OPTIMIZATION OF DEEP FOUNDATION DESIGN IN A SAND, GRAVEL AND COBBLE FORMATION USING PRESSUREMETER TESTING

OPTIMISATION À L'AIDE DU PRESSIOMÈTRE DU DIMENSIONNEMENT DE FONDATIONS PROFONDES DANS DES FORMATIONS DE SABLE, GRAVIER ET GALETS

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ABSTRACT – The resistance of very dense glacial outwash formation consisting of sand, gravel and cobble can be quite high and engineers tend to underestimate it in deep foundation design. More realistic measurements can be achieved with slotted steel casing pressuremeter tests. For the IR-75 project in Dayton, Ohio, USA these materials were characterized using a slotted casing pressuremeter, providing more realistic properties of these dense materials to optimize design.

RÉSUMÉ – La résistance des formations glaciaires de sable, graviers et galets peut être très élevée et est généralement sous-estimée dans le dimensionnement des fondations profondes. Des mesures donnant une estimation plus réaliste peuvent être réalisées au pressiomètre, moyennant des précautions particulières détaillées dans cette communication. Ce type de matériau a été caractérisé pour le projet IR-75 à Dayton (Ohio).

1. Introduction

Proposed reconstruction of I-75 downtown consists of widening an approximately 2.46 km section of IR-75 through the northern section of downtown Dayton, Ohio, USA. The reconstruction also includes reconfiguration of the IR-75/SR4 interchange; reconstruction/construction of 10 bridges (including 2 separate structures spanning the Great Miami River); replacement of the existing Grand Avenue bridge with embankment; reconfiguration/improvements to approximately 0.85 Km of surface streets, mainly at Main Street and Hall Avenue; and construction of 17 retaining structures.

Bridge B-13, a 9-span continuous prestressed concrete I-beam, 319.0-meter (1,048-foot) long bridge with a stub abutment on MSE walls for the rear abutment, and a stub abutment with spill-through slopes for the forward abutment. The piers in the river were located parallel to the Great Miami River, and are full height reinforced concrete walls.

2. Geology

The project lies within the historic flood plain of the Great Miami River and nearby surface soils generally consist of alluvial deposits comprised of silt, clay and fine sand containing varying amounts of organics. Underlying this relatively thin layer of recently deposited alluvium, the subsurface conditions consist of glacial outwash deposited in a buried bedrock valley primarily of sand and gravel with cobbles. Sporadically, relatively thin, discontinuous layers of cohesive glacial till are encountered within the overall outwash matrix. Geologic references indicate that bedrock, consisting of interbedded limestone and shale belonging to the Grant Lake and the Miamitown-Fairview Formations of Ordovician Age, is located at or below MSL Elevation 75 meters beneath the natural ground surface.

3. Subsurface investigations

A total of sixteen borings were drilled for bridge B-13 as a part of the project subsurface investigation. Boring depths ranged from 14.0 to 26.0 meters below the existing ground surface. The borings performed on land were advanced either by an ATV-mounted (all-terrain vehicle) or a truck-mounted drill rig using either a 75- or 100-mm I.D. hollow-stem auger. Borings performed within the Great Miami River were advanced by a skid-mounted drill rig on a barge using a casing advancing roller-bit with 75-mm I.D. casing, with a 75-mm tricone bit. Bentonitic drilling mud was generally introduced into the auger or casing once groundwater was encountered, with the level of mud maintained at or above the groundwater level through boring completion. Disturbed soil samples were retrieved using a 50-mm O.D. split-barrel sampler to the bottom of the boring and then driving the sampler into the soil with a 63.5-Kg hammer freely falling 0.76 meter (ASTM D1586-Standard Penetration Test).

4. Pressuremeter test procedure

The driller advanced either casing or augers approximately 1 meter above the pressuremeter test. Steel casing was used to advance borings performed in the river, and augers were used for borings on land. Mud rotary drilling with a large diameter roller bit was used to remove all of the gravel and cobble fragments inside the casing or augers.

At the pressuremeter test elevation the driller used a 63.5-mm (2.5-inch) carbide button bi-cone bit, using a rotation rate of about 60 rpm, a mud flow rate of about 10 gallons per minute (40 liters per minute), and an advance rate of 6 to 10 minutes per meter (2 to 3 minutes per foot). To minimize vibrations, advance rates were slower where excessive cobbles were encountered. The 63.5-mm (2.5-inch) O.D. slotted casing pressuremeter was lowered to the test elevation, often requiring either pushing or hammering into the test zone. We believe that the test zone was not perfectly straight, or that gravel or cobble fragments jutted into the test zone and had to be displaced to advance the pressuremeter into the test zone. The slotted casing prevented membrane damage during insertion. A photograph of the slotted casing being lowered into the borehole is shown below.

The pressuremeter was calibrated for system compressibility and membrane resistance. At full expansion of 800 cm^3, the membrane resistance of the slotted casing was about 2.5 bars.

Strain-controlled tests were performed using a Texam control unit with a monocell probe. Volume increments of 20 cm^3 were injected into the probe and corresponding pressures were measured. Near the end of the elastic portion of the test, an unload-reload cycle was done. A creep test was performed for 10 minutes at the next volume increment. The test continued until either 800 cm^3 were injected or the membrane burst. Typical test results are shown in Figure 1 and Figure 2.

Photo 1. Lowering the slotted casing PMT into borehole (Lee, 2004)

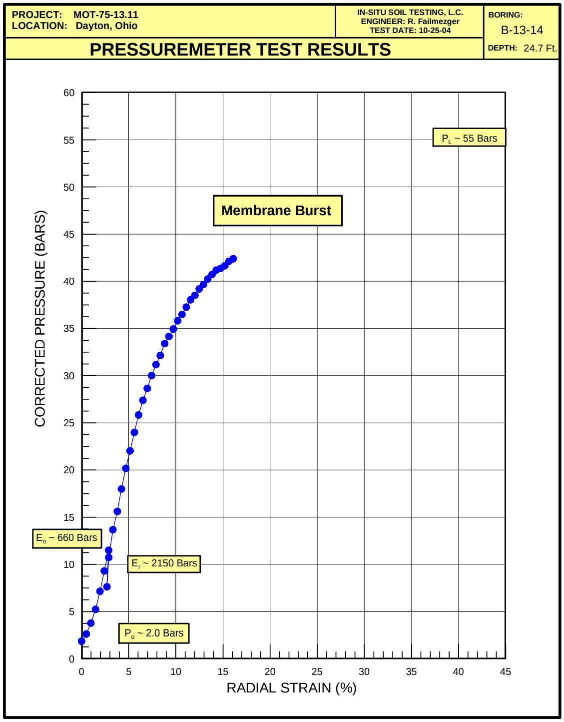

Figure 1. Typical pressuremeter test using slotted casing for I-75 Bridge in Dayton, Ohio

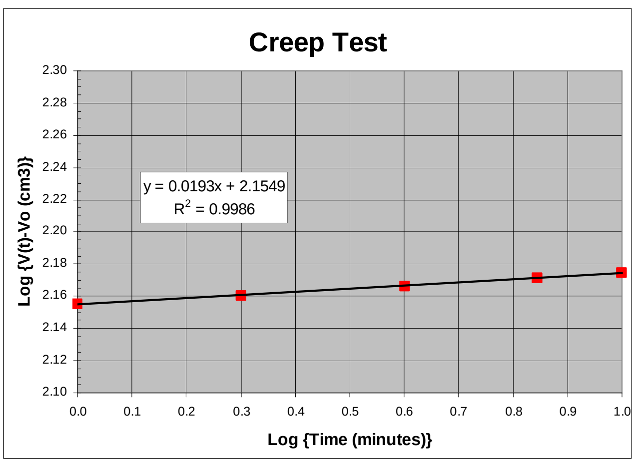

Figure 2. Creep test results from above PMT

Pressuremeter test results are summarized in Table I.

Table I. Summary of PMT Results

Boring Number	Depth (ft/m)	P_o (bars)	E_o (bars)	E_r (bars)	P_L (bars)	Creep Test N	r^2
B-11-02	27.4/8.4		130	640	18		
B-12-01A	15.5/4.7	1.5	530	2380	50	0.0174	0.9946
B-13-08	20.5/6.2	1.3	430	1240	60	0.0123	0.9928
	31.0/9.4	2.0	1460	5250	90	0.0147	0.9943
	40.0/12.2	2.0	540	2140	60	0.0171	0.9972
B-13-13	17.5/5.3	1.1	160	960	24	0.0146	0.9889
	27.5/8.4	1.3	100	460	18	0.0206	0.9913
	43.0/13.1	1.3	240	1130	35	0.0150	0.9986
B-13-14	24.7/7.5	2.0	660	2150	55	0.0193	0.9986
	45.4/13.8	2.5	600	1960	60	0.0155	0.9941
B-14-05	13.0/4.0	1.0	140	700	15	0.0307	0.9873
	23.0/7.0	1.4	510	1970	55	0.0155	0.9960
	33.0/10.1	1.7	870	1240	75	0.0128	0.9888
B-19-06	22.9/7.0	0.8	150	870	26	0.0142	0.9952
	30.9/9.4	1.2	620	3220	60	0.0144	0.9935
RW-15-03A	31.0/9.4	1.6	130	700	32	0.0659	0.9693
RW-16-04A	51.0/15.5	2.1	940	3770	75	0.0166	0.9943
RW-16-04B	27.5/8.4	1.0	360	1640	42	0.0268	0.9924
	33.8/10.3	1.2	620	3890	70	0.0194	0.9942
RW-24-02	31.9/9.7	1.6	110	710	9		
RW-24-04A	22.0/6.7	1.4	430	1700	55	0.0141	0.9987

5. Subsurface conditions

Subsurface conditions under Bridge B 13 piers consisted of the following layers in descending order:

Layer G1: Dense to very-dense with N values between 22 and 60 blows per 30 cm, gray and brown fine to coarse sand and gravel interbedded with discontinuous layers of silt with some fine sand. Cobbles were encountered throughout this stratum.

Layer G2: Dense to very-dense with N values greater than 60 blows per 30 cm, gray fine to coarse sand and gravel. This stratum contains 1.2- to 5.2-meter discontinuous layers of hard gray clayey silt or very-dense with N values over 75 blows per 30 cm, gray fine to coarse sand and clayey silt (A-4a). Cobbles were encountered throughout this stratum.

6. Bridge foundations

The project team selected drilled shafts to support Bridge B-13 piers over the Great Miami River for the following reasons:

- The required lateral capacity of deep foundations to resist bridge lateral loads is relatively high.
- The scour depth for river piers (Piers 3 to 7) is between 1.5 and 4 meters.

- Area geology indicates increasing presence of cobbles and possibly boulders near the river.
- Subsurface conditions indicated in the borings along Bridge B-13 show very high driving resistance to driven piles, particularly within the river piers, and potential early refusal prior to reaching the pile tip elevation required to resist bridge lateral loads.

Piers 3 through 7 (river piers) are located between existing levees. As such, the river piers are subjected to effects of local scour to up to 4 meters. Based on the subsurface conditions, SPT spoon refusal N-values of 50 blows over 15 cm of penetration or less are encountered at pile penetration less than 5 meters, with many cobbles noted throughout the boring logs. Therefore, drilled shafts were selected as the most suitable foundation for piers 3 through 7.

The team evaluated the use of a single row of 1,524-mm (60-inch) diameter drilled shafts to support Piers 3 to 7. The drilled shafts will be constructed with an approximate embedment of 19 meters. This scheme minimizes excavation and any associated sheeting, cofferdams, and dewatering efforts required for construction of piles and pile caps. The drilled shafts can be advanced through cobbles and/or boulders to reach the required tip elevation. This will minimize risk of premature pile refusal, lowers the risk of changing the foundation type and consequent associated schedule delays and cost escalation during construction.

The pier foundation support consists of a single row of 1,524-mm (60-inch) diameter drilled shafts spaced at 156 meter center-to-center spacing. Each drilled shaft is required to carry a design (service) vertical compressive load of 4,693 KN (1,055 kips), a design lateral load of 249 KN, and corresponding shaft-head overturning moment of KN-meter (1,550 ft-kips).

7. Optimization of design using pressuremeter testing

The lateral loads imposed on the foundations controlled the design of the drilled shafts will be resisted by the soil surrounding the drilled shafts. Soil response is analyzed using conventional p-y curves, and site specific PMT-PY curves were evaluated. These curves are a function of soil resistance, as well as shaft diameter. For these analyses, preliminary idealized PMT-PY curves were developed using the results of pressuremeter tests performed in the river borings. Figure 3 presents conventional versus site-specific PMT-PY curves developed based on pressuremeter testing for glacial outwash at Bridge B-13.

The method for developing p-y curves from PMT data suggested by Robertson et al. (1985) was used to derive the p-y curves from the corrected PMT data. A recent application of this procedure was presented by Anderson and Townsend (1999). Generally, this method incorporates the reloading part of the corrected PMT data. It calculates the soil resistance, P, and corresponding deflection, Y, using the following Equations (1) and (2).

P= (Corrected Pressure from PMT)*(Shaft Diameter)*(Reduction Factor) (1)

Y= (Corrected Volume from PMT/(2*Initial Volume))*(Shaft Diameter/2) (2)

The reduction factor (α) was determined based on the soil type and depth where PMT was performed.

The conventional PY curves were used based on internally generated PY curves with soil parameters input in the LPile Plus 4.0[8] (Ensoft, Inc., 2000) computer program.

The response of the drilled shafts to the horizontal and moment loading was analyzed using the LPile Plus 4.0[8] (Ensoft, Inc., 2000) computer program, as presented in figure 4. The program computes shaft deflection, bending moment, shear, and soil response with respect to depth for a given loading. The reinforced concrete drilled shaft is modeled as a beam exhibiting a non-linear bending stiffness. Soil response is modeled as a series of non-linear reaction springs (p-y curves) based on soil properties, shaft diameter, and effective stress.

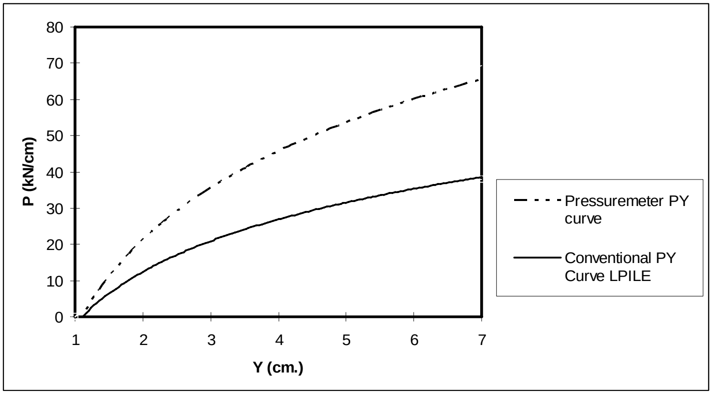

Figure 3. Conventional vs. site-specific PMT-PY curves developed based on pressuremeter testing for glacial outwash at Bridge B-13

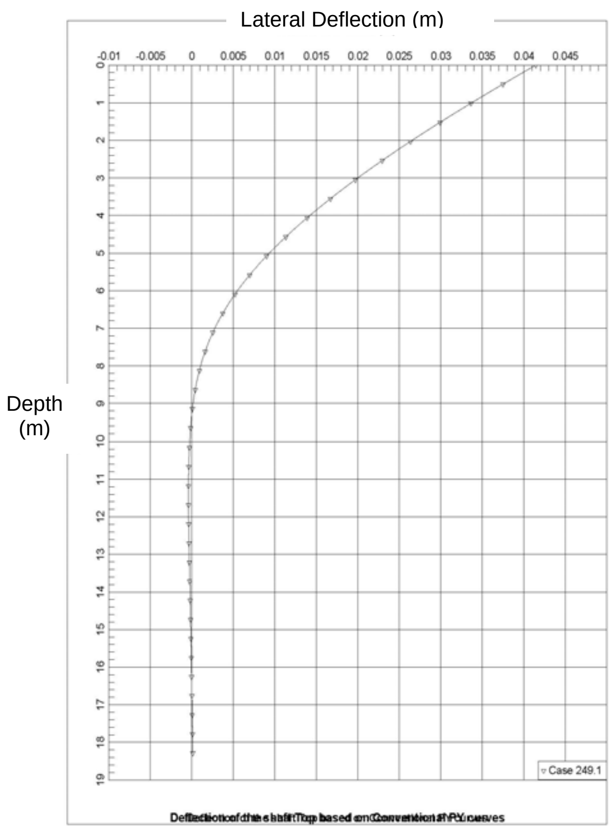

Figure 4a. Lateral movement based on conventional p-y curves

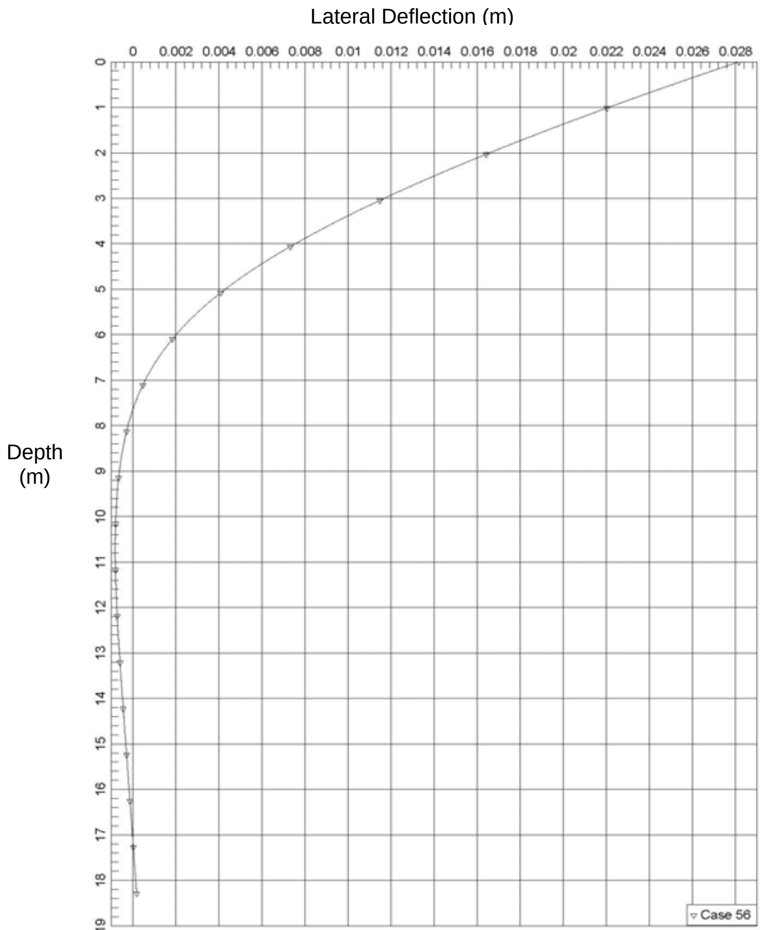

Figure 4b. Lateral movement based on PMT-p-y curves

Site-specific PMT-p-y curves resulted in much higher soil response, as shown in figure 4, and therefore, much smaller shaft top deflection that is within acceptable limits for typical foundations under extreme loading condition. Shaft lateral capacity design solely using conventional p-y curves would have resulted either in additional shafts per pier, or much larger diameter, or possibly deeper, shafts. Therefore, the use of pressuremeter testing will result in substantial construction cost savings.

8. Conclusions

The following conclusions can be drawn from using pressuremeter testing in dense glacial outwash formation to optimize deep foundation design.

1. The slotted casing pressuremeter can be used to measure the strength and deformation properties of sand, gravel, and cobble formation.

2. Pressuremeter test quality is highly dependent on the experience of the engineer performing the test.
3. The PMT-PY curves generally result in higher and more realistic estimate of soil resistance than conventional p-y curves with input soil parameters from laboratory testing. Using PMT-PY curves will reduce the number of shafts per pier by approximately 20 to 30 percent at this site.
4. Lateral load testing is planned during construction. The test data will verify the results of using pressuremeter tests to optimize lateral design of drilled shafts.
5. The shaft vertical capacity also can be optimized using pressuremeter testing. This will be accomplished during final design.
6. For every $1 spent on additional investigation using pressuremeter testing, estimated construction savings ranges between $4 and $7. This $4 to $7 estimate includes the relative cost of lateral load testing to verify pressuremeter test results.

9. References

Anderson J.B., Townsend F.C. (1999) Validation of P-y Curves from Pressuremeter Tests at Pascagoula, Mississippi. *XI Panamerican Conference on Soil Mechanics and Geotechnical Engineering*.

Briaud J.L. (1992) *The Pressuremeter*. A. A. Balkema, Brookfield, VT, USA.

Lee Yong-Woong (2004) Personal correspondence.

Baguelin F., Jezequel J.F., Shields D.H. (1978) *The Pressuremeter and Foundation Engineering*, Clausthal-Zellerfeld, Trans Tech Publications, W. Germany.

Costet J., Sanglerat G. (1975) *Cours Pratique de Mechanique des Sols. Tome 1: Plasticite et Calcul des Tassements*. Dunod.

FHWA (1989) The Pressuremeter Test for Highway Applications, Federal Highway Administration, McLean, Virginia, Publication No. FHWA-IP-89-008.

FHWA (1999) Geotechnical Engineering Circular No. 4 - Ground Anchors and Anchored Systems, Federal Highway Administration, Publication No. FHWA-IF-99-015.

Holtz R. D., Kovacs W.D. (1981) *An Introduction to Geotechnical Engineering*, Prentice Hall, Inc.

Mickelson D.M., Clayton L., Baker R.W., Mode W.N., Schneider A. F. Pleistocene Stratigraphic Units of Wisconsin, University of Wisconsin Extension, Wisconsin Geologic and Natural History Survey, Miscellaneous Paper 84-1, 1-9, A7-1 to A8-3, and A10-1 to A10-4.

Milwaukee Transportation Partners (2002) Geotechnical Reports, Retaining Walls 301, 302, 303 and 308, Milwaukee Transportation Partners for Wisconsin Department of Transportation.

Post-Tensioning Institute (1996) Recommendations for Prestressed Rock and Soil Anchors, Post-Tensioning Institute.

Reese L.C., Wang S.T., Isenhower W.M., Arrellaga J.A., Hendrix J. (2000) Computer Program LPILE Plus (Version 4.0) – A Program for the Analysis of Piles and Drilled Shafts Under Lateral Loads: User's Guide and Technical Manual, Ensoft Inc., Austin, Texas.

Reese L.C., Van Impe W.F. (2001) *Single Piles and Pile Groups Under Lateral Loading*, A. A. Balkema, Rotterdam.

Robertson P.K., Campanella R.G., Brown P.T., Grof I., Hughes J.M. (1985) Design of Axially and Laterally Loaded Piles Using In Situ Tests: A Case History. *Canadian Geotechnical Journal*, Vol. 22, (4), 518-527.

Rose J. (1975) Former Topography of the Eastern Menomonee Valley, Milwaukee, Wisconsin, City of Milwaukee, Department of City Development, Division of Economic Development, Plate I.

Sanglerat G. (1972) *The Penetrometer and Soil Exploration,* Elsevier Publishing Co., Amsterdam.

Terzaghi K., Peck R.B., Mesri G. (1996) *Soil Mechanics in Engineering Practice*, John Wiley & Sons, Inc.

Williams D.E. (1954) *Foundation Conditions in Downtown Milwaukee.* PhD thesis, University of Wisconsin, 136.