On the Seismic Design History and Analysis of the Desjararstifla Dam in Iceland

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SUMMARY:
The design history of the Desjararstifla dam is somewhat unusual in that the seismic design criteria were reviewed during the construction phase based on new geological findings in the dam area. This resulted in earthquake motions to be considered in further analysis of the dam conducted in a program using the equivalent linear model to describe the soil dynamic behaviour. Due to lack of information on the dynamic seismic properties of the dam material, the maximum shear modulus, G_{max}, was estimated by several different approaches aiming at defining the pertinent lower and upper boundaries, representing dams with different stiffness and fundamental periods. Consequently, response in different frequency ranges for the same earthquake motions is observed indicating the sensitivity of the dam's seismic response. Results from the seismic analysis indicated adequate safety of the dam subjected to the design earthquake motion and acceptable permanent deformations.

Keywords: earth-rockfill dam, seismic analysis, dam design; equivalent linear model;

1. INTRODUCTION

The Desjararstifla dam is a part of the 690 MW Kárahnjúkar hydroelectric project (KHP) located in north-east Iceland and is one of three dams retaining the main storage, Hálslón reservoir. (See fig. 1.1). The seismic design history of the dam is somewhat unusual in that the seismic design criteria were reviewed during the construction phase based on new geological findings in the dam area. This resulted in review of the seismic analysis of the dam conducted early on in the design phase.

Figure 1.1. The reservoir area with the three dams, respectively from left; the Desjarárstífla dam, the Kárahnjúkar dam and the Sauðárdalsstífla dam. Hálslón reservoir at about FSL 625 m a.s.l. (fig. Landsvirkjun)

The seismic design history of the Desjarárstífla dam is outlined in this paper. The seismic setting is reflected on considering both the original and revised seismic design critera. Then the main features of the dam are presented followed with a discussion on the selection of dynamic soil properties used in the analysis. The sensitivity of the dam model seismic response is studied using both different material properties and earthquake motions of different frequency content, thereby obtaining responses of the dam model in different frequency ranges. The permanent displacement along specified sliding surfaces are calculated and compared to the maximum allowable displacement of 0.3 m specified in the design criteria for the dam.

2. SEISMIC SETTING

The volcanic zones of Iceland (see Fig. 2.1a) follow the Mid-Atlantic Ridge which marks the boundary of the North-American Plate and the Eurasian Plate. These plates move apart resulting in intrusion of magma on the boundary which again leads to tectonic activity. The earthquakes originating on the spreading zone are relatively small, with magnitudes that seldom exceed five. (Snæbjörnsson et al, 2005) The Hálslón reservoir is located close to the southeast margin of the Northern Volcanic Zone in Iceland (Fig. 2.1a). Fig. 2.1 b shows an earthquake hazard map for Iceland (Sólnes et al, 2004) which indicates that the dam site is located in a low hazard area with defined peak ground acceleration (PGA) of only 0.02-0.05g considering a mean return period of 475 years.

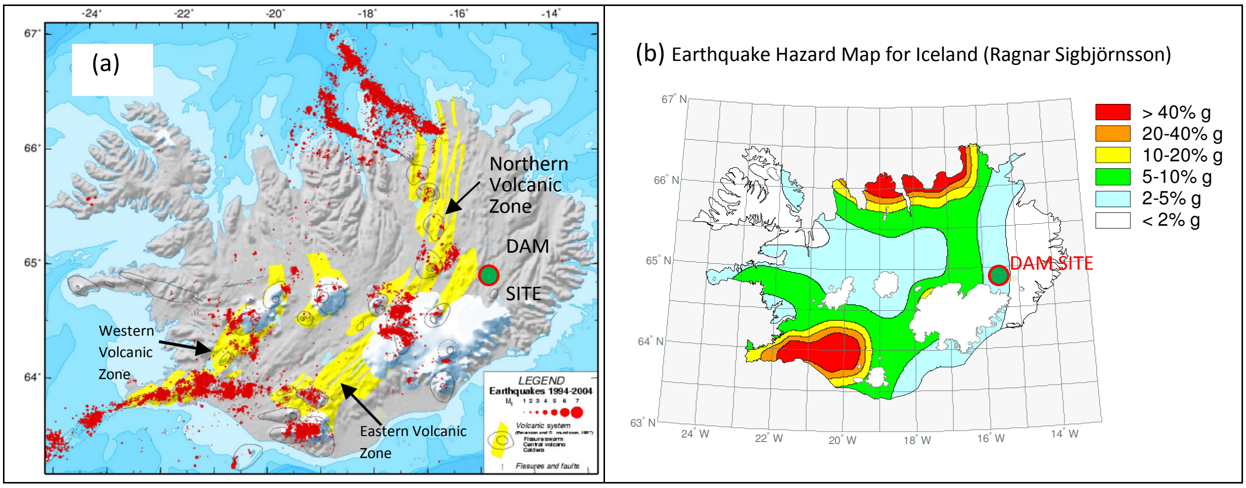


Figure 2.1 (a) Map showing earthquake epicenters (1994-2004) (red dots); volcanic zones in Iceland (yellow) and the dam site (green dot). (fig. Halldórsson (2005). Reference to the dam site and volcanic zones added). (b) Earthquake hazard map of Iceland showing contours of horizontal peak ground acceleration corresponding to mean return period of 475 years, which equals a 10% probability of exceeding the acceleration given by the hazard map during a period of 50 years. (Fig. Sólnes et al. 2004).

2.1 Original seismic design criteria

The seismic design criteria for the Desjarárstifla dam were originally based on earthquake action recommended in a memorandum on the earthquake induced acceleration for the KHP (Sigbjörnsson, 2000) along with the criteria developed in 2001-2002 (Johannesson, 2003) for the largest dam retaining the Hálslón reservoir, Kárahnjúkastifla Dam. Although the dam area was considered to be an area of low seismicity, earthquake of considerable magnitude was not ruled out in the seismic criteria, which in turn was considered quite conservative for the Hálslón area. It was presumed in the design that the upper boundary of the magnitude of a potential earthquake (Maximum Credible Earthquake (MCE)) originating close to the dam area was 5.5-6.0, resulting in a PGA of about 0.26 g (Sigbjörnsson, 2000). About 11 earthquake motions representing this PGA were used, one of them representing a near field motion expecially simulated for the dam site. Furthermore, a far field earthquake of long duration was provided with a PGA of 0.026 g (Sigbjörnsson, 2000). It was decided to scale this event to 0.26 g which provided excitation of totally different frequencies than the near field event and obtain significant response of the dam to the earthquake motion. Additionally, an earthquake motion of short duration and PGA of 0.4-0.5 g was considered and referred to as reservoir triggered earthquake (RTE). Finally, an earthquake motion from a known Californian earthquake (El Centro) was used as an ultimate test on the seismic performance of the dam.

2.2 Revised seismic design criteria

The earthquake action in the area was reviewed by the EERC at the University of Iceland (Snæbjörnsson et al., 2006) following new geological findings in 2004 and 2005, i.e. during the construction phase. The new geological information included faults and lineaments revealed in excavating the dam foundations (Sæmundsson & Johannesson, 2005). Mainly two faults were considered potentially active in the review of the earthquake action (Sigmundsson et al., 2005). Firstly a fault in the foundation of the Kárahnjúkar dam showing a slip of 3 cm in the last 10,000 years, the maximum credible earthquake (MCE) on this fault (strike slip fault) was defined as 5.3M with PGA of 0.23g at the dam site. Secondly, a normal fault some 5 km from the dam site showing a total slip of more than 2 m in three events, the last one 4-5000 years ago, the MCE on this normal fault was defined as 6-6.2M with PGA of 0.3g at the dam site. In view of this, three scenarios were identified: Two earthquakes on the normal fault encountered, one of magnitude M_w=6.0 and the other of magnitude 6.2; Strike slip earthquake of magnitued M_w=5.3 on the Kárahnjúkar Dam fault. (Snæbjörnsson et al., 2006) The extensive review of the earthquake action included simulation of 24 site specific earthquake time series considering these earthquake scenarios. The PGA, frequency content and duration of these were similar to the near field motions originally used in the dam design. Thus, the original seismic design criteria was only slightly affected by these new findings although required review of the seismic analysis of the Desjarárstífla dam. In the review both horizontal and vertical time series were used simultaneously in the analysis (Sigtryggsdottir, 2009).

In addition to the simulated earthquake motions information on five recorded earthquakes (three component acceleration series) from the IESD strong motion database of magnitude M_w=5.7 -6.4 were supplied by EERC of which three were used as a check in the analysis of the Desjarárstífla dam, two stiff soil earthquakes (IESD waveform ID 413 and 414) and one soft soil (IESD waveform ID 879).

Finally, it should be noted that a probabilistic seismic hazard analysis was preferred as a part of the earthquake action review and the presented results were in fair agreement with the hazard map for Iceland shown on Fig. 2.1 b for the 475 year return period but did additionally consider the probability levels of USCOLD corresponding to mean return periods equal to 3,000 to 10,000 years. According to the hazard analyis the PGA for the area is less than 0.1g for the 10,000 year return period. (Snæbjörnsson et al., 2006)

3. THE DAM

The Desjarárstífla dam, is a 70 m high and 1.1 km long earth-rockfill dam. It is the second highest dam in Iceland and the highest such with a central moraine core. Foundation conditions along the dam axis vary markedly, with sound basalt in the east abutment, whereas the west abutment comprises pillow lava mostly overlain by moraine and tillite (Palmason & Sigtryggsd., 2008). Additionally the foundation is transacted by numerous faults, and lineaments (Palmason & Sigtryggsd., 2009) that had to be considered both in the design and monitoring of the dam (Sigtryggsd. & Palmason, 2008). Typical cross section of the dam is shown on fig. 3.1 with downstream slope of 1:1.4 and an upstream slope of 1:1.5. The dam was originally designed as 60 m at the highest cross section but is highest 70 m as constructed. Consequently, some of the results presented apply to a 60 m high dam whereas others apply for a 70 m high dam.

Table 3.1 Material properties for dynamic stability analysis where γ_s and γ_m are the saturated and dry density respectively and ϕ' is the effective friction angle.

Shell: γ_s =20 kN/m³, γ_m =19 kN/m³, ϕ' = 45°,	Core: γ_s = 22 kN/m³, ϕ' = 40°

In the dynamic stability analyses of the dam the material properties in Table 3.1 were used. These material properties are conservative for the dam material. For example an effective friction angle of 47° for the shell and 42°for the core, representing the dam material more closely, were considered in both the static and pseudostatic analysis of the dam. Additionally higher friction angle (~50°) were considered appropriate for critical cases of shallow location of the critical slip surface. (Sigtryggsd. & Palmason, 2004).

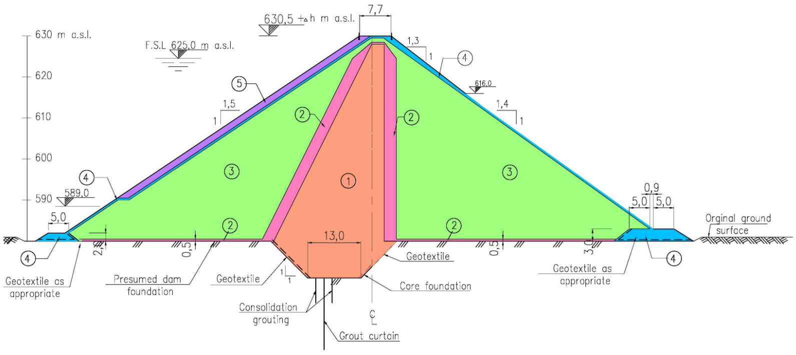


Figure 3.1 A typical dam section showing the essential features. (1) Core; (2) Filter; (3) Shell; (4) Slope protection; (5) Riprap. The core trench shown is limited to the west abutment only.

4. DYNAMIC SOIL PROPERTIES

Dynamic analysis of an earth dam preferably requires knowledge of the dynamic properties of the dam material but in absence of this estimate based on available information from relevant literature can be used. Preparation for the dynamic analyses included estimation of the dynamic soil parameters describing an equivalent linear material model (see for example Kramer, 1996). This includes definition of the K modulus relating the maximum shear modulus, G_{max}, to the mean effective confining stress (see eq. 4.1). Furthermore, selecting appropriate damping functions and G reduction functions from the software used for the analysis. These functions are dependent on i.e. confining stresses and plasticity index. An expression for G_{max} using relationship between this and the initial mean principal effective confining stress is given by eq. 4.1 where the constants n and K (K-modulus) need to be defined for each material:

$$G_{max} = K\,(\sigma_m')^n \qquad (4.1)$$

G_{max}, and/or the K modulus, can be estimated in several different ways. The most reliable means of evaluating G_{max} in situ is by measuring the shear wave velocity, v_s, of the particular soil deposit and use the following relation where ρ is the unit weight of the soil (Kramer, 1996):

$$G_{max} = \rho v_s^2 \qquad (4.2)$$

The shear wave velocity was measured prior to construction at the dam site in Desjarárdalur valley to a depth of about 20 m (Bessason et al. 1998). The measured values considered appropriate for the proposed dam material range from 400 to 600 m/s. Applying the above relationship with ρ =2000 kg/m³ yields G_{max} in the range 320 to 720 MPa. From fig. 4.1(a), displaying the G_{max} versus mean effective principal stress for different K values, a K-modulus of 40,000 yields G_{max} values in this range for for σ_m'= 50 kPa to about 350 kPa. The shear wave velocity for different G_{max} values is also shown on the figure

To further establish appropriate range of shear modulus values the following empirical relationships were used: (1) Empirical formula for G_{max} proposed for sands and gravels by Seed and Idriss 1970 (here converted to SI units):

$$G_{max} = 220\ K_{2,max}\ (\sigma_m')^{0.5} \tag{4.3}$$

where $K_{2,max}$ values for gravels range from 80 to 180 (Seed et al 1984), resulting in K values into eq (4.1) in the range 18 000 to 40 000; (2) Empirical relationship (Kramer 1996) relating G_{max} to the void ratio (e), overconsolidation ratio (OCR) and mean principal effective stress as follows:

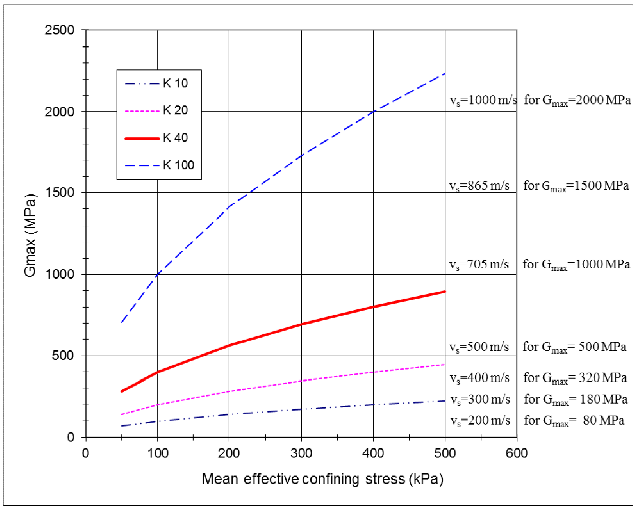
$$G_{max} = 625\ F(e)\ (OCR)^n\ p_a^{1-n}\ (\sigma_m')^n \tag{4.4}$$

With, OCR=1, n=0.5, the atmospheric pressure p_a= 101 kPa, F(e)= $1/e^{1,3}$ (Jamiolkowski et al 1991) and the expected void ratio of the shell material in the range 0.25 to 0.45 the resulting K values into eq (4.1) are in the range 17 800 to 38 100.

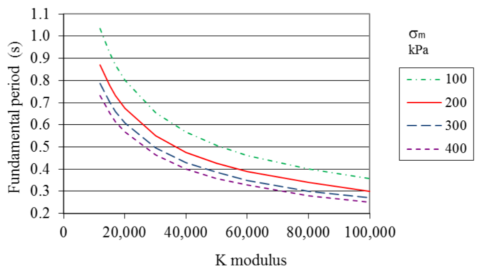
On fig. 4.1(b) the fundamental period of a 60 m high dam is estimated by the empirical formula given by eq. (4.3) below for different; *K* values and mean principal effective stresses (Sigtryggsd. & Palmason 2004). The formula is derived for a homogeneous infinity long dam, where *H* is the height of the dam, v_{ss} is the average shear velocity of soil in the dam, *m* is the power of a shear modulus function and β_1 is constant for the first mode of vibration (Kramer, 1996).

$$T_1 = \frac{16\pi}{(4+m)(2-m)\beta_1} \frac{H}{\bar{v}_{ss}} \tag{4.5}$$

From the figure, assuming that the mean principal effective stress is larger than 200 kPa but less than 400 kPa, a fundamental period in the range 0.55-0.65 s can be expected for a *K* value of 20,000; in the range 0.4-0.5 s for a *K* value of 40,000 and in the range 0.25-0.3, for a *K* value of 100,000.



(a) G_{max} versus σ_m' for different K values



(b) Fundamental period of a 60 m high homogeneous (m=0, β_1 = 2.404), infinitely long dam versus stiffness coefficient K for different values of mean principal stress.

Figure 4.1 (a) G_{max} versus σ_m' for different K values using the formulation G_{max}= $K(\sigma_m')^{0.5}$. The shear wave velocity, v_s, for different G_{max} values and with ρ=2000 kg/m³ are also shown. (K 10 represents K=10.000 etc.) (b) Fundamental period of a 60 m high dam versus stiffness coefficient K. (Sigtryggsd. & Palmason, 2004)

5. SEISMIC ANALYSIS

The seismic analysis of the dam included both pseudo-static and dynamic analyses considering the dam model response to the design earthquake motions and permanent displacement along specified slip surfaces. The pseudostatic analysis was considered as an index of the seismic resistance further established by the dynamic analysis. The dynamic analysis of the dam involved the use of two of GeoSlope's products one for the dynamic analysis of the dam subjected to the earthquake shaking and other for the stability of the dam slope and permanent deformation. (Sigtryggsd. & Palmason, 2004)

The 2-D finite element model of the dam used in the dynamic analysis is shown on fig. 5.1. In the model the actual non-linear behaviour of the soil is approximated by the equivalent linear model. The approximation includes that the strain will always return to zero after cyclic loading and consequently permanent deformation cannot directly be obtained from the analysis. Furthermore, failure cannot occur since a limiting strength is not defined for the linear material. However, stresses and strains are calculated in the soil for every time step, taking into account the degradation in soil stiffness due to the earthquake loading. (See Geoslope manual for Quake/W). The time histories of stresses and strains are used in the stability analysis to look at the variation in slope stability during the earthquake shaking and to estimate the permanent deformation. Thus the stability factor of a specified slope is calculated based on the stress state in the soil at every timestep. (See Geoslope manual for Slope/W). For a stability factor less than 1,0 the average acceleration time history is integrated to find the velocity and then displacement is found by integrating the velocity versus time record. An example of the evaluation of the permanent deformation is shown on fig 5.2 by history plots of the stability factor, acceleration, velocity and deformation.

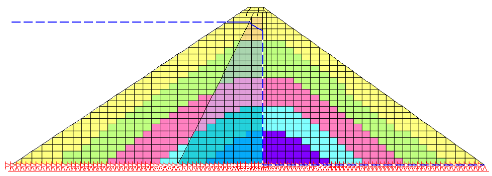


Figure 5.1 2D Finite element model of the dam. Zones with different material properties. G reduction functions and damping functions defined for an effective stress of 50 kPa, 100 kPa, 200 kPa, 300 kPa and 400 kPa.

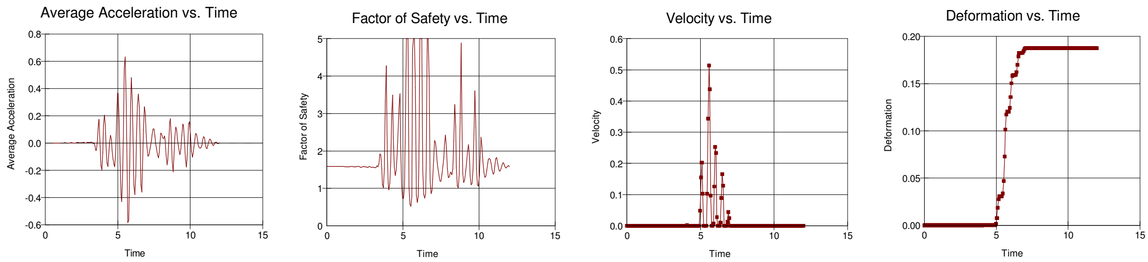


Figure 5.2 Evaluation of the permanent deformation – an example. History plots of the stability factor, average acceleration (m/s²), velocity (m/s) and deformations (m) versus time (in seconds) for sliding surface with 30° inclination subjected to earthquake J1.

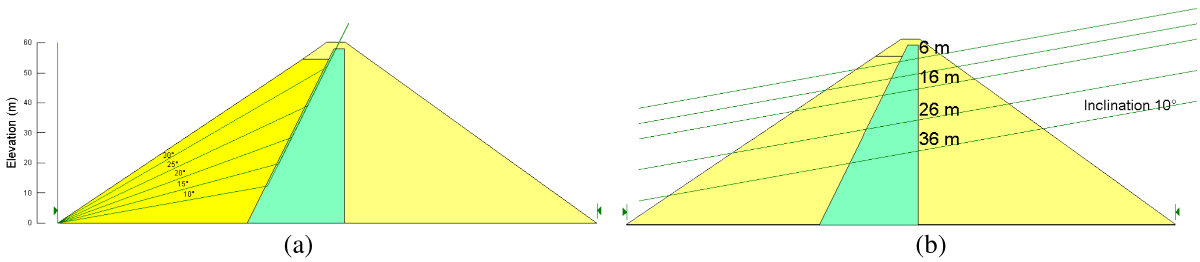
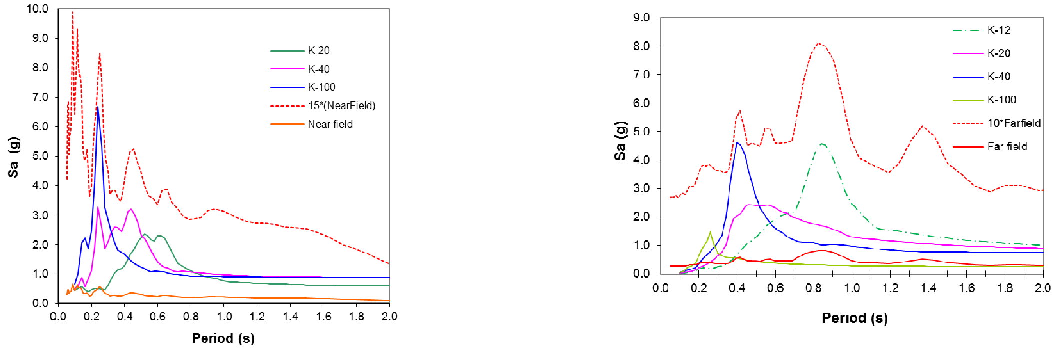


Figure 5.3 Slip surfaces considered to estimate the stability of the dam slope and permanent deformation. (a) Slip surfaces with different inclination in the upstream shell. (b) Slip surfaces with 10° inclination through the dam. The distance from dam crest to the intersection with the boundary between the downstream shell material and the core is labelled .

The sliding surfaces considered in the upstream shell are shown on figure 5.3 (a) and (b). Additionally, sliding surfaces in the downstream shell were considered for the ground motions that resulted in the largest deformations for the upstream shell, but the factor of safety in all cases exceeded 1.0 and thus no permanent displacement were calculated along these. The model of a 70 m high dam considered the same sliding surfaces; however the sliding surfaces shown on fig. 5.3(a) did in that case not originate at the dam toe but with an offset of 15 m horizontally of the dam toe and 10 m vertically. Other sliding surfaces in the upstream slope were also considered for both 60 m high and 70 m high dam but are not discussed here.

A *K* value of 40,000 was considered to be the most appropriate for the shell material. This value defines the upper margin to the maximum shear modulus for gravels according to empirical formulas discussed above. Furthermore, corresponds to the shear wave velocity close to the range considered for the dam material. Thus *K* value of 40,000 was defined for the shell material when calculating the response to earthquake series representing the near field earthquake of the original seismic criteria. Furthermore, when reanalysing the model using the earthquake series according to the revised seismic criteria provided by the EERC, a K value of 40,000 was defined for the shell material.

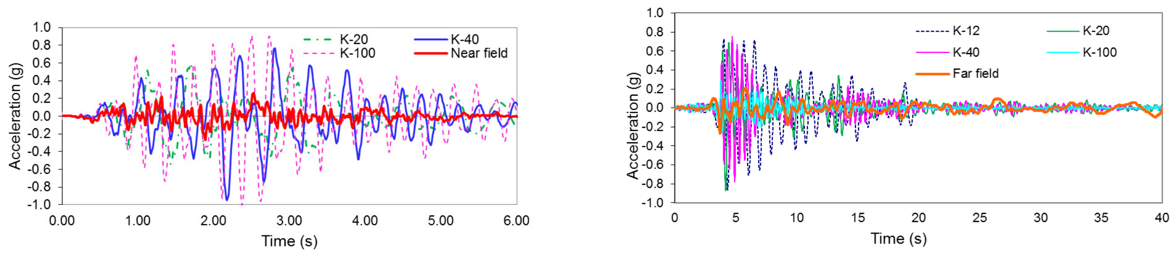


(a) Near field motion with a_{max}=0.26 g. The data series labelled 15*(Near Field) is the spectral ordinates of the input motion scaled 15 times.

(b) Far field motion with a_{max}=0.26 g; The data series labelled 10*Far field is the spectral ordinates of the input motion scaled 10 times.

Figure 5.4 Spectral ordinates of input motion for different stiffness of the dam. K 12, K 20; K 40 and K 100 refer to the *K* value defined for the shell material. K 20 refers to *K*=20.000, K 40 to *K*=40.000 etc. The core was assumed with half the stiffness (K-value) of the shell

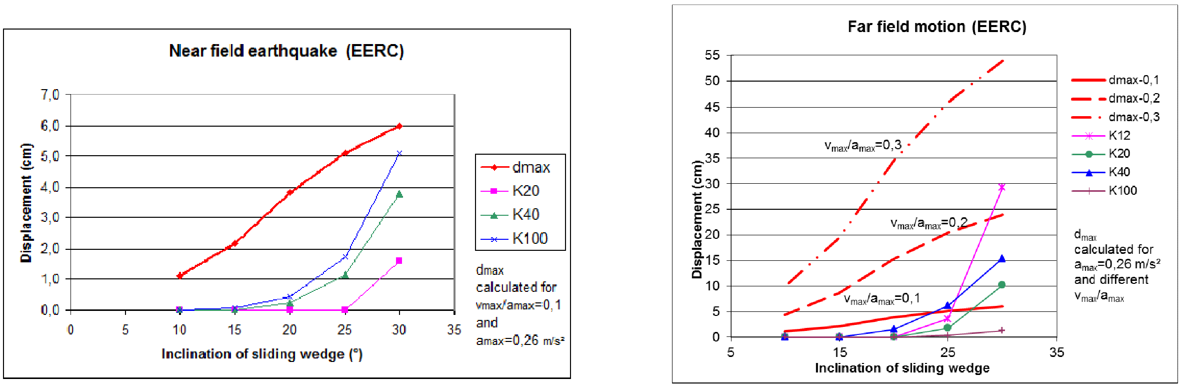
Although K=40 000 was considered appropriate for the shell, the dynamic response to the earthquake motions: near field, far field, RTE and El Centro, were calculated for *K* values of 20,000, 40,000 and 100,000 referred to as K20, K40 and K100 respectively on the figures below. Additionally, for the far field motion, a K value of 12,000 was used. Fig. 5.4 displays the response to the near field and far field earthquakes. From fig. 5.4 the sensitivity in the dam model response to the different type of earthquake motions is evident, furthermore to different dam stiffness. It is interesting to find that for each K value the maximum response of the dam model is in the frequency range indicated in fig. 4.1b) for the same K value. The input motion and the crest responses are shown on fig. 5.5. The magnification of the ground motion with the maximum crest acceleration varies from 0.6 g to 1.0 g for the near field motion with the maximum response in the stiffest dam (K=100,000) whereas the maximum response is in the softest dam for the far field motion and the maximum crest acceleration varying from 0.2 g to 0.8 g for the different dam stiffnesses.



(a) Near field motion with a_{max}=0.26 g.

. (b) Far field with a_{max}=0.26 g

Figure 5.5 Spectral ordinates of input motion and crest response for different stiffness of the dam.



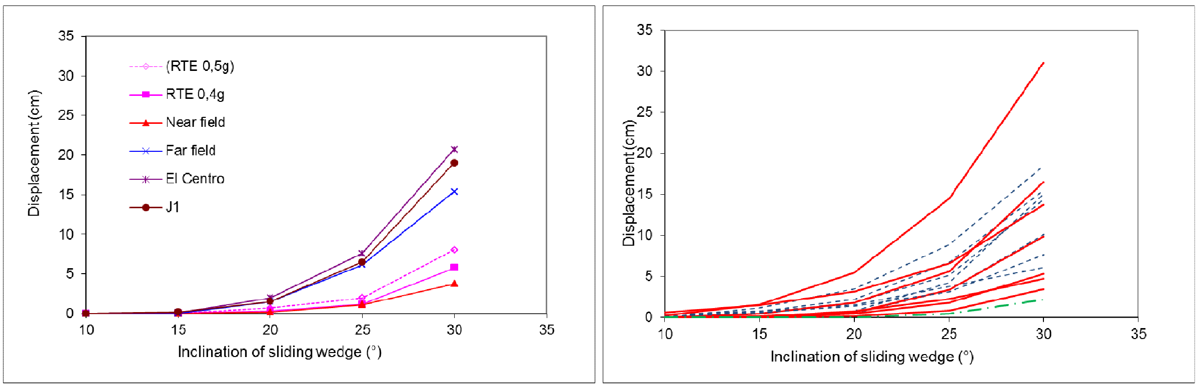
(a) Near field motion v_{max}/a_{max}=0.1; a_{max}=0.26 m/s²

(b) Far field motion v_{max}/a_{max}=0.1, 0.2 and 0.3 (labelled dmax-0,1, dmax-0,2 and dmax-0,3 respectively) and with a_{max}=0.26 m/s²

Figure 5.6 Calculated permanent displacement of sliding wedge versus inclination of the sliding wedge in the shell for different K values. d_{max} is the Newmark upperbound displacement.

The K value for the core was in each case set to half the K value for the shell. Trial runs with other values of *K* for the core were also done to check the sensitivity of the model, disclosing relatively minor effects. Additionally, the effect of a permafrost foundation on a dam with *K* value of 40,000 for the shell and 20,000 for the core was investigated. (Sigtryggsd.&Palmason, 2004).

Fig. 5.6 displays calculated permanent displacement along sliding wedges in the upstream shell resulting from the dynamic analysis for the near field and far field earthquakes. The results are compared to Newmark pseudo-static upper bound displacement, d_{max} (Kramer 1996). The pseudo-static displacement is calculated with a pseudo-static coefficient, k_h, and the ratio of the maximum velocity to maximum acceleration (v_{max}/a_{max}) of the earthquake motion used.



a) Original seismic criteria along with extreme cases considering 60 m high dam model

b) Revised seismic criteria considering 70 m high dam model (Solid line and dashed line: results for the normal faulting earthquake of magnitude 6.2 and 6.0 respectively; Dash-dotted line: results for the strike slip earthquake of magnitude 5.3)

Figure 5.7 Calculated permanent displacement versus inclination of the deformed sliding wedge for the earthquake motions considered for a dam model with K=40.000 in the shell and K =20.000 in the core

Fig. 5.7 displays calculated permanent displacement in a dam model, with K=40,000 defined for the shell and 20,000 for the core, using all the earthquake motions considered. This is within or close to the maximum displacement allowed according to the design criteria for the design earthquake except for one of the revised earthquake motions, for which the permanent displacement is 0.31 m for a slip surface with 30° inclination in the upstream shell. However, as mentioned above the material properties in table 3.1 used in the dynamic stability analysis were considered conservative, and especially so for the shallower slip surfaces.

The maximum values of all analyses are summarized in table 5.1 for the sliding surfaces shown on fig. 5.3 a) and in table 5.2 for the sliding surfaces shown on fig. 5.3 b).

Table 5.1 Summary of critical result from dynamic analysis for calculated maximum permanent displacement for different inclination of the sliding wedge.

	Original criteria 60 m high dam		Revised seismic criteria 70 m high dam	
Inclination of sliding wedge (see fig. 5.3 a)	Extreme case RTE a_{max}=0.5g *K*=100,000	MCE/Design EQ Motion J1 *K*=40,000	Extreme case Recorded EQ EQ-879-YZ K=40,000	MCE/Design EQ $a_{max,x}$=0.3 K=40,000
30°	36 cm	20 cm	44 cm	31 cm
25°	< 20 cm	< 10 cm	< 20 cm	< 15 cm
20°	< 10 cm	< 5 cm	< 10 cm	< 10 cm
15°	< 5 cm	< 5 cm	< 5 cm	< 5 cm
10°	< 5 cm	< 5 cm	< 5 cm	< 5 cm

Table 5.2 Summary of critical result for calculated maximum permanent displacement along sliding surfaces with 10° inclination through the dam.

	Original criteria 60 m high dam		Revised seismic criteria 70 m high dam	
Distance from crest (see fig. 5.3 b)	Extreme case RTE a_{max}=0.5g *K*=100,000	Design EQ Motion J1 a_{max}=0.26 *K*=40,000	Extreme case Recorded EQ EQ-423-YZ K=40,000	MCE/Design EQ $a_{max,x}$=0.3 K=40,000
6 m	< 25 cm	< 10 cm	< 25 cm	< 10 cm
11 m	< 30 cm	< 10 cm	< 25 cm	< 12 cm
16 m	< 25 cm	< 10 cm	< 20 cm	< 10 cm
26 m	< 15 cm	< 5 cm	< 10 cm	< 6 cm
36 m	< 5 cm	< 5 cm	< 5 cm	< 2 cm

6. CONCLUSION

The approach chosen in the seismic design of the Desjfararstifla dam is conservative in more than one respect: Firstly by selecting the maximum credible earthquake (MCE) as the design earthquake for an area of low seismicity. Secondly when considering far field earthquake motions with the same PGA as the MCE earthquake and frequency content corresponding to soft soil conditions. Thirdly, the approach is conservative in the selection of material properties used in the dynamic stability analyses. Finally, the selection of the maximum allowable displacement along the sliding wedges of 0.3 m is conservative compared to the criteria of 0.6 m applied by some (US federal/state) agencies according to FEMA 65 (2005). Here it should be noted that the criteria for the maximum allowable displacement was originally set for the crest displacement but it was later decided to apply the same criteria for the sliding surfaces considered.

Even with this conservative approach the response of the dam models considered for the design earthquake motions are within or close to the design criteria set for the dam with a maximum displacement of 0.31 m calculated for one of the revised earthquake motions. On the other hand for the extreme cases of the earthquake motion the maximum permanent displacement of 0.44 m was calculated for a recorded soft soil motion applying both horizontal and vertical acceleration. Furthermore, the extreme case of a stiff dam (K=100,000) resulted in a permanent displacement of 0.36 m to the short duration RTE motion with a PGA of 0.5 g, i.e. considerable larger than the PGA of 0.3 g for the MCE defined for the area. For all these cases the calculated maximum permanent displacement occurs on a slip surface with inclination of 30°in the upstream shell, i.e. in a shallow location with lower confining stresses where considerable higher friction angle may be expected than the one used in the analysis.

In summary it may be concluded that the calculated permanent displacements, along the sliding surfaces specified, caused by the considered earthquakes are all within tolerable limits, although possibly requiring some remedial measures for the dam following such seismic events, which may be considered extraordinary in this region. Thus the rather steep inclination of the dam´s slopes, i.a. resulting in appreciable cost savings, seems well justified.

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