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ABSTRACT

Recent researches have shown that piles are laterally unsupported in liquefiable soils during most strong earthquakes. If this unsupported length is significantly large, the high axial load on piles may make them more vulnerable to buckling instability. Calculation of buckling instability requires the full unsupported length of pile, which is the sum of pile length above the ground, pile length in the liquefied soil and a depth of fixity below the liquefied soil layer. In this paper, the length of fixity of pile foundations embedded in liquefiable soils has been investigated using a simple numerical method. The finite element program SAP2000 V12 has been used to carry out the parametric analysis. The soil has been modeled using Winkler spring approach, which models the lateral restraining effect of the soil as a set of discrete one-dimensional spring distributed along the length of the pile. The buckling loads of the piles embedded in the soil are evaluated using the eigenvalue analysis. The results are then compared and validated with previous analyses based on empirical, analytical and numerical methods. The sensitivity of the buckling load of the embedded piles are studied with respect to the factors such as the depth of liquefaction, the stiffness of the liquefied soil and the unsupported length of the pile, and the results are discussed.

INTRODUCTION

Piles are long slender structural elements that safely transfer the superstructure loads to the supporting soil through skin friction and end bearing. In service condition, embedded piles get lateral support from the surrounding soil. However, during seismic liquefaction, the lateral support of the piles decreases significantly. In such condition, axially-loaded piles behave like unsupported, beam column structural elements. Bhattacharya et al (2004) suggested that axially loaded piles may collapse as a result of buckling instability if the soil bracing effect is removed due to liquefaction. Reliable methods for estimating the buckling capacity of piles in liquefied soils have not been widely introduced to the industry and are also not included in the recommendations of design codes such as JRA (2002), NEHRP (2000) and Eurocode 8 (1998) etc. Buckling is a non-ductile method of failure which results in a rapid collapse and it should be avoided in the design process. Hence, the present study is aimed at characterizing the buckling load depending on the depth of liquefaction and soil stiffness with the help of a numerical model.

LIQUEFACTION EFFECTS DURING PAST EARTHQUAKES

In the areas of loose, saturated sandy soil (which often prevails throughout the marine environment) strong ground shaking during an earthquake may cause some soils to liquefy due to high pore water pressure generation. Past earthquakes such as 1989 Loma Prieta, 2001 Bhuj earthquake and the 2004 Sumatra earthquake have shown that numerous damages to coastal facilities, like ports, berths and jetties are predominantly due to soil liquefaction. In Loma Prieta earthquake, the most severe damages occurred in Oakland and San Francisco where the poor soil conditions (saturated, loose sand) in this area led to amplified shaking and liquefaction. Fig. 1 shows the lateral spreading of the ground due to liquefaction. Major structures damaged during this earthquake due to liquefaction include buildings, bridges, highways and port facilities.

Fig. 1. Ground failure due to liquefaction during Loma Prieta earthquake 1989 (http://earthquake.usgs.gov/bytopic/photos.html).

During the 2001 Bhuj earthquake, which occurred in the Kachchh region of India, widespread liquefaction has been observed in many places (Fig. 2). According to many residents, fountains of water ranging from 1 to 2 m in height formed during and immediately following the earthquake. In the port of Kandla, the most damages were observed in berthing jetties, oil jetties and warehouses.

Fig. 2. Small Sand blows near Budharmora about 14 km from the earthquake epicenter during 2001 Bhuj earthquake (from 2001 Bhuj, India Reconnaissance Report).

The 2004 Sumatra earthquake, which occurred in the Indian Ocean, resulted in a huge tsunami and affected 12 nations and caused several damages to harbor structures. The major structural damages observed include wharves and jetties which are mainly due to their improper design, poor maintenance and liquefaction.

DEPTH OF FIXITY APPROACH

The stability analysis of fully and partially embedded piles is highly indeterminate and intractable unless some simplifying conditions are imposed, see for example Davisson and Robinson, 1965). Figure 3 shows a free-head pile with an unsupported length L_U and an embedded length L_S. As proposed by Davisson and Robinson (1965), the most desirable simplification is to consider the lower end of the pile as fixed at some depth below the ground surface, this depth is called depth of fixity, L'_S. The pile of Fig. 3 can be considered for the buckling analysis as a simply cantilever of total length $L_{E.}$

$$L_E = L_U + L'_S \tag{1}$$

L_E represents the equivalent length of the cantilever, which is presumed to behave in the same manner of the freestanding pile.

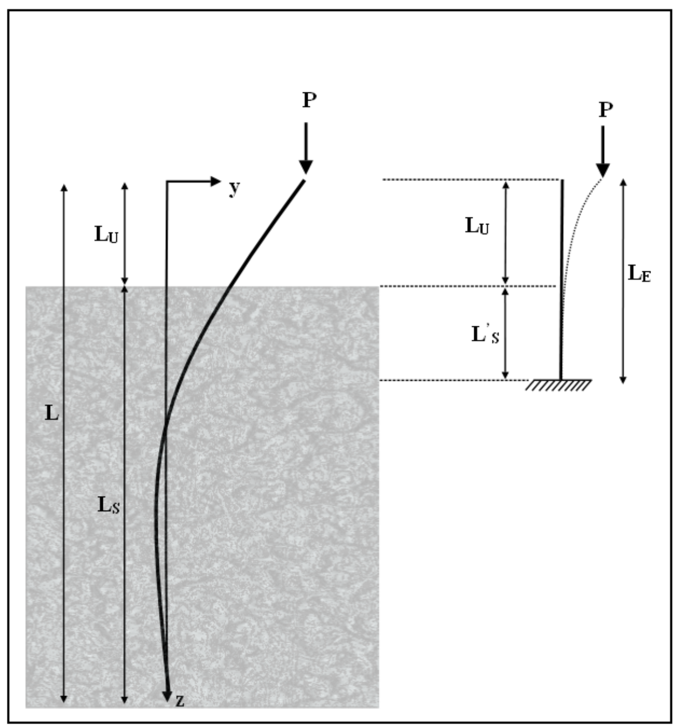

Fig. 3. Partially embedded pile and definition of the depth of fixity using the equivalent cantilever concept.

The depth of fixity, L'_S , can be simply evaluated from the expression of the buckling load valid for a cantilever:

$$P_C = \frac{\pi^2 EI}{4L_E^2} \tag{2}$$

Combining the equation (1) with equation (2), the depth of fixity, L'_S, is given by:

$$L'_S = \sqrt{\frac{\pi^2 EI}{4P_C}} - L_U \tag{3}$$

NUMERICAL ANALYSIS

The numerical analysis has been carried out using the finite element Program SAP2000 V12 (CSI, 2008). The buckling load of the piles embedded in soil has been evaluated using a "linear buckling analysis" which involves the solution of the generalized eigenvalue problem:

$$(S + \tau \cdot S_\sigma) \cdot \psi = 0 \tag{4}$$

Where τ is the diagonal matrix of eigenvalues, ψ is the matrix of the corresponding eigenvector, S is the material stiffness matrix and S_σ is the geometric stiffness matrix. In the linear eigenvalue analysis the eigenvalue τ_i represents the buckling load and the eigenvector ψ_i is the buckling mode. The lowest τ_i gives the first buckling load.

Soil Model

The soil has been modeled using the Winkler spring approach, which models the lateral restraining effect of the soil on the pile as a set of discrete one-dimensional spring distributed along the length of the pile. Each spring is characterized by a constant value, called coefficient of subgrade reaction, k, which represent the ratio between the horizontal pressure at a point (p) along the beam and the horizontal displacement at that point (y).

$$p = k \cdot y \tag{5}$$

Where: p [N/m^2] is the pressure acting on the surface of the pile, k [N/m^3] is the coefficient of subgrade reaction; y [m] is the displacement. Many authors refer to the modulus of subgrade reaction K [N/m^2], which takes in to account the width of pile.

$$K = k \cdot D \tag{6}$$

D is the diameter of the pile.
One of the major definitions in the Winkler soil model that needs careful calculation is the coefficient of subgrade reaction (k). Many researchers carried out in-situ plate bearing tests and proposed different correlation between the mechanical soil's characteristic and the coefficient subgrade reaction. In order to evaluate the coefficient of subgrade reaction, Vesic (1961) proposed the following expression for a beam resisting on isotropic elastic solid as:

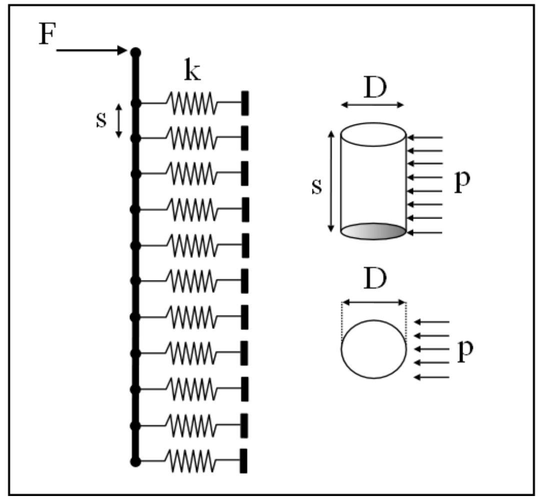

Fig. 4. Illustration of Winkler approach (1867).

$$k = \frac{0.65}{D} \cdot \sqrt[12]{\frac{E_S D}{EI}} \cdot \frac{E_S}{1-\nu_S^2} \tag{7}$$

Where, E_S and ν_s indicate the Young's modulus and the Poisson's ratio of the soil respectively. EI is the modulus of rigidity of the pile. Francis (1964) observed that in the case of a pile foundation, the Vesic's expression needs a correction in order to take into account the different kind of geometry. The author proposed the following expression, which has been used in the numerical analysis.

$$k = \frac{1.30}{D} \cdot \sqrt[12]{\frac{E_S D}{EI}} \cdot \frac{E_S}{1-\nu^2} \tag{8}$$

It can be noted that the value of k in Francis's expression (8) is twice that of the Vesic's one (7). In the present numerical model, the soil springs have been specified with a linear force-deformation relationship. The program considers the total force acting along a particular section of the pile defined by two successive springs. The relationship between the coefficient of subgrade reaction (k) and the stiffness of soil spring (U) can be written as:

$$U = k \cdot D \cdot S \tag{9}$$

Where, S is the spacing of the soil springs. For better approximation of the numerical results, S has been taken as 0.1m and a total of 150 springs have been assigned for a pile of 15m length.

Pile Model

In SAP2000 the pile is modeled by beam-column elements. The cross section of the pile has been assigned as hollow circular. The structural characteristics of the pile are listed in Table 1

Table 1 Structural characteristics of the pile used in the numerical analysis

Young's Modulus [MPa]	Poisson's ratio	Outside diameter [m]	Thickness [mm]	EI [Nm^2]
2000	0.3	1.0	7.8	6000000

The final model developed for the parametric analysis is shown in Figure 5. The boundary conditions used in all cases are fixed at bottom and free at pile's head.

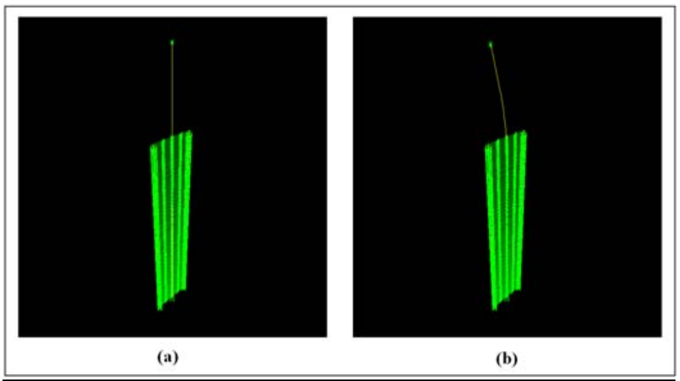

Fig. 5. Model used in SAP2000, (a) initial condition, (b) deformed condition.

Validation

In order to verify the numerical model, the buckling load computed by SAP2000 has been compared with the well-known theoretical solution for different support conditions. The comparison has shown that the difference is very minimal, which gives confidence to use the same model for further parametric study.

Table 2 Comparison of the buckling load computed by SAP2000 with the theoretical solution

Support condition	Difference in % between the buckling load computed by SAP2000 and theoretical solution
Free-fixed column	0.01
Pinned-pinned column	0.06
Fixed-pinned column	0.14
Fixed-sway column	0.23

Many authors have studied the problem of the buckling load of fully and partially embedded piles. The results obtained by SAP2000 have been compared with previous analysis. For this purpose, some non-dimensional variables are introduced as will be discussed later. The buckling load has been normalized by P_E, the Euler buckling load for a hinged-end bar of length L and flexural rigidity EI with no elastic support along its span (Hetényi, 1946).

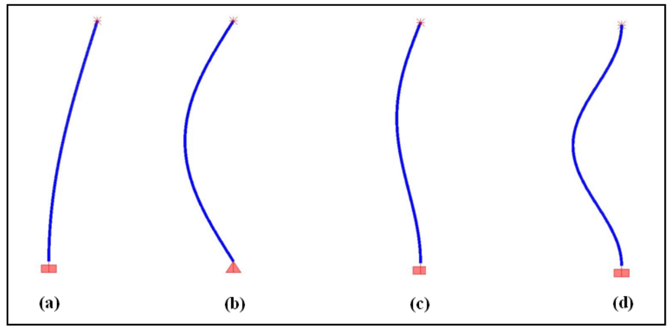

Fig. 6. Buckled shape a) free-fixed column, b) pinned-pinned column, c) fixed-pinned column and d) fixed-sway column.

$$P_E = \frac{\pi^2 EI}{L^2} \tag{10}$$

The soil's stiffness (E_S) has been normalised by λ (Hetényi, 1946):

$$\lambda = \sqrt{\frac{KL^4}{EI}} \tag{11}$$

Finally, the embedment ratio δ has been introduced in order to take into account the grade of pile's embedment in the soil, such as:

$$\delta = \frac{L_S}{L} \tag{12}$$

Where, L_S represents the embedded length of the pile.

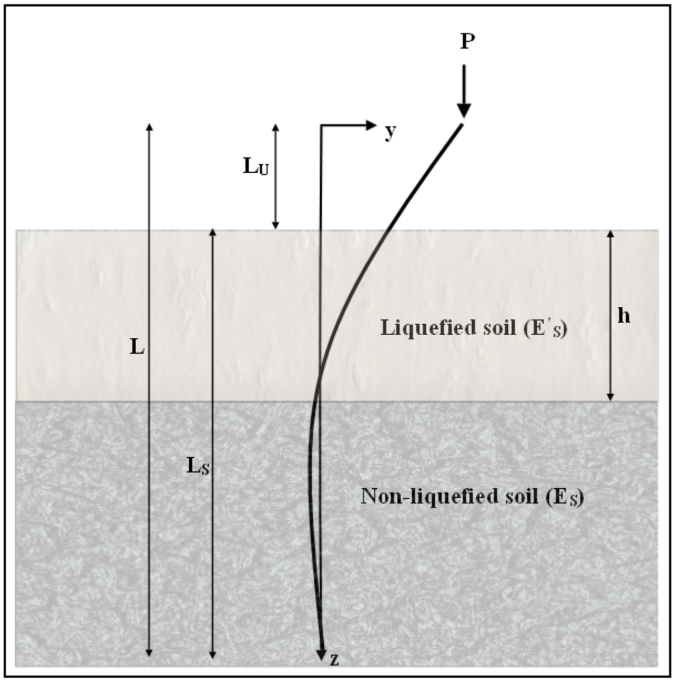

Fig. 7. Visual definition of the variables L, L_U, L_S, E_S, and E'_S.

The results obtained from the numerical study have been plotted in Fig. 8 in terms of non-dimensional variables defined before and compared with other available known solutions as well. For different soil stiffness, the calculated buckling load from the present model is very close to the analytical and empirical solution proposed by many authors. It can also be noted that, for λ less than 200 the Winkler method overestimates the buckling load. This model, hence, can be used as a conservative design value where the normalized soil stiffness (λ) is in the range of 200.

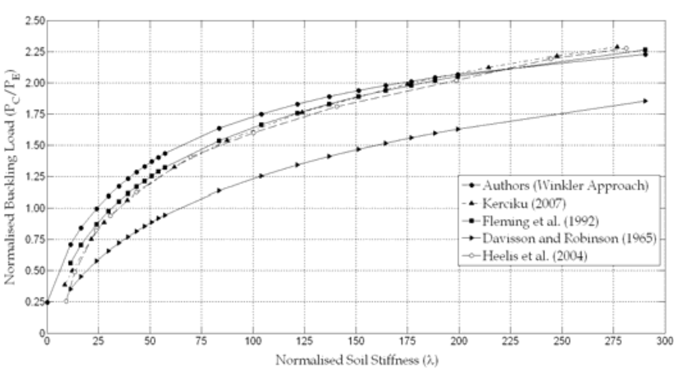

Fig. 8. Comparison in terms of normalized buckling load and soil stiffness for different approaches.

Parametric analysis

The main aim of the parametric analysis is to investigate the sensitivity of the buckling load to the depth of liquefaction, the stiffness of the liquefied soil and the unsupported length of the pile. In this work, the stiffness of the liquefied soil is computed using the stiffness degradation ratio φ, introduced by Yasuda et al (1998).

$$\varphi = \frac{E'_S}{E_S} \cdot 100 \tag{13}$$

E'_S is the degraded stiffness of soil. At full liquefaction, Yasuda et al. (1999) reported that the stiffness of the soil decreased from 10% up to a value close to 0.1%, depending on the density of its non liquefied value. In the parametric analysis three different values of φ have been considered: 10%, 1% and 0,1%. The depth of the degraded soil has been increased in discrete intervals of 1 meter length. For each value of the soil stiffness degradation ratio, φ, three different embedment ratios, δ, given by (12), has been employed: 0.5, 0.75, 1.

Non-dimensional variables

The non-dimensional variables introduced in Table 3 are to characterize the buckling response of pile in a more general way. The results of the analysis are presented for different embedment ratios that will be discussed in the next section.

Table 3 Definition of non dimensional variable

Non-dimension variable	Symbol	Expression
Non-dimension stiffness parameter	R	$\sqrt{\frac{k}{EI}}$
Non-dimensional unsupported length of the pile	J_R	$\frac{L_U}{R}$
Non-dimension depth of liquefaction	H_R	$\frac{h}{R}$
Non-dimension depth of fixity	S_R	$\frac{L'_S}{R}$
Non-dimensional equivalent length of cantilever	E_R	$\frac{L_E}{R}$
Non-dimension buckling load	N	$\frac{P_C}{P_E}$

RESULTS

Figure 9, 10 and 11 plots the results for different values of embedment ratio, δ, defined in equation (12) in terms of non-dimensional depth of liquefaction (H_R) and non-dimensional depth of fixity (S_R). In each case three different soil stiffness degradation ratio φ have been used. The plots show that when the depth of liquefaction is low, e.g., $H_R = 1$ or 2, the depth of fixity variation from $E'_S(10\%)$ to $E'_S(1\%)$ is not high, however, when the depth of liquefaction increases, the ratio of $E'_S(10\%)$ to $E'_S(1\%)$ becomes large. As the depth of fixity is calculated from the pile's buckling response, the depth of liquefaction can be considered to be related nonlinearly to the buckling load of the pile.

Similar results for different values of soil stiffness degradation ratio, φ, defined in equation (13) are plotted in terms of non-dimensional depth of liquefaction (H_R) and non-dimensional depth of fixity (S_R), (Fig. 12, 13, 14). In each case three different embedment ratios (δ) have been considered. As expected, the results show that when the degradation is very high (i.e. 0.1% E_S), the depth of fixity is not very sensitive to the embedment ratio. However, while the soil stiffness degradation is higher (i.e., 1% and 10% E_S), the depth of fixity is nonlinearly related to the depth of liquefaction.

CONCLUSIONS

It has been shown that a fully or partially embedded pile can be analyzed considering a free-standing pile with a fixed base located at some distance below the ground surface. The comparison between the results obtained from the simplified Winkler approach with more sophisticated numerical, analytical and empirical analysis approaches (see Fig. 8) has shown that the simplification taken into account in the Winkler spring approach are not very relevant.

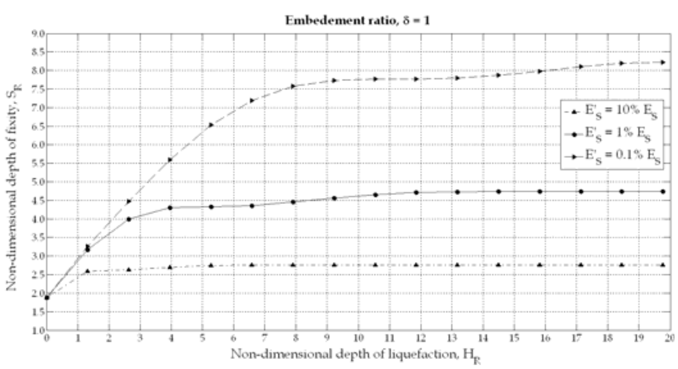

Fig. 9. Results for a fully embedded pile, δ = 1, in terms of non-dimensional depth of liquefaction H_R and non-dimensional depth of fixity S_R.

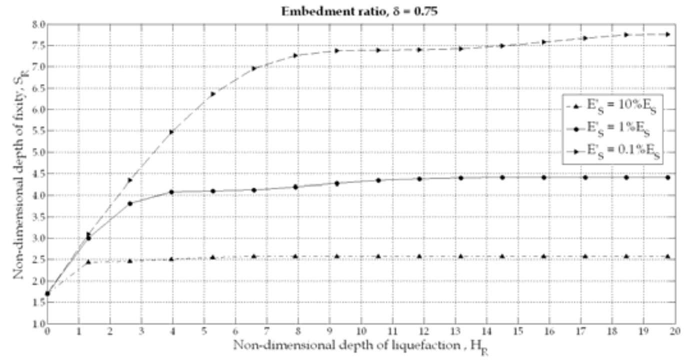

Fig. 10. Results for a partially embedded pile, δ = 0.75, in terms of non-dimensional depth of liquefaction H_R and non-dimensional depth of fixity S_R.

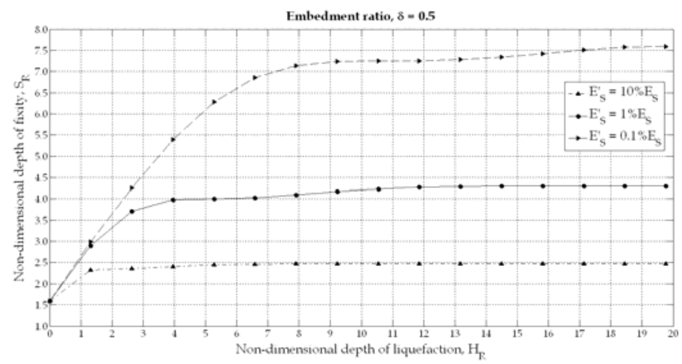

Fig. 11. Results for a partially embedded pile, δ = 0.5, in terms of non-dimensional depth of liquefaction H_R and non-dimensional depth of fixity S_R.

From the results presented in Fig. 9, 10, 11, 12, 13 and 14, it can be concluded that the fixity of the piles embedded in liquefied soil is affected mainly by the depth of liquefaction, h, and soil stiffness degradation ratio φ. After a certain value of the depth of liquefaction, the depth of fixity can be approximated by a constant value depending on the amount of the degradation of the liquefiable layer. Differently, the results appears not to be very sensitive in function of the embedment ratio, δ, this is more evident in the case of the highest stiffness degradation ratio (φ = 0.1%). Further work is required in order to consider the post-buckling behavior, which cannot be considered in a linear buckling analysis as considered in this work.

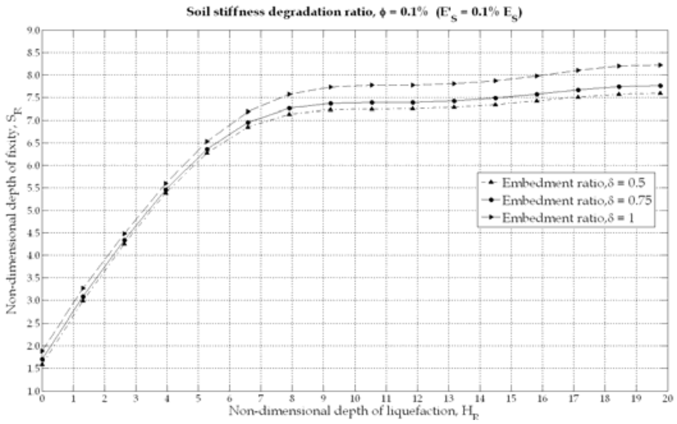

Fig. 12. Results for different embedment ratio, δ, considering a constant soil stiffness degradation ratio, φ = 0.1%.

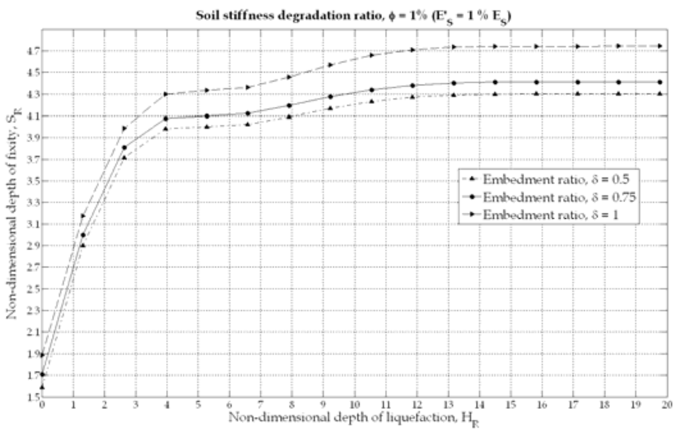

Fig. 13. Results for different embedment ratio, δ, considering a constant soil stiffness degradation ratio, φ = 1%.

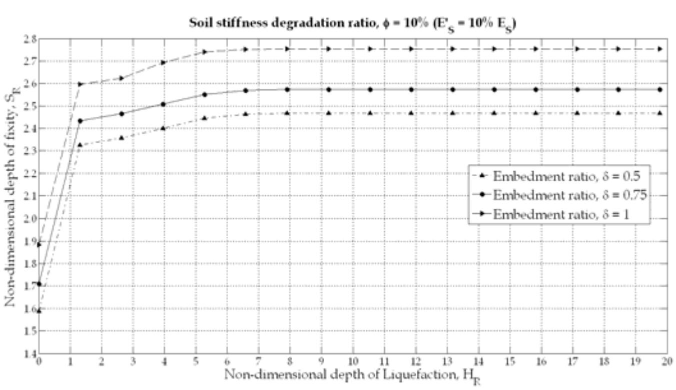

Fig. 14. Results for different embedment ratio, δ, considering a constant soil stiffness degradation ratio, φ = 10%.

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