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# Seismic analysis and design of rockfill dams in the Lower Thjorsa River, Iceland

## Analyse sismique et conception de barrages de gravier sur le cours inférieur de la rivière Thjorsa

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ABSTRACT

In order to harness the hydropower potential of the lower part of the Thjorsa River in South Iceland, two hydropower schemes are proposed in the area, one near the Urridafoss rapids and the other near the Nupur hill. Three different types of rockfill dams for the two schemes are studied in this paper. Two of the dam types have a central core, one made of loessoidal soil and the other of asphaltic concrete. The third dam type has a concrete face on its upstream side. The proposed building sites are in a well known earthquake region called the South Iceland Seismic Zone. In June 2000 two major earthquakes of magnitude 6.6 and 6.5 occurred in the near vicinity of the proposed sites with a maximum registered peak ground acceleration of 0.84g. The dam design as addressed in this paper is twofold. Firstly, the generalized method of slices is used to analyse the stability of the dam slopes and to determine how steep they can be. Secondly, the Finite Element method is used to perform dynamic analysis to determine the permanent displacements due to a proposed earthquake loading as well as liquefaction potential. The largest permanent deformations developed during earthquake will be approximately 0.4 m but should not jeopardize the overall safety of the dam, although some local damage could occur. Liquefaction of the core material is not plausible.

RÉSUMÉ

Dans le but d'exploiter le potentiel hydroélectrique du cours inférieur de la rivière de Thjorsa, dans le sud d'Islande, deux projets d'aménagement sont proposés dans la région; l'un près de la chute Urridafoss, l'autre près de la montagne Nupur. Trois différents types de barrage en gravier sont étudiés dans cet article. Deux de ces types ont un noyau central, l'un fait de terre "loessoidalle", l'autre de béton asphalté. Le troisième type de barrage est couvert de béton sur la face amont. Les sites de l'aménagement sont dans une région de séismes bien connue, la zone sismique sud-islandaise. En juin 2000 deux puissants tremblements se sont produits dans cette zone, de magnitude 6.6 et 6.5, ayant l'épicentre près des sites proposés. L'accélération maximale du sol était mesurée à 0.84g. La conception du barrage se fait sur deux plans, comme exposé dans cet article. Premièrement la méthode généralisée de tranches sert à analyser la stabilité des flancs du barrage et à déterminer la pente possible. En second lieu, la méthode par éléments finis est utilisée pour accomplir l'analyse dynamique, dans le but d'établir les déplacements permanents dus aux heurts d'un séisme éventuel et aussi le risque de liquéfaction. Les déformations permanentes maximales résultant d'un séisme seront d'environ 0.4 m mais ne mettraient pas en risque la sûreté générale du barrage, même si des dégâts localisés pourraient se produire. Liquéfaction du matériau du noyau est peu probable.

## 1 INTRODUCTION

Two hydropower schemes are proposed in the lower part of the Thjorsa River in South Iceland, the Urridafoss project and the Nupur project. They are currently in a design and optimization phase. The hydropower potential in the upper part of the river has already been developed to a large extent. A system of reservoirs has been built to account for seasonal flow variations.

The estimated harnessed flow of Thjorsa at Nupur is 300 $m^3/s$, with a head of 56 m. Total installed capacity will be about 120 MW (Almenna Consulting Ltd., 2003). Two different layouts are being studied, a one step scheme and a two step one. The estimated size of the intake reservoirs for the two power plants is 4.6 $km^2$ and 6.7 $km^2$, with total dam lengths of ca. 2 km for each reservoir. The dams are generally low, with a maximum height of about 15 m.

At Urridafoss, the estimated harnessed flow is 344 $m^3/s$, with a head of about 40 m (Hnit Consulting Ltd., 2003). Installed capacity will be 120 MW. The estimated size of the intake reservoir is 12.5 $km^2$, with a total dam length of 4.3 km. The maximum height of the dam is approximately 14 m.

The building sites are in a well known earthquake region called the South Iceland Seismic Zone. Two major earthquakes of magnitude 6.5 and 6.6 ($M_W$) occurred near the project sites in June 2000. Peak ground acceleration of 0.84g was measured during the earthquakes.

The analysis and design of three different types of rockfill dams for the two hydropower schemes is studied in this paper, especially with regard to seismic effects. Two of the proposed dam types have a central core, one made of loessoidal soil and the other of asphaltic concrete. The third dam type has a concrete face on its upstream side, see Figure 1.

The generalized method of slices is used to analyse the stability of the dam slopes, determining how steep they can be. The Finite Element method is thereafter used to perform dynamic analysis, determining permanent displacements due to earthquake and liquefaction potential in the core material.

## 2 SEISMIC SETTING

Iceland is located on the Mid Atlantic Ridge, on the boundary of the American and the Eurasian Plates. Across Iceland from southwest to the north the plate boundary is displaced to the east through two major fracture zones, the South Iceland Seismic Zone (SIZS) and the Tjornes Fracture Zone in the north (TFZ), see Figure 2 (Einarsson, 1991). The largest historic earthquakes in Iceland have occurred within these zones and have exceeded magnitude 7.

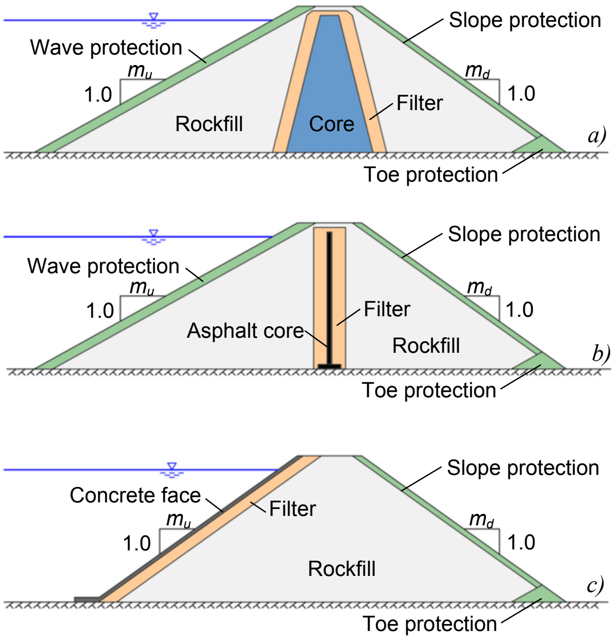


Figure 1. Typical cross sections for the proposed dams in the Lower Thjorsa River. *a)* Rockfill dam with a central core of loessoidal soil. *b)* Rockfill dam with a central core of asphaltic concrete. *c)* Rockfill dam with an upstream concrete face.

The most destructive earthquakes in Iceland have occurred within the SISZ, and major earthquake sequences have affected the area with recurrence intervals between 45 to 112 years. In June 2000, two earthquakes of magnitude 6.5 and 6.6 occurred in the SISZ, near the proposed project sites in the Lower Thjorsa River Basin, see Figure 3.

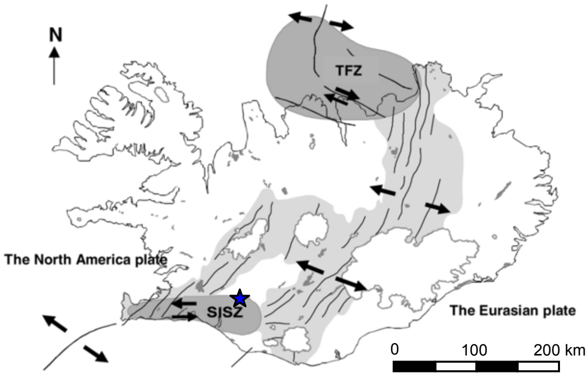


Figure 2. The plate boundaries in Iceland and the two major seismic zones, the SISZ and the TFZ. The approximate location of the Lower Thjorsa hydropower projects is marked with a star. After Bessason and Kaynia (2002).

The river basin, where the dams are to be built, is covered with a 15 – 25 m thick layer of 8000 years old lavaflow resting on finiglacial alluvial and marine sediments of loose sand and gravel with low stiffness compared with the lava. Considerable site amplification has been recorded on the lava, due to the underlying sediment layer (Bessason and Kaynia, 2002).

Eurocode 8, EC8, which applies to the design and construction of buildings and civil engineering works in seismic regions, is used to determine the seismic loading for the dam design, together with the Icelandic National Annex which complements the standard. The annex gives a reference peak ground acceleration $a_{gR}$ = 0.4g, corresponding to a $T$ = 475 years return period. The ground types described in EC8 do not comply with the ground conditions at the building sites in the Thjorsa River area. Acceleration recordings have however shown that a soil factor $S$ = 1.5 is adequate (Bessason and Kaynia, 2002).

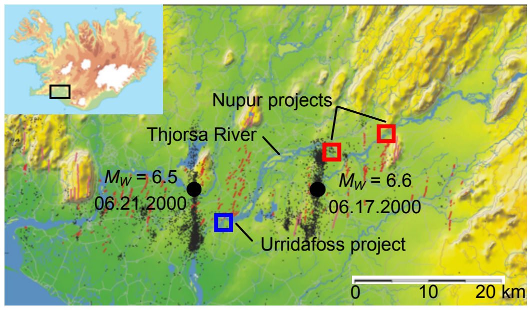


Figure 3. The South Iceland Lowlands. The epicentres of the June 2000 earthquakes are marked with black dots, known faults with red lines. After Almenna Consulting Ltd. (2003).

Figure 4 shows an elastic response spectrum for the site, according to EC8.

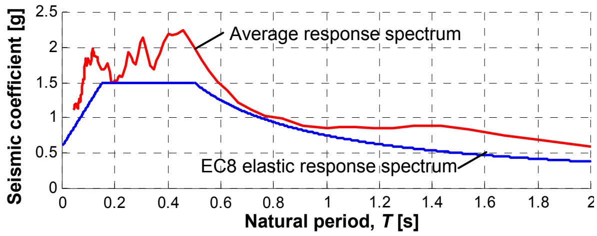


Figure 4. Elastic response spectrum according to EC8 together with the average response spectrum (damping ratio $\xi$ = 5%) of three scaled time histories used for the dynamic analysis.

The dynamic analysis of the dams is performed using three different time histories recorded in the June 2000 earthquakes near the proposed building sites, see Figure 5. The time histories are scaled so their average response spectrum fits with the EC8 elastic response spectrum, see Figure 4.

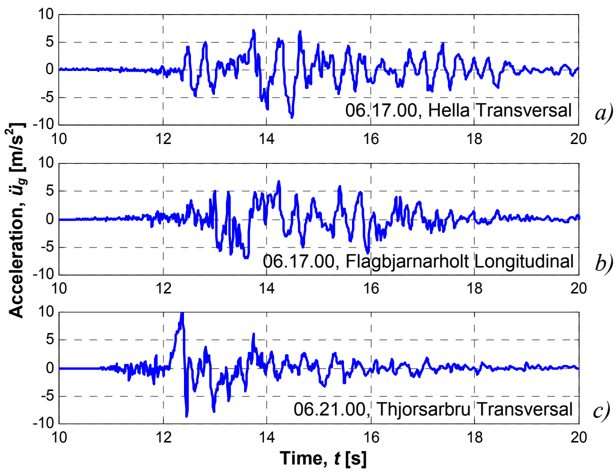


Figure 5. Three measured acceleration time histories, scaled to fulfil EC8 requirements.

## 3 SOIL PROPERTIES

Extensive testing procedure has been carried out on the rockfill and the core material, including both static and dynamic triaxial testing. Based on that the material parameters according to the Mohr-Coulomb soil model have been evaluated, see Table 1.

The maximum shear stiffness of the rockfill material is estimated using the equation

$$G_{max} = \frac{625 \cdot OCR^k \cdot p_a^{1-n} \cdot \left(\sigma_m^{'}\right)^n}{e^{1.3}} = 18000 \cdot \sqrt{\sigma_m^{'}} \qquad (1)$$

where $OCR = 1.0$ is the overconsolidation ratio, $p_a = 100$ kPa is a reference pressure and $\sigma_m^{'}$ is the mean principal effective stress. The stress exponent is taken as $n = 0.5$ (Kramer, 1996).

The maximum shear stiffness of the loessoidal soil used as core is material is estimated according to

$$G_{max} = 3.230 \frac{\left(2.973 - e\right)^2}{1 + e} OCR^k \cdot \sqrt{\sigma_m^{'}} = 4900 \cdot \sqrt{\sigma_m^{'}} \qquad (2)$$

where $OCR = 1.0$ as before.

Table 1. Material properties for rockfill and core used in the analyses.

| | | *Rockfill* | *Core* | |
|---|---|---|---|---|
| Dry unit weight: | $\gamma_{dry}$ | 20.0 | 12.4 | kN/m$^3$ |
| Saturated unit weight: | $\gamma_{sat}$ | 22.5 | 18.1 | kN/m$^3$ |
| Void ratio: | $e$ | 0.45 | 1.17 | - |
| Friction angle: | $\phi$ | 46.0 | 40.0 | ° |
| Dilatancy angle: | $\psi$ | 16.0 | 10.0 | ° |
| Cohesion: | $c$ | 0 | 0 | kPa |
| Poisson's ratio: | $\nu$ | 0.2 | 0.3 | - |
| Hydraulic conductivity: | $k$ | $10^{-2}$ | $10^{-7}$ | m/s |

Dynamic triaxial testing programs have been carried out to estimate the cyclic stress ratio *CSR* for the loessoidal soil proposed to use as core material at the Urridafoss and Nupur sites, in order to estimate the liquefaction resistance of the materials (Almenna Consulting Ltd., 2002). A part of the results for the Nupur area are shown in Table 2. The results are corrected to apply to field conditions according to

$$CSR_{field} = \frac{0.9 \cdot c_r \cdot CSR_{tx}}{\gamma} \qquad (3)$$

with the correction factor $c_r \approx 1.0$ for well compacted materials and $\gamma = 1.25$ is a partial safety factor according to EC8.

Table 2. Dynamic triaxial test results for four soil samples from the Nupur area. Cyclic stress ratio and number of cycles $N$ at 10% double amplitude strain, together with corrected cyclic stress ratio.

| *Sample* | $CSR_{tx}$ | $N$ | $CSR_{field}$ |
|---|---|---|---|
| 1 | 0.406 | 546 | 0.292 |
| 2 | 0.507 | 163 | 0.365 |
| 3 | 0.608 | 106 | 0.438 |
| 4 | 0.710 | 56 | 0.511 |

## 4 DESIGN OF CROSS SECTIONS

The critical steepness of the dam slopes is determined using the generalized method of slices, utilizing the Slope/W software from GeoSlope. The Morgenstern-Price method was used with a half-sine side function where a horizontal pseudo static earthquake force is applied to each slice, see Figure 6.

The safety factors for circular and linear slip surfaces in both the upstream and the downstream slopes of the dams were determined for a full reservoir. Rapid drawdown of the reservoir was also considered. No partial factors are applied to either the soil parameters or the loads, thus the calculated safety factor is the total safety factor.

According to Seed (1979) an acceptable design criterion for an embankment dam subjected to seismic loading is a pseudo static seismic safety factor of 1.15 for a pseudo static seismic coefficient $k = 1.0$, for a magnitude 6.5 event. This applies to cases where crest acceleration does not exceed 0.75g which is not the case here where ground peak ground acceleration of 0.84g has been recorded. To account for the expected high acceleration a seismic coefficient $k = 0.2$ is selected, which is half the reference peak ground acceleration.

A downstream slope of 1:1.4 will have a pseudo static safety factor $FS > 1.15$ for all the three dam types for $k = 0.2$, see Figure 7. Similar analyses show that an upstream slope of 1:1.8 is adequate for the two dam types with the central core and 1:1.4 for the dam with the concrete face.

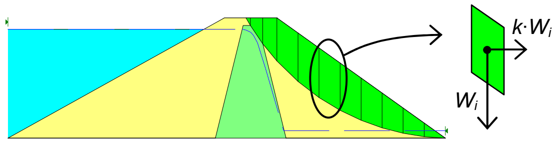


Figure 6. A typical slip surface in the downstream slope of a dam section with a central soil core. A horizontal pseudo static seismic force equal to $k \cdot W_i$ is applied to each slice, where $W_i$ is its weight.

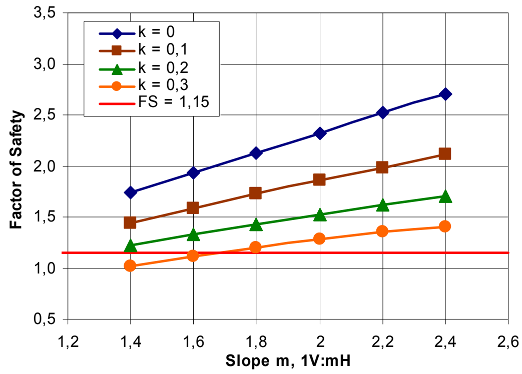


Figure 7. Factor of safety $FS$ for a slip surface in the downstream slope as a function of slope $m$ for four values of the seismic coefficient $k$.

## 5 DYNAMIC ANALYSIS

To check whether the cross sections designed with the slice method can withstand a real earthquake without developing too large deformations a dynamic analysis was performed. This was done using the PLAXIS software, which is a Finite Element (FE) program. Figure 8 shows the 2D model used for the seismic analysis of the rockfill dam with the loessoidal core.

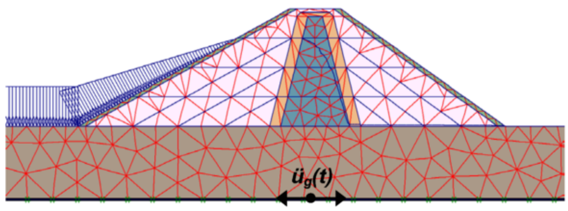


Figure 8. A 2D FE model of a dam cross section. The base of the dam extends one dam width to each side. At the vertical boundaries of the base absorbent boundaries were installed. Soil stiffness increases with depth so the cross section is divided into layers with increasing stiffness.

Shear stiffness of soils generally decreases as shear strains increase. High shear strains can develop in dam sections during earthquakes. It is necessary to take this into account in a realistic analysis. The level of shear strains to be expected in this case is quite high, and according to EC8 and from the triaxial test results it was found appropriate to reduce the stiffness of the soils down to 10% of the maximum shear stiffness as expressed in equations (1) and (2). Material damping is also strain dependent and increases with increased shear strain. A $\xi = 13.5\%$ damping ratio was used.

Permanent deformations of the three dam types considered are shown in Figure 9, for the time series in Figure 5a. The estimated largest deformations are about 0.4 m, but the nature of the deformations seems to bee such that they do not jeopardize the overall safety of the dams.

A nonlinear analysis using a hardening soil model was also carried out for the rockfill dam with the loessoidal core. This resulted in about 50% smaller permanent deformations. It is therefore concluded that the results obtained using the Mohr-Coulomb model are a conservative estimate of the deformations.

Considering the upper two cross sections in Figure 9 it seems that the dam with the loessoidal core is less deformed after the earthquake simulation. Local damage at the top of the asphaltic core above the water table could occur. Similarly, large deformations beneath the top part of the concrete face on the third dam type will probably cause some damage in the concrete slab, but mostly above the water table.

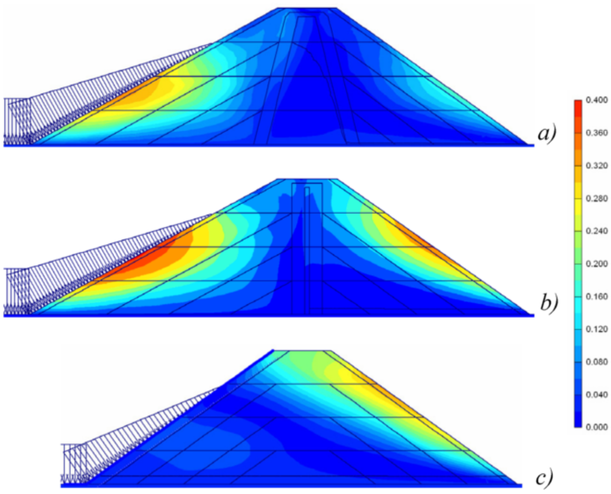


Figure 9. Permanent displacement after applying seismic load, coloured scale in meters. *a)* Rockfill dam with a loessoidal core. *b*) Rockfill dam with an asphaltic core. *c)* Rockfill dam with a concrete face.

Liquefaction occurs in dense materials where pore water pressure can build up during cyclic loading. The only material in the three dam types believed to be susceptible for liquefaction is the loessoidal soil in the core of the first dam type. Figure 10 shows shear stress as a function of time for three points in the core, for the acceleration time series shown in Figure 5c.

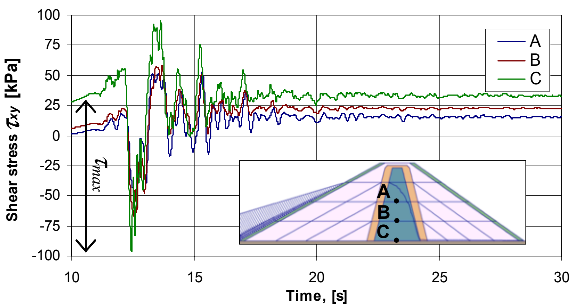


Figure 10. Shear stress development in the core material during earth-quake loading, corresponding to the last time series in Figure 5.

The largest amplitude of the shear strain, $\tau_{max}$, is used to estimate the equivalent cyclic shear stress

$$\tau_{cyc} = 0.65 \cdot \tau_{max} \tag{4}$$

which corresponds to a certain number of equivalent load cycles, $N_{eq}$ = 10 for a size 7.0 earthquake (Seed et al., 1975). The equivalent cyclic shear stress is then converted into the cyclic stress ratio (*CSR*) as

$$CSR = \frac{\tau_{cyc}}{\sigma'_{v0}} \tag{5}$$

where $\sigma'_{v0}$ is the initial vertical effective stress.

The results from the dynamic analysis are shown in Figure 11, for the three points A, B and C and the three time series. The blue line represents corrected triaxial test results. All the results from the dynamic analysis fall beneath that line, so liquefaction of the core material is not plausible.

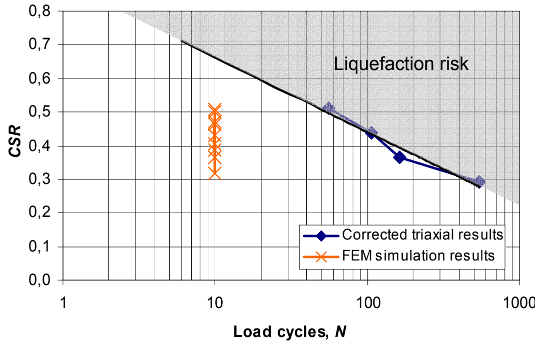


Figure 11. Cyclic stress ratio as a function of equivalent load cycles. Results from dynamic triaxial testing are shown along with FE simulation results.

## 6 CONCLUSION

Dam design in the Lower Thjorsa River Basin is governed by the region's seismic activity. The seismic effects can be accounted for with appropriate design measures. A dam with an upstream slope of 1:1.8 and a downstream slope of 1:1.4 will resist the expected earthquake loading in the region. The largest permanent deformation developed during an expected earth-quake will be approximately 0.4 m. That should not jeopardize the overall safety of the dam, although some local damage could occur. Liquefaction of the core material is not plausible.


## ACKNOWLEDGEMENTS

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