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Designing an earth dam

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Abstract: Dams are structures designed to retain and accumulate water, and the art of designing a dam is linked to the art of controlling the flow of water through the dam-foundation assembly (Cruz, 2004). Studies show that the process of internal erosion is one of the main causes of dam failure worldwide. According to a publication by the U.S. Department of the Interior Bureau of Reclamation (2007), the total number of dam failures related to internal erosion is 49.60%. One of the methods used to control flow is the introduction of internal drainage devices in the dam and foundation which, if properly dimensioned, fulfills the purpose of controlling flow within the massif. In drainage system projects, the assumptions and procedures adopted for sizing must be highlighted, as this system is closely linked to the safety of the structure. Filters must be designed to meet the basic criteria of retention and permeability, which at first appear to be antagonistic criteria. However, these conditions assume that the dimensions of the voids in the filter material must be small enough to retain the larger particles of the protected material and, at the same time, the material must have sufficient permeability to prevent the induction of high percolation forces and hydrostatic pressures applied to the filters.

Key words: dam; strength of materials; drainage; filtration

1 Preliminary studies

The objective of this academic work is not to study the design of a dam as a whole, but rather an anti-design of the dam and its characteristics. Therefore, based on data from the hydrological and climatic studies provided in the aforementioned work, which considered the flow of the river and the meteorological conditions of the region, the main characteristics of the dam and other important hydrological data for the design of the typical section of the dam were obtained.

crista da barragem	473,00
$NA_{máx}$ de operação a montante	469,00
$NA_{mín}$ de operação a montante	459,00
Superfície do terreno natural na seção de maior altura	413,00
NA de jusante	413,00

Figure 1. Input data

Source: Author

2 Geotechnical investigation

During the basic design phase, a survey of the geological-geotechnical conditions of the site was completed. The objective of this geotechnical investigation was to characterize the foundation mass in terms of its support capacity for the structures, and to verify the hydraulic conductivity in the dam area.

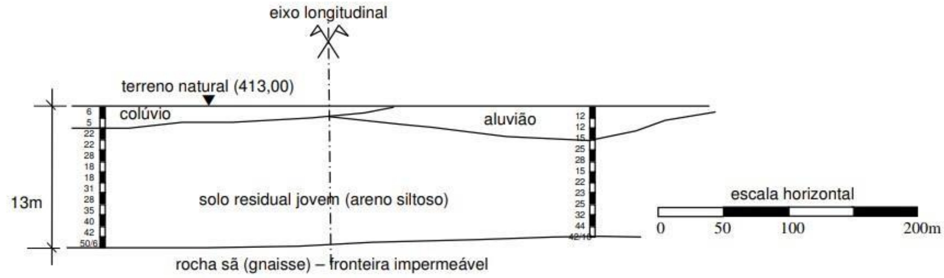

Figure 2. Longitudinal section

Source: Author

3 Geological section

Figure 3 shows (with a horizontal scale equal to the vertical scale) the geological section of the dam axis, showing the layers that make up the subsoil.

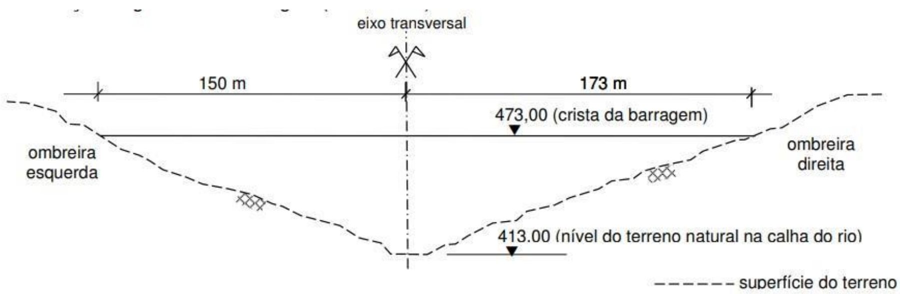

Figure 3. Cross section

Source: Author

4 Laboratory tests

Laboratory tests must be carried out in order to obtain the resistance parameters of the foundation materials and the materials from the borrow pits that will be used in the body of the dam.

Material	γ_{sat} (kN/m³)	k (m/s)	Diâmetro (mm)				
			D_{10}	D_{15}	D_{50}	D_{60}	D_{85}
Solo argilo-siltoso compactado (corpo do aterro da barragem) LL=45 LP=23 C_v= 5x10^{-2} m²/h	19	3 x 10^{-7}	0,003	0,004	0,01	0,03	0,08
Areia fina	18	5 x 10^{-5}	0,09	0,15	0,30	0,35	0,50
Areia média e grossa	18	8 x 10^{-4}	0,25	0,30	0,80	0,95	1,70
Pedregulho	19	5 x 10^{-3}	0,8	1,6	5,5	7,0	10
Brita 1	19	10^{-2}	7	8	16	18	25
Brita 2	19	10^{-2}	22	24	30	32	35
Enrocamento 1	20	10^{-1}	50	90	300	310	400
Enrocamento 2	20	10^{-1}	20	40	200	240	350
Aluvião	17,5	2 x 10^{-6}	0,08	0,12	0,12	0,12	0,14

Figure 4. Geotechnical characteristics

Source: Author

5 Study of alternatives

In a brief study of the geological section in Figure 3 and the surveys carried out in the areas where the dam will be installed, it was observed that the earth dam option would be a good option, given that the foundation support material would be able to absorb the loads well. However, the relevant criteria related to costs were not informed in the project scope, although the estimated deadline is 2 years. Economic criteria are important, since it is necessary to know the quantities of materials available and the distances between the deposits and the construction site, in order to evaluate the best option. Since this information was not provided, we assumed that the most abundant and closest available material would be silty clay soil, so our dam will be of the type with the entire mass in earth, with a chimney drain and a drainage mat.

6 Dimensioning

According to Cruz (1996) the following slope inclinations can be adopted based on experience.

Material da barragem	Talude de Montante	Talude de Jusante
solos compactados	2,5(H):1,0(V)	2,0(H):1,0(V)
	3,0(H):1,0(V)	
solos compactados argilosos	2,0(H):1,0(V)	2,0(H):1,0(V)
	3,0(H):1,0(V)	2,5(H):1,0(V)
solos compactados siltosos	3,5(H):1,0(V)	3,0(H):1,0(V)
enrocamentos	1,3(H):1,0(V)	1,3(H):1,0(V)
	1,6(H):1,0(V)	1,6(H):1,0(V)

Figure 5. Sizing

Source: Author

Since the available material is a silty clayey soil, we chose to adopt Cruz's (1996) suggestion for compacted soils according to Table xx. The geometry of the core was defined by adopting a width of 6 m at the crest with an option for vehicle traffic, according to the recommendation of Eletrobrás (2003), which defines this minimum value as 3.0 m wide, based on construction aspects, and a slope of 4 (V): 1 (H), to avoid the problem of the core hanging on the backboards. This complies with the recommendation of Cruz (2004), which states that, in principle, the width of the clayey core should be used at any point, 30% to 50% of the height of the water sheet above the point. Therefore, the adopted values will be 3.0 (H): 1.0 (V) for upstream slopes and 2.0 (H): 1.0 (V).

7 Flow

Flow lines and equipotentials for calculating the contribution flow of the vertical drain.

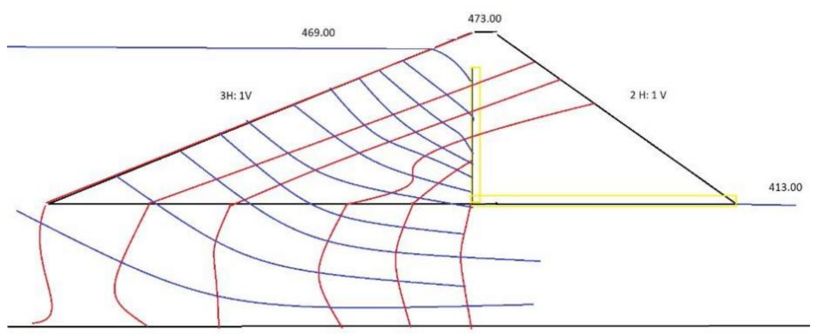

Figure 6. Hydraulic gradient

Calculation of flow rate Q:

$$Q = \frac{khNf}{Nd}$$

$$Q = 3\ x\ 10^{-7} x\ 56\ x \frac{7}{5} = 2{,}4x10^{-5} \frac{m^3}{s}/m$$

As the design flow rate = 10 x Q calculated.

Considering the flow to be practically vertical, how can we then consider that the hydraulic gradient i can be approximately a unitary value?

$$i = \frac{\Delta h}{l} = 1$$

Whereas:

$Q_{PROJ} = 10 \ x \ Q_{CÁL}$

$Q = k.\ i.\ A$

$Q = kv.\ 1.\ Bv$

$$\frac{24x\ 10^{-5}}{8\ x\ 10^{-4}} = 1.\ Bv$$

$Bv = 0{,}30m$ According to Cruz (1998), $B_f > 0{,}8$m we will adopt a drainage mat of 0.90 m.

It is considered that imax along the mat = 10% (normally, due to this, there is a piezometric line above the mat, which has to be taken into account in the stability analysis).

$Q2 = Q1 + Qfundação$

$$Q2 = 24\ x\ 10^{-5} + 5\ x\ 10^{-5} = 29\ x\ \frac{10^{-5}m^3}{s}/m$$

$$B_H = 10.\ \frac{Q}{k_f} = 290x\ \frac{10-5}{8}\ x\ 10_{-4} = 3{,}6\ m$$

Hypothesis 1: the filter works under load (linear variation of HT with L)

$$B_h = \sqrt{\frac{Q.L}{k_{fh}}}$$

$$B_h = \sqrt{\frac{29\ x\ 10^{-5}.\ 123}{8\ x\ 10^{-4}}} = 6{,}67\ m$$

Hypothesis 2:

$$B_h = \sqrt{\frac{2.Q.L}{k_{fh}}}$$

$$B_h = \sqrt{\frac{2\ x\ 29\ x\ 10^{-5}.\ 123}{8\ x\ 10^{-4}}} = 9{,}45\ m$$

All results were above B_h, they were greater than 2.0. We will soon dimension the drainage mat with a sandwich shape.

We will adopt $B_h = 1{,}8\ m$ as the weighted average of the permeability coefficients.

$$k_{eq} = \frac{0{,}5\ x\ \ 10^{-4} + 0{,}5\ x\ \ 10^{-4} + 0{,}8\ x\ 8\ x\ 10^{-2}}{1{,}8} = 0{,}035m/s$$

$$B_H = 10.\ \frac{Q}{k_f} = 10\ x\ 29x\ \frac{10-5}{0{,}035} = 0{,}08\ m$$

Therefore, the drainage mat will be of the sandwich type, being:

$h_{areia}\ grosaa = 0{,}50\ m\ h_{areia}\ grossa = 0{,}50\ m\ hbrita\ 01 = 0{,}80\ m$

Longitudinal dimensioning: direction/sense

For the dimensioning of the transversal drainage, one intermediate cut-off will be adopted between the left shoulder and one cut-off on the right shoulder, in order to reduce the equivalent height of the drainage tape.

Therefore:

$$B_h = \frac{QC}{ki'L}$$

B_h= Drainage mat thickness; C= Longitudinal length of the carpet

i' = tangent of the angle of inclination of the terrain in the longitudinal direction; L= transverse length of the carpet;

1- Ombreira esquerda C1= C2=86,5;
2- L= 123,0 m;
3- i'= 0,35;
4- Q=29 $x\ 10^{-5} m^3/m/s$;

$$B_h = \frac{QC}{ki'L}$$

$$B_h = \frac{29\ x\ 10^{-5} x\ 86{,}5}{0{,}035\ x\ 0{,}35\ x\ 123{,}0} = 0{,}02m$$

5- Ombreira Direita C1= C2=75;
6- L= 123 m;
7- i'= 0,4;
8- Q=29$x\ 10^{-5} m^3/m/s$;

$$B_h = \frac{QC}{ki'L}$$

$$B_h = \frac{29\ x\ 10^{-5} x\ 75}{0{,}035x\ 0{,}4\ x\ 123{,}0} = 0{,}01m$$

Thus an equivalent drain height of B_h =1,80 m in sandwich format will be adopted.

7.1 Filter

The filter material, used downstream of the dam core, has the function of avoiding the grain carrying of the neighboring material to be protected, and therefore, a correct dimensioning of the transition material is essential to ensure the integrity of the dam body. The requirements that the materials used in the transition must meet only concern the granulometry criteria; As the transition section is part of the drainage system, it must meet containment (non-segregation) and permeability requirements. In this study, analyses of filter material were performed considering the criteria shown by Cruz (2004) and the U.S. Army Corps of Engineers (2000). For the filter, the design was performed considering the Cruz method, but respecting the minimum values of the U.S. Army Corps of Engineers method for D15, because the soil to be protected is very thin.

To perform the design of the filter, it was based on the grain size range referring to the clay soil of the core, from the loan area and medium coarse sand of the internal drainage system.

1- D_{15} of the filter has to be $< 5\ x\ D_{85}$ to the solo (core): D_{15} of the filter $< 5\ x\ 0{,}08$ = 0,4 mm.

$D15\ filtro$=0,30mm. OK.

2- $5\ x\ D_{15}$ I only give $< D_{15}$ from the minimum value filter (Corps of Engineers) = 0,1 mm.

5 X 0,004 < 0,30=OK. D ______ $60\ (máx)$ $^{D}10\ (máx)$

3- Non-uniformity coefficient (Cruz) < 20

$C = \frac{0,95}{0,25} = 3,8$= OK!

4- sandy filters no

5- must contain grains with a diameter greater than 2. OK.

6- By Terzaghi's criterion:

$5\ x\ D15\ do\ solo < D15\ do\ filtro < 5\ x\ D85$ do solo

0,02 < 0,30 < 0,04 OK!

The upstream slope will be protected using a special layer of protective rockfill (rip-rap). The wave height was estimated, through hydrological studies, at 1.10 m.

Rip-rap is made up of stones that must be large enough not to move, that can withstand the impact of waves and that can withstand drying and wetting cycles. Sherard et al. (1963) and Mendonça (2013), suggest a correlation between the height of the maximum wave and the average thickness and diameter that the rip-rap should have, as shown in Table 1.

Table 1. Correlation between wave height and rip-rap diameter and thickness (SHERARD et al., 1963 apud MENDONÇA, 2012)

Altura da onda máxima (m)	Diâmetro médio do rip-rap (cm)	Espessura do rip-rap (cm)
< 0,60	25	30
0,60 a 1,20	30	45
1,20 a 1,80	37	60
1,80 a 2,40	45	75
2,40 a 3,00	55	90

Source: Author

Sherard et al. (1963) and Mendonça (2012), also suggest minimum values for the thickness of the transition layer or rip-rap (Table 2) and the layer of soil on the slope that should be executed if the rip-rap does not contain fines.

Table 2. Correlation between wave height and minimum rip-rap transition thickness (SHERARD et al., 1963 and MENDONÇA, 2012)

Altura da onda (m)	Espessura mínima de transição (cm)
< 1,20	15
1,20 a 2,40	22,5
2,40 a 3,00	30

Source: Author

The minimum and average diameters for the chosen material, type 2 rockfill, are limited by the equations:

$Dmáx = 1,5\ .\ D50$

$D_{máx} = 1,5\ .\ 300 = 45\ cm$

$0,6.\ D_{50} > D_{mim} > 2,5\ cm$

$180 >.\ D_{mim} > 2,5\ cm$

The extension of the rip-rap along the height of the slope must follow the following criteria (CORPS OF ENGINEERS, 1971 and MENDONÇA, 2012): From the crest of the dam to 1.5 m below the N min (maximum upstream water level). It can be extended to a lower level if you are concerned about waves during filling.

For a wave height of less than 1.10 m, the average diameter is 30 cm and the rip-rap thickness is 45 cm (Table 1). The maximum and minimum diameters are obtained by the equations above. The maximum diameter value is then 45.0 cm and the minimum is between 2.5 and 18 cm. A rip-rap layer of 45 cm thick was therefore adopted, D_{mim} = 10cm, $D_{médio}$ = 30cm e$D_{máx}$ =45,0 cm, extending it from the 473 m to 460 m mark, and the rip-rap must extend from the crest mark to at least 1.5 m below the NAmín.

On downstream slopes, erosion is prevented by using low vegetation, benches (every 10 m of the dam height) and surface drainage channels.

7.2 Cut off

Since the flow rate at the foundation is greater than 5.0 L/min/m and $k_{fund} > 10^{-4}$ then using the Cedergren (1988) graph, we can estimate the reduction in flow rate along the foundation, thereby also reducing the hydraulic gradient.

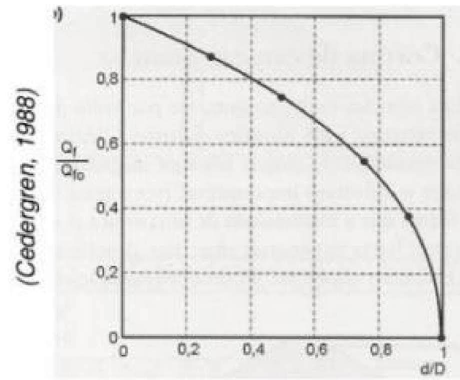

Figure 7. Gradient graph

Source: Author

Where:

Q_{fo} = Trenchless foundation flow rate;

Q_f = Foundation flow rate with trench; D= Foundation thickness; d= Trench thick; In this case we will adopt a trench with the relationship between $\frac{d}{D}$=0.6, which corresponds to a relationship.

of flow equal to $\frac{Q_f}{Q_{fo}} = 0,64 = 0,64 \; x \; 7 = 4,5 \; \frac{l}{mim} /m$

Width of the base of the cut-off according to (Cruz, 1996):

$b = 0,25 \; x \; H_{ÁGUA} = 0,25 \; x \; 56 = 14,0 \; m$

Mín: 4,0 m
Máx: 20,0 m

Taking into account that the section near the foot of the downstream slope has a 3 m thick surface layer of sandy alluvium (γsat = 17.5kN/m^3) which, according to the foundation flow network, will be subjected to a vertical flow with a hydraulic gradient of 1.0, and will be in this condition up to 8 m after the foot of the downstream slope. This layer will not be removed. There is great concern about the stability of the downstream flow since the desired values of i are in the range of i: < 0.4 or 0.5 and $FS = \frac{i_{crit}}{i}$ = 2 or 2.5.

For the case in question:

γsub $\gamma sub - \gamma w$

$i_{crit} =$ 0.75

γw γw

i_{crit} 0.75

$FS =$ 0.75

i 1

The safety factor value is well below the recommended value. The solution is to reduce the hydraulic gradient. Solutions:

1- Design cut off to reduce flow through the foundation;

Considering that with the addition of the cut off there was a 36% reduction in the flow that passes through the foundation, an analysis will be made of the new hydraulic gradient corresponding to the reduction in flow.

Converting 7l/min = 0.000116667 m³/s/m

Q 0,000116667

$i = k\_\_\_ \quad \_\_\_\_\_\_\_\_\_\_ A = 3 \; x \; 10_{-7} x \; A = 1$

$A = 389 \; m^2$

Para a nova vazão com redução de 36% após a instalação do ***cut off***

Q $7{,}5 \; x \; 10^{-5}$

$i = k\_\_\_ \quad \_\_\_\_\_\_\_\_\_\_ A = 3 \; x \; 10_{-7} x \; 389 = 0{,}64$

i_{crit} 0,75

$FS =$ $= \_\_\_ = 1{,}17$

i 0,65

The safety factor was below the recommended 2.5, so for economic reasons, and it is not feasible to extend the cut-off to the foundation, but rather to design a drainage balance berm at the foot of the slope downstream.

Check the upstream bottom survey:

$$FS = \frac{\gamma Sat x \; D'}{ubase}$$

Calculation of pore pressure at the base of the downstream slope.

$H_T = Z_P + H_P$

$(56 + 13) - \frac{56}{7} X \; 7 - 10 = Z_P + H_P$

$3{,}0 \; m = Z_P$

$u_b = Z_P \; x \; \gamma_w$

$u_b = 30$

$$FS = \frac{17{,}5x3}{30} = 1{,}75$$

2- Design a downstream drainage berm. Calculation of the height of the outlet drain to avoid (uplift) the rupture of this (blowout):

We will use a $\gamma_{médio \; filtro} = 19KN/m^3$

$$2{,}5 = \frac{\gamma Sat x \; D' + zbx\gamma b}{ubase}$$

$$2{,}5 = \frac{17{,}5\ x\ 3 + z_b x 19}{30}$$

$z_b = 1{,}2m$

8 Slope stability during construction

The instability situation may occur if considerable positive pore pressures occur in the earth mass, generated by the overload due to the raising of the embankment. In this specific work, where the construction time is very fast (2 years), an undrained situation will be considered (permanent increase in pore pressure after construction). This expectation of behavior is essential for carrying out the stability analysis that coherently represents the behavior that will occur in the field. The type of behavior depends on the construction time of the embankment and the material that makes up the embankment, which in this case is a clayey material with undrained behavior.

For analyses in terms of effective stresses, the pore pressure parameter B (undrained) is used to estimate the excess pore pressure in cases of undrained behavior according to tests in laboratories not provided in this work. However, with triaxial tests and their respective Skempeton parameters, the pore pressures are defined as;

Δu

$B =$ ________

$\gamma aterro.\ h$

In this case for a saturated situation.

9 Stability after rapid downgrade

Upstream landslides due to subsidence, although serious, generally do not place the dam at risk of immediate disaster, since the water level is well below the crest of the affected mass. Sherard (1953) mentions that such landslides are usually deep and associated with low-strength foundations, and that the occurrence of surface failures is much less frequent. The stability of the upstream slope due to subsidence depends on the pore pressures that will exist in the clay mass after subsidence. Stability analyses after rapid subsidence can be made in terms of effective stresses (c' and Φ'), and the pore pressures after subsidence should be considered. To dimension this massif, we will consider a large part of the anchoring upstream of the dam, as the material is available, thus avoiding that, after the rapid lowering of the water level of the dam, it is not at the same speed as the water level of the core and takes a long time to occur, thus avoiding instability to the slope upstream, and avoiding erosion since the dimensioning of the slope protection according to the calculations indicating type 2 rockfill.

10 Final section

Segundo cruz (1996):

1- In dams with H > 25-30m, inclined drain instead of vertical (better stress distribution due to the difference in stiffness that can cause cracking or hydraulic rupture of the drain). De Mello recommends an inclination of the vertical drain upstream of 1(V): 0.5(H).

2- Thickness of vertical or inclined drain > 0.8 m (construction reasons).

3- Horizontal drain thickness < 2.0m (economic reasons) – If this is not sufficient, use a "sandwich drain".

4- Height of foot or outlet drain > 2 x thickness of horizontal drain;

5- Crest width of foot drain > 4.0m.

6- Thickness of transition filters: > 2.0m between rockfill dam and soil core > 0.3m in other cases;

7- Filter at the core-rockfill and core-foundation interface;

8- Maximum inclination of 4:1 (V:H) to avoid the core "hanging" on the backrests;

9- Minimum width of the core (when central) of 30% to 50% of the water height above the considered point, being at least 10 m.

Considering all the dimensions and recommendations from the major bibliographical references, we adopted the following section as being the most efficient section regarding the dimensioning of dam stability, however, more information is needed for economic, social and environmental issues.

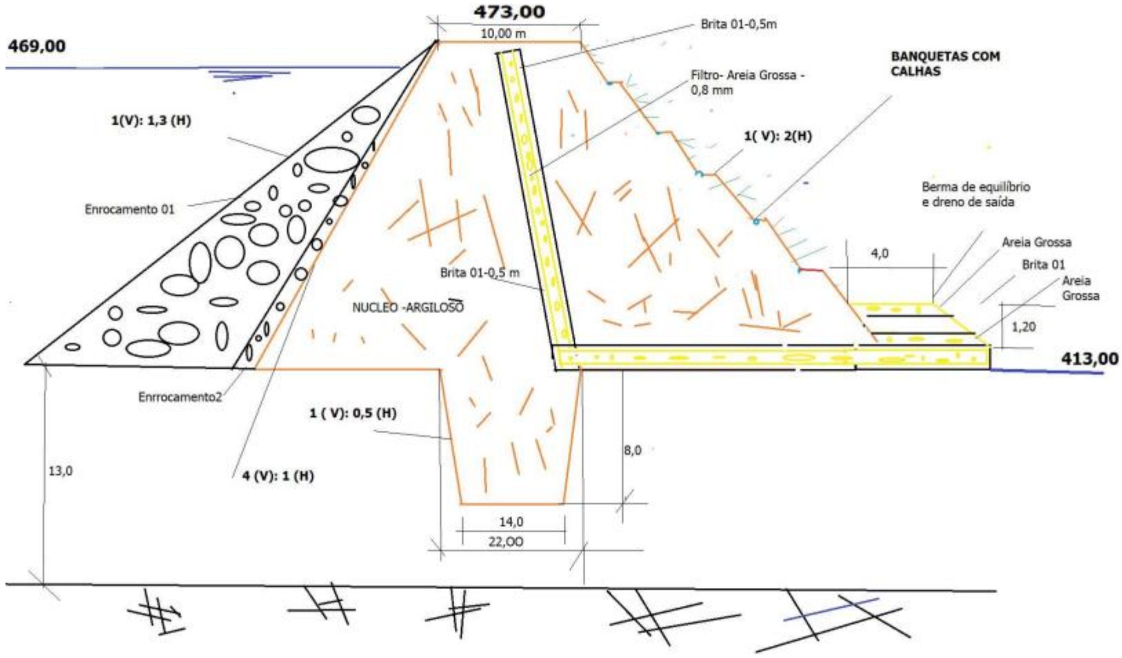

Figure 8. Cross-section detail

Source: Author

Conflicts of interest

The author declares no conflicts of interest regarding the publication of this paper.

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