ANALYSES OF BRIDGE PILES NEAR APPROACH EMBANKMENTS

Fanyu Zhu, Shaheen & Peaker Limited, Toronto, Ontario, Canada
Zuhtu Ozden, Shaheen & Peaker Limited, Toronto, Ontario, Canada
Kenneth R. Peaker, Shaheen & Peaker Limited, Toronto, Ontario, Canada

ABSTRACT

This paper presents analytical results of downdrag and lateral bending loads on driven piles near approach embankments, assuming that the piles are installed without prior surcharging the ground by the approach embankments. The piles are end bearing, founded on bedrock which is overlain by a uniform clay soil layer. The effects of the strength of clay soil and the depth of piles are analyzed. In the analyses, the undrained shear strength of the clay soil ranges from 25 to 100 kPa, and the depth of the piles varies from 10 to 30m. Selection of adhesion between the pile and soil is discussed. In the analyses of the pile stress, a steel H-pile (HP 310x110) has been adopted. The analyses show that pile stress due to downdrag load and lateral bending is related to soil strength and pile length. The maximum stress of the piles due to downdrag load and lateral bending may exceed 100 MPa, which must be considered in the design of the piles.

1. INTRODUCTION

In the construction of a highway crossing where the upper level of ground consists of a weak stratum, the bridge structure can be supported by deep foundations such as driven piles installed underneath the abutments. The placement of approach embankment fill will cause vertical and horizontal movements of the soil at the pile locations. The vertical movement of ground will cause downdrag (negative friction) on the piles, and horizontal movement will induce bending of the piles. This latter aspect is sometimes not recognized or unaccounted for by the engineer due to a lack of simple design methods for analysis.

Downdrag loads resulting from negative friction on piles installed in or through consolidating soils have been analyzed by a number of researchers (Poulos, 1997; Fellenius, 1972; Kuwabara and Poulos, 1989; Chow et al., 1990; Lee, 1993). The downdrag loads are related to the adhesion between the soil and the pile, and to the soil settlement relative to the pile. Negative friction introduces additional loads on the pile, resulting in higher pile stress and increased pile settlement. Negative friction on piles can be significant in some cases and downdrag loads of 2000 kN or more have been reported (Poulos, 2003).

The construction of highway embankments can cause lateral bending of adjacent piles due to horizontal soil displacements resulting from the embankment fill. The pile stresses due to lateral bending can be significant, particularly when soft soils are present and lateral soil displacements are large (Goh et al., 1997). A number of case studies, centrifuge modelling and numerical analyses of pile lateral bending due to embankment construction have been presented in the literature (Heyman, 1965; Springman, 1989; Stewart et al., 1994).

This paper presents the analytical results of downdrag load and lateral bending of driven piles near approach embankments, assuming that the piles are installed through the consolidating clay soil and are founded in the underlying bedrock, without prior surcharging the ground by the approach embankments. In the analyses, the thickness of the consolidating native clay soil ranges from 10 to 30 m. The shear strength and compressibility of clay soil overlying bedrock is assumed to be uniform. The undrained shear strength varies from 25 to 100 kPa. The height of the approach embankment is 8 m. The piles in the analyses are HP310x110 steel piles which are founded in the bedrock. The pile width is B=0.31m, thus resulting in a perimeter of 1.24m; the cross section area of the steel is $0.0141 m^2$, and the moment of inertia is $I=2.36 \times 10^{-4} m^4$.

For the analyses of the bridge piles supporting the abutments, a finite element model has been used to compute the free-field soil displacements in the vertical and horizontal directions at the pile locations. The free-field soil displacements due to embankment fill are obtained without considering the presence of the bridge piles. The computed vertical soil settlement will be used for downdrag analyses of the piles and the horizontal displacements will be employed for lateral bending analyses.

2. FINITE ELEMENT ANALYSES

As shown in Figure 1, the placement of the embankment fill will cause the soil at the pile locations to move vertically and horizontally away from the approach embankments. The displacements of the soil will cause lateral load and downdrag load (i.e. negative skin friction) on the piles installed prior to the embankment construction.

A finite element model has been adopted to estimate the horizontal and vertical displacements of the clay soil at the pile locations due to the placement of the approach embankment fill. In the analysis, a 2-dimensional finite element model established along the centre line of bridge is adopted. The height of the embankment ranges from 8 m adjacent to the abutment wall to 6.0 m at a location 150 m away from the centre line of the underpass.

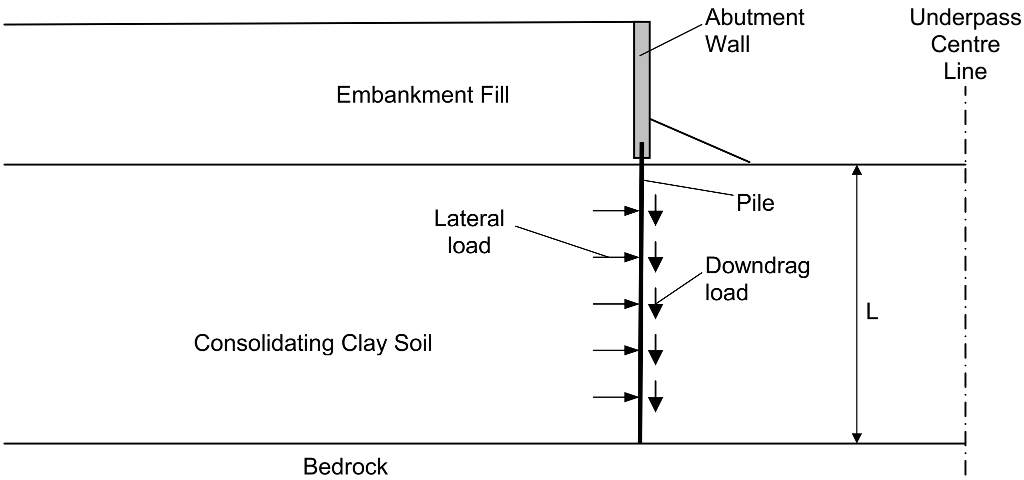


Figure 1: Bridge Piles and Approach Embankment

Three cases of consolidating native clay soil are considered, with undrained shear strengths of 25, 50 and 100 kPa respectively. In the finite element model, the soils are assumed to be linear-elastic. Assumed soil parameters used in the finite element analyses are presented in Table 1.

Table 1: Assumed Parameters of Native Clay Soil

Soil Parameters	**Soil-A**	**Soil-B**	**Soil-C**
Undrained Shear Strength, C_u (kPa)	25	50	100
Young's Modulus, E (MPa)	5	10	20
Poisson's Ratio, ν	0.40	0.38	0.36

Soil parameters used for the approach embankment fill are Young's modulus E=20 MPa, Poisson's ratio ν=0.35, and unit weight γ=20.5 kN/m^3.

Based on the finite element analyses, the settlement and horizontal displacement of the clay soil at the pile locations due to the approach embankment fill are presented in Figures 2 and 3.

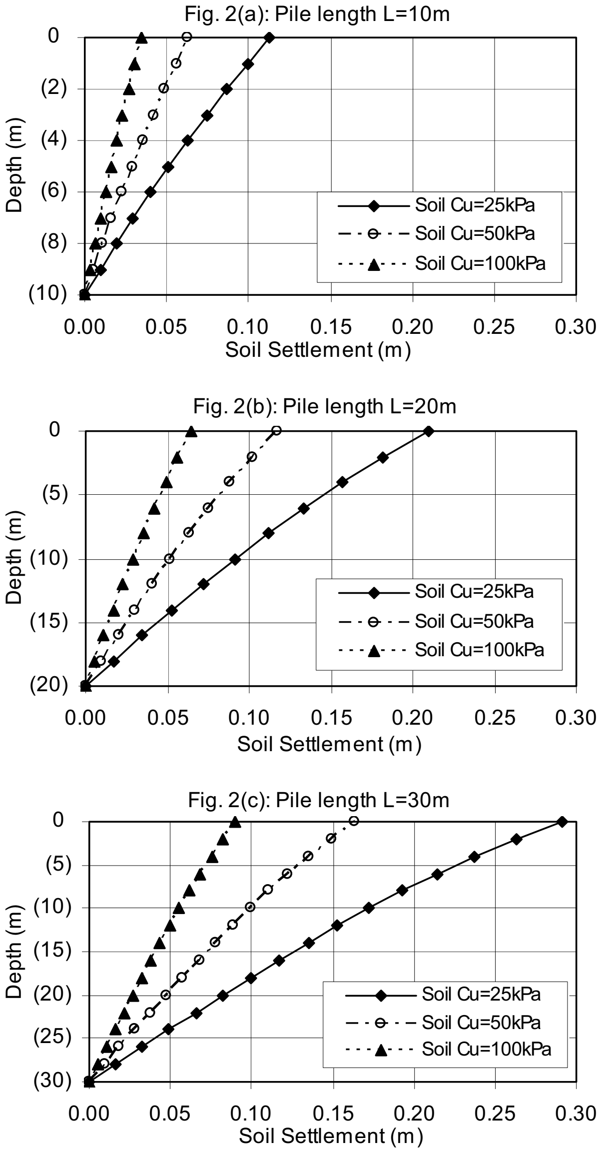
As shown in Figure 2, the maximum settlement of the soil at the pile locations occurs at the ground surface. The settlement decreases with depth and becomes zero at the bedrock surface. The settlement increases with increasing clay thickness (pile length) and with decreasing soil strength. For pile length L=10m, the surface settlement ranges from 0.034m to 0.113m for soil C_u decreasing from 100 kPa to 25 kPa. For pile length L=30m, the surface settlement ranges from 0.090m to 0.291m, decreasing with C_u values.

As shown in Figure 3, the horizontal (lateral) displacement of the soil at the pile locations increases with increasing clay thickness (pile length) and with decreasing soil strength. Considerable lateral movement occurs at the ground surface level. The lateral displacement increases with depth for the upper portion of the piles. After the maximum value which is reached at a depth of 0.25 to 0.5 times pile length, the lateral displacement decreases with depth and becomes zero at the bedrock surface. For pile length L=10m, the maximum lateral displacements are 0.071m and 0.018m for C_u values of 25 kPa and 100 kPa respectively. For pile length L=30m, the maximum lateral displacements are 0.181m and 0.042m, decreasing with soil strength.

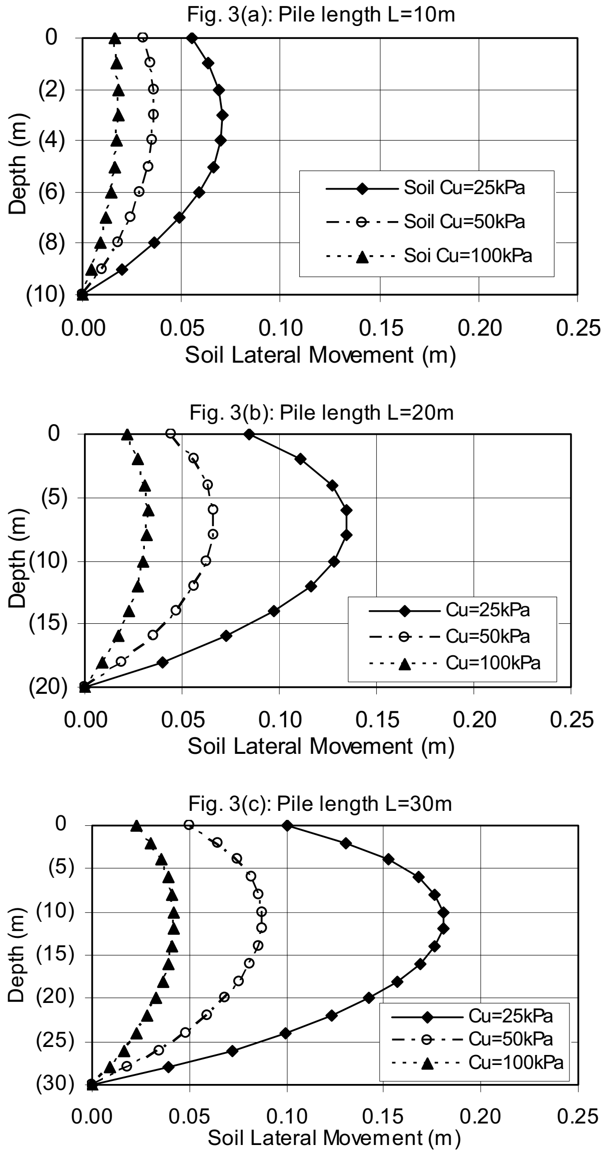
The soil settlement and lateral displacement presented in Figures 2 and 3 are considered free-field soil movements, obtained from finite element analyses without considering the presence of the piles. They will be used in the following sections for the analyses of pile downdrag load and lateral bending. In the analyses, the piles are assumed to be straight and vertical prior to the placement of the embankment fill. The piles are assumed to extend to the surface of the bedrock. The tip of the piles is assumed not to move vertically and horizontally and the top of the abutment wall at the road level is assumed to be fixed horizontally (see Figure 1).

3. NEGATIVE FRICTION ON PILES

Piles installed in or through a consolidating soil layer will be subjected to negative friction (downdrag load) due to the downward movement of the consolidating soil relative to the piles. Downdrag load of piles generally does not reduce the ultimate geotechnical capacity of the piles (Poulos, 1997). In the case of additional downdrag load which is to cause geotechnical failure of the founding soil, the settlement of the pile will increase, which will release the downdrag load until an equilibrium of the axial loads is reached.



Figures 2: Soil Settlements at Pile Locations



Figures 3: Horizontal Soil Movements at Pile Locations

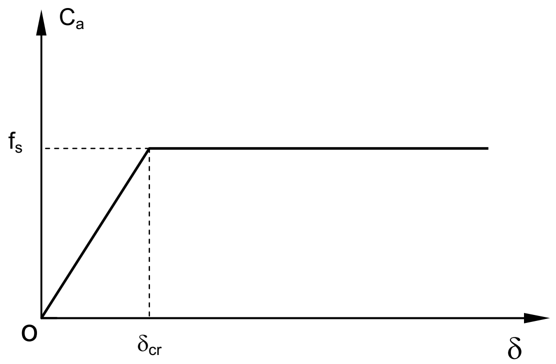
In the design of piles, two main concerns related to downdrag loads are: (1) Structural integrity of the pile section due to additional downdrag load; (2) Additional settlement of the pile due to downdrag load. In this study, the piles are founded in bedrock and the settlement of the pile tip is assumed to be zero. The effects of the downdrag load as well as lateral bending on the structural integrity of the piles (i.e. pile stress) are investigated.

For piles installed in consolidating clay soil, the negative friction is related to the adhesion (C_a) between the pile and the soil and to the soil settlement relative to the pile. The adhesion is generally related to the undrained shear strength (C_u) of the soil and the relative settlement (δ) between the pile and the soil. It is considered that there is a critical relative settlement (δ_{cr}) at which slip between the pile and the soil is to occur. When the relative settlement reaches or exceeds the critical value (δ_{cr}), the mobilized adhesion between the pile and the soil will be the maximum adhesion (i.e. skin friction f_s).

A simplified model as shown in Figure 4 is used in this study to estimate the adhesion C_a between the pile and the soil, expressed as

$$C_a = \frac{\delta}{\delta_{cr}} f_s \le f_s \qquad [1]$$

where f_s represents skin friction between the pile and soil when slip occurs. When the relative settlement δ is less than the critical relative settlement δ_{cr}, the adhesion (C_a) increases linearly with relative settlement (δ). When the δ value is equal to or greater than δ_{cr}, the adhesion is equal to the value of the skin friction f_s.



Figures 4: Adhesion versus Relative Displacement between Soil and Pile

For piles in clay soil, the skin friction f_s is related to the undrained shear strength (C_u). A semi-empirical approach to estimating skin friction is expressed as

$$f_s = \alpha C_u \qquad [2]$$

Where α is an empirical factor. This method for estimating skin friction is called the α-Method (CGS, 1992; Miller, 1997). In addition, the effective stress method has also been adopted to estimate the skin friction on piles (Burland, 1973; Azzouz et al., 1990). Using the data of Tomlinson (1963), Prakash and Sharma (1990) shows that the α values for steel piles are very close to or slightly smaller than those for concrete and timber piles.

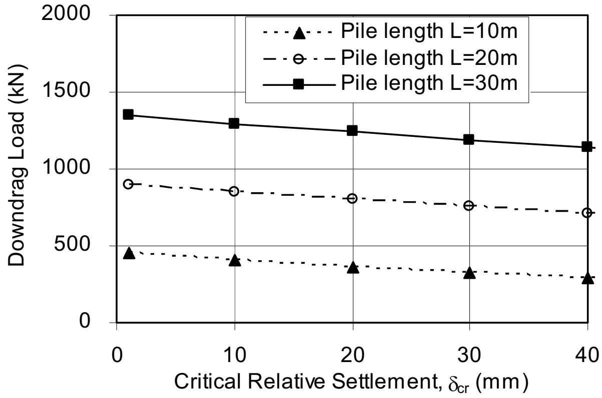
Poulos (2003) presents a relation for the α factor, expressed as

$$\alpha = 0.21 + 0.26 p_a / C_u \le 1 \qquad [3]$$

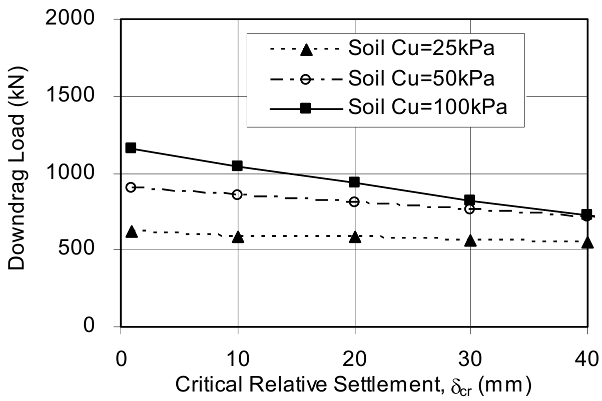
where p_a represents the atmospheric pressure (p_a = 100 kPa). If the α value calculated using Equation 3 is greater than 1, a value of α = 1 is used. Equation 3 is very close to the values presented by Tomlinson (1957). In this study, Equation 3 is adopted to estimate the skin friction between the pile and soil. The calculated values of the skin friction f_s are 25, 37 and 47 kPa for the clay soils with C_u values of 25, 50 and 100 kPa respectively.

The relative settlement required to mobilize full skin friction is generally very small (CGS, 1992; Fellenius and Broms, 1969; Fellenius, 1972). Poulos (2003) indicates that the relative settlement required to mobilize full skin friction (slip) may range from 1 to 5% of pile diameter.

In order to provide an insight into the effect of the relative settlement on downdrag load, piles under the conditions shown in Figure 1 are analyzed for downdrag loads. In the analyses, the pile length L ranges from 10 to 30 m, the undrained shear strength C_u of the soil varies from 25 to 100 kPa. The settlements of the soil at the pile location as presented in Figure 2 are used. The pile is assumed to be HP310x110 pile with a size B=0.31m. For the calculation of the adhesion C_a using Equation 1, the values of the critical relative settlement δ_{cr} at which full skin friction is mobilized (i.e. slip occurs) are assumed to be 1, 10, 20, 30 and 40 mm. The accumulated downdrag loads at the tip of the piles are presented in Figures 5 and 6.



Figures 5: Effect of Slip on Accumulated Downdrag Load at Pile Tip (Soil C_u=50kPa)



Figures 6: Effect of Slip on Accumulated Downdrag Load at Pile Tip (Pile length L=20m)

The results indicate that the accumulated downdrag loads decrease with the increasing value of δ_{cr}, but the effect of δ_{cr} is not very significant. This is due to the fact that the soil settlement at the pile location is generally much greater than the required relative settlement δ_{cr} to mobilize the full skin friction f_s. The selection of δ_{cr} value can only influence the downdrag load at the lower portion of the piles where the soil settlement is small.

For the downdrag load analyses in the following, the critical relative settlement δ_{cr} is assumed to be 15 mm to develop full skin friction. The value of δ_{cr} is about 5% of the pile size B.

4. PILE DOWNDRAG LOADS

Single vertical piles have been used for the downdrag analyses in this study, although a bridge abutment wall is generally supported by a number of vertical piles, as well as in many cases some of the piles may be battered. Pile group effects suppress free-field soil movements and downdrag loads in a pile group are generally smaller than for single piles, especially for the inner piles of the pile group (Kuwabara and Poulos, 1989). Therefore, using single piles for downdrag analyses is conservative.

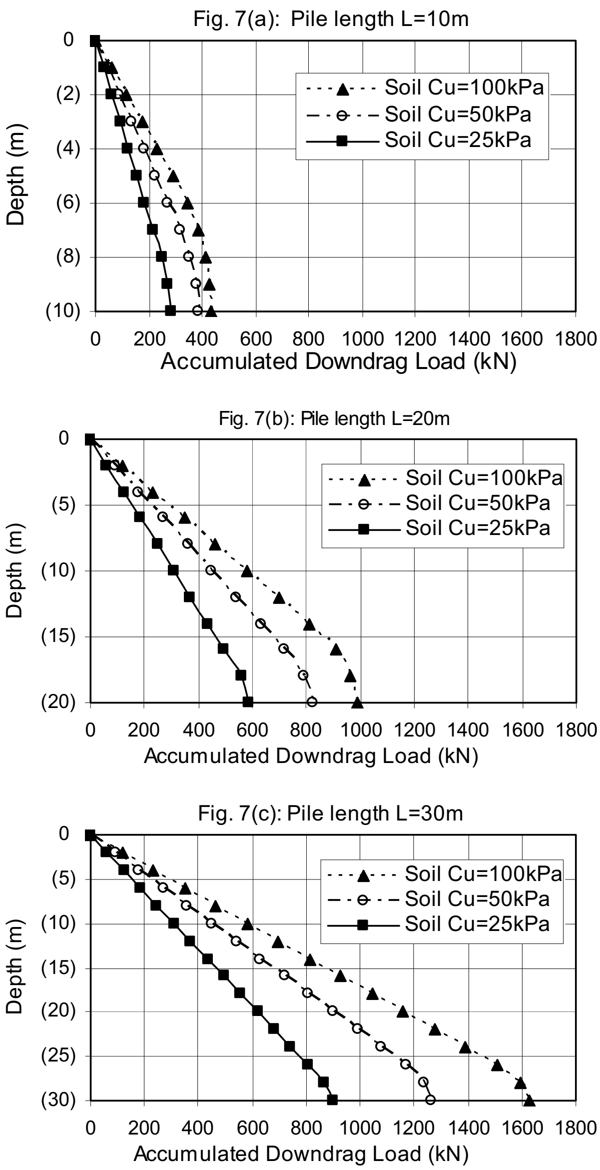


Figure 7: Accumulated Downdrag Load on Piles

In order to calculate the downdrag load, the relative settlement between the soil and the pile can be obtained by deducting steel pile compression due to downdrag load from the free-field soil settlement in Figure 2. The final downdrag load of a pile can be obtained by iterations using hand calculation (spread sheet). General steps for the calculations are as follows:

1. Divide the pile into a number of sections along its length;
2. Assume the initial value of adhesion C_{ao} to be equal to the skin friction f_s;
3. Use the initial adhesion to calculate the accumulated downdrag load along the pile length and then calculate the steel pile compression;
4. Deduct the steel pile compression from the free-field soil settlement as shown in Figure 2 to obtained relative settlement δ between the pile and the soil;
5. Calculate the new adhesion C_{a1} using Equation 1 based on the derived relative settlement δ;
6. Replace the initial adhesion C_{ao} in 1 with the new adhesion value C_{a1} and repeat the steps 2 to 5 until $C_{a1} = C_{ao}$ to obtain the final downdrag load.

The number of iterations required to obtain the final downdrag load is generally 2 to 4. The calculation results of the accumulated downdrag loads are presented in Figure 7.

The calculated downdrag loads generally increase linearly with depth, except for the lower portion of the piles where the increment of the downdrag load becomes smaller. This is due to the fact that the soil settlement at the lower portion of the piles is small and full skin friction between the pile and the soil is not mobilized.

The maximum accumulated downdrag load, which occurs at the pile tip for all piles, increases with pile length, and with soil C_u due to the increase of adhesion C_a with C_u. For pile length L=10m, the maximum accumulated downdrag load ranges from 282 to 433 kN when the C_u values increase from 25 to 100 kPa. For pile length L=30m, the maximum accumulated downdrag load ranges from 899 to 1628 kN, increasing with soil strength

5. PILE LATERAL BENDING

As shown in Figure 3, the soil at the pile location moves horizontally due to the placement of the embankment fill. This will cause lateral bending of the piles. The resulting bending stresses can be significant, particularly in soft soil of which the lateral displacement is large (Goh et al., 1997).

A number of methods for modelling the bending of piles under lateral soil movements have been proposed in the literature (Stewart et al., 1994). For single piles, the elastic continuum approach (Poulos and Davis, 1980), the p-y curve approach (Matlock, 1970), and the modulus of subgrade reaction method are often adopted. For the latter two methods, the pile can be represented by beam elements and the soil can be simulated by a series of linear elastic springs or nonlinear springs (Byrne et al., 1984; Goh et al., 1997). The solutions can be obtained using the finite element technique.

In this study, a finite element model in which the pile is simulated by beam elements and the soil is represented by linear elastic springs is used to estimate the lateral bending of piles under horizontal soil movement. The modulus of (horizontal) subgrade reaction (k_h) method is employed to obtain the values of the spring constant. As recommended by Canadian Foundation Engineering Manual (3^{rd} Ed, 1992), the value of k_h can be obtained using

$$k_h = \frac{67C_u}{B} \qquad [4]$$

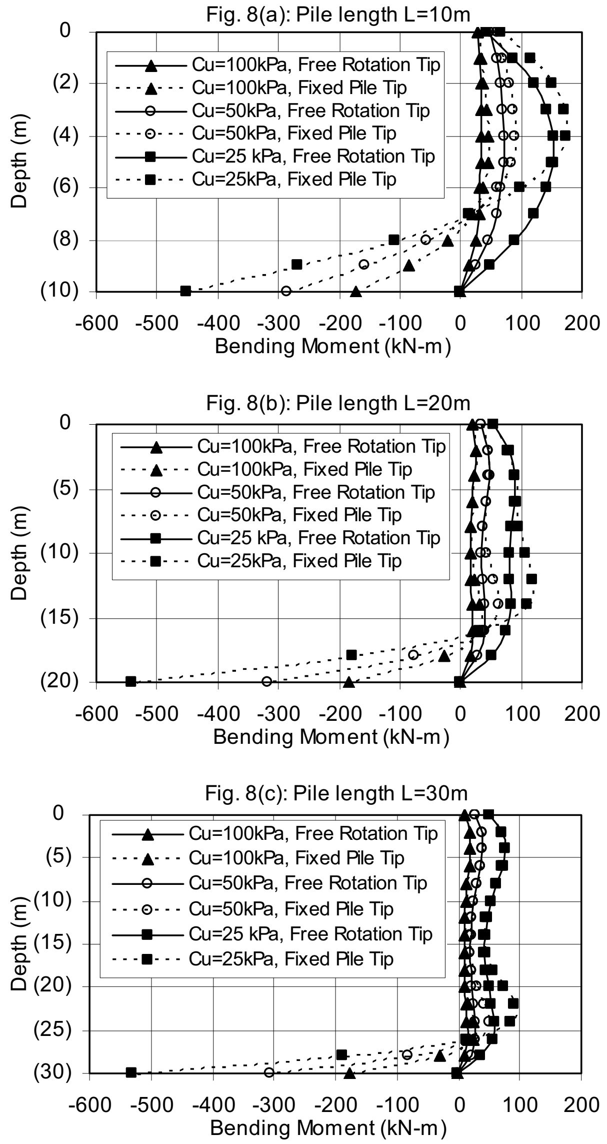


Figure 8: Bending Moment of Piles Subjected to Lateral Soil Loads

where C_u is the undrained shear strength of the soil, and B represents the diameter or width of the pile.

For a spring representing the soil resistance on a pile section of length L_1, the spring constant k_{spr} can be obtained using

$$k_{spr} = k_h B L_1 \qquad [5]$$

It should be noted that using Equations 4 and 5, the estimated soil resistance must be smaller than the ultimate lateral soil resistance p_u. The value of p_u can be approximately estimated using $p_u = 3C_u$ at ground surface and $p_u = 9C_u$ at or below a depth of about 6 times pile diameter (Matlock, 1970).

In the finite element models for the lateral bending analyses of the piles, one end of the horizontal springs is attached to the beam elements of the pile and the other end of the springs is subjected to the horizontal movements of the free-field soil as shown in Figure 3. The bending moments of the piles obtained from the finite element analyses are presented in Figure 8, where the length of the piles ranges from 10 to 30 m and the undrained shear strength of the soil varies from 25 to 100 kPa.

The solid lines in Figure 8 represent the bending moments of the piles of which the tips at the bedrock level are assumed to be free to rotate. In this case, the bending moments are significant for short piles in weak soils, and decrease with increasing pile length and soil strength.

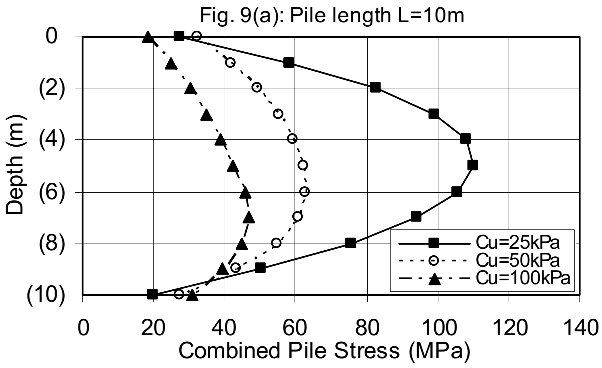
The dashed lines in Figure 8 represent the bending moments of the piles of which the tips are assumed to be fixed and rotations are not allowed. In this case, the bending moments are very large at the lower portion of the piles. At the tip of the piles, large concentrated bending moments are obtained from the analyses. In reality, these large concentrated bending moments are not likely to occur, because the tip of the piles would be in some degree free to rotate and the restraints would be weaker. For end bearing piles founded near the bedrock surface, the tip of the piles may be considered to be free to rotate.

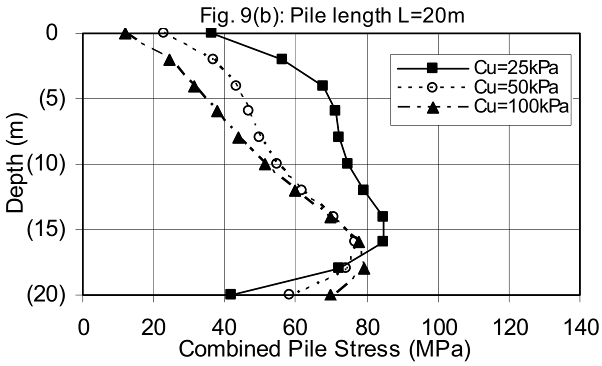
In terms of boundary conditions (restraints) at the tip of the piles, the solid lines in Figure 8 may represent the lower bounds of bending moments of the piles, and the dashed lines can represent the upper bounds of the bending moments. In reality, the bending moment of a pile should be in the range between the lower and the upper bounds, likely to be closer to the lower bound value.

6. PILE STRESS

From the accumulated downdrag loads in Figure 7 and the bending moments in Figure 8, the combined stress of the piles (HP310x110) from downdrag and lateral bending can be obtained, as shown in Figure 9 where the piles are assumed to be free to rotate at the tip. At the top of the piles where there is no accumulated downdrag load, the stress is solely due to lateral bending. At the tip of the piles where the bending moment is zero, the stress results from downdrag load only. Between these two points, the stress of the piles is due to both the downdrag load and lateral bending.

In Figure 9(a) for the piles with a length of 10m, the pile stress is mainly due to lateral bending, especially for the pile in the weak clay soil with C_u=25 kPa. The maximum combined pile stress, which occurs in the middle portion of the piles, ranges from 47 to 110 MPa, increasing with decreasing soil strength.





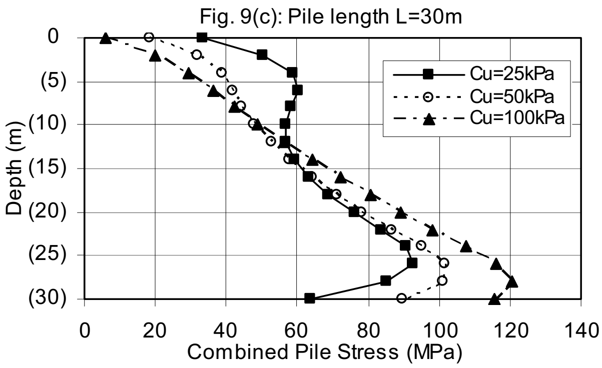


Figure 9: Pile Stress due to Downdrag Load and Lateral Bending (Piles are assumed to be free to rotate at the tip)

For longer piles in Figure 9(b) and 9(c), downdrag load plays a more important role. In Figure 9(b) for the piles with a length of 20m, the maximum combined pile stress, which occurs at the lower portion of the piles, ranges from 79 to 85 MPa, not significantly influenced by soil strength. In Figure 9(c) for the piles with a length of 30m, the maximum combined pile stress, which also occurs at the lower portion of the piles, ranges from 93 to 120 MPa, increasing with soil strength.

In summary, for the 10 to 30m long piles in the clay soil with C_u values from 25 to 100 kPa, the combined pile stress from downdrag and lateral bending ranges from 47 to 120 MPa. The stress from lateral bending is high for short piles in weak soil. For long piles in stronger soil, downdrag load becomes significant.

7. CONCLUSIONS AND RECOMMENDATIONS

This paper presents the analytical results of downdrag load and lateral bending of driven piles near approach embankments, assuming that the piles are installed through consolidating clay soil and are founded in the underlying bedrock. A finite element model has been used to compute the free-field soil displacements in the vertical and horizontal directions at the pile locations. The computed vertical soil settlements are used for downdrag analyses and the horizontal displacements are adopted for lateral bending analyses.

The results of the analyses indicate that the increase of soil strength will lead to increased downdrag load on piles due to higher adhesion between the pile and soil. Lateral bending moments of piles increase with decreasing soil strength as the horizontal movement of soil is smaller in stronger soil. With the increase of pile length, lateral bending stress becomes less significant, while the downdrag load on the piles increases with increasing pile length.

For the 10 to 30m long piles (HP310x110) in the clay soil with C_u values from 25 to 100 kPa, the combined pile stress resulting from downdrag and lateral bending ranges from 47 to 120 MPa. The stress from lateral bending is high for short piles in weak soil. For long piles in stronger soil, downdrag load becomes significant.

To mitigate the bridge pile stresses resulting from downdrag and lateral bending, the embankment fill should be placed considerably prior to the installation of the piles. After the anticipated consolidation of the foundation soil has occurred, the piles can then be installed to minimize the soil movements which will occur after the pile installation.

Another way to reduce the total pile stress (including the stress from working load) is to increase the number of piles or to use heavier sections of piles to provide allowance for the stress due to downdrag load and lateral bending.

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