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Effects of spatial variation in cohesion over the concrete-rock interface on dam sliding stability

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ABSTRACT

The limit equilibrium method (LEM) is widely used for sliding stability evaluation of concrete gravity dams. Failure is then commonly assumed to occur along the entire sliding surface simultaneously. However, the brittle behaviour of bonded concrete-rock contacts, in combination with the varying stress over the interface, implies that the failure of bonded dam-foundation interfaces occurs progressively. In addition, the spatial variation in cohesion may introduce weak spots where failure can be initiated. Nonetheless, the combined effect of brittle failure and spatial variation in cohesion on the overall shear strength of the interface has not been studied previously. In this paper, numerical analyses are used to investigate the effect of brittle failure in combination with spatial variation in cohesion that is taken into account by random fields with different correlation lengths. The study concludes that a possible existence of weak spots along the interface has to be considered since it significantly reduces the overall shear strength of the interface, and implications for doing so are discussed.

1. Introduction

In Sweden and many other regions/countries of the world, there are an increasing number of older dams in need of safety re-assessments to evaluate their compliance with modern safety regulations. One important failure mode considered is sliding. Several techniques for safety assessment with regard to sliding are available, with the traditional limit equilibrium method (LEM) being the most popular, accepted and widely used approach. In LEM, the dam is modelled as a rigid body allowed to slide along its base, and the safety is evaluated by the ratio between the driving forces and the resisting forces. In general, the available shear strength of the dam-foundation contact is expressed by the Mohr–Coulomb failure criterion, and the resisting forces are determined by integrating the normal stresses over the potential sliding plane. The technique is based on the assumption that the shear strength is simultaneously mobilised along the entire sliding surface at the time of failure (Ruggeri et al., 2004). In order for that simplification to be valid, the interface must behave as an elastic-perfectly plastic material. However, tests conducted on concrete-rock cores taken from dams show that an elastic-brittle response is to be expected for cores with bonded interfaces (Rocha, 1964; Link, 1969; Lo et al., 1990; EPRI, 1992). The elastic-brittle response, in combination with the varying stress conditions along the interface, means that a progressive mechanism of failure would be a better description of the interface behaviour. In addition, it is likely that parts with high and low values of cohesion could be expected to appear in clusters with a certain correlation distance. According to Westberg Wilde and Johansson (2013), the reason for this is that the bond strength depends on factors such as the results from cleaning the rock surface prior to the concrete casting, the local rock mass quality and the location of leakage and other degradation processes. Possible spatial variation in cohesion over the interface may introduce weak areas where the failure process can be initiated and contributes further to the uncertainties regarding the failure behaviour of the bonded contact. Since progressive failure can lead to the failure of interfaces which appear to be stable when only the mean value of the peak strength is considered, the simplified Mohr–Coulomb model commonly used in LEM will result in an overestimation of the interface shear resistance and thus dam safety.

Numerical methods, which allow for the incorporation of the deformability of the materials and different sliding and opening criteria for the interfaces, have been implemented for analyses of concrete dams. The constitutive models usually adopted for potential sliding surfaces in the dam body at the dam-foundation interface and in the foundation are of the Mohr–Coulomb type, ruled by the friction angle and cohesion (Foster and Jones, 1994;

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Dawson et al., 1998; Liu et al., 2003; Zhou et al., 2008; Chen and Du, 2011; Jia et al., 2011; Sun et al., 2011). Linear elastic and nonlinear fracture mechanics models, governed by fracture toughness and fracture energy, are also used (Ebeling et al., 1997; Kishen, 2005; Saouma, 2006). When fracture mechanics models are used, the semi-brittle behaviour of bonded interfaces is taken into account. However, when Mohr–Coulomb type constitutive models are used for the concrete-rock interface, the semi-brittle behaviour of bonded contacts is rarely considered. Dawson et al. (1998) and Jia et al. (2011) constituted the rare exceptions. Yet in none of the cases published has the spatial variation in cohesion been included in the analysis, which means that its impact on the assessed dam safety is still uncertain. There is thus a need to generically study and quantitatively determine the impact of brittle failure in combination with a possible variation in the bond strength on the assessed dam safety.

In this paper, the influence of a brittle material model in combination with spatial variation in cohesion on the assessed sliding stability of a hypothetical dam monolith, with dimensions similar to those of a typical Swedish concrete gravity dam, is investigated using numerical experiments. The reason put forward for using numerical methods is that physical observations of dam failure due to sliding are almost impossible to be realized for economic, technical and environmental reasons. The numerical experiments are performed using the three-dimensional (3D) finite difference programme FLAC^{3D} (Itasca Consulting Group Inc, 2011). Mohr–Coulomb constitutive material models are used for the interface to provide a straightforward comparison with the factor of safety (*FS*) obtained using LEM. The distribution of cohesion is estimated from results of direct tensile tests on concrete-rock cores extracted from a concrete dam located in Sweden, and the spatial variation over the sliding surface is taken into account through the use of random fields. Comparisons are conducted between the numerical results obtained using ductile and brittle material models for the interface and values of *FS* determined analytically by LEM to analyse the discrepancy between the different models. Finally, there is a discussion on the results and how possible spatial variation in bond strength could be incorporated into re-assessments of existing concrete dams.

2. Factor of safety

In this section, the definition of *FS* used in this paper is presented. The limit equilibrium and strength reduction techniques, used to determine *FS* in the analytical and numerical analyses, respectively, are also described. A detailed review of the two methods for use in rock engineering and a comparison between them are provided in Ureel and Momayez (2014). Previous studies, within the fields of geotechnical and rock engineering, where comparisons between *FS* obtained using LEM and the strength reduction technique are conducted, can be found in Matsui and San (1992), Cala and Flisiak (2001), Cheng et al. (2007), Chen et al. (2014), etc.

2.1. Limit equilibrium method (LEM)

According to LEM, a structure is stable with regard to sliding when, for any potential sliding surface, the resultant shear stress required for equilibrium (τ) is lower than the available shear strength (τ_F). *FS* is thus determined as the ratio between these quantities, i.e. $FS = \tau_F/\tau$.

Defining the maximum shear strength that can be mobilised using the Mohr–Coulomb failure criterion, the shear strength available locally for each point of the concrete-rock interface is given by

$$\tau_F = c + \sigma_N \tan\phi \tag{1}$$

where σ_N is the effective normal stress; and c and ϕ are the cohesion and internal friction angle of the bonded interface, respectively. In order to estimate the shear force of the total interface (T_F), it is assumed that the ultimate capacity is simultaneously achieved along the entire sliding surface and the normal stress is integrated over the potential sliding plane. This gives

$$T_F = cA + N' \tan\phi \tag{2}$$

where N' is the resultant of the effective forces normal to the assumed sliding plane including the effects of uplift, and A is the area of the sliding surface. By also integrating the resultant shear stresses over the sliding plane, the global *FS* against sliding at the concrete-rock interface, FS_{LEM}, can be determined according to

$$FS_{LEM} = \frac{cA + N' \tan\phi}{H} \tag{3}$$

where H is the resultant of the horizontal loads acting on the structure.

2.2. Strength reduction technique

There are two common techniques to determine failure due to sliding using numerical techniques: reducing the shear strength of the interface until failure occurs, or increasing the applied loads until failure occurs. Since increasing the load could lead to other types of failures, e.g. overturning, which are not considered in this paper, the shear strength reduction technique (SRT) is applied here. SRT is a popular technique when numerical modelling for stability analyses of rock and soil slopes is used and was employed as early as 1975 by Zienkiewicz et al. (1975). In the field of dam engineering, the technique has been applied by Alonso et al. (1996), Liu et al. (2003), Zhou et al. (2008), Chen and Du (2011), and Jia et al. (2011) among others. In this study, the technique is implemented using the 3D explicit finite difference code FLAC^{3D}. FLAC^{3D} and the two-dimensional (2D) code FLAC have been widely used for numerical stability evaluations of rock slopes (Sjöberg, 1999a,b; Latha and Garaga, 2010) and soil slopes (Zettler et al., 1999; Dawson and Roth, 1999), and have also been applied in stability analyses of concrete dams by Kieffer and Goodman (1999), Bu (2001), Léger and Javanmardi (2006), Gustafsson et al. (2010), and Yang et al. (2012) among others.

The analysis is initially carried out with the actual load and resistance parameters of the structure studied to establish the initial stress conditions. After this, the resistance parameters of the interface, ϕ and c, are simultaneously and progressively decreased by a factor *RF* according to Eqs. (4) and (5) while keeping the applied load conditions unchanged. The computation continues until failure occurs.

$$\phi_{RF} = \arctan\left(\frac{\tan\phi}{RF}\right) \tag{4}$$

$$c_{RF} = c/RF \tag{5}$$

The interface strength just prior to failure can be considered as the shear strength required for equilibrium. It then follows, from the definition of *FS* in Section 2.1, that the numerical FS_{SRT} can be determined according to