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Full Length Article

Overhanging rock slope by design: An integrated approach using rock mass strength characterisation, large-scale numerical modelling and limit equilibrium methods

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ABSTRACT

Overhanging rock slopes (steeper than 90°) are typically avoided in rock engineering design, particularly where the scale of the slope exceeds the scale of fracturing present in the rock mass. This paper highlights an integrated approach of designing overhanging rock slopes where the relative dimensions of the slope exceed the scale of fracturing and the rock mass failure needs to be considered rather than kinematic release of individual blocks. The key to the method is a simplified limit equilibrium (LE) tool that was used for the support design and analysis of a multi-faceted overhanging rock slope. The overhanging slopes required complex geometries with constantly changing orientations. The overhanging rock varied in height from 30 m to 66 m. Geomechanical modelling combined with discrete fracture network (DFN) representation of the rock mass was used to validate the rock mass strength assumptions and the failure mechanism assumed in the LE model. The advantage of the simplified LE method is that buttress and support design iterations (along with sensitivity analysis of design parameters) can be completed for various cross-sections along the proposed overhanging rock sections in an efficient manner, compared to the more time-intensive, sophisticated methods that were used for the initial validation. The method described presents the development of this design tool and assumptions made for a specific overhanging rock slope design. Other locations will have different geological conditions that can control the potential behaviour of rock slopes, however, the approach presented can be applied as a general guiding design principle for overhanging rock cut slope.

1. Introduction

1.1. Methodology

The purpose of this paper is to present an efficient, yet geologically representative engineering approach to design a large complex overhanging rock slope. The slope in question is at a scale that exceeds the spacing and persistence of joints present in the rock mass, such that intact rock bridging contributes to the stability of the slope. For this problem, it is not the failure of a single discrete block that governs the stability of the overhang, but the complex interaction of stresses that develop due to the eccentric loading from the cantilevered rock section combined with stress rotations between non-persistent joints in the rock mass. Numerical modelling suggests that the overhangs may fail with a combined shear-tensile failure mechanism. That is to say, the shear failure tends to dominate in the lower one third of the overhangs as rock bridge strength is exceeded, while the tensile failure dominates the upper two thirds of the slope as a result of rotation and overturning. The limit equilibrium (LE) model presented herein captures this unique failure mechanism.

This simplified LE method developed allows for multiple support design iterations to be completed in a time-efficient manner for the proposed overhanging rock slopes. The steps used to develop this engineering tool include:

(1) Detailed field mapping of existing slopes combined with a detailed geo-referenced survey (X, Y, Z), which allows for the

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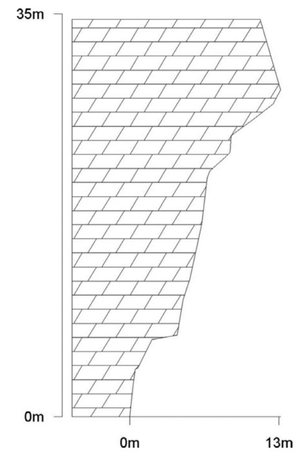
(a)



(b)

Fig. 1. (a) Example of a naturally occurring overhanging rock bluff with basal collapse and an associated blocky basalt overhang; and (b) Critical section through overhanging cliff (after Tsesarsky and Hatzor, 2009).

exact locations of persistent fractures to be spatially represented.

(2) Statistical characterisation of mapping data also allows for geotechnical domains to be delineated for different zones in the rock mass.
(3) Development of a discrete fracture network (DFN) specific to each mapped domain. Since field mapping includes detailed survey of fracture locations, the DFN model is generated with both deterministic fractures (at exact locations of persistent or stability controlling fractures), and stochastically generated fractures to represent joints in three dimensions that are not captured deterministically (calibrated to mapping statistics).
(4) Coupling of the DFN with large-scale explicit numerical strength test simulations to understand possible shear and tensile fracturing processes and to derive synthetic rock mass strength properties that accommodate scale effects.
(5) Full-scale numerical simulations of the slopes to verify the failure mechanism as excavation proceeds.
(6) Development of an LE method to calculate factors of safety (with and without support) that provides a simplification of more complex failure mechanisms and allows for the rapid quantification of rock support as a tool to optimise slope geometries where possible.

It is contended that, provided that the failure mechanism is understood sufficiently, the presented approach allows the designer to make simplified assumptions about the potential shape and location of the failure surface and to apply either shear or tensile strength estimates to discrete parts of the assumed failure surface area. The results of the current study have importance with respect to analysing engineering problems with slopes where confining stresses are very low or absent and failure is not induced due to the development of high compressive stresses or just kinematic block release, but a combination of shear and tensile failures that dictates rock mass behaviour.

This study comes from the work undertaken for infrastructure development where overhanging slopes with complex geometries and varying slope orientations were required and support levels needed to be minimised. The minimised support design criteria required optimising the volume of overhanging rock, which was achieved by leaving a rock buttress at the base of the overhanging slopes. Multiple design iterations were required to achieve this optimisation. While the location of these slopes is not shared, it is contended that, provided that the potential failure mechanism is understood for overhanging slopes, there is (in theory) no limitation on the size of overhanging slopes that can be developed. Rather, the size and length of the anchorage required and the difficulty of construction and associated costs become the limiting factors in the design.

1.2. Current state of practice

Naturally occurring overhanging rock slopes can be found on river banks or sea cliffs (scour), at geological contacts where rock mass strength contrasts are present, at the base of flexural toppling slopes and in the areas where key blocks have kinematic potential to failure from the base of a slope (as shown in Fig. 1). In general, large rock overhangs are inherently unstable, partly due to weathering processes and partly due to fatigue and stress concentrations that develop locally across rock bridges that can cause brittle failure. On rare occasions, overhanging rock slopes are required by design, but typically at a scale where the tensile strength of individual blocks becomes the limiting factor rather than at a scale where the slopes are significantly larger than the scale of fracturing. The case history presented here describes the latter case.

The challenge with modelling of rock overhangs that are three-dimensional (3D) in nature is that most of the existing commercially available modelling codes do not allow for breakage or brittle fracture of rock to be simulated between non-persistent fractures over large distances. Several commercially available codes used to analyse rock overhangs are discussed in this section, and the limitations of each code are presented for completeness.

Tsesarsky et al. (2005) and Tsesarsky and Hatzor (2009) used the discontinuous deformation analysis (DDA) method to assess the stability of a 34 m high overhanging cliff with tightly spaced bedding planes and three sets of vertical joints. The authors illustrated using the numerical method that the stability of the eccentrically loaded overhanging cliff is determined by the depth of face parallel tension cracks, which were observed behind the crest of the cliff, and the critical length of these cracks was assessed using the DDA method. They also illustrated that the main advantage of the DDA method over the available LE methods is that kinematics can be accounted for in the analysis and the failure mode is not assumed prior to running the model. At the time, the DDA method

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