Computers and Geotechnics xxx (xxxx) xxx–xxx

Contents lists available at ScienceDirect

Computers and Geotechnics

journal homepage: www.elsevier.com/locate/compgeo

Research Paper

A constitutive model for soil-rockfill mixtures

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ARTICLE INFO

Keywords:
Soil-rockfill mixture
Constitutive modelling
Subloading surface plasticity
Genetic algorithms

ABSTRACT

Soil-rockfill mixtures (SRMs) are economical and environmentally friendly materials. Due to the absence in the literature of constitutive models specifically developed and tested for SRMs, a new constitutive model, the Subloading Surface Rockfill Model, is presented. This model allows the occurrence of plastic strains inside the yield surface, inducing a smooth elastic/plastic transition. The results of experimental tests for different coarse fractions (CF) of several SRMs performed on samples from Odelouca Dam are compared with those obtained with this model. The model was able to reproduce reasonably well the response of SRMs considering the intrinsic variability of the tested specimens.

1. Introduction

In the last decades, there has been a considerable increase in the use of soil-rockfill mixtures (SRMs) in embankments of high dams and other structures. It is an environmentally friendly material as it includes the excavation products from the spillways, cut-off trenches, outlet works and other appurtenant structures that would have to go to deposit and are instead reused. It is also an economic material since a significant part of it comes from near to the construction site, thereby reducing the costs of transportation.

In this work, the following definition of soil-rockfill mixtures is adopted [1]: (i) fraction retained on ¾″ (19 mm) sieve between 30% and 70%; (ii) fraction passing No. 200 (0.074 mm) sieve between 12% and 40%; (iii) and the maximum particle dimension (D_{max}) less than 2/3 of the embankment layer thickness after compaction and not larger than 0.40 m.

In this study, the coarse fraction, CF, is considered the fraction of the total sample retained on the ¾″ (19 mm) sieve and the finer fraction, FF, is consider the fraction of the total sample passing the same sieve (CF + FF = 1).

A literature review reveals the absence of constitutive models specifically developed for SRMs. Certainly this material presents a behaviour reflecting its two constituents – soil and rockfill. As such, the constitutive model that best reproduces its behaviour will have to take into account some important aspects of both materials. The main objective of this research was to developed a model, which should be as simple as possible but still capable of reproducing well the response of different SRMs subjected to undrained triaxial tests isotropically consolidated to multiple effective stresses and drained triaxial K_0 tests.

In the past SRMs were treated as a “weathered rockfill” or “transition material” and the constitutive models used for this type of material were those used for rockfill. The first models used in rockfill dams were linear elastic. According to some authors ([2–5]) this type of models presented good fitting to the observed results, which is not surprising considering that almost all these analyses were back analysis based on the monitoring results. However, soils present strain irreversibility even at relatively low stress states and the linear elastic models can only give a first approximation to the real mechanical behaviour.

The nonlinear elastic models were also very popular in the simulation of the mechanical behaviour of rockfill. The main objective of these models was to be able to fit the strain–stress curves of the tests. The bilinear model, the K-G model ([6] and [7]), the EC-K0 ([8]) model and the hyperbolic model are all examples of nonlinear elastic models.

In 1963, Konder [9] presented the hyperbolic model when he analysed strain–stress curves of soils subjected to conventional triaxial shear tests. He noted that these curves could be approximated by a hyperbolic function with a horizontal asymptote. Starting from this work, several authors proposed other hyperbolic models ([10–15], among others). Of these, Duncan and Chang model [10] has been the most used.

Examples of numerical analysis of three dams – Borde Seco Dam, Las Cuevas Dam and Alvito Dam, with the hyperbolic model can be found in [16].

A comparison between two different nonlinear elastic constitutive analytical models was presented by [17] for the predictions of the end of construction performance of a central core rockfill dam – Beliche Dam. The models used in these analyses were the K-G model and a modified version of the hyperbolic model of [10]. The hyperbolic model

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http://dx.doi.org/10.1016/j.compgeo.2017.09.009
Received 20 December 2016; Received in revised form 26 July 2017; Accepted 15 September 2017
0266-352X/

Please cite this article as: Brito, A., Computers and Geotechnics (2017), http://dx.doi.org/10.1016/j.compgeo.2017.09.009

gave a less stiff response than the K-G model.

The failure of soils is not normally abrupt with a sudden reduction in stiffness. In fact, in most soils, large plastic deformations occur without complete loss of strength. It is even possible to stabilize failure in soil structures by removing the loading ([18]). Over the past 50 years, critical state models have been used to analyse and explain the behaviour of several materials ([19–26], among others).

The critical state model, known as Cam Clay, was initially developed for soils, in the 50s and 60s by researchers at the University of Cambridge ([19,27,20,28]). This model assumes that if the soil is subjected to an increasing shear strain, it will reach a critical state. A critical state model was also used to model Beliche Dam with a marked improvement over the non-linear elastic models [29].

However, the classic critical state models are hardening elastoplastic models and as such have some limitations. This paper presents a non-classic elastoplastic model, specifically developed for SRMs, that is capable of representing their main aspects of behaviour based on a large number of tests performed at LNEC on samples from Odelouca Dam, a large dam built with these materials. In the following section the model is described in detail. The model has also been used with considerable success in the numerical modelling of the construction of Odelouca Dam, which is described elsewhere [30].

In this analysis, the explicit finite difference program FLAC ([31]) was used. This is a two-dimensional program for geotechnical applications that allows the implementation of constitutive laws by the user.

In this work the usual soil mechanics convention that compressive stresses and strains are positive is adopted. Stresses are effective unless stated otherwise.

2. Subloading surface rockfill model

The Subloading Surface Concept ([32–34]) is a generalization of the conventional elastoplastic models that extends the elastoplasticity theory in such a way that the interior of the yield surface is not a purely elastic domain anymore, instead plastic strains are induced by the stress or strain rate inside the yield surface. So, the conventional yield surface is renamed as the normal yield surface. This concept was developed more deeply in [35] for rate dependent cyclic anisotropic structured behaviour.

In this formulation, differently from the original one by Hashiguchi [33], there is no restriction on the form of the yield surface. The yield function gradient is not normalized and the flow rule is non-associated.

The Subloading Surface Rockfill Model (SSRM) is an extension of the Modified Cam Clay Model (MCCM) with the Subloading Surface Concept, tensile strength, non-associated flow rule and a curved critical state line. Another difference is a non-circular deviatoric cross section, which implies dependence on the invariant θ. Geometrically, the yield surface is translated by ξp_c (constant that defines the effective tensile strength) in the negative direction of the hydrostatic axis. Despite, in principle, tensile strength not being present in SRMs, a very small value is useful to avoid some numerical problems such as singularity at the origin of stress space when computing the value of the scaling factor R and also for materials that present some tensile strength such as soils with structure.

A "continuous plasticity" formulation such as the subloading surface one was adopted partly because simpler hardening elastoplastic models, such as the Cam Clay model, were incapable of reproducing the observed inversion in the direction of the undrained triaxial effective stress paths. Complexities such as cyclic behaviour, rate dependency and anisotropy were also not considered as the model calibration for these aspects of behaviour was not evident from the tests performed. Several possible features of the model were investigated such as keeping an associated flow rule while changing the shape of the yield surface, hardening due to plastic shear strains and a volumetric strain dependent isotropic compression constant, λ^*, to represent particle breaking (clastic behaviour). As these particular features did not contribute to verifiable improvements in the model they were considered unnecessary complications and were not used in the final model.

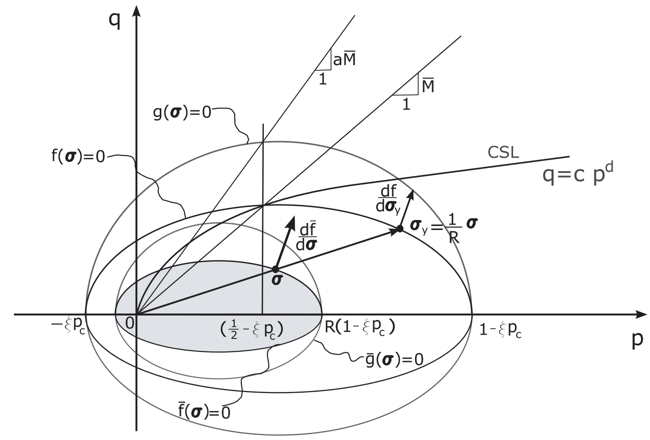

Fig. 1. SSRM yield subloading, plastic potential surfaces and critical state line.

The decreasing gradient of the critical state line is relevant for this type of material because the best fit for a straight CSL gives rise to a significant cohesion value. For the CSL to contain the origin of stress space, it follows that the greatest rate of change in the gradient occurs for small values of the mean effective stress making a curved CSL necessary. Non associativity was needed because the associative version of the model was incapable of reproducing simultaneously conventional triaxial compression and K_0 tests.

The yield surface used by the model, describes an ellipse in (p,q) space (as shown in Fig. 1), similar to the MCCM, and is represented by the following equation:

$$f(\boldsymbol{\sigma}) = \left(\frac{q}{M(\theta,p_c)}\right)^2 + \bar{p}\,(\bar{p} - p_c) = 0 \quad (1)$$

where $\bar{p} = p + \xi p_c$ and the invariants p, q and θ are defined in Appendix A.

The model requires at most ten material constants: λ^*, κ^*, ν, c_R, a, c, d, k, ξ and p_e. The constants λ^* and κ^* are determined so as to fit, respectively, to the slope of the normal compression line and the line of elastic unloading/reloading obtained under isotropic stress conditions in a bi-logarithmic representation ($\ln v - \ln p$), unlike the parameters λ, κ of the Cam Clay Model that are obtained in a semi-logarithmic representation ($v - \ln p$). The advantage of this approach is that the bulk modulus is independent of the specific volume, v, and is given by $K = \frac{p+p_e}{\kappa^*}$ while in the MCCM, the bulk modulus $K = \frac{vp}{\kappa}$ increases for looser states of the soil which is not in agreement with observed soil behaviour. The elastic law is isotropic nonlinear hypoelastic with the shear modulus given by $G = \frac{3(1-2\nu)}{2(1+\nu)}K$. The Poisson's ratio, ν, is constant. The material constant c_R is determined to adjust the evolution of stiffness with strain in the transition from elastic to elastoplastic behaviour. The constant ξ defines the effective tensile strength. p_e is a constant defined to make sure that the bulk modulus (K) is not zero even when the mean stress p is zero. The constant a defines the non-associativity degree with the associated model being recovered for $a = 1$. The constants c and d define the curved critical state in triaxial compression as

$$q = cp^d. \quad (2)$$

When $d = 1$, $c = M$ and $\xi = 0$ the straight critical state line is recovered. In general $d < 1$ as the critical state tangent friction angle decreases with the increase of p. The MCCM with the constants λ^* and κ^*, instead of the usual λ and κ, is obtained for a very large value of c_R, $a = 1$, $d = 1$, $\xi = 0$, $p_e = 0$ and $k = 1$. The constant k is given by