Roller compacted concrete as a solution for safety enhancement of embankment dams against flood events

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Abstract - Some of the older dams currently in operation do not have the capacity to safely convey their peak design flow without the occurrence of overtopping. This is due to the evolution of knowledge regarding the estimation of maximum flood flow and the alteration of the characteristics of catchments contributing to several dams currently in operation.

The solution to this problem can be either the construction of an emergency spillway, raising the top of the dam, the protection of the downstream face of the dam, or some combination of these solutions. These interventions are generally costly, particularly in the case of small dams, where the entities responsible for safety usually have scarce financial resources.

As an alternative to the conventional approaches, the application of roller compacted concrete (RCC) overlays is an expeditious technique which does not require emptying the reservoir. For these reasons, the application of RCC overlays presents itself as a technically and economically interesting solution.

The present thesis aims to contribute to the dissemination and discussion of the possibilities of this method for overtopping protection of small embankment dams, by presenting its main characteristics, hydraulic design guidelines, examples of application and experience of operation. In addition, a preliminary study of the application of this technique is undertaken for an embankment dam in Portugal, including the respective design.

Keywords: roller compacted concrete, embankment dams, rehabilitation, skimming flow, overtopping protection, hydraulic safety reinforcement.

1. Introduction

The evolution of knowledge regarding the calculation of maximum flood flow for the design of dam spillways leads to the consideration of higher design floods than the ones used for a number of dams currently in operation. Several dams cannot pass the design flood without the occurrence of overtopping.

In the case of embankment dams, overtopping leads to severe erosion or even dam failure, which may lead to loss of human lives.

In terms of protection measures, the protection of the downstream face of the dam can sometimes be the most advantageous solution from an economic standpoint, particularly in the case of small dams (McLean & Hansen, 1993).
The development of roller compacted concrete (RCC) construction methods made this a durable, structurally stable and economically competitive solution for dam overtopping protection. RCC is a fast construction process, requiring simple methods and equipment, and it does not require emptying the dam reservoir, thus contributing to the economy of this solution. However, the use of a spillway over an embankment dam should be an exceptional situation, hence these should work as emergency spillways. In general, they are not considered suited to serve as the main spillway.

2. Examples and performance evaluation

RCC has been widely used, particularly in the USA, over the last few decades, for the construction of spillways or for overtopping protection of embankment dams (McLean & Hansen, 1993, Matos, 2003). In addition to the erosion caused by flow passage, many of these dams are also subject to freeze/thaw cycles, due to their location, thus needing protection from these phenomena. The Ocoee Dam no. 2 (Tennessee, USA) was the first case where RCC was used in the downstream face of a dam as an overtopping protection method. It is a timber crib rockfill dam built in 1913. Due to its severe deterioration the dam was rehabilitated in 1980 by placing RCC in its downstream face. Since then the dam has been subject to over 80 days per year of planned overtopping for the practice of white water rafting. The top of the RCC protection has little signs of erosion. However, further downstream, the RCC steps show some erosion, particularly in the uncompacted lift edges. Despite the mentioned signs of erosion, the dam continues to function normally (Abdo & Adaska, 2007). The slight erosion in uncompacted step edges was verified in other dams with RCC protections, such as Brownwood Country Club Dam (Texas, USA), Kerrville Dam (Texas, USA), Lower Lake Royer Dam (Maryland, USA) and Lake Tholocco Dam (Alabama, USA). However, these events did not compromise the normal operation of these dams or the spillways (Abdo & Adaska, 2007).

Red Rock detention basin is a case where flow transports sediments. These sediments can, in some cases, be as large as 90 kg boulders. Inspections in 2005 and 2006 showed that the effect of sediment transport lead to superficial erosion and polish of the steps. Steps showed less than 50 mm of erosion on their outer face. Some spalling occurred in areas where concrete segregation occurred (Abdo & Adaska, 2007).

The information retrieved regarding different cases, some operating for decades, leads to the conclusion that RCC performs well in the long run, even after the passage of high flows that originate significant head over the structures, or flows transporting large sediments.

3. Hydraulics of RCC spillways over embankment dams

3.1. Types of flow over stepped spillways

Flow over stepped spillways can be classified in three main types (Matos, 1999, Fael, 2000, Chanson, 2001, Meireles, 2004): nappe flow, transition flow, and skimming flow. The type of flow occurring in a stepped spillway depends on the discharge and step geometry. To determine the type of flow in a stepped spillway, Matos (1999), based on the experimental results obtained by Ohtsu & Yasuda (1997), suggested the following equations:
In these equations $h_c$ is the critical water depth, $h_d$ is the step height, and $l_d$ is the step length. These are applicable to spillway angles with the horizontal between 19º and 55º. Eq. (1) defines the end of nappe flow and the beginning of transition flow, whereas Eq. (2) defines the end of transition flow and the onset of skimming flow.

3.2. Characterization of the skimming flow along the spillway

To estimate the location of the inception point, $L_i$, and water depth at that point, $h_i$, Meireles & Matos (2009), based on experimental results obtained by André & Ramos (2003) and Cabrita (2007), suggested the following equations:

\[
\frac{L_i}{k} = 5.19F_r^{0.95}
\]

(3)

\[
\frac{h_i}{k} = 0.28F_r^{0.68}
\]

(4)

where $F_r = q/\sqrt{g \sin \theta k^3}$.

Equations (3) and (4) were obtained in a spillway with a slope of 1V:2H and $1.9 \leq F_r \leq 10$. Hunt & Kadavy (2013) considered such range of $F_r$ restrictive and referred that Eq. (3) overestimates $L_i$ for values of $F_r$ over 10. Hunt & Kadavy (2013) and Hunt & Kadavy (2014) obtained experimental data for a broader variety of $F_r$ and $\theta$ and presented the following equations to estimate the location of the inception point:

\[
\frac{L_i}{k} = 5.19F_r^{0.89} \quad , \quad 0.1 < F_r \leq 28
\]

(5)

\[
\frac{L_i}{k} = 7.48F_r^{0.78} \quad , \quad 28 < F_r < 10^5
\]

(6)

For the same range of $F_r$, Hunt & Kadavy (2014) suggested the following relation to calculate the water depth at the inception point:

\[
\frac{h_i}{k} = 0.34(\sin \theta)^{0.13}F_r^{0.62}
\]

(7)

Meireles & Matos (2009) analyzed experimental results from André & Ramos (2003) and Cabrita (2007) and concluded that the normalized water depth, upstream of the inception point ($h_i/h_i$) strongly depends on $L_i/L_i$, but is practically independent from $h_c/h_d$ between 1.27 and 2.85. The authors suggested the following relation, based on the measured values of $h_i$ and $L_i$, for $1.27 \leq h_i/h_d \leq 2.85$:

\[
\frac{h}{h_i} = 0.971 + 0.891e^{-3.41(L_i/L_i)}
\]

(8)

Hunt & Kadavy (2013) showed that for a broader range of $h_c/h_d$, the water depth is a function of this parameter, and based on this principle, Hunt & Kadavy (2014) presented the following equation:
\[ \frac{h}{h_c} = \left( \frac{L}{L_i} \right)^{-0.22} \left[ 0.34 \left( \frac{h_d}{h_c} \right)^{0.063} \left( \cos \theta \right)^{0.063} \left( \sin \theta \right)^{-0.18} \right] \quad (9) \]

which is applicable for or \(0.1 \leq L/L_i \leq 1.0\), \(0.035 \leq h_d/h_c \leq 1.1\) and \(10^\circ \leq \theta \leq 30^\circ\).

The comparison of the two methods shows that, upstream of the inception point, Eq. (8) and (9) lead to similar results of the water depth, for the range of \(h_d/h_c\) tested by Meireles & Matos (2009).

Downstream of the inception point, air entrainment occurs, and it is therefore necessary to define the main features of this type of flow, such as the mean air concentration, \(\bar{C}\), the equivalent clear water depth, \(h_{eq}\), and the characteristic depth, \(Y_{90}\).

Matos et al. (2001) reanalyzed data from other authors and suggested the following expression for the calculation of the mean air concentration on 1V:2H sloping chutes

\[ \bar{C} = 0.262 + \frac{0.158}{(1 + 0.031s')^{-2.398}} \text{, for } 0 \leq s' \leq 100 \quad (10) \]

where

\[ s' = \frac{L - L_i}{h_i} \]

Hunt et al. (2013) used their own experimental data to obtain the following relations

\[ \bar{C} = 0.11 - \frac{0.22}{(L/L_i)} + 0.0645 + 0.216 \left( \frac{h_d}{h_c} \right) + 0.453 (\sin \theta) \quad (11) \]

for \(1.0 \leq L/L_i \leq 2.0\), and

\[ \bar{C} = 0.0645 + 0.216 \left( \frac{h_d}{h_c} \right) + 0.453 (\sin \theta) \quad (12) \]

for \(L/L_i > 2.0\).

Eq. (11) and (12) were obtained for \(0.035 \leq h_d/h_c \leq 1.1\) and \(10^\circ \leq \theta \leq 30^\circ\).

The use of the expression suggested by Matos et al. (2001) leads to considerably different results from those obtained when using Eq. (11) and (12) for very small \(h_i/h_d\) (~1), and for larger \(h_c/h_d\), when \(L/L_i > 3\). However, both methods show that the mean air concentration increases more abruptly in the initial region downstream of the inception point, and in a more gradual manner downstream of this region.

To estimate the equivalent clear water depth, Matos et al. (2001) presented the following equation, based on experimental results obtained by Gaston (1995) and Boes (2000):

\[ \frac{h_{eq}}{h_i} = 0.642 + 0.105 e^{-0.011s'} \text{, for } 0 \leq s' \leq 100 \text{ and } 26.6^\circ \leq \theta \leq 30^\circ \quad (13) \]

However, in Eq. (13), \(h_i\) is calculated using the equation provided by Chanson (1994), which, according to Matos et al. (2001), is expected to overestimate the equivalent clear water depth at the inception point, as well as the location of the inception point, \(L_i\).

Hunt et al. (2013) developed the following expression to estimate the clear water depth, as a function of the critical depth:
\[
\frac{h_{eq}}{h_c} = 0.34 \left( \frac{h_d}{h_c} \right)^{0.063} \left( \cos \theta \right)^{0.063} \left( \sin \theta \right)^{-0.18}
\]

Eq. (14) is applicable downstream of the inception point \((L/L_i \geq 1)\), for \(0.035 \leq h_d/h_c \leq 1.1\) and \(10^\circ \leq \theta \leq 30^\circ\).

As a result of the overestimation of \(h_i\) in Eq. (13), this equation results in considerably different values of the equivalent clear water depth than the ones obtained using Eq. (14).

The characteristic depth, \(Y_{90}\), is given by

\[
Y_{90} = \frac{h_{eq}}{(1 - C)}
\]

where \(h_{eq}\) can be estimated using Eq. (13) (for \(\theta \sim 30^\circ\)) or Eq. (14) and \(C\) can be obtained either using Eq. (10) (for \(\theta \sim 30^\circ\)) or Eq. (11) and (12).

4. Recommendations for hydraulic design

4.1. Crest treatments

Usually for stepped spillways the crest treatments are free uncontrolled overflows that generally take the form of a broad-crested weir. The need to maximize reservoir storage has led to the use of fixed vertical weirs placed on top of a broad crest.

Different crest treatments result in particular features the designer must address. However, the cost of these solutions should not be the deciding factor (Frizell & Frizell, 2015).

4.2. Step height

The height of RCC layers is a conditioning factor to the step height. Usually this height varies between 0.20 m and 0.60 m. To maximize energy loss, Tozzi (1992) concluded that step height should be at least 30% of the critical depth \((h_d/h_c \geq 0.3)\).

4.3. Sidewall design

The sidewall height should in general take into account air entrainment, if the inception point occurs along the spillway, for the design discharge.

Usually, the sidewall height is calculated using the following expression:

\[
h_p = n Y_{90}
\]

where \(n\) is a “safety factor” to account for flow bulking and splash due to air entrainment.

Boes & Minor (2000, 2002) and Boes & Hager (2003) considered that, in the case of embankment dams, \(n = 1.5\). Recently, based in results obtained over the course of several years, in large scale models, Hunt & Kadavy (2016) suggest the following equations to calculate the sidewall height:

\[
\frac{h_p}{Y_{90}} = 1.4 \quad \text{for} \quad \frac{h_d}{h_c} \leq 0.40
\]

\[
\frac{h_p}{Y_{90}} = 2.0 \left( \frac{h_d}{h_c} \right)^{0.37} \quad \text{for} \quad \frac{h_d}{h_c} > 0.40
\]
Equations (17) and (18) are considered adequate for estimating the sidewall height when \( Y_{90} \) is calculated from the equations proposed by Hunt et al. (2013).

If the inception of air entrainment does not take place on the spillway, for the design discharge, Eq. (15) can be applied, replacing \( Y_{90} \) by \( h_{eq} \) and considering \( n=1.5 \), as suggested by Boes and Minor (2000, 2002) and Boes and Hager (2003).

5. **Design criteria and construction aspects**

5.1. **Weir crest and spillway**

It is convenient to protect the weir crest with one of two RCC layers, that can be extended upstream as far as the upper area of the upstream face of the dam. The spillway slope is usually dictated by the downstream face slope of the dam. However, a different slope can be adopted. Usually the spillway angle with the horizontal is comprised between 14° and 30° (frequently 26,6° - 1V:2H). Generally, the step height is 0,20 m or 0,30 m, even though in some cases 0,60 m high steps can be adopted.

The construction process of RCC overlays is very particular, and to ensure equipment operation, RCC layers should have a minimum width between 2,40 m and 2,70 m (in Matos & Meireles, 2006).

5.2. **Drainage system**

A filter/drain system is necessary. This system should be placed as a continuous layer under the spillway and should include a filter adapted to the material of the embankment, constituted by granular materials or, alternatively, geotextile materials that function as a filter in relation to the embankment materials.

5.3. **Spillway treatments**

To avoid deterioration due to climate action or aggressive hydrodynamic actions, a spillway treatment can be considered. Breaching of the RCC overlays should be avoided or minimized to prevent water passing to the embankment material under the spillway.

Several treatments can be used, depending on structural and aesthetic requirements, such as: concrete, pre-fabricate concrete elements and earth either covered with vegetation or not.

The main criteria to take into account when choosing a spillway treatment should be: cost, aesthetic appearance, durability and simplicity of execution (in Matos & Meireles, 2006).

6. **Case study**

6.1. **General remarks**

The object of the case study is Capinha dam, in the district of Castelo Branco, Portugal. The Capinha dam was designed in 1979 and concluded in 1981. It is an 18 m high embankment dam, with a width of 231 m and a 7 m wide crest. The reservoir occupies an area of 9.7 ha and has a volume of 0.522 hm³ and the contributing catchment area is 6.3 km². The dam crest elevation is at 505.00 m, 2.5 m above the maximum storage level. The main spillway is a morning glory spillway at level 502.50 m, with a design discharge of 26.5 m³/s. The bottom discharge has a design capacity of 2.3 m³/s.

HIDRORUMO & AQUALOGUS (1999) revealed that, for heads over approximately 0.90 m, the morning glory spillway behaved as an orifice. In these cases, the increase of discharge capacity is reduced.
Originally, peak design flow was obtained using the Mockus method applied to precipitation calculated based on the precipitation probability lines established for the station of Miuzela, for a return period of 1000 years. Studies by HIDRORUMO & AQUALOGUS (1999) showed that the main spillway could be insufficient for peak flows estimated using other methods like the Rational method or SCS Unit Hydrogram. Using these methods applied to precipitation data acquired in stations that influence the dam contributing catchment, the resulting peak flow could be as much as 1.5 times higher than the flow originally considered for the spillway design.

The studies by HIDRORUMO & AQUALOGUS (1999) concluded that for a return period of 1000 years, the main spillway discharges about 30 m$^3$/s, resulting in a head over the weir of 1.52 m, well above its design head of 0.81 m.

It was therefore considered necessary to build an emergency spillway, to operate only on exceptional flood events, allowing the morning glory spillway to work at its design capacity.

**6.2. Proposed solution**

The use of RCC to materialize an emergency spillway can be an advantageous solution, once it does not require emptying the reservoir, there are no space restrictions downstream of this particular dam, the construction time would be shorter when compared to the construction of a traditional concrete spillway, and the fact that the stepped spillway would increase the energy dissipation thus reducing costs with the energy dissipation basin.

Some precautions should be taken when building the emergency spillway, so that the dam stability is not compromised, such as the placement of a continuous drainage and filter layer under the RCC overlays.

Using the Intensity-Duration-Frequency lines estimated by Brandão et al. (2001) for the station of Covilhã, the peak flow was estimated at about 50.2 m$^3$/s, for a return period of 1000 years.

The maximum flood level was established at 504.40 m to ensure that the main spillway works with a free surface flow above its crest.

The RCC spillway width was defined at 25 m at elevation 502.75 m, 0.25 m above the morning glory spillway’s crest, with a broad crest. The individual and joint rating curves are presented in Figure 1.

![Figure 1](image-url)
Flood wave modelling was done using a Storm Water Management Model (SWMM) and the results are presented in Figure 2.

![Figure 2 – Flood wave modelling results.](image)

The estimated effluent flow was about 40.2 m$^3$/s, of which the morning glory spillway will convey around 24.3 m$^3$/s with a head of 0.77 m, and the emergency spillway will discharge about 15.9 m$^3$/s. Due to RCC overlays minimum thickness, it was established that 0.30 m high steps would be adequate, respecting the previously referred reference for energy dissipation maximization of $h_d/h_c = 0.3$. The step length was set at 0.6 m to maintain the downstream face slope of 1V:2H.

Two solutions are possible: solution 1 - building a localized spillway in a 25 m wide section of the downstream face of the dam; and solution 2 - building a broad crested weir in a 25 m wide section of the crest of the dam and covering the entire downstream face of the dam with RCC overlays.

In the case of solution 1, it is necessary to build sidewalls. The wall height was estimated based on Eq. (17) and (18) resulting on a sidewall height of around 0.50 m.

The Froude number at the toe of the stepped chute was estimated to be 4.2. For this magnitude of Froude number, either a Type IV BUREC energy dissipation basin can be used, or a Type I basin built with RCC, to simplify construction.

The cost of both solutions, using a Type I basin, is presented in Table 1.

<table>
<thead>
<tr>
<th>Item</th>
<th>Cost (€)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Solution 1</strong></td>
<td><strong>Solution 2</strong></td>
</tr>
<tr>
<td>Partial demolition of dam's crest</td>
<td>34 463,00 €</td>
</tr>
<tr>
<td>Pickling of the downstream face</td>
<td>7 000,00 €</td>
</tr>
<tr>
<td>Drain and filter layer</td>
<td>11 200,00 €</td>
</tr>
<tr>
<td>RCC spillway</td>
<td>242 198,00 €</td>
</tr>
<tr>
<td>Spillway sidewalls</td>
<td>5 040,00 €</td>
</tr>
<tr>
<td>RCC Type I energy dissipation basin</td>
<td>31 148,00 €</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td><strong>331 049,00 €</strong></td>
</tr>
</tbody>
</table>
7. Conclusions

Based on the experimental studies developed by other authors, the equations that characterize the skimming flow along a stepped chute were presented, as well as their application to the design of a stepped spillway.

In terms of the case study here presented, two possible solutions were presented. Solution 2 is more onerous once it implies the use of a larger quantity of RCC and a larger area of the drainage and filtering layer. The drainage of the downstream face would also be an added technical difficulty, needing a drainage channel linked to the discharge of the morning glory spillway and an anti-return valve. On the other hand, covering the entire downstream face would not only protect the face of the dam but also allow the flow to be distributed in a direction normal to that of the main flow, resulting in higher energy dissipation and reducing the risk of erosion downstream.

Not only RCC overlays are a possible solution, from the technical standpoint, for the rehabilitation of embankment dams, but its use is less costly than a traditional concrete spillway, which is particularly interesting in small dams and in countries where less financial resources are available.

There are numerous cases in Portugal where this technique could be advantageous when compared with the solutions usually adopted. One of the main reasons for the non-use of this method in the country is the lack of diffusion of RCC overlays and the degree of uncertainty associated to building a spillway over an embankment dam. There is, however, an extensive knowledge of these solutions worldwide, namely in the USA, which can be a starting point towards the development and application of RCC for overtopping protection of small embankment dams in Portugal.

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References


