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Structural reliability of elevated water reservoirs under wind loading

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Abstract

In the field of civil engineering, concrete water storage tanks are considered as hydraulic structures occupying a special place among other structures. The location of these tanks is based on hydraulic considerations related to the desired service pressure for subscribers whose solution is obtained by a compromise with the topographical constraints. In southern Algeria and the highlands; in order to ensure adequate pressure in the drinking water supply networks, the tanks are then elevated to high heights, which puts them under significant stress during windstorms and sandstorms conditions that are frequent in the South Algerian. As the severe weather conditions due to high winds are common, the elevated water storage tanks are designed to withstand to winds speeds included between 25 and 31 m/s according to the Algerian Wind Code. Given the uncertain and randomness of this phenomenon, the classical deterministic calculations of the engineer become limited since they do not integrate the notion of failure probability of the structure. In this study, a probabilistic approach based on Monte Carlo simulations is used to analyze the reliability of elevated water tanks submitted to hazard storm of wind loading. The limit state functions are related to the ultimate and serviceability limit states of the concrete elevated tank under wind analysis. This reliability approach, takes into account mainly two parameters which are the wind speed and the concrete compressive strength considered as random variable. Fragility curves depending on wind zones are obtained, where they demonstrate the dominant failure modes that can cause the structural failure.

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Keywords: Reliability, wind speed, hydraulic load, failure, concrete elevated tanks;

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1. Introduction

Until the 19th century, dimensioning codes of structures were based on empiricism and experience. The adopted principle of safety was that said admissible stresses, which consists to ensure that the maximum stress σ_{max} calculated in a given section under a combination of unfavorable actions, remains below a so-called admissible constraint σ_{adm}. The value of the admissible stress is determined by the ratio of the ruin stress σ_r of the material on a safety factor noted “k” fixed in a conventional manner:

$$\sigma \leq \sigma_{adm} = \frac{\sigma_r}{k} \tag{1}$$

This principle has the advantage of being easy to implement but it remains insufficient. Indeed, it does not allow taking into account the dispersion of each of the parameters involved in the calculation since the same coefficient is assigned to them, which can lead to over-dimensioning. On the other hand, constraint verification is not the only criterion involved in assessing the safety of a construction.

In Algeria, the deterministic methods of structures design under wind action have evolved over time. Until 1944, the French official regulations, which were applied in Algeria, fixed a uniform pressure of the wind on the constructions whatever their form, their height or their situation. This way of doing, reflected improperly the real effects of the wind on buildings and structures, and led to insufficient or excessive safety, depending on the case. At the request of the Ministry of Reconstruction, a commission was created to draw up a wind code taking into account the scientific and statistical data known at that time. Unfortunately, for the buildings, these data were, with rare exceptions, limited to foreign aerodynamic tests, and for the wind speeds to the experience of the technicians of the National Meteorology, because of the absence of archives destroyed during the Second World War.

However, the wind and Snow code (NV 46) was drawn from this incomplete information, in order to quickly put in the hands of the builders a document allowing them to face the task of the reconstruction, without waste of materials and with safety. From that moment, it was expected that these rules should be reviewed after a number of years. To prepare their review an investigation was launched to users in 1956. This survey signaled no serious deficiencies and showed that for ten years the Standard Rules had never resulted in real difficulties while leading to significant savings. In 1965, a new wind and Snow code was born, the RNV 65 which will be modified in 1984, brings some necessary improvements. This code has been oriented towards certain guiding ideas, such as: Facilitate the use of the rules; extend the scope of use of the rules to other structures; take into account the evolution of the type and mode of construction; take into account the evolution of calculation methods and determination of safety; take into account the evolution of ideas on the determination of wind speed. In 1991, after a decade of research, the Eurocode (EN 1991-1-4) is launched, it indicates how to determine the actions of natural wind for the structural calculation of buildings and civil engineering structures, for each of the zones affected by these actions. The wind action is represented by a simplified set of pressures or forces whose effects are equivalent to the extreme effects of the turbulent wind. This action depends on the size, shape, and dynamic properties of the structure. Since 1999, Algeria has adopted a wind and Snow code (RNV99), inspired by the Eurocode rules and the coherence with the verification methods to the limit states. The document is based on a probabilistic approach where the normal and extreme actions of the old rules are replaced by the unique concept of characteristic action defined by reference to a territorial zoning (snow, wind and sand) linked to local climatic specificities.

2. Basics of reliability analysis

In the consideration of the reliability of a structure, the determination of the probability of failure is the central issue. The limit between failure and non-failure is defined as a limit state and the reliability is the probability that this limit state is not exceeded. The limit states are interpreted through the so-called limit state functions whose general form is:

$$G=R-S \tag{2}$$

In which R is the strength or more general the resistance of failure and S is the load or that which is conductive to failure. The basic principle of structural design is that the resistance needs to be higher than the load or in other words that the limit state function is larger than zero (G>0). The main objective of the design is to ensure that the performance criterion is valid throughout the lifetime of a structure. Therefore, a probability of satisfying the preceding criterion is estimated and expresses the reliability of the structure. The probability of failure is:

$$P_f = P(G \leq 0) = P(S \geq R) \tag{3}$$

3. Basics of reliability analysis

The reliability analysis of a structure requires the definition of the different failure modes that are relevant to the corresponding structural components. In this work, the possible failure mechanisms expressed in the limit state function are identified. Based on these limit state functions, the reliability of the system was evaluated. These limit state functions focus on those exceedance implies failure of the support system of the tank.

As far as the support system of the tank is concerned, the failure mechanism to be investigated in this paper is the cracks formation in the concrete, by the compression constraints and tensile stress.

Taking into account the above, the first limit state function of compression can be formed as the difference between the maximum developed stress σ_c and the yield stress σ_c^{adm}, as follow:

$$G_1 = \sigma_c - \sigma_c^{adm} \tag{4}$$

The admissible stress is given by the following relation:

$$\sigma_c^{adm} = 0{,}60.fc_{28} \tag{5}$$

The second limit state function of traction can be formed as the difference between the maximum developed stress σ_t and the admissible tensile stress of the concrete σ_t^{adm}, which is equal to zero according to the Fascicule 74, as follows:

$$G_2 = \sigma_t - \sigma_t^{adm} \tag{6}$$

4. Identification of the random variables

In this study, we will focus on two random variables, the wind speed and the characteristic compressive strength at 28 days. Statistical analyzes on series of measurements carried out on these variables were conducted to define distribution laws that best fit them, in order to generate random draws (Dress, 2007). A Chi-square test is thus performed, in order to compare an observed distribution of a random variable with different known theoretical distribution laws (Normal, log-normal, Gumbel ...). The principle of the adequacy test of Chi-2 consists of comparing observed numbers and theoretical numbers (or calculated) (Baroth et al. 2008). Thus, we define the discriminant function χ^2 which is written as follows:

$$\chi^2 = \sum_{i=1}^{k} \frac{(O_i - E_i)^2}{E_i} \tag{7}$$

Where O_i and E_i designate respectively observed numbers and theoretical numbers. To confirm or refute this hypothesis, the calculated value χ^2 is compared to the value read from the table Chi-2.

The ECE Company in Tizi Ouzou specialized in the construction of hydraulic structures, has given us a series of 121 measurements of characteristic compressive strength of concrete made on its various tanks projects in Algeria. Results of the adjustment test; given in Table 1, show that all distribution laws are accepted to model the distribution. For the rest of the study, the random variable will be generated using the normal distribution, whose graph of the probability density function is shown in Figure 1.

Table 1. Results of chi-2 test of the characteristic compressive strength of concrete

Distribution law	χ^2	$\chi^2(\alpha, \nu)$	**Observations**
Normal law	4,73	9,49	accepted
Log-normal law	1,55		accepted
Gumbel law	4,84		accepted

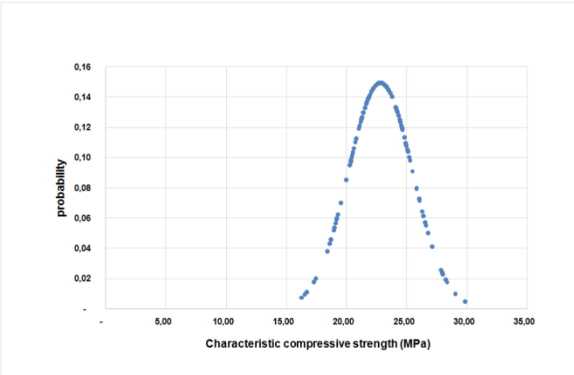

Fig. 1. Density probability curve of the Characteristic compressive strength

African Geosystem Company in Algiers specialized in the design of hydraulic structures, has given us a series of 180 measurements of wind speed, recorded at the meteorological station of Djelfa (Southern Algeria) between 1972 and 1986. Results of the adjustment test; given in Table 2, show that the normal law and the log-normal law can be adapted to model the distribution of the wind speed. For the rest of the study, the random variable will be generated using the normal distribution, whose graph of the probability density function is shown in Figure 2.

Table 2. Results of Khi 2 test of the wind speed

Distribution law	χ^2	$\chi^2(\alpha, \nu)$	**Observations**
Normale law	5,67	7,81	accepted
Log-normale law	6,54		accepted
Gumbel law	13,91		accepted

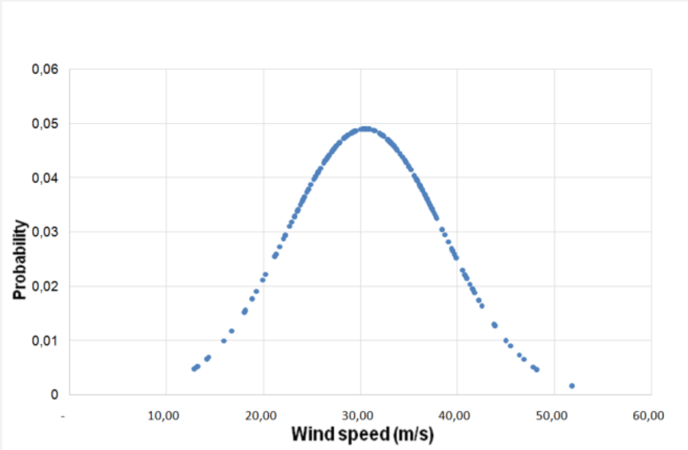

Fig. 2. Density probability curve of wind speed

5. Failure probability assessment of an elevated tank

The analytical assessment of the failure probability of a storage tank is very difficult if not impossible, particularly for failure modes identified in our study. Several numerical approaches based on numerical approximations and integrations are suggested in the literature (Lemaire, 2008), such as the Monte Carlo simulation method, approximations methods of FORM and SORM and the response surface method. In this study, failure probability assessment P_f is conducted with the classical Monte Carlo method, for its simplicity and accuracy of its results. The principle of this method is based on the generation of a large number of random draws which we will note N_{Sim}. The software Matlab® is used for the draws generation. Thus, a ruin indicator I_G is used to define the state of failure system for a given function of state G; such as:

$$I_{G\leq 0} = \begin{cases} 1 \text{ si } G \leq 0 \\ 0 \text{ si } G > 0 \end{cases} \tag{8}$$

The failure probability P_f is given, for each ruin mode, by the following relation (Mébarki et al, 2003).

$$P_f = \frac{\sum_{1}^{Nsim} I_{G \leq 0}}{N_{sim}} \tag{9}$$

To ensure accuracy of results of the failure probability calculation P_f, convergence tests were performed for different limit state functions as shown in Figure 3. Results show that the convergence and the stability of calculations of P_f value are obtained from a number of simulations equal to 2.10^4. Ultimately, the number 3.10^4 will be retained to perform Monte Carlo draws for the rest of the study (Aoues, 2008).

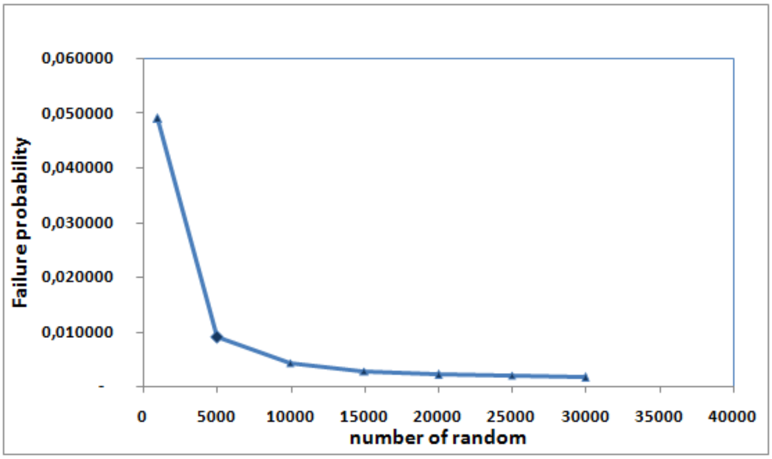

Fig. 3. Convergence and stability of ruin probabilities P_f according to the number of simulations.

6. Results and interpretations

The figure below shows that the failure probability in the limit state of compression in the concrete of the most loaded columns, according to the different wind zones, is null. This proves that there is no risk of concrete failure by compression. The failure probability at the limit state of tensile stress as a function of the different wind zones is illustrated in Figure 4. The results show that the failure probability increase with the wind speed. For the wind zone I, this value is null, it is close to the limit value allowed for civil engineering structures ($10^{-8}< P_f <10^{-3}$) in the wind zone II, and the structure is failing in the wind zone III.

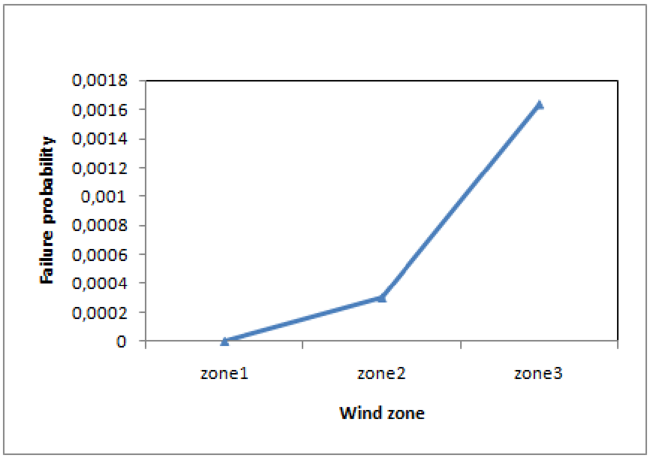

Fig. 4. Failure probability of tensile stress as function of wind zone

6.1. Identification of the most influential variable

Here, we were interested in determining the most influential variable between wind speed and the characteristic strength of concrete. For this, we conducted three different types of analysis of the studied structure, as summarized in the following table, where the corresponding probabilities were evaluated. The results are illustrated in the Fig. 5.

We notice that the two curves corresponding to the analysis type 2 and type 3 are confounded and give failure probability values greater than the values given by the curve type 1. In the analysis type 1, the wind speed is taken deterministic, which underestimate the failure probability, so we can conclude that the most influential variable is the wind speed.

Table 3. The variation of failure probability as a function of the wind zone and the random variable.

Analysis	Variables	Variable type	Wind zone	Failure probability
Type 1	Characteristic compressive strength	Random	Zone 1	0
	Wind speed	Deterministic	Zone 2	0
	Other variables	Deterministic	Zone 3	0,000566667
Type 2	Characteristic compressive strength	Deterministic	Zone 1	0
	Wind speed	Random	Zone 2	0,0003
	Other variables	Deterministic	Zone 3	0,001633333
Type 3	Characteristic compressive strength	Random	Zone 1	0
	Wind speed	Random	Zone 2	0,0003
	Other variables	Deterministic	Zone 3	0,001633333

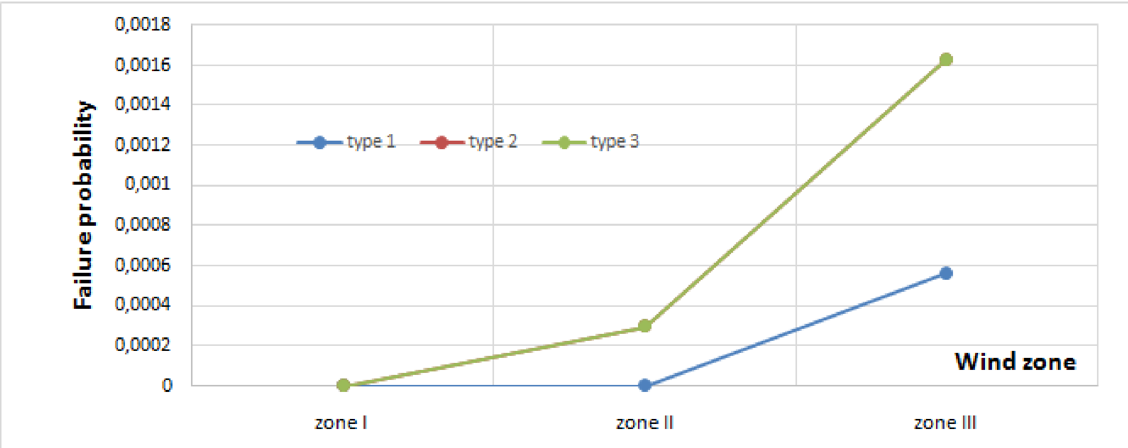

Fig. 5. Failure probability as a function of the wind zone and the type of variable

6.2. Influence of topographical site

The topographical site is an important parameter introduced in the wind analysis of an elevated tank. According to Algerian wind code, a topographical coefficient (C_t) is affected to each site. The evolution of the failure probability at the limit state of tensile is given in the Figure 6 as a function of the topographical site and for each wind zone. In the wind zone III, we notice that the failure probability exceed the admissible value of the probability in all the topographical sites. This leads to the ruin of the elevated tank in this zone. In the wind zone II, the structure enter into failure from a value of topographical site coefficient equal to (C_t=1.3), corresponding to the site around valleys and rivers with funnel effect. In the wind zone I, the failure of the structure is reached only for the mountainous sites.

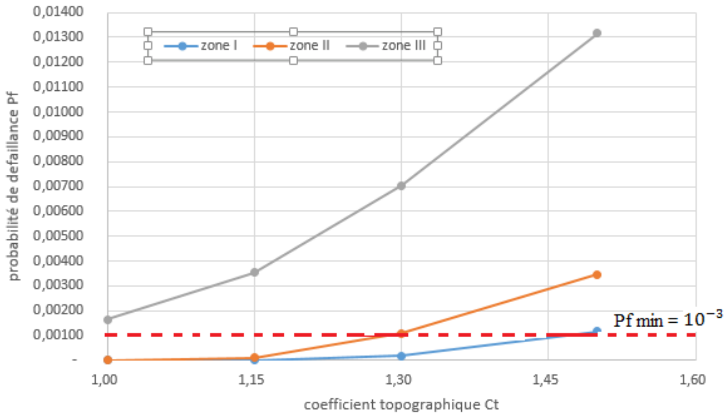

Fig. 6. Failure probability as a function of topographical site.

7. Conclusion

The deterministic approach to the stability analysis of an elevated reservoir located in El-Menea city (Ghardaia, Algeria) under the wind action, using Algerian wind code (RNV 99) showed that the stability of this structure is ensured to the considered failure modes. The reliability analysis taking into account the randomness of the wind speed and the characteristic strength of concrete confirmed the stability of the studied elevated reservoir and highlighted that the most influential random variable on the evolution of the failure probability is the wind speed. The topography of the site on which the reservoir is located is an important factor in the calculation of its stability under the wind action; we have also been able to demonstrate that for a mountainous site, the failure probability exceeds the admissible value for all the wind zones.

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