

# Elevating Geotechnical Excellence: Novel Practices and Innovative Solutions in Hong Kong

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## ABSTRACT

With nearly 50 years of experience as a geotechnical and structural engineer, the author aims to share his noteworthy expertise in this seminar. The paper will provide an overview of three key aspects of geotechnical engineering: Slope Works, Excavation and Lateral Support (ELS) Works, and Foundation. Regarding slope works, the paper will discuss the design development of slope upgrading works and natural terrain mitigation works together with several actual examples showcasing successful approaches to address challenging site constraints. Additionally, the paper will introduce novel materials that are either currently used or under study for slope works. Regarding ELS works, this paper will discuss the Enhanced Control System (5A system) for ground control during ELS. The author will also share his latest experiences in implementing sustained soil nailing as tie-backs, which represents the advancement in ELS design works. For the foundation section, this paper will elucidate the development of "presumed allowable bearing pressures" for rock. Each section will feature actual examples under the author's supervision, showcasing his ability to overcome engineering difficulties and address pertinent safety issues with excellence.

## 1 INTRODUCTION

Hong Kong is a densely populated city and its topography is mostly hilly. Due to the scarcity of flat land, many buildings and infrastructures have to be situated close to or even rightly on the hillside, which in most cases require the careful planning, design and construction of the geotechnical works to achieve safety and economic purposes. In addition, the geology of Hong Kong comprises different types of soil and rock and the bedrock levels vary from ground to more than 100m depth. Wide varieties of foundations have been adopted to suit the different terrain conditions to safely support superstructures ranging from residential houses to high rise buildings. To tackle the challenges comprising not only the technical issues but also the high demands for standards of safety and sustainability, the engineers of Hong Kong have been exercising their expertise and employing their experiences to achieve excellence in these aspects of design and construction. This paper will provide an overview of three aspects of geotechnical engineering works – Slope Works, Excavation and Lateral Support Works and Foundations. This is followed by actual examples that are under the direction of the author and can exhibit excellence in overcoming both engineering difficulties and social issues.

## 2 GEOLOGY OF HONG KONG

As described by Sewell et al. (2000), more than three-quarters of the rock outcrops of Hong Kong are composed of igneous rocks, predominantly volcanic and granitic rocks, while the remaining are sedimentary rocks. Sedimentary rocks are occasionally interbedded and interlaminated with metamorphosed sedimentary rocks e.g. metasiltstone and metasandstone. (GEO, 1992) These rocks are typically deeply weathered, occasionally with weathering front reaching as deep as 100m. (Cheung, 2021) Superficial deposits overlying the bedrocks comprising alluvium, colluvium and other unconsolidated materials are found over the whole territory. These sediments are with weathered profiles of thicknesses varying from a few metres to several tens of metres but can also exceed 200m locally (Fyfe et al., 2000). There are also large reclamation areas along the coasts and in the Hong Kong International Airport at North Lantau with variable deposits comprising sand fill, mixed rock and soil fill as well as construction and demolition debris. A geological map of Hong Kong modified from Sewell et al. (2000) (Pappin et al., 2004) is extracted in Figure 2-1.



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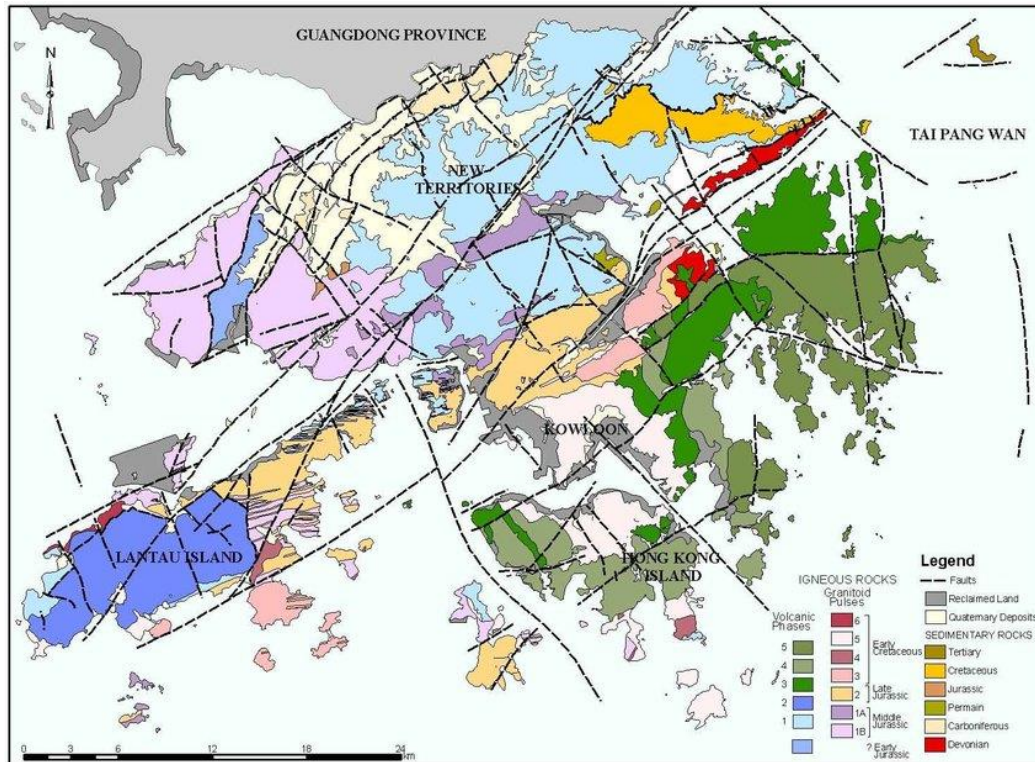


Figure 2-1: Simplified Geological Map of Hong Kong modified from Sewell et al. (2000) (Pappin et al., 2004)

### 3 Slope and Retaining Wall Upgrading Works and Slope Protection

Owing to the hilly terrains and lack of natural flat grounds in Hong Kong, over the years, plenty of cut slopes, fill slopes and retaining walls, collectively known as man-made features, had been constructed across the territories to create flat grounds for urban development and to house the growing population. The amount of geotechnical control, prevailing design standards and available technologies at the time of formation of these man-made features could be vastly different from the present day. Landslides associated with the man-made features formed in the old days prior to establishment of Geotechnical Control Office (GCO) in 1977 (renamed as Geotechnical Engineering Office (GEO)) were not rare. Some notable cases of man-made feature landslides include fill slope landslides in Sau Mau Ping in 1972 and 1976, landslide at the hillside above a cut slope at Po Shan Road in 1972, failure of a masonry retaining wall and the fill platform behind at Baguio Villas in 1992 and failure of a masonry wall and the slope below at Kwun Lung Lau in 1994. A total of 156 people lost their lives in the three landslide events in the 1970s and a total of seven people lost their lives in the two landslide events in 1990s. (GEO, 2024)

In late 1970s, the government launched Landslip Preventive Measures (LPM) Programme, a long-term programme to upgrade the existing substandard government and private man-made features. (DEVB, 2021) In order to upgrade the existing man-made features to meet the current safety standards, different upgrading measures were adopted in public and private projects alike. Some typical examples of upgrading measures are regrading of slope, provision of structural supports and ground improvement. In 2010, GEO broadened the scope of the LPM Programme and launched Landslip Prevention and Mitigation (LPMit) Programme to include studies of natural hillside catchments. (CEDD, 2023)

Recent initiatives in reducing carbon footprint in construction works, as well as the challenges in association with frequent and extreme rainfall events, have prompted the necessity to adopt innovation and novel technology in LPMit Programme. This, together with continuous specialist works in upgrading of man-made features and mitigation of natural terrain hazards, are two of the key elements of the slope safety system in Hong Kong. (Cheung, 2021)

### 3.1 Early development

In the early days of Landslip Preventive Measures (LPM) Programme, i.e. late 1970s to 1980s, upgrading measures adopted for cut slopes and retaining walls were generally limited to cutting back to a shallower angle and supporting the existing slope or retaining wall with a new retaining wall and backfill. Prestressed ground anchors were sometimes used in steep cut slopes or retaining walls with significant height. Fill slopes were stabilized by recompaction of top 3m of loose fill with provision of surface treatments and drainage, based on the recommendations by the Independent Review Panel set up by the government following the major fill slope failure in Sau Mau Ping in 1976. (GCO, 1976 & GEO, 1998)

Each of these early day upgrading measures has its own shortcomings. For example, cutting back to a shallower angle or supporting the slope with a retaining wall and backfill, both involving permanent modification of the existing landscape. The available space at the slope toe or slope crest is modified. Existing trees on the feature and at the crest need to be removed due to excavation or backfilling. As for prestressed ground anchors, life time monitoring works including residual load measurements and special grease checks should be carried out at regular intervals throughout the design life, imposing extra workload on the maintenance party of the anchored slope, especially when the anchor forces were found to drop. Due to this monitoring requirement, permanent prestressed ground anchors have not been adopted since the early 1980s. As for fill slope recompaction, which was traditionally carried out by placing and compacting 200-400mm thick horizontal layers of fill on a benched excavated profile to form a 3 metre thick compacted fill capping, bulk excavation was involved and extensive works area were reserved for stockpiling the excavated ground material. Similar to cutting back mentioned above, the existing landscape of the surroundings was permanently modified while removal of trees were usually involved. In view of these shortcomings, new upgrading measures and construction methods were introduced to the industry.

In the late 1980s, soil nails were first applied in the upgrading measures of cut slopes and retaining walls. Studies were carried out on various aspects of soil nail design, including method of stability analysis, nail-soil interaction, design of soil nail head, durability, pullout resistance, reinforcement materials, drillhole forming techniques, potential damming effect of soil nail grouting and non-destructive testing. Soil nail design has been evolving over the years, with significant improvement in robustness. Efforts are still being made in recent years to improve the design details and materials. For example, studies have been carried out to reduce soil nail head sizes and increase the constructability, alongside studies on potential replacement of steel reinforcement with more sustainable or more durable substitutes.

In the 2000s, development of upgrading measures for fill slope went along two main separate ways- new methods and materials for fill replacement and soil nail with grid beam system. A new technique of fill recompaction emerged in this period. Excavation and recompaction of fill slope were carried out in a pit-by-pit manner (Chan and Chan, 2008). Each pit had a dimension of 1.5m x 1.5m on plan and depth of 3m. High temporary benched cuts can thus be avoided. Instead of using compacted soil as backfill, the use of cement stabilized soil (GEO, 2003a), mass concrete, no-fines concrete and light weight concrete were trialed and was demonstrated to be effective and has become the prevalent trend in the recent years, while new cementitious backfill materials with better performance were being developed and studied in the 2020s.

Meanwhile, on the account that soil nailing was the most commonly used slope upgrading measure in Hong Kong early in this century, the industry also explored the possibility of using soil nail in loose fill slopes. The Hong Kong Institution of Engineers (HKIE) published a study to establish a basic design framework for soil nails in loose fill in 2003. The study covered the following:

- (i) laboratory tests for steady state shear strength of loose fill,
- (ii) proposed use of surface grid beams,
- (iii) recommended minimum relative compaction of 75% for loose fill to be upgraded by soil nails,
- (iv) recommended lower bound shear strengths at the onset and during liquefaction and
- (v) recommended minimum factor of safety for the checking of fill liquefaction using steady state parameters (FOS = 1.1).

In 2011 and 2013, Geotechnical Engineering Office (GEO) and HKIE co-published further studies on the use of soil nails in loose fill. These further studies built on the study in 2003 and further explored the following areas:

- (i) arrangement and inclination soil nails based on numerical analysis,

- (ii) recommended nominal grid beams embedment,
- (iii) recommended method to delineate zones of liquefiable fill and non-liquefiable fill and
- (iv) illustrated worked examples as per the recommendations.

### 3.2 Examples of innovative works

#### 3.2.1 High strength steel bars as soil nails

The subject feature above a private lot along Po Shan Road is a 25m high cut slope formed before mid-twentieth century. Based on records, a total of 22 anchors were installed in 1978. Monitoring and measurement of residual loads of the anchors had been carried out in the 1980s and 1990s, till 2000. The monitoring data showed that there was a decline in anchor forces over the years. (Wong et al, 2011b)

As a long-term solution, soil nails were proposed to substitute the anchors in 2008 under LPM programme. Site constraints including thick layer of colluvial boulders, high groundwater table, existence of ground anchors with sizable anchor heads were carefully considered. Under the conventional approach, soil nails of 40mm diameter high yield steel bars would need to be spaced at less than 2m. This closely spaced arrangement would clash with the existing ground anchor heads. To facilitate widely spaced nails, high strength steel bars of 36mm diameter with yield strength of 1080MPa (the yield strength of normal soil nails was 460MPa at that time) was adopted. The allowable nail force for individual nail was thus increased from 385kN to 732kN, an escalation of 90% as compared to normal soil nails of 40mm. This enables the soil nail spacing to be increased to a range of 2.4m to 4m. Such wider spacing is also necessary in order not to cause damming up the groundwater table at the steep natural terrain behind the subject cut slope. (Wong et al, 2011b)

Owing to the wider soil nail spacing, instead of typical isolated soil nail heads, a grid beam structure was constructed to tie all soil nails together in order to prevent local instability of the feature.

Consideration was also made to the construction method for the soil nailing works. The prevalent drilling method for soil nail was eccentric drilling, also known as Overburden Drilling with EXcentric (ODEX) drilling, which was prone to jamming of drill bits in the layer of colluvial boulders and might lead to excessive ground loss. To prevent such situations from happening during construction at the feature with high geotechnical sensitivity, concentric drilling was adopted. The mechanism of concentric drilling has reduced the width of the gap between the casing pipe and drillhole to the minimum and, thus avoiding the flushing up of material outside the casing and any consequent ground loss. Also, there is no gap between the casing shoe and the pilot bit and therefore jamming of drill bit is highly unlikely. (Wong et al., 2011b) The upgrading works were completed in 2008.



Plates 3-1 & 3-2: Subject government cut slope: before upgrading works with heads of ground anchor exposed on slope surface (left); upgrading works with grid beams exposed on slope surface (right)

### 3.2.2 Retaining walls upgrading works under traffic constraint

The feature comprises two stacking cantilevered reinforced concrete walls – the height of the lower wall is about 10m and that of the higher wall is about 5m. At the toe of the feature is a narrow one way access road acting as an exit from Comfort Terrace to King’s Road. The feature is located at the junction with King’s Road where the access road is on a steep gradient with a sharp turn before turning onto King’s Road.

The conventional upgrading measure for such retaining walls is the installation of soil nails as tie back for the walls. The nails would be spread uniformly on the wall stem. However, the nail heads in this case would make the already sub-standard access road narrower, and might cause encroachment upon the vehicular path.

In view of the above constraints, an innovative solution was proposed. First of all, the two stacking walls were structurally connected at the top of the stem of the lower wall and the toe of the upper wall. Then, tie back soil nails were installed mainly at the upper wall and even with a few at the lower wall, they were located at a minimum of 4m clearance above the road level. The soil nails would then be connected by tie beam and these tie beams would avoid the clashing of vehicles. The works were carried out smoothly in 2011.

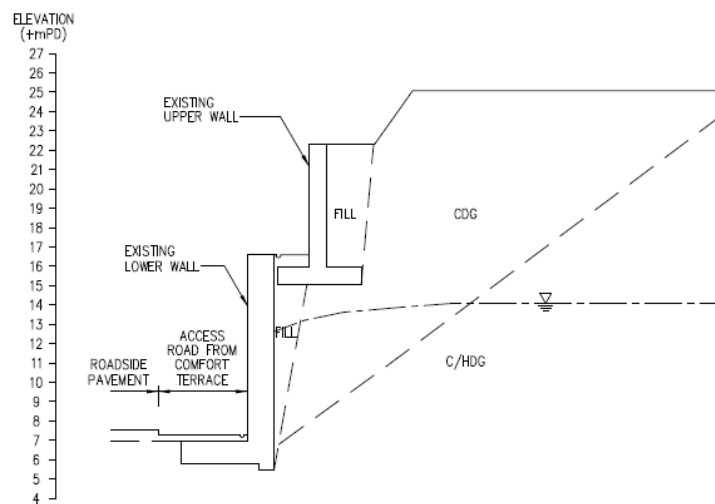


Figure 3-1: Typical section of the retaining wall before upgrading works

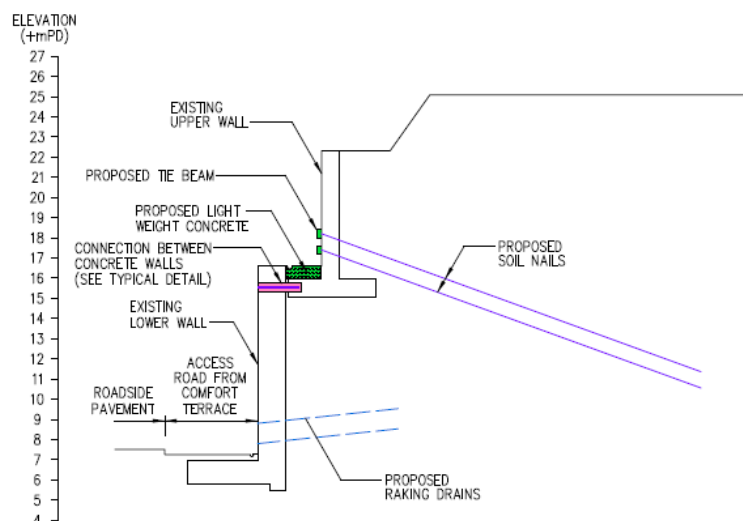


Figure 3-2: Proposed upgrading works at the retaining wall

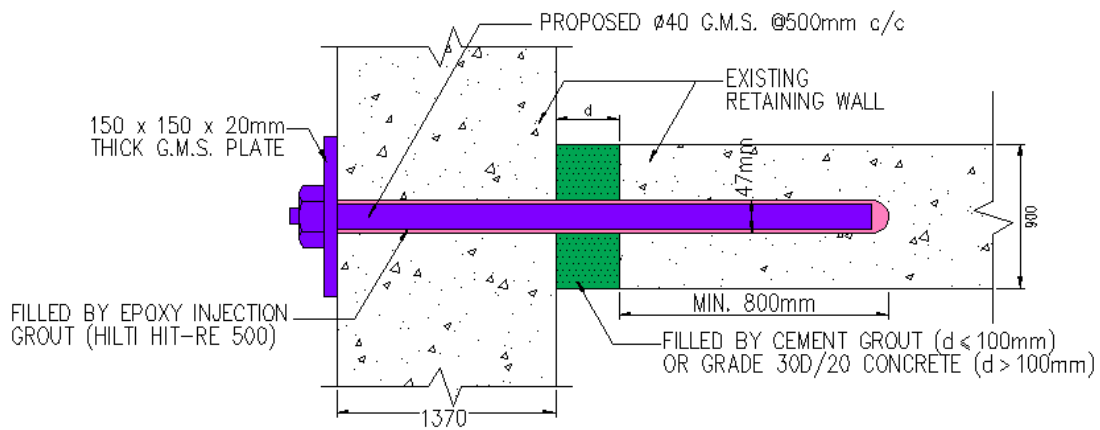


Figure 3-3 Typical details of connection between concrete



Plate 3-3: General view of the retaining wall after completion of upgrading works.

### 3.2.3 Preservation of stone wall tree on old masonry wall

The feature comprises a cut slope with a retaining wall at the toe overlooking Hornsey Road. The retaining wall has likely retained its original masonry appearance since its formation, with a stone wall tree protruding from the southern portion of the feature. The stone wall tree is of the species of *ficus virens* and has an overall height of 18m and average crown spread of 20m.

While designing the upgrading measures, considerations were made to preserve the valuable stone wall tree and the appearance of the masonry wall. Soil nails were adopted as the structural measure stabilize the masonry wall. Professor Jim Chi Yung, a stone wall tree expert, was invited to assess the condition of the tree and provide expert opinions on how to minimize disturbance to the tree. The water-cement ratio of the grout adopted was 0.35, which is lower than normal grout (with water-cement ratio about 0.4) to minimize migration of grout into surrounding soil. For each soil nail, permanent steel casing was provided for the first 5m of drillhole to control grout loss. Soil nail layout in close vicinity of the stone wall tree were handpicked on site together with Professor Jim. To construct the nail heads, - the masonry blocks of the wall affected by soil nails were removed from the wall one by one, and reinstated after the completion of the concealed nail head construction.

To abate sliding force and overturning moment to the masonry wall due to surcharge of the stone wall tree and to minimize the chance of stem failure and uprooting of the tree, pruning was carried out to reduce the tree crown. In future maintenance, the height and crown spread of the tree should be maintained within an allowable maximum value. The upgrading works were completed in 2018. The appearance of the stone wall tree and masonry wall was preserved. The feature won Special Merit in Sustainable Slope Excellence Award in 2020. As of to date, the stone wall tree is still thriving and providing a beautiful heritage-cum-vegetation appearance.

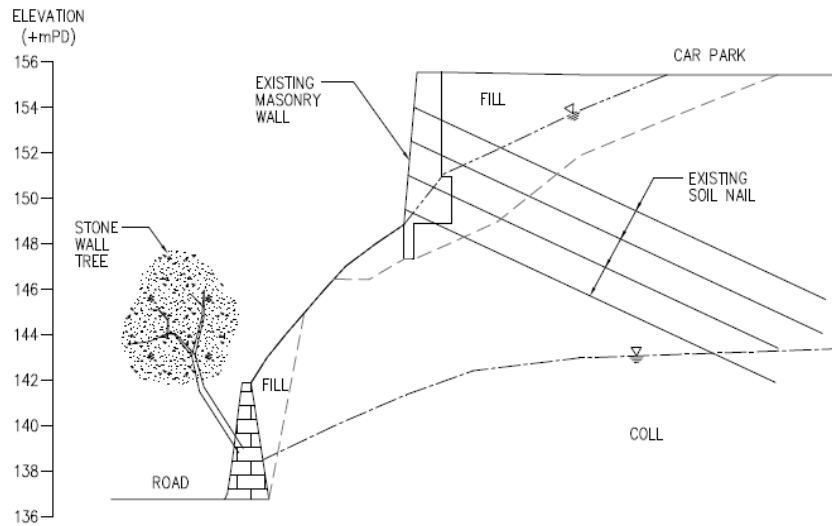


Figure 3-4: Typical section of the cut slope with retaining wall before upgrading work

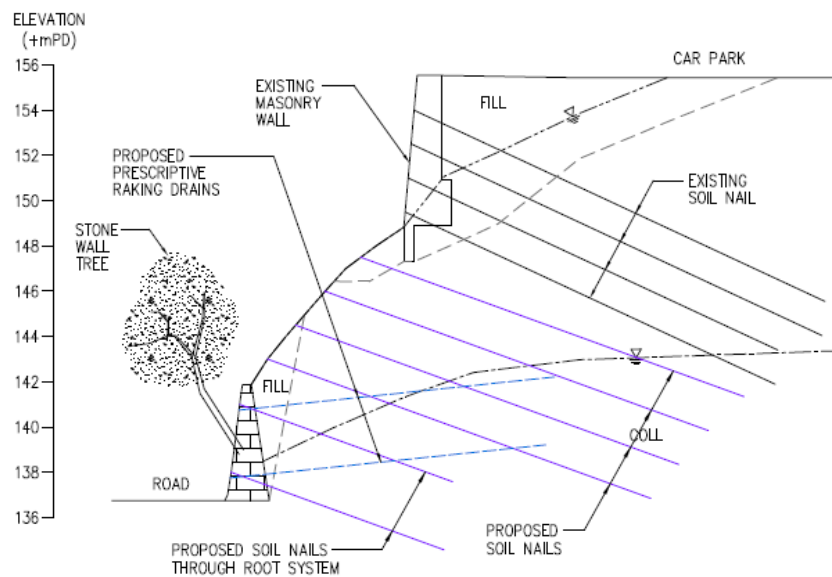


Figure 3-5: Proposed soil nails through root system of stone wall tree



Plates 3-4 & 3-5: Root portion of the stone wall tree: before installation of soil nails (left); after completion of soil nailing works. Locations are soil nail heads are circled in blue. (right)



Plate 3-6: Condition of the stone wall tree in May 2024

### 3.2.5 Rehabilitation of retaining wall with historical values

The subject retaining wall is located within an on-going construction site for private development. The feature was a ruin of a series of back-to-back tenement buildings (tong lau). As of 2018 when the development site was taken over by the private developer, only the back wall and small portions of the side walls of the tenement buildings remained intact in the ruin. The back wall, about 5m in height and 34m in length, retained a former building platform alongside Cochrane Street, while the remnant side walls were connected to the back wall. Composed of brickworks, these walls largely preserved their original appearance. The exact construction year of these tenement buildings is unknown, but Antiquities Advisory Board (AAB) (2016) puts the construction year at 1930s in a historical building appraisal. Although no grading is assigned to the remnant walls as confirmed by AAB in 2017 (AAB, 2024), AAB remarked in the appraisal that the remnant walls could serve as a reminder of a building and living style of pre-war Hong Kong (AAB, 2016).

During the design stage of the private development, efforts were made to preserve the remnant walls including the subject feature as far as possible, alongside their historical values. Assessment reveals that the stability of the subject feature was not up to the current safety standards and requires upgrading works. Typical

upgrading measures of a retaining wall of this height include installation of soil nails and/or thickening of the wall from the wall front. These two typical measures would have an adverse effect on appearance of the wall.

In order to preserve the appearance, upgrading measures could only be carried out behind the wall. In this case, pit-by-pit excavation followed by backfilling with mass concrete, were proposed. However, the current guidelines limit the depth of each excavated pit to 3m whereas the height of the brick wall is about 5m. In order for this scheme to be workable, open excavation was carried out to form a level platform about 2m below the wall crest. Pit excavation could then began at the level platform and the base of the 3m deep pit would meet the wall base. As of to date, the construction works are still on-going.



Plate 3-7: Condition of subject retaining wall and one of the connected side wall before commencement of construction works



Plate 3-8: General view of condition of the subject retaining wall before commencement of construction works

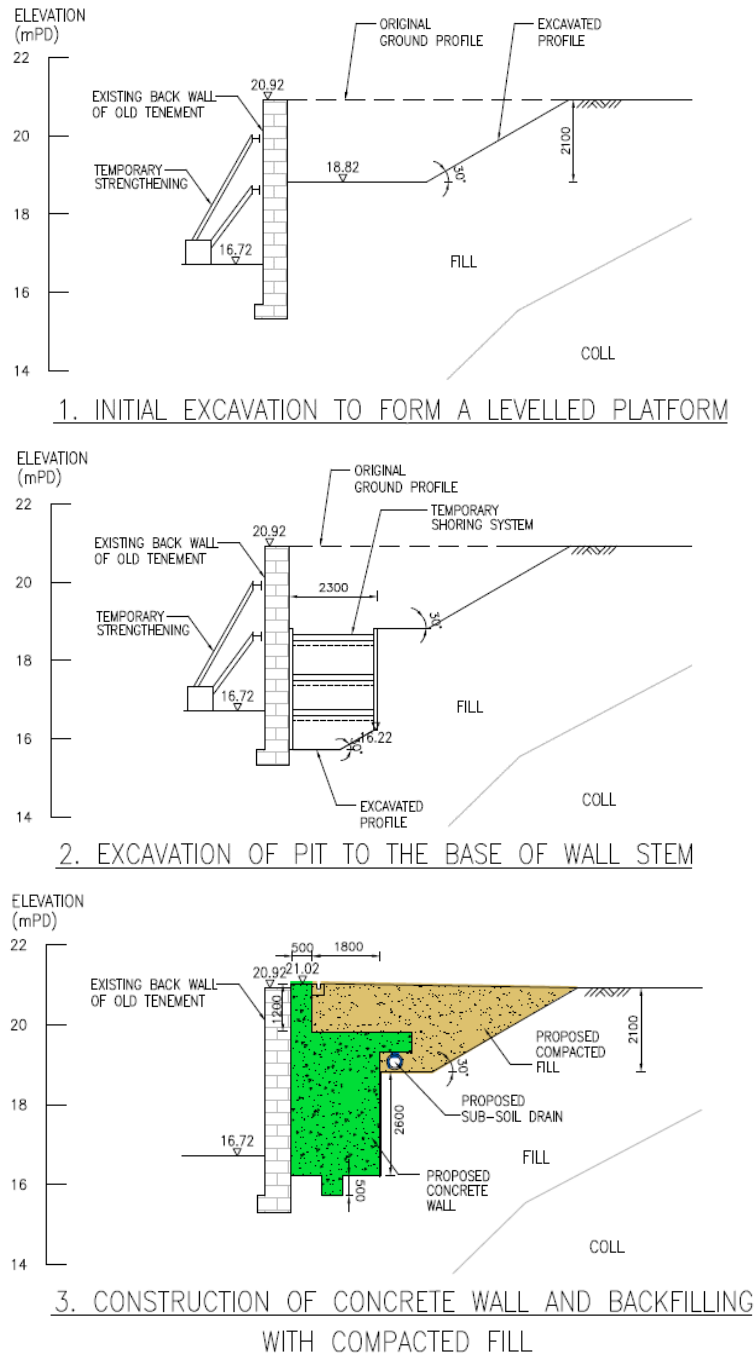


Figure 3-6: Typical section of the upgrading works.

### 3.3 Natural Terrain Hazard Study

Hong Kong is prone to natural terrain landslides, given its high seasonal rainfall, deep weathering soil profile and the vast extent of natural terrain areas (Ko and Lo, 2018). The natural terrain susceptible to landslides pose hazards to people and property in developed land nearby.

In the last century, severe rainstorm events occurred occasionally and caused widespread natural terrain landslides across the territory. Some notable rainstorm events include the June 1966 heavy rainstorm affecting Hong Kong Island of which the landslides and flooding together resulted in 64 fatalities and more than 2000 people homeless, the May and August 1982 rainstorms affecting the whole territory during which more than

1400 natural terrain landslides were triggered and squatter areas were severely damaged and, the November 1993 rainstorm affecting mostly Lantau Island during which more than 800 natural terrain landslides were triggered. In the 2000s, severe rainstorm events became more frequent and extreme. In the June 2008, Hong Kong was struck by the most severe rainstorm at that time, of which the return period of the rolling 4-hour and 24-hour rainfall were inferred to be in the order of 1,000 and 200 years respectively. On Lantau Island alone, more than 2400 natural terrain landslides occurred, resulting to blockage of some roads of sole access and the trunk road to Hong Kong International Airport, cut-off of water supply to villages and evacuation of village residents. (Ho et al., 2016)

In 2010, GEO broadened the scope of the LPM Programme and launched Landslip Prevention and Mitigation (LPMit) Programme to include studies of natural hillside catchments. (CEDD, 2023)

In September 2023, under a single severe rainstorm event, the government received a staggering number of 145 landslide reports within just two days. The 24-hour rainfall recorded in the Eastern and Southern District of Hong Kong exceeded more than 800mm, which was the highest record since rain gauges were installed in the areas. Increasing precipitation with greater extremes is expected in Hong Kong in the coming years, further increasing the landslide risk across the territory. Upscaling the government's LPMit Programme and speeding up the programme might be necessary for protecting public safety.

### *3.3.1 Recent development in NTHS*

Guideline for natural terrain hazard studies (NTHS) was first compiled into GEO Report No. 138 (first edition) for practitioners to follow in 2003. The most popular approach for NTHS has been based on design event approach. Under the first edition of the report, a conservative event with a return period of 100 years and a worst credible event with return period of 1000 years are adopted as the design event for run out analysis of landslide debris (GEO, 2003b).

A major update on the design event approach was released in 2013 in Technical Guidance Note (TGN) No. 36 (GEO, 2013). In the new approach, an event with return period of 100 years based on recent landslides as well as relevant relict landslides with a high degree of certainty is adopted as the design event. This, together with other updates, were compiled into a second edition of GEO Report No. 138 in 2016. Detailed guideline on application of "react-to-known-hazard" principle on natural terrain catchments under LPMit programme is also presented in this new version of the report

### *3.3.2 Typical Mitigation Measures*

Some common mitigation measures for natural terrain landslide hazards in Hong Kong include flexible debris resisting barrier, rigid debris resisting barrier and in-situ stabilization by soil nails. In natural terrain study area comprising numerous catchments and spanning across an extensive area, a hybrid approach consisting of more than one type of mitigation measures should be adopted.

On first look, in-situ stabilization with soil nails is a handy solution which can suit the condition of most natural terrain catchments. However, installation of soil nails involves erection of temporary platforms and access ladders, and might thus cause disturbances to flora and fauna residing in the area. Soil nailing across vast extent of the catchment should be avoided as much as feasible. If condition allows, debris-resisting barrier should be adopted as the mitigation measure instead of soil nailing.

An example including all the recent development on mitigation measures for natural terrain is a Study Area in Clearwater Bay. As shown in Figure 11, a total of 5 hillside catchments require mitigation works based on results of the NTHS. The toe areas of catchments D and E are composed of relatively gentle grounds, which are suitable for construction of debris-resisting barrier. Considering the actual design requirements and ease of access of construction plants and materials by road, rigid-debris resisting barrier is proposed for catchment D, while flexible debris-resisting barrier is proposed for catchment E. Soil nailing is proposed at the remaining catchments A, B and C. The extent of soil nails is confirmed to hazardous areas only, to minimize disturbance to flora and fauna as far as possible.

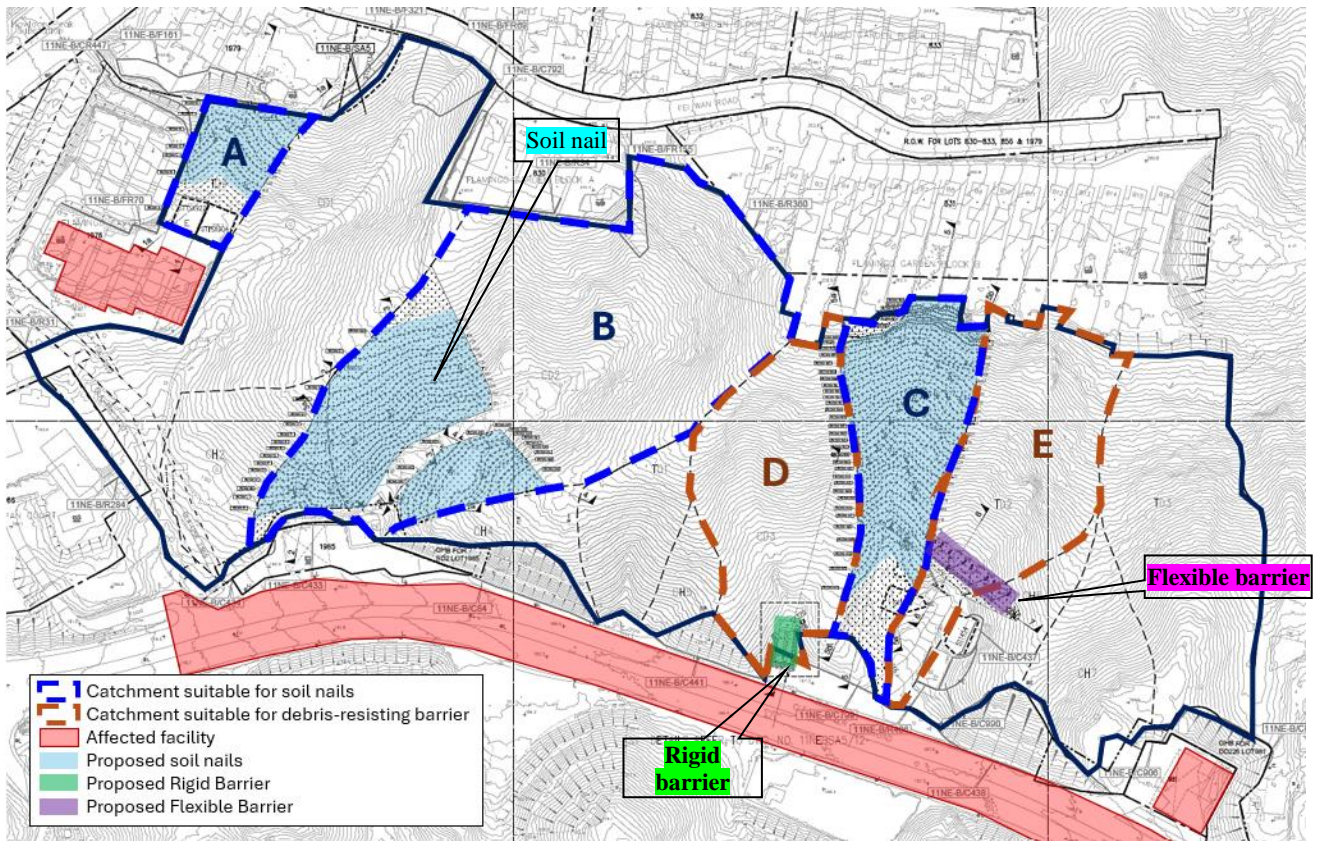


Figure 3-7: Layout of mitigation measures in a Study Area comprising 5 catchments which require mitigation works



Plates 3-9 & 3-10: General view of the rigid barrier at Catchment D of the Study Area in the example, from the facility at risk (left) and from the Study Area (right).



Plate 3-11: General view of the flexible barrier at Catchment E of the Study Area in the example

### *3.4 Prospective advancement in material*

#### *3.4.1 Potential use of fibre-reinforced polymer as reinforcement in soil nail*

Traditionally, steel reinforcement bars are used in soil nailing works in Hong Kong. However, steel reinforcement bars are prone to corrosion and thus steel soil nails require corrugated plastic sheathing for corrosion protection in aggressive grounds. Studies have been carried out to explore possibilities in substituting steel soil nails with fibre-reinforced polymer, which is a composite material made of polymeric resin embedded with fibre and apparently more durable than steel. The initiative was that corrugated plastic sheath protection may not be required for the highly durable fibre-reinforced polymer soil nail in aggressive grounds. (Cheung & Lo, 2005) The light weight of these composite materials also adds to the advantages in the aspect of safety in manual handling. Carbon fibre-reinforced polymer (CFRP) and glass fibre-reinforced polymer (GFRP) were studied in the 2000s and 2020s respectively.

Under the study in the 2000s, a series of laboratory tests including tension test and CFRP-grout bond strength test, trial GFRP soil nail installation and trial GFRP soil nail pull-out test were carried out. Two types of GFRP reinforcement were tested: the flat strip type and the rounded type. The study reveals that the type of CFRP tested has tensile strength around 5 times conventionally used soil nail steel reinforcement bar. The results of CFRP-grout bond strength are comparable with the bond strength between steel reinforcement bar and grout. Trial installation was successful. However, results of the pull-out tests were considered inconclusive since, in some of the tests on flat strip type reinforcement, the CFRP soil nail fails pre-maturely. The pre-mature failure could be explained by damage of carbon fibres due to concentrated stress immediately behind the head adapter during transportation. No pre-mature failure was reported for CFRP soil nails with rounded type reinforcement.

Although the results in laboratory tests indicate a good prospect of CFRP soil nails, the study remarks that the brittle behaviour, low shear strength and low bending capacity have yet been explored, while cast in-situ head adapters are preferred for pull-out tests in further site trial. (Cheung & Lo, 2005)

With the experience on studying CFRP, laboratory studies and site trials on GFRP soil nails has been proposed in the 2023. Reinforcement bar tension tests and GFRP-grout bond strength tests have been proposed to specimens subject to an alkaline environment. GFRP by different manufacturers, resin of different types and reinforcement bars with different surface treatment will be explored. (PolyU, 2023) The site trial is proposed to include a total of 21 trial nails, some of which will be subject to pull-out tests, the remaining of which will be subject to long-term monitoring. All trial nails will be installed with optical fibre sensors for deformation monitoring (CMA, 2023). As of to date, the tests under the laboratory studies and site trial are still on-going.

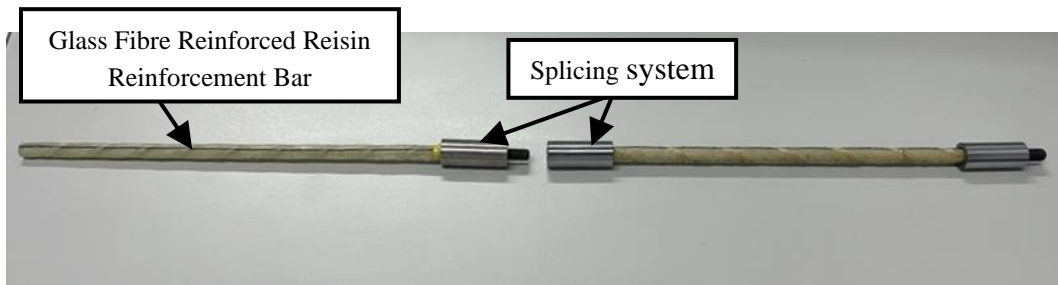


Plate 3-12: Components of glass fibre reinforced resin reinforcement bar

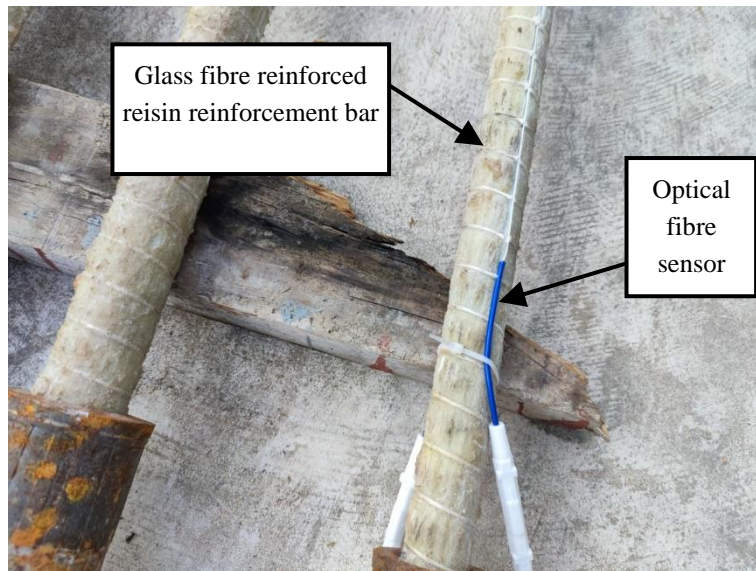


Plate 3-13: Glass fibre reinforced resin reinforcement bar fitted with optical fibre sensor used in site trial

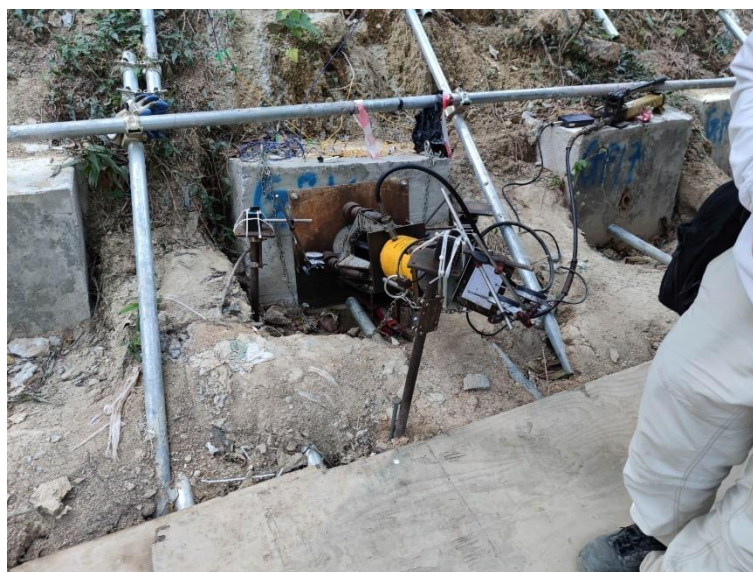


Plate 3-14: Pull-out test for glass fibre reinforced resin soil nail

### 3.4.2 Potential use of engineered cementitious composite (ECC) grout to replace Portland cement grout in soil nail

The initiative of the studies related to ECC grout is similar to fibre-reinforced polymer reinforcement bar mentioned above. ECC, popularly known as bendable concrete, is a class of fibre-reinforced concrete material exhibiting excellent crack control under a strain of several percent. The reinforcement fibre is typically micro fibre and the distribution and flow direction are carefully controlled and monitored. When subject to high tensile load, instead of developing a single crack with macroscopic width, ECC develops numerous parallelly aligned cracks with very small width, typically less than 100 micrometers, in an orientation normal to the tensile loading axis. (Li, 2012) These tight crack width does not increase water permeability of the ECC beyond the level for the uncracked material. (Lepech & Li, 2009) Hence, ECC grout has the potential in protecting steel reinforcement bars against corrosion by limiting infiltration of water and the dissolved corrosive ions. Corrugated plastic sheath protection may not be required for soil nails with ECC grout installed in aggressive grounds.

Professor Victor Li from University of Michigan and Professor Christopher Leung from The Hong Kong University of Science and Technology were invited to carry out Laboratory studies to develop a suitable ECC grout for the use in soil nail construction. Mixes of ECC grout composing of difference mix proportions of fly ash, cement, water and fibre, fibre types and fibre suppliers were subject to workability tests (i.e. flow cone test and flow table test), bleeding test and strength tests (i.e. crushing strength test and direct tensile test). Cracking behavior of grout specimens under direct tensile test was closely monitored. (CMA, Leung & Li, 2018)

Among all mixes, the one showing the most promising laboratory test results, i.e. highest strain compatibility with steel reinforcement, was selected for site trial. Work procedures, work efficiency and workmanship were reviewed. (CMA, Li & Leung, 2019) The results of the study are promising, while further research on the durability of ECC grout in a longer monitoring period will be required. Also, during the site trial, a long ECC mixing and grouting time was noted. The skills required for ECC mixing is also significantly higher than that of conventional cement grout. Optimisation of the work flow for installing soil nails in ECC grout is crucial for its implementation.



Plates 3-15 & 3-16: Filled moulds for simulated grouting exercise (left), mixing of ECC grout during site trial (right).

### 3.4.3 Use of self-compacting cementitious materials for backfill in pit-by-pit works

Although using concrete as backfill in pit-by-pit works was demonstrated to be effective, it also comes with drawbacks like:

- (i) long setting time,
- (ii) difficult for material delivery in remote sites and steep terrain,
- (iii) the need of compaction by concrete vibrator and
- (iv) safety hazards associated with manual handling and working in pits.

In the 2020s, government departments carried out studies to explore the potential use of a self-compacting material (SCM) developed by Nano Advanced Materials Institute Limited (NAMI) as backfill material in road works and pit-by-pit works. The SCM developed by NAMI is composed of cementitious material, fine aggregate, crushed rock fines, water and admixtures. The proportions were adjusted for optimization of its mechanical behaviors. This material was originally developed for the use in road works in which re-excavation is common. The proportions are adjusted to achieve a low compressive strength, while maintaining high flowability and workability, self-levelling, self-compacting and homogeneity. (GEO, 2023a) The qualities of high flowability and self-compacting in SCM are highly favourable in pit-by-pit works, in view of the challenges listed at the beginning of this section.

A sizable loose fill slope was chosen as a pilot site to review the constructability of SCM. This fill slope was designed to be upgraded by pit-by-pit works and nearly 900 m<sup>3</sup> SCM were required for backfilling the excavated pits (Plates 3-17 and 3-18). This site is not close to public road and the works area is limited. The characteristic of high flowability of SCM solved the transportation of backfilling material for over 100-meter travelling distance with level difference exceeding 30-meter. This case is known to be the largest amount of SCM used for slope upgrading works in Hong Kong. This project has also won champion in slope category of HKIE Geotechnical Project Excellence Award in 2023.

The key findings of the studies and the pilot application include a good consistency of SCM and no observation of ground subsidence or signs of distress or detachment after application of SCM, no difficulty in site mixing, efficient placement of SCM due to its high flowability and self-leveling and self-compacting qualities, efficient horizontal pumping over 250m and vertical pumping of over 30m using common equipment, no difficulty in sampling for quality assurance tests. (GEO, 2023a)

In addition to these findings, an apparently much lower amount of heat release was reported during curing when compared to cement grout or concrete, possibly due to the low cement content. This quality, if well investigated and documented in further studies, may be favourable in backfilling works near sensitive utilities. Possibly due to the low cement content in the SCM, much less shrinkage cracks were found in cured SCM backfill than in cement grout or concrete backfill. GEO has issued a Technical Note giving recommendations on the design and use of SCM in upgrading works in 2023. With more successful cases, SCM has the potential to become the prevalent backfill material in pit-by-pit works.



Plate 3-17: Backfilling of a pit by self-compacting material during site trial



Plate 3-18: General view of the fill slope after completion of pit-by-pit excavation and backfilling with self-compacting material, followed by pit planting of shrubs and whips.

#### 3.4.4 Concrete canvas for slope protection

Instead of the conventionally used shotcrete, Concrete Canvas has recently been adopted as slope surfacing materials under emergency works. Concrete Canvas which is known as Geosynthetic Cementitious Composite Mats (GCCMs), consists of three layers, including fibrous top surface, dry concrete mix with 3D fibre matrix PVC backing, and serves different purposes in relation to strength, water proofing and flexibility on the slope surface. (Concrete Canvas Ltd., 2022)

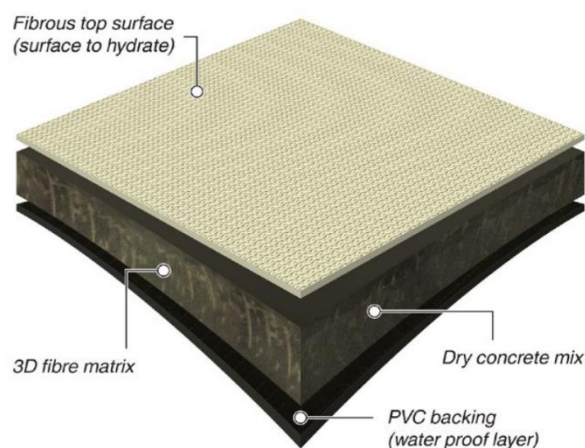


Figure 3-8: Composition of Concrete Canvas (Concrete Canvas Ltd., 2022)

In September 2023, landslides occurred on two man-made slopes in Lamma Island. Emergency repair works for providing hard cover to the failure slope surface are required. However, difficulties arise on carrying out shotcreting works in outlying island due to difficulties in plants mobilization, material delivery and limited working area. It is also required to complete the works in a short time to allow the re-opening of the footpath at the slope toe. Therefore, there was an opportunity of introducing Concrete Canvas as the slope surface cover under this case.

Batched rolls of Concrete Canvas could be delivered on site easily and the corresponding working procedures on handling the Concrete Canvas are relatively simple. Each roll is having roll width and roll length of 1 m and 10 m respectively with thickness of 5 mm and unit weight of  $7\text{kg/m}^2$ . Concrete Canvas is laid on the slope surface with at least 100 mm overlapping between layers and fixed with U-pins. After finish positioning Concrete Canvas, the covered surface is hydrated by spraying with water and bolts are used to fix the joints of Concrete Canvas after setting.

Concrete Canvas will harden to 80% of its 28 days strength in 24 hours and the slope surface could be thereby protecting the slope against erosion, water penetration and weathering effect within a short period of time. (Concrete Canvas Ltd., 2022)

Laying Concrete Canvas on the slope surface provides a quick temporary slope surface cover without involvement of heavy machinery items, such as air compressor and shotcrete machine, and overcome the difficulties on delivering the shotcreting materials to the outlying island.



Plates 3-19 & 3-20: Delivering batched rolls of Concrete Canvas on site (Left) and Fixation of Concrete Canvas on the slope surface (Right)



Plates 3-21 & 3-22: General view of the slope surface before (Left) and after (Right) emergency slope surfacing works using Concrete Canvas



Plate 3-23 : Extent of emergency slope surfacing works under Concrete Canvas

### 3.4.5 Fibre-reinforced shotcrete for slope protection

As mentioned previously, one of the critical issues on the adoption of hard slope surface cover is the crack propagation at the slope surface, resulting in high maintenance cost on cracks repairing works. For shotcreted slope, a layer of steel mesh reinforcement is laid on the slope surface prior to the shotcreting works as crack control measures. However, the laying works involve erection and dismantling of temporary working platforms, and it increases greatly on the time required for the slope surfacing works.

Moreover, the laying works rely heavily on the manual operation, and the corresponding safety risk, such as working at height, manual handling of mobilizing welded mesh and getting injured from protruded welded mesh, may be easily induced.

Therefore, fibre-reinforcement shotcrete is introduced as the slope surfacing cover. Fibre inside the shotcrete mixture may enhance the flexural strength of the slope surface cover and increase the ability of resisting crack propagation and maintaining structural integrity after initial cracking. (CMA, 2020) As such, the adoption of fibre-reinforced shotcrete neglects the need of laying welded mesh and the corresponding safety risks on handling the welded mesh at the slope surface are thereby eliminated.

Wet-mix fibre-reinforced shotcrete, which the shotcreting materials are mixed in the concrete plant and delivered to the construction sites, is commonly adopted in tunneling works. However, the available working space for carrying out the slope upgrading works is usually limited, and there may not be sufficient access and space for delivering the wet-mix fibre-reinforced shotcrete. Therefore, ‘dry-mix’ fibre-reinforced shotcrete, for which the fibre and shotcreting materials are mixed in a small mixing plant on site, is introduced. (CMA, 2020)

As shown in the plates below, fibre is mixed with the shotcrete material in the portable mixing machine on site, and the mixture is applied directly on the slope surface, similar to the working procedures of the shotcreting works. The study is still ongoing and further site trials will be carried out to review the use of ‘dry-mix’ fibre-reinforced shotcrete as the slope surface cover.



Plates 3-24 & 3-25: Mixing of fibre-reinforced shotcrete for shotcreting works



Plates 3-26, 3-27 & 3-28: Typical working procedures of shotcreting works using fibre-reinforced shotcrete. Original slope surface (left); application of shotcrete (middle); shotcreted slope surface (right).



Plate 3-29: Slope surface with covered with fibre-reinforced shotcrete.

#### 4 EXCAVATION AND LATERAL SUPPORT WORKS (ELS WORKS)

Excavation and lateral support (ELS) works are commonplace in the building and infrastructure projects. In recent years, with the growing complexities of the deep excavation works, which are often pushed to the property boundaries and / or in the midst of sensitive structures and utilities, ELS works have become one of the major construction activities in Hong Kong under great concern. For years, the engineers have been exhibiting excellence in designing innovative systems and introducing sophisticated design methods to promote safety, economy and sustainability in the works.

An overview of the design approach in ELS works is first presented, followed by discussions of selected aspects that can demonstrate excellence of the geotechnical engineering practitioners.

##### 4.1 Overview of the Design Approach in ELS Works

Before the computer era in the 1980s, excavations and lateral support systems in Hong Kong comprising either free or propped cantilever walls were mostly analyzed and designed by the Rankine and Coulomb's theories and other conventional approaches described in references such as Terzaghi (1967) and Bowles (1968). In the 1980s, the early editions of Geoguide 1 (GCO, 1982), NAVFAC Design Manual 7.2 (US Government, 1982), CIRIA Report 104 (Padfield and Mair, 1984), and others became the main references for the practitioners in the design of ELS works. In 1990, the then GCO has published the GCO Publication No. 1/90 titled "Review of Design Methods for Excavations". The publication comprises comprehensive descriptions of various ELS works systems and different design approaches as guidance for the engineers. Discussions of some advanced design concepts including the finite element method, elasto-plastic soil model have also been included.

The GCO Publication No. 1/90 (GCO, 1990) adopts the "Global Factor Method" in which unfactored soil strength parameters and loads are used in the stability checks (as described by PNAP APP-57 (BD, 2012)) of an ELS works for compliance with "global factors of safety specified in the guides / codes including Geoguide 1 (GCO, 1982) and Foundation Codes 2004 (CoPF) (BD, 2017). Nevertheless, the CIRIA Report No. C580 (Gaba et al., 2003) was published in 2003 which introduced a Limit State Partial Factor (LSPF) method for embedded retaining walls. In this method, soil structure interaction analysis is carried out for both the serviceability and the ultimate limit state conditions. Soil parameters and loads will be applied with pre-determined partial factors in the design of ELS works. In 2004, GEO of Hong Kong set up a Review Group comprising members from the Buildings Department, consultants and contractors to review the use of the LSPF. The author was a member of the Review Group. After the review, the Buildings Department has subsequently recommended that the method could be adopted for a trial period of two years as indicated in a letter dated 6 January 2005. Some modifications as to the partial factors given by CIRIA Report No. C580 to concur with the practice of Hong Kong are contained in the letter with the requirements including sensitivity analysis for singly propped excavations etc.

Wong & Yau (2005) has assessed the structural implications of the LSPF approach by CIRIA Report No. C580 as compared with the global factor method of GCO Publication No. 1/90 in 3 ELS works cases. The results reveal that the LSPF approach requires shorter embedment lengths of the walls, with reduction in the order of 40% while there are no significant differences in the structural steel contents in the shoring and strutting. Wong

& Yau (2005) thus has demonstrated cost effectiveness of the LSPF approach. In addition, the shorter embedment length can result in lower risk incurred in construction, e.g. minimizing the risk of hard driving of sheet pile in difficult ground.

Upon expiry of the 2 years trial period as imposed by BD's letter dated 6 January 2005, the Buildings Department has further extended the trial period for 3 more years in a letter dated 18 January 2007 to allow time to collect sufficient data for assessing the suitability of the approach. A PNAP APP-57 was finally issued in 2011 which concludes that the LPSF method could be used as an alternative to the global safety method with conformance to the guidance and requirements given in Appendix A of the PNAP. The appendix has excluded the use of the method for walls entirely embedded in soft clay, contains modifications to some of the partial factors given in CIRIA Report No. C580, and requires sensitivity analysis for singly propped excavations etc.

CIRIA Report No. C580 was superseded by CIRIA Report No. C760 (Gaba et al., 2017) in which the contents were modified to be compatible with the relevant Eurocodes. Also its scope of application has been extended to cover soft clays and weak rocks. Contents on some aspects such as environmental protection, derivation of geotechnical parameters are enriched.

In December 2023, GEO published the GEO Publication No. 1/2023 titled "Deep Excavation Design and Construction" (GEO, 2023b) which is a revision to the Publication No. 1/90. The new publication serves to consolidate the experience gained and improvements made since 1990. Clearer and more comprehensive guidelines are given in the Publication. Some advanced design methods are discussed. In particular, the "Partial Factor Method" (Limit State Partial Factor Method) has been well incorporated into the Publication in its 6.4.2 with a full list of minimum partial factors consistent with the Geoguide 1 (2020 Version) (GEO, 2020) for the design of retaining wall. In addition, there is no longer a need to carry out sensitivity analysis and post-construction review, given the experience gained on using the LSPF method since 2005.

#### *4.2 Water Pumping Test*

Before commencing the Excavation and Lateral Support Works (ELS works) below groundwater table, pumping test is normally required to demonstrate the water tightness of the pile wall systems for deep excavation works and the surrounding implications due to dewatering works to ensure safety in construction works proceeding in dry conditions.

For large-sized construction sites with deep excavation, full-scale pumping tests are commonly adopted to study the performance of the pile wall systems and verify the test results if it tallies with the design groundwater drawdown. However, full-scale pumping tests require a long time to complete dewatering and may possibly induce adverse effect to the surroundings. Pumping test may induce a substantial differential piezometric pressure across the embedded wall, which could lead to undesirable settlement when the lateral shoring support has not been installed. Therefore, partial pumping test is often adopted to minimize the potential impacts to the surroundings during dewatering.

Figure 4-1 shows a residential development site at Tuen Mun with an approximate site area of 44,000 sqm and an average excavation depth of 12 m. In view of the sizable construction site, several representative locations are chosen for carrying out pumping tests.

In some cases, partial pumping tests may be considered inadequate for deep excavation with highly sensitive structures at surroundings of the construction site since the uncertainties and the potential risks inducing to the highly sensitive structures can be ascertained to the minimal through implementation of full-scale pumping tests.

A construction site with office development located at Central is having the same site constraints that the highly sensitive structures (i.e. MTR tunnels for Island Line and Tsuen Wan Line) are located only 3 m (minimum allowable for MTR structures) besides the deep excavation works with approximate depth of 25 m (See Figure 4-2).

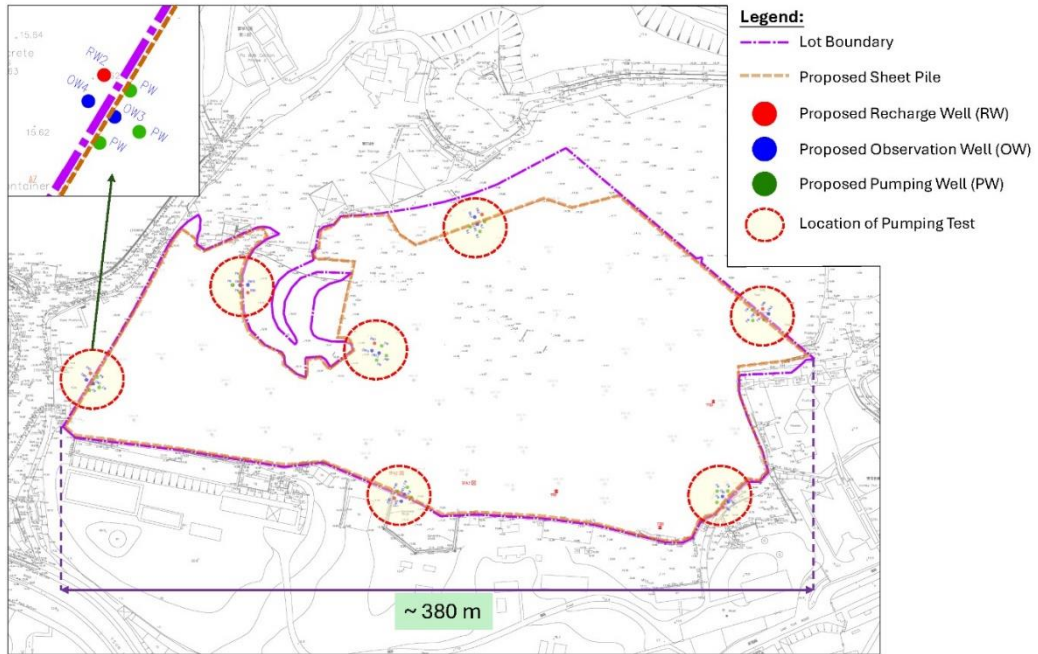


Figure 4-1: Layout Plan of Partial Pumping Tests at Construction Site in Tuen Mun Town

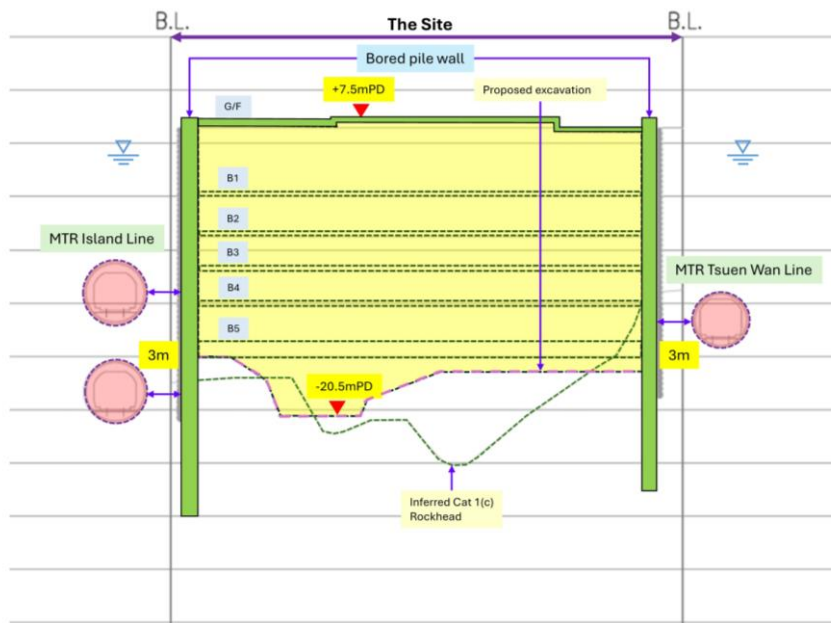


Figure 4-2: Section of Deep Excavation Works at Murray Road Site, Central

To verify the water tightness of the pile wall systems, as well as the implications to the surrounding structures, carrying out full-scale pumping tests in one go, in which the pumps shall be installed below the design excavation level for dewatering, would be the most direct approach for this case. However, the potential impacts to the MTR tunnels are unpredictable, and any excessive groundwater drawdown due to dewatering may induce uncontrollable ground settlement, significant stress changes and causing irrecoverable damages to the sensitive tunnel structures.

Having considered the sequence of work (top-down construction), tightness of the works programme and the safety concerns associated with nearby tunnels, the pumping tests for the development site at Central are divided into three stages. This approach aims to effectively control potential impacts resulting from dewatering

but without hindering the progress of excavation works. The figures below illustrate the steps of staged pumping tests at different ELS stages.

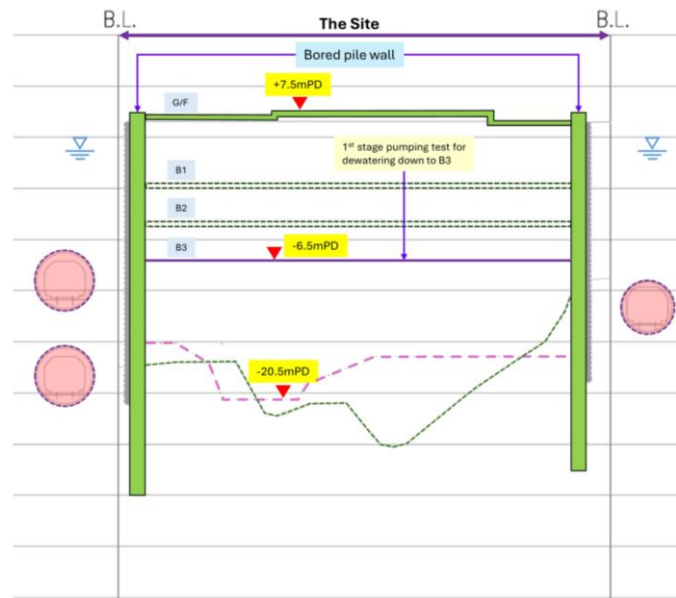


Figure 4-3: 1<sup>st</sup> Stage Full-scale Pumping Tests

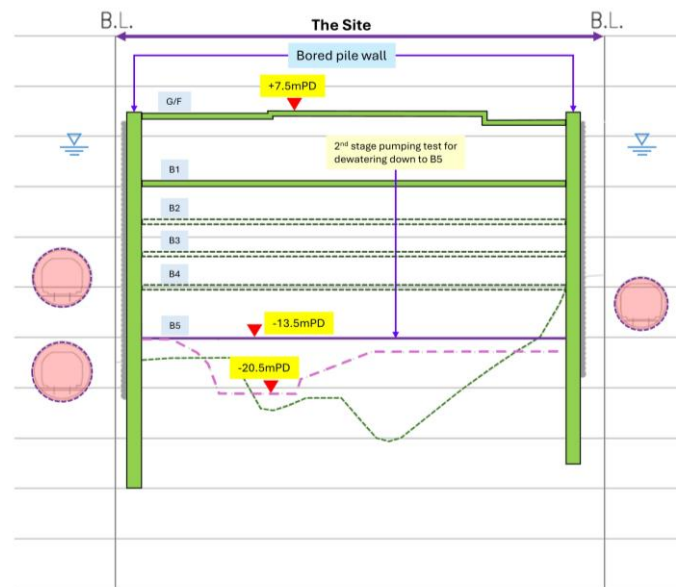
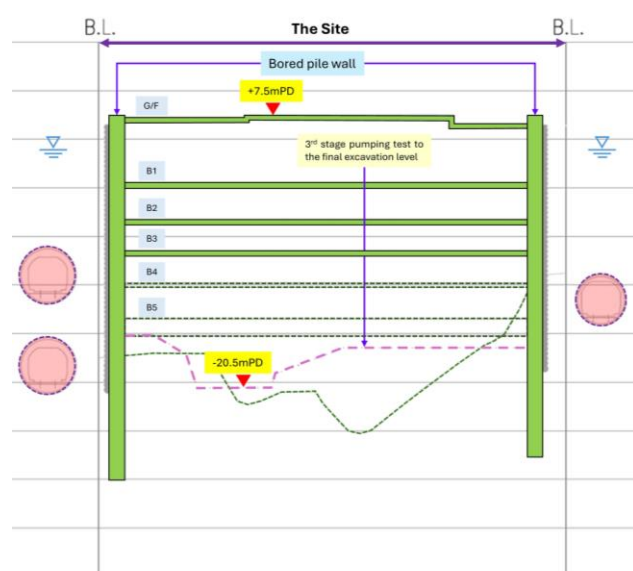


Figure 4-4: 2<sup>nd</sup> Stage Full-scale Pumping Tests

Figure 4-5: 3<sup>rd</sup> Stage Full-scale Pumping Tests

### 4.3 Excavation under Support by Tied-Back Wall

Comparing with the conventional excavation involving shoring and strutting, a retaining wall which is anchored or tied back into the unexcavated ground by rock anchors or soil nails is often a better solution as it provides construction space free of strutting, which is obviously a major benefit for the construction work. It is often a more economical option as large quantities of steel shoring and strutting can be avoided. However, as installation of the anchors beyond the site boundary is required, it often involves land matters which may be difficult to resolve. Therefore, tie back pile wall is much less commonly adopted in Hong Kong than other parts of China or other western countries.

In Hong Kong, the tied back forces of tie-back wall can be provided by soil nails or prestressed anchors which are either rods or strands of high strength steel. An anchor normally comprises a “grouted” length at its end penetrated into bedrock which derives the anchoring force and a “free” length between the bedrock and the wall. However, these anchors are seldom used as permanent supports to the retaining wall which would otherwise require long term monitoring if they are under prestress in accordance with Geospec 1 (GCO, 1989). Normally support to the excavation will finally be taken up by the permanent structure and the anchors are de-stressed and removed. The removal is to avoid obstructions to the future construction work below the adjacent grounds.

The followings are examples of development projects in Hong Kong involving deep excavation with tied-back wall system.

#### 4.3.1 A Residential Development Site at Tsing Yi

The site is located at the foothill of a natural hillside consisting of granitic geology. The development comprises 2 residential towers of 30 storeys high on 2 levels of common podium and one level of basement. The bedrock level is high and the structure can be supported by footings directly founded on bedrock. Vertical excavation of about 23m in depth was required for the construction of the foundation and the podium structure. Behind the excavation is the natural slope which remains about 150m in height after the toe was removed for development.

The excavation was supported by pipe pile walls with prestressed rock anchor during the construction stage. A typical section showing the excavation are shown in Figure 4-6. A photo showing the completed anchor system is shown in Plate 4-1. In the permanent stage, the vertical excavation is supported by the lateral soil loads together with the wind loads are the reinforced concrete podium wall and basement wall. The enormous soil loads together with the wind loads are finally transferred through the podium and basement structures to

the footing and resisted by the friction between the footing and granitic bedrock. Sufficient friction is made possible by the weight of the heavy superstructure.

For the 30 storeys towers, it had been shown that when the superstructure reached 26 storeys, the self-weight of the partially completed building was great enough to mobilize sufficient friction to counteract the earth loads. Therefore the prestressed anchors were destressed and removed after the completion of 26 storeys.

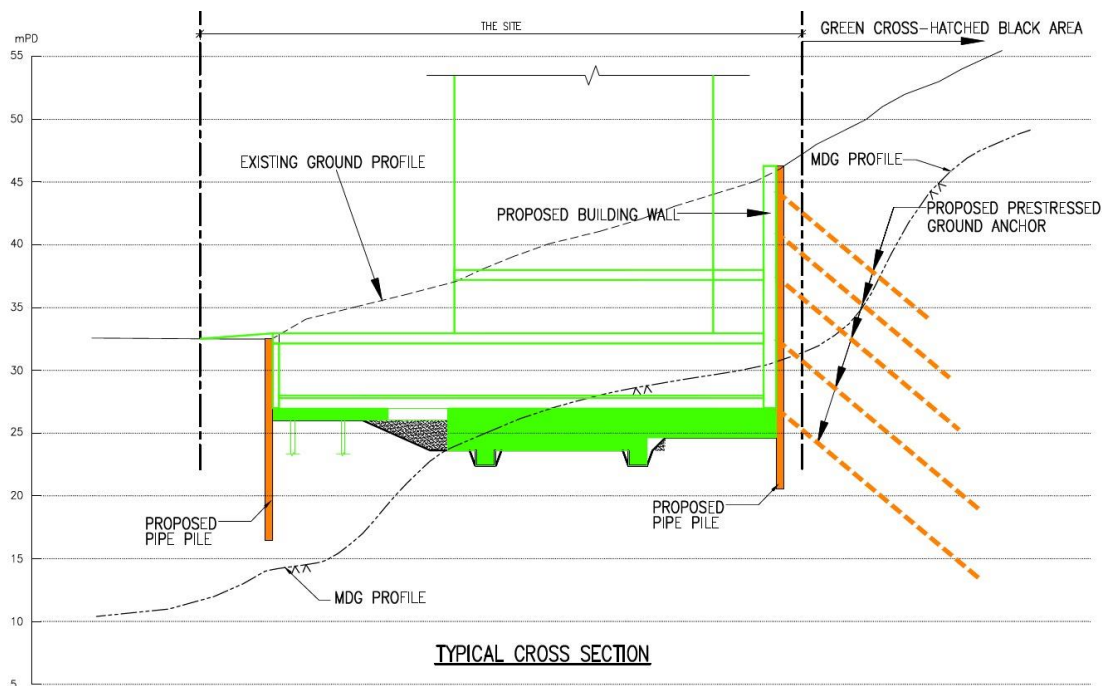


Figure 4-6: A Typical Cross Section of Tie Back Excavation of a Site in Tsing Yi



Plate 4-1: Photo Showing Completed Tie Back System in a Site in Tsing Yi

### 4.3.2 A residential development at Tai Wo Ping

The development site is at the foothill of Beacon Hill. It occupies an area of about 600000 sq. m. The site is entered via a cul de sac at the south and rise by 60m over a length of 250m to the northern lot boundary, the whole site being on hilly terrain. The development comprises 6 nos. of 23-storey buildings at the south and 26 nos. of houses of 3 to 4 storeys high at the north. Vertical excavation depth of 25m to 30m has to be carried out

along the northern boundary of the site. This cutting was temporarily supported by a bored pile wall comprising 3m to 3.5m diameter bored piles with tie backs in the form of prestressed ground anchors anchored from bedrock. A typical cross section across the bored pile wall, and a photo showing the completed bored pile wall are shown in Figure 4-7 and Plate 4-2 respectively.

In the long term, the 25-30m retained height of the anchored bored pile wall is reduced to 15m tall free cantilever wall. The reduction is made possible by casting a purposely 10m thick concrete block (as part of the pile cap) against the lower part of the pile wall, as shown in Figure 4-7. Upon completion of the bored pile foundations and pile caps, the prestressed rock anchors were destressed and removed. The development project was recently completed.

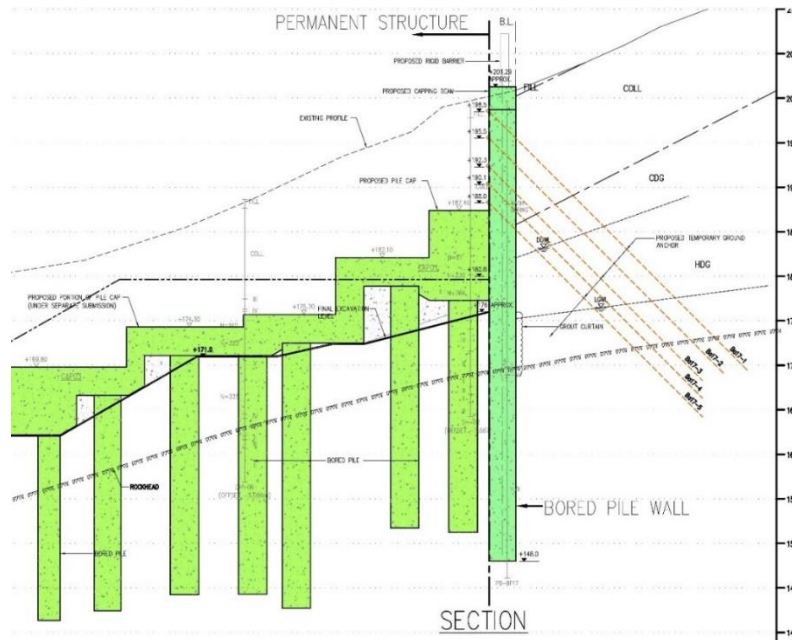


Figure 4-7: Typical Cross Section at the North Boundary of Residential Development in Tai Wo Ping



Plate 4-2: Photo Showing Completed Anchored Bored Pile Wall of Residential Development in Tai Wo Ping

### 4.3.3 A residential redevelopment at Deep Water Bay

The site is fronting onto the upper section of Deep Water Bay Road. It was occupied at the front portion by a single family house and at the rear portion by a series of stacking retaining walls and a cut slope before the redevelopment. The redevelopment comprises two houses and needs cutting into the rear slope to provide more level area. Thus, a vertical excavation of about 17m in height is required.

The proposed building plan layout leaves a space between the vertical cut and rear boundary. The granitic bedrock is also high at this site. Making use of these beneficial factors, a row of temporary soldier pile wall tied back by T40 rebars with anchorage from rock is constructed. A section is shown in Figure 4-8, together with a photo showing the completed tie back wall in Plate 4-3.

Footing foundations on bedrock are constructed to support the superstructure. A vertical cantilever reinforced concrete wall, which is supported by the footing is also cast against the soldier pile wall to replace its lateral load resistance in the long term. This scheme has resulted in significant saving in both time and cost as compared with the conventional shoring scheme.

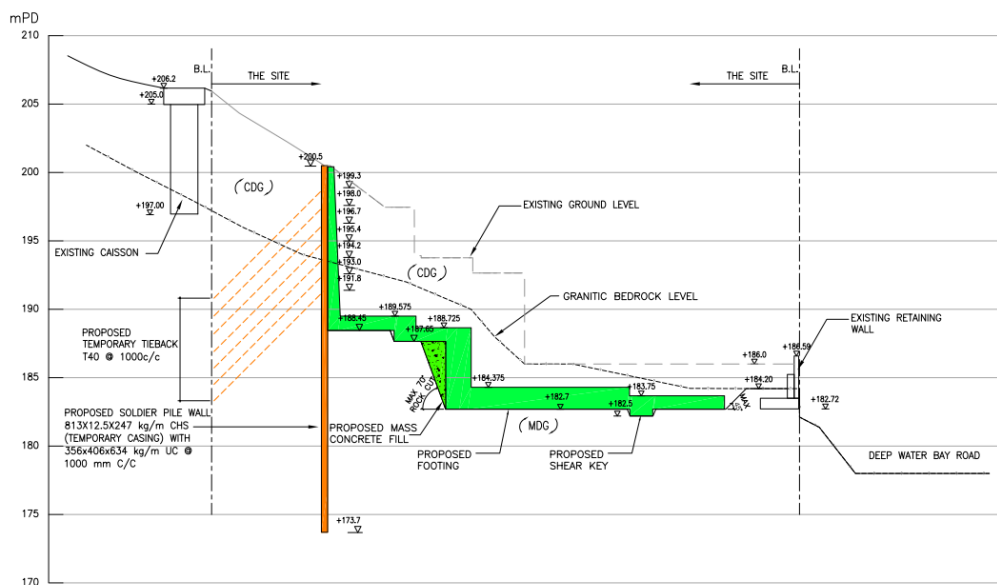


Figure 4-8: Typical Cross Section at the North Boundary of Residential Development at Deep Water Bay



Plate 4-3: Photo Showing Completed Anchored Soldier Pile Wall (with Tie Back) in Residential Development at Deep Water Bay

#### 4.3.4 A residential development at Stanley

A Grade I Historic Building is situated on a platform at the top of the site and needs to be preserved. The platform is supported by fill slopes around it. The development comprises a series of low rise buildings on the fill slopes. In order not to obstruct the heritage building, the top of the proposed buildings around it cannot be higher than its lowest floor level. As such, excavation needs to be carried out deeply into the existing slopes, to result in a vertical cutting about 20m in height. Figure 4-9 shows a typical section showing the existing topography, site geology and proposed formation level.

To support the 20m vertical cut, a row of soldier piles tied back by prestressed ground anchors was proposed. Since the anchors need to be removed before completion of the development, a cantilever reinforced concrete retaining wall supported by the raft footing foundation was proposed to form an integrated structure with the soldier pile. Such integration serves two purposes. Firstly, the bending action to be caused by the enormous soil loads would be resisted mainly by the reinforced concrete cantilever wall, which in turn is supported by a raft footing. Secondly, the sliding resistance to the soil loads would be provided by the shear capacity of the soldier pile wall. Figure 4-11 shows the proposed arrangement.

The issue with tie back walls was that the tie back, whether soil nails with sustainable loads or prestressed ground anchors, need to be monitored during its lifetime. This requirement was waived in 2023 through the revision of Geoguide 7 (GEO, 2023c). Monitoring is now only required during the construction period. Given this relaxation, a revised scheme is devised. This scheme comprises a row of 800mm diameter bored piles with tie back of 4T50 nails. This tied back wall will also act as permanent structure to retain the vertical cut. This will release the building foundation of the burden to resist the soil loads, resulting in a much lighter foundation. The heavy reinforced concrete cantilever wall in the original scheme is also not required. This revised scheme has resulted in significant saving in both time and cost as compared with the original scheme.

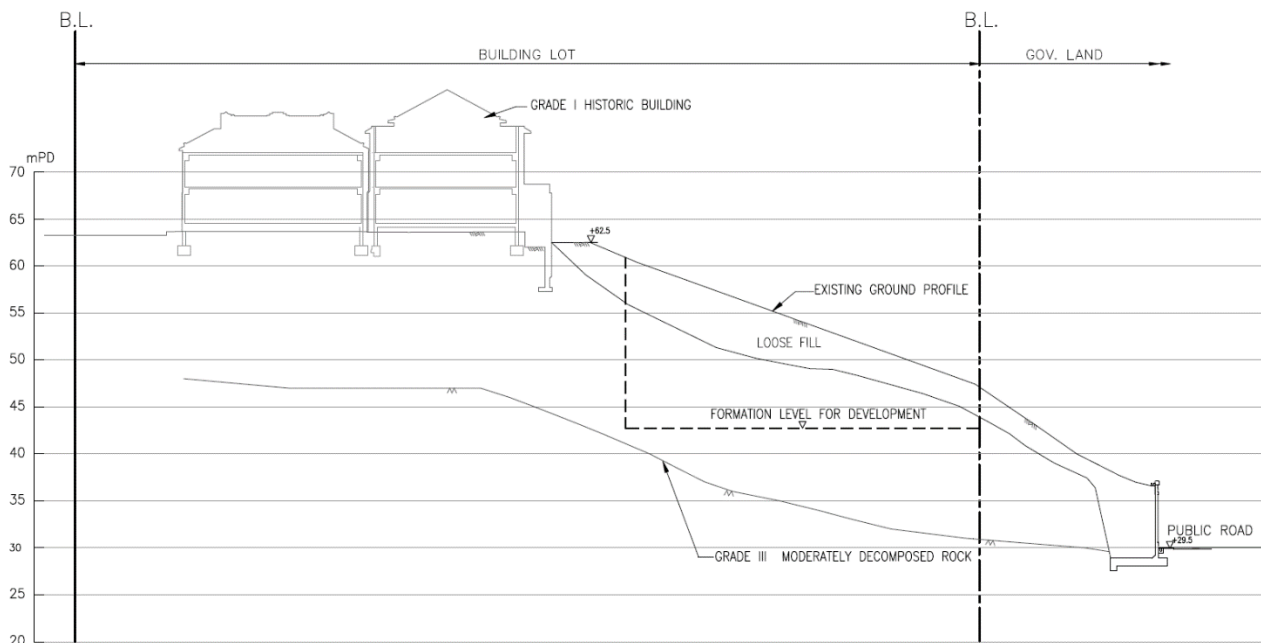


Figure 4-9: Geological Profile of Grade I Heritage Building and the Development in Stanley

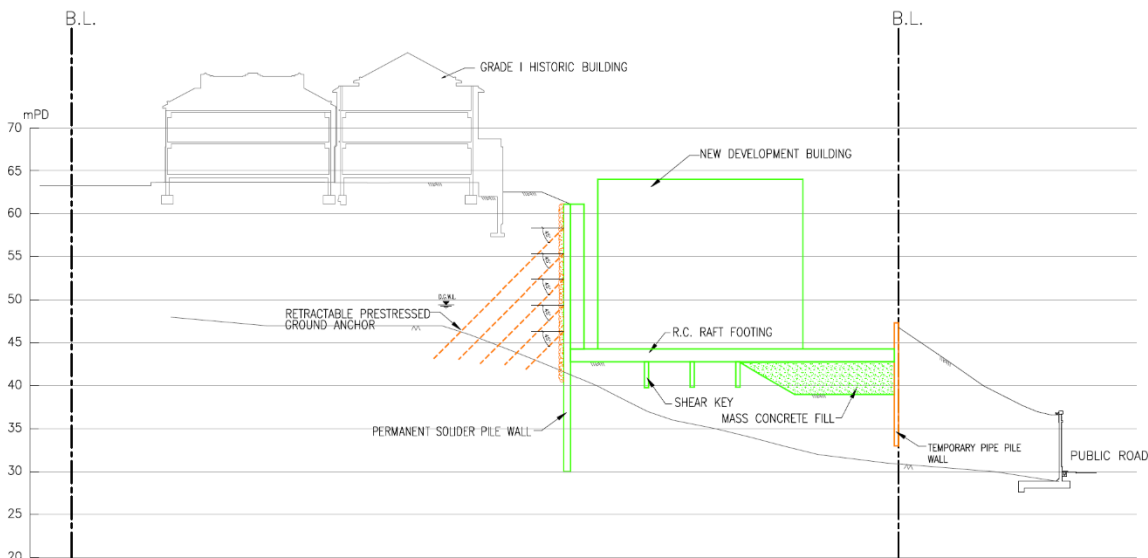


Figure 4-10: Grade I Heritage Building and the Development in Stanley - Scheme I Excavation Support

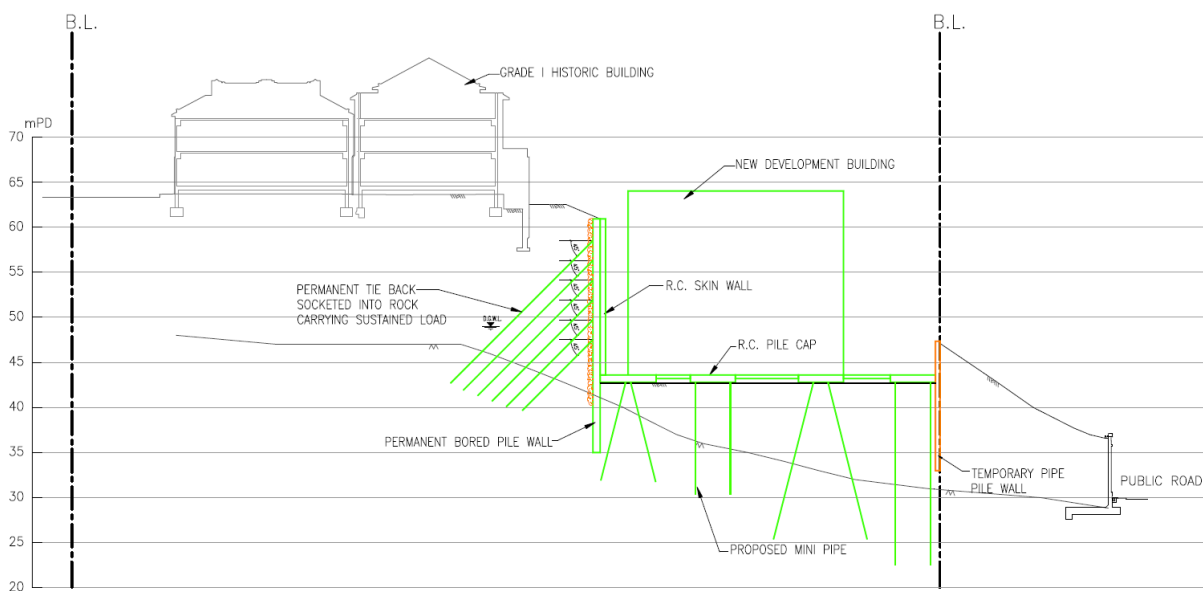


Figure 4-11: Revised Scheme II Support by Soil Nailing

#### 4.4 Introduction of 5A Approach in ELS Works

Prior to the publication of GEO Publication No. 1/2023 (GEO, 2023b), the settlement limit for excavations was empirically set at 25 mm, irrespective of the excavation depth. This approach used a three-tier triggering control mechanism called Alert-Alarm-Action (AAA) Levels, referenced from PNAP APP-137 (BD, 2018), has been commonly adopted for ELS works to monitor if the works have adverse impacts to nearby facilities and pose hazards to the public. In general, the Alert and Alarm levels are set at 50% and 75% of the Action Level respectively, and the Action Level has been taken empirically as 25mm in typical ELS works in Hong Kong. However, the 25mm limit proved impractical for deep excavations in some cases, leading to unreasonable

shoring designs, making ELS works impractical, prolong construction duration and sometimes dangerous for workers.

Response actions at Alert and Alarm Levels suffer from generality, reliance on the project team and contractor, and variability in promptness, adequacy, and significance. This lack of control undermines the purpose of providing advance warning. Stakeholders are often not promptly notified or consulted until the Action Level is reached, leading to unnecessary delays. Conversely, response actions at the Action Level can be overly broad, suspending all works, including essential works to maintain the stability of ELS works. The process of obtaining consent for remedial works after suspension further exacerbates delays. Furthermore, the absence of provisions for revising trigger values creates uncertainty after work resumes.

Since 2018, numerous ELS works projects in both the public and private sectors have encountered the "stop works" action limit. Once this limit is reached, all construction activities are suspended. Subsequent delays occur as time is needed to review the design based on the latest information about site conditions. This creates a snowball effect, as the resumption of construction activities depends on bringing the ground settlement situation under control. In certain projects, the process of resuming work and revising the trigger limit can span several hundred days. These delays have significant impacts on the economy and society, with the public inevitably facing the consequences of postponed construction activities.

The AAA limit is established to monitor the impact of ELS works on the surrounding ground and structures. Exceeding the AAA trigger values, predominantly due to ground settlement, can significantly affect roads and services. To effectively address these issues, specific response actions like repaving cracked road pavements, repairing damaged subgrade, and reinstating deformed pipelines can be easily implemented with prior agreements from stakeholders and authorities. Additionally, specific ground investigations using GCO probes and GPR surveys can be employed at different levels of ground settlement. This comprehensive approach not only addresses serviceability concerns for sensitive receivers but also considers stability requirements for ELS works, streamlining procedures, and improving the control mechanism's effectiveness and efficiency.

GEO (2023b) introduced an alternative 5A control mechanism (five-tier system by subdividing the third-tier response actions into three levels i.e. Alert-Alarm-Action Levels 1-3), to enhance the response actions according to the actual site conditions and increase the awareness of ground settlement monitoring by the stakeholders at different levels.

Table 9.1 of GEO Publication No.1/2023 provides recommended empirical limits for setting trigger values in the 5A approach. The Alert and Alarm levels are generally similar to the 3A approach. However, the Action levels are divided into three categories: Action Level 1, Action Level 2, and Action Level 3. For instance, the ground settlement limit for Action Level 1 is set at 20mm, Level 2 is defined as 0.3%He (limited to 60mm), and Level 3 is established at 0.5%He (limited to 100mm). This means that for a 20m deep excavation, the stop work action Level 3 is set at 100mm in the 5A approach, instead of the 25mm limit in the 3A approach. These revised limits are more reasonable and avoid the need for impracticably closely spaced strut arrangements.

By implementing the 5A approach, the control mechanism considers the actual site conditions and provides more appropriate trigger values. This approach ensures that response actions are tailored to the specific circumstances and allows for a more effective monitoring of ground settlement.

It is important to emphasize that the 5A approach is not simply to relax the "stop" works action limit, but to deal with ground settlement in a more rational way. At Action Level 1, the focus is primarily on addressing serviceability concerns related to road pavement and utilities. This entails carrying out necessary repair works to rectify and repave the pavement, as well as reinstating any deformed pipelines. Prior agreements with utilities undertakers and authorities are sought to ensure proper coordination and approval for these repair activities. Additionally, utilities undertakers and authorities will be notified and consulted if there is a need for repairs or continuous monitoring of the situation. This collaborative approach ensures that the necessary actions are taken to maintain the serviceability and integrity of road pavement and utilities in the face of ground settlement.

Upon reaching Action Level 2 (i.e. 0.3%He), additional investigations such as GPR surveys, CCTV inspections, and probing tests should be conducted to identify subgrade and service issues. The design should be evaluated to determine if cumulative settlement in subsequent excavation stages could exceed the estimated value and whether the performance of ELS remains acceptable.

When Action Level 3 is reached (i.e. 0.5%He), an Emergency Plan should be developed, which includes intermediate stage serviceability checks, a comprehensive investigation plan, and a Works Suspension Plan if necessary.

In comparison to the traditional 3A Approach, the adoption of the 5A Approach also emphasizes the importance of making engineering decisions based on the actual conditions of ELS works. Determining the

action level by considering its correlation with the maximum excavation depth is a more practical approach than adopting the empirical value of 25mm as the Action Level. Moreover, the flexibility of the 5A Approach allows for easier coordination of ELS works and remedial measures to address ground settlement concurrently with construction works, avoiding the need for complete works suspension at all times. The recommended empirical limits in the 5A Approach are summarized in Table 4-1:

Table 4-1 – Recommended Empirical Limits for Setting Trigger Values using 5A Approach in ELS Works, extracted and summarized from Table 9.1 of GEO (2023b)

Instrument	Criterion	Criteria of				
		Alert Level	Alarm Level	Action Level 1	Action Level 2	Action Level 3
Ground Settlement Monitoring Marker	Total settlement	10 mm (or 50% of Action Level 1)	15 mm (or 75% of Action Level 1)	20 mm (Shall be reviewed according to the actual site conditions and the corresponding requirements from the stakeholders)	0.3% $H_e$ (Shall be bounded between 25 mm to 60 mm)	0.5% $H_e$ (Shall be bounded between 30 mm to 100 mm)
Services monitoring marker	Angular distortion	1:600	1:500	1:400	1:350	1:300
Building monitoring marker	Angular distortion	1:1000	1:750	1:600	1:550	1:500
Note: $H_e$ is the maximum excavation depth						

## 5 FOUNDATION WORKS

### 5.1 Historical Review

As rock is the strongest founding stratum for building structures, it has been a practice to determine its allowable bearing pressure as a statutory or codified value for safety and convenience in design. A historical review is first conducted on the statutory or codified provisions of the allowable bearing pressure of rock in Hong Kong.

#### 5.1.1 Igneous Rock

The earliest stipulation of allowable bearing pressure of rock of Hong Kong is that in Building (Construction) Regulation (B(C)R) 1956 (Hong Kong Government, 1956). The B(C)R describes 3 types of rock in its Table VIII which are extracted as follows in Table 5-1.

The descriptions of the rock are too general. There would likely be ambiguity in assigning the type of rock to a founding rock stratum. It was not until 1975 that B(C)R 1975 (Hong Kong Government, 1975) classifies “rock (granite and volcanic)” into 4 types (a) to (d) with quantitative descriptions in terms of total core recovery (TCR) as extracted in Table 5-2.

Table 5-1 – Extract of Table VIII of B(C)R 1956 for Maximum Permissible Pressure on Rock under Foundations

Type of soil under foundation			Maximum permissible pressure
Rock	(a)	Hard, sound rock, massive	40T/sq.ft (4301kPa)
	(b)	Medium hard rock	25T/sq.ft (2688kPa)
	(c)	Soft rock	8T/sq.ft (860kPa)

Table 5-2 – Extract of Table IX of B(C)R 1975 for Safe Bearing Capacities of Rock under Foundations

Type of ground		Safe bearing capacity kPa
Rock (granite and volcanic)		
(a)	massive crystalline rock in hard sound condition (core recovery greater than 85%)	5,000
(b)	medium hard rock (core recovery greater than 50%)	3,000
(c)	soft rock (core recovery of 50% and less)	1,000
(d)	closely jointed and heavily shattered rock	To be assessed after inspection and testing

Under the heading “Rock (granite and volcanic)”, the table applies only to granitic and volcanic rocks which are igneous rocks. The same table has been reproduced in B(C)R 1985 (Hong Kong Government, 1985).

The Buildings Department issued an AP/RSE Practice Note PNAP 141 in December 1990 and October 1995 which contained revisions in rock presumed bearing pressures as extracted in Table 5-3. An additional category of rock having presumed bearing pressure of 7,500kPa has been added. The requirements of rock to attain the presumed bearing values (pressure) are also more comprehensive by including rock grades and strengths as in addition to TCR.

The first Foundation Code of Hong Kong was promulgated in 2004 titled “Code of Practice for Foundations” (CoPF 2004) (BD, 2004). Comparing with PNAP141 for presumed bearing pressures of rock, one more category (1(a)) of igneous rocks (granite and volcanic) is added with the highest presumed allowable bearing pressure of 10,000kPa under more stringent requirements in rock grade, total core recovery and rock strengths. Details for rock are extracted in Table 5-4. For simplicity, only the new Category 1(a) of the CoPF 2004 is listed while the remaining grades 1(b), 1(c) and 1(d) are identical to 1(a), 1(b) and 1(c) of PNAP141 respectively.

In addition, CoPF2004 lists the presumed allowable bond or friction between rock and concrete for piles in its Table 2.2 reproduced in Table 5-5.

Table 5-3 – Extract of Appendix A of PNAP 141 for Rock

Category		Presumed Bearing Value, kPa
1.	Rock (granite and volcanic)	
	(a) Fresh to slightly decomposed strong rock of material weathering grade II or better, with a total core recovery of more than 95% of the grade and minimum uniaxial compressive strength of rock material (UCS) not less than 50MPa (equivalent point load index strength $PLI_{50}$ not less than 2MPa).	7,500
	(b) Slightly to moderately decomposed moderately strong rock of material weathering grade III or better, with a total core recovery of more than 85% of the grade and minimum uniaxial compressive strength of rock material (UCS) not less than 25MPa (equivalent point load index strength $PLI_{50}$ not less than 1MPa). Moderately decomposed, moderately strong to moderately weak rock of material weathering grade III or IV or better, with a total core recovery of more than 50% of the grade.	5,000
	(c)	3,000

Table 5-4 – Extract of Table 2.1 of the Code of Practice for Foundations (2004)

Category	Description of rock or soil	Presumed allowable Bearing Pressure (kPa)
1(a)	Rock (granite and volcanic) Fresh strong to very strong rock of material weathering grade I, with 100% total core recovery and no weathered joints, and minimum uniaxial compressive strength of rock material (UCS) not less than 75MPa (equivalent point load index strength $PLI_{50}$ not less than 3MPa).	10,000

Table 5-5 – Extract of Table 2.2 of the Code of Practice for Foundations (2004)

Category of rock as defined in Table 2.1	Presumed allowable bond or friction between rock and concrete or grout for piles (kPa)	
	Under compression or transient tension	Under permanent tension
1(c) or better	700	350
1(d)	300	150

Wong (2014) summarized the development of the presumed rock bearing pressures used in the foregoing B(C)Rs, codes and PNAPs. The “Code of Practice for Foundations 2017” (CoPF 2017) promulgated in 2017 (BD, 2017) retains the same requirements for presumed allowable bearing pressures of the CoPF 2004 for igneous rock, except that fresh to slightly decomposed strong igneous rock is allowed when assessing the weathering grade of the Category 1(a) rock.

In 2023, the Geotechnical Engineering Office (GEO) issued GEO Technical Guidance Note No. 53 (TGN 53) to supersede the corresponding bearing pressure values in GEO Publication No. 1/2006. The presumed allowable bearing pressures of igneous rocks Categories 1(a), (b) and (c) in CoPF 2017 is increased to 12,500kPa, 10,000kPa and 7,500kPa respectively. In addition, the presumed allowable bond or friction with

rock for Categories 1(a) and 1(b) have also been increased to 1000kPa under compression or transient tension and 500kPa under permanent tension. The Guidance Note justifies the upgrading as follows:

“Over the years, instrumented pile loading tests have been carried out for piles founded on various rock formation, which included the comprehensive pile testing programme for Airport Railway, West Rail and the West Kowloon District Development and private development at Sai Sha. GEO has collected and analysed these pile test data for improving knowledge and practice in foundation engineering. The instrumented pile loading test results indicate that there are substantial safety margin of the proven bearing capacity and shaft friction when compared with the recommendations given in the publication. Hence, updated guidelines on the presumed allowable bearing capacity and bond friction on igneous rocks are derived.”

### 5.1.2 *Meta-sedimentary rock*

Until the promulgation of CoPF 2017, the presumed allowable bearing pressure was applicable to igneous rock only. So before 2017 for sites with meta-sedimentary rock, the allowable bearing pressure need to be determined on a case by case basis. CoPF 2017 classifies meta-sedimentary rock as Category 2 rock with requirements extracted in Table 5-6:

Table 5-6 – Extract of Table 2.1 of the Code of Practice for Foundations 2017

Category	Description of rock or soil	Presumed Bearing Pressure, kPa
2	Meta-Sedimentary rock Moderately decomposed, moderately strong to moderately weak meta-sedimentary rock of material weathering grade III or better, and with not less than 85% TCR of the designated grade.	3,000

There are recent cases that allowable bearing pressures higher than 3000kPa are adopted, which will be elaborated in Section 5.2.1.

### 5.1.3 *Marble*

Marble is formerly limestone that has undergone metamorphosis under heat and pressure by which the rock becomes stronger by being denser and more compact. However, being reactive (though sparingly) with water containing carbon dioxide, marble is usually associated with a karstic upper surface with dissolution features. Large cavities with or without infilled material may also be present within the marble mass affecting foundation design and construction.

The discovery of extensive occurrence of marble rock in some locations of Hong Kong started from the piecemeal identifications of marble bedrock in Yuen Long, as pointed out by Houghton & Wong (1990) in which the special geological features of the district characterized by interbedded meta-sedimentary and marble rocks were described. In Houghton & Wong (1990), special site investigation requirements, construction of the geological site models, foundation design were discussed in detail, with reference to experiences gained from 5 number of sites in the district. With the contents and insights, Houghton & Wong (1990) highlights the appropriate approaches in foundation design and construction for sites with marble geology. Since 1990, the area in North-western New Territories underlain by karstic geology has been designated as Scheduled Area No. 2 and brought under special geotechnical control.

In 1994, GEO issued GEO Report No. 29, promulgating a system developed by Y C Chan for zoning and classification of marble sites. The system is primarily based on a parameter named “Marble Quality Designation” (MQD), which measures quality and extents of fractures of marble rock in individual drillholes. Upon determination of the range of MQD of a marble mass, the “marble class” of the rock mass can be assigned in accordance with a pre-established scale which ranks from “Very good” to “Very poor”. So the system aims to facilitate interpretation of the possible degree and extent of dissolution zones, and assist in the assessment of the relative load-bearing characteristics. The applications of the system to 2 sites have been demonstrated in detail in GEO (1994b) and Chan et al. (1994).

The classification system was incorporated into the CoPF 2017. The Code further ranks suitability of the rock mass in acting as a bearing stratum in accordance with the “marble class” in the Notes under its Table 2.5.

Prescriptive allowable bearing pressures have not been assigned to marble in GEO Report 29 or CoPF 2017. GEO issued Technical Guidance Note No. 26 in 2022. It considers that the foundation properties of sound marble are similar to that of igneous rock and therefore assigns the presumed allowable bearing pressure of 7500kPa and 5000kPa, under the fulfilment of marble class, rock grade, TCR and rock strengths as identical to that of the igneous rocks. The Building Authority included these values of bearing pressure in August 2023 through amendments of CoPF 2017 and PNAP APP-18.

## 5.2 Case studies

### 5.2.1 Large Diameter Bored Piles on Meta-Sedimentary Rock at Kiu Tau Wai, Yuen Long

The bedrock of the site for founding of large diameter bored piles is meta-sedimentary rock, varying at 24m to 52m below ground. In accordance with CoPF 2017, the rock is designated as Category 2 and the presumed allowable bearing pressure is 3000kPa for rock of material weathering grade III or better, and with not less than 85% TCR of the designated grade. Rock strengths in terms of UCS and  $PLI_{50}$  do not form part of the criteria for attaining the presumed allowable bearing pressure of 3000kPa in accordance with CoPF 2017.

Nevertheless, UCS and  $PLI_{50}$  tests have been performed, giving results of 29.4MPa to 127.8MPa and 1.2MPa to 9.9MPa respectively. These rock strengths are higher than the threshold requirements of granite and volcanic rocks for attaining presumed allowable bearing pressure of 5000kPa, which are only 25MPa for UCS and 1MPa for  $PLI_{50}$  under Category 1(c) of the CoPF 2017. So, there should be good justification for increasing the presumed allowable bearing pressure from the codified value of 3000kPa to 5000kPa, so as to achieve economy in design.

Owing to the lack of test data on meta-sedimentary rock, it is considered prudent to include or enhance other criteria apart from strength tests for elevating the allowable bearing pressure to 5000kPa. The minimum TCR is raised from the normally adopted 85% to 90% to provide a higher level of confidence.

Another criterion is the introduction of Rock Mass Rating (RMR) approach. RMR is an integrated parameter for properties of a rock developed by Bieniawski (1973) and (1989). It is the sum of the rating of the rock for 5 aspects including rock core strength, rock quality designation, joint spacing, joint conditions and the presence of ground water which constitutes a comprehensive description of the quality of rock. High rating of RMR indicates strong rock generally. The GEO Publications No. 1/2006 (GEO, 2006) correlates RMR with allowable bearing capacity  $q_a$  in its Table 6.5 and Figure 6.8 extracted as follow. The recommended allowable bearing pressure is a conservative determination as compared with the existing test data. To achieve an allowable bearing pressure of 5000kPa, RMR of 50 is required. Therefore, the criterion of  $RMR \geq 50$  is also included.

The following Table 5.7 summarizes the proposed criteria for determining allowable bearing capacity for the Kiu Tau Wai Site.

Table 5-7: Proposed Criteria for the Presumed Allowable Bearing Pressure of the Kiu Tau Wai Site

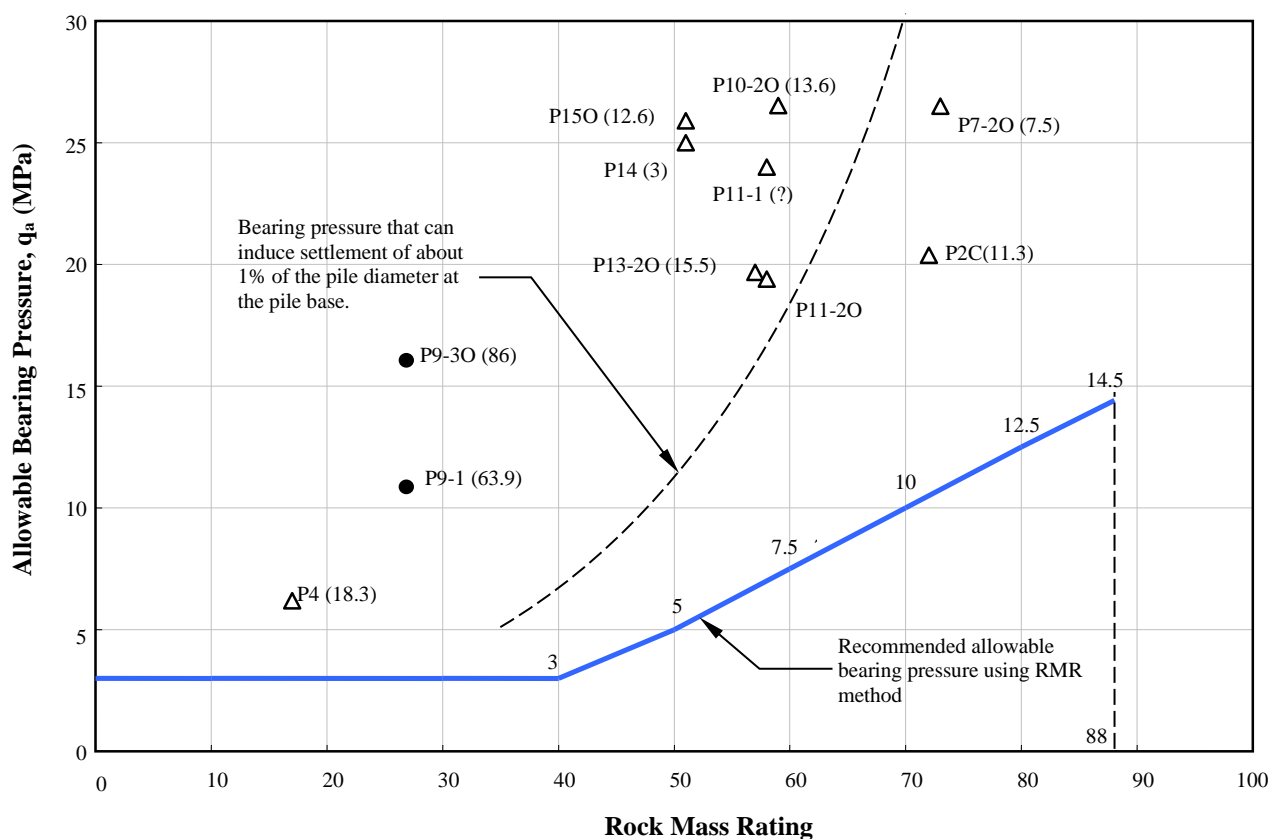
		Value	CoPF 2017 Requirements
<b>Design Parameters</b>	Allowable bearing Capacity	5000kPa	3000kPa
<b>Acceptance Criteria</b>	Uniaxial Compressive Strength (UCS) Or Point Load Index Strength ( $PLI_{50}$ )	Min. 25MPa (UCS) Min. 1MPa ( $PLI_{50}$ )	Nil
	Material Weathering Grade	Grade III or better	Grade III or better
	Total core recovery (TCR)	Min. 90%	85%
	Rock Mass Rating (RMR)	Not less than 50	Nil

Full scale load tests on 2 trial piles with full instrumentations were conducted, each under 3 times of its working load. Performances were satisfactory, showing that the ultimate bearing pressure was well in exceedance of 15000kPa. Based on the load test results, the actual factors of safety are greater than 3 for the allowable bearing pressure of 5000kPa.

**Table 6.5 – Allowable Bearing Pressure Based on Computed RMR Value**

Parameters	Rock Mass Rating (RMR)			
	< 40	50	70	88
Allowable bearing pressure, $q_a$ (kPa)	3,000	5,000	10,000	14,500

- Notes : (1) For RMR < 40, the rock mass should comprise at least 50% of moderately decomposed, moderately strong to moderately weak rocks. Refer to Table 2 of Geoguide 3 (GCO, 1988) for classification of the strength of rock materials. In common granitic and volcanic rocks in Hong Kong, this corresponds to a weathering grade better than IV.
- (2) The rock mass within the zone of influence of the foundation loads should be assessed when computing the RMR values. The minimum zone of influence should not be less than three times the diameter of the pile base.
- (3) Interpolate between allowable bearing pressures for intermediate RMR values greater than 40.
- (4) The ratings for individual parameters are given in Table 6.4.
- (5) This table is applicable where the stability of the rock mass is not subject to the effect of adversely oriented discontinuities.
- (6) If allowable bearing pressure,  $q_a$ , determined by RMR is greater than  $\sigma_c$ , use  $q_a = \sigma_c$ .



- Legend :
- = End-bearing resistance substantially mobilised
  - ) = Degree of mobilisation of end-bearing resistance unknown (i.e. not fully mobilised)
  - (64) = denotes the measured settlement at pile base in mm

- Notes : (1) Refer to Appendix A for details of pile tests.  
 (2) Higher bearing pressure can be used when substantiated by pile loading tests.

**Figure 6.8 – Allowable Bearing Pressure Based on RMR Value for a Jointed Rock Mass beneath Piles**

Figure: 5-1: Extract of Table 6.5 and Figure 6.8 of GEO Publication No. 1/2006 (GEO, 2006)

### 5.2.2 Large Diameter Bored Piles on Sedimentary Rock at Pak Shek Kok

Large diameter bored piles were proposed for a site at Pak Shek Kok where sedimentary rock is at around 60m to 70m below ground. As CoPF 2017 has not included the presumed allowable bearing pressure for sedimentary rock, its value needs to be assessed.

As a start of the assessment, the criteria adopted for the Kiu Tau Wai site above were considered. Since sedimentary rock is generally weaker than meta-sedimentary rock, a smaller allowable bearing pressure of the same grade is anticipated. After careful consideration, for a minimum TCR of 90% and the same strength requirements for that of Category 1 (c) attaining an allowable bearing pressure of 5000kPa for granite and volcanic rock, an allowable bearing pressure of 3000 kPa was proposed. The RMR was lowered from 50 to 40 for 3000 kPa in accordance with Figure 6.8 of GEO Publication No 1/2006. An additional criterion by limiting the Fracture Index (FI) of rock to 16.6, which was based on site specific considerations, was also conservatively imposed. The proposed criteria are listed in Table 5-8 as follows:

Table 5-8: Proposed Criteria for Founding of Large Diameter Bored Piles in Pak Shek Kok Site

Design Parameters	Allowable bearing Capacity	Value
Acceptance Criteria	Allowable bearing Capacity	3000kPa
	Total core recovery (TCR)	Min. 90%
	Weighted-Average of Rock Mass Rating (RMR) from top of Inferred Engineering Rockhead to 5m below or three times the diameter of the pile base whichever the deeper.	Not less than 40
	Fracture Index (FI) of Rock Materials below Inferred Engineering Rockhead of Grade III of better rock.	Not more than 16.6
	Uniaxial Compressive Strength (UCS) or Point Load Index Strength (PLI <sub>50</sub> ) within one time pile base diameter below the pile base	Not less than 25MPa (UCS), Not less than 1MPa (PLI <sub>50</sub> )

### 5.2.3 Shaft Grouted Bored Pile at a Site at Caine Road, Mid-Levels

The bedrock level of the site at Caine Road, Mid Levels is from 80 to 100m below ground where end-bearing pile was not a practicable and economical foundation option. Shaft grouted bored pile was therefore proposed which derived its load bearing capacity mainly from the shaft friction with soil. A typical geological section is shown in Figure 5-2, indicating that the Site is overlaid in succession by fill, colluvium and CDG .

The soil friction on a plain pile shaft is low and the pile capacity with a constructible depth is too small to support the proposed 50-storey building. Therefore, shaft grouted technique was used to increase the pile shaft friction. Shaft grouting is carried out initially by cracking the concrete surface with high water pressure at the initial set of the concrete, and then followed by injecting cement grout into the surrounding soil. The permeation grout along the pile shaft will substantially increase the frictional capacity of the pile.

Shaft grouted bored piles had not been put into actual use in Hong Kong before this project. Several trial piles using the same technique were carried out under West Rail Project but had not been put into practical use (Ng, et al 2001). Shaft grouted barrette had been adopted for several other projects beforehand. Reference was therefore made to the data of those test piles to obtain the design parameters for the shaft grouted bored piles.

The proposed ultimate and allowable frictional coefficients for colluvium and CDG are listed in Table 5-9.

Two instrumented trial piles of 1.2m diameter were carried out in 2011 and the test results indicate that the above ultimate frictional values can be readily attained. (Wong, 2014)

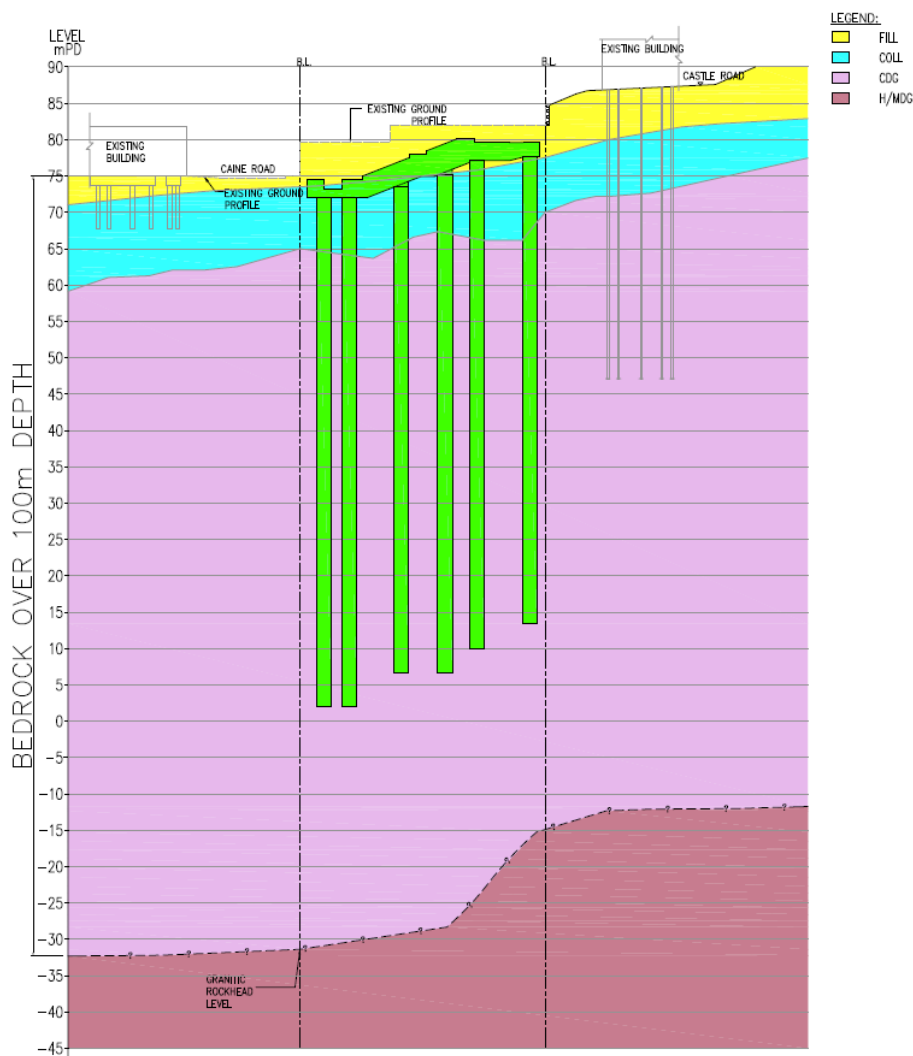


Figure 5-2: A Typical Geological Section through the Caine Road Project

Table 5-9: Proposed Parameters for Shaft Grouted Bored Piles at Caine Road Site

Soil Stratum	Allowable Friction (kPa)		Ultimate Friction (kPa)	
	Related to N	Capping Value	Related to N	Capping Value
Fill	0	0	0	0
Colluvium	1.8N	70	3.6N	140
CDG	1.2N	70	2.4N	140

#### 5.2.4 Shaft Grouted Bored Piles at a site at Wellington Street

Shaft grouted bored piles was again proposed as a site at Wellington Street where the bedrock is more than 150m below ground.

With the test results obtained from the Caine Road site, a higher frictional values for CDG were proposed for this site. The ultimate friction increased from 2.4N to 4.5N with the maximum value also increased from 140kPa to 390kPa. Owing to the increase, the factor of safety was increased from 2 to 3. Two trial piles of 1.2m diameter were carried out to justify the proposed friction values. For one of the trial piles, soft toe was formed and it was intended to load test it to failure. The results show that even when the test load well exceeded 3 times the allowable working load and had reached the limit of the jacking machine, the trial pile was still behaving in a satisfactory manner. The proposed design parameters are summarized below in Table 5-10. If the design

parameters for the Caine Road site were adopted for the Wellington Street site, the pile length would have to be increased from 45m to 65m as indicated in Figure 5-4.

Table 5-10: Proposed Design Parameters for Shaft Grouted Bored Piles in Wellington Street Site

Soil Stratum	Allowable Friction (kPa)		Ultimate Friction (kPa)	
	Related to N	Capping Value	Related to N	Capping Value
Fill	0	0	0	0
CDG	1.5N	130	4.5N	390

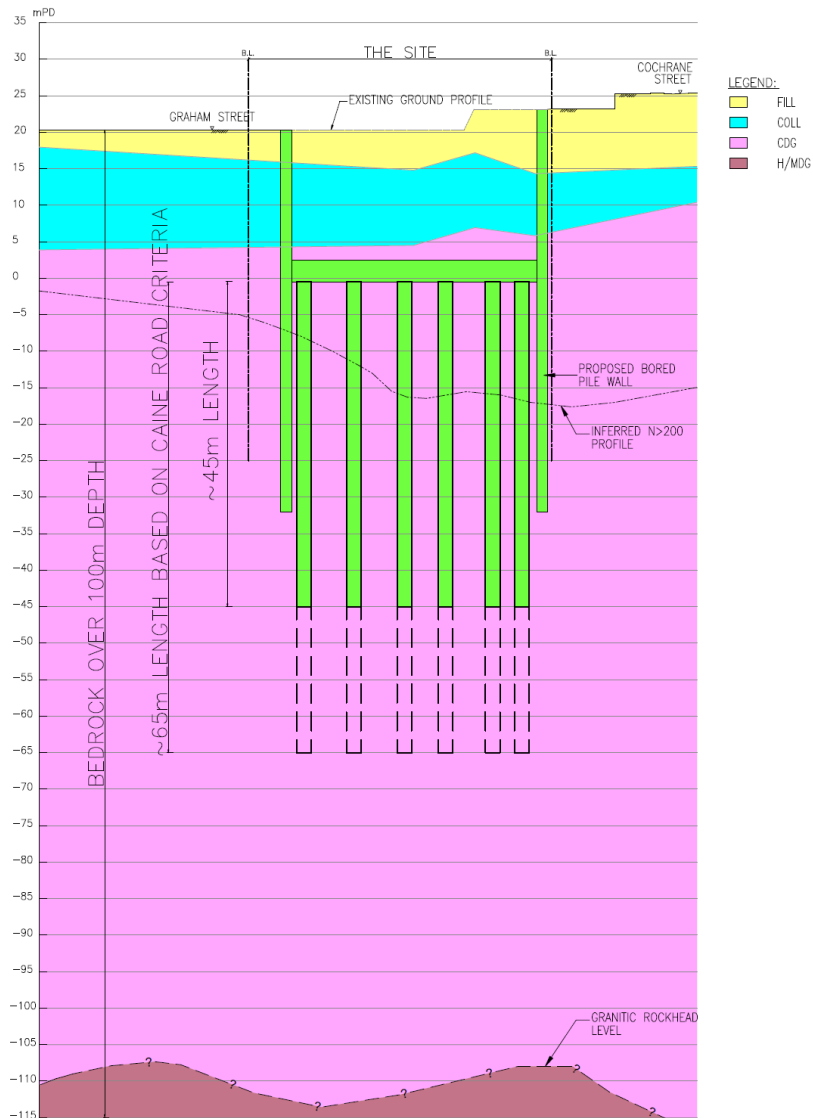


Figure 5-4: Shaft Grouted Bored Piles for Wellington Street Site

### 5.2.5 Shaft Grouted Pre-bored H-Piles at a Site at Staunton Street

Shaft grouted pre-bored H-pile was proposed for a small site at Staunton Street as the bedrock is more than 120m below ground. The shaft grouting technique was similar to that of bored pile. A typical geological section is shown in Figure 5-5.

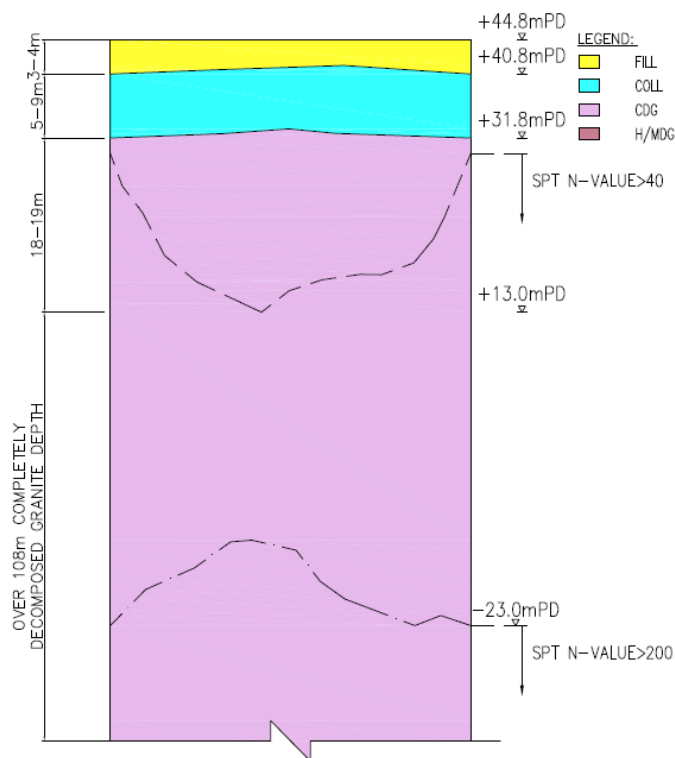


Figure 5-5: Typical Geological Section for Site at Staunton Street

The piles comprise the installation of 305x305 x223Kg/m steel UBP embedded in cylindrical grout columns of 610mm diameter. They are similar to prebored H-piles socketed in rock except that the pile capacity is gained from the enhanced soil friction around the pile shaft.

In determining the friction values, reference was made to those for continuous flight auger pile (CFA pile), for which an allowable frictional coefficient of 1.6N kPa with N capped at 40 were adopted. CFA piles are constructed by a rotating a continuous flight hollow stem auger into the ground to the designed depth. Once the auger head reaches the pile toe level, concrete is pumped through the hollow stem to fill the drilled shaft to form the pile. This pile type was popular in Hong Kong in the 1970s and 1980s when the piling works were near very old buildings as the installation caused little vibration and disturbance. Wong & Tse (2001) has reported the use of CFA piles floating on karstic marble for a site at Yuen Long. The major drawback of CFA piles is when boulders are encountered, the auger has difficulties to go through. After down-the-hole ODEX drilling machine was available in 1980s, it has seldom been used in Hong Kong.

Two trial piles were installed and tested in 2013 to verify the design parameters. The working geotechnical pile capacity was taken as 6000kN and the test load was 3 times this value. The test results were satisfactory. The design parameters are listed in Table 5-11 below:

Table 5-11 – Proposed Design Parameters for Shaft Grouted Bored Piles in Staunton Street Site

Soil Stratum	Allowable Friction (kPa)		Ultimate Friction (kPa)	
	Related to N	Capping Value	Related to N	Capping Value
Fill	0	0	0	0
CDG	1.6N	64	4.8N	192

## 6 CONCLUDING REMARKS

Hong Kong is a densely populated city with limited land and is facing unique challenges with heavy development beside hilly terrain and with varying geology. Geotechnical engineers play an important role in overcoming these challenges effectively. This paper has discussed progression of slope works, natural terrain hazard studies, excavation and lateral support works and deep foundations in Hong Kong. It is noted that the novel practices have been the results of response to the site specific constraints and the initial solutions are normally on a conservative side to ensure safety. Examples are soil nails with sustainable loads, allowable bearing pressure for rock, friction values for shaft grouted piles, etc. As more experience and data are garnered, the initial parameters, which are normally on the conservative side, should be reviewed and updated, if the test data support such improvements. Such continuous evolution can facilitate the resilience and sustainability in the long run.

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