International Geotechnical Symposium on Theory and Practice of Earth Reinforcement / Fukuoka Japan / 5-7 October 1988


# Insitu ground reinforcement for slope improvement in Hong Kong

K.P.Yim, A.T.Watkins & G.E.Powell
*Geotechnical Control Office, Hong Kong*

ABSTRACT: The Geotechnical Control Office of the Hong Kong Government is responsible for the Government's long term Landslip Preventive Measures (LPM) Programme. This programme is set up to reduce the risk to the public from landslide through the rectification of unsatisfactory slopes and retaining walls in the densely populated terrain of Hong Kong. Insitu ground reinforcement techniques using micro/minipiles and soil nails have been applied in the LPM Programme for stabilising slopes and retaining walls. The techniques are particularly useful for stabilizing existing slopes where more conventional regrading of the slope is not possible or where access or site congestion pose difficulties.

The paper describes typical design and construction practice for insitu ground reinforcement techniques using micro/minipiles and soil nails in Hong Kong. Some case histories are presented to illustrate the benefit gained by the application of the techniques, the range of the application, the design principles, and the construction methods and costs.

## 1 INTRODUCTION

Many cut slopes in Hong Kong formed before 1977 were designed empirically to an angle of 10 vertical to 6 horizontal without much regard for geological or hydrological characteristics of the slope (Lumb, 1975). Such slopes generally do not meet current safety standards, and, on investigation, are found to have unsatisfactory factors of safety; accordingly, remedial works are required to upgrade them. By far the simplest means of improving a cut slope is to cut it back to a flatter angle which, despite possibly exposing the slope to a greater risk of infiltration, provides a satisfactory and economic solution. Where space permits, this is the general approach adopted. However, many slopes have substantial buildings above the crest, as well as at the toe, rendering cut back solutions inappropriate. This means that structural buttressing or other support systems is required. More recently, slope reinforcement systems in the form of soil nails, passive tie-backs and pile stitching have become a preferred option. Indeed, in some cases, these methods have been shown to be more economic than a cut back solution.

Many fill slopes constructed prior to 1976 were formed by end tipping. The loose fill was liable to liquefaction following heavy rainfall, and a number of disasters occurred (Vail 1984). The most common form of remedial work to these slopes has been to recompact the outer 3 m of the slope and provide surface and subsurface drainage. Site constraints do not always allow the recompaction approach and other methods, including reinforcement, have been used. Some of these older fill slopes now have well-established tree cover. Apart from the aesthetic value of the trees, the roots provide additional shear resistance near the surface. Engineering solutions which do not involve removal of the trees are therefore favoured in these circumstances.

This paper presents three case histories of stabilization works where reinforcement systems have been used. Site condition, design considerations, economic considerations and performance testing are discussed.

## 2 DESIGN PHILOSOPHY

In all of the cases outlined in this paper, additional shear capacity was provided to slopes which were considered marginally stable, but which had not yet failed. The conditions which would lead to failure on these slopes are the result of saturation of the soil and increased pore water pressures. Such conditions are likely to occur only very infrequently, as a result of rare intense rainstorms.

Investigation of the stability of marginally stable slopes in Hong Kong by means of conventional soil mechanics approach, as outlined in the Geotechnical Manual for slopes (Geotechnical Control Office, 1984), often leads to calculated factors of safety at or below 1.0 (Brand, 1985). However, because the majority of older slopes have been subjected to rainstorms of intensities greater than or equal to 1:10 year return period storms and have not failed, an increased degree of confidence can be held in designing remedial works. The Geotechnical Manual recognises this, and allows for remedial works on existing slopes to provide a 20% improvement in Factor of Safety for high risk to life situations, over the existing worst known condition for which a factor of safety of at least 1.0 may be assumed.

There are currently no codes or official guidance on standards to be applied to the design of insitu ground reinforcement when used for the improvement of existing slopes, although a recent publication by the Geotechnical Control Office (1988) deals with new reinforced fill structures.

The design approach has been to determine the force required on any given potential slip surface necessary to increase the factor of safety to the required standard ($F > 1.2$). The provision of this force is made through the reinforcement, either in shear in the case of minipiles, or through tension in the case of soil nails and passive tie-backs. The design methods for these are briefly outlined in each case history.

The actual margin of safety of the completed works is generally higher than that required to achieve an overall 20% improvement in the factor of safety, since additional allowance is made for possible long term corrosion of the reinforcement. A combination of low design stresses, generally 30% ultimate tensile stress (UTS), together with a sacrificial thickness of 2 mm on each surface is made. All the reinforcement is protected by a grout surround with a minimum thickness of 6 mm. More stringent corrosion protection is provided where adverse ground or water conditions are expected.

The reinforcement is not likely to be stressed to any significant level for the majority of its lifetime, for the reasons outlined above. It is therefore not considered productive to monitor the long term performance of such structures. Rather, a conservative approach is adopted for both design and corrosion allowance. Where possible, design assumptions, such as soil/grout bond and lateral resistance are confirmed on a site-specific basis.

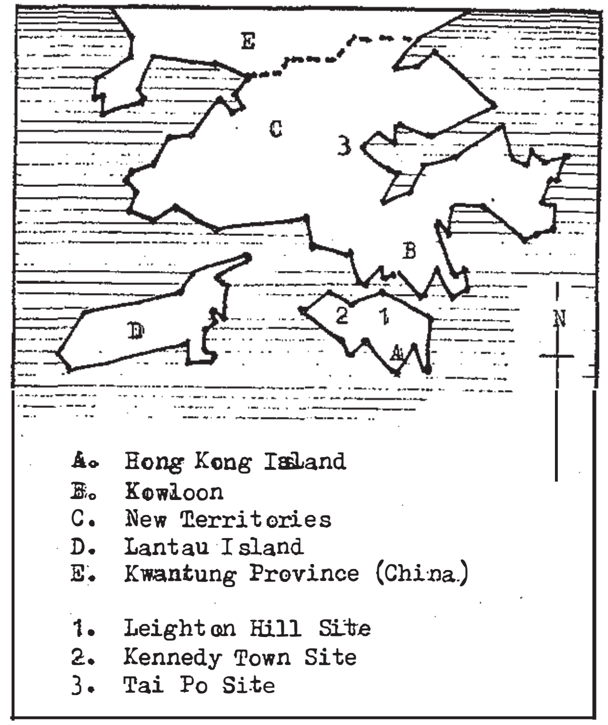


Figure 1. Location plan

## 3 CASE HISTORIES

Each of the following examples discusses a case history where steel inclusions grouted into drilled holes were used to improve the stability of an existing slope. The first case study deals with the reinforcement of a fill slope by the use of a minipile stitching system. The second case study deals with a case where micropile passive tie-backs were to introduce efficiency to a structural retaining and partial cut back system. The third case study deals with a soil-nailed slope. The location of the sites is shown in Figure 1.

### 3.1 Leighton Hill fill slope

At the crest of a 22 m high fill slope, the sole vehicular access to a number of residential buildings is supported on a mass concrete gravity wall. The slope overlooks two primary schools built very close to the toe of the slope, as shown in cross-section in Figure 2.

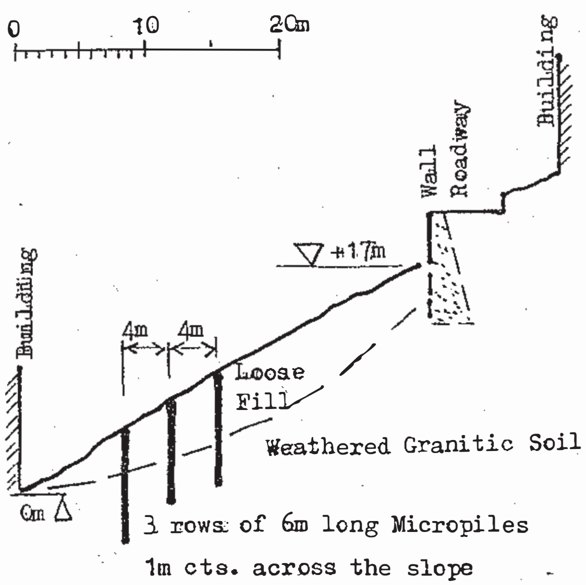


Figure 2. Cross-section of slope at Leighton Hill

Site investigation showed that the fill varied between 3 m to 7 m in depth and was underlain by some 20 m of weathered granitic soil above granitic bedrock. The fill material was decomposed granite in a loose state, with an average insitu dry density of only 1.4 $t/m^3$. Based on consolidated drained triaxial compression tests, the shear strength of the fill was found to be in the region of $c' = 0$, $\phi' = 35^{\circ}$. Stability analysis, using the simplified method of Janbu (1972), showed low factors of safety for trial slips in the fill layer. As the fill slope has a rather dense cover of trees, a study was conducted to quantify the tree root reinforcement effect on the slope stability. This study followed the procedures described by Greenway (1987). However, the tree roots were found to be only effective in increasing shear strength in the top 2 m. The factor of safety for deeper slips was still below the minimum of 1.2 required for existing slopes which pose a high risk to life. Preventive works were therefore necessary.

Because of access constraints and the wish to maintain the existing tree cover, it was decided not to recompact the slope. Instead, a design was adopted involving the installation of minipiles vertically into the slope face in three rows across the slope at 1 m centres. Each pile was designed to be 6 m long and to comprise a 16 mm thick steel hollow circular section of outer diameter of 190 mm encased in a 220 mm diameter grout column. A 2 mm sacrificial thickness allowance was made for corrosion.

The design principle was to increase the shear resistance of the slope by reinforcing the soil in the area of concern. It was postulated that when the soil moved over any potential surface, it would exert pressure on the vertical reinforcement which, in turn, would mobilize reaction from the soil to stabilize the slope. The interaction between the soil and the reinforcement is therefore very similar to that of a pile subjected to lateral force. The piles were analyzed as free head piles using the method of Broms (1964). The pile was also checked by the method derived by Ito & Matsui (1975).

Because of concern that drilling might cause large ground movement in the loose fill air was used as a flushing medium for drilling, and the drilling sequence was staggered so that no drilling was permitted within a distance of 10 m from any uncompleted piles. The lateral resistance of the piles was confirmed by jacking the piles and measuring deformation.

The cost of the completed slope works was \$US50 per $m^2$.

### 3.2 Kennedy Town cut slope

The existing slope was 35 m high, $50^{\circ}$ formed in completely weathered volcanic tuff, below a $25^{\circ}$ natural slope. A multi-storey residential building was located 4 m to 5 m from the toe of the slope. Stability analysis showed that the factor of safety of the soil portion of the slope could be marginal during heavy rainfall.

The conventional solution of cutting back the slope was only found to be practical in improving that part of the slope where bedrock outcropped at the toe of the slope. However, at the central highest part of the slope, where bedrock did not outcrop and the groundwater table was high, stabilization of potential deep-seated slips extending to the toe of the slope would have required cutting to a gradient of $33^{\circ}$. This solution would have meant substantial excavation into the uphill natural slope, which would have been expensive, and would have presented problems in the maintainance of a smooth transition profile to adjacent parts of the slope. For these reasons, and because of other constraints such as access, environmental disturbance, and the risk of disturbing boulders on the natural slopes above, the total cut-back solution was not selected. The design option selected consisted of a $45^{\circ}$ cut back of the upper portion of the central part of the slope, together with provision of a support structure at the toe. This structure comprised a reclining concrete slab cast directly onto the surface of the slope, supported on rock socketed caissons at the toe and with a row of sub-horizontal micropiles acting as a passive anchorage system at roughly mid-span of the slab.

Figure 3 provides a cross-section of the slope and illustrates these works.

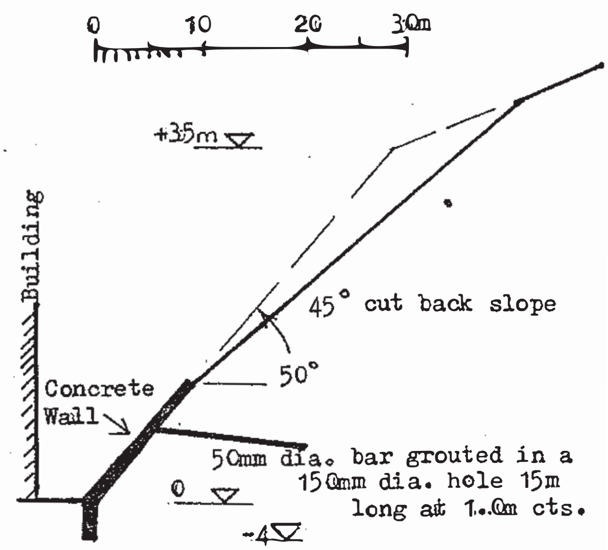


Figure 3. Cross-section of slope at Kennedy Town

The support structure was analysed conservatively as a propped cantilever. It was assumed that, when the soil mass moved over a potential failure surface, it would exert pressure on the reclining slab. The slab would then mobilise resistance from the caissons and the tie-back piles, to restrain the moved soil mass. The earth load on the reclining slab was assumed to act in the horizontal direction. The magnitude was determined by calculating the lateral earth force required to increase of factor of safety of any particular potential failure surface to the required standard. In the absence of knowledge as to how the earth pressure might be distributed, triangular and rectangular earth pressure distributions were both used.

The 1 m diameter rock-socketed caissons, spaced 3 m apart and connected by a capping beam, were designed as laterally loaded piles. Stiffness calculations indicated that the tie-back piles could, within acceptable deformation limits, be expected to resist about half of the earth load and to transmit the load beyond the region where the factor of safety was found less than the required value. The design of the tie-backs followed the anchor design principles outlined in Littlejohn & Bruce (1977) and DD81 of the British Standards Institution (1982). Calculations showed that a high yield steel bar of 50 mm diameter in a 150 mm diameter grouted borehole were required at 1.0 m centres, and that the required movement of the slab at the position of the pile was only some 2 mm to 6 mm in a sub-horizontal direction. This was considered to be an acceptably small movement.

Details of the tie-backs are shown in Figure 4. These are the same as bar anchors, except that the tie-backs were not stressed. Design stresses in the bar were limited to 30% UTS. A 2 mm sacrificial thickness was provided on the steel section and, as an additional protection against corrosion, plastic corrugated sheathing was specified. Testing was carried out to substantiate the pull-out capacity of the tie-back piles. Proof tests to 10% of the piles, whereby a load of up to 1.6 times the design load was applied over 5 cycles, as well, as acceptance tests to other piles where a single load of 1.6 times the design load were applied to the completed pile. Deformation limits were specified as a means of acceptance. The cyclic test sequence is illustrated in Figure 6.

Some thought was given to installing strain gauges to monitor the performance of the wall and tie-back piles in service. However, given that loads in the passive tie-backs would only develop when the factor of safety approached 1.0, it was felt that data useful for design substantiation would not result.

The cost of the slope works described was US$48 per m².

### 3.3 Retaining wall at Tai Po

The 10 m high stone masonry wall, 1 m thick, was built over 40 years ago against a steep slope cut into decomposed Granodiorite. Apart from the facing stones, there was very little interlocking between the blocks which made up the wall as there is no mortar or other packing between the slightly rounded stones which comprised the bulk of the wall. The wall was in poor condition showing signs of lateral and

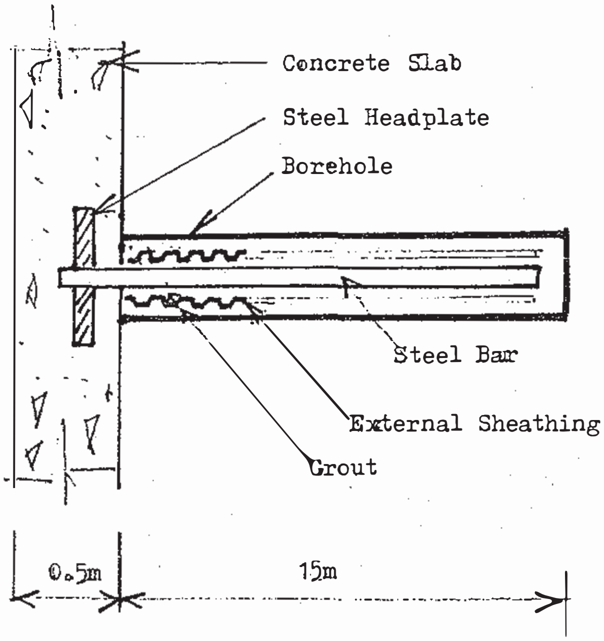


Figure 4. Details of the tie-back micro-pile

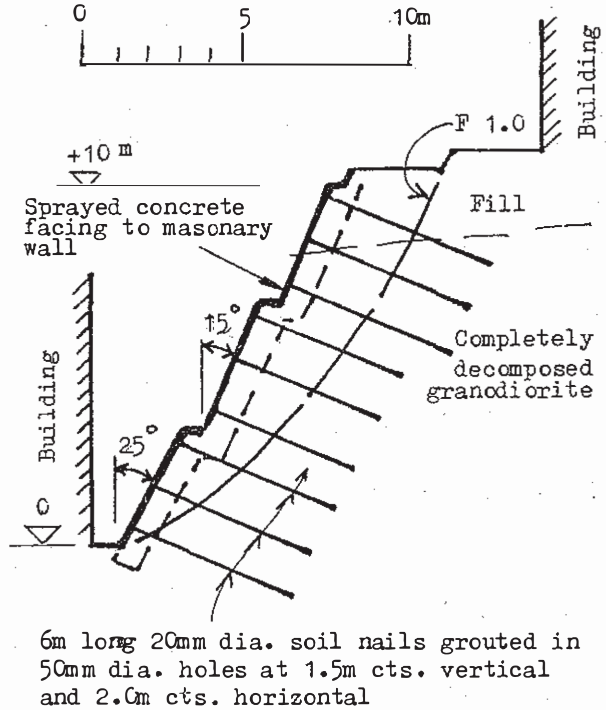


Figure 5. Cross-section of slope at Tai Po

vertical displacement. There are 3 storey village houses near to the top and within 1 m from the toe of the wall. Only pedestrian access was available to the site and because of this, together with the very congested nature of the site, the soil nailing design option had a great deal of appeal. The site is illustrated in Figure 5.

The slope behind the wall was analyzed taking into account the effects of the wall as a surcharge, but ignoring any shear capacity it may provide. Site investigation and subsequent monitoring and testing allowed the development of a geological/hydrological model which revealed a factor of safety of only 1.0 when analysed using Janbu's rigorous method.

To design the soil nails a factor of safety of 2.0 was applied to the pull-out resistance between the soil nail and the ground, and the tensile stresses in the nail were limited to 30% UTS. An additional 2 mm was allowed on bar diameter as sacrificial thickness. The bars used were 6 m long, 20 mm dia. high-yield steel bars conforming to BS4449, spaced 1.5 m vertically and 2.0 m horizontally. A 100 mm thick mesh reinforced facing of sprayed concrete was applied to the outer surface of the wall, and head plates bearing against this surface were fixed to the soil nails.

Using the same analytical method, the horizontal force necessary to upgrade the minimum factor of safety to 1.2 was calculated, and a system of soil nailing was designed which could provide this resistance. In the design of the soil nails, a number of assumptions had to be made, all of which are considered to be conservative. These included :

(i) The required loading is distributed in either a rectangular or triangular distribution.

(ii) The boundary between the active and resistant zones is the slip surface with factor of safety = 1.0. This surface was similar to the Coulomb critical wedge surface.

(iii) All the load is applied to the nail through the 100 mm reinforced sprayed concrete facing and head plates on the nails.

(iv) The resistance provided by the nails follows the form :
Resistance = (c' + k Tan u) x Area
where k is a function of overburden pressure and u is a function of Ø'. Values for these coefficients were based on the measured results reported by Guilloux et al. (1979) and Cartier & Gigan (1983).

The cost of this work is US$65 per m² of slope face.

Tests were required to substantiate design assumptions, and pull-out tests were