Using Load Testing to Save Money and Time on the I-35W Bridge Project

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ABSTRACT: The cost and time of performing design-phase load testing or altering the design based on load tests at the start of construction often inhibits the use of load tests. A case history of a transportation project in the Minneapolis-St. Paul area demonstrates significant monetary and time savings that can come from load testing before, or at the start of, construction. The I-35W Bridge project utilized a bi-directional load test to reduce shaft lengths, which not only saved material costs but also dramatically reduced construction time.

INTRODUCTION

Load testing of deep foundations is commonly limited to resistance confirmation. The design and construction industry generally views the monetary cost and time of performing design-phase load testing as too expensive. Similarly, it views the costs of altering the design based on load tests at the start of construction as too high. Large or complex projects tend to be the exceptions. Using load tests to guide foundation design, a project can realize money and time savings. A design-build transportation project in the Minneapolis-St. Paul metropolitan area demonstrates the potential for monetary and time savings from static load testing.

The Interstate 35W Bridge in downtown Minneapolis was a momentous collapse. Equally significant was the design and construction of a replacement bridge in only 339 days, supported primarily on drilled shafts. The shaft design included a bi-directional static load-test shaft to validate design parameters and construction methods. The test results allowed a significant reduction in shaft length, with a direct cost and time savings.

FOUNDATION DESIGN PROCESS

The idealized foundation design process is similar to the following:

1. Desire for new structure is identified, be it a bridge, hotel, stadium, research facility, manufacturing plant, etc.
2. Working with architects and/or engineers, the owner selects a preliminary shape and size for the new structure, and a project site.
3. The geotechnical engineer determines the type of subsurface exploration best-suited to the project, anticipated soil/rock conditions, along with the number of exploration points, and in-situ and laboratory tests.
4. After completing the subsurface exploration, the geotechnical engineer identifies the foundation systems that are best-suited to the project, and makes preliminary estimates of geotechnical resistance these systems can provide.
5. From here, the process varies a bit from project to project, but generally the project team advances the project design while evaluating the potential risks and benefits, including cost and schedule, of candidate foundation systems.

Eventually, the most-favorable system is identified, construction documents are generated, and the foundation is built. As part of building the foundation, some form of testing is commonly performed to confirm design assumptions. As the following case history shows, there is a significant opportunity to reduce the foundation cost either at this point in the construction, or even earlier in Step 5 of the design process outlined above.

INTERSTATE 35W BRIDGE PROJECT

BACKGROUND

At 6:05 PM on August 1, 2007 a nightmare became reality in Minneapolis, Minnesota. The Interstate 35W (I-35W) Mississippi River bridge, the third-busiest bridge in Minnesota that carries 140,000 northbound and southbound vehicles each day, and serves as a vital link to downtown Minneapolis, collapsed. The collapse sent 111 vehicles, as well as construction workers, equipment and materials 115 feet (35 m) down to the river or its banks. In all, 145 people were injured and 13 people died.

Originally constructed in 1967 with a 50-year design life, Bridge 9340 (the I-35W Bridge designation) was an eight-lane, steel arch truss with 14 spans covering a total length of 1,970 feet (600 m). The main section of the bridge over the Mississippi River had a span of 456 feet (139 m). The Minnesota Department of Transportation (MnDOT) estimated that the out-of-service cost for the bridge was approximately US$400,000 per day. This led to the decision to use the design-build alternative project delivery system for replacing the I-35W Bridge, and MnDOT selected the Apparent Best Value Bid process (Field and Gebhard, 2009).

A joint venture of Flatiron Construction and Manson Construction was the successful bidder and entered into a contract with MnDOT to design and build the replacement bridge. Figg Bridge Engineers and TKDA, Inc. led the design team, with Braun Intertec Corporation (Braun Intertec) providing geotechnical, environmental, and construction materials services. MnDOT wanted the replacement bridge to consist of two identical bridges, one for northbound traffic, and the other for southbound traffic. Requirements for the new bridges included being open in just over a year and having a design life of 125 years. The design-build team selected bridges with four-spans, using both cast-in-place and precast, post-tensioned concrete box girder segments. Each new bridge has five traffic lanes and can accommodate future light-rail transportation. The new bridges are 1,223 feet (373 m) long, with a combined width of 176 feet (54 m), approximately 250 feet (76 m) longer and 75 feet (23 m) wider than the original bridge (Field and Gebhard, 2009).

The project documents allowed the design-build team to select the foundation system to use at some substructures for the new bridge. However, based on the options for bridge types and the preliminary subsurface information available, MnDOT required the river piers to be drilled shafts.

SUBSURFACE PROFILE

Figure 1 illustrates the general subsurface conditions. In general, subsurface conditions at the bridge site consist of fill and surficial soil deposits overlying bedrock layers. Furthest from the Mississippi River channel, the uppermost bedrock is Platteville Limestone that overlies a 3- to 10-foot-

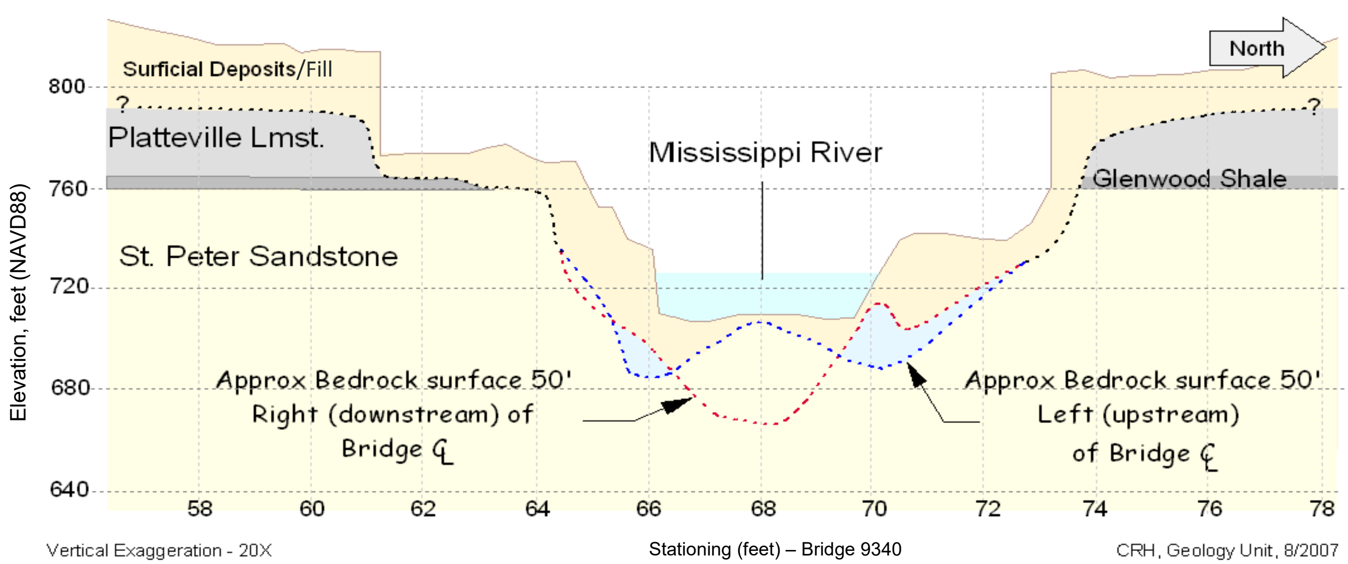

Figure 1. Generalized Subsurface Conditions (MnDOT, 2007)

thick (1- to 3-m-thick) layer of Glenwood Shale. Closer to the Mississippi River, the river has eroded these layers away, likely as the St. Anthony Falls advanced upstream to their present-day location. Under the Glenwood Shale or surficial soil deposits closer to the river channel, the bedrock consists of St. Peter Sandstone. Some rock coring encountered artesian conditions, most notably on the north bank of the Mississippi River.

INITIAL DESIGN

Due to contaminated, uncontrolled fill in the abutment areas, the project team chose to use deep foundations to support all the substructures. At Abutment 1, south of the river, the team chose to use H-piles bearing in the Glenwood Shale. At the other substructures, the project team chose drilled shafts. Of focus for this case history are the drilled shafts for Piers 2, 3 and 4, which obtained support from the St. Peter Sandstone.

The initial estimates for side resistance and end bearing followed the procedures set forth in the American Association of State and Highway Transportation Officials' (AASHTO's) LRFD Bridge Design Specifications (2007), and the Federal Highway Administration's (FHWA's) Drilled Shafts: Construction Procedures and Design Methods (1999). This approach uses the Rock Mass Rating (RMR), and the rock's unconfined compression strength and elastic modulus to determine the unit side and end resistance of the rock. RMR is determined based on five parameters:

1. The rock's strength using either the point load strength index or the uniaxial compression strength
2. The quality of the drill core based on Rock Quality Designation (RQD)
3. Joint spacing in the rock
4. The condition of the rock's joints.
5. Groundwater conditions

Measured RQD values of the St. Peter Sandstone for Piers 2, 3 and 4 varied from 0 to 97 percent. Similarly, the unconfined compression strength varied from approximately 40 to 2,100 pounds per square inch (psi) (276 to 14,479 kPa). Specifically at the test shaft, the RQD varied from 0 to 94 percent, and the unconfined compression strength varied from approximately 40 to 1,900 psi (276 to 13,100 kPa). In general, the RQD and unconfined compression strength increased with depth in the St. Peter Sandstone.

For this project, the project team wanted to use both end bearing and side resistance in the St. Peter Sandstone. Based on the RQD, core recovery amounts, unconfined compression test results and direct shear test results, the project team concluded that the St. Peter Sandstone would behave as a ductile rock mass. This conclusion enabled the use of both side shear and end bearing resistance to axial loading.

The initial, nominal, unit side resistance in the St. Peter Sandstone was estimated to range from 0.5 to 10 kips per square foot (ksf) (24 to 479 kPa). The nominal, unit end bearing was estimated to range from 60 to 150 ksf (3 to 7 kPa). In accordance with project requirements, a resistance factor of 0.50 was used for shafts not designed based on a load test, and a resistance factor of 0.65 was used with the load test. These factors were applied to both side and end resistance. For the anticipated loads and nominal resistances, the project team selected drilled shafts with a rock-socket diameter of 84 inches (2,134 mm) for Piers 2 and 3, and 96 inches (2,438 mm) for Pier 4.

TEST SHAFT

The project specifications required the contractor to perform at least one test shaft for what came to be the location of Pier 3, along the north bank of the Mississippi River. The specifications also required the contractor to construct a method shaft that would allow all parties involved in shaft construction to work out the necessary processes, in addition to validating the contractor's techniques for constructing the shaft. The method shaft was to have a diameter equal to at least 75 percent of the production shaft diameters, and the method shaft and test shaft could be combined. However, the specifications did not require the project team to revise the drilled shaft design based on the results of the load test.

The project team did choose to combine the test shaft and method shaft. With the initial production pile design having a shaft diameter of up to 96 inches, the project team requested and received approval to construct a test/method shaft with a diameter of 78 inches (1,981 mm). The designers and MnDOT determined that scaling effects would be minimal for this difference in diameter. The initial test shaft design had a bottom-of-shaft elevation of approximately 632 feet (193 m), resulting in a length of 53 feet (16 m).

While drilling the test shaft, Case Foundation Company (Case) encountered artesian water pressure, with a head of approximately 15 feet (5 m) above the ground surface. This was approximately 10 feet (3 m) higher than was observed during subsurface explorations. The project team chose to install a second test shaft having the same diameter (78 inches or 1,981 mm), but terminating at Elevation 646 feet (197 m), resulting in a rock socket length of 39 feet (12 m). This allowed the shaft to terminate above artesian conditions.

Loadtest, Inc. performed a 2-level, bi-directional load test. Figure 2 shows the test shaft arrangement with the bi-directional jack assemblies, strain gauge levels, and telltale locations. It also shows a generalized subsurface profile at the test shaft location.

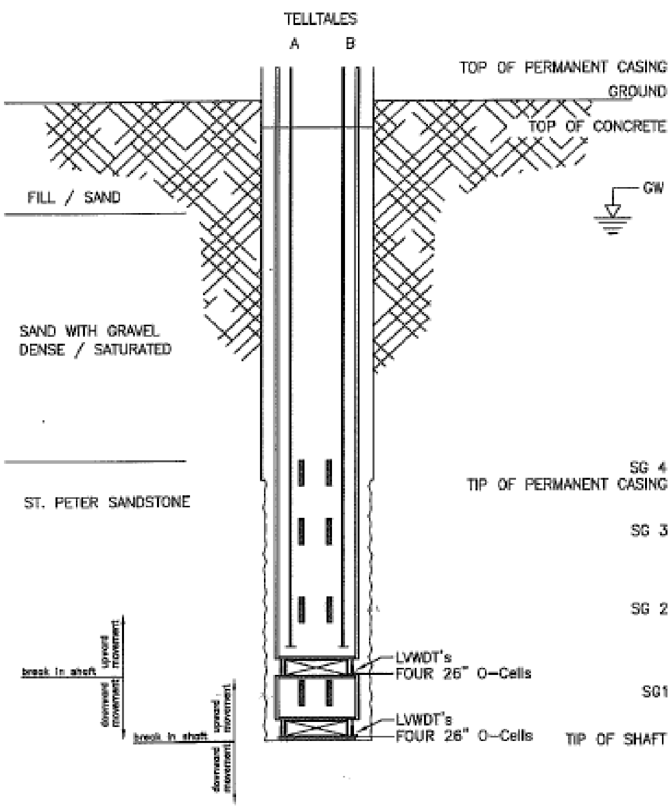

Figure 2. Test Shaft Arrangement (Loadtest, 2007)

The load was applied in three stages to evaluate different zones of interest as follows:

1. In Stage 1, the upper jack assembly was closed and the lower assembly was pressurized, to evaluate end-bearing resistance.
2. For Stage 2A, the lower jack assembly was allowed to drain while the upper assembly was pressurized. This allowed evaluation of side resistance between the two jack assemblies, while not engaging any end bearing below the lower assembly.
3. The last stage, Stage 2B, closed the lower jack assembly, and pressurized the upper assembly, to evaluate shaft resistance above the upper assembly.

Table 1 summarizes the unit resistances estimated for initial design and determined through the load test results for Piers 2, 3 and 4. The table also shows the estimated unit resistances specifically at the test shaft location.

Table 1. Unit Resistance Summary

Design Stage	Nominal Unit Resistance, ksf (kPa)	
	Side	End
Initial	0.5 to 10 (24 to 479)	60 to 150 (2,872 to 7,182)
Final	2 to 40 (96 to 1,915)	90 (4,309)
Test Shaft	2 to 8 (96 to 383)	150 (7,182)

Based on the test results, the final design used a nominal, unit side resistance of 2 ksf (96 kPa) for relatively weaker sandstone. For more-competent sandstone, the final design used a nominal, unit side resistance of 40 ksf (1,915 kPa). The final design included all the shafts bearing in the more-competent sandstone, using a nominal, end bearing resistance of 90 ksf (4,309 kPa).

The final design utilized RQD and core recovery rates to correlate the location of the transition from weaker to more-competent sandstone between the test shaft and the production shafts. In the field, the contractor and inspectors made use of drilling rates, drilling torque and drilling crowd pressure, in addition to the color and appearance of the cuttings.

COSTS AND SAVINGS

Based on load-test results, the redesign resulted in Pier 2 being supported on 78-inch-diameter (1,981-

mm-diameter) drilled shafts, and Piers 3 and 4 being supported on 90-inch-diameter (2,286-mm-diameter) drilled shafts. As-built rock-socket lengths ranged from approximately 38 to 93 feet (12 to 28 m), with lengths in the more-competent sandstone ranging from approximately 11 to 30 feet (3 to 9 m). Total shaft lengths, including overburden and rock socket, ranged from approximately 83 to 113 feet (25 to 34 m).

To estimate the construction cost of a design without benefit of a load test, the final factored design loads, the overburden depth at each shaft location and initial shaft diameter and unit resistance values were used. Based on a combined soil/rock excavation cost of US$45 per cubic foot (US$1,590 per cubic meter), Case's cost for constructing shafts at Piers 2, 3, and 4 is estimated to be US$15,162,976. Case's and Loadtest's cost of constructing both method shafts, and performing the load test, was US$583,000. Case's actual cost of constructing the drilled shafts at Piers 2, 3, and 4 based on the final design was US$7,726,612. This results in a net savings attributable to the load test of approximately US$6,853,364.

Another way to compare costs is to look at foundation support cost (Komurka, 2015). Several components make up a foundation's support cost:

1. Foundation support cost can be evaluated in two ways:
    a. Available foundation support cost is the cost to construct foundations divided by the factored geotechnical resistance of the foundations.
    b. Utilized foundation support cost is the cost to construct foundations divided by the factored structural load on the foundations. As this is the cost that the owner actually pays to resist one unit of load, it is the more-appropriate foundation support cost to evaluate.
2. Cap support cost is the cost to construct the pile cap divided by the factored structural load on the cap.
3. Construction control method (CCM) cost is the cost of all testing divided by the factored structural load on all the foundations to which the testing results apply.

Table 2 summarizes the support costs for the drilled shafts at Piers 2, 3, and 4.

Table 2. Foundation Support Cost Summary

Design Stage	Available Support Cost, US$ / kip (US$ / kN)	Utilized Support Cost, US$ / kip (US$ / kN)	CCM Cost, US$ / kip (US$ / kN)	Total Support Cost, US$ / kip (US$ / kN)[1]
Initial	32.77 (7.36)	39.42 (8.86)	-	39.42 (8.86)
Final	16.70 (3.75)	20.09 (4.52)	1.52 (0.34)	21.61 (4.86)

1. Based on utilized support.

The cap support cost for the initial and final designs is unknown. However, the number of shafts at each pier did not change from initial to final design. Given that the shaft sizes at Piers 2 and 4 are smaller for final design, the cap costs would likely be less for final design. Even disregarding this additional savings from cap costs, the total support cost difference (savings) attributable to the load test is approximately US$17.81 per utilized kip (US$4.00 per utilized kN) of resistance.

Another consideration is the time savings that resulted from the load test. Looking at the drilling rate in the more-competent rock provides some comparison. The initial design would have resulted in drilling 3,114 linear feet (949 m) of more-competent rock. The actual length of drilling more-competent rock was approximately 836 feet (255 m). Case's drilling rate in this material ranged from 1 to 4 feet per hour (0.3 to 1.2 m per hour), meaning that without load testing, drilling would have taken 570 to 2,278 additional hours. The drilled shafts were constructed on a 24-hour basis, seven days a week, except during the week of Christmas, and New Year's Day. Accordingly, drilling would have required an additional 23 to 95 days if a load test had not been performed and the final design had not been adjusted based on the results. Based on MnDOT's out-of-service cost for the bridge of US$400,000 per day, this range of drilling time saved corresponds to out-of-service cost savings ranging from 9.2 to 38 million US dollars.

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