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# Design of Bored Pile Walls for Fill Slope Stabilization in Hong Kong

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**Summary** Bored pile walls have been designed to upgrade two potentially unstable fill slopes in Hong Kong. Each pile wall consists of a row of 0.45 m diameter cast-in-situ bored piles, spaced at 0.9 m centres, and a row of staggered 0.45 m diameter drainage piles on the downslope. The bored piles are socketed into bedrock for about 3 m, while the drainage piles have a length of about 5 m. All piles are restrained by a row of soil nails installed at 30° to the horizontal and socketed into bedrock for about 3 m. This paper describes the procedures employed for the analysis and design of these piles.

## 1. INTRODUCTION

Shek O Road traverses the western side of D'Aguilar Peninsula, Hong Kong, and comprises a narrow two-lane road that forms the only vehicular link to a number of small communities. The road, which is located in Shek O County Park, is about 7km long and was built before 1939 by cut and fill methods, resulting in cut slopes on one side and fill slopes on the other side of the road. Failures of some of the slopes have occurred in the past, usually triggered by heavy rainfall. As part of a major program undertaken by the Geotechnical Engineering Office (GEO) of the Hong Kong SAR Government, landslip preventive measures (LPM) are being undertaken to upgrade many slopes along the road.

The authors have designed bored pile walls to upgrade two of the fill slopes, designated by GEO as Slopes 15NE-B/F23 and 15NE-B/F35, respectively. This paper describes the procedures employed in the analysis and design of these piles. Due to space limitations, only the design of Slope 15NE-B/F35 will be covered in this paper.

## 2. SITE CONDITIONS

In order to establish the site conditions at the slope, a series of site investigations, including geological survey and engineering geological mapping, borehole logging, trial pit testing and piezometer measuring, have been undertaken. The site conditions have been established based on the investigation results and are described below.

Slope 15NE-B/F35 comprises a sidelong embankment, approximately 35m in length and up to 16 m in height, with a slope angle ranging between 36 degrees and 43 degrees. The fill materials, which are estimated to be up to 5m in thickness, comprise rockfill and sandy clay fill with construction waste at the surface. The fill is underlain by completely decomposed (Grade V) fine ash vitric tuff of the Ap Lei Chau Formation of Upper Jurassic Age (referred to as CDT). The CDT is underlain by highly to moderately decomposed volcanic tuff (Grade IV to III, referred to as MDT). There is no evidence of past instability on this slope.

The fill material generally comprises a layer of angular fine to coarse gravel and cobbles, mixed with occasional boulders within a matrix of sandy clay. Beneath this granular layer, the fill becomes predominantly matrix supported in a soft to stiff sandy clay.

The above soils have been classified in accordance with Geoguide 3 (1996).

## 3. GEOTECHNICAL MODEL

For the purpose of stability analysis and design of remedial works, a series of cross sections of the slope have been drawn, based on the site investigation data. Figure 1 depicts a typical cross section, showing fill on the top, underlain by CDT which is in turn underlain by MDT. Also shown in Figure 1 are two assumed ground water levels, one in the CDT representing the ambient ground water level, and the other one in the fill representing the highest possible water level which may be expected under heavy rainfall (a 1 in 10 year return period).

A field and laboratory testing program has also been carried out in order to determine appropriate soil parameters for design. The program includes in-situ density testing, field GCO probe testing, and laboratory classification and index testing (moisture

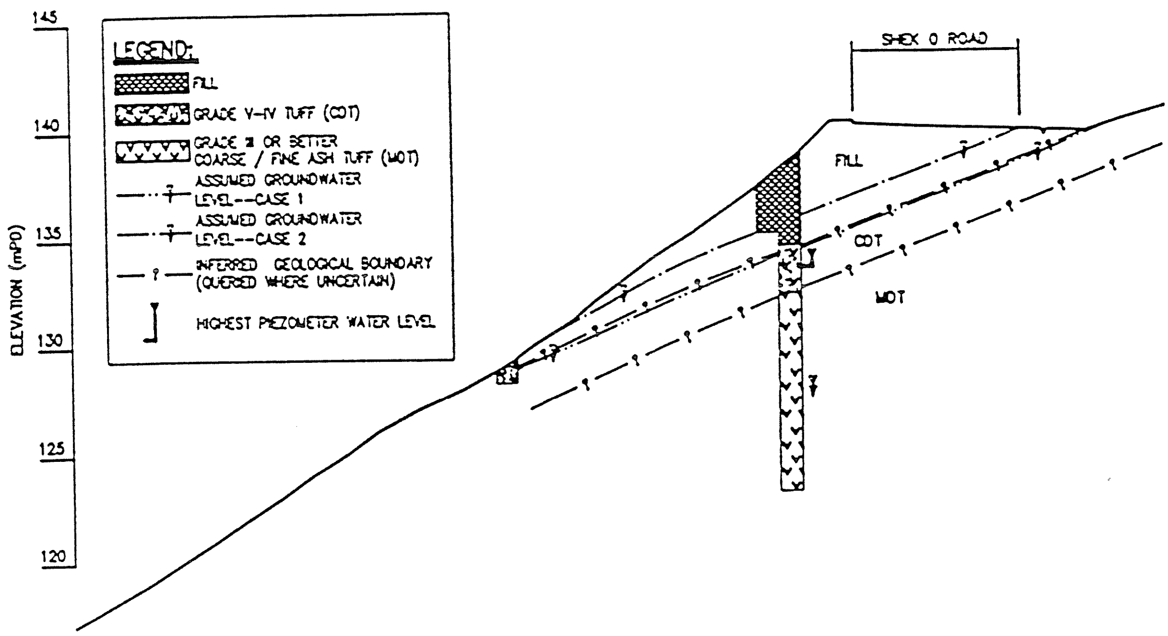


Figure 1 Typical cross-section of existing slope

content, density, sieve analysis and triaxial tests on Mazier samples). Table 1 summarizes the soil parameters adopted based on the testing results and the data recommended in Geoguide 1 (1994). These parameters would appear to be reasonable but may be on the conservative side, as will be shown in a later section based upon the results of stability analysis.

Table 1. Adopted soil parameters.

| Soil Type | Bulk Density ($kN/m^3$) | Effective Cohesion (kPa) | Effective Angle of Friction (degrees) |
|---|---|---|---|
| Fill | 17 | 0 | 35 |
| CDT | 19 | 5 | 35 |
| MDT | 25 | 100 | 40 |

Note that a surcharge of 10 kPa has been considered to represent the traffic loading on the road, as recommended in Geoguide 1 (1994).

## 4. STABILITY ANALYSIS

Stability analysis of the existing slope has been carried out via the proprietary software SLOPE, using Janbu's method and assuming both circular and non-circular slip surfaces. In the analysis, which has been carried out in accordance with the Geotechnical Manual for Slopes (1994), two cases corresponding to different ground water levels have been considered. For Case 1, only the water level in the CDT is considered, representing the existing ground water conditions. For Case 2, both the water levels in the fill and CDT are considered, representing the worst case ground water conditions to be expected. The computed minimum factors of safety for these two cases are summarized in Table 2. As shown, Case 1 gives a factor of safety close to unity, indicating that the slope is below the required standard set out in Geotechnical Manual for Slopes (1994). Given the fact that the slope is still standing, this result may indicate that the adopted geotechnical model is reasonable but may be on the conservative side. On the other hand, Case 2 gives a factor of safety of about 0.75, far less than the required value of 1.2, indicating that the slope may be unstable in the event of heavy rainfall which will cause the ground water to rise. Clearly, LPM upgrading works are required to improve the slope to meet the current standards.

Table 2. Computed factor of safety for existing slope.

| Case | Case 1 | Case 2 |
|---|---|---|
| FoS | 0.98 | 0.75 |

## 5. DESIGN OF PILE WALL

Following the stability studies, a bored pile wall solution was selected to upgrade the slope. As shown in Figure 2, the pile wall consists of one row

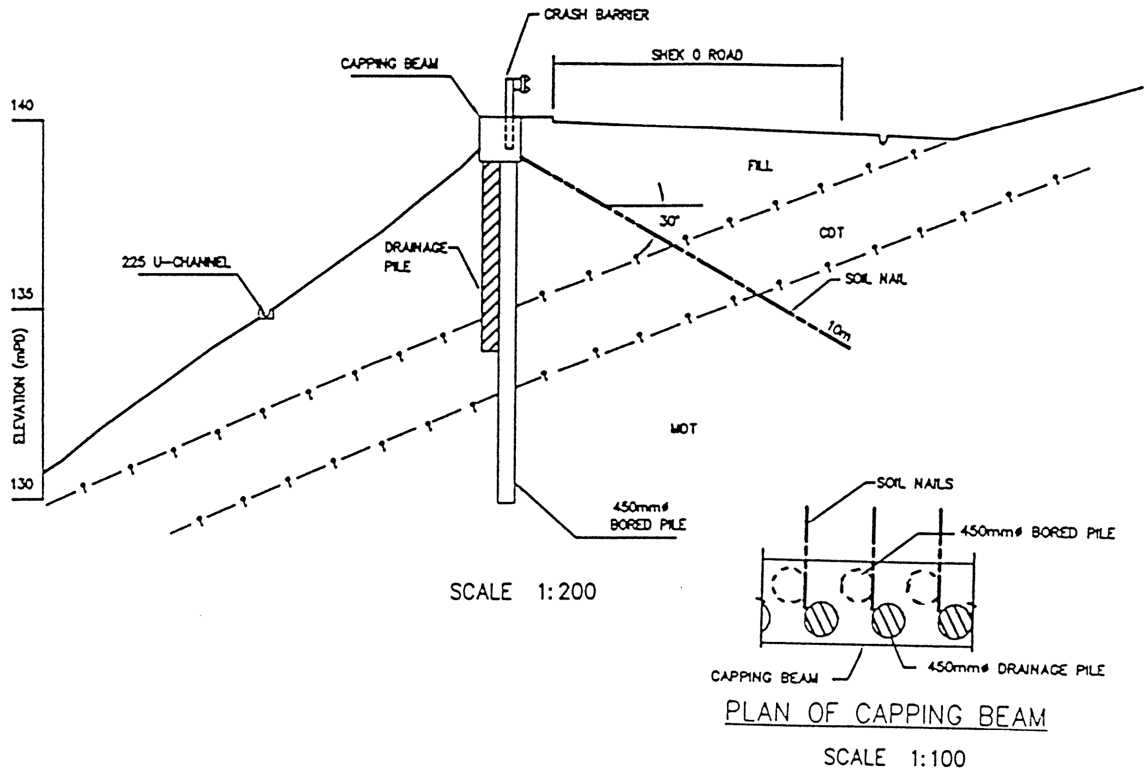


Figure 2 Details of pile wall

of 0.45 m diameter cast-in-situ bored piles, spaced at 0.9 m centres, and a second row of staggered 0.45 m diameter no-fines concrete drainage piles, located on the downslope side. All the piles are connected by a reinforced concrete capping beam, of 1.1 m x 1.2 m in cross-section, and about 35 m in length. The capping beam is restrained by a single row of soil nails, installed at 30° to the horizontal, spaced at 0.9 m centres, and socketed about 3 m into the MDT bedrock. The pile wall is designed to be located at the crest of the slope (or the edge of Shek O Road) to minimize the environmental impact of the upgrading works.

Theoretical analyses have been carried out during the design process for the following purposes:

- to determine an appropriate pile socket length into the MDT bedrock;
- to assess the lateral pile response, in particular, bending moment, shear force and lateral deflection; and
- to assess the stability of the overall slope, incorporating shear forces contributed by the piles.

Note that analysis for soil nails has also been carried out, but is not described in this paper due to space limitations.

## 5.1 Analysis of Piles

In the analysis of the piles, the following two possible mechanisms of pile-soil interaction have been considered:

Mechanism 1: the piles act as a retaining wall and are subjected to forces caused by collapse of the downslope soils which are located above the MDT bedrock; and

Mechanism 2: the piles are subjected to forces caused by ground movements above the MDT bedrock. Note that ground movements will be necessary to generate shear forces from the piles to improve the overall stability of the slope. It is therefore necessary to ensure that the ground movement will be within acceptable limits and that the piles will not fail structurally.

It is clear that Mechanism 1 disregards, while Mechanism 2 considers, the overall stability of the slope. To increase the minimum factor of safety of 0.75 for the existing slope to the required value of 1.2, it is expected that large shear forces will be required from the piles. It is therefore anticipated that the lateral pile response due to Mechanism 2 will be more severe than that due to Mechanism 1. As shown in a later section, this is indeed the case.

The analyses of these two mechanisms are described below.

### 5.1.1 Analysis of Mechanism 1

The analysis of this mechanism has been carried out via the computer program WALLAP. In the analysis, the soils on the downslope side of the pile wall are removed and the lateral pile response is computed for different pile socket lengths, assuming that the pile heads are fully restrained from lateral translation by the soil nails. Figure 3 shows typical pile behaviour computed by WALLAP, for a socket length of 3 m and a total pile length of 9 m.

and lateral deflection tend to decrease, while the maximum shear force tends to increase with increasing socket length. The soil nail force also tends to decrease with increasing socket length.

Based on the above results, a socket length of 1.5 m may be considered sufficient. However, considering that the MDT bedrock is not of good quality and to avoid local rock failure, it was considered prudent to have a greater socket length. In the final design, a socket length of 3 m and a total pile length of 9 m were adopted.

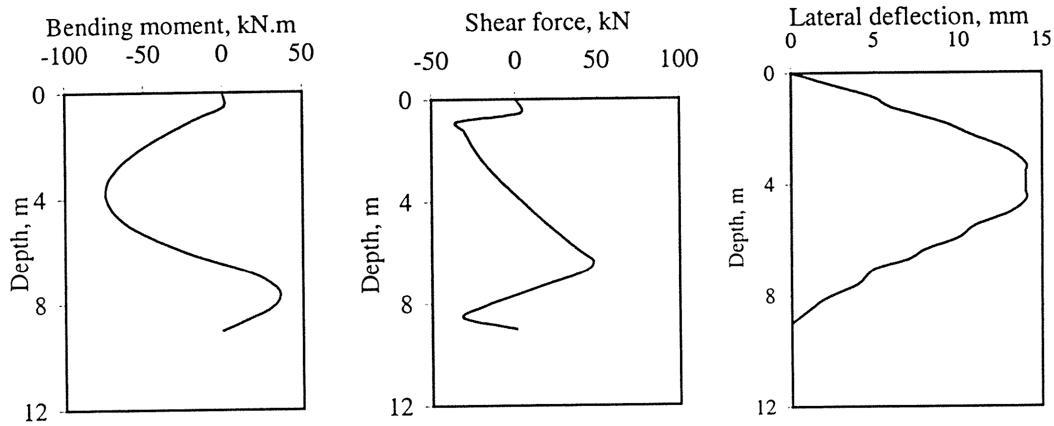


Figure 3 Typical pile behaviour computed by WALLAP

Table 3 Summary of pile response to Mechanism 1

| Pile Length (m) | Socket Length (m) | Maximum Deflection (mm) | Maximum Moment (kN.m) | Maximum Shear (kN) | Nail Force (kN) |
|---|---|---|---|---|---|
| 6.5 | 0.5 | 4702 | 301 | 996 | 1250 |
| 7 | 1 | 21 | 87 | 44 | 48 |
| 7.5 | 1.5 | 17 | 89 | 44 | 48 |
| 8 | 2 | 15 | 86 | 45 | 47 |
| 8.5 | 2.5 | 15 | 80 | 46 | 45 |
| 9 | 3 | 14 | 74 | 48 | 44 |
| 10 | 4 | 13 | 70 | 49 | 42 |

The computed maximum bending moments, shear forces in the pile, lateral pile deflections and soil nail forces corresponding to different pile lengths are summarized in Table 3. It can be seen that the pile responses are very large for a socket length of 0.5 m, but become much smaller when the socket length is greater than 1 m. The maximum bending moment

### 5.1.2 Analysis of Mechanism 2

The analysis of this mechanism has been carried out using a spreadsheet which has been developed based on the design charts presented by Chen & Poulos (1997). These design charts are for estimating the lateral pile response in association with lateral ground movements, assuming elastic behaviour of both the soil and the pile. The spreadsheet requires input of the following information:

- magnitude and distribution of both the soil Young's modulus and the 'free-field' soil movement (the 'free-field' soil movement is that without the presence of the pile);
- length and bending rigidity of the pile; and
- position of the most critical slip surface of the slope without the piles, and the magnitudes of the associated restoring and overturning moments.

The spreadsheet computes the maximum bending moment and shear force in the pile, and also lateral pile movement. Since the magnitude of the ground movement is unknown, the computation requires an iterative trial and error process, which will be further described below. The computed pile bending moment, shear force and lateral deflection are summarized in Table 4. The average Young's modulus has been assumed to be 30MPa in the computation. As shown, this mechanism gives a more severe pile response than does Mechanism 1, and therefore governs the structural design of the pile.

Table 4. Summary of maximum pile response to Mechanism 2.

| Pile Length (m) | Deflection (mm) | Moment (kN.m) | Shear (kN) |
|---|---|---|---|
| 9 | 24 | 126 | 160 |

5.1.3 Assessment of overall slope stability

The general approach employed for the assessment of the overall slope stability, incorporating shear forces contributed by piles, follows closely that described by Poulos (1995). The assessment has also been carried out via the above-mentioned spreadsheet and has involved the following two steps:

1) Estimation of the shear forces required from the piles in order to improve the stability of the existing slope to achieve a target factor of safety of 1.2; and
2) Estimation of the magnitude of the ground movement in order to generate the required shear forces from the piles.

In Step 1, the additional moment required to reach a factor of safety of 1.2 is first calculated as follows:

$$M_{ad} = 1.2M_o - M_r \qquad (1)$$

The required shear force is then calculated as follows:

$$S = \frac{M_{ad}}{Y} \qquad (2)$$

where $M_{ad}$ = additional moment required; $M_o$ = overturning moment; $M_r$ = restoring moment; S = shear force required, and Y = vertical distance from the centre of the slip circle to the horizontal level of the intersection point between the slip circle and the pile.

In Step 2, the magnitude of the soil movement is first assumed and then adjusted until sufficient shear forces are generated from the piles to achieve the target factor of safety. Obviously, engineering judgement needs to be exercised to select a suitable magnitude of soil movement. The design of the piles may need to be altered if acceptable soil movements cannot be obtained.

The results of this analysis are summarized in Table 5. It can be seen that a lateral 'free-field' soil movement of about 30 mm is required to achieve the target factor of safety of 1.2. This soil movement has been considered to be acceptable.

Table 5. Computed factor of safety of improved slope.

| Current Slope | | | Improved Slope | |
|---|---|---|---|---|
| Overturning Moment (kN.m/m) | Restoring Moment (kN.m/m) | FoS | Soil Movement (mm) | FoS |
| 35558 | 2656 | 0.75 | 30 | 1.23 |

## 6. CONCLUSIONS

The procedures employed for the analysis and design of piles to stabilize two fill slopes in Hong Kong have been described. The procedure involves stability analysis of the existing slope, analysis of the lateral behaviour of the piles and stability analysis of the improved slope. For the analysis of piles, two mechanisms of pile-soil interaction are considered. It has been shown that the mechanism associated with ground movements may induce severe pile responses which govern the structural design of the piles. This pile response, as well as the factor of safety of the upgraded slope, is computed via a spreadsheet which incorporates published design charts.

## 7. ACKNOWLEDGMENT

This paper is prepared with the permission of the Director of the Civil Engineering Department, Hong Kong SAR Government. The opinions expressed in the paper are those of the authors.

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