Technical Notes

Criteria for the Analysis of Vulnerability of Water Storage Tanks

Project for the Evaluation of the Seismic Risk for the Network of the Costa Rican Institute of Aqueducts and Sewers

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BACKGROUND

As a natural consequence of the study of seismic risk carried out on the infrastructure of the Costa Rican Institute of Aqueducts and Sewers (AyA) within the Greater Metropolitan Area of San José, specific studies have been planned to analyze specific system components that have scored very high risk values. As water reservoirs are included among these components, this Technical Note aims to pull together the most important aspects to be considered within the necessary structural analyses performed within a formal vulnerability study.

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OBJECTIVE OF VULNERABILITY STUDIES

Vulnerability is the potential for damage inherent to a system or part of a system. More specifically, if we are only talking about seismic hazard, (seismic) vulnerability means the susceptibility to damage from intense ground motion in a system. For the purposes of calculating losses (or risk), vulnerability should be given the form of a mathematical expression to estimate losses as a result of earthquakes characterized by their intensity; this mathematical representation is the vulnerability function. Vulnerability functions relate damage to a technical measurement of the intensity of ground motion, such as peak ground acceleration, or spectral acceleration.

Determining a vulnerability function is not an easy task to perform. In the latest edition of the World Congress on Earthquake Engineering held in Lisbon in 2012, more than 70 papers were presented on the subject of vulnerability, which gives an idea of how current the issue is in academic environments. Strictly speaking, a vulnerability function could be determined, theoretically, by making a nonlinear model of the entire element, and analyzing the responses of this model to different records using different intensities and frequency content. This process, in addition to being long, requires the orderly management of large amounts of information, much larger than that normally used in static or dynamic (spectral mode) analysis.

Experimentally, a vulnerability function can be determined by constructing several specimens of the element to be submitted on a vibrating table to a large collection of records which are representative of the movements actually expected in the location. This procedure is virtually impossible to run because, to avoid biased results, the elements must be all equal, and should be discarded whenever there is a foray into nonlinear behavior.

Between these two extremes, there is a range of possibilities that can be implemented. This paper presents some of the criteria that could be used to establish certain behavioral characteristics of tanks subject to movements of three different levels of intensity, and how these can be used to determine vulnerability functions.

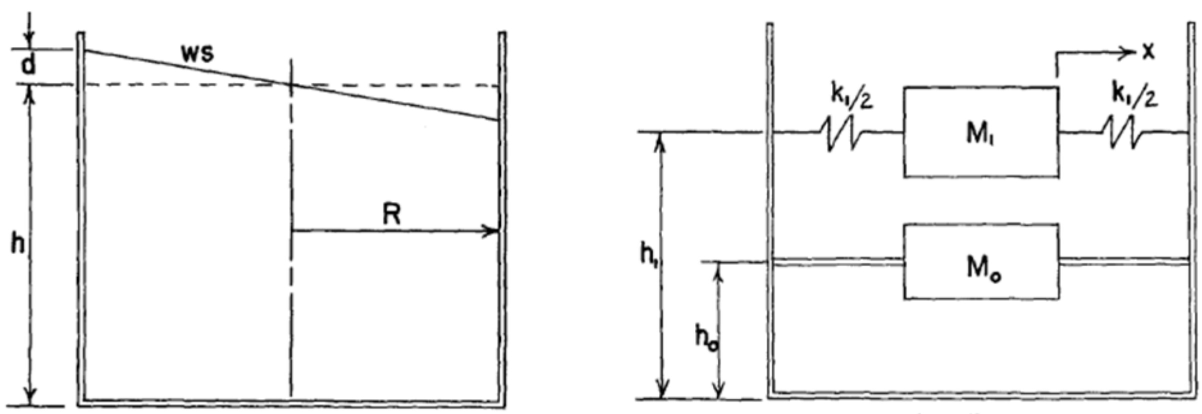
DYNAMIC BEHAVIOR OF TANKS

Tanks are structures that store large volumes of water; therefore the loads that must be considered in the design are directly related to the existence of these water masses.

Tanks Seated on the Ground

From the dynamic point of view, for tanks seated on the ground it should be considered that the ground acceleration causes the volume of water contained to behave in a specific way which is modeled as two masses connected to the tank walls at various heights.

Figure taken from "The Dynamic Behavior of Water Tanks" (George W. Housner, 1963)



ftp://ftp.ecn.purdue.edu/ayhan/Amer/The%20dynamic%20behavior%20of%20water%20tanks%20_%20Housner.pdf

Mass M_0 is a body of water which is rigidly attached to the tank walls. It is known as "Impulsive Mass," and it is located at a height h_0 measured from the base of the tank. The other part of the mass, M_1 is connected by elastic elements of overall rigidity k_1 and it is located at height h_1 This mass, known as "Convective Mass," and the rigidity with which it is attached to the tank walls, cause movement over a long period.

According to Jacobsen, for circular tanks of radius R and water depth h:

$$M_0 = M \frac{\tanh 1.7R/h}{1.7R/h}$$

$$M_0 = M(0.6) \frac{\tanh 1.8h/R}{1.8h/R}$$

$k_1 = 5.4 \dfrac{(M_1^2)}{M} \dfrac{gh}{R^2}$, where g is the acceleration of gravity

$$h_0 = \frac{3}{8} h \left\{ 1 + a \left[\frac{M}{M_1} \left(\frac{R}{h} \right)^2 - 1 \right] \right\}, \text{ where } = 1.33$$

$$h_1 = h \left[1 - 0.185 \frac{M}{M_1} \left(\frac{R}{h} \right)^2 - 0.56ß \frac{R}{h} \sqrt{\left(\frac{MR}{3M_1 h} \right)^2 - 1} \right], \text{ where } = 2.0$$

$T_W = 2\pi \sqrt{\frac{M_1}{k_1}}$ is the vibration period, and M is the total mass of the volume of water.

For rectangular tanks of 2L in length and water depth h:

$$M_0 = M \frac{\tanh 1.7h/L}{1.7h/L}$$

$$M_1 = M \frac{(0.83)\tanh 1.6h/L}{1.6h/L}$$

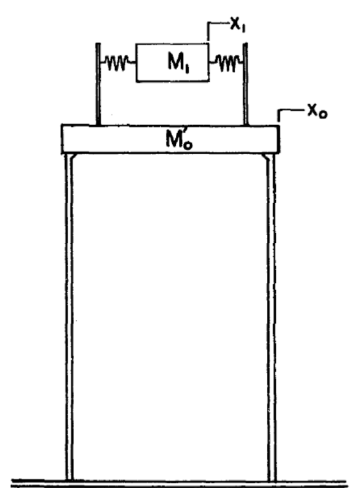
$$k_1 = 3 \frac{(M_1^2)}{M} \frac{gh}{L^2}$$

$$h_0 = \frac{3}{8} h \left\{ 1 + a \left[\frac{M}{M_1} \left(\frac{L}{h} \right)^2 - 1 \right] \right\}, \text{ with } = 1.33$$

$$h_1 = h \left[1 - \frac{1}{3} \frac{M}{M_1} \left(\frac{L}{h} \right)^2 - 0.63ß \frac{L}{h} \sqrt{0.28 \left(\frac{ML}{M_1 h} \right)^2 - 1} \right], \text{ with } = 2.0$$

Elevated Tanks

Elevated tanks have a vertical structure allowing the volume of water to be placed at a considerable height above ground, mainly in order to have a hydraulic strut that allows gravity water distribution. From the dynamic point of view, the systems of elevated tanks can be represented as shown in the figure below, where M_1 is the convective mass calculated for the tank volume (as though it were placed directly on the ground), and M_0 is the mass of the tank's support system plus the impulsive mass of the tank M_0 (calculated for the volume of the tank as if it were placed on the ground). The simplified model shown in the figure is reduced to a system of two degrees of freedom.



ACI350 Proposal

Committee 350 of the American Concrete Institute developed the document called "Seismic Design of Liquid-Containing Concrete Structures", which presents a range of specific technical considerations for the design of concrete structures containing or transporting liquids in general, and water in particular.

Based on Housner's theoretical development, the ACI350 committee proposes specific procedures for analysis and design, which differ in some numerical considerations from the original approach (which is the result of advances in research on the subject) shown in the previous section. To carry out the analysis and design of a tank, it would suffice to consult the documents produced by this committee. However, to understand the dynamic behavior of these structures, referral to Housner's publication is advised.

This document should be the basis of the structural analysis and revisions made to concrete tanks, although many of these considerations may apply to tanks made of other materials, mainly with regard to the determination of seismic impacts on structures on the basis of the seismicity prevalent in the area.

In general, earthquake forces should be determined according to the provisions established in local regulations (see Seismic Analysis), with regard to both elastic and inelastic demands (generally, it is not advisable to determine a standard spectra and use, for example, ductility reduction factors from another standard). For the specific case of Costa Rica, earthquake forces should be determined in strict compliance with the stipulations given in the 2010 Seismic Code, or later versions or it.

The Costa Rica Seismic Code contains the complete criteria and procedures for determining the Seismic Coefficient, *C* which represents the fraction of the mass (or weight) of the structure and content that act horizontally (earthquake force). In the case of tanks, the determination of this mass, as shown in the previous section, should take into account the dynamic behavior of the water content, for which ACI 350 proposes the following:

Rectangular Tanks

$$\frac{W}{W_L} = \frac{Tanh[0.866(L/H_L)]}{0.866(L/H_L)}$$

$$\frac{W_C}{W_L} = 0.264(L/H_L)tanh[3.16(H_L/L)]$$

Where W_i is the Impulsive Mass, W_C is the Convective Mass and W_L is the Total Mass of the liquid content. In the above equations, L is the inner dimension of the tank, measured parallel to the action of the earthquake, and H_L is the height of the water depth.

The height (h_C, the Convective Mass, and h_i, the Impulsive Mass) at which each of these masses are situated (and thus, where the earthquake force should be located) is determined by using the following formulas:

For tanks where

$$\frac{L}{H_L} < 1.333, \frac{h_i}{H_L} = 0.5 - 0.09375\left(\frac{L}{H_L}\right)$$

For tanks where

$$\frac{L}{H_L} \geq 1.333, \frac{h_i}{H_L} = 0.375$$

$$\frac{h_C}{H_L} = 1 - \frac{cosh[3.16(H_L/L)] - 1}{3.16(H_L/L)sinh[3.16(H_L/L)]}$$

The rigidity connecting the convective mass with the tank walls is associated with a period T_C equal to:

$$T_C = \frac{2\pi}{\lambda}\sqrt{L}$$

Where

$$\lambda = \sqrt{3.16gtanh[3.16(H_L/L)]}$$

Circular Tanks

In circular tanks of diameter D the Convective and Impulsive Masses and their corresponding heights are calculated as follows:

$$\frac{W_i}{W_L} = \frac{Tanh[0.866(D/H_L)]}{0.866(D/H_L)}$$

$$\frac{W_C}{W_L} = 0.230(D/H_L)tanh[3.68(D_L/L)]$$

For tanks where

$$\frac{D}{H_L} < 1.333, \frac{h_i}{H_L} = 0.5 - 0.09375\left(\frac{D}{H_L}\right)$$

For tanks where

$$\frac{L}{H_L} \geq 1.333, \frac{h_i}{H_L} = 0.375$$

$$\frac{h_C}{H_L} = 1 - \frac{cosh[3.68(H_L/D)] - 1}{3.68(H_L/D)sinh[3.68(H_L/D)]}$$

The rigidity connecting the convective mass with the tank walls is associated with a period T_C equal to:

$$T_C = \frac{2\pi}{\lambda}\sqrt{D}$$

Where

$$\lambda = \sqrt{3.68g\tanh[3.68(H_L/D)]}$$

In general terms, the period of the Convective Mass, T_C is considerably longer than the period of the Impulsive Mass, which is usually quite short. This difference in vibration periods makes it necessary to use a combination method to calculate the maximum design forces. To do this, after having calculated the earthquake forces for each of the water masses (P_i and P_C), plus the earthquake forces for the mass of the tank walls and ceiling (P_W and P_r), the shear at the base of the tank walls can be obtained by applying a variant of the rule of modal combination of the Square Root of Sum of Squares:

$$V = \sqrt{(P_i + P_W + P_r)^2 + P_C^2}$$

STRUCTURAL ANALYSIS—GENERAL CONSIDERATIONS

The structural analysis of a tank, whether supported or elevated, is actually a theoretically simple procedure, which is made complicated by the shape and behavior of the tank. This is particularly the case with thin-walled tanks, such as metal tanks, where the possibility of failure due to the instability of the walls should be taken into account. The loads to be considered in the structural analysis are as follows:

1. Dead or permanent loads. These are all the loads that act permanently on the structure. Dead loads include the weight of the structural elements themselves, the weight of partition or access elements, such as railings or walls that do not perform structural functions but enable the operation of the tank, or permanent equipment in the tank, such as fillings and surface finish materials. The Costa Rican 2010 Seismic Code states that the weight of the liquid contained in tanks should be considered as a permanent load, although it is not clear how the lateral thrusts produced should be considered. However, since the thrusts exist inasmuch as the weight does, these should be considered as a permanent load.

2. Live or temporary loads. These are loads that are variably present over time on the structure. These charges are normally associated with the occupation or operation of the element, and are usually governed by regulations applicable to conventional constructions or elements, which is not the case with tanks. It should be noted that the water tanks are not constructions within which human activities are permitted, but it is always necessary to consider that there may be some temporary load value for roof-covered tanks, walkways or corridors.

3. Encumbrances. These are loads that occur randomly over the life of a construction, and which are normally associated with highly variable natural phenomena. Encumbrances are earthquakes, extreme winds, forces caused by landslides, falling rain or hail, etc. In the case of the analysis of water tanks in Costa Rica, particularly in the GAM, special attention should be paid to the forces produced by earthquakes. The Costa Rican Seismic Code specifies three levels of threat to structural analysis and design:

    a. A strong earthquake, manifested at an intensity with a 10% probability of being exceeded in 50 years, corresponds to a return period of 475 years.

    b. An extreme earthquake has an intensity which is 25% higher than the strong earthquake. No exceedance probability or return period is specified, but for specific locations and if there is a well-developed seismic hazard study, the return rate to which it corresponds can be determined.

    c. A moderate earthquake has an intensity which is 25% lower than the strong earthquake, although the return period this corresponds to is neither indicated.

In all cases, the intensity referred to in the Seismic Code should represent the Peak Ground Acceleration (PGA).

SEISMIC ANALYSIS

The Costa Rican Seismic Code provides enough information to perform the structural analysis and design of earthquake-resistant tanks. According to this Code, tanks are classified as essential constructions or facilities *(Edificaciones o instalaciones esenciales)* for which a factor of importance I = 1.25 should be considered. Including this factor into the analysis implies that the return period for which these structures are designed is not 475 years, but a larger, indefinite, period. The same occurs with moderate earthquakes and extreme earthquakes, although no return period was determined for these.

The Code provides that for class A structures, *"… given extreme earthquakes (I = 1.25 according to Article 2.3 and Table 4.1), in addition to protecting the lives of residents and pedestrians, attempts should be made to minimize damage to the structure and those components and nonstructural systems capable of seriously disrupting the building's own services and functions."* This means that the Code severely limits the possibility of tanks being included the range of nonlinear behavior. It is not clear in the Code how this should be considered when a tank is being analyzed but this could be one of two alternatives:

1. Considering a very small displacement limit of the upper part of the tank which is consistent with expected behavior when it comes to limiting the amount of damage to the element. This would be the most direct way, but however it is necessary to find these limits in the literature available, subject to the analysis of each case.

2. Considering a reduction value as a result of low nonlinear behavior. This does not imply that the system or material used in the tank should be considered to be fragile, but that by reducing the likelihood of entering the range of nonlinear behavior, this also reduces, indirectly, damage to the element. This would be an indirect but fairly intuitive way of doing it.

The Code does not specify any special consideration for strong or moderate earthquakes, but, as what is expected in the case of extreme earthquakes is moderate nonlinear behavior, for strong earthquakes the behavior should be virtually linear elastic, with very few forays into the nonlinear behavior of the material, while for moderate earthquakes behavior should be completely elastic.

ANALYSIS OF SEISMIC VULNERABILITY OF TANKS

As discussed above, although the purpose of a study or analysis of vulnerability should be to obtain a vulnerability or fragility function, it is difficult to achieve this degree of accuracy within a structural assessment project. For this reason it is proposed that the structural assessment study provide information for at least three limit states affecting tanks, and a brief description of the damage expected for each one. The proposed limit states are as follows (see FEMA 273 and HAZUS MR4 Technical Manual):

1. Event with probability of exceedance of 39% over 50 years, or a return period of 100 years.
2. Event with probability of exceedance of 10% over 50 years, or a return period of 475 years (this is what is usually specified in seismic design codes).
3. Event with a probability of exceedance of 2% over 50 years, or a return period of 2475 years.

In particular, the study is expected to relate the results of the structural analysis of each of these cases, with the hoped-for behavior determined in the same document (HAZUS MR4 Technical Manual), these being:

a. Minimum damage. The tank loses neither content nor functionality. Small cracks and some minor roof damage due to water "splashing" may occur.

b. Moderate damage. There may be considerable damage to the tank but little loss of contents. In steel tanks there may be a degree of instability of the walls (elephant foot) without loss of content.

c. Extensive damage. The tank is severely damaged and should be put out of service. In steel tanks there may be elephant foot style flaws while in concrete tanks there may be large cracks and shear failure.

d. Total damage. The tank collapses and all content is lost.

In all cases, the consultant should make an effort to relate the condition of damage with a relative loss value, ß, which is representative of a limit state.

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