Performance-Based Tsunami Engineering Methodology for Risk Assessment of Structures

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Abstract

Tsunamis are rare destructive phenomena caused by the sudden displacement of a large amount of water in the ocean and can result in enormous losses to coastal communities. The resilience of coastal communities to tsunamis can be improved through the use of risk-informed decision making tools. Performance-Based Engineering (PBE) approaches have been developed for different natural hazards including earthquake, fire, hurricane, and wind to perform probabilistic risk assessment for structures. In this study, a probabilistic Performance-Based Tsunami Engineering (PBTE) framework based on the total probability theorem is proposed for the risk assessment of structures subject to tsunamis. The proposed framework can be disaggregated into the different basic analysis phases of hazard analysis, foundation and structure characterization, interaction analysis, structural analysis, damage analysis, and loss analysis. An application example consisting of the risk assessment of a three-story steel moment frame structure was performed using the proposed framework. The probability of exceedance of the total replacement cost including structural, nonstructural, and content losses were computed.

Keywords:

Tsunami; Performance-Based Engineering; Loss Analysis; Hybrid Fragilities; Monte Carlo Simulation

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1. Introduction

Tsunamis are a series of large sea waves caused by the displacement of a large volume of water caused mostly by submarine earthquakes, but can also include landslides, volcanic eruptions, and meteor impacts. Tsunamis are a high-impact, low-probability event that, although rare, are extremely destructive, resulting in high casualty rates and billions of dollars of economic loss (e.g., Indian Ocean, 2004; Samoa, 2009; Chile, 2010 and Japan, 2011). To improve the resilience of coastal communities to tsunamis, decision makers can benefit from tools that would enable them to make risk-informed decisions. In recent years, probabilistic approaches, such as performance-based engineering (PBE), have gained significant attention and are being used in many specialty areas of civil engineering. The major benefit of a PBE approach is the articulation of performance metrics, which are applicable in the decision making process for hazard risk mitigation [1]. Performance-Based Earthquake Engineering (PBEE) has been at the frontier among natural hazards, starting approximately two decades ago [1-4]. The PBEE framework, implemented by Pacific Earthquake Engineering Research (PEER) center, intended to provide a tool for seismic risk-informed decisions for stake-holders, allowing for a more informative and scientific based approach. This framework is based on the total probability theorem and can be disaggregated into different analysis phases that include hazard analysis, structural analysis, damage analysis and loss analysis [1]. These phases must be carried out in sequence, resulting in an estimation of the frequency for which different levels of decision variables would be exceeded.

Following the PBEE framework and along with its recent applications (e.g. [5-8]) , the PBE approach was used in dealing with other hazards such as Performance-Based Fire Engineering (PBFE) [9, 10], Performance-Based Wind Engineering (PBWE) [11, 12] and Performance-Based

Hurricane Engineering (PBHE), [13]. One major difference between PBEE and the PBE for hazards which deal with fluids, such as PBWE, is the interaction analysis phase that should be performed before the structural analysis phase, accounting for the physical interaction between the structure and the environment [11] [13].

Design guidelines considering tsunami forces are currently very limited in the literature. Cross [14], described tsunami surge forces and predicted the forces occurring when a bore strikes a vertical wall, which were further validated using experimental data. Camfield [15] discussed different methods of predicting tsunami flooding and proposed generating mechanisms of tsunamis and the method of determining the probability of occurrence. Dames and Moore [16] presented different loading requirements that should be considered for tsunami resistant design of residential buildings. Later, the Japan Cabinet Office guideline [17], City and County of Honolulu Building Code [18] and FEMA P646 [19], proposed guidelines for designing vertical tsunami evacuation structures. ASCE 7-16 [20] now has a chapter dedicated to tsunami design, which relies on Probabilistic Tsunami Hazard Analysis (PTHA) to provide tsunami inundation hazard maps for the design of structures.

Preliminary efforts to develop PTHA by Lin and Tung [21], Rikitake [22], and Downes and Stirling [23] used the same approach as Probabilistic Seismic Hazard Analysis (PSHA) [24], to calculate wave heights using a simple source specification and attenuation relationships. Due to lack of historical data for tsunamis, Geist and Parsons [25], proposed another approach for PTHA by combining computational methods and empirical analysis of historical tsunami data. Burbidge et al. [26] used a Green's function summation to perform PTHA, running full probabilistic analyses over a much wider area. This approach was then used by Thio et al. [27] to perform PTHA for California in a PEER report, in which, tsunami hazard maps are used to

compute the tsunami inundation hazard for California [28]. Park and Cox [29] conducted a probabilistic tsunami hazard analysis conditioned on a full rupture Cascadia Subduction Zone event to estimate the annual exceedance probability of five intensity measures closely related to loss of life, immediate response and recover, and damage to the built and natural environment. These intensity measures were the inundation depth, velocity, momentum flux, arrival time, and duration, and were computed throughout the inundation zone. As mentioned earlier, these types of PTHA would be the first step in a fully probabilistic tsunami risk assessment methodology.

The need for Performance-Based Tsunami Engineering (PBTE) has been recognized by researchers for several years (e.g., [30, 31]) and is finding its way to codes and standards such as ASCE 7-16 [20]. However, currently there is no comprehensive PBTE framework in the literature. In this study, a probabilistic framework for PBTE based on the total probability theorem is proposed. Similar to other performance-based approaches (PBHE, PBWE, etc.), the proposed methodology can be disaggregated into different analysis phases each of which is presented herein. The proposed methodology is illustrated by performing a risk assessment of a three-story moment frame steel structure subjected to a tsunami.

2. Tsunami Loads and Effects

Depending on the distance between the origin and the point of interest, tsunamis can be classified into near-field and far-field tsunamis. For the case of far-field tsunamis, the mechanism by which the tsunami is generated, such as a large earthquake, is far enough from the point of interest so that the triggering mechanism itself does not have a direct impact on the structure. For near-field tsunamis, on the other hand, the triggering mechanism is close to the point of interest as is the case when an earthquake occurs offshore of the structure. In this case, the structure experiences the earthquake before being impacted by the tsunami water waves. Near-field tsunamis should be

treated as a multi-hazard event (cascading effects) requiring successive analysis; however, the focus of this paper is limited to the tsunami itself.

Tsunami effects on structures are broadly classified as hydrostatic forces, hydrodynamic forces, waterborne debris impact forces and scour effects [16, 19, 32]. Horizontal hydrostatic forces are typically imposed on the structure when standing water encounters can be calculated as:

$$F_h = \frac{1}{2} \cdot \rho_s \cdot g \cdot B \cdot h_{\max}^2 \quad (1)$$

where ρ_s is the fluid density including sediment, g is the gravitational acceleration, B is the breadth of the building, and $h_{\max}$ is the maximum water height above the base of the wall at structure location. The vertical hydrostatic force (buoyant force) is equal to the weight of water displaced. Buoyant forces are resisted mostly through the weight of the structure and are calculated using the following equation:

$$F_b = \rho_s \cdot g \cdot V \quad (2)$$

in which V is the volume of water displaced by the structure. The capacity of the structure to resist lateral loads may be reduced by buoyancy forces [33]. Hydrodynamic forces are caused by the flow of water and can be computed as:

$$F_d = \frac{1}{2} \cdot \rho_s \cdot C_d \cdot C_o \cdot B \cdot (hu^2)_{\max} \quad (3)$$

where C_d is the drag coefficient, B is the breadth of the building in the plane normal to the direction of flow, h is flow depth, and u is the depth-uniform flow velocity. C_o is a coefficient to account for openings and wall/window failure/breakage [34] and is generally less than unity. The term hu^2 is the specific momentum flux per unit mass and $(hu^2)_{\max}$ is the maximum specific momentum flux per unit mass at any time during the tsunami inundation at the point of interest.

Impulsive forces are caused by the leading edge of a surge of water impacting a structure, which is estimated using the following equation:

$$F_{\text{imp}} = \alpha \cdot F_d \quad (4)$$

where α is an amplification factor [35, 36].

The debris impact forces depend on the density of the debris in the region and the maximum flow velocity, and can be estimated as:

$$F_{\text{deb}} = u_{\text{max}} \cdot \sqrt{k \cdot m_d} \quad (5)$$

where u_{max} is the maximum flow velocity carrying the debris, m_d is the mass of the debris, and k is the effective stiffness of the impacting debris or the lateral stiffness of the impacted structural element(s). The other important aspect that should be considered regarding waterborne debris is the damming effect caused by accumulation of debris. To account for this effect, hydrodynamic force calculated using Eq. (3) can be modified using the breadth of the debris dam (B_d). Other effects of tsunamis on structures that should be accounted for in the analysis are the scour and erosion effects along with soil-foundation-structure interactions.

3. Proposed PBTE Methodology

The proposed PBTE methodology, similar to other PBE frameworks, should involve defining the following:

- An Intensity Measure (*IM*) in probabilistic form, which is a key parameter of the hazard intensity and has a strong correlation with structural response and damage.
- Foundation Structure Parameters (*FSP*), accounting for the relevant properties of the foundation structural system.

- Interaction Parameters (*IP*), describing interaction between the environment and structure.
- An Engineering Demand Parameter (*EDP*), providing a measurable quantity from the structural response subjected to the hazard.
- Damage Measures (*DM*), presenting structural behavior and capacity, including damage based on the *EDPs*.
- Decision Variables (*DV*), which quantifies the *DM*, to determine quantities such as economic loss or determine life safety.

In a PBE approach, typically, the structural risk is measured by the mean rate of exceeding a specified value of a *DV*, within a given reference period, corresponding to a specific performance objective. Similar to the PBWE and PBHE frameworks, the proposed PBTE framework is based on the total probability theorem. The structural risk is defined in terms of a given *DV* as:

$$G(DV) = \int \int \int \int \int G(DV | DM) \cdot f(DM | EDP) \cdot f(EDP | IM, IP, FSP) \cdot f(IP | IM, FSP) \cdot f(IM) \cdot f(FSP) \cdot dDM \cdot dEDP \cdot dIP \cdot dIM \cdot dFSP \quad (6)$$

where $G(\cdot)$ is a complementary cumulative distribution function, $G(\cdot|\cdot)$ is a conditional complementary cumulative distribution function, $f(\cdot)$ is a probability density function, and $f(\cdot|\cdot)$ is a conditional probability density function. Using Eq. (6), probabilistic tsunami risk assessment can be disaggregated into the phases of (1) hazard analysis, (2) structural characterization, (3) interaction analysis, (4) structural analysis, (5) damage analysis, and (6) loss analysis, which is shown schematically in Fig 1. Each phase is discussed in detail below.

Fig. 1. Sequential phases of PBTE framework

3.1. Hazard analysis

Like any other PBE methodology, the PBTE process starts with hazard analysis, providing the mean annual rates of occurrence of exceedance of the *IM*, which is defined as PTHA. The selection of *IM* components depends on the choice of models used to describe the environmental phenomena [13]. All near and far possible tsunami generating sources (including faults, landslides, volcanos, etc.), their properties (e.g., possible moment magnitudes, slip mechanism, source location, recurrence rate, direction, etc.), and possible scenarios should be, in theory, considered in PTHA. Numerical tsunami propagation models, which includes bathymetric/topographic effects, uses inputs from tsunami generation models to compute possible tsunami flow depth and velocity at the location of the structure. Recent methodologies proposing PTHA (e.g., [25-27] [29]) rely mostly on analysis and numerical simulations due to lack of historical data. In these recent methodologies, Monte Carlo simulation is typically employed, in which a large number of unique scenarios are combined, accounting for the likelihood of each scenario, to estimate the magnitude of the *IM*. In PTHA, a logic-tree is typically defined and a tsunami hazard curve is obtained for each path of the logic-tree. It is worth noting that PTHA can account for both epistemic and aleatory uncertainties. Examples of aleatory uncertainties considered in PTHA, are fault magnitude, dip angle uncertainties and random slip distribution uncertainties; while epistemic uncertainties in PTHA include fault segmentation, slip rate and recurrence

models (see e.g. [27]). Outputs of PTHA are the mean annual rates of occurrence of exceedance of *IMs* such as flow depth (h) and flow velocity (u), at the location of the structure. These *IMs* can also be estimated by fitting an appropriate probability distribution to existing historical data or simulations results for the particular location of interest. The statistical description of other *IMs*, such as fluid density including the sediment (ρ_s), impulsive force amplification factor (α), and waterborne debris properties (m_d and k_d) should also be considered in tsunami hazard analysis. Finally, the effect of macroroughness from the built and natural environment on the inundation, and possible flow amplification or reduction (shielding effect), as well as the direction and orientation of the water flow, should be included in the hazard analysis [37-39].

3.2. Foundation and structure characterization

A probabilistic description of the Foundation and Structure Parameter (*FSP*), defining the geometrical and mechanical properties of the foundation, the structure, and its site, which may influence the interaction of the structure with the hazard, is provided in this step. The parameters determining the structural stiffness and strength, as well as geometry of the structure are considered in this phase of PBE methods [11, 13]. Examples of important structural parameters that influence the lateral tsunami load acting on the structure are the breath of structure (B) and the opening coefficient (C_O), which influence the area of contact between the structure and water, but also influence the flow and ultimately the forces developed. Tsunamis can modify soil properties at the site by changing excess pore-water pressure in the soil and changing the consolidation coefficient [33]. Since structural uplift and overturning calculations are important factors for predicting the behavior of structures due to tsunamis, foundation characterization should also be considered.

3.3. Structure-hazard interaction analysis

As mentioned earlier, while the structure-hazard interaction step is not considered in the PBEE framework established by PEER, it is considered in PBWE [11, 12] and PBHE [13] frameworks because of the nature of the hazards in what concerns physical fluid-structure interactions. Physical interaction between the structure and fluid (water) is here described by the interaction parameters (*IPs*). The structure-hazard interactions affect the structural response and performance as well as the intensity and location of the loading. Common *IPs* are: area of contact, water pressure on the structure (affecting tsunami lateral force), volume of the water displaced (affecting buoyancy force), the drag coefficient (C_d) and breadth of debris dam that depends on the *IM* and *FSP*, simultaneously. For example, the volume of the water displaced (V) which will determine the buoyancy force on the structure, is a function of both the flow depth and the opening coefficient or the drag coefficient is a function of geometry and orientation of the structure as well as Reynolds number (which is a function of flow depth and velocity).

The uncertainty in both the *IM* and *FSP* must be taken into account to obtain probability distributions for the *IP*. A fluid-structure interaction analysis can be performed to calculate the water pressure acting on the components of the structure. Another example of this type of analysis is the soil-foundation-structure interaction analysis that is needed to capture the effects of these three components interacting (e.g., [40]). Other important *IPs* that should be accounted for, are erosion and scour effects which can be investigated through fluid-soil-structure interaction analysis.

3.4. Structural analysis

The structural analysis phase provides the probabilistic description of the *EDPs* needed to evaluate the damage to and performance of the structure. To capture all the aspects needed to perform PBTE, a non-linear time-history analysis can be used to calculate the response of the structure. However, because of the loading mechanism for tsunamis, performing multiple non-linear pushover analyses can also provide an adequate level of structural response evaluation and has been used to develop physics-based tsunami damage fragility curves (e.g. [34]). A common *EDP* used in PBEE for buildings is the inter-story drift ratio (IDR), which has a strong correlation with structural damage during earthquakes, and thus specific IDR limits have been proposed for different types of buildings [41]. The component forces of interest for tsunamis, are lateral loads, therefore using IDR as an *EDP* for PBTE, is felt to be logical. In addition, a number of researchers have used IDR for estimating the performance of structures subjected to tsunami loading and developed damage fragility functions (e.g. [42, 43] [34]). It should be noted that typically IDR would be governed by the lateral force resisting component of the structure in the direction of the lateral load. For the earthquake case, since the resisting forces are generated as a function of the inertial forces (mass) that are mainly generated at floor levels, and the inertial forces are transferred to the lateral load resisting elements through the floor diaphragms. However, for the tsunami case, the lateral load is imposed on the structure through the elements (such as walls, columns and beams) that come in contact with water. As long as local failures to these elements do not occur, the lateral force is transferred to the lateral force resisting systems. However, if local failures do occur in these elements such as out-of-plane walls failures, different *EDPs* should be used in combination with IDR. To capture this behavior, detailed 3D numerical simulations and progressive damage analysis should be performed, followed by comparison of

the response with specific performance limits. However, this is beyond the scope of this paper, which focuses on the methodology, and will be addressed in future work by the authors. Performance of the foundation should be evaluated through specific *EDPs* for foundations such as foundation rotation and foundation settlement.

3.5. Damage analysis

Damage analysis provides the probabilistic description of the damage measures (*DM*) conditional on the values of the *EDPs*. Damage analysis should be computed for the structure, the foundation, as well as the non-structural (displacement sensitive and water sensitive) components. Typically, fragility functions are presented in the form of a cumulative probability distribution for a particular damage state conditional on the *IM*. Generally, structural analysis is the most computationally intensive part of a performance-based analysis, and thus fragility functions can be used to substitute both structural and damage analysis phases of PBE frameworks. The concept of developing fragility functions for tsunamis are relatively new [44] and as mentioned earlier, the majority have been empirical functions, based on different *IMs* such as flow depth and flow velocity (e.g., [45-48]). However, using fragilities as a function of an *IM* will lead to one neglecting any changes in structural properties due to the damage (see loops shown in Fig. 1). Alternatively, fragility functions can also be represented as $P(DM/IP)$ instead of $P(DM/IM)$ [13], providing the possibility of modeling the structural system configuration and properties as a function of the level of structural damage. It should be noted that the proposed framework is flexible enough to adopt either of the fragility types ($P(DM/IP)$ or $P(DM/IM)$). Assuming the tsunami lateral force as the *IP*, fragility functions can be developed for the structure by imposing a large number of possible tsunami forces with different inundation

depths and performing non-linear pushover analysis to estimate the peak IDR (see [34] for additional details).

3.6. Successive analysis

Tsunami events often consist of multiple waves impacting a structure and can be in different directions, including the downrush which may have forces in excess of the uprush, which should be considered in the PBTE approach. ASCE 7-16 [20] considered two stages of tsunami flows: (1) incoming, and (2) outgoing flows, for the design of structures in the inundation zone. The density of the structures, shielding effect, density of vegetation, debris sources and other *IMs* will be different for the incoming and the outgoing flows, and thus the structural response may also be different for the incoming and the outgoing flows. Additionally, the initial damage of the structure may modify the structural parameters such as stiffness, strength, and openings in the structure, among others. Hence, the *IMs* and other structural parameters should be updated after the damage analysis, and the steps should be repeated. This is possible in the proposed methodology by the use of two loop chains initiating from damage analysis phase (see Fig. 1). The complete damage a structure would likely modify the flow field during the return flow or during subsequent inundations and could therefore change the *IMs* (flow depth, velocity).

3.7. Loss analysis and decision making

The annual probability of exceedance for the decision variable DV , $G(DV)$, can be estimated in the loss analysis step, where DV is used as an indicator for structural risk. Loss analysis consists of structure and foundation damages as well as non-structural damages. In addition to the direct losses due to the direct physical damage of building (both structural and non-structural), other indirect losses such as business and loss of income can also be considered in this analysis phase. In addition, tsunamis may lead to socio-economic disruptions such as population dislocation and

loss of employment, which can be considered while modeling the time dependent losses or the resiliency of an entire community. Commonly used *DVs* are the repair cost related to tsunami induced damage, life-cycle cost of structures (including construction, maintenance and repair cost after an event), and downtime or the time required to restore the functionality of the structure to its original state. The repair costs are highly uncertain and depend on local labor costs, availability of materials, and local construction practices [49]. The statistical description of the repair cost or loss can be obtained from the literature and/or market. The results of the loss analysis provide risk-informed metrics that can be used by stakeholders, emergency managers, and other community decision makers as a piece of the decision making process pertaining to tsunami hazard for the community.

4. Illustrative Example: Three-Story Office Building

The proposed framework is illustrated through the risk assessment of a hypothetical office building located in San Luis Obispo County, California, within the tsunami inundation zone and near the Diablo Canyon power plant (see Fig. 2). The benchmark building selected is a three-story steel structure, designed for Los Angeles, CA based on UBC 1994 (see [50] for details). The value of the target building was estimated to be around \$11M and the value of the contents is assumed to be \$5.5M [51].

Figure 2. Aerial map showing the location of the hypothetical office building (map data © 2016 Google)

4.1. Hazard analysis

As mentioned earlier, only far-field tsunamis are considered in this study. For illustration and based on the availability of data, the only sources of tsunami that are considered in this study are submarine landslides. For tsunami hazard analysis, the main IM considered in this case study are the flow depth, (h), flow velocity, (u), fluid density including the sediment, (ρ_s), and the impulsive force amplification factor, (α). The effect of tsunami flow debris on the target building was not considered in this study, and thus the waterborne debris properties (m_d and k_d) are neglected. The probabilistic characteristics of h and u were obtained by fitting a Weibull distribution to tsunami simulation records available from PG&E [52]. PG&E [52] performed a detailed PTHA for the Diablo Canyon power plant site where the existing submarine landslides near the site were identified and evaluated for their potential for future landslide activity. The wave heights for 114 individual landslide scenarios were numerically simulated using shallow water long wave hydrodynamic wave equations [52]. The flow depth and flow velocity were modeled as correlated because neglecting the effect of this correlation may produce erroneous results. Thus, to ensure that the dependence structure is accounted for in the risk assessment, the joint probability distribution of h and u was generated and used in this study.

Different techniques can be used to generate joint probability distributions (e.g. Nataf transformation developed by Der Kiureghian and Liu [53]; copula approach proposed by Nelsen [54]). In this study, the copula approach was used to model the joint probability distribution of h and u . A copula is a multivariate joint distribution defined on the n dimensional unit cube $[0,1]^n$ such that every marginal distribution is uniform on the interval $[0,1]$ [54, 55]. According to Sklar's theorem [55], the multivariate joint cumulative distribution function (CDF) of n random variables $X_i (i=1, \dots, n)$, can be expressed as:

$$F(X_1, \dots, X_n) = C[F_1(X_1), \dots, F_n(X_n)] = C(U_1, \dots, U_n) \quad (7)$$

where $F(X_1, \dots, X_n)$ is the joint CDF of variables $X_1, \dots, X_n$, $U_i = F_i(X_i)$ is the marginal CDF of the random variables $X_i (i=1, \dots, n)$ and $C(U_1, \dots, U_n)$ is the copula function. The joint CDF of $X_i (i=1, \dots, n)$ can be determined by the above equation if the marginal distributions of the random variables and the copula function are known. Different types of copulas can be used to describe the dependence between the random variables [56]. The investigation of the efficiency and accuracy of different copulas in modeling the dependence structure of the variables, albeit important, is outside of the scope of this study and will likely be investigated by researchers as more data becomes available. In this study, a Gaussian copula is adopted to model the dependence between h and u . Thus, using the correlation coefficient and the corresponding marginal distributions obtained from PTHA, the joint probability distribution for h and u is obtained (see Figure 3).

Figure 3. Probability of exceedance of flow depth and velocity for the Diablo Canyon power plant site, developed using the copula method

For this case study, the fluid density including transported sediments, (ρ_s), and impulsive force amplification factor, (α), are assumed to follow uniform distributions in the range of [1080 - 1320] kg/m^3 and [1-1.5] respectively.

4.2. Foundation and structure characterization

As mentioned earlier, in this step the geometric and mechanical properties of the structure and its foundation are defined. The height of the building, (H), is 11.89 m (39 ft) and width, (B), is 54.9 m (180 ft), and it is assumed the building has its longer dimension facing the ocean. Some of the details of the building are presented in Figure 4 and Table 1 and the reader is referred to FEMA 355c [50] for additional details on this well documented archetype building.

Figure 4. Floor plans and elevations for model buildings and floor plans showing layout of moment-resisting frames [50] (1ft=0.305 m)

Table 1. Designed sections of the three story building used in this study [50]

Story	Moment resisting frame			Gravity frames	
	Columns		Girder	Columns	Beams
	Exterior	Interior			
1	W14X342	W14X398	W33X141	W10X77	W16X26
2	W14X342	W14X398	W21X62	W10X77	W16X26
3	W14X159	W14X159	W21X62	W10X60	W14X22

In this study, it is assumed that the building has no openings, hence the opening coefficient, C_o , is assumed to be deterministic and equal to 1. In the event of multiple tsunami waves, the opening coefficient should be updated based on the damage state of the building (see Figure 1).

4.3. Intermediate analysis steps

The main *IPs* that control the effect of the tsunami on the structure are the area of contact between the water and structure, the volume of water displaced and total lateral tsunami force. Using the input from hazard analysis and the foundation and structure characterization phases, the area of contact between the water and the structure is determined. Since in this example, only the initial tsunami wave is considered, the area of contact is calculated by assuming the building to be a solid rectangular block. The coefficient of drag, C_d , is assumed to follow a uniform distribution in the range of (1.25-2) [20]. The total lateral tsunami force is calculated using Eq. (3) and (4) accounting for the hydrodynamic and impulsive forces. The buoyancy force is calculated using Eq. (2) based on the volume of the water displaced by the building.

In this approach, structural analysis and damage analysis phases are substituted by the use of fragility functions. To develop these fragility functions, 3D models of the structure developed using A572 Gr. 50 steel for all column, beam and girder sections in the design of the building. Most of the material property uncertainties for steel structures are due to the changes in the yield strength (f_y) of steel [57-59]. In this study, the yield strength is assumed to follow a lognormal distribution with a mean value of 55 ksi (380 MPa) and COV of 10% [59] and other parameters such as modulus of elasticity are assumed to be deterministic ($29000 \text{ ksi} = 2 \times 10^5 \text{ MPa}$). In order to capture the material uncertainty in this study, 100 random structural models were generated, using Latin hypercube sampling, each having unique yield strength, chosen from 100 random yield strength values. Non-linear pushover analyses were performed on each of the 100 models, by varying the tsunami forces with 39 different heights (total height of the building is 39 ft (11.89 m) which was incremented every 1 ft (0.305 m)) as a distributed load. The reason pushover analyses of different heights must be performed is due to the fact that by increasing the

height of the distributed load on a structure, the stiffness and capacity of the structure will decrease, thus altering the accuracy of the fragility [34]. For example, a simple cantilever column (neglecting the axial force) with height h_b , with a modulus of elasticity (E) and moment of Inertia (I), subjected to a uniform distributed load of w with a variable height of h , (i.e.: flow depth) is considered. Maximum deflection of the column and equivalent stiffness of the column can be calculated theoretically. The initial stiffness (k_{int}) can be calculated as:

$$k_{int} = \frac{24EI}{h^2 \cdot (4h_b - h)} \quad (8)$$

Assuming $h_b = 11.89$ m (39 ft), which is the total height of the application example building), the simplified pushover curves can be obtained by assuming a simple bi-linear behavior for the material. Figure 5 shows how one value of force (e.g. the dashed line shown in the figure) can correspond to different flow depths leading to a wide array of possible responses.

Figure 5. Simplified bi-linear push-over curves of a cantilever column based on different flow depths

As mentioned earlier, for each of these 100 structural models, different tsunami flow depths, were considered. For each considered flow depth, 4000 different total tsunami forces in the range of (50 kips (222kN) – 20000 kips (88965 kN)) were calculated, resulting in 156,000

(39x40=156,000) possible combinations. To estimate the responses of the 100 structural models, non-linear push-over analyses were performed for each of these combinations. Plastic hinges based on FEMA-356 were assigned to all of the members, to capture the non-linear behavior of the beams and columns (see Fig. 6). Hinges with coupled axial-force-bi-axial-moment (PMM hinge) behavior (based on section properties) are assigned for each column type. An interaction yield surface in three-dimensional PMM space is specified to these PMM hinges, representing the first occurrence of yielding for different combinations of axial force and the two moments. The interaction surface is defined based on FEMA-356 for steel columns. In addition, simple moment plastic hinges are assigned to the beams based on FEMA-356.

Figure 6. Sample plastic hinges results for distributed load push-over analysis

To obtain the fragility functions in this study, peak inter-story drifts ratios (IDR) are used as the engineering demand parameters (EDPs) for the damage assessment. Since the tsunami loading on the structure is a lateral load similar to the horizontal earthquake inertial forces, the peak IDR would give a reasonable estimate of the structural damage. The peak IDR is then compared against the HAZUS earthquake limit states for high-code seismic design level structures, for each of the four damage states, namely: Slight, Moderate, Extensive, and Complete. Fragility functions for the target structure were developed as a function of IP, tsunami lateral force (see Fig. 7).

Figure 7. Fragility functions based on tsunami total force for damage states of slight, moderate, extensive and complete

However, as mentioned earlier, for estimating the structural response, the height of the applied load is as important as its magnitude. The fragility functions based on force would not fully account for the height of the applied load leading to an underestimation of damage and loss. In this study, a hybrid fragility function, $P(DM|IP,IM)$, is developed by using the joint probability distribution of the lateral force (IP) and flow depth (IM) to provide damage fragility surfaces (see Fig. 8). In addition to structural damage, non-structural damage and content damage are also considered in this study. Unlike structural damages, the non-structural and content damage are mainly due to tsunami inundation and are functions of flow depth. FEMA [60] proposed damage-state parameters (medians, θ , and logarithmic standard deviations, β) for evaluating damages to non-structural systems and contents of the building based on tsunami flooding. For the building being assessed in this study, parameters for Extensive and Complete content damage states are provided by FEMA [60] and parameters for Moderate and Slight damage states are assumed based on the same logic, which is presented in Table 2.

Figure 8. Fragility functions based on tsunami total force and flow depth for damage states of slight, moderate, extensive and complete

Table 2. Damage-state parameters for evaluating non-structural and content damages based on water depth

Components	Slight Damage		Moderate Damage		Extensive Damage		Complete Damage	
	θ (m)	β	θ (m)	β	θ (m)	β	θ (m)	β
Nonstructural	0.9 (3 ft)	0.85	1.8 (6 ft)	0.81	3.6 (12 ft)	0.78	7.3 (24 ft)	0.65
Content	0.3 (1 ft)	0.85	0.9 (3 ft)	0.81	1.8 (6 ft)	0.78	4.6 (15 ft)	0.65

4.4. Loss analysis

Loss analysis of the structure includes structural loss, non-structural loss, and content loss. In this study the decision variable (DV) is taken as the replacement cost of the building. As mentioned

earlier, the total value of the structure is estimated to be \$11M [51]. The statistical description of the repair cost or loss for the building was obtained from FEMA [60]. The replacement cost ratios are assumed to follow a lognormal distribution with mean given in Table 3, and a coefficient of variation (COV) of 10%. The repair cost ratios used in the loss estimations are shown in Table 3 for an office building.

Table 3. Structural, Non-Structural, and Content damage ratios (percent of content replacement cost)

	Slight	Moderate	Extensive	Complete
Structural	0.4	1.9	9.6	19.2
Non-structural	1.6	8.1	30.8	80.8
Content	1.0	5.0	25.0	50.0

A multilayered Monte Carlo simulation was used in this study to estimate the probability of exceedance of loss (see Fig. 9). The flow depth, h , and flow velocity, u , were generated from their joint probability distribution. In addition, the flow density, ρ_s , impact coefficient, α , and coefficient of drag, C_d , were generated from their corresponding distributions. Using the random IMs and $FSPs$ generated, the area of contact, A_c , and the total lateral tsunami force, F_t , was computed using Eq. (4). The fragility functions for each component (structural, non-structural, and content) are used to simulate their corresponding damage state (DS_i). The replacement costs for the component (structural, non-structural, and contents) are then generated based on their corresponding damage state. The single iteration is repeated a large number of times (in this example 10,000) to estimate the probability of exceedance of the total replacement cost (L_i).

Figure 9. Multi-layered Monte Carlo simulation for the loss estimation of structures subject to tsunami

Figure 10, plots, in semi-logarithmic scale, the probability of exceedance of loss for the structural component, the non-structural component, and the content, together with the mean loss (ML) and the standard deviation of loss (SDL), using force-flow depth fragilities. It should be noted that Figure 10 is showing the probability of exceedance of loss conditioned on a tsunami occurring, and the occurrence rate of a tsunami is not accounted for in the analysis. In this example, it can be seen from Figure 10 that only small portion of total loss is associated with the structural loss. This can be explained by the fact that the mean value of the structural component is just 20% of the total value of the structure (see Table 3), while the non-structural component accounts for the remaining 80%. It is observed from Figure 10 that for losses less than \$5.5M content loss is predominant over non-structural loss, which can be explained by comparing the damage limit states shown in Table 2. Additionally, it is seen that the SDL is significantly higher than the ML, which indicates that ML is not sufficient alone to describe the risk of this illustrative example building. To illustrate the effectiveness of the two types of fragility functions explained in the previous phase, the loss analysis was performed for two different cases (function of force alone, and function of force and flow depth). Figure 11 shows the comparison of the probability of exceedance of structural loss for the two cases.

Figure 10. Probability of exceedance of structural loss, structural, non-structural loss, and total loss

Figure 11. Probability of exceedance of structural loss using force-flow depth surface fragility and force fragility curves

It can be seen that using fragility functions based on tsunami force alone would result in an underestimation of the probability of exceedance of structural loss (25% underestimation of ML). This can be explained by the fact that, as stated earlier, the use of fragility functions based on force alone do not fully account for the height of the applied load, which affects the behavior of the structure (see Fig. 8).

4.5. Risk assessment using existing fragility functions

One technique to propagate the uncertainties from the hazard analysis to loss analysis is through the use of fragility functions based on *IMs*. In this approach the probabilistic description of *IMs* are used in combination with fragility functions to obtain the probabilistic description of *DMs* (i.e., fragility functions are used to substitute the structural analysis phase). A similar approach can be used within the proposed methodology by using existing fragility functions to propagate the uncertainties from the hazard analysis to loss analysis (i.e., fragility functions are used to substitute the foundation and structure characterization, interaction analysis, and structural analysis phases as shown in Fig. 12). However, it should be noted that using this approach, only the initial tsunami wave can be considered and any changes in *IMs* and *FSPs* due to structural damage cannot be accounted for in the analysis.

Figure 12. Monte Carlo simulation for the loss estimation of structures subject to tsunami using the alternate approach

As mentioned earlier, in this approach the damage fragilities will be functions of *IMs* (instead of *IPs*). The joint fragility surfaces based on flow depth and velocity, as well as fragility functions based on flow depth alone, were developed for the target building (see [34] for detail on the methodology). Using the joint probability distribution discussed in the hazard analysis combined with *h-u* fragility surfaces, damage states were obtained and their corresponding replacement costs were generated. The loss curve obtained using the above mentioned method is compared with that obtained using fragility functions based on flow depth alone (see Fig. 13).

Figure 13. Comparison of probability of exceedance of structural loss computed using different approaches

As shown in Figure 13, the risk analysis using the two approaches (force-flow depth fragility surfaces and existing $h-u$ fragility surfaces) yield similar results. It is also observed from Figure 13 that using fragility functions based on flow depth alone significantly overestimates the probability of exceedance of structural loss in comparison with other two approaches, using fragility surfaces. This can be explained by the fact that for every flow depth there is a vast range of possible flow velocities that will not be fully accounted for in the fragilities that are a function of flow depth alone.

5. Conclusion

In this study a general Performance-Based Tsunami Engineering (PBTE) methodology is proposed. The proposed methodology is based on the total probability theorem, aimed at capturing all different sources of uncertainty, to perform a full risk assessment of a structure. In this framework the risk assessment is disaggregated into the following probabilistic analysis phases namely, hazard analysis, foundation and structural characterization, interaction analysis, structural analysis, damage analysis, and loss analysis. Each of these analysis phases was briefly

discussed, and the possible parameters that should be considered were identified. The interaction analysis phase, which is one of the novel features developed for PBWE and PBHE approaches over PBEE, was used in this framework to characterize the interaction between the structure and hazard.

The risk assessment of a three-story steel frame office building assumed to be near the coast in San Luis Obispo County, California, was used as an application example for illustrating the proposed framework. Two different types of tsunami fragility functions (based on the total lateral tsunami force and its combination with flow depth) were developed to substitute the structural analysis phase. A multi-layer Monte Carlo simulation approach was used to perform the risk analysis. The results obtained by using the two different fragility functions were compared, and it was observed that using the fragility curves as a function of total tsunami lateral force alone, would underestimate the loss. An alternative approach, using existing fragilities for tsunamis, is also presented and used in the application example. In this alternative approach, fragility functions based on different *IMs* (flow depth alone, flow depth-flow velocity combination) were developed, and used for the loss analysis. It was observed that using flow depth alone for tsunami fragilities would significantly overestimate the structural losses. It is noteworthy that the proposed probabilistic framework is consistent with the state-of-the-art in tsunami hazard and loss modeling, and at the same time is flexible, since it is based on the total probability theorem, which allows for independence of the different analysis components. However, as mentioned earlier, some simplifying assumptions were made in the presented application example to be able to compare the results with other similar approaches, which currently exist in the literature.

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