Artificial accelerograms to estimate damage of dams by using failure criteria

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Molina has got over 30 JCR publications in SCI high impact research journals such as “Journal of Hydrology”, “Environmental Modelling and Software”, “Water Resources Management”.

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**Abstract**

The aim of this paper is to analyse dam’s damage by using two recent methodologies. The first method has been used to define the performance and response curves of concrete gravity dams. The second method defines the seismic input which has been obtained from power spectral density function consistent with the response spectrum. Both methods set themselves as efficient, practical and useful to develop quite complicated analysis as the construction of the stochastic process to define the synthetic earthquake and the estimation of cracks in the dam’s body. These methodologies have been explained and revised to improve their use. The fluid behaviour contained by arch-dams is compared with the fluid behaviour in storage tanks by studying the sloshing phenomenon which is usually neglected for dams. For the mathematical modelling, interactive programming language has been used.
Keywords

Earthquake damage estimation, failure modes, power spectral density function, concrete arch-dams, storage tanks, sloshing mode

1 Introduction

There are several nonlinear models in the literature to estimate the response in terms of concrete dams’ displacements which lead to similar results. Some examples of this are listed as follows; the plastic degradation method, which considers the elasto-plastic modulus’ decreasing during the hysteretic cycles [1,2]; the nonlinear fracture mechanics which defines the potential crack [3]; the modelling of the joints within the dam which transmit the interaction stresses of the blocks [4]; the endurance time acceleration which is based on pushover analysis [5,6]; or the base sliding evaluation even that it is a simplified analysis accounts the nonlinear behaviour of the dam-water-foundation system [7].

There are also several approaches to study the fluid-structure interaction and the hydrodynamic pressures generated by water particle excitation, e.g. the Lagrangian approach, which expresses the fluid behaviour in terms of displacements [8,9]; the Eulerian approach which expresses the fluid behaviour in terms of pressures [10]; and the immersed boundary methods which are based on numerical simulation [11].

The question is: what is the most convenient model to be used in terms of time and reliable results? Nowadays, scholars carry out rather complex studies which are not always possible to be applied in engineering companies. Additionally, there is an increasing number of software; however, engineering companies need optimize time and costs.

In practice, structural analyses results are generally compared with results obtained in traditional methods, e.g. USACE [12]. This is because simple applications, at the expense of results’ accuracy, are easier to control. If such results are compared with specialised verification, the outcome may be optimal.

In recent meetings of the Committee of Large Dams (Meeting of EWG Dams and Earthquakes in Rome in February 2017 and Annual Meeting of SPANCOLD in Madrid in February 2017) it has been discussed how necessary it is to reanalyse the existing dams for preventing further damages.

In Spain in the Andalusian region (southern Spain) the seismic activity is high [13] and several remarkable earthquakes have occurred; therefore, seismic analyses must be constantly carried out in the area. Earthquake
records involving arch dams are very limited in general. The work described in [14] shows some of the important earthquake records of arch dams, presenting some significant remarks.

There are many methods to obtain the seismic input, for instance the probabilistic and deterministic seismic hazard assessment [15], but it would take some months to be done well. Besides, they are usually compared with real accelerograms to identify the most consistent accelerogram to carry out the time-history dynamic analysis [16]; however, in some available database [17] it is possible to find only a short range of real accelerograms.

In this paper two efficient and economical methodologies, in terms of processing time, have been used. The first one defines the Power Spectral Density (PSD) function and the second one estimates earthquake damage of gravity dams. The usefulness of the first methodology is that it provides synthetic accelerograms which allow to carry out the time-history dynamic analysis. In Spain, it is possible to define the synthetic accelerograms using the input data present in the Spanish code [18] by means of stochastic response [19-21]. The usefulness of the second methodology is that it is not necessary to define a nonlinear model, which can be complicated to be developed and to be reliable.

The last part of this paper comprises the fluid behaviour of arch-dams in comparison with the fluid behaviour of storage tanks. A dam-reservoir structure is a liquid storage structure of large scale [22]. The main differences between the arch-dams and tanks are: (i) the volume of water mass against the dam, which is larger in the dam; (ii) the tank’s liquid being affected by the closed walls with regular geometry; (iii) considering an arch-dam as a tank, the internal pressures acting in different directions, whereas in the dam they always go upstream-downstream; (iv) the dam oscillates as a rigid monolith, whereas the (flexible) tank deforms the walls; (v) tanks can be made of concrete or steel; (vi) in tanks the water mass is divided in two parts: the underlying part (impulsive mass) is united rigidly with the walls, whereas the overlying part (convective mass) is united elastically with the walls and oscillates freely [23]. For storage tanks and containers [24,25], this free surface motion is known as sloshing waves (convective mass) and is still under study, whereas for dams it is usually neglected.

2 Case study

To estimate the dams damage and the sloshing mode two type of dams have been chosen. The dams are
located in the Andalusian region (southern Spain) where the relative seismic hazard is high. The studied seismogenic zone where both dams are placed is ZS34 [26] of which some data are shown as following: the tectonic dominant mechanism is reverse fault, the mean maximum moment magnitude (Mw) is 6.6 ± 0.3 and the mean annual rate of exceedance for Mw ≥ 4.0 is 0.198 (= 5 years, i.e. the mean time interval between occurrences of events). In accordance to [27] the two dams (see Figure 1) are considered large because they satisfy the follow condition: h = {89.55 m, 74.0 m} > 15 m and c_r = {57.0 hm^3, 84.0 hm^3} > 1 hm^3. The numbers in the brackets refer to the Concepción Dam and Conde de Guadalhorce Dam, respectively. The parameter c_r is the reservoir capacity and h is the maximum height of the dam. Table 1 shows the description of the data.

3 Materials and methods

3.1 Definition of artificial accelerograms

There are some codes that calculate the elastic spectrum by using curves with three or four branches. In this case, for the last branch, the circular frequency ω tends to infinity. For example, the Brazilian [28], Spanish [18] and Venezuelan [29] codes use three branches whereas the Italian [30] and European [31] ones use four branches. The lack of the last fourth branch could not be much significant because in the flexible structures (with high periods) the accelerations are not transmitted totally to the structure. Consequently, an underestimated pseudo-static force is not much relevant. Usually, the codes with four branches have the fourth branch as the lowest one.

In this section the definition of artificial accelerograms has been carry out by using a methodology that is applicable with the Spanish elastic spectrum [18] with three branches.

The methodology developed by Barone et al.’s (2015) [32] is explained as it follows (see Figure 2). Firstly, the elastic spectrum parameters are chosen, e.g. T_A, T_B, ω_A, ω_B, Peak Ground Acceleration (PGA) and viscous damping ratio ξ. Then, the Peak Factor (PF) expressed by Equation 2 and the parameters to obtain the PSD function expressed by Equation 1 are determined. PF is the maximum probable value of response and is equivalent to the displacement obtained by pseudo-acceleration spectral response times the circular frequency squared.

After that, the deterministic modulation function I(t) and the amplitude of the accelerogram A_i ≈
are calculated. Finally, the artificial accelerograms are computed.

In this analysis, being a stochastic approach, it is possible that the artificial accelerogram has some inconsistencies, e.g. the difference between the maximum acceleration of the artificial accelerogram and the PGA cannot be the same (see Figure 3 (right)), or the final velocity, obtained as integrate of the artificial acceleration, cannot be null. The former is solved scaling all values of the artificial accelerogram. Herein, the maximum scale factor is 2.72 (= 0.924/0.34) higher than the input PGA (despite this they have not been scaled in the current analysis because the focus here is to create cracks – this also justifies the use of the overestimated soil amplification after explained). The latter is solved using the baseline correction. This correction induces important changes in the ground displacement and in the velocity diagrams but in the accelerations, there are not noticeable alterations [34], thus the baseline correction has been neglected.

However, the used analytical model defines the PSD function coherent with response spectrum, therefore the aspect that must be respected must be the distribution of power (amount of energy transferred per unit time) versus the frequency.

The used acceleration value refers to the higher value show in the Spanish code [18] in the Malaga province, that is 0.21 g. The PGA has been calculated considering the structures as special (coefficient equal to 1.3) and the soil amplification as III/IV-type (soil coefficient ~ 1.8 and ground amplification coefficient ~ 1.25). Most of the southern Spain soil is type I(A), I(B) and II, however there are also areas III/IV(A)-type [13]. The calculated PGA is equal to 0.34 g (= 0.21 g x 1.3 x 1.25).

The dynamic amplification factor $\alpha$ is 2.5 and the structure’s natural periods are: $T_A = 0.16$ s ($\omega_A = 39.269$ rad/s) and $T_B = 0.64$ s ($\omega_B = 9.817$ rad/s). To define the structural periods the contribution coefficient $K$, which is considered in the three-branches expression in Spanish code [18], is neglected. In the Spanish code [18], $K$ is listed for each city and rage 1.0 - 1.3. It accounts the seismic hazard but, given that there are many unknown parameters that represent the complexity at defining the seismic hazard [35,36], the authors believe that $K$ has no meaning.

The PSD by Barone et al. (2015) [32] is defined as:
\[
\text{PSD}_0 \left( \frac{\omega}{\omega_B} \right)^{e_1} \quad \text{for } 0 < \omega \leq \omega_B \\
\text{PSD}_0 \left( \frac{\omega}{\omega_B} \right)^{e_2} \quad \text{for } \omega_B < \omega \leq \omega_A \\
\text{PSD}_0 \left( \frac{\omega}{\omega_A} \right)^{e_2} \left( \frac{\omega}{\omega_A} \right)^{e_3} \quad \text{for } \omega > \omega_A
\] (1)

where \( \text{PSD}_0 = 1.554 \times 10^{-4} \text{g}^2/(\text{rad/s}) \) is the peak value of the PSD function at the frequency \( \omega = \omega_B \). These three equations depend on \( \omega \), peak factor, \( \xi \) and \( \alpha \). In this analysis the three exponentials are: \( e_1 = 0.826, e_2 = -1.247, e_3 = -2.592 \). The peak factor [37] is defined as:

\[
\text{PF}(\omega, \xi) = \sqrt{2\ln\left(2\nu_u\left(1 - e^{-\delta^2/4\ln(2\nu_u)}\right)\right)}
\] (2)

where the parameter \( \nu_u \) (when \( \omega = \omega_B \); \( \nu_u = 68.585 \), when \( \omega = \omega_A \)) depends on nominal duration \( T_s \) of the pseudo-stationary part and probability function \( p \). The probability function \( p \) has been assumed equal to 0.367, i.e. the mean value of the peak values can be approximated by the 36.7% fractile of the maxima distribution. The parameter \( \delta = 0.245 \) is called “spread factor” and is correlated to \( \xi = 0.05 \). The nominal duration has been assumed \( T_s = 11 \text{ s} \). The choice of the nominal duration value can be assumed in accordance to the significant duration, which it was shown in previous work [36,38].

The earthquake motion is defined by a non-stationary process and the nonlinear dynamic response of the structures is highly influenced by the non-stationary behaviour of the seismic input; the earthquakes are usually also of too short duration to let the response become stationary. Despite this, in this study the central time interval has been assumed stationary and the end and initial time intervals have been assumed as non-stationary to preserve the increasing and decreasing of the accelerogram in the initial and final phases, respectively.

To simulate the quasi-stationary condition of the real earthquakes it is necessary to define the deterministic modulating function, for this the trapezoidal shape has been chosen [39]; the three branches are 0-4 s, 4-15 s, 15-30 s, where \( t_1 = 4 \text{ s}, t_2 = t_1 + T_s = 4 \text{ s} + 11 \text{ s} = 15 \text{ s} \) and \( t_3 = 30 \text{ s} \).

Finally, to define \( a(t) \), the number of the used harmonic waves (superposition method) with random phases \( \varphi_i \) by [34] is: \( N \geq (150/\pi)t_3 = (150/\pi)30 = 1432 \rightarrow 1500 \), where 150 rad/s is the cut-off circular frequency.
selected.

Figure 3 shows the PSD in function of $\omega$, the artificial accelerogram $a(g)$ in the time domain $t(s)$ and the elastic spectrum (spectral accelerations $S_e$ vs. structural periods $T$)

The shape of the PSD is like other ones in literature [40,41]. PSD is not slope-wise continuous at the control frequencies; this may be attributed to the slope-discontinuities at these frequencies in the response spectrum [42].

### 3.2 Estimation earthquake damage

The estimation earthquake damage of gravity dams is based on the methodology proposed by Alembagheri (2016) [43]. The method consists in calculating the Performance Curve (P-C) and the Response Curve (R-C) of dams.

In the Alembagheri’s work (2016) [43] the pushover curve has been used to define the yielding and ultimate displacement, instead, in this analysis, the pushover curve has been estimated. This is because the dam body under a pushover analysis is subjected at increasing of the tensile stresses, and as the concrete tensile capacity is weak, it is difficult not to obtain poor results. Moreover, the pushover analysis is usually used for new constructions.

The flow chart in Figure 4 shows the methodology of the estimation of the P-C, R-C and the damage index for concrete gravity dams. The procedure of the flow chart is explained as it follows.

To define the Cumulative Inelastic Area (CIA) of the P-C the process is divided into: (i) estimation of $C_u$, $C_y$ and definition of $T_1$ in the trigonometric function (3); (ii) quantification of the number of Demand-Capacity Ratio (DCR); (iii) calculation of the CIA of the P-C curve; (iv) drawing of the curves P-C, 2P-C (two times P-C curve) and 3P-C (three times P-C curve). The area is between $(C_u/C_y)_i$ of step i and $(C_u/C_y)_{\text{max}}$ – in this analysis seven points ($i=7$) have been used (for more details see Figure 3 in [43]).

Because the first vibration mode of the gravity dams is predominant, and each block can be considered as a simple oscillator, DCR can be assumed as a Harmonic Crest Displacement (CDH) time-history with the following equation:

$$\text{CDH} = \frac{C_u}{C_y} \sin \frac{2\pi t}{T_1}$$  \hspace{1cm} (3)
where $C_u$ and $C_y$ are the ultimate and yielding crest displacement capacities and $T_1$ is the dam fundamental period.

To define the R-C the process is like the employed process to calculate P-C, but not the same because, due to the great irregularity of the earthquake accelerations, CIA is complicated to calculate.

To solve this problem, the following approximation has been used: $\text{CIA} = (2 \times 0.05) \times n_p/(2 \times 3) = 0.1(n_p/6)$. The process is: (i) defining the $C_y$; (ii) individualizing the sum of the elastic displacement peaks $n_p$ that exceed $C_{yi}$; (iii) multiplying $n_p$ by two times the step time $d_t = 0.05$ s. To eliminate the descending branches of the time-history and to consider only the positive peak times, the division by two and three, respectively, has been made. The displacement is between $C_{yi}$ of step $i$ and $C_{y, max} \sim$ seven points ($i = 7$) have been used. DCR is between 1.0 and the maximum elastic displacement divided by $C_y$ (for more details see Figure 1 in [43]).

Table 2 shows the data of the dam block considered to carry out the analysis. The aim here is to study the response of the individual block idealized as triangular shape (fundamental triangle) by considering the different structural periods and different $C_u/C_y$ ratio. The structural periods have been considered in a common range for dams, that is 0.25-0.35 s; whereas $C_u/C_y$ ratio range 1.5-4.0, which are the values calculated in the literature [43]. $C_y$ value has been fixed as 1.50 cm. There are occurrences in the literature of three types of concrete gravity dams [43] like the Concepción Dam, attesting the reliability of the data.

4 Results of the damage estimation

To estimate the performance and response curves an analytical analysis has been carry out, therefore the dam-water-foundation interaction has not been considered. However, it is important to notice that, considering dam-water-foundation interaction the structural period increases reaching values of the order of 0.65 s [44].

Figure 5 shows the CDH in the time domain of the studied dam for $T_1 = 0.3$ s. The horizontal dashed lines correspond to $C_u/C_y = 3$ and $C_y = 1.5$ cm. Figure 6 show the elastic displacement in the time domain and the yielding crest displacement $C_y$ for $T_1 = 0.3$ s. The elastic displacement has been calculated by: (mass x acceleration)/stiffness. $n_p$ indicates the number of times that the elastic displacement exceeds $C_{yi}$. 


In Table 3 the area under the P-C, two times P-C and three times P-C of the Concepción Dam are shown. The parameters $\bar{y}_{\text{CIA}}$ and $\bar{x}_{\text{DCR}}$ are the mean values of the CIA and DCR considering the seven points that draw the performance and response curves.

Figures 7-9 show performance curve (dashed line) and response curve (solid line) for the Concepción Dam. The highest response curve and the highest performance curve refer to the response curve calculated by $a(g)$ and to the 3P-C curve, respectively.

After defining the performance and response curves, it is possible to define the Damage Index (DI) by: 
\[
\min(1, A_r/A_p); \text{ where } A_p \text{ is the area under 2P-C (see underlined values in Table 3) and } A_r \text{ is the area under R-C (see underlined values in Table 4). If } DI \leq 0.5 \text{ the dam essentially remains intact or suffers minor damage, if } 0.5 < DI < 1.0 \text{ the dam damage is moderate or operational and if } DI = 1.0 \text{ the dam is subject to severe damage.}
\]

The damages of the dam body usually occur in the base (heel and toe), in the crown and in the points where the slopes change. The cracks can propagate along the whole upstream and downstream face as shown in literature [45-47].

5 Sloshing modes

It is possible to suppose that an arch-dam is a part of a tank circumference [48]. In this analysis, the dam idealized as a tank is the Conde de Guadalhorce Dam.

The parametric equations for the equivalent tank with the reduced radius (Figure 10 (left)) and the tank with the estimated radius (140 m) (Figure 10 (right)), are, respectively: \{Cos(r), Sin(r), z| 0 < r < 2\pi, 0 < z < 1.143\} and \{Cos(r), Sin(r), z| 0 < r < 2\pi, 0 < z < 0.375\}, where $z$ is the vertical axis and $r$ is the radius coordinate.

The height of the equivalent tank has been scaled 1:10 in relation to the fundamental triangle height of the Dam that is 52.5 m ($\rightarrow H_{\text{tank}} = 5.25$ m), whereas the radius is less than 30.5 times the estimated radius of the Dam ($\rightarrow R = 4.59$ m). The criterion used to define the tank radius relies on considering at least 7/8 of the liquid height $H$ by [49]: $p_w = 7/8 \rho \text{ PGA } [1 - (z/H)]^{1/2}$, where $\rho = 1000$ kg/m$^3$ (mass density of the water). Therefore, $R \rightarrow (7/8) \times 5.25 = 4.59$ m.

The impulsive $p_i$ and sloshing $p_c$ [50] pressures for the first mode have been computed by equations:
\[
\frac{p_l(z)}{PGA} = \frac{2\rho H}{I_0\left(\frac{\nu H}{H}\right) - \left(\frac{\nu H}{H}\right)} \frac{\nu_0 \nu}{H} \cos\left(\frac{\nu_0 \nu}{H}\right) I_1 \left(\nu_0 \nu H\right) I_1 \left(\nu_0 \nu H\right) \quad (4)
\]

\[
\frac{p_c(z)}{PSA} = \frac{2\rho H}{(\lambda^2_n - 1) J_1(\lambda_n \nu R)} \cos\left(\frac{\lambda_n \nu^2 R}{R}\right) J_1(\lambda_n) \quad (5)
\]

where \(\nu_0 = 1.5707, I_1 = 1.0538\) and \(I_0 = 1.7187\) correspond to the modified Bessel function of order one and zero, respectively, and \(J_1 = 0.5818\) is the Bessel function of the order one. Pseudo Spectra Acceleration (PSA) is calculated considering the sloshing circular frequency \(\omega_{cn} (= 2\pi/T_{cn})\) by: \(\omega_{cn} = [(g \lambda_n/R) \tanh(\lambda_n H/R)]^{1/2}\), where \(\lambda_n = \{1.841, 5.331, 8.536\}\) and \(n = \{1, 2, 3\}\) for the first three modes. \(g\) is the gravity. All pressures are calculated at the wall of the tank and in the plane of the horizontal seismic action.

For \(H = R = 4.59\) m, the \(\omega_{cn}\) for the first three modes is: \(1.934 \text{ rad/s} (T_{c1} = 3.248 \text{ s}), 3.376 \text{ rad/s} (T_{c2} = 1.861 \text{ s}), 4.271 \text{ rad/s} (T_{c3} = 1.471 \text{ s})\). Considering \(H = 42\) m and \(R = 140\) m, \(T_{c1} = 24.73\) s \((\omega_{c1} = 0.254 \text{ rad/s})\).

PSA, for the soil coefficient equal to 2, is \(0.21 \text{ g} (= 0.34 \text{ g x (2/3.248)})\) [18]. The final PSA is \(0.28g\) \((= 0.21 \text{ g x 1.348})\), where the damping factor is defined by \(10/(5+\xi)^{1/2} = 1.348\) for \(\xi = 0.5\%\) [31].

Table 5 shows the pressure results. The peak sloshing wave is [51]: \(d_{max} = 0.84 \times R \times (\text{PSA/g}) = 0.84 \times 4.59 \times 0.28 = 1.08\) m.

6 Conclusion

This paper describes two methodologies related to the definition of artificial acceleration under a novel analytical model of PSD and estimates the damage of concrete gravity dams. The mainly advantage of the first methodology is that it is well applicable and compatible with different spectra, whereas the mainly advantage of the second one is that it does not need a nonlinear model to define the material behaviour.

Two large dams in the region of southern Spain – built between the years 1921 and 1971 – have been analysed here because in this region the seismic hazard is intense (the greatest recorded magnitude is Mw 6.9).

The first dam, Concepción Dam, has been studied to estimate the damage index under the artificial acceleration consistent with Spanish elastic spectrum. The stochastic analysis used to define the artificial
acceleration is based under assumptions that the time-history earthquake, which is necessary to do the
dynamic analysis, is considered as a realisation of random processes. The obtained maximum value is
0.977g. Because the first vibration mode of the gravity dams is predominant, the damage analysis is based on
the fact that the cumulative inelastic area can be assumed as a harmonic crest displacement time-history. The
obtained damage index ranges from 0 to 0.767, i.e. when the dam is intact and when it is moderately
damaged.

Finally, the second dam, Conde de Guadalhorce Dam, was compared to the storage tank to study the
sloshing effect. Both systems have some differences, but they share some similarities as well. A dam-
reservoir system may become a large-scale storage-tank system. The mean value of the difference between
the dam dynamic pressures and the dynamic pressures of the equivalent tank is 0.946 kN/m². For the
equivalent tank the first sloshing frequency is 0.307 Hz and the height of the waves reaches 1.08 m.

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Figure 1. Photos of the Conde de Guadalhorce Dam (left) and Concepción Dam (right)
Figure 2. Flow chart of the definition of artificial accelerogram
Figure 3. PSD (a), artificial accelerations (b) and elastic spectrum (c). The horizontal dashed lines in (b) refer to PGA = 0.34 g.

Figure 4. Flow chart of the estimation of the damage index.
**Figure 5.** CDH in the time domain of the Concepción Dam for $C_u/C_y = 3$, $C_y = 1.5$ cm and $T_1 = 0.3$ s

**Figure 6.** Elastic displacement of the La Concepción Dam for $C_u/C_y = 3$, $C_y = 1.5$ cm and $T_1 = 0.3$ s ($n_p = 31$).
Figure 7. Analysis results of the Concepción Dam for $T_1 = 0.25$ s. The performance curves are indicated by the dashed lines and the response curves by solid lines.
Figure 8. Analysis results of the Concepción Dam for $T_1 = 0.30$ s. The performance curves are indicated by the dashed lines and the response curves by solid lines.
Figure 9. Analysis results of the Concepción Dam for $T_1 = 0.35$ s. The performance curves are indicated by the dashed lines and the response curves by solid lines.
Figure 10. Geometrical model of the equivalent tank with $R = 4.59$ m, 144 meshes of $0.095 \times 0.523$ m (left); and tank with $R = 140$ m, 144 meshes of $0.031 \times 0.523$ m (right) by software [52].
Table 1. Description of the data

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<td>Province</td>
<td>Malaga</td>
<td>Malaga</td>
</tr>
<tr>
<td>Seismogenic zone</td>
<td>ZS34</td>
<td>ZS34</td>
</tr>
</tbody>
</table>

$^a$The area of the reservoir is calculated for the maximum operating level.

Table 2. Data concerning central block (fundamental triangle) of the Concepción Dam

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Height (m)</td>
<td>77.50</td>
</tr>
<tr>
<td>Base (m)</td>
<td>62.00</td>
</tr>
<tr>
<td>Volume (m$^3$)</td>
<td>2402.50</td>
</tr>
<tr>
<td>Mass density of concrete (kN/m$^3$)</td>
<td>24.0</td>
</tr>
<tr>
<td>Mass (10$^3$ kg)</td>
<td>5766.0</td>
</tr>
<tr>
<td>$T_1$ (s)</td>
<td>0.30</td>
</tr>
<tr>
<td></td>
<td>0.35</td>
</tr>
<tr>
<td></td>
<td>3642.12</td>
</tr>
<tr>
<td>Stiffness (MN/m)$^a$</td>
<td>2529.25</td>
</tr>
<tr>
<td></td>
<td>1858.22</td>
</tr>
</tbody>
</table>

$^a$The stiffness is obtained by equation $T_1 = 2\pi (\text{mass/stiffness})^{1/2}$. 

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Table 3. Performance curve data of the La Concepción Dam

<table>
<thead>
<tr>
<th>T_1 (s)</th>
<th>C_{d}/C_γ</th>
<th>P-C</th>
<th>2P-C</th>
<th>3P-C</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>A_p</td>
<td>\bar{y}_{CIA} (s)</td>
<td>\bar{y}_{DCR}</td>
</tr>
<tr>
<td>1.5</td>
<td>0.378</td>
<td>0.054</td>
<td>1.205</td>
<td>0.756</td>
</tr>
<tr>
<td>0.25</td>
<td>0.808</td>
<td>0.115</td>
<td>2.268</td>
<td>1.616</td>
</tr>
<tr>
<td>4.0</td>
<td>1.086</td>
<td>0.155</td>
<td>2.977</td>
<td>2.172</td>
</tr>
<tr>
<td>1.5</td>
<td>0.546</td>
<td>0.078</td>
<td>1.124</td>
<td>1.092</td>
</tr>
<tr>
<td>0.30</td>
<td>3.0</td>
<td>1.156</td>
<td>0.165</td>
<td>2.106</td>
</tr>
<tr>
<td>4.0</td>
<td>1.550</td>
<td>0.221</td>
<td>2.761</td>
<td>3.10</td>
</tr>
<tr>
<td>0.35</td>
<td>1.503</td>
<td>0.215</td>
<td>1.953</td>
<td>3.006</td>
</tr>
<tr>
<td>4.0</td>
<td>2.017</td>
<td>0.288</td>
<td>2.556</td>
<td>4.034</td>
</tr>
</tbody>
</table>

Table 4. Response curve results of the La Concepción Dam

<table>
<thead>
<tr>
<th>T_1 (s)</th>
<th>C_{d}/C_γ</th>
<th>a(g)</th>
<th>a(g)/1.37</th>
<th>a(g)/2.74</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>A_r</td>
<td>\bar{y}_{CIA} (s)</td>
<td>\bar{y}_{DCR}</td>
</tr>
<tr>
<td>1.5</td>
<td>0.00</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>0.25</td>
<td>0.517</td>
<td>0.074</td>
<td>1.189</td>
<td>0.224</td>
</tr>
<tr>
<td>4.0</td>
<td>0.00</td>
<td>0.167</td>
<td>0.767</td>
<td>0.360</td>
</tr>
<tr>
<td>1.5</td>
<td>1.10</td>
<td>0.157</td>
<td>1.438</td>
<td>0.366</td>
</tr>
<tr>
<td>4.0</td>
<td>0.273</td>
<td>0.128</td>
<td>0.0</td>
<td></td>
</tr>
</tbody>
</table>

Table 5. Dynamic pressures (the unit is kN/m²)

<table>
<thead>
<tr>
<th>z/H</th>
<th>P_i</th>
<th>P_c</th>
<th>P_i + P_c</th>
<th>p_w</th>
<th>p_w - P_i</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>P_i</td>
<td>P_c</td>
<td>p_i + p_c</td>
<td>p_w</td>
</tr>
<tr>
<td>1.00</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>0.50</td>
<td>8.747</td>
<td>4.751</td>
<td>13.498</td>
<td>9.631</td>
<td>0.884</td>
</tr>
<tr>
<td>0.00</td>
<td>12.370</td>
<td>3.267</td>
<td>15.637</td>
<td>13.275</td>
<td>0.905</td>
</tr>
<tr>
<td>Mean pressure</td>
<td>7.231</td>
<td>5.887</td>
<td>13.117</td>
<td>8.177</td>
<td>0.946</td>
</tr>
<tr>
<td>Mean force</td>
<td>25.551</td>
<td>26.643</td>
<td>32.298</td>
<td>27.420</td>
<td>0.946</td>
</tr>
</tbody>
</table>

aThis value has been calculated considering the mean between 0.5 x H x p_{max} and 0.4 x H x p_{max}.

bThis value has been calculated considering the mean between 0.6 x H x p_{max} and 0.5 x H x p_{max}.

cThe mean values are calculated considering the points where their resultant acts, i.e. between 0.4 and 0.6 H.
Table 6. Summary of the data

<table>
<thead>
<tr>
<th></th>
<th>R (m)</th>
<th>H_{tank} (m)</th>
<th>H (m)</th>
<th>T_{11} (s)</th>
<th>m_c (kg)</th>
<th>\xi_c (%)</th>
<th>PSA_{\xi_c = 5.0%} (g)^2</th>
<th>PSA_{\xi_c = 0.5%} (g)^2</th>
<th>d_{max} (m)</th>
<th>x(t) (m)^2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Equivalent tank</td>
<td>4.59</td>
<td>5.25</td>
<td>4.59</td>
<td>3.248</td>
<td>137 \times 10^3</td>
<td>0.5</td>
<td>0.21</td>
<td>0.28</td>
<td>1.08</td>
<td>0.75</td>
</tr>
<tr>
<td>Tank</td>
<td>140.0</td>
<td>52.5</td>
<td>42.0</td>
<td>24.73</td>
<td>2.13 \times 10^6</td>
<td>0.5</td>
<td>0.21</td>
<td>0.28</td>
<td>-</td>
<td>43.40</td>
</tr>
</tbody>
</table>

\(^a\)The sloshing displacement x(t) as a continuum is calculated by equation of motion [53].

\(^b\)Sloshing period of the first mode.

\(^c\)The elastic spectrum is calculated with a viscous damping ratio of 5\% for concrete and 0.5\% for liquid.