Slope Stability Analysis and Stabilization

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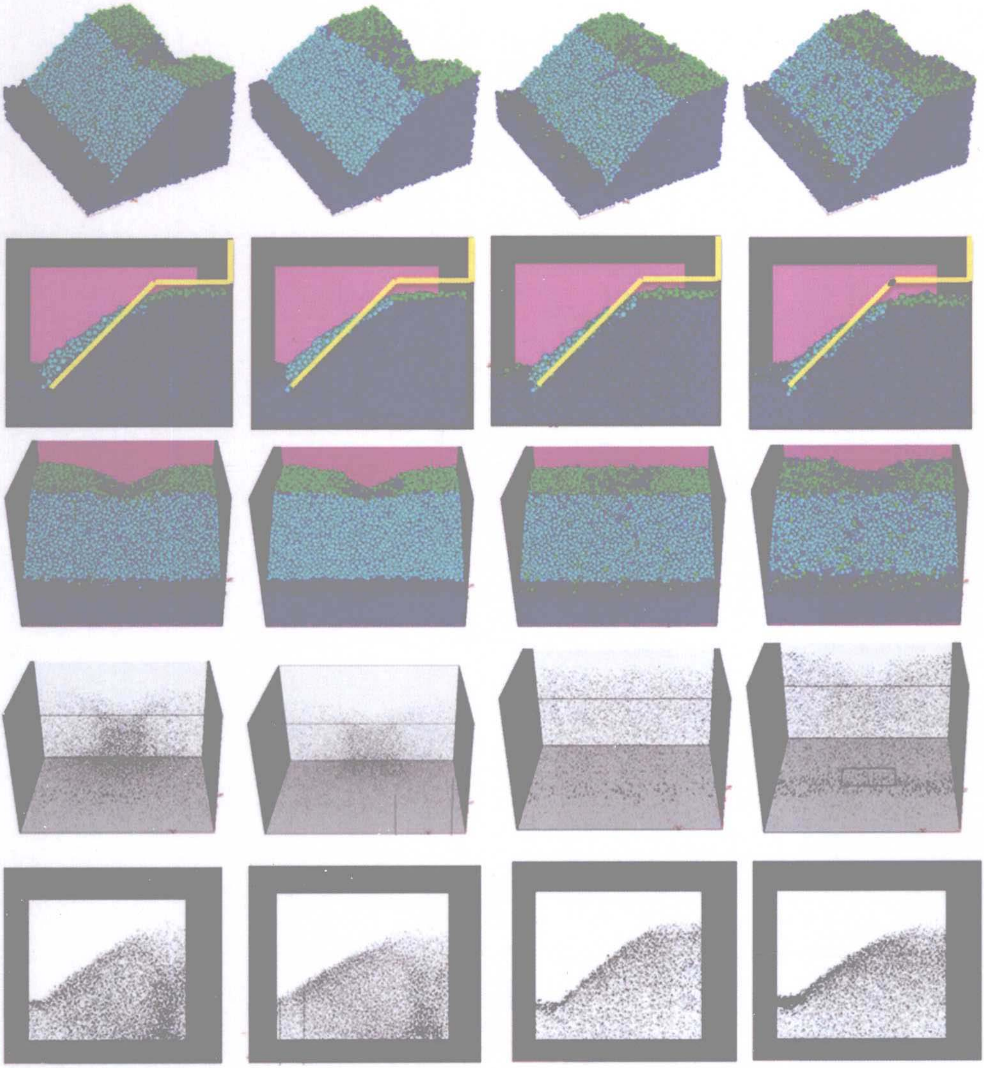
New Methods and Insight

Second Edition

Y.M. Cheng and C.K. Lau



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CRC Press
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Boca Raton London New York

CRC Press is an imprint of the
Taylor & Francis Group, an **informa** business
A SPON BOOK

CRC Press
Taylor & Francis Group
6000 Broken Sound Parkway NW, Suite 300
Boca Raton, FL 33487-2742

Printed on acid-free paper
Version Date: 20140319

International Standard Book Number-13: 978-1-4665-8283-5 (Hardback)

Library of Congress Cataloging-in-Publication Data

Cheng, Y. M.
Slope stability analysis and stabilization : new methods and insight / Y. M. Cheng, C. K. Lau. -- Second edition.
pages cm
Includes bibliographical references and index.
ISBN 978-1-4665-8283-5 (hardback)
1. Slopes (Soil mechanics) 2. Soil stabilization. I. Lau, C. K. II. Title.

TA749.C44 2014
624.1'51363--dc23 2014002523

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Preface

After the publication of the first edition, we received several e-mails regarding some of the background material of slope stability analysis discussed in the book. We also received e-mails from research students enquiring about the procedure of numerical implementation in some of the stability analysis methods.

In the second edition, the more advanced concepts and case studies involved in slope stability analysis have been covered in greater detail based on our research work. In particular, we have added more examples and illustrations on the distinct element of slope, the relation between limit equilibrium and plasticity theory, the fundamental relation between slope stability analysis and bearing capacity problem, as well as three-dimensional slope stability under patch load conditions. The results of some of the laboratory tests that we have conducted are also included for illustration. Most importantly, we have added a chapter detailing the procedures involved in performing limit equilibrium analysis. This should help engineers carry out calculations or develop simple programs to carry out the analysis. Another new chapter deals with the design and construction practice in Hong Kong. This will be useful for those who are interested in slope stabilization works in Hong Kong.

The central core of SLOPE 2000 and SLOPE 3D has been developed mainly by Cheng, while many research students have helped in various works associated with the research results and the programs. We would like to thank Yip C.J., Wei W.B., Li N., Ling C.W., Li L. and Chen J. for helping with the preparation of the book.

Contents

Chapter 1

Introduction

1.1 OVERVIEW

The motive of writing this book is to address a number of issues of the current design and construction of engineered slopes. The book sets out to critically review the current situation and offer alternative and, in our view, more appropriate approaches for the establishment of a suitable design model, enhancement of the basic theory, locating the critical failure surfaces and overcoming numerical convergence problems. The latest developments in three-dimensional (3D) stability analysis and finite-element method will also be covered. It 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, the book will consider the fundamental assumptions of both limit equilibrium and finite-element methods in assessing the stability of a slope, and provide guidance in assessing their limitations. Some of the more up-to-date developments in slope stability analysis methods based on the author's works will also be covered in this book.

Some salient case histories to illustrate how adverse geological conditions can have serious implication on slope design and how these problems could be dealt with will also be given. Chapter 6 touches on the implementation of design on site. Emphasis is on how to translate the conceptual design conceived in the design office into physical implementation on site in a holistic way taking into account the latest developments in construction technology. Because of our background, a lot of cases and construction practices referred to in the book are related to the experience gained in Hong Kong, but the engineering principles should nevertheless be applicable to other regions as well.

1.2 BACKGROUND

Planet earth has an undulating surface and landslides occur regularly. Early humans would try to select relatively stable ground for settlement. As population grows and human life becomes more urbanized, there is a necessity to create terraces and corridors to make room for buildings and infrastructures like quays, canals, railways and roads. Man-made cut and fill slopes would have to be formed to facilitate such developments. Attempts have been made to improve the then rules of thumb approach by mathematically calculating the stability of such cut and fill slopes. One of the earliest attempts was by a French engineer, Alexander Collin (Collin, 1846). In 1916, K.E. Petterson (1955) used the limit equilibrium method to back-calculate mathematically the rotational stability of the Stigberg quay failure in Gothenburg, Sweden. A series of quay failures in Sweden motivated Swedes to make one of the earliest attempts to quantify slope stability by 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), which ensures that the moments are in equilibrium; Janbu extending the circular slip to a generalized slip surface (Janbu, 1973); Morgenstern and Price (1965) ensuring that moments and forces equilibrium are achieved simultaneously; Spencer's parallel inter-slice forces (1967); and Sarma's imposed horizontal earthquake approach (1973). These methods have resulted in the Modern Generalized Method of Slices (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 presented 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, fast Lagrangian analysis of continua (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 also are limitations of these methods, which are covered in Chapter 4. Conversely, other assumptions will be required for the classical limit equilibrium method, which will be discussed in Chapter 2.

In the early days when computer was not available as widely, engineers may have preferred to use the stability charts developed by Taylor (1937), for example. Now that powerful and affordable computers are readily available, practitioners invariably use computer software to evaluate the stability in a design. However, every numerical method has its own postulations and thus limitations. It is therefore necessary for practitioners 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 engineers to have an appropriate design model for the design situation.

There is, however, one fundamental issue that has been bothering us for a long time: all observed failures are invariably 3D in nature, but virtually all calculations for routine design always 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 axisymmetric 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 slopes to fail this way. Now that 3D slope stability analysis has become well established, practitioners would no longer have any excuses to not be able to perform 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 of the engineering problems they are facing 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 of the stability of a slope and whether the computer code calculation is giving us a sensible answer. We hope we can offer a little bit of help in avoiding the current phenomenon where engineers tend to over-rely on a readymade black box–type solution and more on simple but reliable engineering sense in their daily work so that design can proceed with more understanding and less arbitrary leap into the dark.

For a circular slip failure with $c \neq 0$ and $\phi = 0$, if we take moment at the centre of rotation, the FOS will be obtained easily, which is the classical Swedish method, which will be covered in Chapter 2. The FOS from the Swedish method should be exactly equal to that from Bishop's method for this case. On the other hand, the Morgenstern–Price method will fail to converge easily for this case, whereas the method of Sarma will give a result very close to that of the Swedish method. Apart from the closed-form solutions for circular slip for the $c \neq 0$ 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 also 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 the axisymmetric 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 axisymmetric case (Shield, 1955), where Cu is the undrained shear strength of the 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 axisymmetric strain (Cox, 1962; Bolton and Lau, 1993). It is surprising to find that many commercial programs have difficulties in reproducing these classical solutions, and the limit of application of each computer program should be assessed by the engineers.

Similarly, earth pressure problems, both active and passive, would also be a suitable check for the computer code. Here, the slope has an 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 3D, or more precisely the axisymmetric case, for a shaft stability problem (Kwong, 1991).

Our argument is that all codes should be benchmarked and validated by subjecting them to solving the classical problems where *closed-form* solutions exist for comparison. Hopefully, the comparison would reveal both their 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 it is 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 into the dark. So in reality, the less we leap, the more comfortable we would 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 that would help us to assess and evaluate the design situations we are facing so that we can develop a good feel of the problem, which will enable 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 a topographical survey, or more usually an aerial photograph interpretation and subsequent ground-truthing, we can determine its height, slope angle, and whether it has berms and is served by a drainage system or not. In addition, we also need to know its history, both in terms of its geological past, whether it has suffered failure or distress, and whether it has been engineered before. In a nutshell, we need to build a geological model of the slope featuring 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. When the design parameters and boundary conditions are delineated, a design model as defined by the Geotechnical Engineering Office in Hong Kong (GEO, 2007) should be established.

1.6 STATUS QUO

Despite being *properly* designed and implemented, slopes would still become unstable and collapse at an alarming rate. Wong's (2001) study suggests that the probability of a major failure (defined as >50 m^3) of an engineered slope is only about 50% better than that of 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 an established practice for the Geotechnical Engineering Office to carry out landslip investigation whenever there is a significant failure or fatality. It is of interest to note that past failure investigations also suggest that the most usual causes of failure are some *unforeseen* adverse ground conditions and geological features in the slope. It is, however, widely believed that such adverse geological features, though *unforeseen*, should really 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.

1. Examples of adverse geological features in terms of strength are as follows:
   a. Adverse discontinuities, for example, relict joints
   b. Relict instability caused by discontinuities: dilation of discontinuities, with secondary infilling of low-friction materials, that is, soft bands, sometimes in the form of kaolin infill
   c. Re-activation of a pre-existing (relict) landslide, for example, a slickensided joint
   d. Faults

2. Examples of complex and unfavourable hydrogeological conditions are as follows:
    a. Drainage lines.
    b. Recharge zones, for example, open discontinuities, dilated relict joints.
    c. Zones with a large difference in hydraulic conductivity resulting in a perched groundwater table.
    d. A network of soil pipes and sinkholes.
    e. Damming of the drainage path of groundwater.
    f. Aquifer, for example, relict discontinuities.
    g. Aquitard, for example, basalt dyke.
    h. Tension cracks.
    i. Local depression.
    j. Depression of the rockhead.
    k. Blockage of soil pipes.
    l. 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.
    m. Time delay in the rise of the groundwater table.
    n. Faults.

It is not too difficult to set up a realistic and accurate ground model for design purpose using routine ground investigation techniques but for the features mentioned earlier. In other words, it is actually very difficult to identify and quantify the highlighted adverse geological conditions. 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 we do not want to be pedantic, but we still have a real engineering situation to deal with. The impacts would boil down to two types: (1) the presence of narrow bands of weakness and (2) the existence of complex and unfavourable hydrogeological conditions, that is, the transient ground pore water pressure may be high and may even be artesian.

While there is no hard-and-fast rule on how to identify the adverse geological conditions, the mapping of relict joints at the outcrops and the split continuous triple tube core (e.g. Mazier) samples may help identify the existence of zones and planes of weakness so that these can be incorporated properly in 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 like drainage lines, open discontinuities, tension cracks, local depression, etc. More subtle