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Two and three-dimensional numerical analysis of the progressive failure that occurred in an excavation-induced landslide

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ABSTRACT

A finite element approach, in which an elasto-viscoplastic strain-softening model is incorporated, is used to analyse a landslide that occurred after deep excavations had been carried out for the construction of several buildings at the toe of the slope. The soils involved in the landslide were characterized by a pronounced strain-softening behaviour with the shear strength that abruptly reduces as plastic strain increases. As is well-known, a progressive failure may be triggered in these soils by excavation and it can cause the collapse of the slope after some time. The numerical approach used in the present study allows this phenomenon to be properly simulated. Results from both two-dimensional (2D) and three-dimensional (3D) analyses of the landslide are performed. The results from these analyses confirm the occurrence of a progressive failure within the slope. However, the 2D analysis does not completely account for the failure process that occurred, because of the 3D nature of this process that propagated in both upward and lateral directions. A more realistic simulation of the slope failure is achieved by performing a 3D analysis.

1. Introduction

Slope stability analyses are routinely performed under the assumption of plane strain conditions. Even when two-dimensional (2D) conditions are not appropriate (for example in cases where the geometry of the slope and slip surface varies significantly in the third dimension), a three-dimensional (3D) analysis is rarely carried out because it is generally complex and time-consuming. However, all slopes have a finite size and hence a 3D stability analysis should in principle provide more realistic results than a 2D analysis. In particular, the 3D analysis is recommended when the effects of a complex geometry, boundary conditions and spatial variations of soil properties can significantly influence the slope stability analysis.

A great number of 3D methods based on the limit equilibrium approach have been developed (Hovland, 1977; Chen and Chameau, 1982; Azzouz and Baligh, 1983; Leshchinsky and Baker, 1986; Hungr, 1987; Gens et al., 1988; Huang et al., 2002; Bromhead and Martin, 2004; Jiang and Yamagami, 2004). As discussed by Duncan (1996), a common drawback of these methods is that, since they are extrapolated from the classical 2D method of slices, they are conditioned by several approximate assumptions. Recently, Zhou and Cheng (2013) developed a more comprehensive limit equilibrium that can also be used to search automatically for a critical slip surface under 3D conditions. The elasto-plastic finite element method is a powerful alternative to the above-mentioned solutions for performing a slope stability analysis under 3D conditions (Griffiths and Marquez, 2007). In some circumstances, however, even this method may be affected by numerical shortcomings. This is the case of slopes that consist of brittle soils. In these circumstances, in fact, the numerical solution achieved both under 2D and 3D conditions, can be either affected by a lack of convergence or provide results that are strongly influenced by the mesh adopted (Pietruszczak and Mroz, 1981; Leroy and Ortiz, 1989; Brinkgreve, 1994; Pastor et al., 2002; Troncone, 2005; Conte et al., 2010). To overcome these drawbacks, the use of suitable constitutive models was proposed, such as the viscoplastic model (Loret and Prevost, 1990; Oka et al., 1995; Wang et al., 1997), Cosserat model (de Borst, 1991; Tang, 2008) or numerical approaches based on the gradient or non-local theories (Bažant and Chang, 1984; Vardoulakis and Sulem, 1995; di Prisco and Imposimato, 2003; Aifantis et al., 1999; Conte et al., 2013; Murianni et al., 2013).

In this study, an elasto-viscoplastic model is used to perform both 2D and 3D finite element analyses of the Senise landslide (Figure 1), which involved soils with a pronounced strain-softening behaviour. Following Bjerrum (1967), a progressive failure can occur in these soils when an excavation is performed at the toe of the slope. Specifically, some portions of the slope first fail with shear strain that is generally located in a zone of limited thickness (shear zone). As the strain increases within this zone, soil strength reduces from peak towards residual. Owing to a consequent redistribution of stresses, the shear zone propagates in the slope. Thus, a failure surface progressively develops, resulting in the collapse of the slope. The occurrence of a progressive failure at Senise was first assumed by Viggiani and Di Maio (1991) on the basis of some analyses performed using a 2D limit equilibrium method and

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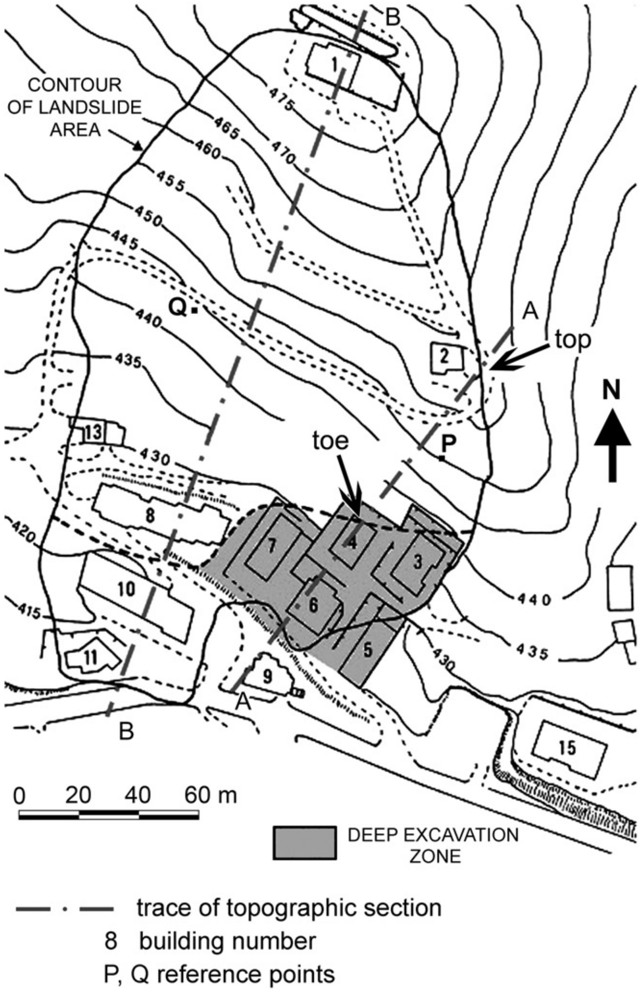

Fig. 1. Plan of the landslide with an indication of the buildings involved and the trace of the cross-sections A–A and B–B.
Adapted from Del Prete and Hutchinson, 1988.

the simplified model by Christian and Whitman (1969). However, the limit equilibrium methods are in principle unsuitable for dealing with such a failure process, and the model by Christian and Whitman (1969) can be only used for a preliminary analysis of this process. More recently, Troncone (2005) performed a 2D finite element analysis of the Senise landslide using an elasto-viscoplastic constitutive model. The study presented by Troncone (2005) corroborates the hypothesis that a progressive failure occurred at Senise owing to the deep excavations carried out at the toe of the slope. Nevertheless, the assumption of 2D conditions is not fully satisfactory for the case study under consideration, owing to the fact that these excavations caused significant changes in the slope geometry. For this reason, both 2D and 3D analyses of the Senise landslide have been performed in the present study, which can be considered an extension of that conducted by Troncone (2005). These analyses have demonstrated the 3D nature of the failure process that occurred at Senise.

2. The Senise landslide

On 26th July 1986, a landslide of large dimensions occurred in Senise which is a village located about 70 km from Potenza, in southern Italy. Owing to this disastrous event, eight people died and several buildings were destroyed or severely damaged. On 6th September 1986, a reactivation of the landslide caused further movements of the failed soil mass. The Senise landslide was studied by several authors who provided a detailed documentation of this event (Del Prete and Hutchinson, 1988; Guerricchio and Melidoro, 1988; Viggiani and Di Maio, 1991; Troncone, 2005). The landslide occurred at Timpone hill the local topography of which was characterized by a height of approximately 70 m and an average slope of approximately 17°. A plan of the landslide is shown in Fig. 1. The dimensions of the landslide were about 150 m in width and 230 m in length. The depth to failure surface varied from 10 m to 15 m from the ground surface. As a result, the volume of the soil involved in the landslide was about 400,000 m^3. Following Cruden and Varnes (1996), the Senise landslide can be defined as a slide that occurred on a thin zone of intense shear strain. The thickness of this zone was in the order of some decimetres. Sliding caused a downward translation of the soil mass of about 32 m (Viggiani and Di Maio, 1991), and involved the buildings numbered from 1 to 13 in Fig. 1. Most buildings were completely destroyed; the others suffered considerable displacements both in the horizontal and vertical directions. A detailed description of the damage caused to these buildings can be found in the abovementioned studies (Del Prete and Hutchinson, 1988; Guerricchio and Melidoro, 1988; Viggiani and Di Maio, 1991).

Before the occurrence of the landslide, deep excavations had been performed at the toe of the slope to allow the construction of the buildings located in this zone. The depth of excavation as estimated by Del Prete and Hutchinson (1988), is specified in Table 1 in which the date of the construction licence for each building is also shown. The buildings were constructed from 1983 to 1986 (Del Prete and Hutchinson, 1988). On the basis of the data shown in Table 1, it can be asserted that some excavations of relatively small depth were performed for the construction of the buildings 8 and 10. On the contrary, important excavations (with a maximum depth of about 10 m) were carried out in the shared zone indicated in Fig. 1, where two high reinforced concrete retaining walls with shallow foundations were constructed. These retaining walls were located at the back of the buildings 3 and 4, and 5, 6 and 7, respectively (Figure 1).

The outcropping geological formation consists of yellowish sand with interbedded thin layers of clayey silt (Aliano formation). These layers, with prevalent south-westward dip, have a thickness ranging from some centimetres to several decimetres, and an average inclination (downslope) of approximately 18° with respect to the horizontal plane. The sand is very dense and characterised by a significant degree of cementation. The Aliano formation overlies a base formation of blue-grey clay.

After the catastrophic event of July 1986 and before the reactivation of September 1986, a site investigation consisting of boreholes, standard penetration tests and laboratory tests was performed. Several piezometers and inclinometers were also installed. The results from the laboratory tests showed that the grain size distribution of the Aliano sand varies from sand with gravel to sand with silt, and that of the clayey silt from sandy and clayey silt to silt with clay. The strength characteristics of the soils were obtained from isotropically consolidated–drained triaxial tests and direct shear tests. A peak shear resistance angle $\varphi'_p = 43$ °, and a peak cohesion $c'_p = 37$ kPa can be assumed for the sand. The peak strength parameters for the clayey silt are

Table 1
Date of the construction permission for the buildings involved in the landslide, and the maximum depth of excavation behind these buildings.
Data from Del Prete and Hutchinson (1988).

Building number	Maximum depth of excavation (m)	Date of construction permission
6	5	Nov. 09, 1982
3	10	Oct. 20, 1982
7	7	May. 31, 1982
4	9	May. 15, 1982
5	7	Mar. 06, 1980
8	1.5	Jan. 21, 1980
10	3	Jan. 21, 1980
11	1	Feb. 04, 1976