# The Use of the Geopier® Intermediate Foundation® System for Support of Seven Wind Turbine Foundations in Eastern Ontario

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ABSTRACT
The use of wind energy in Ontario has been steadily increasing, and multiple farms of a few to hundreds of wind turbines are being constructed at any given time in Ontario. The need for high stiffness foundations due to large overturning moments, and therefore, large edge pressure acting on the base of the foundation can prove challenging when soft or compressible soils at the turbine site do not provide adequate bearing capacity. The use of Geopier® Intermediate Foundations® has been used to provide increased bearing capacity and foundation stiffness for hundreds of wind turbine foundations around the world.

RÉSUMÉ
L'utilisation de l'énergie éolienne en Ontario est en croissance constante, et il arrive que plusieurs parcs éoliens soient en cours de construction en même temps. Les moments de renversement élevés et donc des grandes pressions agissant sur la base des fondations peuvent poser des problèmes pour les fondations dans les sols mous ou compressibles qui ne fournissent pas la capacité portante suffisante. L'utilisation de Geopier® Intermediate Foundations® a été utilisée pour augmenter la capacité portant et la rigidité de la fondation pour des centaines de fondations éoliennes.

## 1 INTRODUCTION

Wind power generation projects have been developing rapidly throughout Canada. Various government incentives are in place to encourage the expansion and development of these projects. The Feed-in Tariff (FIT) program developed by the Province of Ontario has led to a boom in wind power generation.

As of December 31, 2014, Canada's wind energy developments have resulted in a total installed capacity of 9,694 MW (Canadian Wind Energy Association).

These wind farm projects are often located in rural areas and on active agricultural lands. Although these turbine towers have typically light gravity loads, they are subject to large overturning moments. The stresses imposed by these overturning moments can often exceed the bearing capacity of the in situ soils, forcing the project team to look for alternative tower locations or foundation options including ground improvement.

Where existing soils are deemed unsuitable for the support of tower foundations, ground improvement methods provide increased shear strength and reduce soil compressibility, offering cost effective solutions at these sites. Ground improvement techniques have been successfully utilized to support turbine towers and reduce both the project cost and the construction schedule. These techniques can avoid both massive over-excavation and re-engineering of poor soils or costly deep foundation systems.

Recently, a project in Southern Ontario was completed using ground improvement techniques due to poor soil conditions which were unsuitable for the proposed spread footing loads.

### 1.1 Project Description

The project in Southern Ontario consisted of the installation of ten direct-drive wind turbines, each with a capacity of 3 MW for a total farm capacity of 30 MW. With hub heights of about 100 m, the turbines installed at this site are the largest currently installed in Ontario for both tower height and capacity.

The turbine towers were founded on a large 19800 mm octagonal foundations which taper from the footing edge toward center at an embedment depth of approximately 3.5 meters.

### 1.2 Site Description and History

The wind farm, located inside the municipality of South Dundas was constructed on existing agricultural lands mainly used for the production of corn and soybeans as well as dairy farming.

The installation of the turbines, access roads, and electrical collection systems was made feasible though long-term lease and easement agreements with a small group of local land owners. The wind farm development was also made possible through the award of a Feed-in Tariff (FIT) contract.

The project was successfully completed during the winter of 2014/2015.

1.3 Subsurface Conditions

Two geotechnical investigation reports were prepared to outline the expected geotechnical conditions at each tower location and to make recommendations for the design of the tower foundations. Based on the information provided, the soils encountered at the tower sites generally consist of a surficial layer of topsoil underlain by 1.5 to 3.0 m of firm to stiff silty clay, lean clay and fat clay with varying amounts of silt and sand. Zones of the highly variable soils for the seven towers are considered to be soft and sensitive shoreline marine clay deposits. Below these soils is a layer of soft to medium stiff clays with moisture content of 20 to 40 percent to a depth of 4.6 to 12.2 m or clayey sands to a depth of 4.0 to 13.4 m. It should be noted that at one tower location very loose to loose silty clayey sand 8.2 m thick was encountered. Stiff to hard silty clay and lean clay with relatively low moisture content were present below the soft clay or clayey sand layer, and extended to depths of 5.8 to 15.8 m below the existing ground surface, over limestone bedrock in several boreholes.

Groundwater is located between 1.1 m and 7.6 m below the existing ground surface across the entire site. The towers were sited typically along the high bank edge of a natural floodplain.

Three of the ten towers were deemed to have suitable bearing resistance to allow standard spread footings.

For the purpose of this paper, the seven remaining tower sites are divided into two groups based on differences in soil strength as outlined in the table below:

Table 1: Towers Grouped by Soil Conditions

| Tower Group | Tower |
|---|---|
| Group 1 | 5, 7, 10 and 12 |
| Group 2 | 9, 13, 15 |

The subsurface soils for the Group 1 towers were insufficient to support the foundation loads. The SPT ‘N’ values and the virgin / remoulded undrained shear strengths measured during the geotechnical investigation at these sites are shown below in Figures 1 and 2, respectively.

As shown in Figure 2, the sensitivity of the clay ranges from 1.6 to 7.25 averaging 3.5 at the Group 1 towers, indicating that the sensitivity of the clay soils ranges from low to medium.

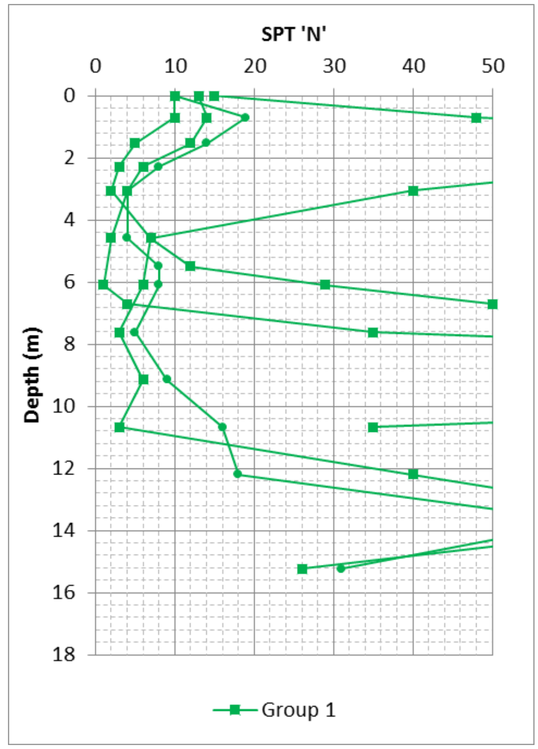


Figure 1. Pre-installation SPT `N` values vs. depth for Group 1

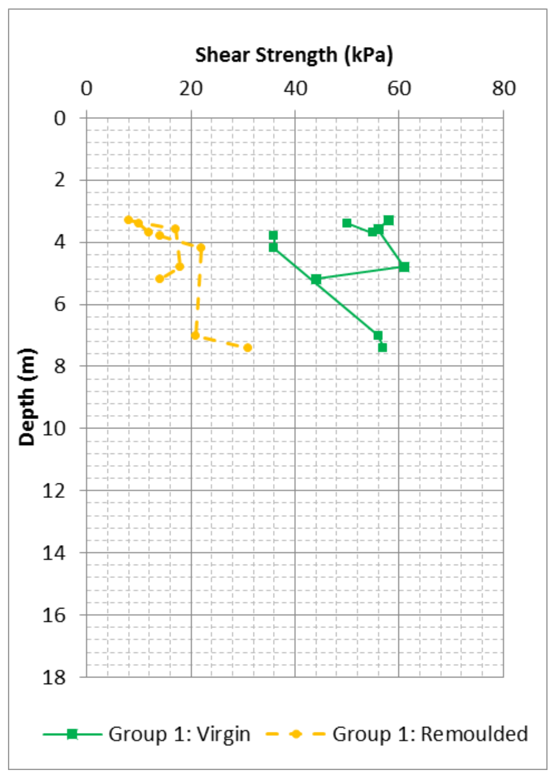


Figure 2. Pre-installation undrained shear strength vs. depth for Group 1

At Tower Group 2 the subsurface conditions were weaker than Tower Group 1. The SPT 'N' values and undrained shear strengths with respect to depth at these tower locations are presented in Figures 3 and 4, respectively.

The sensitivity of the clay ranges from 2 to 8, averaging 3.5 at the Group 2 towers indicating the clay soils to be low to high sensitive.

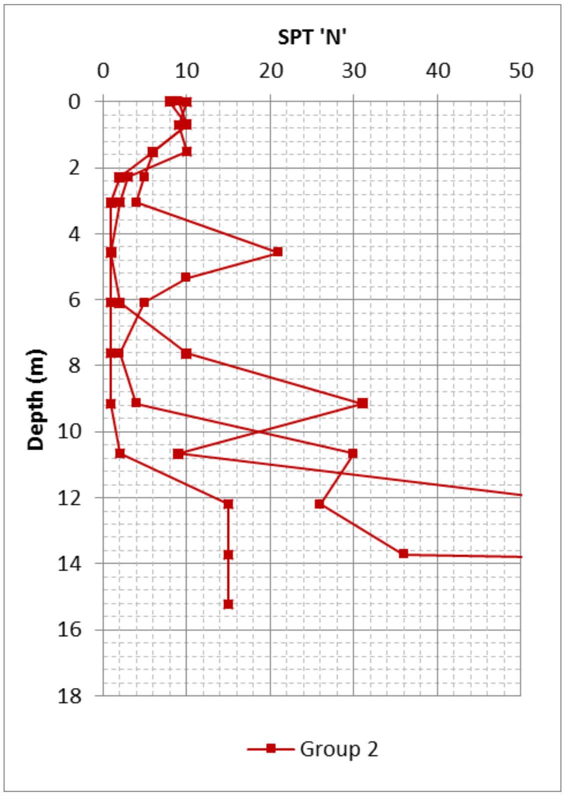


Figure 3. Pre-installation SPT `N` values vs. depth for Group 2

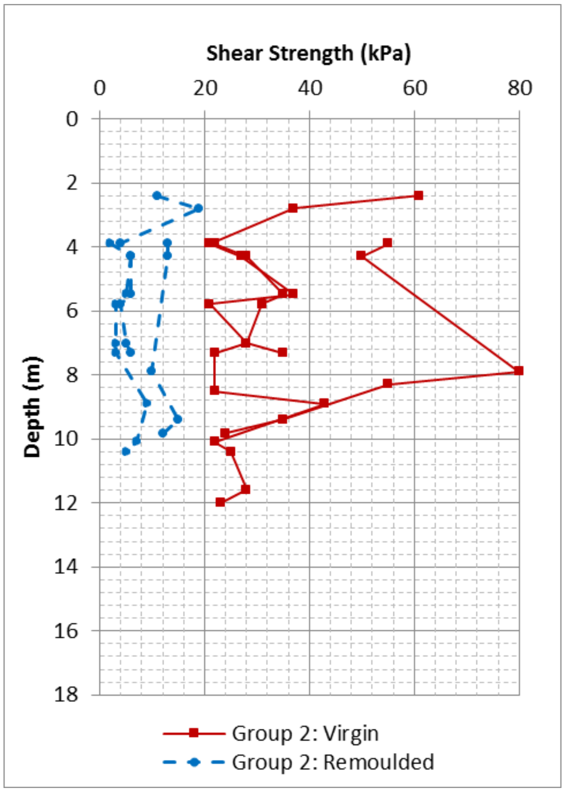


Figure 4. Pre-installation undrained shear strength vs. depth for Group 2

1.4 Design Considerations

The loading conditions for the wind tower foundations considered several loading scenarios resulting from the ultimate parked/idling gusts loads and the ultimate shutdown gust loading, resulting in horizontal shear, vertical compression loads and moments at and about the tower base flange. The typical forces and moments at the tower base flange are presented in Figure 5.

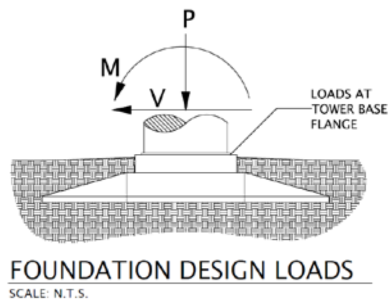


Figure 5. Typical Forces and Moments at a Wind Tower

The magnitude of resultant forces and moments for the envelope loading case yielded an overturning moment of Mr = 83,700 kN.m, a horizontal force, V = 980 kN; and a vertical load, P = 4,100 kN at the extreme wind loading condition. Foundation loads were resolved to a triangular distribution with a pressure of 0 kPa at one edge of the foundation increasing to 190 kPa at the opposite end of the foundation. The manufacturer required a minimum rotational stiffness around the horizontal axis of the tower base of 1500 MNm/deg and the support of the tower foundation was required to control the differential settlement to a maximum rotation at the end of the service life to no more than 0.15 degrees from the horizontal plane.

The potential long-term settlement of the turbine foundations were assessed by considering settlement caused by the one-year extreme load case projected as a sustained load in one direction over the 20 year life of the turbine.

## 2 METHODOLOGY

In order to avoid using more costly deep foundations systems, the project team decided to move forward with various ground improvement methods to reinforce the soils below the tower foundation to satisfy the design requirements.

The turbine foundation was designed to limit the applied stresses at the bottom of the footing to compressive forces only. As a result, the Geopier ground improvement design and techniques did not incorporate any tie down anchors for uplift resistance.

The settlements were analyzed based on applied envelope pressure distributions to calculate upper (Geopier reinforced zone) and lower (native soil) zone settlements as well as the expected differential edge to edge settlements for the improved ground. To evaluate the settlement, the typical settlement criterion was considered, generally established for wind towers, at about 50 mm total settlement and 3mm/m of differential settlement to limit the overall tilt to less than 0.15 degrees.

### 2.1 Geopier Rammed Aggregate Piers® (RAPs)

Rammed Aggregate Piers were installed at the Group 1 tower sites using the GP3® Method, drilling a 0.75 m diameter cavity and ramming thin lifts of well-graded aggregate within the cavity to form very stiff, high-density aggregate piers. The drilled holes typically extended from 3 to 7.5 m below grade and 2 to 6 m below footing bottoms. The first lift of aggregate formed a bulb below the bottom of the pier, thereby pre-stressing and pre-straining the soils to a depth equal to at least one pier diameter below the base of the drill cavity. Subsequent lifts were typically about 300 mm to 600 mm in thickness. Ramming took place with a high-energy beveled tamper that both densified the aggregate and forced the aggregate laterally into the sidewalls of the drill cavity. This action increased the lateral stress in surrounding soil; thereby further stiffening the stabilized composite soil mass. Figure 6 below shows the general installation process used at this tower group.

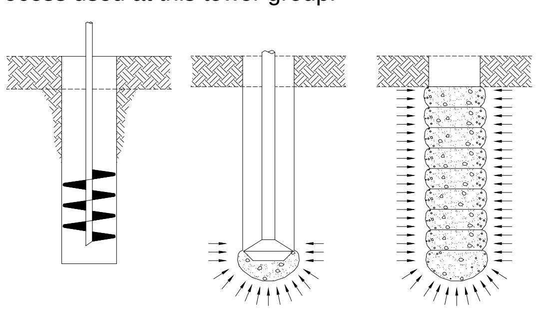

Figure 6. Installation process of the GP3® System

The results of the Geopier installation were significant strengthening and stiffening of the subsurface and increased bearing capacity.

Geopier soil reinforcing elements were designed to control foundation settlements to the project design criteria. Foundation settlements were estimated by summing the estimated settlement in the Geopier reinforced zone (the "upper zone") and the estimated settlement in the zone of soil below the bottoms of the Geopier elements (the "lower zone") in accordance with the methodology described by Lawton et al. (1994).

At one of the Group 1 tower locations, collapsing soils were encountered within a portion of the foundation, and a slightly modified displacement installation method was employed to install the RAP elements instead of the replacement method used in the remainder of the Group 1 towers.

### 2.2 Geopier GeoConcrete™ Columns (GCCs)

At the Group 2 towers, GCC elements were installed consisting of a 14 inch (355 mm) diameter shaft and a 24 inch (600 mm) diameter, 24 inch (600 mm) tall bottom bulb founded on the stiff to very stiff silty clay. A Load Transfer Cushion was constructed on top of the GCC elements. The GCC elements were arranged below footings to support the resultant applied pressures imparted by the compression and overturning moments

## 3 DESIGN OF GEOPIER GROUND IMPROVEMENT FOR WIND TURBINES

The Geopier design for these wind turbine considered all load cases including the static, mean production pressure and abnormal extreme pressures (FitzPatrick, 2009) and was designed to provide adequate bearing capacity and settlement control.

### 3.1 Bearing Capacity

The bearing capacity of the Geopier reinforced foundation was analyzed using standard limit equilibrium analysis. The shear strength used in the limit equilibrium analysis considered a composite shear strength dependent on both the matrix soil and pier strength characteristics.

### 3.2 Settlement Control

Settlement control was estimated using the method by Lawton and Fox, 1994, where, settlements are broken up into two different zones, the Geopier reinforced zone (Upper Zone) and the lower native matrix soil zone (Lower Zone).

The upper zone settlements were estimated using the a ratio of the top of pier stress ($q_g$) and the pier stiffness ($k_g$), where the pier stiffness is confirmed using an onsite modulus test and the top-of-pier stress is calculated as

$$q_g = q \times [R_s/(R_sR_a-R_a+1)] \quad (1)$$

where $q_g$ is the top of pier stress, $R_s$ is pier to soil stiffness ratio, and $R_a$ is the pier to foundation area ratio. For this project upper zone settlements of 13 mm were estimated for the GCCs and 13 to 21 mm were calculated for the RAPs.

The Lower zone settlements were estimated by using conventional settlement analysis approaches, using a Bousinesq stress distribution below the foundations.

## 4 SITE SPECIFIC MODULUS TESTING AND RESULTS

### 4.1 Rammed Aggregate Pier Modulus Testing

Traditional site-specific verification of the RAP design were performed by conducting a full-scale modulus test as depicted in Figure 7.

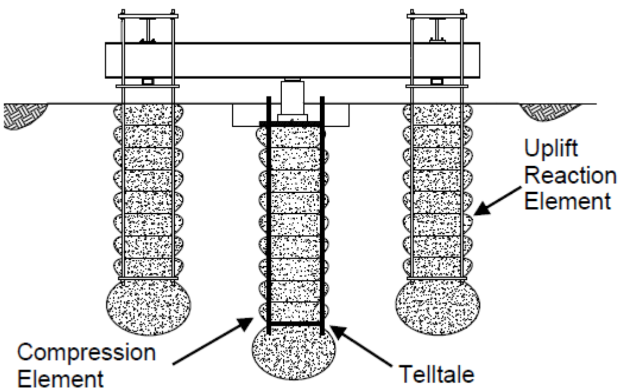


Figure 7. Full scale modulus test setup

#### 4.1.1 Modulus Test Procedure

The modulus test set-up is similar to a pile load test configuration and the test is performed in general accordance with ASTM D-1143. During the installation of the compression test pier, sleeved steel telltales were positioned near the pier bottom and extends to the surface allowing measurements of deflection near the pier bottom. The resulting top-of-pier and telltale deflections were used to evaluate the stiffness of the modulus and deformation behavior of the element.

#### 4.1.2 Modulus Test Results

Full scale modulus testing was conducted at the Group 1 tower sites. The results of this testing (Figure 8) indicated stiff performance of the Geopier elements with total displacement at the top of the pier ranging between 5.5 mm to 10.7 mm at 100% of the design load. The displacement of the bottom of the test pier was also recorded and indicated deflections of less than 0.5 mm. The applied load on the Geopier test element was increased to 150% of the design load for one of the two full scale modulus tests and total top of pier displacements of about 21 mm and bottom of pier deflection of less than 1 mm were recorded. The second Geopier test element was loaded to a top of pier stress equal to 200% of the design load and total top of pier displacements were recorded as less than 22 mm. The bottom of pier displacements at the 200% load increment were not measured due to interference with the strain gauges; however, the total bottom of pier displacement at 175% of the design load was measured as less than 0.4 mm.

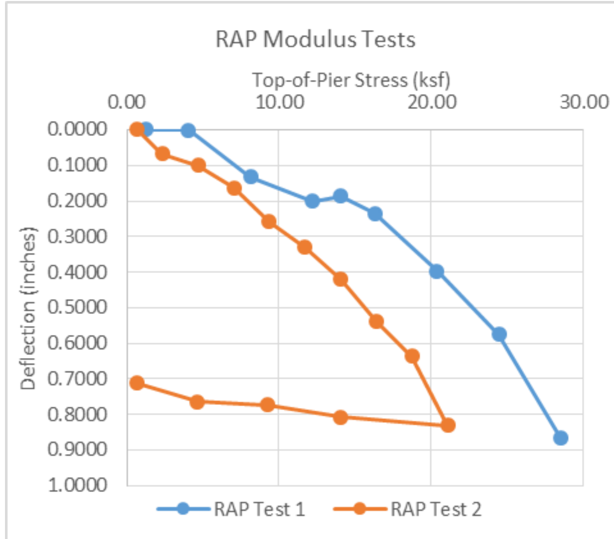


Figure 8. Rammed Aggregate Pier Modulus Test Results

### 4.2 GeoConcrete Column Modulus Test Procedure and Results

Traditional site-specific verification of the GeoConcrete elements was performed by conducting a full-scale modulus test as depicted in Figure 7 with the exception that the uplift reaction elements and the compression test element were installed using the Geopier GeoConcrete installation method. Uplift capacity for the modulus test reaction anchors was generated by installing a central uplift threadbar through the GeoConcrete element as opposed to more than one bar on the outer edges of the element as depicted in the Figure.

#### 4.2.1 Modulus Test Procedure

The modulus test set-up was conducted in a similar manner to the Rammed Aggregate Pier modulus test. During the installation of the compression test pier, a sleeved steel telltale was positioned near the pier bottom and extended to the surface allowing measurements of deflection near the pier bottom. Plots of the stress versus deflection for both the top of pier and telltale responses were constructed from the modulus test results and used to evaluate the stiffness of the modulus and deformation behavior of the GeoConcrete element.

#### 4.2.2 Modulus Test Results

Full scale site-specific modulus testing was conducted at each of the three tower site where Geopier GeoConcrete Columns were installed to support the tower base reactions to the project requirements. The results of this testing (Figure 9) indicated very stiff performance of the GeoConcrete elements with total displacements at the top of the pier ranging between 5.5 mm to 6.9 mm at 100% of the design load. The displacement of the bottom of the test pier was also recorded and generally mirrored the response of the top of the pier.

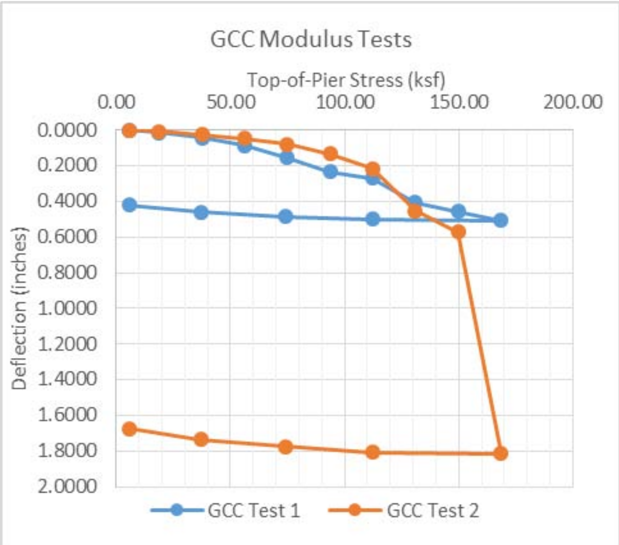


Figure 9. GeoConcrete Column Modulus Test Results

## 5 DISCUSSION

The modulus and deformation behavior measured during the full scale testing of both the Rammed Aggregate Pier elements and the GeoConcrete elements described above exceeded the project requirements. The soft consistency and sensitivity of the clay required ground improvement in order to used standard spread footings. Where lower soil sensitivity and higher shear strength was encountered, the RAP system was able to develop adequate shaft friction to shed the load applied to the tops of the piers; however, where the lower shear strengths and higher sensitivity soils were encountered enough shaft friction could not be achieved resulting in the need for the use of the GCC system, where the piers founded on a more competent soil.

The total deformation was measured to be below the anticipated values and no significant movement (plunging) of the bottom of these elements were recorded. The stiff response measured at the bottom of the Rammed Aggregate Pier test elements indicates that the designed shaft lengths were sufficient to transfer the applied load though friction along the shaft length and no significant tip stress was generated below the pier.

## 6 CONCLUSIONS

The installed ground improvement systems exceeded the vertical, horizontal, and rotational deformation requirements and were determined to be appropriate to support the design loads of the largest wind turbines installed in Ontario to date. The Geopier system was able to reinforce the soft sensitive clay, allowing for the turbines to be founded on spread footings rather than more costly deep foundations.

## 7 ACKNOWLEDGEMENTS


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