# Geotechnical Aspects of a Major Site Formation in Hong Kong

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**Summary** This paper presents the case history of a large Design/Build site formation contract recently completed in the east Kowloon region of Hong Kong. Site formation works comprised the excavation of a seven-hectare building platform designated for major high-rise residential development into existing hillside, requiring large-scale slope formation and stabilisation works. The main focus of the paper is a description of the geotechnical aspects of the site formation works, with particular emphasis on the various measures adopted by the designer with respect to slope stabilisation, much of which required design as construction proceeded. Also discussed briefly are aspects of the design/build contract which facilitated integration of an on-going design process into construction.

## 1. INTRODUCTION

The long-sustained economic growth experienced in Hong Kong and recent government directives regarding the provision of public housing into the next millennium have resulted in a number of large-scale site formation projects being brought on-line to produce land suitable for high-rise residential development. The naturally hilly terrain of Hong Kong dictates that useable land must either be won from the ocean by means of reclamation, or from existing hillside through excavation, with the associated formation of significant man-made slopes.

The Po Lam Road Site Formation Contract was let by the Civil Engineering Department of the Hong Kong Government under a Design/Build arrangement in March 1995. Construction activities commenced almost immediately, with final design and construction completed over a period of approximately three years, culminating in site handover to the Hong Kong Housing Authority in early April 1998. The Design/Build form of the contract was chosen primarily as a means to early procurement of the completed works, on a project which required significant ongoing geotechnical design activities as construction proceeded.

This paper describes the nature of the site formation works and the geotechnical aspects of the design and construction process, including the site geology and groundwater regime as well as the impact of site topography and other existing conditions. It provides discussion on slope formation, with particular emphasis on the slope treatment and stabilisation techniques adopted by the designer. These have included geogrid reinforced fill, soil nailing and extensive use of rock dowels.

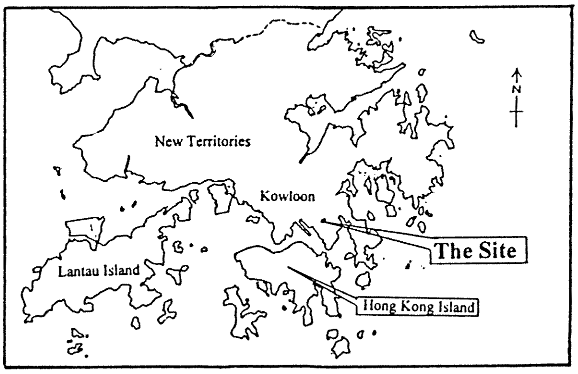


Figure 1. Site location plan.

## 2. THE SITE

### 2.1 Site Description

The contract works area occupies approximately 13 hectares and is located on southerly sloping hillside in the Sau Mau Ping area of eastern Kowloon in Hong Kong (refer Figure 1). The site is bound by heavily used distributor roads to the north and west, and the toll plaza for the Kowloon to Tseung

Kwan O tunnel link, constructed on a filled valley, to the south.

The ground elevation prior to commencement of works ranged from around +170mPD (metres above principal Datum) at a small knoll in the central eastern portion of the site to +80mPD along the southern boundary. A major natural drainage path was located to the east of the knoll and two smaller gullies were located to the west. Previous land-usage identified through aerial photographic interpretation (API) included open container storage and a concrete batching plant. Substantial existing cut slopes in soil and rock were located along much of the western site boundary. An existing carriageway traversed the site from north to south, with a number of associated cuttings present along the southern portion.

The entire southern portion of the site, as indicated by API, had at some stage been occupied by 'squatters', resulting in numerous small terraces, platforms and pathways in the hillside. The natural slopes angles in this area ranged up to 30° to 40° and were very heavily vegetated - this area required significant ongoing design work as construction moved to this area.

## 2.2 Geology and Ground Investigation

Desk study revealed that the site geology comprised an insitu weathering profile of fine to medium grained granite. This was confirmed by ground investigation, which indicated that the weathering profile was reasonably variable in thickness (3m to 20m), but generally conformed with the typical pattern for the bedrock type, as reported, for example, by GCO (1984), comprising material of weathering grades VI and V (residual soils and completely weathered bedrock) overlying corestones of weathering grades IV to II (highly to slightly decomposed bedrock) in a grade V matrix, and essentially competent bedrock of weathering grades III and II. A thin veneer of fill material was recorded locally, associated with previous terrace formation, mainly by squatters.

## 2.3 Groundwater

Piezometer records indicated a variable groundwater regime within the site. The ambient groundwater table was generally located +/- 5m relative to rockhead. Obviously, previous site activities would have had a substantial influence on the groundwater conditions, as would the future development (including significant cuttings) and this was borne in mind during the design process.

# 3. DESIGN PHASE

## 3.1 Design Brief

The design brief, as contained in the Employer's Requirements presented the following as the main project objectives:

- The formation of a building platform approximately 7 hectares in area at an elevation of +120mPD.
- The provision of a temporary access carriageway along the northern (Po Lam Road) boundary.
- The upgrading of the existing carriageway remaining after platform excavation for use as an Emergency Vehicular Access (EVA) from Sau Mau Ping Road.
- The investigation and assessment of all sloping ground and retaining walls within the site to confirm adequate long term stability.
- The provision of a stormwater drainage system to the platform area, sloping ground and carriageways.
- The provision of soft landscaping to excavated and made ground beyond the platform area.

## 3.2 Conforming Design

The conforming design is subdivided into the following areas, for ease of description:

- Northern Slopes
- Po Lam Road Access
- Emergency Vehicular Access
- Southern Slopes
- Stormwater Drainage

The permanent works design is presented in plan view in Figure 2.

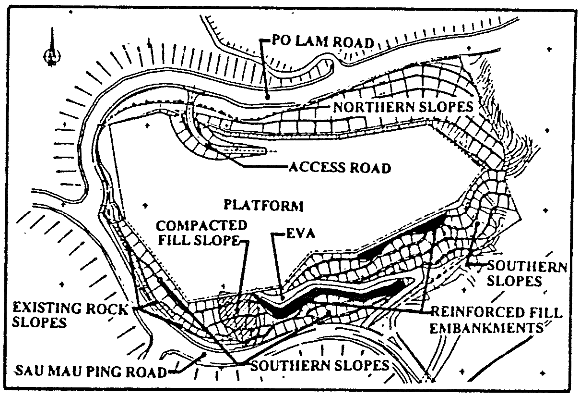


Figure 2. Site formation works design.

The northern slopes resulting from excavation to the platform level, and which reached a maximum height of approximately 50m, were designed as stand-alone cut slopes in the weathered granite and intact bedrock beneath. The design was based on providing 1.5m wide berms for maintenance access and stormwater drainage at 7.5m vertical intervals, and applying a maximum gradient in soil of 1V:1.5H and 2V:1H in bedrock. The soft slope gradient was determined from stability analyses carried out using a pc-based software package approved by the Hong Kong Government, based on drained shear strength parameters of $c' = 5$ kPa and $\phi' = 35°$. These values fall within typical published ranges in Hong Kong and were confirmed by laboratory testing. A design groundwater table located 2m above rockhead was adopted, which allowed for 'perching' during the wet summer months. The final geometry adopted for the northern slopes, comprised up to five benches of soft slopes and a single rock slope bench immediately above platform level. This geometry acknowledged the need for an open, well ventilated and naturally well lit area for development, and was considered more aesthetically pleasing than other past examples of site formation in Hong Kong. The design brief made allowance for the assessment and design of stabilisation measures for the rock cut slopes during the construction period, as the slopes were exposed.

A typical section through the northern slopes is presented in Figure 3.

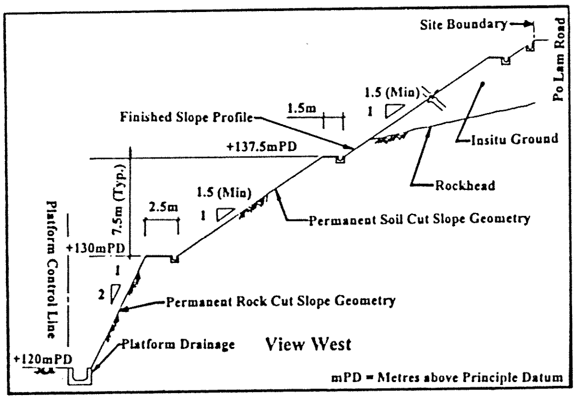


Figure 3. Typical section - northern slopes.

The Po Lam Road Access was designed as a simple earth embankment with side slopes of 1V:2H and concrete pavement in the carriageway for durability, since the prime function of the access would be to carry construction vehicles during the subsequent building contract period.

The design of the EVA essentially comprised a flexible overlay to existing pavement and the provision of drainage, kerbing and guard barriers.

The southern slopes comprised the major challenge to the designer. This was due to the extremely variable topography, much of which was relatively steep, the variable ground profile, heavy vegetation and lack of access which prevented detailed investigation during the design period. However, on the basis of what survey information was available, and making reasonable allowance for the anticipated thickness of unsuitable material located above the insitu ground profile, a design slope layout was developed on the basis of maximising the area of stand-alone cut slopes, with maximum gradients as outlined earlier, to minimise formation costs. The resulting layout dictated the inclusion of a number of additional engineered features in order to achieve the specified platform area at the required elevation and maintain safe slope geometry throughout.

These included:

- The filling of a valley at the south-western end of the platform area to achieve formation within the platform area, with a formed fill slope gradient of 1V:2H extending to the site boundary.
- The filling of two small valleys in the central southern portion of the platform area to again achieve formation and the retention of the fill material by a geogrid reinforced fill embankment, necessary for geometry reasons and chosen on aesthetic as well as economical grounds.
- The support of the EVA carriageway by a second geogrid reinforced fill embankment.
- The application of soil nailing to reinforce locally oversteep sections of soil cut slopes and maintain overall stability. Generic designs were compiled to facilitate site design decisions as construction proceeded.

The design also included the upgrading of existing rock cuts located along the western site boundary, in much the same manner as for the northern rock cuts described above.

The stormwater drainage system, designed on a 200 year return period, comprised a comprehensive network of open and covered channels located on the slopes and around the platform perimeter. Points of discharge utilised existing facilities, both man-made and natural, in the southern and western parts of the site.

Soft landscaping elements comprised hydroseeding with a grass and tree seed mix.

## 4. PROGRAMME

Due to the Design/Build form of the contract and the fact that the major portion of the site formation works comprised a large earthmoving operation, initial construction activities comprising site clearance and removal of soft material were able to commence immediately and in parallel with the detailed design. The period for completion of design was originally anticipated at around 6 month's duration, which included time for a rigorous checking and approval route. This comprised review by an independent checker and geotechnical checking by the Government's Geotechnical Engineering Office (GEO). The design was also passed to all relative Government Departments for comment, prior to issue to the client department for final approval. The Employer's Requirements permitted the design to be split into a number of packages to allow a staged submission process, each detailed design package preceded, in any case, by a submission for approval-in-principle, and this was the option pursued. Via this route the main design phase actually covered a period of approximately 18 months, with approximately 70% of the submissions gaining final approval within 8 to 9 months of commencement. Due to the non-critical nature of the design to the major excavation phase of construction, the additional time required to gain design approval was absorbed by the programme and impacted little on site activities.

Blasting of rock within the platform area commenced in October 1995, roughly in line with the completion of bulk removal of overburden and following successful application for a Government issued Blasting Permit. Blasting continued late into the contract period, with final platform formation being completed in September 1997.

Slope formation commenced in early September 1996, with separate sub-contracts being let in tandem for the northern and southern slopes. Final slope works were completed in late March 1998, with the site being handed over in early April. The landscaping contract completion date is June 1998, with an ensuing 12-month maintenance period on all works.

The major design and construction activities are presented in Figure 4.

A photograph of the site formation near completion is presented in Figure 5.

## 5. CONSTRUCTION PHASE

### 5.1 Bulk Earthworks

The overall volume of material excavated from the site totalled approximately 2 million $m^3$ of soil and rock in roughly equal quantities, which, apart from the filling material required for the southern slopes formation, was disposed of off-site. The soft overburden was deposited in a public dumping facility in nearby Junk Bay, while the excavated rock of adequate quality was sold to a nearby quarrying facility, where it was processed for use as construction aggregate.

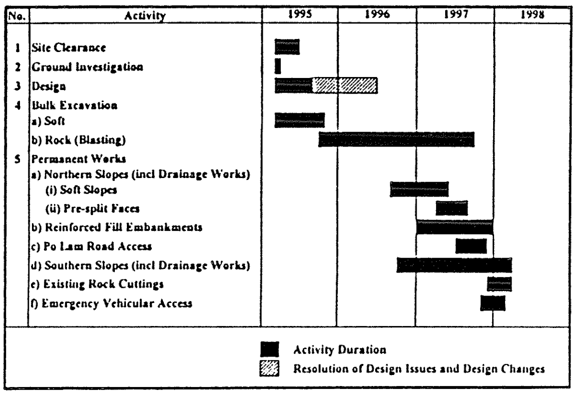


Figure 4. Programme.

Excavation of the overburden material was performed primarily by hydraulic excavators directly loading dump trucks and lasted for a period of approximately 30 months.

Forming part of the application for the Blasting Permit, a blast appraisal report was compiled which considered the allowable size of blast in relation to adjacent sensitive structures and slopes. The permissible size of blast was limited to a large extent by the presence of a gas main located along the northern and western site boundaries necessitating that peak particle velocities be limited to 10mm/s. This impacted significantly on output rates.

The extent of bulk blasting was maintained a minimum distance of 10m in any direction from a permanent slope face in order to reduce the likelihood of fragmentation encroaching beyond the finished slope profile. Controlled blasting incorporating pre-split was employed to form the main 2V:1H rock cuttings.

Blasting continued almost throughout the slopes formation period, the last shot being fired on 27 September 1997.

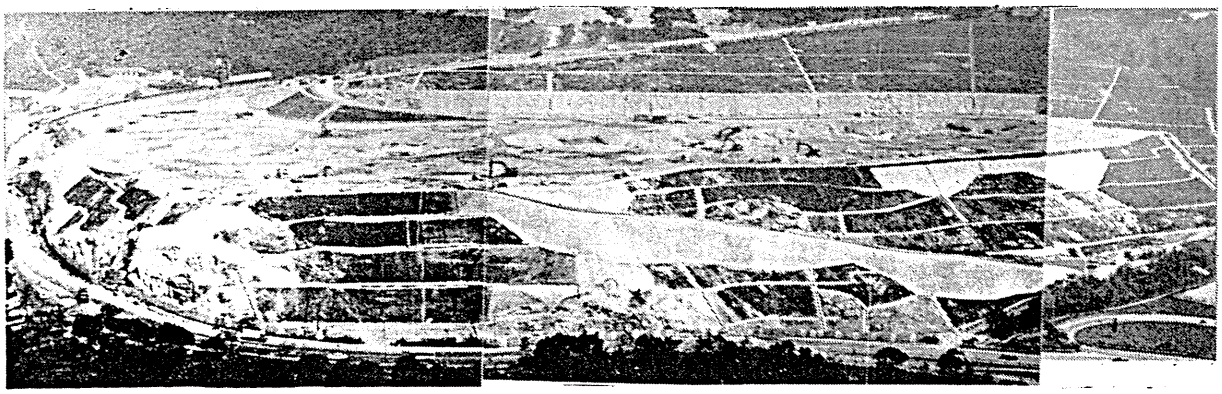

Figure 5. Site formation works nearing completion (March 1998).

### 5.2 Soft Cut Slopes

An overburden of approximately 6 m was left in place above the final slope profile during bulk earthworks, which was subsequently removed in a top-down operation during the slopes formation phase.

An interactive approach was maintained between designer and contractor, the latter providing the proposed final profile and ground survey information for checking in advance of formation, in order to permit fine-tuning of the final layout by the designer. This process was essential for the successful formation of the southern slopes, where the conforming design required refinement in a number of areas. It also permitted advantage to be taken of the natural landform, reducing excavation volumes and producing a more pleasing visual appearance.

Construction of drainage works followed closely behind the slope formation process, and these activities were in turn followed by hydroseeding of the slope face at an early stage.

The total face area of soft cut slopes formed during construction amounted to around 50,000 $m^2$.

### 5.3 Fill Slopes

Fill slope formation comprised an initial phase of placement and compaction of fill material, overfilled to the final profile, following preparation of the natural interface by removing unsuitable material, benching and placement of granular filter and drainage layers. Once platform level was achieved, trimming to the final profile was carried out in the same manner as for the soft cut slopes. A minimum thickness of fill, including the filter/drainage layers, of 3m was maintained throughout the section.

### 5.4 Geogrid Reinforced Fill Embankments

Reinforced fill embankments were designed on the basis of providing sufficient density of reinforcement (in terms of tensile strength and vertical spacing) to maintain an adequate degree of stability against shear failure developing along surfaces within the reinforced zone (internal stability), and sufficient length of reinforcement to maintain an adequate degree of stability against shear failure developing along surfaces beyond the reinforced zone (external stability). A partial safety factor approach based on force equilibrium was adopted for the internal stability analyses, described by Wong (1995), and a conventional global approach for the external stability analyses.

The use of proprietary reinforced earth systems in Hong Kong is effectively controlled by a government based prior approval system, with an Endorsement Certificate, for use in design and construction, being issued following satisfactory completion of the documentation process for a particular product. The standard specification adopted for construction of reinforced earth embankments is the Hong Kong Government Model Specification (GEO (1989)) which includes requirements for verifying design assumptions regarding the shear strength of the fill and the interaction of the geogrids and the fill through direct shear box testing.

The primary geogrids adopted for the embankment construction were from the "Tensar" range of uniaxial grids produced in polyethylene by Netlon Ltd. Given the relatively gentle face gradient chosen for the embankments (1V:1H), the permissible vertical spacing of grids in the upper portion of the reinforced zone was 1m, necessitating the inclusion of lighter, biaxial grids as secondary reinforcement midway between primary grids to reduce the likelihood of small ravelling type failures on the embankment face.

The maximum height of embankment indicated as being required at design stage was around 11m. However, a section with a maximum height of 12.5m was developed to allow for deviation in the actual ground surface, and the section incorporated a 1m thick granular replacement foundation to act as a bridging layer across any soft material encountered at founding level in the valleys. The maximum length of geogrids required was 10m and the minimum vertical spacing was 300mm.

A typical section of the reinforced fill block geometry as presented in Figure 6.

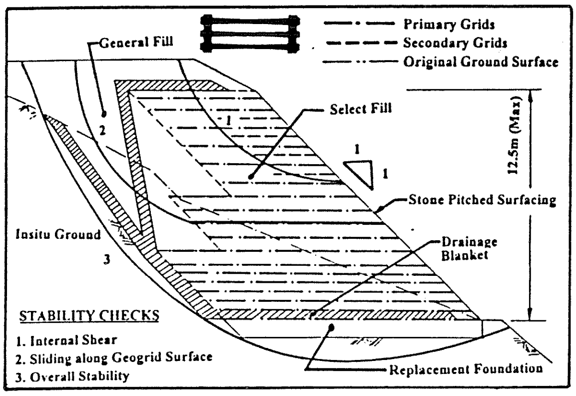


Figure 6. Typical section - geogrid reinforced fill embankments.

Because of the gentle face gradient, the geogrids were able to be simply terminated at the slope face, rather than having to use the "wrap-around" necessary for steeper structures. Due to concerns held by the end-user over vandalism of the structures causing damage to the grids, a stone pitched facing was adopted, which was both non-combustible and aesthetically more pleasing than a shotcreted finish.

The construction of the two embankments proceeded in much the same fashion as for the unreinforced fill slope described earlier, with the grids being placed to pre-determined setting out details. Proof testing of the on-site source of the select fill material for the reinforced zone was carried out during bulk earthworks operations and comprised the determination of particle size distribution, Atterberg Limits, the friction angle of the fill material and the coefficient of friction between geogrids and the fill material. Compliance testing of the select fill performed during the filling operations included classification, relative density and compaction tests.

The combined length of the two embankments was 350m with an average height of 10m. A total of 25,000m$^3$ of reinforced fill was placed.

## 5.5 Soil Nailing

Soil nailing as passive soil reinforcement was employed in areas where soft slopes had to be formed at gradients in excess of the design maximum. The generally accepted principle of soil nailing in the Hong Kong perspective is well described by Watkins and Powell (1990), and is that of an unstressed reinforcing element which provides a stabilising tensile resistance to potential shear failure developing along surfaces within the reinforced zone. Resistance to pullout is developed by skin friction over the length of nail embedded beyond a failure surface.

Soils nails generally comprise a galvanised high yield steel bar of diameter ranging from 25mm to 40mm, which is fully grouted into a pre-drilled hole of around 100mm diameter and inclined at angles of 10° to 20° below horizontal. The nail is provided with a reinforced concrete head, and nail spacing is typically of the order of 2m on a horizontal and vertical staggered grid. The length of soil nail varies with each application, though a rough estimate of 80% of the slope height, assuming full embedment in soil is considered reasonable. The soil nails used here ranged from 6m (with rock embedment) to upwards of 15m.

Typical Soil Nail details are presented in Figure 7.

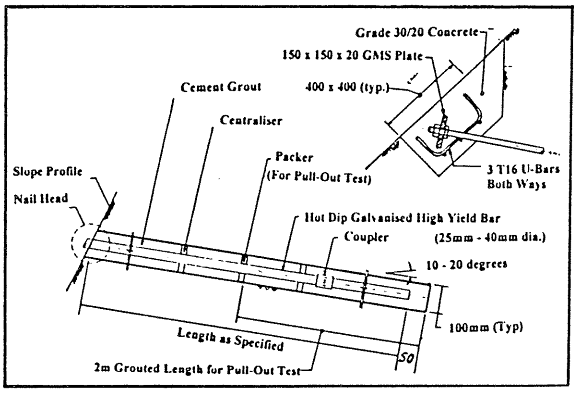


Figure 7. Soil nail details.

Load testing of sacrificial nails, grouted over the lower 2m, was carried out at an overall frequency of 6% of working nails, with at least 1 test for each individual zone of nails. The load test comprised three cycles of loading by hydraulic jack to a maximum load equal to 1.5 x the design working load for the 2m grouted length. Each cycle was held constant for a one hour period. Bar extension was monitored throughout. Acceptance criteria was for movement along the grout/soil interface to be not more than 0.3% of the grouted length.

A typical soil nail pullout test Load/Extension plot is presented in Figure 8. The total number of soil nails installed at the site was 300 and length totalled 3,500m.

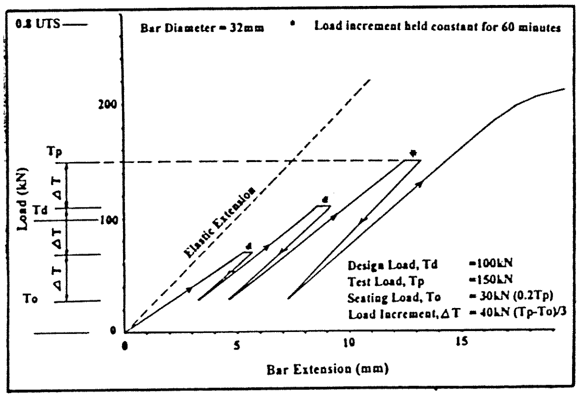


Figure 8. Typical soil nail pullout test load/extension plot.

## 5.6 Rock Slopes

The rock slopes incorporated in the permanent works comprised a total face area of 5,000m$^2$ and fell into three basic categories. These were:

- Pre-split cuttings formed at a gradient of 2V:1H along the northern edge of the platform (10m max height)
- Existing rock cuttings located along the western site boundary for which no further excavation was proposed (approx. 20m max height)
- Sundry rock cuttings and exposures formed in the southern portion of the site which were left oversteep in the interests of reducing excavation volumes (2 to 3m max. height)

All of the rock cuttings were assessed with regards to overall stability on a progressive basis during the construction phase, and to a programme synchronized with the contractor's mobilisation of resources to install stabilisation measures.

The rock mass characteristics of each cutting, or section of cutting, were logged through visual appraisal and line surveys along the toe with respect to joint orientation. Potential instability was assessed on the basis of stereographic construction and further visual appraisal as a confirmatory measure. Stabilisation measures were generally applied prescriptively where only minor instabilities were present, and only in the event of a major potential instability was a rigorous assessment carried out.

The primary means of stabilisation, where required, was untensioned, grouted high yield bars, or "dowels", as described by Bjurstrom (1973), chosen on the basis of simple installation and there being no requirements for testing, maintenance and/or monitoring. Bar diameter ranged from 32mm - 40mm and lengths were typically of the order of 2 to 3m. Dowels were applied singly, and on a uniform grid (pattern dowels) where the exposed face was generally fractured or more highly weathered than the surrounding areas.

Reinforced sprayed concrete, or shotcrete, was applied extensively as surface protection. Relief drains were also incorporated to provide discharge to identified seepage paths. To maintain a uniform appearance at platform level, the decision was taken to shotcrete the majority of the pre-split face, though shotcrete was kept to a minimum on the existing cuttings for aesthetic reasons.

## 6. CONCLUSIONS

This paper has presented the case history of a large site formation project, recently completed under a Design/Build arrangement in Hong Kong. A description of the project design and construction phases has been presented, with focus provided on geotechnical elements of the permanent works. The successful interaction between the designer and contractor has been highlighted, in particular, the use of a designer on site to facilitate geotechnical assessments and enable a rapid and on-going construction process.

## 7. ACKNOWLEDGEMENTS

The paper is published with the permission of the Civil Engineering Department of the Hong Kong SAR Government.

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