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Seismic analysis of gravity dams: a comparative study using a progressive methodology

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ABSTRACT: In structural analysis, it is widely accepted that simplified methods, that require fewer resources, should produce more conservative results, while more sophisticated methods, which require greater resources, allow the calculation of more realistic results. For gravity dams, (i) pseudo-static methods, (ii) pseudo-dynamic methods, (iii) linear time history analysis, and (iv) non-linear time history analysis are common techniques. The aim of this paper is to investigate the effectiveness of a progressive analysis methodology, using the four mentioned methods (i-iv) for seismic analysis, and define the scope of its applicability. The study of 52 gravity dam models, representing dams with different heights and properties, was performed by this progressive methodology, regarding the stress distribution along the base of the dam, the sliding safety factor (SSF) and the permanent sliding of the dam. The results show that the pseudo-static method gives a good approximation of the SSF, but the stresses are generally lower, when compared with the values obtained by more sophisticated methods. The pseudo-dynamic method gives a good approximation of the SSF and the stresses on the base of the dam, while the linear time history analysis did not give additional information for the dams analysed with SSF > 1when compared with the pseudo-dynamic method. The non-linear time history analysis confirms the dam behavior against a sliding failure scenario, even after the stress redistribution, giving an actual time-history of the SSF and stress during an earthquake. The results show that the proposed progressive methodology, is a beneficial procedure to conduct gravity dam safety assessment.

KEY WORDS: Gravity dams; seismic analysis, progressive methodology.

1 INTRODUCTION

Generally the seismic analysis of dams begins with simplified methods and, if necessary, progresses through more sophisticated methods. For gravity dams, (i) pseudo-static methods, (ii) pseudo-dynamic methods, (iii) linear time history analysis, and (iv) non-linear time history analysis are common techniques [1]. Numerous aspects influence the selection and application of the analysis method, most of them related to the dam-reservoir-sediment-foundation interactions. For example, the representation of the hydrodynamic effect of the reservoir, the application methodologies of the seismic input ground motions, the type and amount of equivalent viscous and radiation damping, the consideration of internal uplift pressures, and the possibility of cracking. Many regulations suggest the assessment of the dam against an earthquake with moderate intensity, for which the dam must present an elastic behavior, and also an earthquake with high intensity for which the dam can suffer some damage, but keeping the ability to retain the reservoir. The criteria for assessing the behavior of the dam are both local, such as the stress response of the structure in relation to the strength of the material, and also global, such as the possibility of permanent sliding of the dam or through sections of the dam.

It is widely accepted that simplified methods, that require fewer resources, should produce more conservative results, while more sophisticated methods, which require greater resources, allow the calculation of more realistic results. The aim of this paper is to investigate the effectiveness of this progressive analysis methodology and define the scope of its applicability. The study of 52 gravity dam models, representing dams with different heights and properties, was performed, using the four mentioned methods (i-iv) for seismic analysis, regarding the stress distribution along the base of the dam, sliding safety factor and the permanent sliding of the dam. In terms of geometry, the considered gravity profiles ranged between small dams (15 m high) and very large dams (100 m high). The properties of the materials were adopted considering the case of aged or damaged dams, dams in good condition, good quality rock mass foundations and weak rock mass foundations.

2 DESCRIPTION OF THE MODELS

2.1 Dams geometry and materials properties

Four different dams size were considered (Figure 1), representing a small dam, 15 m high, a medium dam, 30 m high, a large dam, 50 m high, and a very large dam, 100 m high. The base of the dams was defined from a downstream slope of 0.8:1 (H:V), and the upstream face is assumed vertical for simplicity. The crest width is adjusted according to the dam height. The main dimensions are presented in Table 1. The material properties of the dams were selected to represent two scenarios. The first scenario represents the case of a dam in a good condition, characterized by a Young's modulus of 30 GPa. The second case is intended to represent an aged dam or a damaged dam, characterized by a Young's modulus of 15 GPa.

For the material properties of the rock mass, a similar approach was adopted, considering the case of a rock mass with good quality, moderately fractured, characterized by a Young's modulus of 20 GPa and a rock mass highly fractured, characterized by a Young's modulus of 10 GPa. For all scenarios, the material density is 2400 kg/m^3 for the concrete and 2500 kg/m^3 for the rock mass. All properties are listed in Table 2.

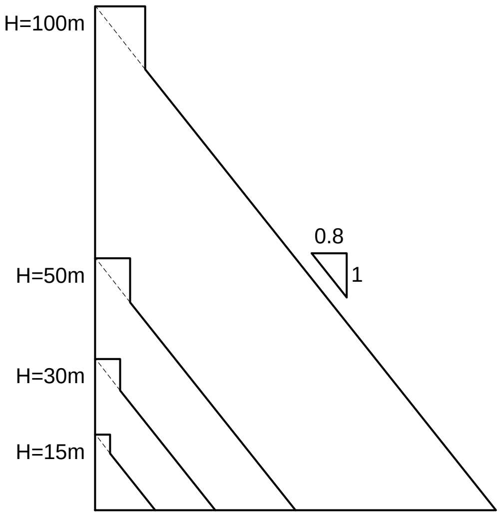

Figure 1. Geometry of the dams.

Table 1. Main dimensions of the models.

Description	Height	Base width	Crest width
Small dam	15 m	12 m	3 m
Medium dam	30 m	24 m	5 m
Large dam	50 m	40 m	7 m
Very large dam	100 m	80 m	10 m

Table 2. Materials properties.

Description	Young's modulus [GPa]	Density [kg/m^3]	Poisson's ratio
Dam – Good condition	30	2400	0.2
Dam – Old or damaged	15	2400	0.2
Rock mass – Moderately fractured	20	2500	0.2
Rock mass – Highly fractured	10	2500	0.2

2.2 Static and dynamic loads

In addition to the dead weight, the hydrostatic pressure was applied using the elevation of the crest, in each case, as the reservoir level (Figure 2). For the uplift, a triangular diagram was adopted, with null pressure at the downstream toe and 30% of the hydrostatic pressure at the upstream heel (Figure 2). This reduction of 70% was assumed to take in account the effect of the grout curtain and the drainage system, reducing the flow rate and the uplift on the dam-foundation interface. It is assumed that the uplift load keeps unchanged during the earthquake. Table 3 summarized the total hydrostatic pressure and uplift loads, dead weight and the ratio between uplift and self-height. For all cases, the uplift is about 12.5% of the dead weight.

Table 3. Hydrostatic pressure ($P_{h,h}$), uplift (U), dead weight (W) and the ratio between uplift and self-height (U/W).

Height	$P_{h,h}$ [kN]	U [kN]	W [kN]	U/W
15 m	1125	270	2160	0.125
30 m	4500	1080	8640	0.125
50 m	12500	3000	24000	0.125
100 m	50000	12000	96000	0.125

For the hydrodynamic effect of the reservoir, the Westergaard's solution [1] was adopted (Figure 2), using the following equation:

$$m_{a,i} = \frac{7}{8}\rho_w\sqrt{Hy_i}A_i \tag{1}$$

where $m_{a,i}$ is the added mass in the horizontal direction for point i; ρ_w is the water density; H is the reservoir elevation; y_i is the vertical height measured from the reservoir elevation at point i; and A_i is the influence area of point i.

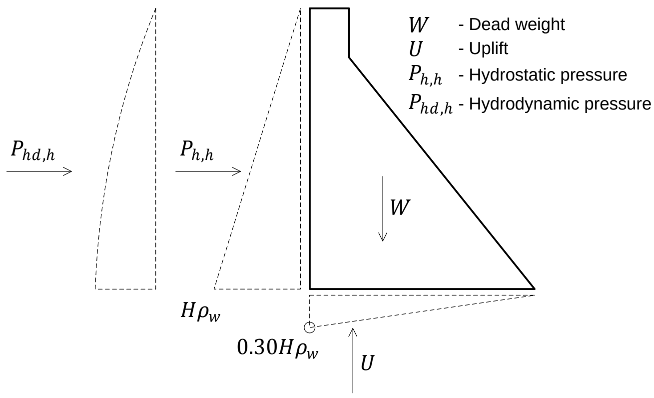

Figure 2. Static and dynamic loads.

The mass of the dam and the added mass for each case are listed in Table 4 that also shows the ratio between the added mass and the mass of the dam. For all cases, the added mass represents approximately 61% of the dam mass.

Table 4. Dam mass (m_d), added mass (m_a), and the ratio between the added mass and the dam mass (m_a/m_d).

Height	Dam mass (m_d) [10^3 kg]	Added mass (m_a) [10^3 kg]	m_a/m_d
15 m	216	132	0.61
30 m	864	562	0.61
50 m	2400	1462	0.61
100 m	9600	5848	0.61

From the geometry of the model, the material properties, and the added mass, the fundamental period of vibration was calculated, using a simplified method described in reference [1]. Figure 3, representing a dam 100 m high, with a foundation 280 m wide and 150 m high, shows the fundamental mode of vibration. Four models for each dam height were considered: model 15/10, E_c=15 GPa and E_r=10 GPa, model 15/20, E_c=15 GPa and E_r=20 GPa, model 30/10, E_c=30 GPa and E_r=10 GPa, and model 30/20, E_c=30 GPa and E_r=20 GPa, where E_c is the Young's modulus of the concrete and E_r is the Young's modulus of the rock mass. The results are summarized in Table 5.

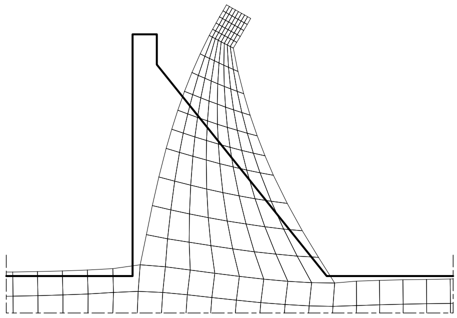

Figure 3. Fundamental mode of vibration, representing a dam 100 m high.

Table 5. Fundamental period of vibration.

Height	Period			
	15/10	15/20	30/10	30/20
15 m	0.075 s	0.068 s	0.064 s	0.056 s
30 m	0.150 s	0.136 s	0.129 s	0.112 s
50 m	0.410 s	0.227 s	0.214 s	0.186 s
100 m	0.500 s	0.454 s	0.429 s	0.372 s

The seismic load was defined considering the case of a dam located in Évora, Portugal, and laid out on a rock mass foundation. For a long distance earthquake scenario and for a 1000 years return period, according to the Portuguese National Annex of Eurocode 8 (EC8) [2], the peak ground acceleration (PGA) is approximately 1.6 m/s^2. Two artificial accelerograms were generated (Figure 4), 42 seconds long, to match the elastic response spectrum proposed in EC8 (Figure 5). A 5% viscous damping ratio, proportional to the mass and centered on the fundamental frequency of vibration, was adopted.

As mentioned before, the seismic analysis was carried out using four different methods: a pseudo-static method (PS), a pseudo-dynamic method (PD), a linear time history analysis (L) and a non-linear time history analysis (NL). The first two methods, pseudo-static and pseudo-dynamic, were solved using the numerical tool CADAM [3]. For the last two methods, linear time history analysis and the non-linear time history analysis, the studies were developed using a numerical application developed by means of the discrete element method (DEM), to carry out structural and hydraulic analysis of gravity dams [4]. CADAM uses the gravity method and the stresses are calculated using the beam theory. The L analysis is a finite element analysis, whose model is composed by two continuous meshes representing the dam and the foundation. Between the dam and the foundation, an elastic interface was assumed, with a normal stiffness of 20 GPa/m and a shear stiffness of 7 GPa/m. The NL models are similar to the L models, except the constitutive model of the joint on the foundation plane, between the dam and the rock mass, assuming a non-linear behavior, with a friction angle of 45º, null cohesion and no tensile strength.

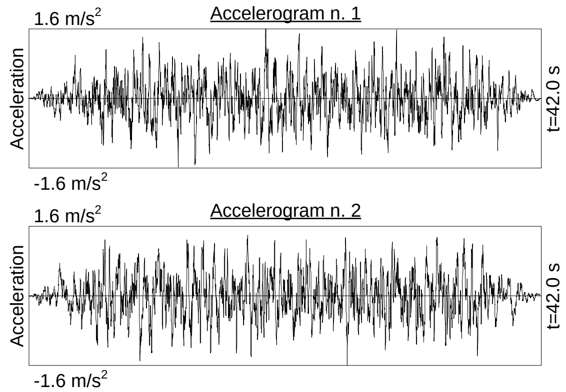

Figure 4. Accelerograms with a PGA of 1.6 m/s^2.

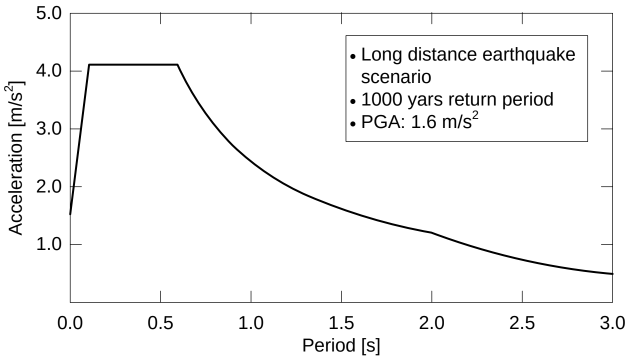

Figure 5. Shape of the elastic response spectrum.

2.3 *Static loading*

In the first stage only the static loads, dead weight, hydrostatic pressure and uplift, were considered. For all model cases, the sliding safety factors (SSF) are 1.80 for the dam 15 m high, 1.76 for the dam 30 m high, 1.74 for the dam 50 m high and 1.71 for the dam 100 m high. The stresses are plotted in Figure 6.

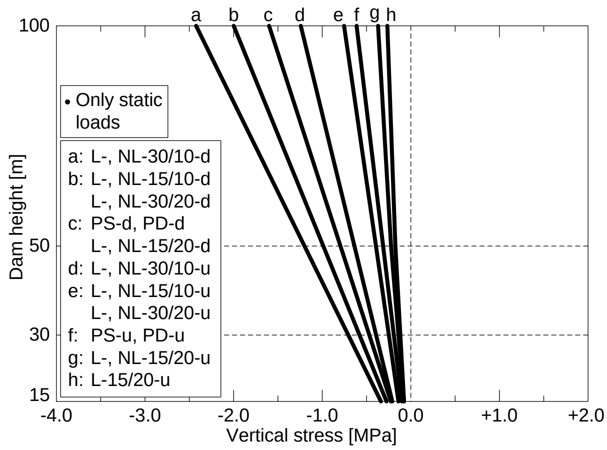

Figure 6. Vertical stress considering only the static loads (u - upstream; d – downstream).

3 EVALUATION OF THE RESULTS ACCORDING TO THE SEISMIC ANALYSIS METHODS

3.1 General considerations

For the seismic analysis, the main results include the vertical stresses and the safety factor as described for the static loading. For the linear time history analysis and the non-linear time history analysis, the stresses and the sliding safety factor were registered as time-histories. For this reason, the stresses are characterized by the minimum and maximum values recorded during the earthquake. In the case of the sliding safety factor, only the minimum value is relevant to assess the stability of the dam.

3.2 Seismic analysis using a pseudo-static method

In the pseudo-static analysis, the inertia forces result from the product of the mass times the acceleration. The dynamic amplification of the inertia loads along the height of the dam and the oscillatory nature are neglected. For the horizontal direction, an acceleration of 1.6 m/s^2, the PGA, was applied in the stress analysis, while an acceleration of 1.1 m/s^2, sustained acceleration taken as 2/3 of the peak acceleration value, was used in the stability analysis. For the vertical direction, the acceleration was reduced by a factor of 2/3, being, in the stress analysis and in the stability analysis, respectively, 0.5 and 0.4 m/s^2. The values obtained for the SSF are presented in Table 6, and the vertical stresses at the upstream heel and at the downstream toe are plotted in Figure 7. Comparing to the results from the static loading, the SSF is reduced approximately 35%, the vertical stresses at the upstream heel are tensile stresses, while the compressive stresses at the downstream toe were magnified around 65%.

Table 6. Sliding safety factors obtained from the pseudo-static seismic linear analyses.

Model	15 m	30 m	50 m	100 m
PS	1.19	1.17	1.15	1.13

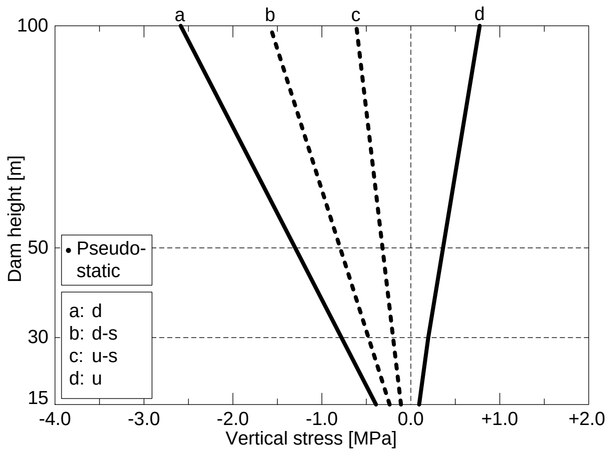

Figure 7. Vertical stresses of the pseudo-static seismic analysis (u - upstream; d - downstream; s - static loading).

3.3 Seismic analysis using a pseudo-dynamic method

The pseudo-dynamic seismic analysis is similar to the pseudo-static seismic analysis except that it recognizes the dynamic amplification of the inertia forces along the height of the dam, by the simplified response spectra method as described by Chopra [5]. The dynamic amplification is considered only in the horizontal direction. The dynamic flexibility of the dam-foundation is modeled with the Young's modulus of the concrete and the rock mass. The oscillatory nature of the inertia forces is not considered, the horizontal and vertical loads are continuously applied. For this reason, the stress analysis is performed using the peak spectral acceleration while the stability analysis is performed using the sustained spectral acceleration. Those values are, respectively, 4.1 m/s^2 and 2.7 m/s^2, keeping the same horizontal and vertical ratio for the peak and sustained ground acceleration described before for the pseudo-static seismic analysis. Table 7 shows the SSF. The reduction from the SSF obtained in the static loading is around 33%. The stresses are plotted in Figure 8. In the upstream heel, the stresses are tensions. In the downstream toe the compressive stresses are magnified by a factor around 2.1. The materials properties have a slight impact on the results.

Table 7. Sliding safety factors obtained from the pseudo-dynamic linear seismic analyses.

Model	15 m	30 m	50 m	100 m
PD-15/10	1.25	1.17	1.14	1.11
PD-15/20	1.26	1.17	1.14	1.11
PD-30/10	1.28	1.18	1.15	1.12
PD-30/20	1.29	1.18	1.15	1.12

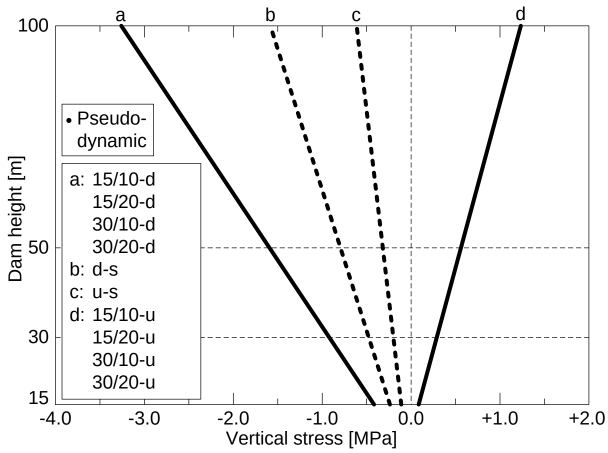

Figure 8. Vertical stress of the pseudo-dynamic seismic analysis (u - upstream; d - downstream; s - static loading).

3.4 Seismic analysis using a linear time history method

A finite element model was employed in the linear time history analysis. The two accelerograms presented in the section 2.2 were used. The accelerogram n. 1 was applied to the horizontal direction, while the accelerogram n. 2 was applied to the vertical direction, with a reduction of 2/3. The seismic analysis was developed by applying an equivalent time-history of shear and vertical stress at the lower foundation elements. Through the integration of the time-history of the horizontal unbalanced forces, the permanent displacement can be estimated, when the safety factor is below the unity.

The SSF are presented in Table 8. The vertical stresses are presented in four different figures, according the materials properties, from Figure 9 to Figure 12. The compressive stress at the upstream heel and at downstream toe are similar, with a maximum of -3.5 MPa, in the case of the model 30/10 (E_c=30 GPa and E_r=10 GPa), representing a dam 100 m high. The maximum tensile stress at the upstream heel is +1.6 MPa, for the model 15/10 (E_c=15 GPa and E_r=10 GPa), representing a dam 100 m high.

Table 8. Sliding safety factors obtained from the seismic linear time history analyses.

Model	15 m	30 m	50 m	100 m
L-15/10	1.07	1.12	1.11	1.15
L-15/20	1.04	1.23	1.12	1.20
L-30/10	1.05	1.17	1.12	1.16
L-30/20	1.06	1.19	1.16	1.22

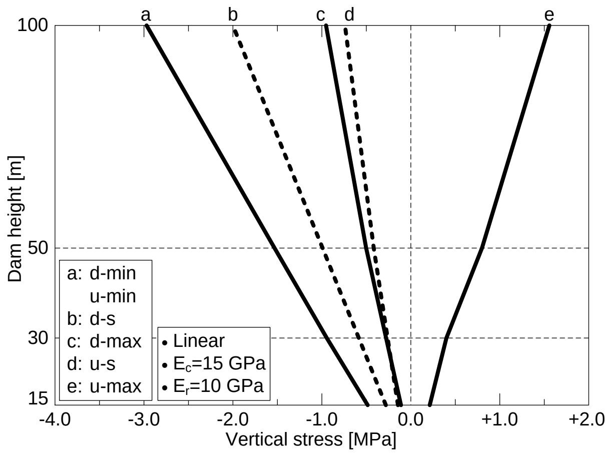

Figure 9. Vertical stress of the linear time history analysis for the case E_c=15 GPa and E_r=10 GPa (u - upstream; d - downstream; s - static loading).

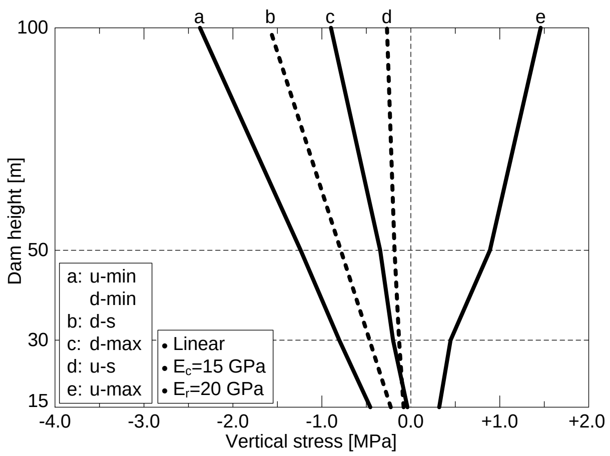

Figure 10. Vertical stress of the linear time history analysis for the case E_c=15 GPa and E_r=20 GPa (u - upstream; d - downstream; s - static loading).

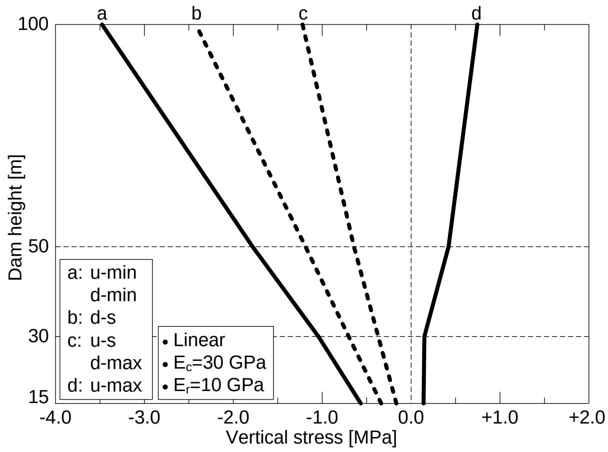

Figure 11. Vertical stress of the linear time history analysis for the case E_c=30 GPa and E_r=10 GPa (u - upstream; d - downstream; s - static loading).

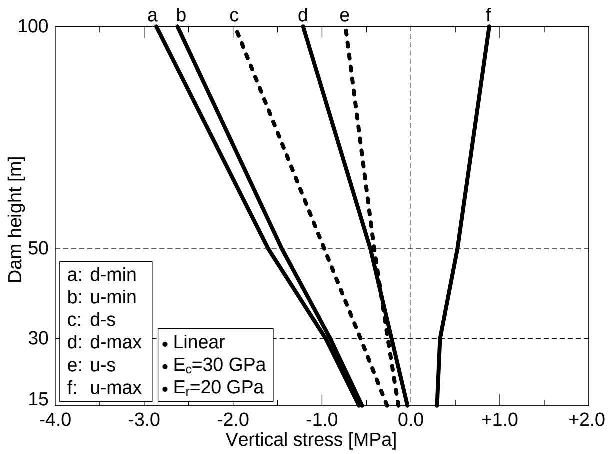

Figure 12. Vertical stress of the linear time history analysis for the case E_c=30 GPa and E_r=20 GPa (u - upstream; d - downstream; s - static loading).

3.5 Seismic analysis using a non-linear time history method

The model used for the non-linear time history analysis is very similar to the one considered to the linear time history analysis. The main difference is concerned with the properties of the joint between the dam and the foundation. A non-linear model was assumed, with a friction angle of 45º, null cohesion and no tensile strength. The SSF are presented in Table 9. The vertical stresses are plotted according to the materials properties, from Figure 13 to Figure 16.

Table 9. Sliding safety factors obtained from the seismic non-linear time history analyses.

Model	15 m	30 m	50 m	100 m
L-15/10	1.08	1.13	1.16	1.10
L-15/20	1.11	1.23	1.12	1.22
L-30/10	1.05	1.17	1.11	1.16
L-30/20	1.06	1.19	1.17	1.24

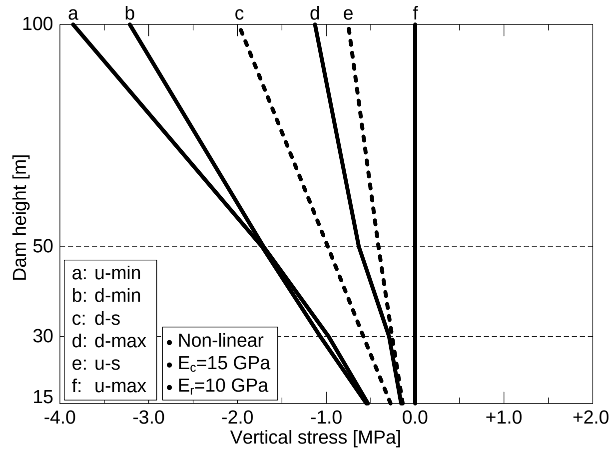

Figure 13. Vertical stress of the non-linear time history analysis for the case E_c=15 GPa and E_r=10 GPa (u - upstream; d - downstream; s - static loading).

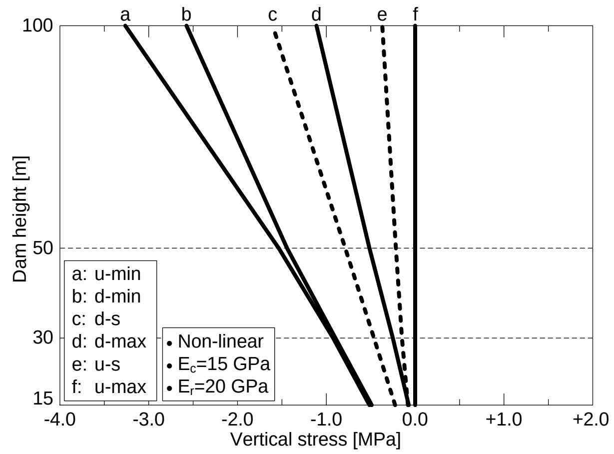

Figure 14. Vertical stress of the non-linear time history analysis for the case E_c=15 GPa and E_r=20 GPa (u - upstream; d - downstream; s - static loading).

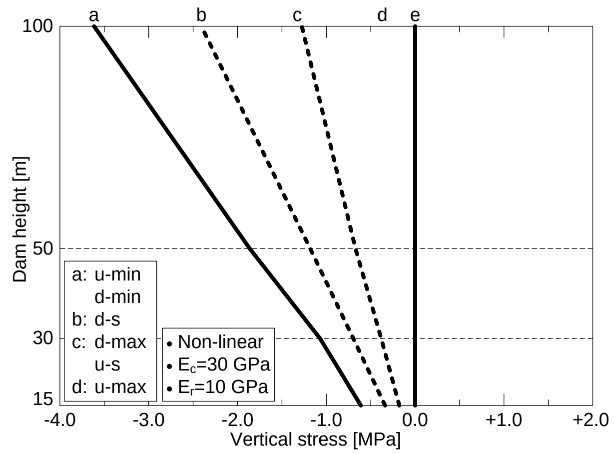

Figure 15. Vertical stress of the non-linear time history analysis for the case E_c=30 GPa and E_r=10 GPa (u - upstream; d - downstream; s - static loading).

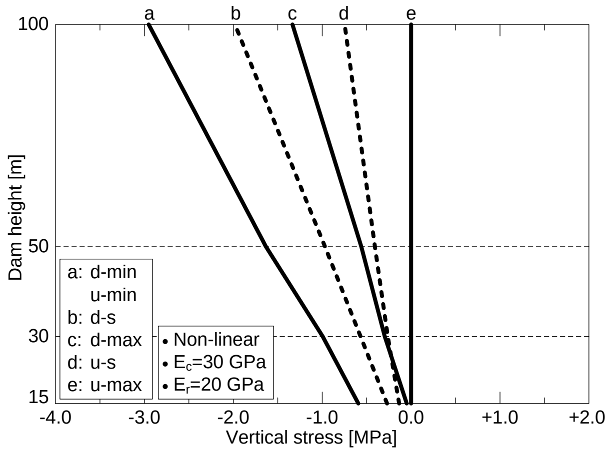

Figure 16. Vertical stress of the non-linear time history analysis for the case E_c=30GPa and E_r=20GPa (u - upstream; d - downstream; s - static loading).

The maximum compressive stress at the upstream heel is found in the case of the model 15/10, with a value of -3.9 MPa, representing a dam 100 m high. The maximum tensile stress on the base of the dam is null, because the nonlinear constitutive model of the joint, between the dam and the foundation, does not admit tensile stresses.

4 EVALUATION OF THE RESULTS ACCORDING TO THE DAM HEIGHT

4.1 Results for the dam 15 m high

The results obtained for the static and seismic analyses are presented, respectively, in Table 10 and in Table 11. Considering the seismic analyses, the SSF obtained from NL models, are smaller than those from the others methods. The dam-foundation interface opening acts as a base isolation system reducing the inertia forces transmitted to the dam. Great variations on the stress, both at the upstream heel and at the downstream toe, can also be found. The local permanent displacements, observed after the earthquake, were measured at the downstream toe. The SSF are above the unity in all instants, probably the sliding did not occur simultaneously in all points of the dam base.

Table 10. SSF, vertical stress and permanent sliding for the static loading, for the dam 15 m high.

Model	Vert. stress upstream [MPa]	Vert. stress downstr. [MPa]	SSF	Perm. displ. [mm]
PS	-0.11	-0.22	1.80	0.0
PD	-0.11	-0.22	1.80	0.0
L-15/10	-0.15	-0.28	1.80	0.0
L-15/20	-0.08	-0.23	1.80	0.0
L-30/10	-0.21	-0.34	1.80	0.0
L-30/20	-0.14	-0.27	1.80	0.0
NL-15/10	-0.15	-0.28	1.80	0.0
NL-15/20	-0.09	-0.23	1.80	0.0
NL-30/10	-0.21	-0.34	1.80	0.0
NL-30/20	-0.14	-0.27	1.80	0.0

Table 11. SSF, vertical stress and permanent sliding for the seismic analyses, for the dam 15 m high.

Model	Vert. stress upstream [MPa]		Vert. stress downstr. [MPa]		SSF	Perm. displ. [mm]
	Min.	Max.	Min.	Max.		
PS	0.10		-0.39		1.19	0.0
PD-15/10	0.11		-0.44		1.25	0.0
PD-15/20	0.10		-0.42		1.26	0.0
PD-30/10	0.09		-0.41		1.28	0.0
PD-30/20	0.08		-0.40		1.29	0.0
L-15/10	0.20	-0.49	-0.11	-0.47	1.07	0.0
L-15/20	0.31	-0.44	-0.04	-0.48	1.04	0.0
L-30/10	0.14	-0.57	-0.12	-0.57	1.05	0.0
L-30/20	0.29	-0.55	-0.04	-0.59	1.06	0.0
NL-15/10	0.0	-0.55	-0.16	-0.53	1.08	< 0.1
NL-15/20	0.0	-0.52	-0.08	-0.50	1.11	0.6
NL-30/10	0.0	-0.59	-0.15	-0.63	1.05	0.1
NL-30/20	0.0	-0.59	-0.05	-0.59	1.06	1.2

4.2 *Results for the dam 30 m high*

Table 12 shows the results for the static loading and Table 13 shows the results for the seismic analyses. For the seismic analyses, the results seem to be relatively consistent between all the models. The SSF are quite similar and the stresses, excepted for the model PS, are equivalents. The permanent displacements are small and not of engineering significance. The pseudo-dynamic model is a reasonable approximation of the actual seismic behavior of the dam 30 m high.

Table 12. SSF, vertical stress and permanent sliding for the static loading, for the dam 30 m high.

Model	Vert. stress upstream [MPa]	Vert. stress downstr. [MPa]	SSF	Perm. displ. [mm]
PS	-0.21	-0.45	1.76	0.0
PD	-0.21	-0.45	1.76	0.0
L-15/10	-0.27	-0.58	1.76	0.0
L-15/20	-0.13	-0.47	1.76	0.0
L-30/10	-0.41	-0.70	1.76	0.0
L-30/20	-0.26	-0.57	1.76	0.0
NL-15/10	-0.27	-0.58	1.76	0.0
NL-15/20	-0.15	-0.47	1.76	0.0
NL-30/10	-0.41	-0.70	1.76	0.0
NL-30/20	-0.26	-0.57	1.76	0.0

Table 13. SSF, vertical stress and permanent sliding for the seismic analyses, for the dam 30 m high.

Model	Vert. stress upstream [MPa]		Vert. stress downstr. [MPa]		SSF	Perm. displ. [mm]
	Min.	Max.	Min.	Max.		
PS	0.22		-0.78		1.17	0.0
PD-15/10	0.32		-0.94		1.17	0.0
PD-15/20	0.32		-0.94		1.17	0.0
PD-30/10	0.30		-0.93		1.18	0.0
PD-30/20	0.30		-0.93		1.18	0.0
L-15/10	0.39	-0.99	-0.26	-0.90	1.12	0.0
L-15/20	0.44	-0.79	-0.20	-0.82	1.23	0.0
L-30/10	0.15	-1.04	-0.33	-1.02	1.17	0.0
L-30/20	0.32	-0.91	-0.22	-0.96	1.19	0.0
NL-15/10	0.0	-1.08	-0.30	-0.98	1.13	< 0.1
NL-15/20	0.0	-0.94	-0.26	-0.90	1.23	< 0.1
NL-30/10	0.0	-1.08	-0.37	-1.07	1.17	0.1
NL-30/20	0.0	-1.00	-030	-0.99	1.19	0.2

4.3 *Results for the dam 50 m high*

The results obtained for the static and seismic analyses are presented, respectively, in Table 14 and in Table 15. The comments to the results are similar to the ones expressed for the dam with 30 m. The SSF are equivalents and stresses are similar, excepted for the model PS, which values are lower than those achieved by the others methods. A global displacement of the dam through the foundation plane is unlikely.

Table 14. SSF, vertical stress and permanent sliding for the static loading, for the dam 50 m high.

Model	Vert. stress upstream [MPa]	Vert. stress downstr. [MPa]	SSF	Perm. displ. [mm]
PS	-0.33	-0.76	1.74	0.0
PD	-0.33	-0.76	1.74	0.0
L-15/10	-0.42	-0.99	1.74	0.0
L-15/20	-0.18	-0.79	1.74	0.0
L-30/10	-0.66	-1.19	1.74	0.0
L-30/20	-0.41	-0.97	1.74	0.0
NL-15/10	-0.42	-0.99	1.74	0.0
NL-15/20	-0.22	-0.79	1.74	0.0
NL-30/10	-0.66	-1.19	1.74	0.0
NL-30/20	-0.41	-0.97	1.74	0.0

Table 15. SSF, vertical stress and permanent sliding for the seismic analyses, for the dam 50 m high.

Model	Vert. stress upstream [MPa]		Vert. stress downstr. [MPa]		SSF	Perm. displ. [mm]
	Min.	Max.	Min.	Max.		
PS	0.38		-1.29		1.15	0.0
PD-15/10	0.57		-1.60		1.14	0.0
PD-15/20	0.57		-1.60		1.14	0.0
PD-30/10	0.54		-1.57		1.15	0.0
PD-30/20	0.54		-1.57		1.15	0.0
L-15/10	0.80	-1.46	-0.50	-1.58	1.11	0.0
L-15/20	0.88	-1.23	-0.35	-1.25	1.12	0.0
L-30/10	0.42	-1.75	-0.62	-1.81	1.12	0.0
L-30/20	0.52	-1.45	-0.46	-1.60	1.16	0.0
NL-15/10	0.0	-1.73	-0.64	-1.72	1.16	0.2
NL-15/20	0.0	-1.53	-0.52	-1.44	1.12	0.3
NL-30/10	0.0	-1.82	-0.69	-1.91	1.11	0.3
NL-30/20	0.0	-1.62	-0.56	-1.66	1.17	0.2

4.4 *Results for the dam 100 m high*

Table 16 shows the results for the static loading and Table 17 shows the results for the seismic analyses. For the seismic analyses, the pseudo-static and the pseudo-dynamic methods seem to be conservative relative to the SSF, because the results for the SSF are smaller than the results obtained by the linear time history and by the non-linear time history analyses. The stresses are similar, excepted for the PS analysis, whose stresses, both in upstream and downstream faces, are lower. The permanent displacement is also small.

Table 16. SSF, vertical stress and permanent sliding for the static loading, for the dam 100 m high.

Model	Vert. stress upstream [MPa]	Vert. stress downstr. [MPa]	SSF	Perm. displ. [mm]
PS	-0.60	-1.57	1.71	0.0
PD	-0.60	-1.57	1.71	0.0
L-15/10	-0.74	-2.00	1.71	0.0
L-15/20	-0.27	-1.59	1.75	0.0
L-30/10	-1.24	-2.42	1.71	0.0
L-30/20	-0.74	-1.99	1.75	0.0
NL-15/10	-0.76	-2.01	1.71	0.0
NL-15/20	-0.38	-1.61	1.71	0.0
NL-30/10	-1.24	-2.42	1.71	0.0
NL-30/20	-0.76	-1.99	1.71	0.0

Table 17. SSF, vertical stress and permanent sliding for the seismic analyses, for the dam 100 m high.

Model	Vert. stress upstream [MPa]		Vert. stress downstr. [MPa]		SSF	Perm. displ. [mm]
	Min.	Max.	Min.	Max.		
PS	0.78		-2.58		1.13	0.0
PD-15/10	1.27		-3.28		1.11	0.0
PD-15/20	1.27		-3.28		1.11	0.0
PD-30/10	1.22		-3.23		1.12	0.0
PD-30/20	1.22		-3.23		1.12	0.0
L-15/10	1.56	-2.99	-0.95	-2.95	1.15	0.0
L-15/20	1.46	-2.41	-0.91	-2.34	1.20	0.0
L-30/10	0.74	-3.52	-1.20	-3.40	1.16	0.0
L-30/20	0.88	-2.63	-1.22	-2.87	1.22	0.0
NL-15/10	0.0	-3.87	-1.13	-3.23	1.10	0.3
NL-15/20	0.0	-3.26	-1.11	-2.58	1.22	0.2
NL-30/10	0.0	-3.70	-1.33	-3.56	1.16	0.4
NL-30/20	0.0	-2.95	-1.35	-2.97	1.24	0.3

5 CONCLUSIONS

A set of 52 seismic analyses of gravity dam models, using four different methods, were carried out. It was found that (i) the pseudo-static method model gives a good approximation of the sliding safety factor (SSF), but the stresses are generally lower, when compared with the values obtained by more sophisticated methods; (ii) the pseudo-dynamic analysis gives a good approximation of the SSF and stresses on the base of the dam, the materials properties have small influence on those results; (iii) for the dam analysed with SSF $>$ 1 the linear time history analysis did not give additional relevant information when compared with the pseudo-dynamic method; and (iv) the non-linear time history analysis confirmed the dam response against a sliding failure scenario, even when stress redistribution is taking place, giving an actual time-history of the stress during an earthquake and permanent displacements.

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