Comparing Downdrag Design: Eisenhower Bridge of Valor

Derrick D. Dasenbrock, PE, D.GE
Federal Highway Administration Resource Center, Minneapolis, Minnesota USA

Liang Chern Chow, PE
Haley & Aldrich, Inc., Walnut Creek, California, USA

ABSTRACT: The new Eisenhower Bridge of Valor replaces the 57-year-old bridge over the Mississippi River in Red Wing, Minnesota. Old bridge inspection records indicated up to 4 feet of settlement at the north end of the old bridge through its service life. During design of the new bridge, the MnDOT focused on understanding and addressing the persistent settlement issue, as it was suspected to have caused failure of the original bridge's pile foundation and could also affect the new bridge. The site's geology, along with documented settlement issues, made this an almost ideal setting for examining pile downdrag loading. To evaluate actual downdrag response and compare measured values with predictions, four HP 14×117 steel piles were instrumented with a series of strain gauges. Two piles were located within the bridge footprint, and two were located outside the footing within the approach embankment fill. For performance comparison, two instrumented piles were driven to the top of bedrock with the aid of PDA, and two instrumented piles were driven to a depth of 5 ft above bedrock. As of 2022, the bridge is complete, and more than 3 years of data have been collected to capture the pile strain responses, including removal of a soil surcharge and in-service conditions. This paper discusses the pile monitoring results under these conditions and compares three design methods: the existing explicit method in AASHTO LRFD Bridge Design Specifications, the neutral plane (NP) method, and the MnDOT simplified neutral plane method for dragload evaluation.

1 INTRODUCTION

Bridge No. 25033 (a.k.a. Eisenhower Bridge of Valor) was built to replace the old overhead steel truss bridge constructed in 1960. The new bridge is approximately 1,630 feet long and is located just upstream from the in-place bridge, which remained in-service during construction of the new bridge. The slight realignment of US 63 in the north abutment area, needed to construct the new bridge adjacent to the old bridge, required the placement of a relatively high and wide embankment fill, over known highly to moderately compressible alluvial deposits. In addition to the design considerations related to embankment settlement and slope stability, downdrag load (dragload) imposed on bridge piles was also a known issue that needed to be addressed in the foundation design.

MnDOT had been evaluating pile performance at sites favorable for downdrag conditions for several years and this bridge project presented an opportunity to further examine the effects of soil settlement on the loads imparted into bridge foundation piles and their distribution. In addition to examining the magnitude and distribution of loads and the foundation response, the measurements obtained from the monitoring program also provided an opportunity for comparison of the appropriateness of different design methods, after the project was complete. The overall instrumentation program is described in detail by Dasenbrock et al. (2021). In the subsequent discussion, 'north abutment' refers to the project study area of the new bridge.

2 GENERAL SITE CONDITIONS AND PREVIOUS WORK

2.1 Subsurface Conditions

Surficial soils in the Red Wing bridge project area consisted of coarse sand and gravel alluvial deposits from Glacial River Warren and modern river channel deposits of sand and gravel with areas of silt, clay, and organic soils. Seventeen (17) foundation borings that included Standard Penetration Tests, split spoon and thin-walled sampling, and triple barrel rock coring with NX size were performed for the new bridge. The borings and soundings encountered typical alluvial materials, including soft silty clays, highly organic to organic silt loams and silty clay loams, and granular soils underlain by sedimentary bedrock. The north abutment and embankment area's site geology consisted of an upper 20 to 25 feet thick very loose silty sand layer underlain by 40 to 50 feet of very soft to soft, organic to highly organic silty clay. Below these soils, medium dense to very dese sand and gravel was found down to the top of Cambrian sandstone of the Franconia Formation at approximately 120 to 130 feet below ground surface (bgs). In general, the groundwater level was near the river level, which seasonal fluctuations varied from 1 to 14 feet bgs from 2017 to 2020.

2.2 Foundation Design and Recommendations

Significant settlement from the proposed embankment fill was expected to induce substantial dragload in driven piles. Using the neutral plane method and the effective stress method for the static pile analysis, the nominal dragload was determined to be as much as 700 kips with an assumed neutral plane at a depth of 85 feet below the bottom of footing or the bottom of the silty clay layer (elevation 595). This analysis assumed that embankment fill is placed and allowed to remain in place long enough to fully consolidate the underlying soils.

Recommendations for the north abutment foundation suggested the use of 14×117 of steel H-piles driven to rock. Long-term settlement at the existing north abutment and the northern approach embankment was well documented. The foundation recommendations included a permanent 700 kips predicted dragload to be incorporated into the structural design of the piles. During the design phase, both settlement and slope stability were examined.

The proposed north abutment approach embankment included new fill of almost 22 feet above the current grade at the bridge abutment, tapering lower to match the existing grade several hundred feet further north. The fill was about 70 feet wide with 1V:3H side slopes. The compressible soils were estimated to consolidate nearly 5% in response to the new loading, resulting in an estimated primary settlement of 40 inches under the new roadway centerline. In addition, estimated secondary settlement was in the range of 2 to 4 inches, occurring over the next 10 to 20 years in service. A surcharging program consisting of placement of an additional 5 to 10-foot-thick layer of fill and prefabricated vertical drain installation was used to accelerate primary consolidation of the north approach embankment in conjunction with a staged embankment construction plan and a 6 to 12-month waiting period.

This design solution, while significantly reducing in-service embankment settlement, did not address the added pile loading (dragload). As the north abutment piling was intended to be driven to rock, downdrag (settlement) was not significant because pile settlement would be constrained by the bedrock acting as a stiff end-bearing layer. The added loading on the proposed piles and the need for a sufficient number of piles of sufficient pile section to carry the added loading was, therefore, a primary design consideration at the north abutment.

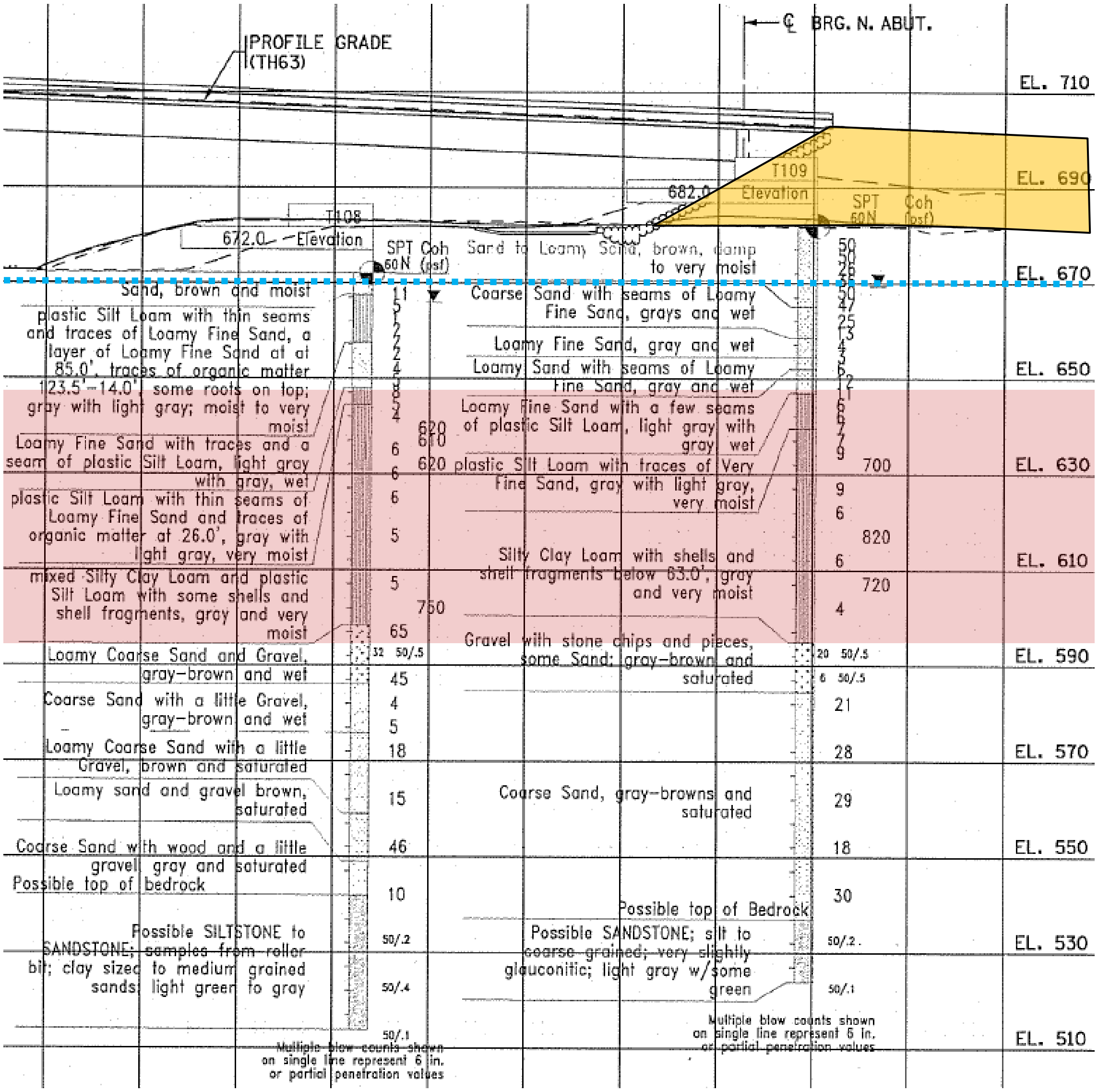

Figure 1. A portion of construction plan sheet 298 of the bridge construction plans showing soil borings advanced near the north abutment of the new bridge. The new fill is highlighted in orange shading and the compressible soil layer is shown by the red shading. The approximate water elevation is 670 ft.

2.3 Geotechnical Instrumentation

As part of the north approach embankment construction, a monitoring program was included in the project to capture the embankment responses. The plan was designed to monitor vertical displacements (settlement), pore pressure, and slope movements. In addition, an independent instrumentation program with the specific purpose to evaluate pile downdrag response was included: four HP 14×117 steel piles were instrumented with a total of 84 vibrating wire (VW) strain gauges. In addition to VW strain gages welded along the webs of the four H-piles, eight (8) earth pressure cells were installed in the embankment backfill near the pile cut-off elevation. Considerable attention to detail was needed for both gauges and cable protection throughout the pile installation process, especially as each of the four piles was constructed with two pile splices. Pile details are presented in Table 1 and the pile arrangement is shown in plan view in Figure 2. In essence, Pile “1” represents all production piles or typical bridge piles encased within the pile cap, bearing on rock; Pile “2” represents a pile that was driven “short”; the other two piles repeat the pile tip conditions but without structural top load. Figure 3 presents two stages in the construction process of the north abutment, soon after the construction of the abutment stem and wing walls (adding structural load) and after embankment backfilling (adding dragload). Results of the monitoring program are shown for the four instrumented piles in the plots in Figure 4.

Table 1. Instrumented Pile Layout.

Pile	Location	Pile Head Condition	Pile Tip Condition	Approx. Length
1	Inside pile cap	Support structural load	Driven to rock	150 feet
2	Inside pile cap	Support structural load	Driven to 5′ above rock	145 feet
3	Outside of pile cap	No structural load	Driven to rock	150 feet
4	Outside of pile cap	No structural load	Driven to 5′ above rock	145 feet

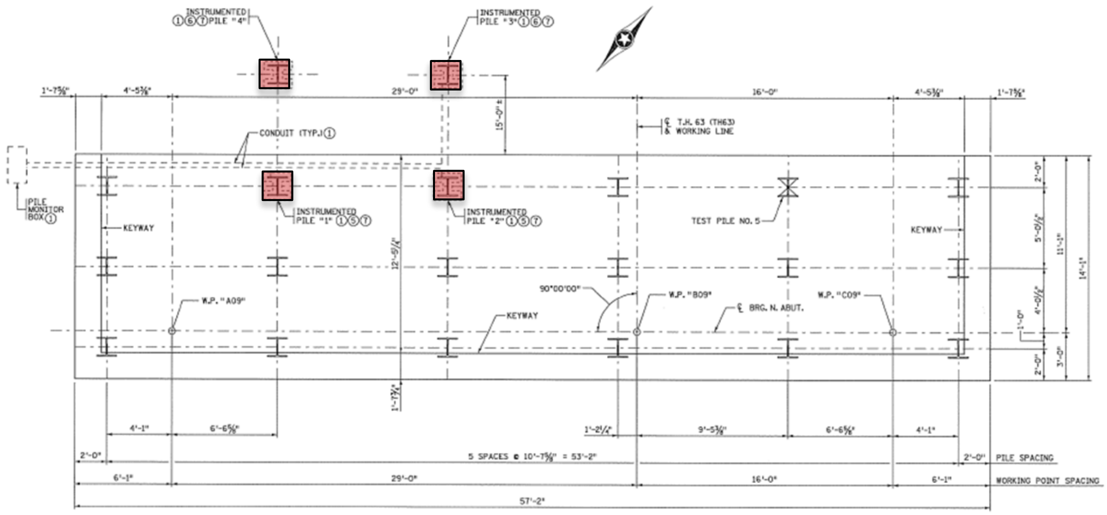

Figure 2. The north abutment used three rows of piling with six HP 14×117 piles in each row. Two additional reference piles (Pile “3” and “4”) were included in the performance monitoring program; these piles were installed external to the foundation in the approach embankment backfill to provide a reference without structural (top) load. Instrumented piles are shown in red.

Figure 3. Construction of the north abutment is shown (at left) prior to embankment backfilling. At right, new fill can be seen to the left of the abutment. The bridge beams are not yet placed.

3 PILE PERFORMANCE

3.1 Pile Strain Measurements and Dragload Response

Similar to other recent pile performance monitoring projects in Minnesota, documented by Dasenbrock and Budge (2011) and Budge et al. (2015, 2016, 2017, 2019), at site conditions favorable for dragload, placement of new fill around piling driven through a compressible subsurface layer resulted in additional loading induced by soil settlement. The raw measurements from the VW strain gages are in microstrain and are subsequently converted to load using a representative area and modulus for the steel H-piles. The results, measured over three years from 2017 to 2020, were plotted with depth and time to demonstrate load accumulation and shedding along the length of the pile and how the load distribution changed as construction stages progressed (see Figure 4). The plots also include the estimated pile resistance (dashed lines) using the λ-method based on the soil data with pile tip resistance matched to the measured loads. In this case, Pile "1" mobilized about 70% of the estimated tip resistance, while Pile "4" nearly mobilized full estimated tip resistance. It is assumed that side resistance above the neutral plane were fully mobilized due to accumulation of large settlement from the new embankment loading. The plots also show that the added dragload is a permanent load (although, dragload and overall pile load can reduce in magnitude, such as when the surcharge fill was removed after Day 770). The graphical plots also show that there is load shedding above the top of rock in the two piles extending to rock (the NP is about 30 feet above rock for Pile "1" and "3"). Assuming the NP is at the top of rock is likely to be overly conservative in many cases as described in the MnDOT Geotechnical Engineering Manual, Appendix F-3, and by Lucarelli et. al (2015).

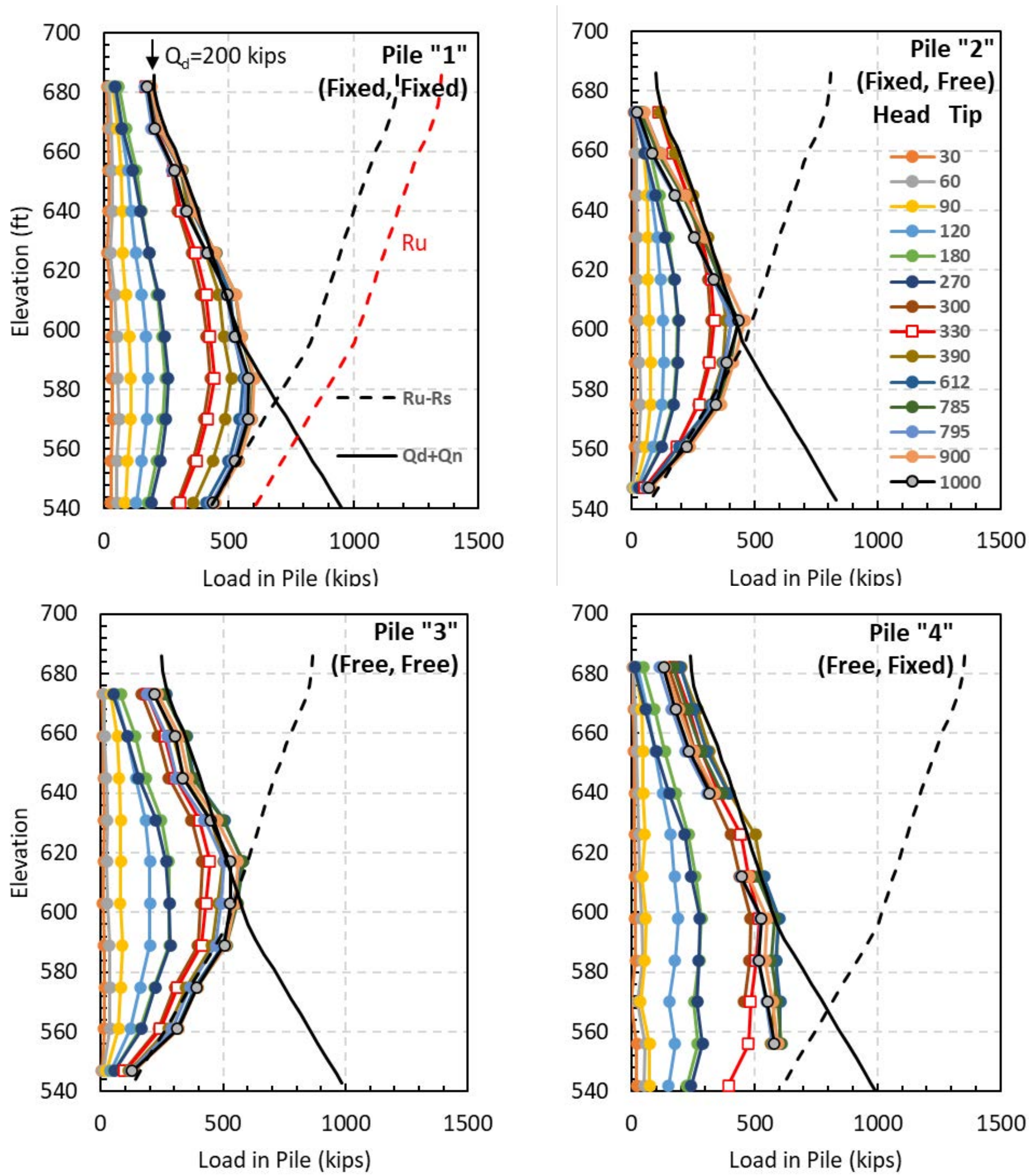

Figure 4. The shape of the load profile was different for each pile depending on the top and toe fixity conditions. Differences in load magnitude, rate, can be seen among the four piles.

3.2 The Neutral Plane Location

The location of the neutral plane corresponds to the point of maximum dragload and maximum pile load. The calculated position is where the black solid line intercepts the dashed line as shown in Figure 4, e.g., at around elevation 575 feet for Pile "1". For this project, the resolution of the neutral plane was aided by the relatively frequent spacing of the gauges at every 10% of the pile length. The instrumented piles revealed distinct distributions of load relative to pile boundary conditions of top (head) load and tip bearing. Pile load distributions changed over time, in both shape and magnitude, in response to various construction stages, including structural loading, the accelerated consolidation of the underlying soils, embankment loading, surcharging, and surcharge removal. For example, the comparatively large gap between Days 270 and 300 in Figure 4, illustrates the pile response during the abutment fill placement.

Load in the piles increases as new load is added to the pile head (structural load) or through the addition of fill placement, where the load is transferred to the piles through soil-pile side shear interaction above the neutral plane. These loads are then supported by side shear resistance below the neutral plane and end bearing. Observationally, in Figure 4, the shape of the side resistance curve changes with additional loading and load shedding. Changes in the slope of the loading curves indicate changes in the mobilized % of side shear (either accumulation or shedding) along that portion of the pile. In Figure 4, changes in end bearing are indicated by changes in position of the base of the curves along the x-axis. As seen in previous MnDOT performance monitoring projects, the location of the neutral plane is relatively stable after pile installation, through construction, and application of service loads.

4 EVALUATING AND COMPARING DESIGN METHODS

Nominal pile resistance (ultimate capacity) is evaluated to ensure adequate geotechnical strength for supporting structural (top) loads. The estimation process typically encompasses the modeling of site soils and the geometry of proposed piling for available side and base resistances with consideration of an appropriate factor of safety or reliability, e.g., resistance factor. In situations with insufficient axial capacity, the piling depth could be increased, or the size or number of foundation elements could be increased. In most circumstances, geotechnical strength limit state (geotechnical failure) evaluation only considers structural top loads and the in-place soil stratigraphy. Occasionally, there would be more extensive analysis for conditions such as scour, dynamic loading, liquefaction, or other unusual circumstances such as eccentric loading during construction and uplift.

The current AASHTO LRFD Bridge Design Specifications provide an explicit method for evaluating downdrag (Article 10.7.3.7) as well as an option to use the neutral plane (NP) method (Article 3.11.8). MnDOT adopted the NP Method for bridge foundation design based on an alternative design approach (as described in Siegel et al., 2014) and performance monitoring, with the release of its 2017 Geotechnical Engineering Manual, where section 6.9 and Appendix F describe downdrag load, dragload, and associated considerations (including a specified LRFD resistance factor) and calculations.

In LRFD, downdrag is considered at the service limit state, as it is related to settlement and serviceability of the structure. Downdrag load (dragload) is also taken account for the structural strength of piling; additive loads from adjacent settling soil can be relatively large and potentially more significant than other axial loads (such as live load). While not clearly addressed in the current AASHTO specifications, it is important to recognize that when load is applied to the pile head, downward pile movement reduces downdrag loading by mobilizing positive side resistance to counter the additional applied top load. As more top load is applied, the pile deflects which changes the relative movement of the pile and the soil. As applied load approaches the geotechnical limit strength and pile resistance is fully mobilized to resist driving forces, the length of pile over which dragload acts becomes smaller and dragload eventually reduces to zero at the geotechnical strength limit, where all side resistance (and toe resistance) is fully mobilized.

The results from the pile performance monitoring program in this study provide an opportunity to evaluate three dragload estimation methods at the service limit state.

4.1 The Neutral Plane (NP) Method

The neutral plane method uses graphical plotting of two curves: 1) sustained top load or unfactored dead load combined with the negative skin friction ($Q_d + \Sigma Q_n$), and 2) positive side resistance combined with the mobilized tip resistance (R_u - ΣR_s). The geotechnical resistance can be estimated from SPT, CPT, or other static design methods. A conceptual example of this plot is presented in Figure 5. The sustained load (Q_d) is the constant axial compressive load and therefore most closely associated with AASHTO's permanent load. The location of the neutral plane is at where the two curves intercept. A significant attribute of the NP method, unlike the AASHTO method (Article 10.7.3.7), negative skin friction or dragload is not part of the evaluation of the geotechnical strength limit state. The nominal geotechnical resistance is the combination of the cumulative side resistance along the entire pile plus the available end resistance. The NP method recognizes at geotechnical failure, the entire pile element will be moving downward relative to adjacent soil in which case, the side resistance will all be positive relative to the entire pile. While this is true at the geotechnical strength limit state, at other limit states, the dragload is considered as an internal force and must be accounted for in pile structural capacity.

Another important feature of the NP method is that live or transient load does not reduce the geotechnical resistance of the pile. The is because if transient load is added to sustained load, the (R_u - ΣR_s) curve shifts right, and as long as the total top load is below R_u, there is no geotechnical failure. Regarding the effect of transient load on the forces in the pile, the interaction is complex and the maximum compressive force in the pile will depend on the location of the neutral plane, toe stiffness, and other factors. Transient load may compress the pile and temporarily reduce the dragload component, shifting the neutral plane slightly, but the overall load in the pile will be similar (with the added transient load replacing the dragload). Additional discussion on the impact of stiffness with respect to dragload is provided by Lucarelli et al. (2015).

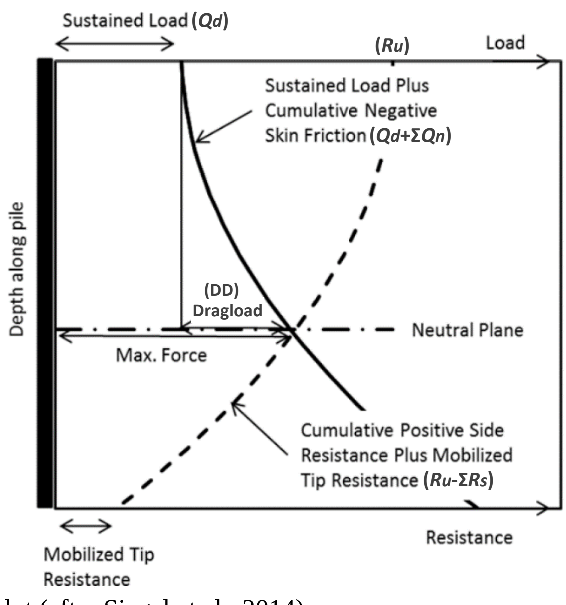

Figure 5. Neutral plane plot (after Siegel et al., 2014).

The pile monitoring program provided valuable insights about various boundary conditions and their corresponding responses. Based on the measured loads at the top gauge at each pile, the final sustained loads were in the order of 200, 100, 250, and 240 kips for Pile "1" through "4". By matching the lower portion of measured pile loads with the estimated resistances using λ-method, the mobilized tip resistances were 70%, 30%, 50% and nearly 100% for the piles, respectively.

Consequently, the dragload measured in service, in order of each pile 1 through 4, were around 500, 350, 300, and 560 kips, for use in the checking structural capacity of HP 14×117. The estimated pile resistance under geotechnical strength limit state (ultimate capacity) is plotted in the red dashed line for Pile "1" (similar to the dashed line for Pile "4" with tip resistance nearly fully mobilized) for reference in Figure 4. As more top load is applied to each pile, the loading curve translates to the right, intersecting the fully mobilized resistance curve; dragload does not reduce the geotechnical resistance of the piles as all side resistance eventually becomes available to support pile top loads.

4.2 AASHTO Explicit Method

FHWA and other guidance documents have suggested that dragload be evaluated if settlement of soils surrounding piling exceeded 0.4 inches, which downdrag is assumed to fully develop. The current AASHTO LRFD design specifications describe a method in Article 10.7.3.7, where load, resistance, and bias factors are applied in the design process, and the downdrag zone (above the NP location) should be neglected when calculating nominal pile driving resistance required (pile capacity). The aggregate soil-settlement-induced load applied to the pile along the pile length above the NP is portrayed as an additional top load, resulting in the following equation (1):

$$R_{ndr} = R_{Sdd} + \left[\frac{\Sigma\gamma_i Q_i}{\varphi} + \frac{\gamma_p DD}{\varphi}\right] \tag{1}$$

where R_{ndr} = Nominal pile driving resistance required; R_{Sdd} = Side resistance over the downdrag zone; $\Sigma\gamma_i Q_i$ = factored load per pile; γ_p = load factor for downdrag (note: only α Tomlinson and λ-method are listed in AASHTO Table 3.4.1-2); φ = Resistance factor; and *DD* = dragload per pile. This equation indicates that the specifications neglect the resistance within the downdrag zone (i.e., R_{sdd}) in the geotechnical resistance and includes a factored dragload ($\gamma_p DD$) in determining the nominal pile driving resistance required (R_{ndr}). The pile penetration depth, therefore, is adjusted proportionally to meet R_{ndr}. Neither of these design simplifications represent the actual soil-structure interaction problem well. In some cases, the explicit method can result in overly conservative of required nominal pile resistance, e.g., Olson (2022).

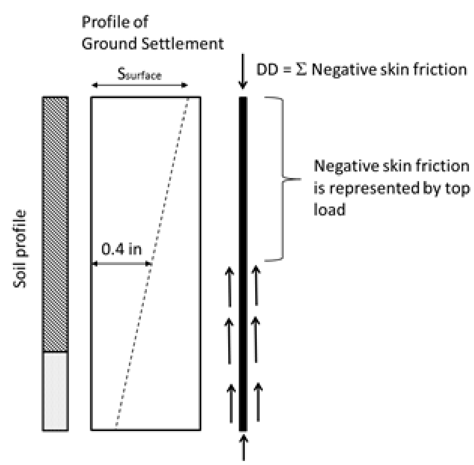

Figure 6. Simplified conceptual illustration of the explicit AASHTO approach for evaluating dragload (after Siegel et. al 2014).

It is perhaps easier to illustrate an example using Pile "1" at the north abutment as it best represents typical bridge foundation. Because AASHTO allows the use of neutral plane method, R_{Sdd} and *DD* are both around 500 kips. By using the λ-method to estimate nominal pile resistance for driven pile, the resistance factor (φ_{stat}) is 0.40 and maximum load factor for *DD* is 1.05 (AASHTO Table 3.4.1-2), resulting in $\gamma_p DD$ = 525 kips. Other relevant design information includes factored design load ($\Sigma\gamma_i Q_i$) = 411 kips per pile. By plugging all these numbers into the equation, the nominal pile resistance required (R_{ndr}) is 2,840 kips per pile! From here, we see that R_{ndr} is double of estimated pile nominal resistance or R_u of Pile "1" in Figure 4. The dragload could be higher if designer assumes a 100% mobilized tip resistance (shift the dashed line to the right). In this case, several viable options to meet R_{ndr} are to increase pile size, increase the number of piles or reduce the resistance factor by implementing other evaluation method. Note that due to the presence of bedrock, driving the piles deeper at this site is not an option.

4.3 MnDOT Estimated Dragload Design Process, Compressible Layer Approximation

In addition to more rigorous methods, the MnDOT 2017 Geotechnical Engineering Manual, Appendix F, describes a simplified method for preliminary evaluation of dragload, where downdrag conditions are both favorable and unfavorable, The general method, described in sections F-1 and F-2, uses a normalized chart for pile depth (z/D) and degree of skin friction mobilization, modified from Sun et al. (2015), applicable to various types of soil strength data such as SPT or CPT data.

Using the MnDOT simplified estimation method for favorable downdrag conditions at this site, the site stratigraphy (location and thickness of the compressible layer), pile geometry (for side shear properties), and assumed neutral plane location at the base of the compressible layer, the dragload [negative side friction, (NSF)] at the service limit can be approximated as 100% mobilized side resistance along the top 60% of the pile length (z/D = 0.6) and 50% mobilized side friction over the next 20% (z/D = 0.6 to 0.8) of the pile length, as measured from the pile head. Fully mobilized side resistance uses the same value as in capacity calculations for the strength limit state, the 50% mobilized side friction value is approximated as half of the average ultimate value. Resisting shear forces [positive side friction, (PSF)] are estimated as 50% of the fully mobilized side resistance for the remaining pile length (z/D = 0.8 to 1.0). Alternately, the resisting force is estimated as 50% of the calculated ultimate side friction acting over the bottom 20% of the pile length above the pile toe. Refer to Figure 7 and Table 2.

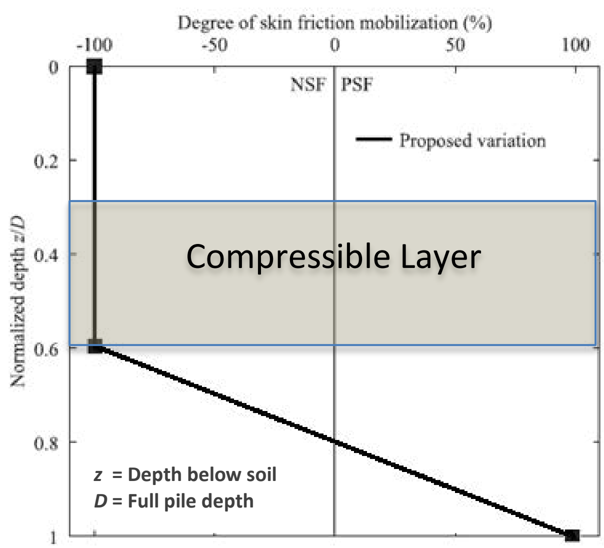

Figure 7. Plot of the % of PSF and NSF for piles at the north abutment using the MnDOT simplified method for favorable dragload conditions, based on site stratigraphy.

The degree of skin friction mobilization based on position using normalized pile depth. The point of transition to PSF is approximated at the base of the compressible layer. Above the base of the compressible layer at z/D = 0.6, negative skin friction is taken to be 100% (fully) mobilized; below the base of the compressible layer exists a “transition zone.” The neutral plane is at ½ the remaining pile length between the base of the compressible layer and the pile toe, shown at z/D = 0.8. The degree of mobilization is 0% at the neutral plane; at this location there is no relative movement of the soil and the pile. Note that in when applying this diagram, PSF is not 100% (fully) mobilized until the pile toe (after MnDOT, 2017). The mobilization values are used with the unit side friction over the corresponding normalized length of the pile. For a 150-foot pile, each 10% represents 15-foot section of pile. Summing the unit side friction values and multiplying by the percent mobilization provides the estimated value.

The perimeter surface area of an HP 14×117 pile is about 7.2 ft^2 per foot of embedded length. For the instrumented piles, about 92 feet of piling is located above the base of the compressible layer, with the remaining portion (about 52 feet) of the pile extending through generally coarse sandy material to the pile toe. As the soil profile changed somewhat with depth, an average 1.0 ksf was taken from the CPT sleeve friction data as representative of the side friction from the ground to the base of the compressible layer for purposes of the following calculation: (92 feet) × (7.2 ft^2/ft) × (1.0 ksf) results in 660 kips of fully mobilized dragload. If the remaining pile dragload in the partially mobilized region is calculated as (30 feet) × (7.2 ft^2/ft) × (1.7 ksf) × (0.5) = 180 kips of additional dragload in the partially mobilized zone, then the resulting estimate is about 840 kips. This over-predicts the measured dragload from the performance monitoring program. The over prediction could come from many sources, such as the simplified diagram, side resistances used, or the H-pile surface area (assumed as the perimeter area, rather than box area).

This simplified method provides value as a preliminary check (as is its suggested purpose by the MnDOT manual) on the potential dragload magnitude at the service limit state and structural strength limit state for pile design It is recommended that if the dragload is large that it is calculated by more rigorous spreadsheet or software methods.

Table 2. Degree of skin friction mobilization using MnDOT simplified estimate for favorable downdrag conditions for piling in the north abutment of BR 25033.

Normalized Length	NSF/PSF Mobilization	CPT Average Sleeve Friction (psi)
0 – 10%	100% - 0%	18
10 – 20%	100% - 0%	8
20 – 30%	100% - 0%	8
30 – 40%	100% - 0%	3
40 – 50%	100% - 0%	3
50 – 60%	100% - 0%	3
60 – 70%	75% - 0%	12
70 – 80%	25% - 0%	12
80 – 90%	0% - 25%	18
90 – 100%	0% - 75%	18

Notes (on pile and strata locations and elevations):
1. Pile head: 687 ft.
2. Compressible layer (top to base): 645 to 595 ft.
3. Assumed NP location (based on the method): 595 ft.
4. Pile "2" and "3" toe: 543 (5′ above bedrock estimate) - 144 ft pile length — 92 ft of NSF; 52 ft of PSF (64%).
5. Pile "1" and "4" toe: 538 (bedrock estimate) - 149 ft pile length — 92 ft of NSF; 57 ft of PSF (62%).

The performance monitoring at the Red Wing bridge project showed that piles behave differently in the same geologic setting depending on the toe fixity and % tip mobilization. One complexity in applying the NP method is that the percentage of mobilized tip stress is needed. Tip mobilization can be estimated from local or previous experience, or the use of t-z and q-z curves and iterative methods in spreadsheets or computational modeling software. A discussion of load-movement response and t-z and q-z functions is provided by Fellenius and Rahman (2019).

A method to quickly estimate the % tip mobilization at the service limit state is using the diagrams and relationships outlined in the Appendix F-2 of the MnDOT Geotechnical Engineering Manual and Equation 2 below:

$$NSF + PSF + (\%\, Tip\ Mobilization * Base\ Resitance) = Unfactored\ ServiceLoad \qquad (2)$$

where NSF is the mobilized side resistance above the NP, PSF is the mobilized side resistance below the NP, and the Base Resistance is the fully mobilized Geotechnical Strength Limit.

Calculating the NSF and PSF values are outlined in section 4.3 and shown in Table 2 (above). Some judgment can also be applied, noting that harder and stiffer bearing materials should result in greater tip mobilization. For this project, more tip mobilization would be expected for the piles bearing on the sandstone bedrock, as compared to the piles purposefully stopped short of rock. The measurements shown in Figure 4 show this anticipated result to be confirmed by the measurements. Significantly more load was measured in the bottom gages in piles 1 and 4, bearing on rock, as compared to piles 2 and 3.

4.4 Additional Observation on Piles 3 and 4

Measurements showed the piles located outside the structural footing attracted significant loading near the top from the influence of the surrounding embankment loading. This load attraction is consistent with observations from the design of column supported embankments. The loads in Pile 4 (free head, fixed toe) were the largest measured, perhaps as the toe was fixed and there was no structural top load acting to reduce the dragload in that pile (top loads compress the pile and the elastic shortening changes the relative movement between the pile and the surrounding soil, reducing the dragload).

As dragload is applied by soil-structure shear forces, so long as piles are designed with sufficient internal structural capacity, to utilize all potential geotechnical resistance, piles should not fail structurally. The total internal pile load, including dragload at a given depth along the pile at a service limit state can't exceed the pile internal load (at that location along the pile) at the geotechnical strength limit state. As additional loads are added, the NP moves up, relative pile movement with respect to the surrounding (settled) soil changes, dragload reduces, and loading curves begin to take the shape of curves at the strength limit state (dashed lines in Figure 4).

5 CONCLUSIONS

The Eisenhower Bridge of Valor's pile performance project provided valuable insight into the magnitude and behavior of downdrag loading among four piles with different top and base conditions. The measurements obtained allowed a comparison between the AASHTO LRFD Bridge Design Specifications downdrag method [current as of the 2020 9^{th} edition, Article 10.7.3.7], the neutral plane method (currently adopted by Departments of Transportation in several states), a simplified NP prediction method, and measured pile in-service performance.

The main contrast between the AASHTO explicit method and the neutral plane (NP) method is how downdrag loading is evaluated at the service and strength limit states. In AASHTO, dragload is considered at the geotechnical limit state, whereas NP method excludes dragload at the geotechnical limit state. As presented as an example herein, the consideration of factored load and dragload, as well as neglecting side resistance within the downdrag zone to estimate required pile resistance, produced an excessive design demand using the AASHTO method. The NP method treats the distributed loading and load shedding along the length of the piling more realistically than the explicit method. The graphical plots included in the NP design methods match well with plots generated from measured field behavior at this site. While determining the % tip mobilization (by judgment, local performance experience, or using computer software) adds complexity, the NP design method has two significant advantages over the AASHTO method. Recognizing there is no dragload at the strength limit state eliminates unnecessary overdesign at this limit state. Secondly, the "dragload region" above the estimated location of the neutral plane is available to provide side resistance; it is not discounted or ignored for contributing resistance to support other loads at the strength limit state. Use of the NP method has positive implications for labor, time, and material savings on deep foundation projects.

At present, there can be a significant inconsistency in the dragload calculated by the two methods described in the AASHTO specifications. Improvement in the specification language, and associated resistance factors, such as providing resistance factors for the NP method, is needed to aid practitioners. Adoption of the NP method as the primary (or only) method for calculating dragload and downdrag would significantly improve the state of practice by providing a more consistent approach which is more representative of the nature of the soil-structure interaction.

MnDOT used the NP method to estimate dragload for this bridge, consistent with the 2017 MnDOT Geotechnical Engineering Manual, and it agreed well with measured values from the performance monitoring program. Outcomes from this project support the continued application of the NP method in practice. Finally, the MnDOT simplified method can provide a relatively fast check on the magnitude of the dragload to determine if additional more rigorous methods are needed, or if the dragload is relatively small and not significant to the foundation design.

6 ACKNOWLEDGEMENTS

The authors wish to thank the MnDOT Office of Materials, Geotechnical Section and Bridge Office, Wisconsin DOT, American Engineering Testing, Inc., and Zenith Tech for their contributions and support in the project. Special thanks to the field engineering team who successfully installed and protected the instrumentation and cabling during construction.

7 REFERENCES

Budge, A.S., Dasenbrock, D.D., Mattison, D.J., 2015. A synthesis of pile performance monitoring projects in downdrag environments in Minnesota, *Proceedings of IFCEE 2015*, March 17-21, 2015, San Antonio, TX.

Budge, A.S. and Dasenbrock, D.D., 2016. The downdrag response of driven pile bridge foundation in Minnesota, *Proceedings of GeoVancouver 2016*, October 2-5, 2016, Vancouver, B.C., Canada.

Budge, A.S. and Dasenbrock, D.D., 2017. Variability of pile-soil interaction: a comparison of behavior seen in instrumented pile pairs at three MnDOT bridge sites, *Proceedings of Geotechnical Frontiers 2017*, March 12-15, 2017, Orlando, FL.

Budge, A.S. and Dasenbrock, D.D., 2019. The Dragload Response of a Driven Pile Bridge Foundation in Minnesota Given “Ideal” Downdrag Conditions, *Proceedings of the University of Minnesota 67th Annual Geotechnical Engineering Conference*, February 22, 2019, St. Paul, MN.

Dasenbrock, D.D., Budge, A.S., Borree, B.R., 2011. Behavior of a MnDOT bridge foundation subject to downdrag, *Proceedings of the University of Minnesota 59th Annual Geotechnical Engineering Conference*, February 25, 2011, St. Paul, MN.

Dasenbrock, D.D., Chow, L.C., Van Heuveln, S.D., 2021. Evolution of the dragload behavior during construction of four H-Piles with different top load and end fixity conditions at a bridge abutment with embankment fill, *Proceedings of IFCEE 2021*, May 10-14, 2021, Dallas, TX.

Fellenius, B.H. and Rahman, M.M., 2019. Load-movement response by t-z and q-z functions. Geotechnical Engineering Journal of the SEAGS & AGSSEA Journal, September 2019, 50(3) 11-19.

Lucarelli, A., Blanksma, D., Dasenbrock, D., & Peterson, R. (2015). Effects of Tip Stiffness on Piles with down Drag. Transportation Research Record, 2511(1), 45–52.

Minnesota Department of Transportation (MnDOT) Bridge No. 25033 Bridge Construction Plans, 300 sheets, 2016.

MnDOT 2017 Geotechnical Engineering Manual (Including Appendix F)

Olson, S.J., 2022. Driven piles design: an alternative considering pile behavior under service and drag loads, *Proceedings of Geo-Congress 2022*, GSP 332, March 20-23, 2022, Charlotte, NC.

Siegel, T.C., Lamb, R., Dasenbrock, D., Axtell, P.J., 2014. Neutral plane method for drag force of deep foundations and the AASHTO LRFD Bridge Design Specifications. *Proceedings of the University of Minnesota 62nd Annual Geotechnical Engineering Conference*, February 28, 2014, St. Paul, MN.

Sun, T.K., Yan, W.M., Dong Su, 2015. Fully coupled consolidation analysis of shear strength mobilization and dragload of a pile subject to negative skin friction, *International Journal of Geomechanics*, 15(3), 2015.

SP 2515-21 Bridge 25033 TH63/Mississippi River and CP Rail Foundation Analysis and Design Recommendations for Abutments and Piers 3-6, May 15, 2015, by Rich Lamb.

SP 2515-21 North Approach Embankment for Bridge 25033 and Proposed 825th Street Embankments Foundation Investigation and Foundation Recommendations, June 18, 2015, by Rich Lamb.