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Paper Title: Bonner Bridge Replacement Project - Pile Driving Experience

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The Bonner Bridge Replacement project is a 2.7 mile bridge over the Oregon Inlet on the Outer Banks of North Carolina. The new bridge will replace the existing Bonner Bridge built in 1963. The existing bridge has experienced multiple storm, scour and ship impact events which have resulted in numerous bridge closures and underpinning of some foundation piles. The new bridge foundations consist of three pile types: 36-inch square precast, pre-stressed concrete with a 21-inch diameter void; 54-inch diameter cylinder piles; and 20-inch square precast, pre-stressed concrete. This paper will describe the methods selected for production pile installation and the successes/problems encountered with the planned methods. Results from the static load testing and PDA testing results will also be discussed in detail.

Introduction

The Bonner Bridge Replacement project was awarded by the North Carolina Department of Transportation (NCDOT) as a design build project to PCL Civil Constructors, Inc. and HDR Engineering, Inc. in July 2011. Upon award of the project the design build team preceded with final design and construction planning with construction anticipated to begin sometime in 2012. However, the project construction was delayed due to environmental concerns expressed by the Southern Environmental Law Center. This delayed the start of construction until approximately January, 2016. In the interim the preconstruction load test program was completed which allowed for final design drawings and documents to be completed.

Project Background

The original bridge design likely had little or no analysis of the area scour conditions and the effect such conditions would have on the bridge foundations. As such the current existing bridge has experienced numerous scour events which have reportedly undermined the existing bridge pile foundations. Such undermining has resulted in numerous bents requiring underpinning. Some bents have been underpinned multiple times, as shown in Figure 1. As such, the proposed bridge replacement was required to be designed for extreme scour conditions. The anticipated scour critical elevations along the bridge alignment range from approximately elevation -22 to -85. Such deep anticipated scour elevations resulted in pile foundations to be extended much deeper than would be required for axial pile compression loading and in additional analyses for pile capacity assessment based upon dynamic pile testing and CAPWAP analysis. The additional analysis will be discussed in detail.

Site subsurface conditions generally consist of medium dense sands (SPT blow counts of 30 blows per foot or less) to approximately elevation -85, underlain by dense to very dense sand. Intermittent thin layers of clay soils are also present across the bridge alignment but these are generally minor. The dense to very dense sands below elevation -85 generally have STP blow counts in excess of 30 blows per foot and up to over 100 blows per foot.
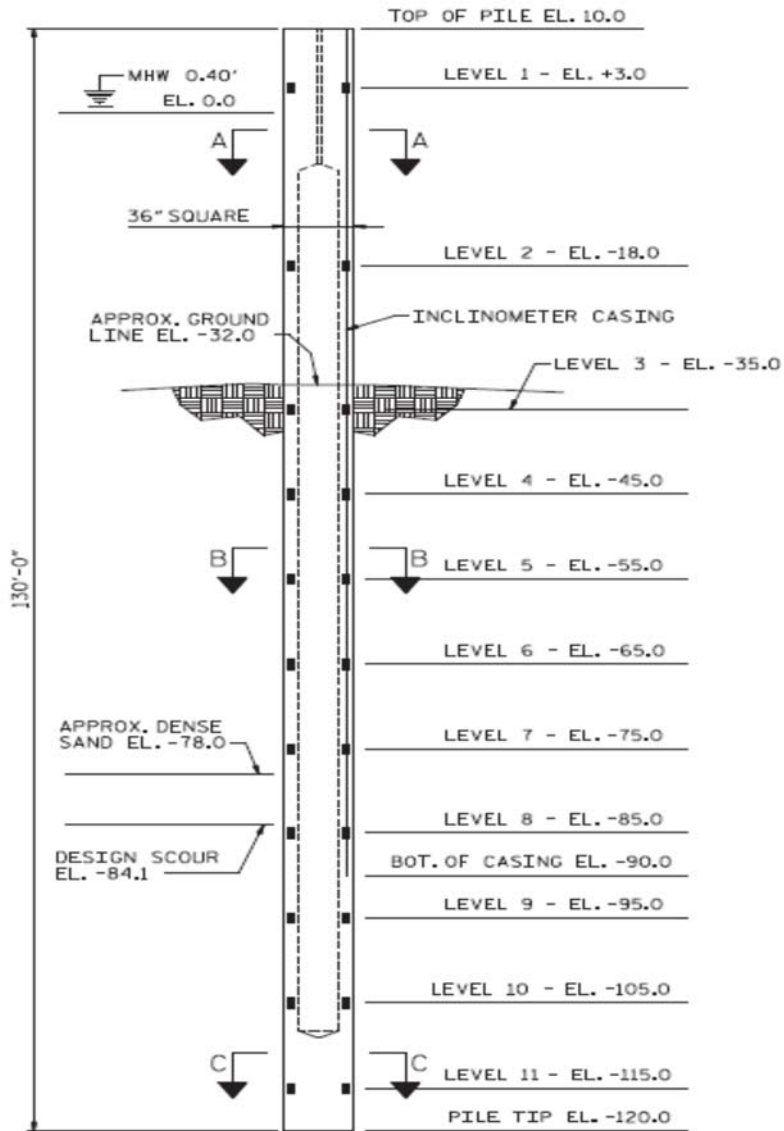
Figure 1. Typical underpinning of original bridge bent.

Preconstruction Load Test Program

As required by the NCDOT request for proposal, a preconstruction load test program was planned in the navigation section of the proposed bridge alignment. This section was considered to have the critical loading conditions for the bridge design. The load test program was planned to consist of driving two 36-inch square prestressed, precast concrete piles and one 54-inch diameter cylinder pile. One of the 36-inch piles was planned to be statically load tested for compression pile capacity and lateral pile capacity. The test piles were planned to be driven to a final tip elevation of approximately EL-120 and therefore were constructed with a total length of 130 ft. The installation of the piles was planned to be accomplished by jetting to a pile toe elevation of approximately EL -110 and then driving to a final pile toe elevation of EL -120.

The first test pile was installed to a final toe elevation of EL-120 by jetting the pile to a toe elevation of approximately EL-110. However, the jetting system was allowed or required to jet below the pile toe to an approximate elevation of EL-111. This apparently resulted in disturbance of the lower soils as indicated in the dynamic testing performed for this pile. The second test pile was also jetted to an approximate pile toe elevation of EL-110.5 but the jetting system was revised such that advancing the pile was achieved without the need to advance the jetting system below the pile toe. This pile was subsequently driven to a final pile toe elevation of EL-120 using an APE D 225-42 single acting diesel hammer. The blow count at final driving was approximately 62 blows per foot, with a hammer stroke of approximately 7.5 ft. CAPWAP analysis indicated an end of drive pile capacity of approximately 2100 to 2300 kips.

Static load testing of the test pile was performed approximately three weeks after initial driving was completed. It should be noted that the test pile was instrumented with vibrating wire strain gages at 11 locations along the length of the pile as shown in Figure 2. At each of the 11 elevations, four vibrating wire strain gages were attached to the pile reinforcing cage as shown in the figure and a picture of the gage attachment is shown in Figure 3.

TOP OF PILE EL. 10.0

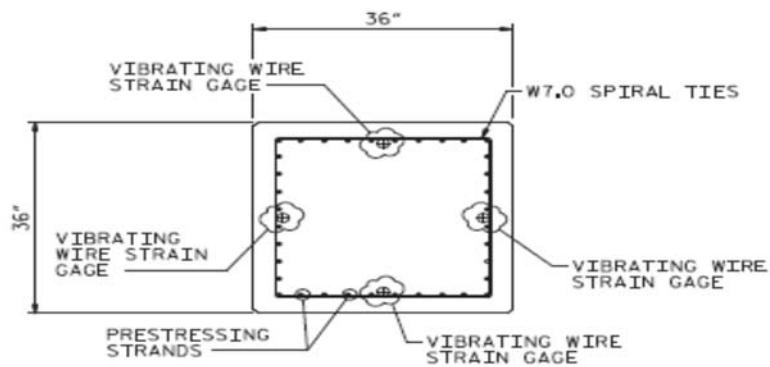MHW 0.40'
EL. 0.0

LEVEL 1 – EL. +3.0

A

A

36" SQUARE

LEVEL 2 – EL. –18.0

APPROX. GROUND
LINE EL. –32.0

INCLINOMETER CASING

LEVEL 3 – EL. –35.0

LEVEL 4 – EL. –45.0

130'-0"

B

B

LEVEL 5 – EL. –55.0

LEVEL 6 – EL. –65.0

APPROX. DENSE
SAND EL. –78.0

LEVEL 7 – EL. –75.0

LEVEL 8 – EL. –85.0

BOT. OF CASING EL. –90.0

DESIGN SCOUR
EL. –84.1

LEVEL 9 – EL. –95.0

LEVEL 10 – EL. –105.0

C

C

LEVEL 11 – EL. –115.0

PILE TIP EL. –120.0

ELEVATION OF TEST PILE

36"

VIBRATING WIRE
STRAIN GAGE

W7.0 SPIRAL TIES

36"

VIBRATING
WIRE STRAIN
GAGE

VIBRATING WIRE
STRAIN GAGE

PRESTRESSING
STRANDS

VIBRATING WIRE
STRAIN GAGE
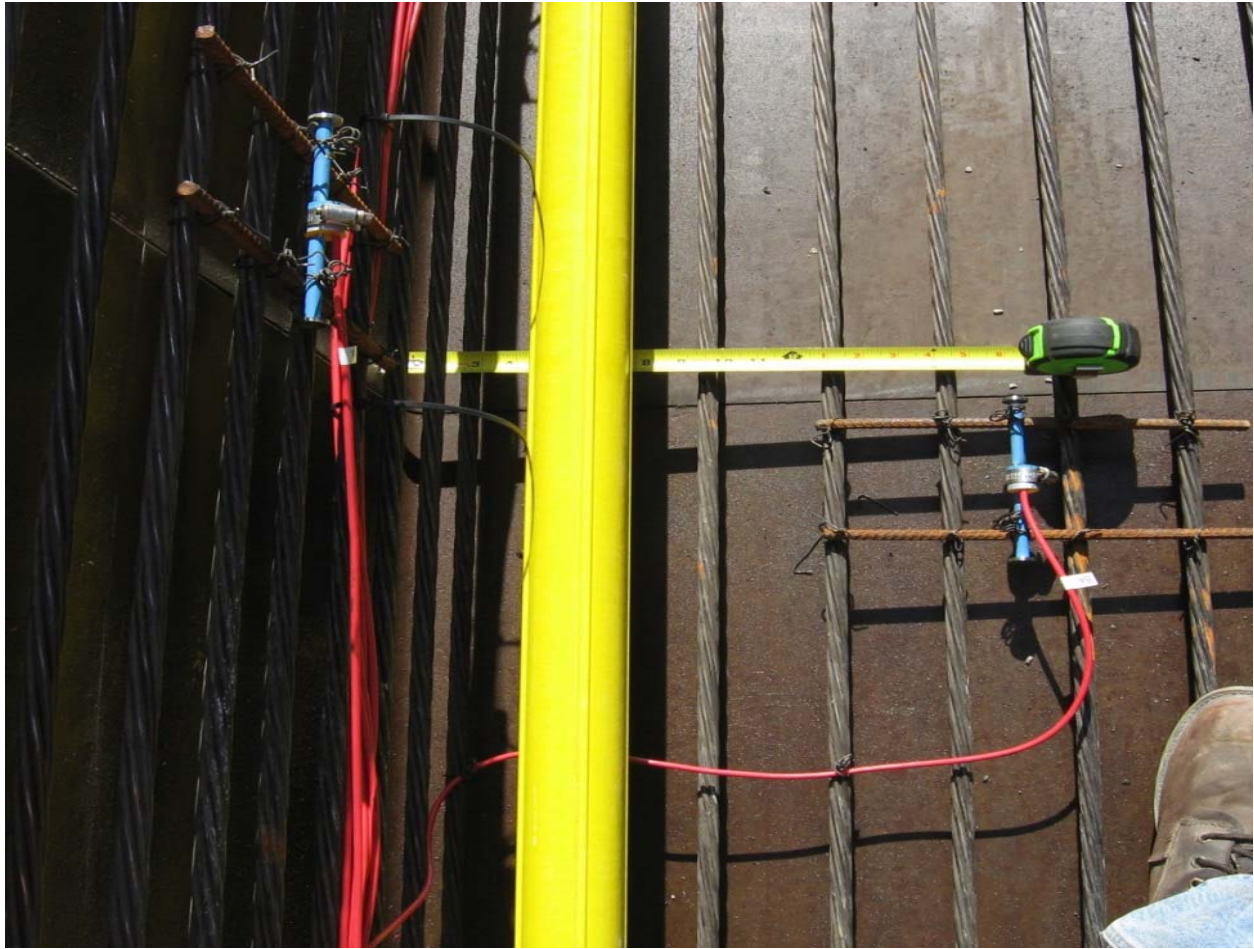
Figure 2. Static Load test pile instrumentation.

Figure 3. Vibrating wire strain gage and inclinometer tubing attachment.

Static load testing was performed on the static load test pile with the maximum loading achieved of approximately 1100 tons (2200 kips). However, the reaction system was not able to provide a greater resistance and the failure load of the static load test pile was not achieved. Based upon the load deflection curve it was estimated that a failure load of about 1400 tons (2800 kips) might be appropriate as shown in Figure 4.

Results from the vibrating wire strain gages are provided in Figure 5, and indicate the apparent soil resistance with respect to depth. These results show very poor agreement between the vibrating wire average load at the first location (only seven feet below the pile top) and the load cell. This apparent error was due to the fact that the load test pile was not completely vertical when driven which appears to have resulted in an eccentric loading of the test pile. Results for the vibrating wire strain gages along each pile face are provided in Figures 6a, 6b, 6c and 6d and show that loading of the pile was not axial during the static load test. It is apparent that the South and East faces of the pile were placed in compression while the North and East faces of the pile were placed in tension during the load testing. This loading was further confirmed by lateral movement of the pile during the load test which was monitored by dial gages. The apparent lateral movement was approximately 1 inch at the maximum applied load.

Bonner Bridge Load Test Program
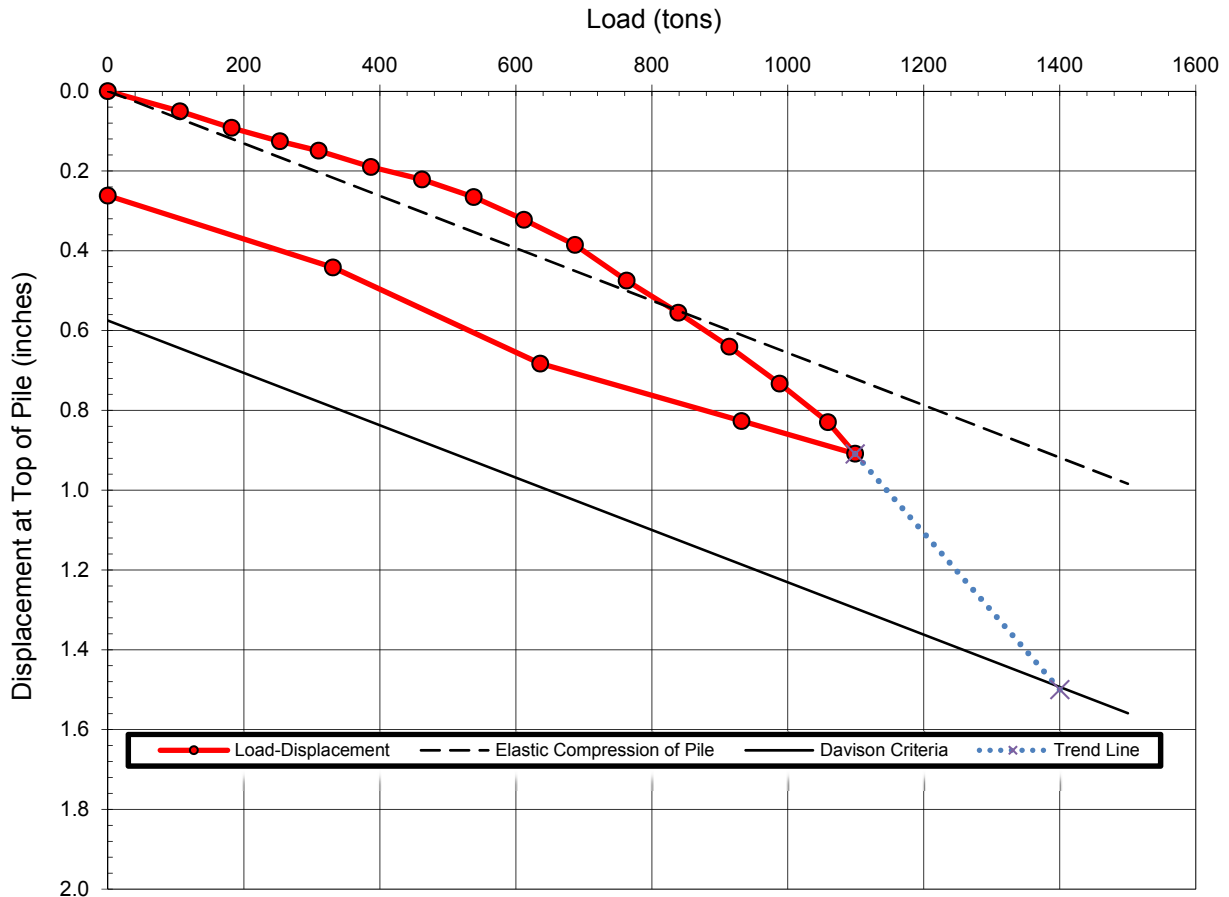## Load - Displacement - SLT 2-28-13

Figure 4. Static load testing results.

Static load test results were not ideal for this project. However, this was mostly due to the rather difficult working conditions present at this job site. Working in approximately 40 ft of water with daily tidal activity and very strong currents made installation of the test pile difficult. In addition, jetting of the 36 inch square concrete pile through the dense to very dense sands resulted in less than ideal pile driving conditions. All of these factors contributed to the pile not being installed in a vertical orientation which made load testing difficult. Interpretation of the static load test results were also made more difficult due to these conditions.

Although these conditions limited the results of the static load testing, it was clear that the piles would be able to achieve the desired driving resistances which ranged from 1000 to 2000 kips. In addition, the load deflection curve indicated that the majority of this resistance would be distributed as end bearing resistance as the curve starts to move toward the elastic compression line at approximately 600 ton loading. This would indicate that approximately 600 tons (1200 kips) of the soil resistance was distributed as skin friction. This soil resistance distribution is also indicated from the load transfer data presented in Figure 5. Therefore, the soil resistance distribution was expected to be approximately 40% skin friction and 60% end bearing.

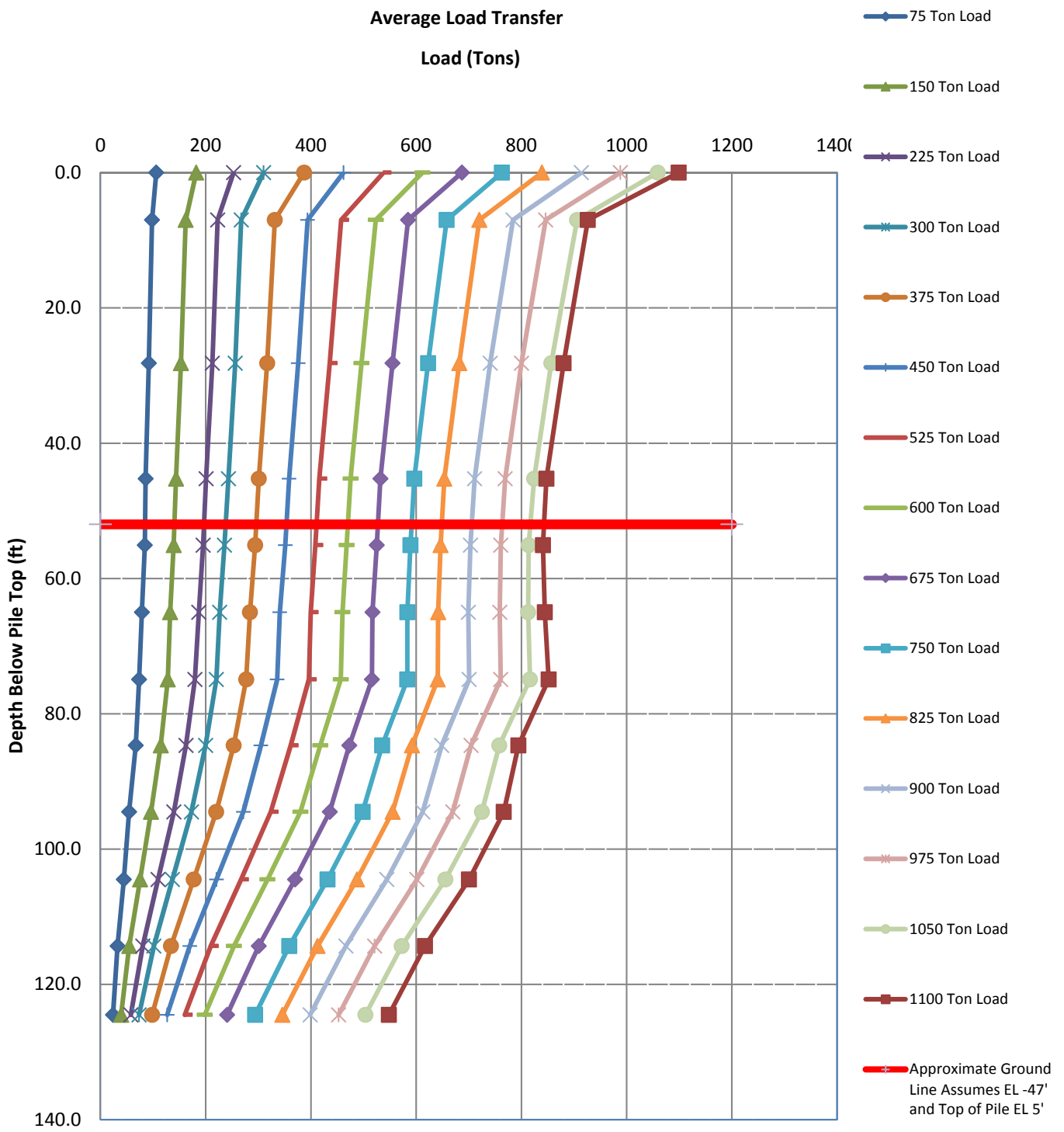## Average Load Transfer

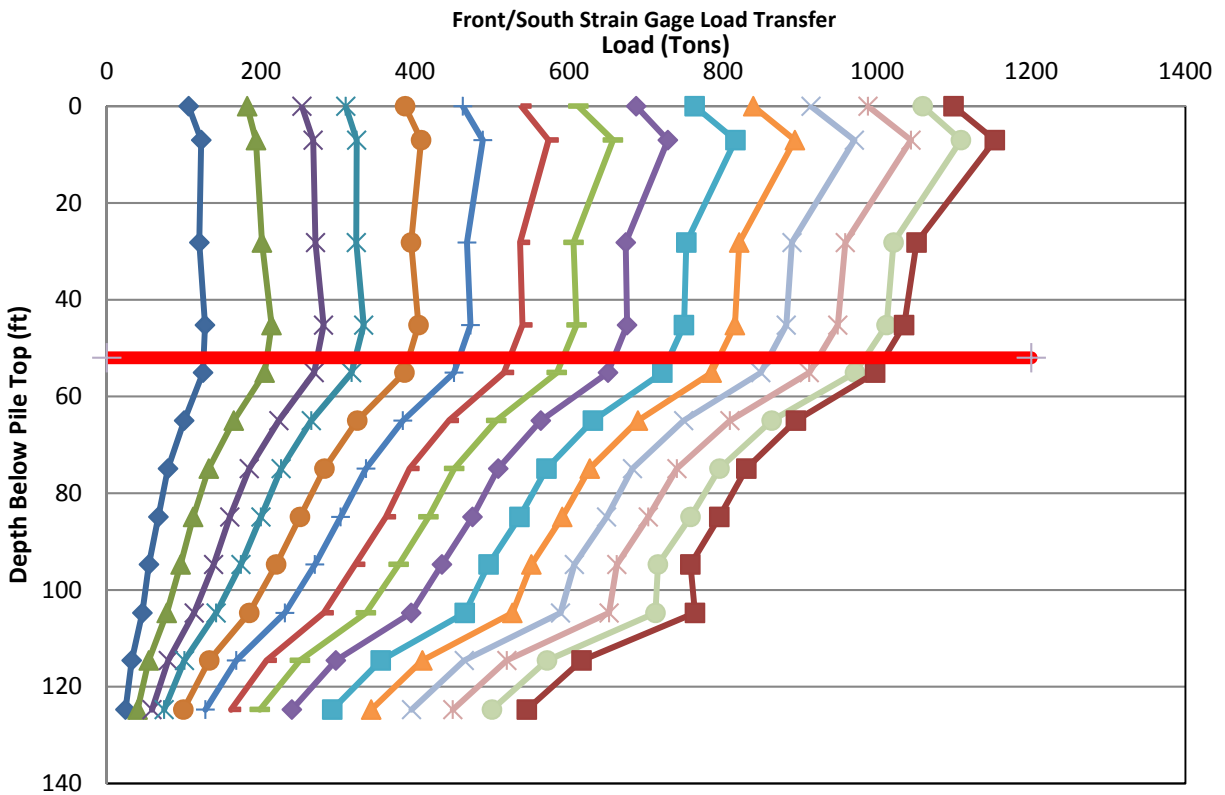### Load (Tons)

Figure 5. Average Load Transfer data.
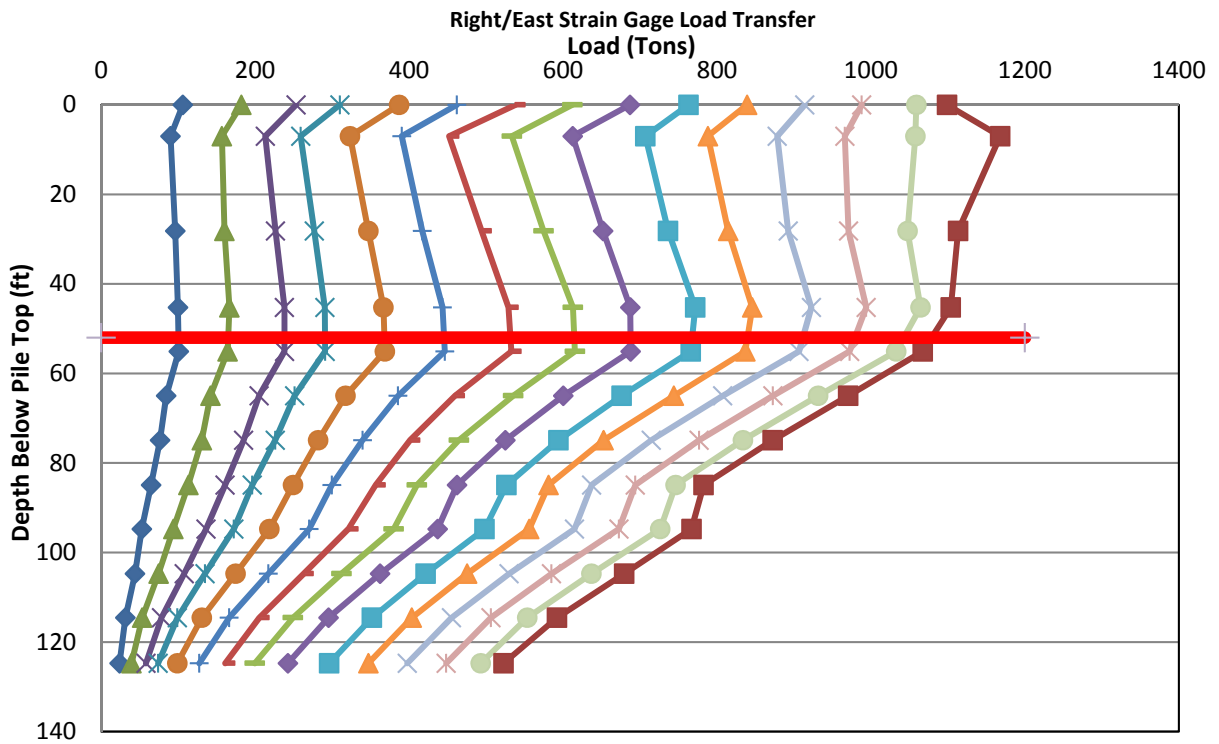
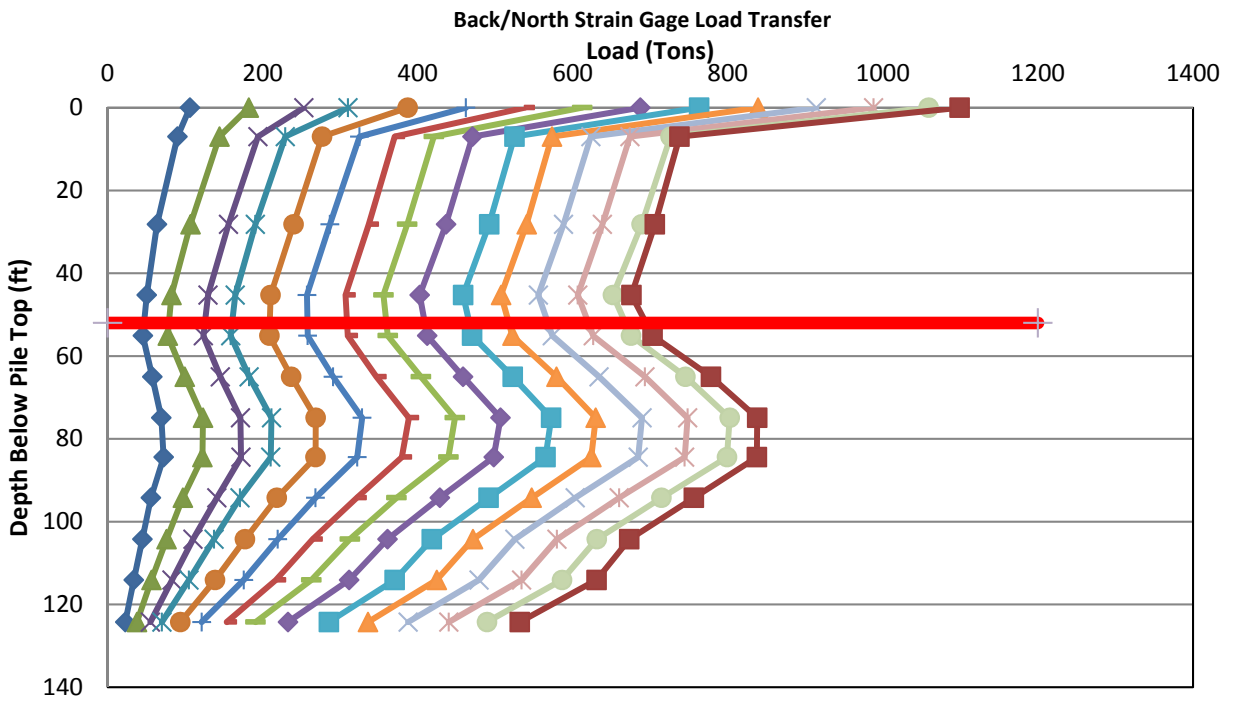Figure 6a. Load Transfer data.

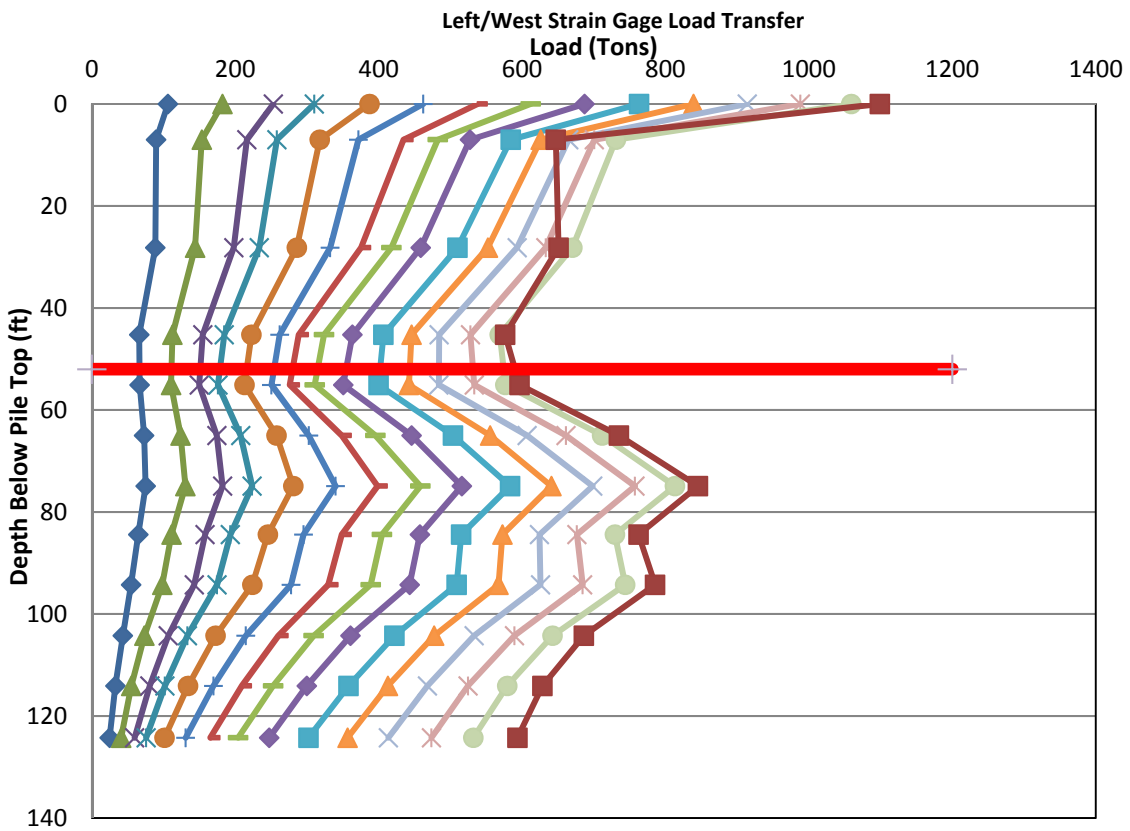Figure 6b. Load transfer data.

Figure 6c. Load transfer data.

Figure 6d. Load transfer data.

Production Pile Installation, Testing and Assessment

A major concern for the project was the installation of the 36-inch square piles in the navigation zone, through the upper dense sand deposits and to the required minimum tip elevations. While jetting of the piles was the quickest and easiest method, it was unclear to the project team how the jetting process would work for these piles, as all piles were required to be driven on a 2:12 batter. Research of other projects where jetting of large diameter concrete piles on a batter indicated that this type of installation, while common for vertical piles, was typically not done for piles driven on batters. As such, the project team developed an installation plan that consisted of the installation of a template that would accommodate steel pipe sections. The steel pipe sections would be vibrated to a penetration of about 50 or 60 feet and would provide a pile guide for jetting and driving the piles. A schematic of the planned installation setup is shown in Figure 7 and a picture of the steel pipe jetting and driving guide is shown in Figure 8.
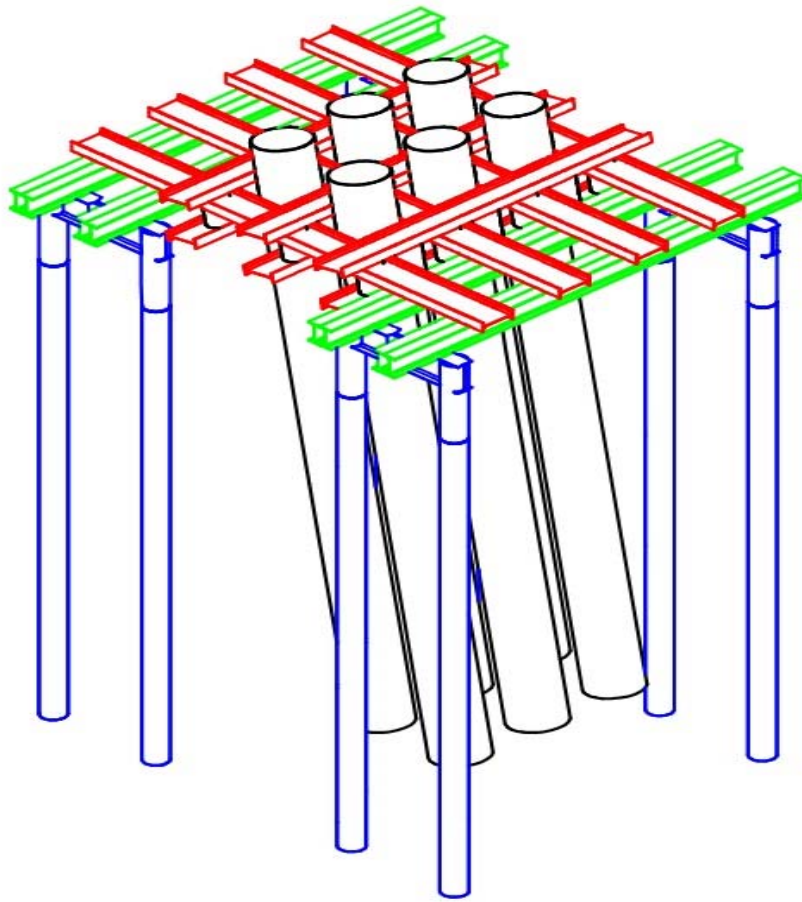

Figure 7. Pile installation template/guide.

Figure 8. Steel pipe jetting and pile driving guide.

Jetting was provided through steel pipe and was controlled by tubular channels welded to the steel pipe sides. This allowed for more accurate guiding of the jetting system. The jetting was performed by two 3-stage jet pumps connected in manifold, with each pump capable of providing 1000 gallons per minute. In addition, compressed air was used to assist the jetting system. A single 1600 cfm air compressor was used in combination with the jet pumps. This system worked very well for pile installation to approximately 10 to 20 feet above the estimated pile tip elevations. In fact, the system has worked with only minor issues and adjustments, and jetting of the piles to any tip elevation desired was easily accomplished. The most difficult adjustment has been to try to predict the ideal elevations to jet the piles to in order to minimize the driving time while not having any detrimental effects on the soil bearing resistance.

Extensive dynamic pile testing was required to evaluate the pile capacities after installation. The primary concern for this testing was how to evaluate the pile capacity for the extreme scour event. As noted earlier, a large proportion of the bridge foundations would need to be suitable to withstand an anticipated scour event to approximately EL-85. This would require as much as 80 feet of soil resistance to be removed. This removal of both soil resistance and effective overburden needed to be assessed by Pile Driving Analyzer (PDA) testing. As such, the design team developed a method to provide just such an assessment.

PDA results needed to exclude all resistance in the upper scour prone soils, and it was deemed appropriate to reduce the underlying skin friction resistance by the reduced effective overburden. It was proposed by the design team that the method to calculate such reductions would be as follows:
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1. Calculate average skin friction from CAPWAP analysis over the pile penetration from the design scour elevation (DSE) to the pile tip.
2. Calculate the effective overburden at the time of PDA testing for the portion of the pile from DSE to pile toe elevation.
3. Calculate the β value for this penetration based upon the CAPWAP skin friction and the effective overburden at the time of pile installation.
4. Calculate the reduced overburden stress which would result due to the design scour elevation and

the subgrade elevation at the time of pile driving for the soil layers between the DSE and pile toe elevation. The reduced overburden is calculated based upon a scour hole with 2H:1V side slopes.

5. Multiply the reduced effective overburden by the β factor calculated above to obtain a reduced unit skin friction long term for the design scour condition.

6. Calculate the long term skin friction as the unit skin friction long term times the pile perimeter and pile embedment below the DSE to the pile toe elevation.

7. Subtract the calculated long term skin friction the total skin friction obtained from CAPWAP analysis skin friction. This resistance is the Unfactored Scour Resistance (USR) to be used in the required driving resistance (R_{ndr}) equation below.

8. Calculate the Required Driving Resistance, R_{ndr} as indicated below:

R_{ndr} = (Factored Axial Resistance + Factored Dead Load) / Phi factor + Unfactored Scour Resistance

The total measured resistance (skin + toe) from the CAPWAP analysis must equal or exceed the calculated R_{ndr} value. The final driving criteria (ie – blow count and hammer stroke) may include an expected soil setup which may need to be confirmed based upon restrike testing of some or all of the production piles.

Pile driving for this project is currently ongoing and approximately one half of the pile bents have been installed. In general, the above method of pile assessment has been very successful with CAPWAP analyses being used to calculate the R_{ndr} values for each pile bent and driving criteria or pile assessment being provided for those piles not tested using dynamic pile testing. PDA and CAPWAP analyses have generally indicated that pile capacities on the order of 2000 to 2500 kips have been achieved and R_{ndr} criteria based upon the design scour elevation have been calculated for all piles of the bridge bent. The driving criterion has been based upon both the R_{ndr} values and refined wave equation analysis which is based upon the PDA testing results.

Conclusions

This project has demonstrated that the proposed pile installation methods used for installing the 36-inch square, pre-stressed concrete piles has been very successful with little or no detrimental effects for the overall pile performance. Specifically, jetting of the 36-inch piles through the pipe templates on a 2:12 batter has allowed installation to the planned pile tip elevations. This is considered significant due to the dense sand layers which required penetration and the fact that jetting of the battered piles did not result in reduced pile capacities for surrounding piles. In general, jetting of the piles to a tip elevation approximately 10 ft above the final required tip elevation has worked well and allowed for driving of the piles to the desired penetration and pile capacity with the least amount of effort.

The proposed analysis for evaluation of the pile capacity while considering the design scour elevation has also been quite successful. CAPWAP results along with refined wave equation analyses have been provided for each bent location installed to date and this data along with the above described evaluation method has been used to accept/approve the pile installations.