

## Polymeric Shell-Confined Aggregate Pier Ground Improvement Method to Support Bridge Embankments over Soft Clay Soil

Tony Sangiuliano, P.Eng.<sup>1</sup>; Jason Brown, P.Eng.<sup>2</sup>; Brian C. Metcalfe, P.E., M.ASCE<sup>3</sup>; and Kord J. Wissmann, Ph.D., P.E., D.GE., M.ASCE<sup>4</sup>

<sup>1</sup>Ministry of Transportation Ontario, 145 Sir William Hearst Ave., Room 223, Toronto, ON M3M 0B6. E-mail: [Tony.J.Sangiuliano@ontario.ca](mailto:Tony.J.Sangiuliano@ontario.ca)

<sup>2</sup>GeoSolv Design/Build Inc., 122 Creditstone Rd., Vaughan, L4K 1P2. E-mail: [jason@geosolv.ca](mailto:jason@geosolv.ca)

<sup>3</sup>Geopier Foundation Company, Inc., 130 Harbour Place Dr., Suite 280, Davidson, NC 28036. E-mail: [bmetcalfe@geopier.com](mailto:bmetcalfe@geopier.com)

<sup>4</sup>Geopier Foundation Company, Inc., 130 Harbour Place Dr., Suite 280, Davidson, NC 28036. E-mail: [kwissmann@geopier.com](mailto:kwissmann@geopier.com)

### Abstract

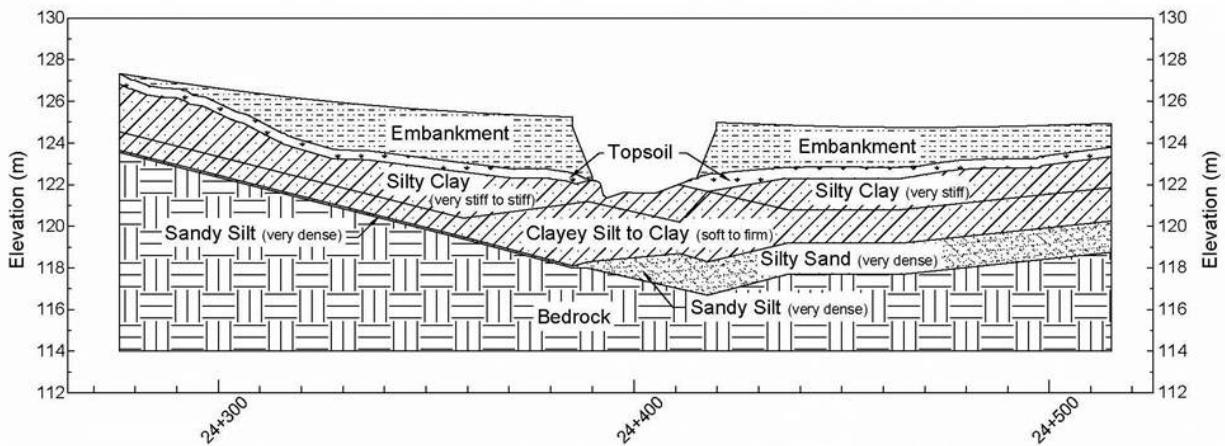
Densified aggregate piers have been widely used for ground improvement since the mid 1990's. The piers are typically constructed by backfilling cylindrical cavities with densified stone using a vertical ramming apparatus. The strength and compressibility of densified aggregate pier systems are confining stress dependent and tend to have low capacities in highly compressible soil because of their tendency to bulge into weak soil. This paper describes the design and construction of a densified aggregate pier system with polymeric shells for confinement in soft soil for a highway embankment in Seeley's Bay, Ontario, Canada. The method allows for the insertion of high density polyethylene (HDPE) sleeves into the ground through the soft materials using a specially adapted mandrel. This paper is of particular significance because it presents significant insight into an effective ground improvement method in weak and sensitive soil subject to shear strength degradation by traditional aggregate pier methods.

### INTRODUCTION

The Ministry of Transportation of Ontario (MTO) specified highway improvements for Highway 15 between Seeley's Bay and Crosby Creek, Ontario. The improvements included a new bridge and associated approach embankment construction over Crosby Creek. The geotechnical investigation identified the soil conditions below the bridge approach embankments as unsuitable for support of the embankments in their current condition. The MTO and geotechnical consultant sought to provide various options to support the embankments including, remove and replace with structural fill, preload and surcharge, light weight fill, wick drains, conventional aggregate piers and confined aggregate piers. The MTO ultimately selected ground improvement methods using a combination of conventional and confined aggregate piers to support the two approach embankments to provide increased bearing capacity, global stability, settlement control, and time rate of settlement.

**Subsurface conditions.** The subsurface conditions at the approach embankments generally consist of surficial topsoil underlain by 1 m of very stiff to stiff silt and clay over 2 m of sensitive soft to medium stiff silt and clay (Figure 1). The silt and clay deposit was underlain by dense silty sand to sandy silt till and/or bedrock. Groundwater was located within 0.5 metres from

ground surface. Oedometer and in-situ vane shear testing was performed on the sensitive silt and clay. The vane shear testing indicated peak undrained shear strengths of 20 to 60 kPa and remolded undrained shear strengths of 5 to 19 kPa in the silty clay, corresponding to sensitivities of 2 to 6.



**Figure 1. Subsurface conditions along embankment profile.**

**EMBANKMENT DESIGN**

MTO specified performance requirements for settlement control and global stability for the up to 4-metre high approach embankments and a minimum embankment design life of 75 years. The MTO specified factors of safety for static global stability of 1.3 and settlement specified as maximum post construction and differential settlements varying as the distance from the abutment increased as shown in Table 1.

**Table 1. Roadway post-construction total and differential settlement criteria.**

Distance from Abutment (m)	Maximum Total and Differential Settlement (mm)
0 – 20	25
20 – 50	50
50 -75	100
>75	200

Geotechnical analyses determined that with no ground improvement, primary consolidation settlement would be approximately 100 mm, and secondary compression settlements would be up to 60 mm over 20 years. Stability analysis indicated unacceptable global stability factors of safety at the abutment locations if founded on the existing soil conditions.

**Foundation support alternatives.** Given the required magnitudes of settlement and factors of safety against global stability, several options were considered for foundation support, including excavation and replacement of the sensitive silty clay and clay, lightweight fill, preloading with or without surcharge and/or wick drains and ground improvement. The advantages and disadvantages of each option were evaluated and the MTO decided to tender the project using a design/build ground improvement specification.

**Ground improvement.** The use of the densified aggregate pier system was selected as the preferred ground improvement option. Densified aggregate piers are constructed by compacting thin lifts of aggregate in cylindrical cavities. Compaction is performed by direct vertical ramming with a specially designed tamper head. The construction process may be performed using replacement (predrilled) or displacement techniques. The stiffness of the piers is dependent on the density of the compacted pier and lateral stress confinement provided by the matrix soil. Therefore, the softer and weaker the matrix soil, the more compressible the composite aggregate pier and matrix soil system. The use of densified aggregate piers in soft and organic soils typically results in the need for very close spacing of the aggregate piers and/or acceptance of larger settlement tolerances for moderate to heavy loads or large area pressures because of the propensity of the aggregate to bulge into the soft soils at higher loads.

Because the existing soil beneath the approach embankments consists of soft sensitive clay and given the varying settlement criteria, the use of a new confined densified aggregate pier, combined with more traditional densified aggregate piers was selected to support the embankments. The confined aggregate pier consists of the use of a High Density Polyethylene (HDPE) sleeve that provides lateral confinement of the pier through the soft soil, creating a very stiff element, allowing for better settlement control than standard densified piers.

The elements are constructed by inserting a hollow mandrel within 475 to 600 mm diameter conical HDPE sleeves and driving the sleeves to the design depths using a large static force augmented by high frequency vertical impact energy (Figure 2). The mandrel includes a hopper (at the top end) and serves as a conduit for delivering aggregate through the mandrel shaft and a specially designed valve (at the bottom end).

The HDPE sleeve remains in place after driving to the design depth. Aggregate is then placed inside the hopper propagating to the bottom of the mandrel and into the sleeve. The aggregate is compacted in lifts by incrementally raising and lowering of the mandrel using hydraulic crowd force and vertical ramming from the high frequency vibratory hammer. The process densifies the aggregate vertically and forces the aggregate laterally into confining sleeve, causing the sleeve to expand slightly outward against the soft matrix soil. Once the aggregate is compacted within the sleeve, the mandrel is incrementally raised and lowered, allowing the aggregate to flow into the displaced cavity above the sleeve and forming dense aggregate pier lifts in the stiffer upper soil. A schematic of the finished construction process is shown in Figure 3.

The need for the utilization of the confining sleeve is based on the applied top of pier stress, the shear strength of the specific soil in a given area, and the required amount of settlement control for the project.



Figure 2. Confined aggregate pier construction.

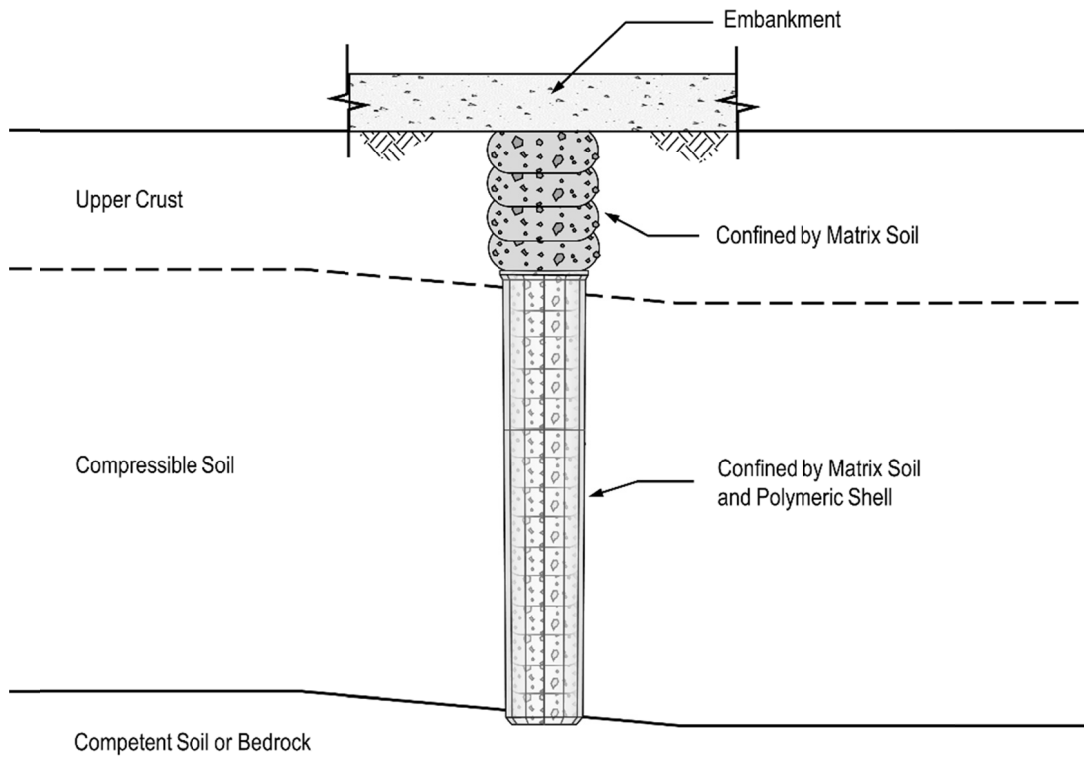
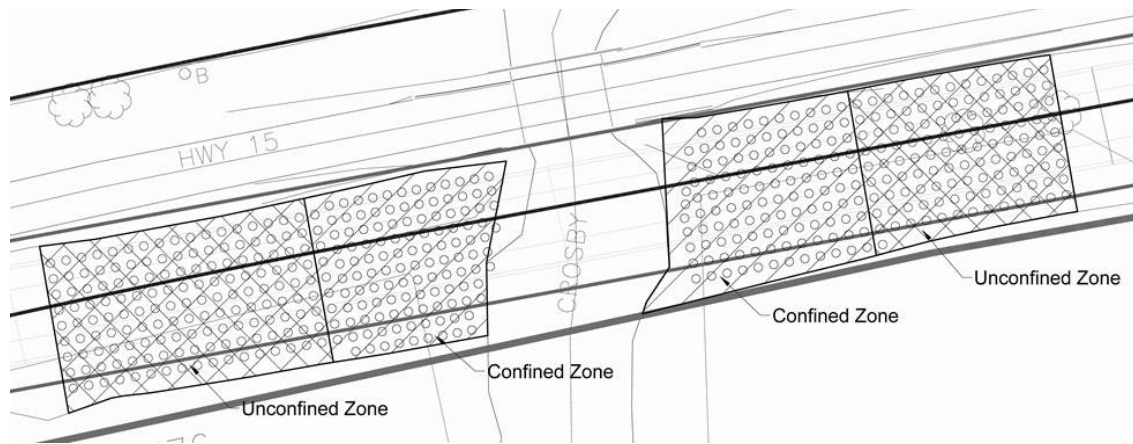


Figure 3. Schematic of finished construction process.

### DESIGN OF THE AGGREGATE PIER SYSTEM

**Ground improvement layout and design.** The final ground improvement design for the Crosby Creek approach embankments consisted of approximate 550 mm diameter confined aggregate pier elements installed near the abutment at a spacing of 2.1 m to 2.3 m center to center to a distance of 25 m away from the abutment where settlement tolerances are lower. At lateral distances of greater than 25 m from the abutment, 600 mm diameter traditional aggregate pier elements were installed on a spacing of about 2.3 m between 25 m and 65 m from the abutments, as shown in Figure 4 below.



**Figure 4. Ground improvement layout.**

**Design for settlement.** Design for settlement control is carried out using the method proposed by Wissmann, et.al., 2002, where settlements are evaluated in both the aggregate pier reinforced zone (Upper Zone) and the lower native matrix soil zone (Lower Zone). The elements extend to hard materials and thereby the lower zone is expected to be negligible.

The Upper Zone settlement calculations are completed by implementing a composite elastic modulus  $E_{comp}$ , computed from the weighted average of the elastic modulus of the aggregate pier element and the equivalent elastic modulus of the matrix soil:

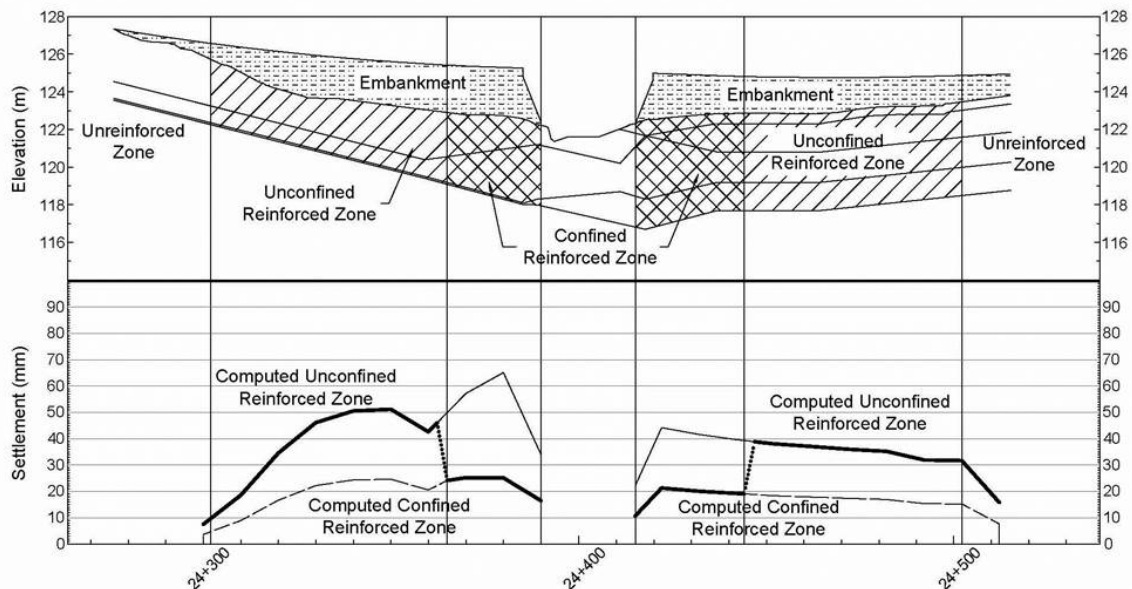
$$E_{comp} = (1 - R_a)E_m + R_aE_g \tag{1}$$

Here,  $E_g$  is the elastic modulus of the aggregate pier element,  $E_m$  is the elastic modulus of the matrix soil, and  $R_a$  is the ratio of the area coverage of the aggregate pier elements to the gross footprint area. Design parameter values used for the Crosby creek project are shown in Table 2 below.

**Table 2. Design parameter values.**

	Parameter	Design Values
Upper Zone (Bearing)	Aggregate Pier (with confining sleeve) Elastic	190 MPa
	Aggregate Pier (without confining sleeve) Elastic	72 MPa
	Matrix Soil Elastic Modulus, $E_m$	2 MPa
Lower Zone (Compressibility)	LZ Elastic Modulus, $E_{Lz}$	Incompressible

Settlement estimates for the approach embankments were performed using commercially available software. Settlements were calculated using elastic modulus relationships, as described above. The applied stresses within the composite soil profile were estimated using Boussinesq elastic theory. The estimated settlement profiles along the centerline of the approach embankment for both confined aggregate piers and for traditional unconfined aggregate piers are shown in Figure 5. The design settlement of the embankment within 25 m of the abutment was limited to 25 mm using confined aggregate pier elements. Embankment settlement at lateral distances greater than 25 m from the abutments was controlled to about 50 mm using unconfined aggregate piers.



**Figure 5. Computed settlement profile along centerline of approach embankments using confined and traditional aggregate piers.**

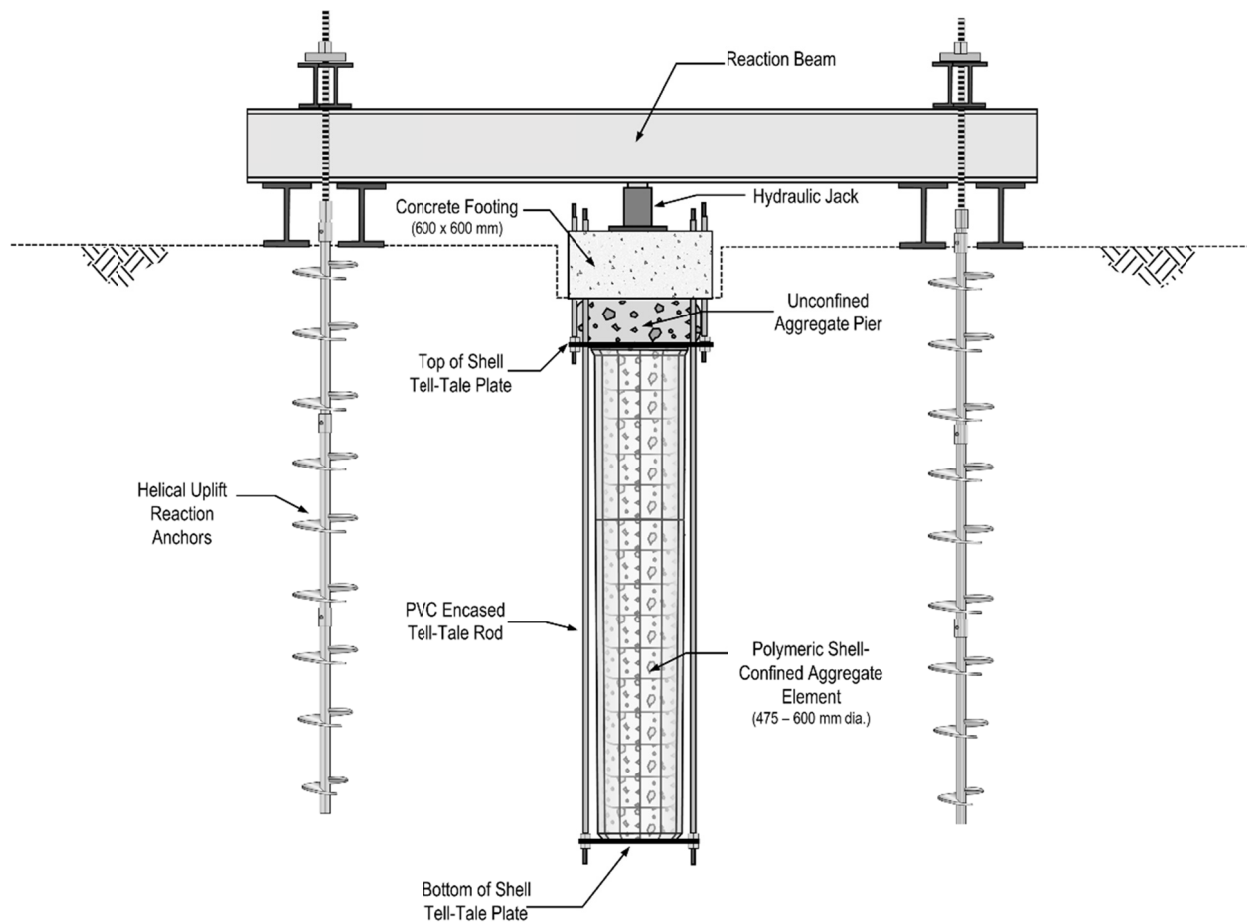
**Confirmation Testing.** Site-specific verification of the aggregate pier designs is performed by conducting a full-scale modulus test (Figure 6) to verify the element stiffness. The modulus test set-up is similar to a pile load test configuration and is performed in general accordance with ASTM D-1143. During the installation of the compression test pier, sleeved steel telltales were positioned at the pier bottom and at the top of the confining sleeve. The telltale rods are sleeved through a 600 mm by 600 mm square footing constructed 600 mm below grade. The top of the confining sleeve was located 200 mm below the bottom of the concrete footing. Measurements of compression are made for the 200 mm thick unconfined aggregate pier / gravel pad, the top, and bottom of the confined aggregate pier to evaluate the stiffness and deformation behavior of the combined element.

The results of the modulus testing for the Crosby Creek confined aggregate pier (Figure 7) indicated a total top-of-pier displacement of 23 mm, which included a top-of-confined pier displacement of 10 mm and bottom-of-pier displacement of 6 mm. These results show that more than one-half of the total settlement is attributable to the 200 mm unconfined pier “stem” emphasizing the need to obtain good compaction of the upper aggregate materials.

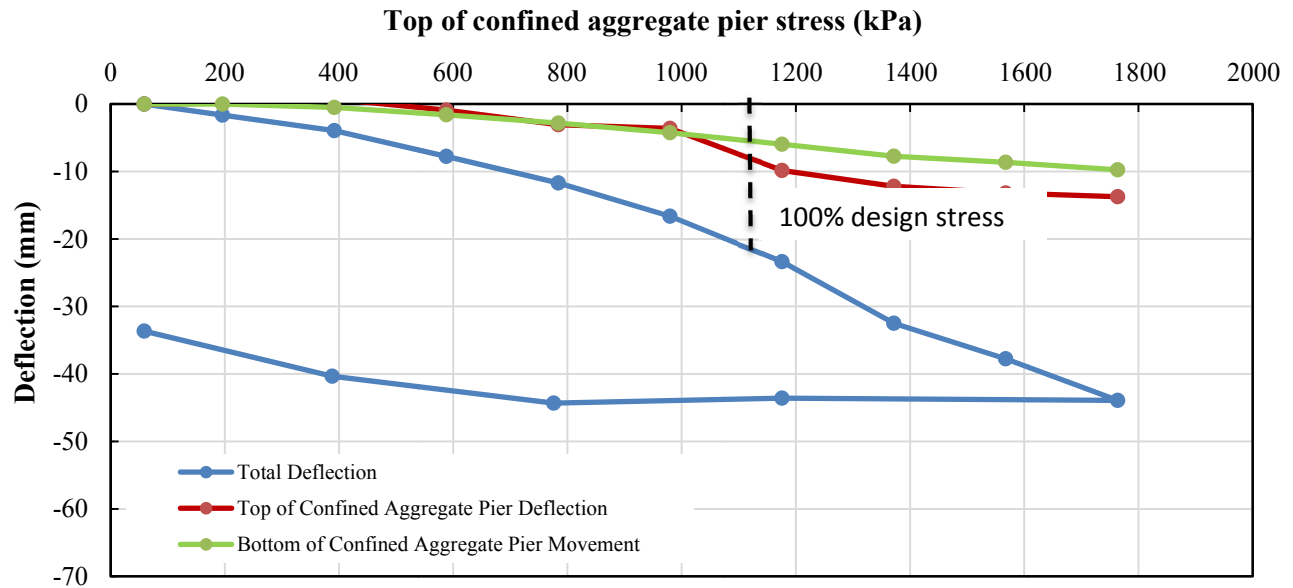
The elastic modulus of the confined and unconfined aggregate pier can be estimated using Hooke’s Law:

$$S = \frac{\bar{\sigma}H}{E_g} \quad [2]$$

Here,  $S$  is the compression of the confined or unconfined aggregate pier,  $\bar{\sigma}$  is the average stress applied and  $H$  = the length of the pier. For conditions where the average pier stress is one-half of the top of pier stress, the computed elastic modulus for the confined pier is then 635 MPa. For conditions where the average pier stress is equal to the top of pier stress, the computed elastic modulus for the confined pier is 1270 MPa. These values are 3 to 8 times the assumed design value.



**Figure 6. Full scale modulus test setup.**



**Figure 7. Confined aggregate pier modulus test results.**

**Design for Global Stability.** The installation of the aggregate pier system increases the composite shear strength parameter values within the aggregate pier-reinforced zones. The composite shear strength parameter values are estimated using the following equations (Barksdale and Bachus 1983, Mitchell et al. 1981, FitzPatrick and Wissmann 2002):

$$\phi_{\text{comp}} = \tan^{-1}[R_a \tan \phi_g + (1 - R_a) \tan \phi_m], \quad [3]$$

$$C_{\text{comp}} = [R_a c_g + (1 - R_a) c_m], \quad [4]$$

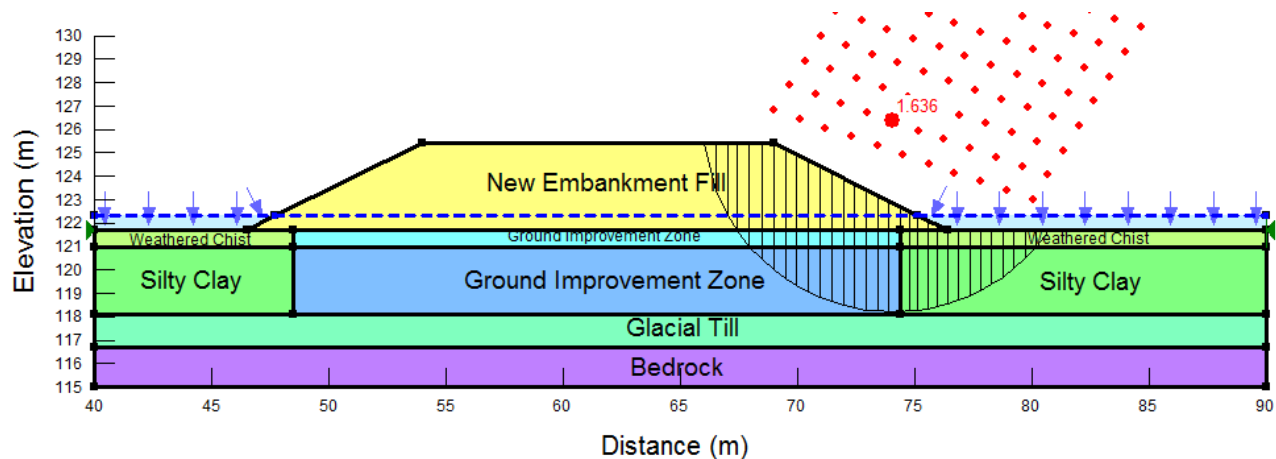
Here,  $R_a$  is the area replacement ratio,  $\phi_g$  is the friction angle of the aggregate pier,  $\phi_m$  is the friction angle of the matrix soil,  $c_g$  is the aggregate pier cohesion and  $c_m$  is the matrix soil cohesion. The aggregate pier shear strength is represented by effective stress (drained) design parameter values because of the drainage afforded by the aggregate. The confined aggregate pier portion is also represented by drained parameter values because of the confinement provided by the HDPE sleeve and negligible pore pressure increases within the element. Of interest, this approach conservatively ignores the added shear resistance provided by the ductile HDPE confining sleeve. The design matrix soil parameter values may be evaluated using both undrained (short-term) and drained (long-term) parameter values.

Limit equilibrium analyses were performed using commercially available software to evaluate the factors of safety against global stability for the embankments. The parameter values used for the analysis of each wall section are shown in Table 3 and results of the stability analysis for undrained soil parameter values and the 4-metre-high embankment section are shown in Figure 8. The computed factor of safety of 1.6 is greater than the specifications value of 1.3.



**Table 3. Undrained soil parameter values for global stability**

Type of Soil	Su (kPa)	$\phi$ , (°)
Confined Aggregate Pier	0	45
Matrix Soil (silty clay crust)	128	0
Matrix Soil (sensitive silty clay)	45	0

**Figure 8. Results of static undrained global stability analysis.**

## PERFORMANCE

Optical surveys were performed to monitor the settlement of the embankments. The measurements showed that settlements were less than 20 mm within the confined aggregate pier zone at distances within 25 m from the abutments and less than 40 mm within the unconfined aggregate pier zone at distances greater than 25 m from the abutments. The measured embankment performance is in close agreement to the predicted performance shown in Figure 5. Inclometers installed adjacent to the embankments near the abutments showed less than 25 mm of lateral movement with most of the lateral movement confined to the upper 2 metres of the soil profile.

## CONCLUSION

The combined use of the confined and unconfined aggregate piers allowed for the ability to tailor the pier spacing and type of aggregate pier systems to achieve the variable settlement criteria. The design-build efforts provided for a cost-effective foundation support solution that met the required schedule and performance needs of the MTO when compared to traditional foundation support solutions.

## REFERENCES

American Society for Testing and Materials. "Standard Test Methods for Deep Foundations Under Static Axial Compressive Load." ASTM D1143 / D1143M-07(2013). ASTM International.

- Barksdale, R.D. and Bachus, R.C. 1983. Design and Construction of Stone Columns, *FHWA/RD 83/026*, Vol. I. Report No. 1, Federal Highway Administration, 210 pp.
- FitzPatrick, B.T. and Wissmann, K.J. 2002. *Technical Bulletin No. 5 – Geopier Shear Reinforcement for Global Stability and Slope Stability*, Geopier Foundation Company, Inc. Blacksburg, Virginia.
- Mitchell, J.K. 1981. Soil Improvement: State of the Art, *Tenth International Conference on Soil Mechanics and Foundation Engineering*, Session 12. Stockholm, Sweden. June 15 – 19.
- Wissmann, K.J., FitzPatrick, B.T., White, D.J., and Lien, B.H. 2002. Improving global stability and controlling settlement with Geopier soil reinforcing elements, *Proceedings, 4th International Conference on Ground Improvement*, Kuala Lumpur, Malaysia, 26 – 28 March.