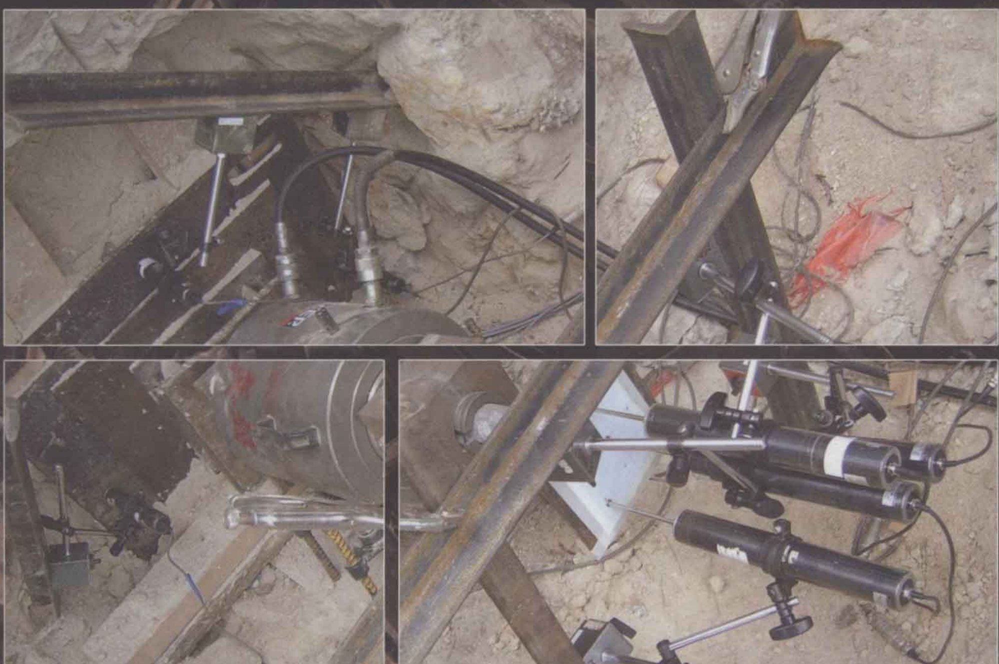
SLOPE STABILITY ANALYSIS AND STABILIZATION

New Methods and Insight



Y.M. Cheng and C.K. Lau

Slope Stability Analysis and Stabilization

New methods and insight

Y.M. Cheng and C.K. Lau

Routledge
Taylor & Francis Group
LONDON AND NEW YORK

First published 2008
by Routledge
2 Park Square, Milton Park, Abingdon, Oxon OX14 4RN

Simultaneously published in the USA and Canada
by Routledge
270 Madison Ave, New York, NY 10016, USA

Routledge is an imprint of the Taylor & Francis Group, an informa business

Typeset in Sabon by
Book Now Ltd, London
Printed and bound in Great Britain by
Antony Rowe, Chippenham, Wilts

British Library Cataloguing in Publication Data
A catalogue record for this book is available from the British Library

Library of Congress Cataloging in Publication Data
Cheng, Y.M.
Slope stability analysis and stabilization: new methods and insight Y.M. Cheng and C.K. Lau.
p. cm.
Includes bibliographical references and index.
1. Slopes (Soil mechanics) 2. Soil stabilization. I. Lau, C.K. II. Title.

TA749.C44 2008
624.1′51363—dc22 2007037813

ISBN10: 0–415–42172–1 (hbk)
ISBN10: 0–203–92795–8 (ebk)

ISBN13: 978–0–415–42172–0 (hbk)
ISBN13: 978–0–203–92795–3 (ebk)

Slope Stability Analysis and Stabilization

Preface

To cope with the rapid development of Hong Kong, many slopes have been made for land development. Natural hillsides have been transformed into residential and commercial areas and used for infrastructural development. Hong Kong's steeply hilly terrain, heavy rain and dense development make it prone to risk from landslides. Hong Kong has a high rainfall, with an annual average of 2300 mm, which falls mostly in the summer months between May and September. The stability of man-made and natural slopes is of major concern to the Government and the public. Hong Kong has a history of tragic landslides. The landslides caused loss of life and a significant amount of property damage. For the 50 years after 1947, more than 470 people died, mostly as a result of failures associated with man-made cut slopes, fill slopes and retaining walls. Even though the risk to the community has been greatly reduced by concerted Government action since 1977, on average about 300 incidents affecting man-made slopes, walls and natural hillsides are reported to the Government every year.

There are various research works associated with the theoretical as well as practical aspects of slope stability in Hong Kong. This book is based on the research work by the authors as well as some of the teaching materials for the postgraduate course at Hong Kong Polytechnic University. The content in this book is new and some readers may find the materials arguable. A major part of the materials in this book is coded into the programs SLOPE 2000 and SLOPE 3D. SLOPE 2000 is now mature and has been used in many countries. The authors welcome any comment on the book or the programs.

The central core of SLOPE 2000 and SLOPE3D was developed mainly by Cheng while many research students helped in various works associated with the research results and the programs. The authors would like to thank Yip C.J., Wei W.B., Sandy Ng., Ling C.W., Li L. and Chen J. for help in preparing parts of the works and the preparation of some of the figures in this book.

1 Introduction

1.1 Introduction

The motive for writing this book is to address a number of issues in the current design and construction of engineered slopes. This book sets out to review critically the current situation and to offer alternative and, in our view, more appropriate approaches to the establishment of a suitable design model, the enhancement of basic theory, the locating of critical failure surfaces and the overcoming of numerical convergence problems. The latest developments in three-dimensional stability analysis and the finite element method will also be covered. This book will provide helpful practical advice in ground investigation, design and implementation on site. The objective is to contribute towards the establishment of best practice in the design and construction of engineered slopes. In particular, this book will consider the fundamental assumptions of both limit equilibrium and finite element methods in assessing the stability of a slope and give guidance in assessing their limitations. Some of the more up-to-date developments in slope stability analysis methods based on the authors' works will also be covered in this book.

Some salient case histories will also be given to illustrate how adverse geological conditions can have serious implications for slope design and how these can be dealt with. The last chapter touches on the implementation of design on site. The emphasis is on how to translate the conceptual design conceived in the design office into physical implementation on site in a holistic way, taking account of the latest developments in construction technology. Because of our background, a lot of cases and construction practices referred to in this book are related to experience gained in Hong Kong, but the engineering principles should nevertheless be applicable to other regions.

1.2 Background

Planet Earth has an undulating surface and landslides occur regularly. Early humans tried to select relatively stable ground for settlement. As populations grow and human life becomes more urbanized, terraces and corridors have to be created to make room for buildings and infrastructures such as quays, canals, railways and roads. Man-made cut and fill slopes have to be formed to facilitate such developments. Attempts have been made to improve upon the rule-of-thumb approach of previous generations by mathematically calculating the stability of such cut and fill slopes. One of the earliest attempts was by the French engineer Alexander Collin (1846). In 1916, using the limit equilibrium method, K.E. Petterson (1955) mathematically back calculated the rotational stability of the Stigberg quay failure in Gothenburg, Sweden. A series of quay failures in Sweden provided the impetus for the Swedes to make one of the earliest attempts at quantifying slope stability using the method of slices and the limit equilibrium method. The systematical method has culminated in the establishment of the Swedish Method (or the Ordinary Method) of Slices (Fellenius, 1927). A number of subsequent refinements to the method were made: Taylor's stability chart (Taylor, 1937); Bishop's Simplified Method of Slices (Bishop, 1955) ensures the moments are in equilibrium; Janbu extended the circular slip to generalized slip surface (Janbu, 1973); Morgenstern and Price (1965) ensured moments and forces are simultaneously in equilibrium; Spencer's (1967) parallel inter-slice forces; and Sarma's (1973) imposed horizontal earthquake approach. These various methods have resulted in the modern Generalized Method of Slices (GMS) (e.g. Low *et al.*, 1998).

In the classical limit equilibrium approach, the user has to a priori define a slip surface before working out the stability. There are different techniques to ensure a critical slip surface can indeed be identified. A detailed discussion will be given in Chapter 3. As expected, the ubiquitous finite element method (Griffiths and Lane, 1999) or the equivalent finite difference method (Cundall and Strack, 1979), namely FLAC, can also be used to evaluate the stability directly using the strength reduction algorithm (Dawson *et al.*, 1999). Zhang (1999) has proposed a rigid finite element method to work out the factor of safety (FOS). The advantage of these methods is that there is no need to assume any inter-slice forces or slip surface, but there are also limitations to these methods which are covered in Chapter 4. On the other hand, other assumptions will be required for the classical limit equilibrium method that will be discussed in Chapter 2.

In the early days when computers were not as widely available, engineers preferred to use the stability charts developed by Taylor (1937), for example. Now that powerful and cheap computers are readily to hand, practitioners invariably use computer software to evaluate the stability in design. However, every numerical method has its own postulations and thus limitations. It is therefore necessary for the practitioner to be fully aware of them, so that the method can be used within its limitations in a real design situation. Apart from the numerical method, it is equally important for the engineer to have an appropriate design model for the design situation.

There is, however, one fundamental question that has been bothering us for a long time and this is that all observed failures are invariably 3D in nature but virtually all calculations for routine design assume the failure is in plane strain. Shear strengths in 3D and 2D (plane strain) are significantly different from each other. For example, typical sand can mobilize in plane strain up to

6° higher in frictional angle when compared with the shear strength in 3D or axi-symmetric strain (Bishop, 1972). It seems we have been conflating the two key issues: using 3D strength data but a 2D model, and thus rendering the existing practice highly dubious. However, the increase in shear strength in plane strain usually far outweighs the inherent higher FOS in a 3D analysis. This is probably the reason why in nature all slopes fail in 3D as it is easier for a slope to fail this way. Now that 3D slope stability analysis has been well established, there is no longer any excuse for practitioners not to do the analysis correctly, or at least take the 3D effect into account.

1.3 Closed-form solutions

For some simple and special cases, closed-form but non-trivial solutions do exist. These are very important results because apart from being academically pleasing, these should form the backbone of our other works presented in this book. Engineers, particularly younger ones, tend to rely heavily on code calculation using a computer and find it increasingly difficult to have a good feel for the engineering problems they face in their work. We hope that by looking at some of the closed-form solutions, we can put into our toolbox some very simple and reliable back-of-the-envelope-type calculations to help us develop a good feel for the stability of a slope and whether the computer code calculation is giving us a sensible answer. We hope that we can offer a little bit of help to engineers in avoiding the current tendency to over-rely on ready-made black box-type solutions and use instead simple but reliable engineering sense in their daily work so that design can proceed with greater understanding and fewer leaps in the dark.

For a circular slip failure with $c \neq 0$ and $\phi = 0$, if we take moment at the centre of rotation, the factor of safety will be obtained easily. This is the classical Swedish method that will be covered in Chapter 2. The factor of safety from the Swedish method should be exactly equal to that from the Bishop method for this case. On the other hand, the Morgenstern–Price method will fail to converge easily for this case while Sarma's method will give a result very close to that from the Swedish method. Apart from the closed-form solutions for the circular slip for $c \neq 0$ and $\phi = 0$ case which should already be very testing for the computer code to handle, the classical bearing capacity and earth pressure problem where closed-form solutions exist may also be used to calibrate and verify a code calculation. A bearing capacity problem can be seen as a slope with a very gentle slope angle but with substantial surcharge loading. The beauty of this classical problem is that it is relatively easy to extend the problem to the 3D or at least/axi-symmetric case where a closed-form solution also exists. For example, for an applied pressure of 5.14 Cu for the 2D case and 5.69 Cu for the axi-symmetric case (Shield, 1955), where Cu is the undrained shear strength of soil, the ultimate bearing capacity will be motivated. The computer code should yield FOS = 1.0 if the surcharge loadings are set to 5.14 Cu and 5.69 Cu, respectively. Likewise, similar bearing capacity solutions also exist for frictional material in both plane strain and axi-symmetric strain (Cox, 1962 or Bolton and Lau, 1993). It is surprising to find that many commercial programs have difficulty in reproducing these classical solutions, and the limit of application of each computer program should be assessed by the engineers.

Similarly, the earth pressure problems, both active and passive, would also be a suitable check for the computer code. Here, the slope has a slope angle of 90°. By applying an active or passive pressure at the vertical face, the computer should yield FOS = 1.0 for both cases, which will be illustrated in Section 3.9. Likewise, the problem can be extended to the 3D, or more precisely axi-symmetric, case for a shaft stability problem (Kwong, 1991).

Our argument is that all codes should be benchmarked and validated through being required to solve the classical problems where 'closed-form' solutions exist for comparison. Hopefully, the comparison will reveal both their respective strengths and limitations so that users can put things into perspective when using the code for design in real life. More on this topic can be found in Chapter 2.

1.4 Engineering judgement

We all agree that engineering judgement is one of the most valuable assets of an engineer because engineering is very much an art as well as a science. In our view, however, the best engineers always use their engineering judgement *sparingly*. To us, engineering judgement is really a euphemism for a leap in the dark. So, in reality, the fewer leaps we make, the more comfortable we will be. We would therefore like to be able to use simple and understandable tools in our toolbox so that we can routinely do some back-of-the-envelope-type calculations to assist us in assessing and evaluating the design situations we are facing so that we can develop a good feel for the problem, thus enabling us to do slope stabilization on a more rational basis.

1.5 Ground model

Before we can set out to check the stability of a slope, we need to find out what it is like and what it consists of. From the topographical survey, or more usually an aerial photograph interpretation and subsequent ground-truthing, we can tell its height, its sloping angle and whether it has berms and is served by a drainage system or not. In addition, we also need to know its history, in terms of its geological past, whether it has suffered failure or distress and whether it has been engineered previously. In a nutshell, we need to build a geological model of the slope that features the key geological formations and characteristics. After some simplification and idealization in the context of the intended purpose of the site, a ground model can then be set up. Following the nomenclature of the Geotechnical Engineering Office in Hong Kong (GEO, 2007), a design model should finally be made, when the design parameters and boundary conditions are also delineated.

1.6 The status quo

A slope, despite being 'properly' designed and implemented, can still become unstable and collapse at an alarming rate. Wong's (2001) study suggests that the probability of an engineered slope failing in terms of major failures (defined as >50 m^3) is only about 50 per cent better than a non-engineered slope. Martin (2000) pointed out that the most important factor with regard to major failures is the adoption of an inadequate geological or hydrogeological model in the design of slopes. In Hong Kong, it is established practice for the Geotechnical Engineering Office to carry out a landslip investigation whenever there is a significant failure or when there is fatality. It is of interest to note that past failure investigations suggest the most usual causes of the failure are some 'unforeseen' adverse ground conditions and geological features in the slope. It is, however, widely believed that such 'unforeseen' adverse geological features, though unforeseen, really should be foreseeable if we set out to identify them at the outset. Typical unforeseen ground conditions are the presence of adverse geological features and adverse groundwater conditions.

(I) Examples of adverse geological features in terms of strength are the following:

1 adverse discontinuities, for example, relict joints;
2 relict instability caused by discontinuities: dilation of discontinuities with secondary infilling of low-frictional materials, that is, soft band, some time in the form of kaolin infill;
3 re-activation of pre-existing (relict) landslide, for example, slickensided joint;
4 faults.

(II) Examples of complex and unfavourable hydrogeological conditions are the following:

1 drainage lines;
2 recharge zones, for example, open discontinuities, dilated relict joints;
3 zones with large difference in hydraulic conductivity resulting in perched groundwater table;
4 a network of soil pipes and sinkholes;
5 damming of the drainage path of groundwater;
6 aquifer, for example, relict discontinuities;
7 aquitard, for example, basalt dyke;
8 tension cracks;
9 local depression;
10 depression of the rockhead;
11 blockage of soil pipes;
12 artesian conditions – Jiao *et al.* (2006) have pointed out that the normally assumed unconfined groundwater condition in Hong Kong is questionable. They have evidence to suggest that it is not

uncommon for a zone near the rockhead to have a significantly higher hydraulic conductivity resulting in artesian conditions;
13 time delay in the rise of the groundwater table;
14 faults.

It is not too difficult to set up a realistic and accurate ground model for design purposes using routine ground investigation techniques, but for the features mentioned above. In other words, in actuality, it is very difficult to identify and quantify the adverse geological conditions listed above. If we want to address the 'So what?' question, the adverse geological conditions may have two types of quite distinct impacts when it comes to slope design. We have to remember that we do not want to be pedantic but we still have a real engineering situation to deal with. The impacts boil down to two types: (1) the presence of zones and narrow bands of weakness and (2) the existence of complex and unfavourable hydrogeological conditions, that is, the transient ground porewater pressure is high and may even be artesian.

Although there is no hard-and-fast rule on how to identify adverse geological conditions, the mapping of the relict joints at the outcrops and the split continuous triple tube core (e.g. Mazier) samples may help to identify the existence of zones and planes of weakness so that these can be properly incorporated into the slope design. The existence of complex and unfavourable hydrogeological conditions may be a lot more difficult to identify as the impact would be more complicated and indirect. Detailed geomorphological mapping may be able to identify most of the surface features, such as drainage lines, open discontinuities, tension cracks, local depression and so on. More subtle features would be recharge zones, soil pipes, aquifer, aquitard, depression of the rockhead and faults. Such features may manifest as an extremely high-perched groundwater table and artesian conditions. It would be ideal to be able to identify all such hydrogeological features so that a proper hydrogeological model can be built up for some very special cases. However, under normal design situations, we would suggest a redundant number of piezometers are installed in the ground instead so that the transient perched groundwater table and artesian groundwater pressure, including any time delay in the rise of the groundwater table, can be measured directly using the compact and robust electronic proprietary groundwater pressure monitoring devices, for example, DIVERs developed by Van Essen. Such devices may cost a lot more than the traditional Halcrow buckets but can potentially provide the designer with the much needed transient groundwater pressure in order that a realistic design event can be built up for the slope design.

While the ground investigation should be planned with the identification of the adverse geological features firmly in mind, one must be aware that engineers have to deal with a large number of slopes and it may not be feasible to screen each and every slope thoroughly. One must accept, no matter what one does, some will inevitably be missed from our design. It is nevertheless still best practice to attempt to identify all potential adverse geological features so that these can be properly dealt with in the slope design.

As an example, a geological model could be a rock at various degrees of weathering resulting in the following geological sequence in a slope, that is, completely decomposed rock (saprolite) overlying moderately to slightly weathered rock. The slope may be mantled by a layer of colluvium. To get this far, the engineer has had to spend a lot of time and resources already. But this is probably still not enough. We know rock mass behaviour is strongly influenced by discontinuities. Likewise, when rock mass decomposes, they would still be heavily influenced by relict joints. An engineer has no choice, but has to be able to build a geological model with all the salient details for his design. It helps a lot if he also has a good understanding of the geological processes and this can assist him in finding the existence of any adverse geological features. Typically, such adverse features are the following: soft bands, internal erosion soil pipes and fault zones and so on, as listed previously. Such features may result in planes of weakness or create a very complicated hydrogeological system. Slopes often fail along such zones of weakness or as a result of the very high water table or even artesian water pressure, if these are not properly dealt with in the design through the installation of relief wells and sub-horizontal drains. With the assistance of a professional engineering geologist if required, the engineer should be able to construct a realistic geological model for his design. A comprehensive treatment of engineering practice in Hong Kong can be found in GEO Publication No. 1/2007 (GEO, 2007). This document may assist the engineer in recognizing when specialist engineering geological expertise should be sought.

1.7 Ground investigation

Ground investigation is defined here in the broadest possible sense as involving desk study, site reconnaissance, exploratory drilling, trenching and trial pitting, in situ testing, detailed examination during construction when the ground is opened up and supplementary investigation during construction planned, supervised and interpreted by a geotechnical specialist appointed at the inception of a project. It should be instilled in the minds of practitioners that a ground investigation does not stop when the ground investigation contract is completed but should be conducted throughout the construction period. In other words, mapping of the excavation during construction should be treated as an integral part of the ground investigation. Greater use of new monitoring techniques like differential Global Positioning Systems (GPS; Yin *et al.*, 2002) to detect ground movements should also be considered. In Hong Kong, ground investigation typically constitutes less than 1 per cent of the total construction costs of foundation projects but is mainly responsible for overruns in time (85 per cent) and budget (30 per cent) (Lau and Lau, 1998). The adage is that one pays for a ground investigation, irrespective of whether one is having one or not! That is, you either pay up front or else at the bitter end when things go wrong. So it makes good commercial sense to invest in a thorough ground investigation at the outset.

The geological model can be established by mapping the outcrops in the vicinity and the sinking of exploratory boreholes, trial pitting and trenching. A pre-existing slip surface of an old landslide where only residual shear strength is mobilized can be identified and mapped through the splitting and logging of a continuous Mazier sample (undisturbed sample) or even the sinking of an exploratory shaft.

In particular, Martin (2000) advocated the need to appraise relict discontinuities in saprolite and the more reliable prediction of a transient rise in the perched groundwater table through the following:

1 more frequent use of shallow standpipe piezometers sited at potential perching horizons;
2 splitting and examining continuous triple-tube drill hole samples, in preference to alternative sampling and standard penetration testing;
3 more extensive and detailed walkover surveys during ground investigation and engineering inspection especially natural terrain beyond the crest of cut slopes. Particular attention should be paid to drainage lines and potential recharge zones.

1.8 Design parameters

The next step would be to assign appropriate design parameters for the geological materials encountered. The key parameters for the geological materials are shear strength, hydraulic conductivity, density, stiffness and in situ stress. Stiffness and in situ stress are probably of less importance compared with the three other parameters. The boundary conditions are also important. The parameters can be obtained by index, triaxial, shear box and other in situ tests.

1.9 Groundwater regime

The groundwater regime would be one of the most important aspects for any slope design. As mentioned before, slope stability is very sensitive to the groundwater regime. Likewise, the groundwater regime is also heavily influenced by the intensity and duration of local rainfall and the drainage provision. Rainfall intensity is usually measured by rain gauges, and the groundwater pressure measured by standpipe piezometers installed in boreholes. Halcrow buckets or proprietary electronic groundwater monitoring devices, for example, DIVER by Van Essen and so on, should be used to monitor the groundwater conditions. The latter devices are essentially miniature pressure transducers (18 mm OD) complete with a datalogger and multi-years battery power supply so that they can be inserted into a standard standpipe piezometer (19 mm ID). They usually measure the total water pressure so that a barometric correction should be made locally to account for the changes in the atmospheric pressure. A typical device can measure the groundwater pressure once every 10 min. for 1 year with a battery lasting for a few years. The device has to be retrieved from the ground and connected to a computer to download the data. The device, for example, DIVER, is housed in a strong and watertight stainless steel housing. As the metallic housing acts as a Faraday cage, the device is hence protected from stray electricity and lightning. More details on such devices can be found at the manufacturer's website (http://www.vanessen.com). One should also be wary of any potential damming of the groundwater flow as a result of underground construction work.

1.10 Design methodology

We have to tackle the problem from both ends: the probability of a design event occurring and the consequence should such a design event occur. Much more engineering input has to be given to cases with a high chance of occurring and a high consequence should such an event occur. For such sensitive cases, the engineer has to be more thorough in his identification of adverse geological features. In other words, he has to follow best practice for such cases.

1.11 Case histories

Engineering is both a science and an art. Engineers cannot afford to defer making design decisions until everything is clarified and understood as they need to make provisional decisions in order that progress can be made on site. It is expected that failures will occur whenever one is pushing further away from the comfort zone. Precedence is extremely important in helping the engineer know where the comfort zone is. Past success is obviously good for morale but, ironically, it is past failures that are equally, if not more, important. Past failures are usually associated with working at the frontier of technology or design based on extrapolating past experience. Therefore studying past mistakes and failures is extremely instructive and valuable. In Hong Kong, the GEO carries out detailed landslide investigations whenever there is a major landslide or landslide with fatality. We have selected some typical studies to illustrate some of the controlling adverse geological features mentioned in Section 1.6.

1.11.1 Case 1

The Shum Wan Road landslide occurred on 13 August 1995. Figure 1.1 shows a simplified geological section through the landslide. There is a thin mantle of colluvium overlying partially weathered fine-ash to coarse-ash crystallized tuff. Joints within the partially weathered tuff were commonly coated with manganese oxide and infilled with white clay of up to about 15 mm thick. An extensive soft yellowish brown clay seam typically 100–350 mm thick formed part of the base of the concave scar. Laboratory tests suggest that the shear strengths of the materials are as follows:

CDT: $c' = 5$ kPa; $\phi' = 38°$
Clay seam: $c' = 8$ kPa; $\phi' = 26°$
Clay seam (slickensided): $c' = 0$; $\phi' = 21°$

One of the principal causes of the failure is the presence of weak layers in the ground, that is, clay seams and clay-infilled joints. A comprehensive report on the landslide can be found in GEO's report (GEO, 1996b).

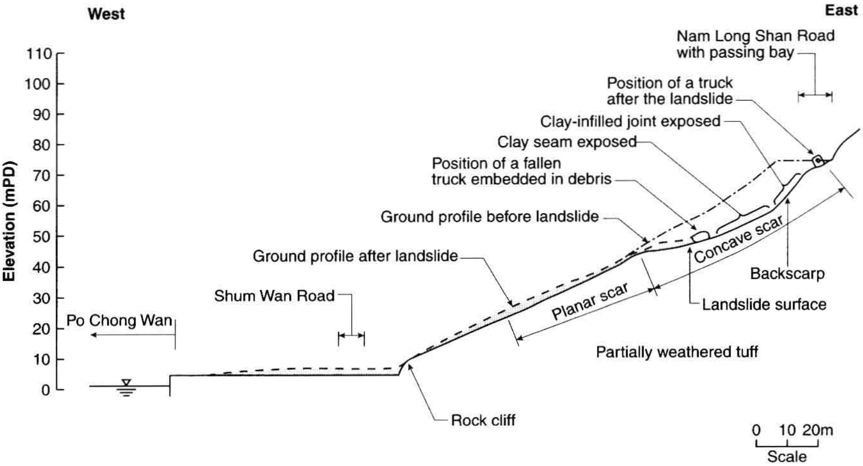


Figure 1.1 The Shum Wan Road landslide occurred on 13 August 1995 in Hong Kong.

Source: Reproduced by kind permission of the Hong Kong Geotechnical Engineering Office from GEO Report (1996b).

1.11.2 Case 2

The Cheung Shan Estate landslip occurred on 16 July 1993. Figure 1.2 shows the cross-section of the failed slope. The ground at the location of the landslip comprised colluvium of about 1 m thick over partially weathered granodiorite. The landslip appears to have taken place entirely within the colluvium. When rainwater percolated the colluvium and reached the less permeable partially weathered granodiorite, a 'perched water table' could have developed and caused the landslip. More details on the failure can be found in the GEO's report (GEO, 1996c).

1.11.3 Case 3

The three sequential landslides at milestone 14 $\frac{1}{2}$ Castle Peak Road occurred twice on 23 July and once on 7 August 1994.

The cross-section of the slope before failure is shown in Figure 1.3. The granite at the site was intruded by sub-vertical basalt dykes of about 800 mm thick. The dykes were exposed within the landslide scar. When completely decomposed, the basalt dykes are rich in clay and silt, and are much less permeable than the partially weathered granite. Hence, the dykes act as barriers to water flow. The groundwater regime was likely to be controlled by a number of decomposed dykes resulting in a damming of the groundwater flow and thus the raising of the groundwater level locally. The

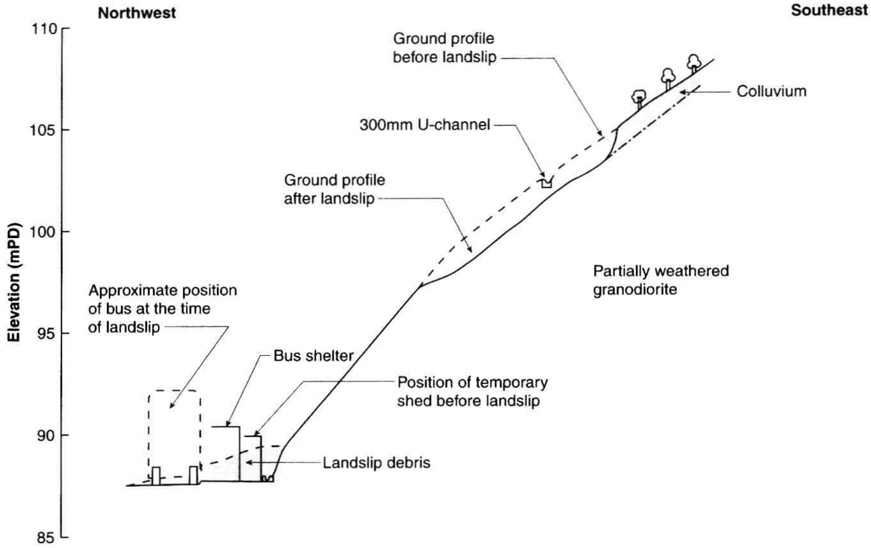


Figure 1.2 The Cheung Shan Estate landslip occurred on 16 July 1993 in Hong Kong.

Source: Reproduced by kind permission of the Hong Kong Geotechnical Engineering Office from GEO Report No. 52 (1996c).

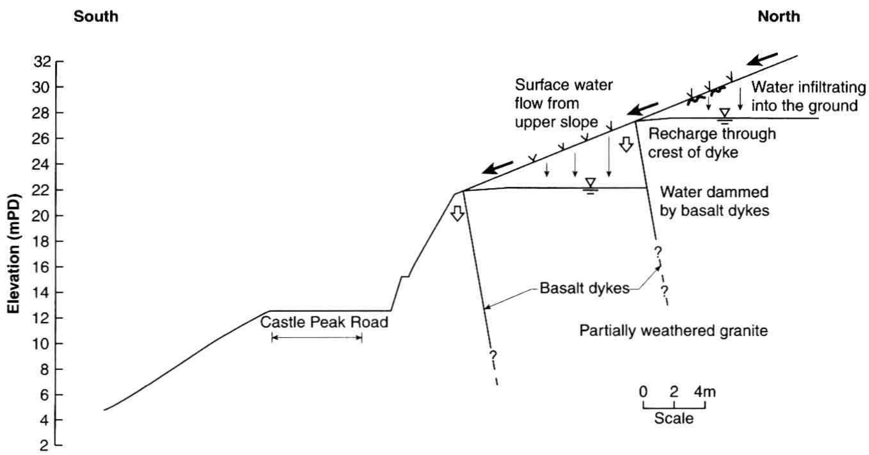


Figure 1.3 The landslide at Castle Peak Road occurred twice on 23 July and once on 7 August 1994 in Hong Kong.

Source: Reproduced by kind permission of the Hong Kong Geotechnical Engineering Office from GEO Report No. 52 (1996c).

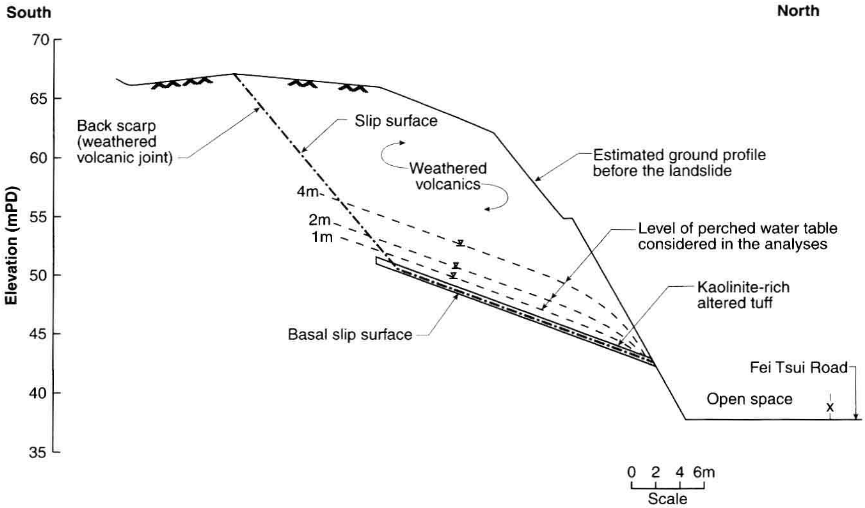


Figure 1.4 The Fei Tsui Road landslide occurred on 13 August 1995 in Hong Kong.
Source: Reproduced by kind permission of the Hong Kong Geotechnical Engineering Office from the GEO Report (GEO, 1996a).

high local groundwater table was the main cause of the failure. More details can be found in the GEO's report (GEO, 1996c).

1.11.4 Case 4

The Fei Tsui Road landslide occurred on 13 August 1995. A cross-section through the landslide area comprises completely-to-slightly decomposed tuff overlain by a layer of fill of up to about 3 m thick as shown in Figure 1.4. A notable feature of the site is a laterally extensive layer of kaolinite-rich altered tuff. The shear strengths are

Altered tuff: $c' = 10$ kPa; $\phi' = 34°$
Altered tuff with kaolinite vein: $c' = 0$; $\phi' = 22\text{–}29°$

The landslide is likely to have been caused by the extensive presence of weak material in the body of the slope triggered by an increase in groundwater pressure following prolonged heavy rainfall. More details can be found in the GEO's report (GEO, 1996a).

1.11.5 Case 5

The landslides at Ching Cheung Road that involved a sequence of three successively larger progressive failures occurred on 7 July 1997 (500 m^3), 17