GEOTECHNICAL SPECIAL PUBLICATION NO. 126

GEOTECHNICAL ENGINEERING FOR TRANSPORTATION PROJECTS

VOLUME TWO

PROCEEDINGS OF GEO-TRANS 2004

July 27–31, 2004
Los Angeles, California

SPONSORED BY
The Geo-Institute of the American Society of Civil Engineers

EDITED BY
Mishac K. Yegian
Edward Kavazanjian

Published by the American Society of Civil Engineers

SETTLEMENT MONITORING OF LARGE BOX CULVERT SUPPORTED BY RAMMED AGGREGATE PIERS– A CASE HISTORY

David J. White[1] M., ASCE and Kenneth Hoevelkamp[2] A.M., ASCE

ABSTRACT: Subsurface conditions at the site of a large box culvert constructed beneath a deteriorating bridge consisted of highly compressible alluvial clay. Concerned with settlements estimated up to 50 cm, engineers considered completely removing and rebuilding the bridge structure. With estimated settlements still exceeding 10 cm, rammed aggregate piers were selected for installation beneath the box culvert to control differential settlement and prevent downdrag on the existing bridge pier foundations. Despite construction challenges including high water table, very soft soil conditions, and low clearance for machinery beneath the bridge structure, it was determined that the benefits of the box culvert and embankment (i.e. ease of future roadway expansion and continual service of the highway throughout construction) outweighed the cost of replacing the bridge structure. Approximately 250 rammed aggregate piers were installed in a grid pattern with lengths ranging from 2.3 m to 6.7 m depending on embankment fill heights. Performance was monitored with settlement plates and compared to predictions.

INTRODUCTION

This paper describes a case history where rammed aggregate piers were used to control settlement of a 4.2 m wide x 3.6 m high x 50 m long box culvert constructed beneath a three-span bridge on Iowa Highway 191 south of Neola, Iowa. The purpose of the box culvert construction was to eliminate replacement of the previously widened, deteriorating bridge built in 1927. Rammed aggregate piers were installed to reduce total and differential settlement of the culvert, and also to prevent downdrag on the existing bridge pier foundations. Soil conditions at the site consisted of highly

[1] Assistant Professor of Civil Engineering, Iowa State University, Ames, IA 50011-3232, djwhite@iastate.edu
[2] Associate Project Engineering, Geopier Foundation Company, West Irvine, CA 92614

compressible alluvial clay overlying glacial till and weathered shale bedrock. Construction began in July 2001 and was finished in December 2001. Backfilling operations began in November 2001 and were completed in about three weeks. The embankment fill reached a maximum height of 7.5 m beneath the bridge.

A wide range of in situ and laboratory tests were conducted prior to construction to characterize soil conditions. Further, full-scale load tests were performed to better characterize individual pier behavior. Settlement pins were installed within the floor of the box culvert to monitor settlement during and after filling operations. The results of this project are presented by describing the site conditions, load test results, design assumptions, and performance (i.e. settlement) monitoring. Figure 1 shows the project site during initial grading operations and after completion of the box culvert.

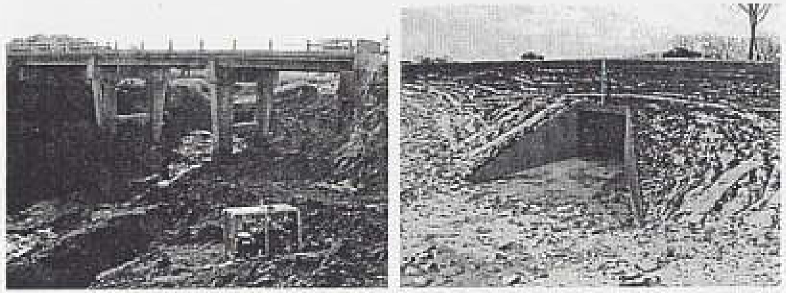

FIGURE 1. Initial Site Preparation and after Box Culvert Construction

SUBSURFACE INVESTIGATION RESULTS
Soil Index and Compressibility Parameters

Atterberg limits tests, natural water content and dry density measurements were performed on samples up to about 10 m below the bottom of the proposed elevation of the box culvert. Figure 2 presents the plasticity index, moisture content, and liquid limit values determined at each depth. Average liquid limit and plasticity index values are 44 percent and 16 percent, respectively. In-situ moisture content decreased with depth from about 42 percent to 32 percent—resulting in an average liquidity index of about 0.8. Void ratio decreased with depth from about 1.1 to 0.8. Hydrometer analysis shows that the alluvial clay is composed of about 74 percent silt size particles and about 26 percent clay size particles and classifies as CL.

One-dimensional consolidation tests were conducted to determine the compressibility of the soft alluvial clay. Results were used to provide an estimate of primary consolidation settlement and the time-rate of settlement. Tests were performed on 75 mm diameter tube samples obtained from depths of 3.7 m and 4.0 m. Results indicate that the coefficient of consolidation (c_v) averaged 0.07 ± 0.03 m^2/day. Void ratio, e, versus the logarithm of applied effective pressure results provided in Figure 3 show that the alluvial clay layer is normally consolidated. The average compression index (c_c) is about 0.32.

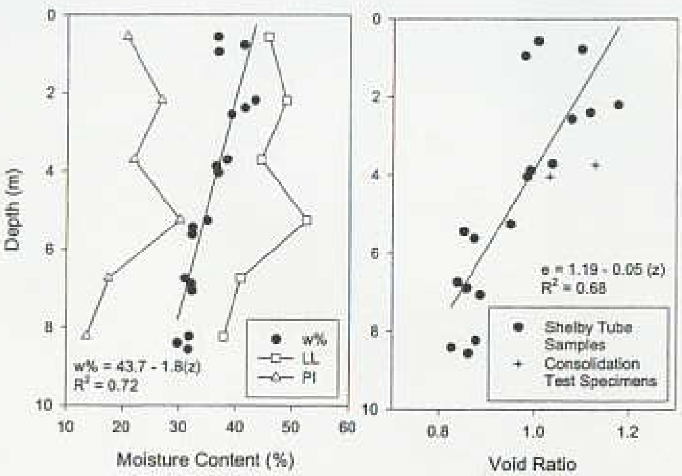

FIGURE 2. In situ moisture content, LL, PI, and dry density

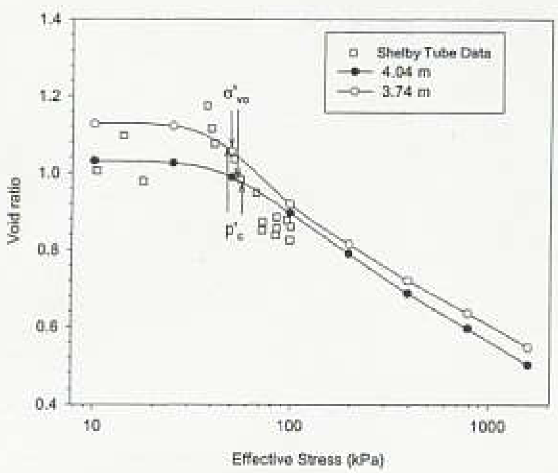

FIGURE 3. e-log(p) Curves and Void Ratio Measurements from Several Field Samples

Soil Profile

Cone penetration tests (CPTs) were performed prior to construction. Results are presented in Figure 4. The parameters displayed include: q_T for corrected tip resistance, f_s for sleeve friction, R_f for friction ratio. Overall, the profiles indicate a layer of fill averaging 1.2 m thick underlain by 12.5 m of alluvial clay underlain by 2 m of glacial till outwash overlying weathered shale bedrock. The alluvial clay was of primary interest for this project, due to its high compressibility. Based on CPTs, Dilatometer tests (DMTs), and unconsolidated-undrained (UU) triaxial compression tests, undrained shear strength (s_u) varied from about 10 kPa to 30 kPa . Additional details of the subsurface investigation are described in White *et al.* (2003).

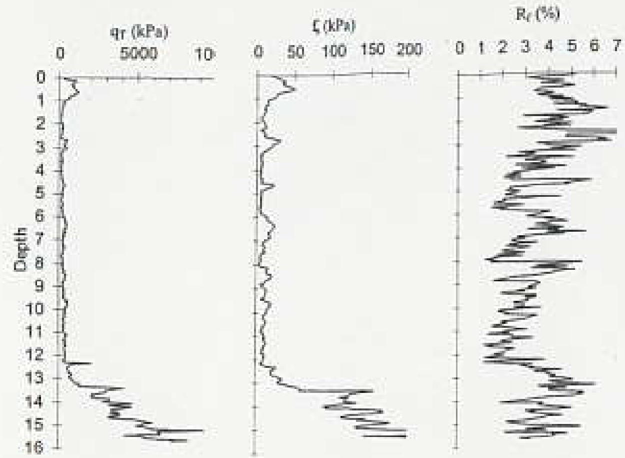

FIGURE 4. Cone Penetration Test Results

LOAD TEST RESULTS

Individual load tests were performed on two pier elements of 0.76 m diameter. Pier No. 1 was installed to a depth of 2.74 m and Pier No. 2 was installed to a depth of 5.05 m. A telltale was installed near the bottom of each load test pier to measure tip movement. Figure 5 shows the applied stress at the top of the pier versus settlement at the top and bottom of the piers. For Pier No. 1 loading was aborted at a stress of about 390 kPa. Top of pier settlement reached 80 mm at this point while the telltale settled about 43 mm. Stiffness varied from about 74 to 5 kPa/mm (also MN/m^3) over the applied stress range. For Pier No. 2, loading was aborted at a stress of about 460 kPa. Top of pier settlement reached 22 mm while the telltale settled only 1.4 mm. The difference in top of pier settlement and telltale settlement suggests pier bulging. Stiffness varied from about 164 to 28 kPa/mm (also MN/m^3).

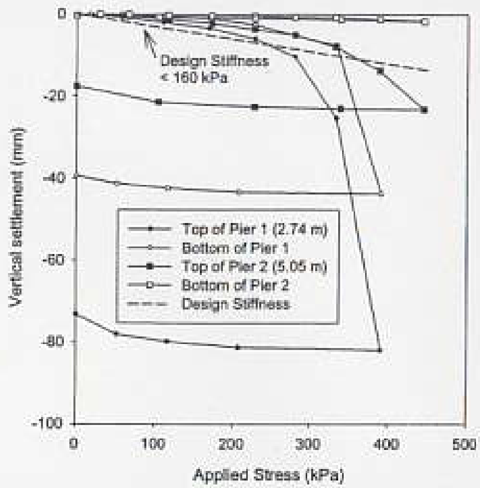

FIGURE 5. Load Test Results for 0.76 m Dia. by 2.74 m and 5.05 m Long Piers.

SOIL REINFORCEMENT DESIGN

Unreinforced Culvert

Consolidation test results shown in Figure 3 were used to estimate the magnitude of primary consolidation settlement for the unreinforced culvert. Assumptions used in these calculations were: (1) The alluvial clay layer was normally consolidated; (2) The drainage distance equals half the thickness of the alluvial clay layer or about 3.75 m — assumes glacial sand at the bottom of the alluvial clay layer would act as a drainage pathway; (3) The applied stress at the ground surface due to box culvert and embankment construction would be about 160 kPa; and (4) A stress increase of 125 kPa was estimated at the mid-height of the alluvial clay layer. Using these assumptions, it was estimated that the unreinforced culvert could settle up to 50 cm. A period of about 170 days was estimated to reach 90 percent primary consolidation.

Reinforced Condition

To estimate settlement for the reinforced condition, stiffness of the pier elements and stress concentration must be estimated. For soft clay soils, Fox and Cowell (1998) recommend using a design stiffness of about 33 MN/m^3. Full-scale load test results (Fig. 5) verified that this value was reasonable. For the case of "rigid" footing over the piers, stress concentration is established with the expression (Lawton *et al.* 1994):

$$q_g = q\,[R_s/(R_s\,R_a - R_a + 1)] \qquad (1)$$

where q_g is the stress applied to the tops of the piers elements and q is the average applied stress at the bottom of the box culvert. A stiffness ratio (R_s) was established using computations derived from the in situ test results. Using an area replacement ratio (R_a) of 25 percent (equilateral center-to-center pier spacing of 1.5 m) (see design layout Fig. 6) and a maximum culvert-bottom stress of about 160 kPa, a top-of- stress (q_g) of 490 kPa is computed. The settlement of the reinforced zone (s_g) is then computed as:

$$s_g = q_g/k_g = 490 \text{ kPa} / 33 \text{ MN/m}^3 = 1.4 \text{ cm} \qquad (2)$$

Based on an assumed stress distribution and soil modulus values (2000 kPa to 3000 kPa), additional settlements on the order of 11 cm were computed to occur below the 6.7 m long elements. Calculated design lengths and anticipated settlements are provided in Table I. Based on these calculations, the design criteria of less than 15 cm of total settlement and less than 10 cm of differential settlement would be satisfied. Design calculations performed by the authors' for Zones A through D are further described in White *et al.* (2003).

For estimating the time rate of settlement, solutions are given by Han and Ye (2001) who identify two mechanisms that contribute to settlement rate reduction: (1) The presence of the vertical drainage element, which reduces the flow distance for the dissipation of excess pore water pressure, and (2) The concentration of stress to the relatively stiff pier elements, which reduces the consolidation settlement of the compressible matrix soils. With reinforcement, 90 percent primary consolidation was estimated at about 10 days.

TABLE 1. Comparison of Design Settlement Calculations and Actual Field Measurements

Design Section Zone	Design Shaft Length (m)	Design Bearing Pressure (kPa)	Estimated Upper Zone Settlement (cm)	Estimated Lower Zone Settlement (cm)	Estimated Total Settlement (cm)	As-Built Shaft Length (m)	Average Measured Total Settlement (cm)
A	6.71	163	1.4	10.3	12.7	6.7	10.1
B	5.79	123	1.1	12.0	13.0	5.8	9.1
C	4.27	82	0.7	11.9	12.6	4.3	8.5
D	0.91	41	0.4	10.3	10.6	2.3	5.6

PERFORMANCE MONITORING

Total and differential settlement of the box culvert and settlement of the bridge were considered during the performance-monitoring phase of this project. Optical surveys were used to monitor the box culvert and bridge structure. Figure 7 and Table 1 show the survey log along the length of the box culvert. The data in Figure 7 indicate a maximum settlement of 11.5 cm in Zone A and a maximum differential settlement of 7.9 cm between Zone A and D. Measurements show that the culvert settled 1 cm before backfilling, and continued to settle during fill placement which was completed in about 20 days.

Although the project met the criterion for differential settlement, the piers in Zone D were too long. The design called for four zones of different pier lengths. Varying the pier lengths was intended to compensate for the non-uniform fill height over the culvert, i.e., shorter piers where there is less overburden. Review of the pier installation records reveals that piers in zones A, B, and C were constructed as specified. However, zone D piers were constructed to an average length of 2.3 m, which is more than twice the design length of 0.9 m. Zone D was designed to settle about 10.6 cm, but with longer than specified piers it only settled about 4 cm, contributing to increased differential settlement. In effect, the box culvert curled up at the ends, which is believe to have contributed to the opening of small gaps along construction joints. The gaps are larger at the bottom of the culvert than the top.

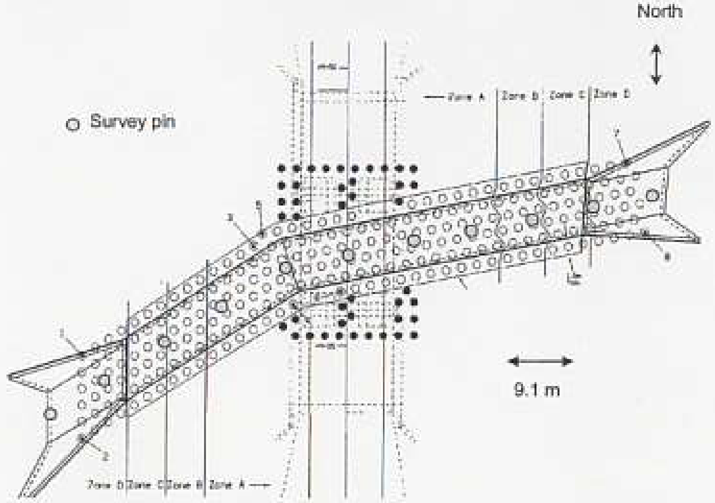

FIGURE 6. Installation Detail and Settlement Pin Locations

SUMMARY AND CONCLUSIONS

The following conclusions are based on the information gathered throughout the course of this investigation:

- The reinforced box culvert settled about 12 cm compared to the predicted unreinforced settlement of about 50 cm.
- Measured settlement is lower than the design criteria of 15 cm of total settlement and 10 cm of differential settlement.
- The increase in pier construction length in Zone D likely resulted in an additional 3 cm to 4 cm of differential settlement.

- Settlement rate calculations within the alluvial clay layer revealed that the reinforcement increased the time rate to reach 90 percent primary consolidation from 170 days to about 10 days.

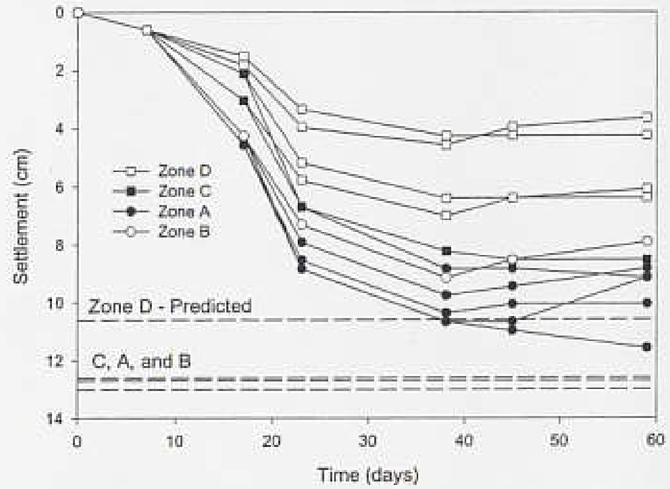

FIGURE 7. Settlement Monitoring Results for Box Culvert

ACKOWLEDGEMENT

The Highway Division of the Iowa Department of Transportation and the Iowa Highway Research Board sponsored this study under contract TR-443. The opinions expressed in this paper are those of the authors and not necessarily those of the sponsoring agency.

REFERENCES

Fox, N.S. and Cowell, M.J. (1998). *Geopier Foundation and Soil Reinforcement Manual*. Geopier Foudnation Company, Inc. Scottsdale, AZ.

Han, J. and Ye, S.L. (2001). Simplified method for consolidation rate of stone column reinforced foundations. *ASCE Journal of Geotechnical and Geoenvironmental Engineering*. Vol. 127, No. 7.

Lawton, E. C., Fox, N. S., and Handy, R. L. (1994). "Control of settlement and uplift of structures using short aggregate piers." *In-situ Deep Soil Improvement*, ASCE, 121-132.

White, D.J., Gual, A., Hoevelkamp, K. (2003). *Highway Applications for Rammed Aggregate Piers in Iowa Soils*, Final Report, Iowa DOT Project TR-443, CTRE Project 00-60, April.