Seismic slope stability analyses

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EPFL / ENAC / GC section – Master semester 2 and 4 – 2024-2025

1. Introduction

Earthquakes may significantly affect the stability of slopes. Fast generation of excessive pore water pressures (undrained conditions) can cause a severe reduction of the slope stability. Some soils may undergo significant reduction of their shear strength during the dynamic loading (e.g. collapse/liquefaction of loose sands; quickclays)

Two main methods of analysis can be used.

I. Dynamic analysis: it is a complete analysis of the slope behaviour under the seismic loading. A comprehensive procedure for seismic analysis would require:

1) The anticipated acceleration-time input for the slope $\mathbf{a}(t)$.
2) A proper characterization of the behaviour of the involved soil under cyclic loading, through laboratory testing and the choice of a proper constitutive law.
3) A static FEM simulation to evaluate the initial stress state for the slope, together with the pore water pressures.
4) A dynamic FEM simulation subjecting the slope to the dynamic condition (accounting for the loss of shear strength related to the increasing of pore water pressure and the cyclic soil response).
5) The stability of the slope can be computed considering the change in the stress distribution resulting from the dynamic FEM analysis.

A dynamic analysis involves a level of complexity so that is seldom employed. However it should be performed when problematic soils are involved, significant reductions of shear strength are expected, the landslide-related risk is high. Dynamic analyses are normally performed for dams due to the high cost of the construction and the possibility to analyse the behaviour of the compacted material in the laboratory.

II. Analysis based on the LEM: the dynamic load is represented by a corresponding static force (pseudostatic analysis) and the factor of safety is computed. When the landslide body is mobilized by the seismic action, the cumulated displacement can be computed with the Newmark method.

2. The pseudostatic analysis

In the pseudostatic analysis, the dynamic load due to the earthquake is represented by a horizontal static force $A_{h,d}$ (seismic action). The module of this force is equal to the weight *W* of the considered body multiplied by a seismic coefficient (*k*):

$$A_{h,d} = kW \quad (2.1)$$

The seismic coefficient can be thought as the ratio of the horizontal ground acceleration produced by the earthquake (a_g) to the gravity acceleration (g):

$$A_{h,d} = \frac{a_g}{g} W = a_g M \quad (2.2)$$

where *M* is the mass of the body.

Table 2.1 reports values of seismic coefficients and suggested values of the factor of safety from various studies. It is advisable to refer to the norms of the Country of reference (see exercise 7 for the Swiss case).

The seismic action is considered in the limit equilibrium analysis as a static force; an expression of F accounting for this force is derived. In the following, the expression of F is obtained for the cases of an infinite slope in dry conditions or a rigid block on an inclined slope (**Figure 2.1**).

Table 2.1. Suggested seismic coefficients from various studies (Jibson 2011).

Investigator	Recommended pseudostatic coefficient (k)	Recommended factor of safety (FS)	Calibration conditions
Terzhagi (1950)	0.1 (R-F = IX) 0.2 (R-F = X) 0.5 (R-F > X)	> 1.0	Unspecified
Seed (1979)	0.10 (M = 6.50) 0.15 (M = 8.25)	>1.15	< 1 m displacement in earth dams
Marcuson (1981)	0.33-0.50 × PGA/g	> 1.0	Unspecified
Hynes-Griffin and Franklin (1984)	0.50 × PGA/g	> 1.0	< 1 m displacement in earth dams
California Division of Mines and Geology (1997)	0.15	>1.1	Unspecified; probably based on < 1 m displacement in dams

R-F is Rossi-Forel earthquake intensity scale

M is earthquake magnitude

PGA is peak ground acceleration

g is acceleration of gravity

Equilibrium in the direction parallel to the slip surface:

$$T = W \sin\alpha + kW\cos\alpha \qquad (2.3)$$

Equilibrium in the direction perpendicular to the slip surface:

$$N' = W\cos\alpha - kW\sin\alpha \qquad (2.4)$$

For a planar slip surface:

$$T = \frac{T_f}{F} = \int_0^L \frac{\tau_f}{F} dl = \frac{c'L + N'\tan\varphi'}{F} \qquad (2.5)$$

Factor of safety:

$$F = \frac{T_f}{T} = \frac{c'L + (W\cos\alpha - kW\sin\alpha)\tan\varphi'}{W\sin\alpha + kW\cos\alpha} \qquad (2.6)$$

In the case of a planar slip surface and $c' = 0$, the critical or yield acceleration a_{yield} (i.e. the value of a_g for which $F = 1$) can be computed as:

$$a_{yield} = \frac{\tan\varphi' - \tan\alpha}{1 + \tan\varphi' \tan\alpha} g \tag{2.7}$$

When a circular failure mechanism in a saturated slope is considered, an undrained analysis may be performed (**Figure 2.2**):

$$F = \frac{c_u L_a r}{Wb + kWa} \tag{2.8}$$

where L_a is the length of the arc.

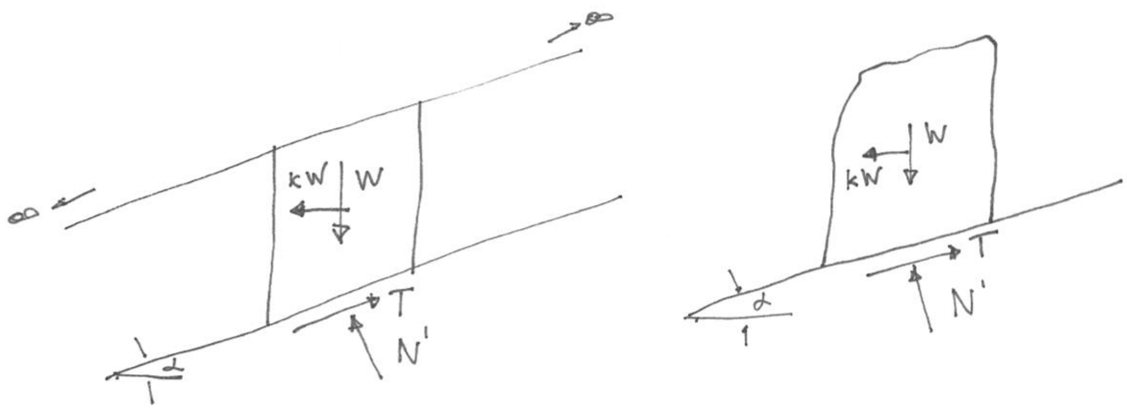

Figure 2.1. Outline for computing *F* for a dry infinite slope and a rigid block subjected to seismic action.

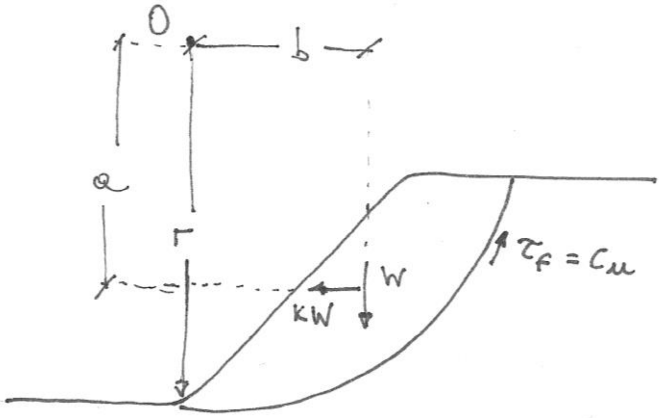

Figure 2.2. Outline for computing *F* for a circular slip surface – undrained analysis accounting for the seismic action.

Experience shows that the pseudostatic analysis can be over-conservative in many situations and unconservative in others. Its main limitation is related to the fact that the reduction in available shear strength is not considered. As a consequence it should not be used when the involved soils may generate significant pore water pressures or can reduce their shear strength properties during the seismic loading (more than 10 - 15%).

3. The Newmark analysis

Newmark method (1965) allows computing the cumulated displacement of a sliding slope during a seismic event. It is based on a simple model of a rigid block sliding on a planar surface. The model is a strong simplification of the real slope behaviour and should not be used when significant changes in pore water pressures and shear strength are expected or when the landslide body may undergo internal deformations.

With reference to **Figure 2.1**, the block will start its motion after the base acceleration exceeds its yield acceleration. When the block begins to move downslope, all the available shear strength is mobilized ($T = T_f$). For the case $c' = 0$, the resulting force in the direction parallel to the slope is:

$$R = W(k\cos\alpha + \sin\alpha - \cos\alpha\tan\varphi' + k\sin\alpha\tan\varphi') \qquad (3.1)$$

The block acceleration in the direction parallel to the slope is:

$$\ddot{x}(t) = R/M = a_g(t)(\cos\alpha + \sin\alpha\tan\varphi') + (\sin\alpha - \cos\alpha\tan\varphi')g \qquad (3.2)$$

Velocity is calculated by integrating the acceleration time history:

$$\dot{x}(t) = \int_0^t \ddot{x}\,dt \qquad (3.3)$$

When the velocity becomes negative during the integration, (i.e the block would tend to move uphill), the velocity must be set equal to zero. It would be in fact not correct to continue to account for all the available shear strength when computing the acceleration of the block; moreover, if the block tends to move uphill, the available shear strength would be mobilized in the opposite direction with respect to **Figure 2.1**.

The cumulated displacement in the direction parallel to the slope is calculated by integrating the velocity time history:

$$x(t) = \int_0^t \dot{x}\,dt \qquad (3.4)$$

Figure 3.1 shows the evolution of velocity and displacement for a rigid block on a slope with α = 22° and φ' = 30° subjected to a sinusoidal ground acceleration time history. The following trends are observable:

1. The ground acceleration does not exceed the critical acceleration and the block moves together with the bedrock ($T < T_f$).
2. As the acceleration reaches the critical value, the block starts to move; velocity increases and displacement cumulates.
3. When the ground acceleration becomes smaller than the critical value, the motion is decelerated; in this phase the available strength is higher than the shear force needed to maintain the block in equilibrium; however all the strength is mobilized since the block is still moving.
4. When the velocity has a zero value, the block stops moving. The block remains in equilibrium until the ground acceleration exceeds the critical value again.

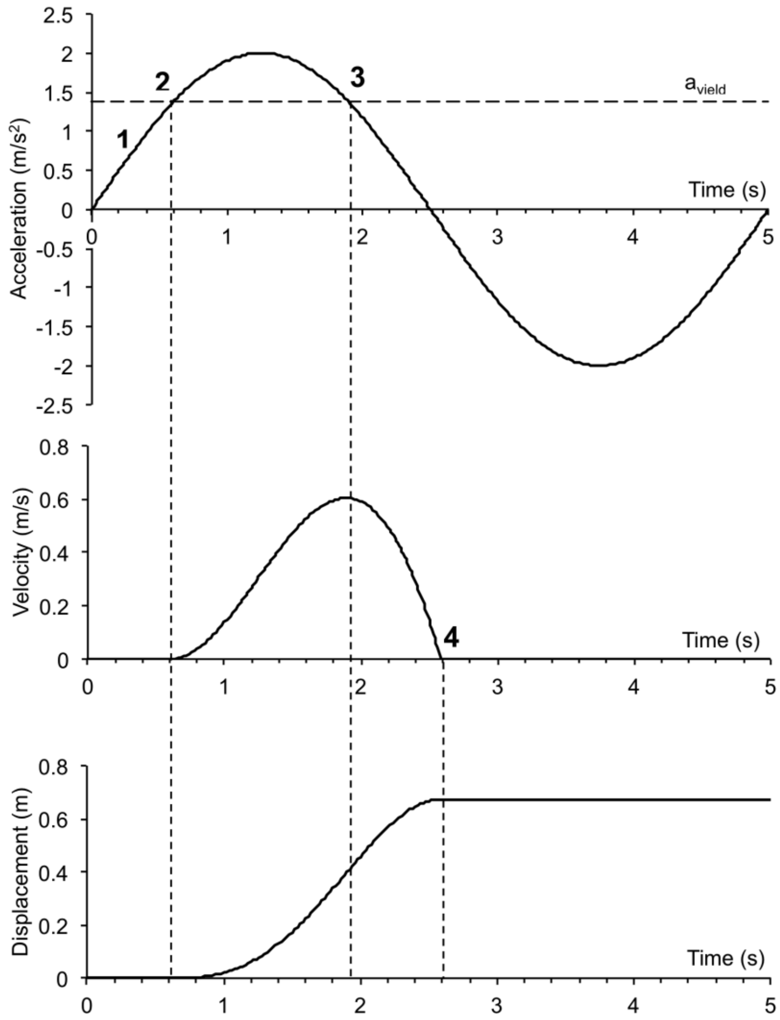

Figure 3.1. Application of the Newmark method for a rigid block subjected to a sinusoidal seismic input.

References

Jibson, R. W. (2011). Methods for assessing the stability of slopes during earthquakes — A retrospective. *Engineering Geology*, *122*(1-2), 43–50.

Newmark, N.M., (1965). Effects of earthquakes on dams and embankments. Geotechnique 15, 139-159.