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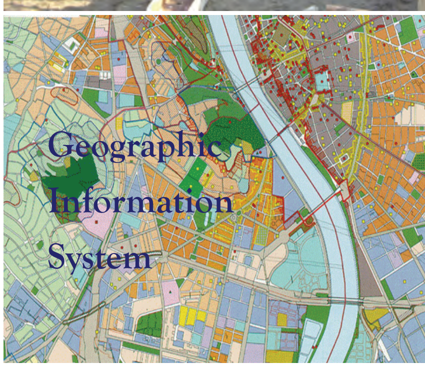
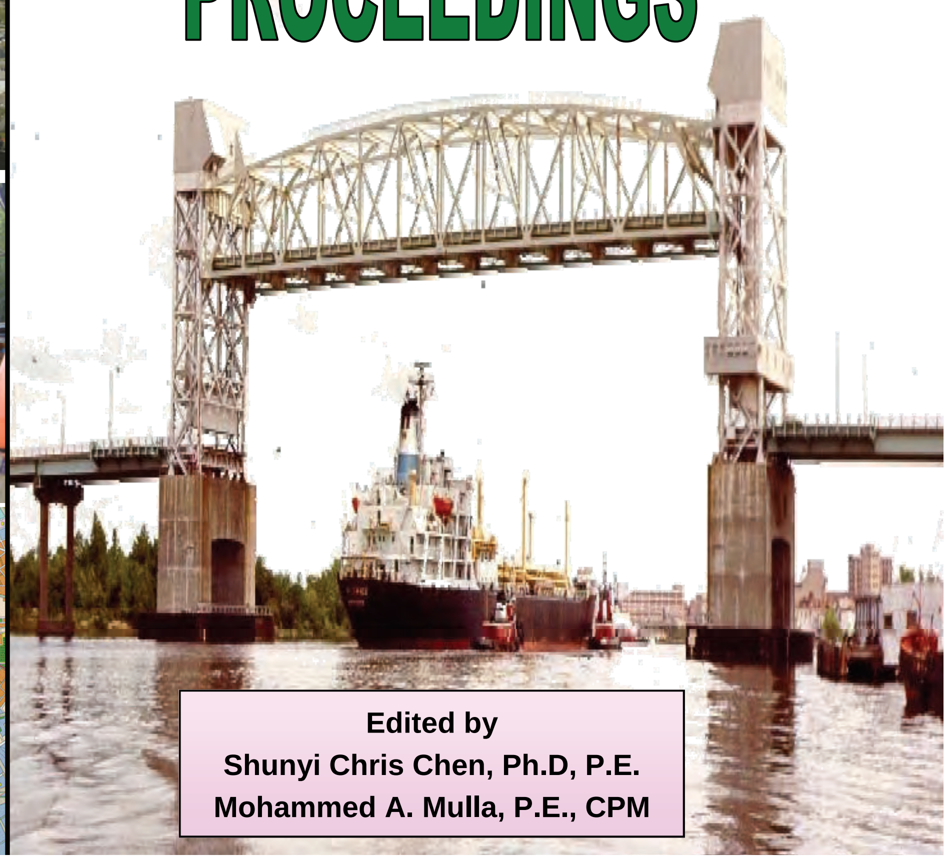


PROCEEDINGS



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LRFD in Practice – A Case Study for Foundation Designers

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ABSTRACT

The kcICON project in Kansas City, Missouri is a $150 million design-build project that will include replacing the I-35 over the Missouri River with a new signature structure. The foundations for the majority of the bridge are large diameter drilled shafts, and driven H-piles support the abutments. The H-piles were originally designed using allowable stress design (ASD) - conventional geotechnical foundation design using factors of safety to determine the allowable pile capacity. During pile driving, the pile driving analyzer (PDA) was used; however, the PDA results showed significantly lower pile capacity than anticipated. The low driving resistance occurred in loose to dense sands below the water table, even at depths of over 80 feet.

In order to evaluate the design and driving results, the geotechnical design team considered several methods of analysis to predict the pile capacity, including three different static analysis methods and wave equation analysis of piles (WEAP). The foundations were also evaluated using LRFD, even though LRFD was not part of the project design specifications for the kcICON project. The structural engineers provided the LRFD loads, which were higher than the ASD loads used for the original design. The geotechnical designers then evaluated the pile lengths using LRFD methods and resistance factors.

The presentation will present an interesting, real-life comparison of design methods, loads, and pile lengths using ASD and LRFD. The differences between the methods will be demonstrated, and the suitability of field inspection methods and acceptance procedures will be discussed.

INTRODUCTION

The kcICON project in Kansas City, Missouri is a $240 million design-build project that will include replacing the I-35 over the Missouri River with a new signature structure. The foundations for the majority of the bridge are large diameter drilled shafts. Driven HP 14x73 piles support the abutments (or end bents).

Soil borings were drilled both pre-bid and post-bid. The general subsurface conditions predominantly consist of loose to dense alluvial sand deposits (SP, SM, and SC) with interbedded lenses and thin strata of clay and silt (CL and ML). Underlying the alluvial deposits are shale and limestone bedrock of the Pleasanton Formation at depths of approximately 110 feet or more at the South End Bent and 135 feet or more at the North End Bent.

During the proposal phase, the design/build team was given the option to base the design on allowable stress design (ASD) as contained in AASHTO Bridge Design Specifications, 17th Edition, 2002 or load factor and resistance design (LRFD) as contained in AASHTO LRFD Bridge Design Specifications, 4th

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Edition, 2007. The team elected to use ASD for the project, and ASD was thus specified in the design-build contract.

PILE STATIC DESIGN

The foundation scheme for the end bents was a single line of vertical HP14x73 piles. The center to center spacing between piles was about eight pile diameters. The design service loads were 186 kips per pile at the South End Bent and 156 kips per pile at the North End Bent. Two designs were conducted, one for piles installed after the approach embankment fill was placed and allowed to settle, and the other for piles installed prior to placement of the fill. Ultimately the sequence called for the piles to be driven after the fill was placed. The controlling event for pile resistance was the flood case. The reduced effective stress caused by the high water reduced the available axial resistance during the flood. The piles therefore had to be driven to a tip elevation that had a resulted in a much higher resistance for the static load case to provide the required resistance for the flood case. Both abutments are located on the protected side of federal levees, so scour is not predicted at those locations.

The static calculation method employed for design was the Beta (or effective stress) method in general accordance with that presented in the Design and Construction of Driven Pile Foundations, FHWA Publication No. NHI-05-042, April 2006 (p. 9-53). The original design used a factor of safety of 2.25 based on the use of dynamic pile testing for verification of resistance during or after pile installation. This factor of safety was used for the flood case so that load restrictions on the bridge are avoided during floods (e.g., the groundwater level for design was at Elevation 752 ft, as opposed to its normal level of about Elevation 720 ft). Corrugated metal pile (CMP) sleeves were placed at the pile locations prior to placing the embankment fill, and the CMP sleeves were backfilled with sand after the piles were installed. As such, the design neglected axial resistance developed in the fill.

The original ASD-based pile design for the North End Bent is presented in Figure 1 as an example. The figure shows a target driving resistance of 490 kips to achieve the required factor of safety during the flood case. A similar design graph was provided for the South End Bent with a target resistance of 540 kips.

DYNAMIC TESTING

During pile driving at the North End Bent, the pile driving analyzer (PDA) was used to verify the pile resistance. While the end of initial drive (EOID) resistance was expected to be somewhat lower than the required driving resistance, it was anticipated that re-strikes would demonstrate the required resistance was achieved or exceeded.

One pile was driven to the estimated tip elevation of 693 feet. The EOID dynamic capacity was 195 kips, well below the target driving resistance of 490 kips. A second pile was then driven to a tip elevation 30 feet below the estimated tip elevation (an extra 30 feet of embedded pile). The EOID dynamic capacity was 250 kips, still well below the target and not significantly higher than the first pile that was 30 feet shorter. Six days later, a re-strike was conducted on the longer pile. A CAPWAP analysis was performed on the re-strike, yielding a dynamic capacity of 280 kips, a 12% increase from the EOID but still well below the required 490 kips.

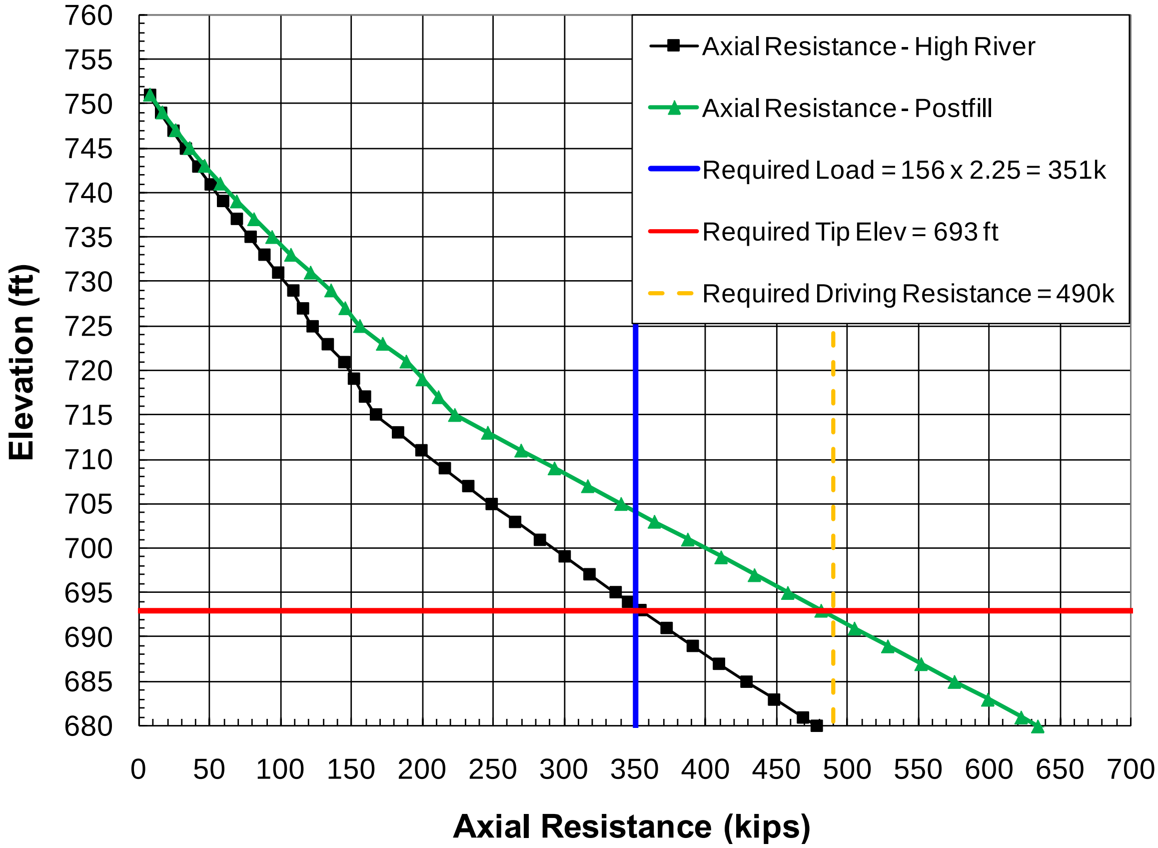

Figure 1: Axial Resistance for HP14x73 Piles at the North End Bent

The second author observed similar dynamic behavior of H-piles not tipped in dense or hard bearing material on a previous project a few miles upstream of the kcION project. In the 2002/2003 time-frame, U.S. Army Corps of Engineers Missouri River Levee (L-385) was constructed in Riverside, MO. The soils at L-385 are similar to those found at the North End Bent as both projects are in the Missouri River flood plain and consist of recent alluvial deposits. Five HP 14x102 test piles were installed at the L-385 project. Each test pile included dynamic measurement (PDA and associated CAPWAP) at the time of driving and on several re-strikes at various times after initial drive. Static load tests in accordance with ASTM D 1143 (Quick Method) were also performed. All five test piles yielded *dynamic* capacities less than the capacity obtained from top-down *static* load testing. The average underestimate was 31% (underestimates ranged from 26% to 37%, with a standard deviation of 5%).

Based on the data collected at L-385, it seemed apparent that the dynamic capacity measurement significantly underestimated the static capacity of the H-piles tipped in medium-dense sand. It was beyond the scope of the kcICON project to determine the cause of the under prediction, nor is it within the scope of this paper. However, possible causes include:

1. The sands experiencing significantly more loss of shear strength during driving due to increased pore pressures at the pile-soil interface.

2. The dynamic action of the test during driving results in the base resistance being engaged only by the area of the steel section (e.g., only the web and flanges, not the "plugged section"). During a static test the rate of loading is applied several orders of magnitude slower than during a pile hammer blow, allowing a portion or all of the "plugged" section of the pile to be engaged with base resistance.

Ultimately, the design/build team deemed field verification using high-strain dynamic tests with signal matching (PDA and CAPWAP) as unreliable in these soils with this pile type. This assessment was based on the local experience mentioned above, comparison with additional static calculations using several different analysis methods, comparison of dynamic test results on concrete piles at adjacent structures for the project, and engineering judgment of the observed performance.

REVISED DESIGN METHODOLOGY

The original ASD-based pile design at the end bents required a factor of safety equal to 2.25 based on verification by dynamic testing. Since the dynamic testing was deemed unreliable, an alternate method to verify that the piles were acceptable was needed. Performing a static load test was not feasible at the abutments because of the proximity to traffic on the adjacent freeway, the newly-placed MSE abutment wall and fill, and the design/build, so the team first sought an analytical solution.

The first alternate considered was to establish the pile tip elevations using a factor of safety equal to 3.0 based on static calculations in the absence of in-situ testing, as allowed by the AASHTO ASD code. To achieve a factor of safety equal to 3.0 under the extreme flood condition (FS = 4.0 under normal groundwater conditions), the piles would have to be driven an additional 15 feet beyond the initial design tip elevations, as illustrated in Table 1.

Table 1: Revised Pile Lengths using ASD Design

Location	*Initial Pile Tip Elevation (feet), FS =2.25*	*Revised Pile Tip Elevation (feet), FS =3.0*	*Length Increase (feet)*
North End Bent	693	678	15
South End Bent	693	678	15

The piles at the North End Bent were installed to the revised tip elevation based on the FS=3.0 analysis. The South End Bent Piles were not scheduled to be installed until several months later.

After the revised ASD analysis was completed and new driving criteria submitted, there were several discussions among the design/build team, MoDOT, and FHWA. Eventually FHWA suggested that an additional analysis should be performed using the LRFD method (AASHTO LRFD Bridge Design Specifications, 4th Edition, 2007) and compared to the ASD calculations. The LRFD code allows in Section C10.7.3.8.6a (Page 10-98) that "… a static analysis may be used to establish pile installation criteria if dynamic methods are determined to be unsuitable for field verification of nominal axial resistance."

To utilize the LRFD approach, the structural designer had to provide factored loads computed according to the LRFD method. These loads differed from the loads calculated using ASD because of the different load factors and limit states applied. Table 2 shows the difference in the controlling load for ASD and LRFD. The required resistance for ASD is thus the design load times the factor of safety (in this case, 2.25 and 3.0), whereas the required resistance for LRFD is based on a factored resistance that must be greater than or equal to the factored load.

Table 2: Design Load Comparison for ASD and LRFD

Location	*Design Load – ASD (kips)*	*Factored Load – LRFD (kips)*
North End Bent	156	218
South End Bent	186	241

As suggested by FHWA, the axial resistance of the piles was calculated using the Nordlund method in accordance with FHWA Manual No. NHI-05-042, April 2006, Section 9.7.1.1c (Pages 9-25 to 9-41). Hand calculations were verified with the FHWA software DRIVEN, which also uses the Nordlund method in sands.

A summary of the revised computations is listed below.

1. The resistance factor required using the Nordlund method is 0.45 (Table 10.5.5.2.3-1, Page 10-39) for a redundant pile group. The required nominal axial resistance per pile was 536 kips (241k ÷ 0.45) at the South End Bent and 484 Kips (218k ÷ 0.45) at the North End Bent.

2. All design values were chosen in accordance with Section 9.7.1.1c of the FHWA Manual. The design values were selected based on the corrected SPT blowcounts.

3. The "box" area of the HP14x73 piles was used for both side resistance and base resistance as allowed in the FHWA manual (Section 9.10.5) due to anticipated plugging of H-piles under static loading in cohesionless soils with large Pile Length-to-Pile Diameter ratios. The L/D

ratio for the piles is greater than 70, or more than twice as long as needed for a plug per FHWA guidelines.

4. The controlling case considered groundwater at El. 747 ft., which corresponds to the 100-year flood as reported in the project hydraulic report. The design extreme flood event yields groundwater at El. 752.6 ft., only 5.6 ft. higher. This case, with a resistance factor equal to 1.0, did not control.

5. The variability was accounted for by creating a separate soil zone for each SPT blowcount (i.e., a compilation of 5 ft. soil layers, each with its own design values based on its own N). Individual sets of computations were conducted for both borings at each end bent. The nominal axial resistance computed using the soil properties from each boring only varied by 1 kip.

6. The additional resistance that will be provided in side resistance by the clean sand placed inside the 30 ft. CMP sleeves was neglected, as per the original design.

7. The weight of the embankment was included by using a Boussinesq stress distribution beneath the corner of a rectangular ramp loading. Hence, the lowest in-situ stress condition was used in the analysis of each pile along the line.

At the North End Bent, the piles were already installed to the previous design tip elevation of 678 feet. The LRFD analysis showed that the nominal pile resistance for piles at that tip elevation exceeded the required nominal resistance of 484 kips. At the South End Bent, the LRFD analysis showed that a pile tip elevation of 675 feet would yield the required nominal pile resistance. This elevation was 18 feet deeper than required for the original design, and only 3 feet deeper than the ASD FS=3 analysis required. Table 3 illustrates the revised pile tip elevations. The South End Bent piles were subsequently installed to the LRFD tip elevation.

Table 3: Pile Lengths using LRFD Design

Location	***Pile Tip Elevation (feet) ASD FS =2.25***	***Pile Tip Elevation (feet) ASD FS =3.0***	***Pile Tip Elevation (feet) LRFD φ=0.45***	***Increase in Length (feet) ASD FS 2.25 vs. LRFD φ=0.45***
North End Bent	693	678	678	15
South End Bent	693	678	675	18

CONCLUSIONS

This paper has presented a case history of evaluation of driven H-piles by both ASD and LRFD design methods. The piles were originally designed utilizing ASD with a factor of safety commensurate with using dynamic testing to verify the pile design. After the dynamic testing was deemed unsuitable for verifying the pile resistance at this site, pile tip elevations were re-evaluated using LRFD design methods according to the AASHTO 2007 LRFD code. By the time the LRFD analysis was performed, the piles at the North End Bent had been installed to the tip elevations as determined using ASD design with a factor of safety of 3 (for no field testing case). The LRFD analysis showed that the as-installed pile tip elevations at the North End Bent were acceptable for the factored design loads and factored pile resistance values. The design pile tip elevations for the South End Bent (not installed at the time) were re-evaluated using LRFD procedures and the pile lengths increased 18 feet from the original ASD design (FS = 2.25), which was only 3 feet longer than the revised ASD design (FS = 3).

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