Safety assessment of a masonry dam from the 18th century

Susana Carrega
Instituto Superior Técnico, Lisboa, Portugal

July 2018

Abstract

Dams were amongst the first structures built by humankind and are extremely important for a country's development. Due to the catastrophic consequences of a dam´s failure, its safety is of the utmost importance. The safety of Monte Branco dam, a masonry gravity dam with a small building attached to its downstream face, is assessed. This dam was built in 1749 and is located near Borba, upstream of the Tapada Real de Vila Viçosa. The safety of the dam is evaluated according to the Portuguese Regulations and is carried out in two parts: hydraulic safety and structural safety. In the first part, the maximum design flood, the maximum design reservoir elevation (MDRE) and the peak flow discharge are estimated. In the second part the static and seismic loads and the loading scenarios are defined. Afterwards, factors of safety for the global stability (overturning, sliding and uplifting) and the maximum stresses (tensile and compressive stresses) are calculated, considering different water levels. These stresses are then compared to the yielding stresses of the masonry. Also, it was determined if the building attached to the downstream face of the dam is critical to its global stability. The structural analysis is performed through simplified methods and finite elements models using SAP2000.

Keywords: safety assessment; masonry dam; flood; hydraulic safety; structural stability; earthquake

1. Introduction

Masonry dams usually have a gravity profile with either a straight plan development or a small arch with a polygonal development. These dams are usually ancient, as new building materials, such as concrete, led masonry construction to decline. Thus, these dams were built according to the scientific knowledge then available, so certain aspects were not considered (earthquakes, uplift, magnitude of the floods) which sometimes resulted in slimmer profiles than needed or in spillways with lesser discharge capacity than required. The catastrophic results of a dam's collapse justify the thorough studies that dams, especially antique dams, are subjected to.

2. Regulations for masonry dams

When this thesis was being concluded the Portuguese dam Regulation was Regulamento de Segurança de Barragens (RSB) (from 17/10/2007). Recently it was updated on March 28th, 2018. Monte Branco dam, which was previously covered by RSB (it was not labeled a large dam, but was covered by the large dam Regulation), is now labeled a small dam and is now covered by the updated version of the Regulation for small dams, Regulamento de Pequenas Barragens (RPB). But, since the design manual for the updated version of RPB hasn't been released yet and since the updated version of RSB doesn't differ much from the previous one, the new versions of the Regulations weren't used.

3. Case study

Monte Branco dam is a masonry gravity dam which has a small building attached to its downstream face which is thought to function as a buttress. The exterior area of the building is 5,40x9,80 m2 and the height is 5,30 m (Figure 3.1). The total thickness of the perpendicular walls to the dam is 1,80 m and the foundation slab is thought to be 0,70 m thick. This dam, built in 1749, is located near Borba, upstream of Tapada Real de Vila Viçosa and has a small arch, as shown on Figure 3.2. It's composed by schist masonry and cemented by a superior quality mortar. The angle of internal friction on the dam-foundation interface is 45º. A spillway with irregular geometry was dug on each of the abutments. The normal design reservoir elevation (NDRE) is that of the elevation of the right spillway's crest: 305,80 m [1].

Figure 3.1 – Downstream view of the building.

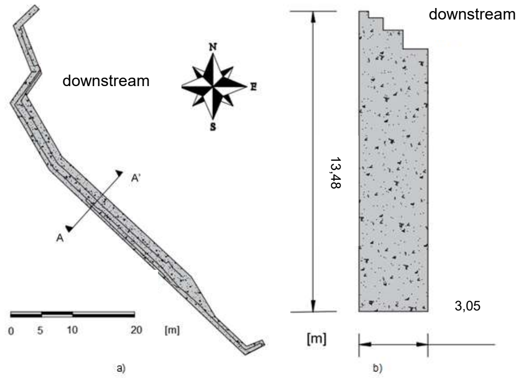


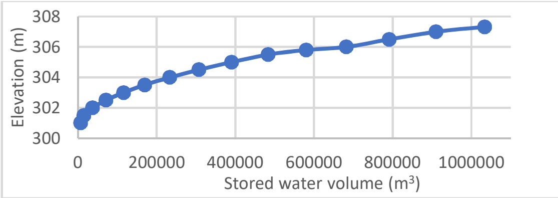
Figure 3.2 – Dam's plan and middle cross section.

The mortar covering the upstream face is slightly deteriorated near the NDRE and apart from some small cracks the masonry seems to be reasonably well preserved [1] .

There's reference of two outlets but only the existence of one of them is confirmed, with 0,10 m diameter. Its location, near the dam's base, indicates that it also functioned as a conduct [1].

The adopted yielding stress of the schist masonry is 6 MPa for the compressive stress and 0,30 MPa for the tensile stress [2]–[4].

The curve of stored reservoir volumes is shown in Graph 3.1 and the watershed in Figure 3.3.



Graph 3.1 – Stored water volumes curve.

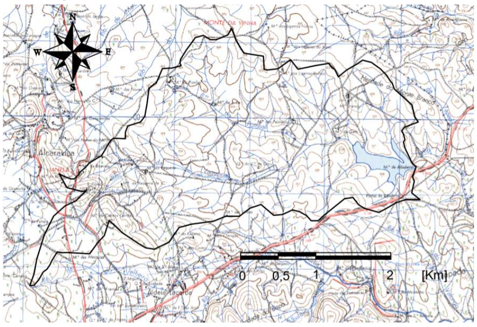


Figure 3.3 – Watershed of Monte Branco dam.

4. Hydraulic safety

4.1. Daily maximum annual rainfall

Daily maximum annual rainfall samples from Vila Viçosa's climate station (21M/01UG), closest to the dam's basin, were submitted to a statistic analysis. Then, the daily maximum annual rainfall regarding the return periods of 100, 500 and 1000 years was computed according to Gumbel's Law [5] (using Gringorten's formulation), as shown on Table 4.1.

Table 4.1 – Daily maximum annual rainfall.

Return period, Tr	Non-exceedance probability, *F(x)*	Maximum annual rainfall, Pdma (mm)
T= 100 years	0,99	94,76
T=500 years	0,998	113,45
T=1000 years	0,999	121,48

4.1.1. Design rainfall

Design rainfall is the maximum rainfall with the span of the time of concentration, for any given return period. This way, it's certain that all the watershed area contributes to the water flow at the dam site, resulting in a larger peak flood. The time of concentration, *tc*, is "the time that the raindrop that falls furthest in the watershed from the dam takes to reach it" [6]. For Monte Branco dam this is approximately of 2,3 hours. The design rainfall is based on the estimation of daily maximum annual rainfall and can be calculated according to IDF curves, adjusted to Portugal's watersheds [7], [8]:

$$i = at^b \quad (4.1)$$

where i (mm/h) is the average rainfall intensity that occurs during t minutes, a and b depend on the return period, the length of the rainfall and on the udographic station (Table 4.2). Monte Branco dam's watershed is in Portalegre's udographic station area. For each station, the ratio of two rainfalls, P_t/P_{24} (P_t: rainfall lasting t hours, P_{24}: rainfall lasting 24 hours) is constant, doesn't depend on the return period and is equal to $\mathrm{P}_{\mathrm{BH}_t}^{\mathrm{Tr}}/\mathrm{P}_{\mathrm{BH}_{24}}^{\mathrm{Tr}}$ (BH indicates that the rainfall occurs only in the dam's watershed and $\mathrm{P}_{\mathrm{BH}_{24}}^{\mathrm{Tr}}$ is the daily maximum annual rainfall calculated earlier), as long as the same return period is used. P_t/P_{24} is obtained from [7]:

$$\frac{P_t}{P_{24}} = \frac{a_t\, 60^{b_t}\, t^{(1+b_t)}}{a_{24}\, 60^{b_{24}}\, 24^{(1+b_{24})}} \quad (4.2)$$

Table 4.2 shows the values of a and b, for Portalegre's station.

Table 4.2 – Values of a and b - Portalegre's station.

IDF 18M01 Portalegre						
Time	Tr=100 years		Tr=500 years		Tr=1000 years	
	a	*b*	*a*	*b*	*a*	*b*
30 min-6 h	884,39	-0,739	1156,70	-0,756	1275,50	-0,761
6-48 h	392,58	-0,603	448,78	-0,597	473,11	-0,596

Design rainfalls, $\mathrm{P}_{\mathrm{BH}_{tc}}^{\mathrm{Tr}}$, are calculated multiplying P_t/P_{24} and $\mathrm{P}_{\mathrm{BH}_{24}}^{\mathrm{Tr}}$ and are: $\mathrm{P}_{\mathrm{BH}_{2,3}}^{100}$=43,0 mm, $\mathrm{P}_{\mathrm{BH}_{2,3}}^{500}$=51,9 mm and $\mathrm{P}_{\mathrm{BH}_{2,3}}^{1000}$=56,3 mm.

4.1.2. Design peak flood discharge

4.1.2.1. Rational method

For small to medium watersheds and if the rainfall's intensity is considered uniform and it lasts *tc* hours, then the peak flood discharge (Q_p), for a given return period, is calculated by:

$$Q_p = CiA \quad (4.3)$$

where A is the watershed's area and C depends on the type of soil and land use as it reflects rainfall losses and water soil storage. For areas smaller than 500 km^2 C can be considered as equal to 0,8 giving plausible peak flood discharge results [9].

4.1.2.2. Flood hydrograph

Flood hydrographs are chronographs of instant incoming flows, obtained from design rainfall's hyetographs. Two hyetographs with the duration of tc (138 min) for each return period were considered, with three rainfall blocks with similar durations: one with even and one with uneven rainfall. The hyetographs were obtained using the IDF curves referring to Portalegre's station. To compare peak flood discharges obtained from this method to those obtained from the rational method, the rainfall was multiplied by 0,8.

The hydrographs were obtained from the software HEC-HMS using the unit hydrograph model from Soil Conservation Service with a 0,6tc lag. The 1000 years' flood hydrograph is shown on Graph 4.3. The design peak flood discharges for each of the methods used are shown in Table 4.3:

Table 4.3 – Design peak flood discharges.

Design peak flood Qp (m^3/s)				
Return period Tr (years)		100	500	1000
Rational method		36,17	43,62	47,33
HEC-HMS	Even	36,50	43,90	47,20
	Uneven	44,10	53,40	57,50

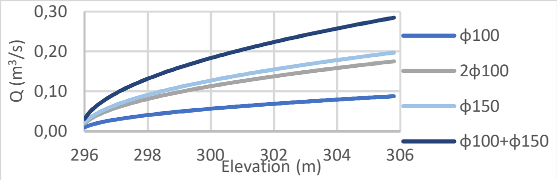
4.2. Discharge curves: spillways and conducts/outlets

To assess the emptying conditions of the dam, the discharge flows of some hypothetical conducts, with different diameters where studied, as shown on Graph 4.1, according to the following equation:

$$Q_v = C_c A\sqrt{2gH} \quad (4.4)$$

where Q_v is the discharged flow, C_c is the contraction coefficient which in this case is equal to 0,8 as it is an orifice in a thick wall [10], A is the area of the hole, g is the gravitational acceleration and H is the distance between the center of the hole and the water surface.

The graph shows that considering the hole whose existence is confirmed (0,10 m of diameter) it takes 86 days to completely empty the reservoir, starting from NDRR.



Graph 4.1 – Discharge curves for the hypothetical geometries of the outlets/conducts.

Spillways should have the capacity to discharge the design flood. As aforesaid their geometry is irregular so to be able to compute their discharge flow "equivalent sections" with the same areas (A), water heights (h) and equivalent width (b_{eq}) should be used, as shown on Figure 4.1.

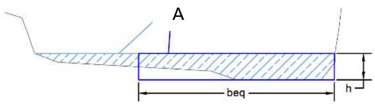


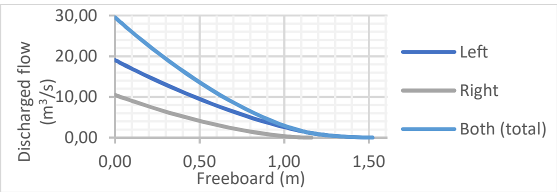
Figure 4.1 – Right spillway. Equivalent area.

The right spillway has a broad crest and the left one is thought to have a crest [1] so their discharge flow (Q) is computed by formulas (4.5) and (4.6):

$$Q = \left(\frac{2}{3}\right)^{3/2} b\sqrt{g}h^{3/2} \qquad (4.5)$$

$$Q = bc\sqrt{2g}h^{3/2} \qquad (4.6)$$

where b is the width of the spillway, g is the gravitational acceleration, h is the water height above the crest and c is the discharge coefficient and is equal to 0,48 [10]. Graph 4.2 shows the discharge flow for each of the spillways and for both.



Graph 4.2 – Spillways' discharge curves.

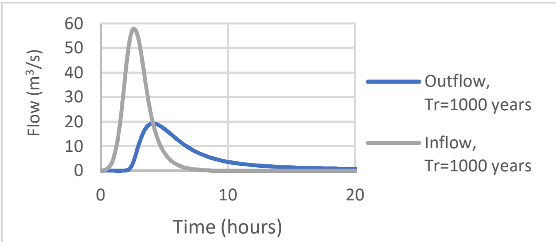
4.3. Outflow discharges

Hydraulic safety of Monte Branco dam was assessed from outflow hydrographs, built from reservoir stored water volumes curve and spillways' discharge curve, which were then compared to inflow hydrographs. The initial water level was assumed to be that of NDRE and the following two equations were used to compute the water volume of the reservoir:

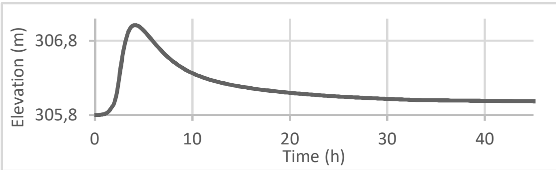
$$V_{stored} = V_{initial} + Q_{inflow} - Q_{outflow} \qquad (4.6)$$

$$V_{i+1} = V_i + \left(\frac{Q_{ii} + Q_{ii+1}}{2}\right)\Delta t - \left(\frac{Q_{oi} + Q_{oi+1}}{2}\right)\Delta t \quad (4.7)$$

where *i* and *i+1* denote consecutive steps for each *Δt* interval (15 min). This algorithm was automatized in Excel and for each computation step the stopping criterion was a maximum error of 0,001 m3/s between the outflow discharge given by (4.7) and the one given by the spillways discharge curve. Graph 4.3 shows the inflow and outflow hydrographs for the 1000 years' return period and Graph 4.4 shows the reservoir water levels for the 1000 years' design peak flood.



Graph 4.3 – Inflow and outflow hydrographs for the 1000 years' return period.



Graph 4.4 – Reservoir water level variation for the 1000 years' return period.

These graphs show that the maximum design reservoir elevation (MDRE) is 307,01 m, lower than the lowest point of the top of the dam (307,32 m) so overtopping does not occur for any design peak flood.

4.4. Freeboard

Freeboard is the distance between the crest of the dam and the maximum reservoir water elevation. It must be sufficient to prevent overtopping of the dam that could result from wind-generated wave action (wind setup and wave runup). Monte Branco dam has the following freeboards: 1,52 m for NDRE and 0,31 m for MDRE).

Fetch (Ft) is the maximum uninterrupted straight over-water distance, perpendicular to the axis of the dam and it's 0,36 km for Monte Branco dam. Due to the funneling effect, the direction of the design wind is close to that of the fetch and its velocity is 70 km/h for NDRE and 120 km for MDRE. The significant wave height (average height of the highest third of all waves), wave run up and wave setup were computed according to the methodology presented in *Memória n. º 828 – A folga em barragens* [11] and with an additional 0,30 m to due to seismic zones, the minimum freeboard is 0,78 m (<1,52 m) for NDRE and 1,19 m (> 0,31 m) for MDRE. Overtopping occurs for MDRE but being an extreme loading scenario with a very low probability of occurrence and given that overtopping doesn't significantly damage masonry dams, it is not a key safety issue.

5. Structural safety

In this part two structural models will be analyzed: gravity dam and dam with the building attached to its downstream face.

5.1. Loads and loading combinations

The following loading conditions were used in the safety assessment of Monte Branco dam:

- Usual loading condition: (i) NDRE with dead loads, uplift (and silt): (ii) the latter plus the effects of the Operation Basis Earthquake (OBE).
- Extreme loading condition: (i) MDRE with dead loads, uplift (and silt); (ii)The latter plus the effects of the Maximum Design Earthquake (MDE).

The stability criteria, including factors of safety (FS), are listed in Table 5.1 [12].

Table 5.1 – Safety criteria imposed by the Portuguese Regulation.

Loading conditions			Usual	Extreme
FS_{Stress}		Dam	2,4 - 4	< 4 (1)
Sliding	$FS_{Cohesion}$	Foundation	3 - 5	0
	$FS_{Angle\ of\ internal\ friction}$	Foundation	1,5 - 2	1,2 - 1,5
		Dam	-	1,5 - 2
(1) Factor of safety against material crushing on slender dams				

5.1.1. Return periods for seismic loading

The two design earthquakes are the Operation Basis Earthquake (OBE) and the Maximum Design Earthquake (MDE). In the Portuguese dam Regulations' design manual [13] there are no guidelines to estimate the return periods for the design earthquakes, so international regulations will be used. OBE has a return period of 145 (i.e. a 50% probability of being exceeded in 100 years) and is that for which there should be no severe damage to the dam [14]. The MDE is that for which some damage can be accepted but for which there should be no uncontrolled release of water from the reservoir. Either deterministic or probabilistic approaches can be applied to evaluate the MDE. The probabilistic approach takes into consideration the hazard: for dams whose failure would present a great social hazard the MDE will be characterized by a long return period – 10000 years. Monte Branco dam's risk class is moderate so the return period associated with the MDE is 2000 years (i.e. a 5% probability of being exceeded in 100 years) [14].

5.1.2. Design ground accelerations

Seismic action can be represented by the ground acceleration induced by it. The design ground acceleration (for type A soil – rock) is obtained by multiplying the importance factor, γ_I, by the reference peak ground acceleration, a_{gr} [15]. Importance factors relate to the consequences of a structural failure and are associated with four importance classes. Structures with higher importance have higher importance classes linked, and consequently higher importance factors which, in turn, mean higher return periods. Importance factors are computed by:

$$\gamma_I \sim \left(\frac{T_{LR}}{Tr}\right)^{-\frac{1}{k}} \qquad (5.1)$$

where T_{LR} is the reference return period (475 years, importance class II - γ_I=1), Tr is the return period for the earthquake and the value of k is 1,5 for Type 1 earthquake and 2,5 for Type 2 earthquake[1]. The ratio of the vertical design ground acceleration, a_{vg}, to the horizontal is 0,75 for Type 1 earthquake and 0,95 for Type 2 [15]. Table 5.2 shows the aforesaid parameters values.

Table 5.2 – Seismic ground accelerations for Type 1 and Type 2 earthquakes.

	AS1		AS2	
Seismic zone	1.5		2.4	
a_{gr} (m/s^2)	0,600		1,100	
γ_I	OBE	MDE	OBE	MDE
	0,453	2,607	0,622	1,787
a_g (m/s^2)	0,272	1,564	0,684	1,955
a_{vg} (m/s^2)	0,204	1,173	0,650	1,857

The table shows that Type 2 earthquake is more demanding to the structure, as ground accelerations as greater than those of Type 1. Furthermore, as it will be shown, the frequency of the structure is rather high, so it's more vulnerable to the effects of earthquake Type2.

5.1.3. Water-structure interaction

The seismic action disturbs the water in the reservoir causing dynamic pressures to be added to the static ones. Westergaard [16] studied this, and proposed two different formulations: hydrodynamic pressures, I_{wd} , for pseudo-static analysis (5.2) and added water masses, m_{wi}, for dynamic applications (5.3):

$$I_{wd} = \frac{7}{12} a_h \gamma_w h_w^2 \qquad (5.2)$$

$$m_{wi} = \frac{7}{8} \rho_w \sqrt{h_w (h_w - z_{wi})} A_i \qquad (5.3)$$

where a_h is the horizontal seismic coefficient, γ_w the water weight, h_w the water height, m_{wi} the mass to be added to i, ρ_w the water density, z_{wi} the water height above i and A_i the area of influence of i. The application point of (5.2) is 0,4 h_w.

5.2. Seismic coefficient method

This is a conservative method also known as pseudo-static method. Seismic loading is treated as an inertial force applied to the center of gravity of the dam, as it is considered a rigid body. Seismic forces are computed according to the principle of mass times the earthquake acceleration and the seismic coefficient is the ratio of the earthquake acceleration to gravity [17]. The seismic horizontal coefficients are 0,070 for OBE, 0,199 for MDE and the seismic vertical are 0,066 for OBE and 0,190 for MDE (type 2 earthquake).

The global stability of the structure is assessed for sliding (5.4), overturning (5.5) and uplifting (5.6):

$$FS_{Sliding} = \frac{\frac{\tan(\phi)\cdot(\sum F_V - U)}{\gamma_\phi} + \frac{c \cdot A_c}{\gamma_c}}{\sum F_H} > 1{,}0 \qquad (5.4)$$

$$FS_{Overturning} = \frac{\sum M_{Est}}{\sum M_{Inst}} > 1{,}0 \qquad (5.5)$$

$$FS_{Uplifting} = \frac{\sum F_V}{U} > 1{,}0 \qquad (5.6)$$

where ϕ is the friction angle , $\sum F_V$ is the sum of the vertical forces and $\sum F_H$ of the horizontal forces, U is the uplift, c is the cohesion on the sliding surface A_c, $\sum M_{Est}$ and $\sum M_{Inst}$ are the sum of the stabilizing moments and the non-stabilizing moments.

The vertical stresses will also be assessed as the yielding tensile stress of masonry is low, and the safety of dams is limited by these stresses.

5.2.1. Analysis results

Only the dam's middle cross section (the tallest, 13,48 m) was analyzed, with 1 m thickness. The path of the seismic forces was the most negative for each case. The results shown on Table 5.3 and Table 5.5 are for the gravity dam model and on Table 5.4 and Table 5.6 for the model with the dam

[1] In mainland Portugal. In Azores the value of k is 3,6, regardless of the Type of earthquake.

and the building. Figure 5.1 shows the used models.

Table 5.3 – Dead loads and seismic inertia loads.

			Gravity dam	Dam with building
Self weight		W (kN/m)	904,51	1220,76
		Distance (m)	1,53	5,81
Inertial seismic forces	Horizontal, $F_{sis,h}$ (kN/m)	OBE	63,16	85,24
		MDE	180,43	243,52
		Distance (m)	6,74	5,68
	Vertical, $F_{sis,v}$ (kN/m)	OBE	60	80,98
		MDE	171,41	231,34
		Distance (m)	1,53	5,81
Note: Moments are calculated at the downstream toe of the dam.				

Table 5.4 – Water loads (static and dynamic).

Water level (m)	Hydrostatic pressure		Uplift			Hydrodinamic loading		
			U (kN/m)	Distance (m)		OBE	MDE	Distance (m)
	I_w (kN/m)	Distance (m)		Gravity dam	Dam with building	I_{wd} (kN/m)		
307,01	846,30	4,34	198,40			68,95	196,96	5,20
305,80	696,20	3,93	179,95			56,72	162,02	4,72
305,00	605,00	3,67	167,75			49,29	140,80	4,40
304,50	551,25	3,50	160,13			44,91	128,29	4,20
304,00	500,00	3,33	152,50			40,73	116,36	4,00
303,50	451,25	3,17	144,88	2,03	7,41	36,76	105,02	3,80
303,00	405,00	3,00	137,25			32,99	94,25	3,60
302,50	361,25	2,83	129,63			29,43	84,07	3,40
302,00	320,00	2,67	122,00			26,07	74,47	3,20
301,50	281,25	2,50	114,38			22,91	65,45	3,00
301,00	245,00	2,33	106,75			19,96	57,02	2,80

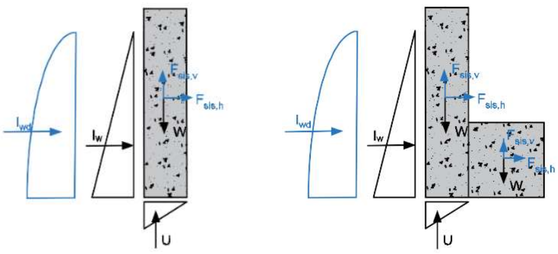


Figure 5.1 – Loads on the two models: static in black and dynamic in blue.

The former tables show the significant buttress effect of the adjacent building, as the FS increases when the global structure is considered.

Table 5.5 – Safety factors (gravity dam model).

Water level (m)	Sliding			Overturning			Uplifting		
	Static	Dynamic-OBE	Dynamic-MDE	Static	Dynamic-OBE	Dynamic-MDE	Static	Dynamic-OBE	Dynamic-MDE
307,01	0,7	0,4	0,4	0,3	0,3	0,2	4,6	3,5	2,4
305,80	0,7	0,5	0,4	0,4	0,4	0,3	5,0	3,8	2,6
305,00	0,8	0,6	0,5	0,5	0,4	0,3	5,4	4,0	2,7
304,50	0,9	0,7	0,6	0,6	0,5	0,3	5,6	4,1	2,7
304,00	1,0	0,8	0,6	0,7	0,5	0,4	5,9	4,3	2,8
303,50	1,1	0,8	0,7	0,8	0,6	0,4	6,2	4,4	2,9
303,00	1,3	0,9	0,7	0,9	0,6	0,4	6,6	4,6	2,9
302,50	1,4	1,1	0,8	1,1	0,7	0,5	7,0	4,8	3,0
302,00	1,6	1,2	0,9	1,3	0,8	0,5	7,4	5,0	3,1
301,50	1,9	1,3	1,0	1,5	0,9	0,5	7,9	5,2	3,2
301,00	2,2	1,5	1,1	1,7	1,0	0,6	8,5	5,4	3,3

The maximum and minimum stresses (tensile and compressive) are shown in Table 5.7 and Table 5.8 for the gravity dam model and for the model with the adjacent building.

Table 5.6 – Safety factors (model with the building).

Water level (m)	Sliding			Overturning			Uplifting		
	Static	Dynamic-OBE	Dynamic-MDE	Static	Dynamic-OBE	Dynamic-MDE	Static	Dynamic-OBE	Dynamic-MDE
307,01	1,0	0,6	0,5	1,4	1,1	0,8	6,2	4,4	2,8
305,80	1,0	0,8	0,6	1,7	1,3	0,9	6,8	4,7	3,0
305,00	1,2	0,9	0,7	2,0	1,5	1,0	7,3	4,9	3,1
304,50	1,3	1,0	0,7	2,3	1,7	1,1	7,6	5,1	3,1
304,00	1,4	1,1	0,8	2,5	1,8	1,2	8,0	5,2	3,2
303,50	1,6	1,2	0,9	2,8	2,0	1,3	8,4	5,4	3,2
303,00	1,8	1,3	1,0	3,2	2,1	1,3	8,9	5,6	3,3
302,50	2,0	1,4	1,0	3,6	2,3	1,4	9,4	5,8	3,4
302,00	2,3	1,6	1,1	4,0	2,5	1,5	10,0	6,0	3,5
301,50	2,6	1,8	1,2	4,6	2,8	1,6	10,7	6,2	3,5
301,00	3,0	2,0	1,3	5,2	3,0	1,7	11,4	6,5	3,6

Table 5.7 – Vertical stresses (gravity dam model).

Water level (m)	Static (MPa)		Dynamic-OBE (MPa)		Dynamic-MDE (MPa)	
	Min.	Max.	Min.	Max.	Min.	Max.
307,01	-2,73	2,14	-3,25	2,66	-4,23	3,64
305,80	-2,12	1,53	-2,59	2,00	-3,46	2,86
305,00	-1,78	1,19	-2,22	1,62	-3,02	2,43
304,50	-1,59	1,00	-2,01	1,42	-2,78	2,19
304,00	-1,42	0,83	-1,82	1,23	-2,56	1,97
303,50	-1,27	0,67	-1,65	1,06	-2,36	1,77
303,00	-1,13	0,53	-1,50	0,90	-2,18	1,59
302,50	-1,00	0,41	-1,36	0,76	-2,02	1,43
302,00	-0,89	0,29	-1,24	0,64	-1,88	1,29
301,50	-0,79	0,19	-1,13	0,53	-1,75	1,16
301,00	-0,70	0,11	-1,03	0,44	-1,64	1,05

Table 5.8 – Vertical stresses (model with the building).

Water level (m)	Static (MPa)		Dynamic-OBE (MPa)		Dynamic-MDE (MPa)	
	Min.	Max.	Min.	Max.	Min.	Max.
307,01	-0,48	0,19	-0,56	0,27	-0,70	0,44
305,80	-0,40	0,11	-0,47	0,18	-0,70	0,28
305,00	-0,35	0,07	-0,42	0,13	-0,63	0,22
304,50	-0,33	0,04	-0,40	0,11	-0,60	0,19
304,00	-0,31	0,02	-0,37	0,08	-0,56	0,17
303,50	-0,29	0,00	-0,35	0,06	-0,53	0,14
303,00	-0,27	-0,02	-0,33	0,04	-0,51	0,12
302,50	-0,25	-0,04	-0,31	0,02	-0,48	0,10
302,00	-0,23	-0,06	-0,29	0,00	-0,46	0,08
301,50	-0,22	-0,07	-0,28	-0,01	-0,44	0,07
301,00	-0,21	-0,08	-0,26	-0,03	-0,42	0,05

The buttress effect is also observed when analyzing the stresses (negative are compressive ones and positive the tensile ones). The distribution of stresses is as expected: compressive stresses on the toe and tensile stresses on the heel. Also, the maximum and minimum stresses don't occur simultaneously as the results are shown as an envelope of stresses. The results provided by this method may not be the most accurate (specially the stresses) as it is a simplification of the reality where the arch curvature of the dam is not considered nor the fixation on the abutments, so the dam is being analyzed as a cantilever. The model in which the building is considered is more rigid and safer than reality because only the part being supported by the building is being analyzed which is more rigid and that explains the substantial difference between the tensile stress distributions of the two models.

5.3. Finite element analysis in SAP2000

5.3.1. Modeling

The 3D modeling using finite elements aims to represent the mechanical characteristics of the dam as close as possible to the reality but with some simplifications: the irregular steps on the crest of the dam were modeled as 3 regular steps (width and height) with the lower step at 305,80 m (NDRE). The crest's height is irregular, so it was modeled with its average height. The finite element mesh was modeled as orthogonal and with constant element height, to make it easier to compute and add the water masses, and with three elements on the width. Restrains to all degrees of freedom exist on the interface of the dam with the foundation and abutments as the rocky massif is very sturdy and compact [1] so there's no associated spring effect. The dam and building schist masonry was modeled as homogeneous isotropic, with Young's module equal to 1 GPa and weight equal to 22 kN/m^3..

The deformed shapes for the first mode, for the two models (gravity dam and dam with the building) are shown in Figure 5.2. The translation associated with the first mode is along the axis of the valley.

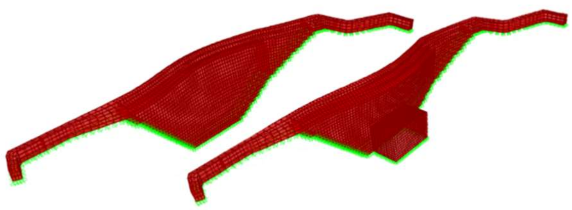


Figure 5.2 – First modal shapes for the gravity dam (left) and dam with the building (right).

The first frequencies are 3,77 Hz for the gravity dam model and 4,14 Hz for the model with the building. The latter model has a higher frequency for being more rigid than the former. For the dynamic scenarios, the added water masses decrease the frequency of the model.

5.3.2. Response spectrum analysis

The dynamic analysis determines the structural response using a response spectrum for the dynamic input, for which the results are computed for each vibration mode, then combined to produce a single, positive and maximum result. This method is suitable to compute dynamic results for dams as it only considers the elastic behavior of the structure. The seismic action is considered through vertical and horizontal response spectrums as indorsed by Eurocode 8 [15], for Type 2 earthquake with the soil factor, S, equal to 1 (rock) and a modal damping of 5 %.

5.3.3. Analysis results

The results shown are for the dam´s central cross section, which is where the maximum stresses occur. Since the dynamic analysis doesn't provide principal stresses, these must be calculated, using Mohr's circle, as plane strain typically develops in these types of structures where one dimension is significantly greater than the other two. Table 5.9 shows the maximum and minimum principal stresses, which occur on the heel and toe of the dam, for the gravity dam model and Table 5.10 for the model with the building. Graph 5.1 shows only the maximum principal stresses for the latter model. Table 5.9 and Table 5.10 show that the building reduces by 35 % the tensile stress level on the dam so it's evident that it works as a buttress.

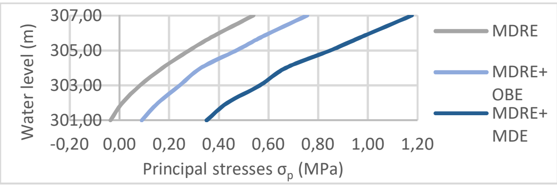
Table 5.10 and Graph 5.1 show that the allowable compressive stress is not exceeded for any water level for any scenario. However, the allowable tensile stress is exceeded even for the lower water levels for dynamic scenarios, such as for the results from the pseudo-static analysis, so the Dynamic-MDE scenario is the limiting scenario for the safety of the dam. Graph 5.1 also shows that the same water level is safe for static loading, where compressive stresses occur in the heel, for lower water levels. The yielding stress of the masonry is only reached for water levels above NDRE. Except for the 1755 earthquake, since the construction of the dam there haven't occurred any earthquakes with the intensity of the design earthquakes, and, over the years, the reservoir elevation has been at NDRE and possibly at MDRE without the collapse of the dam. Table 5.11 shows the eccentricity of the forces for the lower water elevations of dynamic scenarios, where lower tensile stresses occur.

Table 5.9 – Maximum principal stresses for the middle cross section, for the gravity dam model.

Water level (m)	Static		Dynamic-OBE		Dynamic-MDE	
	Min.	Max.	Min.	Max.	Min.	Max.
307,01	-1,27	0,78	-1,55	1,07	-2,11	1,61
305,80	-1,05	0,55	-1,31	0,81	-1,83	1,33
305,00	-0,92	0,43	-1,17	0,68	-1,67	1,15
304,00	-0,78	0,28	-1,01	0,52	-1,47	0,98
303,00	-0,65	0,16	-0,87	0,37	-1,29	0,79
302,00	-0,55	0,06	-0,75	0,26	-1,16	0,65
301,00	-0,47	-0,01	-0,66	0,16	-1,04	0,53

Table 5.10 – Maximum principal stresses for the middle cross section of the model with the building.

Water level (m)	Static		Dynamic-OBE		Dynamic-MDE	
	Min.	Max.	Min.	Max.	Min.	Max.
307,01	-0,97	0,54	-1,16	0,76	-1,54	1,18
305,80	-0,82	0,38	-1,00	0,58	-1,36	0,98
305,00	-0,74	0,28	-0,91	0,47	-1,25	0,85
304,00	-0,64	0,17	-0,80	0,33	-1,11	0,67
303,00	-0,55	0,08	-0,70	0,24	-0,99	0,57
302,00	-0,48	0,01	-0,62	0,16	-0,89	0,43
301,00	-0,42	-0,04	-0,54	0,09	-0,80	0,35



Graph 5.1 – Tensile stresses on the heel of the dam for the model with the dam and the building.

Table 5.11 shows that for the Dynamic-MDE scenario, the resultant force acts within the base of the dam only for water elevations below 302 m, but at 303 m water level it's only 0,32 m outside of the base but still well inside the base of the dam with the building. That indicates that the dam suffered a slight rotation towards downstream that is being supported by the building and explains a slight cracking occurring near the heel, which does not result in the collapse of the dam, but in the local yielding of the masonry (around 0,40 m), as shown in

Figure 5.3. For the Dynamic-OBE at the water level of 303 m the yielding tensile stress isn't exceeded, but the allowable stress is. Yet, the Portuguese Regulations state that for extreme loading scenarios some damage can occur if it doesn't end up in the collapse of the structure, so it's reasonable to keep the water level at 303 m, which still provides 0,12 hm^3 of water storage.

Table 5.11 – Eccentricity for dynamic scenarios on the middle cross section.

Water level (m)	Dynamic-OBE			Dynamic-MDE		
	N (kN/m)	M (kNm/m)	e (m)	N (kN/m)	M (kNm/m)	e (m)
303,00	694,65	729,51	1,05	653,53	1208,56	1,85
302,50	700,05	664,28	0,95	676,47	1118,66	1,65
302,00	705,45	599,05	0,85	699,41	1028,76	1,47
301,00	690,98	490,95	0,71	689,61	894,42	1,30

Note: M was calculated at the center of the base of the dam.

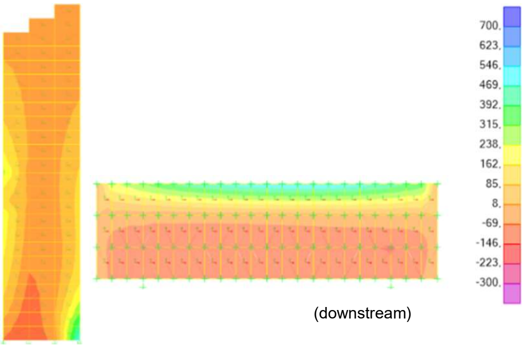
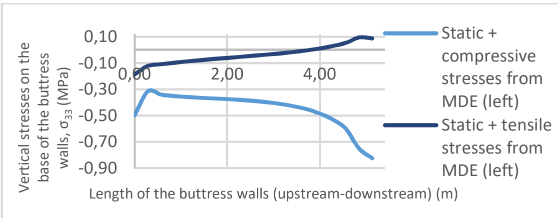


Figure 5.3 – Maximum stresses distribution for Dynamic-MDE scenario, for 303 m of water level (model with building).

The stresses on the left buttress wall (more loaded) for Dynamic-MDE scenario are displayed in Graph 5.2, which shows that the occurring stresses (compressive and tensile) are relatively low compared to yielding stresses, which indicates that the buttresses are safe and able to uphold the dam.



Graph 5.2 – Stresses on the left buttress wall for Dynamic-MDE scenario.

6. Conclusions and future work

This work addresses the main aspects of the safety of dams, namely the hydraulic and structural safety. First, it was concluded that Monte Branco dam is not overtopped by the design floods but may be by wind-generated waves when in MDRE, which is not severe for masonry dams. If the emptying time of the reservoir needs to be shortened it's recommended the installation of siphons on the top of the dam. In the second part it was concluded that the building is critical for the stability of the dam as it provides a buttress effect, rises the factors of safety and lowers the stresses. For water levels of 303 m the dam is safe for Static and Dynamic-OBE scenarios and is reasonably safe for Dynamic-MDE scenario.

It would be interesting to obtain the yielding stresses and assess the conservation state of the masonry, based on tests and define a methodology of intervention to fix the areas where the mortar is eroded and cracking exists. Additionally, depending on the latest version of the Regulations or if the water storage needs to be larger, the installation of metallic buttresses could be considered. Also relevant would be to analyze the dam with a non-linear approach, with a deterministic definition of the design earthquakes and a 3D modeling of the foundation and abutments to obtain more reliable results.

References

[1] Tetraplano, "Barragem do Monte Branco - Relatório da inspeção detalhada," 2002.

[2] L. Robles & F. Peña, "Comportamiento estructural de presas antiguas de mampostería", 2016.

[3] R. S. Barros, D. V. Oliveira, H. Varum, C. A. S. Alves & A. Camões, "Experimental characterization of physical and mechanical properties of schist from Portugal," Constr. Build. Mater., vol. 50, pp. 617–630, 2014.

[4] USBR, "Design of gravity dams. Design manual for concrete gravity dams." Denver, 1976.

[5] C. C. L. B. Pereira, "Análise de precipitações intensas," Tese de Mestrado. Instituto Superior Técnico, 1995.

[6] A. de C. Quintela, "Hidrologia e Recursos Hidricos," Lisboa: Folhas de apoio à disciplina de Hidrologia e Recursos Hídricos. Associação de Estudantes do Instituto Superior Técnico, 1996.

[7] M. M. Portela, "Estimação de precipitações intensas em bacias hidrográficas de Portugal Continental," Recur. Hídricos, vol. 27, no. 1, pp. 15–32, 2006.

[8] C. C. L. B. Pereira, R. Rodrigues, & J. Pinto da Costa, "Análise de fenómenos extremos. Precipitações intensas em Portugal Continental," Lisboa, DSRH, p. 64, 2001.

[9] A. de C. Quintela, "Hidrologia de águas superficiais," in Curso Internacional de Hidrologia Operativa, 1984.

[10] A. de C. Quintela, Hidráulica, 11o ed. Lisboa: Fundação Calouste Gulbenkian, 1981.

[11] R. Martins, "A folga em barragens: Memória n.o 828." Laboatório Nacional de Engenharia Civil, Lisboa, 2002.

[12] Portaria n.o 846/93. Lisboa: Diário da República, 1a série-B - N.o 213, 1993.

[13] Decreto-Lei 344/2007. Lisboa: Diário da República, 1a série - N.o 198 - 15 de Outubro de 2007, 2007.

[14] ICOLD, "Selecting seismic parameters for large dams - Guidelines Bulletin 72." 2009.

[15] NP EN 1998-1, Eurocódigo 8: Projecto de estruturas para resistência aos sismos, Parte 1: Regras gerais, acções sísmcas e regras para edifícios. Instituto Português da Qualidade, 2009.

[16] H. M. Westergaard, "Water Pressures on Dams during Earthquakes," Trans. Am. Soc. Civ. Eng., vol. 98, pp. 418–433, 1933.

[17] USACE, "Gravity Dam Design." Washington, DC, 1995.