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SEISMIC SAFETY OF ROCKFILL DAMS

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Abstract. Structural stability of concrete or masonry arch dams can be assessed using the standard Finite Element techniques. The sufficient safety is provided as long as the superposed stresses from all relevant loading cases remain smaller than the material resistance. For earth- and rockfill dams the slope stability has to be proved. Taking into account only the static loads, the method of Krey/Bishop can be used to determine the critical slope exhibiting the minimum safety. The standard way to determine the slope stability under dynamic loading conditions is by a quasistatic Krey/Bishop method. In this method the maximum acceleration of the dam gravity center is multiplied by the mass and the resulting force is compared with the restraining forces given by friction and cohesion along one slope curve. This paper presents an alternative approach, which is based on a full dynamic response calculation of the dam structure in the time domain under all relevant loads.

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1 Introduction

German authorities require to perform detailed verification about the structural safety of existing water reservoir dams regularly. In the framework of this verification scheme, a number of dams located in the Sauerland region, which are owned and operated by the Ruhrverband Essen, have been explored. Following the experiences with the 1992 Roermond earthquake, the surveillance authorities imposed higher requirements on the earthquake loading scenarios. The applied earthquake loading conditions will be discussed in detail in chapter 2.

Chapter 3 describes the Finite Element discretization and the relevant static loading conditions, presenting the special problem of hang-up effects which can be observed in rockfill dam structures with a soft clay core and a stiff rock filling.

Structural stability of rockfill dams has to be proven by verification of an adequate slope stability. The standard procedure, taking into account only static loading conditions, has been given by Krey/Bishop. This method can not be directly extended to dynamic problems. In a first approximation a quasistatic Krey/Bishop procedure can be used to determine minimum slope stability under dynamic loading. The basics of this simplified approach, which is usually taken for seismic safety calculations of dam structures, will be explained in chapter 4.

Chapter 5 presents an alternative approach, which is based on a full dynamic response calculation in time domain, taking into account all relevant static and dynamic loads.

2 Earthquake Loading

A detailed investigation about the seismic activity in the Sauerland has been performed by the Geological Institute of the University Cologne [1]. The critical load case has been defined, in accordance with the surveillance authorities, as the safety earthquake with a frequency of occurrence to be once every 1000 years. Following current regulations for nuclear power plants, only this seismic load case has to studied. The following vertical and horizontal peak accelerations have to be considered for the investigated site (table 1):

peak accelerations [m/s^2]		
	horizontal acceleration	vertical acceleration
University of Cologne	0.45	0.25
Surveillance authority	0.56	0.18

Table 1: Peak accelerations in vertical and horizontal direction [m/s^2]

The difference in the prescribed peak accelerations can be explained by a reduction factor of 80%, which has been applied by the Geological Institute of the University of Cologne, in order to consider recent measurements. Measurements of the 1992 Roermond earthquake have been showing that the predicted peak accelerations by the Murphy&O´Brien formula

$$\log(a_h) = 0{,}25I + 0{,}25 \qquad a_h \text{ in cm/s}^2, \tag{1}$$

overestimated the measured results by 25% to 60%. The seismic assessment of the dam has been based on conservative peak accelerations of the surveillance authority. Figure 2 presents the appropriate response spectra.

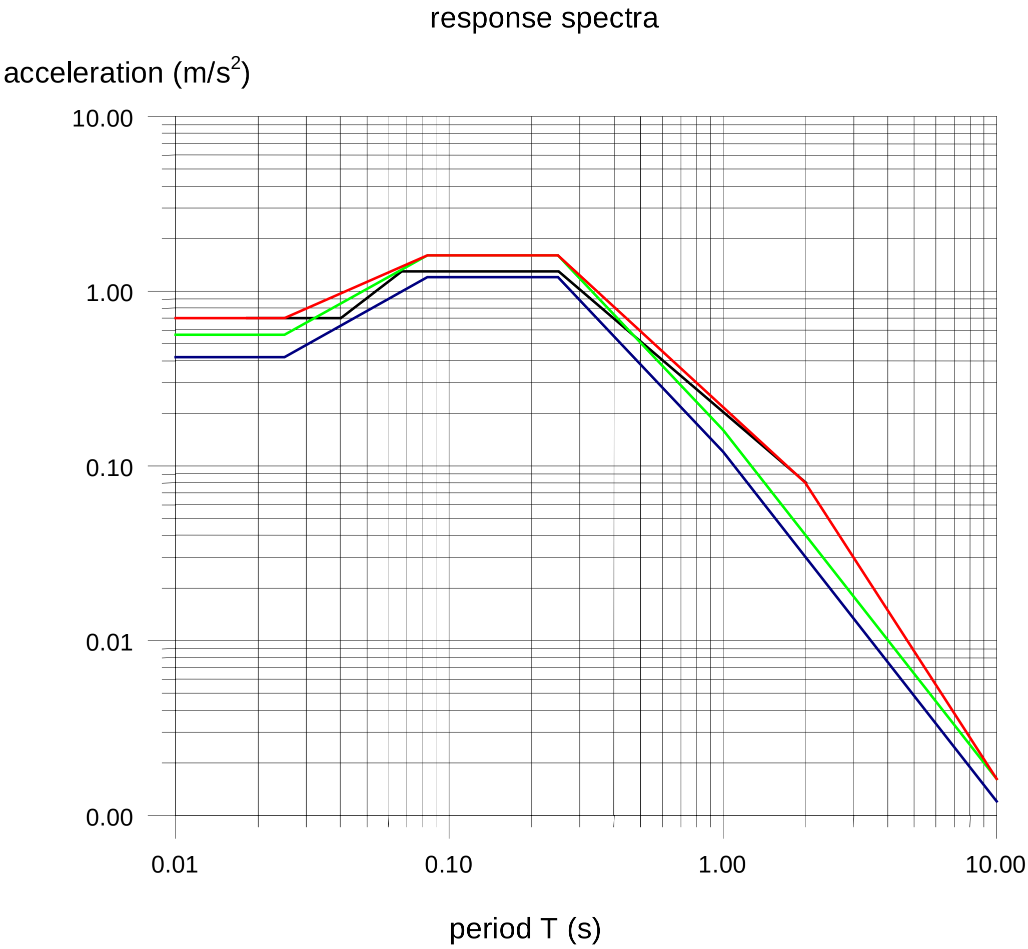

Figure 1: Response spectra

The blue line has been proposed by the Geological Institute of The University of Cologne. Transforming this spectrum to the higher peak acceleration of the surveillance authority, leads to the spectrum marked with a green line, which keeps the same shape as the original one, but shows higher acceleration values over the whole period range. The response spectrum given by the surveillance authority has been based on a proposal by Hosser [2] and is shown as black line in figure 2. As a conservative assumption the envelope of all three proposed spectra, shown as red line in figure 2, has been taken to generate artificial acceleration-time histories. These acceleration-time histories, describing the free-field response, have been transferred to the bedrock level by a deconvolution calculation.

3 Finite Element Model

3.1 Geometry and Finite Element Discretization

The investigated dam structure is a rockfill dam of 8.67 m height with clay core (figure 2). The subsoil has been taken into account to a depth of 7.50 m, where the bedrock is reached.

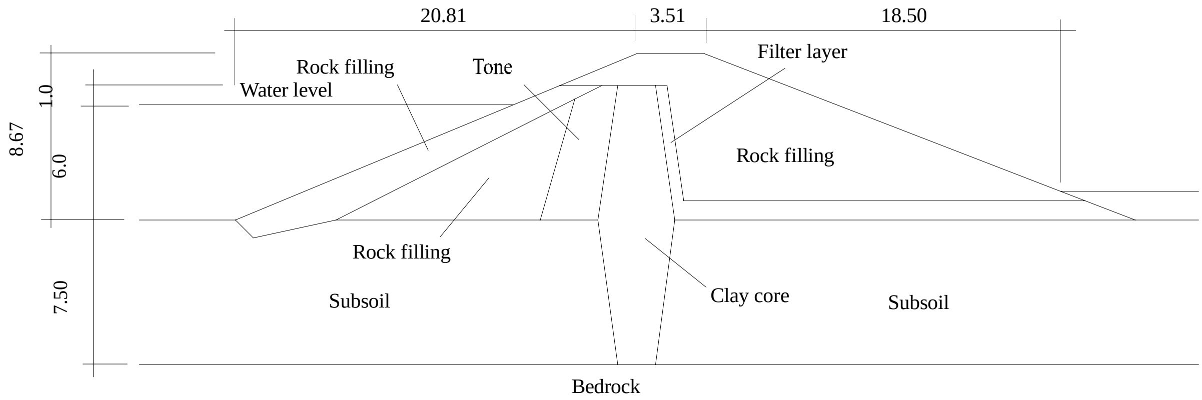

Figure 2: Dam geometry

The dam has been modeled as a 2D plane strain Finite Element model. The discretization with 2104 bilinear elements can be seen in figure 3. The acceleration-time histories have been applied to the bedrock level as vertical and horizontal accelerations.

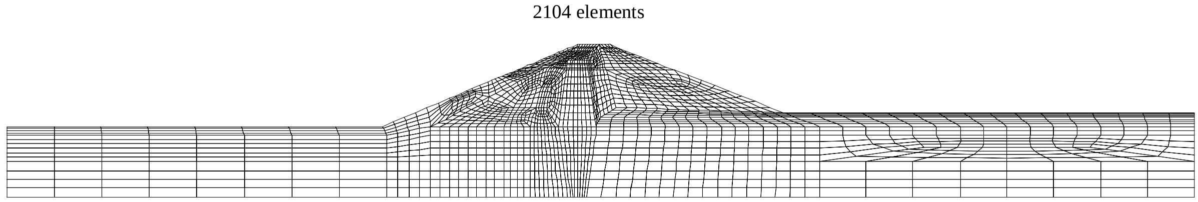

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Figure 3: FE-Discretization

3.2 Dead load and hydrostatic water pressure

Static loading consists of the dead load of the dam structure and the hydrostatic water pressure, acting on the clay core. Depending on the stiffness ratio between clay core and rockfill dam, we can observe that the soft clay core hangs up the surrounding rockfill region. Due to the utilized linear material models tension stresses will develop in the clay core. This behavior is contradictory to a physically reasonable material behaviour, where tension stresses in the soil will lead to cracks perpendicular to the principal stress direction. Due to stress redistribution only compression stresses will develop in the dam structure. The stress distribution due to dead load computed with initial material stiffness is shown in figures 4 to 6. The tension stresses at the upper left corner of the clay core can be clearly observed.

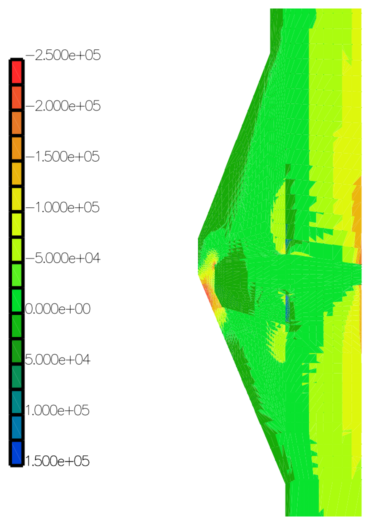

Figure 4: Stress distribution σ_{xx} [N/m^2]

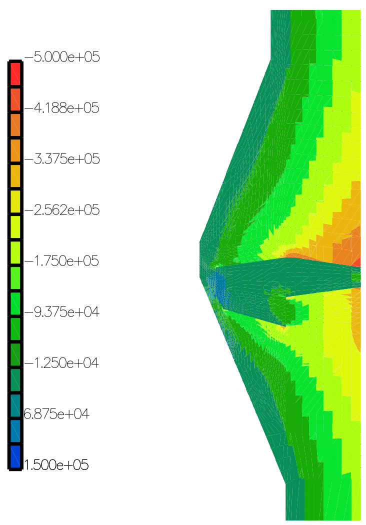

Figure 5: Stress distribution σ_{yy} [N/m^2]

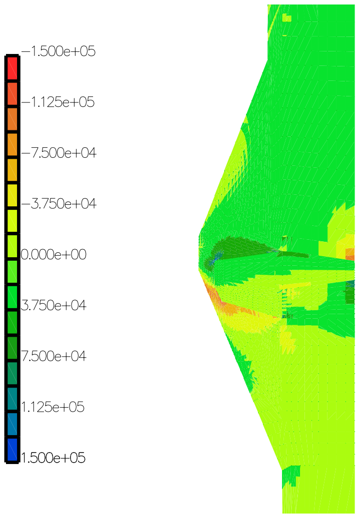

Figure 6: Stress distribution σ_{xy} [N/m^2]

The displacement plot is shown in figure 7.

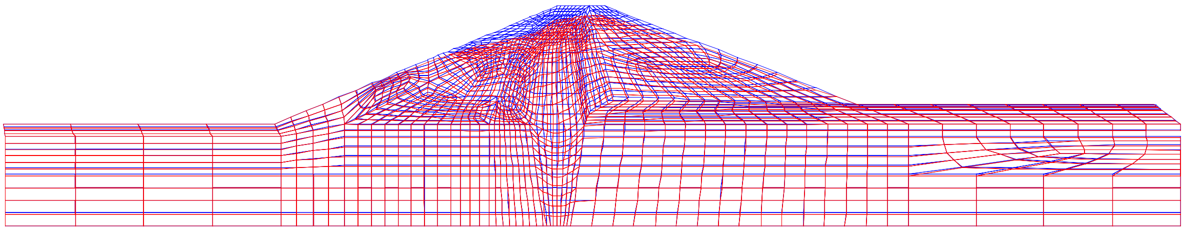

Figure 7: Displacement plot (scaling factor 100)

In a first approximation to overcome the tension stress, the stiffness of the specific subregions have been scaled with respect to their density. The stiffness and density of the rock filling is used as reference. This procedure leads to a higher stiffness coefficient of the clay core, which prevents the evolution of tension stress under dead load. It should be mentioned that this stiffness adaptation has been taken as first step in order to overcome the hang-up problem without leaving the linear elastic material range. Figures 8 to 10 demonstrate the resulting stress distribution, showing that the vertical stress σ_{yy} now tends to develop a nearly linear increase over the height of the dam. This "quasi-hydrostatic" stress distribution can be also explained physically by consolidation process, developing over longer time periods [3].

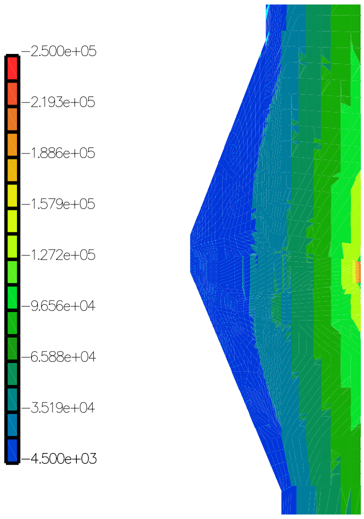

Figure 8: Stress distribution σ_{xx} [N/m^2]

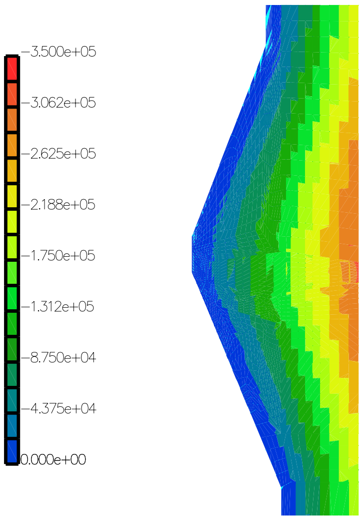

Figure 9: Stress distribution σ_{yy} [N/m^2]

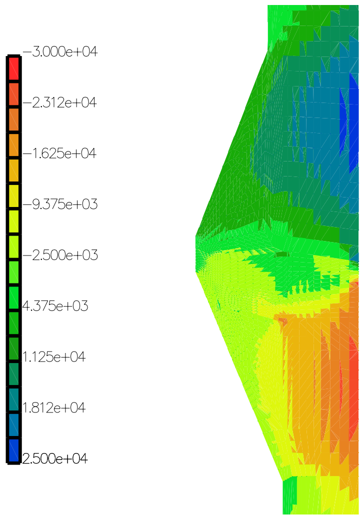

Figure 10: Stress distribution σ_{xy} [N/m^2]

The corresponding displacement plot is shown in figure 11.

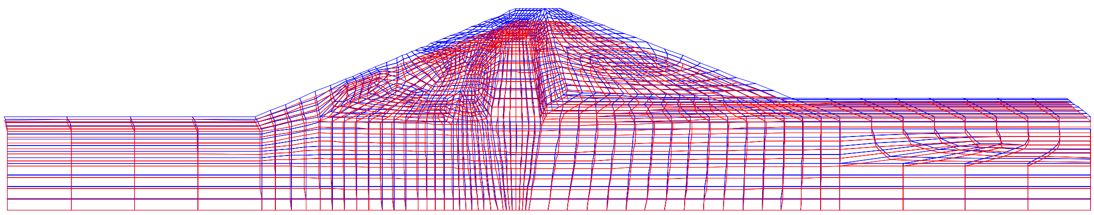

Figure 11: Displacement plot (scaling factor 100)

The stiffness modified model has been also taken to calculate the structural response under hydrostatic pressure, which has been applied as a normal pressure distribution, linearly varying with height, to the clay core.

4 Krey/Bishop method for determination of slope stability

The Krey/Bishop methods supposes the development of a circular shaped slope curve, along which parts of the dam will slide against each other. Each sliding part is defined as a rigid solid. The forces driving the sliding part to move are given by dead load (in a purely static loading case) and mass inertia effects (in a dynamic load case). The restricting forces are given by the cohesional and frictional forces along the slope (see figure 12). The criteria for slope stability is fulfilled as long as the driving forces remain smaller than the restricting forces.

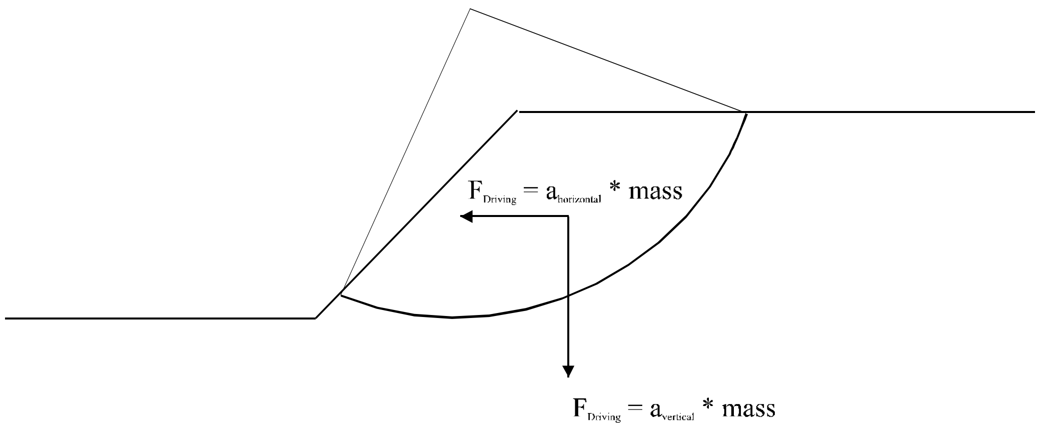

Figure 12: Krey/Bishop method for determination of slope stability

It now becomes obvious, that in case of dynamic loading conditions the question arises which inertia forces should be considered. In most cases published in literature, the maximum accelerations in the studied time interval have been taken to compute the driving forces [4]. This method allows certainly to study the worst case scenario, finding the minimum safety factor for slope stability, but in the authors opinion it shows two major disadvantages. The time dependency of the slope stability vanishes, by studying only one specific situation in time, and more realistic stress distribution, which is provided by the FE calculation is not considered. Both disadvantages can be overcome by the approach presented in the next chapter.

5 FE-approach for determination of slope stability

Using the results of the time domain FE-computation, each time step is clearly defined by its displacement, strain and stress state. These variables can be evaluated along arbitrary slope line cutting through the dam structure, e.g. along every circular shaped slope line. The stress tensor along the slope line can be transformed into normal and tangential components, as shown in figure 13.

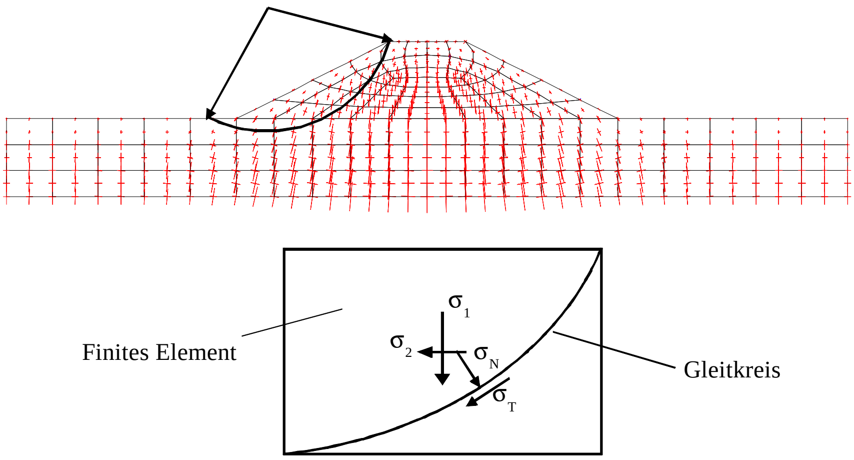

Figure 13: Circular slope and normal/tangential stress components

The slope stability can be now defined by comparing the driving forces, given by integration of the tangential stress component along the slope line

$$T_{Driving} = \int_L \sigma_T \, dL \quad , \tag{2}$$

with the resisting forces, resulting from sum of friction and cohesion forces

$$T_{Resistance} = \int_L \sigma_N \tan\varphi' + c' \, dL \quad . \tag{3}$$

The resulting safety factor for the slope stability is then given by

$$\eta = \frac{T_{Resistance}}{T_{Driving}} \quad . \tag{4}$$

By variation of the central point and radius of slope line the minimum safety factor in every time step can be determined. Figure 14 is showing the result for the safety factor for two different discretizations of the dam structure, fixed central point and varying radius [5]. The result demonstrates, that the minimum safety factor is obtained for a slope line which cuts the dam close to the surface. This result is reasonable for soil structures without cohesion.

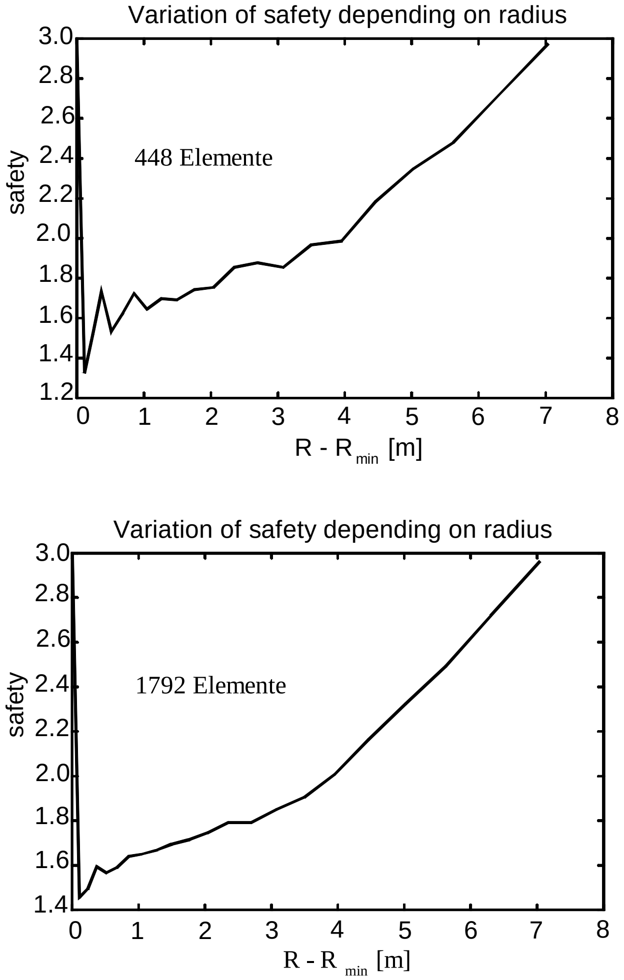

Figure 14: Safety factor for fixed central point and varying radius

In order to obtain the minimal safety factor for the dam under combined static and dynamic loading the static and dynamic response have to be superposed. In every time step the minimal safety factor and the appropriate slope line can then be detected by the just described procedure. The result can be plotted as a three-dimensional surface in the s-t-coordinate system (see figure 15). Every point on this surface characterizes the minimal safety factor for the associated central point of a specific slope line. The information about the associated radius can not be detected from this plot.

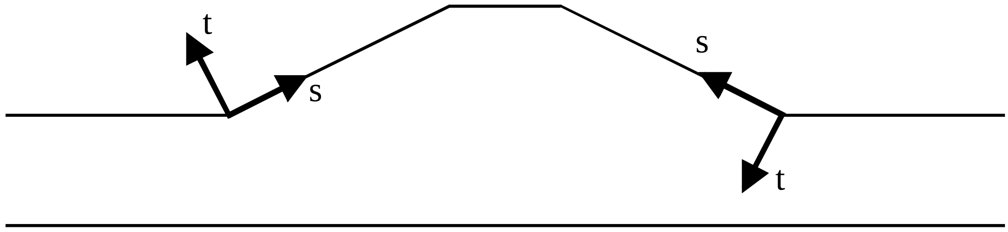

Figure 15: s-t-coordinate system

Figure 16 shows this safety surface at time t = 0.0 seconds, where only static loads have been applied.

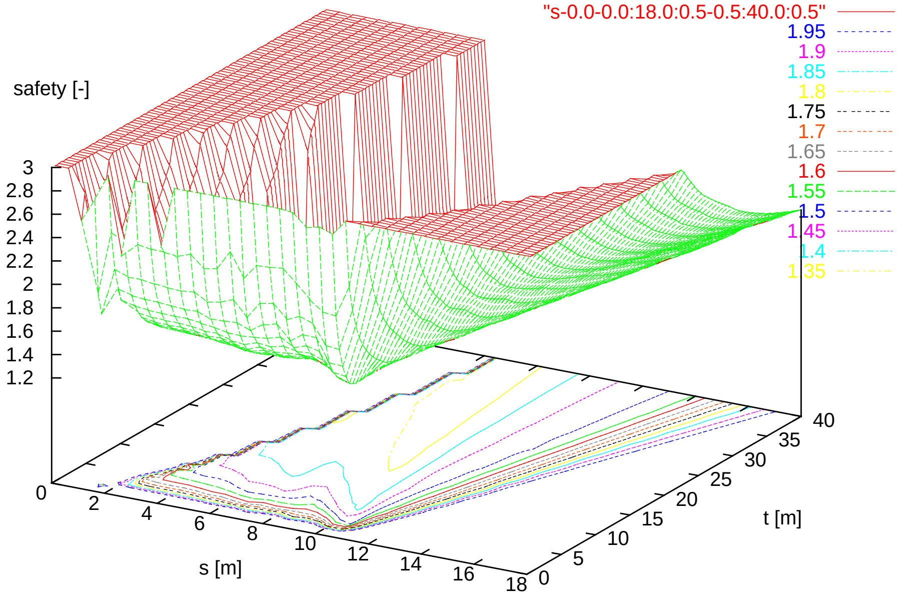

Figure 15: safety surface at time t = 0.00 seconds

It can be seen, that a zone of minimal safety factor η = 1.35 develops for s = 8.00 - 10.00 m and t = 15.00 - 35.00 m.

We now apply the earthquake loading and study the evolution of the safety factor for fixed slope lines over time (figure 17). This plot shows the time-dependent behavior of the slope stability.

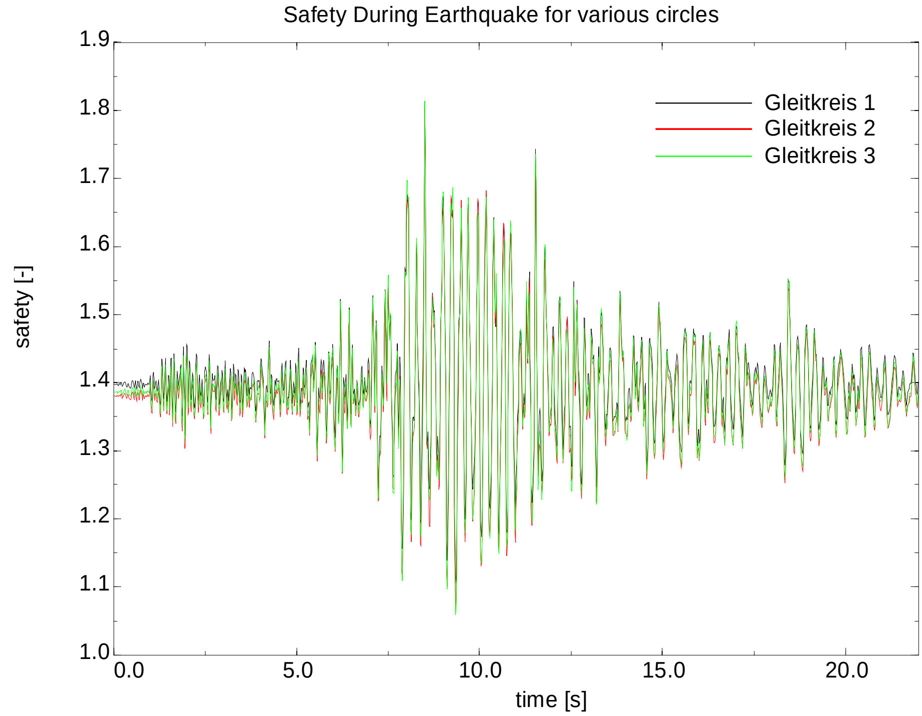

Figure 17: Time-history of safety factor for three different slope lines

6 Conclusions

The paper demonstrated a new approach to compute the structural safety of rockfill dams under static and dynamic loads. The search for determining the minimal safety factor is based on the stress results obtained from a FE-calculation in time domain. With this procedure the history of safety can be traced over arbitrary time intervals.

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