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A Program Development for Prediction of Negative Skin Friction on Piles by Consolidation Settlement

압밀침하를 고려한 말뚝의 부마찰력 예측 프로그램 개발

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요 지

재하 하중에 의해 압밀 되는 지반에 관입 된 말뚝의 지지력을 예측하고자 MATLAB을 이용한 GUI환경에서 Pile NSF(Pile Negative Skin Friction)프로그램이 개발되었다. 본 연구에서 제안된 방법은 일차원 유한변위 압밀이론이 적용될 수 있도록 비선형 하중 전이법에 의한 일차원 토질-말뚝 모델 프로그램인 OpenSees 를 확장하였다. 개발된 프로그램은 압밀과정 중에 발생하는 토질-말뚝의 경계면 변화는 물론 유한 점토층의 저감이 고려되는 Mikasa의 유한 변위이론을 융합하는 특성을 가지고 있다. 더 나아가 말뚝 타설 후에 재하성토에 의해 발생하는 지반의 압밀상태도 해석시에 고려할 수 있는 특징을 지니고 있다. 본 연구에서 제안된 방법에 의한 프로그램 해석은 부마찰력이 발생하는 말뚝에 대하여 말뚝 장기시험 사례 결과와 비교하여 타당성이 검증되었고, 압밀침하를 반영한 말뚝의 부마찰력 예측치는 측정된 결과와 잘 일치하고 있음을 보여주고 있다.

Abstract

The microcomputer program PileNSF (Pile Negative Skin Friction) is developed by the authors in a graphical user interface (GUI) environment using MATLAB® for predicting the bearing capacity of a pile embedded in a consolidating ground by surcharge loading. The proposed method extends the one-dimensional soil-pile model based on the nonlinear load transfer method in OpenSees to perform an advanced one-dimensional consolidation settlement analysis based on finite strain. The developed program has significant features of incorporating Mikasa's finite strain consolidation theory that accounts for reduction in the thickness of the clay layer as well as the change of the soil-pile interface length during the progress of consolidation. In addition, the consolidating situation of the ground by surcharge filling after the time of pile installation can also be considered in the analysis. The program analysis by the presented method has been verified and validated with several case studies of long-term test on single piles subjected to negative skin friction. Predicted results of negative skin friction (downdrag and dragload) as a result of long term consolidation settlement are shown to be in good agreement with measured and observed case data.

Keywords : Finite strain consolidation theory, Negative skin friction, Nonlinear load transfer method, Piles

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1. Introduction

The problem of a pile embedded in a consolidating soil that is subjected to surcharge loading from fill is treated in this paper. It has long been recognized that when piles are situated in a consolidating soil-mass, a downward force or negative skin friction is induced in the pile because of the shearing stresses from downward movement of the soil relative to the pile (Poulos and Davis, 1980; Fellenius, 2006). Typical methods for the analysis of negative skin friction on piles employ a two-dimensional (2D) or three-dimensional (3D) finite element mesh of the soil-pile continuum in a coupled approach (Indraratna et al., 1992; Jeong et al., 1997; Comodromos and Bareka, 2005). A one-dimensional (1D) soil-pile model with the load transfer method is used by Alonso et al. (1984) to predict the negative skin friction on single piles in an uncoupled analysis with the classical Terzaghi (1943) assumptions of one-dimensional consolidation theory with an elastoplastic and bilinear transfer function for the soil-pile interface. Wong and Teh (1995) also utilized the load-transfer approach using a hyperbolic criterion for the soil spring element model at the pile shaft.

This paper suggests the use of a simplified one-dimensional soil-pile analytical model using the nonlinear load-transfer method that is uncoupled with the one-dimensional consolidation theory in terms of finite strain (Mikasa, 1965) for the prediction of negative skin friction in single piles. The microcomputer program PileNSF (Pile Negative Skin Friction) is developed by the authors in a graphical user interface environment for this objective. As compared to the coupled and continuum approach, the uncoupled method of analysis is capable of independently incorporating an advanced settlement theory that considers the finite reduction in thickness of the clay layer during the consolidation process, in contrast to the infinitesimal strain and constant thickness assumed by the Terzaghi theory of consolidation. In addition, where in most cases piles are usually driven or installed after some time from the start of surcharge load or fill, the method can consider the conditions of delayed pile installation as one of its significant and unique features. It is also noteworthy to mention that Mikasa's finite strain theory has the advantage of being used in conjunction with settlement or strain data being mostly available in the field rather than on excess pore pressures with the Terzaghi theory. In this study, a finite difference program is made for a one-dimensional consolidation analysis of a homogeneous clay layer that is overlain by sand or fill layers, and/or underlain by sand layers or rock (Fig. 1), that is integrated with the finite element program OpenSees (2000) for the load transfer analysis and prediction of the induced negative skin friction from the soil settlements. The integrated software program, PileNSF, thus analyzes the problem of negative skin friction on single piles in a two-step approach: (1) finite difference method for the analysis and prediction of effective soil stresses and settlements based on Mikasa's (1965) generalized one-dimensional consolidation theory in terms of finite strain, and (2) nonlinear load-transfer and finite element analysis with the open-source program OpenSees (2000) as the executable, for the prediction of pile settlements and forces that is subjected to axial load at the pile head and/or imposed displacements from the consolidation settlement of the surrounding soil layers. The integrated analysis program is developed in a graphical user interface (GUI) environment using MATLAB® that can as well serve the individual purpose and independent function of which is developed, that is: (a) as a 1D consolidation analysis program for a homogenous compressible soil (clay) subjected to surcharge load, or (b) as a pre-processor and postprocessor for OpenSees in the prediction of load capacity and settlement of single piles subjected to axial load and imposed soil

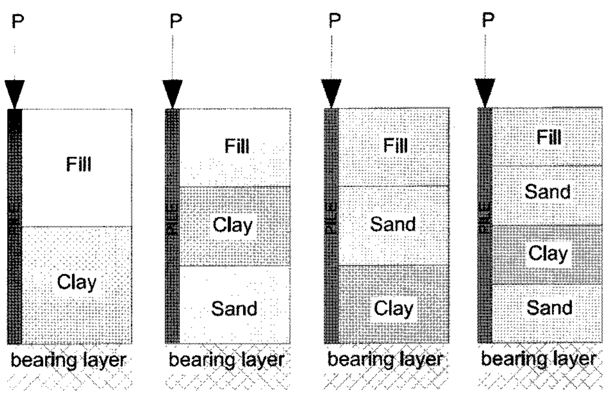


Fig. 1. Example soil-pile profiles analyzed by the program PileNSF

displacements. The various features of the program are demonstrated and validated in a case history of long-term pile test and negative skin friction measurement found in the referred literature.

2. One-dimensional Soil-Pile Model and Method of Analysis

Figure 2 (b) shows the one-dimensional finite element model based on the method of nonlinear load transfer, used for the analysis of a single pile embedded in a compressible layer that is subjected to consolidation from a surcharge load on its surface (Fig. 2a). The load-transfer method is probably the most widely used technique to study the problem of single axially loaded piles, and is particularly useful when the soil behavior is clearly nonlinear and when the soil surrounding the pile is stratified. The side soil springs represent the resistance of the soil-pile interface in skin friction (T-z spring) and a vertical Q-z spring is used to model the load-deformation property in end-bearing of the layer at the bottom of the pile. These soil springs are nonlinear representations of the soil reaction T at the sides or Q for the tip, versus displacement z. The free ends of the vertical side springs are subjected to imposed vertical displacements from settlements (S) of the surrounding soil at any time during the progress of consolidation, and the other ends attached to the pile. The ultimate properties of the side springs are determined from the effective stresses at the time when negative skin friction is to be evaluated.

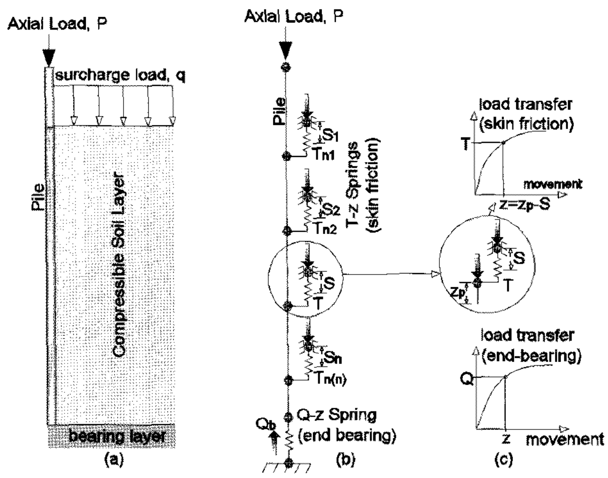


Fig. 2. (a) Pile in compressible soil layer undergoing consolidation settlement from surcharge load, (b) One-dimensional soil-pile discretization, and (c) Typical nonlinear load transfer curves

The transfer of load through shear along the sides of the shaft is given by the following differential equation (Reese and O'Neill, 1987),

$$EA\frac{d^2 z_p}{dz^2} = P.f_s \tag{1}$$

where z_p = movement of the pile or shaft at depth z, A = cross-sectional area of pile, E = modulus of elasticity of the pile material, P = pile perimeter, f_s = shear force per unit area of load transfer from the shaft to the soil at depth z. With the presence of on-going settlements around the vicinity of the pile from soil consolidation, if we let S = soil settlement and z_p = pile movement, then a criteria for T-z load transfer can be established based on the following;

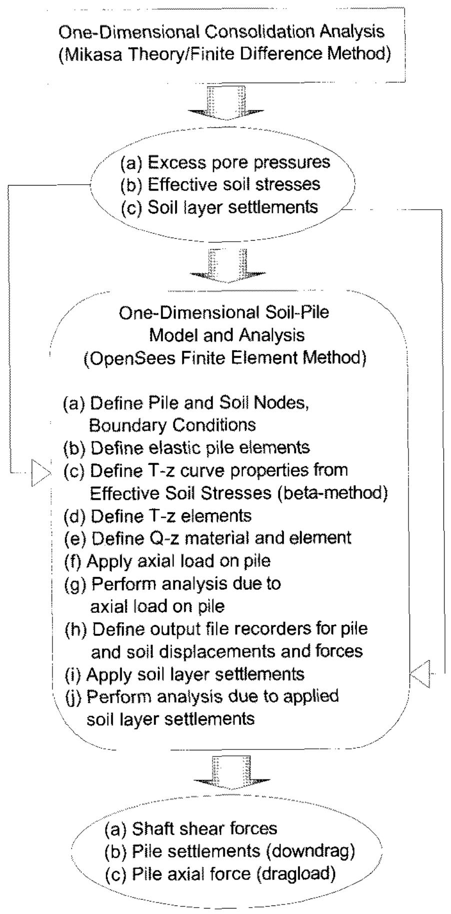


Fig. 3. Flowchart for uncoupled analysis procedure

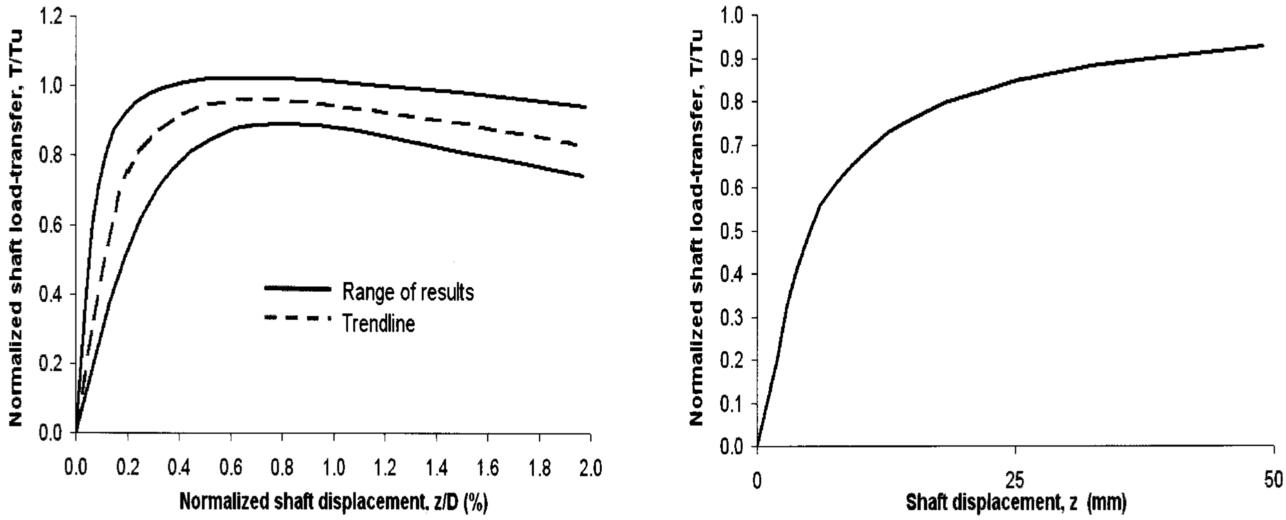
(a) $(S > z_p)$, T = negative (downward drag force)
(b) $(S < z_p)$, T = positive (upward shaft resistance)
(c) $(S = z_p)$, T = 0 (equilibrium, location of neutral plane)

Thus, the shear force per unit area of load transfer f_s in Eq. (1) can also be viewed and written in a generalized form as a function of the pile and soil movements, z_p and S, respectively, as

$$f_s = f(z_p - S) \tag{2}$$

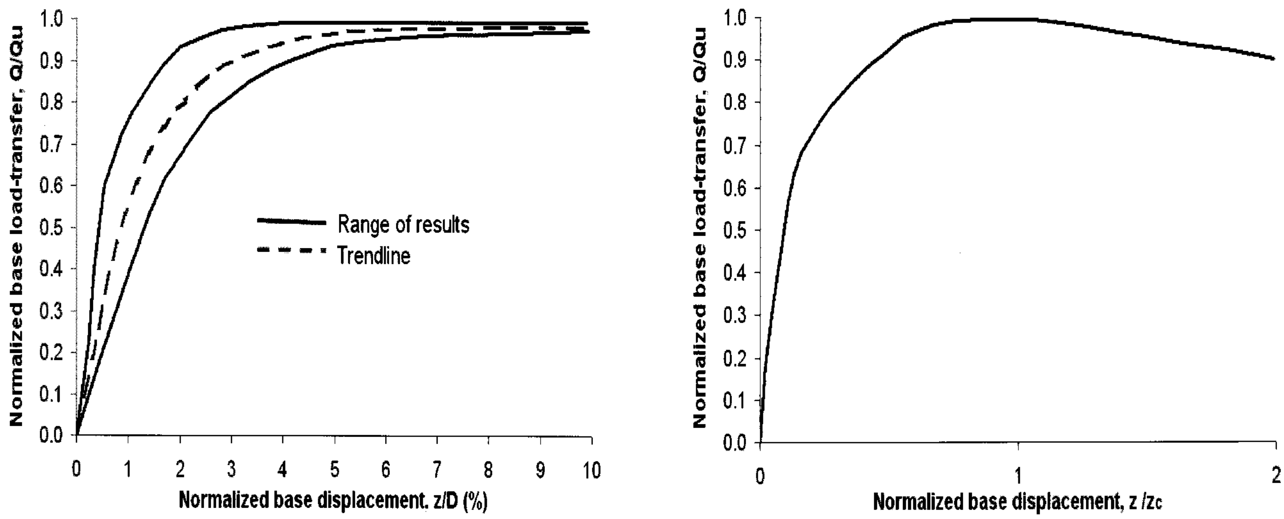
The general flowchart for the uncoupled analysis procedure is shown in Fig. 3. The analysis and finite element model shown in Fig. 2 (b) are being implemented through the open-source software program OpenSees (2000) that is integrated in the program PileNSF. For the consolidation analysis, the soil profile is defined in layers and sublayers along with their consolidation properties and drainage boundary conditions.

In this study, the backbone of the nonlinear T-z curve for clay is approximated from Reese and O'Neill's (1987) relation (Fig. 4a) and the backbone of the nonlinear T-z



(a) (b)

Fig. 4. Normalized nonlinear load-transfer curves in skin-friction (T-z curves): (a) T-z curve for clay (Reese and O'Neill, 1987), (b) T-z curve for sand (Mosher, 1984)



(a) (b)

Fig. 5. Normalized nonlinear load-transfer curves in end-bearing (Q-z curves): (a) Q-z curve for clay (Reese and O'Neill, 1987), (b) Q-z curve for sand (Vijayvergiya, 1977)

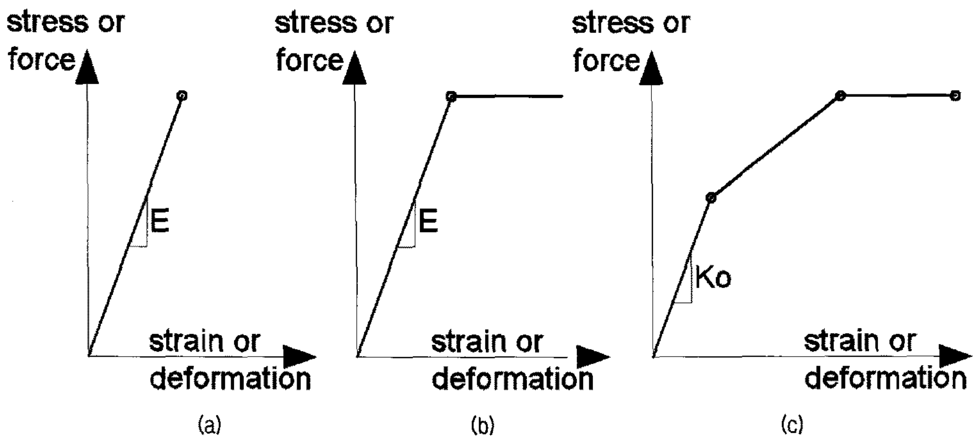


Fig. 6. (a) Elastic (Linear), (b) Elastic-plastic (Bilinear), and (c) Tri-linear load-deformation relation for end-bearing resistance (Q-z curve) using uniaxial material models in OpenSees (Mazzoni, et al., 2006)

curve for sand is approximated from Mosher's (1984) relation (Fig. 4b), all of which are being implemented using the *TzSimple1* materials (Boulanger, 2003a) in OpenSees. Similarly, the backbone of the nonlinear Q-z curve approximates Reese and O'Neill's (1987) relation for drilled shafts in clay (Fig. 5a) and the backbone of the Q-z curve for sand is approximated from Vijayvergiya's (1977) relation (Fig. 5b), all of which are being implemented using the *QzSimple1* materials (Boulanger, 2003b) in OpenSees. In addition, the end-bearing properties of layer at the base of the pile can also be either rigid or compressible with applicable load-deformation properties of the Q-z curve that can be defined as elastic (linear), elastic-plastic (bilinear), or trilinear curve as shown in Figs. 6 (a), 6 (b), and 6 (c), respectively.

Only closed-ended piles are considered that can either be of concrete or steel that has elastic section properties. The time of installation of the pile can be specified anytime from the start of surcharge loading and the output time specified at which negative skin friction is to be evaluated. Significant outputs from consolidation analysis are the settlement profile end effective stress at any time. Significant outputs from negative skin friction analysis are the downdrag (settlement) and dragload (axial force) on the pile. The structure of the input file used for the nonlinear load transfer analysis in OpenSees is shown in Fig. 7. A more detailed explanation of the various finite modeling and analysis commands that is implemented in this study can be found from the OpenSees user manual (Mazzoni et al., 2006).

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##########################################################
# UNCOUPLED ANALYSIS OF NEGATIVE SKIN-FRICTION ON PILES #
# BASED ON FINITE STRAIN CONSOLIDATION THEORY AND THE   #
# NONLINEAR LOAD-TRANSFER METHOD                        #
##########################################################
# units m, kN, kPa, s
wipe
#
model basic -ndm 2 -ndf 3
source Nodes.tcl
source Constraints.tcl
geomTransf Linear 1
source PileElements.tcl
source TzMaterials.tcl
source TzElements.tcl
source QzMatElement.tcl
source AxLoadPatterns.tcl
source Analysis.tcl
loadConst -time 0.0
source SDispPatterns.tcl
source Recorders.tcl
source Analysis.tcl
#
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Fig. 7. Structure of the input file for nonlinear load-transfer analysis in OpenSees

3. One-dimensional Consolidation Theory in Terms of Strain

Considering that numerous consolidation tests and field observations have produced the data mostly on compression strain or settlement rather than on excess pore pressure, it is then more convenient to express consolidation in terms of strain according to Mikasa's (1965) generalized consolidation equation theory (Eq. 3).

$$\frac{\partial \varepsilon}{\partial t} = C_v \frac{\partial^2 \varepsilon}{\partial z^2} \tag{3}$$

Equation 3 holds the same fundamental assumption as that of the Terzaghi theory wherein the coefficient of consolidation, C_v and coefficient of volume compressibility, m_v are constant during the consolidation process, the influence of self-weight consolidation is neglected, and the change in thickness of a clay layer is negligible, that is, an infinitesimal strain is assumed. The relationship between the strain with the change in vertical effective stress $\Delta\sigma'_v$ at the depth and the time under consideration neglecting the effect of self-weight is given as,

$$\varepsilon = m_v \Delta\sigma_v' \tag{4}$$

Equations (3) and (4) relate the soil settlements and change in effective soil stresses through the strain ε as a function of time t and depth z during the consolidation process. In the case of highly compressible clays and soft soils in reclaimed grounds, z coordinate of each clay element changes its value during consolidation process. To effectively consider the reduction in thickness during the consolidation process, a new notion of original coordinate z_o is introduced, which is the z-coordinate in the original state in which the clay is supposed to have the same volume ratio f_o everywhere, and Mikasa's consolidation Eq. (3) is then transformed into Eq. (5) that can be integrated for the case of finite strain.

$$\frac{\partial \zeta}{\partial t} = C_v \zeta^2 \frac{\partial^2 \zeta}{\partial z_0^2} \tag{5}$$

In Eq. (5), t = elapsed time, $\zeta = f_o/f$ (consolidation ratio) (where f denotes the volume ratio that is related to the void ratio and is equal to (1+e), and f_o = original volume ratio). The consolidation ratio ζ is related to the nominal (arithmetic) strain $\bar{\varepsilon}$ or natural (logarithmic) strain ε by,

$$\zeta = \frac{1}{1-\bar{\varepsilon}} = \exp(\varepsilon) \tag{6}$$

The solutions of the consolidation Eqs. (3) and (5) can be approximated by a numerical technique through the finite difference method that is also presented in detail by Kim and Mission (2009). For both Eqs. (3) and (5), the number of layer subdivisions n and size of time step Δt are carefully selected so that the stability of the solution process may be maintained (Verruijt, 1995). The settlements can be calculated from the strains by approximating the integral with the trapezoidal method,

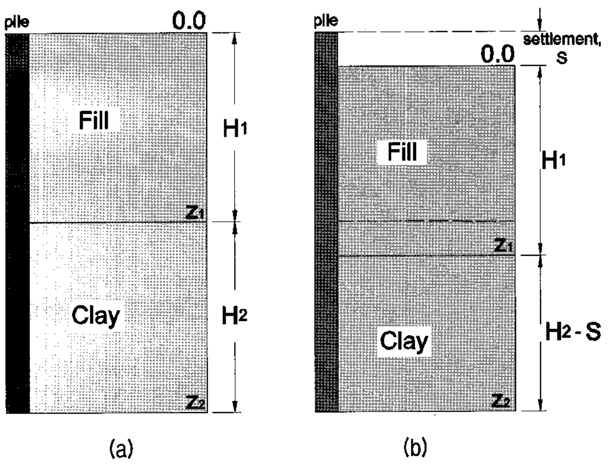


Fig. 8. Effect of large settlements on soil-pile interaction length: (a) before settlement, and (b) after settlement

$$S = \int_0^{H_o} \varepsilon \cdot dz_o \approx 0.5(\varepsilon_1 + \varepsilon_2)\Delta z_o + \ldots + 0.5(\varepsilon_{n-1} + \varepsilon_n)\Delta z_o \tag{7}$$

Considering the finite reductions in thickness of the compressible clay layer with time during the progress of consolidation, the changes in the soil-pile interaction length within the clay layer should also be accounted for as shown in Fig. 8. When large settlements in the clay layer exist, the calculation for the ultimate or limiting soil-pile interface shear strength (in unit of force) that is tributary for a given soil-pile node of the T-z soil spring would be overestimated if this reduction in length is not taken into account. This variability and reduction of the soil-pile interface is being considered in this study for the theoretical computations of the ultimate soil-pile skin-friction resistances in the clay layers.

4. Effective Stress Approach to Soil-Pile Interface Shear Strength

Although it is well-accepted in engineering practice that the load-transfer for piles in sand is proportional to the effective overburden stress, case histories and research findings however have demonstrated that actual shaft resistance in clays is also proportional to the effective overburden stress, where reasonable agreements have been obtained with correlations through the beta-coefficient (Fellenius, 2008) (Eq. 8). In the calculation of the ultimate or limiting skin-friction f_s with the β-method, the appropriate effective stresses σ'_v are those prevalent at the time when downdrag is to be computed (Alonso et al., 1984).

$$f_s = \beta \sigma_v' \qquad (8)$$

In this study, it is assumed that this limiting skin friction is the same both for the positive (upward) and negative (downward) direction of side shear. The coefficient β is a function of the soil type, pile material and surface roughness, and the method of pile installation. Fellenius (2008) expressed the beta-coefficient as a function of several parameters,

$$\beta = M \tan\phi' (1 - \sin\phi')(OCR)^{0.5} \qquad (9)$$

where M = tanδ'/tanϕ', ϕ'= effective soil friction angle, δ'= effective interface friction angle that is a function of pile surface texture and material, and OCR = overconsolidation ratio.

For sand, β is a function of the lateral earth pressure

Table 1. Recommended Input Parameters for Sand: Lateral Earth Pressure Coefficient (Poulos and Davis, 1980; Tomlinson, 1986)

Compactness	Relative Density, Dr (%)	Lateral Earth Pressure Coefficient, Ks
Very loose	0 to 15	0.6 to 1.0
Loose	15 to 35	1.0 to 1.4
Medium dense	35 to 65	1.4 to 1.6
Dense	65 to 85	1.6 to 2.0
Very dense	85 to 100	2.0 to 2.4

Table 2. Recommended Input Parameters for Sand: Interface friction angle (Kulhawy, 1984)

Pile material	Ratio of interface friction angle δ to soil friction angle ϕ	Typical Field Analogy
Rough concrete	1.0	Cast-in-place
Smooth concrete	0.8 to 1.0	Precast
Rough steel	0.7 to 0.9	Corrugated
Smooth steel	0.5 to 0.7	Coated
Timber	0.8 to 0.9	Pressure-treated

coefficient, K_s, and the interface friction angle, δ, expressed as,

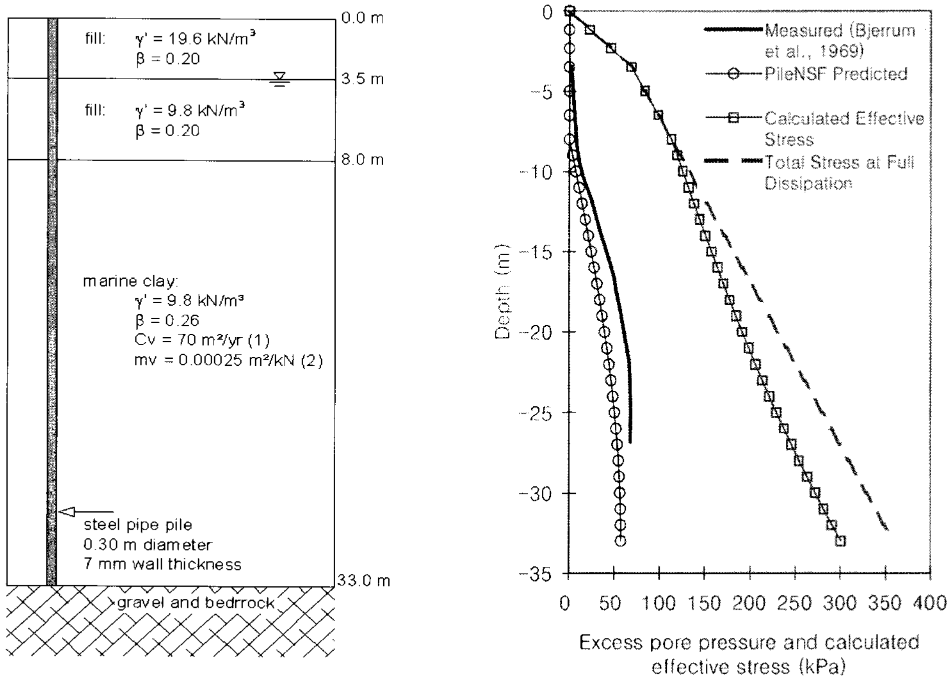
$$\beta = K_s \tan\delta \qquad (10)$$

Different values of K_s and δ have been reported in the literature (Poulos and Davis, 1980; Tomlinson, 1986; and Kulhawy, 1984) for different types of pile material and property of sand as given by the recommended values in Tables 1-2.

5. Model Application and Validation with Field Test Case Studies

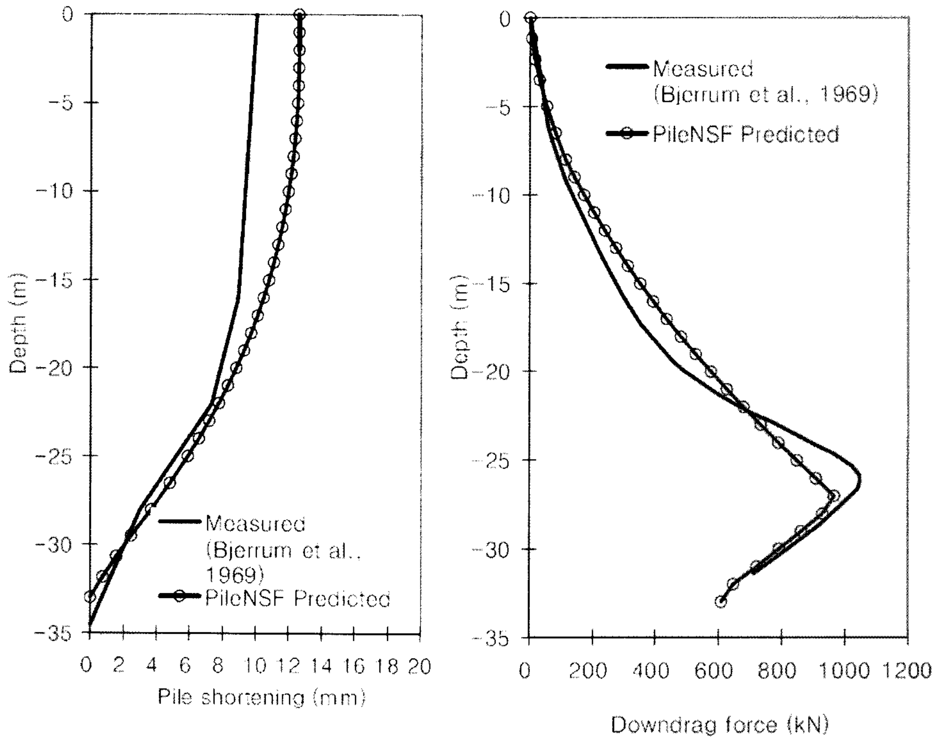
Case study 1- Pile A, Heroya Site, Norway (Bjerrum et al., 1969):

Bjerrum et al. (1969) reported full-scale investigation on an uncoated test pile (Pile A) under 8 m of fill overlying a layer of silty clay about 25 m thick. The fill had been in place for about 3 years prior to installation of the test piles. The properties of the soil-pile profile are shown in Fig. 9 (a) where peizometer measurements at the site indicated one-way drainage conditions. For the numerical analysis, the fill layer was subdivided into 6 sublayers, the clay layer subdivided into 25 sublayers, and a time step Δt = 0.005year was selected. The pile material was modeled as elastic using the given cross-sectional area, section moment of inertia, and modulus of elasticity for steel pipe. The consolidation coefficient C_v was adapted from Poulos and Davis (1980) and the compressibility coefficient m_v adopted from Alonso et al. (1984) that gives the predicted profile of excess pore pressure after 18months from pile driving as shown in Fig. 9 (b). Consolidation analysis uses the finite strain method and accounts for the reduction in soil-pile interaction length from soil settlements. The ultimate load transfer resistances in skin friction for the fill and sand layer were computed from the calculated effective stresses through their given beta coefficients. Load transfer curves in skin friction (T-z curve) for the fill layer were modeled using Fig. 4 (b) and for the clay layer using Fig. 4 (a). The end bearing load transfer at the base was modeled as elastic with Fig. 6 (a) using the reported 25 mm penetration at the pile toe at an end-bearing load of about 540 kN. The relative soil settlements imposing downdrag on the pile were calculated from the difference between the soil settlement profile at 3 years from the start of pile driving to 4.5 years at the end of pile monitoring. These relative



(a) (b)

Fig. 9. Case study 1 - Pile A, Heroya site, Norway (Bjerrum et al., 1969): (a) Soil-pile profile: (1) after Poulos and Davis (1980), (2) after Alonso et al. (1984), (b) Distribution of excess pore pressure and calculated effective stress.



(a) (b)

Fig. 10. Case study 1 - Pile A, Heroya site, Norway (Bjerrum et al., 1969): (a) Predicted and measured pile shortening, (b) Predicted and measured downdrag force

soil settlements have induced the downdrag and pile shortening on the pile (Fig. 10a), as well as dragload (Fig. 10b) wherein predictions by the PileNSF program have produced fair agreement with measured data.

Case study 2 – Pile C, Heroya Site, Norway (Bjerrum et al., 1969):

Another test pile, Pile C, a 300 mm steel pipe pile with 7 mm wall thickness was reported by Bjerrum et al. (1969) at the Heroya site subject to about 8 m of fill as shown in Fig. 11 (a). It was also assumed that full slip occurred between the fill and the pile where the pile was kept free from fill by the enlarged point. Full slip was modeled by using a zero value of beta coefficient for the soil-pile interface shear at the fill contact. The pile was modeled as elastic using the cross-sectional properties of the steel pipe material. Skin friction at the pile-fill and pile-clay interfaces were modeled using Mosher's (1984) and Reese and O'Neill's (1987) load transfer relations, respectively. A rigid end-bearing was assumed at the base where the pile was driven to bedrock, and the support point at the pile tip was modeled by a pin constraint. The pile was driven at 3 years after the surcharge filling and monitored for about 1-1/2 years. The relative settlement during the 18 month of monitoring had imposed the downdrag and dragload on the pile as shown in Figs. 11 (b) and 11 (c), respectively, wherein a fair agreement with predicted and measured data is observed.

Case study 3 - Pile G, Sorenga Site, Norway (Bjerrum et al., 1969):

Bjerrum et al. (1969) reported monitoring of downdrag force on a test pile (noted as Pile G), a pipe pile 500 mm and 8 mm wall thickness. This was also discussed by Wong and Teh (1995) in terms of the pile being driven to bedrock at 40 m. Figure 12 (a) shows property of the soil profile and the pile with the water table located at 2 m below the ground. The beta-coefficient for the fill is assumed to be 0.20, which is adapted from Poulos and Davis (1980). At the location of Pile G, the fill had been in place for 70 years and consolidation under this fill was almost complete. Nevertheless, two years after pile installation the pile head and the surrounding ground were found to have settled about 13.8 mm and 70 mm, respectively. In the present study, no consolidation analysis was performed for this case and predictions for downdrag and pile shortening were carried out to verify the applicability of

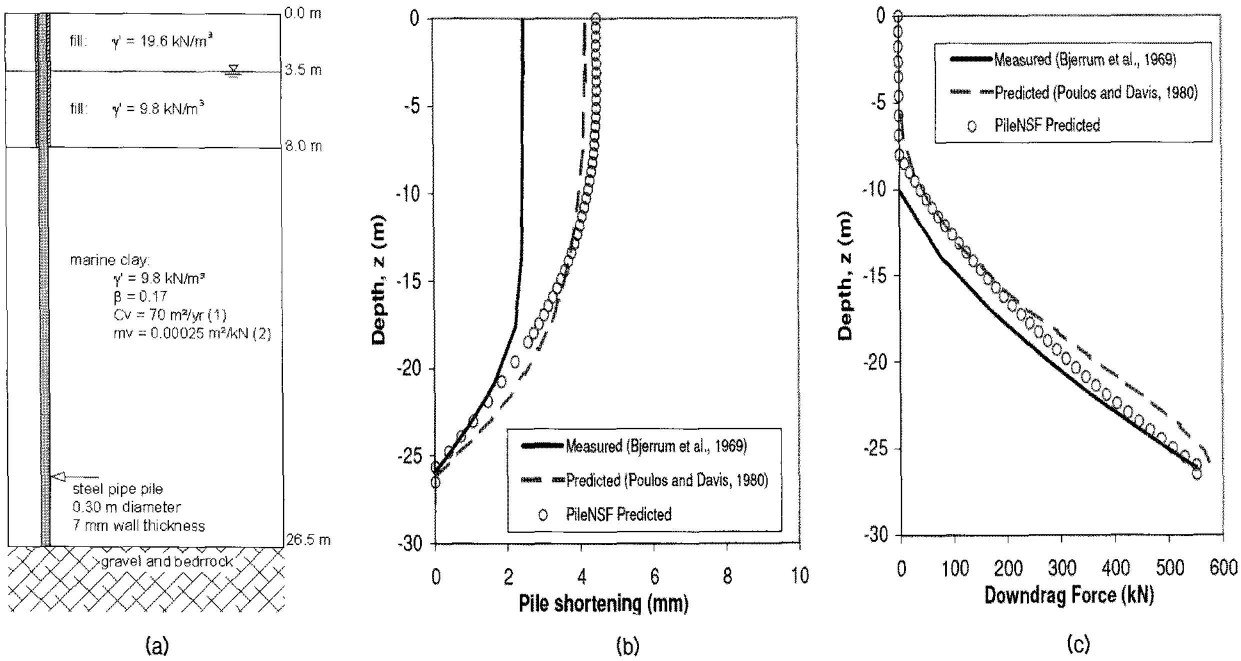


Fig. 11. Case study 2 – Pile C, Heroya site, Norway (Bjerrum et al., 1969): (a) Soil-pile profile: (1) after Poulos and Davis (1980), (2) after Alonso et al. (1984), (b) Predicted and measured pile shortening, and (c) Predicted and measured downdrag force

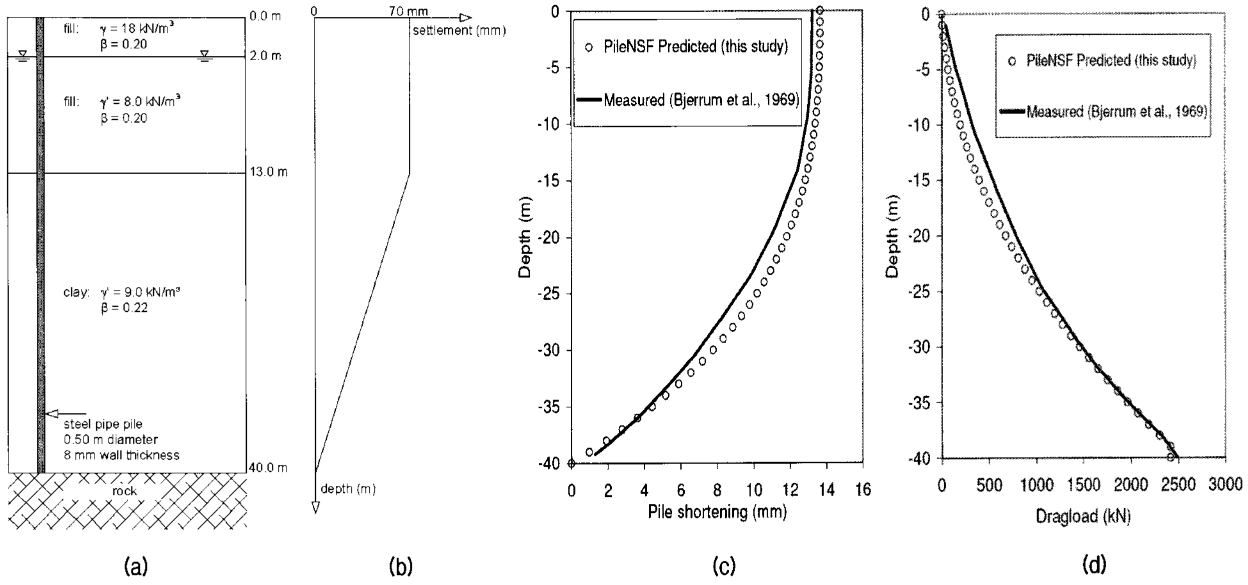
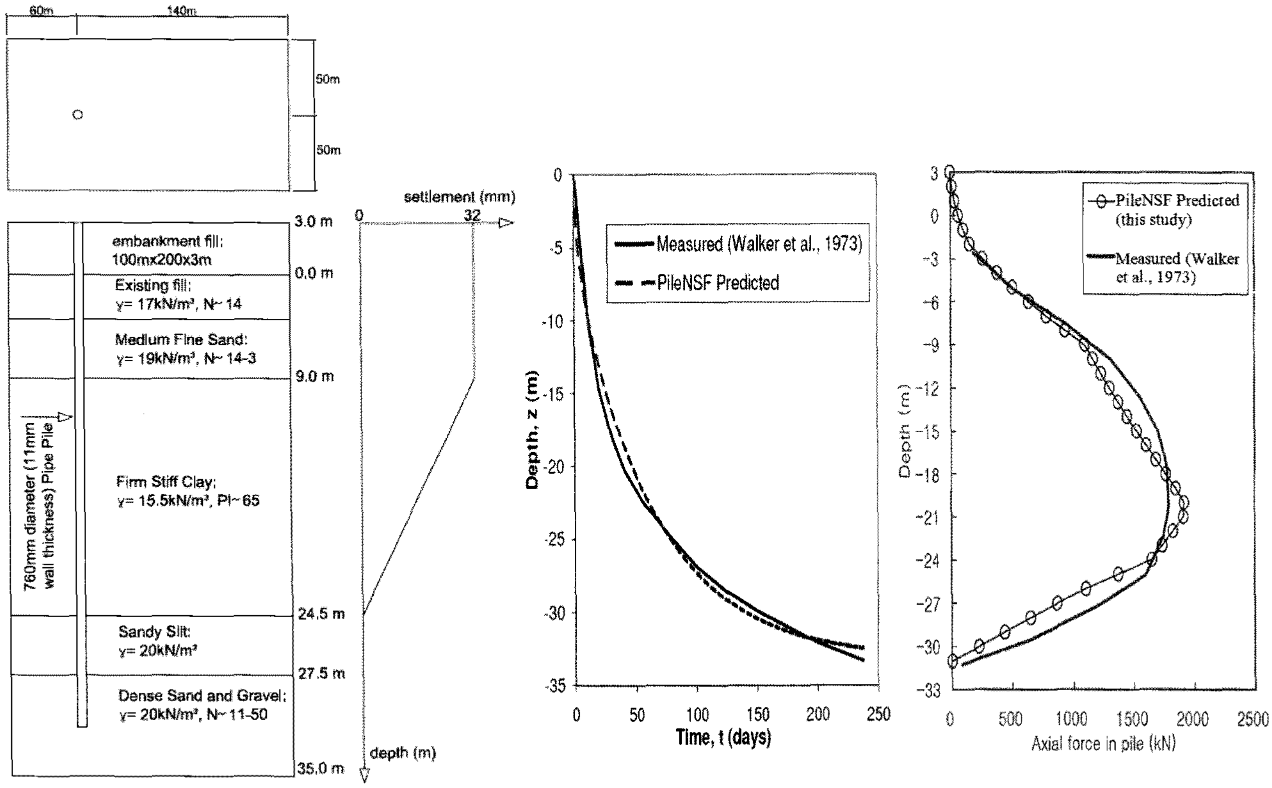


Fig. 12. Case Study 3 - Pile G, Sorenga Site, Norway (Bjerrum et al., 1969): (a) Soil-pile profile data, (b) Settlement profile, (c) Predicted and measured pile shortening, and (d) Predicted and measured dragload on pile

the model presented in Fig. 2 (b). The steel pipe pile was modeled as elastic, and Mosher's (1984) T-z load-transfer relation for sand (Fig. 4b) was used for the fill layer and Reese and O'Neill's (1987) relation was used for the clay layer (Fig. 4a). A rigid bearing stratum was assumed so that the base of the pile in the model may be supported by a pin constraint and the settlement of the soil assumed to be constant in the fill layer and decrease linearly from 70 mm at the surface of the clay layer to zero at the pile tip. The predicted pile shortening and downdrag are shown in Figs. 12 (c) and 12 (d), respectively that are in good agreement with the measured data by Bjerrum et al. (1969).

Case study 4 - 760mmx34m test pile in embankment fill (Walker et al., 1973):

Walker et al. (1973) presented a documented case of downdrag measurement on an uncoated steel pipe pile driven closed-ended through a highly stratified soil. The pile of 34 m in length, 760 mm in diameter, and 11 mm in wall thickness was driven into a soil profile consisting of a 2 m of recent fill, and 7m of sand overlying firm silty clay, sandy silt, and dense sand and gravel (Fig. 13a). A test embankment (100 m × 200 m × 3 m high) was constructed after the pile was driven and the pile was monitored for about 8 months. The stress increase due to the surcharge embankment was initially assumed to be constant with depth for the consolidation analysis, which was verified using Boussinesq's solution. Back-calculated constant coefficients m_v=0.000042 m^2/kN and C_v=135 m^2/year were used to predict the settlement shown in Fig. 13 (c), which is in close agreement with the measured settlement. The properties of the fill and sand were derived based on the SPT-N data of the soil profile shown in Fig. 13 (a). The water table was deemed to be located at 1.5 m below the original ground surface. Data on the silt layer were not available and assumed to have properties of sand with a relative density of 25%. An over-consolidation ratio (OCR) of 1.57 derived from consolidation test at a depth of 18.5 m was used for the entire clay layer with an average plasticity index of about 65. The coefficients of lateral earth pressure K_s were assumed to be 0.60, 1.0, 1.4, and 1.6 for the fill, sand, sandy silt, and dense sand, respectively, based on the recommended values shown in Table 1. An interface friction angle of about 0.70 and 0.80 of the angle of internal friction were used for the fill-medium fine sand and sandy silt-dense sand, respectively, based on the recommended values shown in Table 2. Based on these parameters, the limiting friction resistances for the clay and sand at various depths were computed using Eqs. (8), (9), and (10). The property of the end-bearing layer at



(a) (b) (c) (d)

Fig. 13. Case study 4 - 760 mm x 34 m test pile in embankment fill (Walker et al., 1973): (a) Properties of soil-pile profile, (b) Profile of predicted settlement, (c) Predicted and measured total settlement, (d) Predicted and measured dragload on pile

the tip of the pile was evaluated based on a corrected SPT-N value of 22 and correlated property ϕ' of 39° for sand (Wong and Teh, 1995). The steel pile material was modeled as elastic (E=200,000 MPa) and nonlinear load transfer curves in skin friction (T-z curve) for the fill and sand layers were modeled using Fig. 4 (b) and that for the clay layer using Fig. 4 (a), while nonlinear load transfer curve for the dense sand in end-bearing (Q-z curve) was modeled using Fig. 5 (b). Downdrag analysis was performed with the analytical model shown in Fig. 2 (b) under the imposed displacements due to the soil settlements of the layers (Fig. 13b) at the end of 8 months of monitoring. A fair agreement is shown with the predicted maximum dragload of about 1900 kN compared to the measured maximum downdrag of 1800 kN as shown in Fig. 13 (d).

Case study 5 - 610 mm diameter steel pipe pile in existing sand fill (Fukuya et al., 1982)

Downdrag measurement on a test pile located at a recently reclaimed land in Tokyo bay was reported by Fukuya et al. (1982). The steel pipe pile was 37.5 m long, 610 mm in diameter and with a wall thickness of 9.5 mm that was driven in October-November 1963 through 7 m of sand fill and 30.5 m of silt, clay, and gravel into a bearing stratum of sand (Fig. 14a). The water table was assumed to be at 2 m below the ground surface. The pile was monitored for about 41 months until March 1976 where the ground surface settled approximately 290 mm during this period. The measured settlement profile in March 1976 is shown in Fig. 14 (b), which shows that primary consolidation is almost complete. Downdrag analysis is performed at the end of the monitoring period using the measured settlement profile at the soil layers. In the computation, the effective stress parameters for the sand fill, gravel, and bottom dense sand layers were based on SPT N-values and beta coefficients were based from derived data that were evaluated by Wong and Teh (1995). The steel pipe pile was modeled as elastic (E=200,000 MPa) and the nonlinear load transfer curves in skin friction for the sand fill, sandy silt, and gravel were based on Mosher's

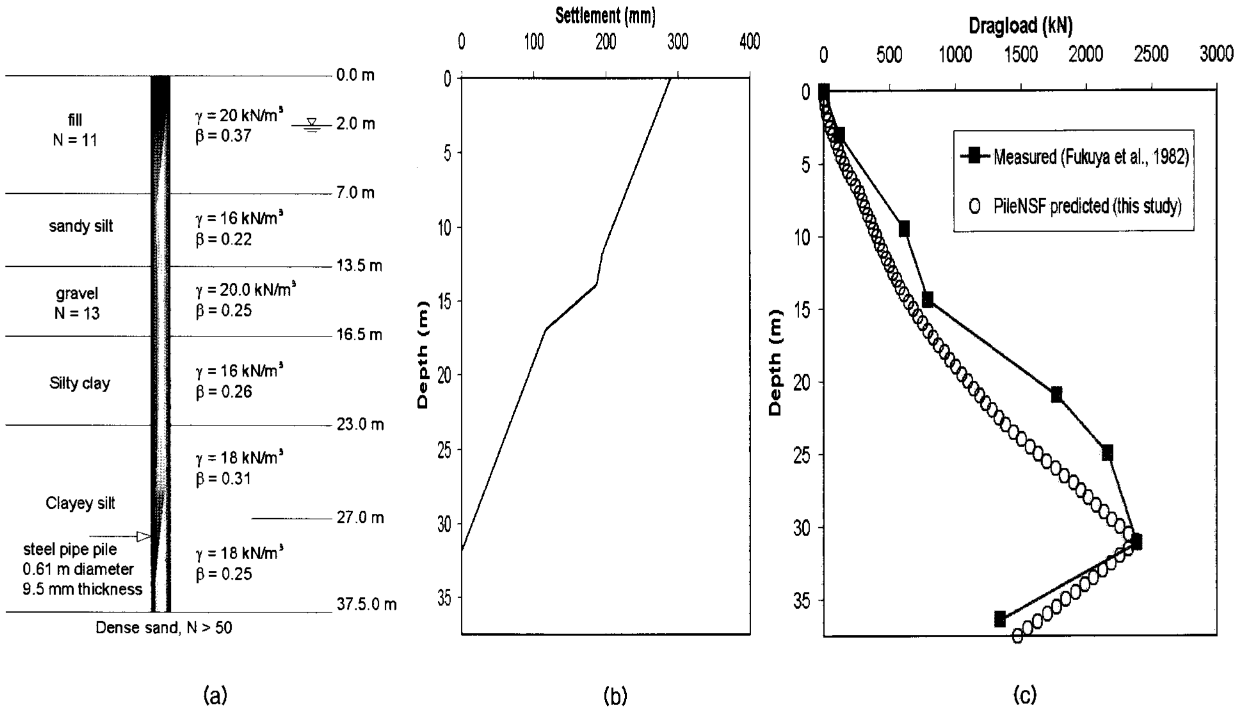


Fig. 14. Case study 5 - 610 mm diameter steel pipe pile in existing sand fill (Fukuya et al., 1982): (a) Properties of soil-pile profile (after Wong and Teh, 1995), (b) Measured settlement profile at end of observation, (c) Predicted and measured dragload on pile

(1984) T-z curve relation (Fig. 4b) while that for silty clay and clayey silt were based on Reese and O'Neill's (1987) T-z curve relation for piles in clay (Fig. 4a). Nonlinear load transfer curve in end bearing was based on Vijayvergiya's (1977) Q-z curve relation for piles in sand (Fig. 5b). The predicted and measured variation of downdrag with depth is shown in Fig. 14 (c), in which a fair agreement is shown with the maximum predicted downdrag of 2,386 kN that compares favorably with the maximum measured downdrag of 2,390 kN.

6. Conclusions and Future Developments

This paper presents a simplified one-dimensional soil-pile analytical model using the nonlinear load-transfer method that is uncoupled with the generalized one-dimensional consolidation theory in terms of finite strain by Mikasa (1965) for the prediction of negative skin friction in single piles. The microcomputer program PileNSF (Pile Negative Skin Friction) is developed by the authors in a graphical user interface environment using MATLAB® for evaluating the problem of a pile embedded in a consolidating soil that is subjected to surcharge loading from fill. The program developed has a significant feature of incorporating an advanced settlement theory that considers the variability of the thickness of the compressible layer during consolidation. In addition, the finite reduction of the soil-pile interface length at any time during the progress of consolidation in the clay layer is also accounted for in the calculation of the limiting shaft resistances in the compressible soil layer. A feature of delayed pile installation from the start of the surcharge loading from fill can also be considered in the analysis. The presented model and solution method have been verified and validated with several case studies of long term measurement of test piles subjected to negative skin friction. Predicted results of consolidation settlement, downdrag, and dragload by the program are shown to be in good agreement with measured data and field observations. Considering that the program presently handles only a homogeneous single sublayer of clay that is underlain or overlain by sand layers, future developments are currently being undertaken to consider analysis of pile negative skin friction under surcharge consolidation in a multilayer compressible soil with nonuniform consolidation parameters.

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