferred rockhead/final excavation levels were baselined as the overall volume of rock for different types of excavation. In achieving the estimates, available boreholes in the vicinity of the site have been reviewed individually and the percentage of corestone and rock in grades better than IV were recorded in respect to the founding levels of the nearest diaphragm, relevant defined rockhead and/or the nearby final excavation level.

4 CASE 3 – BORED PILES CONSTRUCTION OF A BRIDE

Case 3 involves the construction of a bridge which was proposed to be supported by large diameter bored piles. Verification of the ground conditions to provide competent foundations for piles is critical to foundation design for the bridge. Factors such as strength, quantity and extent of unsuitable rock, corestones, boulders, and AHM are likely to affect the bored pile foundation works and subsequently the time and cost of construction.

According to the findings of previous ground investigation works and geological publications, the proposed bridge will be constructed over a zone where complex geology, including pockets of marble with cavities and deeply weathered fault zones, has been proved and the alignment will likely be intersected by several major faults. One of the challenges for preparation of the GBR is that along the alignment no difficult ground condition was identified by the project specific boreholes and geophysical surveys. However, it is considered adverse geological features such as faults, highly weather joints, corestones may exists and can significantly affect the construction works. Besides, other unfavourable geological features such as calcareous meta-sedimentary rocks, karst deposits, cavity deposits etc. may exist and impose significant adverse impact on the foundation works. Therefore, it is considered necessary to include a GBR in the tender in one way as a representation of what is assumed will be encountered for the purpose of defining the indications of the contract and in the other way to provide a platform for sharing of risks between the client and the contractor. The following items were baselined in the GBR: a) Top level of Completely and Highly Decomposed Rock; b) Level of foundation rockhead; and c) corestones.

Considering the above, the baseline approach is to determine the profiles of the top levels of the saprolite and foundation rockhead along the alignment based on the available boreholes information and to baseline the profiles with allowance of variations above or below the baselined profiles. For the corestone issue, a percentage of rock within the insitu materials has been estimated by reviewing the content of corestones encountered by project specific and archival borehole data.

5 CONCLUSION

The baselines approaches for key geotechnical issues may vary site by site due to different types of critical ground conditions, construction types, construction methods and site constraints for different sites. Despite this, experiences from previous studies may help ones to inspire appropriate baseline approaches for ones' own cases and avoid the drawbacks made by others in previous cases. Different baseline approaches may be adopted in different previous studies for similar constructions at similar ground conditions due to, for example, site specific constraints, quality and quantity of the GI information and preferences of the clients on the baseline approaches. No matter what types of baseline approaches, the clients should be properly briefed so that they can fully understand what sort of risks they are sharing with the contractors. Although GBRs have been adopted in tunnelling related projects for a while in Hong Kong, it is still relatively new in deep excavation and foundation projects. However, with more GBRs being prepared for and adopted at especially large scale deep excavation and foundation projects, it is anticipated that the application of GBRs on deep excavation and foundation projects will be more common and better understood..

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Advanced 3D Modelling for Interactions of Landslide Debris and Flexible Barriers

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ABSTRACT

Hong Kong has a high proportion of land area that consists of relatively steep natural terrain. This land area is susceptible to landslides that can travel long distance. Inevitably housing development in the vicinity of these natural hillsides is subjected to such natural terrain hazards. Flexible barriers could be a feasible option for mitigating natural landslide risk. However, there still lacks a comprehensive and recognised design standard for flexible debris-resisting barriers.

The development of such design standard and appropriate design methodology for flexible barriers requires an understanding of both landslide dynamics and the interaction of landslide debris and flexible barriers. Driven by the initiative of the Geotechnical Engineering Office (GEO), Civil Engineering and Development Department, Arup is undertaking a pilot numerical investigation of the debris/flexible barriers interaction. Such modelling work is technically challenging as it calls for coupling of debris mobility analysis and structural assessments of flexible barriers. In the investigation, the advanced multi-purpose finite element program, LS-DYNA, has been used to carry out the coupled analysis.

This paper describes the modelling methodology and the simulation technique adopted for the investigation.

1 INTRODUCTION

1.1 Purpose of the Investigation

Numerical simulations of debris mobility have been successfully carried out in the past both locally and in the overseas (Sun, 2007). Similarly, various attempts in the academia as well as in the industry have also been made in the structural modelling of the flexible barrier behaviour under impact such as rock fall or debris flow, for example by Volkwein (2004) and Zhou et al. (2011). However, the coupled modelling of both remains a challenge and has yet to be done. The purpose of this investigation is to propose an appropriate modelling methodology for such coupled analysis. Laboratory experiments, field tests and case histories have been used to verify the validity of the proposed simulation technique. It is hoped that new insights and findings will be obtained through this investigation, and hence assists in the development of the design standard for flexible barriers against landslide debris.

1.2 Framework of the Investigation

The investigation is divided into different stages, with each stage targeting the development of the modelling technique for a specific aspect of the debris-structure interaction analysis. This approach allows for the review of interim results and necessary adjustments on the strategy of the investigation for subsequent stages. The stages are as follows:

i. Numerical simulation of debris mobility;
ii. Structural modelling of the flexible barrier;
iii. Interaction of landslide debris and flexible barrier;
iv. Parametric study of different sizes and velocities of debris flow impacting a flexible barrier.

With the anticipation of highly three-dimensional and complex debris-structure interaction as well as the non-linear response of the structural components of the flexible barrier, the multi-purpose finite element program LS-DYNA has been chosen as the numerical package for the investigation. The program is frequently used for the solving of complex real world problems such as automotive crash analysis, metal forming, drop testing, seismic analysis and blast loading. It has also been used extensively in civil engineering related design and analysis, including rail, civil, structural, wind and vibration engineering.

2 DEBRIS MOBILITY SIMULATION

2.1 Benchmark Cases

Benchmark cases have been selected for the verification of the developed debris mobility simulation technique. The simulation of those cases enables the identification of the appropriate numerical techniques, constitutive models and rheological parameters for modelling debris mobility for the investigation. The cases chosen include laboratory flume tests as well as previous debris flow cases in Hong Kong, and are as follows:

i. The experimental tests on granular avalanches across irregular three-dimensional terrains reported by Iverson et al. (2004);
ii. 2008 Yu Tung Road debris flow; and
iii. 2005 Kwun Yum Shan landslide.

2.2 Simulation Technique

All the benchmark cases were simulated using similar numerical techniques. 3D topography was generated using the available data and represented using rigid shell elements. Vertical rigid shell elements were used to model the head gate or the confining tube for retaining the initial debris mass at its source location before the triggering of the debris flow. At specified time, the head gate or the confining tube was prescribed with a displacement such that it moved away from the debris mass. This allowed the debris mass to freely slide down the slope (Figure 1).

The debris material has been modelled in LS-DYNA as an Arbitrary Lagrangian-Eulerian (ALE) material. It adopted an adaptive meshing technique such that the large deformation nature of the debris material can be simulated accurately. The internal strength of the debris material was specified by assigning a set of apparent total stress strength parameters in terms of friction angle and cohesion. The Drucker-Prager yield criteria were adopted. The assignment of the strength parameters were directly taken from the data provided for the case of experimental tests, or determined by repeating the tests with different values until the result matched the observed flow characteristics and flow speed.

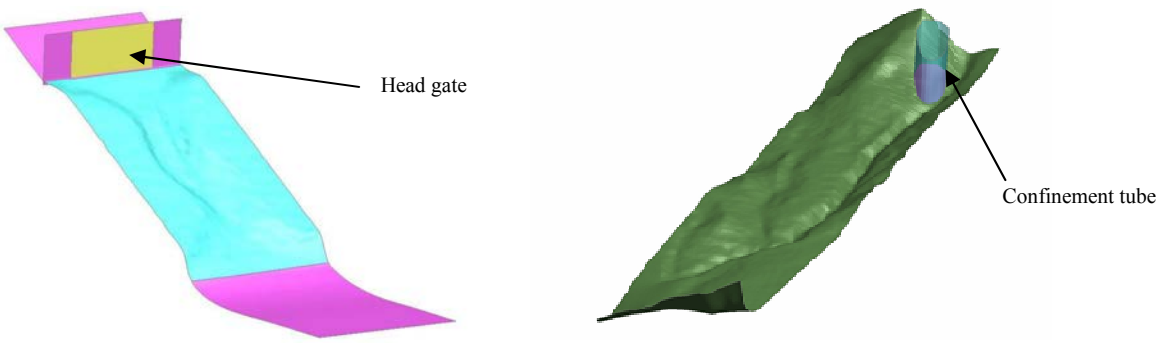


Figure 1: 3D Topography and the head gate / confinement tube developed -
Left: for the experimental tests by Iverson et al. (2004); Right: 2008 Yu Tung Road debris flow

The flow resistance was first assumed to be pure friction and developed from the basal contact resistance between the debris and the topography. Friction coefficients were assigned to the topography based on the

available information or observation. Other rheology models, such as resistance terms analogous to the Voellmy turbulent coefficient, were also developed to investigate the appropriateness of using different rheological parameters for simulation of local debris flow cases.

For the simulation of the previous debris flow cases in Hong Kong, entrainment and secondary slides which were not included in the model.

2.3 Simulation Results

Figures 2 provides a side-by-side comparison of the LS-DYNA simulation results on the experimental tests and the vertical orthophotographs extracted from Iverson et al (2004) for Experiments A. The displacement of the granular avalanche front as obtained from the experiment and the LS-DYNA simulation is also compared.

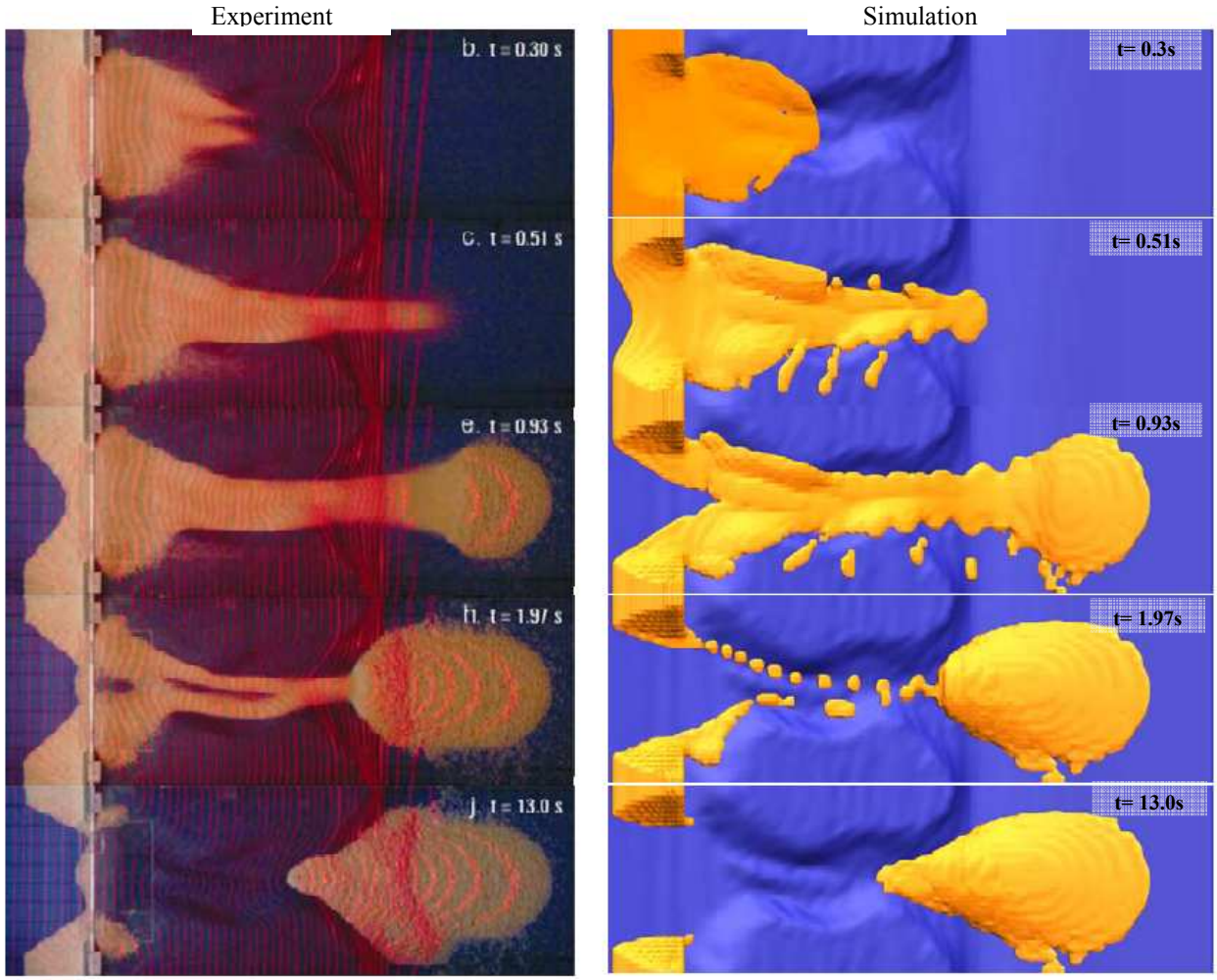


Figure 2: Side-by-side comparison of the flume test experiment result
(extracted from Iverson et al., 2004 and the LS-DYNA simulation for Experiment A)

Figure 3 shows the frontal displacement against time plot for various combinations of debris internal friction angles, basal friction angles and turbulent coefficients (if used) for the 2008 Yu Tung Road debris flow. From the available video recording, it was evident that the debris travelled from chainage 320 m to 530 m in approximately 20 to 21 seconds, which was equivalent to an average velocity of about 10 m/s. These two chainages have been highlighted in the plot for reference. It can be seen, that adopting pure frictional rheology with internal friction angle of 1.25° and contact friction of 25°, as well as the rheology with a Voellmy-like resistance term using internal friction angle of 1.25°, contact friction of 11° and turbulent coefficient of 500 provided the best estimation of the debris mobility among all runs in terms of the average velocity moving

from chainage 320 m and 530 m. However, it is noted that the latter combination provided a more accurate estimate on the distance between the front and the tail of the landslide debris when it reached approximately chainage 520 (Figure 4).

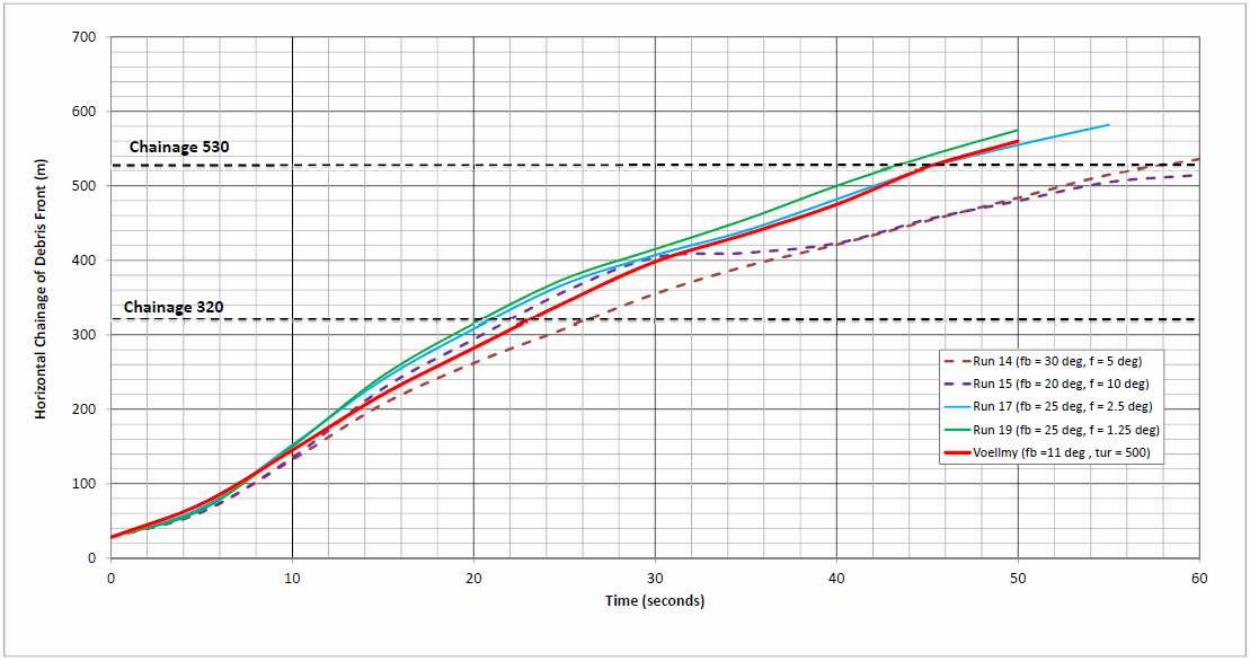


Figure 3: Frontal displacement vs. time plot

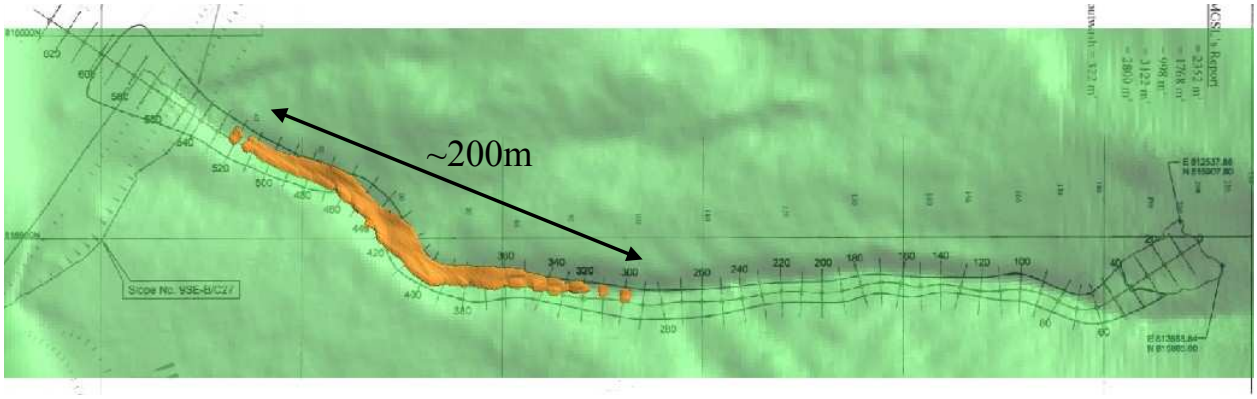


Figure 4: Plan view showing the distance between front and tail of the landslide debris estimated using Voellmy-like rheology in LS-DYNA

3 MODELLING OF FLEXIBLE BARRIERS

3.1 Benchmark Cases

A benchmarking exercise has been carried out based on the field tests conducted by the Swiss Institute for Snow and Avalanche Research (SLF Davos) on flexible barriers, originally reported by Grassl (2002) and later in the PhD thesis on Numerical Simulation of Flexible Rockfall Protection Systems by Volkwein (2004). Different rockfall scenarios were tested in the filed tests. To simulate the rockfall scenarios, an 820 mm diameter ball weighed 825kg made of steel fibre reinforced concrete was adopted as the weight. Tests were performed with the concrete ball dropping at different heights (16 m & 32 m) on to the test rigs where a flexible barrier was installed to (Plate 1).

Plate 1: The specifically-built test rig for rock fall tests (Extracted from Volkwein, 2004)

3.2 Simulation Technique

In the LS-DYNA simulation, all structural components of the flexible barrier were modelled explicitly using beam elements. Non-linear load-deformation relationship curves were specified for some of the structural components such as the steel rings and brake rings that exhibited highly non-linear behaviours. A friction coefficient of $\mu = 0.1$ was adopted for all material contacts.

The model was initialised by applying the gravity load to the test rigs, structural components of the flexible barrier, as well as the concrete ball. The concrete ball was then allowed to fall freely and impact on the flexible barrier. The position, velocity and potential energy of the concrete ball were tracked and recorded. The deformation of the ring net and the movement of the steel cables were calculated. In addition, the tension forces developed within the steel cables were tracked and recorded for comparison with the test data.

3.3 Simulation Results

The visual illustration in the LS-DYNA simulation for the deformation of the flexible barrier during the impact of the rockfall is shown in Figure 5. It can be seen that the explicit modelling of the structural components in LS-DYNA, for example, steel rings and shackles, enables the accurate prediction on the movement of the ring net and the plastic deformation of rings. The extension and sideways movement of the steel cables have also been predicted accurately being very similar to the actual tested barrier. The position and total energy of the ball respect to time were compared between the LS-DYNA simulation and the rockfall field tests. These results for the cases of 16 m drop without brake rings and 16 m / 32 m drop with brake rings are shown in Figures 6 to 9, illustrating the accuracy of LS-DYNA simulation.

4 INTERACTIONS OF LANDSLIDE DEBRIS AND THE FLEXIBLE BARRIER

4.1 Benchmark Cases

The debris flow occurred at the test site in Illgraben, Switzerland was selected as the benchmarking case for the verification of the simulation of debris-structural interaction. A full-scale flexible barrier (Plate 2) was installed at the test site with various instrumentation devices along the debris flow track such as geophones, videocams, force plate and load cells. Data including flow depth, debris density, and tension developed in the barrier ropes were recorded during a number of debris flow cases.

4.2 Simulation Technique

The technique developed for simulating debris mobility and modelling the flexible barrier described in Section 2.2 and 3.2 were adopted for the simulation of the benchmark case. Since each ring of the steel net was modelled explicitly with beam elements, it was possible for the debris to pass through the net. However, it was observed in the real case most of the debris would be retained behind the net. In order to ensure no leakage of

debris during the impact, a special no-strength membrane was developed and covered the steel net (Figure 10), such that the debris impact pressure is probably captured and transferred to the flexible barrier structure.

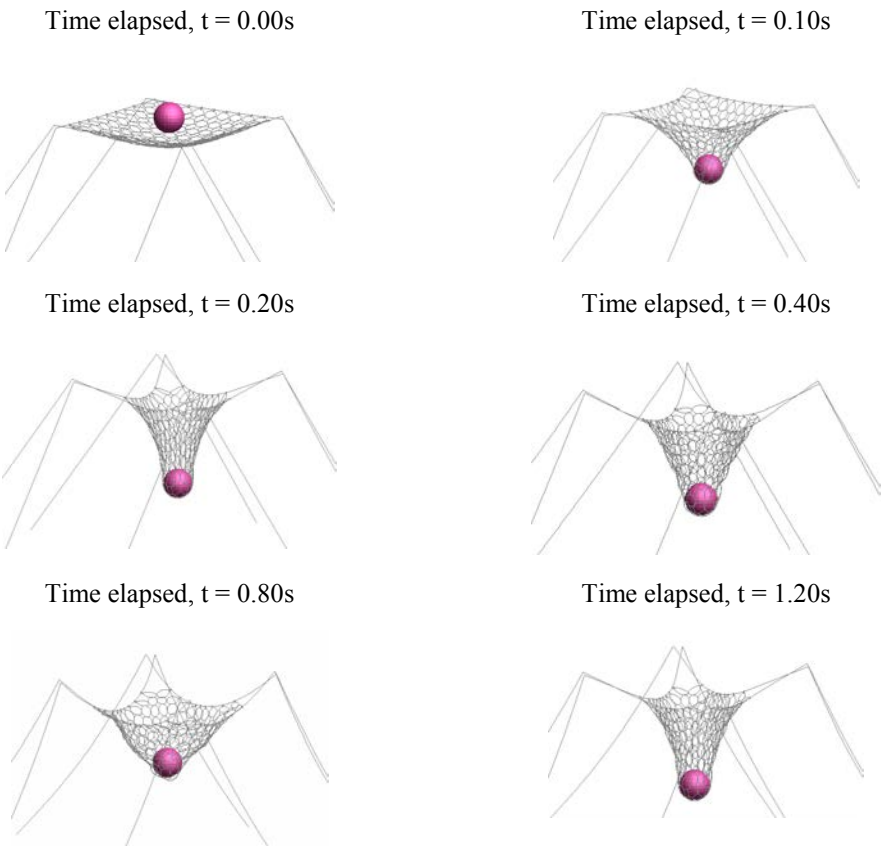
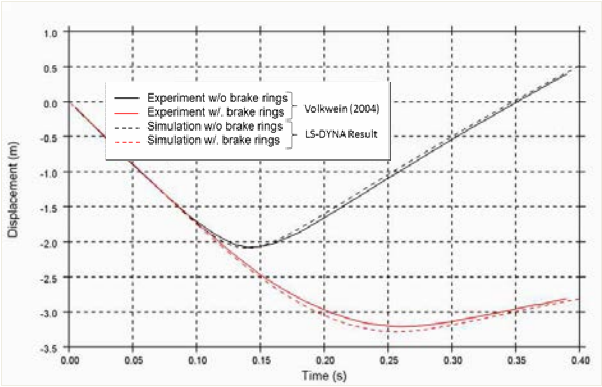


Figure 5: Simulated deformation of the flexible barrier during the 16 m drop rock fall test with brake rings (steel frame of the test rig not shown for clarity) simulation



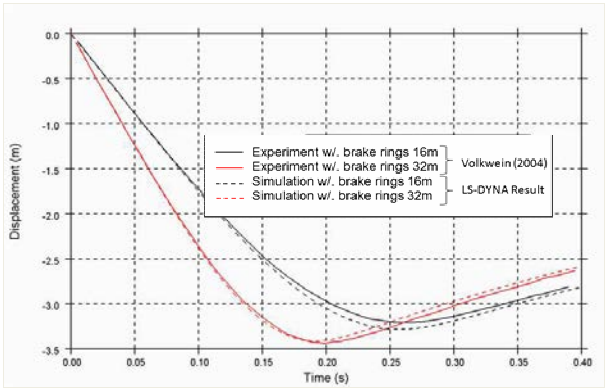
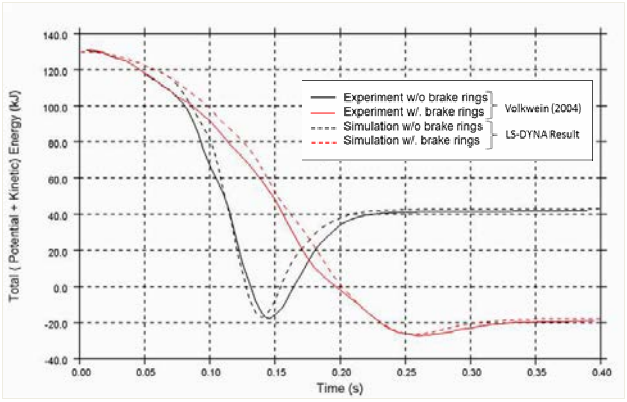


Figure 6 (left): Displacement-time curves of 16 m drop case (both with and without brake rings)

Figure 7 (right): Displacement-time curves of 16 m and 32 m drop cases (both with brake rings)



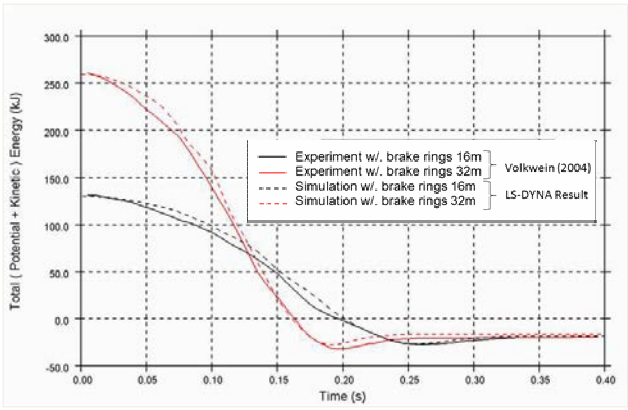


Figure 8 (left): Total energy - time curves of 16 m drop case (both with and without brake rings)

Figure 9 (right): Total energy - time curves of 16 m and 32 m drop cases (both with brake rings)

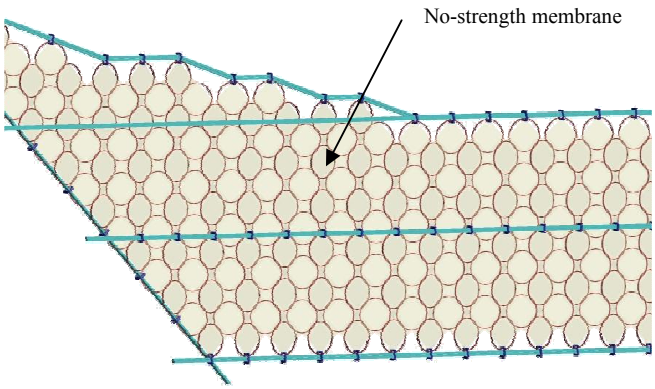


Plate 2 (left): A full scale flexible barrier installed at Illgraben, Switzerland (extracted from Geobrugg)

Figure 10 (right): Modelling of no-strength membrane covering the flexible barrier steel net

On the other hand, considering the extensive length of the debris flow track in the Illgraben test site, it would be impossible to model the entire length in the simulation. Therefore, it was decided only a short length of the track behind the flexible barrier was modelled. To compensate for not providing the required running length for the debris to obtain the correct speed and hence the impact energy right behind the flexible barrier, the debris was introduced in the model continuously with an adjustable initial velocity.

4.3 Simulation Results

At the time of writing this paper, the LS-DYNA simulation and the verification of the modelling technique is in progress. Interim results showed that the technique of covering the flexible barrier with a no-strength membrane was able to prevent the leakage of debris material out of the steel net of the flexible barrier. Furthermore, it was able to transfer the impact pressure on to the flexible barrier structure. By adjusting the initial velocity of the debris flow and the internal shear strength of the debris material, the simulation was also able to replicate the run-up and pile-up behaviour of the debris deposition mechanism behind the flexible barrier (Figure 11).

Despite the plausible interim results, further work will be required, including the comparison of the simulation results against the instrumentation measurement available for the Illgraben test site, to fully

validate the appropriateness and accuracy of the proposed modelling technique for the interaction of the debris material and the flexible barrier structure.

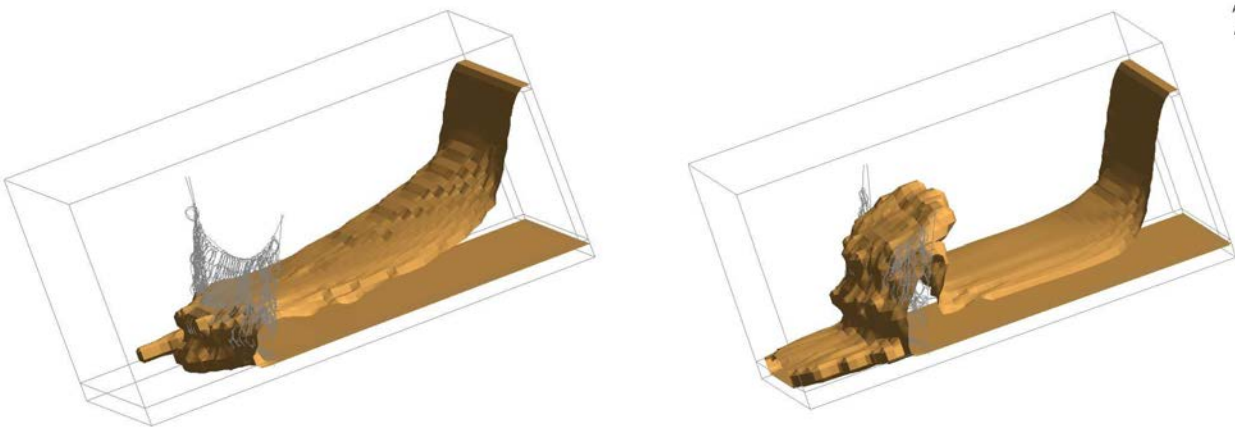


Figure 11: Interim simulation of the interaction of landslide debris and the flexible barrier -
left: thick debris (high internal friction angle) exhibits run-up deposition mechanism behind the barrier
Right: lean debris (low internal friction angle) exhibits pile-up deposition mechanism behind the barrier

* Some debris escaped from the bottom of the barrier as the steel net was slightly lifted up from the ground during the impact

5 CONCLUSION

The investigation described in this paper has explored the appropriate numerical analysis technique to investigate the interaction of landslide debris and flexible barrier. Selected benchmark cases have been adopted to verify the accuracy of the methodology and technique developed. Although the investigation is still in progress, it has demonstrated that advanced 3D modelling could be used to study the behaviour of the debris flow and the load-deformation characteristics of flexible barriers, in a way that previously had not been done by other less sophisticated analyses. It will enable the better understanding of the debris resisting mechanism using flexible barriers, and the development of a rational design method for such natural terrain hazard mitigation systems.

ACKNOWLEDGEMENTS

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Slope Maintenance in Public Housing Estates – Prevention is Better than Cure

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ABSTRACT

As a significant proportion (some 30 per cent) of Hong Kong's population lives in public housing, the safety of slopes located within public housing estates has all along been a prime concern for the Housing Department (HD), the executive arm of the Hong Kong Housing Authority (HKHA). The Department subscribes to implementing comprehensive maintenance measures to ensure slope safety. Through procurement of the geotechnical and maintenance services of Slope Maintenance Term Consultancies (SMTC), slope inspections are conducted regularly via Annual Geotechnical Inspection (AGI) programmes, assessments of slope stability are undertaken from time to time and assessments of possible impacts arising from buried water-carrying service leakage are carried out as when necessary. Regarding AGI, the HD has adopted stringent standards in its AGI programmes under which slope inspection frequencies meet, and even transcend, the requirements specified in Geoguide 5. Additional to slope inspections, slope upgrading works are undertaken where appropriate.

Consequent upon implementation of the holistic preventive measures as outlined above, landslides in public housing estates have become a rarity since 2000. Other than reaping the benefit of slope safety, greening coming alongside upgrading works to slopes has helped to enhance the living environment of many public housing estates. This paper aims to share the practical experience of slope maintenance in public housing estates with respect to slope inspection, slope maintenance and upgrading, and improvements to slope appearance.

1 INTRODUCTION

In the 1950s, the rapid growth in Hong Kong's population saw an expansion of urban development that involved extensive building works on hillsides. To cope with the development, many hillsides were cut and filled. At the same time, before the 1970s there were many squatters built on steep hillsides, most of which comprised flimsy huts with unscrupulous slope cuttings and inadequate drainage systems. In the 1960s, there were hardly any statutory mechanisms to control slope formation works. Nearly all the slopes at that time were not subjected to any rigorous geotechnical investigation and design checking.

It was not until the occurrence of some fatal landslides, e.g. the landslides that happened in the former Sau Mau Ping Estate in 1972 and 1976, that both the government and the general public became more aware of the importance of putting in place maintenance measures to achieve slope safety. Against this background, the Geotechnical Control Office (GCO) was established in 1977 to exercise geotechnical control over private and public works so as to ensure that all new slopes are constructed to high safety standards. In 1991, the Office re-titled itself as the Geotechnical Engineering Office (GEO) to reflect more adequately the various engineering functions delivered. After a deadly landslide that struck Kwun Lung Lau (an estate built and managed by the Hong Kong Housing Society) in 1994, it was realized that enhancement of regular slope maintenance to include leakage detection for buried water-carrying services plays a vital role in maintaining the good performance of slopes.

The huge and ever-expanding public housing population and the large amount of slopes located in public housing estates have prompted the HD to adopt a "prevention is better than cure" strategy to guard against slope failure which may otherwise lead to loss of life and property. Since 1995, the Department has established a dedicated team – currently named as the Geotechnical Management Unit (GEMU) – whose task

is to design and implement proactive maintenance programmes to ensure the safety performance of all slopes in public housing estates.

According to HD's Slope Record 2012, there are totally some 1,500 slopes scattered in 126 estates. Based on the geographical distribution of these slopes, the Department has divided the territory into four maintenance regions (namely Regions A to D) and approximate equal numbers of slopes are maintained by each region as illustrated in Figures 1 and 2 below:

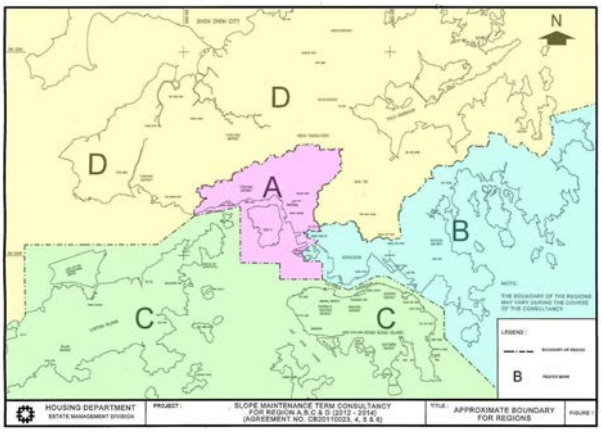

Figure 1: Four maintenance regions (Regions A to D)

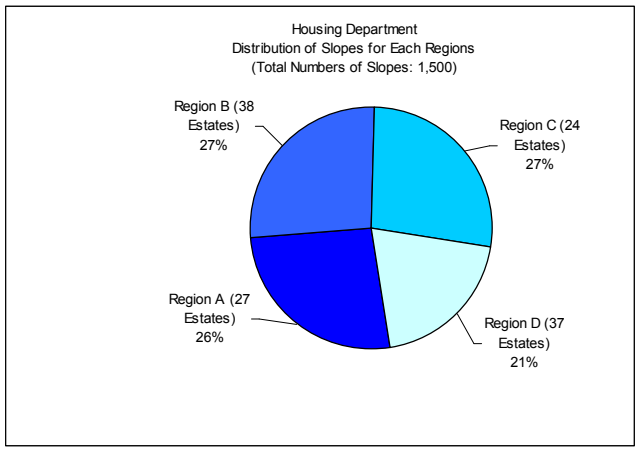

Figure 2: Distribution of slopes in each maintenance region

To help monitor the performance of slopes within its estates, the HKHA has engaged geotechnical consultants to provide professional services to the GEMU via four SMTC. Under the current practice, each SMTC looks after one Region. The general scope of services for each geotechnical consultancy includes regular inspection of slopes according to the AGI Programme, emergency engineer inspection for the landslide incidents; information search; slope database management; assessment of existing stability of slopes and retaining walls; detailed design of upgrading works; leakage detection for slopes, retaining walls and buried water-carrying services within public housing estate boundary; acting as the Contract Manager for the HKHA's Term Contract for Slope Maintenance and Improvement Works (TCMW), etc.

The vast number of public housing estates and the great number of people residing therein (some two million as at March 2012) have presented a need for the HKHA to conduct rigorous slope inspections to assess the performance of slopes. These inspections are generally carried out via three tiers as shown in Table 1 below:

Table 1: Three tiers of regular slope maintenance in public housing estates

Type of Inspection	Frequency of Inspection	
	Consequence-to-life Category 1	Consequence-to-life Category 2 or 3
Annual Geotechnical Inspection by STMC	Once every year	Once every three years
Routine Maintenance Inspection by HD's building maintenance staff	At least three times each year and immediately after the lowering of a "Black" or "Red" rainstorm warning.	At least three times each year and immediately after the lowering of a "Black" or "Red" rainstorm warning.
Daily Inspection by HD's estate management staff	Daily	Daily

2 ANNUAL GEOTECHNICAL INSPECTION

In recognition of the need to tackle slope maintenance and detailing, the HKHA has set up a unique and comprehensive maintenance programme implemented through the SMTC. Under this programme, geotechnical consultants are required to identify not only signs of distress during the AGI, but also changes that can affect stability with due regard to the assumptions made in the design of the slope or retaining structures. Examples of adverse changes are additional surcharge imposed by new developments, diversion of a watercourse towards the slope or retaining structures, or an increase in height or gradient of the slope or retaining structures.

To ensure slope safety, the HKHA has set up a high-standard slope maintenance guideline for compliance by the SMTC. According to this guideline, the extent of AGI includes surface inspection, abnormal seepage inspection and attendance to other safety matters. Since many slopes are located in densely populated public housing estates, the risk to life and economy is high and thus the frequencies of inspection for these slopes have been made higher than the requirements stipulated in Geoguide 5. For slopes falling within consequence-to-life category 1, the frequencies of AGI have been enhanced from five years (under Geoguide 5) to one year (under the HA's Slope Maintenance System). Similarly, for slopes under consequence-to-life category 2 or 3, the frequencies of inspection have been increased from five years to three years.

Experience gained from AGI reveals that surface drainage plays a major role in slope failures, particularly those shallow, small-scale failures. The main failure mechanism involves surface runoff leading to erosion or concentrated water ingress during intense rain. Inadequate slope maintenance generally takes the form of blocked or disrupted drainage channels, damaged hard surface cover (e.g. shotcrete) and blocked weepholes. These defects will cause washout failure to slopes due to overspill from blocked or damaged crest drains, infiltration into overstep slopes through bare or poorly maintained vegetated ground or cracked surface protective cover, build-up of water pressure behind hard surface cover with blocked weepholes, or leakage from poorly maintained buried water–carrying services. Therefore, visual inspection and subsequent maintenance recommendations should be directed principally towards measures that minimise the infiltration of surface water and scouring by surface water flow. Plates 1 and 2 illustrate how a slope with surface drainage differs "before" and "after" the implementation of maintenance works.

Plate 1: Slope no. 7SW-C/C1393 before maintenance

Plate 2: Slope no. 7SW-C/C1393 after maintenance

3 LEAKAGE DETECTION AND REPAIRING WORKS

Leakage from buried water-carrying services may erode the soil and create void underground. It may destroy the soil structure and the soundness of the ground. Unfortunately, failure of slope caused by the leakage of buried water-carrying services has no notable signs. Following the fatal landslide incident in Kwun Lung Lau in 1994, a program of direct monitoring and repairing of buried water-carrying services (which might otherwise impact on slope stability) was launched and developed by the government. Currently, the checking of buried water-carrying services are based on the "Code of Practice on Monitoring and Maintenance of Water-Carrying Services Affecting Slopes" published by the Environment, Transport and Works Bureau (ETWB) in November 2006, which provides guidance on the monitoring and maintenance of buried water-carrying services affecting slopes. Based on the guidelines concerned, the frequencies of monitoring required to be carried out to slopes are summarized in Table 2 below.

Table 2: Required frequencies of monitoring for buried water-carrying services

Slope Type / Utility Type	Cut Slope (Year)	Compacted Fill Slope (Year)	Fill Slope (Year)	Retaining Wall (Year)
Non-pressurized Drains	10	5	5	10
Pressurize Drains	5	5	5	5

Leakage detection and associated repairing works comprise necessary planning, reviews, investigations, studies, consultation, site inspections, co-ordination, assessment and design work. Since many years ago, the HKHA has been implementing leakage detection and repairing works for buried water-carrying services within public housing estates. The first CCTV inspection contract for underground drainage pipes was awarded in the early 1980s. After the Kwun Lung Lau landslide incident, another contract for leakage detection on buried water-carrying services affecting slopes was implemented. Between 1997 and 2000, leakage detection was conducted regularly under rolling Leakage Detection Term Contracts. Starting from 2002, leakage detection has been included as part of the essential works required from TCMW. Plates 3 and 4 illustrate how a buried water-carrying pipeline condition differs "before" and "after" the implementation of repairing works.

Plate 3: Buried water-carrying pipeline near slope no. 11NW-A/F280 before repairing

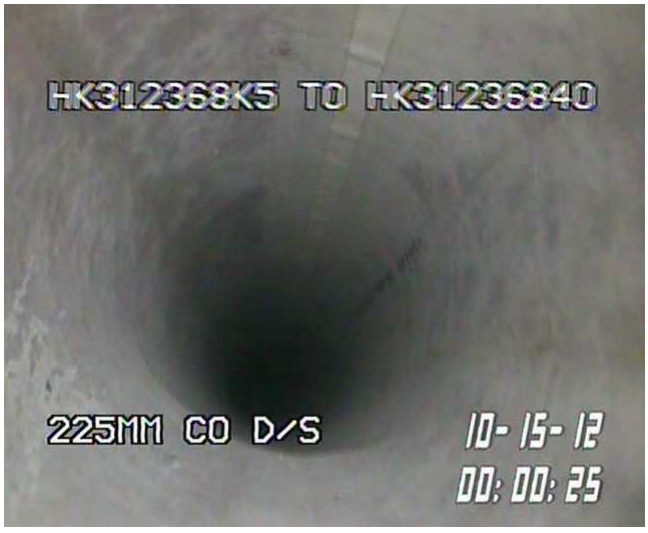

Plate 4: Buried water-carrying pipeline near slope no. 11NW-A/F280 after repairing

4 LANDSLIP PREVENTIVE AND MITIGATION MEASURES

In order to prevent, or to reduce the risk of landslides arising from slopes in public housing estates, the HKHA has introduced an Enhanced Maintenance Programme (EMP) whereby slopes are selected and prioritized for landslide preventive/mitigation works. The priorities of the slopes selected for upgrading are determined on the basis of: (i) Score achieved under the Combined New Priority Classification System, (ii) Consequence-to-life Category and (iii) Pre/Post GEO slopes.

The geotechnical consultants under STMC are permitted to gain necessary access to reviews, investigations, studies, consultation, site inspections, and co-ordination on the previous history and existing conditions of all slopes managed by the HKHA for the purpose of verifying slope information for the EMP. Such slope information is updated periodically and the priority ranking of slopes for landslide preventive/mitigation action is reviewed accordingly for endorsement by the HKHA.

When a slope is selected for implementation of landslide preventive/mitigation works, a stage study will be carried out by through the SMTC concerned. Stage study in general comprises ground investigation (GI), slope stability assessment and slope upgrading works design. If the geotechnical consultant decides to carry out GI works for getting more information about the underground conditions and soil design parameters, he/she will need to submit a GI plan to the Independent Checking Unit (ICU) of the HD in accordance with the Technical Memorandum for Supervision Plans and the Code of Practice for Site Supervision and other relevant ICU Instructions. Upon completion of the GI, the geotechnical consultant carries out rigorous slope stability assessment and works out a slope upgrading design for submission to the ICU for its approval. ICU's consent for commencement of the approved works shall then be sought. Upon obtaining such consent, slope upgrading works will be carried out via TCMW with site supervision being undertaken by the geotechnical consultant. After completion of the slope upgrading works concerned, as-built-drawings will be prepared and submitted to the ICU for application of a completion certificate (i.e. the GEO Checking Certificate in accordance with ETWB TCW No. 20/2004). Finally, a slope maintenance manual will be prepared by geotechnical consultant. Illustration of the difference of a slope before and after the implementation of upgrading works is shown via Plates 5 and 6.

Plate 5: Slope no. 11NW-A/CR702 before upgrading

Plate 6: Slope no. 11NW-A/CR702 after upgrading

5 SLOPE GREENING WORKS

Greening can be functioned as an urban lung to provide visual and psychological comfort and relief, which are vital to health and well-being of residents in the public housing estates. It can also improve the microclimate by absorbing heat and thus reducing the temperature of hard surface, and by enhancing the humidity. Vegetation cover on slope face can help moderate soil erosion and strengthen slope stability. It can also maintain the ecology of the area and conserve wildlife. The HKHA recognizes all these merits and carries out greening works to upgraded slopes where feasible in order to enhance the comfort and quality of public housing estates as well as their surrounding environment. The effect of slope greening is shown via Plates 7 and 8 below.

Plate 7: Slope no. 11NW-A/F206 before upgrading & greening

Plate 8: Slope no. 11NW-A/F206 after upgrading & greening

6 SITE ACCESS AND SAFETY PRECAUTIONS

In tackling the inaccessibility of some densely vegetated slopes in public housing estates, the HKHA adopts the guidance given via GEO Report No. 136 ("Guidelines on Safe Access for Slope Maintenance") on the provision and arrangement of access for slope maintenance that is safe for maintenance personnel, visually pleasing and where necessary, secured against trespassers, for slope maintenance works. In line with this guideline, proper maintenance ladders and fencing have been constructed along slope face and berms to facilitate routine slope maintenance inspections and minor routine slope maintenance works. Some illustrations in this regard are shown in Plates 9 and 10.

Plate 9: Slope no. 7SW-C/C153 before installation of hand-railing

Plate 10: Slope no. 7SW-C/C153 after installation of hand-railing

7 TREND FOR SLOPE MAINTENANCE IN PUBLIC HOUSING ESTATES

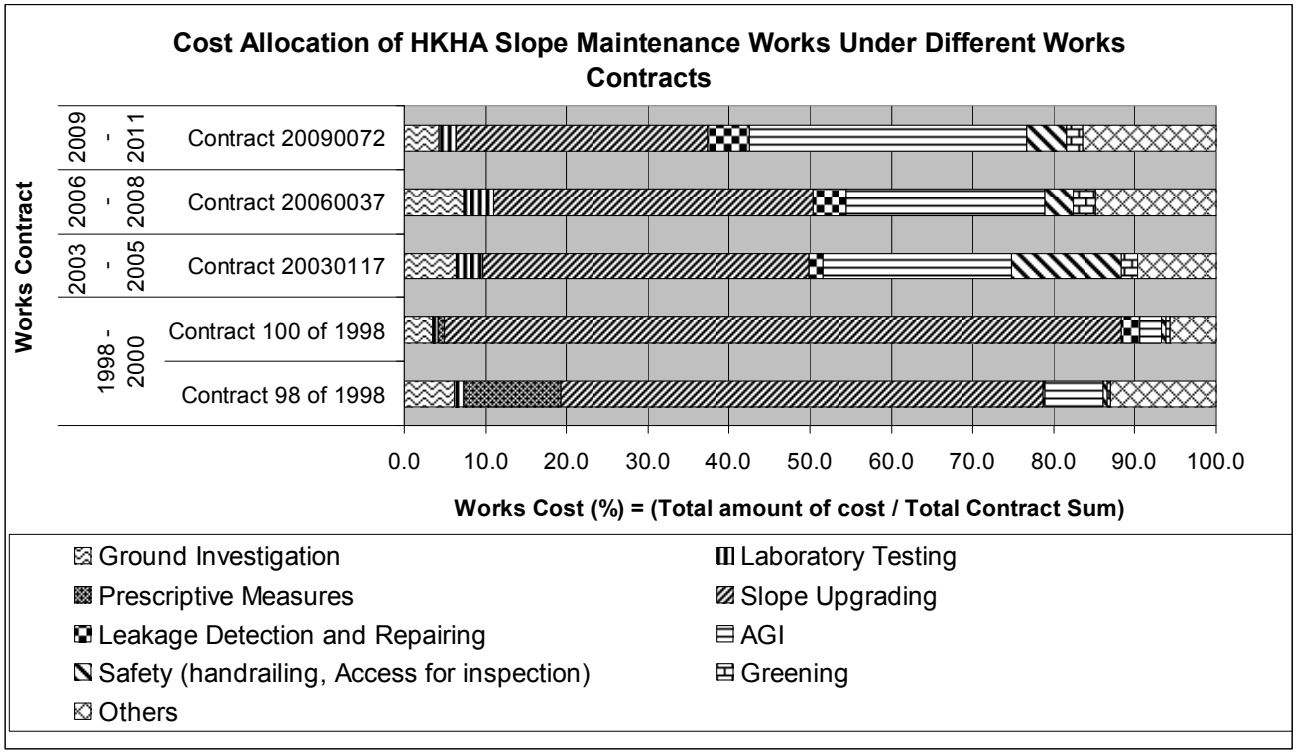

Figure 3: Percentage of expenditure for different TCMW

An examination of the cost allocation for HKHA slope maintenance works under different TCMW from 1998 to 2011, as shown in Figure 3 above, reveals that –

Before the 2000s,

(a) about 70 per cent of the expenditure on major works was related to slope upgrading;
(b) only 0.4 per cent to 0.6 per cent of the contract sum was spent on the provision of safety measures for future slope inspections and routine maintenance; and
(c) the expenditure on greening works accounted for only 0.5 per cent of the contract sum.

In the early 2000s,

(a) the expenditure on slope upgrading dropped from 70 per cent to 35 per cent of the contract sum;
(b) there was an increase in maintenance works expenditure to about 15 per cent of the contract sum for AGI;
(c) the expenditure on provision of site safety measures increased significantly from 0.5 per cent to 7 per cent of the contract sum; and
(d) there was also a rise in the expenditure on greening, from 0.5 per cent to 2.3 per cent of the contract sum.

The difference in expenditure patterns between the two periods as mentioned indicates that since the early 2000s, large amounts of inspected slopes in public housing estates have been found to be in satisfactory conditions and routine maintenance works have been effective in sustaining the safety standards set by the HKHA. There is, therefore, an obvious trend for the Authority to move towards enhancing the serviceability and visual appearance of its estate slopes and retaining walls in order to create and sustain a better living environment to the tenants.

8 CONCLUSION

To avoid risks to life posed by some 1,500 estate slopes in public housing estates, the HKHA and its geotechnical consultants work hand in hand to ensure the safety performance of these slopes through comprehensive maintenance programmes. Such efforts have borne fruit. On average less than one minor slope failure incident without any casualties occurred each year between 2000 and 2008, and since 2008, the situation has further improved to virtually zero incidents.

A review of HKHA's expenditures regarding TCMW shows that slope upgrading works have decreased, from about 70 per cent of the contract sum before the 2000s, to some 35 per cent in recent years. It is evident that as a result of the extensive slope improvement works carried out towards the end of the 1900s, many sub-standard slopes and retaining walls in public housing estates have been upgraded. And attributable to the rigorous and effective slope maintenance measures implemented since the mid-1990s, most slopes have been kept to satisfactory conditions, hence obviating the need for further upgrading works. As prevention is always better than cure, signs abound that the HKHA will continue to take maintenance as a vital strategy to safeguard the performance of slopes in public housing estates.

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The Characteristics of Cavernous Marble and Their Influence on Foundation Design in Hong Kong

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Fugro (Hong Kong) Limited

ABSTRACT

Housing development in Hong Kong has been very rapid in the past years. Many high-rise buildings have been built in the marble and marble clast-bearing volcanic rocks in Northwest New Territories. All these developments required detailed ground investigation to study the geological setting which in turn accumulated a wealth of geological data and extensive experience of deep foundation design. The karstic cavity of buried carbonate rock has been proven to have significant influence on geological models and foundation engineering. Since the lithology and geological structure is highly complex and most rocks are identified by naked eye, a considerable amount of rock identification has remained doubtful for a long time. The marble clast-bearing volcanic rocks have very often been described as sedimentary rocks for more than a decade. Up to the present, these descriptions in some geological investigation reports and publications still prevail. They may significantly affect the accuracy of the geological model and estimation on the extent of karst development underneath deep foundations. This paper presents the author's interpretation of the rock unit in light of new geological evidence.

1 INTRODUCTION

Since the discovery of marble in Fairview Park and Yuen Long in 1977, the author has consistently studied the characteristics of marble and related karstic cavity. Until now, the studies have mainly included carrying out geological mapping of the marble area in Northwest New Territories, examination of more than 10,000 drillhole logs and cores and collation of valuable geological data. The cavernous marble which would mainly impose problems to foundation engineering is layered marble. The lithic clast marbles are relatively minor effects. The layered marble and lithic clast marble have different engineering properties and varied extents of karstic development (Figure 1).

2 LAYERED MARBLE

The karstic cavities mainly occur in layered marble of Lower Carboniferous (Figure 2). The geological age is from 358.9 ±0.4 to 346.7±0.4 Ma. They are divided into two parts: the lower part is Long Ping Member composing of dark grey dolomitic marble with silicified marble and minor thin bedded siltstone which occurs at Yuen Long Estate, Long Ping Estate and Shui Bin Wai. The original rocks are interpreted as impure limestone including siliceous or clayey limestone, formed in a lagoon closed to shallow sea. These marble contains not only calcium and magnesium carbonate but also a considerable quantity of silicate and insoluble material. The insoluble materials make up 8% to 33% of the rock (Frost 1992). The depth of the marble is from -20mPD down to -150mPD. Karstic cavities in most boreholes are less than 1m high and the maximum recorded cavity height is 6.8m.

The upper part is Ma Tin Member comprising mainly white, massive, pure marble with some dark grey dolomitic marble. It occurs at Yuen Long, San Tin to north of Lok Ma Chau. In the pure marble the calcium carbonate content ranges from 93.62% to 96.43% and in the dolomitic marble the calcium and magnesium carbonates comprise over 99% of the rock (Langford *et al* 1989). This marble is derived from limestone formed in a shallow sea environment. The depth of the marble is from -8.3mPD to -90mPD. These marble are readily susceptible to dissolution. The marble generally characterized by highly to intensely karstic development. Most cavities intersected in boreholes vary in height from 1m to 2m, but the maximum recorded height of a cavity is 24.5m whilst the maximum proven length of a subterranean stream course is 45m. The

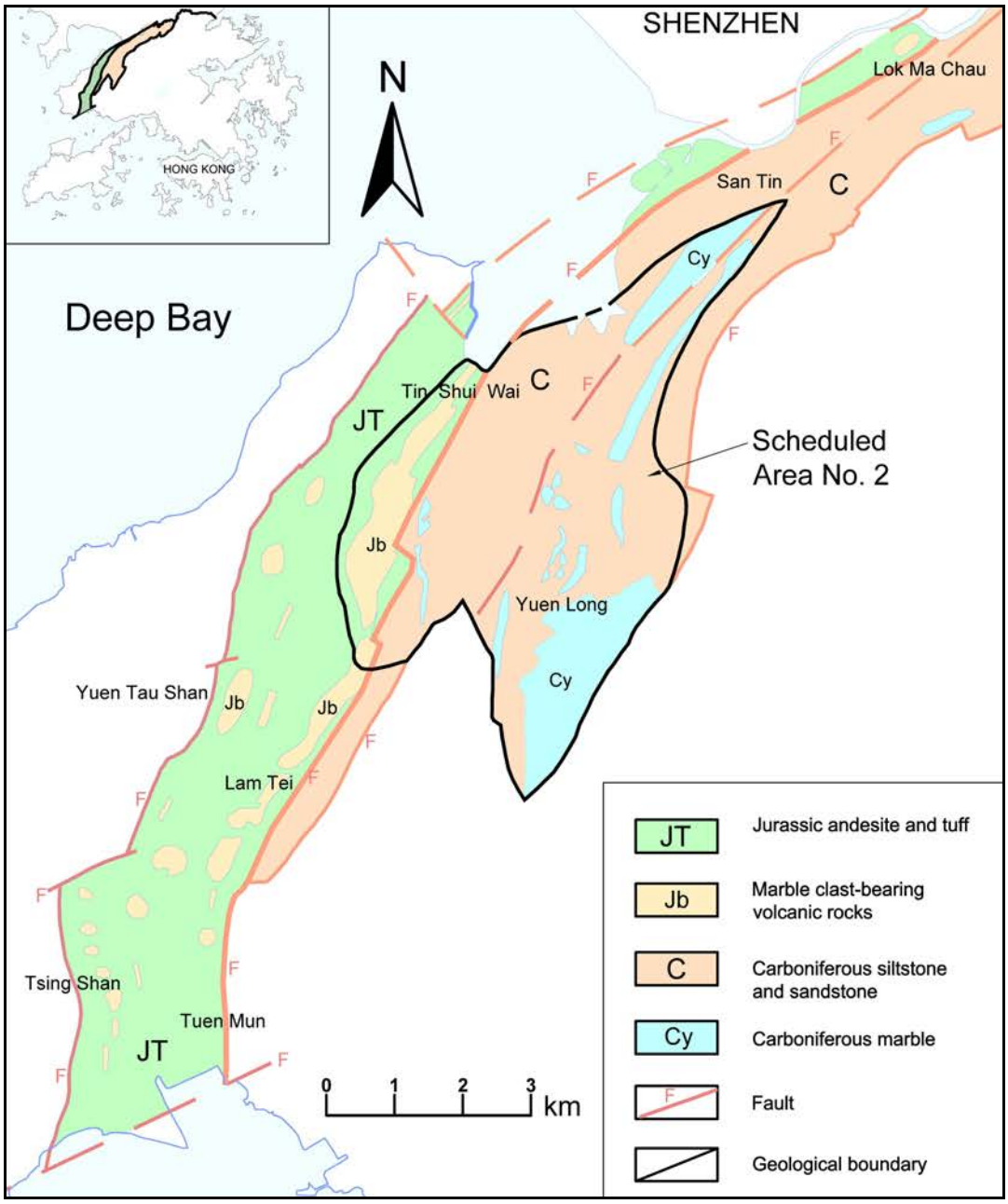

Figure 1: Distribution of Carboniferous marble and Jurassic marble clast-bearing volcanic rock

present of such karstic features can seriously affect foundation engineering.

The surface karst deposits are always exposed in the weathering zone of the layered marbles. It comprises dark brown, reddish brown to bright red silty clay with marble fragments varying from 1m to 20 m in thickness.

Generally, this deposit underlies the alluvium, but it has been previously described as debris flow deposits. Actually they are insoluble residues which, also termed terra rossas, are quite different from the marble clast-bearing volcanic rock because the latter cannot form these weathering products.

3 LITHIC CLAST MARBLE

This rock formation was formed by volcanic eruption in Jurassic age and the marble occurs as lithic clasts embedded in volcanic rock of the Tuen Mun Formation. It is present from Tuen Mun passing through Lok Ma Chau to Shenzhen (Figures 1 & 3). Most lithic clast marbles are buried under superficial deposits with only

some outcrops exposed in volcanic plug pinnacles, such as at Tsing Shan and Por Lo Shan. Some outcrops also occur in lava dykes such as at Yick Yuen Estate and Tin Shui Wai, and minor crop out in fallout deposits. The rock in the plugs and dykes presents as marble clast-bearing andesite or diatreme breccia. The plugs are elliptical or circular in shape and are 50m to 120m long and 20m to 50m wide. Crystal tuff and andesitic lava are present around the plugs. The lava dykes are 1m to 4m wide and 20m to hundreds metres long trending NNE and are parallel to each other. Other than marble, the lithic clasts also consist of andesitic lava, tuff, siltstone, sandstone and vein quartz. Generally, lithic clasts are subangular to subrounded varying from 10mm to 300mm across, occasionally up to 500mm. The contacts between the lava and marble clast always form altered rims. The relationship between the wall rock and the plug or dyke is an intrusive contact.

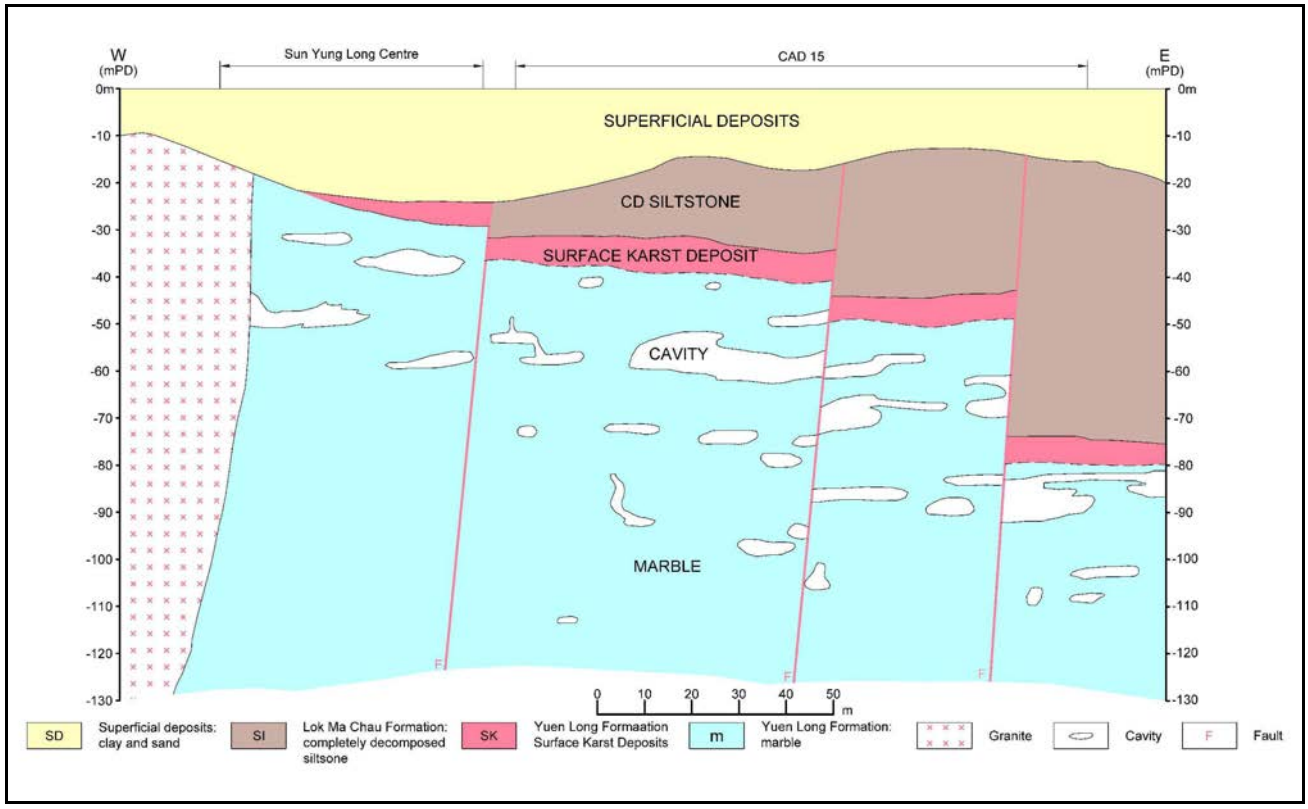

Figure 2: Geological section showing the layered marble with cavities of Yuen Long Formation at Yuen Long Town

Research results of thin section identification and chemical analysis of more than 40 rock samples, proved that the cement materials are andesite or dacite lava which are insoluble. Only the marble clasts are easily soluble but are not interconnected with each other, thereby restricting karst development to honeycomb weathering. Most cavities are less than 0.2m high, locally up to 0.5m. Large cavities are rare and only occur in fault zones. Since the lava and tuff are also susceptible to weathering, the rockhead level of sound rock can reach depths of -20mPD to -40mPD and may occasionally extend to -150m deep within fault zones.

Owing to rare outcrops and insufficient borehole data, the marble clast-bearing rock has not been fully understood by earlier geologists, such as Allen & Stephens (1971), Geotechnical Control Office (1988), Frost (1988) and Darigo (1988). Since mid-1989, 45 drillholes sunk in Tin Shui Wai and the surrounding area by GCO which confirmed the marble clast-bearing rock to be volcanic in origin. The rock type has since been revised and shown as Tuen Mun Formation in papers (Darigo 1990, Fig. 2), (Frost 1990, Fig. 2) and in the 1:20,000 scale solid geological map of San Tin (Geotechnical Engineering Office 1994). Subsequently, Irfan's study of rock strength properties also confirmed the volcanic origin (GCO 1990). After further detailed study, the Tsing Shan Formation has been merged into Tuen Mun Formation as shown in the 1:100,000 Scale Geological Map of Hong Kong (GEO compiled by Kirk *et al* 2000).

4 SOME RECOGNITION PROBLEMS IN MARBLE CLAST BEARING VOLCANIC ROCKS

4.1 Differentiation Between The Vent Breccia /Diatreme Breccia and Conglomerate / Breccia.

The marble clast-bearing rocks at Tsing Shan Monastery have been described as conglomerates and breccias. These rocks may have been formed following the collapse of a volcanic edifice, or by fluvial activity on the margin of a crater lake (GEO 2008). Meanwhile some authors suggested that the marble clast-bearing rock was mainly formed by redeposition within a volcaniclastic and/or epiclastic matrix, but no evidence was provided to substantiate epiclastic rocks. Fisher & Schmincke (1984) document that an epiclastic volcanic fragment is produced by weathering and erosion of volcanic rocks reworked in valleys, streams, lakes and beach environments. After detailed field observation and studying of data from 20 boreholes and more than 10 geochemical test and thin section identification results (Li 2010), it is suggested that the monastery is situated above a palaeovolcanic plug which is a 120m long and 50m wide raindrop-shaped structure composed of explosive breccia or diatreme breccia (Figures 4 to 6). Lithic clasts are subangular to subrounded, 10mm to 300mm across containing marble, sandstone, siltstone and lava fragments. The petrological identification and chemical analysis results confirm that the cement materials are basaltic andesite and dacite. The relationship between the plug and wall rocks are intrusive contact. The contact between marble clasts and andesitic lava formed skarnizated margin. Crystal ash tuff and andesitic lava around the plug form a stratovolcano.

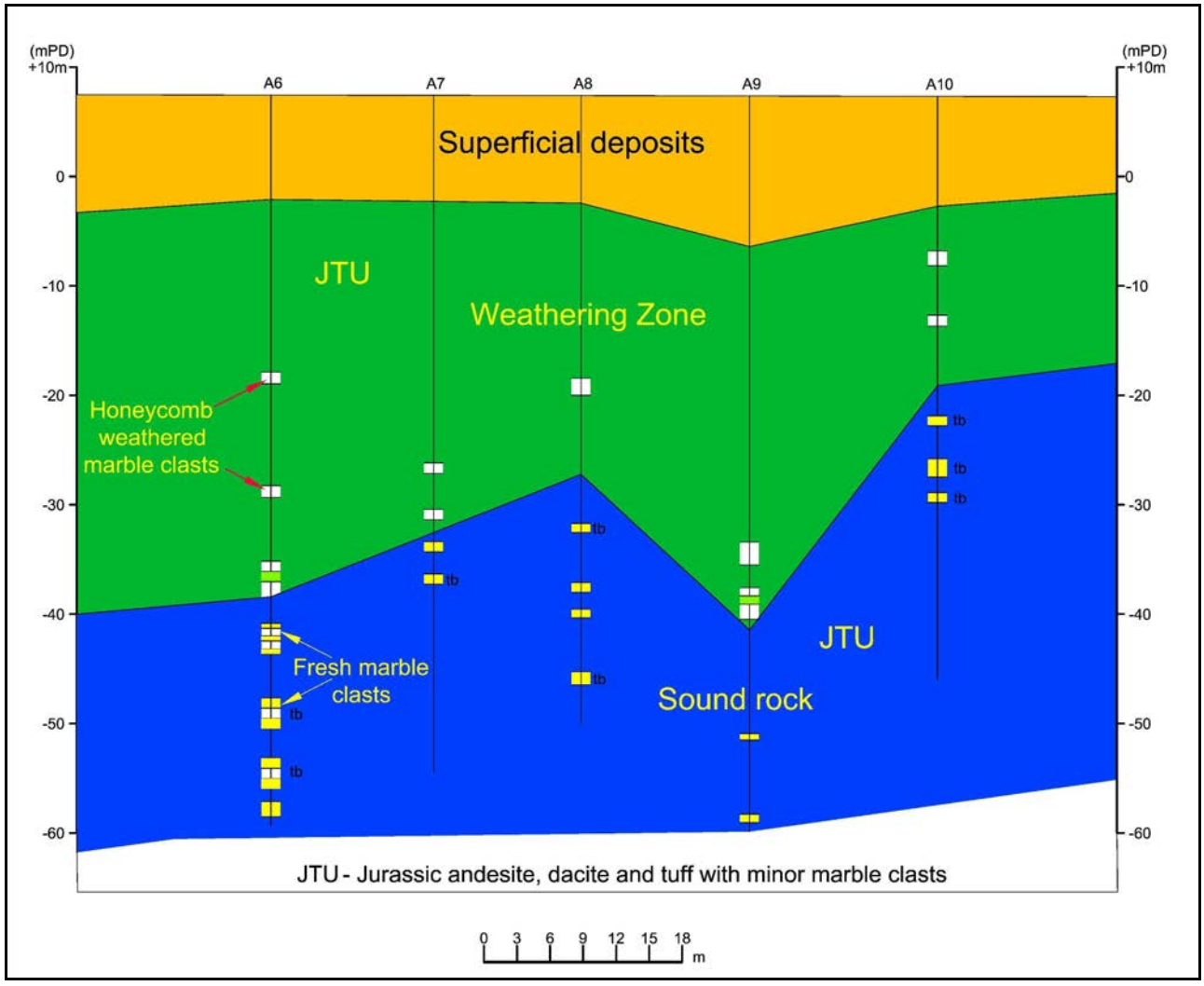

Figure 3: Geological section showing the marble clast-bearing volcanic rocks of Tuen Mun Formation at a Tin Shui Wai site

4.2 Problems in distinguishing Marble Clast-bearing Andesite from Marble Layer Interbedded with Siltstone.

The marble clasts-bearing andesite or vent breccia has been interpreted as marble layer interbedded with siltstone (Fletcher 2004). Subsequent thin sections identification and chemical analysis have proved that the marble clasts were embedded in andesite or dacite lava and are in fact marble clast-bearing andesite.

Plenty borehole logs have over estimated the cavity size and thickness of lithic clast marble (Figure 7). A typical example from Tin Shui Wai recorded a 12m thick marble which containing a 7.84m high cavity. The photographs of this borehole and surrounding boreholes have been reviewed in detail. It is found that individual marble clasts are generally a few centimeters, occasionally to tens centimeters thick. It does not form a thick bedded layered marble. The large cavity as shown in the aforesaid borehole logs is unreliable. On the contrary, a more than 50m thick layered marble containing a cavity up to 3.5m high has been described as marble clasts in a geological report (Sze 2006).

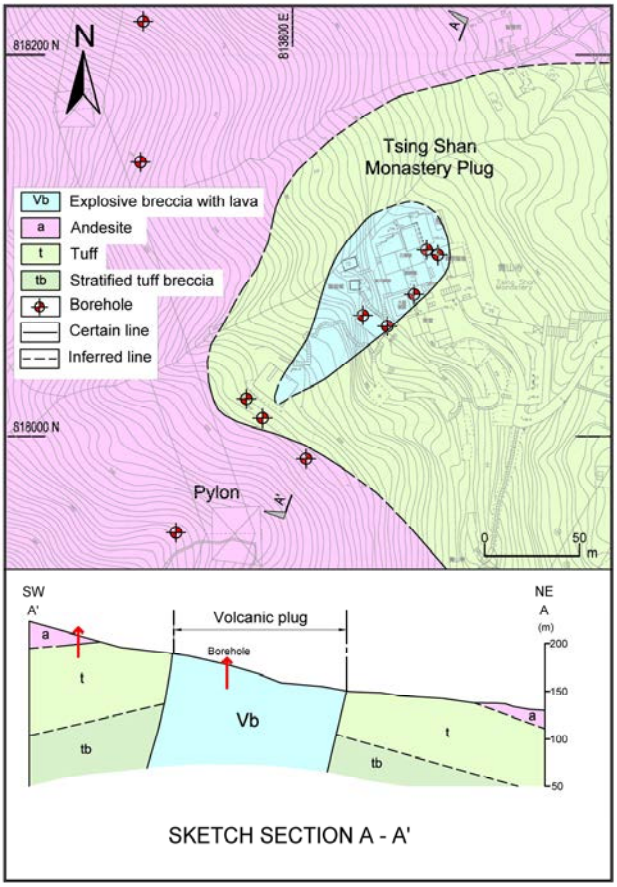

Figure 4: Tsing Shan volcanic plug comprises marble clast-bearing andesite and surrounding tuff and andesite lava

Figure 5: Contact between explosive breccia and tuff of Tsing Shan Monastery Plug

Figure 6: Marble clast-bearing andesite of Tsing Shan Monastery Plug

4.3 Problems in identifying Mylonitized Marble Clasts from Thin Bedded Marble.

Where marble clasts occur within or close to a ductile fault zone, the marble clasts were subject to mylonitization and became flattened, elongated and foliated. The correct rock name should be mylonitized marble clasts-bearing andesite or tuff breccia (Figure 8).

4.4 *Representing the Lithic Clast Marble as Carboniferous Sedimentary Rocks.*

The lithic clast marble-bearing rocks in most Tin Shui Wai area have been demonstrated as Jurassic volcanic rocks. However, they have subsequently been described otherwise as Carboniferous layered marble with siltstone of shallow sea deposits (Fletcher 2004 & GEO 2007). These result in the occurrence of a considerable borehole logs with questionable descriptions in the same area.

5 SUGGESTED METHODOLOGY FOR RECOGNITION OF ROCK CLASSIFICATION

- *Field observations*: The mode of occurrence and contact relationship between different rock types have to be studied in detail. The marble clast-bearing volcanic rocks have dual characteristics. They reveals a sedimentary nature such as sorting, bedding and roundness whilst also possessing volcanic characteristics such as the high temperature lavas contain volatiles forming alteration and fumaroles structures.

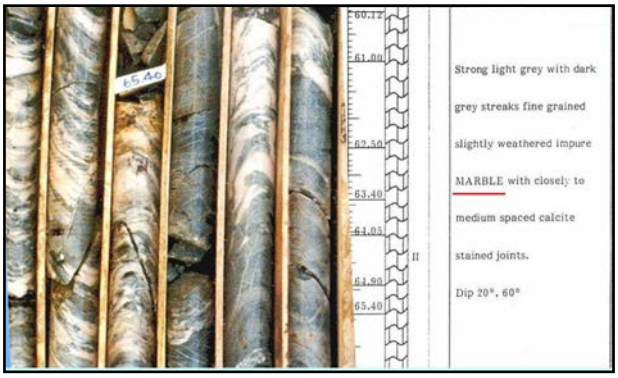

Figure 7: *(left)* The marble clasts-bearing tuff breccia was described as layer marble in some borehole logs

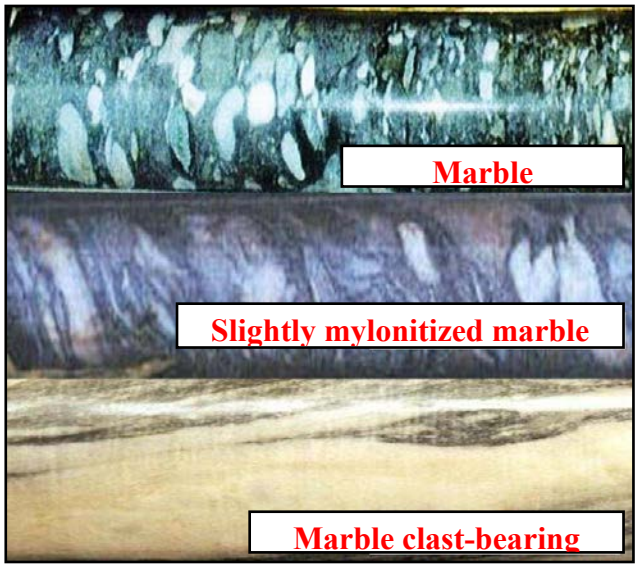

Figure 8: *(right)* Various degree of mylonitization of marble clast-bearing andesites

- *Geochemical analysis and thin section identification of fine-grained cement materials:* It is impossible to recognize the fine-grained andesite and dacite by naked eye alone.
- *Understanding of regional geology:* The marble layer only occurs in the Carboniferous oceanic deposit whereas the marble clast-bearing volcanic rock is only formed in Jurassic volcanic eruption.
- *Study on volcanic facies:* Generally a volcanic eruption forms various facies at difference distances from the volcano. For instance, the explosive breccia of diatreme facies occurs in the central zone, the lava flow facies in the proximal zone and the air fall facies of fine ash tuff in the distal zone. They are contemporaneous products and do not have a lower/upper strata relationship.

6 CONSEQNENCES

Quality of geological data may seriously affect the quality of future geological mapping. After examination of 627 drillholes logs containing marble clast-bearing rock from mid-2002 to 2007, more than 60% of the rock descriptions were found to be questionable. The marble clast-bearing volcanic rocks which occur within the volcanic plugs or dykes have been commonly described as sedimentary conglomerate or breccia.

The quality of geological data results affects the quality of geological models, which in turn may affect the foundation design. Potential effects may include increased time and cost for ground investigation and foundation works.

7 CONCLUSIONS AND RECOMMENDATION

Over the past 35 years, abundant experience in geological investigation and foundation design in cavernous marble sites for housing development has been accumulated. Meanwhile, the characteristics, engineering properties and differences between lithic clast marble and layered marble have been better understood through detailed field works and laboratory studies.

After examining more than one thousand borehole logs containing lithic clast marble, only honeycomb cavities occur in the weathering zone. No significant solution phenomena are present and no evidence for cavity-related subsidence has been identified below their sound rock level. Fletcher (Arrow Geoscience Ltd 2003a & b) carried out geological investigations at Tin Shui Wai and obtained the same conclusion. He defined the rockhead at a level beneath which is a minimum of 5m of moderately decomposed to fresh rock and the Total Core Recovery is above 85%. Those building construction sites were completed for more than three years, no settlement and geological problems have been reported. Fletcher's opinion is reasonable. It is suggested that the requirement of rockhead level of marble clast-bearing volcanic rock should be the same as that of other volcanic rocks.

The preparation of the Scheduled Area No.2 boundaries was based upon geological information available before 1992. Over the past 20 years plenty ground investigation data has become available. These data can help to better delineate the boundary between the Tuen Mun Formation and the Yuen Long Formation and hence better define the area potentially underlain by cavernous marble.

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