Advancement in Geotechnical Practice for Smarter and Greener Projects Delivery

Sammy PY CHEUNG *, Lawrence KW SHUM, Raymond CH KOO

Civil Engineering and Development Department, Geotechnical Engineering Office, Government of Hong Kong SAR

*Corresponding author
doi: https://doi.org/10.21467/proceedings.159.16

ABSTRACT

The disastrous landslides in 1972 proved to be the turning point in the evolution of geotechnical engineering in Hong Kong, as the Government decided to establish the Geotechnical Engineering Office (GEO) to manage the geotechnical hazards. Geotechnical profession in Hong Kong has prospered ever since and over the years, local geotechnical practice has been subtly put together with the collaborating efforts from the Government, academia and practitioners. Geotechnical engineering is a challenging discipline, as it deals with natural material that are highly variable in their compositions, characteristics and engineering properties. Many methodologies and analyses in geotechnical engineering are not exact sciences and have been developed based on experience, simplifications and assumptions. Inevitably, geotechnical practice is embedded with some degree of conservatism to allow for the uncertainties. On the other hand, developments in Hong Kong have always been squeezed into a tight construction programme and are subject to a highly regulatory framework. These constraints may have impeded the advancement of geotechnical practice from innovative perspective. In recent years, the Government has made significant investments on infrastructural developments to compete with other international financial centres. There are increasing demands for the industry to boost the productivity whilst enhancing safety, quality and sustainability in the delivery of construction projects. Maintaining normalcy in geotechnical practice cannot meet the infrastructural investments and demands of society. Innovation in practice has always been a priority in the GEO and this always calls for a paradigm shift to our understanding of the geotechnical practice. The GEO has been working with practitioners, academia and other Government authorities in materialising advancements that would enable a smarter, leaner and greener project delivery portfolio. This paper discusses the rationale and considerations behind some of the advancements that have important benefits in realising leaner and greener construction when executing geotechnical works in site formation, excavation and foundation.

Keywords: Site formation

1 Introduction

Land is valuable commodity and it has long been the Government major revenue. Unfortunately, Hong Kong has a mountainous topography and the scarcity of flat land had plagued Hong Kong since the mid-nineteenth century. In the early days, reclamation on the central harbourfront provided the much-needed land for commercial activities. Hillsides on the Hong Kong Island were also terraced for buildings and roads to provide dwellings for people. The population bloom started in 1950s when Hong Kong began to develop as a manufacturing centre. There was an immense pressure for making more land to support the rapid economic growth and accommodate the population. The rapid surge of population in the 1960s and 1970s had resulted in intense urbanization on the fringes of the hillsides in many parts of Hong Kong. Many new migrants were living in flimsy squatter structures erected illegally on the hillsides. Inevitably, many slopes were formed in association with these hillside developments.

Proceedings DOI: 10.21467/proceedings.159; Series: AIJR Proceedings; ISSN: 2582-3922; ISBN: 978-81-965621-6-8

Unlike today geotechnical standards, little attention was given to the nature of soils when filling the valleys or cutting the hillsides in the old days. Slope design was simply a matter of constructing them in accordance with standard details based on experience, which were considered acceptable in most cases. Eves (1913) described road cutting as steep as 75 degree as the practice at that time. The steep cutting had the advantage of having a very small surface exposed to rain, although landslides were frequent. However, as there was little wheeled traffic at that time, landslides did not cause much damage and delay. Pedestrian could climb over the debris, which were often left for weeks. The slope design practice had changed starting from 1950s (Lumb, 1972), as steep cutting behind terraced building sites produced a change in the consequence of a slope failure. Landslide was no longer a matter of inconvenience and there was a real danger of loss of life for dwellers living at the toe of the steep cutting. Around the post-war years, cut slope was commonly formed to a batter of 10:6 with a berm of 1 to 2 m wide added at every 7.5 m interval in height. If failure occurred during construction, the batter was reduced to 1:1 giving an average slope angle of 40 degree. Fill embankment was formed by end-tipping without much compaction. Chunam plaster was usually applied to the slope surface for preventing infiltration of rainwater into the soils. Weep holes were added in subsequent practice, which allowed groundwater to seeping out of the slope.

Given the rugged topography, seasonal heavy rainfall and slope formation practice in the past, landslides are common in Hong Kong. Records show that more than 470 people have been killed in fatal landslides since 1948. The disastrous landslides occurred in 1972 and 1976 had resulted in more than 160 fatalities and that galvanised the Government's determination to tame the landslide problem in Hong Kong. Thus, the Geotechnical Engineering Office (GEO) was established in 1977 as the centralised regulating body and has been given the mandate to manage the landslide risk. The Slope Safety System formulated by the GEO comprises three main strategies, including (a) regulating the design and construction of new geotechnical works; (b) retrofitting substandard slope and undertaking slope maintenance; and (c) reducing the consequence of landslides. The GEO faced many challenges upon its establishment, as there was an absence of geotechnical community to provide adequate input to the geotechnical works, let alone the sets of acceptable standards to follow. It was, therefore, an important task to set geotechnical standards that best suit local condition and environment. The GEO has a dedicated team with specific role of technical development for production of guidance documents. They include the authoritative series of Geoguides, Technical Guidance Notes, GEO publications and technical circulars, etc. The guidance documents cover a wide variety of pertinent subjects, encompassing geology, ground investigation, slope engineering, excavation and lateral support, foundation and landscaping. Practitioners follow these guidance documents in the design and construction of geotechnical elements which are subject to the regulatory control exercised by the GEO. Besides technical considerations, other administrative matters (e.g. land use planning and zoning) may also influence the technical solutions. Over the years, geotechnical practice has been subtly put together that is unique to Hong Kong.

The construction cost in Hong Kong is amongst the highest in the world, especially when compared with other cities of similar economic scale. The situation is expected to be worsen in the coming years, as the Government is committed to invest heavily on infrastructures and housing developments; and the local construction industry faces many problems such as an aging labour force and escalating inflation. The Government has pushed for the transformation of the construction industry to survive the productivity crush. New digital construction methods are promoted to improve project delivery capability and facilitate offsite construction that reduces reliance on manual labour. Besides these initiatives, there is a narrative that changes in the regulatory control could significantly leverage the productivity gains from leaner construction and catalyze productivity improvement. There is no dispute that construction works are subject to a highly regulatory framework in Hong Kong, which involves the enforcement of the Buildings Ordinance for private developments and the administrative instructions imposed on public works projects. The GEO is certainly part and parcel of this regulatory framework, for the role as the technical advisor to the Building Authority and other Government project offices on geotechnical matters.

The GEO always embraces innovations that can bring advancement to the geotechnical engineering. Whilst most people view innovations as applying novel technologies for new services, innovations also mean implementing new ideas and processes to existing services that leads to productivity enhancement and financial gains. This is particularly relevant to the prevailing geotechnical practice that have different degree of conservatism and perception on safety margin on the design. It is well recognised that not only the geotechnical standards drafted by the GEO but also the interpretation of them by practitioners, including those in regulatory authorities, have profound influence on shaping the local geotechnical practice. The GEO sees that maintaining normalcy in geotechnical practice is not sustainable to satisfy the expectation of society and the productivity gains necessitated for the huge infrastructural investment. It has been opportune that the GEO has been working with practitioners, academia and other Government authorities in recent years, and made advancements in the geotechnical practice that greatly benefit the execution of site formation, excavation and foundation. Some advancements call for paradigm shift on the understanding and rationale of the prevailing practice; and some rest on readjusting the perception on safety margins.

2 Recent Advancement in Site Formation Practice

2.1 Control of moisture content of fill material at time of deposition

In projects involving substantial filling works, fill compaction is typically controlled by product specifications in accordance with the General Specification (GS) on Civil Engineering Works (CEDD, 2020), where special or general fill material are available and specified by the Engineer. The GS places a restriction, as a compliance condition, on the moisture content (MC) of the fill material to be controlled within ±3% of optimum moisture content (OMC) during deposition. Samples should be collected and delivered to designated laboratory for determining the MC within one hour after deposition. Practitioners raised the concern that the collection of samples and testing the MC at deposition has a great implication on the filling process. In addition, this procedure brings significant workload to the Public Works Laboratory (PWL), which is the only authorised laboratory for conducting compliance tests in public works projects. In the event that the MC at deposition is not determined, the MC obtained in the subsequent sand replacement test (SRT) is sometimes used as a substitute for checking the compliance requirement. Some engineers decided to remove the compacted layer when receiving a non-compliance report based on the MC obtained in the SRT, despite the fact that it did not equate to the MC of the fill material at deposition.

Proceedings DOI: 10.21467/proceedings.159
ISBN: 978-81-965621-6-8

Series: AIJR Proceedings
ISSN: 2582-3922

The GEO has taken the initiative to study the effect of the MC on the engineering properties of the fill material compacted to the requirements. Four selected types of fill material were compacted to a target relative compaction (RC) of 95% at different MCs. Triaxial tests were then carried out to study the strength properties (Chung & Chu, 2020). Figure 1 shows the particle size distribution of the four types of fill material used in the study. They were compacted with MC beyond the tolerance of ±3% from OMC. The test results show no evidence that there is appreciable difference on the strength and stiffness of the fill material which is compacted to a target RC of 95% with MC beyond the tolerable range of ±3% OMC. Figure 2 shows the results of the triaxial tests conducted on the compacted samples. The peak friction angles achieved are generally greater than the strength parameters of compacted fill

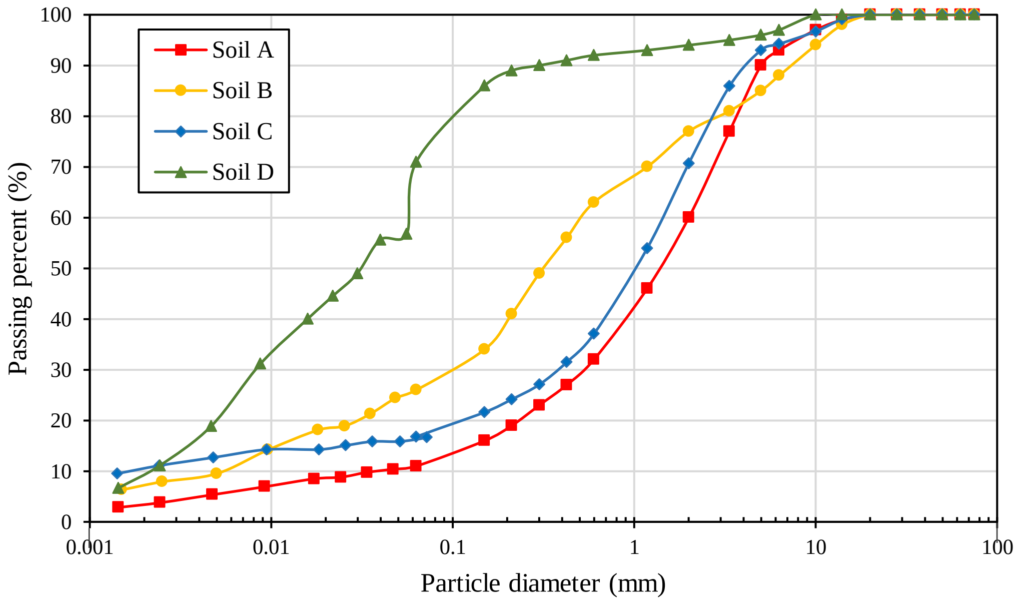

Figure 1: *Particle size distribution curves of the four soils tested*

commonly used in the design. Besides, a review of widely adopted local and international specifications on the control of filling works has been conducted. It is found that only the GS for Civil Engineering Works specifies the MC of fill material at deposition as a compliance requirement.

After benchmarking with local and international specifications as well as reviewing the laboratory test results of the selected fill material, it is considered that the compliance criterion on the MC of fill material at deposition is not essential to forming a stabilized fill. Arrangement has been made to remove such specific requirement from the GS.

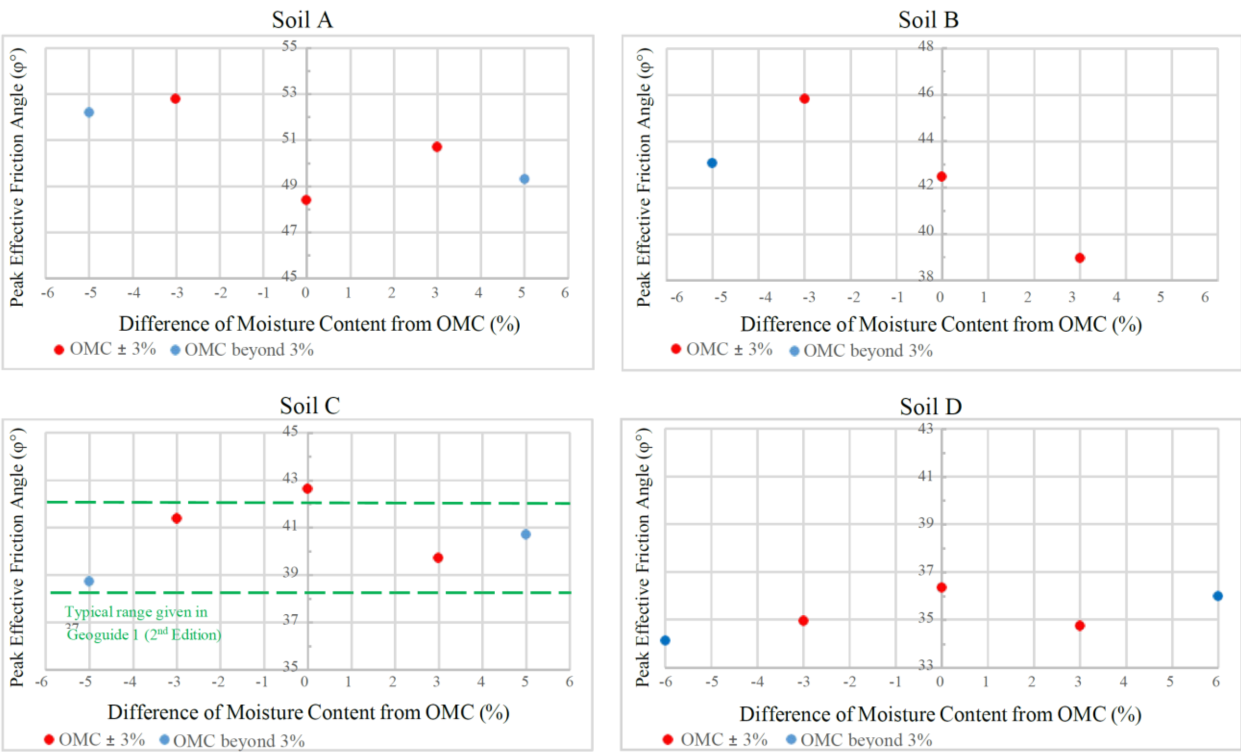

Figure 2: *Variation of effective peak friction angle (φ°) at different moisture content*

2.2 Slope with soil nails installed on Government land

In recent years, more sites on hillsides are earmarked for public and private developments, which always necessitate substantial site formation works. It has been the customary obsession by practitioners that all engineering works should be restricted within the site boundary and that has been cast in stone. Permission to install structural support elements (e.g. soil nails) outside the site will rarely be granted by relevant authorities. Such constraint always results in the adoption of costly and extensive bored pile walls along the peripheral of the site.

In fact, such perception is not unique to site formation works for new developments. It has also plagued many slope owners who have been served with Dangerous Hillside Order (DHO) to investigate and upgrade their man-made slopes. In majority of these DHO cases, the slopes are formed along the peripheral of the site boundary to maximize the building areas. Upon completion of the buildings, there is practically no access or space for constructing massive bored pile walls (Figure 3). Installation of soil nails is considered as the only plausible and affordable solution. However, obtaining approval for installing the soil nails outside the lot boundary had deterred many geotechnical professional to pursue this sensible solution. Such problems have profound implications on public safety, as the slope owners or the geotechnical professional engaged would have much difficulty in overcoming these hurdles. The statutory order has specific time limit that requires the slope owner to comply with and it is undesirable to allow the order be extended for a long period of time.

In view of the practical difficulty, the GEO has reached out to relevant Government departments, including Lands Department (LandsD) and Highways Department (HyD), and worked out the principles that could facilitate slope owners to pursue slope remedial scheme with soil nails installed outside the site. In gist, on the advice of the GEO, LandsD would consider positively for applications involving DHO and proactive action by the owner to improve the stability of their slope. On the other hand, the

Proceedings DOI: 10.21467/proceedings.159
ISBN: 978-81-965621-6-8

Series: AIJR Proceedings
ISSN: 2582-3922

GEO drafted relevant lease clauses to indemnify the Government's right and liability in the event of any damages caused by the lot owner installing soil nails in Government land. In addition, the lease clauses also have provisions on the scenario when the adjoining areas are to be developed and the liability and impact of the construction on the installed soil nails.

Figure 3: *Congested space and access available for upgrading substandard slope*

Soil nailing technique has been introduced to Hong Kong since 1980s and the experience suggests that there is little or no maintenance required for soil nails installed in the soil mass. In the newly drafted "Soil Nailing Works Clause", it no longer imposes any maintenance responsibility to the owners regarding any additional area occupied by the soil nails. This relieves the worries of the slope owner on any additional liability, particularly where the land above the soil nails is natural hillside. Similar principle is also acceptable to situation where soil nails are to be installed underneath the public road

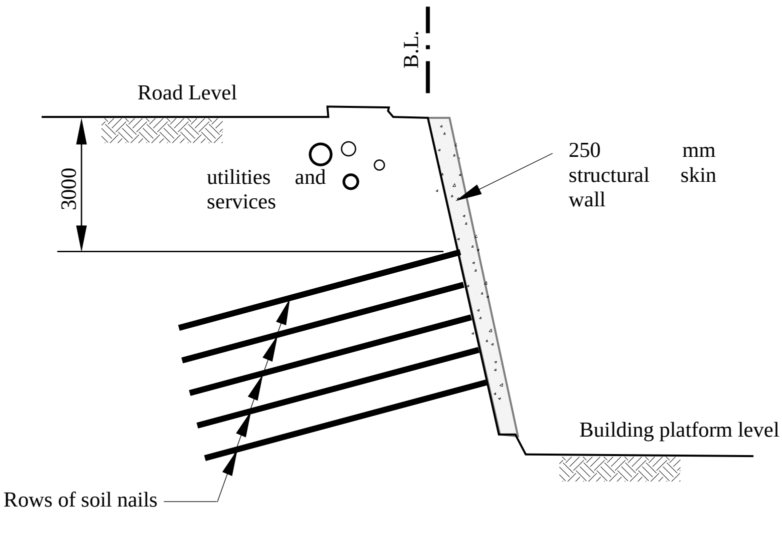

Figure 4: *Stabilization scheme for cut slope abutting public road with soil nails installed outside lot boundary at Kwun Tong*

maintained by HyD. In fact, GEO helped owners and Buildings Department to resolve seven DHO cases that had been held in abeyance for some years, since approval to install soil nails underneath the public road were difficult to obtain. HyD took the same sympathetic attitude on slope upgrading works that would improve slope safety. A general principle has been established that a 3m deep exclusion zone measuring from the road surface is provided in the slope design, where no soil nails should be installed. Figure 4 shows the conceptual stabilisation scheme that was used in slope upgrading works at a Kwun Tong site. The primary objective of the exclusion zone is to provide adequate space for installing and maintaining services and utilities underneath the road pavement.

The same concept has been extended to public works projects, particularly the site formation works for public housing developments, as this involves the public coffers. Substantial saving in cost and time could be achieved if soil nailed slopes, instead of bored pile walls, are used to form the building platforms. The land authority, on the advice of the GEO, will incorporate suitable clauses in the land instrument to define the responsibility for damages and problems associated with the soil nailing works in the Government land. A vivid example of applying this concept involved the site formation works for a public housing at Fanling. With the collaborative efforts from the project office, policy bureau, consultants and the GEO, soil nailed slopes were adopted in lieu of bored pile walls for forming the building platforms. Such optimisation had resulted in 50% saving in cost and 6 months advancement in the programme of the formation works.

In fact, a forward-looking mindset in the planning of development site in hillside can help minimising many geotechnical works. Where new site is identified for development, particularly on the hillside, the proposal is circulated to Government departments for comment and advice. The GEO is usually asked on the views of applicable geotechnical clauses to be included in the land instrument and any geotechnical constraints on developing the site. Besides the usual advice, the GEO now takes a proactive approach to postulate the scenario of geotechnical works that are likely needed for the development and identify any improvement on the site layout that could minimise the geotechnical works. Such consideration should be made even at the planning stage when the zoning proposal is being circulated. Any planning constraint (e.g. rezoning of green belt) should be resolved earlier which, otherwise, may limit the practicality of revising the boundary at a later stage. Figure 5 shows the initial circulation of a proposed land sale site that would be carved from existing hillside. The planner intended to align the lot boundary of the site along the ridgeline of the man-made slope adjoining a public road. However, this would leave a truncated slope that needs to be supported by either a retaining wall or a substantial cut slope within the proposed site. There was no reason for leaving a portion of this slope from technical consideration. Hence, on GEO's advice, the site boundary was moved further southwest to align with the public road. This gives more flexibility on the site formation works to form the platform levels that match the road level and eliminate the need for any retaining structure. Such concept is equally important to engineers when planning the project boundary and the works area required for the project. In determining the land requirement, the engineer should also optimise the site layout, with the focus on minimising the geotechnical works, especially for hillside development where the substantial cutting is always necessary.

Proceedings DOI: 10.21467/proceedings.159
ISBN: 978-81-965621-6-8

Series: AIJR Proceedings
ISSN: 2582-3922

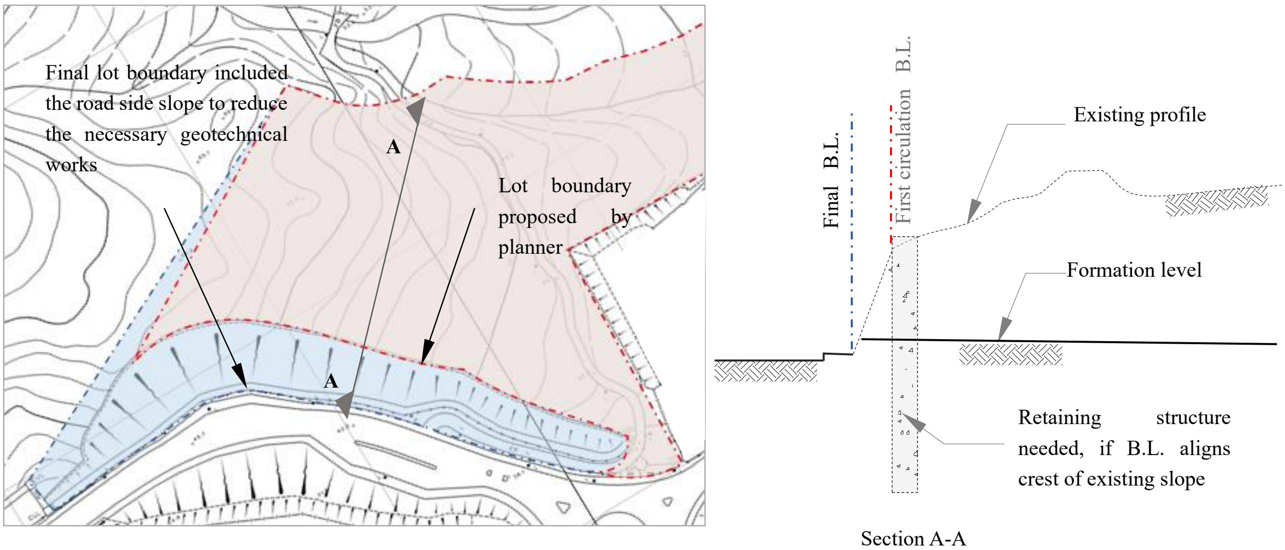

Figure 5: *Smarter planning of the site layout and lot boundary to minimize the geotechnical works necessary for the development*

2.3 Soil nails with sustained load

While preference is given to use soil nailed slope in site formation works, there is a need to review the requirements for soil nails subjected to sustained load, in order to facilitate the wider adoption of such scheme. The use of soil nails as structural support to excavation is common in overseas practice. Unlike stabilising existing slope, soil nail that is installed in tandem with slope excavation, is considered to have been mobilised with sustained load during its service life. Concerns on soil nails with sustained load relate to the durability, deformation and creep behaviour. Geoguide 7 (GEO, 2007) transpires these into additional provisions which, amongst other requirements, include the need for monitoring the movement of the soil nails for at least two wet seasons after construction. Some practitioners hesitate on such requirement, as it will bring uncertainty at the completion stage of the project. Any remedial works after the project completion would have huge implication to the users of the land. There was a case in which the engineer decided to form the slope to its final profile and installed the soil nails as a stabilisation measure to an existing slope. As such, it was presumed that the soil nails were installed for stabilizing an existing slope and therefore, the soil nails did not carry any sustained load. The construction sequence was peculiar, as it involved forming the slope at a reduced safety factor and erecting temporary platforms for installing the soil nails. These were riskier operations that could be prevented.

The need for post-construction monitoring was debated when the GEO was preparing the Geoguide 7 (GEO, 2007) in 2005. It was considered at that time that local volcanic and granitic soils generally behave as granular material, but limited data has shown that the plasticity index (PI) of the fine portion of the weathered soils could be higher than 20. This PI threshold has been adopted in overseas design guidelines for fine-grained soils that would exhibit creep displacement in soil grout interface under sustained load. As there was little information on the creep behaviour of local soils, it was decided to include creep test as part of the pull-out test in the construction of soil nails in Hong Kong. The intention was to obtain enough data for further review at a later time. In general, creep potential is considered unlikely when the creep displacement is less than 2 mm per log cycle of time in 6 to 60 minutes period when the soil nail is subject to a constant load.

A review of the creep tests conducted in slope upgrading works under the Landslip Prevention and Mitigation Programme (LPMitP) was carried out. Altogether, 286 no. of tests in different types of soils were reviewed and analysed. Figure 6 shows the distribution of the slopes from which the results of the pull-out tests were collected. The creep tests followed the procedures as stipulated in the Geoguide 7 (GEO, 2007) and creep extension was measured between 6 to 60 min with the nail force maintained at T_{DL2}. T_{DL2} is the design load of the soil nail with a safety factor on bond friction. Figure 7 depicts the extension of 143 soil nails at the creep test stage. The observed maximum creep displacement over this period is less than 0.1 mm, which is well below the 2 mm extension as the creep potential threshold. All creep tests reviewed so far do not show any creep potential of concern.

In the control framework for soil nails with sustained load, deformation analysis is required to investigate the effect of movement caused by the slope engineering works when there are sensitive receivers in adjoining ground, such as buildings and utilities. This is similar to other construction activities, e.g. excavation and lateral support works and foundation works, to ensure that the movements induced are within the tolerable limit of the sensitive receivers. Monitoring the deformation of the soil nails is reasonable within the construction stage but long-term monitoring is considered not necessary for the creep behaviour.

Given the preliminary finding on the review of creep tests, it appears plausible to dispense with the post-construction monitoring. In fact, pull-out tests and creep tests are usually carried out prior to the construction of the working soil nails. Therefore, there is an opportunity to identify any creep potential of the soil nails and implemented precautionary measures to address this problem, e.g., increasing the number or the bonded length of the soil nails to reduce the bond friction, or installing the bonded section at different type of soils where possible. The GEO will complete the review with more creep tests on different types of soils before concluding on the requirement of this post-construction monitoring. As a simple clarification, this post-construction monitoring should not be applicable to soil nails socketed in rock.

Proceedings DOI: 10.21467/proceedings.159
ISBN: 978-81-965621-6-8

Series: AIJR Proceedings
ISSN: 2582-3922

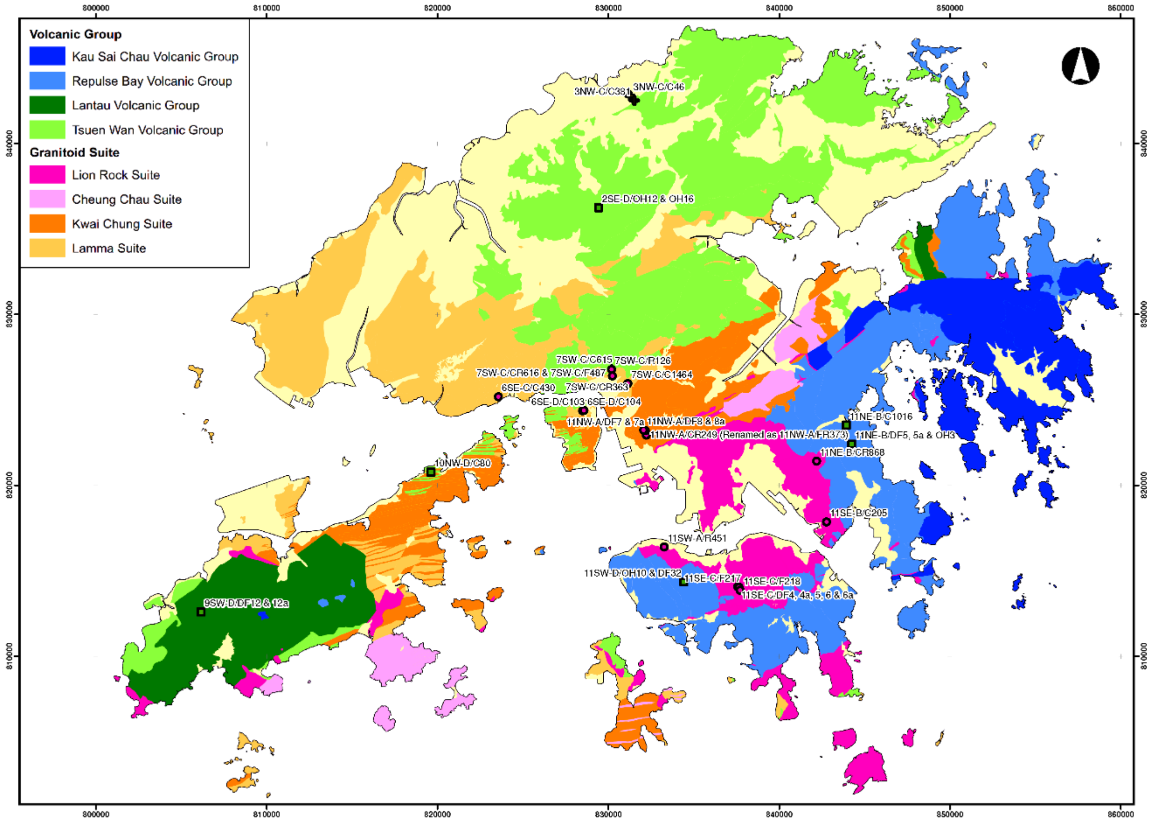

Figure 6: *Review of pull-out tests of soil nails installed in slope upgrading works*

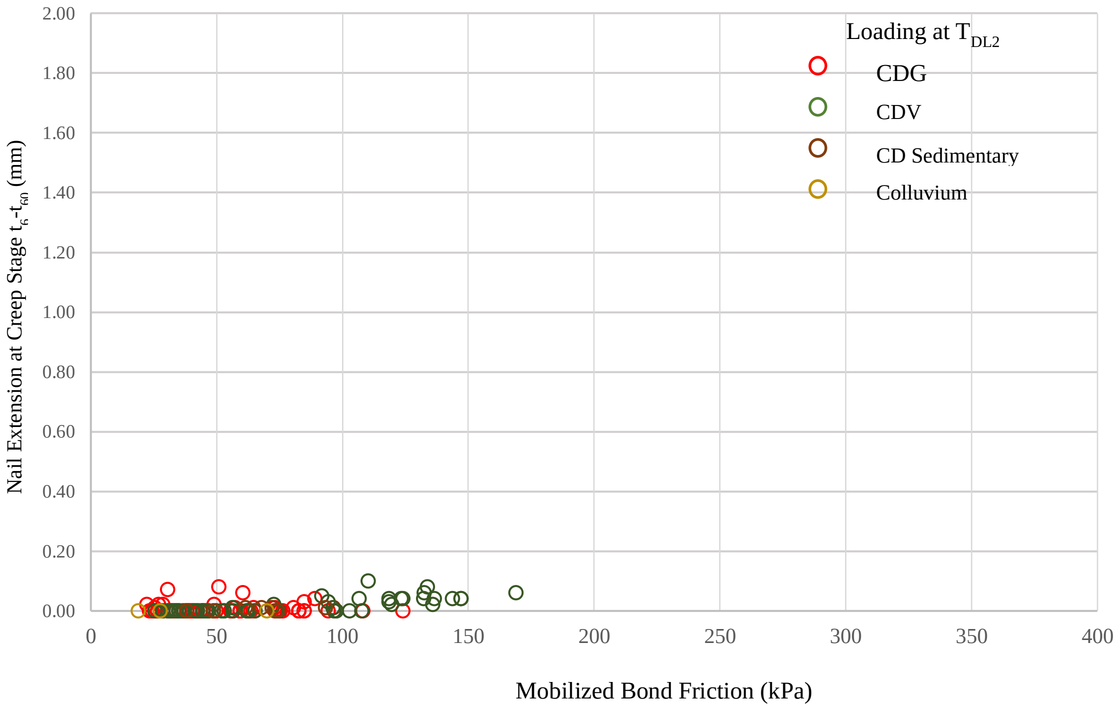

Figure 7: *Creep extension of soil nails at T_{DL2} loading stage between t_6 and t_{60}*

3 Recent Advancement in Deep Excavation Practice

GCO Publication No. 1/90 (GEO, 1990) provides a review of the state-of-the-art practice of the design methods for excavation. Since its promulgation in 1990, it has been used by practitioners as a key reference document in excavation design. However, there have been much advance in the design and construction of excavation methods since then, notably the application of Partial Factor Method (PFM) as stipulated in the CIRIA Report No. C580 on Embedded Retaining Wall – Guidance on Economic Design (A. R. Gaba *et al.*, 2003) published by the Construction Industry Research & Information Association (CIRIA C580). The Buildings Department has imposed additional provisions for using the guidance given in CIRIA C580 to suit local setting and limited experience at that time (BD, 2012). In 2017, a review of the excavation practice was carried out by a task force led by the Geotechnical Division of the HKIE, which made explicit recommendations for the advancement of practice in excavation design and construction. The GEO is currently revising the GCO Publication No. 1/90 and will take cognizance of the recommendations. The revision is supported by a working group comprising practitioners and representatives from relevant government departments. The revised publication is scheduled for completion in 2023. The debate and rational on some of the advancements and improvements that would be promulgated in the revised publication are documented below.

3.1 Additional provisions on application of partial factor method

The new publication will incorporate the application of the PFM in the design of excavation works, which are largely based on the experience of using CIRIA C580 with additional provisions as given in the Practice Notes No. PNAP APP-57 (BD, 2012). This will also incorporate relevant recommendations given in the updated version of the guidance (A. Gaba *et al.*, 2017). Having gained much experience in the application of PFM since 2011, practitioners and the GEO generally agreed that certain additional provisions as stipulated in Practice Notes APP-57 could be dispensed with. These include the requirements for a post-construction performance review, sensitive analysis of collapse or excessive deformation caused by incorrectly installed strut.

When executing geotechnical works, a fundamental principle is that their performance should be continuously monitored during the construction stage and compared with the design assumptions, especially any variation to the ground conditions. Precautionary and remedial measures should be implemented timely where necessary. The regular review is usually undertaken by the senior professional of the designer's firm under the qualified supervision system. Thus, post-construction review of the temporary ELS works does not provide any value-added benefit in safeguarding public safety.

A high standard of quality supervision is put in place in local construction projects to ensure that the works are carried out in accordance with the approved construction sequences. The risk of incorrectly installed strut should be managed by proper site supervision, rather than relying on the tolerance provided in the design. Similarly, better site management should be imposed to guard against any accidental removal of struts. In recent years, it has been common to adopt innovative solutions to manage construction risks, e.g. sensors and alarm system that could provide warning to the operator of the possibility of hitting nearby person and other machinery. In fact, such worries on irregularity should not be applied to design method based on PFM only.

3.2 Rock socket design

Geoguide 1 (GEO, 1994) stipulates that, where no adverse rock discontinuities are identified in the ground investigation, the rock socket depth should be designed based on preventing bearing failure of

Proceedings DOI: 10.21467/proceedings.159
ISBN: 978-81-965621-6-8

Series: AIJR Proceedings
ISSN: 2582-3922

the rock mass. However, in practice, discontinuity-controlled failure is often considered by assuming the presence of the most adversely oriented discontinuity. It is mainly because rock discontinuity survey is seldom conducted at the ground investigation stage and designer is indifferent to the missing of such important piece of information. In some cases, even rock discontinuity survey is conducted and major joint sets identified, some practitioners and regulating authority still question the possibility of the presence of individual yet adversely oriented discontinuity that could be missed by the survey. Such perception often resulted in excessively conservative design of rock socket length.

In the revised publication, the importance of conducting rock discontinuity survey at the ground investigation stage is reiterated, when it is anticipated that the embedded wall will penetrate the rock formation for stability. Without such information, it would be difficult to make a proper engineering judgement as to the presence of any adverse discontinuity that would affect the stability of the embedded wall. A high concentration of discontinuities in a stereo plot should be the evidence indicative to the potential of persistence of that joint set (Figure 8). The rock cores should be inspected and the weathering condition of the discontinuity also tells the likelihood of its persistence.

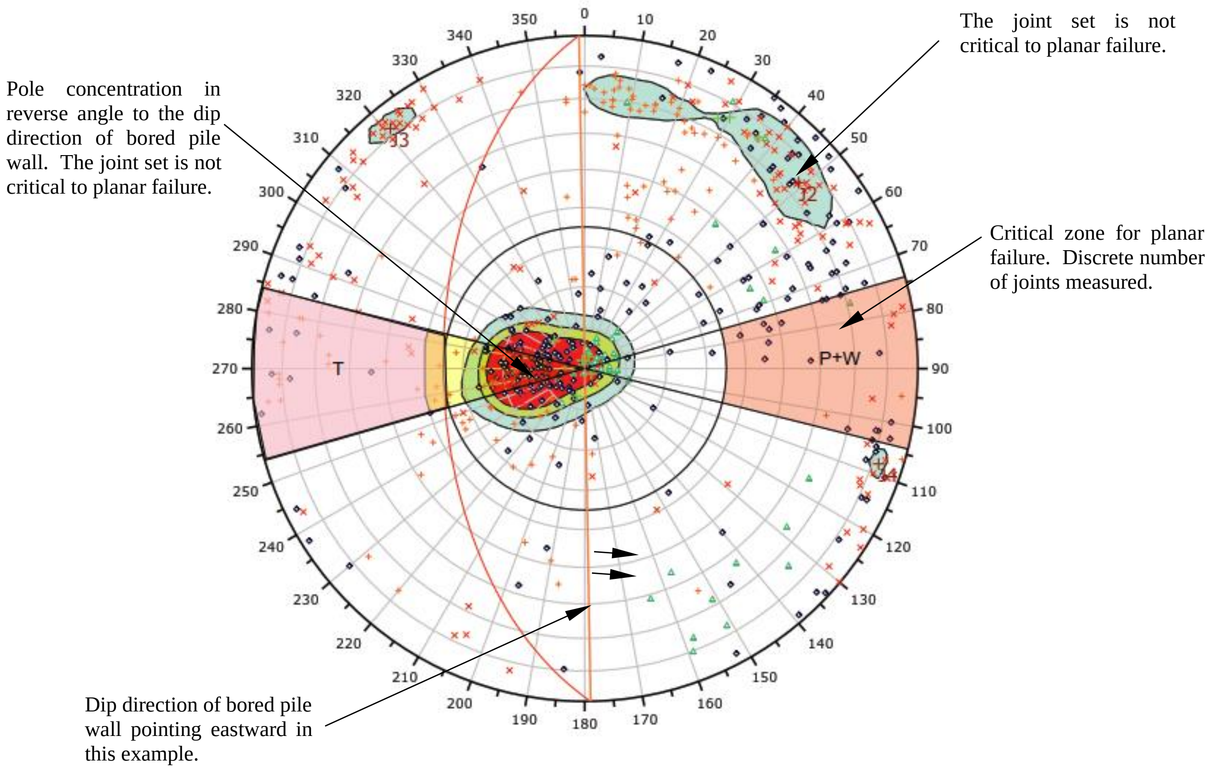

Figure 8: *Rock discontinuity survey to identify critical discontinuities for assessing the likelihood of discontinuity-controlled failure*

In a recent site formation project involving the use of bored pile walls socketed into slightly and moderately decomposed tuff, the initial design assumed the presence of the most adverse joint at an inclination of about 16 degree to horizontal, although there were results of rock joint survey. The design included a discontinuity-controlled failure analysis following the recommendation of Geoguide 1 (GEO, 1994) and a socketed length of about 20 m was proposed for a 15 m cantilevered bored pile wall. It is interesting, yet difficult to comprehend, to note that the length of this assumed persistent discontinuity would run into more than 150 m in horizontal distance for a 6-degree discontinuity plane, which was considered as an exceptional joint set. In assessing the likelihood of such failure mechanism and its stability subject to the horizontal load from the embedded wall, the building load or foundation piles will certainly give confidence on whether such a discontinuity-controlled planar failure is plausible. Figure 9 shows the section of the bored pile wall scheme that could help the engineer in making sound engineering judgement on the potential of such individual persistent rock discontinuity-controlled failure. A holistic assessment of the likelihood of such failure is important to derive a sensible and cost-effective solution. In this project, the GEO has proactively engaged the designer to revisit the assumptions and review the rock discontinuities pattern. The collaborative effort has resulted in reduction of the rock socket lengths of the bored pile wall.

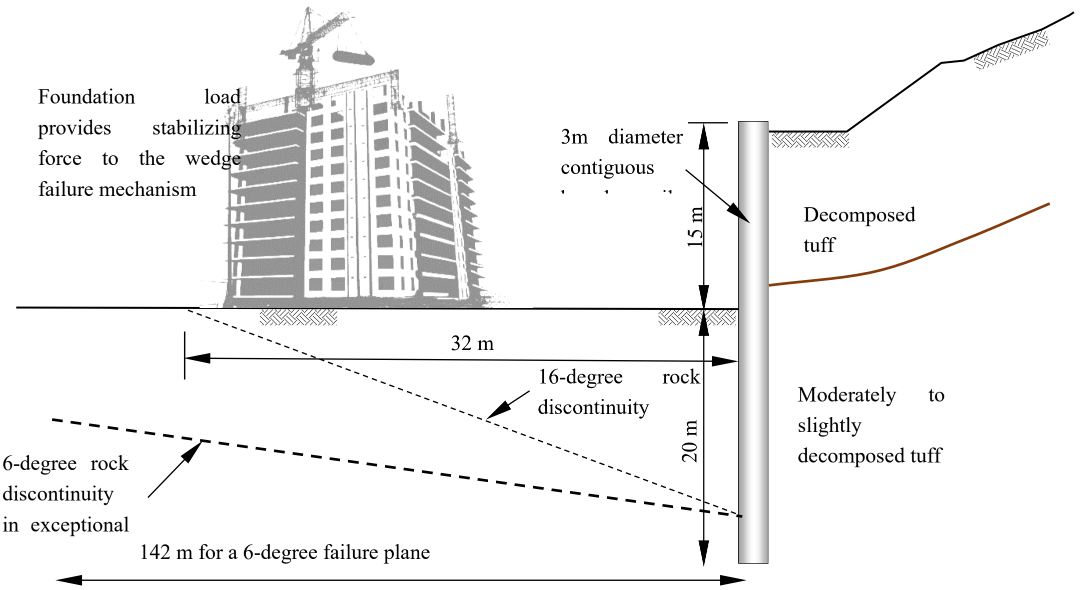

Figure 9: *Holistic assessment of the likelihood of wedge failure due to presumed adverse rock discontinuity plane*

3.3 Pumping test for deep excavation

Full-scale pumping test is often specified with the presumption that it is used for validating the permeability of the soil mass assumed in the groundwater seepage analysis prior to bulk excavation. However, majority of the pumping test was, in fact, used to conclude the insignificant effect caused to adjoining area by the dewatering, rather than review the assumption of the soil mass permeability. A major disadvantage of such arrangement is that dewatering would induce significant lateral deflection to the embedded retaining wall, particularly for deep excavation projects. In urban setting where excavation is commonly surrounded by sensitive structures and utilities, e.g. old buildings on shallow foundations, MTR facilities, gas mains etc., it is more desirable and prudent to adopt construction sequences that would minimise any ground deformation. However, there is currently no clear guidance on when such a field validation is necessary and full-scale pumping test is conducted indiscriminately, even for ELS works that are primarily designed as a ‘dry’ excavation, for which the design intent is to cut-off water seepage into the excavation. There seems to be no real benefit of conducting pumping test to prove the permeability and seepage quantity for such a ‘dry’ excavation system. For ELS works, it is almost mandatory to implement an instrumentation and monitoring scheme to ensure that the excavation works will not cause any damage or adverse effect on adjoining ground, facilities and structures. Therefore, any adverse effects due to dewatering would be safeguarded by the monitoring and action plan at the bulk excavation stage. Dewatering carried out in tandem with the strut installation can reduce

Proceedings DOI: 10.21467/proceedings.159
ISBN: 978-81-965621-6-8

Series: AIJR Proceedings
ISSN: 2582-3922

the lateral deformation of the embedded retaining wall. The necessity of conducting a full-scale pumping test prior to bulk excavation should be carefully assessed and is not preferred in excavation system where it is designed to be fully enveloped with an impermeable barrier.

Where the engineer considers desirable to conduct a full-scale pumping test, there is a potential saving by streamlining the test procedures. At present, the procedures in pumping tests for excavation works involve holding the steady state seepage condition for a 72-hour period, before allowing the recovery of the groundwater table. A review of 24 pumping tests conducted in recent deep excavation projects was conducted to explore the necessity of holding the steady stage seepage for such a long period. Figure 10 shows the normalised dewatering curves extracted from the reports of these pumping tests. The steady state seepage condition, once achieved, remained stable for the entire steady state seepage to 72 hours. Unlike the purpose of water yield test, the pumping test is not meant to determine the continuous extraction rate for a prolonged period. Therefore, it has been proposed in the revised publication that the holding period for steady state seepage could be reduced to 24 hours. This allows the commencement of recovery stage at an earlier time.

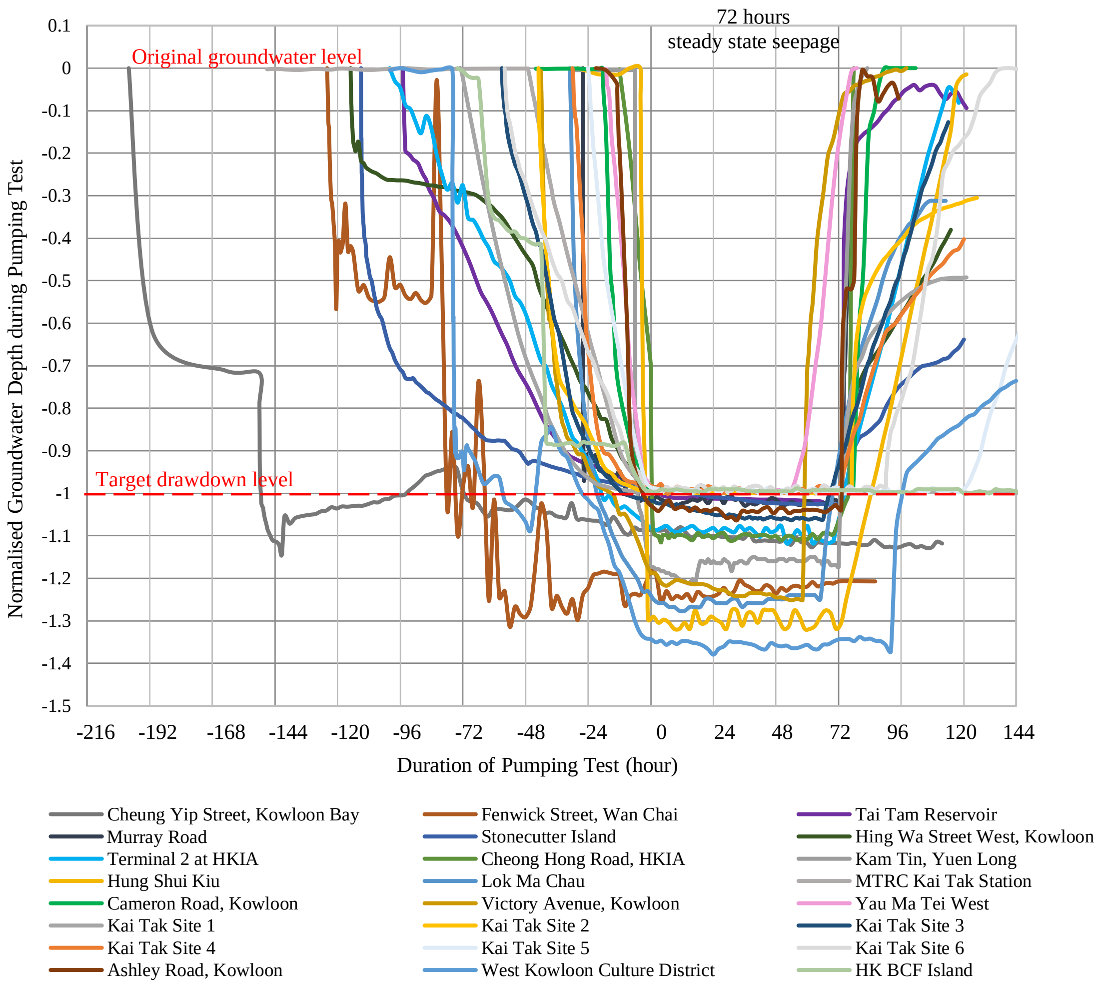

Figure 10: *Consolidated plot for performance of dewatering during pumping test results*

3.4 Response and control mechanism on ground settlement

ELS works need to be cautiously carried out to ensure that the induced impact on the nearby sensitive receivers is kept within an acceptable level. A three-tier triggering control mechanism, i.e. Alert-Alarm-Action (AAA) Levels, with corresponding response actions is usually specified to forewarn any excessive ground movement and safeguard the sensitive receivers during construction. At Action Level, the control mechanism usually calls for suspension of all site works. Works suspension is very disruptive on the construction programme and always produce a negative impression to the public on the engineers and the project proponent.

A performance review of recent deep excavation projects was conducted. The projects were selected based on the availability of quality monitoring data. It is found that the ground settlement induced by the ELS works generally varied from 0.3% to 0.5% of the maximum excavation depth, depending on the compactness of the soils, excavation system and construction sequences. In recent years, private developments always include a basement for the benefit of concession in gross floor area. Hence, it is common to find development projects with a 2 to 3-level basement that typical involve 20 m excavation. If the empirical value of 25 mm is still adopted as the Action Level for ground settlement, there is a high possibility that this Action Level would be breached at certain stage of excavation. Some designers took the risk and looked for negotiating with relevant stakeholders and authority when this Action Level was reached. However, it is highly undesirable to have an excavation with active dewatering maintained for an extended period of time, which will prolong the risk of affecting the nearby facilities. Also, it would bring uncertainty to the construction programme, as there were many cases that it took months, if not years, to get approval to recommence the project. On the other hand, the relaxation of the Action Level is often accused by the public of moving the goalposts and this has a definite negative impact on the professional image of all parties involved in the project.

Some designs strengthened the support systems (e.g., using preload of more than 1,500 ken/m or putting strut layers at every 1.5 m vertical intervals) to meet the empirical limit. However, this will not only give rise to overdesign and end up with increased construction cost and time, it may even result in a counterproductive design that will bring constructability and safety issues. For example, the installation of longer or larger pipe piles involves the use of compressed air, which could pose construction risks to workers or the public, if the boring operation is not properly controlled. There are few incidents that were caused by deep excavation works as documented in GEO (2020). The preloading of the strut would induce reverse deformation of the embedded retaining wall and there had been reported cases of breakage of welding connections for those struts and walings installed at the earlier stages and at higher levels.

Based on past records, many excavations were suspended due to the exceedance of the Action Level for ground settlement of 25 mm, rather than the building structures which are commonly supported on deep foundations. Carriageway, pavement, footpath and playground surface can be readily repaired in case there are serviceability concerns (e.g. cracks or uneven surface). Therefore, it is sensible to understand the serviceability limit of road and pavement and the recommended repair strategy, such that a reasonable response could be formulated.

The Guidance Notes for Road Inspection Manual (Report No. RD/GN/016C) (HyD, 2016) recommend that depression larger than 20 mm may pose a safety hazard to pedestrians and repair works should be carried out if necessary. On the other hand, there may be concern on the integrity of the paving material when there is significant ground settlement. The National Standards of the People's Republic of China published the Specifications for Design of Highway Subgrades (JTG D30-2015) (MOT, 2015) and recommend the allowable differential settlement between bridges and road abutment to be 100 mm and total settlement of 300 mm for general road pavement for carriageway. These guidance documents provide the basis for proposing a more representative value on when repair to the road and pavement is considered necessary.

In fact, the control mechanism can be devised to address two separate issues. One is primarily related to the serviceability issue, e.g. driving comfort and pedestrian safety, distorted utilities. Repairing the ups and downs of the road or pavement could be easily completed at affordable time and cost. It takes few hours to open up the road and repair any leakage of drains and water mains. A smarter arrangement for addressing these concerns is to put in place a good communication with the relevant stakeholder (e.g. HyD maintenance party) to agree the actions necessary (e.g. repaving the pavement or readjusting the distortion of settled services) when the serviceability limit (e.g. 20mm) has been reached.

The other issue comes to play when the estimated ground movement has been exceeded, which cast doubt on the design assumption and the overall stability of the ELS works. Usually this limit will be higher and is comparable to the value of 0.3% of the excavation depth; or when the tolerable limit of the ground settlement has been reached. Under such circumstance, no further construction activities that will aggravate the ground settlement should be carried out near the identified safety hazard area. Investigation and design review should be carried out to find out the cause, estimate further movements and assess the impact to the nearby sensitive receivers based on the performance of ELS works. Such investigation should also look for the presence of any underground voids and cavities caused by the excavation works. However, it is important to note that works that are contributing to the stability or performance of the excavation system, should be continued. There were many cases where all works were suspended at the stage where the walings and struts were yet to be properly installed. They were important to safely transfer the load across the excavation.

Such control mechanism is more practical and sensible. It should avoid setting an unrealistic Action Level on ground settlement that is bound to be triggered. The repair work would be carried out anyway, but at a much-controlled manner, rather than under a situation where the site works are suspended and public announcement made.

4 Recent Advancement in Foundation Practice

GEO Publication No. 1/2006 gives technical guidance on the design and construction of foundations and some recommendations were based on the Code of Practice for Foundation (CoPF) (BD, 2004). In 2017, Buildings Department published the second edition of the CoPF and as a consequential change, the GEO considers that there is a need to update the publication to align the technical standards for private and public projects. In addition, it is also opportune to enhance the guidance to improve the productivity and economy of foundation works, as the industry has carried out many studies and instrumented pile loading tests since 2006.

4.1 Presumed bearing capacity on igneous rocks

The publication gives recommendations on the presumed allowable bearing capacity and bond friction for foundations rest on different categories of igneous rocks, which follows the CoPF (BD, 2004). Practitioners always question whether higher allowable presumed bearing capacities could be adopted, which are used in other places with similar geological formations and foundation practice, particularly for competent rocks such as Category 1(a), 1(b) and 1(c) igneous rocks. The prevailing guidelines given in the publication for the presumed allowable bearing capacity are 10,000 kPa, 7,500 kPa and 5,000 kPa, respectively.

In order to explore the feasibility of enhancing the bearing capacity, the instrumented pile loading tests that were conducted since 2006 were collected and consolidated into the pile database documented in the publication. The pile database now comprises more than thirteen cases of instrumented pile loading tests founded on igneous rocks. Figure 11 presents the proven bearing capacity from these instrumented pile loading tests versus with the uniaxial compressive strength (UCS) of the rock mass underneath the founding level of piles. In addition, the instrumented piles that were founded on Cat 1(b) and Cat 1(c) rocks are also labelled for reference.

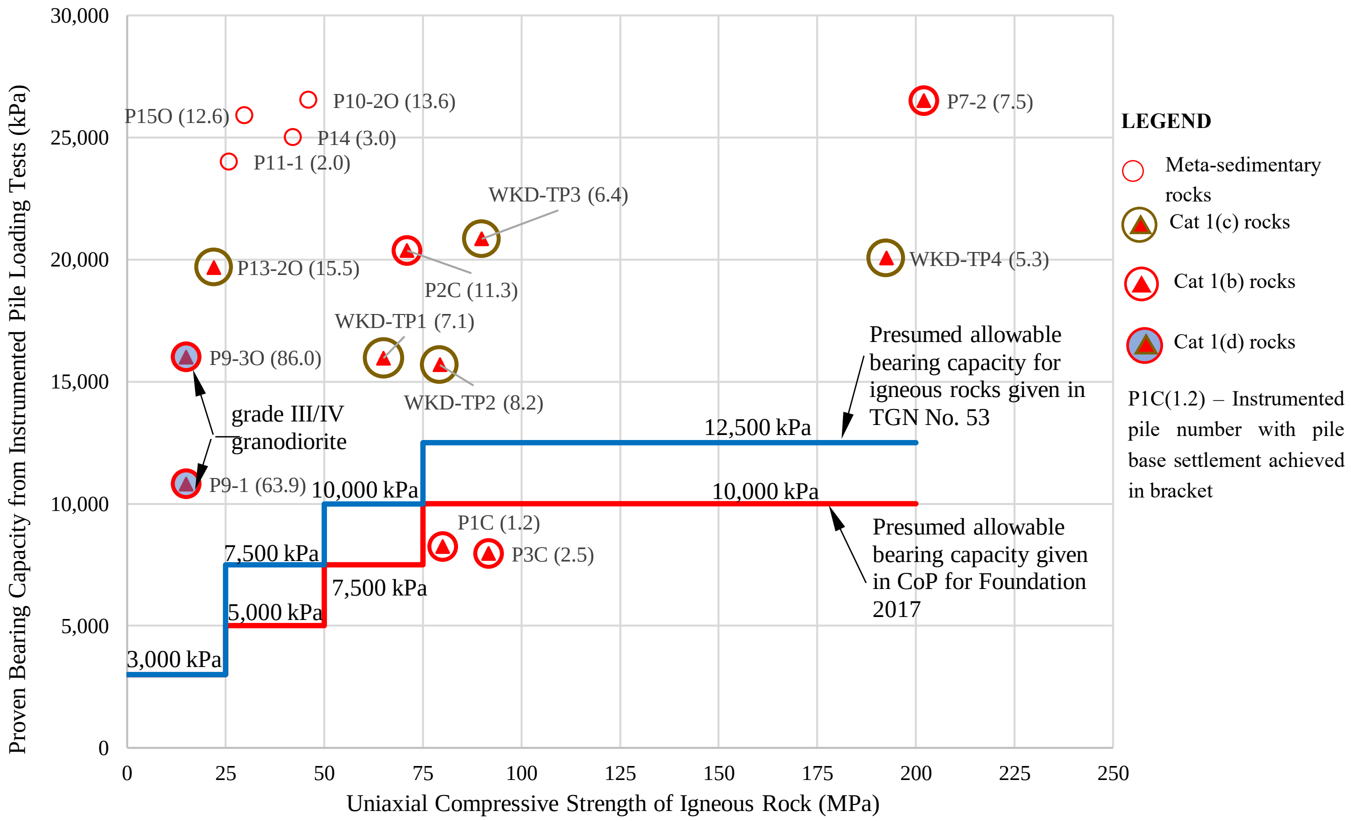

Figure 11: *Presumed allowable bearing capacity for foundations on igneous rocks (TGN No. 53, GEO, 2022)*

Based on the instrumented pile loading test data and the interpretation reports of these pile loading tests, the measured settlements at the pile base ranged from 1.2 mm to 15.5 mm for piles founded on Cat 1(c) or better rock, with the maximum pile base settlement less than 1% of the base diameter. Most of these pile tests indicates a low mobilisation of the bearing capacity, that were attributed mostly to the limits of the kentledge load or the capacity of the Osterberg load cell set up for the tests. For example, pile loading test Nos. P1C and P3C only recorded 1.2 mm and 2.5 mm base settlement when the maximum kentledge load had reached. It is anticipated that much higher bearing capacity would be proven, should a larger kentledge load be provided. Most of the proven bearing capacity for piles founded on Cat 1(c) or better rock was well above 15,000 kPa, except pile test Nos. P1C and P3C as explained above. It is obvious that there is still a significant margin for increasing the presumed bearing capacity for foundations rested on competent igneous rocks.

The settlement of foundation founded on Cat 1(b) and Cat 1(c) rocks has been estimated with the circular loaded area ranging from 1 m to 4.5 m in diameter, which covers the common dimensions of bored piles used in Hong Kong. The settlement computed for applying 7,500 kPa on Cat 1(c) is about 21 mm for a pile with a base dimension of 4.5 m. This corresponds to a mobilisation ratio of only 0.46%

Proceedings DOI: 10.21467/proceedings.159
ISBN: 978-81-965621-6-8

Series: AIJR Proceedings
ISSN: 2582-3922

of the width of the foundation. The maximum settlement computed for applying 10,000 kPa on Cat 1(b) rock is about 8 mm for the same dimension of loaded area. In the estimation, it is assumed that rock within 600 mm from the founding level should not be non-intact (i.e., the rock cores are not fragmented). Highly decomposed materials are assumed to be existed at the top of each 1 m core length. The settlements computed are the upper-bound values, as the most unfavourable distribution of weaker material and the lower bound rock modulus of 5 GPa for igneous rocks based on Geoguide 1 (GEO, 1994) are adopted in the computation. The settlements estimated are less than 1% of the foundation width, for the increased bearing pressure of 7,500 kPa and 10,000 kPa on Cat 1(c) and Cat 1(b) rocks, respectively.

For foundations founded on fresh rock satisfying the Cat 1(a) rock that composes of strong to very strong igneous rock with 100% TCR of rock with UCS greater than 75 MPa, there is hardly any base settlement needed to be assessed. An allowable presumed bearing capacity of 12,500 kPa is proposed for foundations founded on Cat 1(a) rock in the revised guidelines. The consideration is mainly to have a comparable compressive stress on the shaft of the concrete piles, which usually adopts Grade 60 concrete. The revision of the presumed allowable bearing capacity is promulgated in the Technical Guidance Notes No. 53 (TGN No. 53) (GEO, 2022) and summaries in Table 1.

Table 1: *Presumed allowable bearing pressure and bond friction of igneous rock given in TGN No. 53 (GEO, 2022)*

Category	Description of Igneous Rock	Presumed Allowable Bearing Pressure (kPa)	Presumed Allowable Bond Friction (kPa)
1(a)	Fresh to slightly decomposed strong to very strong granite or volcanic rock of material weathering grade II or better, with 100% TCR of the designated grade which has a minimum UCS of rock material not less than 75 MPa (or an equivalent point load index strength PLI50 not less than 3 MPa)	12,500	1,000 (under compression or transient tension) 500 (under permanent tension)
1(b)	Fresh to slightly decomposed strong granite or volcanic rock of material weathering grade II or better, and with not less than 95% TCR of the designated grade, which has a minimum UCS of rock material not less than 50 MPa (or an equivalent point load index strength PLI50 not less than 2 MPa)	10,000	
1(c)	Slightly to moderately decomposed moderately strong granite or volcanic rock of material weathering grade III or better, and with not less than 85% TCR of the designated grade, which has a minimum UCS of rock material not less than 25 MPa (or an equivalent point load index strength PLI50 not less than 1 MPa)	7,500	700 (under compression or transient tension) 350 (under permanent tension)

4.2 Presumed bond friction on igneous rocks

The current guidelines only provide the presumed allowable bond friction for pile socketed into Cat 1(d) and Cat 1(c) or above rocks. It is evident that saving can be further achieved by differentiating the presumed allowable bond friction for Cat 1(c) and Cat 1(d), as the later has a higher uniaxial compressive strength (UCS). The instrumented pile loading tests conducted for the railway projects had also investigated the bond friction in the rock socket. Figure 12 presents the proven bond friction for piles socketed into various rock formations. The bond friction on majority of the pile loading tests was not fully mobilized. It can be observed that the proven bond frictions are generally greater than 1,800 kPa for piles socketed into rocks with UCS greater than 50 MPa.

ICE (2012) documented a number of correlations between the UCS of the rock and bond friction based on load test data on socketed piles in overseas projects. Horvath and Kennedy (1979) gives the lower bound values amongst the reported correlations. This is largely consistence with the bond friction obtained from the instrumented pile loading tests conducted in Hong Kong. By applying a mobilisation factor of 1.5 on the proven bond friction, a higher presumed allowable bond friction of 1,000 kPa could be adopted for pile socketed into Cat 1(b) or better rock, which should have UCS greater than 50 MPa.

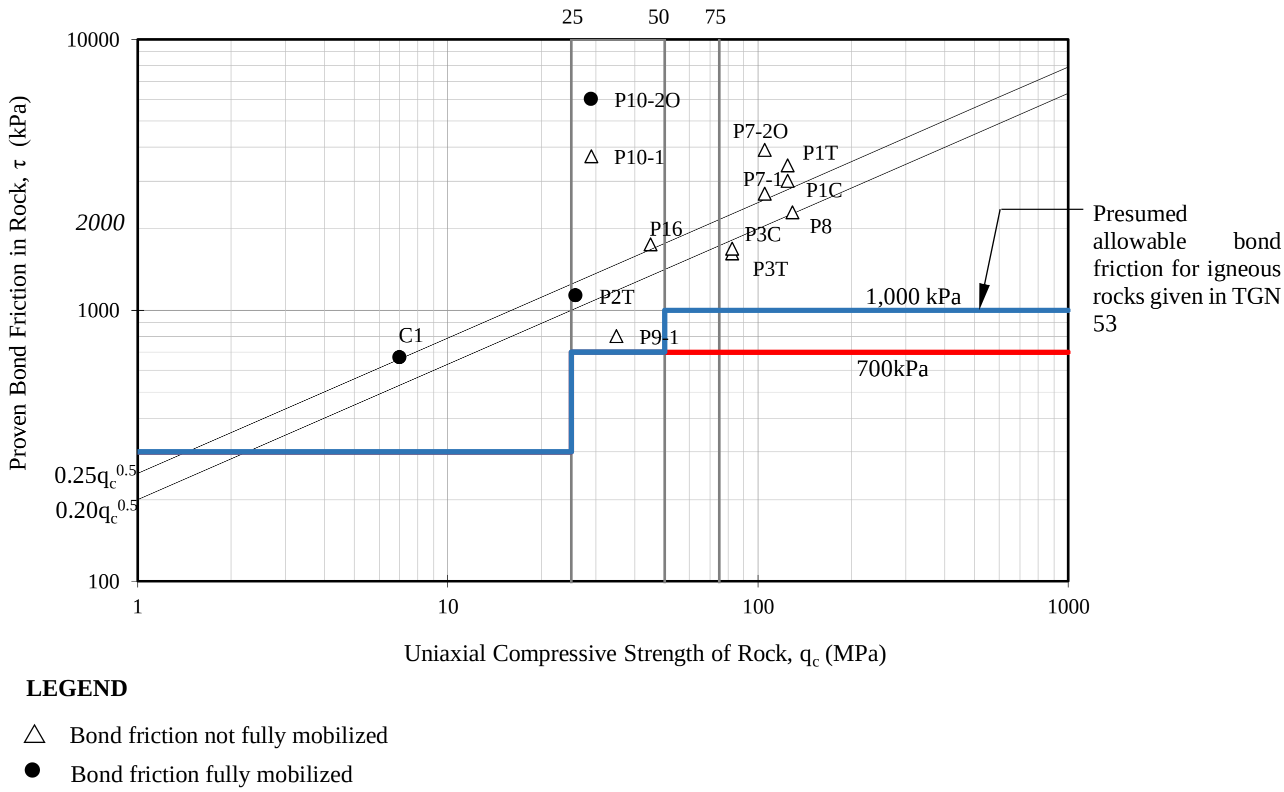

Figure 12: *Presumed allowable bond friction for foundations on igneous rocks (TGN No. 53, (GEO, 2022))*

The tension capacity under permanent load case is likewise taken as 500 kPa accordingly. The recommended changes are also promulgated in TGN No. 53 and given in Table 1.

Although there are thirteen instrumented pile loading test cases carried out in different projects, it is important to note that the foundations of all these private and public projects were constructed based on the higher bearing capacity as a result of the instrumented pile loading tests. There had been no report of defects or excessive settlement arising from the use of the increased bearing capacity.

4.3 Presumed bearing capacity on marble and marble bearing rocks

The CoPF published in 2017 (BD, 2017) includes a new Category 2 rock that is described as 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. A presumed allowable bearing capacity of 3,000 kPa is specified for foundations founded on such rock. Since its promulgation, practitioners interpreted that marble was also a type of meta-sedimentary rock and therefore, the recommended presumed value for Category 2 rock is also applicable to the design of piles founded on

Proceedings DOI: 10.21467/proceedings.159
ISBN: 978-81-965621-6-8

Series: AIJR Proceedings
ISSN: 2582-3922

marble rock formation. The consequence of this interpretation had reduced the presumed allowable bearing capacity and bond friction for piles founded on marble formation to 3,000 kPa and 300 kPa (friction under compressive), respectively. It was quite a frustration to practitioners as the foundation design used to accept the presumed bearing capacity of 7,500 kPa and 5,000 kPa for competent marble rocks prior to this interpretation.

Marble is a metamorphic rock composed largely of recrystallized carbonate minerals and their engineering properties are very different from meta-sedimentary rocks. GEO Publication No. 2/90 (GEO, 1990) documents the foundation and engineering properties of marble and other rocks in Yuen Long Formation. Figure 13 was reproduced from the publication which shows the engineering classification of marble in terms of elastic modulus (E) and UCS. This figure is also supplemented with the strength properties (E and UCS) of Grade II and III igneous rocks based on the laboratory tests conducted in the PWL and tabulated in (Irfan & Powell, 1991)and Lee (2019). It provides a comparison and illustrates that the elastic modulus of marble and marble bearing rocks are stronger than that of igneous rocks and generally have high moduli.

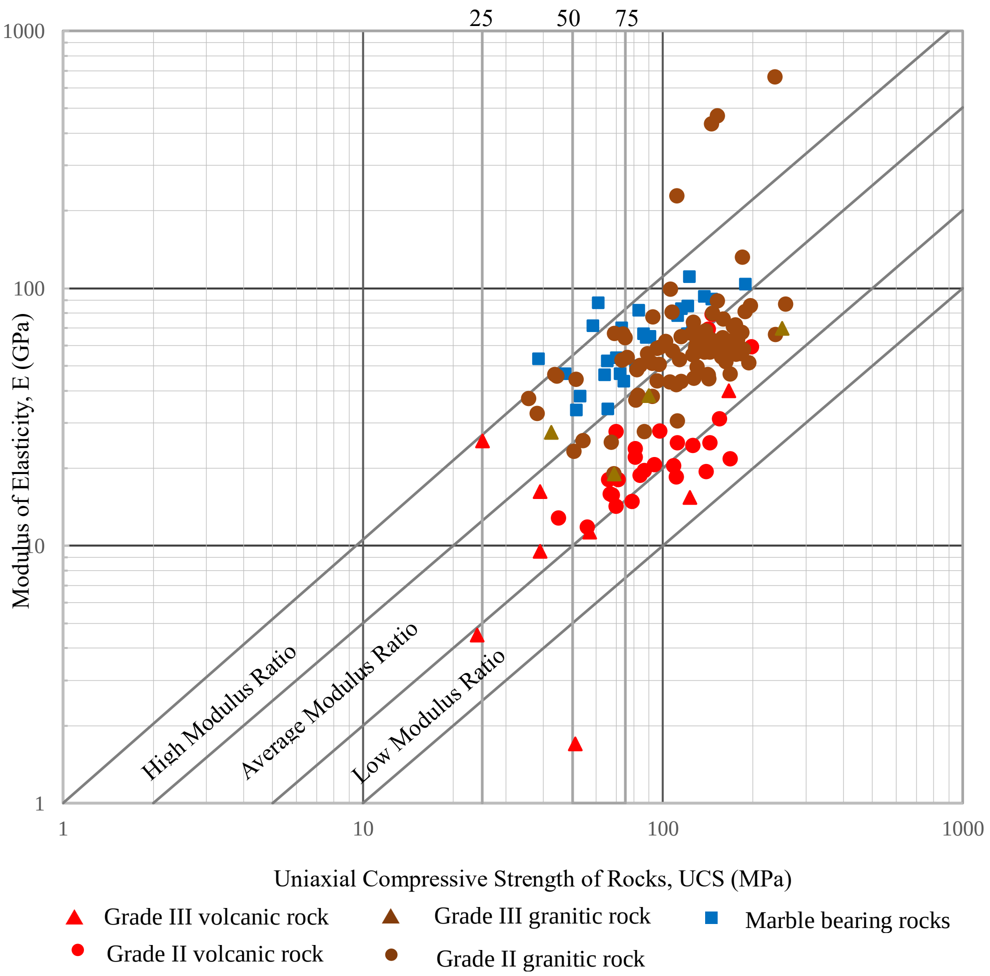

Figure 13: *Engineering classification of igneous rocks and marble in terms of modulus of elasticity and uniaxial compressive strength*

The GEO saw that the misunderstanding had caused significant impact on the foundation design in Scheduled Areas underlain by marble. Not only the number of bored piles is increased, so is the construction risk associated with the piling works. In 2020, GEO clarified and the BD published an addendum to exclude marble and marble-bearing rock from the Category 2 rock. However, the industry was still not given any presumed values for the design of piles on marble rock. Practitioners continued to adopt allowable bearing capacity of 3,000 kPa, which could avoid the need for instrumented pile tests and lengthy approval process.

High-rise buildings exceeding twenty storeys constructed in Scheduled Areas underlain by marble rock formation with cavities are subject to long-term building settlement monitoring. Table 2 tabulates those Yuen Long projects that were carried out between 2010 and 2017 and the presumed bearing pressure adopted in the foundation design. No undue building settlement have been observed for these high-rise buildings and the performance of the foundations are considered satisfactory. The end bearing pressures adopted in these projects ranged from 5,000 kPa to 7,500 kPa.

Table 2: *Long term building settlement monitoring for selected buildings at Yuen Long*

Project Site	Type of Pile Foundation	Founding Criteria	Date of Approval	Adopted Presumed Values	Building Settlement Monitoring (after OP)
Building at Tai Tong Road	Bored piles with bell out	Class II Marble or better, UCS > 50MPa and TCR > 95%	April 2010	End-bearing 7,500 kPa	4mm (4/2014 – 8/2021)
Building at Fung Cheung Road	2.75m and 3m dia. Bored piles with bell out	Class II Marble, UCS > 25MPa and TCR > 85%	December 2010	End-bearing 5,000 kPa	9mm (9/2014 – 8/2021)
Building at Long Ping	2.5m dia. Bored piles with bell out	Class II Marble, UCS > 25MPa and TCR > 85%	July 2013	End-bearing 5,000 kPa	8mm (2/2018 – 8/2021)
Building at On Ning Road	2.5m and 3m dia. Bored piles with bell out	Class II Marble, UCS > 25MPa and TCR > 85%	June 2012	End-bearing 5,000 kPa	2mm (6/2021 – 8/2021)

For piles founded on rock formation, the settlement of the rock mass can be estimated based on the elastic theory (GEO, 2006). Assuming a bored pile with a bell-out diameter of 4.95 m and applying a bearing pressure of 7,500 kPa to the rock mass with a lower bound elastic modulus of 5 GPa for Grade III igneous rock, the cumulative settlement in the rock mass is only about 4 mm. The typical elastic modulus of marble and marble bearing rocks is in fact higher than 30 GPa from Figure 13 and is certainly greater than the assumed 5 GPa used in the estimation. Hence, it is comfortably to conclude that the settlement for piles found on sound marble (i.e., Class I and II) with UCS and TCR comparable to the igneous rocks are considered capable of taking 7,500 kPa.

Given that the engineering properties of sound marble is comparable or better than igneous rocks, and the experience of using presumed values similar to igneous rocks, it is considered plausible to provide clear guidance on the presumed values and founding criteria for piles founded on sound marble rocks. There would be substantial saving in terms of cost and time in the construction of the foundation in marble area. GEO has updated the TGN No. 26 that clarifies that piles founded on sound marble (i.e. Marble Class I or II) are considered acceptable and provides the presumed allowable bearing capacity that can be used. With the revision of the presumed allowable bearing capacity of igneous rocks as given in TGN No. 53, there are scope for further revising the same for marble and marble bearing rocks. The GEO would further examine the suitability of aligning the recommended capacity between igneous rocks and marble rocks.

5 Paradigm Shift on Exercising Geotechnical Control

In recent years, the GEO emphasizes adopting proactive approach in managing requests for geotechnical advice, whether these are formal submissions of detailed design by consultants and registered geotechnical engineer, circulation of land instruments and planning documents; and feasibility and technical studies by other project offices. Whilst public safety remains the primary objective of the regulatory control exercised by the GEO, consciousness of practicality and cost benefits of the geotechnical works are equally important and ought to be exercised. The GEO is committed to continue the journey of promoting the role of regulator and facilitator for the betterment of our society.

Practitioner also plays an important role in shaping the geotechnical practice. There are too many occasions that designers substantially revise the design when confronted by the regulating authority, without arguing and defending the design intent, which they have spent months in developing the scheme and design. Rationale based on sound engineering judgement is the best defense and it does not hurt to bring the matters to the appropriate levels of authority for resolution. The GEO has promoted openness with fellow practitioners in recent years. In fact, pre-submission meeting mechanism has been well documented in the Practice Notes No. ADM-19, BD (2023) and encouraged by the GEO. An Expert Checking Panel mechanism has also been set up in the GEO that allows practitioners to propose innovative ideas and methods to be adopted in geotechnical works. The mechanism aims to provide in-principle agreement to the proposal and give directive on auditing the critical issues at detailed design stage. This gives more rooms for practitioners to discuss and justify their innovative proposals.

The advancements in geotechnical practice achieved in the recent endeavors as encapsulated in this paper are the collaborative efforts by all relevant parties, including practitioners in the industry and regulating authority. From innovative perspective, the progressive mindset helped breaking new ground on geotechnical practice that have significant benefits. There is no magic in these efforts, most issues are tackled from basic principles, engineering judgement and the determination to solve problems. A final concluding remark on advancement in geotechnical practice rests on the perception of “how safe is safe” and the risk aversion nature of fellow practitioners, regardless of which organizations they serve. This is a phycological barrier for many people and makes practicing in the geotechnical discipline even more interesting and challenging. Rather than taking them for granted, confronting the practice that are adopted as routine is the beginning of any advancement.

6 Declarations

6.1 Acknowledgement

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

6.2 Publisher’s Note

AIJR remains neutral with regard to jurisdictional claims in published maps and institutional affiliations.

How to Cite

CHEUNG *et al.* (2023). Advancement in Geotechnical Practice for Smarter and Greener Projects Delivery. *AIJR Proceedings*, 192-214. https://doi.org/10.21467/proceedings.159.16

References

BD. (2004). *Code of Practice for Foundations*. Buildings Department, Government of Special Administrative Region.

BD. (2012). Practice Note for Authorized Persons, Registered Structural Engineers and Registered Geotechnical Engineers APP-57. In *Buildings Department, The HKSAR Government*. https://www.bd.gov.hk/doc/en/resources/codes-and-references/practice-notes-and-circular-letters/pnap/APP/APP057.pdf

BD. (2017). *Code of Practice for Foundations 2017*. Buildings Department, Hong Kong Government of Special Administrative Region. http://www.bd.gov.hk.

BD. (2023). *Building Approval Process Practice Note for Authorized Persons, Registered Structural Engineers and Registered Geotechnical Engineers ADM-19*.

Chung, P. W. K., & Chu, F. L. F. (2020). Review of Quality Control of Fill Compaction Works in Hong Kong. *The HKIE Geotechnical Division 40th Annual Seminar 2020*, 18–27. www.arup.com

Eves, G. W. (1913). THE CANTON-KOWLOON RAILWAY: BRITISH SECTION. (INCLUDING APPENDIX). *Proceedings of Institution of Civil Engineering*, *192*(1913), 190–204. https://doi.org/10.1680/IMOTP.1913.17621

Gaba, A., Hardy, S., Doughty, L., Powrie, W., & Selemetas, D. (2017). Guidance on embedded retaining wall design. In *Ciria c760*. CIRIA. https://www.ciria.org/ItemDetail?iProductcode=C760&Category=BOOK

Gaba, A. R., Simpson, B., Powrie, W., & Beadman, D. (2003). *Embedded retaining walls : guidance for economic design*. CIRIA. https://www.ciria.org/CIRIA/CIRIA/Item_Detail.aspx?iProductCode=C580&Category=BOOK

GEO. (1990). *Review of Design Methods for Excavations*. Geotechnical Engineering Office, Civil Engineering Department. https://www.cedd.gov.hk/filemanager/eng/content_144/ep1_90.pdf

GEO. (1994). *Geoguide 1 Guide to Retaining Wall Design* (2nd ed.). Geotechnical Engineering Office, Civil Engineering and Development Department, HKSAR Government. https://www.cedd.gov.hk/filemanager/eng/content_106/eg1_20200601.pdf

GEO. (2006). *FOUNDATION DESIGN AND CONSTRUCTION*. Civil Engineering and Development Department, HKSAR Government. https://www.cedd.gov.hk/filemanager/eng/content_148/ep1_2006.pdf

GEO. (2007). *Geoguide 7 Guide to Soil Nail Design and Construction*. Civil Engineering and Development Department, HKSAR Government. https://www.cedd.gov.hk/filemanager/eng/content_117/eg7_20170918.pdf

GEO. (2020). Supplementary Guidelines for Foundation Design in Areas Underlain by Marble and Marble-bearing Rocks. In *Geotechnical Engineering Office* (Issue 5, pp. 45–48). https://www.cedd.gov.hk/filemanager/eng/content_427/TGN26-1A_rev.pdf

GEO. (2022). *(TGN 53) Supplementary Guidelines for Foundation Design and Construction*.

Horvarth, R. G. and Kenney, T. C. (1979). Shaft resistance of rock socketed drilled piers. Proceedings of the ASCE Annual Convention, Atlanta, Georgia. Pre-print No. 3698. https://cedb.asce.org/CEDBsearch/record.jsp?dockey=0031931

HyD. (2016). *GUIDANCE NOTES ROAD INSPECTION MANUAL (RIM)*. https://www.hyd.gov.hk/en/technical_references/technical_document/guidance_notes/pdf/gn016c.pdf

ICE. (2012). ICE Manual of Geotechnical Engineering: Geotechnical engineering principles, problematic soils and site investigation. In *Institute of Civil Engineers Manuals series*. Institution of Civil Engineers. https://books.google.com/books/about/ICE_Manual_of_Geotechnical_Engineering_G.html?id=oI0-XwAACAAJ

Irfan, T. Y., & Powell, G. E. (1991). *Foundation Design of Caissons on Granitic and Volcanic Rocks* . Geotechnical Engineering Office, Civil Engineering and Development Department, HKSAR Government. https://www.cedd.gov.hk/filemanager/eng/content_161/er8_section1.pdf

Lee, W. Y. (2019). A study on brittleness of granitic rocks in Hong Kong. *The University of Hong Kong*. http://hub.hku.hk/handle/10722/284437

Lumb, P. (1972). Slope Failures in Hong Kong. *Quarterly Journal of Engineering Geology and Hydrogeology*, *8*, 31–65. https://hkss.cedd.gov.hk/hkss/filemanager/common/publications-resources/list-of-technical-papers/401_Lumb%20(1975)_Slope%20failures%20in%20Hong%20Kong.pdf

MOT. (2015). *Specifications for Design of Highway Subgrades*. Ministry of Transport of the People's Republic of China.

Proceedings DOI: 10.21467/proceedings.159
ISBN: 978-81-965621-6-8

Series: AIJR Proceedings
ISSN: 2582-3922