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A BOUNDARY ELEMENT ANALYSIS OF SOIL-PILE INTERACTION UNDER LATERAL CYCLIC LOADING IN SOFT COHESIVE SOIL

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Abstract

The environment prevalent in ocean necessitates the piles supporting offshore structures to be designed against lateral cyclic loading initiated by wave action. Such quasi-static load reversal induces deterioration in the strength and stiffness of the soil-pile system introducing progressive reduction in the bearing capacity associated with increased settlement of the pile foundation. To understand the effect of lateral cyclic load on axial response of single pile in soft clay, a numerical model has been developed. The theoretical results are compared with the available experimental results so as to validate the theory. This paper presents a brief description of the methodology developed, analysis and interpretations of the theoretical results obtained and the relevant conclusions drawn there from.

Keywords*:* Amplitude; cohesive soil; cyclic load; frequency; pile

1. Introduction

Offshore structures, namely, oil drilling platforms, jetties, tension leg platforms etc. are mostly supported on pile foundations. Apart from the usual super structure load (dead load, live load, etc.), these piles are subjected to continuous lateral cyclic loading resulting from ocean waves. As reported by other researchers, this type of loading induces progressive degradation of the foundation capacity associated with increased pile head displacement. A comprehensive review of literature indicates that limited research works have been done in the related areas. The contributions made by Matlock [1], Poulos [2], Purkayastha and Dey [3], Narasimha Rao et al. [4], Dyson [5], Basak and Purkayastha [6] and Randolph [7] are worthy of note. Some of the works were theoretical while the others had been experimental (laboratory and field based investigations). As pointed out by Poulos [2], basically the following three reasons have been identified for such degradation of strength and stiffness of pile-soil system: (i) Development of excess pore water pressure generated during cyclic

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loading in progress. (ii) General accumulation of irrecoverable plastic deformation of soil surrounding the pile surface. (iii) Rearrangement and realignment of soil particles surrounding the pile surface. The offshore pile foundations need to be designed considering two phenomena: adequate factor of safety against ultimate failure and acceptable deflection at pile head. The aim of this investigation reported herein is to develop a theoretical model to analysis and understand the effect of lateral cyclic loading on the performance of pile foundation under axial static loading.

2. Mathematical Formulations

The theoretical investigation that is reported here was aimed at developing a theoretical methodology for analyzing the effect of lateral cyclic loading on axial static response of *single pile in clay*. Initially, analysis of a single pile under static lateral load was carried out. Further extension was made to incorporate the effect of lateral cyclic loading. The methods suggested by Poulos [2,8] were followed as preliminary guideline with relevant modifications. For computation, boundary element analysis was used.

2.1 Assumptions

The analysis was based upon the following assumptions: (i) The subsoil is a homogeneous, isotropic, semi-infinite, elastic-perfectly plastic material. When elastic, it has a constant Young's modulus (E_s) and Poisson's ratio (μ) which remain unaffected by the presence of the pile. (ii) The pile behaves as an elastic beam. (iii) Possible horizontal shear stresses between the soil and the sides of pile are ignored. (iv) The soil strictly adheres to the pile surface during loading - static or cyclic. For evaluating the cyclic axial capacity of the pile, however, the effect of formation of gap at pile-soil interface near G.L. was considered. (v) Pile tip is free to translate and rotate.

2.2 Pile under lateral static load

The idealized problem is depicted in Figure1. The single, vertical pile was idealized as a thin vertical plate of width'd', length 'L' and flexural rigidity 'E_pI_p'. Lateral static load 'H' is applied at a height of 'e' above G.L. The embedded portion of the pile is longitudinally discretized into finite number of elements, say (n+1). These elements were equal in length except the two extremes which were half the length of the other elements. In case of a free headed pile, a clockwise moment 'M' was applied at the pile head. Any i-th pile element was subjected to a lateral soil pressure p_i which was assumed to act uniformly over the surface of the entire element. Initially, the focus was to evaluate the displacements of the soil and the pile at the central nodal points of each element and to apply a condition of displacement compatibility.

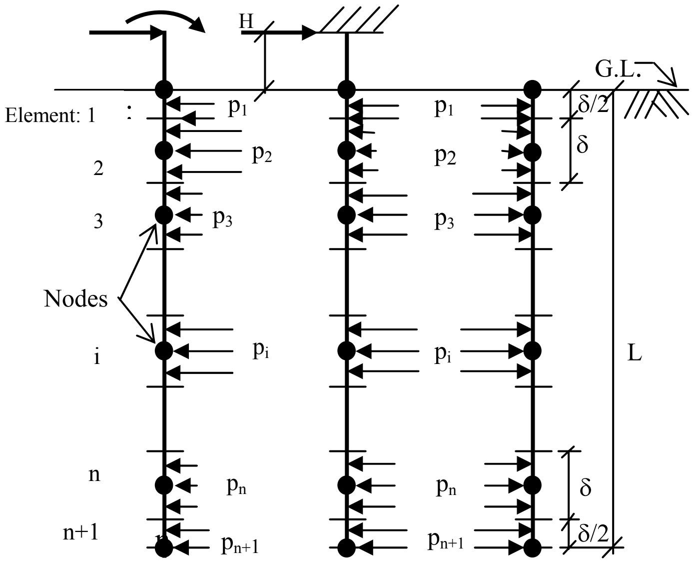

Figure 1. The idealized problem for stresses acting on: (a) Free headed pile. (b) Fixed headed pile. (c) Soil adjacent to pile surface

The soil displacements were obtained by integrating the equation of Mindlin, 1936 (see [8]) over each element. The expressions already provided by Douglas and Davis, 1964 (see [8]) is used for this purpose. The soil displacement relations were expressed in matrix form as follows:

$$\{\boldsymbol{\rho}_S\} = [\mathbf{I}].\{\mathbf{p}\} \tag{1}$$

where $\{\boldsymbol{\rho}_S\}$ and $\{\mathbf{p}\}$ were the vectors representing nodal horizontal displacements of soil at the interface and the elemental soil pressures respectively and $[\mathbf{I}]$ was a square matrix representing the soil displacement influence factors as obtained from integration of Mindlin's equation.

The pile nodal displacements, on the other hand, were evaluated by expressing the standard fourth order differential equation of an elastic beam in finite difference form. The expressions were written in a matrix form as follows:

$$\{\mathbf{p}\} = [\mathbf{F}].\{\boldsymbol{\rho}_p\} \tag{2}$$

where $\{\boldsymbol{\rho}_p\}$ was the vector for nodal pile displacements and $[\mathbf{F}]$ was the finite difference coefficient matrix.

In accordance with the condition of displacement compatibility, the soil nodal displacements and that of the pile should be equal. Thus,

$$\{\boldsymbol{\rho}_S\} = \{\boldsymbol{\rho}_p\} \tag{3}$$

The nodal displacements were therefore replaced by the unknown soil pressures. These expressions, together with the two more expressions regarding the horizontal load and moment equilibrium conditions of the pile were written in matrix form as follows:

$$[\mathbf{C}]\{\,\mathbf{p}\,\}=\{\mathbf{D}\} \tag{4}$$

where, [**C**] was a square matrix termed as coefficient matrix and {**D**} was the relevant augment vector.

This equation was solved to obtain the initial values of the unknown soil pressures. These elastic soil pressures p_i as obtained from Eq. (4) above were then compared with a specific yield pressure p_{iu} relevant to any i-th element. If $|p_i| \geq |p_{iu}|$, the interface soil adjacent to the element were considered to have been yielded and the element was assumed to have been *failed*, and $|p_i|$ was replaced by $|p_{iu}|$. The soil pressures for the remaining *unfailed* elements were computed from initial elastic analysis in the same manner as described above. The procedure was recycled till the elastic soil pressures in non of the elements were observed to exceed the yield pressure.

Unless specified otherwise, the values of p_{iu} considered in present analysis was in accordance with the recommendations of Broms [9,10]. The values adopted in present analysis are summarized below:

(a) **For clay**: $2c_u$ at G.L., increasing linearly up to a limiting value of $9c_u$ at a depth of three times the pile diameter, the value being retained for greater depths, c_u being the undrained cohesion of the clay.

(b) **For sand**: The ultimate lateral soil pressure at a certain depth is three times the Rankine's passive earth pressure at that depth.

After computing {p}, the nodal displacements ρ_i were obtained from the pile displacement relations. The expressions for pile head deflection ρ_h was evaluated by considering the flexure equation of the pile above G.L.. The final expressions for different pile conditions were expressed as follows:

$$\rho_h=\rho_1(1+e/\delta) - \rho_2(e/\delta) + e\delta(M+He)/(2E_pI_p)+(3M+2He)e^2/(6E_pI_p) \tag{5(a)}$$

$$\rho_h= \rho_1+He^3/(3E_pI_p)+e^2\delta^2 d\{0.5(p_1+p_{n+1})+(\sum_{n}^{i=1} p_i)\}/(2E_pI_p) \tag{5(b)}$$

Nodal bending moments and shear forces developed in the pile were calculated by considering the condition for equilibrium of the portion of pile below G.L. To evaluate the ultimate lateral capacity of pile, the value of H was increased in small steps. The load corresponding to which all the pile elements fail or the maximum nodal bending moment or shear force exceeds the yield values for the pile in particular, whichever is less, had been chosen to be the ultimate static lateral capacity of the pile.

2.3 Pile under lateral cyclic load

The cyclic response of the pile in clay is governed by two significant phenomena: (i) Degradation of p_{iu} and E_s at the nodal points and the effect of the loading rate. (ii) Shakedown effect induced from the gradual accumulation of irrecoverable plastic deformation developed in the soil at the interface as reflected by development of soil-pile gap in the vicinity of G.L.

Cyclic analysis in regards to the soil degradation may be carried out in two alternative approaches, viz. cycle-by-cycle analysis or composite analysis. In the former case, enormous computational effort is need especially when the number of cycles is quite large. In the later case however, the values of cyclic strength and stiffness of soil are to be adjusted after completion of all load cycles. This is an approximate method although has been reported by other researchers to yield quite promising results. The second approach being faster is adopted in present analysis.

The degradation of soil strength and stiffness was quantified by a term soil degradation factor D_{si}, defined as the ratio of the cyclic to pre-cyclic values of nodal soil strength and stiffness. As stated by Idriss et al. [11], for soft clay, $D_{si} = N^{-t}$, where, N is the number of cycles and t is denoted as degradation parameter. As recommended by Vucetic et al. [12], the parameter't' happened to be a unique function of the cyclic normal strain that the soil was subjected to at the nodal points. For clay, the following correlation was considered: $t=\in/(A + B\in)$, '$\in$' being the nodal normal strain and 'A' and 'B' were the two soil parameters whose values were to be determined from cyclic undrained tri-axial (CyUT) or cyclic undrained direct simple shear (CyUDSS) tests. It should be mentioned at this stage that the nodal strains in soil were computed as one-sixth the nodal displacement normalized by external pile diameter, after Poulos [2].

Since the undrained strength and stiffness of cohesive soil increases linearly with the logarithm of loading rate, as per Poulos [3], the nodal rate factor was expressed as, $D_{ri} =1+F \log_{10}(\lambda_i / \lambda_d)$, where, λ_i was the nodal strain rate, λ_d was a datum strain rate and 'F' is a constant termed as rate factor. The value of 'F' should be calculated from the results of any undrained monotonic test carried out at different rates. The term λ_I was estimated in present computation as the average value for one quarter cycle which introduces the relation λ_i=4 f α_i, although for better results, it is hereby recommended by the author to use different values at different instants in a complete cycle since the velocities at the nodes vary at every instant. Hence, the combined nodal soil degradation factor was formulated as,

$$D_i = [1+F \log_{10}(\lambda_i / \lambda_d)] [N^{- \in_i/(A + B\in_i)}] \tag{6}$$

It has already been stated earlier that the effect of soil pile separation has *not* been taken into consideration during cyclic loading in progress. However, while calculating the axial post-cyclic capacity of piles, the effect of shakedown was incorporated. Starting from the uppermost soil element, a soil-pile gap was supposed to be developed for those elements, where yielding took place. This assumption was not in agreement with that proposed by Poulos and Davis [8] who had recommended the separation to take place at any depth behind the pile whenever half the lateral soil pressure induced at that depth exceeds the corresponding in-situ stress which the author does not believe to be appropriate enough on the basis of the initial analysis carried out by him indicating gap to proceed to a considerable depth which was unrealistic. However, the cyclic axial capacity of the pile was calculated considering no contribution on the frictional resistance at the interface where gap has developed and degraded values of soil strength and stiffness for the remaining portion of the interface where no soil-pile separation developed. The lateral capacity of pile after completion of all load cycles will be hereafter denoted as 'post-cyclic' capacity rather than cyclic capacity and will indicate the pile capacity after cyclic loading when drainage has not yet taken place. This is not in agreement with that of the other researchers (e.g., Dyson [6] and Randolph [7]) who denoted the post-cyclic capacity as the value after drainage took place after cycling. Also, unless stated otherwise, henceforth in this paper, the term 'degradation factor' will indicate the degradation factor for axial pile capacity.

2.4 Computational algorithm

The analysis was carried out step-by-step sequentially. The steps of computation in the present analysis were as follows: (i) Analysis under lateral static load was conducted using initial input parameters for soil, pile and load. The unknown lateral soil pressures and nodal displacements were calculated. (ii) The combined nodal soil degradation factors were calculated and the new values of p_{iu} and E_{si} were evaluated. (iii) Steps (i) and (ii) above were repeated until the desired convergence was achieved. (iv) The depth up to which soil-pile gap developed during cyclic loading is calculated. (v) The post-cyclic axial capacity of pile is calculated considering the effect of gap and the degraded values of soil strength and stiffness at the interface. (vi) The degradation factor D_f for ultimate axial capacity of pile, defined as the ratio of its post-cyclic to pre-cyclic values, was evaluated.

The degree of convergence considered in the present computational analysis was 0.1%. It was observed that further reducing of the value of this quantity initiated enormous computational effort and in extreme case, convergence could not be achieved. The axial capacity of pile was calculated following standard method considering the skin friction, end bearing and self weight of the pile. The method is available in any standard text book [3].

2.5 Development of computer software

The entire computation was carried out using a user-friendly computer software LCYC developed by the author in Fortran-77 language. The flowchart of the software developed is presented in Figure 2.

3. Results and Discussion

The theoretical methodology and the relevant computer software LCYC was utilized to obtain a number of computational results. The results were compared with available experimental results and relevant observations were made to arrive at specific conclusions. These results are discussed in detailed in the preceding section.

3.1 Model tests of Dyson

Dyson [5] carried out experimental investigation on the pile-soil interaction due to lateral loading (monotonic and cyclic) in sand and silt. All tests were carried out in a beam centrifuge machine at 100g and 160g. The results obtained were compared with the existing models and relevant conclusions were arrived at.

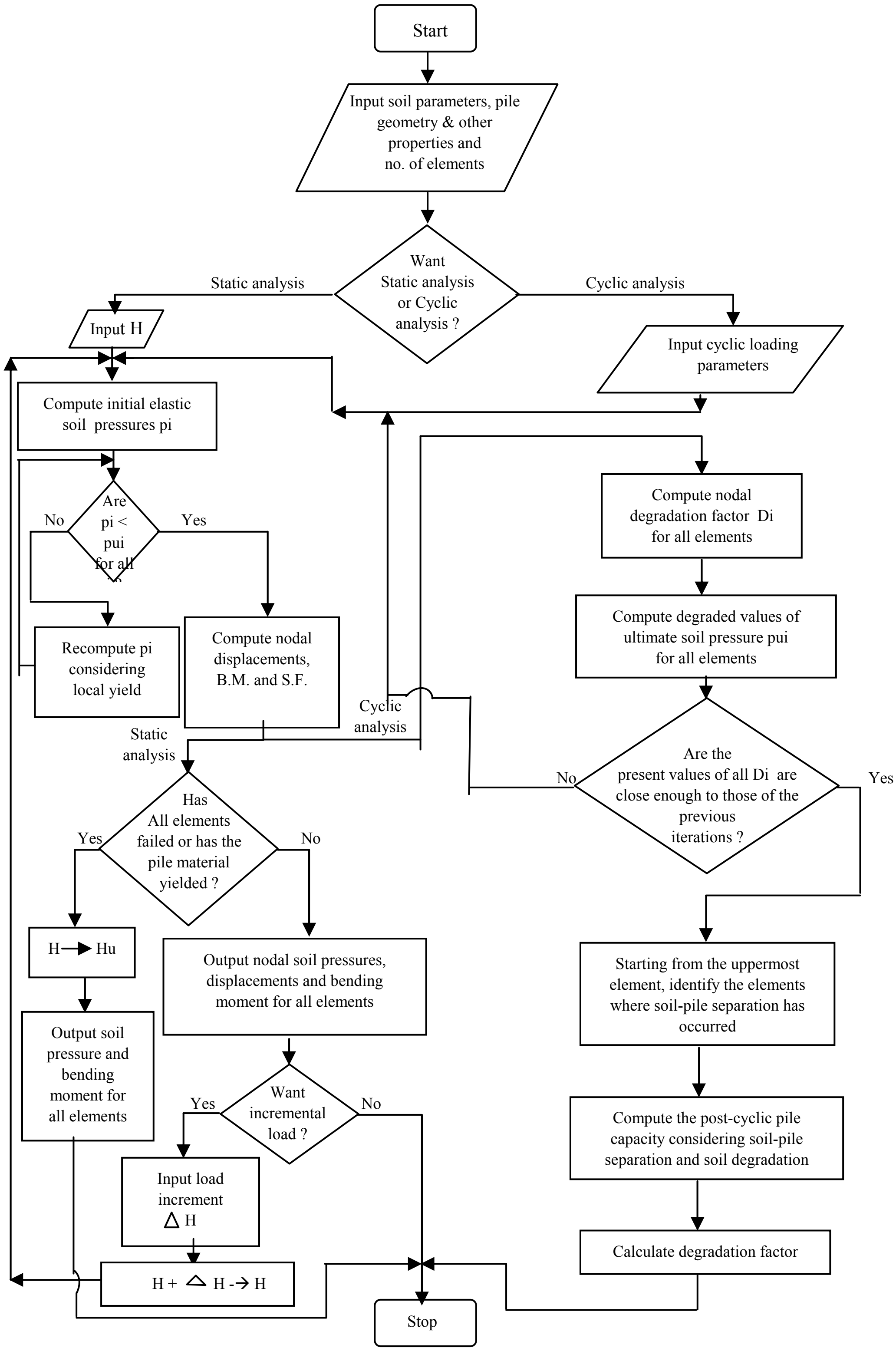

Figure 2. Flowchart of the software LCYC

Although the soil material was actually saturated silt, Dyson [5] pointed out that its behavior was more closely relevant to the conventional behavior of soft clay. Hence, the computational model developed under this project, which is specifically for saturated clay, has been used to study the analytical results in case of cyclic loading. The t-Є variation has been assumed reasonably.

Figure 3 represents the computed and the experimental cyclic pile deflection profile at N=100. It should be mentioned at this stage that no rotation has actually taken place at the pile head due to the chosen boundary condition although it appears because of Excel package that notable head rotation has taken place. Notable deviation between the computed and experimentally derived deflection profiles may be observed in the vicinity of G.L. which progressively diminished with increase in depth. However, for z/L>0.3, the deviation is quite small.

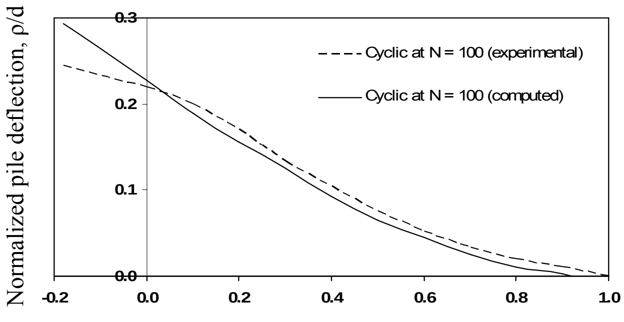

Figure 3. Comparison of computed static and cyclic pile deflections with available experimental results of Dyson [5]

The computed and the experimental profiles for the pile bending moment at N=100 are depicted in Figure 4. The basic natures of the two curves are similar but the deviation in magnitude is fairly large, the computed values being on the higher side. The contra-flexure has been observed to occur at a normalized depth of 0.75 against the experimental value of 0.3.

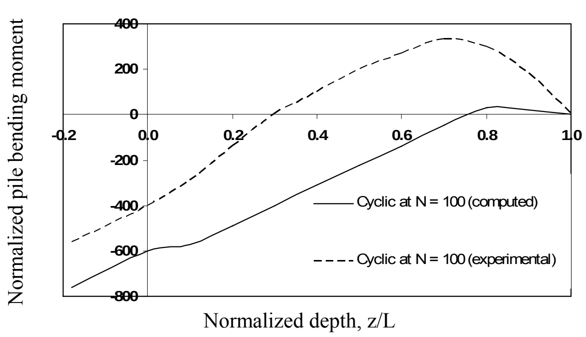

Figure 4. Comparison of computed static and cyclic pile bending moments with available experimental results of Dyson [5]

3.2 Laboratory tests of Basak

Basack [14] carried out detailed experimental investigations to understand the effect of lateral cyclic loading on axial response of piles in soft cohesive soil. Experiments were performed with 2 x 2 stainless steel pipe pile embedded in saturated uniform soft clay bed. The pile head conditions were partially fixed.

For computations, a few soil parameters were needed to be assumed in addition. These were: α = 0.9, γ = 18 KN/m^3 and μ = 0.5. As recommended by Poulos and Davis [8], the value of E_S for was taken as 8000 KPa for c_U = 20 KPa.

For cyclic analysis, the values of A and B were arbitrarily chosen as 0.105 and 81.82 respectively, in absence of cyclic tri-axial test data. As observed from the experimental results, the variation of degradation factor was in the range of 0.8-0.975 in spite of higher level of amplitude (76%). This indicates that the degradation of strength and stiffness of soil which took place adjacent to interface was sufficiently low. Therefore, the degradation parameter't' and hence the curve assumed for cyclic analysis has been kept sufficiently low.

The values of Fρ was assumed reasonably as 0.1. From UU tri-axial test results, the value of λ_r was calculated as 0.07895.

The computed load-deflection responses for single pile are presented in Figure5. The curves were observed to be fairly hyperbolic in nature.

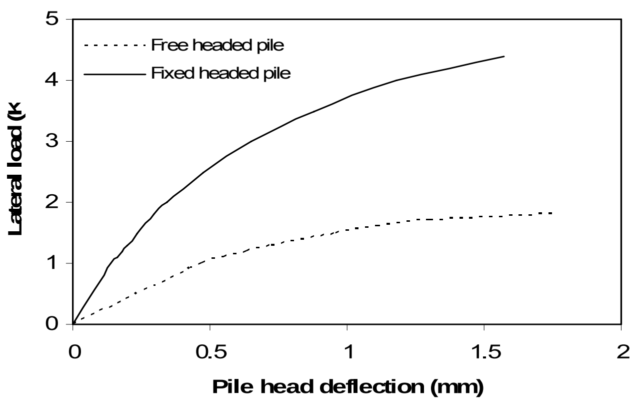

Figure 5. Computational load-deflection curves [5]

The average deviation of the values of degradation factors were within about 15-20 % in comparison to the experimentally obtained values, the theoretical values being on the lower side. It seems that such deviation occurred primarily because of the assumed values of A and B, in absence of cyclic tri-axial test data, whereas in reality, the actual pattern of variation of 't' with 'γ_C' might not be truly hyperbolic and might have a different range of values. The variation of theoretical degradation factors with no. of cycles, frequency and amplitude are depicted in Figures 6-8. It was observed that the pattern of variation of the computed values of degradation factors with the cyclic loading parameters was quite similar to those obtained experimentally.

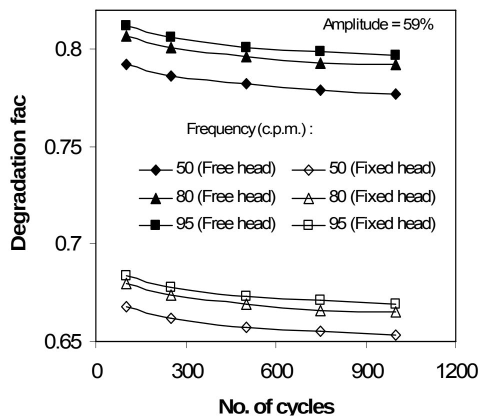

Figure 6. Variation of computed degradation factor with number of cycles

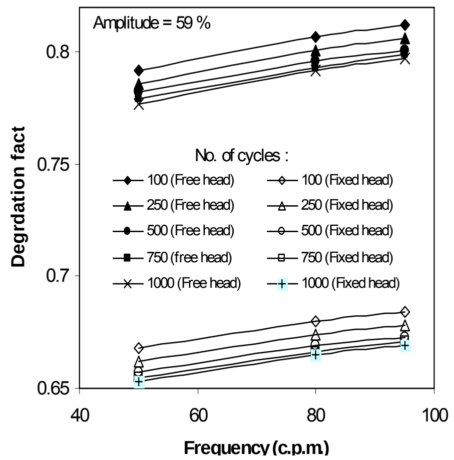

Figure 7. Variation of computed degradation factor with frequency

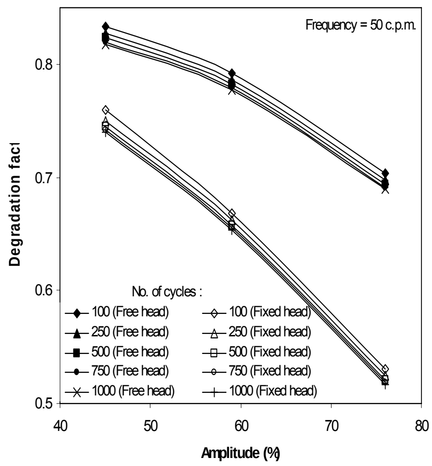

Figure 8. Variation of computed degradation factor with amplitude

4. Conclusion

From the entire experimental investigation, the following conclusions may be drawn:

Under the effect of lateral cyclic loading, the ultimate axial capacity of pile foundation alters. For soft clay soil, the degradation factors are less than unity indicating deterioration of soil pile interactive performance. In order to understand the effect of lateral cyclic loading on axial performance of single pile in clay, a theoretical model based on boundary element analysis was developed. A computer software LCYC was developed to carry out the analyses. The model and the software developed were observed to yield quite promising results. Apart from the prediction of load-deflection response, bending moment and displacement profiles along embedded pile length and ultimate capacity in case of pile under lateral static loading, computation of post-cyclic axial pile capacity and hence the degradation factor and its variation with cyclic loading parameters were some of the essential features that were taken care of by the software.

The theoretical results were compared with available experimental results and reasonably good agreements were noted. However, the methodology requires accurate data related to t-ϵ variation of the soil under consideration to be obtained from laboratory testing of undisturbed samples. As observed from theoretical analysis, the degradation factor decreases with no. of cycles and increases with frequency asymptotically. The pattern of variation of the degradation factor with amplitude was found to be non-linear.

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