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SEISMIC DESIGN OF EMBANKMENTS – NUMERICAL AND ANALYTICAL STUDY

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Abstract: In this paper, a dynamic shear strength analysis was performed in a finite elements environment to evaluate the stability of embankments under seismic loadings. In addition, a dynamic factor of safety depending on the results of the numerical analysis was defined. To study these parameters' effects on the embankment's stability, a parametric study concerning the embankment inclination and the soil type was performed. Finally, the numerical analysis results were compared with the results of the pseudo-static analysis according to the EC8. The results of this study show the significance of the numerical seismic analyses in comparison with the analytical calculations.

Keywords: embankments, dynamic shear strength, seismic loadings, numerical analysis, stability, sound absorption coefficient.

ПРОЕКТИРОВАНИЕ НАСЫПЕЙ С УЧЕТОМ СЕЙСМИЧЕСКИХ ВОЗДЕЙСТВИЙ – ЧИСЛЕННО-АНАЛИТИЧЕСКОЕ ИССЛЕДОВАНИЕ

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Аннотация: В статье выполнен анализ динамической сдвиговой прочности в программной среде метода конечных элементов для оценки устойчивости насыпей при сейсмических нагрузках. При этом, по результатам численного анализа определен динамический коэффициент запаса прочности. Для изучения влияния этих параметров на устойчивость насыпи проведено параметрическое исследование наклона насыпи и типа грунта. В завершении, результаты численного анализа сравнивались с результатами псевдостатического анализа в соответствии с ЕК8. Результаты этого исследования показывают значимость численного сейсмического анализа по сравнению с аналитическими расчетами.

Ключевые слова: насыпи, динамическая прочность на сдвиг, сейсмические нагрузки, численный анализ, устойчивость, коэффициент звукопоглощения.

INTRODUCTION

The propagation of the earthquake-induced waves due to an earthquake event affects the stability of geotechnical structures and causes different damages and ground settlements. The degree of influence depends on the geology, topography of the influenced area, and the behavior and properties of the local soil under cyclic loadings and it is highly related to the ground motion parameters. In the historical developments of the procedures to analyze the seismic stability of embankments, the pseudo-static approach of Terzaghi (1950) based on the principle of D'Alembert in the mechanics was used very often. The effects of an earthquake in this approach and its following improvements are represented by constant horizontal and/or vertical accelerations, which produce inertial forces F_h and F_v acting through the centroid of the failure mass. The stability of the slope can be then evaluated by resolving the static and pseudo-static forces on the potential failure mass. Simultaneously, the permanent deformations can be estimated using the sliding block method developed by Newmark (1965). In this method, the permanent deformations can be calculated by time-dependent double integrating of the accelerations, which are above a determined value called the yield accelerationa_y, which makes the pseudo-static factor of safety equals to unity.

The above-mentioned methods are very simple to comprehend and to be applied due to their similarity to the static slope stability analyses and because they can be used to analyze the stability of slopes with sliding surfaces of different forms (planer, circular, or arbitrary failure surfaces). However, the simplification of the very complex effects of the earthquake using only pseudo-static accelerations is very rough. In addition, these methods assume the soil behavior to be rigid which means that they do not consider the realistic soil behavior. Furthermore, these analyses can be unreliable for soils that build up large pore pressures or show more than 15% degradation of strength due to earthquake shaking.

Despite the drawbacks and shortcomings of these methods, many regulations such as the Euro code (EC8) or the German standard (DIN EN 1998-5: 2010-12) suggest their applications to study the seismic stability of slopes and embankments taking into consideration their application limits.

In contrast to the conventional methods, the numerical methods consider the realistic behavior of the soil according to the utilized constitutive models. In addition, the analyses can be achieved in the time domain using real acceleration-time histories. Further, the values of the expected permanent deformations can be realistically determined.

Although the application of the finite element method is simple, it requires good knowledge of numerical modeling, an accurate understanding of the problem to be solved, and deep comprehension of the material laws, which govern the soil behavior. In the following, the most important requirements for numerical modeling under dynamic loading will be discussed.

1. THE APPLICATION OF THE FINITE ELEMENTS METHOD IN THE DYNAMIC ANALYSIS

The dynamic calculations are mainly concerned with the phenomenon of wave propagation. Therefore, the dimensions of the model must be chosen so that the wave reflections at the model boundaries won't influence the important area of the model, which includes the studied structure. This leads to a model with big size and a large number of elements. In addition, the movements of the soil at the model boundaries must be represented realistically using suitable boundary conditions. An additional, artificial, and a special type of boundary condition, which can absorb the earthquake-induced energy waves, must be also used. As a consequence of complying with the above-motioned requirements, the energy waves that propagate inside the model are continuously removed from it.

The mesh coarseness has also a great influence on the results. A coarse mesh with a small number of nodes causes the displacement components of high frequencies to be filtered. The proposed maximum dimension of an element must be between 1/8 Kuhlemeyer und Lysmer (1973) and 1/5 Lysmer et al. (1975) of the shortest wavelength.

After creating the model and generating the mesh, the mass, damping, and stiffness matrices of all elements are calculated and the global equation of motion of the model is time-dependent solved to determine the vectors of acceleration, velocity, and displacement of every node in the model.

2. THE MODEL DESCRIPTION

To achieve the main aims of this study, a plane strain model was created using the program PLAXIS 2D - version 9.0. The soil layers were simulated with volume elements of 15 nodes. The dimensions, boundary conditions, and the coarseness of the mesh were chosen so that the already mentioned requirements are met. The model has a total width of 305 m and a total height of 110 m. The width and the height of the slope are 105 m and 60 m and it has an angle of β=⟦30⟧^o, so that the lateral boundaries and the basis of the model are at distances of 100 m and 50 m away from the slope as shown in figure 1.
The option 'standard Fixities' was used to define the allowed movements at the model boundaries so that the lateral points of the model can only move vertically while the base points were totally fixed.
The seismic action was applied at the lower boundary of the model through prescribed displacement. Simultaneously, energy absorbent boundaries were added at the model vertical sides to eliminate the effects of the reflected waves on the model boundaries. Only the horizontal component of the earthquake was considered in the performed calculations. Figure 2 depicts the model with its boundary conditions. The mesh was generated with a fine coarseness.

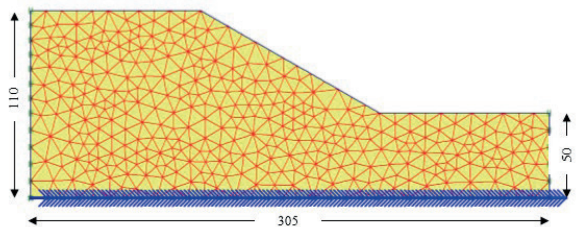


Figure 1. The created model, its dimensions and mesh (without scale, all units are in m)

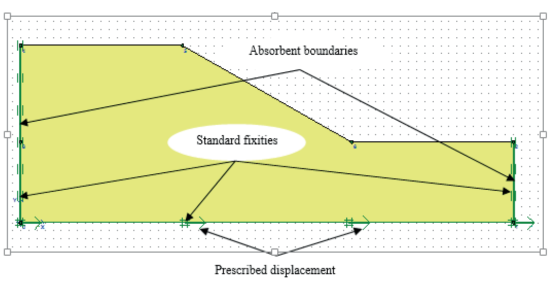


Figure 2. The boundary conditions of the model

3. THE CONSTITUTIVE MODEL HSS

The applied hardening soil small strain model (or abbreviated HSS-model) has the same features as the hardening soil model. In addition, it provides new features added by Benz (2007). The yield surface of the HSS-model can be extended depending on the plastic strain without exceeding the Mohr-Coulomb limit conditions. Due to the consideration of the double hardening, this model can't only account for the elastic and plastic strains (ε_e) and (ε_p), but it also divides the plastic strains into deviatoric and isotropic portions depending on the load direction. The soil stiffness calculated in every calculation step depends not only on the stress level but is also related to the level of the shear strain (γ) so that the value of the shear modulus (G) is greatly increased when the shear strain decreases as illustrated in Figure 3.
As it was observed from the laboratory experiments and the field tests, soil shows high stiffness under dynamic loads, which act within a short time and in a small strain domain. Since the HSS-model can take into account the dependency of the soil stiffness on the level of the shear strain, the use of this material model for the dynamic calculations is ideal. In addition, the HSS-model can also account for hysteretic damping according to the shear strain, so that the damping capacity of the soil (ψ) increases with the increase of the shear strain as shown in Figure 4.

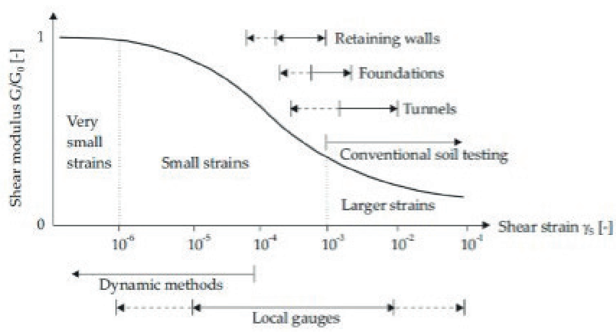


Figure 3. Characteristic stiffness-strain behavior of soil with typical strain ranges

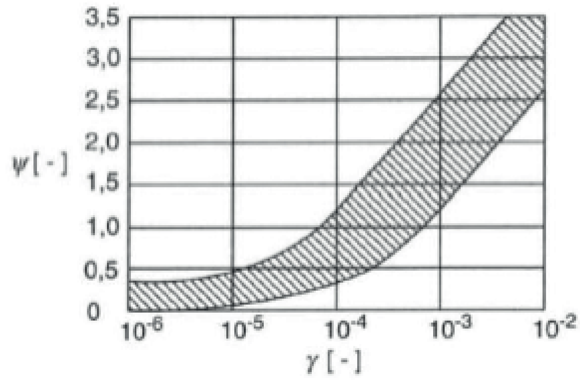


Figure 4. The soil damping capacity related to the shear strain

Depending on the previously mentioned explanations and features of the HSS-model, it can be concluded that this material law is suitable to analyze the studied problem. However, the shortcomings and the limitations of this model should also be taken into consideration. The HSS model cannot generate the accumulated strains due to multiple load cycles. In addition, it cannot generate the excess pore water pressures in the undrained calculations, which leads to the inability of the HSS-model to account for liquefaction, which is the most important phenomenon in the dynamic analysis.

4. THE DYNAMIC SHEAR STRENGTH REDUCTION ANALYSIS

The calculations were carried out stepwise. In the first step, the model has constructed then a static shear strength reduction (abbreviated SSR) analysis was performed to calculate the static factor of safety FS_{stat} and finally the seismic analysis was carried out. Table 1 shows the soil properties of the used dense, well-graded sand (SW).

The applied seismic acceleration-time history as depicted in figure 5 has a maximum acceleration of $PGA = -2.4\frac{m}{s^2}$, a duration of $T = 23.43s$, and a local magnitude of M=5.40.

Table 1. Properties of the SW soil

Soil property	Symbol	Unit	value
Soil unit weight	γ_b	$[\mathrm{kN/m^3}]$	20
Secant stiffness in standard drained triaxial test	E_{50}^{ref}	$[\mathrm{kN/m^2}]$	30000
Tangent stiffness for primary oedometer test	E_{oed}^{ref}	$[\mathrm{kN/m^2}]$	30000
Unloading/reloading stiffness	E_{ur}^{ref}	$[\mathrm{kN/m^2}]$	90000
Power for stress-dependency of stiffness	m	[-]	0.6
Cohesion	C_{ref}	$[\mathrm{kN/m^2}]$	0.1
Friction angle	φ	[°]	40
Dilatancy angle	ψ	[°]	6
Shear strain at which $G_s = 0.772G_0$	$\gamma_{0,7}$	[-]	1.0*10^{-4}
Shear modulus at very small strain	G_0^{ref}	$[\mathrm{kN/m^2}]$	1.0*10^{5}
Poisson's ratio	υ_{ur}	[-]	0.2
Reference stress for stiffness	p_{ref}	$[\mathrm{kN/m^2}]$	100
Failure ration q_f/q_a	R_f	[-]	0.9
Interface strength	R_{inter}	[-]	1

In each subsequent calculation step, the previously done process (excluding the calculation of the static factor of safety) was repeated but using manually reduced shear parameters according to the Fellenius rule (Fellenius, 1927). The shear strength parameters were reduced using a reduction factor RF started from 1,000 (no reduction) and increased with a rate of 0,025 in each following step till the failure was observed. This analysis, in which the shear strength parameters were step by step reduced and the seismic analyses were carried out, is called the dynamic shear strength reduction analysis or abbreviated as dynamic SSR-Analysis. This analysis is just an extension of the static shear strength reduction analysis to the dynamic conditions based on the Fellenius rule.

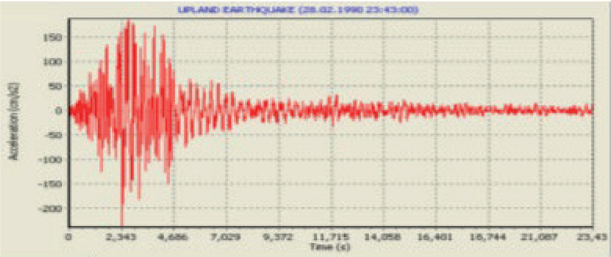


Figure 5. The applied acceleration-time history

To investigate the effect of embankment slope angle on the results, similar calculations with four different models with different slope angles ($\beta = 25^o, 28^o, 32^o$ and 35^o) were performed. Three new models with cohesive, stiff, high plasticity clay (CH) with different slope angles ($\beta = 21^o, 23^o$ and 25^o) were created to investigate the influence of the soil cohesion on the embankment stability under seismic conditions. In these models, the soil parameters listed in Table 2 were used.

Table 2. Properties of the CH soil

Soil property	Symbol	Unit	value
Soil unit weight	γ_b	$[kN/m^3]$	18
Secant stiffness in standard drained triaxial test	E_{50}^{ref}	$[kN/m^2]$	15000
Tangent stiffness for primary oedometer test	E_{oed}^{ref}	$[kN/m^2]$	15000
Unloading/reloading stiffness	E_{ur}^{ref}	$[kN/m^2]$	45000
Power for stress-dependency of stiffness	m	[-]	0.8
Cohesion	c_{ref}	$[kN/m^2]$	15
Friction angle	φ	[°]	25
Dilatancy angle	ψ	[°]	0
Shear strain at which $G_s = 0.772 G_0$	$\gamma_{0,7}$	[-]	$8.0*10^{-5}$
Shear modulus at very small strain	G_0^{ref}	$[kN/m^2]$	$60*10^3$
Poisson's ratio	υ_{ur}	[-]	0.2
Reference stress for stiffness	p_{ref}	$[kN/m^2]$	100
Failure ration q_f/q_a	R_f	[-]	0.9
Interface strength	R_{inter}	[-]	1

5. ANALYSIS OF THE RESULTS

After the end of each calculation step, the developments of the plastic points (Mohr-Coulomb plastic points) and the incremental strains ($\Delta\varepsilon$) resulted from the seismic analyses were visually observed by creating time-dependent animations. In addition, the values of the total displacement at the top point of the model resulted from the seismic analyses ($U_{tot,dyn}$) were read and noted.

The development of failure surfaces inside the embankment body during the seismic phases of calculations was utilized to assess the stability of the embankment depending on the visual observations of the time-dependent developments of the plastic points and the incremental strains resulted from the dynamic SSR-analysis. According to these visual observations, the slope failed when a continuous failure surface appeared at a particular time during the dynamic analysis. In this context, the time-dependent development of the incremental strains must also be considered because the plastic points can't be used alone as a failure criterion.

Figures 6 and 7 show the failure surface resulted from the plastic points compared with the observation of incremental strains at the same time increment for the model with the slope angle of $\beta = 30^o$.

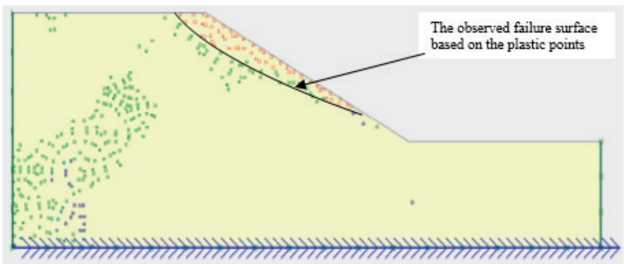


Figure 6. The failure surface resulted from the visual observations of the plastic points (the red points)

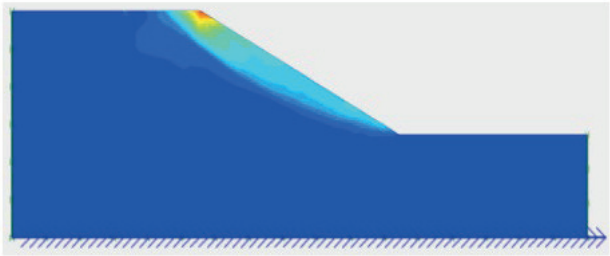


Figure 7. The failure surface resulted from the visual observations of the incremental strains

Furthermore, the development of the total dynamic displacements $U_{tot,dyn}$ with the increasing values of the reduction factor RF during the dynamic SSR-analysis was also observed. Figure 8 illustrates the curve representing this relationship for the model with the slope angle of $\beta = 30^o$.
Depending on the principles of the shear strength reduction analysis, a dynamic factor of safety (FS)_dyn, which represents an amount that can be used to evaluate the stability of an embankment under seismic conditions, can be defined as the shear strength reduction factor RF, at which a continuous failure surface during the dynamic SSR-analysis was observed. The dynamic factor of safety can also be obtained from the curve representing the development of the dynamic displacements during the SSR analysis. In this case, the dynamic factor of safety is defined as the shear strength reduction factor, at which this curve leaves its semi-linear development and turns into an exponential shape. At this value of RF, the values of the dynamic displacements increase suddenly very rapidly. This outcome was found out by noticing the coincidence between the appearance of a failure surface based on the plastic points and the incremental strains from one side and the development of the shape of the curve representing the development of the total dynamic displacements during the seismic SSR analysis on the other side.

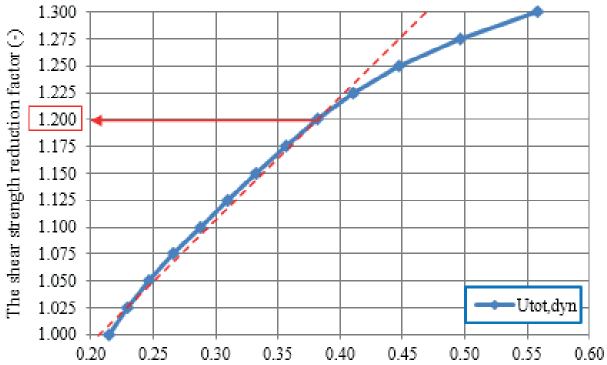


Figure 8. the development of the total dynamic displacement during the dynamic SSR-analysis

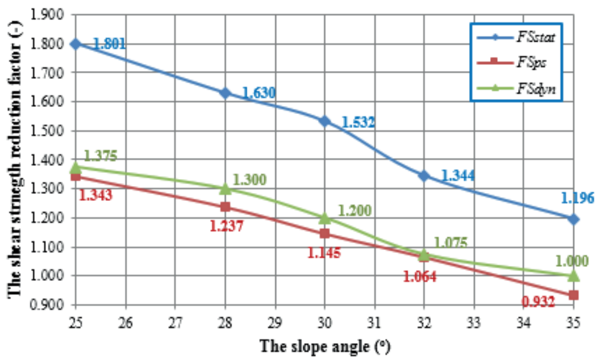
6. THE ANALYTICAL ANALYSIS

The same models discussed in the SSR analyses were created again using the program "Phase02-Slide" in order to compare the results of the performed dynamic analyses with the results of the conventional pseudo-static method. Figure 9 depicts the model of the slope angle of $\beta = 30^o$. The pseudo-static analysis used in this paper benefited the slices method of Bishop (1955) in the pseudo-static conditions by adding inertial forces acting in the centroid of each slice within the potential failure body and resolving the static and pseudo-static forces to calculate the pseudo-static factor of safety FS_{ps}. To determine the value of the pseudo-static coefficient k_h, the response spectrum method according to Eurocode 08, Part 5 was utilized.

7. DISCUSSION OF THE RESULTS

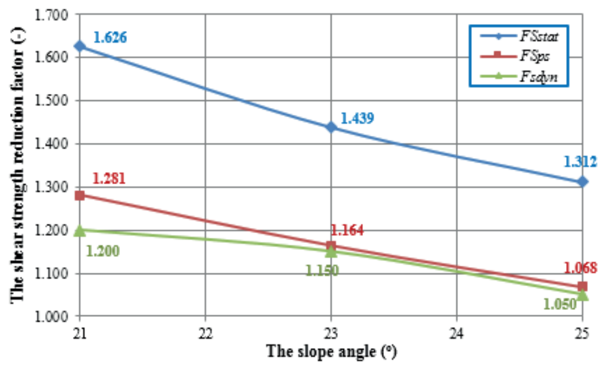
Figures 9 and 10 show the changes in the values of the static FS_{stat}, pseudo-static $FS_{ps}$and dynamic FS_{dyn}factors of safety with the increasing values of the slope angle (β) for the previously described models.
It is clear that the values of the static factor of safety decrease when the values of the slope angle increase. This is also to be noticed in the values of dynamic and pseudo-static factors of safety, so that the steeper the slope is, the lower the dynamic safety level of the embankment is. Furthermore, the

cohesion of the soil plays an important role in increasing slope stability. This fact can be shown by observing the rate of reduction in the values of the dynamic factor of safety with the increasing values of the slope angle for the CH-clay models in comparison with the models of SW-sand.



<u>Figure 9.</u> The decrease of the factors of safety with the increase of the slope angle for the models of the SW-soil

By comparing the results of the pseudo-static analysis with the result of the dynamic finite elements calculations, a rough coincidence can be noted between the results of both analyses, so that the pseudo-static analysis can only provide a rough prediction of the embankment slope stability under seismic conditions. It can also be noted that the results of the pseudo-static analysis may not always be on the safe side, since the results of this type of analysis are highly dependent on the input value of the pseudo-static coefficient



<u>Figure 10.</u> The decrease of the factors of safety with the increase of the slope angle for the models of the CH-soil

8. CONCLUSION

In this paper, a full description of the dynamic shear strength reduction analysis implemented in a finite elements environment was explained and applied to investigate the stability of embankments under seismic conditions. In addition, the paper provides information about the definition of a dynamic factor of safety as a measure for the evaluation of seismic slope stability. Furthermore, a parametric study containing the soil parameters and the embankment slope angle was carried out to predict the influences of these parameters on the stability of the embankments under seismic conditions. Finally, the results of the static, pseudo-static, and dynamic analyses were compared and the main features and interpretations of the results were discussed.

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