Analysis of a geomembrane face rockfill dam during earthquake loading

Analyse d'un barrage de roches à parement de géomembrane durant une charge sismique

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ABSTRACT

The paper represents an analysis of the suitability of a geomembrane face rockfill dam in highly seismic area. This rockfill dam type has a geomembrane for watertightness on its water side with geotextile and thin layers of filter materials on both sides to protect it. The rockfill dam is a part of three proposed hydropower schemes in the lower part of the Þjórsá glacial river in SW-Iceland. The hydropower plants and all related structures are situated in the South Iceland Seismic Zone where two major earthquakes of magnitude 6.6 and 6.5 (M_W) with a registered peak ground acceleration of 0.84g occurred in June 2000. Further a 6.3 event occurred in 2008. The analysis of the dam is split into two parts. Firstly the stability of the dam is determined with the method of slices. Secondly the finite element method is used to estimate the deformations induced by earthquake loading where the behaviour of the geomembrane and the surrounding soil is given special attention in regards of possible slipping. The main result revealed that the proposed dam will withstand the earthquake load without major damage or total collapse. Largest relative deformations along the geomembrane, caused by the applied earthquake loads, have been estimated to be approximately 60 cm. These deformations are rather large but since they are local, the total stability of the dam is not threatened. A necessary repair would though have to be carried out in the occurrence of such an event.

RÉSUMÉ

Ce papier présente une analyse de la pertinence d'un barrage de roches à parement de géomembrane dans une région de haute sismicité. Ce type de barrage de roches a une géomembrane pour assurer l'étanchéité sur son parement amont, avec du géotextile et de minces couches de filtre sur les deux côtés pour la protéger. Le barrage de roches fait partie de trois projets hydroéléctriques dans la basse partie de la rivière glaciale Þjórsá dans le sud-ouest d'Islande. Les centrales hydroéléctriques et tous les structures complémentaires se trouvent dans la Zone Sismique d'Islande du Sud où deux séismes violents, de magnitude 6,6 et 6,5 (M_W), se sont produits en juin 2000. L'accéleration maximale enrégistrée était de 0,84g. L'analyse du barrage est divisée en deux parties. En premier lieu, la stabilité du barrage est déterminée par la méthode des tranches. En second lieu, la méthode des éléments finis est utilisée pour estimer les déformations provoquées par la charge sismique, en prêtant une attention spéciale au comportement de la géomembrane et du sol environnant pour détecter un glissement éventuel. Le résultat principal démontre que le barrage proposé résistera au charge sismique sans effondrement totale, ni même dégâts importants. Les déformations relatives les plus grandes sur la géomembrane, produites par les charges sismiques, sont estimées à environ 60 cm. Ces déformations sont assez larges mais étant bien circonscrites la stabilité totale du barrage n'est pas menacée. Dans un tel cas une réparation s'imposerait.

Keywords : Dams, design, geomembranes, earthquakes

1 INTRODUCTION

Three hydropower schemes, the Hvammur project, the Holt project and the Urriðafoss project, are being considered in the lower part of the Thjorsa River in South Iceland. The project area is located within the South Iceland Seismic Zone (SISZ) which is the most active seismic zone in the country. In the summer of 2000 two earthquakes of magnitude 6.6 and 6.5 (M_W) occurred in the area. Further a 6.3 event occurred in 2008. Therefore earthquake load is considered critical in the design of dams and other constructions in this area.

The head of the three schemes is between 18 - 32 m with a harnessed flow of 300 – 340 m^3/s. Their total installed capacity is of about 120 MW. The land at the project sites is relatively flat, so the dams need to bee quite long, with total dam lengths of ca. 2 km for each reservoir. The intake reservoirs are ranging from 4.6 to 12.5 km^2. The dams are generally low, with a maximum height of about 17 m where they cross the river and decrease then gradually as they form the reservoirs.

The lack of availability of good quality dense core material for central earth core dams have made it necessary to look at other alternatives of dam types. Concrete face earth dams and dam replacing the central core with asphalt concrete are considered to expansive as the structures are rather low (Arnorsson and Erlingsson 2005). One alternative, studied here, is to use a geomembrane in the upstream part of the structure. Therefore, the behaviour of a geomembrane face rockfill dam for the Hvammur project with regard to seismic effects is studied in this paper. The geomembrane, located at the dam's upstream part, is waterproof and therefore most of the dam is unsaturated. Friction between the geomembrane and the soil is highly important in this dam type in regards of possible slipping. The finite element method is used to analyse the dynamic behaviour of the structure for estimating possible permanent deformations.

2 GEOLOGICAL CONDITIONS

The hydropower projects are located in a highly seismic area in the southern part of Iceland. This area is called the South Iceland Seismic Zone, (see Figure 1). In the year 2000 two earthquakes of magnitude 6.5 and 6.6 occurred in this zone (Sigbjörnsson 2002). Further a 6.3 event occurred in 2008 in the western part of the area.

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The dams are to be built in the Thjorsa river basin which is covered with 15-25 m thick layer of ca. 8000 years old lava resting on sediments of loose sand with low stiffness compared with the lava. Eurocode 8 (EC8) is used to determine the seismic load for the design, resulting in a reference peak ground acceleration of $a_{gR} = 0.4$.

The dynamic analysis is performed by using three different time histories recorded during the June 2000 earthquakes, see Figure 2. The time histories have been scaled so their average spectrum fits with the EC8 response spectrum. A soil factor S = 1.5 was considered suitable for these large earthquakes (Bessason and Kaynia 2002).

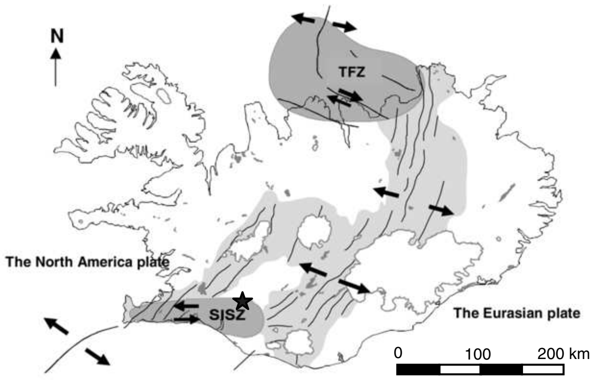

Figure 1. The plate boundaries in Iceland and the two major seismic zones, the SISZ and the TFZ. The approximate location of the Lower Thjorsa hydropower projects is marked with a star. After Bessason and Kaynia (2002).

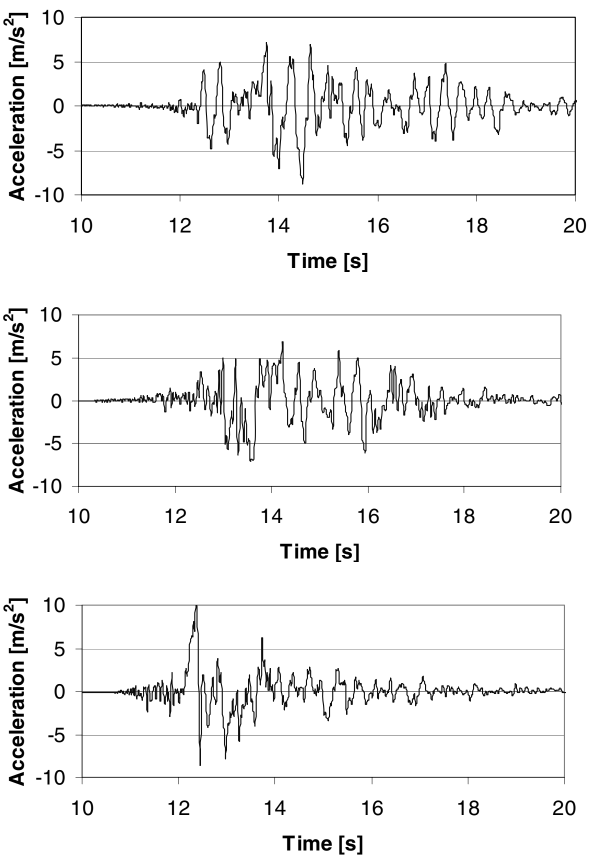

Figure 2. Three scaled time histories. The time histories from 2000 were scaled with the following factors a) Hella, June 17th k = 1.87, b) Flagbjarnarholt, June 17th k = 2.26 and c) Thjorsa Bridge, June 17th k = 1.34.

3 GEOMEBRANE FACE ROCKFILL DAM

Geomembranes have been used in earthfill dams since the year 1959. They are used for water tightness and as such they are quite effective since they have very low permeability. Polyvinyl chloride (PVC) and High density polyethylene (HDPE) are the most common geomembranes used in dam structures. These geomembranes are generally stronger and have higher friction angle than other membranes such as Chlorosulfonated polyethylene (CSPE) and Ethylene propylene diene monomer (EPDM). When a geomembrane is placed in a slope (f. ex. in a dam) a high friction angle between the membrane and its surrounding soil is an important property (Sembenelli and Rodrigues 1996). Figure 3 shows a typical layout of a geomembrane face rockfill dam.

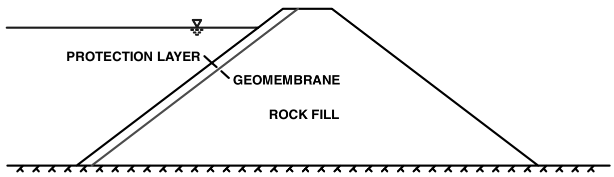

Figure 3. Geomembrane face rockfill dam.

Friction between a geomembrane and its surrounding soil is highly dependent on the surface characteristics of the geomembrane. It is also dependent on various properties of the soil. Therefore it is necessary to have good understanding of the interaction between the soil and the membrane. A friction coefficient between the two layers can be defined as

$$E = \frac{\tan(\delta)}{\tan(\phi)} \qquad (1)$$

where δ is the friction angle between a geomembrane and its surrounding soil and φ is the friction angle of the soil. Here a friction coefficient was given values between 0.65 and 1.0 to investigate the dam's sensitivity to it during a seismic analysis. Table 1 shows friction angle, δ, and frictional coefficient, E, between different types of geomembranes and three types of sand. Later, in the dynamic analysis, the values for PVC geomembrane were used as a base for the analysis where a fixed value of the friction coefficient E = 0.85 was used.

Table 1. Frictional characteristics between different geomembranes and various soils. After Martin et al (1984).

	Soil type					
	Concrete Sand (ϕ = 30°)		Ottawa Sand (ϕ = 28°)		Mica Schist Sand (ϕ = 26°)	
Geomembrane	δ	E	δ	E	δ	E
EPDM-R	24°	0.77	20°	0.68	24°	0.91
PVC						
Rough	27°	0.88	-	-	25°	0.96
Smooth	25°	0.81	-	-	21°	0.79
CSPE-R	25°	0.81	21°	0.72	23°	0.87
HDPE						
Smooth	18°	0.56	18°	0.56	17°	0.63

The most common earthfill dams in Iceland are Earth Core Rockfill (ECR) dams. However, if it is not possible to find suitable material for the core in an economic distance from the dam site other options are looked for. Alternative solutions are to use unconventional core materials or a watertight layer is put on the waterside of the dam for waterproofing. In this project a geomembrane face rockfill dam was studied. It has a geomembrane for waterproofing on its waterside. The geomembrane can be placed in the middle of the dam as well. However it is generally not considered as effective since it can be hard to access the membrane to fix possible leaking of the dam.

The membrane is the critical layer of the structure and it is important to protect it from puncturing. Therefore it was proposed to place a geotextile on each side of the membrane. This is of special interest in seismic areas were slippage might occur along the membrane during a seismic event. The geotextile protects the geomembrane from puncturing against sub-angular particles and minimizes the danger of tearing or puncturing during transportation and placement, see Figure 4 (Akber et al 1985).

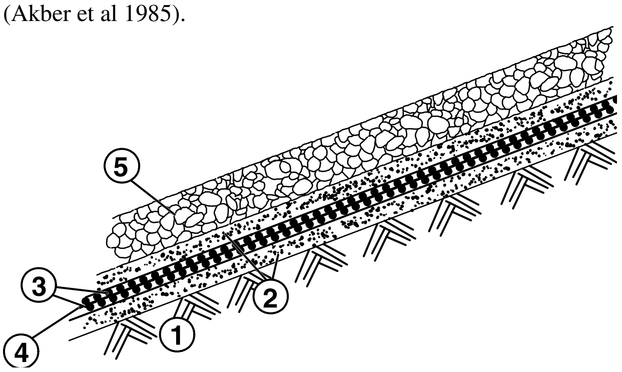

Figure 4. Details of the geomembrane facing. 1, Earthfill; 2, crushed gravel; 3, non-woven geotextile; 4, geomembrane; 5, rockfill. After Girard et al (1990).

4 DESIGN OF CROSS SECTION

The design of cross sections deals mostly with determining the critical slope. Here the method of slices was used utilizing the software Slope/W from GeoSlope. The Morgenstern-Price method was used with a half-sine side function. A horizontal pseudo static seismic force, F_h, was applied to the structure

$$F_h = k \cdot W \tag{2}$$

where k is seismic coefficient. A seismic coefficient k = 0.2, which is half the reference peak ground acceleration, was chosen in this case [8]. According to Kjærnsli et al. (1992) a safety factor FS = 1.15 is suitable in the dam design process for earthquake load (Kærnsli et al 1992).

The geomembrane is modelled as a thin soil layer having the same properties as the soil lying next to it except that it has a reduced friction angle. The shape of the critical slope on the waterside is such as shown on Figure 5 while the shape of the critical slope on the downstream is circular.

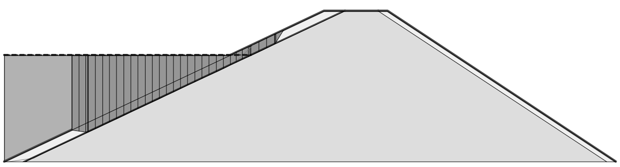

Figure 5. A typical slip surface in the upstream slope of the dam.

The analysis was carried out using different friction coefficients and side slopes. Results can be seen on Figure 6. For a friction coefficient, E = 0.85, which seems a proper value for PVC geomembrane (see Table 1) an upstream slope of 1:2.1 has a factor of safety FS > 1.15. A downstream slope of 1:1.5 was found to be suitable in the analysis. These side slopes were used in the dynamic analysis.

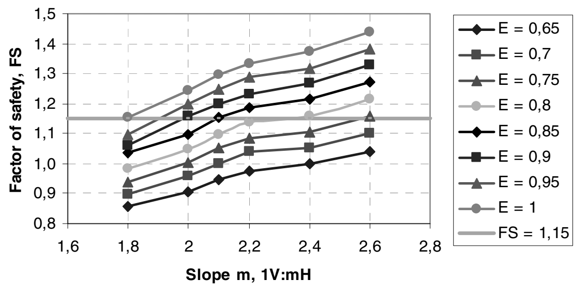

Figure 6. Factor of safety as a function of side slope for different values of friction coefficient of slip surface.

5 DYNAMIC ANALYSIS

A dynamic analysis was thereafter carried out to study the behaviour of the structure during a dynamic event. The FE-program PLAXIS was used with a Mohr-Coulomb soil model, see Figure 7. An interface element, which allows different movements on each of its side, was used to model the behaviour between the geomembrane and the ambient soil. The earthquake load (as a time history) was applied to the base of the model. At the vertical boundaries of the model absorbing boundaries were installed to prevent the earthquake waves from rebounding. The dam considered was 17 m high. Table 2 shows the material properties used in the analysis.

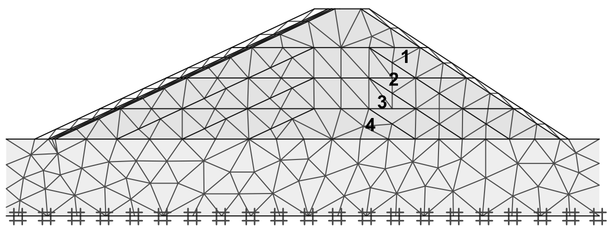

Figure 7. An FE model of the dam. The dam structure was divided into four layers (the numbers on the figure) with increasing shear stiffness as the shear stiffness of soils increases with depth.

Table 2. Material properties for the rockfill.

Dry unit weight	γ_{dry}	20.0	kN/m^3
Saturated unit weight	γ_{sat}	22.5	kN/m^3
Void ratio	e	0.45	
Cohesion	c	1.0	kPa
Friction angle	ϕ	46	°
Dilatancy angle	ψ	16	°
Poisson´s ratio	ν	0.2	

The shear stiffness of the soil is dependent on the state of stress as follows (Kramer 1996)

$$G_{max} = 625 \cdot \frac{1}{e^{1,3}} \cdot OCR^k \cdot p^{1-n} \cdot \left(\sigma_m^{'}\right)^n = 18000 \cdot \sqrt{\sigma_m^{'}} \tag{3}$$

where e is the void ratio of the material, OCR = 1.0 is the overconsolidation ratio, p_a = 100 kPa is a reference pressure and $\sigma_m^{'} = \left(\sigma_1^{'} + 2\sigma_3^{'}\right)/3$ is the mean principal effective stress. The value for n is taken as 0.5. The increase in shear stiffness with increasing effective stress was taken into account by dividing the dam structure into four layers with increasing stiffness. Shear stiffness of soils is further dependent on the shear strains in the way that it decreases as the strains increases. Based on EC8 guidelines the value of the stiffness was reduced to 10% of the stiffness found in equation (3).

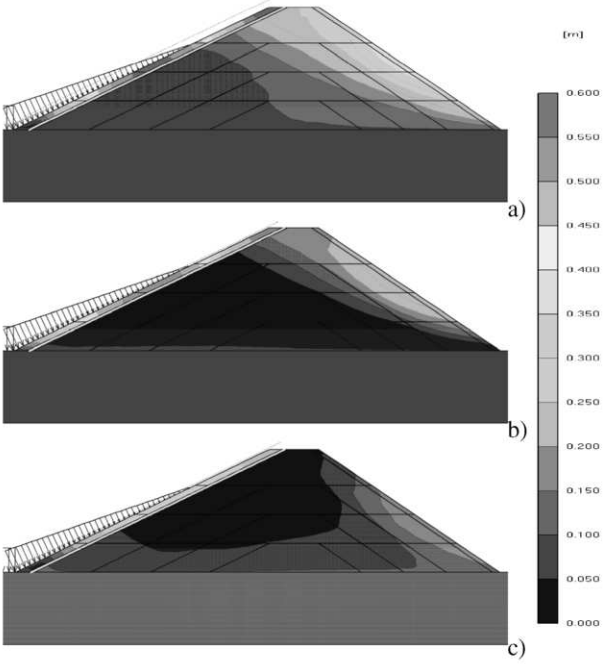

Figure 8. Plastic deformations. Scale is in meters. a) Time series from Hella. b) Time series from Flagbjarnarholt. c) Time series from Thjorsa bridge.

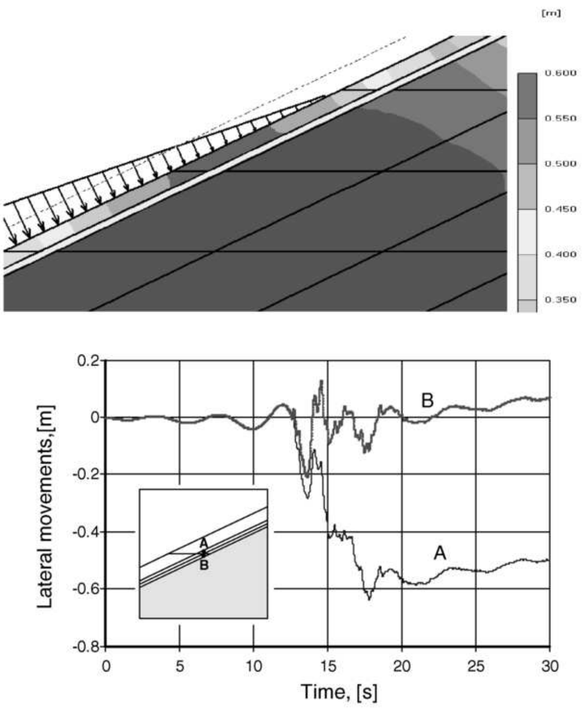

Figure 9. a) Close up of the largest deformation from the time history at Hella. b) History of the lateral movement of two points A and B on each side of the geomembrane during the earthquake loading.

Accumulated permanent deformations at the end of the dynamic analyses using three time histories are shown on Figure 9. The scale represents displacements in meters. A 2.5 mm thick PVC geomembrane with a frictional coefficient E = 0.85 was used. The largest deformations are due to slip between the membrane and its surrounding soils. These deformations are local and appear on the waterside of the dam where the geomembrane is located. The time series at Hella (see Figure 2a) gave the largest deformations, with a peak value of about 60 cm, see Figure 9. These deformations are achieved because the soil slips on the geomembrane during the earthquake load. It is believed that these deformations will not affect the total stability of the dam. The membrane will probably be locally punctured by sharp particles and therefore no longer watertight but the estimated tension forces in the membrane were small and much lower than the tension strength of the membrane. As the membraine is placed close to the surface makes it possible to carry out necessary repair or even replacing the membraine with a new one. No effort was made here to estimate possible piping followed such an event. A separate analysis is needed for that.

6 CONCLUSIONS

The paper discuss the suitability of using a geomembrane face rockfill dam as a part of a hydropower scheme in highly seismic area. To withstand the seismic loading applied to the structure, the upstream slope has to be 1:2.1 and the downstream slope 1:1.5. Applying a registered seismic time histories and using a friction coefficient E = 0.85 between the geomembrane and the surrounding soil the largest plastic deformation was estimated to be approximately 60 cm. These deformations are local and appear on the waterside of the dam where the geomembrane is located. It is believed that these deformations will not threaten the total stability of the dam since they are local. In the occurrence of such an event it would be necessary to carry out a repair. The behaviour of the slip movements is highly dependent on the friction coefficient. It is therefore important to carry out detailed study of the proposed friction coefficient to confirm the observed behaviour of the structure in this study.

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