VULNERABILITY ASSESSMENT OF RC BUILDINGS AND WAREHOUSES DUE TO LIQUEFACTION DISPLACEMENTS

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ABSTRACT

Lessons learnt from recent strong seismic events have demonstrated that even moderate levels of earthquake intensity can cause liquefaction, potentially leading to induced soil settlements and lateral spreading that may result to severe physical damages and important economic and societal losses. However, the published literature in the evaluation of the induced physical damages to building structures is generally inadequate. The objective of this study is to develop an efficient analytical methodology for assessing the vulnerability of typical port structures, i.e. low-code reinforced concrete (RC) buildings and warehouses, subjected to liquefaction-induced displacements. Specifically, a series of nonlinear static time-history parametric analyses of representative structural typologies are carried out. The input to the structural model are various sets of displacement loads imposed quasi-statically directly at its supports (shallow foundations), to simulate the differential displacement demand of the structure impact by liquefaction. The building’s response is then assessed using appropriate engineering demand parameters (EDPs) that are statistically correlated with predefined damage states to construct the fragility functions. Log-normally distributed fragility curves are finally derived for different damage states using nonlinear regression analysis, for the different structural typologies representative of typical seaport structures. They could be used in quantitative risk assessment studies to assess the vulnerability of typical low-code RC buildings and warehouses exposed to liquefaction hazard along European-Mediterranean and other regions of similar facilities worldwide.

Keywords: Liquefaction; Differential ground displacements; Building damage; Fragility curves, Port structures

1. INTRODUCTION

Experience gained from recent strong seismic events revealed the need for the estimation of the effects of liquefaction-induced displacements especially on seaport structures. By far, the most widespread source of seismic damage to port structures and infrastructure is the liquefaction of loose, saturated soils that often prevail at coastal areas (Pitilakis et al. 2014). Occurrence of liquefaction phenomena can cause severe physical damages and important economic and societal losses as seen recently after the earthquakes in Kocaeli (Turkey) 1999; Maule (Chile) 2010; Christchurch (New Zealand) 2011; Tohoku (Japan) 2011 and Emilia-Romagna (Northern Italy) 2012 (e.g. Bray and Dashti 2014; Lai et al. 2015). However, while methodologies for the assessment of liquefaction potential and the resulting ground deformation have been the focus of research for many years, when it comes to determining the impact these deformations will have on existing structures and the subsequent liquefaction-induced structural damages, the published literature is rather limited (e.g. Bird et al. 2006).

To bridge the gap, the objective of this study is to develop an efficient analytical methodology for assessing the vulnerability of typical port structures, i.e. low-code reinforced concrete (RC) buildings of various heights and warehouses with surface foundations, which are representative critical structures of Thessaloniki port, subjected to liquefaction-induced displacements (settlements and lateral spreading). The proposed methodology is based on an uncoupled approach, where the soil and

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the structure are studied separately, and the soil liquefaction-induced permanent ground displacements are imposed to a finite element model of the structure. In particular, a numerical parametric investigation of the selected structural typologies is performed considering different combinations of statically applied differential displacements at the foundation level. The building's response is then assessed using appropriate engineering demand parameters (EDPs) that are statistically correlated with predefined damage states to construct the fragility functions. Different damage levels are defined to describe e.g. slight, moderate, extensive and complete damage states of the port structures. The outcome of the proposed vulnerability assessment method is the derivation of log-normally distributed fragility curves, for the different structural typologies, as a function of the liquefaction-induced differential displacement. They could be used in quantitative risk assessment studies to assess the vulnerability of typical low-code RC buildings and warehouses exposed to liquefaction hazard along South Europe and other regions of similar facilities worldwide.

2. METHODOLOGY

The proposed methodology is applicable for the vulnerability assessment of RC frame buildings and steel-frame warehouses subjected to liquefaction-induced differential displacements. It involves a comprehensive set of nonlinear numerical computations and adequate statistical analysis. The schematic framework of the proposed methodology is illustrated in Figure 1.

![Figure 1. Flowchart of the proposed framework of the methodology](image)

In the proposed vulnerability assessment method, several building typologies (i.e. low-code frame RC buildings and a typical steel-frame warehouse) representative of Thessaloniki’s port critical structures were considered to apply the proposed method. Nonlinear constitutive models were used to govern the building’s response. For assessing building response to liquefaction-induced differential displacement, the foundation system is also of importance (e.g. Bird et al. 2006). The structural response and damage to the differential ground deformation induced due to soil liquefaction depends primarily on the foundation type (e.g., shallow flexible foundations, shallow stiff/rigid foundations and deep foundations). In this study, the focus is on buildings with shallow flexible foundations (i.e. isolated footings for the RC buildings and strip footing for the steel-frame warehouse) which allow columns moving differentially and hence permitting the computation of structural deformation demand and
fragility using a numerical and/or analytical approach. In this case, it was assumed that failure takes place on the building’s structural elements and the load bearing capacity of the foundation is adequately designed to resist the liquefaction induced deformation. It is also worth noting that in the proposed method the vulnerability was assessed only for the effect of the liquefaction-induced differential displacement, 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. The expected differential settlements or differential lateral movements, which commonly occur due to the heterogeneity in soil stiffness and stratigraphy, become the major cause of damage to buildings (Bird et al. 2005, 2006), as the absolute (uniform) displacements on their own are often insufficient to describe the main damage patterns of the structures. Thus, one may start with the quantification of the absolute free field ground deformations using one of the available empirical or numerical methodologies and continue with the evaluation of the corresponding differential movements beneath the foundation of a building. The estimation of the differential ground movements entails an even greater uncertainty than the estimation of absolute movements. This is principally due to the lack of sufficient and reliable geotechnical data (e.g. Bird et al. 2006). Thus, unless an extensive geotechnical investigation at the building area is to be performed, the distribution of differential displacements over the footprint of the building in terms of a percentage of the absolute ones can be reasonably assumed (e.g. California Geological Survey 2008).

Nonlinear static time-history analyses of the considered building typologies were performed using an appropriate finite element code to assess the structure’s response and fragility due to the liquefaction-induced differential displacement. Different combinations of differential displacements were imposed as quasi-static loads at the structure’s supports to simulate the differential displacement demand of the structure impact by liquefaction, considering only the vertical component of the differential displacement (i.e. to represent settlements in level ground) or taking also into account the horizontal differential displacement with a vertical component (i.e. to represent lateral spreading in sloping ground or in the vicinity of a free face). In the latter case, two different scenarios were analysed: (i) the horizontal component is equal to the vertical one and (ii) the horizontal component is equal to twice the vertical one. For each of the previous differential displacement profiles the maximum differential displacement was assumed to be applied either at the middle or edge column at the foundation level while a triangular distribution of the differential displacement is considered in all cases.

Different failure mechanisms of the considered RC structural models were examined including a global flexural failure of the reinforced concrete frames and a local shear failure (attainment of shear capacity) of the columns. In the former case the engineering demand parameter (EDP) is expressed in terms of maximum values of steel and concrete material strain. Four damage states were defined describing slight, moderate, extensive and complete structural damage of the buildings based on the existing literature (e.g. Crowley et al. 2004, Bird et al. 2005, 2006, Fotopoulou and Pitilakis 2017) and expert judgement. In the latter case, the computed shear forces in the columns were used as EDP. One damage state was considered in this case describing complete shear failure defined as being attained when at least 50% of all columns in a storey have reached their shear strength (e.g. Kappos et al. 2006, Celarac and Dolsek 2013).

The vulnerability was assessed through probabilistic fragility functions, which describe the probability of exceeding each limit state under a range of liquefaction-induced differential displacements. Log-normally distributed fragility curves for the selected buildings subjected to settlements and lateral spreading differential deformation were constructed for the different damage states using nonlinear regression analysis, considering the most adverse failure mechanism (due to flexure or shear). The differential displacement is used as intensity measure (IM) that adequately correlates with structural deformation and damage. Various sources of uncertainty were taken into account in the analysis with respect to the structural capacity and the damage states (defined empirically) and the demand (defined analytically).
3. NUMERICAL ANALYSIS

3.1 Selection of the structural typologies

The Port of Thessaloniki, one of the largest transit-trade ports in Greece and the Aegean Sea basin, is a crucial point for supply chains, import-export trade and transportation. The interruption of its port structures functionality can have severe effects on the economy and on the social and environmental growth of the broader area of interest. Critical buildings of Thessaloniki’s port mainly consist of low- and pre- code RC buildings and steel-frame warehouses. The SYNER-G (www.syner-g.eu) taxonomy was used to describe the different RC building typologies. In particular, three moment resisting frame (MRF) RC buildings (without masonry infills) of various heights were selected as reference structures (Kappos et al. 2006). 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. It is reasonably assumed that the selected poorly designed buildings are likely to have poorly designed foundations. Thus, a flexible foundation system (i.e. spread footings) is considered allowing columns move differentially. In this case, the various modes of differential deformation can produce structural damage (e.g., in terms of cracks) to the building members (Bird et al. 2005; 2006). The warehouses, which are basically industrial steel-frame structures, constitute a category of their own and are founded on strip footings. Figure 2 illustrates representative cross-sections of the studied RC building typologies (Kappos et al. 2006) and the typical steel-frame warehouse provided by the Thessaloniki Port authorities and reproduced by AUTH.

3.2 Structural modelling

The selected typologies were modelled as two-dimensional (2D) structures, using the fibre-based finite element code Seismostruct 2016 (SeismoSoft 2016), which is widely and successfully used in several structural earthquake engineering applications either for research or professional engineering purposes. The code is capable of predicting the large displacement behaviour of space frames subjected to static or dynamic loads allowing both geometric nonlinearities and material inelasticity to be captured. 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 is obtained through the integration of the nonlinear uniaxial stress-strain response of the individual fibres into which the section is subdivided. In this study, the frame sections were discretized into 300 fibres. For the RC structures, inelastic displacement-based (DB) formulations were implemented for the nonlinear beam-column frame element modelling, that are more capable to avoid strain localization effects due to softening behaviour (Calabrese et al. 2010) which are expected to govern the response of the structural elements. More specifically, in the DB finite element formulation using nonlinear models, a refined discretisation of the structural elements (typically 4-5 sub-elements with two integration points each per sub-element) is required in order to accept the assumption of a linear curvature field inside each of the sub-domains. Thus, the beam-column frame elements were subdivided into five segments with the first and last sub-element’s length being equal to twice the assumed plastic hinge dimension (Calabrese et al. 2010). The latter was estimated based on an empirical relationship (Paulay and Priestley 1992). Considering that the maximum inelastic deformation has generally shown to be concentrated at the end sub-elements of the beam-column structural members, the subdivision of the remaining part of the element is of less importance, provided that the element has a softening response as in our case. 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 $E_c = 1.7586 \cdot 10^5$ 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 $E_s = 2.0 \cdot 10^5$ 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. Cross-sections of the (a) 2-storey, (b) 4-storey, (c) 9-storey MRF RC buildings, designed by the 1959 Greek seismic code and the (d) warehouse
Regarding the warehouse modelling, the uniaxial bilinear model with kinematic strain-hardening was employed for the steel material ($E_s = 2.1 \cdot 10^5\,\text{MPa}; \, f_y = 235\,\text{MPa}; \, \mu = 0.01$). Columns were modelled using force-based (FB) inelastic frame elements with 5 integration sections, as there is no softening behaviour of steel members, while trusses were modelled through truss elements (truss). Inelastic FB formulations were also implemented for the nonlinear RC connecting beam frame element modelling.

The number of fibres used in section equilibrium computations in both cases was set to 300. The masses were applied as distributed along columns and beams (by assigning the specific weight of steel material) plus concentrated vertical loads on joints due to the existence of trusses on the normal direction (Figure 3).

Nonlinear static analyses of the studied building typologies were performed to account for the response to the seismically induced liquefaction differential displacement. Different sets of quasi-static displacement loads were imposed directly at the supports (shallow foundations), to simulate the differential displacement demand of the structure impact by liquefaction. More specifically, for each structure, a series of approximately 100 nonlinear static analyses was carried out for gradually increasing level of displacement (to cover the whole range from elasticity to structural collapse) imposed at the foundation level considering different differential displacement profiles, i.e. settlements with the maximum differential displacement applied at the edge or middle column, lateral spreading with the horizontal component of the differential displacement ($h$) being equal to or twice the vertical ($v$) one ($h=v$ or $h=2v$) and the maximum differential displacement vector applied at the edge or middle column.

The differential displacement demand at the foundation level was assumed to follow a triangular distribution in all analysed cases. Figure 3 presents indicatively the imposed displacement loading pattern considered for the nonlinear static analysis of the RC 2-storey structural model and the warehouse. The fragility curves were derived for the most adverse cases where the maximum differential displacement (settlement or lateral spreading) was applied at the middle or edge column.

![Figure 3](image-url)
4. FRAGILITY CURVES

4.1 Definition of limit states

The definition of realistic limit damage states is of paramount importance for the evaluation of fragility curve parameters. In the first phase of this study, the occurrence of shear failure in the RC elements was neglected, assuming that the lateral resistance was 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 the available literature (e.g. Crowley et al. 2004; Fotopoulou and Pitilakis 2013) for the RC MRFs and on the nonlinear static analyses and engineering judgment for the warehouse (Karafagka et al. 2016). They describe the exceedance of minor, moderate, extensive and complete damage of the structures. According to NIBS (2004), “Steel Light Frames” structures are mostly single storey structures combining rod-braced frames in one direction and moment frames in the other. Due to the repetitive nature of the structural systems, the type of damage to structural members is expected to be rather uniform throughout the structure. Consequently, warehouses were considered as “Steel Light Frames” structures. A qualitative description of each damage state for the RC and the steel light frames is provided in Crowley et al. (2004) and NIBS (2004) respectively. The limit state values finally adopted are presented in Table 1. In order to minimize the uncertainties associated with the selection of the appropriate damage state limits for the warehouse, nonlinear static analyses were performed to define structure-specific limit state values for each damage state. Figure 4 illustrates the definition of limit states on the corresponding liquefaction capacity curves derived from the nonlinear static analyses for the warehouse subjected to differential settlements and lateral spreading due to liquefaction. In particular, the first limit state is specified as steel bar yielding while mean values of post-yield limit strains for steel reinforcement are suggested for the rest limit states. It is noted that the liquefaction 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 nonlinear static analyses conducted.

<table>
<thead>
<tr>
<th>Limit states</th>
<th>Steel strain (ε_s)</th>
<th>MRF bare frames</th>
<th>Warehouse</th>
</tr>
</thead>
<tbody>
<tr>
<td>Limit state 1</td>
<td>0.002</td>
<td>0.00112</td>
<td></td>
</tr>
<tr>
<td>Limit state 2</td>
<td>0.0125</td>
<td>0.0125</td>
<td></td>
</tr>
<tr>
<td>Limit state 3</td>
<td>0.025</td>
<td>0.03</td>
<td></td>
</tr>
<tr>
<td>Limit state 4</td>
<td>0.045</td>
<td>0.055</td>
<td></td>
</tr>
</tbody>
</table>

Figure 4. Definition of limit states on liquefaction capacity curves for the warehouse subjected to (a) differential settlements and (b) to lateral spreading due to liquefaction
In the second phase of this study, one damage state was assigned describing the complete shear failure of the RC structures, since shear failure of columns is also likely to occur. Complete shear failure is assumed when at least 50% of the columns in a storey reach their shear capacity (Kappos et al. 2006; Celarec and Dolsek 2013). 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, was evaluated using the results of the nonlinear static time-history analysis and compared to the shear capacity of the columns.

4.2 Generation of fragility curves

Fragility curves describe the probability of exceeding predefined damage states (DS) for given levels of intensity expressed in this study in terms of liquefaction-induced differential displacement. The results of the nonlinear static analysis (differential displacement – maximum material strain and differential displacement – shear forces) were used to derive fragility curves expressed as two-parameter lognormal distribution functions according to Equation 1:

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

where, $\Phi$ is the standard normal cumulative distribution function, IM is the intensity measure given in terms of liquefaction-induced differential displacement, $\bar{IM}$ and $\beta$ are the median and log-standard deviation values respectively of the building fragilities for each damage state $i$ and DS, is the damage state. The median values of differential displacement corresponding to the predefined damage states were determined based on a statistical analysis of the nonlinear static analysis results (differential displacement - maximum strain pairs). In particular, a quadratic fit of the logarithms of the differential displacement- maximum strain simulated data, which minimizes the regression residuals, was adopted. Figure 5 presents a representative plot (in log-log scale) of damage evolution in terms of maximum strain as a function of differential displacement for the 4-storey frame building subjected to differential settlements with the maximum differential displacement applied at the middle column. The corresponding limit values of maximum steel strain defined for each damage state are also shown.

Figure 5. Differential displacement vector- steel strain relationship for the 4-storey frame building subjected to differential settlements with the maximum differential displacement applied at the middle column

The various 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 three components of uncertainty associated with the definition of the limit state value $\beta_{LS}$, the capacity of each structural type $\beta_c$, and the demand $\beta_D$ (NIBS 2004). The uncertainty in the definition of limit states was defined
analyzing the comparison of seismic shear demand and differential settlements. In building being more vulnerable compared to the low-rise MRF frame structures. Also, regarding the warehouse, it is shown that it is by far less vulnerable than the RC MRF frame structures.

In the following, some comparisons with field observations where widespread liquefaction occurred may further increase the validity of the proposed fragility curves. More specifically, after the Christchurch, New Zealand 2010 and 2011 earthquakes, a six-story building founded on isolated spread footings with tie beams and perimeter grade beam, subjected to an overall differential settlement of 25 cm, was considered uneconomical to repair after the 22 February 2011 earthquake (Bray and Dashti 2014). This observation is in accordance to the proposed fragility curves for the mid-rise MRF building, which suggest a median differential settlement of approximately 30 cm for the complete damage state. Also, the proposed fragility curves for the port warehouse were finally compared with the results of geotechnical field investigations for the Port International de Port-au-Prince following the 2010 Haiti earthquake which caused catastrophic ground failures in calcareous-sand artificial fills at the seaport, including liquefaction, lateral spreads and differential settlements. In particular, the port steel-frame warehouse founded on strip footings with width approximately equal to 37 m suffered significant damage but no collapse. According to a detailed survey, the maximum differential settlement in an internal bay of the warehouse was equal to 57.3 cm while the differential lateral displacement equal to 94.9 cm. (Green et al. 2011). These findings are in good agreement with the proposed fragility curves for the warehouse, which suggest extensive damage be expected for a differential displacement vector equal to 110 cm.

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 complete shear failure occurs only for the 2-storey and 9-storey RC frame buildings subjected to lateral spreading. It is also noted that for the studied low-code high-rise building shear failure would be the prevailing damage mechanism of the structure while for the low-rise building the flexural failure is expected to come first. This may be attributed to the P-delta effects that affect the performance of the high-rise MRFs at high deformation levels amplifying the seismic shear demand and therefore resulting to a more fragile failure mechanism (e.g. Fenwick et al. 1992).
Figure 6. Fragility curves for the (a) 2-storey, (b) 4-storey, (c) 9-storey RC frame buildings and (d) the warehouse subjected to differential displacements due to liquefaction.
5. CONCLUSIONS

Within the framework of this study, analytical fragility functions were developed for low-code RC MRF buildings of various heights and a typical steel-frame warehouse with a shallow foundation system, all representative of typical port structures in South Europe subjected to liquefaction-induced differential displacements. Parametric nonlinear static time-history analyses were performed for each building typology. Various differential displacement profiles were assumed including settlements and lateral spreading with the maximum differential displacement imposed quasi-statically at a middle or edge column at the foundation level. 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 differential displacement.

It is shown that the vulnerability of low-code MRF RC buildings and port steel-frame warehouses may be considerably influenced by the existence or not of a horizontal component of the differential displacement (lateral spreading or settlements). In addition, the height of the low-code MRF RC building may significantly affect the structural performance. In general, the high-rise MRF RC buildings are more vulnerable compared to the low- and mid-rise ones, especially for the lateral spreading cases. It is also observed that the warehouse is by far less vulnerable than the RC MRF frame structures. Finally, shear failure was the dominant damage mechanism only for the considered low-code high-rise MRF building subjected to lateral spreading. Preliminary comparisons made between the numerically derived fragility curves and field observations are generally satisfactory considering the important uncertainties involved, enhancing the reliability of the proposed fragility curves. The proposed curves could be used within a quantitative risk assessment framework to assess the vulnerability of typical low-code RC frame buildings and port steel-frame warehouse in Greece and in Southern Europe in general exposed to liquefaction-induced differential displacements. 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 the combined effects of ground shaking and soil liquefaction.

6. ACKNOWLEDGMENTS

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7. REFERENCES


