International Conference on Case Histories in Geotechnical Engineering

(2013) - Seventh International Conference on Case Histories in Geotechnical Engineering

01 May 2013, 2:00 pm - 4:00 pm

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April 29 - May 4, 2013 CHICAGO
Seventh International Conference on Case Histories in Geotechnical Engineering
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PERFORMANCE MONITORING OF A BRIDGE ABUTMENT SPREAD FOOTING FROM CONSTRUCTION THROUGH SERVICE

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ABSTRACT

The use of spread footings over compressible soils is becoming more common for Minnesota Department of Transportation bridges as technologies improve to better predict, mitigate, and evaluate settlement. In August of 2011 the north abutment of a new bridge crossing I-494 was constructed over compressible soils following a soil fill preload, designed to reduce the foundation settlement from several inches to less than one inch, to meet project requirements.

Spread footing foundations are seldom outfitted with instrumentation; adequate performance is frequently assumed based on the decision to use shallow foundations. Here, a monitoring plan was developed to validate the preloading technique for mitigating otherwise unacceptable deformations, assess the efficacy of shallow foundation monitoring methods, and gain a better understanding of shallow foundation behavior with time. Instrumentation consisted of two earth pressure cells, a horizontal MEMS SAA deformation monitoring array, and four optical survey reflectors which were installed during the construction of the foundation and abutment wall.

During the course of construction, portions of the abutment backfill soil volume were placed and removed to accommodate the construction of the bridge deck and the adjacent wall footings. The effect of the various loading and unloading conditions was observed on the sensors. The abutment foundation performance over the construction timeline is discussed, including apparent loading, deflection, and rotation. The data from the manually observed survey targets is compared to the automated data from the SAA and earth pressure cells.

INTRODUCTION

Construction of a new Minnesota Department of Transportation (MnDOT) bridge on spread footing foundations over mixed native deposited and engineered fill soils with moderately good strength and settlement characteristics provided an opportunity to monitor and evaluate shallow foundation performance from the initial phase of construction through early service of the structure. In addition to validating the selection and use of a spread footings design by showing the installation met the governing service limit state requirements, the project provided an opportunity to compare the observed and predicted performance and compare the performance of the structure through two different settlement monitoring systems. The established settlement tolerance of 1 in (25 mm) meant that any system needed to have relatively high precision. The settlement monitoring system specified by the contract was survey-grade monitoring of optical targets placed on the bridge. Large excavations, utility conflicts, adjacent retaining wall construction, contractor equipment staging areas, embankment preloads, and other site constraints made designing a complementary high-precision settlement monitoring program a challenge. A ShapeAccelArray (SAA) in-place deformation monitoring system and two earth pressure cells were placed below the north abutment foundation to measure deformation and pressure below the footing. The SAA system is described in general by Danish et al. (2008) and in use on other MnDOT projects (Dasenbrock et al., 2011).

The overall program was a partnership between the design build contractor, their geotechnical sub-consultant, and the project owner. In addition to successfully comparing two techniques for monitoring small settlements, additional insight was gained on the behavior of the underlying foundation soils during the construction sequence. Although all the foundations for the bridge were placed on spread footings, the overall study and the focus of this paper is on the north abutment foundation and the observations made during its construction.

BACKGROUND

As part of a design build project, a new bridge was constructed in the fall of 2011 carrying Washington Avenue South traffic over Interstate 494 (I-494) in Eden Prairie, Minnesota. Located just to the east of a major interchange reconstruction of U.S. Trunk Highway 169 (T.H. 169) and I-494, the new structure was part of a local road access improvement program to allow for better traffic navigation through the area. Figure 1 shows a visualization of the completed structure, prepared prior to construction. The contractor who won the apparent best value selection for the entire project was a joint venture between C.S. McCrossan and Edward Kraemer & Sons. The original proposal price for the new interchange construction was $125.3 Million.

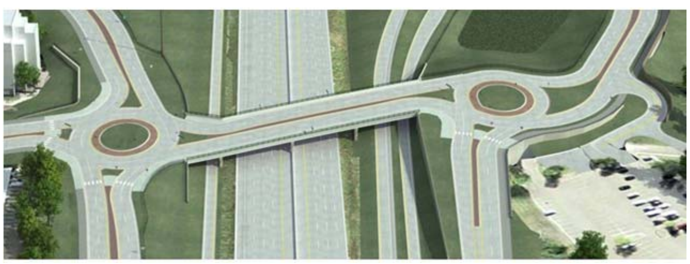

Fig. 1. The north abutment separates the north roundabout (left) from the US 169 south to I-494 east ramp.

The Washington Avenue bridge carries 4 lanes of local traffic across eastbound and westbound I-494 and two freeway access ramps. The bridge consists of two cast-in-place abutments, three cast-in-place piers, and four simple bridge spans. The bridge is on a slight skew and the northernmost span widens to accommodate a roundabout located just north of the north bridge abutment (Figure 2). The bridge has the designation 27R29. The north abutment runs generally east-west in orientation and has a parallel retaining wall on each side. Retaining wall 18 is located immediately west of the north abutment, and retaining wall 20 is on the east side of the abutment. Five standard penetration test (SPT) borings and four Cone Penetration Test (CPT) soundings were completed by consultants for MnDOT near the proposed bridge embankments, abutments and piers in April and June of 2004.

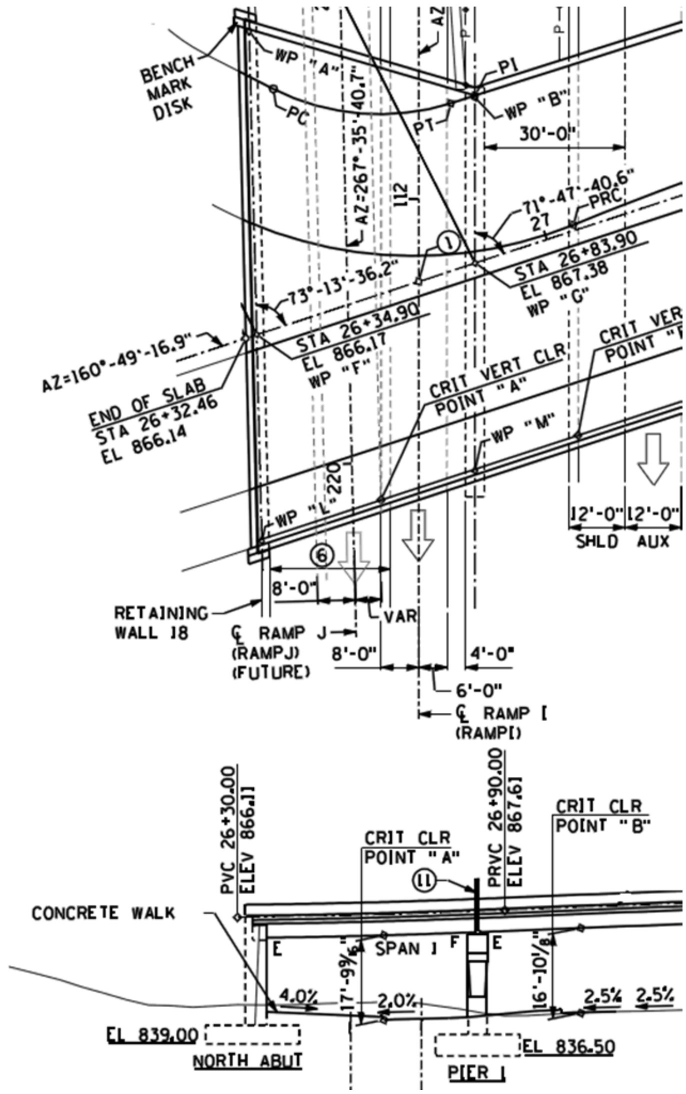

Fig. 2. Plan and profile view showing the general geometry of the north abutment, span 1, and pier 1 areas of BR 27R29.

SPREAD FOOTING DESIGN

The Washington Avenue bridge borings typically encountered remnant pavement sections or topsoil over fill underlain by alluvial and glacially deposited soils. Based on the original investigation the Joint Venture's geotechnical design firm, Braun Intertec Corporation (Braun) suggested that shallow foundations could be used for support of all five foundations to meet the required tolerance of no more than 1 in (25 mm) of settlement. In the winter of 2012, to supplement the original investigation, four SPT borings were advanced by Braun near Pier 1 and Pier 3 of the bridge. Additionally, two pressuremeter (PMT) test borings were advanced, one at each abutment. The soils at the north abutment are described in greater detail in the following section.

Subsurface Characterization at the North Abutment

The north abutment borings encountered a 4-6 in (100-150 mm) layer of slightly organic clay loam, clay loam or loamy sand topsoil over fill soils consisting of clay loam, sandy clay loam, slightly plastic sandy loam silt loam and sand to depths ranging from 4 to 12 ft (1.2 to 3.7 m) below existing grade (elevation 843.0 to 834.5 feet). Below the fill soils, sand, loamy sand, sandy loam and clay loam with layers of silt loam and gravel with sand were encountered to terminations depths of 111, 121 and 53.5 ft (34, 37, and 16 m). SPT N-values within the native sandy soils ranged from 4 to 88, indicating very loose to very dense relative densities. SPT N-values within the native clayey soils ranged from 12 to 37, indicating stiff to hard consistencies. CPT soundings advanced to characterize the site indicated the subgrade was interpreted to be sand, sandy loam, sandy clay loam and clay loam in behavior. The PMT boring performed near the north bridge abutment and embankment indicates the soil conditions are similar to the SPT borings. The PMT boring was sampled at 2.5 ft (0.75 m) intervals with a test plan based on the anticipated footing width of the north abutment. Groundwater elevations were noted on the SPT boring logs between elevations of 830 and 837.5 feet. Pore pressures shown on the CPT logs display the water table approximately between elevations 825 and 835 feet (Braun 2011).

Layers of sandy clay loam, clay loam and clay layers were periodically encountered in the borings at the north embankment. Two thin wall samples were obtained and unconfined compression tests were performed in general accordance with AASHTO T208. The results of those tests yielded undrained shear strengths as shown in Table 1. A summary of the PMT data is shown in Table 2.

Table 1. Summary of undrained shear strengths from Boring B-114 at the north abutment

Boring	Depth of Sample (ft.)	Undrained shear strength, Su, (psf)
B-114	23	1460
B-114	28	2330

Table 2. Summary of pressuremeter testing at the north abutment (Boring PMT-1).

Test Depth (ft)	Geologic Material	MnDOT Classification	Limit Pressure (tsf)	Modulus (tsf)
21.8	Glacial Till	Slpl Sandy Loam	7.0	46.2
29.4	Glacial Till	Sandy Clay Loam	10.6	43.9
49.5	Glacial Outwash	Sand	15.9	70.0

Bearing Capacity

The resistance factors for evaluation of the strength limit state performance limits were based on the Load and Resistance Factor Design (LRFD) code. The Strength Limit State (Bearing) was evaluated using a resistance factor of 0.45, associated with an investigation using SPT methods. Bearing resistance was checked, however, it did not control the design.

Settlement Predictions

Based on 15 ft (4.6 m) of new backfill placed behind the north embankment for the roadway approach, total settlement of 2.0 to 2.5 in (50 to 65 mm) was estimated. Settlement, due to bridge loading only, was estimated to be less than 1 in (25 mm). Settlement associated with the granular soils was anticipated to occur rather quickly as construction progressed. Based on the embankment heights, SPT N-values, unconfined compression tests and moisture contents, approximately 0.25 to 0.50 in (6 to 13 mm) of primary and secondary consolidation was predicted in the layers where the unconfined testing was performed.

Total anticipated foundation settlement was calculated based on three methods. The first method was the Hough method with Boussinesq and Westergaard models using SPT values from the soil borings. The second method was the CPT method or Constrained Modulus method, utilizing the in-place elastic modulus of the soil calculated from CPT data. The third method was the Menard method, based on pressuremeter determinations of soil parameters collected in the field and modified from the SPT values from the soil borings. After these three methods were evaluated, the results were averaged to determine an average service limit state prediction.

The service limit state (settlement) was expected to control the design. Based on the pressuremeter test results, it was recommended that the average service limit state prediction be used for design of the North Abutment. As a maximum settlement of 1 in (25 mm) was specified for the project corridor- a preloading scheme was implemented after performing subcutting to remove soft soils located directly below the proposed footing and prior to footing construction. Excavation depths with corresponding removal elevations are outlined in Table 3.

Table 3. Footing Elevations and Anticipated Soil Removals/Replacements for North Abutment

Soil Borings	Anticipated Bottom of Footing Elevation (ft)	Anticipated Excavation below bottom of footing (ft)	Corresponding Bottom Elevation (ft)
B-113	838	0	838
B-114		10	828
PMT-1		10	828

The design specified that after the removal of the soft in-situ soils, the base of the excavation was to be surface compacted and evaluated by a geotechnical engineer prior to fill placement. If soil corrections extended below the groundwater table, a crushed rock or coarse sand having less than 50 percent of the particles by weight passing a #40 sieve, and less than 5 percent of the particles passing a #200 sieve was to be used to provide a stable excavation base prior to establishing final grades. Groundwater, if encountered, was to be drawn down 2 ft (0.6 m) below the excavation during the work.

PROCEEDING WITH THE DESIGN

The draft foundation recommendation report was submitted to MnDOT for review on January 26, 2011. Comments were returned and the final foundation recommendation report was dated February 2, 2011 and submitted for approval on February 3, 2011. The design of the structure was being prepared concurrently. The north abutment foundation consisted of a reinforced concrete spread footing, 20 ft wide by 116 ft long, and 3.5 ft thick (6 m by 35 m by 1 m). The abutment stem wall was 4.67 ft (1.4 m) wide and constructed to a variable height, roughly 21 ft (6.4 m) at the center.

The final LRFD design for the north abutment called for a factored bearing design pressure of 3.4 ksf (170 kPa) based on a Strength I case 4 load combination. The effective footing width was calculated to be 19.2 ft (5.9 m). The service loading would be expected to be between 2.5 and 3.0 ksf (125 and 150 kPa).

On August 4, 2011 the excavations to remove unsuitable soils from below the footing influence area were performed, see Fig. 1. Based on field observations, some questionable soft native soils were encountered along the eastern half of the footing. These sandy loam soils were excavated to depths where the relative density was judged to be suitable for engineered fill placement. Soils removed from below the foundations were backfilled with Select Granular Modified Sand, with 10% or less passing the #200 sieve (0.075 mm). The field observations also noted three utilities including water, gas, and a Transite (cement-asbestos) pipe at elevations that conflicted with construction of the footing. The utilities would remain in place during the preload and be lowered or abandoned during footing construction so as not to conflict with the footing.

<u>Embankment Preload, Waiting Period, and Settlement Plate Monitoring</u>

The north approach embankment was to consist of Select Granular Borrow or Granular Borrow material. With a grade raise up to 15 ft (4.6 m) at the north abutment, embankment settlement was expected. A soil preload was specified to promote consolidation settlement prior to the construction of the north abutment spread footing foundation. The preload was defined to have a top at the future roadway elevation and a width and length based on a vertical projection of the footing with side slopes extending down at a 1:1 (Horizontal:Vertical) or flatter slope. Excavation for the preload construction, as seen in Figure 3, began in early August of 2011.

It was recommended that an embankment construction waiting period of four weeks be performed, measured from the time the embankment preload had been fully constructed. Settlement would be monitored and that construction could proceed earlier if it was shown that settlement had measurably ceased. The preloading operation during the prescribed waiting period was expected to reduce the majority of the consolidation of the foundation soils prior to the final embankment construction. To monitor and evaluate the rate and magnitude of settlement, three ‘traditional’ settlement plates were specified for placement within the abutment preload areas. The plates consisted of plywood bases with riser pipes affixed to the center of the plates. The plates were installed at the base of the preload backfill and as the backfill height increased, extensions to the riser pipes were added. The plates were surveyed at regular intervals during the waiting period. Settlement plates were allowed for monitoring of temporary embankments and other ground improvement areas. Structural deflections, described later, were monitored using optical target reflectors attached to the structure.

Fig. 3. Excavation of the north abutment area of BR 27R29 prior to placing preload, August 04, 2011.

Based on the construction scheduling and settlement plate data, the north abutment preload lasted approximately one month and construction began after the planned four week waiting period. Monitoring was started using 3 settlement plates on August 5, 2011. Fourteen readings were taken over the next twenty days until the final reading was taken on August 25, 2011. Readings from September 19 through September 25 had remained constant, after observing 2.16 in (55 mm) of movement on the eastern plate (near retaining wall 20), 2.28 in (58 mm) of movement on the center plate (behind the bridge abutment), and 1.74 in (44 mm) of movement on the western plate (near retaining wall 18). The deformation behavior agreed with the original predictions which indicated the material on the eastern side of the foundation was generally poorer in nature.

Following the preload, the north abutment footprint was excavated. On September 9, 2011 the full bottom of the excavation was reviewed and the observations indicated that all the potentially soft or problematic soils had been removed prior to the embankment preload to the satisfaction of the geotechnical engineer. The footing subgrade was compacted, tested, and prepared for footing construction. The excavation was constructed somewhat larger than the proposed footing to provide working space for the foundation formwork. During the construction of the footing formwork, MnDOT began placing performance monitoring instrumentation at the base of the footing.

Contractor's Spread Footing Monitoring Program

Where shallow foundations were used to support bridges, the design build contract, through the use of an Alternative Technical Concept (ATC), established optical targets and periodic survey readings as minimum requirements for monitoring three dimensional movements of spread footing foundations. Prior to the abutment being cast and in order to monitor settlement in the early stages of construction, a target was affixed to a metal post at each of the two front corners of the footing. The footing was designed to be 116 ft (35 m) long and 20 ft (6 m) wide and is oriented roughly east-west for the long axis, see Fig. 2. After the stem was cast and the forms removed, two additional targets were affixed near the top of the east and west ends of the abutment wall. When the new targets were established on the abutment they were immediately used to continue the settlement survey performed by the targets cast in the footing. The targets on the footing were then removed and the base of the footing was covered with soil. The targets were monitored from September 19, 2011 until January 6, 2012.

MnDOT's Spread Footing Monitoring Program

MnDOT, in an effort to gain additional performance data from spread footing supported structures, added an independent instrumentation program at the north abutment. Prior to the rebar being placed in the footing a 40 m (131 ft) ShapeAccelArray (SAA) was installed in a protective conduit just below the foundation. The use of the conduit would allow the potential for the in-place SAA sensor to be removed after the study and repurposed elsewhere. The SAA used here had 0.5 m (1.64 ft) segments, allowing deformation to be monitored at 80 locations along the length of the array. The SAA was positioned such that a fixed reference end was located midway between the footings for the north abutment and Pier 1 (the northernmost bridge pier) and approximately 6 ft (1.75 m) below the ground surface. The array passed into the excavation for the footing through the western toe area and curved in an arc until reaching the center of the foundation where it was positioned to follow the stem on the eastern half of the footing, ending just inside the eastern formwork. The array placement is shown in Fig. 4; the data collection cabinet can be seen just to the right of pier 1 (at right in the photo).

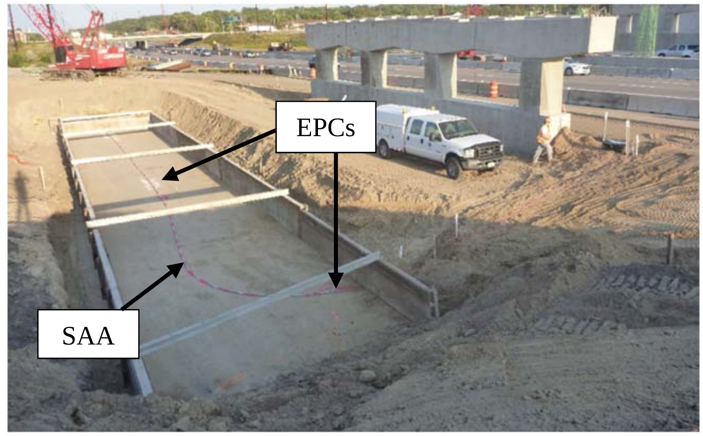

Fig. 4. SAA and EPC sensors were placed at the North Abutment footing. The SAA gray conduit was painted pink and white to help identify/protect it. One EPC was placed near the center of the footing; a second EPC was placed near the edge of the toe of the footing at the SAA exit (September 12, 2011).

Although settlement could have been monitored using a variety of hydraulic systems such as settlement cells and borehole settlement cells, the authors have found these types of monitoring systems to be susceptible to a number of external influences that can easily corrupt their data to a point where the results are no longer useful. These types of systems were not considered appropriate for use at this site.

With the intent of gaining some general insight to the load distribution on footing, two earth pressure cells (EPC) were installed beneath the footing. The loading would be compared with the observed deformations measured by the SAA. One EPC was placed near the center of the footing (centered in both north-south and east-west directions). The second EPC was positioned about 1.5 ft (0.5 m) inside the edge of the toe where the SAA exited the front of the footing in the southwest corner. The two EPC sensors were of the same type and sensitivity and intended to give an idea as to the ratio of pressure distribution at the center and near the toe of the footing and detect any changes during the loading sequence. The EPC sensors were not installed to provide an accurate measure of the footing pressure, although a comparison of the measured value to the predicted footing pressure is described later. A discussion on the accuracy of earth pressure cells and problems associated with their proper calibration is given by Labuz and Wachman 2011.

Construction

Following the soil preload, foundation excavation, and necessary ground improvement actions, the foundation area was compacted and formwork set up for the 116 ft long by 20 ft wide by 3.5 ft thick footing (35 m by 6 m by 1 m). On September 12, 2011 the SAA and EPC sensors were installed the same afternoon the formwork was placed. Reinforcing rebar was placed the following two days. The footing was poured on September 16, 2011 and is shown in Figure 5.

Fig. 5. The footing is cast with an optical target located in the foreground, September 20, 2011.

Over the next two weeks portions of the abutment stem were formed and poured. The final section of the abutment stem was completed by October 1, 2011. The bridge beams were placed on October 6, 2011, as seen in Figure 4. Backfilling took place between October 11 and 12, and the bridge deck was cast during an overnight pour beginning on October 17, 2011. Intermittently, to accommodate the adjacent retaining wall construction, portions of the bridge abutment backfill were removed and replaced, see Figures 7 and 8 and 9.

Fig. 6. North Abutment beams were placed October 12, 2011.

Fig.7. Adjacent retaining wall construction continues adjacent to the North Abutment, October 15, 2011.

Temporary pavement was placed behind the abutment for a number of months and the final approach panel was poured on August 3, 2012, roughly 10 months after construction of the bridge. Refer to Table 4 for a presentation of key construction milestones.

Table 4. Bridge 27R29 Construction Timeline

Activity	Date	Foundation Comments
Excavation and Preload Start	8/5/2011	
Preload Ends; Foundation Preparation	8/25/2011	
SAA/EPC Monitoring Installed	9/12/2011	Monitoring Reference Date
Footing Rebar Placed	9/14/2011	
Footing Poured	9/16/2011	
Electronic Cell/Web Monitoring Started	9/21/2011	
Abutment Stems Poured	9/24/2011 to 10/1/2011	Day 12
Beams Placed	10/4/2011	Day 22
Abutment Backfilling	10/11/2011 to 10/12/2011	Day 29
Decking	10/17/2011 to 10/18/2011	Day 34
Some Backfill Removed	11/06/2011	Approximate Day 55
Backfill Restored	11/22/2011	Approximate Day 70
SAA/EPC Monitoring Removed	6/05/2012	266 days of monitoring
North Approach Panel Pour	8/03/2012	

Fig.8. Adjacent retaining wall construction occasionally required the removal of some of the bridge backfill as seen in this photo from November 11, 2011.

Optical Target Monitoring

A total of twenty three survey shots were taken during the construction monitoring period by the contractor to monitor settlement for quality control. The resolution of the survey readings was 0.01 ft (3 mm); some fluctuation was observed where elevations appeared to increase during periods where a decrease would be expected. The movements did not appear to be consistent between the measurements of the east and west targets, although this was likely due to the precision and resolution issues more than any differences in movement. The overall optical target precision appeared to be about 0.01 ft (3 mm) with an accuracy within 0.02 ft (6 mm). The survey reading results from the north abutment targets are included as Table 5. The first 9 readings were taken based on the reflectors mounted above the footing while abutment stem construction proceeded. When the stem was cast and the formwork was removed, readings were then taken based on the stem targets. Total measured deformation of the North Abutment, based on the optical survey data, was between 0.02 and 0.03 feet (3 mm to 4.5 mm), well within the project specifications of 1inch (25.4 mm).

Table 5. North Abutment Survey Target Readings*

DATE	Location	West Target Elevation (ft)	East Target Elevation (ft)
9/19/2011	Footing	842.53	842.45
9/20/2011	Footing	842.53	842.46
9/21/2011	Footing	842.51	842.44
9/22/2011	Footing	842.52	842.45
9/27/2011	Footing	842.53	842.45
9/28/2011	Footing	842.53	842.45
9/29/2011	Footing	842.52	842.45
10/1/2011	Footing	842.52	842.45
10/3/2011	Footing	842.53	842.44
10/6/2011	Stem	859.40	859.37
10/11/2011	Stem	859.41	859.37
10/14/2011	Stem	859.42	859.38
10/18/2011	Stem	859.40	859.35
10/28/2011	Stem	859.41	859.37
11/4/2011	Stem	859.40	859.35
11/17/2011	Stem	859.39	859.34
11/23/2011	Stem	859.38	859.35
12/2/2011	Stem	859.40	859.35
12/9/2011	Stem	859.38	859.35
12/16/2011	Stem	859.38	859.34
12/23/2011	Stem	859.39	859.35
12/30/2011	Stem	859.39	859.36
1/6/2012	Stem	859.38	859.35

Fig.9. Adjacent retaining walls construction is substantially completed at the North Abutment, November 22, 2011.

ESTIMATING BRIDGE LOADS AND COMPARING LOADING WITH EARTH PRESSURE CELL RESPONSE

Based on the unit weights of the construction materials and the bridge geometry, estimates of the contributory bridge loads were made. The total weight was estimated at 6500 kips (30 MN). With a bearing area of the footing of 2320 square feet (215 square meters), a rough estimated loading was calculated as 2.8 kips per foot (neglecting additional active pressure soil loading, guardrail, railing, the approach panel, and other items) which agrees well with the estimated service loadings of 2.5 to 3.0 kips per foot (120 kPa to 145 kPa).

Both EPC sensors appeared to show consistent loading behavior, reading up to about 500 psf (25 kPa) for about two weeks. After this time there were some spikes observed on the EPC at the toe of the footing, which are believed to be caused by the construction of the stem, which was offset slightly toward the toe, as shown in the abutment diagram, Figure 13. Casting and placing these elements probably induced slight outward rotations, as shown in the early EPC data plotted in Fig. 10. About 0.01 to 0.02 in (0.25 to 0.50 mm) of movement is observed on the SAA at this time.

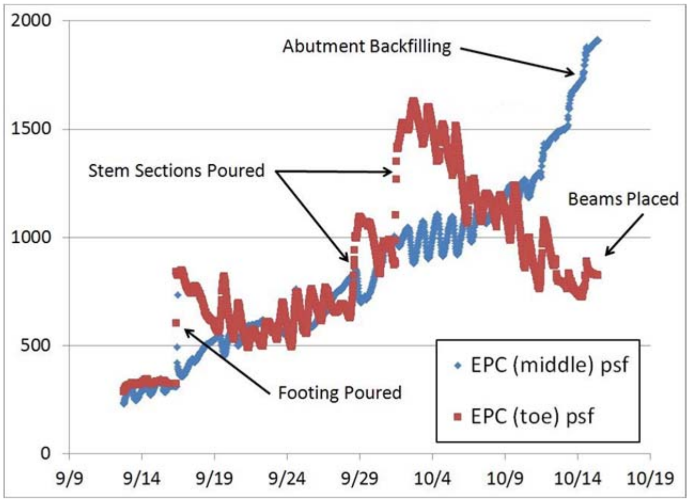

Fig. 10. Early EPC data showing trends in movement at the center of the foundation (blue) and the toe (red).

Coinciding with the placement of the bridge beams and embankment backfill operation, the next major increase in pressure occurs in mid-October, at about day 30 in the construction timeline. At this time a change of about 0.08 to 0.12 in (2 to 3 mm) of downward deflection is observed by the SAA sensor. The largest changes in pressure, and deformation, were observed during this period. Unfortunately, for the monitoring program, the contractor was beginning to place backfill on the footing heel at the same time the bridge beams were set, making it difficult to resolve the cause of the observed pressure responses during this period. It is proposed that there would be an overall deformation and pressure increase but the weight of the beams may cause some forward rotation of the stem while the backfill would cause backward rotation. The change in EPC pressures is shown in Fig. 11 and the corresponding SAA deformation is shown in Fig. 12.

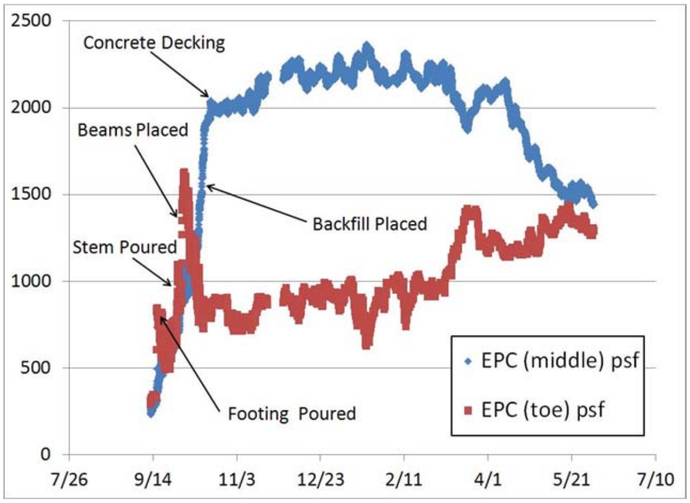

Fig. 11. The full EPC dataset shows differing trends in movement observed at the center of the foundation (blue) and near the foundation edge at the toe (red).The most significant difference in behavior is observed during, and after, the backfill placement.

As beams were set and backfill was placed, the pressure on the middle EPC continued to increase, but the pressure on the toe EPC changed from increasing to decreasing. After backfilling was substantially complete, the EPC placed at the toe of the footing had a steady-state reading of about 750 psf (36 kPa), while the EPC under the abutment stem showed a loading of 2250 psf (110 kPa). The results appear to indicate some possible amount of backward rotation into the fill. As time progressed, from 175 to 250 days during the monitoring period, the earth pressure readings began to become more uniform with both sensors having a pressure reading of about 1000 psf (50 kPa) at the end of the monitoring period at 266 days.

The effective footing width was given in the design plans as 19.2 feet; the plan width of the footing is 20 feet, so the loading eccentricity was very small. This appears to correlate well with the pressures observed at the end of the monitoring program where they were similar at about 1500 psf (72 kPa).

As the EPC sensors were not field calibrated at the site, the measured values are not expected to accurately represent the actual in-situ pressures below the foundation. However, as the sensors were installed similarly, it is anticipated that their relative behavior will be similar and the pressure ratios may be compared, as presented in Fig. 10 and Fig. 11.

For comparison of the measured pressures and deformations associated with loading events, a table of component weights was developed. Table 6 shows the estimated weight of each of the major structural components and the percentage of the overall loading that each represents. The soil backfill (as seen by the EPC response in Fig. 12, makes up the single largest contribution at about 40% of the foundation load. This is of particular interest for projects where the "construction point concept" to assess only settlement that may be problematic for a bridge superstructure could be employed, as described in the FHWA document "Selection of Spread Footings on soils to Support Highway Bridge Structures." The timing of the pressure and deformation responses seen in each sensor type is in excellent agreement with the observed loading events. The trends of observed increases in earth pressure to the trends of increases in the magnitude of deformation also appear to be in in very good agreement.

Table 6. Estimates of Loading based on Materials and Geometry.

Loading	Volume ft^3	Material/ (assumed unit weight, pcf)	Estimated Weight (kips)	% of Total Foundation Load
Footing	8120	Concrete (150)	864	13
Footing Cover (toe)	5760	Soil (120)	691	10.6
Stem	10850	Concrete (150)	1624	25
Parapet	637	Concrete (150)	96	1.5
Beams †		Reinforced concrete 613.5 lb/lineal ft.	153	2.6
Deck*‡ (Trapez oidal)	2983	Concrete (150)	448	6.9
Soil Backfill (heel)	21840	Soil (120)	2621	40.4

*Beams and deck were assumed to be on a simple span with ½ the load applied to the north abutment.
† 10 beams, each about 50 feet long at 613.5 lbs/lineal foot
‡ Deck was assumed as 50 feet long, 8 inches deep, with widths of 75 feet at pier 1 and 104 feet at the north abutment.

As discussed in Samtani and Mertz (2010), Sargand et al. (1999) and Sargand and Masada (2006) reported measurements of contact pressure and settlement for highway bridges in Ohio and found variations in footing pressure and similarly small deformations which occurred coincident with each loading event, consistent with the observations in this study.

ShapeAccelArray (SAA) Data

The SAA system acquired data 4 times daily through the reporting period. The majority of observed movement was seen during the backfilling period. Smaller movements were observed when the stem was cast, beams set, and when the bridge deck was poured. The precision of the instrument, approximately 1.5 mm (0.06 in) (Danish 2010), also allowed very small deformations to be effectively monitored for this study. All of the major movements were seen in the first 60 days during the construction period. After the initial construction period, deformations appeared to be limited to less than 0.1 in (2 mm). After the bridge abutment was substantially complete, the deformation behavior appeared very stable during the construction timeline period from about 180 days to 266 days. Although continued monitoring for long term behavior would have been of interest, SAA and EPC monitoring was discontinued after 266 days when the SAA sensor was exhumed for potential re-use on another portion of the project. The accompanying data collection earth station was also removed for re-use elsewhere.

The SAA data showed good correlation between the loading events and observed deformations. A plot of the SAA data can be seen in Fig. 12. Less settlement was observed by the SAA close to the toe edge of the foundation (the top, purple trace, in Fig. 12.) Overall, the SAA behavior was generally very regular, as seen by the behavior of the nodes relatively distant from the free edge of the foundation (all colors except the purple trace, at top, in Figure 12.)
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Deformation Monitoring

The majority of the deformation measured at the BR 27R29 site was measured on the 3 settlement plates during the 20 day soil preload, before any structural construction began. The majority of the movement there (1.5-2.25 in or 38-57 mm) was observed in the first 5 days of the preloading. The observed deformation responses appeared to have occurred either instantaneously or very fast, consistent with expectations for the mostly sandy soil site behavior.

Based on the rough bridge loading model (developed using plan geometry and estimated unit weights as presented in Table 6), about 13% of the deformation can be expected from the weight of the footing, 10% from the backfill over the toe, 25% from the weight of the stem, and 1.5% from the weight of the parapet. A diagram showing these components, totaling nearly 50% of the structural weight, is included as Fig. 13. It is important to note if survey targets or other monitoring had only been completed after the stem was cast and the formwork removed as much as 50% of the immediate and short-term settlement (attributed to the structural foundation) could have been expected to have already occurred. Not that here, targets were set on the footing, but the observed deformation was small, most likely due to the success of the preload.

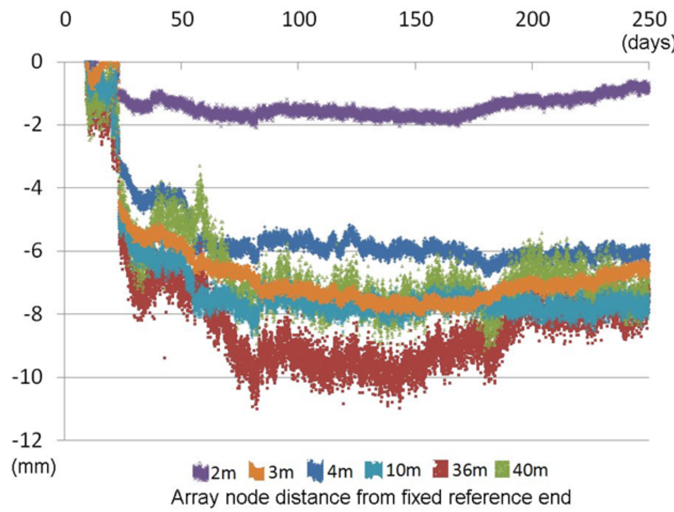

Fig. 12. Settlement of the ShapeAccelArray at the North Abutment plotted for 6 selected nodes (of 80) along the length of the 132 ft. (40 m) array.

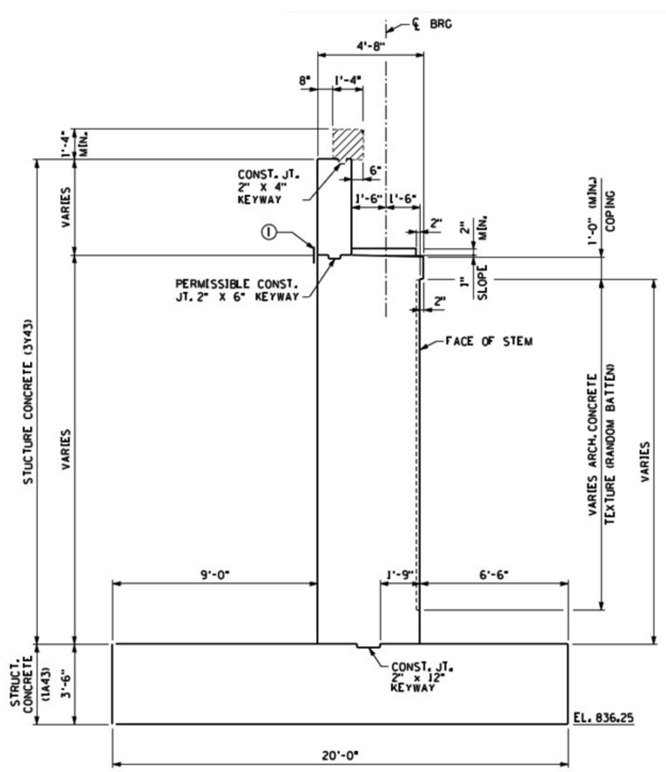

Fig. 13. Diagram of the North Abutment structure, showing the geometry and relative size of the footing and stem.

Another 40% of the loading may be attributable to the soil backfill, leaving only 10% of the loading attributable to the beams and the deck. This agrees well with the overall observations that showed the majority of the observed deformation was observed when the footing and stem were cast and the soil backfill placed. Relatively little deformation was seen when the beams were placed and the concrete deck was poured.

As seen in Fig. 12, there are some small differences observed in the overall deformation along the SAA as it enters the footing from the toe edge and arcs into a line below the abutment wall. This appears to suggest the footing has some small amount of flexibility and differences in deformation appear to be up to 0.16 in (4 mm) across the length of the array placed below the foundation. As the array was laid on the top of the well compacted foundation material in a circular conduit, it is believed to have been cast intimately with the base of the footing. Considering the size of the footing in relation to the observed deformations, it seems reasonable to consider it a rigid body.

Figure 12 also shows that the maximum downward deformations are also observed between the period of 75 and 175 days after footing preparation; after this time there appears to be some measured upward deformation. This can also be seen in Figure 14 which depicts all the nodes along the SAA instrument at 3 different points in time, December 2011, April 2012, and June 2012. Figure 14 shows that there appears to be slightly more time dependent movement along the array embedded further into the foundation, and that the movement is curiously upward with respect to the earlier datasets. The cause of this is not well understood, but potentially involves some settlement of the reference end over time, temperature effects, or possibly some rebound in the soil below the foundation due to time dependent effects. The overall shape of the array appears similar and vertically offset, so tilting of the array reference end appears less likely. Additionally, the readings from the reference end to about node 25 are similar and they appear to be further apart for the portions of the array that are directly under the stem. The error is on the order of the sensitivity of the array, so it is not judged to be problematic or significant with respect to the overall findings.

Total foundation deformations measured by the SAA appeared to be between 0.2 to 0.3 in (6 to 8 mm). These movements were measured after the footing pour and may not represent additional small early deflections; these deformations may have been masked by soil compaction performed by the contractor during their construction operations near the reference end of the SAA. Roughly 3 days of SAA data, where no abutment construction was performed, was edited for clarity as the data appeared to be highly scattered and erratic. The EPC data presented is complete.

The optical targets provided useful data to assess that the footings were not settling problematically or outside project tolerances. Due to the small movements and the fact that precision and accuracy were about the same order of magnitude, it was difficult to confirm the rate of settlement or any meaningful trends with respect to load-deformation relationships. Although better than traditional settlement plates, the precision of the optical target system did not allow for resolution of very small movements. The system does appear useful to confirm large movements are not occurring or for monitoring larger movements, such as those experienced by the settlement plates during the preload monitoring period. Optical systems with enhanced precision using fixed manual or robotic total stations and reflective prisms appear to be more appropriate for monitoring very small deformations. Concurrently, as long as the SAA instrument can be located where the reference end is fixed and the entire length of the array is not subject to damage or conflict with other contractor operations, the SAA system appears to capture movements on the order of millimeters with more resolution and overall precision and accuracy.

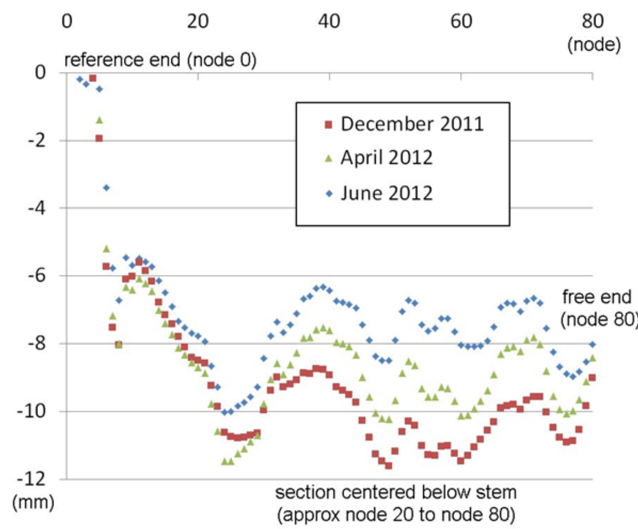

Fig. 14. Movement of all ShapedAccelArray (SAA) modes with time. Much of the deformation occurred when backfill loads were initially placed, although there were also some subtle changes over time.

CONCLUSIONS

Deformation measured by reading the optical targets was about 0.1 ft (2.5 mm) to 0.2 ft (5.0 mm). The movements were difficult to interpret as the total settlement was small and the resolution was 0.1 ft (2.5 mm). The SAA data appeared to indicate (after adjusting for early errors) that total deformations ranged between a minimum of 0.25 in (6 mm) and a maximum of 0.4 in (10 mm). The SAA system provided a good check on the data integrity of the optical targets.

The earth pressure cells appeared to perform well to assess the relative motion of the bridge abutment foundation. As anticipated, when the earth backfill was placed behind the wall to build the approach embankment, the wall appeared to rotate slightly backward into the fill. This movement relieved some of the pressure at the toe of the footing and contributed to larger settlements along the far portions of the SAA (located at the center of the footing, below the stem). There was excellent agreement among the observed times associated with the loading events, and observed pressure and deformation responses in the EPC and SAA sensors. Although the overall trends agreed well with predicted behavior, there was less agreement with respect to the magnitudes of pressure and movement between the sensors.

The SAA system appeared to provide good quality, stable, data. Due in part to the high sampling rate and considerable number of data points, the SAA data was easier to interpret than the optical target datasets. The SAA data showed good correlation between the loading events and observed deformations. The precision of the instrument also allowed very small deformations to be effectively monitored for this study. As compared to traditional horizontal traversing probe systems, system automation also provided a significant amount of cost savings.

The deformations observed were well within the project tolerances. The soil preloading appears to have been very effective in reducing the observed settlement from as much as several inches (~50 mm), observed during the preloading phase, to within 0.25 in (6 mm), observed during the final foundation construction phase.

The monitoring program was successful in showing the foundation performance was well within project tolerances and the use of spread footings combined with the ground improvement plan met the project need for serviceability without the extra cost associated with deep foundation systems. Given that the majority of the loading appears to be associated with "early" loading from the footing, stem, parapet, and soil, it seems reasonable that shallow foundations could be employed even at more marginal sites- particularly those where the settlement is immediate in nature. If a well-designed monitoring program is employed and some accommodations for final adjustments of the parapet and beam seats allow for larger settlement to be accommodated prior to placing girders and the deck, perhaps even sites with relatively large deformations could incorporate spread footing foundations in their design. The referenced adjustments to the parapet and beam seats are generally considered more critical due to the bolting of the diaphragms and other stress considerations- but these items were found to contribute relatively little to the overall structural dead load. Although settlement tolerances for spread footing foundations may be able to be revised, based on a magnitude observed after settlement-intolerant structures are placed, this deformation may not be practical due to added risk, or the cost and complexity of monitoring programs with greater resolution, accuracy, precision, and redundancy.

Monitoring the shallow foundations with optical targets was cost effective and provided useful data to assess that the footing(s) were not settling problematically or to an extent that was outside project tolerances. However, in environments where deformations are small or the project is critical in nature, systems with high precision such as millimeter (0.05 in) precision total station systems or ShapeAccelArray systems should be considered to ensure high quality data is captured at a sufficient resolution to meet project needs.

Based on the success of this monitoring program, similar shallow foundation performance monitoring is recommended as part of regular quality control and assurance programs and to help assess current design methods and LRFD resistance factors associated with shallow foundation construction.

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