Seismic response analysis of a river embankment on deep sedimentary strata

Analyse de réponse sismique d'un barrage de terre fluvial sur des couches sédimentaires profondes

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ABSTRACT

The seismic response analysis of a river embankment on very deep sedimentary strata in a region of medium seismicity is considered. A synthetic seismic input motion is determined from code response spectra with an extrapolation to consider the higher safety level. Dynamic soil properties are estimated from the limited information available. The response analysis is performed using a FEM mesh with a limited subsoil foundation depth whereby the base motion is determined by applying a deconvolution of the deep-stratum free-field response. An effective acceleration value for the dam slope is calculated by an appropriate procedure for subsequent use in pseudo-static slope stability analyses.

RÉSUMÉ

L'analyse de la réponse sismique d'un barrage de terre fluvial sur des couches sédimentaires très profondes dans une région d'activité sismique moyenne est présentée. Un mouvement sismique synthétique est déterminé du spectre sismique de la norme avec une extrapolation pour considérer le niveau de sécurité élevé. Les propriétés dynamiques des sols sont estimées à l'aide de l'information limitée disponible. L'analyse de la réponse est conduite à l'aide d'un réseau des éléments finis avec une profondeur limitée du sol de fondation. Le mouvement de la base est déterminé en appliquant une déconvolution du mouvement à la surface du sol de la couche profonde. Une valeur de l'accélération effective pour le talus du barrage est calculée par une méthode appropriée pour l'utiliser ensuite dans une analyse pseudo-statique de stabilité.

Keywords : Embankment, dam, seismic response, dynamic soil properties

1 INTRODUCTION

Various levels of modelling fidelity can be applied for assessing the seismic response behaviour of embankments, namely i) a pseudostatic method with an ultimate limit state analysis of slope stability, ii) simplified calculation of permanent displacements of potentially sliding masses, iii) numerical analysis in terms of total or effective stresses, Sêco e Pinto (1993), Finn (1998). In practice, these methods are often used in a complementary manner. For example, the value of acceleration that is representative for the sliding mass in the pseudostatic approach is calculated by applying either equivalent linear or nonlinear FEM procedures or simplified methods based on the shear-beam model for regular cross sections geometries, Cascone & Rampello (2003). Furthermore, the Newmark procedure is used to estimate the permanent displacement of sliding masses after the acceleration time histories have been determined from a dynamic response analysis.

The pseudo-static approach is still the basic requirement in many regulatory codes, and is considered appropriate for embankments of medium height. The option of applying sophisticated nonlinear 2D FEM procedures using suitable constitutive models for the earth materials composing the dam body is restricted to critical projects with large heights.

The essential input parameter in the pseudo-static slope stability analysis is the horizontal seismic coefficient k_h. Even today there is no consensus on the methodology to be followed in the selection of a representative value. Given that the potential sliding soil mass is not rigid, a phase difference between points within this mass will occur yielding to an inertial force that is significantly smaller than that implied by the rigid-block assumption. Furthermore, the response of the dam depends on the vibration characteristics of the underlying foundation.

A particular case for the latter effect refers to very deep sedimentary strata, such as those encountered along the Rhine River Valley in Germany. Starting with the response spectra defined in seismic building codes for this type of soil, a practical procedure is proposed to determine appropriate values for the seismic coefficient. The investigation is restricted to embankments of small heights where the approximation of in-phase movement of the failure surface is reasonable, and is applied here for a region of moderate seismicity.

2 GEOLOGICAL SETTING

The region considered belongs to the Upper Rhine Graben. As a major extensional rift system it is one of the major active tectonic structures in Western Europe. The seismic hazard is considered to be moderate. The total thickness of the tertiary and quaternary sediments along the graben reaches values up to 3000 m. In the broader region of the site the base of the tertiary lies between 200 m and 1000 m. The thickness of the quaternary is approx. 100 m. A typical cross section is given in Figure 1.

3 SUBSOIL AND EMBANKMENT STRUCTURE

The foundation subsoil at the dam location has been investigated only to a limited extent. The top layer of thickness from 0.5 to 1.5 m consists of clay and silt, and is underlain down to a depth of at least 160 m by sand and gravel layers. Grain size for 50% passing d_{50} range from 6 to 25 mm. In-situ sounding tests by dynamic probing (DPH) showed values of

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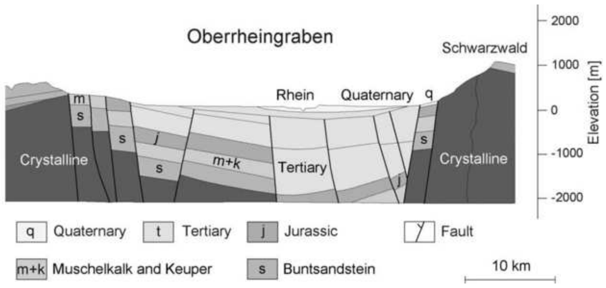


Figure 1. Typical geological cross section of the Upper Rhine Graben.

relative density $D_r = 0.48$. For use in the slope stability analyses an angle of friction equal to 34° was selected.

The embankment dam running along the Rhine river has a height of 12 m relative to the river bed level, a crest width of 6.5 m, and up-/downstream slope inclinations of 25°, Figure 2.

The core is composed of alternating cohesive and non-cohesive layers of sandy silt and gravely sand, respectively. The upstream and downstream shells contain coarser gravel material. On the upstream side a 1 m thick silty layer has been placed between core and shell to provide additional sealing. Due to the composition of the dam body the sealing effect is considered insufficient thus necessitating the installation of a cut-off wall.

DPH soundings in the shell indicated well compacted material. For the whole dam an average value of relative density $D_r = 0.63$ was selected. Typical grain size distribution curves for the core materails are: sandy silt with $d_{10} = 0.002$ to 0.04 mm; $d_{60} = 0.06$ to 0.2 mm; sandy gravel with $d_{10} = 0.06$ to 0.6 mm; $d_{60} = 15$ to 25 mm.

Shear strength parameters φ [°] / c [kN/m^2] for the various earth materials were: 31.5/5 for the core; 25/5 for the silty layer; 36.5/0 for the shell.

The upstream water level as well as the seepage line in the dam are plotted in the cross sectional view in Figure 2.

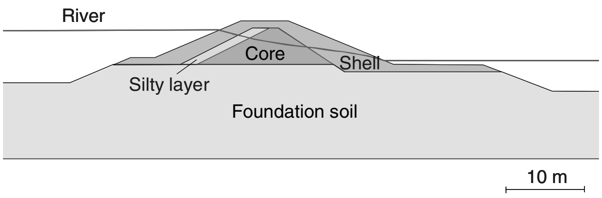


Figure 2. Cross sectional view of the river embankment.

4 SEISMIC INPUT

Since the stability of the particular dam was not considered critical, a detailed seismic hazard analysis study has not been commissioned. Proof of stability is performed solely on the basis of the relevant regulatory building codes and state-of-the-art methodologies. The relevant Code DIN 4149:2005-04 closely follows the specifications of Eurocode EC8 with design spectra considering the subsoil conditions at the various geographical units. In the present study we will use information from both codes in a complementary fashion.

In a first step the subsoil conditions are assessed. According to EC8, Part 1, Table 3.1 the foundation subsoil is classified as ground type C. DIN 4149 includes a refinement of the classification for the foundation soil by introducing sub-categories. Here, sub-category C-S referring to fine grained soil with shear wave velocity ranging from 150 to 350 m/s applies.

Next, we select the seismic input motion. According to the applicable code, here DIN 4149, the site is located in a zone of EMS intensity $7.5 \leq I \leq 8.0$ with peak ground acceleration for sound rock (a_g for class A ground of EC8) equal to

$$a_g = 0.8 \text{ m/s}^2$$

The reference return period in that national seismic building code is equal to 475 years. However, for the particular water-retaining earth structure additional national specifications apply that dictate a higher return period for the design earthquake. The extrapolation to higher return periods requires knowledge of the magnitude recurrence law for the particular region and the magnitude-intensity relationship. To accomplish this task the assistance of a seismologist is needed. EC8, Part 1 offers an alternative by adjusting the importance factor γ_I according to

$$\gamma_I \sim (T_{LR}/T_L)^{-1/k} \tag{1}$$

where T_L is the return period for which the extrapolation is applied, T_{LR} is the reference return period, and $k = 3$.

Here, we select a safety level described by a return period of $T_L = 950$ years, yielding an importance factor

$$\gamma_I = (475/950)^{-1/3} = 1.26$$

This value compares well to the estimate based on empirical seismological relationships for the broader region.

Hence, for the specified safety level the peak ground acceleration is:

$$a_g \cdot \gamma_I = 0.8 \cdot 1.26 = 1.0 \text{ m/s}^2$$

For the purpose of conducting a seismic response analysis of the dam, an accelerogram is required. In the absence of a project-specific seismological study a representative time history is determined using the code SIMQKE, Gasparini and Vanmarcke (1976). The target spectrum is that of the applicable code for the particular subsoil class, as given in Figure 3. This response spectrum considers site effects by means of a soil parameter S. Here $S = 0.75$. The resulting time history is given in Figure 4 with a peak ground acceleration value

$$a_{max,\ freefield} = a_g \cdot \gamma_I \cdot S = 0.8 \cdot 1.26 \cdot 0.75 = 0.75 \text{ m/s}^2$$

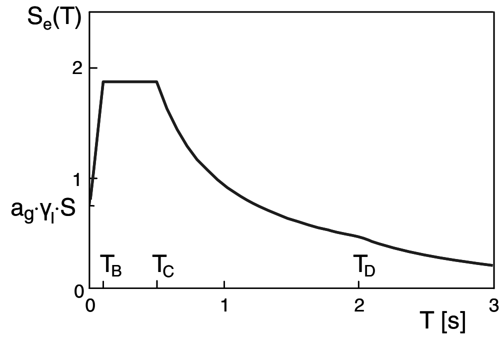


Figure 3. Horizontal elastic response spectrum S_e (T) according to applicable national code (DIN 4149, subsoil class C-S): $S = 0.75$, $T_B = 0.1$ s, $T_C = 0.5$ s, $T_D = 2.0$ s, and 5% damping.

5 DYNAMIC SOIL PROPERTIES

The dynamic soil properties both for the foundation soil and for the inhomogeneous dam material are estimated from empirical correlations. The small strain shear wave velocity for soils spanning a wide range of grain sizes is determined using the empirical relation by Kokusho and Yoshida (1997):

$$v_S = [120 + (420 \cdot U_c / (U_c+1) - 120)\, D_r] \cdot [\sigma_v \cdot \sigma_h / p_0^2]^{0.125} \tag{2}$$

where v_S is given in m/s, U_c is the coefficient of uniformity of the soil, σ_v and σ_h the in-situ vertical und horizontal stress, and

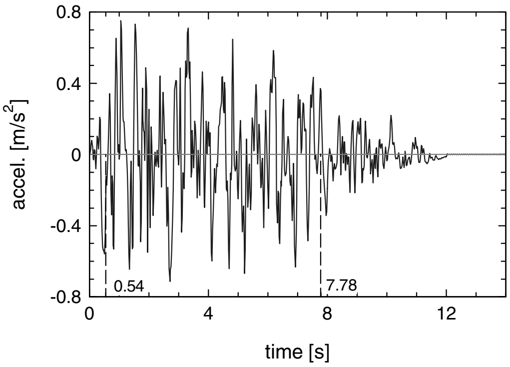


Figure 4. Synthesized design acceleration time-history for safety level corresponding to a return period of 950 years with indication of strong ground motion duration.

p_0 the atmospheric pressure. For the foundation material, on average, $U_c = 30$. The mean in situ stress is estimated at $\sigma_v = 180$ kN/m^2. The respective horizontal stress is determined assuming a coefficient of earth pressure at rest $k_0 = 0.5$. From this, we obtain $v_S = 275$ m/s. The accuracy of the above equation is checked against the well-known equations for sands with a typical value of $U_c = 8$ showing very good agreement.

The associated value for the core material is determined as the average of a sandy gravel with $U_c = 30$ according to equation (2), and of a silt sand with a void ratio $e = 0.4$ according to the well-known equations for normally consolidated cohesive soils equal to 263 m/s.

For the cell material and the sealing silt layer we obtain analogously small strain values of 220 m/s and 240 m/s, respectively.

Reduction factors to account for nonlinear effects are obtained from Table 4.1, EC8, Part 5. For the foundation soil the acceleration level is set equal to the level of seismic input motion while for the dam structure the expected average ground acceleration ratio is estimated at 0.15. This leads to a reduction factor for the shear wave velocity equal to 0.9 for the foundation soil and 0.8 for the dam structure. Damping ratio is also determined from EC8, Part 5 to 3% and 4.5% for the foundation soil and the dam, respectively.

For Poisson's ratio a value of 0.3 is selected for all materials.

6 DYNAMIC RESPONSE ANALYSIS AND SEISMIC COEFFICIENT

The FEM-Code PLAXIS, Brinkgreve and Vermeer (2002), is used for the 2D-analysis that is conducted directly in the time-domain. Since such algorithms use frequency dependent Rayleigh damping formulations, appropriate numerical values have to be determined for the Rayleigh damping coefficients α and β in order to model frequency independent damping. Most commonly they are computed to give the required levels of damping at the resonant frequency of the layered soil profile and the average frequency of the incident ground motion. Here α/β are set equal to 0.30/0.0019 for the foundation, and 0.44/0.0029 for the dam materials. It is inevitable that the response is filtered out in the high frequency range.

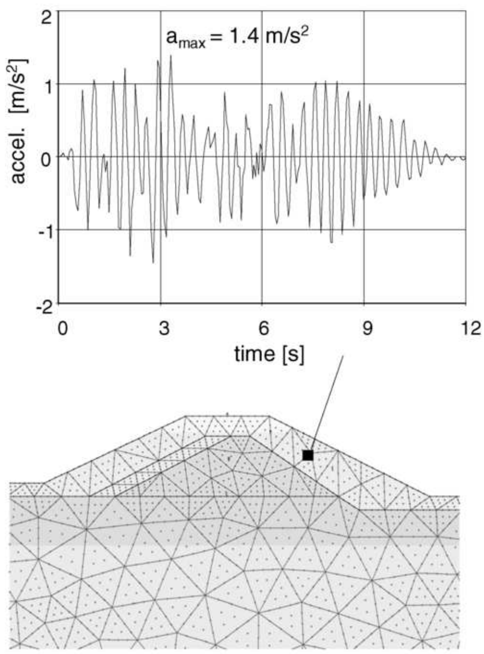
Since the depth of bedrock is unknown, in order to proceed with the dam calculation an appropriate FE-system with a finite thickness has to be established. The motion at the base should yield the spectrum-compatible synthesized motion at the surface. For the FE-model (without dam structure) a depth of 15 m is selected. By deconvolution using SHAKE, Schnabel et al. (1972), a base motion for a 1D-model is first calculated. The resulting time-history is then entered into the 2D-FEM model and the motion at the surface is computed. This process is repeated until the maximum acceleration at the surface of the 2D-FEM model is equal to that of the synthesized spectrum-compatible motion. In order to simulate the 1D conditions the mesh width of the 2D-model was set eight times its height. This iterative procedure yields

$$a_{\text{max, base FEM}} = 0.54 \text{ m/s}^2$$

The subsequent 2D dynamic analysis for the dam structure resting on a 15 m thick soil layer on bedrock yields at the dam crest a peak acceleration value of 1.69 m/s^2, while for the dam slope an average representative value of the peak acceleration is determined:

$$a_{\text{max,dam slope}} = 1.4 \text{ m/s}^2$$

The corresponding accelerogram given in Figure 5 as calculated by 2D PLAXIS shows a different time-history pattern compared to that obtained from the 1D SHAKE free-field analysis, demonstrating the influence of the Rayleigh damping assumption and of the wave absorbing boundary conditions employed along the sides of the mesh.



$$a_{rms} = \sqrt{\frac{1}{t_d}\int_0^{t_d}[a(t)]^2 dt} \approx 0.5 \cdot a_{max}$$

Figure 5. Embankment response and effective acceleration for slope stability analysis.

The effective acceleration value within the dam is taken as the RMS value of the acceleration time history over the strong motion duration t_d. The latter is obtained from a Husid plot as the time required to build-up between 5 and 95% of the total Arias intensity of the record, Trifunac and Brady (1975). Here, $t_d = 7.2$ s, cf. Figure 4, and the calculated reduction factor is for the particular synthesized motion approx. equal to 0.5, Figure 5, yielding an average effective acceleration for the embankment slope

$$a_{\text{dam slope}} = 0.5 \cdot 1.40 = 0.7 \text{ m/s}^2$$

This corresponds to an effective horizontal seismic coefficient k_h= 0.07 for use in quasi-static slope stability calculations.

In order to set-up a simplified procedure applicable to a variety of boundary conditions using the code spectrum as input for the seismic motion, the following method is proposed. Amplification effects in the dam are determined assuming homogeneous dam material with shear wave velocity $v_{S,dam} = 200$ m/s. The eigenperiod of the dam is determined from the following approximate solution, Gazetas (1987):

$$T_{dam} = 2.5 \cdot H / v_{S,dam} = 2.5 \cdot 11.5 / 200 = 0.14 \text{ s} \quad (3)$$

The response spectrum of the applicable code for the site-specific ground conditions yields for this period an amplification factor of A = 2.5, Figure 3, that is herewith assigned to the dam crest. A rough approximation consists in setting the effective amplification over the embankment height equal to the average between the free-field value (A = 1) and that of the crest (A = 2.5), i.e. equal to 1.75. The amplification value obtained is used in the slope stability analysis:

$$a_{max,\,dam} = a_{max,\,freefield} \cdot 1.75 = 0.75 \cdot 1.75 = 1.3 \text{ m/s}^2$$

This value is a good approximation to the value of 1.4 m/s^2 obtained from the dynamic 2D-FEM analysis as given above.

For slope stability calculations EC8, Part 5 recommends the application of a reduction factor of 0.5 to the peak ground acceleration when use is made of simplified pseudo-static analyses. Note that this refers solely to slopes, i.e. it does not consider the amplification effects expected in embankment dams.

7 CONCLUSIONS

The analysis procedure presented herein allows for consideration of very deep sedimentary strata, as typically encountered in graben structures. In the form presented herein, the simplified method is applicable to low-rise dams (e.g. up to 15 m) in regions of medium seismic hazard. Both, the more rigorous as well as the simplified method for estimating an effective acceleration for pseudo-static slope stability calculations may be extended to include larger dams and stronger seismic motions.

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