Technical Notes

Analysis of the Results of the Network Seismic Risk and Vulnerability Functions Evaluation Performed by the Costa Rican Institute of Aqueducts and Sewers

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BACKGROUND

With support from the World Bank, the Costa Rican Institute of Aqueducts and Sewers (AyA) undertook the development of a project to estimate the seismic risk of its infrastructure including 65 water storage tanks located in the cities of San José and in some outlying residential areas. Thanks to financial support from the World Bank Water Partnership Program (WPP) and the Government of Australia, AyA was also able receive technical assistance from the Mexican company Evaluación de Riesgos Naturales (ERN).

The result of this project was a risk estimate using two indicators: Annualized Loss Expectancy (ALE), and Probable Maximum Loss (PML). The first of these indicators is evaluated for both the entire portfolio and individually for each component within the AyA infrastructure network, while the second indicator is an estimate of the entire portfolio.

The overall results or the aggregate ones at portfolio level have allowed AyA to acquire knowledge about the risk situation of those of its assets exposed to the action of earthquakes. However, a review of ALE values obtained for individual components yielded some anomalous results, and thus a deeper analysis of all the stages involved in the risk assessment was proposed to identify the reason for these results. This paper presents the analysis performed, the findings, and the steps required to achieve the objective.

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RESULTS OF SEISMIC RISK ASSESSMENT FOR WATER AND SANITATION INFRASTRUCTURE IN COSTA RICA

The document *"Evaluación sísmica de los sistemas de agua potable y saneamiento de la Gran Área Metropolitana, San Isidro e Higuito—Plataforma CAPRA Modelación Probabilística de Escenarios de Riesgo para Centroamérica,"* prepared by the Research and Development Division of the Office of Environment, Research and Development of the AyA, presents the results of the risk assessment carried out on the company's infrastructure in parts of the metropolitan area of San José and the outlying towns of San Isidro and Higuito using the CAPRA Platform, a tool developed by the World Bank.

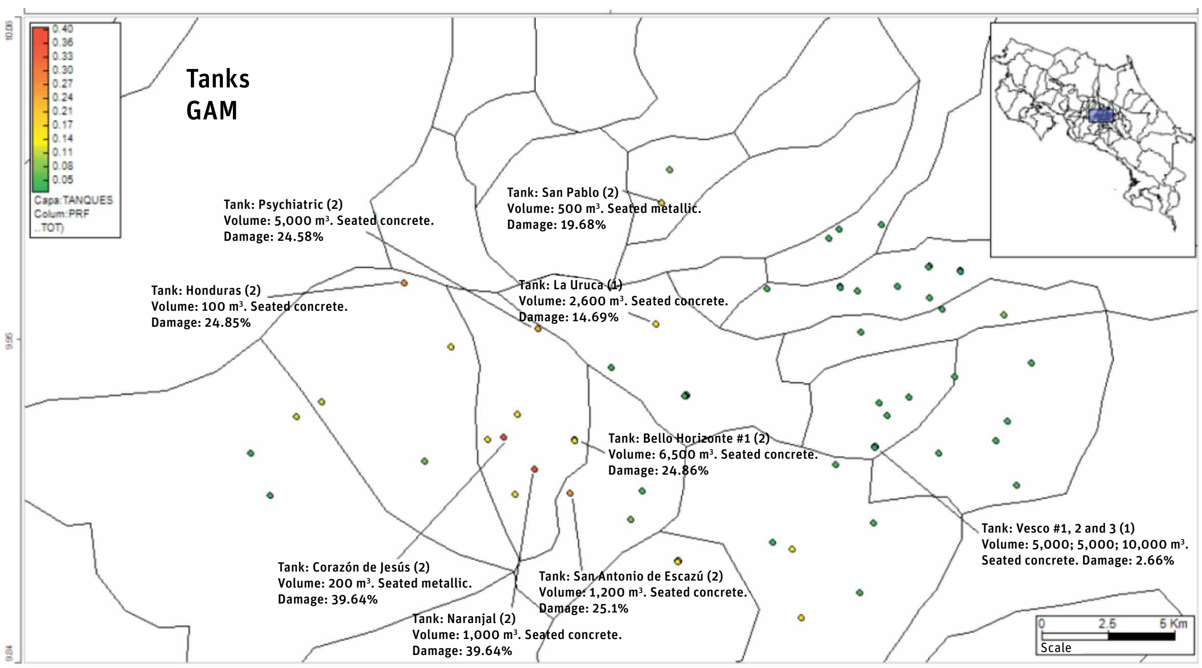
According to this document, after a detailed evaluation of more than 29,000 likely earthquake scenarios, the ALE values obtained for some water reservoir tanks are shown in Table 1.

Table 1 ALE obtained for some reservoir tanks

Name	Volume (m^3)	Type	Damage expected
Corazón de Jesús	200	Seated metallic	39.64%
Naranjal	1,000	Seated metallic	39.58%
San Antonio de Escazú	1,200	Seated concrete	25.10%
Bello Horizonte	6,500	Seated concrete	24.86%
Honduras	100	Seated concrete	24.86%
Psychiatric	5,000	Seated concrete	24.58%
La Uruca	2,600	Seated concrete	14.69%
San Pablo	500	Seated metallic	19.68%
Vesco #1, 2 and 3	5,000; 5,000; 10,000	Seated concrete	2.66%

It is noted that both reinforced concrete tanks and metal ones have very high ALE values, and there is also certain proximity between them (see Figure 1).

Figure 1 Tank location



These results are noteworthy for two reasons:

- The values are much higher than usual for this type of evaluation. For example, the ALE values of tanks evaluated in the study made on the company SEDAPAL of Lima varied between 1.8% and 11%.
- The results do not represent the reality of these elements. An element which has an ALE of 39% suggests that its condition is such that, on average, each year a seismic event damages it so that an equivalent investment of 39% of its value is required in order to restore it to the same condition it was before the event. Clearly, this has not happened in the case of these tanks. Most tanks listed have remained fully operational during the last 20 or 30 years and have undergone moderate seismic events without showing any serious damage.

In view of this, we proceeded to review all aspects of tank risk assessment so as to understand the reason for these results. The procedure was as follows:

1. Field visit
2. Review of information about site effects
3. Review of threat information
4. Review of vulnerability functions

FIELD VISIT

A field visit took place from the 4th to the 6th of February 2013, where besides visiting San José, the trip was used to check other aspects of risk assessment, such as site effects. On Tuesday, February 5th, two tanks were visited, one made of concrete (at the Escazú Cemetery, known as “Red Cross”) and one made of metal (Naranjal), both typical of the types of tank that AyA uses and for which unusually high ALE levels were recorded.

The Naranjal tank is a steel tank seated on a concrete platform located in an area with a number of slopes. The general appearance of the tank is good, there are no obvious cracks or instabilities of the walls, and as such, no loss of content is appreciated. The welded joints seem healthy and show no signs of fatigue damage, and at first sight seem to be of good quality without the presence of any rust or traces of bubbles (Figures 2, 3, 4, and 5). For this tank, risk assessment indicates an ALE of 39.58%, which does not adequately represent the tank's situation.

Figure 2 **View from the outside of the metal Naranjal tank. At first glance it looks in good condition, with no visible areas of corrosion nor any loss of content.**

Figure 3 **Detail of the metal walls of the Naranjal tank. The sheets forming the tank wall can be seen, as well as the welded seams, and the ripples which occur naturally in these kinds of welded joints.**

Figure 4 **The Naranjal tank sits on a base of reinforced concrete, with no apparent anchor. At the time of inspection, the tank was almost full, and no loss of content could be seen at any point of the tank.**

Figure 5 **To the naked eye, the welded joints seem to be in good condition, although it is always possible to find some flaws in the welding work, which are, nonetheless, not serious.**

The other tank visited was at Escazú Cemetery (known as “Red Cross”), made up of reinforced concrete, rectangular in shape, and with a lid. Several construction problems were found in this tank, mainly related to activities of filling compaction, which had created relative movements along the sidewalks with respect to the tank itself—that, however, have not affected it—and that have only shown up as problems with the pipe connections. Besides, in the tank roof there is evidence of the inadequate application of coating, and on the walls there are traces of leaks. According to AyA, the construction of this tank was carried out by a company that made several errors and incurred in construction deficiencies which were repaired later. The tank is fully operational.

Figure 6 Side view of the Escazú Cemetery concrete tank

Figure 8 Evidence of compaction problems in the neighborhood of the Escazú Cemetery concrete tank

Figure 7 Another side view of the Escazú Cemetery concrete tank. Note the stains left by leaks on the wall.

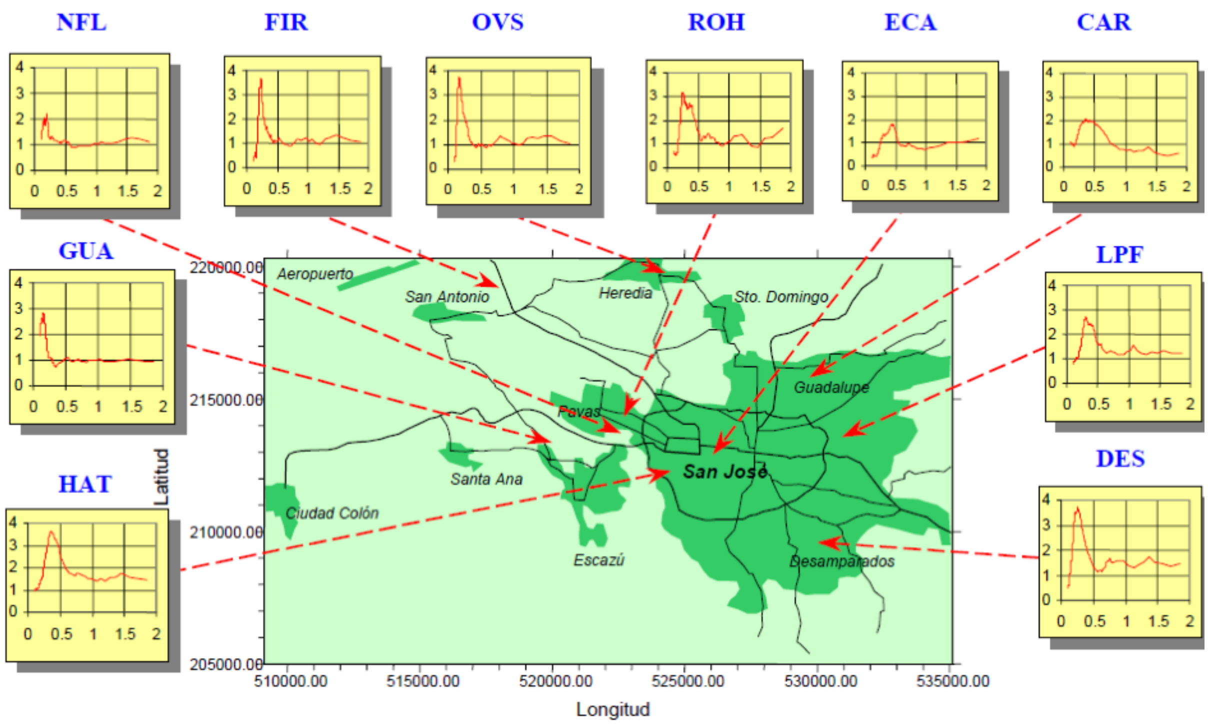
Figure 9 Picture of the Escazú Cemetery tank roof. Repairs are visible in well-defined areas.

It is not known whether the constructive state of this tank was taken into consideration for the risk assessment, although probably not, as is usually the case in a study of this type when the number of components to be considered is so great that it is not possible to inspect each one and take their real and current condition into account. It is common to consider vulnerability functions which are representative of the structural system and modifying them according to some aspects, such as the age of the tank and its general condition, as was done with the AyA study. Despite the condition of the tank, it is difficult to accept that its ALE is 15.18% (not shown in Table 1).

REVIEW OF INFORMATION ABOUT SITE EFFECTS

In view of the apparent proximity of tanks with a high ALE, a possible explanation for these results could be related to local effects or site effects. To explore this possibility, we used site effects studies developed by the University of Costa Rica, which considered local effects due to soil deposits (Figure 10).

Figure 10 **Amplification functions in different parts of the Greater Metropolitan Area of San José. The acronyms identify the place where the evaluation was conducted, while the abscissa corresponds to structural periods and the ordinates to intensity amplification factors.**



According to these studies, the amplification caused by the presence of soil varies to a little under 4 (Figure 10). The value of this amplification depends on the type and depth of the soil deposit, and the structural vibration period of the element being analyzed, in this case the water tanks or reservoirs. To determine how soil type contributes to risk estimates, an assessment of the ALE was made without regard to amplification acceleration caused by local effects and then compared with the original assessment (only for tanks within the Greater Metropolitan Area). The results indicated that the increase in the ALEs were due to the inclusion of the site effect in the original study which in some cases was nearly 7 times.

Figures 11 and 12 show the amplification values depending on the structural period for different levels of ground acceleration determined by the ERN study consulting team.

Figure 11 Amplification functions for different soil types, structural period and Intensity (Part 1)

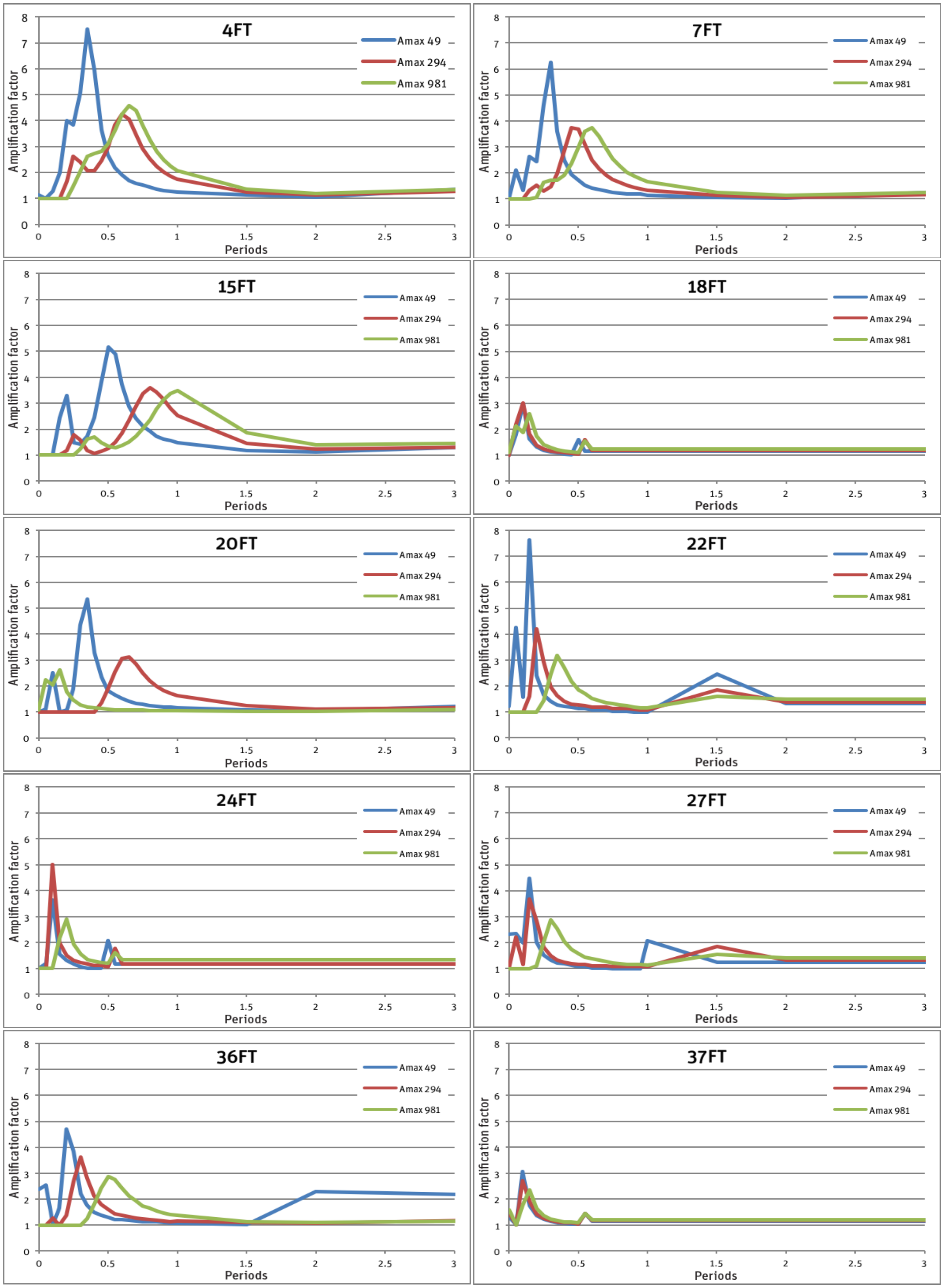
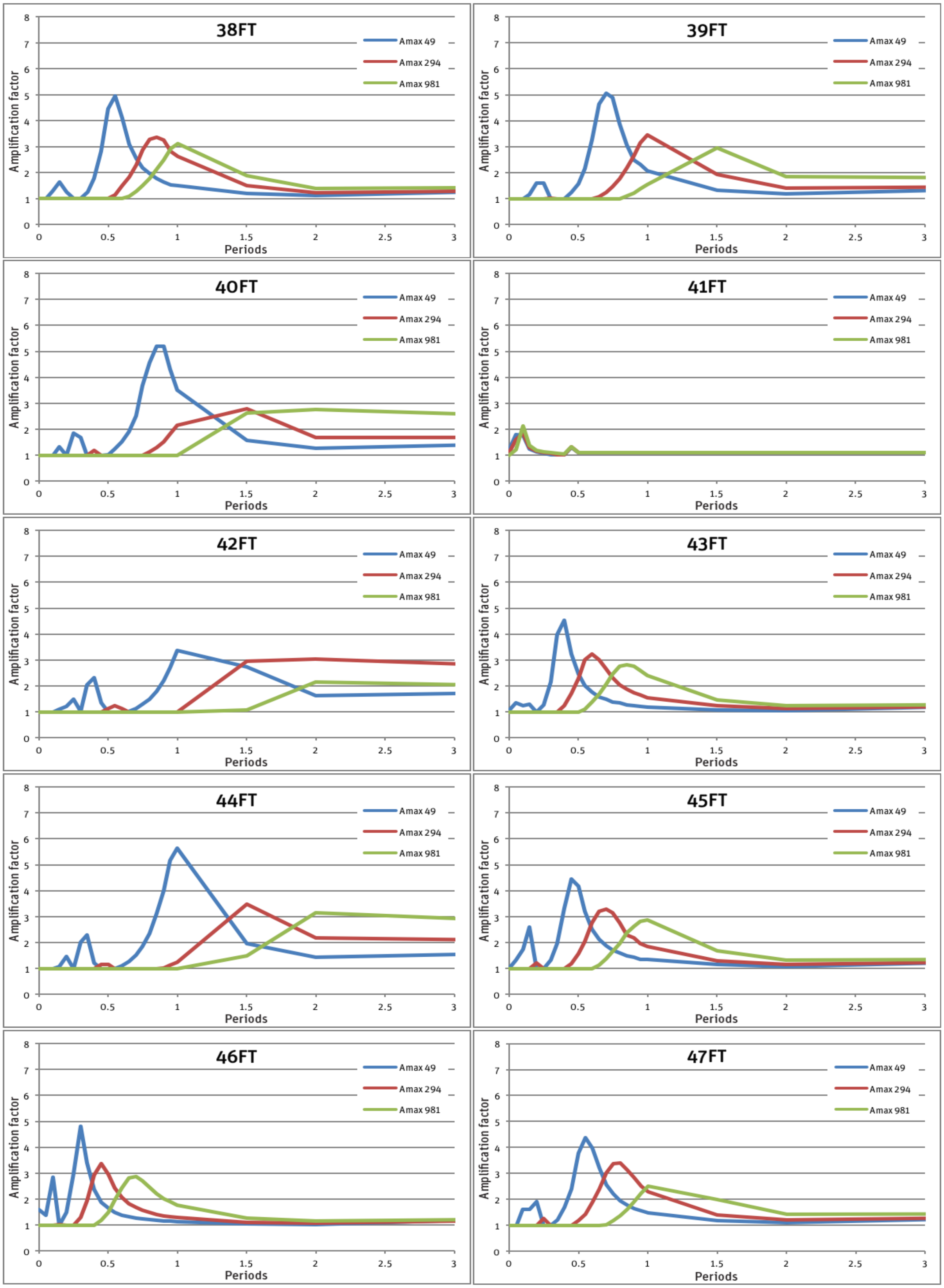


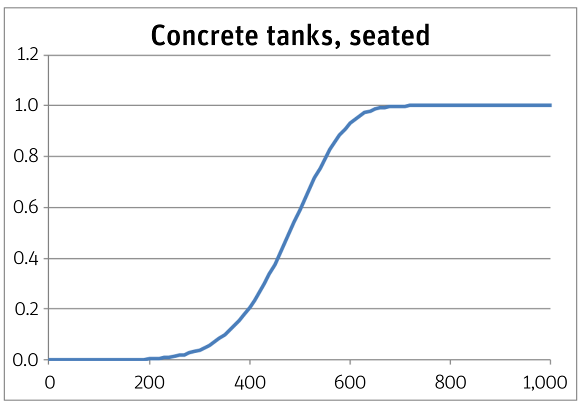
Figure 12 Amplification functions for different soil types, structural period and Intensity (Part 2)



We can see that the maximum amplifications due to the ground are around 7.5 (4FT); however, the vulnerability functions used for tanks use peak ground acceleration (PGA) as a measure of intensity, i.e., the corresponding amplification is period 0, which uses a maximum value of about 2.4 for 27FT and 36FT soils. This means that the amplifications caused by soil type during the acceleration suffered by tanks (up to 2.4) are much smaller than the increases in expected damage (up to 7). It should be noted that the increase in the values of expected damage is not generally proportional to increases in acceleration, and, depending on the vulnerability curve type of the tank, small increases in acceleration can lead to further increases in the expected damage, and hence, local effects may explain an increase in ALE, but not necessarily explain increases as high as those presented in this case.

The typical shape of a vulnerability function (which establishes a relationship between the expected percentage of damage vs intensity expressed in PGA) is as follows:

Figure 13 **Typical form of a vulnerability function, where the abscissa corresponds to peak ground acceleration (PGA) and the ordinates to the expected damage percentage.**



Then, in order for small increases in the value of acceleration (about 2.4) to produce large increases in expected losses, unamplified loss values should be significantly smaller than the values of amplified losses. This can occur, as shown in Figure 13, if the unamplified intensity is less than about 200 Gal, in which case, the expected value of damage is very close to zero, and when it is amplified, the intensity rises to 500 Gal, associated to an expected loss value of about 50%. This highly disproportionate behavior is characteristic of some ductile structural systems, where the occurrence of a catastrophic and sudden failure is possible. This does not seem to be the case with tank structures, where the internal forces that develop in the tank often tend to be bending moments and axial tensile forces (and in very few cases, compression).

In conclusion, the amplification values that were used for loss calculations correspond to the levels reported in study results, so the sharp increase in PAE tanks **is not related** to the amplifications produced from local effects, but probably to the structure of the vulnerability functions.

REVIEW OF THREAT INFORMATION

It is necessary to rule out that acceleration levels (threat) are disproportionately high in the Costa Rican model. To this end, the CAPRA post-processor was used to generate curves for different return periods of isoacceleration (Figures 14, 15, 16, and 17).

Figure 14 **Isoacceleration curves for a return period of 50 years**

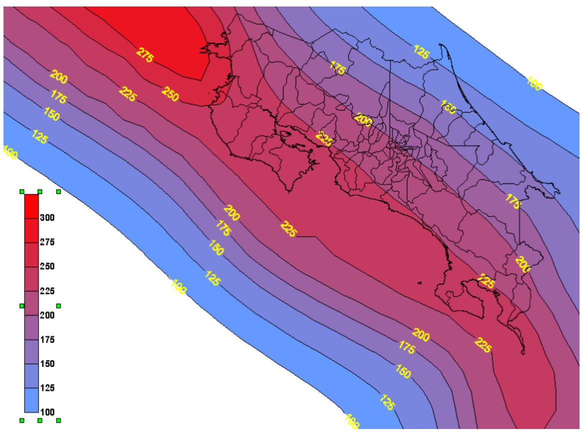


Figure 15 **Isoacceleration curves for a return period of 100 years**

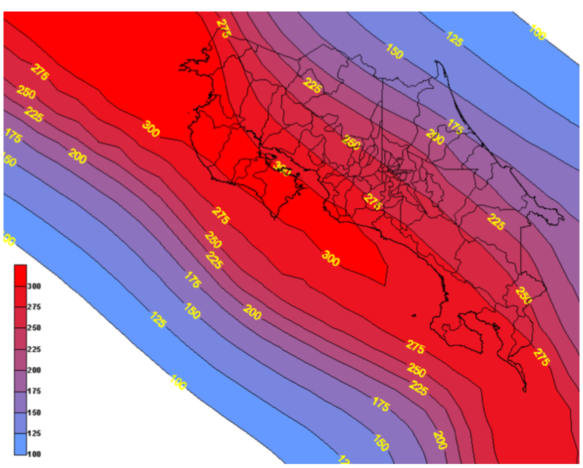


Figure 16 **Isoacceleration curves for a return period of 500 years**

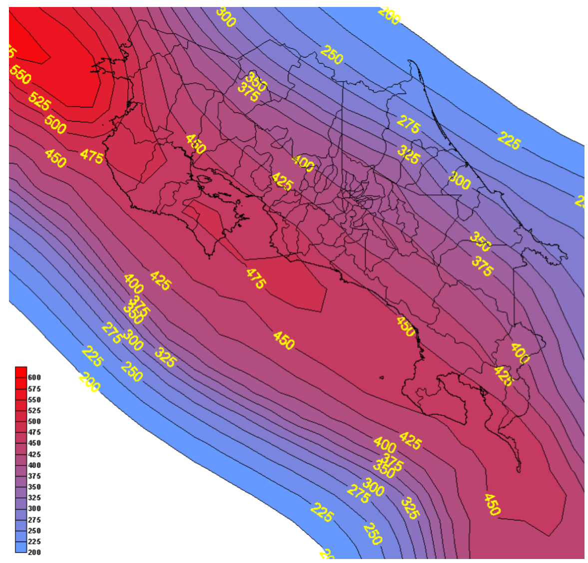
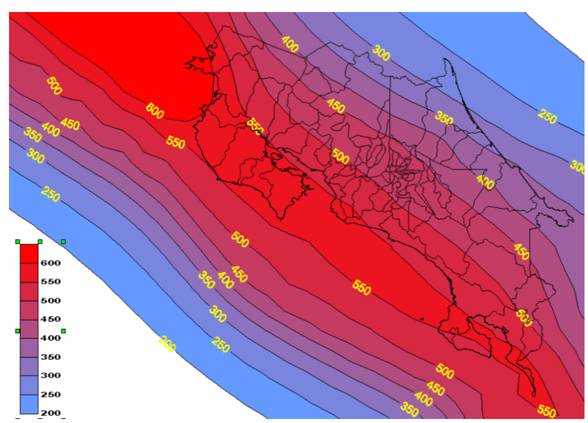


Figure 17 **Isoacceleration curves for a return period of 1,000 years**



PGA values in San José are approximately 200 Gal for a return period of 50 years, 250 Gal for a return period of 100 years, 400 Gal for a return period of 500 years, and 475 Gal for a period return of 1,000 years.

From the above charts, it can be seen that the intensity levels, at least for PGA, are in the same order as the threat in other regions where there have been similar risk studies, such as Peru, despite the fact that no studies have observed such high ALE levels in tanks.

Therefore, the threat levels are not the reason for the ALE values obtained in the Costa Rican study.

REVIEW OF VULNERABILITY FUNCTIONS

To verify the vulnerability functions proposed by AyA for analyzing the seismic risk of infrastructure, the case of seated metal tanks with an anchor has been taken under consideration in the first place.

For this type of structure, as for all other structural systems, the method established by HAZUS to determine damage functions or features of fragility has been followed. To determine fragility functions, HAZUS proposes a fairly large collection of values for the parameters that define the different conditions of damage for analysis. These parameters have not been revised, although it is likely that, as the threat conditions are not the same in Costa Rica as in the United States, the values should be slightly readjusted.

The passage of fragility functions to vulnerability functions was confirmed as having been correctly implemented by AyA. The main source of uncertainty in this conversion is undoubtedly the relative damage (RD) reference values associated with different conditions of damage. These RD values are the most important functions linking fragility to vulnerability functions, hence their importance.

In the case of steel tanks which are seated and anchored, RD values considered by the AyA are shown in Table 2.

Table 2 Relative damage values used for water and sanitation system components

Condition of Damage	RD	Description
Minor damage	20%	Tanks suffer minor damage without loss of content or functionality. There is minor damage to the roof of the tank due to water splashing, small cracks in concrete tanks or wrinkles on the walls of steel tanks.
Moderate damage	40%	There may be considerable damage to the tank but little loss of contents. In steel tanks, the walls present a degree of instability (elephant foot) but there is no loss of content, or there may be mild cracking with little loss of content in concrete tanks.
Extensive damage	80%	The tank is severely damaged and should be put out of service. In steel tanks, the walls present a significant degree of instability (elephant foot) with loss of content. The bars in wooden tanks or the walls of concrete tanks are deformed.
Complete damage	100%	Tanks collapse and lose all their contents.

It should be noted that there are very few references to RD values which have widespread acceptance among professional engineering circles. In addition to this, the definition of damage provided by HAZUS does not offer many objective technical data to relate loss to, say, return periods or some other measure of recurrence. Given this, it is clear that determining RD values for each condition of damage will always be a subjective exercise associated with high uncertainties. Despite this, in the table of values above, a value of RD 0.2 (20%) for a condition of minor damage may, at first, be consider to be very high.

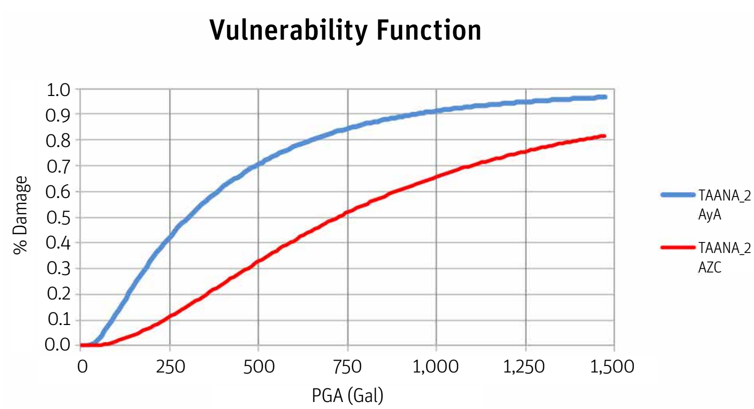
Let us consider this particular case. In a condition of damage graded as "minor," a relative loss of 20% in a tank means that a fifth of the value of the tank is lost or unusable, but that it has not lost any content or functionality. This combination of conditions does not seem to be consistent, as a much smaller relative value of damage was expected. To illustrate this, a relative damage value equal to 2% will be proposed for minor damage. All other conditions of damage can be analyzed in this way (Table 3).

Table 3 **Relative damage values proposed to illustrate changes in vulnerability functions**

Condition of Damage	RD (AyA)	RD (AZC/illustrative proposal)
Minor damage	20%	2%
Moderate damage	40%	15%
Extensive damage	80%	50%
Complete damage	100%	100%

With these proposed values, vulnerability curves are modified in the manner shown in Figure 18.

Figure 18 **Vulnerability functions for seated unanchored metal tanks proposed by AyA (blue), and for this report (red)**

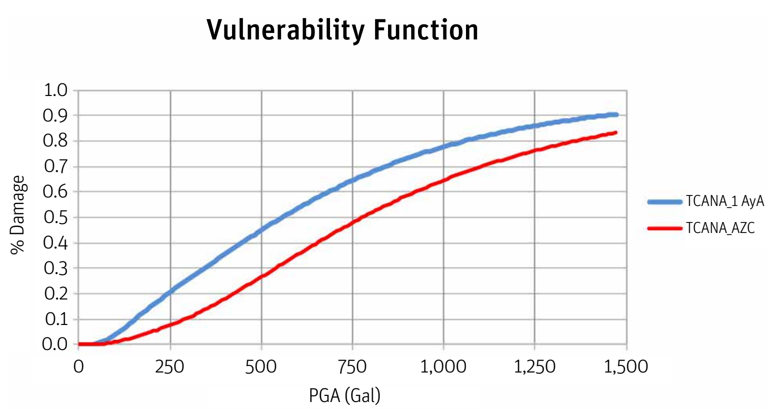


The blue curve is the one originally used by AyA, while the red curve plots the relative damage values proposed for this report. This modification generates a curve which, in general, uses lower relative damage values than the original, although the steepness of the curve is maintained in some sectors, which means that the fault mode shares the same essential behavior. It can be seen that when the intensity of the event is small (say less than 490 Gal), the values of the modified curve are significantly lower than those of the original, and, in particular, when the values are less than 250 Gal in intensity, this difference may be considerable. Since the ALE is a weighted calculation of losses that can be caused by all events, considering their annual occurrence frequency, then it is very likely that these small but very frequent events will have an important influence on the final risk premium estimate, which explains the very high value of losses in the estimates of the AyA, which are unrelated to the actual condition of these tanks.

To confirm this, we proceeded to make an assessment of all GAM tanks, using, for the Naranjal tank (inspected metal tank), a vulnerability function such as that shown in Figure 18. The ALE of this tank was found to be 8.65%, which is probably closer to the real condition of threat and vulnerability.

For the other tank inspected, a vulnerability function resulting in an ALE of 15.18% was originally used. To redefine the vulnerability function, the same RD values were used as those used in the case of the metal tank, which resulted in the vulnerability function shown in Figure 19.

Figure 19 **Vulnerability functions for seated unanchored reinforced concrete tanks proposed by AyA (blue), and for this report (red)**

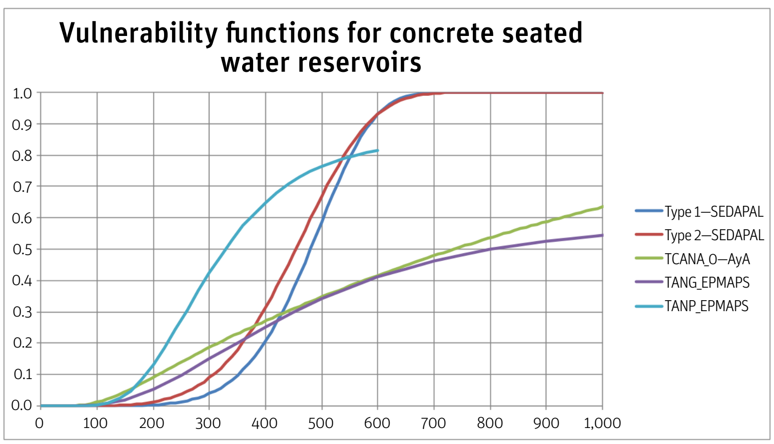


As in the case above, there is a slight reduction in the relative loss values obtained with the new function with respect to the one originally employed by AyA. The use of this vulnerability function leads to an ALE of 5.93%.

COMPARISON WITH VULNERABILITY FUNCTIONS IN OTHER STUDIES

In addition to this study by AyA, there are the results and vulnerability functions included in the studies undertaken by the SEDAPAL, which is the Lima Drinking Water and Sewerage Services, and by the Public Metropolitan Water Supply and Sanitation Company (EPMAPS) of Quito. The vulnerability functions of the seated storage tank elements employed in each of these studies were analyzed. It should be stated that although this analysis is illustrative, comparing vulnerability functions should not be done directly without taking into consideration the specific technical aspects of each country or region, as well as local design and construction regulations, which are aspects related to the construction practices prevalent in each country, in addition to threat levels, among others.

Figure 20 **Vulnerability function, where the abscissa corresponds to peak ground acceleration (PGA) (0–1,000 Gal) and the ordinates to the expected damage percentage**



The following vulnerability functions were compared:

1. Type 1—SEDAPAL. This function is applicable to the reinforced concrete tanks in the SEDAPAL network, which are "small," and seated on the ground. The study does not indicate whether or not it has a lid, but given the features of this tank, it is very unlikely to have a cover. These so-called "small" tanks do not resemble those reviewed in the AyA study.

2. Type 2—SEDAPAL. This function is applicable to the reinforced concrete tanks in the SEDAPAL network, which are "large," seated on the ground, and may or may not have a lid (this is not indicated in the report).

3. TCANA_O—AyA. This is a vulnerability function for concrete tanks seated on the ground in the AyA network.

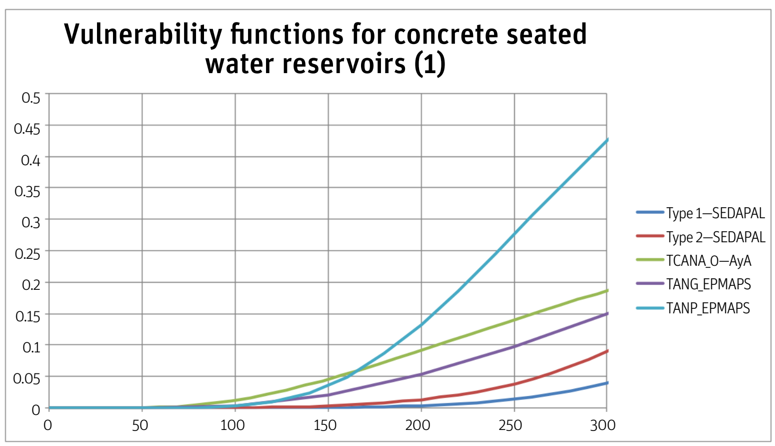
4. TANG—EPMAPS. This corresponds to tanks (the material is not specified, but is assumed to be concrete) with a lid, seated on the ground.

5. TANP—EPMAPS. This corresponds to tanks (the material is not specified) without a lid, seated on the ground.

It can be seen that there is a great diversity of forms and loss values for the same intensity in the different curves. However, two types of curves can be distinguished: the ones of SEDAPAL on the one hand, and those of EPMAPS and AyA on the other. The vulnerability functions used in the SEDAPAL study are very different in terms of values from the curves in the EPMAPS and AyA studies, which is probably due to the various references used to determine these functions. SEDAPAL curves are characteristic of more fragile structural systems than those of AyA and EPMAPS, which means that in the SEDAPAL study it has been considered that the failure of a tank occurs relatively suddenly, without warning and within well-defined intensity limits, whereas AyA and EPMAPS curves suggest that the tanks will tend to fail gradually, slowly, showing the deterioration associated with the various levels of damage reached during successive load cycles.

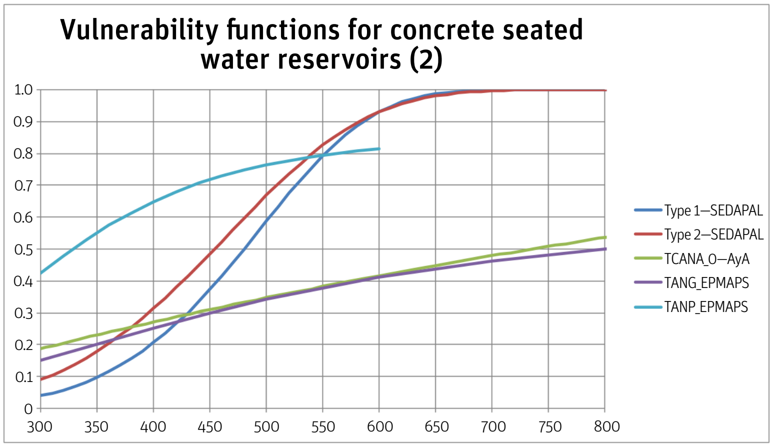
If the curves for levels of moderate or low intensity are examined in greater detail (Figure 21), this shows that the curve given by AyA generates higher values than the other curves (for the moment, we will leave aside the TANP-EPMAPS curve values). Concerning some of the curves, mainly TYPE 1—SEDAPAL, the loss values for the same intensity are significantly higher using the TCANA_O curve—AyA. Since it is very likely that the calculation of ALE will be strongly influenced by events in this range of intensities, it is to be expected that the AyA results will be greater than those obtained using the other vulnerability functions.

Figure 21 **Vulnerability functions, where the abscissa corresponds to peak ground acceleration (PGA) (0-300 Gal) and the ordinates to the expected damage percentage**



Now, let us look at the range of highest intensity (Figure 22). It can be seen that in a small initial interval, things remain the same as for low intensities, but suddenly the SEDAPAL curves increase their slope and significantly raise loss values, while the AyA and EPMAPS curves maintain an almost constant slope. Thus, for high intensities, the SEDAPAL vulnerability functions will produce much higher damage values than those of AyA and EPMAPS. This is relevant to the calculation of the ALE, where the interest is usually set on very high losses, associated with very rare events and therefore, very intense ones.

Figure 22 **Vulnerability functions, where the abscissa corresponds to peak ground acceleration (PGA) (300-800 Gal) and the ordinates to the expected damage percentage**



Accordingly, it can be concluded that for SEDAPAL, tanks are elements that work well at low intensities, but that when the intensity is increased slightly, widespread damage can occur violently, while for AyA and EPMAPS, the deterioration is gradual. It is of interest to establish which of the two approaches bears a greater relation with the true and actual behavior of concrete tanks when subjected to ground movements.

In concrete tanks, the dimensions of the tank walls are such that the possibility of brittle failure is minimal. Other fault types appear unlikely, such as tumbling or the failures at the level of the mechanical anchorage to the base, mainly because concrete tanks are nearly always built monolithically with the base incorporated, i.e., the base and walls form a heavy rigid continuum, which needs very high ground accelerations to flip it or make it crack at the point where the walls meet the floor slab.

In concrete tanks, earthquake-related tank failures seem to be more associated with stress overexertion (whether pure or induced by bending moments) on the tank walls. In any of these cases, failures initially manifest themselves as a vertical or horizontal crack (depending on the direction in which the failure starts) associated with major deformations in the tension of the reinforcing steel. This means that, as loads are increased, tank walls deform, stressing the materials; and, since concrete itself has a very poor response to stress, it is the reinforcing steel which significantly bears these tensions, causing the concrete to crack. The increase in the size of the cracks is proportional to how the steel behaves in its elastic range. As the steel enters its nonlinear behavioral range, it is plasticized and begins to experience large deformations and therefore, large increases in the size of the crack. This process continues until the deformation of the steel reaches very high values, which is when there is an apparent hardening of the steel, able to withstand higher loads with only slight increments in deformation. This behavior interval is relatively short, and generally precedes a fault or break in the steel under tension.

In light of this failure mechanism, one could conclude that none of the curves shown adequately reflect the behavior, because, although the structure is relatively ductile, in the AyA and EPMAPS curves the influence of the flow of the steel under tension is not observed, and moreover, the development of this failure mechanism is not so violent as to produce functions with such a pronounced curve as the SEDAPAL slope. Now this failure mechanism is purely structural and does not take into account other failure criteria that may be relevant, such as loss of content through cracks appearing in concrete walls.

RECOMMENDATIONS

Accordingly, it is recommended that new vulnerability functions be obtained, following the procedure employed by AyA but trying to document or better relate the different levels of damage to relative loss values and taking into account the characteristics of seismicity inherent to Costa Rica.

In essence, the idea is to determine the RD values that best describe the dynamic behavior of the tanks during events of different intensities. In order to establish this, the response of the structure to specific requests should be taken into account, considering the existence of a large amount of water as content and the levels of threat to the location of the tank (including design and construction practices). All these aspects can be found in the Technical Note "Criteria for the Analysis of Vulnerability of Water Storage Tanks."

In the case of steel, it is necessary to consider that brittle failure may occur, such as the instability of the walls (elephant foot, or buckling at the bottom of the tank wall where the axial compression due to the overturning moment is at maximum level), or failure due to the anchors shearing off, or failures in welds. It is usual for tanks to be designed to be very conservative regarding these failure modes, favoring the ductile behavior of the structure. In this case, as is the case with concrete tanks, tank walls will deform radially as the earthquake loads increase until the steel reaches its limit of elasticity, and the subsequent process is similar to that described above with the difference being that in these tanks there is no cracking of the walls. The possibility that the tanks may fail due to steel wall tension is relatively low due to the high strength of steel. After that, it is feasible that other failure modes may begin to acquire greater importance. The most common failure is the elephant foot and the failure of the weld seams. So, the most suitable vulnerability function for this type of structure should show little damage in the first interval, scaling to relatively high intensities, which depends on local analysis and design specifications, and then a brittle behavior (large curve slope); meaning curves which are very similar to SEDAPAL's, but where the increase in slope values is manifest in higher intensity values. Of course, this must be calibrated with the resistance values used for designing the tanks.

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