



Level Osterberg Cell on site for Minneapolis's I-35W replacement bridge

Load Test Saves Money and Time at Minnesota Bridge Replacement

On August 1, 2007, the I-35W Bridge over the Mississippi River near downtown Minneapolis collapsed during the evening commute, tragically killing 13 people and injuring more than 100 others. The bridge was part of a main artery through Minneapolis and one of the most heavily used bridges in Minnesota. Nearly 140,000 vehicles crossed the bridge each day and the State of Minnesota estimated the additional cost to motorists to detour around the bridge at approximately \$400,000 per day. Construction of a new bridge in the shortest time possible was critical. An Osterberg load test supplemented preliminary boring of the subsurface and determined that the final shaft length could be shortened by about 30 m (100 ft), which contributed to significant cost savings for the project.

When the bridge collapsed, about 305 m (1,000 ft) of the more than 579 m (1,900 ft) long, multi-lane bridge went down. About 137 m (450 ft) of the main span fell roughly 34 m (110 ft) into the Mississippi River. With out-of-service time a critical issue for the local economy, the Minnesota Department of Transportation (Mn/DOT) chose to utilize design-build contracting to expedite construction of a new bridge. The project was awarded to the design-build team of Flatiron-Manson, a joint venture, with Figg Bridge Engineers (FIGG)/TKDA leading the design.

As part of the construction of the I-35W replacement bridge, Flatiron-Manson, FIGG and **Braun Intertec** (the geotechnical engineering firm for the design-build team), concluded that a drilled shaft foundation system would be the most economical means to support the river span of the new four-span bridge shown in Figure 1. To satisfy the requirements of the Request for Proposal (RFP), and to optimize the skills of the team, the design selected was a post-tensioned, box girder bridge. The river span would be precast segments and the approach spans would be cast-in-place. The design also included a separate bridge of the same construction to carry traffic over 2nd Street. As part of the design, driven H-piles would support Abutment 1 while drilled shafts would support the remaining substructures. The final design included supporting Abutment 5 and the 2nd Street abutments on 1.2 m (4 ft) diameter drilled shafts with 1.1 m (3.5 ft) diameter rock sockets. Drilled shafts with 2.1 m (7 ft) diameters and 2.0 m (6.5 ft) diameter rock sockets would support Piers 2 and 3, while 2.4 m

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(8 ft) diameter shafts with 2.3 m (7.5 ft) diameter rock sockets would support Pier 4. Therefore, the full-scale load test the design-build team elected to evaluate consisted of a drilled shaft diameter of 2.0 m (6.5 ft).

Due to the estimated axial and lateral loads at the drilled shafts, and the expected subsurface conditions based on preliminary soil borings, the engineers expected drilled shaft depths of more than 43 m (140 ft). Because of the limited nature of the preliminary soil borings, the design-build team conducted a supplemental geotechnical evaluation. They focused on refining the interpretation of the St. Peter Sandstone strengths and lengthening the borings to develop more confidence in the engineering characteristics of the bedrock near the drilled shaft tip elevations.

On October 10, 2007, the Minneapolis office of Braun Intertec mobilized four drill rigs to help the team evaluate soil and bedrock conditions beneath the foundations for the new bridge. The drill crews were challenged because wreckage from the collapsed bridge was still being removed from the site and the new bridge footings would be in areas of contaminated soil, with portions being designated as a Superfund site. As the entire bridge project progressed, drilling was performed amid a continuous and growing influx of contractors, equipment and stockpiled bridge materials. The drilling operations for geotechnical and environmental purposes explored more than 60 locations, which required more than 944 m (3,100 ft) of soil drilling and 274 m (900 ft) of rock coring. Containing the contaminated drilling spoils involved using three, 3,785 liter (1,000 gallon) plastic tanks and more than a hundred 208 liter (55 gallon) steel drums.

The site consisted of mostly coarse-grained soils (sands and gravel) overlying limestone and sandstone bedrock. The relative density of the coarse-grained soils ranged from loose to very dense and generally increased with depth. The standard penetration test borings encountered the first bedrock at depths between 1.5 m (5 ft) and 15.2 m (50 ft). Platteville Limestone was typically found on the higher banks north and south of the river. From Abutment 1 to the south edge of the river, the borings did not encounter the limestone, which was likely quarried and removed from this area. The Platteville Limestone found was typically about 9.8 m (32 ft) thick. A 0.6 m (2 ft) to 0.9 m (3 ft) thick layer of Glenwood Shale underlay the Platteville Limestone and then transitioned to St. Peter Sandstone, typically around elevation 760.

The supplemental investigation identified two significant issues: one was that the St. Peter Sandstone was stronger at depth than originally anticipated; the second was that a shaley layer at depth was acting as an aquatard confining up to 4.6 m (15 ft) of artesian water pressure near the anticipated tip elevation.

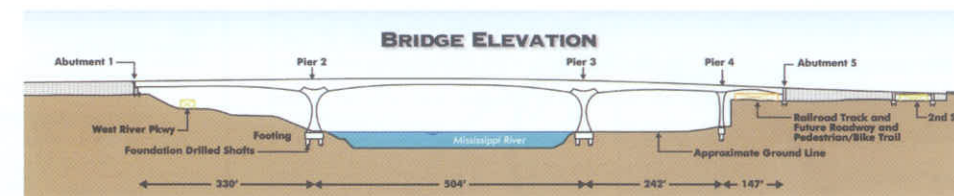


Figure 1: I-35W Bridge profile

Load Test Design

The RFP required the test shaft to be installed to confirm load carrying capacities. The shaft had to be at least 3/4 the diameter of the largest production shaft and be loaded to at least 1.5 times the nominal resistance of the production shafts. Based on these requirements, the available subsurface information and the factored bridge loads, the design test shaft had a diameter of 2.0 m (6.5 ft), a length of 29.6 m (97 ft) and load cells designed to exert over 111.2 MN (25,000 kips) of load into the shaft. The team designed the test shaft to mobilize the skin friction over the entire length of the rock socket and the end bearing. As such, the test shaft was sacrificial and not used to support the proposed bridge.

The boring near the test shaft location at Pier 3 encountered about 16.7 m (55 ft) of sands and gravel with cobbles and boulders overlying highly weathered to weathered sandstone from the St. Peter Formation to a depth of about 25.6 m (84 ft). The highly weathered to weathered sandstone transitioned into fresh sandstone with Rock Quality Designations (RQDs) ranging from 5 to 92 and the majority being over 70. At a depth of about 31.7 m (104 ft) below the ground surface, roughly elevation 639, the boring found a layer of shaley sandstone that was about 1.5 m (5 ft) thick (the aquatard layer). Fresh sandstone with RQDs ranging from 70 to 94 was below the shaley sandstone.

The design-build team selected Loadtest, Inc. to aid in designing the load test, and to instrument the test shaft and analyze the results. The final design of the test shaft incorporated two levels of Osterberg Load Cells (O-cells®) for bi-directional loading: one level at the shaft tip and the other level located approximately 3.0 m

(10 ft) above the tip. Each level included four, 16.0 MN (3,600 kip), 66 cm (26 inch) diameter O-cells arranged in a cloverleaf pattern shown in Figure 2. The test itself consisted of three separate stages. In the first stage, the lower level was pressurized in 15 load increments and then depressurized in 4 decrements in order to assess the tip resistance. The second stage, called Stage 2A, pressurized the upper level O-cells in a similar manner as the first stage while the lower level O-cells were allowed to drain. This approach allowed assessment of the side friction resistance between the two O-cell levels with no load transferred to the tip. The third stage, called Stage 2B, locked the lower-level O-cells and continued pressurizing the upper-level O-cells in additional eight increments in



Osterberg Cells being placed

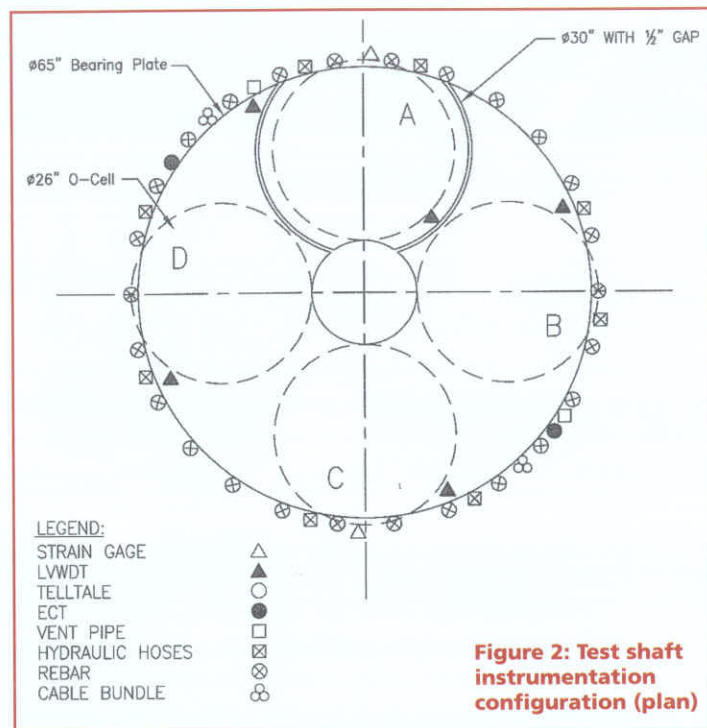


Figure 2: Test shaft instrumentation configuration (plan)

order to assess the side friction resistance of the shaft above the upper level of O-cells.

In addition to the eight O-cells, the test shaft contained instrumentation designed, assembled and installed by Loadtest. Figures 2 and 3 show the locations and typical arrangement of the instrumentation used to measure O-cell expansion, vertical shaft displacements and side shear transfer above and below the upper level of the O-cells. Specifically, the instrumentation consisted of:

- Linear vibrating wire displacement transducers
- Embedded compression telltales
- Vibrating wire strain gauges
- Bourdon pressure gauges
- Vibrating wire pressure transducers
- Automated digital survey levels

Load Test Results

The load test was performed on Thanksgiving Day 2007. Loadtest conducted the test under the observation of representatives from the Federal Highway Administration (FHWA), Case Foundation Company (the drilled shaft contractor), Mn/DOT and Braun Intertec. The measured loads and displacements were sufficient to mobilize both skin friction over the length of the rock socket and the end bearing such that nominal skin resistance and end bearing values could be determined.

The evaluation of unit side resistance involves calculation of the net load, defined as the gross O-cell load minus the weight of the shaft above the O-cell. The development of the equivalent top-loaded settlement curves for each level of O-cells also used the net load. The shaft load (buoyant weight) was calculated as 1.67 MN (375 kips) above the lower level of O-cells and 1.53 MN (344 kips) above the upper level of O-cells.

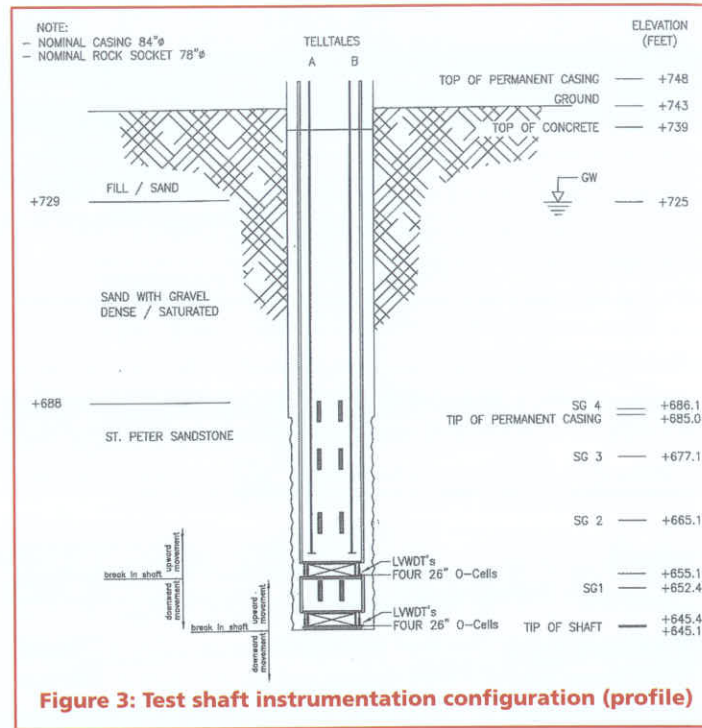


Figure 3: Test shaft instrumentation configuration (profile)

For Stage 1, the maximum, sustained, bi-directional load applied to the lower side shear and shaft tip was 37.95 MN (8,532 kips). At this maximum load, the displacement under the lower O-cell level was 58.42 mm (2.3 in). From the developed load-displacement curve, the determined load at the tolerable toe displacement of 15.24 mm (0.60 in) was 13.34 MN (3,000 kips). Applying this load uniformly across the cross section of the 2 m (6.5 ft) diameter shaft results in an end bearing resistance of 12,311 kPa (257 ksf).

For Stage 2A, the maximum, bi-directional load was 35.58 MN (8,000 kips) with a measured displacement of 17.27 mm (0.68 in). Stage 2B applied a maximum, bi-directional load of 55.88 MN (12,563 kips) to the shaft located above the upper level of O-cells. The recorded displacement was 16.29 mm (0.64 in). The assessment of unit side resistance involves calculating loads based on strain gauge data and approximations of shaft stiffness. Load is calculated by multiplying the estimated stiffness by strain in the shaft. Using the calculated load and surface area of the shaft



Load Test Thanksgiving Day dinner

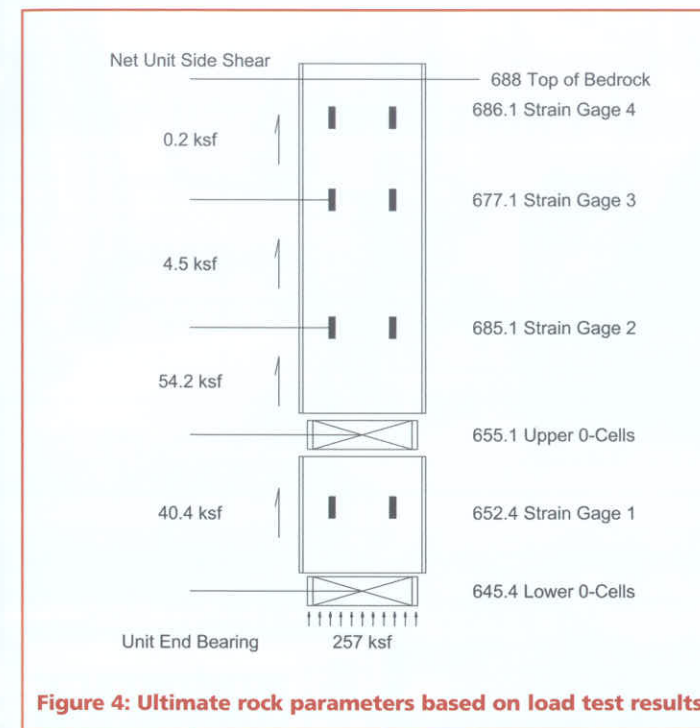


Figure 4: Ultimate rock parameters based on load test results

between adjacent strain gauge levels enables determination of unit side resistance between the strain gauge levels. Figure 4 shows the results of unit side resistance calculations.

Production Shaft Design

The design team followed the methodology outlined in the 2007 American Association of State Highway and Transportation Officials Load and Resistance Factor Design Bridge Design Specifications and the FHWA manual, *Drilled Shafts: Construction Procedures and Design Methods (1999)* during preliminary design of the drilled shafts. For shafts founded in rock, this methodology uses relationships to the unconfined compressive strength of the rock, which Braun Intertec gathered during the subsurface exploration program. Based on this empirical approach the estimated skin friction between elevation 688 and 665 was 95 kPa (2 ksf) and 479 kPa (10 ksf) between elevation 665 and 600. Below elevation 665, the estimated end bearing was 5,267 kPa (110 ksf).

Using the recommended resistance factor of 0.50, which is applicable with no load testing, the parameters result in shaft lengths up to about 43 m (140 ft).

In addition to justifying the use of a higher resistance factor of 0.65, the load test analysis resulted in significantly higher unit skin resistance values for the lower portions of the shaft. Based on the test results and subsurface data, the final design used skin friction of 120 kPa (2.5 ksf) between elevation 688 and 665 and 1,915 kPa

(40 ksf) between elevation 665 and 600. The final design also used an end bearing resistance of 4,309 kPa (90 ksf) below elevation 665, based on the test results and subsurface data. Thus the unit skin resistance for the upper portions of the shaft was marginally higher than estimated in the design phase, while the end bearing resistance was slightly lower.

Using the final design parameters with a resistance factor of 0.65 resulted in average shaft lengths of about 30 m (100 ft), well below the originally expected 43 m (140 ft) drilled shaft length.

Conclusions

The multi-level test identified multiple zones of rock strength allowing the design-build team to optimize the drilled shaft design. The final design depths enabled the toe of the shafts to stay above the shaly sandstone, which meant the drilled shaft contractors did not have to deal with artesian pressures during construction and allowed the actual shaft length to be shortened by around 30%. This effectively saved millions of dollars in construction costs and schedule impacts.

Although load tests, and even O-cell load tests, are common for projects of this magnitude, multi-level tests are relatively rare. At the time of this test, the full load range of the design was one of the largest, multi-level O-cell test loads known to have been performed. Following the test, the design-build team then worked 20 to 24 hours a day, 7 days a week (except for a brief holiday at Christmas) to complete installation of the drilled shafts by early-February 2008. The design and construction of the entire bridge took only about 11 months, a far shorter time than other projects of this magnitude. The new bridge opened to traffic on Sept. 18, 2008, 97 days before the contract deadline.

Drilling production shaft

