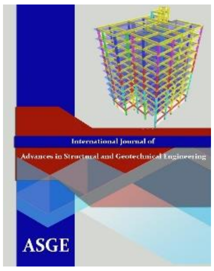

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Behavior of Single Pile in Consolidating Clay under the Application of Axial Load

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ABSTRACT

Negative skin friction (NSF) is considered one of the most popular problems in the design of piled foundations in consolidating soils. NSF develops on piles when the settlement of the surrounding soils is greater than that of the piles. When the relative shear displacement (γ_E) increased than limiting value the slip at soil-pile interface is induced. As a result an additional compression force on the pile called drag-load (P_{DL}) and an extra pile displacement called down-drag (W). Both NSF and down-drag (W) are time dependent. However, the mechanism of NSF mobilization on pile is still not well understood and often several pile design codes provide different recommendations to calculate NSF. At the meantime, codes dealing with down-drag calculations are scarce. In the present study, the behavior of single piles embedded in consolidating clay is analyzed by three dimensional finite element model using (ABAQAUS, 6.14). In this model, clay was simulated using Cam Clay model (CCM) to represent the soil strength while the friction at soil-pile interface and sand was represented by Mohr-Coulomb model (MCM). The pile was described by a ٣D linear elastic model. In the analysis, one dimensional consolidation theory was coupled with the NSF developed along the pile. The analysis revealed drag-load, down-drag, soil settlement and excess pore water pressure at different degrees of consolidation (U). An extensive parametric study was carried out to investigate the effect of spatial parameters on drag-load (P_{DL}) and down-drag (W). The numerical results indicated that; when designing a pile foundation in consolidating soil it is crucial to take into account the pile-soil-fluid water interaction to achieve the actual performance of pile-soil system and to avoid overestimating drag-load on piles.

Keywords: *Drag-load, Down-drag, Consolidating clay, Slip condition, Axial load.*

1.Introduction:

Negative skin friction (NSF) induced on single piles in consolidating clay was realized since sixties of the last century and attracts the attention of many researchers, [1], [2], [3], [4], [5], [6], [7], [8], [9], [10], [11], [12], [13], [14] and [15]. Piles in consolidating clay are subjected to drag-load (P_{DL}) and down-drag (W), the first affects the structure deficiency of the piles while the later affects the serviceability of the piles. The investigation of down-drag (W) of single pile has received little attention from researchers, [3], [16], [6], [17], [18], [13] and [14]. The awareness of the performance of single pile in consolidating clay is a crucial aspect in the design of pile foundation. Poulos,[19] and Fellenius,[12] reported that down-drag (W) of pile should be included in pile design. Despite of the research effort, Lee,[20] pointed out that the assessment of drag-load (P_{DL}) imposed on a pile in consolidating clay and the exhibited down-drag (W) are neither based on unsuitable factors nor realistic design approaches. The researchers reported from field studies that, piles got out of service due to excessive down-drag (W), [21], [22], [8], [23] and [24]. While, Lee,[20] pointed out that down-drag (W) may input some problems on pile serviceability. Development and magnitude of drag-load (P_{DL}) on single pile is dependent on soil model of pile-soil interface and the method of analysis. Notably that the performance of single pile in consolidating clay was achieved by different methods; analytical method,[25], [26] and [27], simplified linear elastic analysis, [28] and [29] and no-slip linear elastic finite element analysis,[20] and [30]. The later method revealed results which overestimate the drag-load.

2. Verification and modelling:

2.1. Geometry and model discretization:

A soil domain of cylindrical shape having a diameter of 60m, which is equal to 50 times the pile diameter, and a height of 35.0m, in which the pile is contained, was discretized. The thickness of soil domain below the pile tip was 12m while pile length 23m. Due to symmetry only one-quarter of the pile and soil domain was modeled. The clay and sand domain were simulated using C3D4P (a 4-node linear tetrahedral, coupled displacement-pore water pressure elements), while the pile was simulated using (a 4-node linear tetrahedral elements). More than 41214 elements were used to discretize the pile and soil domain containing 11006 nodes. The mesh was staged refinement by using elements most refined along pile-soil interface and the size of elements gradually increased as the distance increased radially from the pile center line.

2.2. Boundary condition:

The vertical boundary of soil domain is located far away from the pile by a distance equal to 25 times the pile diameter (30m), while the bottom boundary at depth equal to about 1.5 times pile length. The vertical and radial displacements of soil elements at bottom boundary were restrained by the means of pinned supports. The soil elements along the vertical boundaries of soil medium were restrained against radial displacements; only vertical displacement is allowed using roller supports on the side boundary of the soil domain. Top boundary of soil domain is free to move at vertical and radial directions. At the top and bottom of clay layer, the excess pore water pressures were set equal to zero at any time.

2.3. Interface modeling:

The interface between the pile and the soil was modeled using surface to surface algorithm in (ABAQUS, 6.14). Surfaces are in contact where the relative displacement (Δ) between master node (Pile) and slave node (Soil) is less than 5mm. If shear displacement becomes more than 5mm, the slip between surfaces (soil and pile) will occur. The interface elements which are of zero thickness transfer shearing force across the interface between pile and soil. Friction between the pile-soil interfaces before slippage was simulated by Mohr-Coulomb Model (MCM) with friction interface angle (δ). In the present study (μ) was set equal to 0.3 at pile-soil interface for clay and sand.

2.4. Constitutive model and material parameters:

The soil is represented with appropriate constitutive parameters in the numerical simulation. The subsoil soft clay is simulated by Cam Clay Model, (CCM). Three parameters are implemented in the model λ, k and m. The parameter m is the slope of critical state line in $q - P'$ space.The pile is simulated as a 3D linear elastic material. Sand layer is simulated by Mohr- Coulomb Model (MCM). The model is configured for flow of water to complete dissipation of excess pore water pressure. The flow of water is kept on during the analysis, and the excess pore water distributions within the clay layer were computed at time intervals. In the analysis, the flow of water is kept on, while the properties of clay λ, k and m are kept constant independent of the effective stress variation. The drained-coupled analysis is simultaneous action of pore water fluid for with the volumetric change of clay soil; therefore pore water flow is simultaneous actions with the drag load, and down drag of the pile. The inelastic behavior of material is accompanied by volume change. Dilation angle ψ of zero is set for clay and 10° for sand.

The initial condition of soil is considered isotropic. In the developed three dimensional model, the consolidation of soft clay occurs by means of pore water fluid flow in vertical direction. Since, the clay is double open faces, flow of water takes place in vertical direction only. The analysis is carried out up to a degree of consolidation of clay equal to 90%, to save computation time.

2.5. Loading and solution steps:

In the numerical analysis, the effect of pile installation on soil properties is disregarded, so the pile is wished-in-place in soil domain. Two cases of pile loading are considered, simultaneous loading and post consolidation loading. For both, the first step of the analysis is the geostatic deformations of soil domain. At the end calculation of first step, numerical analysis indicated negligible deformations. During the geostatic step the interaction between pile and soil is allowed, as well as all boundary conditions are implemented. In case of simultaneous loading, where the pile head load (P_{HL}) and surcharge load on ground surface (q) were applied simultaneously, the second step is the application of surcharge load (q) and pile head load (P_{HL}) at once. Pile head load (P_{HL}) is applied as uniformly distributed load over the pile head cross section while the surcharge load (q) is applied on unlimited area on ground surface. The pile head load (P_{HL}) and surcharge load (q) are kept constant. The consolidation process continues up to predefined time corresponding to a specified degree of consolidation (U). On reaching specified time the output results are harvested. Different elapsed time intervals corresponding to different degree of consolidation (U) are considered. Each time interval is started from initial conditions. But in case of post consolidation loading the second step is the application of surcharge load (q) only as distributed load on ground surface, then consolidation continues up to a predefined time corresponding to degree of consolidation (U) equal to 90%. At this stage of consolidation the pile head load (P_{HL}) was applied and the process continues up to a predefined time.

2.6. Verification of FE model:

Lam [13], carried out axis-symmetric modeling using ABAQUAS software and conducted laboratory tests using centrifuge. The pile was circular of diameter 1.2m wished-in-place in 18m of soft clay followed by12m of sand. Water level was at ground surface. The pile is a friction pile of 17.7m length with pile tip 0.30 m above the clay-sand interface. Soil domain was 24m diameter with soil properties shown in table (1). Surcharge load of 45kPa was applied on ground surface. Analyses were continuing to time equal to 60 months which represent 90% degree of consolidation. In the verification process both axisymmetric and 3D analysis were carried out and compared by Lam [13]. Drag-load verses depth were shown in figures (1). The figure indicates reasonable agreement between measured shaft load compared with results obtained using 3D and axisymmetric analysis implemented in the present study. However, 3D analyses resulted in closer shaft loads to those measured.

Table 1: Constitutive model parameters of single pile verification.

Properties	Clay	Sand	Pile
Unite weight γ (kN/m^3)	16.3	19.4	27
Modulus of elasticity E (kN/m^2)	λ = 0.14 k = 0.012 m = 0.98	1.2 E 5	7 E 7
Poisson's ratio (ϑ)	0.35	0.3	0.35
void ratio (e_0)	1.6	0.4	0.2
Frictional angle at critical state ($\emptyset'$)	25°	29.7°	-
Angle of dilation (ψ')	0	8.3°	-
Coefficient of earth pressure at rest K_0	0.58	0.39	-
Permeability $K_s(m/s)$	1e -8	1e -5	1e -10
$\gamma_{critical}$ (mm)	5.0	5.0	-

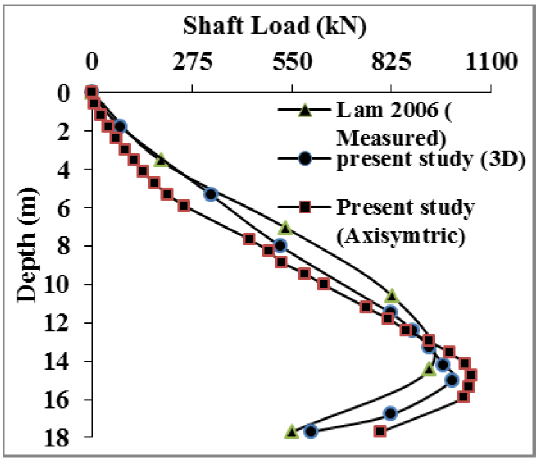


Fig. 1 Comparison of shaft load at degree of consolidation 90% from Lam [13] and present study.

3. Computed results:

3.1 Effect of pile head load:

3.1. 1 Simultaneous loading:

The axial load from superstructure on the pile in consolidating clay increased the downward pile displacement. Which resulted an increase in the pile tip load (P_{TL}) and the positive friction load (P_{FL}) and at a specified degree of consolidation (U) the shear displacement (ε_{γ}) at soil-pile interface within the clay layer shall be decreased, and consequently the drag-load (P_{DL}) decreased.

To investigate the effect of pile head load on the mobilized drag-load (P_{DL}) and down-drag (W) of the pile, comparative study was carried out on two similar piles of total length 23m, 5m of the pile length was impeded in sand, figure (2). In the first case study, the pile was loaded simultaneously with pile head load (P_{HL}) and surcharge load (q) on ground surface. The pile head loads (P_{HL}) were 1000, 1500 and 2000kN. In the second case study, the pile is free-head-load pile. In both cases two surcharge loads of 30 and 40kPa were considered. Table (2) presents the constitute parameters of soil and pile.

Figure (3) shows the accumulative shaft load (P_S) verses Z/L of free-head-load pile, while figure (4) shows the accumulative shaft load (P_S) verses Z/L of a pile of pile-head-load (P_{HL}) of 2000kN. The figures are at different degrees of consolidation and different surcharge load (q). The figures indicated that the accumulative pile shaft load (P_s) increased nonlinearly with depth, to peak drag-load value ($P.P_{DL}$), then decreased to pile tip load (P_{TL}) value. The nonlinearity in accumulative shaft load (P_s) reflects the nonlinearity of (NSF) distribution along the pile. Therefore, NSF is not directly proportional with vertical effective overburden stress. This may be due to that the NSF may lead to a reduction in effective overburden pressure.

In case of free-head-load pile the relative depth of peak drag-load ($P.P_{DL}$) is the location of neutral plane (N.P) is 0.8. This relative depth (Z_{HL}/L) moves little upward to (Z_{HL}/L) of 0.75 corresponding to pile head load (P_{HL}) of 2000kN. The relative depth of (N.P) is more or less equal to the relative depth of clay-soil interface. Therefore, the depth of (N.P) in case of end bearing pile is at the clay-sand interface. The depth of N.P is not appreciably affected by pile head load up to a load equal to 2000kN.

The peak drag-loads ($P.P_{DL}$) of free-head-pile load and pile loaded with different head load up to 2000kN were calculated and normalized as expressed in equation (1).Then drawn against the degree of consolidation (U), figure (5). The figure indicated that, the peak drag-load ($P.P_{DL}$) imposed on the pile decreased as the pile head load increased. The peak drag-load ($P.P_{DL}$) developed on the pile at (U) equal to 90% was 887.2kN and 1105.8kN in case of surcharge load (q) 30 and 40kPa, respectively and the head-pile load (P_{HL}) equal to 2000kN. The pile head load of 2000kN represents 2.54 and 1.8 times the long term drag-load, even though the pile head load was unable to eliminate appreciable percentage of the drag-load.

$$(P.P_{DL})_N = \frac{P.P_{DL}}{\pi * D * H^2 * \gamma_{eff}} \qquad \textbf{(1)}$$

Where;
D Pile diameter,
H Thickness of clay layer,
γ_{eff} Effective unite weight which expressed as ($\gamma_{wet} - \gamma_{water}$).

Table 2: Constitutive model parameters of numerical analysis model.

Properties	Clay	Sand	Pile
Unite weight γ (kN/m^3)	16.3	19.4	27
Modulus of elasticity E (kN/m^2)	λ = 0.14 k = 0.012 m = 0.98	1.2 E 5	7 E 7
Poisson's ratio (ϑ)	0.45	0.35	0.15
void ratio (e_0)	1.6	0.4	0.2
Frictional angle at critical state ($\emptyset'$)	25^0	45°	-
Angle of dilation (ψ')	0	10°	-
Coefficient of earth pressure at rest K_0	0.58	0.5	-
Permeability $K_s(m/s)$	1e -9	1e -5	1e -13
$\gamma_{critical}$ (mm)	5.0	5.0	-

3.1. 1.1 Effect of surcharge load (q):

Figures (3) and (4) revealed that the drag-load (P_{DL}) imposed on the pile and the peak drag-load ($P.P_{DL}$) increased with the increase of surcharge load (q). But the figures indicated that the peak drag-loads at U equal to 40% in case of free-head-load pile are 83% and 86% of that imposed on the pile at U equal to 90% in case of surcharge load 40kPa and 30kPa, respectively. In case of pile head load 1000, 1500 and 2000kN, the above values became (84% & 82%), (83%&81%) and (79%&81%). Figure (5) shows that the peak drag-load ($P.P_{DL}$) increased with the increase of (U) up to U equal to 40%. For (U) bigger than 40%, the peak drag-load ($P.P_{DL}$) increase with nearly constant rate. The pile head load has not affected the progress of the drag-load. Also the degree of consolidation (U) has not affected the location of the (N.P). Therefore, the drag-load imposed on the pile is mainly due to the surcharge load (q).

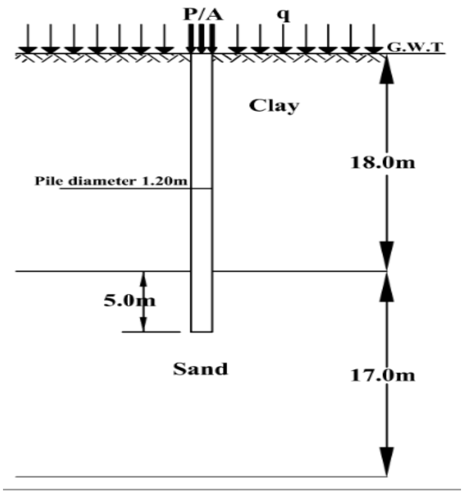


Fig. 2 Soil profile.

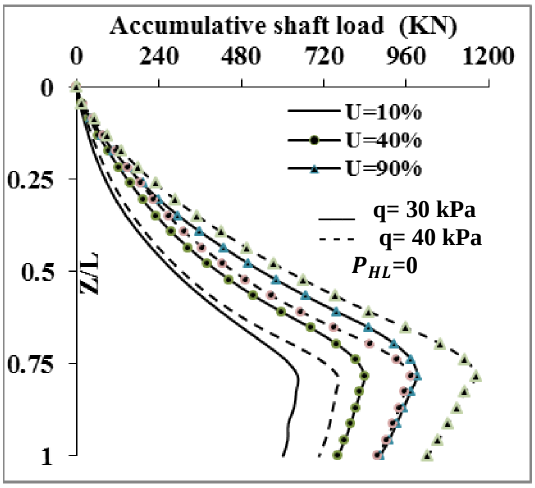


Fig. 3 Distribution of accumulative shaft load versus depth at different U for pile head load 0kN.

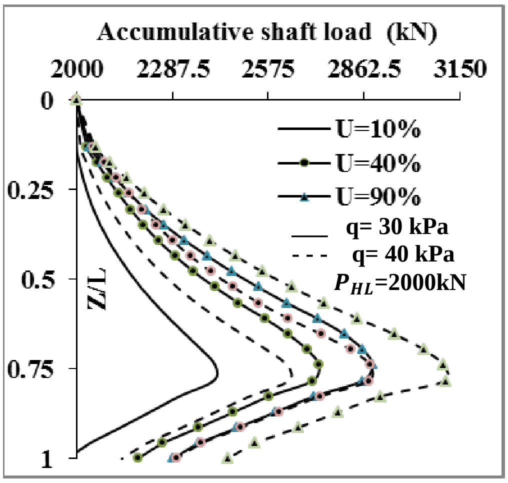


Fig. 4 Distribution of accumulative shaft load versus depth at different U for pile head load 2000kN.

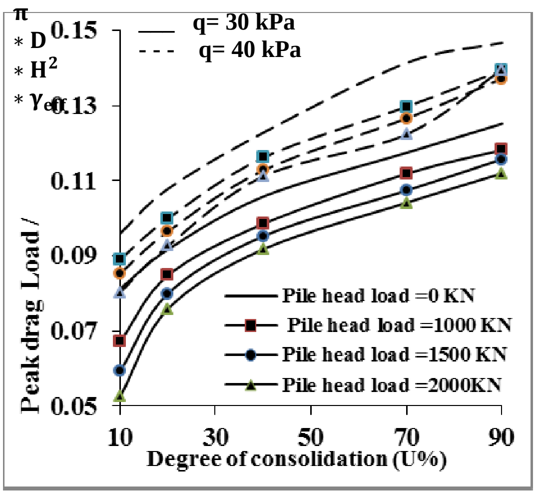


Fig. 5 Normalized peak drag-load versus degree of consolidation at different pile head load.

3.1. 1.2 Negative skin friction (NSF):

Figures (6) and (7) present the distribution of (NSF) and (PSF) along free-head-load pile and pile loaded with 2000kN on pile head. The figures indicate the positive skin friction (PSF) increased with the increase of pile head load due to the increase of shear displacement (ε_γ) along the pile below the (N.P). Also the positive skin friction increased with the increase of the degree of consolidation U, due to the increase of drag-load which tends to push the pile downward. The (NSF) increased with depth below ground surface attaining a peak value at relative depth (Z_{NF}/L) equal to 0.7 to 0.62 at U equal to 10% and 90% respectively, in case of free-head-load pile. The long term (NSF) at U equal to 90% is not appreciably affected by pile head load (P_{HL}). At the meantime, the (PSF) increased with the increase of pile head load.

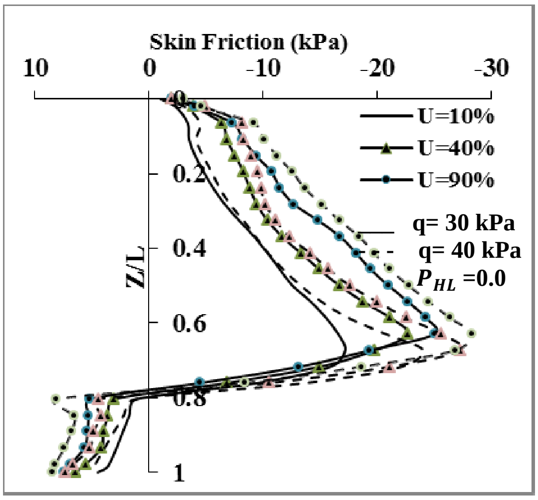


Fig. 6 Distribution of skin friction versus depth at different U for pile head load 0kN.

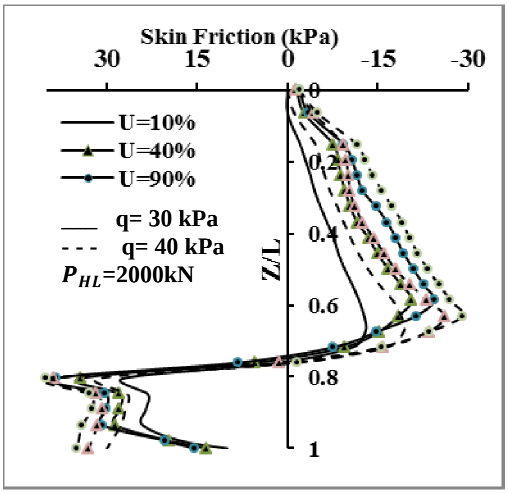


Fig. 7 Distribution of skin friction versus depth at different U for pile head load 2000kN.

3.1. 1.3 Excess pore water pressure (u):

The excess pore water pressure was normalized with the surcharge load (q) in figures (8) and (9), for free-head-pile load and for pile with pile head load equal to 2000kN. The figures indicated that the excess pore water pressure depends upon the magnitude of surcharge load, degree of consolidation and mostly independent of pile head load (P_{HL}). The peak of excess pore water pressure takes place at mid height of clay layer. Still at U equal to 90%, the excess pore water pressure represent 12.5% of that induced at U equal to 10%. Therefore, drag-load imposed on the pile still in progress.

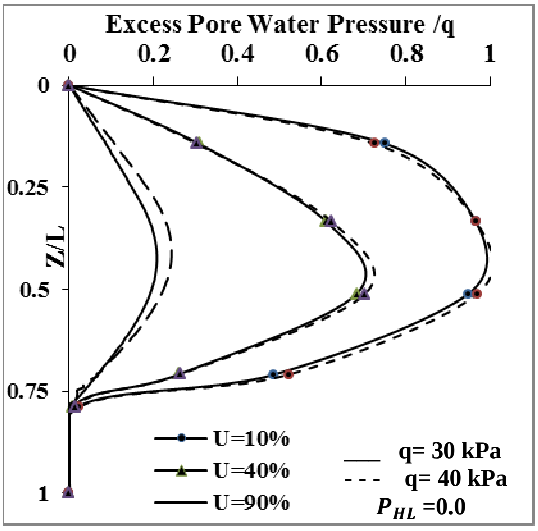


Fig. 8 Normalized excess pore water pressure versus depth at different U for pile head load 0kN.

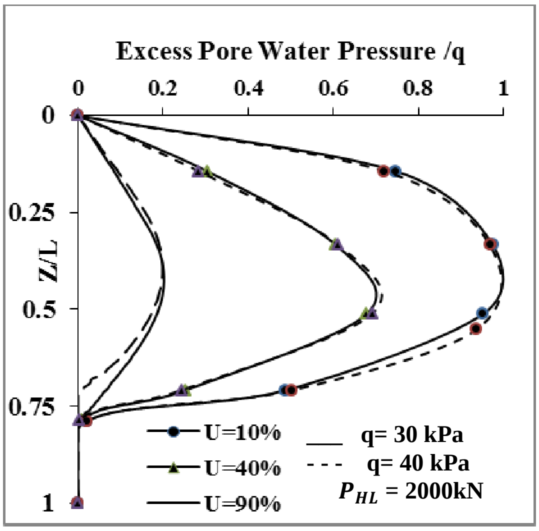


Fig. 9 Normalized excess pore water pressure versus depth at different U for pile head load 2000kN.

3.1. 1.4 Mobilization length (L_m):

The mobilized length (L_m) of the pile, along which the negative skin friction was developed and attained the maximum limiting value was obtained and normalized to pile length (L_m/L_{pile}), then drawn verses the degree of consolidation, figure (10). The figure indicates that the mobilized length attained maximum value and stabilized at degree of consolidation equal to 40% in case of surcharge load 30kPa, and at degree of consolidation equal to 20% in case of surcharge load equal to 40kPa. Beyond these degrees of consolidation the mobilized length is almost of constant value and becomes less dependent on the pile head load. Therefore the drag-load induced on the pile attained most of its value at degree of consolidation 40% in case of surcharge load 30kPa and at degree of consolidation equal to 20% in case of surcharge load equal to 40kPa. The pile head load tends to decrease the peak drag-load; therefore at specified degree of consolidation the mobilized length decreased as the pile head load (P_{HL}) increased but this decrease is not appreciable.

The transition zone is the length which the negative skin friction decreased from the limiting value to vanish at neutral plane. The mobilized length in addition to the transition zone is of length equal to the length measured from ground surface up to neutral plane. The negative transition zone length normalized to pile length verses degree of consolidation was shown in figure (11). The variation of negative transition length with the degree of consolidation and pile head loads are with reverse to those of the mobilization length. The length of transition zone decreased with the increase of (U) and attained to stabilized length at (U) equal to 20% in case of (q) equal to 40kPa, and at (U) equal to 40% in case of (q) equal to 30kPa.

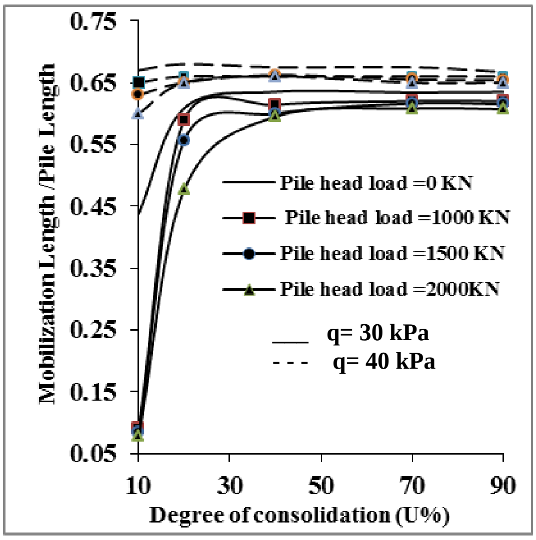


Fig. 10 Normalized mobilization length verses degrees of consolidation at different pile head load.

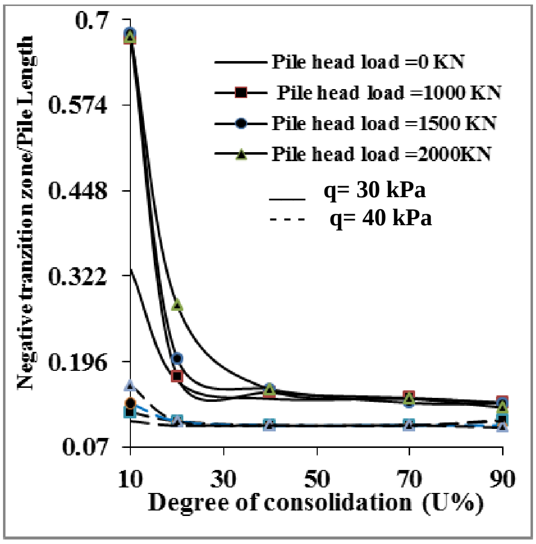


Fig. 11 Normalized negative transition zone verses degrees of consolidation at different pile head load.

3.1. 1.5 Normalized down-drag (W_n):

The down-drag of the pile (W) was normalized with the soil surface settlement, far away from the pile due to soil consolidation. The normalized down-drag of pile of length 23.0m, with 5m embedment in sand are presented in figure (12). The figure revealed that (W_n) decreased with the increase of the degree of consolidation (U). As the degree of consolidation (U) increased the down-drag (W) of the pile increased due to the increase of drag-load (P_{DL}), also the soil surface displacement (S) due to consolidation of clay increased but (W_n) decreased. In case of free-head-pile load there is no appreciable effect of surcharge load (q) on the values of (W_n). With the increase of pile head load, the down-drag (W) increased and consequently the normalized down-drag (W_n) increased. This is attributed to that the pile head load (P_{HL}) has not appreciable effect on the consolidation settlement of clay, while as the pile head load (P_{HL}) increased the down-drag (W) increased. At surcharge load (q) equal to 30kP, the values of (W_n) exhibited by the pile at (U) equal to 40% varies from 61.14% to 57.7% of that at (U) equal to 10% according to pile head load (P_{HL}). The lower percentage corresponding to smaller pile head load and the higher percentage to higher pile head load. At U equal to 40% and surcharge load (q) equal to 40kPa the values of (W_n) varies from 59.13% to 54.7% of that at (U) equal to 10%. At U equal to 70%, these percentage of (W_n) decreased to some value between 46% to 42% in case of (q) equal to 30kPa, while in case of (q) equal to 40kPa, these percentages become some value between 44% to 38.8%.

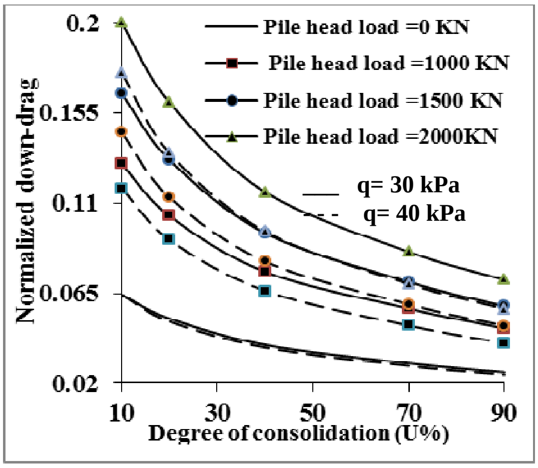


Fig. 12 Normalized down-drag verses degrees of consolidation.

3.2.1 Post consolidation loading:

To facilitate the construction in swap area and low level land, the ground surface has to be backfilled. Therefore, the clay may attain a specified degree of consolidation (U) before loading the piles by the superstructure load. Therefore, consolidation process of clay takes place before the piles are subjected to dead load (P_{HL}) from super structure, and lock-in drag-loads (LP_{DL}) was acting on the piles. To simulate this problem in the numerical analysis, the pile was loaded after the soft clay has attained 90% degree of consolidation under the action of surcharge load (q). Two pile head loads were implemented in the study 1000kN and 2000kN. The surcharge load (q) was set equal to 30kPa. These cases were compared with the case of simultaneous loading. The achieved results from the two-case study were presented on graphs, the y-axis (ordinate) represent the parameter under study while the x-axis (coordinate) represent the degree of consolidation in case of simultaneous loading and represent time elapsed in case of post consolidation loading.

The peak drag-load ($P.P_{DL}$) imposed on the pile either simultaneously loaded with surcharge load (q) placed on ground surface or post consolidation loading is not affected by the time history of pile loading. The peak drag-load ($P.P_{DL}$) is the same, figure (13). The lock-in drag-load in case of post consolidation loading decreased instantaneously after the application of the pile head load (P_{HL}), then increased again to the values corresponding to simultaneous loading. The decrease in peak drag-load ($P.P_{DL}$) is attributed to the immediate pile displacement resulted from pile-head-load (P_{HL}). The pile displacement reduced the shear displacement (ε_{γ}) mobilized along pile-soil interface and consequently reduced the (NSF) which had been developed along the pile-soil interface. The built up of peak drag-load ($P.P_{DL}$) of post consolidation loading is attributed to that the consolidation settlement of clay still in progress. The ($P.P_{DL}$) acting on the free-head-pile load was some value between (2 to 7%), and (12 to 15%) more than that imposed on piles of simultaneously loading in case of pile head load 1000kN and 2000kN, respectively, figure (13).

Figure (14) present the time history of soil surface settlement which increased by 15% more than that displacement at U equal 90%. The increase of surface soil settlement is due to post consolidation pile head load and the progress of clay consolidation. Once more the pile head load had not appreciable effect on consolidation settlement of clay layer keeping in mind that the soft clay still proceeding in consolidation and the increase of surface soil displacement is expected. However in 2910 days; which represent the time interval after (U) equal to 90% the soil surface displacement increased by 15%, but there is no increase in drag-load. This can be attributed to that the shear displacement causing the limiting (NSF) was attained in relatively shorter time.

It is worth noting that the down-drag (W) exhibited by piles, receiving lock-in drag-load (LP_{DL}), decreased by 30% to40% according to pile head load (P_{HL}), once the load was applied on the pile head. This is due to decreasing in drag-load (P_{DL}), pulling down the pile, figure (15). The post consolidation loaded pile exhibit down-drag from the commencement of the load application, which is equivalent to a degree of consolidation 96% after 2910 days, about 93% of that takes place in simultaneous loading at U equal 90%, figure (15). The drag-load (P_{DL}) and down-drag (W) of the pile are independent of the superimposed load time history.

Figure (16) presents that the long term mobilization length of negative skin friction of the pile simultaneously loaded and post consolidation loading. The mobilization length of both simultaneously loaded pile and post consolidation loaded pile are the same, figure (16).The mobilization length of post consolidation loaded pile was created by the surcharge load on ground surface.

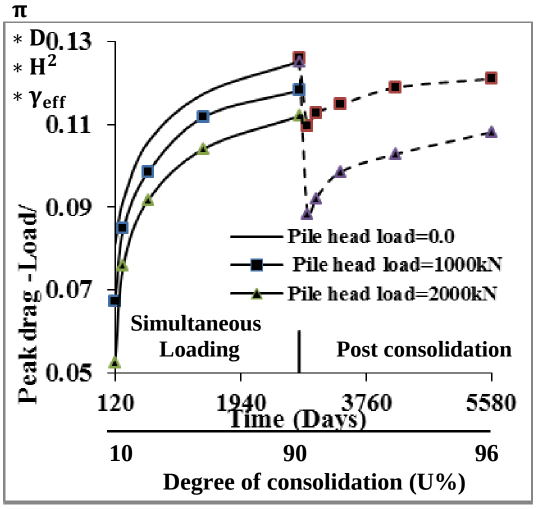


Fig. 13 Normalized peak drag-load verses time and degrees of consolidation at different (P_{HL}).

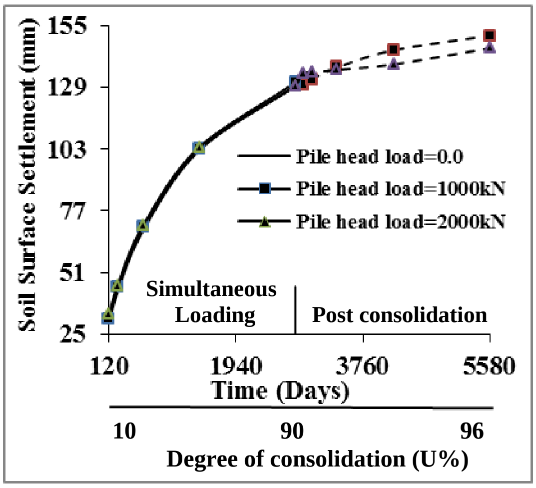
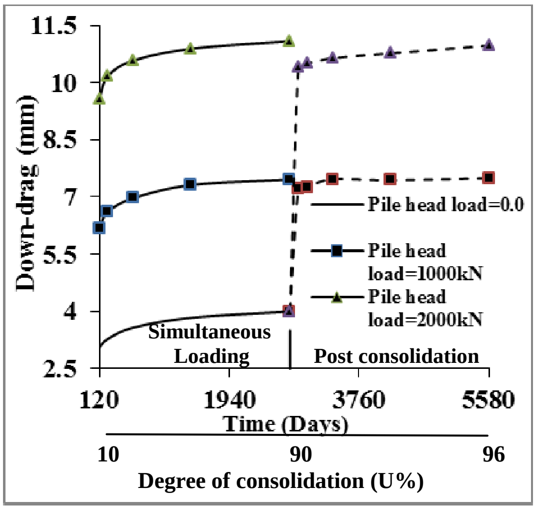


Fig. 14 Soil surface settlement verses time and degrees of consolidation at different (P_{HL}).



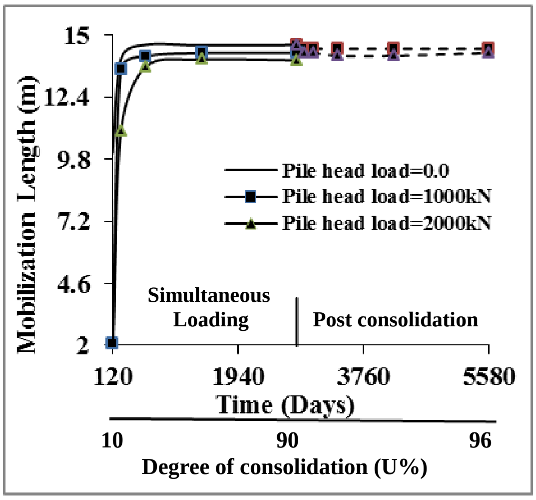


Fig. 15 Down-drag verses time and degrees of consolidation at different (P_{HL}).

Fig. 16 Mobilization Length verses time and degrees of consolidation at different (P_{HL}).

4. Summary and conclusions:

Single pile in consolidating clay was simulated and analyzed using finite element taking into consideration linear elastic-plastic interface. The mobilized (NSF) coupled with excess pore water pressure was considered. The course of investigation revealed the following conclusions:

1. The distribution of accumulative shaft load (P_S) and the mobilized (NSF) along the pile are nonlinear.
2. The depth of neutral plane (N.P) of end bearing pile is located very close to clay-sand interface. The negative skin friction (NSF) increased with depth attaining its peak value at relative depth (Z_{NF}/L) ranging from 0.62 to 0.7, depending upon pile head load.
3. The location of neutral plane (N.P) is independent of pile head load (P_{HL}) and degree of consolidation (U).
4. The peak drag-load ($P.P_{DL}$) imposed on a pile decreased as the pile head load (P_{HL}) increased. The decrease of drag-load (P_{DL}) is not as such that very big pile head load (P_{HL}) is required to eliminate the drag-load (P_{DL}) totally. Pile head load (P_{HL}) of about two times the peak drag-load ($P.P_{DL}$), only eliminate about 3%to 6% of the long term peak drag-load ($P.P_{DL}$) imposed on free-head-load pile.
5. The positive skin friction (PSF) developed along the pile below (N.P) increased with the increase of pile head load (P_{HL}) and degree of consolidation (U). Meaning positive skin friction (PSF) is time dependent.
6. The drag-load (P_{DL}) imposed on a pile in consolidating clay is mainly due to surcharge load (q). Eighty percent of drag-load (P_{DL}) was mobilized when the degree of consolidation (U) achieved 40%, meaning after 600days. The pile head load (P_{HL}) has no effect on the rate of the mobilization of drag-load (P_{DL}).

7. Most of the drag-load (P_{DL}) imposed on pile takes place in the early stage of consolidation irrespective of pile length.
8. The excess pore water pressure (u) depends upon the magnitude of surcharge load (q) and the degree of consolidation (U) where excess pore water pressure (u) independent of pile head load (P_{HL}).
9. The mobilization length (L_m) established along the pile earlier as the surcharge load (q) increased, the maximum mobilized length was attained at U equal to 40% and 20% in case of surcharge load 30kPa and 40kPa, respectively. The maximum mobilization length is independent of pile head load (P_{HL}).
10. The down-drag (W) exhibited by the pile still in progress as long as the long term consolidation of clay in progress.
11. The lock-in peak drag-load (LP_{DL}) mobilized on free-head-load pile, at U equal to 90% decreased instantaneously at the time of the acting pile head load (P_{HL}), then built up again to attain the same value. The drag-load imposed on the pile is not dependent on the time-load history of pile head load (P_{HL}).The down-drag (W) of the pile is independent of the load history of pile head load (P_{HL}) either loaded simultaneously with surcharge load (q) or loaded at degree of consolidation (U) of 90%.
12. The down-drag (W) depends on the elastic shortening of the pile under the acting loads and on the magnitudes of pile tip load (P_{TL}), positive friction load (P_{FL}) and drag-load(P_{DL}).

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