Implications of soil variability for geo-computations

Michael A Hicks, Christakis Onisiphorou, Kristinah Samy & William A Spencer

School of Mechanical, Aerospace and Civil Engineering
The University of Manchester
M60 1QD
michael.hicks@manchester.ac.uk

ABSTRACT

The importance of soil heterogeneity in geo-computations is highlighted for three slope stability applications. The effect of heterogeneity on material behaviour is demonstrated, the implications for design are reviewed, and the case for 3D modelling is illustrated. The need for high performance computing in larger simulations is also noted.

1. INTRODUCTION

There is now a growing awareness that the spatial variability (i.e. heterogeneity) of geomaterials should be properly accounted for in geotechnical and geo-environmental computations. This is recognised by new codes of practice, including Eurocode 7, and it has prompted *Géotechnique* to organise a Symposium in Print in 2005 on "Risk and Variability in Geotechnical Engineering" [2, 3]. This paper summarises the basic principals of stochastic analysis, and reviews three numerical examples illustrating the practical and computational implications of spatial variability.

2. STOCHASTIC MODELLING OF SOIL HETEROGENEITY

In the paper, random field theory is used for modelling material variability, while finite elements are used for computing geo-structural response. This approach is based on defining material properties by their statistics: first, there are the mean, μ, and standard deviation, σ, which define the probability density function and combine to give the coefficient of variation, V; and second, there are the vertical and horizontal scales of fluctuation, θ_v and θ_h, which define the distances over which there is significant correlation of material property values. The statistics are used to generate random fields of property values, and these are then used in conventional finite element simulations. This takes place within a Monte Carlo framework: that is, for a given set of material statistics, multiple realisations are carried out, leading to probabilistic descriptions of geo-performance.

3. NUMERICAL ILLUSTRATIONS

3.1 EFFECT ON MATERIAL BEHAVIOUR

Hicks and Onisiphorou [5] used a sophisticated double-hardening soil model to investigate the failure of a 26 m high underwater sandfill berm [4]. This case history involved the statistical interpretation, in terms of state parameter [1], of 71 cone penetration tests, as well as calibration of the soil model (also in terms of state parameter) against 74 triaxial tests. The berm was then analysed by (a) generating random fields of state parameter, based on the state parameter statistics, (b) back-figuring the material property fields, using the state-parameter-dependent calibration, and (c) using finite elements to compute geo-structural response (assuming undrained gravitational loading).

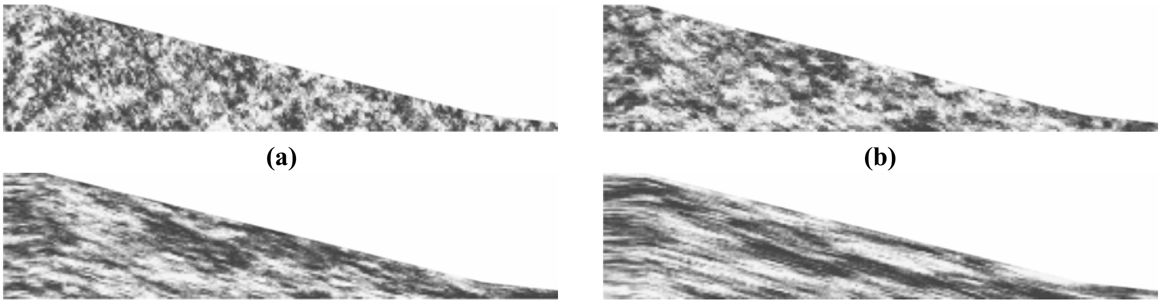
(a) (b)
(c) (d)

Figure 1. Typical random fields of state parameter: (a) $\xi = 1$; (b) $\xi = 2$; (c) $\xi = 4$; (d) $\xi = 8$ (Hicks and Onisiphorou [5]).

Figure 1 shows typical random fields of state parameter for the berm, for different degrees of anisotropy of the heterogeneity, $\xi = \theta_h / \theta_v$. In this figure, the lighter zones correspond to lower densities (and hence weaker materials), and vice versa for darker zones. Figures 2(a) and 2(b) show typical deformation mechanisms for a predominantly dilative fill, for isotropic and anisotropic spatial variability respectively, and for the same μ, σ and θ_v (i.e. only θ_h is different). For the isotropic case (Figure 2(a)), tensile pore pressure changes in the denser parts of the fill lead to local increases in strength, and these zones tend to hold the structure together. (This is despite localised zones of failed material, as indicated by the higher (i.e. lighter) shear strain invariant contours.) The result is a stable slope, and a global load–settlement response that is similar to the deterministic solution based on the mean property values. In contrast, for the more realistic anisotropic case (Figure 2(b)), the deterministic load–settlement response (based on the mean) represents an upper bound solution, with the stability of the slope for some realisations being dominated by the weakest materials. In this instance, global failure of the slope is possible, due to liquefaction propagating along semi-continuous loose zones. For mildly dilating fills, this can result in a diffuse failure mechanism (Figure 2(c)), closely resembling that for a deterministic analysis based on material properties for a very loose soil (Figure 2(d)).

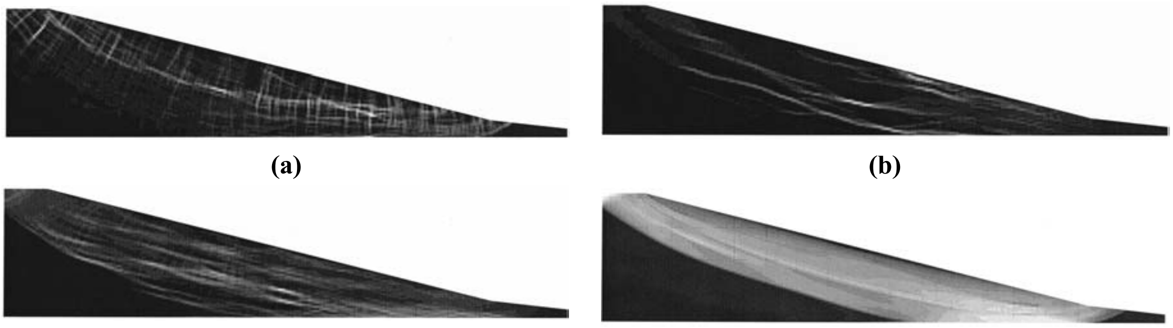
(a) (b)
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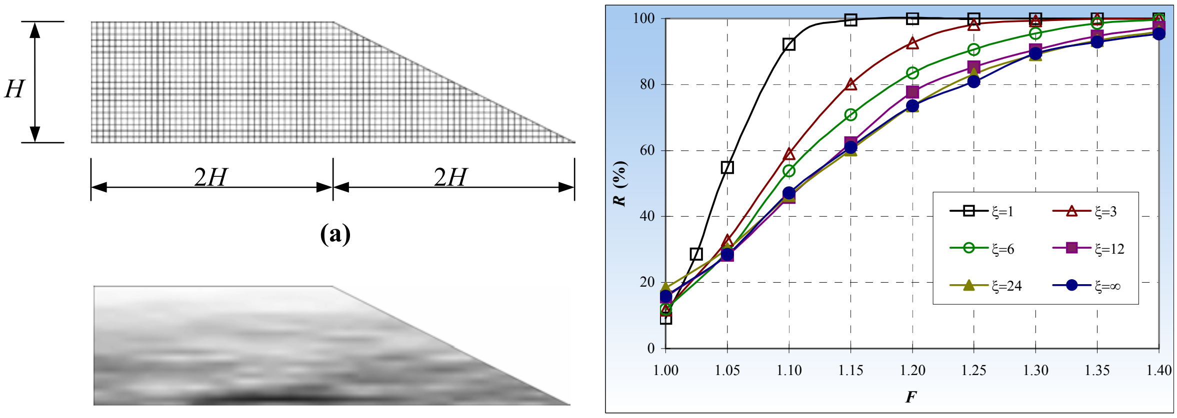
Figure 2. Influence of sand state on deformation mechanism, as indicated by contours of shear strain invariant: (a) dilative fill with $\xi = 1$; (b) dilative fill with $\xi = 8$; (c) mildly dilative fill with $\xi = 8$; (d) very loose fill (deterministic solution) (Hicks and Onisiphorou [5]).

Hicks and Onisiphorou [5] demonstrated that the undrained behaviour of a sand at the macro-scale was significantly different to the behaviour of the same sand at the laboratory scale (for the same mean property values). In particular, they concluded that it is possible for a predominantly dilative fill to liquefy, due to the presence of semi-continuous loose zones arising from deposition-induced anisotropy.

3.2 IMPLICATIONS FOR DESIGN

Figures 3(a) and 3(b) show the finite element mesh and a typical random field used in a simpler slope stability application: that is, for a cutting in an undrained clay, characterised by a spatially varying undrained shear strength, c_u. (In Figure 3(b), the darker zones represent higher values of c_u, and the mean and standard deviation of c_u are proportional to depth with $V = \sigma / \mu$ remaining constant.) Hicks and Samy [6–10] demonstrated that, for a given set of statistics (μ, σ, θ_v and θ_h), there are an infinite number of possible random fields (that are similar in appearance). As each field leads to a different solution, for example when used in a finite element analysis, enough realisations need to be carried out to ensure convergence of the output statistics.



(a)

(b)

(c)

Figure 3. Gravity loading of an undrained cutting characterised by a spatially varying c_u: (a) finite element mesh and problem geometry; (b) typical random field of c_u; (c) influence of ξ on reliability against global factor of safety (Hicks and Samy [8]).

Figure 3(c) shows some results for the problem illustrated in Figure 3(b) [8]. That is, the reliability (or probability of failure not occurring), R, has been expressed as a function of the global factor of safety against failure, F. Each curve has been produced by carrying out multiple realisations of structural performance, and is based on the same μ, σ and θ_v: hence, the only difference between the curves is the value adopted for the horizontal scale of fluctuation, as defined by ξ. For given values of ξ and F, the reliability represents the percentage of realisations in which the slope remained stable under gravitational loading.

The results show that, for a given mean strength and target reliability, the required factor of safety is highly influenced by the degree and nature of the spatial variability [6–10]. Moreover, Hicks and Samy [9] demonstrated that it is also dependent on the geometry of the slope itself. The results highlight that a global factor of safety, on its own, tells us nothing about the probability of failure, and that a "one factor of safety fits all" approach can be seriously flawed. Hicks and Samy [8, 10] demonstrated how stochastic analysis may be used to derive reliability-based characteristic values and factors of safety in line with the requirements of Eurocode 7.

3.3 3D MODELLING OF SLOPE STABILITY

The need for 3D modelling is demonstrated in Figure 4. This shows a long slope, of constant cross-section, and with a constant mean undrained shear strength distribution along its length. As in the previous examples, it has been subjected to gravitational loading. Figure 4 shows the deformed meshes and displacement contours for two realisations in which slope failure occurred: note that the only difference between the realisations is the distribution of strong and weak zones (relative to the mean strength). It is seen that slope failures can be shallow or deep, and that multiple failures may occur. However, all failures are three dimensional, even though the slope is 2D from a deterministic point of view (that is, based on its geometry and mean strength distribution). Hence, for slope stability computations, stochastic analysis leads to the probability of failure (i.e. risk). For 3D simulations, it also leads to the predicted volumes of failed material (including their probability of occurrence).

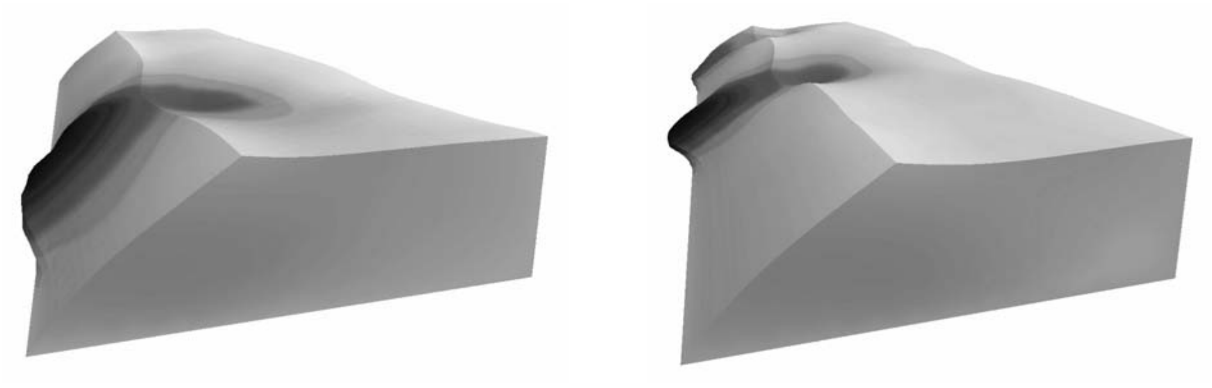


Figure 4. Deformed meshes and displacement contours for 3D failures in a 2D slope (two realisations).

4. CONCLUDING COMMENTS

The variability of geomaterials (and geo-based contaminants) can have a significant influence on geotechnical and geo-environmental problems. Stochastic analysis enables realistic modelling of such problems by (a) simulating the heterogeneity, and (b) quantifying its effects within a probabilistic (risk-based) framework. For smaller problems, analyses can be run efficiently on a PC. However, the need for very fine meshes and multiple realisations makes larger (e.g. 3D) problems inaccessible for many users at present. There is therefore a need for high performance computing (and advanced visualisation techniques) in such cases.

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