VULNERABILITY ASSESSMENT OF LOW-CODE REINFORCED CONCRETE BUILDINGS SUBJECTED TO TSUNAMI LOADING

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ABSTRACT

While lessons learnt from recent devastating tsunami events revealed the need for the estimation of the effects of tsunami wave on coastal structures, only a limited number of tools to estimate the potential impacts of tsunami are available until now. This study aims at developing an efficient analytical methodology for assessing the vulnerability of typical seaside low-code reinforced concrete buildings subjected to tsunami. Tsunami loading is taken into account based on FEMA (2008) recommendations for gradually increasing tsunami inundation depths. Tsunami nonlinear static analyses are performed and appropriate tsunami capacity curves are derived. Appropriate structure-specific limit states are defined on the tsunami capacity curves in terms of threshold values of material strain. For the complete damage state, shear failure is also considered, since the collapse of structures may be caused by the occurrence of either a flexural or shear failure in structural components. The proposed methodology results to the development of log-normally distributed fragility curves for different structural damage states, as a function of the inundation depth, considering different sources of uncertainty. It is shown that the height of the structure represents a key factor affecting the vulnerability assessment. The proposed methodology and the derived fragility curves could be used within a quantitative risk assessment framework to assess the structural damages of typical low-code RC buildings in South Europe subjected to tsunami loading.

Keywords: Tsunami fragility curves; Inundation depth; Structural damage states; Shear failure; RC structures

1. INTRODUCTION

Tsunami risk assessment has got increased interest the last few years due to the occurrence of recent devastating tsunami events (e.g. Indian Ocean 2004, Chile 2010 and Japan after the 2011 Great East earthquake) that affected coastal areas around the world causing significant losses to buildings and infrastructure (e.g. Koshimura et al. 2009; Mas et al. 2012; Suppasri et al. 2013). Also, probabilistic approaches to tsunami hazard are now becoming a standard (e.g. Geist and Lynett 2014), and they are now being applied consistently at the global (http://www.globaltsunamimodel.org) and regional scales (http://www.tsumaps-neam.eu/), and at the very local scale (Lorito et al. 2015), including also a full treatment of aleatory and epistemic uncertainty (Selva et al. 2016). However, only a limited number of tools to estimate the potential impacts of tsunami are available until now.

Considering the tsunami impact, Dalrymple & Kriebel (2005); Stansfield (2005); Ghobarah et al. (2006) focused their studies on qualitative damage analysis, while Rossetto et al. (2007) and Kaplan et al. (2009) categorized building damage using field survey data from the 2004 Indian Ocean tsunami. Existing tsunami fragility functions for buildings and infrastructures exposed to tsunami hazard are principally based on empirical data (hazard-damage relationships from previous tsunami events) and/or expert judgment (Peiris 2006; Leone et al. 2011; Reese et al. 2011; Mas et al. 2012; Suppasri et al. 2015), and as such there are limitations in their general application as they are highly specific to a particular seismo-tectonic environment. In addition, the development of analytical tsunami fragility

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functions is more limited and most of them are referring to the collapse damage state (Park et al. 2012; Nanayakkara and Dias 2016; Petrone et al. 2017).

To bridge the gap, the objective of this study is to develop an efficient analytical methodology for assessing the response and finally the vulnerability of typical seaside low-code reinforced concrete (RC) buildings subjected to tsunami. A comprehensive set of numerical analyses is performed considering statically applied tsunami loads for gradually increasing tsunami inundation depths according to FEMA (2008) recommendations. In particular, the structures are subjected to buoyant and hydrodynamic forces combined with forces due to debris, all of which constitute tsunami load effects. To minimize the uncertainties related to the definition of damage limit states, tsunami nonlinear static analyses are performed and appropriate tsunami capacity curves are derived. Structural limit states are defined on the tsunami capacity curves in terms of threshold values of material strain. Four damage states are proposed in this study associated with none to slight, moderate, extensive and complete structural damage of the structure. The final goal of the study is the derivation of log-normally distributed fragility curves, which describe the probability of exceeding a certain limit state of the structure, versus the inundation depth, and explicitly account for uncertainties in demand and capacity. For the development of fragility curves, an appropriate relationship between the numerically calculated material strain and the gradually increasing inundation depths is established through nonlinear regression analysis. In addition, considering that the selected structures are designed according to low-level seismic code provisions, especially under the effect of tsunami, they may fail under element shear. Thus, an investigation is also conducted to assess whether failure due to shear or bending prevails. In the former case, the fragility curves for the complete damage state are re-derived considering local shear failure. The derived numerical fragility curves are finally compared with a few available empirical ones as well as the results of a relevant numerical study. The proposed fragility curves could be used in quantitative risk assessment studies to assess the vulnerability of typical low-code RC buildings subjected to tsunami loading.

2. METHODOLOGY

The proposed methodology is applicable for the vulnerability assessment of low-code RC frame buildings subjected to tsunami forces. It involves a comprehensive set of nonlinear parametric numerical computations and adequate statistical analysis. The layout of the proposed methodology is illustrated in Figure 1.

![Flowchart of the proposed methodology](image)

In the proposed vulnerability assessment method, low-code RC frame buildings were considered to
apply the proposed method. Nonlinear constitutive models were used to simulate the behaviour of materials since cracking and irreversible deformations are normally expected to govern the structure’s response. Fixed-base conditions were assumed for all models used in the analysis. It is also worth noting that in the proposed method the vulnerability was assessed only for the effect of the tsunami forces, assuming that the building’s structural members have not sustained any possible initial damage (e.g. in terms of stiffness and strength degradation) due to ground shaking.

Tsunami loading was determined based on FEMA (2008) recommendations and proper engineering judgment. In particular, each structure was subjected to buoyant and hydrodynamic forces combined with forces due to debris, all of which constitute tsunami load effects. The computed forces were then directly applied as input time-variant static loads to an appropriate nonlinear structural model for gradually increasing inundation depths and the structure’s response in terms of material strain (i.e. the engineering demand parameter, EDP) for the statically applied tsunami loads was estimated. Subsequently, appropriate limit damage states were defined to account for the flexural structural failure in terms of threshold values of the material strain of the structure, based on nonlinear static analyses, engineering judgment and the literature (e.g. Crowley et al. 2004; Fotopoulou & Pitilakis 2013). Four damage states were proposed in this study associated with none to slight, moderate, extensive and complete structural damage of the structure. Except for flexural failure, failure due to shear (the attainment of shear capacity in a structural member) was also considered and the structure’s demand in terms of shear forces (i.e. EDP) was estimated. One damage state was defined in the latter case describing the complete shear failure of the structures.

The vulnerability was assessed through probabilistic fragility curves, which describe the probability of exceeding each limit state. The inundation depth is used as intensity measure (IM) that adequately correlates with structural damage. Two components of uncertainty were taken into account in the analysis with respect to the structural capacity (defined empirically) and the demand (defined analytically in terms of material strain and shear forces). For the development of fragility curves, an appropriate relationship between the numerically calculated material strain or shear force (i.e. the EDP) and the gradually increasing inundation depths (i.e. the IM) was established through nonlinear regression analysis. Log-normally distributed fragility curves were finally derived for the selected buildings subjected to tsunami forces for the different damage states.

3. NUMERICAL ANALYSIS

3.1 Selection of the structural typologies

Three moment resisting frame (MRF) RC buildings (without masonry infills) of various heights were selected as reference structures (Kappos et al. 2006) to illustrate the proposed vulnerability assessment methodology. They have been designed with a low seismic code according to the 1959 Greek seismic regulations (‘Royal Decree’ of 1959) in which the ductility and the dynamic features of the structures are neglected. The first building typology is a two-storey three-bay frame model that is representative of low-rise (1-3 stories) MRF RC buildings. The second one is a four-storey three-bay frame structure that represents mid-rise (4-7 stories) MRF RC buildings, while the third one is a nine-storey three-bay frame model that is considered typical of high-rise (8+ stories) MRF RC buildings. In all cases the height of the 1st storey is 4.5m and that of the upper storeys is 3.0m. Figure 2 illustrates representative cross-sections and the floor plan of the studied RC building typologies (personal communication Kappos A. and Panagopoulos G.).

3.2 Structural modelling

Two-dimensional numerical simulations of the selected building typologies were conducted using the fibre-based finite element code SeismoStruct v7.0 (SeismoSoft 2015), which is widely and successfully used in structural earthquake engineering. Inelastic force-based formulations were implemented for the nonlinear beam-column frame element modelling. Distributed material inelasticity was applied based on the fibre approach to represent the cross-sectional behaviour (Neuenhofer & Filippou 1997). Each fibre was associated with a uniaxial stress-strain relationship and
the sectional stress-strain state of the beam-column elements was obtained through the integration of the nonlinear uniaxial stress-strain response of the individual fibres into which the section was subdivided. In the present analysis, the frame sections were discretized into 300 fibres. The concrete fibres were modelled based on the uniaxial nonlinear model proposed by Mander et al. (1988), assuming a constant confining pressure for the confined concrete core fibres throughout the entire stress-strain range. Five model calibrating parameters were defined to fully describe the mechanical characteristics of the material: the compressive strength ($f_c$), the tensile strength ($f_t$), the modulus of elasticity ($E_c$), the strain at peak stress ($\varepsilon_c$) and the specific weight ($\gamma$). The compressive strength of the concrete and modulus of elasticity were set equal to $f_c = 14$ MPa and $E_c = 1.7586 \times 10^4$ MPa respectively. The strain at peak stress $\varepsilon_c$, that is the strain corresponding to the point of peak compressive stress ($f_c$) was taken equal to $\varepsilon_c = 0.002$. A zero tensile strength of the concrete material ($f_t = 0$ MPa) was assumed. For the reinforcement, a uniaxial bilinear stress-strain model with kinematic strain hardening was utilized. Five model calibrating parameters were again defined including the modulus of elasticity ($E_s$), the yield strength ($f_y$), the strain hardening parameter ($\mu$), the fracture/buckling strain ($\varepsilon_{ult}$) and the specific weight ($\gamma$). The modulus of elasticity and the yield strength were taken equal to $E_s = 2.0 \times 10^5$ MPa and $f_y = 400$ MPa respectively. The strain hardening parameter ($\mu$) defined as the ratio between the post-yield stiffness and the initial elastic stiffness of the material was assumed equal to $\mu = 0.01$. Finally, the strain at which fracture or buckling occurs ($\varepsilon_{ult}$) was taken equal to 0.1. The total mass was applied as uniformly distributed along the beams (as distributed loads) and columns (by assigning the specific weight of steel and concrete material).

![Figure 2](image-url)

Figure 2. Cross-sections of the (a) 2-storey, (b) 4-storey and (c) 9-storey RC MRF buildings designed with the 1959 Greek seismic code and (d) plan view of the three MRFs

Nonlinear static time-history analyses were performed for all models while static analyses were
carried out before the onset of tsunami nonlinear analysis to account for the gravity forces. Tsunami forces calculated according to FEMA (2008) were statically imposed on the structures at the location of each load’s resultant depending on the inundation depth (h). In particular, the hydrodynamic (F_{d}) and impulsive (F_{s}) forces were applied at h/2 from the base of the structure, while the debris impact force (F_{i}) was applied at h from the base of the structure. Buoyant forces (F_{b}) were applied at the base of the RC buildings. As noted previously the uplift forces (F_{u}) were applicable only in the cases where the inundation depth exceeds the height of the first floor (i.e. for the 4-storey and 9-storey MRFs). The amplitude of the tsunami forces increases with increasing inundation depth. All loads were applied in proportion to each other assuming linearly increasing time-variant loads with a constant time step up to the (maximum) calculated values. The analyses were performed for different levels of inundation depth (at least 20 levels), varying from very small values (e.g. h=0.5m), which result to negligible structural damage to large ones (e.g. h=6m) which may lead to significant structural damages and potential collapse. All the combinations of tsunami loading for the different analysed building types are presented in Figure 3.

Figure 3. Tsunami loading for the a) 2-storey, b) 4-storey and d) 9-storey RC MRFs, where d is lower than the height of the first floor and for the c) 4-storey and e) 9-storey RC MRFs, where d is higher than the height of the first floor (uplift forces are applicable). F_{b}, F_{d}, F_{s}, F_{i} and F_{u} are buoyant, hydrodynamic, impulsive, debris impact and uplift tsunami-induced force components respectively.
4. FRAGILITY CURVES

4.1 Definition of limit states

The definition of realistic limit damage states is of paramount importance for the construction of fragility curves. In the first phase of this study, the occurrence of shear failure in the elements was neglected, assuming that the lateral resistance is controlled by the flexural failure in the beams or columns. Four limit states (LS1, LS2, LS3 and LS4) were defined in terms of limit values of steel and concrete material strains based on nonlinear static analyses (tsunami time history analyses) for the various structure typologies and engineering judgment (e.g. NIBS 2004; Crowley et al. 2004; Fotopoulou and Pitilakis 2013).

In order to minimize the uncertainties associated with the selection of the appropriate damage state limits, nonlinear static analyses were performed for the different structures to define structure-specific limit state values (in terms of strains) for each damage state. Figure 4 illustrates the definition of limit states on the corresponding tsunami capacity curves derived from the tsunami nonlinear static analyses for the various structure typologies. The limit state values finally adopted are presented in Table 1. It is noted that the tsunami capacity curves were not extracted from a single nonlinear static analysis as for a seismic capacity curve (derived from a pushover analysis) but from the total number of the tsunami nonlinear static time history analyses. This is done considering that the location and amplitude of the applied tsunami forces changes as a function of the inundation depth. It is seen that for all analysis cases, steel strain (εs) gives more critical results. Hence, hereafter, the proposed limit damage states are defined in terms of steel bar strain. In particular, the first limit state is specified as steel bar yielding while for the rest limit states, mean values of post-yield limit strains for steel reinforcement are suggested.

![Figure 4](image_url)

Figure 4. Definition of limit states on tsunami capacity curves for the a) 2-storey, b) 4-storey and c) 9-storey RC MRF buildings subjected to tsunami
### Table 1. Definition of limit states for the RC MRFs.

<table>
<thead>
<tr>
<th>Limit states</th>
<th>Steel strain ($\varepsilon_s$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Limit state 1</td>
<td>0.002</td>
</tr>
<tr>
<td>Limit state 2</td>
<td>0.0125</td>
</tr>
<tr>
<td>Limit state 3</td>
<td>0.025</td>
</tr>
<tr>
<td>Limit state 4</td>
<td>0.045</td>
</tr>
</tbody>
</table>

Failure due to shear was also considered in the second phase of the study, since the collapse of structures may be caused by the occurrence of shear failure. Shear failure is identified as the attainment of shear capacity in a member calculated according to Annex A of Eurocode 8- part 3 (CEN 2005). In particular, the demand on the buildings columns, in terms of shear forces, is evaluated using the results of the nonlinear static time-history analysis and compared to the shear capacity of the columns. One damage state was assigned describing the complete shear failure of the structures. Complete shear failure is assumed when at least 50% of the columns in a storey reach their shear capacity (Kappos et al. 2006; Celarec & Dolsek 2013). For the selected buildings, the first shear failure is always observed at the 1st storey (left) column (Col 1-1), where tsunami forces are applied.

### 4.2 Generation of fragility curves

Fragility curves describe the probability of exceeding predefined damage states (DS) under a tsunami event for given levels of intensity expressed in this study in terms of inundation depth. The results of the nonlinear numerical analysis (inundation depth - steel strain values) were used to derive fragility curves expressed as two-parameter lognormal distribution functions. Equation 1 gives the cumulative probability of exceeding a DS conditioned on a measure of the tsunami intensity $IM$:

$$P[DS_i/IM]=\Phi\left(\frac{\ln(IM) - \ln(IM_i)}{\beta}\right)$$

where $\Phi$ is the standard normal cumulative distribution function, $IM$ is the intensity measure of the tsunami expressed in terms of inundation depth (in units of m), $\ln(IM_i)$ and $\beta$ are the median values (in units of m) and log-standard deviations respectively of the building fragilities for each damage state $i$ and $DS_i$ is the damage state. The median values of inundation depth corresponding to the prescribed damage states were determined based on a regression analysis of the nonlinear static analysis results (inundation depth - steel strain pairs) for each structural model. More specifically, a second order polynomial fit of the logarithms of the inundation depth - steel strain data, which minimizes the regression residuals, was adopted in all cases. Figure 5 shows indicatively the derived inundation depth – steel strain relationships for the 4-storey RC MRF building.
The uncertainties were taken into account through the log-standard deviation parameter $\beta$, which describes the total dispersion related to each fragility curve. The higher its value, the higher the slope of the fragility curve and consequently the greater the dispersion. It contains two components of uncertainty associated with the capacity ($\beta_C$) of each structural type and the demand ($\beta_D$). The log-standard deviation value in the definition of the capacity was assumed to be equal to 0.3 for low-code buildings (NIBS 2004). The uncertainty in the demand was considered by calculating the dispersion of the logarithms of inundation depth - steel strain simulated data with respect to the regression fit (Cornell et al. 2002; Fotopoulou & Pitilakis 2013). Under the assumption that these two log-standard deviation components are statistically independent, the total log-standard deviation was estimated as the root of the sum of the squares of the component dispersions. The computed log-standard deviation $\beta$ values of the curves vary from 0.33 to 0.43 for the studied structural models. Figure 6 illustrates the computed sets of fragility curves for the RC MRF buildings with their log-normally distributed fragility parameters (median m and log-standard deviation beta) in terms of inundation depth. An important observation for the studied typical low code MRF RC buildings is that the total height of the structure may considerably influence its vulnerability to tsunami forces. In particular, the high-rise RC MRF buildings have lower vulnerability compared to the low-rise ones.

Figure 6. Fragility curves for the a) 2-storey, b) 4-storey and c) 9-storey RC MRF buildings subjected to tsunami forces

For the cases where at least 50% of the columns in a storey reach their shear capacity, new fragility curves for the complete damage state due to shear were derived. Figure 7 presents the comparison between the new fragility curves for the complete damage state due to shear failure (SF) with the corresponding ones due to flexural failure (FF). It is shown that shear failure is the prevailing failure mechanism of the structure for all the RC MRF buildings, regardless of their height.
A preliminary comparison was also made between the proposed tsunami fragility curves and the few available empirical ones of Suppasri et al. (2013) obtained using field survey data from the 2011 Great East Japan tsunami for RC structures. Unfortunately the typology of RC buildings in Japan and Greece (Europe as well) is not that same and the definition of damage states is also different. However if we will make the comparison for the complete damage state, where the definition of damage states is comparable, the comparison between the numerical and empirical curves is rather good (Figure 8). In particular, the proposed fragility curve for complete damage state of the 2-storey RC MRF building stands between the empirical ones for complete and collapse damage states.

The differences can be attributed to various reasons mainly methodological but also to the fact that the empirical fragility curves, carefully selected to represent as closely as possible our typologies, were constructed based on hazard-damage relationships from previous tsunami events and expert judgment and they are highly specific to a particular tsunami and structural characteristics. In addition, the proposed fragility curves refer to low-code RC buildings in contrast to the empirical ones that include RC buildings of different levels of design codes. Moreover, the empirical curves were derived based on damage data from various RC building typologies while in our case representative RC MRF typologies were studied. Thus the empirical curves are characterized by a higher level of uncertainty indicated by the higher slope of the curves. Therefore, only preliminary comparisons can be made.

The numerical tsunami fragility curves for the 2-storey RC MRF building subjected to tsunami forces were finally compared with the results of a relevant numerical study conducted by Foytong et al. (2015). They analyzed a generic one-storey building to capture responses under tsunami forces. It was shown that the longitudinal reinforcement in the middle column yielded at an inundation depth of 1.8 m, while at a tsunami inundation depth of 2.4 m, the lateral resistance increased and controlled by the shear failure. The median values of inundation depth, proposed in our study, corresponding to the prescribed damage states associated with none to slight, moderate, extensive and complete flexural structural damage of the 2-storey RC MRF building are equal to 1.85m, 2.38m, 2.56m, and 2.71m.
respectively. In addition, the median of the proposed fragility curve for the complete damage state due to shear failure of the 2-storey RC MRF building is equal to 2.40m. These values are close to the key-point results (where the behaviour of the structure changes) derived by Foytong et al. (2005), also enhancing the reliability of the proposed curves.

![Graph showing comparison of fragility curves](image)

Figure 8. Comparison of the numerical tsunami fragility curves for the 4-storey RC MRF building with the empirical ones of Suppasri et al. (2013) for RC-structures

5. CONCLUSIONS

Within the framework of this study, analytical fragility functions were developed for low-code MRF RC buildings of various heights subjected to tsunami forces. Parametric nonlinear static time-history analyses were performed for each building typology. Statically applied tsunami loads were determined according to FEMA (2008) recommendations imposed at an appropriate nonlinear structural model for gradually increasing inundation depths. For the complete damage state, flexural or shear failure of the RC structures was considered. Probabilistic fragility curves for different structural damage states were finally derived as a function of the inundation depth.

It is shown that the structural performance and vulnerability of low-code RC MRF buildings may be considerably influenced by the height of the structure, with the high-rise structures presenting lower vulnerability compared to the low-rise ones. It is also observed that shear failure was the dominant damage mechanism of the structure for all the MRF RC buildings subjected to tsunami forces, regardless of their height. Preliminary comparisons made between the proposed fragility curves and a few available empirical ones as well as the results of a relevant numerical study are generally satisfactory, enhancing the reliability of the proposed fragility curves.

The proposed methodology and the derived fragility curves could be used within a quantitative risk assessment framework to assess the structural damages of typical low-code RC buildings in Greece and in Southern Europe in general exposed to tsunami forces. Given the significance of this study’s findings, future research should address fragility taking into account additional structural configurations including soil nonlinearity and soil-structure interaction (SSI), as well as other parameters involved, which have not been considered in this analysis like scouring effects and the structural deterioration effects from prior ground shaking or even aging effects.

6. ACKNOWLEDGMENTS

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7. REFERENCES


