ALTERNATIVE DESIGN APPROACH FOR DRAG LOAD AND DOWNDRAG OF DEEP FOUNDATIONS WITHIN THE LRFD FRAMEWORK

Timothy C. Siegel, P.E., G.E., D.GE, Dan Brown and Associates, PC, Knoxville, TN, USA
Rich Lamb, P.E., Minnesota Department of Transportation, Maplewood, MN, USA
Derrick Dasenbrock, Minnesota Department of Transportation, Maplewood, MN, USA
Paul J. Axtell, P.E., D.GE, Dan Brown and Associates, PC, Overland Park, KS, USA

The axial resistance provided by deep foundations may be divided into two components: side resistance and tip resistance. The direction that the side resistance acts depends on the relative movement between the deep foundation and the adjacent soil. That is, when the foundation moves downward relative to the soil, then the side resistance is positive and acts upward. Conversely, when the soil moves downward relative to the foundation the side resistance is negative and acts downward. Research supports that both positive side resistance and negative skin friction develop in essentially all deep foundations. The side resistance distribution is a function of the soil strength and stiffness, the applied top loads, and whether the top load is sustained, transient, or a combination of sustained and transient loads. Consideration of drag load and downdrag has become more convoluted with implementation of geotechnical aspects into the AASHTO LRFD Bridge Design Specifications and its use of load and resistance factors. This paper examines the current AASHTO Specifications and presents an alternative design approach for drag load and downdrag using the LRFD framework. The alternative design approach is illustrated by two examples involving drag load and downdrag.

Introduction

The axial resistance provided by deep foundations may be divided into two components: side resistance and tip resistance. The direction that the side resistance acts depends on the relative movement between the deep foundation and the adjacent soil. That is, when the foundation moves downward relative to the soil, then the side resistance is positive and acts upward. Conversely, when the soil moves downward relative to the foundation the side resistance is negative and acts downward. Research supports that both positive side resistance and negative skin friction develop in essentially all deep foundations. The side resistance distribution is a function of the soil strength and stiffness, the applied top load, and whether the top load is sustained, transient, or a combination of sustained and transient.

Consideration of drag load and downdrag has become more convoluted with implementation of geotechnical aspects into the LRFD Bridge Design Specifications and its use of load and resistance factors. This paper examines current LRFD design specifications and presents an alternative design approach for drag load and downdrag using the LRFD framework. The alternative design approach is applied to several examples where drag load and/or downdrag may control design.

Review of Terms

Negative skin friction – Side resistance mobilized as the ground moves downward relative to the deep foundation.

Drag load – Axial compressive load induced on a deep foundation due to accumulated negative skin friction. AASHTO Specifications (2012) use the term “downdrag” or “downdrag load”.

Neutral plane – Location along the deep foundation at which the sustained forces (i.e., drag load plus sustained structure load) are in equilibrium with the combination of the upward (positive) side resistance below the neutral plane and the tip resistance.

Downdrag – Downward movement of a deep foundation that results from ground settlement. The downdrag is equal to the settlement of the ground at the location of the neutral plane.

Residual load – Axial load exerted on a pile that is neither due to a top load or associated with resisting a top load. Examples of residual load are drag load and the resistance mobilized to oppose the drag load.

Geotechnical axial nominal resistance – Top load at which the deep foundation will no longer satisfy static equilibrium and will experience continued downward movement. It is equal to the sum of the fully mobilized side and tip resistances.

Structural axial nominal resistance – Ultimate structural strength of the deep foundation cross-section.

Permanent loads – Loads and forces that are, or are assumed to be, either constant upon completion of construction or varying only over a long interval of time (AASHTO, 2012.)

Transient loads – Loads and forces that can vary over a short time interval relative to the lifetime of the structure (AASHTO, 2012.)

Literature Review

Hanna and Tan (1973) recognized that deep foundation installation by driving or by casting in-place resulted in a complex foundation-soil interaction. They performed laboratory tests on instrumented long slender piles that confirmed that piles under zero top load were not stress free. The results of their tests support that a downward force (drag load) developed along the upper section of pile from the surrounding settling ground. The magnitude of the drag load was opposed by upward soil resistance acting on the lower section of pile. Hanna and Tan referred to these as "residual loads."

Briaud and Tucker (1984) proposed a method for evaluating the resistance distribution for driven piles in sand that explicitly considers residual stresses. Their discussion is limited to driven piles although the rebound during driving and reconsolidation of the soil after driving are identified as contributors to the development of residual stresses.

Fellenius (2001a) presented an analysis of instrumented bored piles that were dynamically tested and illustrated the presence of residual load. Fellenius (2001b) also analyzed strain gage data from statically loaded piles which showed that residual load develops in bored piles. On the basis of long term monitoring of driven piles, Fellenius (1998; 2006) concluded that essentially all piles will progress toward equilibrium where the sustained top load and the cumulative negative skin friction (*i.e.,* drag load) will act downward and be opposed by the positive shaft resistance and mobilized tip resistance. The location where the negative skin friction transitions to positive shaft resistance is known as the neutral plane. In recognition of the importance of the residual load to deep foundation behavior, Fellenius developed the unified design of deep foundations for the rational consideration of the interaction between drag load, applied sustained and transient loads, and downdrag.

There are a number of possible explanations for the presence of residual loads (i.e., drag loads and the associated opposing side/tip resistances) in deep foundations. For driven piles, residual loads may be expected as a result of rebound. Changes in stress and volume within the deep foundation-soil system are believed to be primarily responsible for the development of long term residual loads as the pile progresses toward a state of stress equilibrium with the surrounding soil over time. Fellenius (1989) identifies changes in effective stress during reconsolidation of the soil as a possible cause of residual load development. Hayes and Simmonds (2002) conclude that the physical expansion and contraction of the concrete during curing are also responsible for the development of residual loads in drilled shafts.

Perhaps the most fundamental explanation is that residual loads develop as a result of the stiffness contrast between deep foundation and soil (Fellenius, 2002). For the pile-soil system to reach a state of equilibrium, the soil resistances and internal pile loads must be in balance and the relative soil-foundation movements must be compatible. As result, the side resistance is fully mobilized, either upward (positive side resistance) or downward (negative skin friction), and tip resistance is mobilized to a degree consistent with the tip movement.

Conceptual Model

Figure 1 shows a conceptual deep foundation model that graphically illustrates the relationship between the permanent load ($Q_{permanent}$), the negative skin friction, the positive side resistance, the tip resistance (R_{tip}), the drag load, the downdrag (S_{pile}), and the neutral plane. All loads should have a load factor of unity. Using load factors greater than one in this approach will distort the maximum compressive load and the location of the neutral plane.

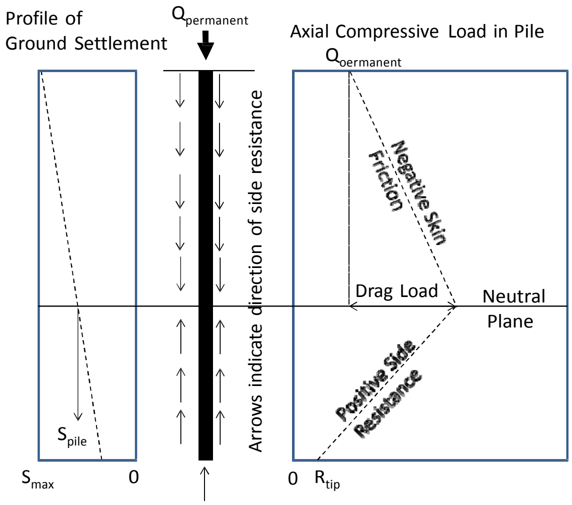

FIG 1. Conceptual Deep Foundation Model

By extending the concepts represented in the model shown in Figure 1, the following conclusions can be made (Fellenius, 1989; Fellenius, 1998):

- At the geotechnical strength limit state, the side resistance is positive over the entire deep foundation and the drag load is zero;
- Transient component of the top load is resisted by the temporary reversal of the side resistance from negative to positive along a portion of the deep foundation above the neutral plane;
- In most cases, the maximum compressive load in the deep foundation is the sum of the permanent component of the top load and the drag load and occurs at the neutral plane, and;
- The settlement of the deep foundation is equal to the ground movement at the neutral plane.

Commentary on Drag Load and Downdrag and the AASHTO LRFD Specifications

Numerous design approaches have been proposed for the consideration of drag load and down drag (Fellenius, 1989; Briaud and Tucker, 1997; Poulos, 1997; Dumas, 2005; Hannigan et al., 2005; Brown et al., 2010; AASHTO, 2012). Detailed discussion of all of them is beyond the scope of this paper. The following discussion focuses on the AASHTO Specifications (2012) and the unified design of deep foundations (Fellenius, 1989; Fellenius, 1998) as the objective herein is to provide a rational, alternative design approach for consideration of drag load and downdrag within the LRFD framework.

The AASHTO Specifications explicitly note four conditions for where drag load may develop; however, as described herein, drag load will develop for all deep foundations and should be estimated using the unified design of deep foundations (Fellenius, 1989; Fellenius, 1998).

The magnitude and distribution of the drag load should be calculated using the unfactored permanent top load. Conceptually, this is most closely matched by the Strength IV limit state load combination with all load factors equal to unity. The computed drag load should then be factored and included in the structural strength limit state design of the pile section.

The AASHTO Specifications include the drag load as an applied load in the analysis of the geotechnical strength limit state. As previously stated, the side resistance is positive over the entire deep foundation for this condition and the drag load is zero. Therefore, the drag load should not be included in the load combinations when considering the geotechnical strength limit state.

The AASHTO Specifications include the drag load as an applied top load in the analysis of the geotechnical service limit state. Although the drag load should be part of the calculation for determining the location of the neutral plane, it should not be included in the top load combinations for settlement analysis. The drag load does indirectly influence the geotechnical service limit state because the deep foundation settlement is equal to the ground movement at the neutral plane.

In other words, ground movement at the neutral plane (and, as a result, pile settlement) can result from a number of conditions that are not appropriately represented as a top load. These conditions include but are not limited to lowering of the water table, overlying fills, adjacent excavations, and liquefaction-induced soil compression.

While the AASHTO Specifications recognize that liquefaction can induce downdrag and re-distribute the drag load, it contains some shortcomings. Fellenius and Siegel (2008) address these shortcomings with a geotechnical analysis for deep foundations during a liquefaction event.

Design Examples

Two hypothetical design examples are presented in the Appendix that illustrate the proposed rational, alternative design approach for drag load and downdrag within the LRFD framework. Brief descriptions of the design examples are presented in the following paragraphs.

New Fill Above Pile Group in Sand. This design example illustrates the load conditions of a pile group in sand and the changes that result from the placement of new fill above the top of the pile. The same general conclusions described within the example apply to other soil types although the methods for estimating the side resistance, tip resistance, and settlement may be different.

Rock Bearing H Piles Installed Through Soft Clay. This design example illustrates a “classic” drag load condition where end bearing piles are installed through soft clay and extend up through new fill. In this example, the clay and the new fill induce negative skin friction into the piles.

Concluding Remarks

The authors recognize the consideration of drag load and downdrag has become more convoluted with implementation of geotechnical aspects into the AASHTO LRFD Bridge Design Specifications and its use of load and resistance factors. This paper examines the current AASHTO Specifications and presents an alternative design approach for drag load and downdrag using the LRFD framework. The proposed rational, alternative design approach is applied to two hypothetical examples: (1) the placement of new fill above a group of piles in sand, and (2) rock bearing H-piles installed through soft clay.

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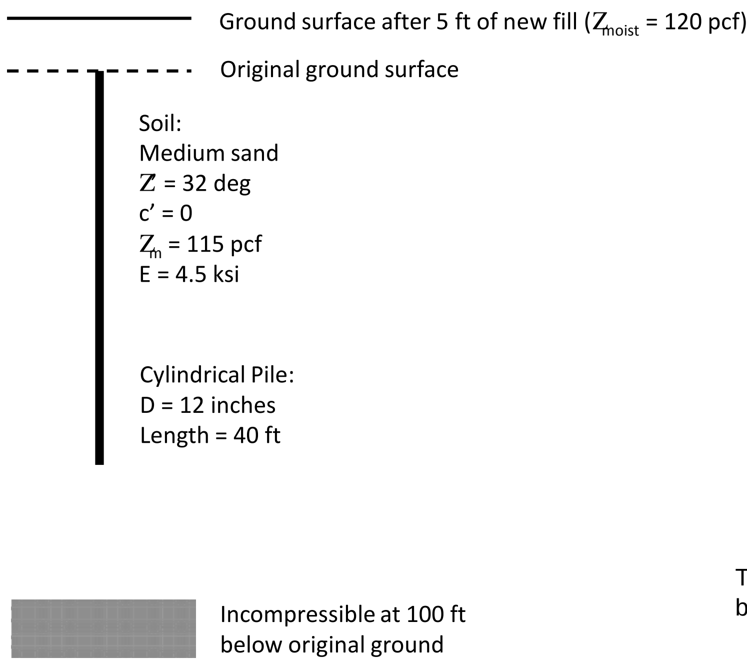
Appendix: Design Examples

Example 1 - New Fill Above a Pile Group in Sand
Calculation of Drag Load and Downdrag using the LRFD Framework

Description:

A group of cylindrical piles is installed in medium dense sand for a bridge abutment. Each pile is 12-inches in diameter and 40 ft in length. At some time after installation, a 5-foot thick layer of new fill is placed over the area causing additional ground settlement. The drag load and downdrag for each pile in the pile group resulting from the new fill are calculated as follows.



The water table is well below the pile tip.

$Q_p := 40\text{tons}$

Given: Factored load per pile for the Strength IV load combination excluding "downdrag load" (DD) . Preferably, the load factors (γ) are 1 and the contributions from transient loads are negligible.

$T_{fill} := 5\text{ft}$

Thickness of the new fill placed in the area of the subject pile group.

$\gamma_{moist} := 120\text{pcf}$

Moist unit weight of new fill placed in the area of the subject pile group.

$\gamma_p := 1.25$

Load factor for dragload force DD when using the β method per the AASHTO Specifications (2012). (Use this for settlement and structural calculations.)

$\Delta\sigma_v := T_{fill} \cdot \gamma_{moist} = 600 \cdot \text{psf}$

Increase in vertical stress due to placement of a new fill. This is appropriate for a fill placed over a very wide area. Stresses from discrete surface loads (e.g., narrow fills) can be estimated using the procedures in Samtani and Nowatzki (2006).

$E_{soil} := 4.5\text{ksi}$

Elastic modulus of in-place sand for settlement calculations for a medium sand from Table C10.4.6.3-1 in AASHTO (2012).

Step 1. Calculate the nominal/unfactored shaft and tip resistances for the pile prior to placement of the new fill using an appropriate methodology. The following nominal resistances were computed using the Beta method for shaft resistance and the Nordland/Thurman method for tip resistance. Any accepted method for calculating pile resistance may be used; however, the results should be nominal/unfactored resistance values. Otherwise, calculation of the neutral plane depth will not be accurate.

$\beta := 0.5$ in this example

Nominal side resistance: $R_{shaft} := A_{side} \cdot \beta \cdot \sigma_{v\_eff}$

where A_{side} is the area of the side of the pile

β is the beta coefficient from AASHTO (2012)

σ_{v_eff} is the effective vertical stress

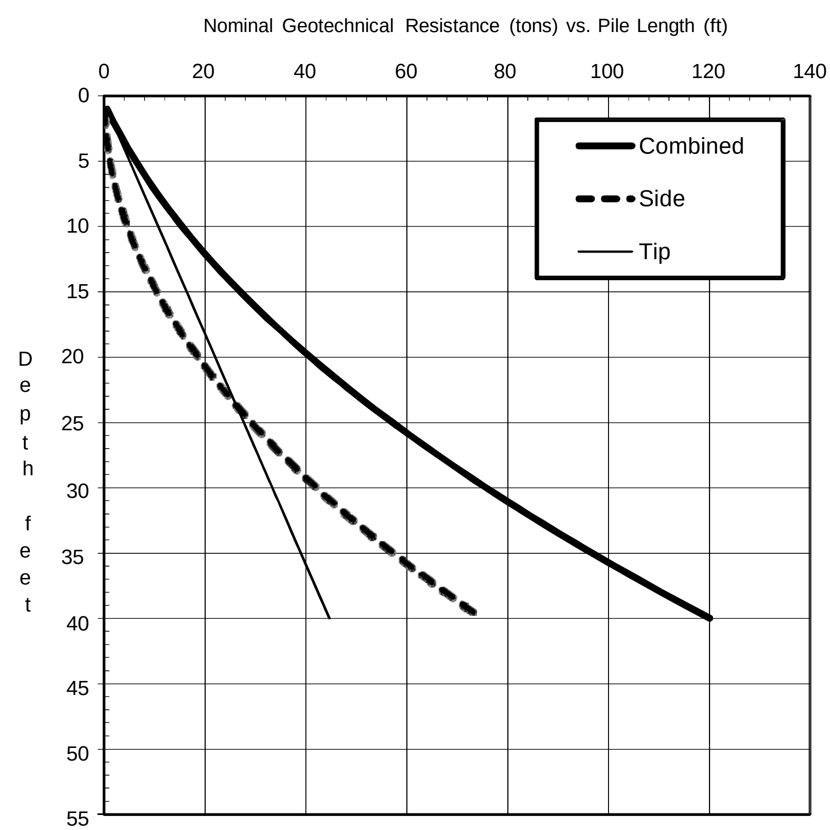
Nominal tip resistance: $R_{tip} := A_{tip} \cdot \alpha_1 \cdot N_{q\_prime} \cdot \sigma_{v\_eff}$

where A_{tip} is the area of the pile tip

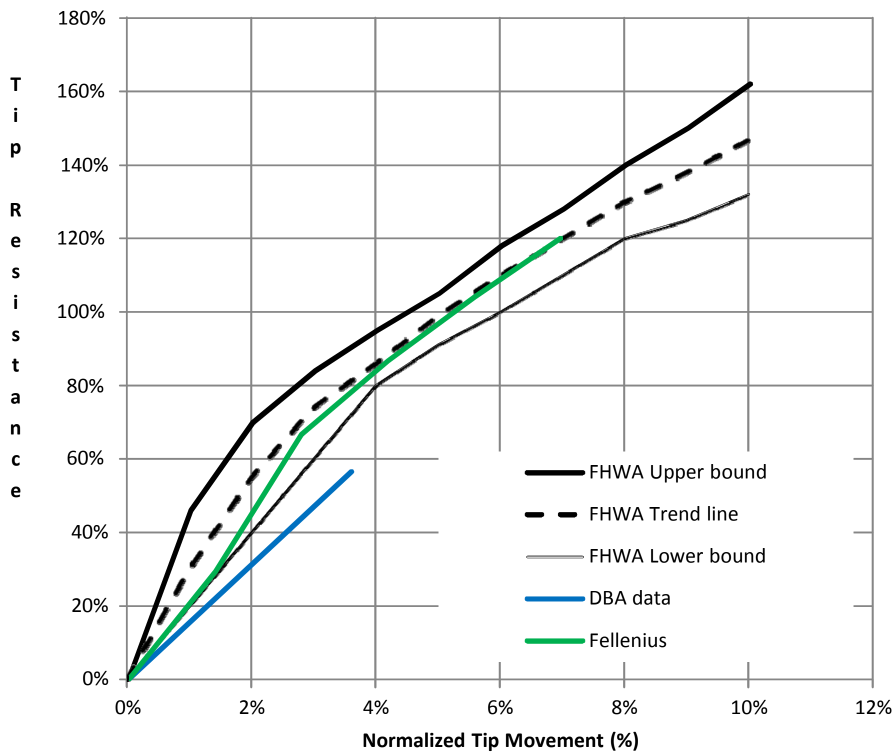
α_1 is coefficient from AASHTO (2012)

N_{q_prime} is a coeffcient from AASHTO (2012)

σ_{v_eff} is the effective vertical stress



Note. Only a small amount of relative movement between the pile and surrounding soil will fully mobilize the side resistance. A significantly greater amount of relative movement between the pile and the surrounding soil is required to mobilize substantial tip resistance. The following figure shows the normalized relationship between tip resistance (in percent of nominal value) and relative movement (in percent of pile diameter).

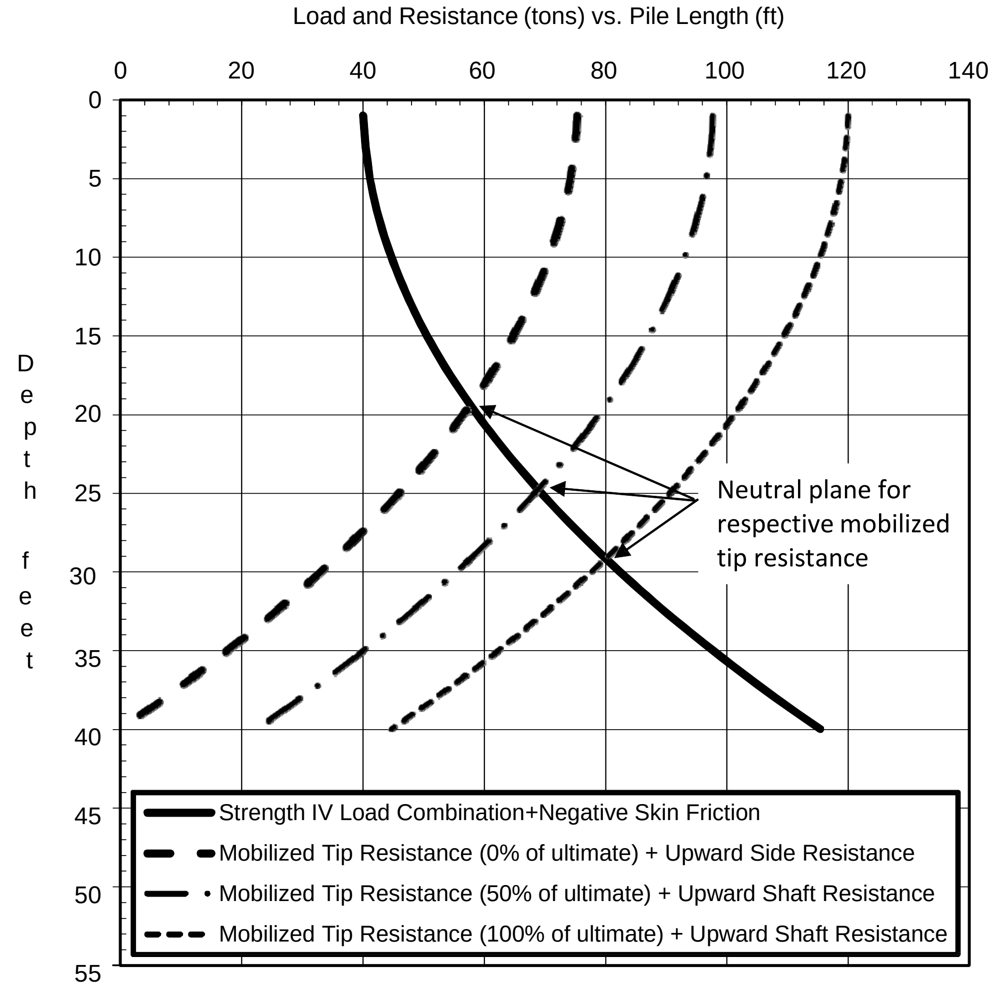


The vertical axis exceeds 100% because the FHWA defines the tip capacity as the tip resistance corresponding to a normalized tip movement of 5%. In reality, additional resistance is mobilized at greater tip movements.

Step 2. Plot the downward load (the Strength IV load combination without DD plus the cumulative side resistance acting downard - a.k.a. negative skin friction) and the upward resistance (the estimated mobilized tip resistance plus the upward nominal side resistance). The intersection of the two curves is the estimated depth of the neutral plane. As shown in the plot on the next page, the neutral plane increases in depth as the mobilized tip resistance increases. For this example, the depth range of the neutral plane is 1/2 of the pile length (with zero mobilized tip resistance) to about 3/4 of the pile length (fully mobilized tip resistance) of the pile length.

Meyerhoff (1976) concluded that pile groups in sand (and clay) could be estimated using the Equivalent Pier Foundation Method where the settlement is estimated by analyzing for an "equivalent footing" at a location of 2/3 along the pile length. The Equivalent Pier Foundation Method is described in NAVFAC Design Manual 7.02 Foundations and Earth Structures.

Fellenius (1989) proposed that the pile and pile group settlement be computed by locating the "equivalent footing" at the neutral plane depth. More recently, Fellenius (personal communication) has suggested that the lower portion of the piles (beneath the neutral plane) provide reinforcement so that this equivalent footing approach is conservative.



Note: All values in this plot are unfactored.

For this example: Assumed Mobilized Tip Resistance = 50% Nominal Tip Resistance
(There is little information available for estimating the mobilized tip resistance for piles designed for a combination of side and tip resistance. Fortunately, the analysis results are generally not sensitive to this value.)

$D_{neutral\_plane} := 24ft$ Depth of neutral plane

$Q_{max} := 68tons$ The load in the pile at the neutral plane and the maximum axial compressive (unfactored) force in pile for structural analysis of the pile section.

$Q_{drag\_load} := Q_{max} - Q_p = 28 \cdot tons$ Drag load (i.e., the downdrag force, DD, in the AASHTO Specifications) on pile prior to placement of new fill

Technically, a drag load (i.e., the downdrag force, DD, in the AASHTO Specifications) exists within the pile prior to placement of the new fill. This is not currently recognized in the AASHTO Specifications.

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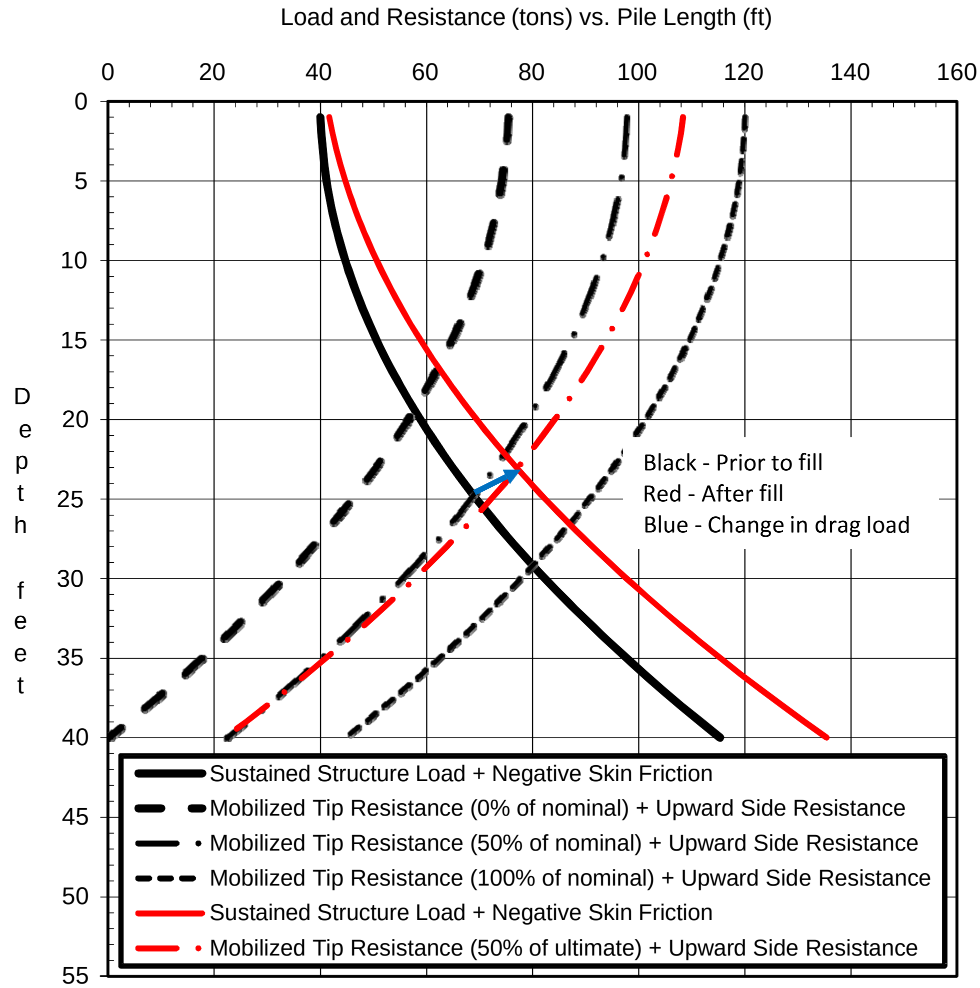
Step 3. To calculate the additional drag load that results from the placement of the new fill, re-plot the downward load and upward resistance considering the increase in the vertical and horizontal effective stresses. This is illustrated in the following plot where the black lines represent the conditions prior to fill placement and the red lines are the conditions after fill placement.



$\Delta Q_{drag\_load} := 8\text{tons}$

The increase in the drag load due to the placement of the new fill (rightward shift in the location of the neutral plane as represented by the blue arrow in the above plot).

$Q_{max} + \Delta Q_{drag\_load} = 76 \cdot \text{tons}$

The maximum axial compressive force in the pile is increased by ΔQ_{drag_load} as a result of placement of the new fill.

Note: The drag load (i.e., the downdrag force, DD, according to AASHTO) is 36 tons per pile. Transient loads of 36 tons or less (per pile) will temporarily replace the drag load with negligible movement. If the transient loads exceed the drag load, then the pile will move downward and mobilize additional tip resistance.

Step 4. To calculate the downdrag due to placement of the new fill, compute the downward soil movement at the depth of the neutral plane.

$D_{rock} := 100ft$ — Depth to rock surface

$D_{neutral\_plane} := 23ft$ — Depth of neutral plane after placement of new fill

$T_{soil} := D_{rock} - D_{neutral\_plane} = 77 \cdot ft$ — Thickness of compressible soil beneath neutral plane

$\delta_{downdrag} := T_{soil} \cdot \frac{\gamma_p \cdot \Delta\sigma_v}{E_{soil}} = 1.1 \cdot in$ — Downdrag which is the downward movement of the soil at the neutral plane. The increase in vertical stress due to the new fill is factored.

In this example, the placement of new fill after pile installation results in the following:

1. There is a slight increase in the nominal geotechnical resistance for each pile of the group as a result of the increase in effective vertical stress;

2. There is an increase in the drag load (i.e., the downdrag force, DD, according to the AASHTO Specifications) of 8 tons;

3. As a result of the increase in drag load, there is an 8 ton increase in the axial compressive force in each pile of the group at the neutral plane;

4. As a result of the settlement induced by the new fill, the pile/pile group experiences downdrag of 1.1 inches which is equal to the downward soil movement at the neutral plane, and;

5. There is an increase in the elastic shortening of each pile in the group due to an increase in axial compressive force in the piles. This is usually very small and negligible for practical purposes.

Although the soil for this example is sand, the general methodology applies to all soil types. The α or λ methods may be used instead of the β method. Furthermore, the settlement may be computed using a variety of conventional methods appropriate to the respective soil type as presented in the AASHTO Specifications(2012).

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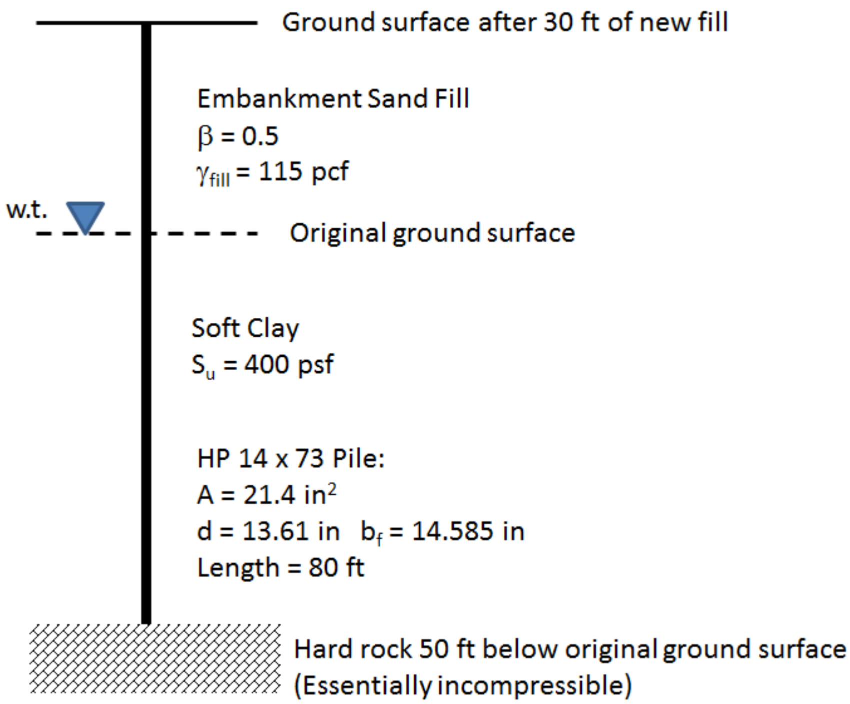
Example 2 - Rock Bearing H Piles Installed Through Soft Clay
Calculation of Drag Load and Downdrag using the LRFD Framework

Description:

A group of H piles is installed through a soft organic clay and bear on rock as shown in the sketch below. The piles are HP 14 x 73 sections with a yield strength of 36 ksi. The drag load for a pile in the pile group resulting from the new fill are calculated as follows.



$Q_p := 50\text{tons}$ — Given: Factored load per pile for the Strength IV load combination excluding "downdrag load" (DD) . Preferably, the load factors (γ) are 1 and the contributions from transient loads are negligible.

$T_{fill} := 30\text{ft}$ — Thickness of the new fill placed in the area of the subject pile group.

$\gamma_{moist} := 115\text{pcf}$ — Moist unit weight of new fill placed in the area of the subject pile group.

$\gamma_p := 1.4$ — Load factor for dragload force DD per AASHTO when using the α method per AASHTO Specifications (2012). (Use this for settlement and structural calculations.)

By observation, the neutral plane will develop at the top of the rock surface because the rock easily has sufficient strength to resist the combination of the permanent loads and the side resistance acting downward (i.e., the drag load). Typically, determination of the pile settlement requires calculation of the downward ground movement at the neutral plane. In this case, the downward ground movement at the neutral plane (and the pile settlement) is essentially zero.

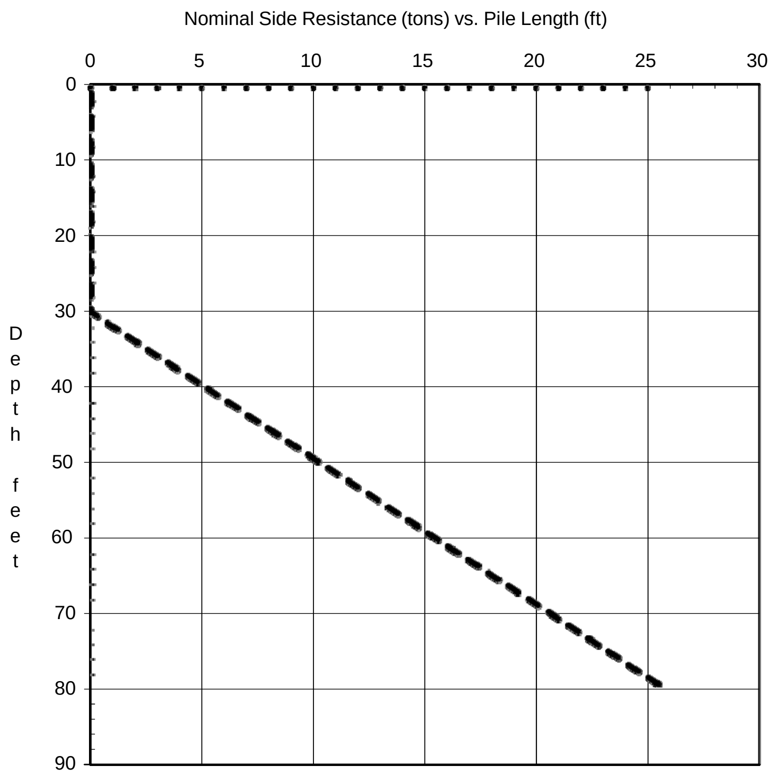
Step 1. Calculate the nominal/unfactored shaft and tip resistances for the pile prior to placement of the new fill using an appropriate methodology. The following nominal resistances were computed using the Alpha method for side resistance in the clay. All loads should have a load factor of unity. Using load factors greater than one in this approach will distort the maiximum compressive load and the location of the neutral plane.

Nominal side resistance in clay: $R_{shaft} := A_{side} \cdot \alpha \cdot S_u$

where A_{side} is the area of the side of the pile

α is the alpha coefficient from AASHTO (2012)

S_u is the undrained shear strength



Note that the depth references the top of pile and the resistance from the new fill has been ignored.

According to AASHTO, the nominal resistance of piles driven to point bearing on hard rock where pile penetration into the rock formation is minimal is controlled by the structural limit state.

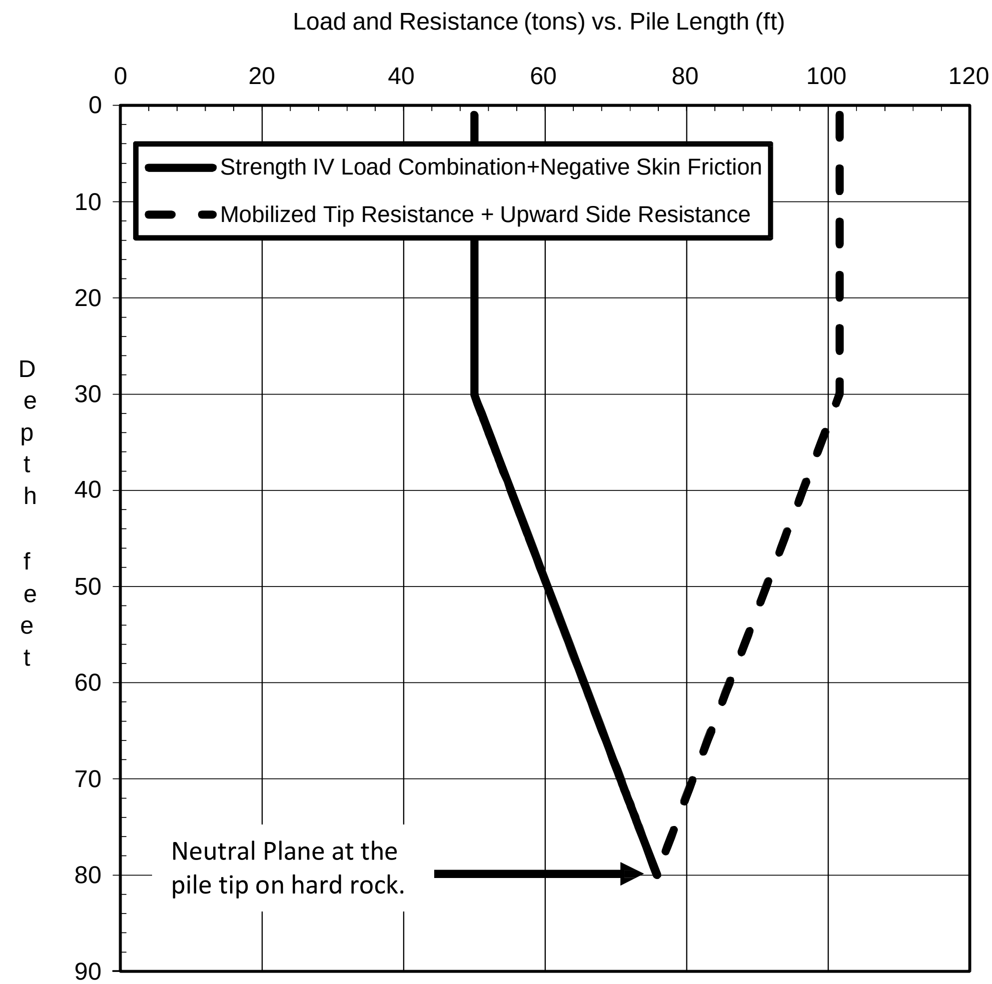
$\phi_c := 0.5$ Resistance factor for axial resistance of H-piles in compression and subject to damage due to severe dring conditions where use of a pile tip is necessary.

$f_y := 36ksi$ Yield strength of steel

$R_{pile\_section} := \phi_c \cdot f_y \cdot A_{tip} = 193 \cdot tons$ Factored structural compressive axial resistance of pile section

Step 2. Plot the downward load (the Strength IV load combination without DD plus the cumulative side resistance acting downward - a.k.a. negative skin friction) and the upward resistance (the estimated mobilized tip resistance plus the upward nominal side resistance). The intersection of the two curves is the estimated depth of the neutral plane.

As shown in the plot below, the neutral plane for this example is located at the tip of the pile (i.e., essentially the surface of hard rock). In this example, the resistance of the hard rock surface mobilizes as necessary to resist the pile load at very small pile movements.



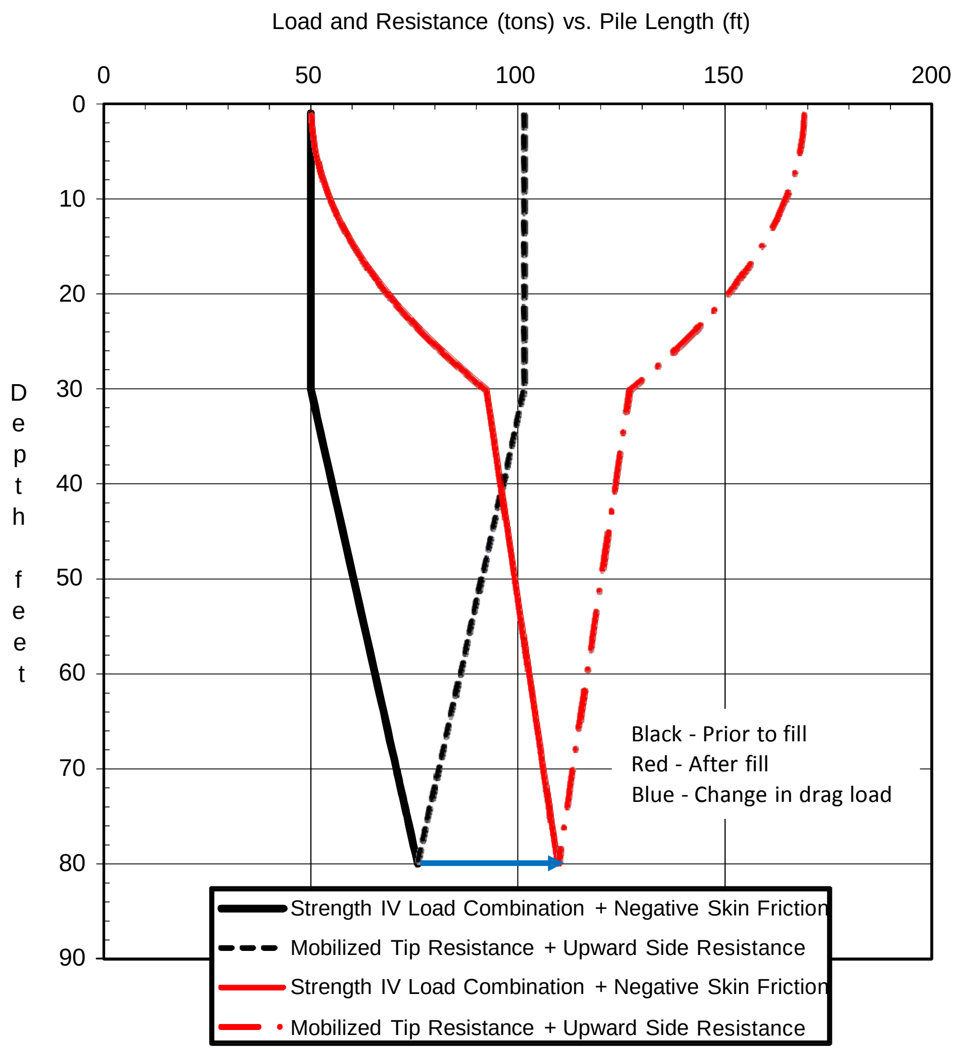
$D_{neutral\_plane} := 80ft$ Depth of neutral plane

$Q_{max} := 75.8tons$ Maximum axial compressive (unfactored) force in pile for structural analysis of the pile section.

$Q_{drag\_load} := Q_{max} - Q_p = 25.8 \cdot tons$ Drag load (i.e., the downdrag force, DD, in AASHTO) in pile prior to placement of new fill

Technically, a drag load (i.e., the downdrag force, DD, in AASHTO) exists within the pile prior to placement of the new fill. This is not currently recognized in the AASHTO Specifications (2012).

Step 3. To calculate the additional drag load that results from the placement of the new fill, re-plot the downward load and upper resistance. This is illustrated in the following plot where the black lines represent the conditions prior to fill placement and the red lines are the conditions after fill placement.



$\Delta Q_{drag\_load} := 33.9 tons$

The increase in the drag load due to the placement of the new fill (rightward shift in the location of the neutral plane as represented by the blue arrow in the above plot).

$Q_{max} + \Delta Q_{drag\_load} = 109.7 \cdot tons$

The maximum axial compressive force in the pile is increased by ΔQ_{drag_load} as a result of placement of the new fill.

Note: The drag load (i.e., the downdrag force, DD, according to AASHTO) is 59.7 tons per pile. Transient loads of 59.7 tons or less (per pile) will temporarily replace the drag load.

Step 4. Check the structural resistance available for the pile section.

$DD := Q_{drag\_load} + \Delta Q_{drag\_load} = 59.7 \cdot tons$ Drag load on pile after fill placement

$P_u := Q_{max} + \gamma_p \cdot DD = 159.4 \cdot tons$ Factored maximum axial compressive load in pile

$R_{pile\_section} = 192.6 \cdot tons \quad > \quad P_u = 159.4 \cdot tons$ The factored structural compressive axial resistance of the pile section exceeds the factored axial load and therefore the pile section would be considered acceptable for this load combination.

In this example, the placement of new fill after pile installation results in the following:

1. There is an increase in the nominal geotechnical resistance for each pile of the group as a result of the higher effective vertical stress and greater pile-soil contact area;

2. There is an increase in the drag load (i.e., the downdrag force, DD, according to the AASHTO Specifications) of approximately 34 tons;

3. As a result of the increase in drag load, there is a 34 ton increase in the axial compressive force in each pile of the group at the neutral plane, and;

4. There is an increase in the elastic shortening of each pile in the group due to an increase in axial compressive force in the piles. This is usually very small and negligible for practical purposes.

References

AASHTO. 2012. LRFD Bridge Design Specifications. Parts 1 and 2, American Association of State Highway and Transportation Officials.

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