



Tsunami fragility for anchored floating-roof atmospheric tanks

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ABSTRACT

One of the issues for quantitative risk assessment in the process industry, is incorporating so-called NaTech risk, that is considering the potential consequences of natural hazards, whose impact on facilities can cause accidents such as explosions, fires, or toxic substance release. In this context, this paper explores the behaviour of anchored atmospheric storage tanks under tsunami, with the goal of quantifying their vulnerability via parametric fragility functions. Three archetype tanks with different aspect ratios were considered, while tsunami vulnerability was evaluated for different filling levels. The effects of tsunami-induced pressure on the tanks were calculated numerically following an equivalent-static approach, based on nonlinear analyses of finite element models. Each tank was subjected to various combinations of tsunami inundation levels and increasing flow velocities, obtaining structural response of each tank's steel shell and anchorage system. Parametric tsunami fragility curves were obtained, also with the aid of a site-specific conditional tsunami hazard model, surrogating vector-valued hazard analysis. It was found that the predominant failure mode depends on the aspect ratio and filling level, with emptier tanks being more vulnerable overall. The fragilities were then used to quantify the risk associated with loss of content due to structural failure, and the results were also compared those from existing models.

1. Introduction

One of the open issues in quantitative risk assessment (QRA; [Fabricio et al., 2005](#)) in the process industry are cases of natural events, such as landslides, extreme winds, tsunamis, floods or earthquakes, triggering extensive industrial accidents ([Campedel, 2008](#)). The main feature of such accidents is that damage sustained by the industrial equipment (e.g., structural damage or rupture of connecting pipelines) can cause the release of hazardous substances into the environment. This loss of containment, in turn, may cause a series of cascading effects such as chain fires, blasts, and dispersion of toxic clouds, with potential loss of human life, which collectively go under the name of so-called NaTech events (e.g., [Krausmann et al., 2011](#)).

Analytical procedures for NaTech probabilistic QRA require a characterization of the natural hazard at the site of interest, vulnerability models for the industrial components, and consequence models for industrial accidents. In this context, the focus of this work is on the vulnerability of anchored floating-roof atmospheric storage tanks impacted by tsunamis. The relevance of these issues resides with industrial facilities, often located in coastal zones due to access to

maritime transportation, and the vulnerability of atmospheric storage tanks against content release following damage to the structure and connections induced from external pressure and drag forces during flood-like events (e.g., [Cozzani et al., 2010](#); [Krausman, 2010](#)).

The final objective of the numerical analysis in this study, was to derive fragility functions for NaTech risk analysis, that is vulnerability models providing the conditional probability of loss-of-containment inducing structural damage, given a specific level of tsunami intensity. In this respect, the present study comes in continuation of past works that studied the seismic vulnerability of such tanks ([Iervolino et al., 2004](#); [Salzano et al., 2003](#)), and aims to cover additional failure mechanisms, levels of tank contents and sources of uncertainty with respect to previous similar works ([Basco and Salzano, 2016](#)). More specifically, finite element models (FEM) of various cylindrical storage tanks were developed, following the state-of-the art in seismic modelling of tanks, ([Bakalis et al., 2017a; 2017b](#)), and were subjected to an equivalent-static representation of tsunami-induced actions, which is an approximation that may impose a limitation on this work. These models considered the response of the anchorage system to tsunami-induced drag and buoyant forces, stresses due to potential loss of contact of the base plate with the

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foundation, and stability issues of the shell walls due to external pressure from tsunami inundation.

The remainder of this article is structured so that a presentation of the chosen methodology comes first, replete with modelling assumptions for the tsunami-induced loads on the tanks, structural behaviour of the latter and the failure modes considered. Additionally, the geometry and some detailing information of the case-study tanks are reported. Subsequently, the numerical models of these structures are described, with a focus on sources of non-linear behaviour built into said models. Finally, the results of the analyses and the procedure employed to develop parametric fragility functions from said results are presented. The paper concludes with a discussion of these results, a brief illustrative application that allows comparison with the few existing models from the literature, and some final remarks.

2. Methodology

Three archetypal floating-roof atmospheric steel storage tanks, together with a simplified representation of tsunami-induced loads on these structures, were considered. They have been designed for the purposes of this investigation according to European building standards or Eurocodes (CEN, 2017; CEN, 2005; CEN, 2017; CEN, 2006), with a focus on dimensioning the thickness of the cylindrical shell, the width and thickness of the annular foundation plate and the anchors. Subsequently, structural analyses of a FEM were used to determine the conditional probability that at least one of three possible failure modes can be activated for given tsunami loading parameters and quantity of liquid content in the tank. The three failure modes considered, illustrated in Fig. 1, are.

- buckling of the cylindrical shell wall caused by the external pressure;
- shear failure of the anchors due to drag forces;
- axial tension failure of the anchors due to uplift forces from buoyancy.

It was assumed that activation of any of these failure modes will likely cause leakage due to rupture of connections with piping systems, so that the occurrence of at least one is tantamount to reaching a loss-of-content damage state. The uncertainties in structural properties considered in the fragility derivation include the axial and shear resistance of the anchors, and the level of geometric fabrication imperfections of the tank shell that drive stability. Uncertainty in the tsunami-induced actions is also taken into account, as will be elaborated in what follows.

2.1. Tsunami actions on the tanks

A typical approach for representing tsunami effects on structures, in terms of forces, consists in separating these effects into a hydrostatic and a hydrodynamic component. Literature suggests that dynamic fluid-structure interaction effects can often be neglected, and both components can be represented by equivalent quasi-static loads (Araki et al., 2017; Foster et al., 2017), which is also the approach adopted by a number of building codes/guidelines that consider tsunami-induced actions on structures (ASCE/SEI 7-22, 2022; FEMA, 2008) and by engineering applications considering structural response to tsunamis (Attary et al., 2017; Baiguera et al., 2019; Del Zoppo et al., 2023; Macabuag et al., 2014; Petrone et al., 2017) and flood risk of atmospheric tanks (Landucci et al., 2012).

This quasi-static force approach was also adopted in the presented study, as shown in Fig. 1 (a), where the tsunami and liquid content pressures considered are represented on a vertical section of a generic cylindrical tank with diameter D , and height H . More specifically, the hydrostatic component consists of an external lateral pressure, $P_{ws}(z)$, and a corresponding buoyant pressure $S = P_{ws}(z = 0)$:

$$P_{ws}(z) = \rho_w \cdot g \cdot (h_w - z), z \in [0, h_w], \quad (1)$$

where ρ_w is the mass density of water, g is the gravity acceleration, z is the distance from the base plate and h_w is the depth of tsunami water level with respect to the grade plane at the structure. In this case, a value

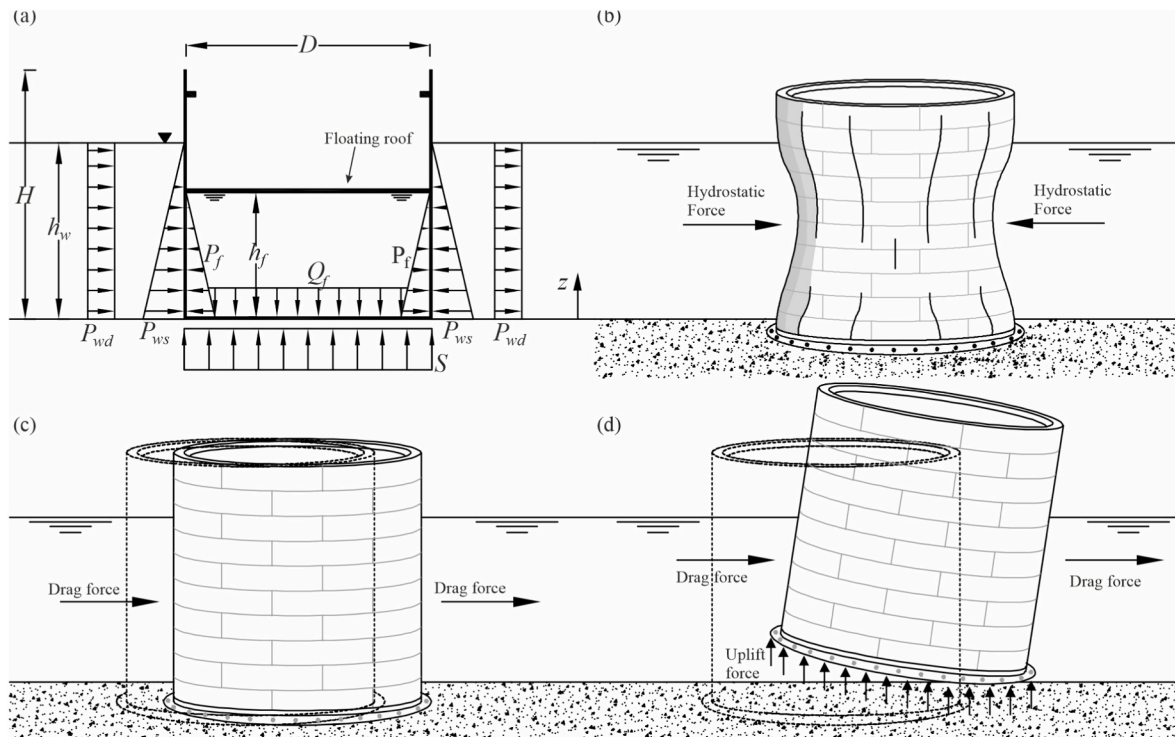


Fig. 1. – (a) equivalent static tsunami load and schematic representation of possible failure modes considered: (b) shell buckling, (c) the shear failure of the anchors, and (d) the tensile rupture of the anchors.

of $\rho_w = 1200 \text{ kg/m}^3$ is assumed, corresponding to seawater with sedimentary content (FEMA, 2008). The resulting pressure distribution is linear along the height and axisymmetric around the circumference of the storage tank.

Additionally, the liquid content acts on the tank wall with an internal hydrostatic pressure, P_f , given by:

$$P_f(z) = \rho_f \cdot g \cdot (h_f - z), z \in [0, h_f], \quad (2)$$

where, $\rho_f = 810 \text{ kg/m}^3$ is the density of the contained liquid, considering an average value for flammable substances typical of the petrochemical industry (Council Directive 67/548/EEC, 1997; Cozzani et al., 2010), and h_f is the liquid content height. The vertical component, acting on the base plate, is denoted as $Q_f = P_f(z = 0)$.

In the context of the quasi-static representation of tsunami-induced forces adopted in this study, the horizontal resultant of hydrodynamic pressure acting on the tank, that is the drag force, F_{wd} , can be computed as:

$$F_{wd} = \frac{1}{2} \cdot D \cdot h_w \cdot C_d \cdot \rho_w \cdot v_w^2, \quad (3)$$

where C_d is the drag coefficient and v_w is flow velocity averaged over the cross section perpendicular to the flow. This simplified approach is mainly representative of the quasi-steady flow phase of tsunami inundation (Harish et al., 2021). However, during incidence of a tsunami wave with a vertical structure, the steady-state flow phase is preceded by a transient one, where flow depth is non-constant along the structure. During that phase, the hydrostatic component of external pressure is not yet axisymmetric and exhibits a horizontal resultant force that adds to total drag and overturning moment (Tsutsumi Y et al., 2023), which may partially persist into the steady-state phase at higher flow velocities. Nevertheless, drag forces during initial impact and transient flow phase, could be influenced by dynamic fluid-structure interaction effects and the presence of additional structures/obstacles around the tank, respectively, falling beyond the scope of the computational arsenal deployed for the present study. Therefore, a limitation of this work may be that the adopted simplified equivalent-static model of tsunami actions on the tanks is representative of quasi-steady flow conditions, characterized by low-enough Froude numbers, and assumes that potential failure mechanisms will be triggered during that phase, with harder to model transient effects upon impact being outside the scope of this approximation.

With this caveat in mind, a value of two was adopted for the drag coefficient in the subsequent analyses, which is larger than what is typically measured experimentally for cylindrical tanks (Araki et al., 2017) and is usually associated with rectangular obstacles (Foster et al., 2017; McGovern et al., 2023). The reasoning behind this choice is to possibly compensate for cases where the initial inundation phase drag forces could overshoot those from the subsequent quasi-steady state flow and govern the shear response of the anchorage system. On the other hand, some trial calculations showed that transient non-axisymmetric hydrostatic pressure is never critical for shell buckling due to meridional compression or uplift-related failure.

2.2. Failure criteria

The failure modes taken into account for the fragility analysis of the tanks were shell buckling under compressive circumferential stresses, shear failure of the anchorage system and uplift due to anchor failure under tension. It was considered that any of these situations would potentially lead to displacements that are too large for the pipe-to-tank connections to accommodate (O'Rourke and Liu, 1999; Paolacci et al., 2018; Vathi et al., 2017). For this reason, it was assumed that exceedance of each tank's capacity against any of these failure modes by tsunami-induced demand, would lead to large deformations and/or displacements, compromising one or more connections with consequent

loss of content.

2.2.1. Shell buckling

Regarding the failure mode governed by shell buckling, it was necessary to account for shell geometry imperfections during erection, which are mainly concentrated at the locations where curved steel plates are welded together to form the cylindrical shell. The present study adopted the Eurocode 3 part 1-6 (CEN, 2017) procedure for evaluating the buckling capacity of the steel cylindrical shell. This approach has been found suitable by results from stochastic FEM analyses in the literature (Hübner et al., 2006), and can be further justified by the fact that cylindrical tanks subjected to tsunami- or flood-induced external pressure are expected to reach instability under stresses in the elastic range, unlike the seismic case where buckling is often observed in locations where plastic strains have been developed (Bakalis and Karamanos, 2021). In this context, buckling capacity was expressed in terms of the critical circumferential stress leading to instability, $\sigma_{\theta,C}$, as:

$$\sigma_{\theta,C} = \alpha_{\theta} \cdot \sigma_{\theta,cr}, \quad (4)$$

where $\sigma_{\theta,cr}$ is the critical circumferential compressive buckling stress according to the procedure given in Eurocode 3-1-6 and α_{θ} accounts for geometric imperfections, considering the quality of fabrication. More specifically, Eurocode considers three classes of fabrication quality: excellent quality, with a corresponding α_{θ} value of 0.75, high quality with 0.65, and normal quality with 0.50.

Buckling of cylindrical shells has been widely studied in the literature and, although there are works dealing with uncertainty quantification for geometric imperfections of cylinders under meridional compression (Fina et al., 2023), such information is more scarce in studies that investigate circumferential compression, that is the case at hand (Bernier and Padgett, 2019; Combescure and Gusic, 2001). Herein, it was assumed that α_{θ} can be modelled as a Gaussian random variable (RV) with a mean of 0.65, which corresponds to high fabrication quality according to the Eurocode, and a standard deviation of 0.125. This assumption approximately corresponds to a 38 % probability of the α_{θ} coefficient falling between the Eurocode's normal and high fabrication quality classification and a 29 % probability of being between high and excellent fabrication quality.

2.2.2. Axial bearing capacity of the anchorage system

Regarding the shear and axial bearing capacity of the anchorage system, consisting of steel anchor bolts embedded in the concrete of the foundation raft, recourse was made to literature recommendations that link capacity to the nominal design strength, based on experimental data, and also provide a measure of its dispersion (Eligehausen et al., 2006). More specifically, for the axial bearing capacity two failure modes were considered: steel rupture of the anchor bolts and concrete cone breakout. Fig. 2 shows a representation of concrete cone breakout in panel (a) and tensile rupture of the steel in panel (b).

The tensile capacity of an anchor bolt, N_C , was taken as the lowest value between that associated to concrete cone breakout, $N_{u,c}$, and the tensile strength of the anchor bolt itself, $N_{u,s}$, according to:

$$\begin{cases} N_{u,c} = \alpha_c \cdot 16.8 \cdot L_B^{1.5} \cdot f_{ck}^{0.5} \\ N_{u,s} = 0.9 \cdot A_s \cdot f_{ub} \\ N_C = \min\{N_{u,c}, N_{u,s}\} \end{cases} \quad (5)$$

In the equation, $N_{u,c}$ is calculated according to a semi-empirical formulation based on experimental data for headed stud fastening systems (Zeitler and Wörner, 1995), where f_{ck} is the concrete characteristic compressive strength, L_B the embedment length, both expressed in units of N and mm, and α_c is a Normal RV with mean 0.99 and standard deviation 0.178. For $N_{u,s}$, the mean ultimate stress of the steel anchors, f_{ub} , was considered along with the nominal cross-sectional area of an anchor bolt, A_s (Eligehausen et al., 2006; CEN, 2005).

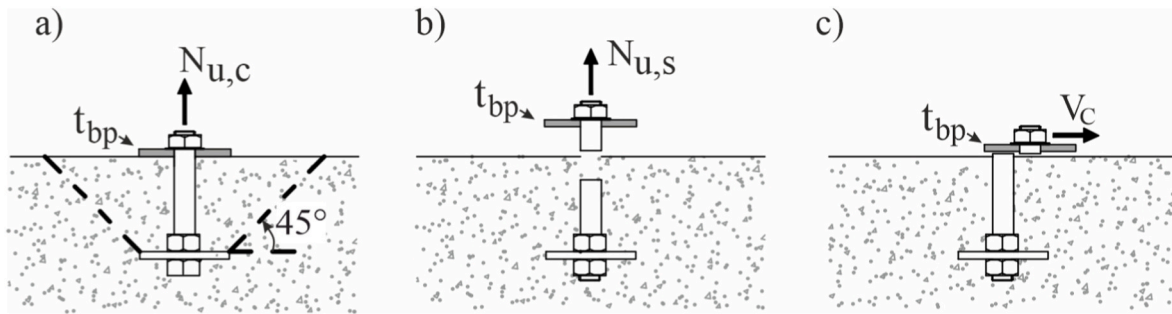


Fig. 2. Failure-inducing forces and stresses: (a) concrete cone breakout due to anchor bolt tension, (b) rupture-inducing tension force, (c) rupture-inducing shear force.

2.2.3. Shear capacity of the anchorage system

A semi-empirical model was also used for the anchors' shear capacity, V_C (Fuchs and Elgehausen, 1986):

$$V_C = \alpha_V \cdot A_s \cdot f_{ub}, \quad (6)$$

where α_V is another Normal RV with mean 0.68 and standard deviation of 0.238 (Rehm et al., 1988). Fig. 2 panel (c) shows a shear rupture of the anchor bolt.

It is also worth mentioning that past studies have shown that the base plate, under uplift forces, can yield prior to anchor failure (Mia and Kameshwar, 2023), which is confirmed also by the analyses conducted in this study. However, it is unlikely that this localized plasticity will lead to steel rupture and loss of content.

2.3. Design of archetype tanks

Floating-roof atmospheric storage tanks, whose main scope is the storage of highly flammable liquids such as gasoil, fuel, or combustible oils, are generally designed according to the provisions for steel thin-shell structures. The structure consists of a cylindrical steel shell, made of plates, welded to a base plate and featuring a floating plane roof that can adapt to the level of the liquid content. The base plate was designed thicker around close to the shell, to anticipate concentration of stresses due to potential localized uplift, resulting in an annular plate with greater thickness than the internal circular base plate. The annular plate extends beyond the outer face of the vertical shell to accommodate anchor bolt installation and mitigate local uplift effects. The primary objective of adopting a floating roof solution is to avoid gas accumulation under the roof, from the evaporation of the liquid contents, that may pose a blast hazard. Another objective is the prevention of gas leakage from the roof-shell junction, thanks to the roof's flexible sealing joint. These tanks can be closed off by a fixed roof or just rely on the floating roof whose motion beyond the top is impeded by a stiffening ring.

The three archetype storage tanks considered in the study were chosen to represent a different height-to-base-radius ratio, or *slenderness* (Table 1). Equivalent static wind and seismic actions for the design are calculated according to the Eurocodes and their Italian national annexes, (Consiglio Superiore dei Lavori Pubblici, 2018; CEN, 2004; CEN, 2006) for a reference coastal site in northern Sicily (Italy) as a plausible site for an oil refinery; design details are available in Vitale (2024). Thus,

seismic design of the tanks at the site of interest was based on a 0.16 g peak ground acceleration with 475-year return period on rock, while site subsoil conditions at the coastal site were assumed classified under category C, following the Eurocode 8 soil classifications (CEN, 2013). Given that the archetype tanks were presumed to operate under atmospheric pressure, their steel cylindrical shell wall, base plate and anchorage system are designed in accordance with the provisions of Eurocode 3 (CEN, 2017; CEN, 2005; CEN, 2017). For the preliminary design phase, the recommendations of the widely used *American Petroleum Institute standards*, API, (API 650, 2018) were also followed, regarding the width of the external annular base plate and minimum thicknesses of the cylindrical steel shell wall and base plates. The vertical shell was designed thicker than the annular base plate, so as to lead potential yielding under seismic conditions away from the wall base and towards the base plate (Calvi and Nascimbene, 2023).

Some schematics of a tank are shown Fig. 3, including horizontal and transverse sections along with the configuration near the anchor bolts. These sketches help to define the notation used herein for the geometry parameters, that is the thicknesses of the cylindrical shell near the base, t_w , the base plate, t_{bp} , and the annular plate, t_{ap} , the width of the annular plate, w_{ap} , the anchor bolt diameter, d_B , number, n_B , spacing, i_B , and embedment depth in the concrete foundation raft, L_B . A freeboard of 1 m was assumed for all tanks, that is $h_{fmax} = H - 1$ m. Although it is typical to reduce the wall thickness at the upper courses of large-diameter tanks, a constant t_w was assumed for the archetype designs; however, as the analyses (to follow) showed that the critical locations were at the lower courses, where the maximum thickness would be present regardless, the final results can be considered valid also for cases with tapered t_w .

The tanks were designed with a steel grade of S355 (CEN, 2022), while the anchor bolts with a bolt class of 8.8 (CEN, 2005), both according to Eurocode classifications. Table 1 reports the geometrical features of the tanks, which are the outcomes of nonlinear elastic analyses of FEM models, as described in next section, using the combinations of actions of so-called Eurocode 0 (CEN, 2023) and also indications about seismic action of Eurocode 8-1 (CEN, 2013). The FEM analysis enabled the optimization of the structure's plate thicknesses and determined the suitable number and diameter of anchors to resist seismic and wind design actions.

Table 1
Geometrical characteristics of archetype tanks.

Tank	H [m]	R [m]	H/R [-]	t_w [mm]	t_{bp} [mm]	t_{ap} [mm]	w_{ap} [m]	d_B [mm]	n_B [-]	i_B [mm]	L_B [mm]
Slender	11	5	2.20	20	8	10	1	27	12	2640	216
Intermediate	13	12	1.08	25	15	20	1.8	30	45	1750	240
Squat	15	28	0.54	30	20	25	2	39	120	1470	312

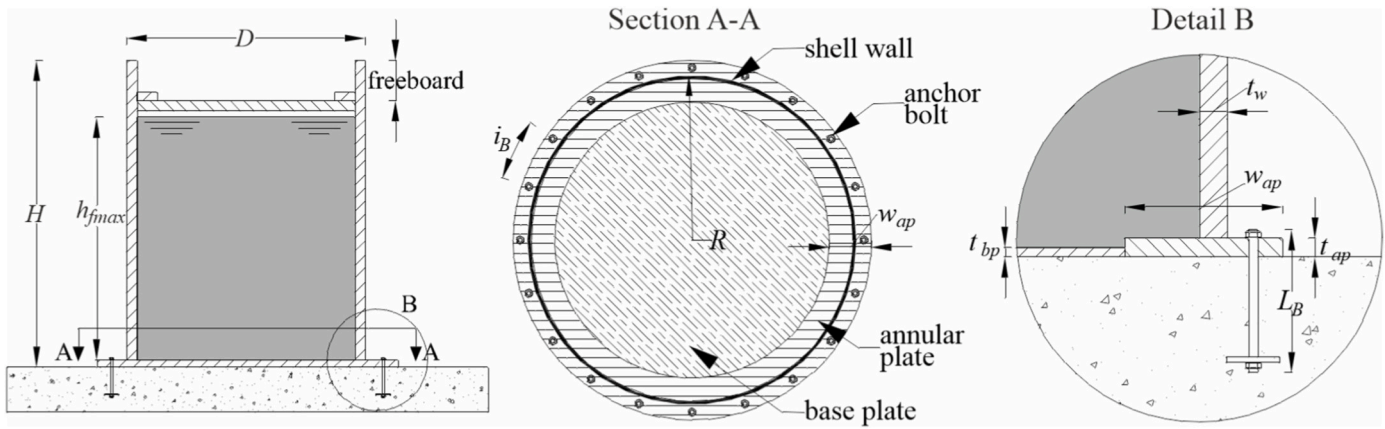


Fig. 3. Schematic representation of a floating-roof anchored atmospheric storage tank.

3. Finite element models and analysis

The three archetype tanks were modelled using four-node elements available in the SAP2000 software (Computers and Structures Inc, 2021), that combine plate bending and membrane action formulations. The model mesh discretization was increased towards the base of the cylindrical shell, adapting to the expected concentration of bending-induced stresses, as shown in Fig. 4. The presence of a stiffening ring beam along the rim of the tank, that also acts as a stopper to the floating roof, was modelled by means of a kinematic constraint of the top nodes that preserves the undeformed circular shape.

In cylindrical tanks, the boundary conditions in the vicinity of the annular plate can be affected by localized uplift. Past work has analysed the behaviour of the partial uplifting of the base plate for cylindrical tanks subjected to internal hydrodynamic pressures induced by seismic actions (Malhotra and Veletsos, 1994a, 1994b). Consequently, contact between the base plates with the foundation mat was modelled via unilateral compression-only *Winkler springs* (Bakalis et al., 2017a) corresponding to a coefficient of subgrade reaction of 50 MN/m³.

The axial behaviour of bolts, consistent with the design assumption of no preload, was modelled via tension-only bilinear yielding springs. Shear behaviour was assumed linear elastic. Given that friction forces between base plate and foundation mat were not included in the model, the consideration of possible failure modes involving bolt shear was addressed in analysis post-processing, to follow.

For each archetype tank, a series of analyses were carried out, by varying the tank's content level and the tsunami height and velocity. The liquid content levels covered the entire allowable range, starting from an assumed 1 m minimum filling level and going up to each tank's h_{fmax} . On the other hand, tsunami inundation depth values spanned a range from a lowest-considered value of 1 m up to each tank's full height. Internal liquid and inundation levels were varied at 1 m increments, with the exception of cases where the uplift force was almost approaching the tank and contents' self-weight, where a finer step was adopted to allow better investigation of base plate deformations during imminent loss of contact with the foundation mat. Finally, seawater velocities considered

ranged from 0 to 5 m/s, at 1 m/s increments. For the slender tank only, velocity was considered up to 10 m/s, to better map the development of tensile forces imposed on the anchors.

The output extracted from each analysis run for post-processing were the circumferential compressive stresses of the steel cylindrical wall shell, σ_θ and the tensile forces at the anchor bolts, F_T . For the circumferential stress averaged-across-wall-thickness, σ_θ , values were obtained from membrane force resultants of the finite elements, evaluated at the standard two-by-two Gauss integration points of each element, as shown in Fig. 5. For the fragility analysis, the maximum value of circumferential compressive stress over all shell elements of the wall, $\sigma_{\theta,D}$, was considered as the demand parameter for the shell buckling damage state.

Fig. 6(a) shows the gradual uplift response of the intermediate tank's base plate, as the tsunami depth increases from 2.7 m to 2.8 m, while tsunami velocity is fixed at 1 m/s and the tank is filled up to one-third of its maximum storage capacity. Depending on each tank's filling level, buoyant forces from the tsunami wave can lead to an almost complete loss of contact of the base plate to the foundation. For these cases, it was

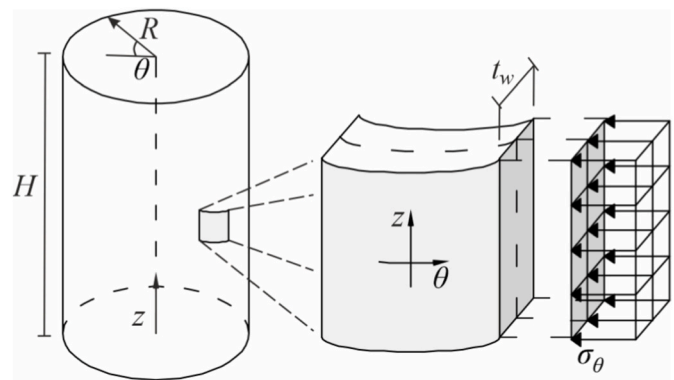


Fig. 5. – Circumferential stresses corresponding to membrane forces.

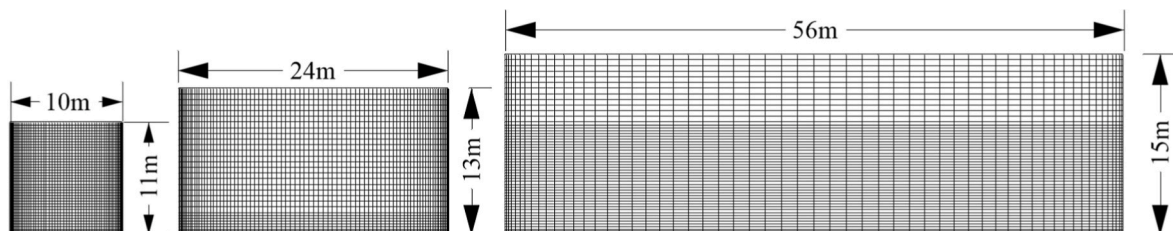


Fig. 4. – FEM meshes of tanks.

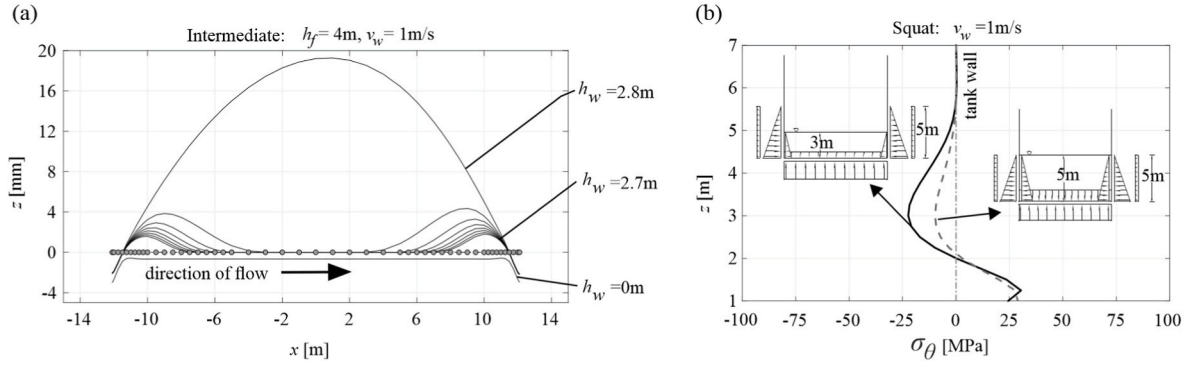


Fig. 6. – (a) Base plate uplift of intermediate tank. (b) Membrane circumferential stresses along the shell wall of squat tank.

necessary to include large-displacement geometric non-linearity in the analysis, for the model to account for the stiffening effect of membrane forces generated in the deflected base plate.

Panel (b) of the same figure shows the values of the membrane circumferential stresses at the nodes of the shell elements of wall, along the direction of flow, for a given flow velocity and 5 m of tsunami inundation depth. The two cases shown in the figure refer to a squat tank with filling levels of 3 m and 5 m. In both cases, the uplift force exceeded the tank's self-weight, and the maximum compressive circumferential stresses occurred in a zone around 3 m high from the base of the tank. A validation analysis was also carried out comparing the circumferential stresses coming from linear elastic analyses of the tank shell walls, with analytical solutions (Billington, 1965), considering the boundary condition of the shell as fixed.

Fig. 7 shows the results in terms of $\sigma_{\theta,D}$ for five filling levels of the slender archetype tank, for various tsunami inundation depths, at a velocity value of 1 m/s. The results of the nonlinear elastic analysis in large displacements show a trend ascendent and linear for a given filling level. As expected, the largest circumferential compressive stress, given a value of h_w , is associated to the tank being almost empty (white circles in the figure), while the cases shown where $\sigma_{\theta,D}$ is null correspond to the shell remaining in tension from the hydrostatic action of the liquid content.

Fig. 8 shows the tensile force of two anchors in the slender tank, for varying tsunami flow velocity in the abscissa, five levels of tsunami inundation depth and two filling levels of 1 m and 9 m. In each panel, a black dot on the circumference indicates the position of the anchors with respect to the direction of flow, which is indicated with an arrow. The figure shows the combined effects of uplift and drag forces on the anchorage system, for the two cases of an almost empty (panel a) and an almost full tank (panel b).

For the anchor located in the downstream part, there is tension due

to buoyancy at the low filling level, which decreases with increasing velocity and consequent drag force. The figure shows that for velocity values exceeding 6 m/s, tension in that anchor becomes null, meaning that the base plate around it has regained contact with the foundation mat. The second panel of the figure enables exploring the behaviour of an anchor on the meridional normal to the flow direction under full-tank conditions. In this case, it takes an inundation depth of 8 m or more for the uplift forces to overcome gravity loads, and for velocities between 4 m/s and 5 m/s, the anchor at that location is unloaded, because the drag forces' overturning moment creates a contact surface with the foundation mat, which has a neutral axis approximately coinciding with the corresponding diameter. For velocities higher than 5 m/s the neutral axis moves downstream, and that anchor returns to tensile force. Possible anchor failure is not considered in the FEM; instead, these forces constitute the initial conditions for a post-process progressive collapse calculation that accounts for uncertainty in anchor capacity, which will be described later on.

4. Fragility functions via conditional hazard

Fragility functions are needed for the probabilistic risk analysis of tsunami-triggered NaTech events. Fragilities provide the probability of structural damage leading to a loss-of-contents event, termed herein the probability of *failure*, conditional to natural event *intensity measure* (IM) and, in this study, also tank's filling level. Among scalar IMs, earlier works dealing with tsunami risk employed peak coastal tsunami amplitude (Geist and Parsons, 2006; Sørensen et al., 2012), whereas recently (Selva et al., 2016; Volpe et al., 2019) there is a trend to express tsunami hazard using maximum inundation depth at the structure's site, $h_{w,max}$, and the (maximum) momentum flux, m_f , which is defined as:

$$m_f = \rho_w \cdot h_w \cdot v_w^2 \quad (7)$$

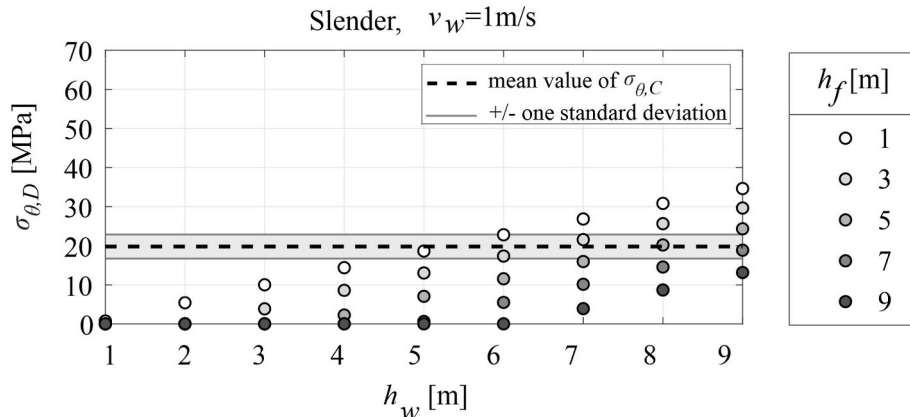


Fig. 7. – Maximum circumferential compressive stress in the shell wall for different levels of tsunami inundation depth and levels of liquid contents for slender tank.

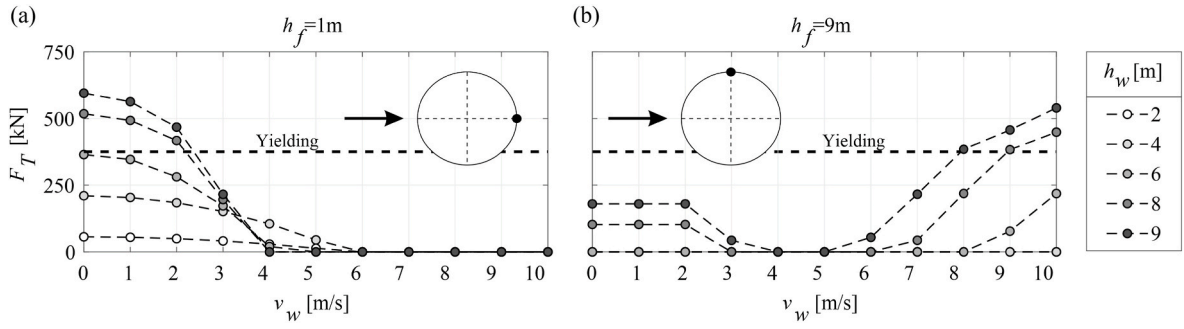


Fig. 8. – Tensile force of two anchors in the slender tank, for some values of tsunami inundation depth and flow velocity, for (a) an almost empty tank; (b) an almost full tank.

Equations (1) and (3) reveal that the maximum drag force produced is proportional to the peak value of momentum flux m_{fmax} , the maximum uplift force due to buoyancy is proportional to h_{wmax} and the circumferential compression stress demand on the shell walls generally depends on a combination of the two. The three damage states of shell buckling, uplift and sliding, the corresponding engineering demand parameters, such as $\sigma_{\theta,D}$ and anchor tensile or shear forces, depend on all three of drag force, uplift force and external circumferential pressure, which renders the choice of a scalar IM non-trivial. In fact, as Fig. 9 shows the h_w , v_w and m_f time-histories of a shallow-water tsunami simulation (Volpe et al., 2019) at the Italian coastal site of Milazzo, the maxima of h_w and m_f do not occur simultaneously, since v_w does not reach its peak value at the same time as h_w . To overcome these issues, herein a *conditional hazard* approach (Iervolino et al., 2010) was adopted, which enables considering multiple IMs (i.e., surrogating vector-valued hazard analysis), one of which is defined as *primary* and the others *secondary*, in a probabilistically consistent manner, yet keeping the advantage of fragility being expressed in terms of a scalar IM.

The *primary* tsunami IM is h_{wmax} , thus fragility functions can be denoted as $P[F|H_{Wmax}, \Psi]$, where $\Psi = h_f/h_{fmax}$ is the percentile filling level of the tank, which can also be treated as a RV, being generally unknown at the time of potential tsunami arrival, same as H_{Wmax} that is the RV associated to the realizations of h_{wmax} . To compute $P[F|H_{Wmax}, \Psi]$ accounting for all failure modes, $P[F_i|H_{Wmax}, \Psi]$ the fragility for each failure mode, $P[F_i|H_{Wmax}, \Psi]$, is calculated first as:

$$P[F_i|H_{Wmax}, \Psi] = \int \int_{\tilde{V} M_{Fmax}} P[F_i|\tilde{V}, M_{Fmax}, H_{Wmax}, \Psi] \cdot f_{M_{Fmax}|\tilde{V}, H_{Wmax}} \cdot f_{\tilde{V}|H_{Wmax}} \cdot d\tilde{V} \cdot dM_{Fmax}, \quad (8)$$

where $i = 1, 2, 3$ indicates the failure mode, with F_1 being shell buckling, F_2 anchorage tension failure, and F_3 sliding after shear rupture of the anchors. Moreover, in the equation, $\tilde{V}$ indicates the RV of the velocity at the time when h_{wmax} is reached, and M_{Fmax} denotes the RV maximum momentum flux; the lower-case symbols $\tilde{v}$ and m_{fmax} are used to represent realizations of the two RVs. Finally, $P[F_i|\tilde{V}, M_{Fmax}, H_{Wmax}, \Psi]$ are the conditional failure probabilities given maximum momentum flux,

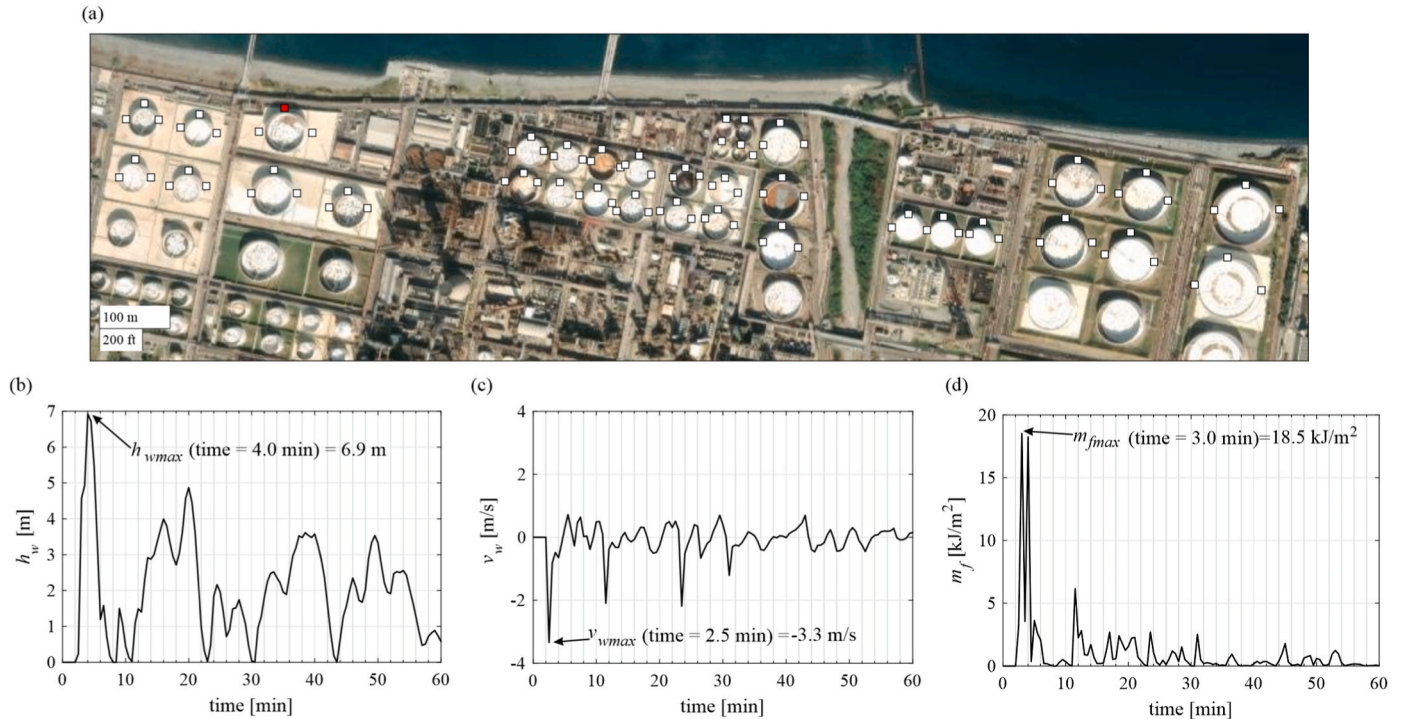


Fig. 9. – (a) White boxes represent ninety-five inland control points where the simulations are saved. The red box indicates the one point for which one simulation's time histories are shown: (b) the tsunami depth; (c) the flow velocity; and (d) the momentum flux. (For interpretation of the references to colour in this figure legend, the reader is referred to the Web version of this article.)

maximum inundation depth with corresponding velocity, and filling level, while the terms $f_{M_{Fmax}|\tilde{V},H_{Wmax}}$ and $f_{\tilde{V}|H_{Wmax}}$, represent the conditional probability density functions (PDFs) of M_{Fmax} , given $\tilde{V}$ and H_{Wmax} . Both were derived from a site-specific conditional hazard model (see the following section). In other words, taking advantage of conditional hazard, the fragility functions were developed for each failure mode, considering a primary IM, H_{Wmax} , and two secondary IMs, M_{Fmax} and $\tilde{V}$. After the secondary IMs were marginalized out, via integration using their conditional distributions given the value of the primary one, the resulting functions were fitted via probabilistic parametric models for simpler handling.

Once the fragilities for each failure mode were obtained, they were combined into a single function, under the conditional independence assumption:

$$P[F|H_{Wmax}, \Psi] = 1 - \prod_{i=1}^3 (1 - P[F_i|H_{Wmax}, \Psi]). \quad (9)$$

4.1. Site-specific conditional tsunami hazard

The conditional hazard is needed to derive the conditional distributions of the secondary IMs discussed in the previous section. To this aim, the probabilistic tsunami hazard analysis (PTHA) performed for an oil refinery site located on the northern Sicilian coast of Italy (Volpe et al., 2019) is considered. This PTHA concerns coastal locations around the Mediterranean basin and is based on numerical simulations of tsunami generation from a stochastic catalogue of seismic events and subsequent wave propagation towards the coastlines. More specifically, for the case of Milazzo, propagation up to the 50 m offshore isobath was simulated for each stochastic tsunami event. Subsequently, a representative subset of 1154 tsunami events was selected via clustering analysis, for which explicit modelling of inland inundation was performed, using a mixed finite-difference-finite-volume method for solving the shallow-water equations (de la Asunción et al., 2013). For these inland propagation simulations, storage tanks were considered as rigid obstacles. Thus, for each one of the selected tsunami events in the explicit inland propagation subset, the time-histories of inundation depth and average flow velocity were calculated and saved at 95 points located around the petrochemical plant's storage tanks, as shown in Fig. 9 (a). Therefore, a total of 55686 (95 times 1154) time-histories of tsunami inundation depth and flow velocity are available. Fig. 10 provides a summary of the h_{Wmax} and m_{Fmax} values obtained from these simulations, at all recording points, in the form of histograms. As expected by typical recurrence relationships of earthquakes, the figure shows that lower tsunami intensities are better represented in this data than higher ones.

Towards conditional hazard analysis, m_{Fmax} and $\tilde{v}$ values from simulations were binned at intervals of h_{Wmax} values 0.1 m wide. The sample conditional mean and standard deviation of their natural logarithms are calculated per bin, as shown in Fig. 11, where each circular marker corresponds to bin statistics. Then, to simplify subsequent calculations,

the conditional averages of the logarithms of velocity at maximum depth and maximum momentum flux, are fitted against this data to obtain the following equations:

$$\begin{cases} \mu_{\ln(\tilde{v})|h_{Wmax}} = \alpha_1 \cdot \ln(h_{Wmax}) + \beta_1 \\ \mu_{\ln(m_{Fmax})|h_{Wmax}} = \alpha_2 \cdot \ln(h_{Wmax}) + \beta_2 \end{cases} \quad (10)$$

As shown in the figure, linear models were also tried for the (log) standard deviations, but the slopes resulting from the fits were such that the approximation of constant conditional standard deviations was adopted, with $\sigma_{\ln(\tilde{v})|h_{Wmax}} = 0.9$ and $\sigma_{\ln(m_{Fmax})|h_{Wmax}} = 1$. Regarding the notation used here, $\alpha_1, \alpha_2, \beta_1, \beta_2$ are model coefficients, $\mu_{\ln(m_{Fmax})|h_{Wmax}}$ and $\sigma_{\ln(m_{Fmax})|h_{Wmax}}$ are the mean and the standard deviation of logarithm of the momentum flux given tsunami depth, while $\mu_{\ln(\tilde{v})|h_{Wmax}}$ and $\sigma_{\ln(\tilde{v})|h_{Wmax}}$ are the mean and the standard deviation of the logarithm of flow velocity simultaneous to the maximum inundation depth. In these equations, units for depth are m, for velocity m/s and for momentum flux kJ/m².

Conditional hazard also requires the conditional correlation coefficient, ρ , between the logarithms of momentum flux and flow velocity, given inundation depth:

$$\rho = \frac{COV(\ln(\tilde{v}), \ln(m_{Fmax})|h_{Wmax})}{\sigma_{\ln(\tilde{v})|h_{Wmax}} \cdot \sigma_{\ln(m_{Fmax})|h_{Wmax}}}, \quad (11)$$

where the operator COV indicates the covariance. A linear model was also fitted in this case:

$$\rho = a \cdot h_{Wmax} + b, \quad (12)$$

where a and b are coefficients estimated via regression, Fig. 12 (a), given in Table 2, along with the coefficients of the models for the conditional means.

At this point, the bivariate Gaussian model, on which the conditional hazard model is based, can be computed for each value of the primary IM. More, specifically, assuming that the logarithms of M_{Fmax} and $\tilde{V}$ are jointly Gaussian, conditional on $H_{Wmax} = h_{Wmax}$, their distribution is completely defined by the parameters modelled via Equations 10–12, with the corresponding probability density function shown, as an example, in Fig. 12 (b) for $h_{Wmax} = 3$ m. Under this assumption, the conditional density $f_{M_{Fmax}|\tilde{V},H_{Wmax}}$ required by Equation (8) correspond to a lognormal distribution with parameters of logarithmic mean and standard deviation, $\mu_{\ln(m_{Fmax})|\tilde{v},h_{Wmax}}$ and $\sigma_{\ln(m_{Fmax})|\tilde{v},h_{Wmax}}$ respectively, given by:

$$\begin{cases} \mu_{\ln(m_{Fmax})|\tilde{v},h_{Wmax}} = \mu_{\ln(m_{Fmax})|h_{Wmax}} + \\ + \rho \cdot \frac{\sigma_{\ln(m_{Fmax})|h_{Wmax}}}{\sigma_{\ln(\tilde{v})|h_{Wmax}}} (\ln(\tilde{v}) - \mu_{\ln(\tilde{v})|h_{Wmax}}) \\ \sigma_{\ln(m_{Fmax})|\tilde{v},h_{Wmax}} = \sigma_{\ln(m_{Fmax})|h_{Wmax}} \cdot \sqrt{1 - \rho^2} \end{cases} \quad (13)$$

Another model that is later used to derive $P[F_2|H_{Wmax}, \Psi]$ is the

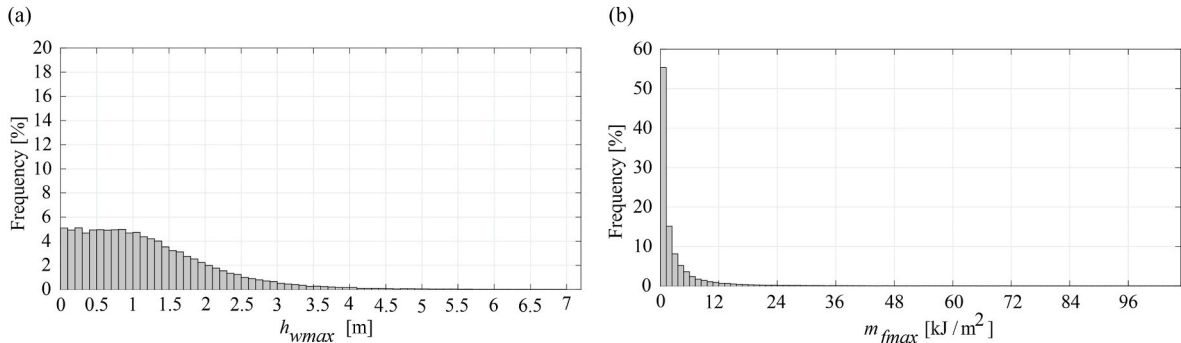


Fig. 10. – (a) Histogram of the maximum tsunami height. (b) Histogram of the maximum momentum flux.

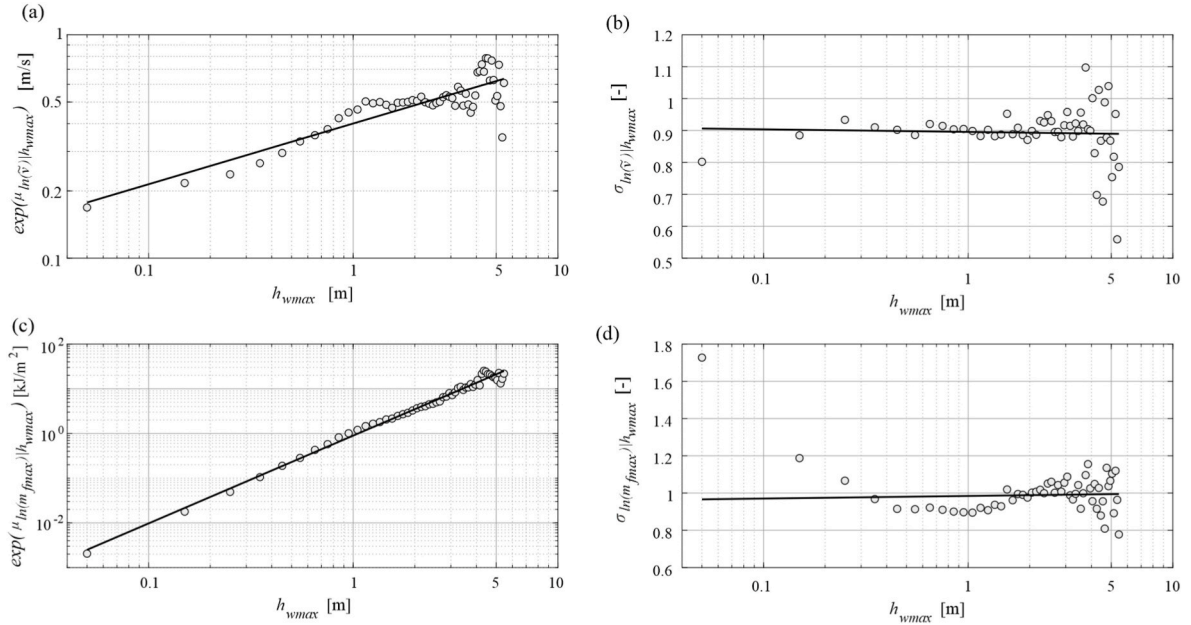


Fig. 11. –Median (a) and standard deviation (b) of the natural logarithm of $\tilde{v}$ given h_{wmax} . Median (c) and standard deviation (d) of the natural logarithm of m_{fmax} given h_{wmax} .

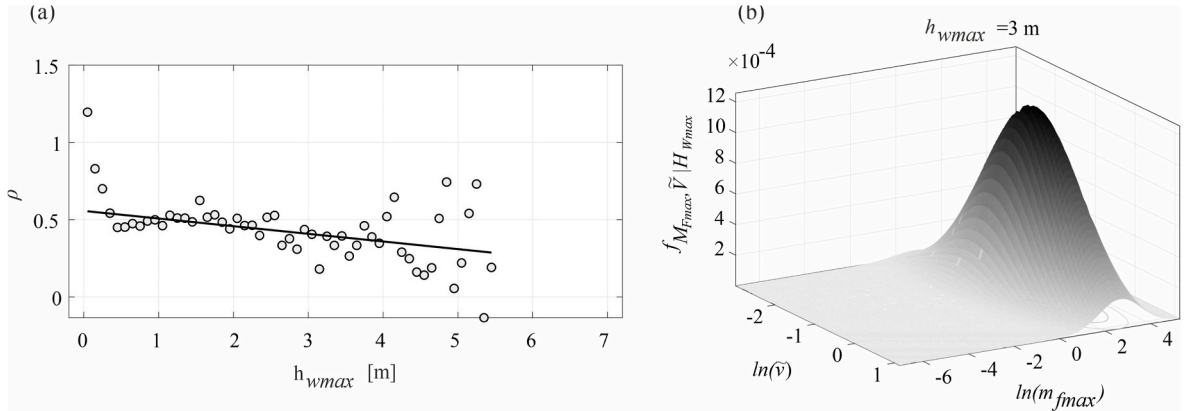


Fig. 12. – (a) Linear regression of the correlation coefficient between $\tilde{v}$ and m_{fmax} , given h_{wmax} . (b) Conditional joint probability density function of m_{fmax} and the $\tilde{v}$ conditional to h_{wmax} .

Table 2

Regression coefficients.

α_1	α_2	β_1	β_2	a	b
0.2716	1.9635	−0.9149	−0.1120	−0.0618	0.5883

conditional probability distribution of inundation depth, $\tilde{h}_w$, at the time when momentum flux reaches its maximum value:

$$\ln(\tilde{h}_w) = 0.35 \cdot \ln(m_{fmax}) - 0.41 + \varepsilon \cdot \sigma_{\ln(\tilde{h}_w)|m_{fmax}}. \quad (14)$$

In this equation (see Fig. 13), the linear term $0.35 \cdot \ln(m_{fmax}) - 0.41$ represents the conditional mean of $\ln(\tilde{h}_w)$, while the conditional standard deviation $\sigma_{\ln(\tilde{h}_w)|m_{fmax}}$ is provided by the regression's standard error, equal to 0.52.

4.2. Failure-mode-specific fragility functions

According to Equation (4), a conditional failure probability needs to

be attributed to each of the three failure modes. For shell buckling, it has been already mentioned that the uncertainty associated with the fabrication process is accounted for by assuming the geometric imperfection coefficient α_0 a normal RV. Therefore, the probability of failure due to shell buckling can be calculated as:

$$P[F_1|\tilde{V}, M_{fmax}, H_{wmax}, \Psi] = \Phi\left(\frac{\sigma_{\theta,D} - 0.65 \cdot \sigma_{\theta,cr}}{0.125 \cdot \sigma_{\theta,cr}}\right), \quad (15)$$

where $\sigma_{\theta,D}$ is the circumferential compression stress demand caused by each pair of values of $H_{wmax} = h_{wmax}$ and $\tilde{V} = \tilde{v}$, taken from the static analysis of the FEM.

The failure probabilities involving the anchorage system, resulting in uplift and/or lateral displacement (i.e., sliding), were calculated by means of Monte Carlo simulations, whose flowchart is given in Fig. 14. In each simulation, a certain level of liquid content and a value of m_{fmax} was assigned. Then, a value of $\tilde{H}_W = \tilde{h}_w$ was sampled from the Gaussian conditional distribution from Equation (14), given the maximum momentum flux value, and a value for the friction coefficient, μ_d , was sampled from a normal distribution with mean 0.5 and coefficient of

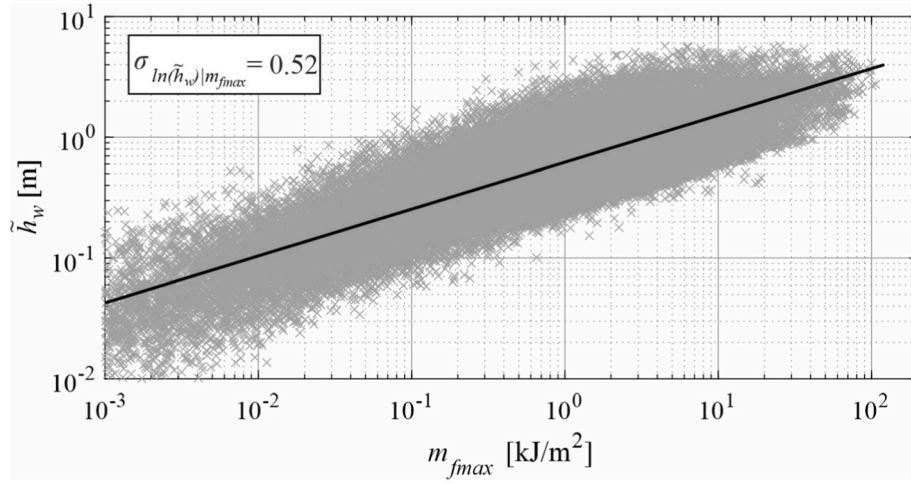


Fig. 13. – Regression line of the logarithm of inundation depth during maximum momentum flux against maximum momentum flux.

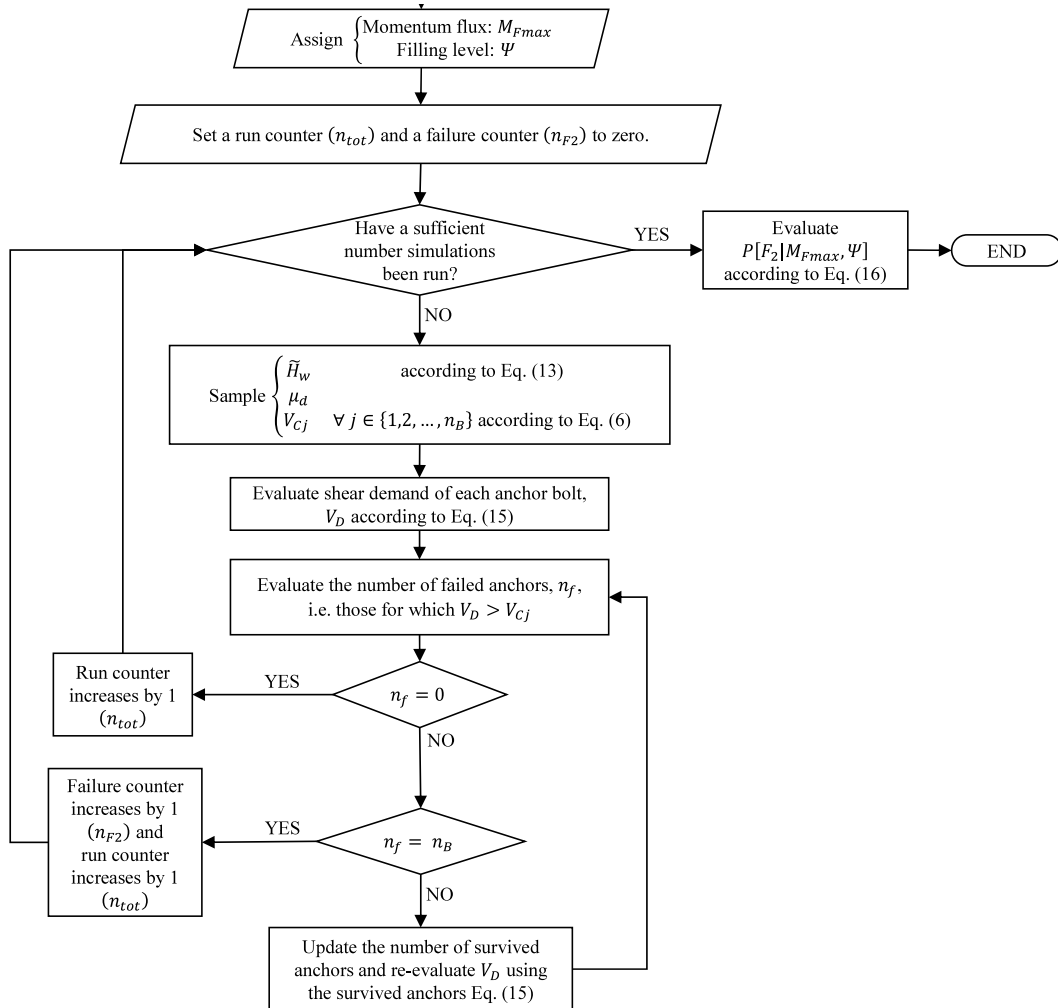


Fig. 14. – Flowchart for the evaluation of the conditional probability of shear failure of the anchor bolts.

variation of 0.3 (Iervolino et al., 2004). Finally, n_B values of anchor shear resistance were sampled from the distribution implied by Equation (6), V_{Cj} with $j = 1, 2, \dots, n_B$. The implication of this operation is that the shear resistance of each anchor bolt was assumed independent from those of the others and identically distributed. At this point an iterative

limit-analysis procedure is followed. At the start of each iteration, the shear demand V_D of each anchor bolt was calculated according to:

$$V_D = \begin{cases} 1/n_B \cdot \{2 \cdot C_d \cdot m_{fmax} \cdot R - \mu_d \cdot [(Q_f - S) \cdot \pi \cdot R^2 + G_1]\}, & \text{when } S \cdot \pi \cdot R^2 < (G_1 + Q_f \cdot \pi \cdot R^2), \\ 1/n_B \cdot (2 \cdot C_d \cdot m_{fmax} \cdot R), & \text{when } S \cdot \pi \cdot R^2 \geq (G_1 + Q_f \cdot \pi \cdot R^2). \end{cases} \quad (16)$$

which assumes equal partition of the resultant horizontal force among the anchors, that is drag force minus possible friction force. In this equation, G_1 is the self-weight of the empty tank that, for the tanks examined herein, assumed values of 538 kN, 2466 kN, and 10198 kN respectively for the slender, the intermediate, and the squat tank. If $V_{C,j} \geq V_D$ for all anchors, then no further iterations are necessary, and the outcome is that the anchorage system survives. If $V_{C,j} < V_D$ for one or more anchors, those anchors were considered to have failed and were removed, whereupon the number of anchors, n_B , was updated and V_D was re-evaluated using Equation (16) with the new number of surviving anchors. If subsequent iterations resulted in zero surviving anchor bolts, the simulation was deemed to have resulted in failure of the anchor system. Finally, the conditional failure probability was estimated as the ratio of the number of simulations that indicate failure of the anchorage system, n_{F2} , divided by the total number of simulations, n_{tot} :

$$P[F_2|\tilde{V}, M_{Fmax}, H_{Wmax}, \Psi] = P[F_2|M_{Fmax}, \Psi] \approx \frac{n_{F2}}{n_{tot}} \quad (17)$$

An analogous Mont Carlo was pursued for the case of tension failure of the anchors due to uplift forces (Fig. 15). In this case, the simulation started from a given set of h_{wmax} and $\tilde{v}$ values, for which finite element analysis results were already available, providing an axial force demand for each anchor bolt, $N_{D,j}$, with $j = 1, 2, \dots, n_B$. Then, for each anchor bolt a capacity $N_{C,j}$ was sampled according to Equation (5) and another iterative procedure began: if $N_{C,j} \geq N_{D,j}$ for all anchors, the outcome was that the anchorage system did not experience tensile failure, and no more iterations were needed. On the other hand, if $N_{C,j} < N_{D,j}$ for a number of anchors, then those anchor bolts were considered to have failed and the sum of their axial force demand was added to those of the remaining anchors, assuming that the base plate connection points with the anchor bolts remain on a plane, whereupon the new demands were again compared to the capacities. If all bolts failed in tension after a number of iterations, then that simulation was tagged as a failure of the anchorage system leading to tank's flotation. At the end of a number of n_{tot} simulations with n_{F3} recorded failures, the conditional failure probability was estimated according to:

$$P[F_3|\tilde{V}, M_{Fmax}, H_{Wmax}, \Psi] = P[F_3|\tilde{V}, H_{Wmax}, \Psi] \approx \frac{n_{F3}}{n_{tot}}. \quad (18)$$

It should be noted that, in these simulations, any intra- and inter-component correlations (Bakalis, 2024) were neglected, with

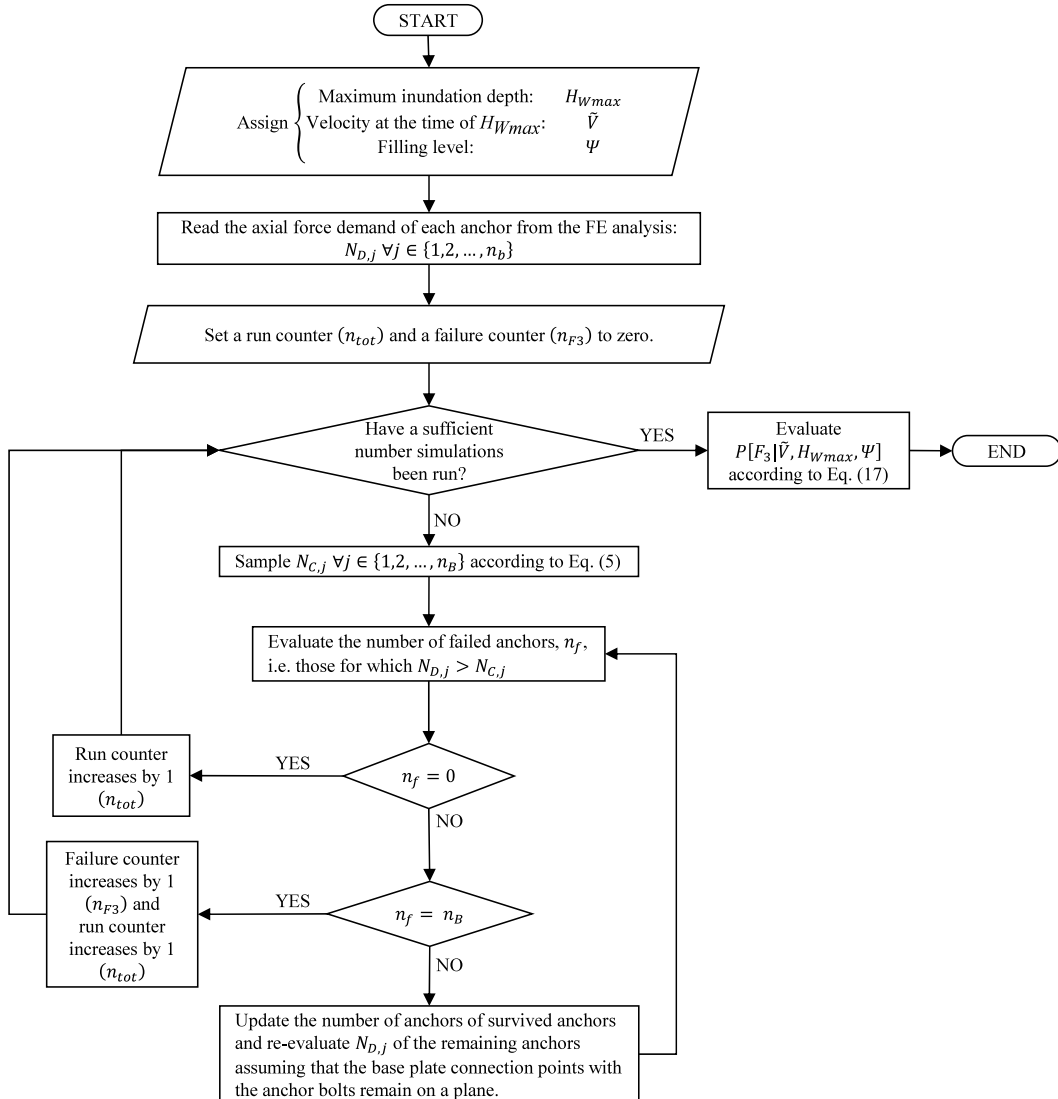


Fig. 15. Flowchart for the evaluation of the conditional probability of tension failure of the anchor bolts.

intra-component correlation referring to the stochastic dependency among the two capacity parameters for a single anchor and inter-component correlation representing the spatial dependency of a single capacity parameter among all anchors. This can be justified for two reasons. First, it was observed from the simulations results that tension failure was mainly governed by concrete cone breakout, while the shear failure is due to the shear rupture of the steel. For this reason, tension and shear strength of an individual anchor are assumed uncorrelated because the tension failure happens mostly on the concrete side. Thus, the intracomponent correlation between two strengths is not considered. Regarding the inter-component correlation, trial simulations with varying degrees of correlation were carried out, all leading to similar results and it was thus decided to neglect it altogether.

All fragility curves obtained according to the procedure described above, were adapted to a parametric model. The parametric model that describes the best fit model for all failure modes, is the *Weibull* cumulative distribution function (CDF), defined through two dimensionless parameters, that is, the filling level and the inundation level of the tank, $Y = h_{wmax}/h_{fmax}$, under the assumption that the tank cannot be completely submerged:

$$P[F_i|Y, \Psi] = 1 - \exp\left(-\left[\frac{Y}{a_i(\Psi)}\right]^{b_i(\Psi)}\right), i = 1, 2, 3 \quad (19)$$

where $a_i(\Psi)$ and $b_i(\Psi)$ are the Weibull scale and shape parameters, respectively. These six parameters (two per failure mode) were calibrated for various levels of filling level, as shown in Fig. 16 for the intermediate tank geometry. The figure was structured in way that the first row shows the values of a while the second one, values of b ; instead, each column is referred to a failure mode. From the figure results that for shell buckling, both parameters have linear trend; while for the tension and shear failure criterion, a is linear function while b is a parabolic function. The equations of the parameters of the Weibull CDF, as a function of the filling level, are shown in Table 3. The results of the fit are shown in

Table 3

Scale and shape parameters of Weibull CDF for all tank geometries and failure modes.

Tank	Shell buckling	
Slender	a_1	$0.59 \cdot \Psi + 0.57$
	b_1	$26.49 \cdot \Psi^3 - 20.92 \cdot \Psi^2 + 12.01 \cdot \Psi + 5.47$
Intermediate	a_1	$0.67 \cdot \Psi + 0.24$
	b_1	$18.63 \cdot \Psi + 6.48$
Squat	a_1	$0.66 \cdot \Psi + 0.10$
	b_1	$41.69 \cdot \Psi + 9.00$
Tensile failure of anchor bolts		
Slender	a_2	$0.69 \cdot \Psi + 0.35$
	b_2	$-22.69 \cdot \Psi^2 + 33.30 \cdot \Psi + 16.88$
Intermediate	a_2	$0.67 \cdot \Psi + 0.20$
	b_2	$-66.11 \cdot \Psi^2 + 144.91 \cdot \Psi + 25.65$
Squat	a_2	$0.67 \cdot \Psi + 0.12$
	b_2	$-103.76 \cdot \Psi^2 + 443.87 \cdot \Psi + 35.32$
Shear failure of anchor bolts		
Slender	a_3	$0.26 \cdot \Psi^2 + 0.52 \cdot \Psi + 1.36$
	b_3	$-0.35 \cdot \Psi^2 + 0.88 \cdot \Psi + 2.97$
Intermediate	a_3	$0.77 \cdot \Psi + 1.25$
	b_3	$-0.51 \cdot \Psi^2 + 1.43 \cdot \Psi + 3.09$
Squat	a_3	$0.90 \cdot \Psi + 1.31$
	b_3	$-0.92 \cdot \Psi^2 + 2.24 \cdot \Psi + 3.29$

Fig. 17, for the case of intermediate tank when is half full.

4.3. Tank fragility functions

Fig. 18 shows the fragility curves for slender (panel a), intermediate (panel b) and squat (panel c) tanks given inundation and filling level, from Equation (19). Each curve is shown refers to a given filling level. The curves show a decrease in vulnerability when the tanks are fuller. In fact, Fig. 18 (d), shows a quasi-linear trend of the tanks' median capacities, in terms of normalized inundation depth, with the filling level.

It is also interesting to discuss the relative influence of each failure

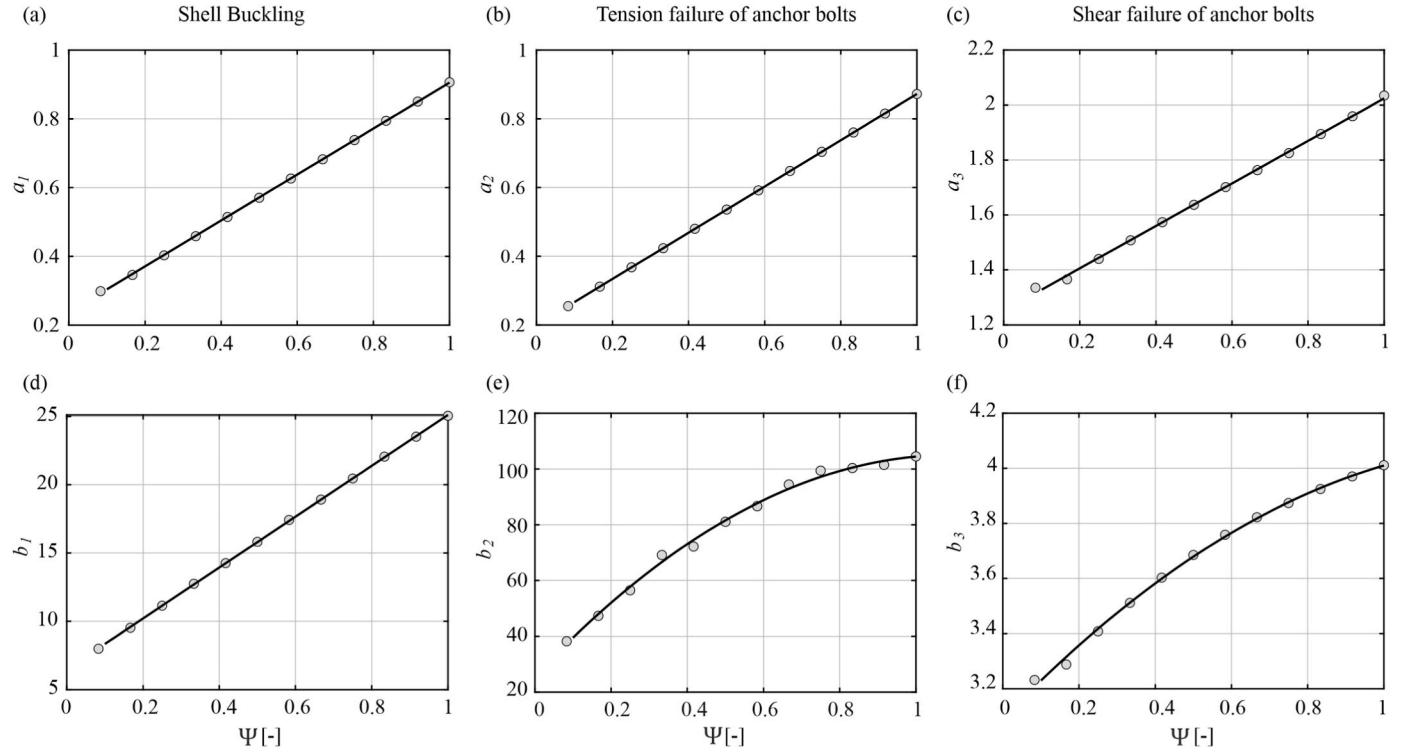


Fig. 16. Parameters of Weibull CDF at varying of the filling levels, for intermediate tank. In the first row, the scale parameter a for failure mode: (a) shell buckling; (b) tension of anchor bolts; (c) shear of anchor bolts. In the second row, the shape parameter b defined for: (d) shell buckling; (d) tension of the anchor bolts; (e) shear of the anchor bolts.

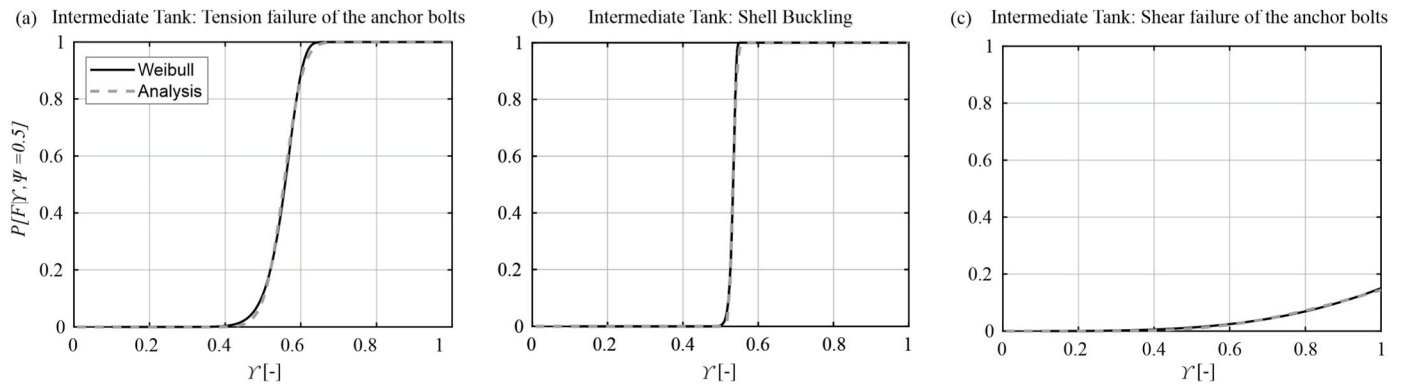


Fig. 17. Validation of the fit model, for intermediate tank, for: (a) shell buckling; (b) tension failure of anchor bolts; and (c) shear failure of the anchor bolts.

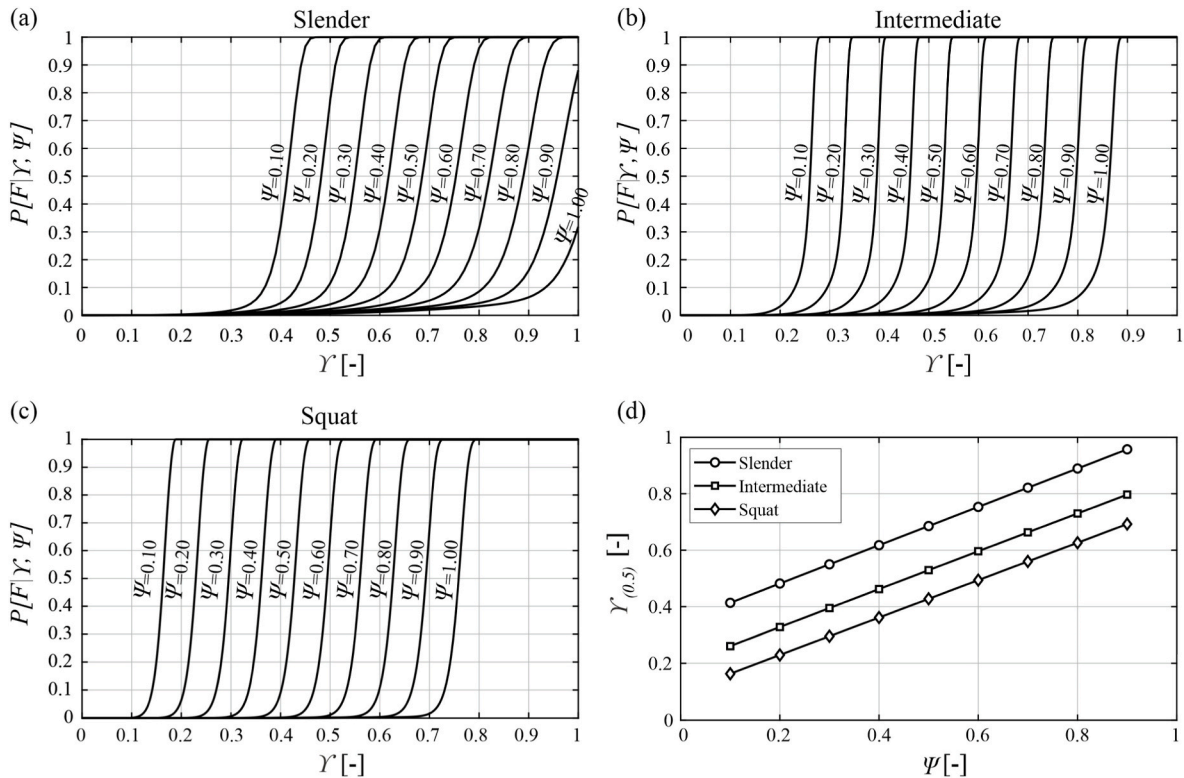


Fig. 18. Fragility curves at varying of the filling level for: (a) slender tank; (b) intermediate tank; (c) squat tank. (d) Inundation level corresponding to the probability of failure of fifty percent versus the filling level.

mode to overall fragility, which varies with inundation depth and level of tank contents, which is highlighted in Fig. 19. The figure shows some examples of fragility curves for three tanks, for different filling levels. More specifically, each row of panels corresponds to different filling levels, while the columns to the three tank geometries. Each panel shows a dotted curve that is the overall fragility function considering the interaction of all failure modes, while the dark blue curve is fragility curve for the shell buckling failure mode, the blue curve for tension failure of the anchors and the cyan curve to shear failure. It is observed that the mechanism that controls the failure for the slender is the tension of the anchors, panel (a); while, for the squat tank, the dominant failure mode is the shell buckling, panel (c). In these panels, it is seen as the overall curve (dashed line) is almost completely overlapped to the curve of the predominant failure mode. Conversely, the intermediate tank presents more than one mechanism that governs the overall fragility curves for low liquid levels. In more detail, the panels (b), (e) and (h) show the trend that it is common to all curves of the intermediate tank

up to 50 % of filling where for $P[F|Y, \psi] \leq 0.2$ the fragility is what it would have if it were considered only the shell buckling, while for $P[F|Y, \psi] > 0.2$ the curve carries on with a trend identified from fragility of only resistance mechanisms in tension.

5. Structural failure rate and comparison with models from the literature

The analytical fragility curves presented here can be used to calculate tank failure rates of tanks at the site of interest, which is an intermediate result needed for NaTech quantitative risk assessment. In the context of this work, the failure rate, λ_f , is the mean annual number of tsunami events that lead to release of contents due to structural damage of the tanks, and is a measure of pure-structural risk. Tsunami hazard at the site is expressed via the annual rate of exceeding various H_{wmax} levels, indicated as $\lambda_{H_{wmax}}$, which is typically termed the site's *hazard curve*. The failure rate can be calculated by integrating the product

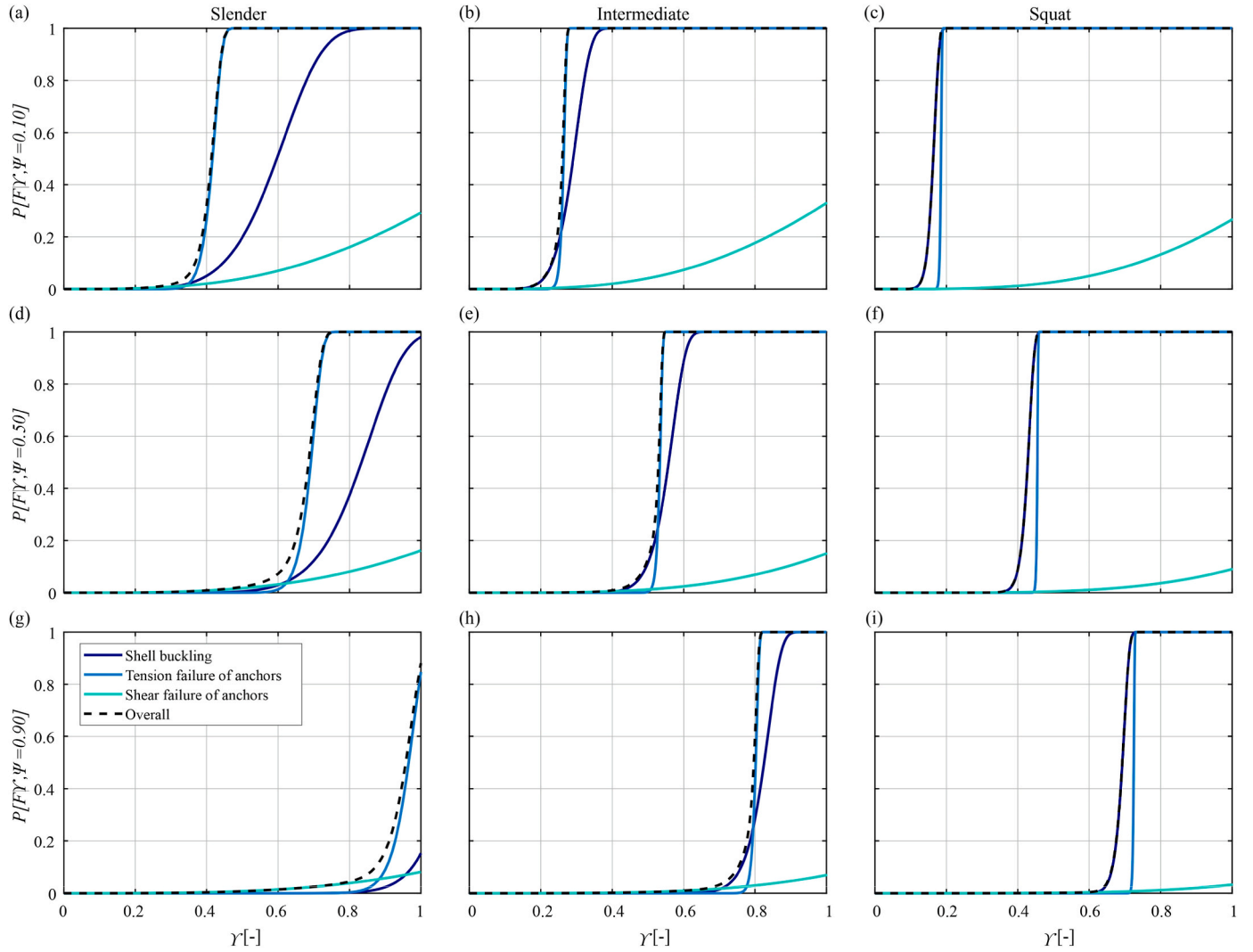


Fig. 19. Fragility curves for individual failure mode. Fragilities for filling level of 10 % for (a) slender, (b) intermediate, (c) squat tanks. Fragilities for filling level of 50 % for (d) slender, (e) intermediate, (f) squat tank. Fragilities for filling level of 90 % for (g) slender, (h) intermediate, (i) squat tanks.

between the fragility curve, and the absolute value of the differential of the hazard curve, $|d\lambda_{h_{Wmax}}|$:

$$\lambda_f = \int_0^{h_{fmax}} P[F|Y, \Psi] \cdot |d\lambda_{h_{Wmax}}| + \lambda_{H_{Wmax} > h_{fmax}} \quad (20)$$

which assumes that inundation depths exceeding the tank's height will lead to failure with certainty (Iervolino et al., 2018). The failure rates for the three archetypes tanks can be calculated for the site of Milazzo to compare with tsunami vulnerability from literature. Basco and Salzano (2016) provided tsunami log-normal fragility functions for 115 tanks with different geometries, having a filling level of 80 % with a density of the internal liquid of 750 kg/m^3 . In fact, in that work the fragility functions are expressed in terms of maximum momentum flux, thus the comparison between fragilities cannot be direct, unless a fragility intensity measure conversion is performed (Suzuki and Iervolino, 2020).

The three tanks from Basco and Salzano (BS) closest in geometry to the archetypes considered here are chosen for the comparison. Table 4 shows the geometric characteristics of these tanks, as well as the median value $\mu_{\ln(m_{fmax})}$ and standard deviation $\sigma_{\ln(m_{fmax})}$ defining the fragility.

The failure rates are calculated assuming that all tanks are located at the red dot in the top right corner of the panel (a) of Fig. 20 and that the filling level is known to be constantly 80 %. The hazard curves are

Table 4

Geometry features of selected tanks from database of Basco and Salzano (2016) and corresponding log-normal distribution fragility function parameters.

D [m]	H [m]	t_w [mm]	$\mu_{\ln(m_{fmax})}$	$\sigma_{\ln(m_{fmax})}$
10.5	9	8.4	2.60	0.48
24	12.6	12.5	3.00	0.50
48	7.20	12.5	2.30	0.50

shown in Fig. 20 in terms of exceedance rates of both maximum tsunami height and maximum momentum flux, $\lambda_{h_{Wmax}}$ and $\lambda_{m_{fmax}}$ respectively, and are taken from the work of (Volpe et al., 2019). The results are given in Table 5, with the first column considering the fragility of BS, whereas the second those from the present work. The differences can be judged as limited, given the profound difference in approach between the two studies.

6. Conclusions

The present study proposes a methodology for analytical derivation of tsunami fragility curves for anchored atmospheric storage tanks, based on numerical modelling of the structural response to an equivalent and quasi-static approximation of tsunami actions. These fragilities

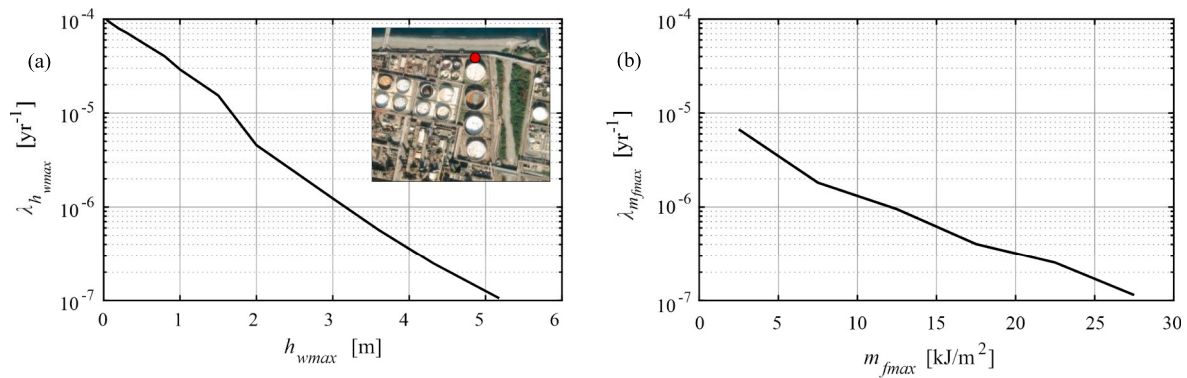


Fig. 20. Tsunami hazard curves in terms of (a) maximum tsunami height, and (b) maximum momentum flux (Volpe et al., 2019).

Table 5

Comparison between tsunami failure rates evaluated with fragility function of (Basco and Salzano, 2016) and analytical fragility.

	$\lambda_{f,BS} [yr^{-1}]$	$\lambda_f [yr^{-1}]$
Slender	$6.79 \cdot 10^{-7}$	$2.17 \cdot 10^{-7}$
Intermediate	$3.12 \cdot 10^{-7}$	$4.00 \cdot 10^{-7}$
Squat	$1.14 \cdot 10^{-6}$	$8.09 \cdot 10^{-7}$

consider failure modes that can ostensibly cause release of contents due to structural damage and can be therefore used in the context of NaTech quantitative risk assessment. The static equivalent tsunami action approximation could introduce some limitation to the scope of these fragilities, as it implies that failure is expected during the quasi-steady flow phase of tsunami inundation and that the flow's Froude number is not that high as to cause persistent water level difference upstream and downstream of the tank. Three archetype geometries were considered as well as various levels of liquid contents present in the tanks. Other than providing fragility curves to be used in NaTech practice, this work shows that.

- conversely to the case of ground shaking, tsunami vulnerability for these tanks increases the emptier they are;
- the prevalent failure mode can change with filling level, tank geometry and tsunami intensity; for slender tanks, the predominant failure mode is the tension failure of the anchor bolts; for squat tanks, on the other hand, shell buckling governs the structural behaviour under tsunami action; while for intermediate tanks, predominance of one failure mode over the other is not observed;
- comparison with the few cases where existing fragility models from the literature are available, showed consistent trends with varying tank slenderness, and failure rates within the same orders of magnitude.

It should also be mentioned that the use of site-specific conditional hazard models was necessary to evaluate the vulnerability of the storage tanks. The features of conditional hazard can be site-specific, which may hinder exportability of fragilities for applications elsewhere. Nevertheless, because the provided models show agreement with other models in the literature in terms of structural risk, for cases that permit comparison, the theoretical site-specific limitation may be less stringent in practice.

CRedit authorship contribution statement

Antonio Vitale: Writing – original draft, Formal analysis, Data curation. **Georgios Baltzopoulos:** Writing – review & editing, Writing – original draft, Methodology, Conceptualization. **Iunio Iervolino:**

Writing – review & editing, Supervision, Methodology, Conceptualization.

Declaration of competing interest

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

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Data availability

Data will be made available on request.

References

- API 650, 2018. Welded Steel Tanks for Oil Storage. American Petroleum Institute, Washington D.C., USA.
- Araki, S., Kunimatsu, W., Nishiyama, S., Furuse, T., Aoki, S., Kotake, Y., 2017. Experimental study on tsunami wave load acting on storage tank in coastal area. J. Loss Prev. Process. Ind. 50, 347–354. <https://doi.org/10.1016/j.jlp.2016.10.004>.
- ASCE/SEI 7-22, 2022. Minimum Design Loads and Associated Criteria for Buildings and Other Structures. American Society of Civil Engineers, Virginia.
- Attary, N., Unnikrishnan, V.U., van de Lindt, J.W., Cox, D.T., Barbosa, A.R., 2017. Performance-based Tsunami Engineering methodology for risk assessment of structures. Eng. Struct. 141, 676–686.
- Baiguera, M., Rossetto, T., Robertson, I., Petrone, C., 2019. A nonlinear static procedure for the tsunami design of a reinforced concrete building to the ASCE7 standard. In: Rossetto, T. (Ed.), SECED 2019 Conference. Society for Earthquake and Civil Engineering Dynamics, London.
- Bakalis, K., 2024. Incorporating modeling uncertainties in the collapse assessment of capacity-designed steel moment frames under seismic loading. J. Struct. Eng. 150.
- Bakalis, K., Fragiadakis, M., Vamvatsikos, D., 2017a. Surrogate modeling for the seismic performance assessment of liquid storage tanks. J. Struct. Eng. 143, 04016199. [https://doi.org/10.1061/\(ASCE\)ST.1943-541X.0001667](https://doi.org/10.1061/(ASCE)ST.1943-541X.0001667), 1-04016199.13.
- Bakalis, K., Karamanos, S.A., 2021. Uplift mechanics of unanchored liquid storage tanks subjected to lateral earthquake loading. Thin-Walled Struct. 158. <https://doi.org/10.1016/j.tws.2020.107145>.
- Bakalis, K., Vamvatsikos, D., Fragiadakis, M., 2017b. Seismic Risk Assessment of Liquid Storage Tanks via a Nonlinear Surrogate Model. Earthq Eng Struct Dyn 46, 2851–2868. <https://doi.org/10.1002/eqe.2939>.
- Basco, A., Salzano, E., 2016. The vulnerability of industrial equipment to tsunami. J. Loss Prev. Process. Ind. 50. <https://doi.org/10.1016/j.jlp.2016.11.009>.
- Bernier, C., Padgett, J.E., 2019. Buckling of aboveground storage tanks subjected to storm surge and wave loads. Eng. Struct. 197. <https://doi.org/10.1016/j.engstruct.2019.109388>.

- Billington, D.P., 1965. Thin Shell Concrete Structures. McGraw-Hill, United State of America.
- Calvi, G., Nascimbene, R., 2023. Seismic Design and Analysis of Tanks. John Wiley & Sons, New Jersey.
- Campedel, M., 2008. Analysis of Major Industrial Accidents Triggered by Natural Events Reported in the Principal Available Chemical Accident Databases. European Commission Joint Research Centre, Bologna.
- CEN, 2023. Eurocode 0: Basis of Structural Design. European Committee for Standardization.
- CEN, 2017. EN 1993-4-2: Design of Steel Structures - Part 4-2: Tanks. European Committee for Standardization.
- CEN, 2022. Eurocode 3: Design of Steel Structures - Part 1-1: General Rules and Rules for Buildings. European Committee for Standardization.
- CEN, 2004. EN 1991-1-4: General Actions - Wind Actions. European Committee for Standardization.
- CEN, 2005. EN 1993-1-8: Design of Steel Structures - Part 1-8: Design of Joints. European Committee for Standardization.
- CEN, 2013. Eurocode 8: Design of Structures for Earthquake Resistance - Part 1: General Rules, Seismic Actions and Rules for Buildings. European Committee for Standardization.
- Combesure, A., Gusic, G., 2001. Nonlinear buckling of cylinders under external pressure with nonaxisymmetric thickness imperfections using the COMI axisymmetric shell element. *Int. J. Solid Struct.* 38, 6207–6226. [https://doi.org/10.1016/S0020-7683\(00\)00359-0](https://doi.org/10.1016/S0020-7683(00)00359-0).
- Computers and Structures Inc., 2021. SAP2000 Integrated Software for Structural Analysis and Design.
- Consiglio Superiore dei Lavori Pubblici, 2018. Norme Tecniche per Le Costruzioni. Gazzetta Ufficiale della Repubblica, Italiana, Rome, Italy (in Italian).
- Council Directive 67/548/EEC, 1997. The Approximation of Laws, Regulations and Administrative Provisions Relating to the Classification, Packaging and Labelling of Dangerous Substances. Brussel.
- Cozzani, V., Campedel, M., Renni, E., Krausmann, E., 2010. Industrial accidents triggered by flood events: analysis of past accidents. *J. Hazard Mater* 175, 501–509.
- de la Asunción, M., Castro, M.J., Fernández-Nieto, E.D., Mantas, J.M., Acosta, S.O., González-Vida, J.M., 2013. Efficient GPU implementation of a two waves TVD-WAF method for the two-dimensional one layer shallow water system on structured meshes. *Comput. Fluids* 80, 441–452. <https://doi.org/10.1016/j.compfluid.2012.01.012>.
- Del Zoppo, M., Rossetto, T., Di Ludovico, M., Prota, A., 2023. Effect of buoyancy loads on the tsunami fragility of existing reinforced concrete frames including consideration of blow-out slabs. *Sci. Rep.* 13.
- Eligehausen, Rolf, Mallée, Rainer, Silva, J.F., 2006. Anchorage in Concrete Construction. Ernst & Sohn, Berlin, Germany.
- Fabbrocino, G., Iervolino, I., Orlando, F., Salzano, E., 2005. Quantitative risk analysis of oil storage facilities in seismic areas. *J. Hazard Mater* 123, 61–69.
- FEMA, 2008. Guidelines for Design of Structures for Vertical Evacuation from Tsunami. Federal Emergency Management Agency, Redwood City, CA.
- Fina, M., Wagner, W., Graf, W., 2023. On polymorphic uncertainty modeling in shell buckling. *Comput. Aided Civ. Infrastruct. Eng.* 38, 2632–2647. <https://doi.org/10.1111/mice.13054>.
- Foster, A.S.J., Rossetto, T., Allsop, W., 2017. An experimentally validated approach for evaluating tsunami inundation forces on rectangular buildings. *Coast. Eng.* 128, 44–57.
- Fuchs, W., Eligehausen, R., 1986. Tragverhalten Und Bemessung Von Befestigungen Ohne Randeinfluß Unter Querzugbelastung (Load-bearing Behaviour and Design of Fixing Without Edge Influence Under Shear Loading). Report. No. 0/8-86/12. Institut für Werkstoffe im Bauwesen, Universität Stuttgart, not published (in German).
- Geist, E., Parsons, T., 2006. Probabilistic analysis of tsunami hazards. *Nat. Hazards* 37, 277–314.
- Harish, S., Sriram, V., Schüttrumpf, H., Sannasiraj, S.A., 2021. Tsunami-like flow induced force on the structure: prediction formulae for the horizontal force in quasi-steady flow phase. *Coast. Eng.* 168, 103938. <https://doi.org/10.1016/j.coastaleng.2021.103938>.
- Hübner, A., Teng, J.G., Saal, H., 2006. Buckling behaviour of large steel cylinders with patterned welds. *Int. J. Pres. Ves. Pip.* 83, 13–26.
- Iervolino, I., Fabbrocino, G., Manfredi, G., 2004. Fragility of standard industrial structures by a response surface-based method. *J. Earthq. Eng.* 12, 927–945.
- Iervolino, I., Giorgio, M., Galasso, C., Manfredi, G., 2010. Conditional hazard maps for secondary intensity measures. *Bull. Seismol. Soc. Am.* 100, 3312–3319.
- Iervolino, I., Spillatura, A., Bazzurro, P., 2018. Seismic reliability of code-conforming Italian buildings. *J. Earthq. Eng.* 22, 5–27. <https://doi.org/10.1080/13632469.2018.1540372>.
- Krausman, E., 2010. Analysis of Natche Risk Reduction in EU Member States Using a Questionnaire Survey. Publications Office. <https://doi.org/10.2788/82675>.
- Krausmann, E., Renni, E., Campedel, M., Cozzani, V., 2011. Industrial accidents triggered by earthquakes, floods and lightning: lessons learned from a database analysis. *Nat. Hazards* 59, 285–300.
- Landucci, G., Antonioni, G., Tugnoli, A., Cozzani, V., 2012. Release of hazardous substances in flood events: damage model for atmospheric storage tanks. *Reliab. Eng. Syst. Saf.* 106, 200–216.
- Macabuag, J., Rossetto, T., Lloyd, T., 2014. Sensitivity analyses of a framed structure under several tsunami design-guidance loading regimes. In: 2nd ECEES Conference. European Association for Earthquake Engineering, Istanbul, Turkey.
- Malhotra, P.K., Veletsos, A.S., 1994a. Uplifting analysis of base plates in cylindrical tanks. *J. Struct. Eng.* 120, 3489–3505.
- Malhotra, P.K., Veletsos, A.S., 1994b. Beam model for base-uplifting analysis of cylindrical tanks. *J. Struct. Eng.* 120, 3471–3488.
- McGovern, D.J., Allsop, W., Rossetto, T., Chandler, I., 2023. Large-scale experiments on tsunami inundation and overtopping forces at vertical sea walls. *Coast. Eng.* 179.
- Mia, M.M., Kameshwar, S., 2023. Fragility assessment of bottom plates in above ground storage tanks during flood events. *J. Loss Prev. Process. Ind.*, 104975.
- O'Rourke, M.J., Liu, X., 1999. Response of Buried Pipelines Subject to Earthquake Effects. MCEER, Buffalo, NY.
- Paolacci, F., Corritore, D., Caputo, A.C., Bursi, O.S., Kalemli, B., 2018. A probabilistic approach for the assessment of LOC events in steel storage tanks under seismic loading. In: ASME 2018 Pressure Vessels and Piping Conference.
- Petrone, C., Rossetto, T., Goda, K., 2017. Fragility assessment of a RC structure under tsunami actions via nonlinear static and dynamic analyses. *Eng. Struct.* 136, 36–53.
- Rehm, G., Eligehausen, R., Mallée, R., 1988. Befestigungstechnik (Fixing Technology). Betonkalender. Ernst & Sohn, Berlin.
- Salzano, E., Iervolino, I., Fabbrocino, G., 2003. Seismic risk of atmospheric storage tanks in the framework of quantitative risk analysis. *J. Loss Prev. Process. Ind.* 16, 403–409. [https://doi.org/10.1016/S0950-4230\(03\)00052-4](https://doi.org/10.1016/S0950-4230(03)00052-4).
- Selva, J., Tonini, R., Molinari, I., Tiberti, M.M., Romano, F., Grezio, A., Melini, D., Piatanesi, A., Basili, R., Lorito, S., 2016. Quantification of source uncertainties in seismic probabilistic Tsunami Hazard analysis (SPHTA). *Geophys. J. Int.* 205, 1780–1803. <https://doi.org/10.1093/gji/ggw107>.
- Sørensen, M.B., Spada, M., Babeyko, A., Wiemer, S., Grünthal, G., 2012. Probabilistic tsunami hazard in the Mediterranean Sea. *J. Geophys. Res. Solid Earth* 117.
- Suzuki, A., Iervolino, I., 2020. Intensity measure conversion of fragility curves. *Earthq. Eng. Struct. Dynam.* 49, 607–629. <https://doi.org/10.1002/eqe.3256>.
- Tsutsumi, Y., Aoki, S., Araki, S., 2023. Practical and precise formula for wave forces on single cylindrical tank by Run-up tsunami. In: Proceedings of the 11th International Conference on Asian and Pacific Coasts. Kyoto, Japan, pp. 807–816.
- Vathi, M., Karamanos, S.A., Kapogiannis, I.A., Spiliopoulos, K.V., 2017. Performance criteria for liquid storage tanks and piping systems subjected to seismic loading. *Journal of Pressure Vessel Technology, Transactions of the ASME* 139. <https://doi.org/10.1115/1.4036916>.
- Vitale, A., 2024. Earthquake and Tsunami multi-hazard Risk Analysis of a Petrochemical Plant (Phd Thesis in Ingegneria Strutturale Geotecnica E Rischio Sismico. Tutors: Iunio Iervolino, Georgios Baltzopoulos, Pasquale Cito). Università degli Studi di Napoli Federico II, Naples.
- Volpe, M., Lorito, S., Selva, J., Tonini, R., Romano, F., Brizuela, B., 2019. From regional to local SPHTA: efficient computation of probabilistic tsunami inundation maps addressing near-field sources. *Nat. Hazards Earth Syst. Sci.* 19, 455–469. <https://doi.org/10.5194/nhess-19-455-2019>.
- Zeitler, R., Wörner, J.-D., 1995. Tragverhalten von Dübelverankerungen in hochfestem Beton (Behaviour of anchors in high strength concrete). In: Beton- Und Stahlbetonbau. John Wiley & Sons, Ltd, pp. 241–244 (in German).
- CEN, 2006. EN 1998-4: Design of Structures for Earthquake Resistance - Part 4: Silos, Tanks and Pipelines. European Committee for Standardization.
- CEN, 2017. EN 1993-1-6: Design of Steel Structures - Part 1-6: Strength and Stability of Shell Structures. European Committee for Standardization.