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Site-Response-Based Capacity Degradation for Foundation Design in Seismic Active Areas

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ABSTRACT: Understanding foundation capacity degradation during earthquakes in seismic active areas is crucial for the stability and safety of offshore structures. This paper introduces a site-response-based method to evaluate seismic-induced foundation capacity degradation for granular soil dominated sites.
The paper initially presents a sensitivity study performed adopting both equivalent linear and non-linear site response methods which model the ground motion propagation through the soil column, allowing the determination of the seismic induced cyclic shear stresses at each depth of the soil model. The cyclic induced strength degradation can be estimated applying a pore pressure accumulation procedure, based on site-specific pore pressure development contours, following the methodology outlined by Andersen (2015).
The study outlines differences between the equivalent linear and non-linear site response modelling of soils, and impact of each approach on acceleration/shear stress amplification through the soil column. Practical approaches based on cyclic laboratory testing are proposed for incorporating pore-pressure-induced strength degradation into geotechnical design of open ended piles, focusing on the axial and lateral responses during the earthquake. These findings aim to enhance the resilience of geotechnical structures in earthquake-prone regions.

Keywords: liquefaction, earthquake, pile design, site response analysis, silty sand.

1 INTRODUCTION

The stability and safety of offshore structures in seismically active regions are of paramount importance, particularly in areas where granular soils are dominating. Seismic loads can induce significant foundation capacity degradation, leading to catastrophic failures if not properly accounted for in the design and analysis of offshore foundations. The soil strength degradation in sand dominated sites is typically related to the accumulation of pore pressures during the seismic events, leading to a reduction of effective stresses until full liquefaction in the worst cases. Many authors (Seed 1985, Robertson and Wride 1998, Youd et al. 2001, Zhang 2002, Boulanger and Idriss 2014) have proposed methodologies on how to estimate the soil liquefaction potential; these studies are proposing semi-empirical correlations based on information from liquefaction phenomena observed at onshore sites within the upper ~30 meters of the soil column whereas foundation for large offshore structures could reach larger penetrations.

The Japan Road Association (JRA, 2002) provides a methodology that may be followed for the evaluation of the strength reduction in sands in the context of seismic assessments. These guidelines provide reduction factors to be applied on the unit skin friction depending on the depth (larger reduction factor at shallow depth), on the dynamic shear strength ratio representative of the cyclic resistance of the soil and on the safety factor against liquefaction (ratio between cyclic resistance ratio CRR and cyclic shear stress ratio CSR)

In the field of deep offshore foundations, it is therefore important to investigate the occurrence of liquefaction but also to recognize the conditions of pore pressures generated within the soil, even if they do not trigger liquefaction.

2 SEISMICITY AND DESIGN REQUIREMENTS

Seismic active areas are typically situated in complex geological settings, influenced by the interaction of major tectonic plates.

Design codes and regulations are available worldwide, especially in the areas more prone to seismic events. The seismicity at site could be reasonably described by the peak ground acceleration at outcrops defined at different periods, the return period of the associated seismic event and the magnitude of the seismic input. Most of this information are generally obtained by Seismic Hazard Analyses, based on regional seismic hazard assessments, incorporating data on historical earthquakes, tectonic settings and soil characteristics.

These analyses can be performed under either deterministic hazard assessment (DHA) or probabilistic seismic hazard assessment (PSHA) approaches. The latter type of analyses provides the advantage of treating the seismic event with a statistical approach to predict the likelihood of different levels of ground shakings to occur at a specific site and with a given return period. Each standard defines seismic design cases or limit states to be satisfied in the event of an earthquake. This paper does not address a specific limit state case but proposes a general approach to study the soil degradation due to a seismic event. This methodology can be adopted for different design cases or limit states.

2.1 Seismic input

The standard procedure for the evaluation of a local site response analysis typically involves the execution of the procedure on at least 7 time histories (E-W, N-S, UP-DOWN components) compatible with the design Uniform Hazard Response Spectra (UHRS) for the earthquake design level under consideration, typically obtained from a PSHA. For clarity, the analyses and results in this paper focus on 1 (one) single time history to illustrate each step of the proposed procedure, which in this case is taken as the one providing the maximum PGA between E-W and N-S components).

The selected seismic input is provided in Figure 1, representative of time history at bedrock with a PGA of 0.3g. A sensitivity assessment of the results to the PGA is not explicitly assessed in this paper (Section 4), however it is expected that any effect of non-linearity would prevail at higher levels of PGA, meaning that for relatively low PGA, equivalent linear and non-linear seismic assessments might provide relatively similar findings.

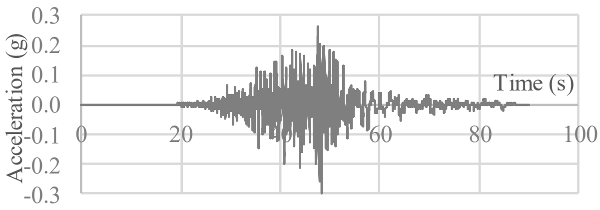

Figure 1. Seismic time history

3 SOIL PROPERTIES

3.1 Soil stratigraphy

Figure 2 provides the typical CPT and shear modulus at small strain profiles considered in this study, with the black lines representing the mean values of a wider range of measured values. The envelopes of these ranges are represented with grey lines. An idealized stratigraphy consisting of an alternance of loose to medium dense silty sands has been considered. The soil profile reveals a general increase in cone resistance and shear modulus with depth, suggesting progressive densification and strengthening of the material. In the upper layer, Unit S1 (0–20 m), both parameters increase rapidly, indicating a transition from softer to firmer materials. Unit S2 (20–60 m), shows fluctuating increases in resistance and a more gradual rise in shear modulus, suggesting heterogeneity within this zone typically associated to heavy presence of silt within the main sand soil matrix. In the deeper layer, Unit S3 (60–100 m), both parameters stabilize, reflecting a more uniform and denser material. Overall, stiffness and resistance increase progressively with depth.

Figure 2

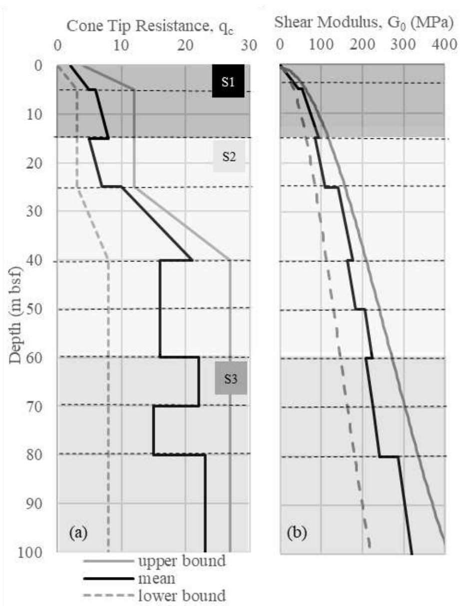

Figure 2. Selected soil profiles, cone tip resistance and shear modulus at small strain vs depth, respectively plot (a) and (b)

3.2 Cyclic and Dynamic parameters

Susceptibility of soils due to cyclic loading can be investigated by means of advanced cyclic laboratory tests to be scheduled for a range of testing conditions including consolidation stresses and cyclic/average stress or strain conditions. When aiming at characterizing the strength behaviour under cyclic loading it is common practice to conduct tests in stress control of cyclic and average stresses, in undrained conditions. These tests are typically conducted in direct simple shear (DSS) or consolidated undrained triaxial (CUTX) mode. The results of these tests are consequently interpreted to build iso-strain and iso-pore pressure contours that can be directly used as tools for the prediction of shear strength degradation (Andersen, 2015). Cyclic contours are also available in literature for clays and sands as function of the stress-history parameters/overconsolidation ratio (OCR), relative density (D_r) and of the normalized cyclic shear strength at failure after a conventional number of cycles N=10 (Andersen, 2015). The same author provides means to scale the cyclic strength (and contours) to take into account for fines content or consolidation stress. The contours available in literature can be generally adopted for an early stage of the project developments in the lack of site-specific results. Despite it is necessary to conduct site specific cyclic laboratory testing program to obtain the cyclic characterization of the soils, the literature reference remains a solid basis to verify validity and coherence of the site specific results.

For the purpose of this study, the contour diagram of the pore pressure for normally-consolidated sands published by Andersen (2015) has been considered (Figure 3). For simplicity one single diagram has been considered to model the three units (S1, S2, S3).

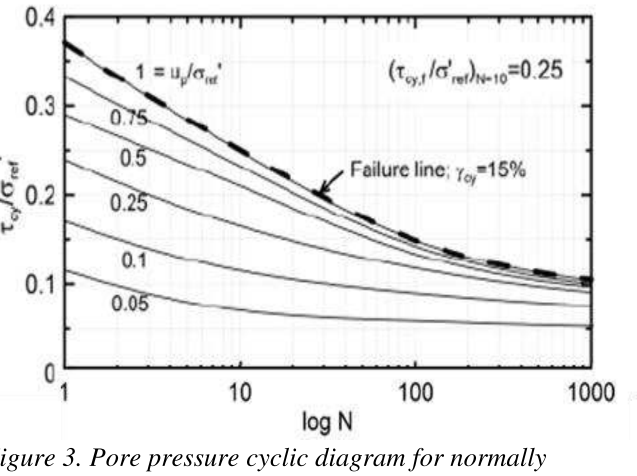

Figure 3. Pore pressure cyclic diagram for normally consolidated sand and silt – DSS tests (Andersen, 2015)

The dynamic properties of the three units are also reported in Figure 4, where the shear modulus degradation and damping increase curves versus shear strain have been selected from Darendeli (2001).

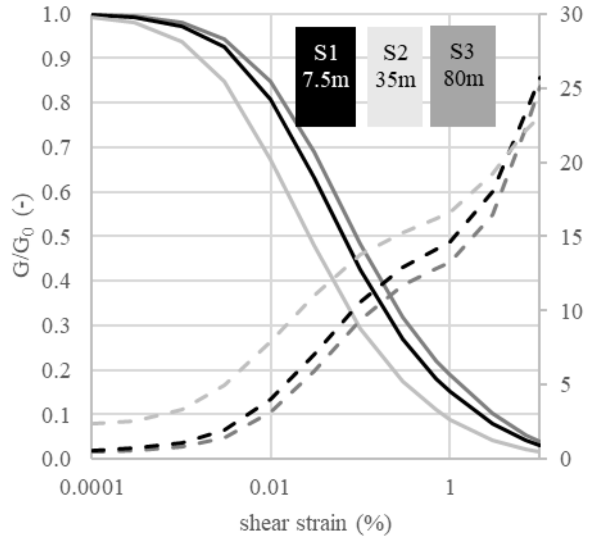

Figure 4. Normalized shear modulus and damping vs shear strain for S1, S2, S3

4 FREE-FIELD SITE RESPONSE ANALISIS (SRA)

Site response analysis assesses the alteration of seismic waves as they propagate through soil layers from the bedrock to the ground surface. This analysis is pivotal in geotechnical earthquake engineering, as it accounts for local soil conditions that can significantly amplify or attenuate seismic motions. The dynamic response of soils is influenced by factors such as layering, stiffness, damping properties, and the nonlinear behavior of soils under cyclic loading. In this paper the site response analyses have been performed by both Equivalent Linear and Non Linear analyses using DEEPSOIL.

The equivalent linear (EQL) method approximates the nonlinear behavior of soils by iteratively adjusting their linear elastic properties based on the level of induced shear strain during seismic loading. This approach simplifies the complex stress-strain relationship of soils into a sequence of linear analyses with updated soil properties. In the cases where non-linear effects are dominant (i.e. liquefaction), a non-linear analysis (NL) is recommended, as it incorporates the nonlinear and hysteretic behavior of soils under seismic loading using advanced constitutive models. This method accounts for the strain-dependent stiffness and damping properties without simplifying assumptions inherent in the equivalent linear approach.

The results of the site response analyses for the EQL and NL (General Quadratic/Hyperbolic Model GQ/H) cases (for a PGA of 0.3g) are reported in Figure 5 in terms of acceleration spectra at seabed. The comparison of the two spectra indicates a significant difference between the resulting spectra at seabed, with the main difference in the range from 0.2s to 2s.This means that for the case under consideration, performing an EQL analysis would provide a conservative estimation of the accelerations (and consequently stresses and strains).

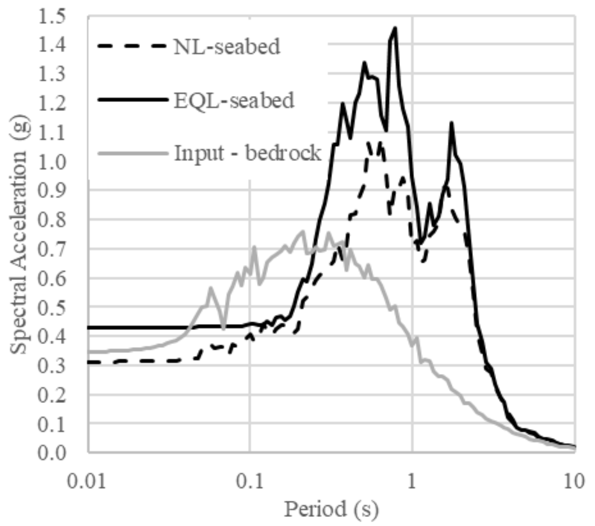

Figure 5. Spectral acceleration comparison between nonlinear and equivalent linear analyses

5 PROPOSED SOIL DEGRADATION APPROACH

The method proposed in this paper is articulated in a series of steps as summarised in the following:

a. Conduct a free-field local site response analysis (EQL or NL) to estimate the acceleration and shear stress time histories at various depths along the soil column (Section 4) as well as the maximum shear stress versus depth,

b. Transform the irregular acceleration time history at each depth into parcels of constant cyclic stress, following a rainflow counting approach, as outlined by Andersen (2015) and shown in Figure 6,

c. Estimate the accumulated normalized pore pressure R_u following the pore pressure accumulation procedure, as proposed by Andersen (2015), at each depth of the soil profile,

d. Estimate the profile of pore pressure build up at each depth of the soil profile. From the normalized pore pressure factor R_u the 'effective stress factor' λ is calculated as:

$$\lambda = (1 - R_u) \quad (1)$$

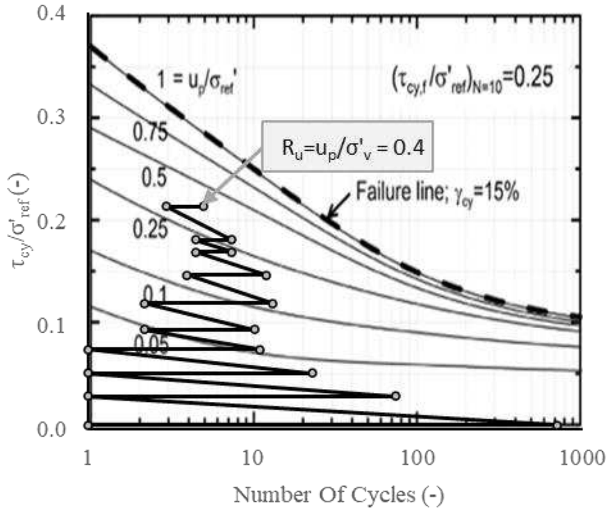

Figure 6. Example of Pore pressure accumulation procedure at 32m below seafloor (bsf)

The main results of the above steps are provided in Figure 7. The cyclic stress ratio profile resulting from the non-linear (NL) site response analysis reduces with depth from 0.35 to around 0.12 at 100m. The computed degradation factor profile versus depth is also plotted following the methodology proposed in this paper and the methodology as proposed by the Japanese Road Association (JRA, 2002) based on a conventional liquefaction assessment following Robertson (1998).

The comparison of the two approaches shows that the JRA tends to underestimate the degradation of soil resistance subjected to seismic event and it considers no degradation below 22m below seafloor (bsf). On the opposite side, the evaluation performed following the method proposed in this paper provides a much severe degradation profile where full liquefaction is predicted for the upper soil Unit S1, significant pore pressure build up from 15 to 25m bsf and much lower degradation until the full depth of the considered soil column. Based on this assessment it could be drawn that following a simplified approach as the one proposed by JRA (2002) may lead to severe underestimation of the effects of the earthquake. This outcome seems to contradict the general idea that semi empirical correlation used for liquefaction study could be conservative in pile design therefore it the execution of more refined procedures is encouraged (or recommended in highly seismic active areas).

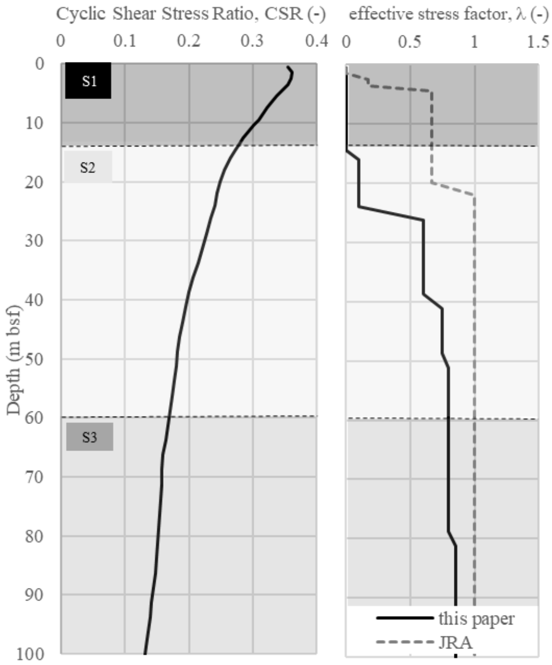

Figure 7. Normalized shear stress and 'effective stress factor' λ versus depth bsf

6 ENGINEERING APPLICATION

This section provides an example of potential application of the proposed degradation factor in offshore open ended steel piles design in sandy and silty soils. The pore pressure build up during the earthquake may affect both axial (capacity) and lateral (stiffness) behaviour of slender piles leading to potential partial loss of capacity and possible noticeable displacement during the earthquake. It shall be noted that the earthquake represents a severe scenario and combination with environmental events with adequate return period shall be considered (taking into account for the reduced likelihood of a combination of multiple extreme events to occur simultaneously).

6.1 Axial Capacity

Axial capacity evaluation under seismic loads should account for the combined beneficial strain rate effect and detrimental strength degradation due to cyclic strength degradation effects. On sands, particularly if loose, the effects of seismic loadings on unit skin friction would be associated primarily with pore pressures development. The seismic axial capacity can be obtained as follows:

$$R_{seismic} = R_{liquefied} + \lambda\,(R_{static} - R_{liquefied}) \qquad (1)$$

The liquefied soil resistance can be obtained from site specific undrained laboratory tests on representative granular samples or from Olson and Stark (2002) formulation, where representative ranges for liquefied undrained shear strength $s_{u,liq}$ of granular soils are provided.

It is possible to establish a first order estimate for a seismic reduction factor (to be used solely for general assessment analysis), in a preliminary design stage based on a number of assumptions as follows:

- The liquefied resistance $R_{liquefied}$ is assumed to be proportional to the liquefied undrained shear strength $s_{u,liq}$,
- The static resistance R_{static} is assumed to be proportional to soil-soil effective shear resistance $\sigma'_v \tan\varphi'$,
- Frictional capacity dominates the axial resistance.

Under the above assumptions, a degradation factor 'DF' can be computed as follows:

$$DF = \frac{\frac{s_{u,liq}}{\sigma'_v} + \lambda\cdot\left(tan\ \phi' - \frac{s_{u,liq}}{\sigma'_v}\right)}{tan\ \phi'} \qquad (2)$$

By means of this factor, the seismic capacity can be estimated from the static capacity as follows:

$$R_{seismic} = DF\ \cdot R_{static} \qquad (3)$$

6.2 Soil Pile Interaction

The interaction between the soil and the pile through the pile interface is typically modelled by means of the T-Z and P-Y curves for the axial and lateral loading conditions. The pore-pressure build up induced by the seismic event would generally impact these interaction curves.

6.2.1 Recommendations for T-Z curves

Two sets of T-Z curves can be defined for sands and silty sands:

- Low estimate set to account for the reduction in the effective stresses, calculated as follows:

  $$f_d = \lambda\cdot f_{sand,static} \qquad (4)$$

  where $f_{sand,static}$ is the static unit skin friction as defined for the static condition in sand.
- High estimate set to conduct sensitivity stiffness analyses. This set corresponds to the conventional formulation of T-Z as given in the API RP 2GEO (2021).

6.2.2 Recommendations for P-Y curves

Complex interactions are expected on piled foundations during a seismic event with reference to the lateral behaviour, in particular:

- Inertial loading: effect due to the structure mass, typically affecting the upper ten pile diameters below seabed,
- Pile curvature: effect due to free-field displacement (flow) of the soil around the pile. For slender and flexible piles the effects of soil movement would be minimal, while for stiff large diameter piles the loading from the soil flow may become significant and should therefore be considered as actual loading conditions on the pile.

The determination of the horizontal loading response on a pile is not straightforward and typically varies along shaft.

The API RP 2GEO (2021) cyclic formulation for (P-Y) curves can be considered as a starting point with appropriate modifications to take into account for the pore pressures generated during the seismic event. Kagawa et al. (1980) demonstrated that the normalized shear modulus of a sand tends to decrease with the square root of the pore pressure generated in the soil (Figure 8).

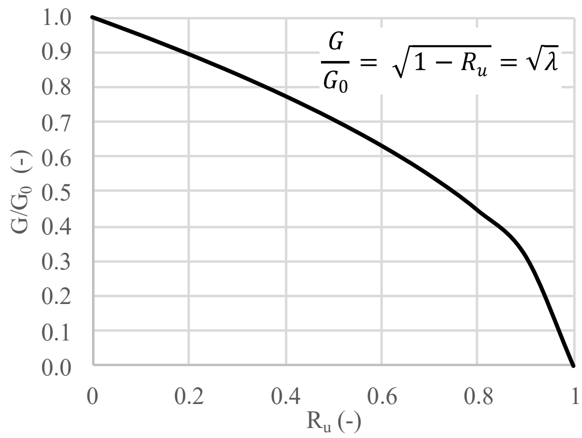

Figure 8. Normalized shear modulus vs 1-λ

The reduction in stiffness induced by the pore pressure could be therefore modelled by applying a reduction to the P component of the P-Y curves equal to the square root of the 'effective stress factor' λ. This proposed approach would lead at the same time to a reduction of the ultimate lateral resistance as well as to a reduction of the stiffness typically providing a conservative response.

7 CONCLUSIONS

This paper presents a procedure to estimate soil strength degradation in offshore foundations caused by seismic events through accounting for pore pressure accumulation with depth. The case study comparing the proposed methodology against the Japan Road Association (JRA, 2002) approach reveals that conventional methods may significantly underestimate degradation effects at depths greater than 20m. The proposed four-step method provides a more comprehensive assessment, predicting full liquefaction in the upper soil unit and significant pore pressure buildup in deeper layers. Engineering applications are presented for both axial capacity calculation using a seismic reduction factor and soil-pile interaction through modified T-Z and P-Y curves. This paper is however not considering kinematic effects, which can provide different soil-pile interaction and waves propagation. As recommended by many standards, both kinematic and inertial analysis shall be conducted and might be considered/implemented in the future as an extension to the proposed approach.

REFERENCES

API RP 2GEO, 2021. API RP 2GEO Geotechnical and Foundation Design Considerations (Reaffirmed, January 2021) - ISO 19901-4:2003 (Modified).

Andersen, K.H., 2015. Cyclic soil parameters for offshore foundation design, in: 3rd ISSMGE McClelland Lecture. Presented at the ISFOG.

Boulanger R.W. and Idriss I.M. (2014). CPT and SPT based liquefaction triggering procedures. Report No. UCD/CGM-14/01. Center for Geotechnical Modeling, University of California, Davis.

DEEPSOIL V7, Nonlinear and Equivalent Linear Seismic Site Response of One-Dimensional Soil Columns, University of Illinois at Urbana-Champaign.

Darendeli M. B., "Development of a New Family of Normalized Modulus Reduction and Material Damping Curves", The university of Texas at Houstin.

JRA. 2002. "Specifications for Highway Bridges." Maruzen, Tokyo.

Olson, Scott & Stark, Timothy. (2002). Liquefied strength ratio from liquefaction case histories.

Canadian Geotechnical Journal - CAN GEOTECH J. 39. 629-647. 10.1139/t02-001.

Youd, T.L., Idriss, I.M., Andrus, R.D., Arango, I., Castro, G., Christian, J.T., Dobry, R. et al. 2001. Liquefaction resistance of soils: summary report from the 1996 NCEER and 1998 NCEER/NSF workshops on evaluation of liquefaction resistance of soils. Journal of Geotechnical and Geoenvironmental Engineering 127 (10): 817–833.

Kagawa, T., et al. 1980. Lateral Load-Deflection relationships of piles subjected to dynamic loads. Soil and Foundations 20 (4): 19–36.

Robertson, P.K. and Wride (Fear), C.E. 1998. Evaluating cyclic liquefaction potential using the cone penetration test. Canadian Geotechnical Journal 35 (3): 442–459.

Seed, H. B., Tokimatsu, K., Harder, L. F., & Chung, R. M. ,1985, The Influence of SPT Procedures in Soil Liquefaction Resistance Evaluations. Journal of Geotechnical Engineering, ASCE, 111, 1425-1445.

Zhang, G., Robertson, P.K. and Brachman, R.W.I., 2002, Estimating Liquefaction induced Ground Settlements From CPT for Level Ground, Canadian Geotechnical Journal, 39(5): 1168-1180.