Saving a Bridge Foundation

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ABSTRACT: Many bridge foundations are not reused. For a variety of reasons it is easier to replace rather than reuse the bridge foundations. This paper discusses methodology for evaluating the existing pile foundations. Pile resistances were estimated using the wave equation and compared to the factored pile loads determined from Group Analyses. Geofoam lightweight backfill was used to allow reusing the piles. Some piles were exposed, investigated and found to be in pristine condition.

INTRODUCTION

Many existing bridge pile foundations cannot be saved or re-used. Incomplete historical data, difficult foundation accessibility, limitations in non-destructive testing (NDT) technology, and increased modern highway loads create conditions that make it easier, but not necessarily more cost effective, to replace the bridge and the foundations completely.

A project was recently completed in Massachusetts where the existing timber pile foundations were re-used beneath a two span bridge that was replaced. The driving logs, soil information, and hammer information, were obtained from the Mass DOT archives. The pile resistances were back-estimated by modeling the pile driving log data, hammer information, and subsurface soil conditions into the wave equation. Group analysis of the abutment and pier piles were performed to determine the new individual pile loads. The pile resistances were then compared to the factored pile loads. The pier piles could be re-used as-is. However, in the abutments, the loads slightly exceeded the resistances. Geofoam lightweight backfill was used as a solution to reduce the loads and allow re-use of those piles as well.

After the engineering study proved that the piles could be successfully re-used, the piles were physically evaluated to confirm integrity. Four test pit excavations were completed and the piles were exposed, visually observed, cored, and NDT was performed to verify lengths. The piles were found to be in pristine condition and the tested lengths matched those recorded in the logs.

PROJECT HISTORY AND INFORMATION

The bridge was constructed in 1951 and consisted of a two span structure founded on timber piles. The distance between the abutments is around 130 feet and the clearance of the bridge over the road is around 18 feet. The timber piles were composed of oak with 12-inch butt diameter and 8 inch tip diameter. Figure 1 provides a plan and elevation view of the bridge and Figure 2 portrays a photo of the bridge.

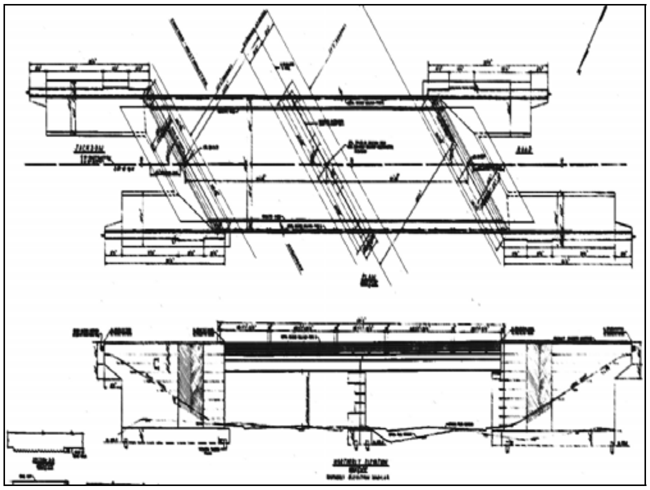


Figure 1. Bridge Plan and Elevation

Figure 2. Bridge Photo

Bridge inspections by Mass DOT revealed that the bridge superstructure was in poor condition and needed to be replaced. The condition of the timber pile foundations was unknown. Mass DOT contracted Lin Associates to determine whether the bridge should be completely replaced or rehabilitated (remove superstructure and retain pier and abutment footings and piles).

GTR was retained by Lin Associates to evaluate the re-use of the timber piles. The project team felt that if the piles could be re-used under the new loading conditions, the existing borings were sufficient and new borings would not be needed.

The subsurface conditions at the bridge based on borings taken in 1949, generally consist of around 10 to 15 feet of sand-gravel fill. Around 5 to 10 feet of loose to medium dense, fine yellow sand was encountered below the fill. The N-values ranged between 5 and 15 blows per foot (bpf) in the fill/fine sand. Sand and gravel (up to 10 feet thick) underlies the fine yellow sand and was encountered around 20 to 25 feet below grade (or 15 to 20 feet below the bottom of the substructures). The N-values ranged between 10 and 20 bpf. The timber piles were driven into this layer. Clayey Sand with varying amounts of gravel underlies the sand and gravel and becomes denser with depth. This layer extends to refusal at elevations varying between +100 feet to +130 feet (100 to 130 feet below grade). Groundwater was identified in the borings around 20 feet below grade. The simplified subsurface profile is presented in Figure 3.

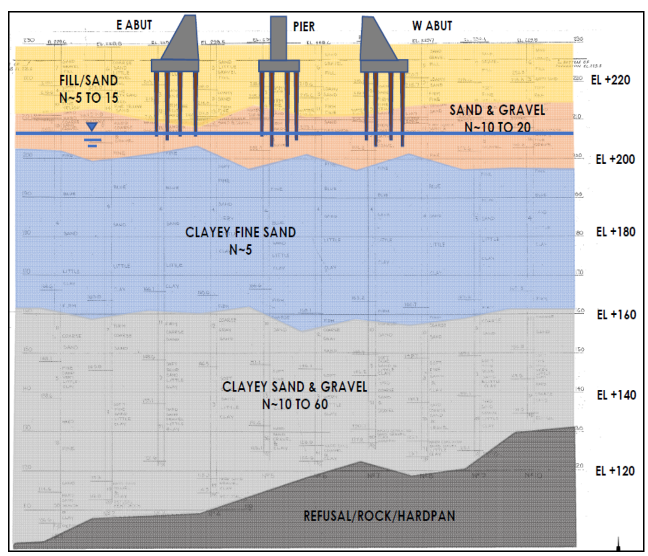


Figure 3. Soil Profile

The cost and schedule advantages of re-using the piles becomes more pronounced when considering that the new piles most likely would have to be driven significantly deeper to bedrock. In order to save the timber pile foundations the records of the as-built foundation geometry, driving logs, and any load test results would be needed.

Fortunately, the Mass DOT archives contained the records of the as-built abutment and pier information and the pile driving logs. Load test information could not be located, however, the construction records mentioned that load tests were performed on a sacrificial pile in the pier and on production piles in the abutments. The original pile design load was 30 kips, which was increased to 36 kips after the load testing program.

Figure 4 presents information from the pile driving logs in the west abutment, where the pile penetration length and blow count over the last foot of driving were recorded for each pile.

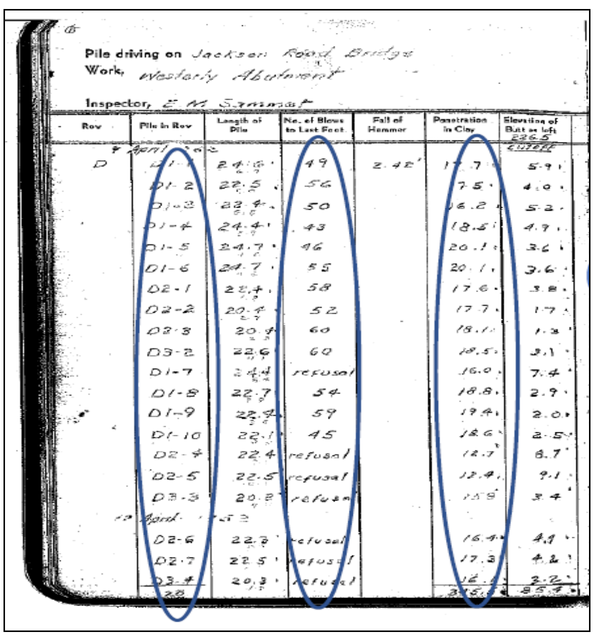


Figure 4. Pile Driving Logs

The piles were driven to penetrations between 10 and 25 feet below foundation grade and blow counts ranging between 32 blows per foot (bpf) and refusal (assumed to be 120 bpf). A Vulcan 02 hammer (rated energy of 7.3 kip-ft) was used to drive the production piles. The information on the hammer type is one of the most critical components of performing this study, besides knowing the soil conditions, pile length and blow count.

The as-built plan of the easterly abutment is shown in Figure 5. This provided the necessary information on the pile, abutment, and pier geometry. Batter piles (1H:12V or 1H:6V) were used for resisting horizontal loads.

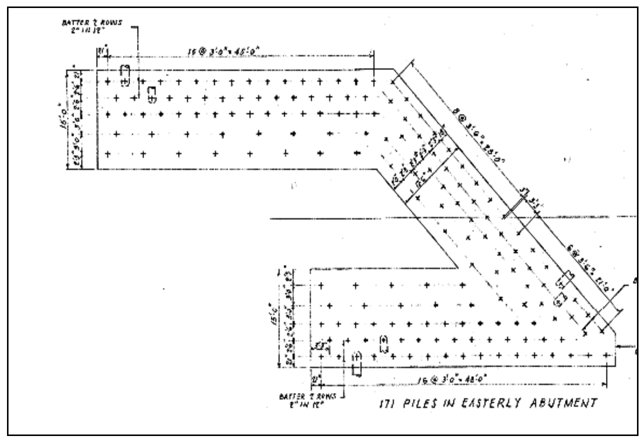


Figure 5. As-built Foundation Plan

DESIGN APPROACH

In order to re-use the pile foundations, the pile capacity (or nominal resistance) had to be determined. The general approach for estimating the nominal pile resistance was performed as follows:

1. Develop rough soil profile for each substructure.
2. Determine range of pile resistances for various subsurface conditions and pile penetration lengths. Several static capacity methods in the AASHTO 2012 code were

used to develop a range of resistances and soil distributions (end bearing vs friction).

3. Perform wave equation analyses to develop blow count vs resistance relationships using the results of the analyses in Step 2. Several cases were performed based on penetration length, percent end bearing, and hammer efficiency (vertical vs batter pile) variations. For the range of blow counts recorded, the resulting wave equation back-calculated resistances ranged from 65 to 125 kips. Figure 6 shows the relationship between blow count and capacity (nominal resistance) for one case.

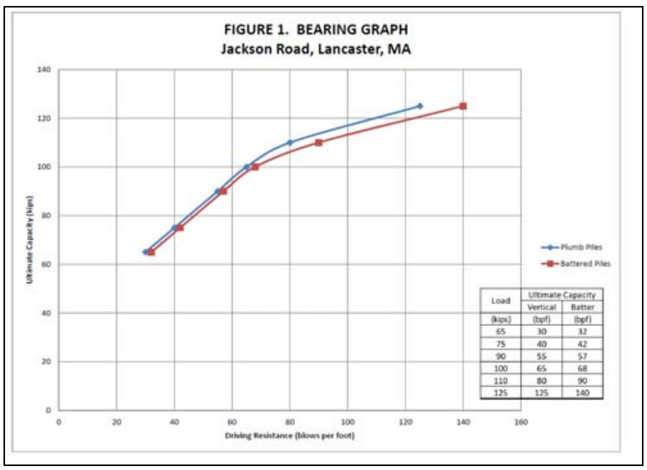
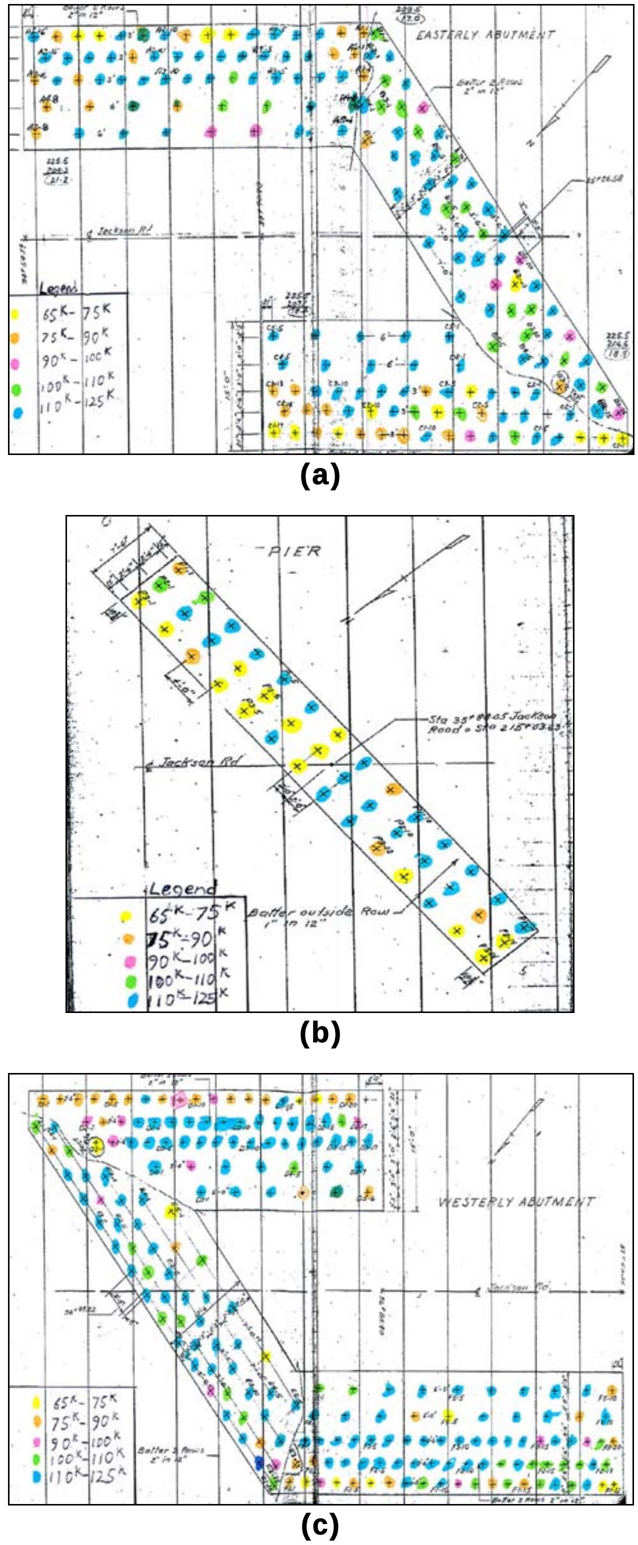


Figure 6. Wave Equation Bearing Graph

4. The piles in each substructure were then assigned a capacity range based on their blow count. Figures 7a, 7b, and 7c show the range of nominal resistances associated with each pile. The lowest blow count piles had resistances ranging between 65 and 75 kips (denoted by the yellow piles) while the piles driven to refusal had the highest capacity range of 110 to 125 kips (denoted by the blue piles).

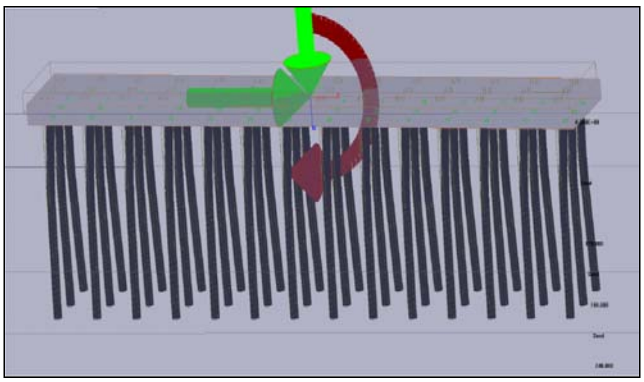
These nominal resistances are representative of conditions at the end of driving. The pile tips were driven into a hard coarse sand and gravel through fill and fine sand. Long term pile capacity could be higher or lower (i.e. setup or relaxation) than the end of driving capacity. In our opinion, for this project, the long term capacity is expected to be about



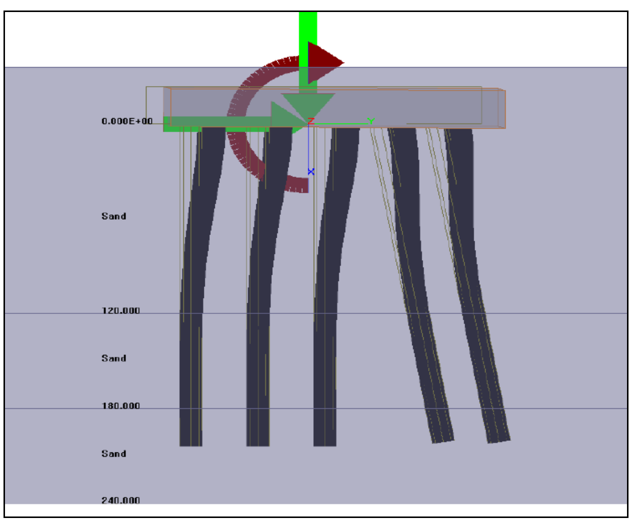
Figures 7a, 7b, 7c. Wave Equation Nominal Pile Resistances at Pier and Abutments

the same as the capacity at the end of driving, since the borings do not indicate a soil that has been known to relax at the pile tip (hard coarse sand/gravel). In addition, the piles were not driven though a soil that contains a lot of silt and clay suggesting minimal setup with time.

5. Group Analyses were performed on each substructure with the as-built pile geometry, soil conditions, and new loads. The controlling load cases provide the load or force on each pile. This load is the factored pile load. Figures 8a and 8b illustrate the pile groups analyzed for the pier and one of the abutments.



(a)



(b)

Figures 8a and 8b. Group Analysis of Pier and Abutment

RESULTS AND SOLUTION

The factored axial loads from step 5 were compared to the range of nominal resistances from step 4. A resistance factor of 0.5 was used based on wave equation analysis to determine the factored pile resistance. The summary of the results of the factored pile forces (loads) and capacities (resistances) are presented in Table 1.

	Total force	Factored Pile Capacity (kips)		
Pile #	(kips)	Min.	Max.	Result
1	17.2	38	45	OK
2	20.0	50	55	OK
3	22.8	55	63	OK
4	25.6	55	63	OK
5	28.4	55	63	OK
6	31.3	55	63	OK
7	34.1	33	38	OK
8	36.9	55	63	OK
9	39.7	38	45	OK
10	42.5	55	63	OK
11	45.3	55	63	OK
12	48.1	55	63	OK
13	51.0	55	63	OK
14	53.8	55	63	OK
15	17.2	50	55	OK
16	20.0	55	63	OK
17	22.8	55	63	OK
18	25.6	33	38	OK
19	28.4	33	38	OK
20	31.3	33	38	OK
21	34.1	33	38	OK
22	36.9	55	63	OK
23	39.7	55	63	OK
24	42.5	55	63	OK
25	45.3	55	63	OK
26	48.1	55	63	OK
27	51.0	38	45	CHECK
28	53.8	33	38	CHECK
29	17.2	33	38	OK
30	20.0	33	38	OK
31	22.8	38	45	OK
32	25.6	33	38	OK
33	28.4	33	38	OK
34	31.3	55	63	OK
35	34.1	33	38	OK
36	36.9	55	63	OK
37	39.7	55	63	OK
38	42.5	38	45	CHECK
39	45.3	33	38	CHECK
40	48.1	55	63	OK
41	51.0	33	38	CHECK
42	53.8	33	38	CHECK
	1490	1898	2173	

Table 1. Pier Load vs Resistance Results

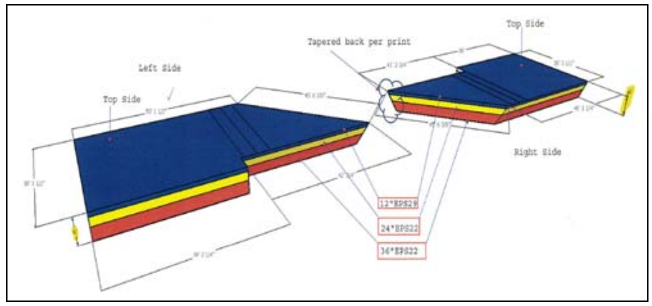
Figures 7a through 7c present a color coded plan of the piles in the two abutments and the pier. The color code represents the distribution of the pile capacity (nominal resistance) for the piles based on the blow count. As some piles may have resistances lower than required, while other piles may have been overdriven, it may be possible to take averages in each substructure over all or for a group of piles. This may depend on the relative rigidity of the pile cap and its ability to redistribute loads for piles in an area or a group. As shown in Table 1, the minimum factored resistance of 1898 kips for the pier is greater than the total factored axial load of 1490 kips.

The pier factored resistances exceeded the factored axial loads for most piles and overall for the total group. Some of the abutment piles were overloaded considering the new load combinations and dead weight of the backfill. In order to reduce the dead loads on the abutments, Geofoam lightweight fill was used. Figures 9a through 9c show the Geofoam fill design and on-site placement.

ADDITIONAL CONSIDERATIONS

The factored geotechnical resistance is determined by multiplying the capacity (nominal resistance) by a resistance factor. Since these piles are existing piles, there is minimal guidance for selecting a resistance factor. The piles were load tested at the time they were installed (resistance factor of 0.75 based on 2012 AASHTO). However, the load test results were unavailable and not totally applicable to the current requirements. Table 2 provides the 2012 AASHTO LRFD resistance factors. For this project a resistance factor of 0.5 was selected based on the wave equation method to confirm pile capacity. However, there is some flexibility on the selected resistance factor, and for this case, a value somewhere between 0.5 and 0.6 may have been more appropriate.

More research into this area is needed to further explore and establish guidance on the application and selection of resistance factors for existing piles.



(a)

(b)

(c)

Figure 9a, 9b, and 9c Geofoam for Abutments

AASHTO LRFD Bridge Design Specifications (2012)
Table 10.5.5.2.3-1– Resistance Factors for Driven Piles - Bearing Resistance.

Condition/Resistance Determination	Resistance Factor
1 Successful SLT on at least one pile per site condition & PDA testing on at least 2 piles per site condition (min 2% total)	0.80
1 Successful SLT on at least one pile per site condition	0.75
PDA Testing on 100% of Production Piles	0.75
PDA Testing with CAPWAP @ BOR	0.65
WEAP (No load testing, EOD Only)	0.50
Static Analysis Methods	0.25 to 0.5
FHWA Modified Gates Dynamic Formula (EOD Only)	0.40
ENR Dynamic Formula (EOD only)	0.10

Table 2. LRFD Resistance Factors

VERIFYING PILE INTEGRITY

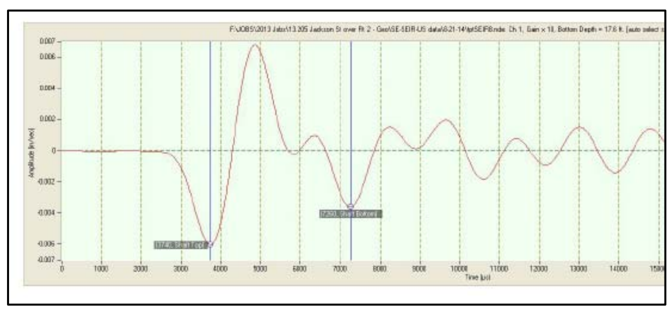
Once the piles were evaluated and determined to be acceptable for re-use, a program was established to verify the condition and integrity of the piles. A test pit was performed at each corner of each abutment, for a total of four test pits. One timber pile was exposed and evaluated in each test pit. Exposing the pier piles was deemed too difficult as the pier was in the middle of the highway and inaccessible. The conditions of the pier piles was expected to be similar to the abutment piles as the bottom of pier was the same elevation and in similar near subsurface conditions.

The soil around the piles and under the abutment caps was composed primarily of fine to medium sand with trace silt. Groundwater was not present at the time of the test pits. The timber piles were evaluated in the following ways:

1) Visual observation,
2) Core testing and evaluation,
3) Resistograph testing, and
4) Sonic Echo Method integrity testing to confirm length.

Overall, the exposed timber piles in the test pits appeared to be in pristine condition, based on visual observation, core samples, resistograph testing, and non-destructive integrity testing. Figures 10a through 10c provide a typical photo of the timber piles, sonic echo integrity testing results, and resistograph testing results, respectively.

(a)



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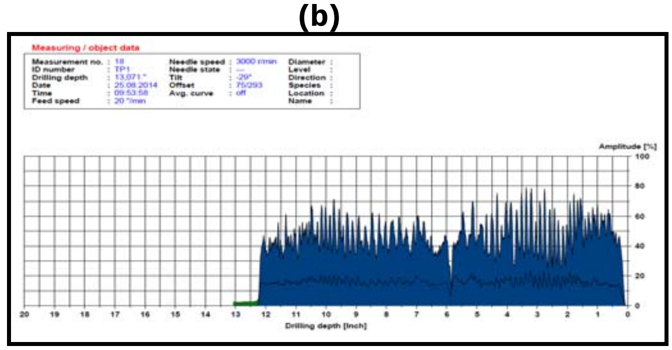

(c)

Figure 10a, 10b, 10c. Pile Investigation and NDT

CONCLUSIONS

The methodology discussed above enabled the re-use of the timber piles and cap substructures for the new bridge superstructure. Some important conclusions drawn from the project include;

1. Availability of the driving records and as-built foundation geometry are critical and required for the proposed approach of using the wave equation to back-calculate the resistances and Group to model the new loads under as-built conditions.
2. More research and guidance on the selection of an appropriate resistance factor for existing foundations is needed. For the proposed approach, a value of 0.5 was used. However, it seems feasible to use a value somewhere between 0.5 and 0.6 considering the methods and data available for this project. The resistance factor selected should consider the number and type of past tests as well as the future planned tests.
3. Access to the piers for physical and/or NDT investigation allowed confirmation to accept the piles and should be performed to have confidence in the future performance.

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