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Estimated and Measured Settlements of Shallow Foundation Supporting Bridge Substructure

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ABSTRACT

Shallow foundations have numerous advantages when compared to deep foundations; primarily: low cost, fast construction and being environment friendly. Shallow foundations on soils are underutilized to support highway bridge substructures. This is due to a limited performance data and overestimation of settlements. Despite the success of previous shallow foundation studies, more research is needed to evaluate the performance of shallow foundations as a highway bridge foundation. To study the field performance of shallow foundations on soil, the central pier footing was instrumented and monitored during the construction of a two-span highway bridge in Columbus, Ohio. A-2-4 and A-3a soil types were encountered in a borehole. The field instrumentations consisted of multiple sensors and stations for recording contact pressure under the footing, settlement of the footing and tilting of pier columns tied to the footing. A USGS quality benchmark was incorporated to establish a solid permanent benchmark at the site. The field performance data collected in the study provided further insight into how contact pressure, footing settlement and column/wall tilting were correlated with each other throughout various construction stages. The study also produced outcome on general reliability of the settlement prediction methods outlined in the AASHTO LRFD design specifications and provided enhancement to the elastic half-space method.

KEYWORDS: Shallow foundations, Settlement, Instrumentation, Bridge, Tilting.

INTRODUCTION

Cost of shallow foundations is typically 30% to 80% less compared to the cost of deep foundations (Amar et al., 1984), and about 50% of the bridge construction cost is in the foundation (Briaud, 1997). However, shallow foundations have not been often utilized for supporting highway bridge structures. This is because bridge engineers generally believe that bridges supported on shallow foundations tend to develop settlement/scour problems and become high maintenance items in their infrastructure inventory. In order to dispel this common negative notion regarding shallow foundations among bridge engineers, their satisfactory field performance must be demonstrated through well-planned and documented field case studies.

The authors have been conducting shallow foundation studies in Ohio for over fifteen years. Outcome of their previous research efforts on this

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subject was presented in journal publications (Sargand et al., 1999, 2003). The latest study, which was completed a few years ago for the Ohio Department of Transportation (ODOT) and the Federal Highway Administration (FHWA), had the following major objectives:

- instrument and monitor field performance of spread footing foundations at two new highway bridge construction sites in Ohio; and
- evaluate the reliability of the spread footing performance prediction methods outlined in the AASHTO LRFD bridge design specifications.

PREVIOUS STUDIES

Bozozuk (1978) examined the 1975 survey data obtained by Transportation Research Board (TRB) Committee A2K03. He noticed that in some cases nearly equal magnitude movements took place among the bridges supported by spread footings and by pile foundations. A plot of the survey data produced vertical and horizontal tolerable movements of 3.9 inches (99 mm) and 2 inches (51 mm), respectively.

Walkinshaw (1978) reviewed the data for thirty five bridges supported by spread footings in the western states. He noted a poor riding quality which resulted when vertical movement exceeded 2.5 inches (64 mm).

Keene (1978) studied case histories of spread footing use in Connecticut and noted some cases in which post-construction settlement of as much as 3 inches (76 mm) occurred with no damage to the bridges. He stressed the importance of staged construction practices to minimize post-construction settlement.

DiMillio (1982) surveyed the conditions of 148 bridges supported by spread footings on compacted fills in Washington. All were in good conditions, posing no safety or functional problems. He found that the bridges could tolerate easily differential settlements of 1 to 3 inches (25 to 76 mm). He estimated that spread footings were 50% to 60% less expensive than pile foundations.

Moulton et al. (1982) reviewed the data for 204 bridges in West Virginia, which experienced movements and damages in some cases. They saw that the average vertical and horizontal movements were at least 4 inches (102 mm) and 2.5 inches (64 mm), respectively, among the cases, regardless of the foundation type. This finding dispels the common notion that shallow foundations are more prone to settlement than deep foundations.

LATEST FIELD CASE STUDY

Project/Site Descriptions

As part of the latest study, one spread footing foundation was instrumented with sensors and monitored throughout construction stages at a major highway bridge construction site in Ohio. At this site, a two-span bridge structure FRA-670-0380 was erected to allow crossing of High Street over Interstate Highway I-670 in Columbus, Franklin County, Ohio. Table 1 summarizes basic specifications of the bridge.

Subsurface exploration work was conducted at the site, using a truck-mounted rig equipped with a safety hammer. A-2-4 and A-3a soil types were encountered in a borehole. Table 2 summarizes the soil boring log data. Neither groundwater table nor bedrock was encountered in the borehole.

Field Instrumentation Plans

Field instrumentation plans were developed for the FRA-670-0380 site to monitor the performance of the central pier foundation. Figure 1 illustrates the overall field instrumentation schemes. The types of instrumentation included Geokon Model 4800E soil pressure cells, settlement monitoring points encased in riser pipes and column tilt stations (based on a tiltometer by SINCO). The following sections describe each type of field instrumentations.

Instrumentations for Contact Pressure

Earth pressure cells (Model 4800E) were purchased from Geokon, Inc. (New Lebanon, NH) and had a full scale of 689 kPa with a resolution of 689 Pa. Each pressure cell was attached to a small concrete block prior to the footing construction and placed sensitive face down against a sand bedding layer within the footing construction area. This arrangement was necessary to keep the pressure cell from being disturbed during concreting of the footing, to ensure that the pressure cells become integrated into the shallow foundation structure and to minimize undesirable bridging effects on their pressure readings. Obviously, all the pressure cells were located over the bottom surface of the footing. A minimum of five (5) sensors was needed to gain insight into contact pressure distribution changes during construction.

Following any site visit, soil pressure was computed from the field readings coming from each cell using the following formula:

$$P = G(R_0 - R_i) + K\,(T_i - T_0); \qquad (1)$$

where P = soil pressure (kPa); G = pressure calibration constant (kPa/digit); R_0, R_i = initial, subsequent transducer frequency reading (digit); K = temperature calibration constant (kPa/°C rise); and T_0, T_i = initial, subsequent transducer temperature (°C).

Table 1. Basic Information for Highway Bridge FRA-670-0380

Bridge Spans	North Span: 31.4 m South Span: 30.5 m
Bridge Deck Width	23.77 m
Footing Width (B) & Length (L)	2.44 m & 12.19 m
Footing Thickness	0.91 m
Embedment Depth	1.52 m
Max. Design Bearing Pressure	192 kPa

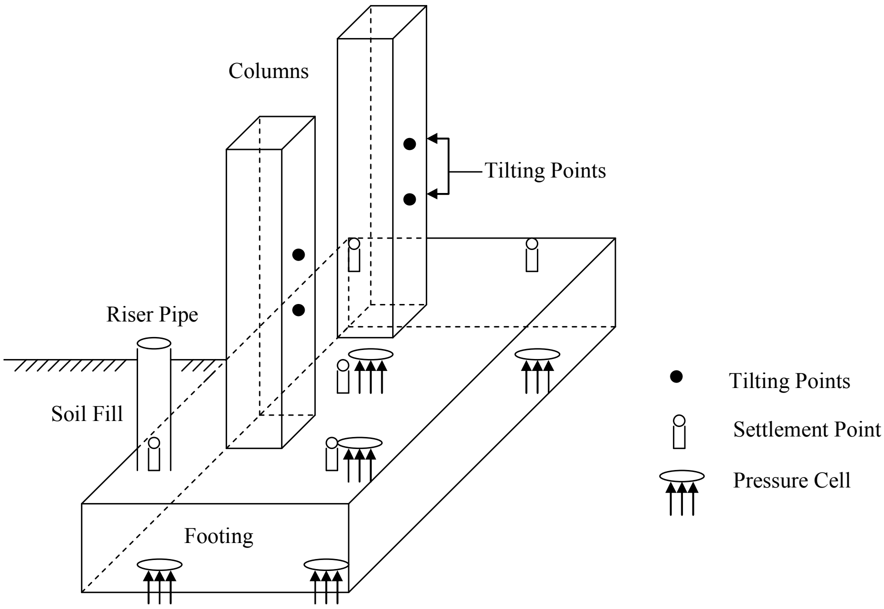

Figure 1: Overall Field Instrumentation Scheme for Spread Footing

Table 2. Subsurface Exploration Data

Depth Below Bottom of Footing (ft)	Soil Descriptions	Ave. SPT-N (uncorrected)
0	Gray sandy silt w/ trace gravel (A-4a)	51
5.0 (1.5 m)	(same as above) (A-4a)	65
10.0 (3.0 m)	(same as above) (A-4a)	73
15.0 (4.6 m)	Gray gravel, some sand and little silt (A-2-4)	63
20.0 (6.1 m)	(same as above) (A-2-4)	84
25.0 (7.6 m)	(same as above) (A-2-4)	70

Instrumentation for Settlement

The footing received a total of five settlement monitoring points installed over the top surface of the footing to detect both average and differential settlement movements. The locations of these points matched the locations of earth pressure cells underneath the footing for developing correlations between the soil pressure and settlement data. A 152-mm diameter PVC pipe coupler was embedded into concrete at each point, so that a solid PVC riser pipe could be installed to provide a continued access to the point even after backfilling. During each visit, conventional optical surveying technique was employed to measure elevations of the points with respect to a USGS quality benchmark established at the site.

Instrumentation for Pier Wall/Column Rotation

A tilt measurement station was set up on the east and west footing columns. An accelerometer-based tilt-meter from Slope Indicator, Inc. (Seattle, WA) was used at each tilt station to obtain tilt measurements taking place along the direction of the footing width. In any field visit, the wall/column tilting angle θ was computed using the field data through:

$$\theta = \sin^{-1}\left[\frac{(+R)-(-R)}{2}\right]; \qquad (2)$$

where θ = angle of tilt (from true vertical; in degrees); +R = positive side accelerometer reading (digit); and –R = negative side accelerometer reading (digit).

Construction History

Table 3 summarizes the construction history data. It took a total of 151 days for the bridge to be officially opened to general traffic.

Contact Pressure and Footing Settlement

A visit to the site was made to collect all sensor/instrument readings, typically one week from the time the footing underwent each major construction stage. Table 4 summarizes the average field measured contact pressure values at different stages of construction for the spread footing. Figure 2 shows the variations of the average contact pressure with time at different construction stages. The contact pressure distribution started out relatively uniform during the early stages of construction but became progressively more non-uniform after Stage 4 (backfilling). The soil pressure mound developed a definite tilt from the north to the south. Stage 7 (bridge deck construction) induced the largest increase in the contact pressure, followed by Stage 6 (placement of beams).

Figure 3 presents three-dimensional views of the contact pressure distribution over the footing bottom face. This plot was made by locking the southwest corner of the footing at the coordinate origin and rotating the footing in 90° increments to show the field measurements from various angles. The contact pressure distribution started out relatively uniform during the early stages of construction but became progressively more non-uniform after Stage 4 (backfilling). The soil pressure mound developed a definite tilt from the north to the south. Stage 7 (bridge deck construction) induced the largest increase in the contact pressure, followed by Stage 6 (placement of beams).

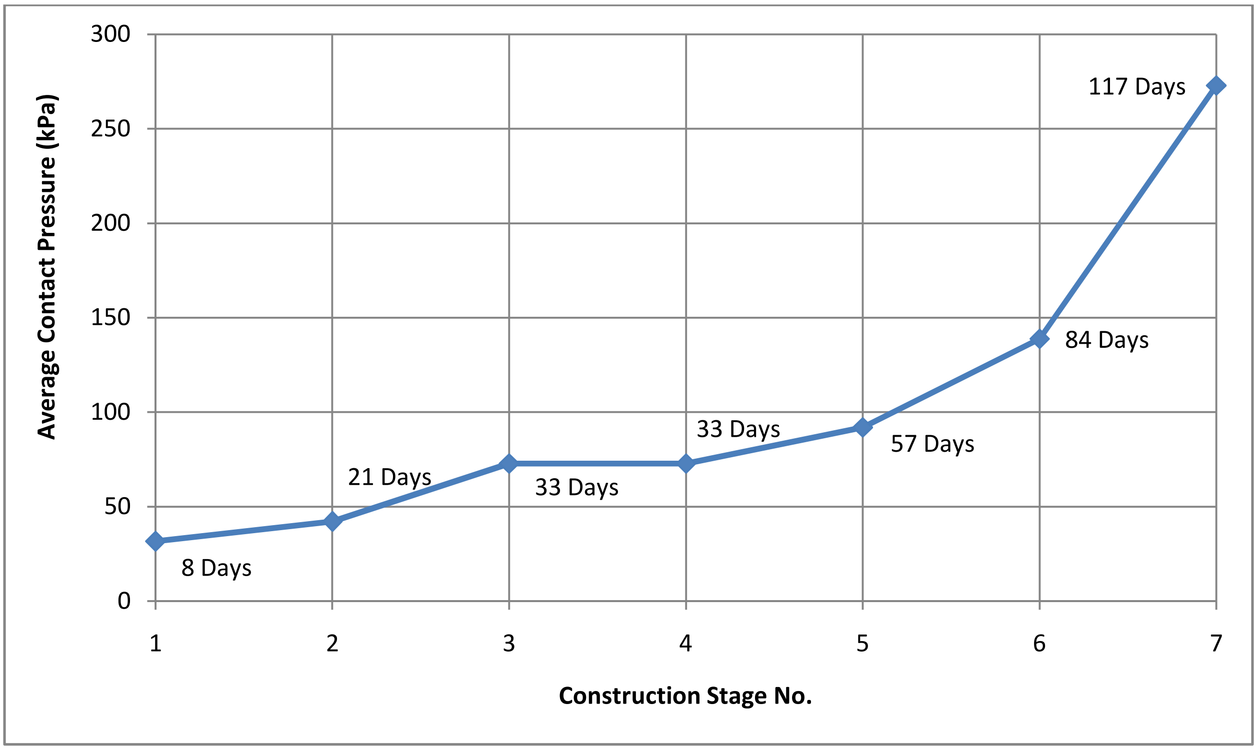

Figure 2: Variations of Average Contact Pressure with Time at Different Construction Stages

Table 3. Construction History Data

Stage	Stage Description	Days	Notes
0	Excavation of footing const. area	0	---
1	Concreting of footing	8	57 m^3 of concrete poured
2, 4	Concreting of (pier columns + cap)	21	24 m^3 of concrete poured
3	Backfilling over footing	33	Soil cover thickness 0.6 m
5	Barrier wall	57	7.5 m^3 of concrete poured
6	Girder & cross beam placement	84	181,820 kg of steel
7	Bridge deck construction	117	204 m^3 of concrete poured

Table 4. Summary of Field Measurements of Contact Pressure

Stage No.	Stage Description	Average Contact Pressure (kPa)	
		Cumulative	Increase
1	Footing construction	31.6 kPa	31.6 kPa
2	Pier column construction	42.1 kPa	10.5 kPa
3	Backfilling over footing	72.8 kPa	30.6 kPa
4	Pier cap construction		
5	Barrier wall construction	91.9 kPa	19.2 kPa
6	Placement of girder & cross beams	138.8 kPa	46.9 kPa
7	Bridge deck construction	272.9 kPa	134.1 kPa
8	Bridge open to traffic	272.9 kPa	0.00

Table 5. Summary of Field Measurements of Footing Settlement

Stage No.	Stage Description	Settlement (mm)	
		Cumulative	Increase
2	Pier column construction	1.5	1.5
3	Backfilling over footing	2.3	0.8
4	Pier cap construction		
5	Barrier wall construction	1.3	-1.0
6	Placement of girder & cross beams	2.8	1.5
7	Bridge deck construction	5.1	2.3

Table 5 summarizes the average footing settlement values detected at different stages of construction. Figure 4 shows the variations of the average settlement with time at different construction stages. The maximum average settlement was measured to be only 5.1 mm.

Figure 5 presents three-dimensional views of the footing settlement behavior. Examination of this plot shows that the footing tilted increasingly more downward toward the north side as the construction progressed. The maximum degree of tilt was close to 0.4%. Based on Table 5, Stage 7 (bridge deck construction) produced the largest increase in settlement, followed by Stage 2 (pier column construction) and Stage 6 (placement of beams).

During stage 5 (barrier wall construction), the central pier footing was tilted somewhat while settling under load. Because of this behavior, two of the settlement points moved upward. The overall average settlement value is zero when the movements of all five points are used. The average value becomes 1.3 mm if upward movements are not included in the computation. Another reason for the small amount of settlement is that this construction stage did not apply as much loading as other stages did.

Figure 6 shows variations of average contact pressure with average settlement at different construction stages. The trend is almost linear after construction stage 5. However, before this stage, the data didn't show any trend because of the tilting effect on the settlement.

Column Tilting

Two spans of unequal lengths met at the central pier footing at the FRA-670-0380 site. The north span was 31.4 m and the south span was 30.5 m. This condition generated a net overturning moment of 2,660 kN-m. By selecting Poisson's ratio of 0.3 and a soil modulus ranging from 33 to 56 MPa, the titling angle was estimated to vary between 0.16 and 0.28 degree. This range complied well with the field measurements, which showed that both of the monitored columns inclined slightly (0.13° for east column, 0.24° for west column) toward the north.

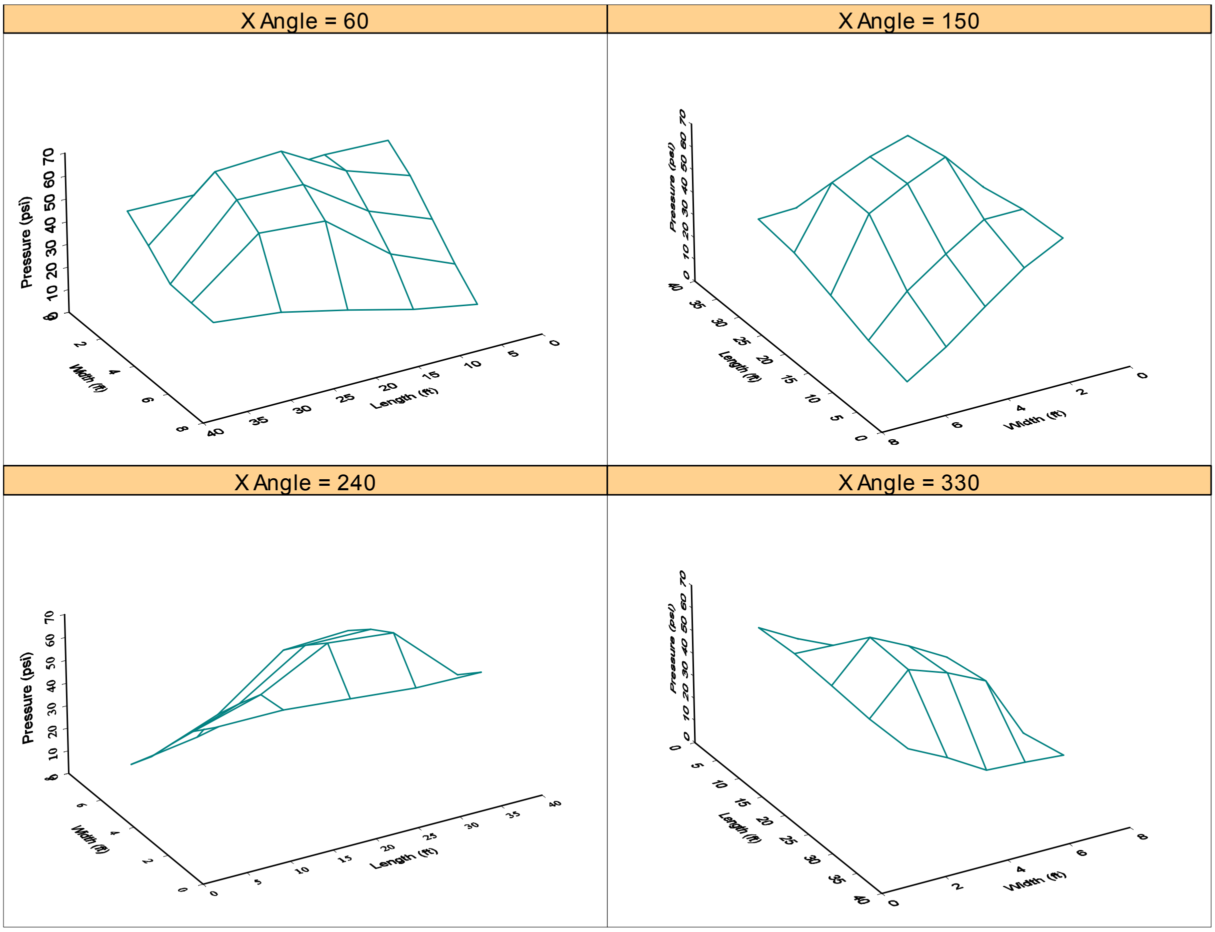

Figure 3: Contact Pressure Distribution (Stage 8: Bridge Open to Live Loads)

Correlations between Field Performance Data

In general, the settlement data and column tilting data agreed with each other. Both the footing and columns tilted slightly toward the north, which indicates that the footing and columns acted as one rigid body. This tilting behavior also suggests higher contact pressure development on the north side. However, soil pressure measurements taken in the field were not compatible with the other performance measurements. This outcome may imply that settlement and column tilting data normally reflect the global behavior of the footing structure while each soil pressure cell reading tends to be influenced not only by the global rigid body movement/rotation of the footing under the applied load, but also by the stiffness of the bearing soil that exists underneath.

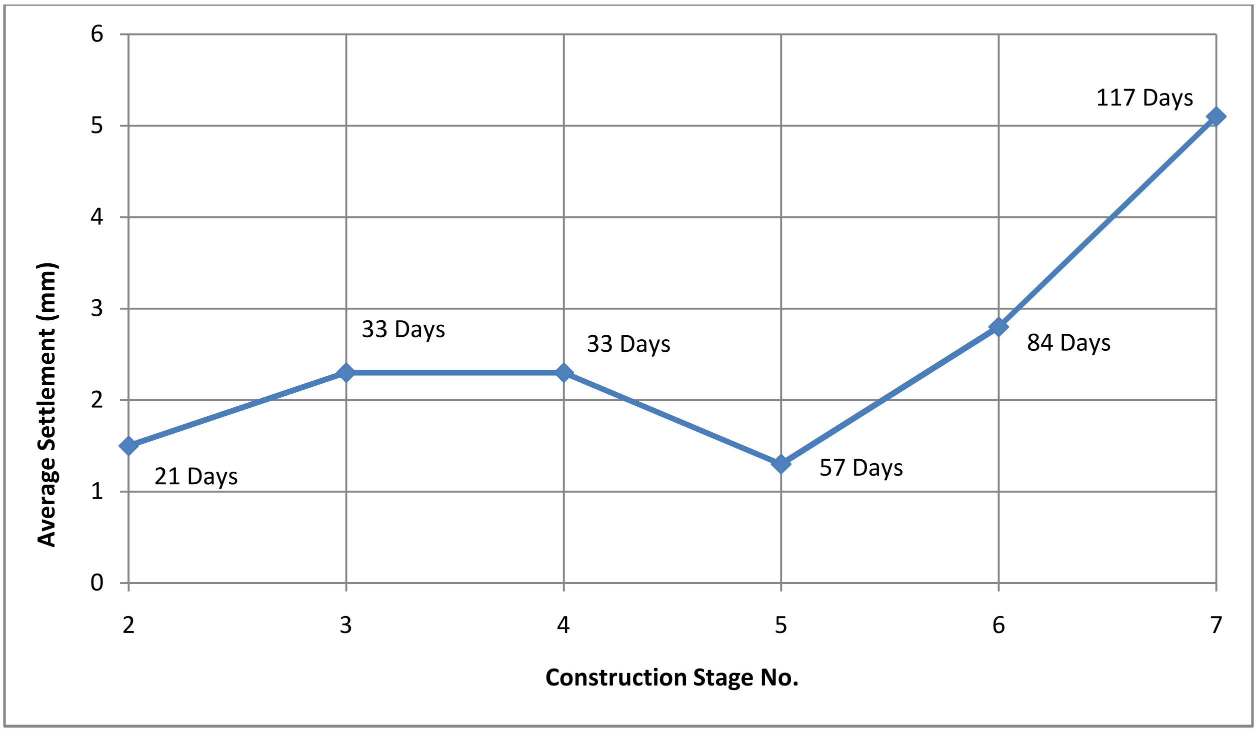

Figure 4: Variations of Average Settlement with Time at Different Construction Stages

Table 6. Comparison between Measured and Estimated Average Settlement Performance

Const. Stage No.	**Stage Description**	**(Estimated/Measured) Settlement Ratio for:**				
		Elastic Method w/E_s =			**Hough Method w/ C′ =:**	
		$(E_s)_{min}$	**$(E_s)_{max}$**	**$(E_s)_{sab}$**	**164**	**236**
2	Pier Column Construction	0.82	0.33	0.59	0.46	0.32
3	Soil Backfilling	NA	NA	NA	NA	NA
4	Pier Cap Construction	1.67	0.91	0.66	0.90	0.62
5	Barrier Wall Construction	5.69	2.15	1.43	2.09	1.45
6	Girder Beam Placement	4.62	1.75	0.94	1.53	1.06
7	Deck Construction	5.09	1.94	0.95	1.51	1.05
Average		3.58	1.42	0.91	1.30	0.90

[Note]: $(E_s)_{min}$ = 23.9 MPa; $(E_s)_{max}$ = 62.1 MPa; $(E_s)_{Sab}$ = 139.75 MPa. "NA" = Not Available.

Reliability of Selected Footing Design/ Analysis Methods

The American Association of State Highway & Transportation Officials (AASHTO) has been setting national and international standards for the design of bridges since 1931. In 1987, the AASHTO subcommittee reassessed U.S. bridge specifications and reviewed foreign design specifications and codes. A recommendation was made to adjust the conventional Working Stress Design (WSD) so that the variability in the loadings and the material properties are reflected. With this new philosophy, the Load and Resistance Factor Design (LRFD) was implemented on the basis of recent developments in structural engineering and statistical methods. Section 10.6 of AASHTO LRFD bridge design specifications document (2010) presented two methods for estimating the settlement performance of spread footings resting on cohesionless soils. The first method is based on the analysis of an elastic half space, and its key formula is given by Eq. 3:

$$S_e = \frac{q_o\sqrt{A}}{E_s\beta_z}\left(1-\upsilon_s^2\right); \quad (3)$$

where: q_o = applied vertical stress; A = footprint area of footing (L·B); L = footing length; E_s = elastic modulus of soil; β_z = rigidity factor; and υ_s = Poisson's ratio of soil.

Es estimation is difficult, because true undisturbed sampling of cohesionless soils is neither simple nor practical for usual foundation design. The AASHTO provided guidelines on Young's modulus Es and Poisson's ratio υ using the information available from U.S. Department of the Navy (1982) and Bowles (1988) and on the rigidity factor using the approach outlined in Kulhawy (1983). The elastic half-space method is overpredicting the settlement when using the E_s values recommended by U.S. Department of the Navy (1982) and Bowles (1988). A different method for estimating the Es needs to be used to enhance the AASHTO guidelines for predicting the settlement.

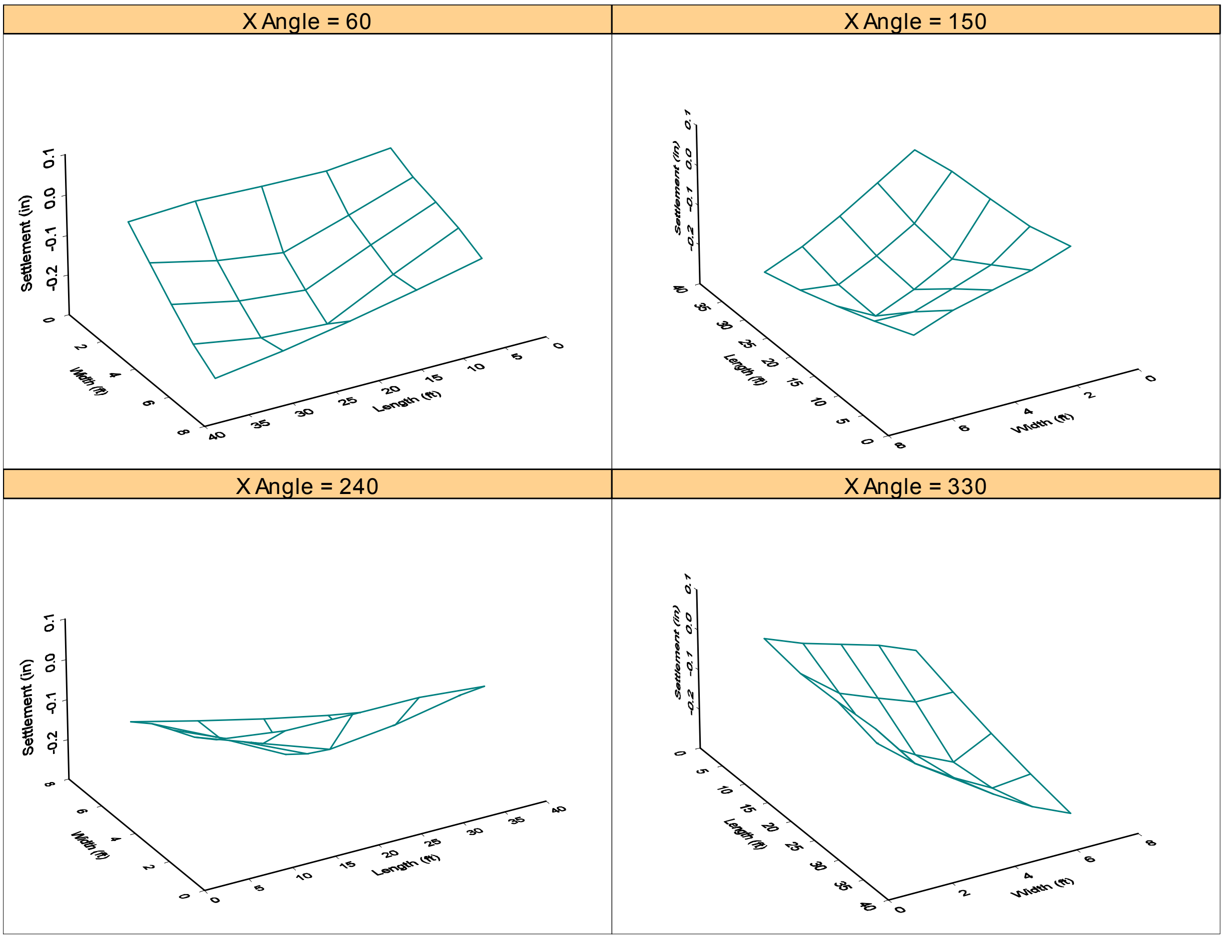

Figure 5: Settlement Distribution (Stage 7: Bridge Deck Construction)

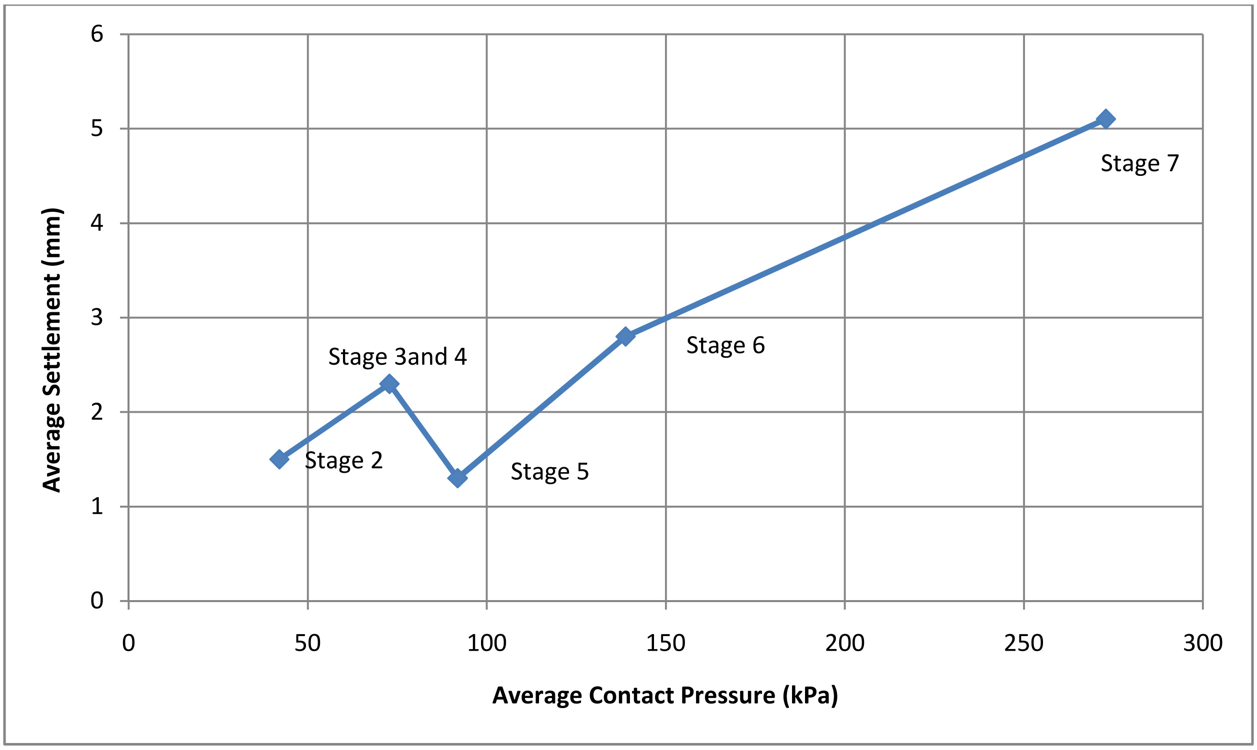

Figure 6: Variations of Average Contact Pressure with Average Settlement at Different Construction Stages

The research team estimated E_s using a method proposed by Sabatini et al. (2002) to enhance the AASHTO method for estimating the footing settlement. This method is recommended by the Engineering Circular No.5 (Evaluation of Soil and Rock Properties). The method is provided below:

$$E_s = \left(\frac{E}{E_o}\right) E_o \quad (4)$$

$$\left(\frac{E}{E_o}\right) = 1 - \left(\frac{1}{FOS}\right)^{0.3} \quad (5)$$

$$E_o = 2G_o\,(1+v) \quad (6)$$

$$G_o = 15{,}560(N60)^{0.68}; \quad (7)$$

where: E_s= Elastic modulus of soil; E_o= Small-strain elastic Young's modulus; G_o= Small-strain shear modulus; $\left(\frac{E}{E_o}\right)$= Modulus degradation value; v= Poisson's ratio $0.1 < \nu < 0.2$; FOS= Factor of Safety for the structure; N_{60}= SPT number corrected to an equivalent rod energy ratio of 60%.

The second method is based on the work of Hough (1959), which can provide immediate settlement of a footing on cohesionless soils. The Hough method calculates the settlement by:

$$S_e = \sum_{i=1}^{n} \Delta H_i = \sum_{i=1}^{n} H_c \frac{1}{C'} \log\left(\frac{\sigma'_o + \Delta\sigma_v}{\sigma'_o}\right); \quad (8)$$

where: ΔH_i = elastic settlement of layer; n = number of soil layers within zone of influence; H_c = initial layer thickness; C' = bearing capacity index; σ'_o= initial vertical effective stress; and $\Delta\sigma_v$ = increase in effective stress.

In the Hough method, the zone of influence is considered to extend to the depth equivalent to three times the footing width. When the soil in the influence zone is subdivided, each soil layer should be a maximum of 3.0 m thick. The correlation between the blow counts and the bearing capacity index C′ values is available and is based on the original results obtained by Hough (1959) and the modifications made by Cheney and Chassie (2000).

For each prediction method, a few variations in the value of the soil modulus (E_s or C′) chosen resulted in a range of settlement behavior.

Table 6 shows the results. For the elastic method, the case with the elastic modulus of soil calculated using the method proposed by Sabatini et al. (2002); denoted as $(E_s)_{sab}$, produced the best outcome. For the Hough method, the case with the higher C′ value also produced better settlement predictions. Both methods shared a tendency to underpredict the actual settlement during the early stages and overpredict the field performance during the later stages. This implies that the soil modulus increased in the field under higher loads. The settlement prediction methods specified by the AASHTO LRFD design guidelines are reasonable, but further refinements may be needed.

SUMMARY AND CONCLUSIONS

The researchers continued their comprehensive study of spread footing foundations. In their newest research project, funded by the Ohio Department of Transportation (ODOT) and the Federal Highway Administration (FHWA), one spread footing foundation existing at a major highway bridge construction site in Ohio was instrumented and monitored. Subsurface soil conditions at the site were favorable to the use of shallow foundations. This was evidenced by the fact that uncorrected SPT-N values are all well above 50 blows/ft. The spread footing field performance data were then compared with results provided by the geotechnical methods outlined in the recent AASHTO LRFD bridge design specifications to assess general reliability of the LRFD design guidelines. Enhancement to one of the AASHTO methods to estimate the settlement was proposed in this paper.

The analysis of the footings at the site showed that they can be regarded as a rigid structure. Also, the results of the current study indicated that the methods presented in the AASHTO LRFD design specifications (2010) were satisfactory. However, this paper proposed enhancement to the elastic-half space method by using Sabatini et al. (2002) method to calculate Es. According to a quick review of the most current version of the AASHTO LRFD specifications (2010), descriptions of the elastic settlement prediction method and the Hough method have not been modified.

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