Article

Analytical Fragility Curves of Pile Foundations with Soil-Structure Interaction (SSI)

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Abstract: Pile foundations is a well-studied technique with many applications and its benefits on structures have been widely studied in the literature. In particular, the mutual effects of pile flexibility and soil deformability may significantly modify the seismic behaviour of superstructures. In order to consider the uncertainties that are connected with these issues, the paper applies the probabilistic-based approach of fragility curves by proposing three limit states based on ductility factor. Non-linear dynamic analyses were performed with OpenSees PL to assess the potentialities of three pile configurations founded on three cohesionless soil with different deformability.

Keywords: soil structure interaction; analytical fragility curves; pile foundation; ductility factor; numerical simulations; Openses

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1. Introduction

The role of Soil Structure Interaction (SSI) is fundamental to define the seismic fragility of pile foundations that depend on the mutual flexibility of the pile and soil and the location and length of the plastic hinges. Inelastic responses and damage mechanisms of laterally loaded piles have been described in the literature (i.e., [1,2]). In particular, [3] describes in detail the development of yielding points along the piles due to lateral seismic forces. Pile flexibility drives the seismic response of entire systems (soil-foundation-structure) as demonstrated in previous earthquakes [4,5] and investigated in several contributions, such as [6]. Since inelastic deformations may occur in inaccessible areas (with no visual inspection), stringent ductility limits need to be defined as prescribed by [7] and the New Zealand provisions for the seismic design of highway bridge foundations [8].

Many contributions have aimed to investigate pile foundation responses under seismic loading conditions considering experimental tests such as [9,10] that performed centrifuge studies or [11] that carried out shaking table tests. In addition, analytical methodologies were proposed such as [12] that developed an exact formulation for a piled raft system resting on elastic Winkler foundations.

Numerical studies were also performed from the first attempts (i.e., [13] that applied the Winkler modelling approach). Several contributions performed numerical simulations of the inelastic deformation and displacement ductility of laterally loaded piles, such as [14,15] that adopted the beam-on-Winkler-foundation model. The authors of [16] applied a kinematic cantilever model and [17] focused on the inelastic seismic response of bridges supported by free-standing extended pile-shafts. The authors of [18,19] performed numerical simulations to assess the displacement ductility capacity of fixed-head piles in cohesive soils. Recent contributions were based on more developed platforms, such as [20–22]. 2D finite elements were performed by [23] that applied the DYNAFLOW platform, [24] that carried out nonlinear plane-strain seismic analyses and more recently by [25] that performed a fully nonlinear 2D FE study with several pile-deck connections. Several 3D finite element models were conducted, such as [26] that investigated the Port of Oakland Berth recorded seismic response and [27] for a push-over seismic evaluation study. In addition, several studies applied Openses such as [28] and the more recent [29]

that assessed the salient characteristics associated with the pile-ground and pile-to-pile interaction mechanisms. More recently, [30] adapted the previous kinematic model [3] to assess the displacement ductility factors for piles foundations in cohesionless soils. Many numerical models (i.e., [31–36]) applied deterministic approaches that may not account for the uncertainties related with modelling of SSI ([37,38]).

In this regards, even if fragility curve is a well-known approach to assess the structural damage of various systems ([39,40], very few contributions have applied this method to assess pile foundations. In particular, [41] applied Opensees to develop fragility curves for a typical wharf structure, [42] derived fragility curves of 3D models of buildings considering (among the others) a pile foundation system with soil-pile interaction modelled through a 2D finite element soil-block. The authors of [43] performed seismic fragility analyses for a case study of sheet-pile wharves in Port of Hualien in Taiwan by applying the FEA software PLAXIS 2D. The recent [44] applied the methodology to pile foundations of unreinforced masonry (URM) buildings in the Groningen region (northern Netherlands) by proposing hysteretic single-degree-of-freedom (SDOF) models by applying the cloud method with linear regression and Conditional Spectrum (CS) for the ground motion selection.

The present paper aims to propose the fragility curves of single-pile schemes founded on different sand layers characterized by increasing soil deformability. Secondly, it is well noted that the mechanism of piles depends on their flexibility in terms of L/D (ratio between pile length and pile diameter) that is fundamental to define the performance of the entire pile-foundation-soil system. Therefore, the paper proposes several case studies of pile foundations characterized by different values of L/D to assess the mutual role of pile flexibility and soil deformability on the entire system (soil-pile-superstructure).

2. Methodology

This paper aims to investigate the performance of three pile configurations with different flexibility ($L/D = 5, 10$ and 20). The behaviours of the piles depend on the mutual behaviour between the piles and the soil, the so-called kinematic interaction that consists of a series of complex non-linear mechanisms that may be activated by an earthquake [45]. Overcoming the previous contributions that considered deterministic approaches, the paper applies analytical fragility curves as a probabilistic-based methodology to express the probability of damage at certain levels of intensities by defining damage states (DS). They represent graphical relationships between the conditional probability of exceeding predefined damage states (DS) for given levels of EDP (engineering demand parameters) and are used to estimate the amount of damage for a particular level of shaking defined with intensity measures (IM).

In particular, for pile foundations, inelastic deformations are due to displacements imposed on the foundation, the mutual stiffness and strength of the pile and soil, and the location and length of the plastic hinges. Therefore, the definition of damage levels is particularly challenging since it depends on the pile flexural yielding. This paper applies the Ductility Factor (DF) proposed by [30] as the reference parameter to define limit states. In particular, the depth at which pile yielding is expected to occur is fundamental to defining the design ductility factor. For example, the seismic design of highway bridge foundations in New Zealand [8] consider several design displacement ductility factors taking 2 m as the reference depth. In this paper, the equivalent elastoplastic yield displacement Δ_y was idealized by considering a bilinear elastoplastic force–displacement response of the pile. Therefore, the ductility factor DF is defined as the ratio of the ultimate displacement (Δ_u) and the elastoplastic yield displacement (Δ_y):

$$DF = \frac{\Delta_u}{\Delta_y} \quad (1)$$

where the ultimate displacement Δ_u was calculated as the maximum lateral displacement for each of the applied input motions and the yield displacement is calculated by considering the expected yield strength of the reinforcement, calculated as:

$$f_{ye} = 1.13f_y \quad (2)$$

where f_y is the steel yield strength.

In particular, Limit State 3 (LS3) was defined for $DF = 2$, proposed herein as a design value to ensure good seismic performance of the pile for soils with friction angle between 30° and 33° [30]. A moderate level (LS2) was proposed here as 50% of the previous value ($DF=1$) and a slight level (LS1) with $DF = 0.30$. The adopted methodology is based on the calculation of linear regressions to define the logarithmic mean (μ) and standard deviation (β) of the Peak Ground Acceleration (PGA), selected as the representative intensity measure (IM).

The seismic scenario is described by the longitudinal component of 100 input motions, selected from the PEER NGA database (<http://peer.berkeley.edu/nga/>) and applied at the base of the models. The motions were divided into 5 bins of 20 motions each with characteristics: moment magnitude (M_w) 6.5–7.2 and closest distance (R) 15–30 km, M_w 6.5–7.2 and R 30–60 km, M_w 5.8–6.5 and R 15–30 km, M_w 5.8–6.5 and R 30–60 km, and M_w 5.8–7.2 and R 0–15 km, as already applied in [46]. Figure 1 shows the cumulative density functions (CDFs) of the 100 motions and for the Peak Ground Acceleration (PGA) selected as the reference intensity measure. In particular, fragility may be defined as the conditional probability of failure for selected parameters in order to account for several sources of uncertainty (i.e., modelling, representation of the demand and capacity, soil parameters). In the proposed study the only random parameter is the seismic demand that is described with PGA. The set of input motions was selected following Eurocode 8 (CEN, 2003, [47]): (1) it consists of at least seven recordings and (2) the average spectral ordinates of the selected record set were matched to the target code-based spectral shape. The set was selected to have significantly dispersed values of intensities to affect the dynamic characteristics of the system and at each acceleration value a , the fragility was represented by a subjective probability density function.

Figure 1. Cumulative distributions for the considered IM: PGA.

3. Finite Elements Models

This paper aims to study single piles as parts of a group of piles (i.e., Figure 2). 100 non-linear dynamic analyses were performed with parallel computation by applying the well-credited platform Opensees PL [48]. Computational time was reasonably controlled (less than 72 h) by considering some assumptions: (1) The 25 m layer was considered homogeneous for each of the three selected soils (see below). (2) Symmetry conditions were considered, allowing to investigate a representative “cell”, following the approach already performed in [28] (Figure 3). (3) Pile deformations were calculated at every points during the analyses but saved only at specific sections along the pile. (4) A bilinear material ($f_y = 420$ MPa, $f'_c = 34.5$ MPa) was used to represent the concrete.

Figure 2. Example of a full 3D mesh.

Figure 3. Cell mesh: (a) 3D view, (b) vertical view, (c) plan view.

The half-mesh of the representative cell of soil was constrained with the periodic-boundary technique, thanks to symmetry conditions (see simplification 2), as shown in [49]. Accounting for the symmetry, left and right boundary nodes were tied together both longitudinally and vertically using the penalty method [50] with 10^{-4} tolerance, as a compromise (more details in [51,52]). The circular piles were modelled with 3D fiber sections, force-based beam-column elements [48]. As mentioned earlier, concrete materials for the core and the cover portions of the cross sections have been modelled with bilinear

relationships to reduce computational time and define yielding conditions. Steel bars were modelled with Steel02 material [50]. Figure 4 shows the moment-curvature response (the same in all directions because of the circular shape) of the pile cross section under the constant axial load P defined in Table 1 (formula 3). The interaction between the pile and the soil was realized with zerolenght elements [50] that connect the soil domain 3D brick elements at the outer nodes of the rigid beam-column links (set normal to the pile longitudinal axis) by allowing the relative soil-pile displacements (slippage) that affects considerably SSI effects mainly under large deformations.

Figure 4. Moment-curvature for: (a) PL5, (b) PL10, (c) PL20.

Table 1. Piles Characteristics.

	D (m)	L (m)	L/D	Ag (m ²)	I (m ⁴)	P(MPa)
PL5	2.0	10.0	5	3.142	0.785	2.03
PL10	1.0	10.0	10	0.785	0.049	8.12
PL20	0.5	10.0	20	0.196	0.003	32.50

The piles were loaded with a vertical axial force defined as the 30% of the axial load capacity, generally considered for normal design of concrete piles [30]:

$$P = 0.30f'_c A_g \quad (3)$$

where: A_g is the gross cross sectional area of the pile; and f'_c the concrete compressive strength (34.5 MPa).

The soil mesh was built up with 3193 nodes and 2640 non-linear “Bbar brick” elements and was calibrated with a convergence study as a compromise between the MESH1 (2139 nodes and 1680 elements) and MESH3 (9331 nodes and 8280 elements), by comparing the longitudinal displacement at the center of the pile at surface level (Figure 5). The meshes were tested by verifying that the accelerations at the surface of the mesh near the lateral boundaries were identical to those simulating free-field conditions.

Figure 5. Convergence Study: longitudinal displacements at the center of the pile at surface level (for mesh1, mesh2 and mesh3).

Every element is built with 20 nodes with three translational Degree of Freedom (DOF), recorded using OpenSees Node Recorder [50] at the corresponding integration points. The number of elements was defined on the bases of the wavelength of the seismic signal and the maximum frequency (15 Hz) above which the spectral content of the input may be considered negligible [53]. Following [54], the characteristics of the input motions (i.e., wavelength and maximum frequency) were used to define the number of elements whose dimensions were increased with the distance to the centre of the piles. Lysmer-Kuhlemeyer (L-K) viscous dashpots [55] were defined at the base boundaries (along the longitudinal direction) to accurately model radiation damping so as to prevent the reflection of the seismic waves back into the soil medium after being incident on the far-off boundaries. Three 25m-homogeneous cohesionless dry (no pore pressure) layers (named STIFF, MEDIUM and SOFT) with different increasing deformability were performed to consider the non-linear effects of the soil, such as amplitude-dependent amplification (or deamplifications), plastic accumulations of ground deformation, and permanent movements (rotations, longitudinal displacements) at the pile level. The PressureDependMultiYield (PDMY) model was applied in order to consider the non-linear mechanisms of hysteretic response and radiation damping based on the multisurface-plasticity theory for frictional cohesionless soils proposed by [56]; more details in [48]. Figure 6 shows the non-linear backbone curves, represented by hyperbolic relations defined by the strain shear modulus and ultimate shear strength, as described in [48]. The representative parameters are shown in Table 2.

Table 2. Soil Parameters (PDMY model).

	Stiff	Medium	Soft
Mass density (kN/m ³)	20	19	17
Shear Modulus (kPa)	1.0×10^5	7.7×10^4	5.8×10^4
Bulk Modulus (kPa)	3.1×10^5	2.2×10^5	1.75×10^5
Friction Angle (°)	37	33	29
Num. Yield Surface	20	20	20

Figure 6. Backbone curves.

4. Results

In this section, the results are shown and discussed. Figures 7–9 show the results of the 900 (100×3 : 100 for each of the selected pile cases and for the 3 soil conditions) non-linear analyses with the considered limit states. Soft soil was shown to be the most detrimental condition for all the models.

The results were applied to build linear regressions (R^2 values in Table 3) and calculate the logarithmic mean (μ) and standard deviation (β) of PGA, necessary to define the probability of exceedance (PE):

$$P[D \geq C_i | \text{PGA}] = \phi\left(\frac{\ln(\text{PGA}) - \mu}{\beta}\right) \quad (4)$$

where: P is the probability of the structural damage (D) exceeding the i-th damage state (C); ϕ is the standard normal cumulative distribution function;

Table 3. R^2 values.

	Stiff	Medium	Soft
PL5	0.875	0.896	0.719
PL10	0.559	0.591	0.547
PL20	0.872	0.871	0.709

Figure 7. PL5: PGA(g) vs. Ductility Factor.

Figure 8. PL10: PGA(g) vs. Ductility Factor.

Figure 9. PL20: PGA(g) vs. Ductility Factor.

4.1. PL10

This section shows the results for PL10 in terms of fragility curves, as defined in the previous section. In particular, Tables 4–6 show μ and β values for the considered soil conditions. It is worth noting that the standard deviation affects the slope of the curves while the mean values increase with soil deformability, shifting the curves toward more vulnerable responses.

Table 4. PL10 STIFF: lognormal parameters.

Parameter	LS1	LS2	LS3
μ	0.173 g	0.180 g	-
β	0.634	0.654	-

Table 5. PL10 MEDIUM: lognormal parameters.

Parameter	LS1	LS2	LS3
μ	0.154 g	0.165 g	0.180 g
β	0.577	0.606	0.640

Table 6. PL10 SOFT: lognormal parameters.

Parameter	LS1	LS2	LS3
μ	0.109 g	0.146 g	0.165 g
β	0.509	0.576	0.609

The role of SSI effects at different intensities for PL 10 is shown in Figures 10–12. When STIFF conditions are considered (Figure 10) the probability of exceedance (PE) does not depend on the limit states, meaning that the level of damage is very low (compared with Figure 8) and LS3 is not reached under stiff conditions (Figure 10). For medium and soft conditions, DF is more distributed and fragility curves for the various LS are sparser. For example, for soft conditions, the PE of 50% (mean value, see Table 6) is reached for 0.109 g, 0.146 g, and 0.165 g, respectively for LS1, LS2, and LS3.

Figure 10. PL10-STIFF: PGA (g) vs. Probability of exceedance (PE).

Figure 11. PL10-MEDIUM: PGA (g) vs. Probability of exceedance (PE).

Figure 12. PL10-SOFT: PGA (g) vs. Probability of exceedance (PE).

If the same LS are compared for different soil deformability (Figures 13–15), it is possible to see the effects of SSI in increasing the damage. In particular, Figure 13 shows that soil deformability is relevant for the first part of the curves (below 0.40 g), while for bigger intensities, the curves are less influenced by SSI. For example, for LS1 and $\text{PGA} = 0.20 \text{ g}$, PE values are 0.560, 0.666 and 0.894, for stiff, medium and soft conditions, respectively. For LS2, soil deformability is also significant with a maximum difference for $\text{PGA} = 0.20 \text{ g}$: 0.539, 0.590 and 0.705 for stiff, medium and soft conditions, respectively. For LS3 (Figure 15), stiff conditions do not reach the limit state and the difference between medium and soft conditions are still important (i.e., for PE values are $\text{PGA} = 0.20 \text{ g}$: 0.552 and 0.633 for medium and soft conditions, respectively), see Table 7.

Table 7. PL10 (PGA: 0.20 g).

PE	LS1	LS2	LS3
STIFF	0.560	0.539	-
MEDIUM	0.666	0.590	0.552
SOFT	0.894	0.705	0.633

Figure 13. PL10-LS1 (various soil conditions): PGA (g) vs. Probability of exceedance (PE). **Figure 14.** PL10-LS2 (various soil conditions): PGA (g) vs. Probability of exceedance (PE).

Figure 15. PL10-LS3 (various soil conditions): PGA (g) vs. Probability of exceedance (PE).

4.2. PL5 and PL20

Pile flexibility is herein considered by discussing the results for PL5 (the least flexible case) and PL20 (the most flexible case).

Figures 16–18 show that PL5 is more sensitive to soil conditions than PL10, in particular for LS1. The results also show that for soft and medium soil conditions, the values of PE are significantly distant for the various LS. In particular, for PGA = 0.10 g and LS1, PE values are 0.215, 0.511 and 0.836, while for PGA = 0.20 g and LS2, PE values are: 0.203, 0.648 and 0.782 for stiff, medium and soft conditions respectively. PL5 under stiff conditions reaches LS3, as it is possible to see in Figures 7 and 18.

Figure 16. PL5-LS1 (various soil conditions): PGA (g) vs. Probability of exceedance (PE).

Figure 17. PL5-LS2 (various soil conditions): PGA (g) vs. Probability of exceedance (PE).

Figure 18. PL5-LS3 (various soil conditions): PGA (g) vs. Probability of exceedance (PE).

The results for PL20 (the most flexible case) are shown in Figures 19–21, where it is possible to see that for LS1 (Figure 19, slight damage), the curves are still different from each other, with mean values of 0.181 g, 136 g, and 0.095 g for stiff, medium, and soft conditions, respectively. For LS2 (Figure 20), the values of probabilities for soft conditions are significantly different from those for medium and stiff conditions and this is true for all intensities. For example at PGA = 0.30 g, PE are 0.749, 0.788, and 0.850 for stiff, medium, and soft conditions, respectively. Figure 21 shows that stiff conditions do not reach LS3 (compared with Figure 10), while the SSI effects are significant for all the intensities, with the maximum difference for PE are at 0.30 g with 0.749 and 0.850 for medium and soft conditions, respectively.

Figure 19. PL20-LS1 (various soil conditions): PGA (g) vs. Probability of exceedance (PE).

Figure 20. PL20-LS2 (various soil conditions): PGA (g) vs. Probability of exceedance (PE).

Figure 21. PL20-LS3 (various soil conditions): PGA (g) vs. Probability of exceedance (PE).

Figures 22–24 show the role of pile flexibility for several soil conditions at the intermediate level LS2 (DF = 1). When stiff conditions are considered, the flexibility of the pile does not seem to affect the fragility of the piles (same results for PL5, PL10, and PL20). For medium and soft soil conditions, the flexibility reduces pile vulnerability by shifting the curves toward less vulnerable behaviours (right part of the diagram). For example, for PGA = 0.25 g, for medium conditions: 0.767, 0.742, and 0.674 while for soft conditions: 0.888, 0.836, and 0.764, respectively, for PL5, PL10, and PL20 (see Table 8).

Figure 22. STIFF-LS2: PGA (g) vs. Probability of exceedance (PE).

Figure 23. MEDIUM-LS2: PGA (g) vs. Probability of exceedance (PE).

Figure 24. SOFT-LS2: PGA (g) vs. Probability of exceedance (PE).

Table 8. Comparison: PL5, PL10, and PL20 for MEDIUM and SOFT conditions (PGA = 0.25 g).

PE	PL5	PL10	PL20
MEDIUM	0.767	0.742	0.675
SOFT	0.888	0.836	0.764

5. Conclusions

The study conducted in this paper may be viewed as an original contribution to the seismic assessment of pile foundations with a probabilistic-based approach based

on developing analytical fragility curves. The mutual effects of pile flexibility and SSI were considered by performing non-linear dynamic analyses on three pile configurations and three soil conditions (with increasing deformability). The FE computational model is based on an advanced constitutive formulation that enables to drive the assessment of the non-linear behaviour of the soil and of the piles. In particular, the paper assesses the role of pile flexibility on the performance of the soil-pile complex by referring to the ductility factor and three limit states that were proposed herein. In this regard, the ultimate aim of this study is to propose analytical fragility curves for the assessment of piles that may be applied to perform more typologies of configurations and different seismic conditions. This will be the objective of future work.

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