Seismic risk evaluation of water elevated tanks

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ABSTRACT
Maintaining the functionality of the water lifeline networks is a primary objective, not only in periods of ordinary management but also during emergencies. This paper presents the seismic risk analysis of Reinforced Concrete (RC) water elevated tanks belonging to the lifeline network managed by AcqueVenete S.p.A. In particular, the first results for five water elevated tanks located in low seismicity areas of Polesine, Italy, were presented. These structures are part of a database collecting about 100 water elevated tanks of sixteen different structural typologies, designed in the absence of specific seismic design rules. The analyses carried out made use of a probabilistic approach starting from a set of pushover analyses, from which inelastic Single Degree of Freedom (SDOF) systems, one for each structure, were defined. These SDOF systems were then subjected to nonlinear dynamic analyses for a number of scaled accelerograms, according with the Multiple Stripe Analysis technique. The fragility curves obtained, combined with the hazard curves, provided the mean annual frequency of the particular damage state considered (e.g. collapse). From here, it was possible to quantify for each structure the average annual loss index by integrating the expected annual loss curve.

1 INTRODUCTION

Numerous earthquakes around the world, such as the 1964 Alaska, 1964 Nigata and 1995 Kobe, Japan, 1971 San Fernando, 1979 Imperial Valley, 1989 Loma Prieta and 1994 Northridge, California, and the more recent 2012 Emilia (Brunesi et al., 2015), Italy, earthquakes, caused large and severe damages to water elevated tanks (Haroun and Ellaithy, 1985; Kafle et al., 2011). The seismic performance of these structures is crucial for the functioning and management of post-quake emergency services. The 1906 San Francisco earthquake, only to mention a catastrophic example, produced dramatic economic losses due to fire, because of the lack of water caused by the collapse of the water tanks.

An elevated tank is a relatively simple structure comprised of a RC stem topped by a reinforced concrete container. Several studies investigated the behavior of water tanks under seismic action. Housner (1963) analyzed the behavior of water tanks with a simple model of fluid-structure interaction based on a two-mass system. This model allowed taking account in a simplified form of the effects due to the sloshing wave. After that, some studies investigated the fluid- and soil-structure interactions, which typically characterize water elevated tanks. For example, Livaoğlu and Doğangün (2006) developed a simplified seismic analysis procedure considering both these phenomena.

Currently, quantifying earthquake-induced losses has become an important challenge for communities in seismic areas. The key parameter commonly used to quantify and compare the seismic performance of buildings is known as the Expected Annual Loss (EAL) (Calvi, 2013; Liel and Deierlein, 2013; O'Reilly and Calvi, 2019).

The evaluation of the EAL is a decision variable of Performance-based earthquake engineering (PBEE) (Deierlein et al., 2003; Krawinkler and Miranda, 2004). The main objective of this framework, developed by Pacific Earthquake Engineering Research (PEER) center, is to quantify the seismic performance of structures using performance measures in order to provide engineers, stakeholders, investors and all possible decision makers with a decision variable. This probabilistic framework considers uncertainties in

the seismic hazard, seismic response, damage estimation, and risk estimation and allows these uncertainties to be propagated and rationally accounted for.
This contribution shows the first results of the numerical analyses conducted on five different water elevated tanks (Figure 1) situated in Italy. All these structures, conceived in the absence of specific design rules for seismic resistance, present nowadays a significant material degradation. This evidence induced AcqueVenete S.p.A. to promote research on the seismic risk of lifeline networks.
A rapid tool to evaluate the risk of collapse or the EAL of the tanks was defined according with the method outlined by Silva et al. (2019). To obtain the fragility curves for the tanks, we first carried out a set of pushover analyses. From these analyses, a SDOF system was obtained for each tank. Then, each SDOF system was subjected to a probabilistic assessment based on nonlinear dynamic analyses. The following sections describe the investigated structures (§ 2) and the corresponding Finite Element (FE) models (§ 2.1), the seismic performance assessment (§ 3) and the EAL estimation (§ 4.2).

2 CASE STUDIES: ITALIAN WATER ELEVATED TANKS

From a database of about 100 water elevated tanks under control of the managing institution AcqueVenete S.p.A. five different structural typologies were selected. These typologies can be considered as the prevalent typologies of water elevated tanks present in the province of Rovigo.
The selected tanks can be subdivided into two categories depending of the supporting structure, which may be a RC shell or a RC frame (Table 1).

Table 1. The five water elevated tanks considered

Site location	Year of construction	Type of RC supporting structures
Ariano nel Polesine - Rivà	1960	shell
Castelnovo Bariano - Centrale	1974	shell
Corbola - Centrale	1958	frame
Occhiobello – S. M. Maddalena	1985	shell
Taglio di Po - Polesinello	1983	shell

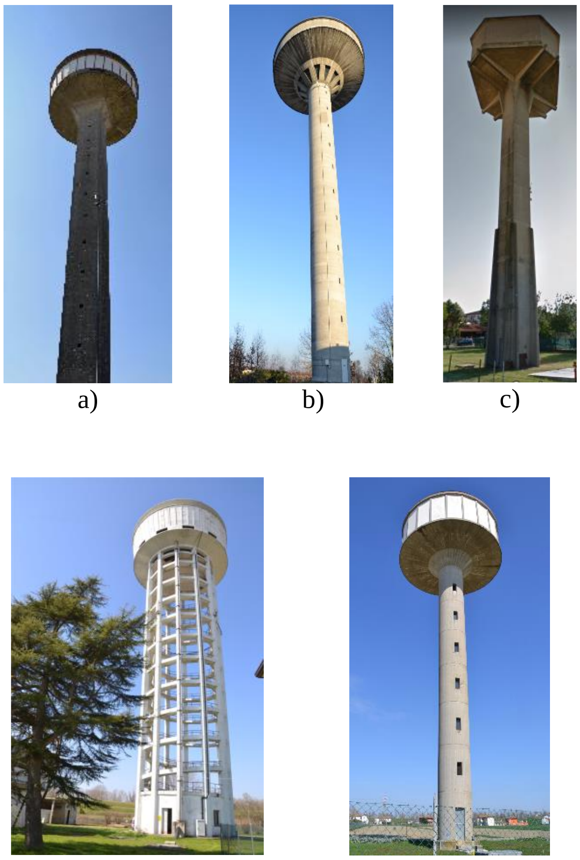
a) b) c) d) e)

Figure 1. Water elevated tanks considered, located in a) Ariano nel Polesine – Rivà, b) Castelnovo Bariano – Centrale, c) Occhiobello – S. M. Maddalena, d) Corbola – Centrale, e) Taglio di Po – Polesinello

2.1 *Numerical Modeling*

For evaluating the vulnerability of these structures, a Finite Element (FE) model is developed (Figure 2). For each typology, we performed several detailed FE analyses both non-linear static and linear dynamic employing with the commercial software MIDAS Gen. The linear dynamic analysis is employed for understanding vibration modes of structure and their effective masses (Tables 2-3). The non-linear static analyses are used for obtaining the simple degree of freedom (SDOF) that it is employed later in nonlinear time-history analysis. In particular, in the case of a tank with RC shaft supporting structure, a quick method for pushover analysis was adopted. This method consists in idealizing the structure as a single cantilever beam including geometric and material nonlinearities, the latter through a fiber model. In the case of a tank with frame supporting structure, we used shell elements for the tank whereas beam elements were used for the frame. Also in this case, both material and geometric nonlinearities were accounted for. All these models were considered

fixed to the soil and the soil-structure interaction was neglected. This assumption is partially justified because these structures present rigid pile foundations. Another aspect analyzed was the fluid-structure interaction. We analyzed the tanks under different conditions: empty, full with the sloshing effects and without them.
The sloshing effect of the tank is analyzed according to Eurocode 8 provision (CEN, 2006).
To obtain the structural response of a Multi Degree Of Freedom (MDOF) system through a pushover analysis, it is necessary to connect the response of this system to a SDOF system. This SDOF system is usually obtained directly by the pushover curve of the base shear versus top displacement. Such an operation requires that the structure presents one vibration mode with an effective mass not smaller than 60%. Otherwise, higher modes should be taken into account, for example by making use of a multi-modal techniques (Minghini et al., 2014).
In this contribution, a bilinear idealization of the SDOF system is used as required by the Italian Building Code. The main parameters for this idealization are the SDOF oscillator's mass m*, yield strength F_y^*, yield displacement δ_y* and backbone parameters. Force F* and displacement d* are related to MDOF system by modal participation factor Γ. Another important parameter is the period of SDOF system T*. The definition of F_y^* and δ_y* depends on the approximation of the pushover curve. In this case, the capacity curve is obtained with energetic equivalence. The parameters and the capacity spectra are constructed in acceleration–displacement response spectra (ADRS) format. All these parameters are used for performing the nonlinear time-history analysis (NLTHA) over one single degree of freedom (SDOF) system (§ 4.2).

Table 2. Results of the linear dynamic analysis for empty water tanks.

Site location	Period, T_n (s)	Effective mass ratio (%)
Ariano nel Polesine - Rivà	1.40	79.62
Castelnovo Bariano - Centrale	1.98	76.03
Corbola - Centrale	1.71	62.07
Occhiobello – S. M. Maddalena	0.77	64.13
Taglio di Po - Polesinello	1.73	75.13

Table 3. Results of the linear dynamic analysis for full water tanks.

Site location	Period, T_n (s)	Effective mass ratio (%)
Ariano nel Polesine - Rivà	1.89	86.72
Castelnovo Bariano - Centrale	2.91	85.36
Corbola - Centrale	2.53	70.34
Occhiobello – S. M. Maddalena	1.52	81.12
Taglio di Po - Polesinello	2.41	82.87

3 SEISMIC PERFORMANCE ASSESSMENT

For each of the water elevated tanks examined, a seismic vulnerability analysis is done in agreement with the Italian Building code.
For each water elevated tank outlined previously a survey of damageable structural elements was listed on information gathered in-situ surveys. Also, before the survey of the building, a collection of available information such as design documentation, architectural and structural drawings are collected in order to verify the geometry, position and dimensions of structure elements.

3.1 *Characterization of site hazard*

The first step in PBEE framework was to obtain the seismic hazard curve for all of the tanks considered in this study. For each location, with the probabilistic seismic hazard analysis (PSHA) and with Openquake it was possible to obtain the site hazard curve (Pagani et al., 2014). Figure 2 shows the five hazard curves for the sites considered.

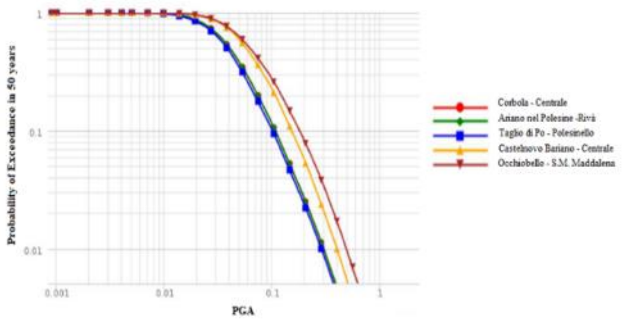

Figure 2. Seismic hazard curves for the five site of water tank with intensity measure as PGA.

3.1.1 *Intensity measure*

The intensity measure (IM) describes the level of ground shaking and quantifies the seismic hazard. A desirable IM should be an efficient and sufficient predictor. According to the Italian Building code, peak ground acceleration (PGA) is the key parameter for seismic design. However, already in 1952, Housner pointed out that peak ground acceleration “is not a good measure of the intensity of shaking as regards effects on structures” (Housner, 1965).

3.2 *Characterization of structural response*

The nonlinear dynamic analyses of the SDOF systems idealizing the tanks were performed by the open source software OpenSees (McKenna et al., 2000). To perform the analysis, we need the capacity curve, idealized by five relevant points of S_d-S_a coordinates, the value of damping (in this case we assumed 5%), the period of the structure and the level of degradation in the cyclic rule. The five points of the capacity curve are needed to represent the material hysteretic behavior with the Piching4 Opensees material command. The capacity curve is an input that can describe the envelope response of the hysteretic behavior.
It was then necessary to specify a damage model such as spectral displacement, capacity curve-based and drift based damage criterion. In this contribution, the capacity curve-based damage criterion is used. With this criterion is possible to estimate a set of damage thresholds. For more details see (Erberik, 2008).
Four damage levels are identified as limit states: DS1 slight damage (SLO), DS2 moderate damage (SLD), DS3 extensive damage (SLV), DS4 complete damage (SLC). These limit states are described in Italian national code. In Table 4 the damage states are exposed with the relative displacement.

Table 4. Damage states and relative displacements

Damage state	$S_{d,i}$
DS1 slight	$0.7d_y$
DS2 moderate	$1.5d_y$
DS3 extensive	$0.5(d_y+d_u)$
DS4 complete	d_u

These displacements are directly identified on the capacity curve as a function of the yielding d_y and the ultimate d_u displacement from the idealized elasto-perfectly plastic capacity curve obtained from non-linear static analysis.
Figures 3-4 show the influence of the filling conditions on the capacity curves of two different water elevated tanks. The different conditions are referred to as empty, sloshing and full water tank.

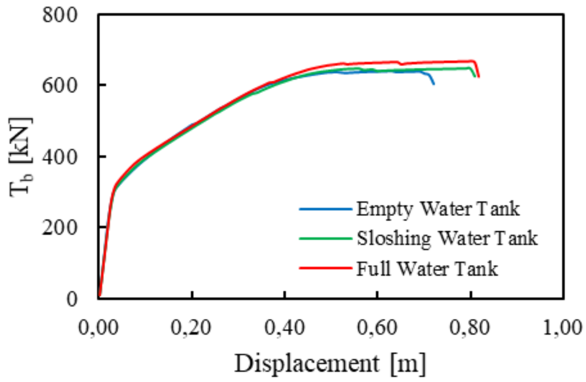

Figure 3. Capacity curves for different filling conditions for a water elevated tank (e.g. Corbola Centrale).

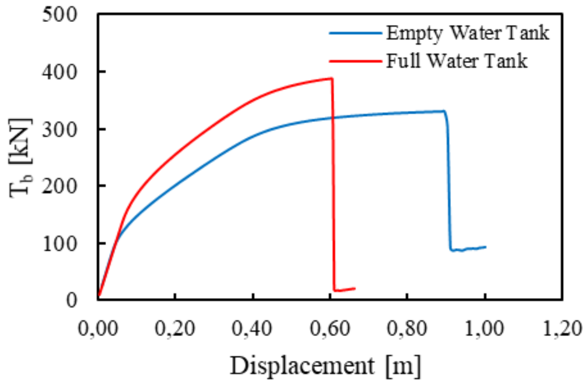

Figure 4. Capacity curves for different filling conditions for a water elevated tank (e.g. Ariano nel Polesine - Rivà).

Defining an Engineering Demand Parameter (EDP) is necessary to identify the damage state obtained from each given analysis. For the structures investigated in this research, the horizontal displacement on the top of SDOF system is used as EDP.

3.3 *Multiple Stripe Analysis*

In order to estimate fragility parameters from structural analysis, a multiple stripe analysis (MSA) is used. With this method, time history analysis are performed for a specific set of IM levels, each of which corresponds to a unique ground motions set (Jalayer, 2003).
This method is used because it produces more efficient fragility estimates than incremental dynamic analysis for a given number of structural analyses (Baker, 2015).

3.4 *Fragility function*

A fragility function is defined as a conditional probability of failure. This type of curves can be built by relying on a lognormal cumulative distribution function:

$$P\left(C \middle| IM = im\right) = 1 - \Phi\left(\frac{\ln(im) - \mu_{\ln(im)}}{\beta}\right) \tag{1}$$

where $\mu_{ln(im)}$ and β represent the central tendency and the dispersion parameters of the cumulative standard normal distribution Φ.

Parameters $\mu_{ln(im)}$ and β must be calibrated from structural analyses results. There are different statistical methods for estimating parameters from a dataset: for instance, the method of moments and the maximum likelihood method.

In this contribution, the maximum likelihood method is used. All the methods try to have some desirable proprieties such as being unbiased (the estimator do not systematically overestimate or underestimate the true parameter's value), efficient (the estimators have small variance) and consistent (as the number of data goes to the infinity, the estimator converges to the true parameter).

In Figure 5, an example of fragility curves for different damage states is exposed. For these tanks, we use five damage states. It is possible to see that the tank is vulnerable for low accelerations and it start slight and moderate damage for very low accelerations.

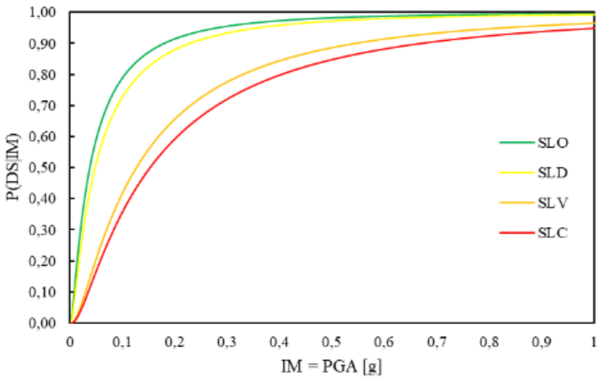

Figure 5. Fragility curves for different damage states for the full water tank of Corbola – Centrale.

Figure 6 shows a comparison between the different fragility curves of the same water elevated tank but with the different condition (empty and full). In this figure, the collapse is considered as damage state. It is possible to see how the full water tank is more vulnerable than the empty water tank. In fact, if we take the same IM, the full water tank has a higher probability of collapsing than the empty tank.

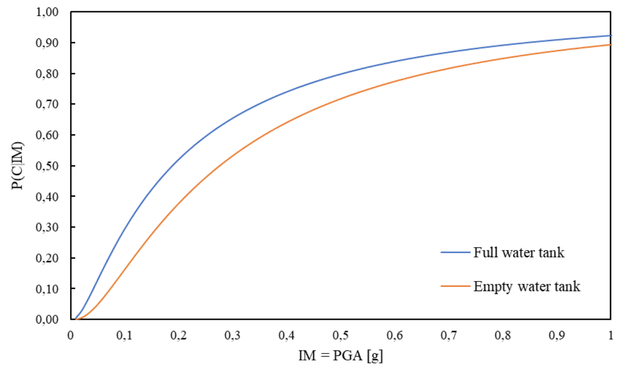

Figure 6. Fragility curves for collapse damage state for a water tank with different tank condition (e.g. Corbola – Centrale)

3.5 Collapse Estimation

For evaluating the level of safety of a structure against collapse the mean annual frequency of collapse (λ_C) is an important and efficient metric for the seismic risk (Eads et al., 2013; Miranda et al., 2017a). In this procedure, two elements are needed to calculate λ_C: the seismic hazard curve, which gives information on site hazard and the collapse fragility curve, which describes the probability of collapse conditioned to ground motion intensity. Using numerical integration, the assessment of the probability of collapse can be computed as follow:

$$\lambda_c = \sum_{i=1}^{\infty} P\left(C \mid im_i\right) \cdot \left| \frac{d\lambda_{IM}\left(im_i\right)}{d\left(im\right)} \right| d\left(im\right) \tag{2}$$

where *P(C|im)* will collapse when subjected to an earthquake with ground motion intensity level *im*, $d\lambda_{IM}(im)/d(im)$ is the slope of the seismic hazard curve at the site

This parameter and its deaggregation process can give important information for identifying the contribution of different levels of ground motion intensity to total collapse risk.

3.6 Loss estimation

Loss estimation represents an important step for the risk assessment of the structure. For this estimation, a key decision variable is the expected annual loss (EAL). This value is computed as the sum of expected losses at a given level of ground motion intensity and then integrating over the mean annual frequency of exceeding of all possible intensities. The mean annual frequencies of exceeding of all possible intensities are obtained by the seismic hazard curve.

The general formulation of EAL can be computed as follows:

$$EAL = \int E\left[L_T \middle| IM\right] \left|\frac{d\lambda}{dIM}\right| dIM \qquad (3)$$

where $E\left[L_T \middle| IM\right]$ represents the total expected direct losses for giving IM for the determinate site described as follow

$$E\left[L_T|IM\right] = E\left[L_T|NC, IM\right]\left(1 - P\left[C|IM\right]\right) + P\left[C|IM\right] \cdot \mathrm{Rep}C \qquad (4)$$

where *P[C|IM]* represents the probability of collapse for a given level of IM, *NC* correspond to no collapse cases and *RepC* represents the replacement cost of building. In absence of the specific *RecC* for these types of structures in first approximation, we employ the replacement cost (%RC) provided by the Italian seismic risk classification guidelines.

4 RESULTS

4.1 Collapse assessment

Applying the procedure described in Section 3.5 to the five tanks we find a low annual probability of collapse (Table 5). The results are in agreement with the low seismicity of the area, despite the high degradation of structures.

Table 5. Mean annual frequency of collapse (Miranda et al., 2017b).

Site location	λ
Ariano nel Polesine - Rivà	0.17 %
Castelnovo Bariano - Centrale	0.32 %
Corbola - Centrale	0.25 %
Occhiobello – S. M. Maddalena	0.17 %
Taglio di Po - Polesinello	0.23 %

4.2 Expected annual loss assessment

In 2017 in Italy the High Council of Public Works (Ministry Decree n.58 28/02/2017) approved the *"Guidelines for seismic risk classification of the constructions"* that define the general principles to classify the seismic risk of buildings. According to the Italian guidelines for seismic risk classification, different limit states have been defined: Operational (OLS) and Damage Limitation (DLLS) at Serviceability Limit State (SLS), Life Safety (LSLS) and Collapse (CLS) at Ultimate Limit States (ULS). Furthermore, there are two conventional limit states: Initial Damage Limit State (IDLS) and Reconstruction Limit States (RLS). For each limit state, a repair cost is associated. The repair costs (%RC) used in these cases are calibrated on the recent reconstruction process of L'Aquila (Del Vecchio et al., 2018).

In Figure 7, a comparison between the conventional approach described in the Italian guidelines (Cosenza et al., 2018) with the PBEE approach is shown. As it can be readily seen, the conventional approach provides results similar to the PBEE approach for the frame supported Corbola Centrale water elevated tank.

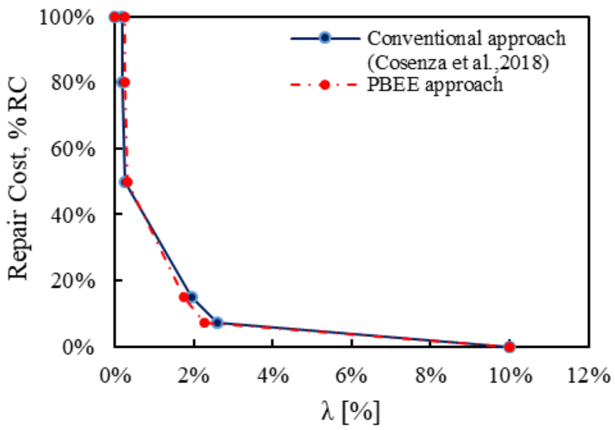

Figure 7. EAL curves for a water elevated tank (e.g. Corbola Centrale – water elevated tank)

This classification is described in terms of EAL (namely *Perdita Annuale Media attesa*, PAM, in the Italian guideline, Ministry Decree n. 58 28/02/2017 (MIT, 2017)), used to provide an overall rating on a letter scale from A+ to G, similar to energy classification for buildings (Cosenza et al., 2018).

In Table 6 we show a comparison between the class EAL of Italian seismic classification with the EAL calculated with the PBEE-based procedure.

Table 6. EAL values computed using the proposed method and EAL classes from Italian guidelines.

Site location	EAL	$\text{CLASS}_{\text{EAL}}$
Ariano nel Polesine - Rivà	1.37 %	B
Castelnovo Bariano - Centrale	1.12 %	B
Corbola - Centrale	1.17 %	B
Occhiobello – S. M. Maddalena	1.06 %	B
Taglio di Po - Polesinello	1.08 %	B

These results show how these structures, built in very low seismic areas, are little vulnerable to seismic hazard. The values of EAL reported in Table 6 are obtained without considering structural degradation. Future research will examine the effect of material degradation on the seismic class classification.

5 SUMMARY AND CONCLUSIONS

In this paper, we present the seismic assessment of five existing water elevated tanks located in the province of Rovigo. These case studies represent a small part of the Italian stock of water elevated tanks. The results obtained from the analyses show how this simplified method is consistent with the method proposed by the guidelines.

Future studies will analyze the probability of collapse and evaluate the EAL for other water tank typologies. The repair costs for this category of structures will also be estimated. Of considerable interest will be also the evaluation of the impact of the damage of these structures on the whole water network. The risk analysis of the water network is an important challenge for government agencies, such as civil protection or managing authority, and decision makers.

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