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GEOTECHNICAL OPPORTUNITIES ON A FAST-TRACK BRIDGE PROJECT

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ABSTRACT

The bridge for Nine Mile Road over Interstate 75 in Hazel Park, Michigan was destroyed by a tanker fire. The loss of the bridge was considered an emergency situation. Therefore, the bridge replacement was put on a fast-track schedule.

Geotechnical engineering challenges included the design of shallow and deep foundations, design of light-weight backfill behind abutments, design of temporary earth retention systems to minimize traffic disruption during construction, and coordinating design changes during construction based on variable subsurface conditions. The design was based on the Bridge Design Specifications from the American Association of State Highway and Transportation Officials (AASHTO) Load and Resistance Factor Design (LRFD).

Since the project involved replacement of a former bridge, the LRFD design could be compared with the previous foundations that were designed decades earlier. Thus, a summary was developed that identifies how the foundation types and sizes using LRFD methods changed, or remained unchanged, relative to the former bridge design using the Allowable Stress Design (ASD) method.

The Michigan Department of Transportation (MDOT) elected to conduct the bridge replacement using the design-build approach. Total time to complete the design and construction of the new bridge: 65 calendar days.

INTRODUCTION

On July 15, 2009, a fuel tanker burst into flames after losing control while traveling southbound on Interstate 75 in Hazel Park, Michigan. The accident occurred under the Nine Mile Road bridge over I-75. Fortunately, no lives were lost but the damage to the bridge was extensive. After extinguishing the fire and assessing the damage, it was determined that the existing bridge was beyond repair and a new bridge would need to be constructed.

Geotechnical engineering played a key role in the foundation design, abutment backfill, temporary earth retention design, and in assessing soil and foundation capacities based on field conditions encountered during construction. In addition, the new bridge was designed using the latest design methodology, Load Factor and Resistance Design (LRFD). This required adapting conventional geotechnical engineering practice to support the new design method.

The use of light-weight backfill and Pile Driving Analyzer (PDA) testing were implemented to address the project's geotechnical challenges. The light-weight fill helped to expedite the backfilling process behind the abutments, and keep lateral soil pressure low so the abutments could be supported on shallow foundations. The PDA testing helped realize the maximum in-place capacity of the piles for a future center pier, while installing the piles under a tight schedule.

Fig. 1 Tanker Fire Under Bridge

The bridge project was awarded to a design-build team on September 30, 2009, and design commenced immediately on the award date Construction started on the week of October 12, 2009. The bridge was opened for traffic on December 11, 2009. The total time to complete the bridge design and construction was 65 calendar days.

This paper describes the components of the bridge design that were the responsibility of the geotechnical engineer. Also, it describes how the subsurface analyses were performed to develop recommendations for shallow and deep foundation design, and temporary earth retention design. The final foundation design, based on the LRFD design methodology, was compared with some of the former bridge foundations which were designed using the conventional Allowable Stress Design (ASD) methodology. In summary, the LRFD design produced a foundation system that was larger than the foundations designed using the ASD method.

SHALLOW FOUNDATIONS

The new bridge foundation design consisted of a shallow spread foundation system, similar to the foundation system that supported the former bridge. Foundations for the former bridge were removed, and the new foundations were designed using the Bridge Design Specifications from the American Association of State Highway and Transportation Officials Load Factor and Resistance Design.

New shallow foundations were required for the two abutments (abutment A – west side, and abutment B – east side) and one row of center piers (pier 1A). Proposed bottom of footing elevation would be about 10 feet below the ground surface of the highway under the bridge. Subsurface conditions were obtained from three soil borings performed at highway level along the alignment of the new bridge. Figure 2 provides a general soil and groundwater profile at the boring locations.

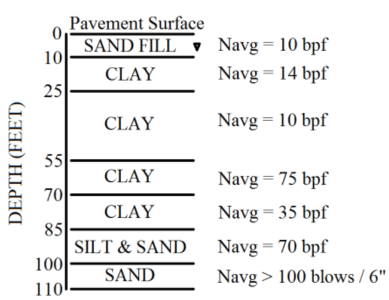


Fig. 2 Subsurface Profile

Subsurface conditions to a depth of about 50 feet below the ground surface were analyzed for the shallow foundation design. According to the borings, foundation bearing soils would consist of very stiff clay. The very stiff clay stratum was underlain by a soft to stiff clay stratum that began about 15 feet below design bottom of footing elevation. The soft to stiff clay stratum was underlain by a hard clay stratum that began about 45 feet below design bottom of footing elevation. Groundwater was encountered about 5 to 10 feet below the ground surface, but was perched (or entrapped) in the granular fill overlying the less permeable natural clay. Long-term groundwater levels were greater than 50 feet below the existing ground surface.

The following soil parameters were applicable for analyzing the bearing capacity of the shallow foundations:

- Undrained shear strength, c = 2,500 psf
- Soil Unit Weight = 120 pcf

The bearing capacity was determined from Section 10.6.3.1.2a-1 of the AASHTO LRFD Bridge Design Specifications (refer to Equation 1 below). A resistance factor of 0.45, per Section 10.5.5.2.2-1 of the Bridge Design Specifications, was applied to q_n to obtain the factored bearing capacity, q_f. The factored maximum foundation pressure (based on the strength I limit state) provided from the structural engineer for each abutment and pier foundation, along with the calculated q_f, are shown in Table 1.

$$q_n = cN_{cm} + \psi D_f N_{qm} C_{wq} + 0.5\psi B N_{\gamma m} C_{w\gamma} \quad (1)$$

$$qf = qn * \varphi_b \quad (2)$$

Since the bearing soils consisted of clay, and in accordance with the Bridge Design Specifications, no depth factor was assigned to the cohesion component of q_n. The absence of this depth factor reduced q_n by about 10 percent for footings at abutment A and pier 1A, and about 25 percent for the footing at abutment B. The embedment depth for the footing supporting abutment B was about 27 feet.

Table 1. Factored Bearing Capacity vs. Maximum Bearing Pressure

Foundation	**Factored bearing capacity, q_f**	**Factored max. bearing pressure**
Abutment A	7,517 psf	4,810 psf
Abutment B	7,878 psf	7,850 psf
Pier 1A	6,524 psf	6,330 psf

As indicated in Table 1 above, the factored bearing capacity was greater than the factored maximum foundation pressure for each foundation. It is important to note that q_f is barely greater than the factored maximum foundation pressure for abutment B. This is due to the relatively high soil overburden pressure (because of the significant embedment depth) applied to the footing load, and the absence of a depth factor when calculating q_n. The use of light-weight backfill behind abutment B was critical to increasing the factored maximum foundation pressure, so that the shallow foundation design could be implemented.

Calculating sliding resistance capacity was based on the following soil parameters:

- Clay soil-to-foundation sliding coefficient = 0.35
- Maximum adhesion value = 750 psf
- Passive earth pressure coefficient = 3.0
- Lateral soil bearing pressure (for keyway) = 4,000 psf

Sliding resistance for the proposed shallow foundation was calculated based on Section 10.6.3.4-1 of the Bridge Design Specifications (refer to Equation 3 below). Resistance factors for soil-to-foundation interaction, and for passive resistance, were 0.85, and 0.5, respectively.

$$\varphi R_n = \varphi_\tau R_\tau + \varphi_{ep} R_{ep} \tag{3}$$

The factored maximum sliding force (based on the strength I limit state) was calculated by the structural engineer for each abutment foundation. The factored sliding resistance values were greater than the factored maximum sliding force, because of the use of light-weight backfill behind the abutments. In addition, the structural engineer designed a 3-foot deep keyway for the abutment foundations to achieve the required sliding resistance.

Fig. 3 West Abutment Footing and Wall

Settlement estimates for the proposed shallow foundations were calculated based on Section 10.6.2.4.1 – Settlement Analyses, Section 10.6.2.4.3 – Settlement of Footings on Cohesive Soils, and Section 10.6.2.4.2-1 (for elastic settlement) of the Bridge Design Specifications. The equations in those sections were used for calculating elastic settlement, settlement from primary consolidation, and settlement form secondary consolidation. Table 2 summarizes the results of the settlement analyses, along with the total estimated settlement.

Table 2. Settlement Analysis Summary

Footing	Elastic	Primary Consolidation	Secondary Consolidation	Est. Total Settlement	Est. Differential Settlement
Abut. A	0.08"	0.36"	0.23"	0.67"	0.34"
Abut. B	0.12"	0.22"	0.22"	0.56"	0.28"
Pier 1A	0.13"	0.50"	0.25"	0.88"	0.44"

Total estimated settlement varied from 0.56 inches to 0.88 inches. Estimated differential settlements (over a 30-foot length) were one-half of the total settlement. The maximum acceptable settlement for the shallow foundations was 1.0 inch for total settlement, and 0.5 inches for differential settlement.

Of the three soil-related categories that were analyzed (bearing capacity, sliding resistance, and settlement), it was determined that sliding resistance controlled the size of the foundations for the abutments, and settlement controlled the size of the pier foundation.

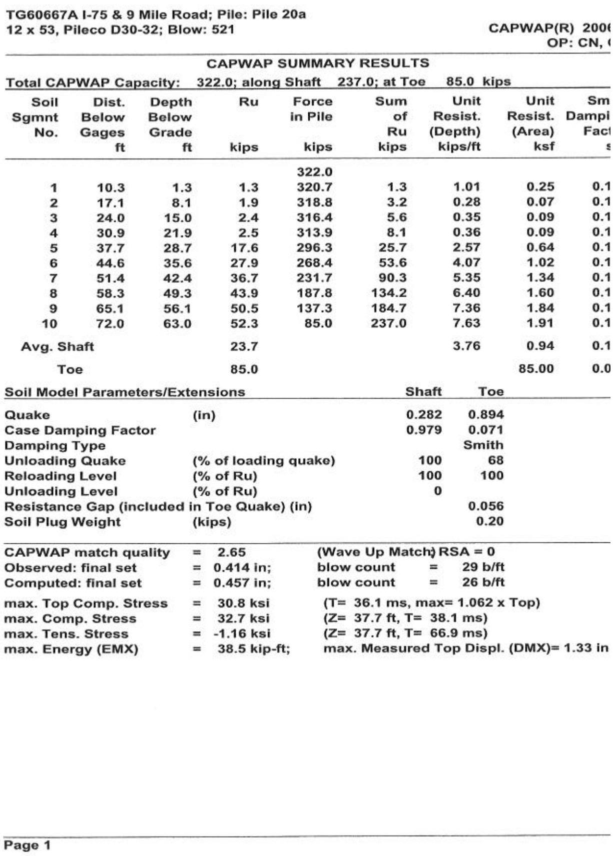
DEEP FOUNDATIONS

Deep foundations were installed for the future center pier. The purpose for the deep foundations was to provide a rigid foundation that would experience minimal movement once subjected to the full weight of the bridge dead and live loads. Specifically, construction of the future center pier would be completed during future highway expansion and reconfiguration project, but without removing the bridge deck. Therefore, the deep foundation system was designed to limit predicted movement to less than 0.5 inches once the center pier began support the bridge in the future.

The pile capacity and pile length was analyzed using equations from FHWA Driven 1.2 software. Both side friction and end bearing were used to obtain the predicted pile capacity. The results of the analysis indicated that HP12x53 steel H-piles could achieve a required nominal driving resistance of 400 kips for piles that were 60 to 70 feet long.

Confirmation of the design capacity of the piles is typically performed by a static load test, in which a cribbing and weight system is staked over the pile and a hydraulic jack pushes against the system while measuring the downward deflection of the pile. Since the project had an expedited schedule and confined lateral space, a conventional static load test of the piles was not desired. Therefore dynamic load testing was performed during the pile installation process using a Pile Driving Analyzer (PDA). The PDA is a computer that calculates results from velocity and force signals obtained by strain transducers and accelerometers attached to the top of the pile. The Case Method is used to assess the axial capacity of the piles, as well as assess shaft integrity (driving stresses), hammer energy transfer, and other related measurements.
A PILECO D30-32 hammer drove the H piles. PDA tests

were performed in the field on two production piles. Test results were transmitted remotely in real-time to an off-site location and were refined and analyzed using a Case Pile Wave Analysis Program (CAPWAP®). The analysis indicated the nominal dynamic capacity of a test pile was 322 kips at 63 feet below grade. The maximum recorded driving energy from the hammer was 38.5 kip-ft. Results of the CAPWAP® analysis are provided in the Figure 4.



TG60667A I-75 & 9 Mile Road; Pile: Pile 20a
12 x 53, Pileco D30-32; Blow: 521

CAPWAP(R) 200(
OP: CN, (

CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 322.0; along Shaft 237.0; at Toe 85.0 kips

Soil Sgmnt No.	Dist. Below Gages ft	Depth Below Grade ft	Ru kips	Force in Pile kips	Sum of Ru kips	Unit Resist. (Depth) kips/ft	Unit Resist. (Area) ksf	Sm Dampi Fact s
				322.0				
1	10.3	1.3	1.3	320.7	1.3	1.01	0.25	0.1
2	17.1	8.1	1.9	318.8	3.2	0.28	0.07	0.1
3	24.0	15.0	2.4	316.4	5.6	0.35	0.09	0.1
4	30.9	21.9	2.5	313.9	8.1	0.36	0.09	0.1
5	37.7	28.7	17.6	296.3	25.7	2.57	0.64	0.1
6	44.6	35.6	27.9	268.4	53.6	4.07	1.02	0.1
7	51.4	42.4	36.7	231.7	90.3	5.35	1.34	0.1
8	58.3	49.3	43.9	187.8	134.2	6.40	1.60	0.1
9	65.1	56.1	50.5	137.3	184.7	7.36	1.84	0.1
10	72.0	63.0	52.3	85.0	237.0	7.63	1.91	0.1
Avg. Shaft			23.7			3.76	0.94	0.1
Toe			85.0				85.00	0.0

Soil Model Parameters/Extensions		Shaft	Toe
Quake	(in)	0.282	0.894
Case Damping Factor		0.979	0.071
Damping Type			Smith
Unloading Quake	(% of loading quake)	100	68
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	0	
Resistance Gap (included in Toe Quake) (in)			0.056
Soil Plug Weight	(kips)		0.20

CAPWAP match quality = 2.65 (Wave Up Match) RSA = 0
Observed: final set = 0.414 in; blow count = 29 b/ft
Computed: final set = 0.457 in; blow count = 26 b/ft
max. Top Comp. Stress = 30.8 ksi (T= 36.1 ms, max= 1.062 x Top)
max. Comp. Stress = 32.7 ksi (Z= 37.7 ft, T= 38.1 ms)
max. Tens. Stress = -1.16 ksi (Z= 37.7 ft, T= 66.9 ms)
max. Energy (EMX) = 38.5 kip-ft; max. Measured Top Displ. (DMX)= 1.33 in

Page 1

Fig. 4 CAPWAP® Results

The efficiency of the pile hammer was analyzed using GRLWEAP™ software for the purpose of establishing the pile driving criteria based on the actual measured PDA test data. A portion of the analysis for nominal (ultimate) pile resistance relative to blows-per-foot from the D30-32 hammer is shown in Figure 5. To achieve a nominal (ultimate) resistance of 322 kips with the D30-32 hammer, a target of 29 blows-per-foot would be required, at a hammer stroke of 8.34 feet, and would produce 41.36 kip-ft of driving energy. Since the measured driving energy from the PDA testing was somewhat less (e.g. about 36 to 38.5 kip-ft), the target driving criteria was adjusted to 33 blows-per-foot.

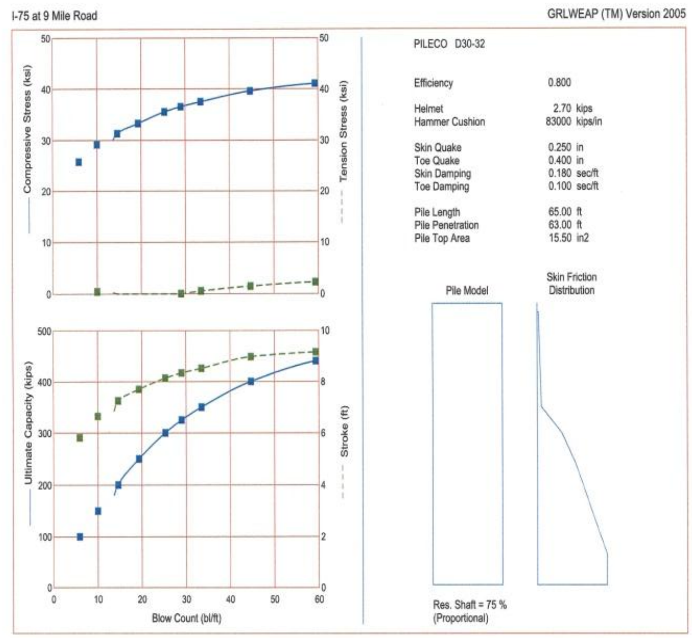


Fig. 5 GRLWEAP™ Results

An attempt was made to achieve a higher nominal resistance by driving one of the test piles to 70 feet below grade, and then re-striking the pile shortly thereafter. Note that a wait time of several days before re-striking the pile was desired to allow pore-water pressures to dissipate and increase frictional resistance along the pile shaft. However, due to schedule constraints, the re-strike occurred on the same day. PDA test results from the re-strike operation indicated the nominal driving resistance of the pile was 325 kips. The limited gain in driving resistance with depth was consistent with the findings from the soil borings, which indicated a decrease in soil strength, and a change in soils from clay to wet sands, from about 70 to 100 feet below grade. Therefore, the design and construction teams were presented with two options: 1) use a reduced nominal resistance and add more piles, or 2) drive the piles deeper (to about 105 feet below grade) where nominal resistance would increase substantially upon driving into the glacial till. Since size of the pile cap was unaffected by adding more piles, and due to the additional time required for splicing to drive piles deeper, the teams elected to reduce the nominal resistance and add more piles.

LIGHT-WEIGHT BACKFILL

The use of light-weight backfill, which consisted of expanded polystyrene (EPS) blocks, behind the new abutments was advantageous for the following reasons:

- Lateral earth pressure against the abutment walls was significantly reduced, thereby reducing the size of the abutment foundations.

- EPS blocks could be placed against the abutments walls immediately after the wall forms were stripped (no wait time for concrete curing).

- EPS blocks could be placed in inclement and/or below-freezing weather conditions.

While the EPS blocks were expensive relative to a conventional sand backfill and compaction operation, the ability to support the abutments on a shallow foundation system (in lieu of a deep foundation system) and the time savings in backfill placement during construction, resulted in a net advantage for the project budget and construction schedule.

The calculated unit weight of the EPS blocks was about 1.5 pcf. The design unit weight for determining the lateral earth pressure against the walls was 10 pcf (accounting for some moisture absorption). The base course of blocks was placed about one foot above the top of the abutment footings, and continued horizontally from the abutment walls to the back edge of the foundations. Each subsequent course of EPS blocks extended beyond the back edge of the footings so that the soil backfill against the end of the blocks formed a 1 horizontal to 1 vertical bench style slope. A relatively small K_a value of 0.08 was assigned to calculate the design lateral force on the backside of the blocks and the abutment walls.

A 30 mil PVC liner was placed over the top course of blocks and against the ends of the top two courses of block. The top course of block was about 8 feet below design final grades at the top of the walls. The total thickness of the EPS system was up to 19 feet. Well-draining granular backfill was placed around the EPS blocks, along with an underdrain system.

TEMPORARY EARTH RETENTION SYSTEM

Temporary earth retention was required to construct both the new and future center piers. Retained earth heights of about 10 feet, or less, were necessary to allow vertical excavation adjacent to I-75, thus limiting disruption to highway traffic. The earth retention consisted of both cantilevered and braced systems using continuous steel sheet piles. The cantilever wall was designed for the new center pier, and the braced wall was designed for the future pier. Deflection was the controlling factor in both design cases.

The cantilever wall consisted of 20-foot long PZ-22 steel sheet piles that retained up to 9 feet of earthen subgrade with a live load highway traffic surcharge. Predicted deflection at the top of the wall was about ½ inch.

The braced wall was not constructed but needed to be designed for a future condition that involved moving the center pier east so the highway could be expanded and reconfigured. The purpose for the braced wall would be to provide working room for the installation of a future pile cap that would be immediately next to, and 5 feet deeper than, the bottom of the new center pier. In addition, the wall design needed to consider that the future center pier foundation would be constructed without removing the bridge deck.

The braced wall design consisted of 20-foot long PZ-22 steel sheet piles with HP10x42 walers and struts (spaced at 12-foot centers) that would retain up to 14 feet of earthen subgrade and provide temporary lateral support for the existing center pier foundation until the future pier could be constructed and secured to the bridge deck. Predicted deflection at the top of the wall was less than 1/8 of an inch.

COMPARE LRFD AND ASD DESIGN METHODS

The former bridge for Nine Mile Road was designed in 1964 using the Allowable Stress Design (ASD) methodology. The replacement bridge was designed using the LRFD methodology. While there were some differences in the new design (which was based on a reconfigured abutment layout), some design comparisons could be made on the center pier foundation, and the allowable/factored capacity of the steel H pile foundations.

Fig. 6 Bridge Deck Under Construction

Shallow Foundations

The former bridge design used a shallow foundation system that consisted of five strip footings for two abutments and three piers (four span bridge). The abutment foundations were located near the top of the embankments that sloped down to the highway. Two of the three piers were located at the toe of the embankments on the east and west sides, and the remaining pier was located in the median of the highway.

The new abutments were located near a former abutment on one end, and near a former pier on the other end. Also, the new abutments were designed to retain about 25 feet of earthen subgrade and light-weight backfill, whereas the former abutments were near the top of the embankment and retained only about 4 feet of earthen subgrade. This reconfiguration of the new abutments, and embankments, created a unique design relative to the former bridge layout. Therefore, comparing the new abutment design (LRFD) with the former design (ASD) was not practical.

However, the new center pier was reconstructed near the former location of the existing center pier. The pavement section for the old and new bridge was nearly the same. The design live load and deflection criteria for the former bridge was similar to those for the new bridge. There was, however, a difference in the length and number of spans, and a minor difference in bridge width. The new bridge deck spans (about 90 to 120 feet) were larger than the former bridge spans (about 30 to 80 feet). The new bridge deck is about 73 feet wide, whereas the former bridge deck was about 66 feet wide. Overall, the tributary area for the center pier of the new bridge was 7,665 square feet, which is about 49% larger than the tributary area for the former center pier (at about 5,150 square feet).

The former center pier footing width, based on the ASD methodology, was 9 feet. Using the LRFD methodology, the new center pier footing width was 14 feet or about 55% larger. Given the difference in tributary areas between the new and former center pier foundations, the LRFD-based footing size is generally consistent, but slightly larger than the ASD-based footing size. Other comparisons on this project between the two methodologies indicate that sizing the center pier foundation using LRFD were about 5 to 15 percent greater than using ASD. A primary reason for the increase in footing size appears to be connected with the LRFD requirement to analyze footings based on their effective width, not their total width. The soils analysis using LRFD did not appear to have an effect on increasing, or decreasing, the size of the center pier foundation.

Deep Foundations

A comparison between LRFD and ASD methodologies could also be made for the deep foundation system. This comparison consisted of establishing the predicted ultimate/nominal pile resistance at 400 kips, and using the two design methodologies to obtain an allowable/factored pile capacity.

Since PDA testing was implemented for this project, the LRFD value for the resistance factor for driven piles, φ_{dyn}, was 0.65. Another resistance factor, per an LRFD-based special provision for the project, was applied and required the nominal resistance of the test pile be 110 percent of the nominal pile resistance of the production piles. In addition, the nominal resistance was reduced by an additional 10 kips to account for the existing soil overburden (as about 10 feet of soil would be removed around the piles in the future as part of the highway expansion/realignment project). When considering the nominal resistance value of 322 kips measured from the PDA testing, the factored nominal axial pile resistance (R_R) was 183 kips.

For the ASD methodology, the ultimate pile capacity was reduced by 10 kips to account for the existing soil overburden, and then was divided by a factor of safety of 3.0. Therefore, the allowable pile capacity was 130 kips.

A summary of the analyses based on LRFD and ASD methodologies is provided in the following table:

Table 3. LRFD vs. ASD – Drive H-Pile Foundation

Design Method	Est. Nom. / Ult. Resist.	Nom. Resist. (PDA test)	Special Provision Reduction	Overburden Reduction	Resist. Factor / Safety Factor	Factored Resist. / Allow. Capacity
LRFD	400 kips	322 kips	29 kips	10 kips	0.65	183 kips
ASD	400 kips	322 kips	---	10 kips	2.25	138 kips

The LRFD design methodology, coupled with the PDA testing, increased the usable capacity of the piles by about 33% when compared with the ASD design methodology. While this study was limited in comparing the two design methods, this evidence supports the conclusion that the LRFD-based design realizes greater pile capacity that the ASD-based design. The primary reason for this difference is that the LRFD resistance factor (0.65) is the equivalent of a factor of safety of about 1.54, compared to the ASD factor of safety of 2.25.

Fig. 7 Completed Bridge

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