

Lateral and uplift resistance of Geopier-supported footings

Resistencia a cargas laterales y de arranque de zapatas apoyadas sobre Geopier

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This paper discusses the behavior of Geopier-supported shallow foundation systems when subjected to lateral and uplift (tensile) loads. Lateral and tensile loads are often applied to foundation systems when the supported structures are subject to wind or seismic events and also to lateral earth pressures. Geopier - supported shallow foundations provide resistance to lateral loads using mechanisms identical to those applicable to conventional shallow footings. These mechanisms include passive earth pressure adjacent to the footings and sliding resistance along the base of the footings. However, because of the high stress concentration to the Geopier elements and the high friction angle of the Geopier aggregate, greater resistance is achieved in comparison to a footing supported by soil not reinforced by Geopier elements. Uplift anchors are incorporated into Geopier elements to resist tensile loads. The anchors consist of a steel plate installed at the bottom of the piers and threaded bars connected to and extending from the embedded plate to the overlying footing. This paper describes methods used to design Geopier-supported footings to resist lateral and uplift load demands. Results of lateral and uplift loading tests using full-scale footing are also presented.

Keywords: lateral and uplift loads, shallow foundations, compacted aggregate piers Geopier, full scale lateral and uplift load tests

Este artículo trata sobre la respuesta a cargas laterales y de arranque (tensión) de sistemas de fundaciones superficiales apoyados sobre pilas de agregado compactado (Geopier). Las cargas laterales y de tracción se aplican comúnmente a sistemas de fundaciones que soportan estructuras sometidas a viento y eventos sísmicos así como a empujes laterales. Fundaciones superficiales apoyadas sobre Geopiers resisten cargas laterales por medio de mecanismos similares a los aplicados en zapatas superficiales convencionales. Estos mecanismos incluyen empuje lateral sobre las zapatas y resistencia al deslizamiento a lo largo de la base de la zapata. Sin embargo, debido a la alta concentración de tensiones en los elementos Geopier y al alto valor del ángulo de fricción del agregado, se alcanza una mayor resistencia en comparación a una zapata apoyada sobre suelo no reforzado con elementos Geopier. Anclajes de arranque son incorporados en los elementos Geopier para resistir las cargas de tracción. Los anclajes están compuestos de una placa de acero instalada al fondo de la pila y barras con resaltes conectadas a esta placa se extienden hasta alcanzar la zapata sobreyacente. El artículo describe métodos usados para diseñar zapatas apoyadas sobre Geopier que resistan tanto cargas laterales como de arranque. También se presentan resultados de ensayos de carga lateral y de arranque en terreno a escala natural.

Palabras clave: cargas laterales y de arranque, fundaciones superficiales, pilas de agregado compactado Geopier, ensayos de carga lateral y de arranque a escala natural

Lateral load demands

Lateral load demands on structures, retaining walls, and buildings can be generated by horizontal earth pressure, wind and earthquakes. Lateral loads transmitted through

a structure are resisted by the foundation system. Geopier-supported shallow foundations resist lateral loads with mechanisms identical to those applicable to conventional shallow footings (Figure 1):

- Passive earth pressures adjacent to the footing.
- Base sliding resistance along the bottom of the footing.



The combination of stress concentration to the stiff Geopier elements and the high friction angle of the Geopier aggregate allows for the development of a significantly greater amount of lateral load resistance than developed by footings not supported by *Geopier* elements.

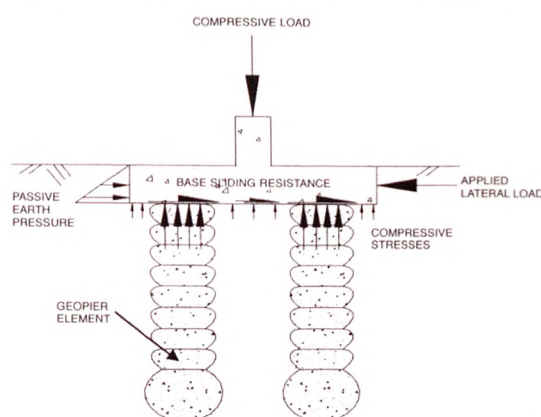


Figure 1: Lateral load resistance of *Geopier*-supported footing

Construction

Geopier elements are constructed by drilling out a volume of compressible soil to create a cavity and then ramming select aggregate into the cavity in thin lifts. *Geopier* construction results in a very dense aggregate column, wherein the aggregate tends to dilate when subject to shearing stresses. This construction process allows for a high level of confidence in the design friction angle used for rammed *Geopier* aggregate. *Geopier* construction is again touched in the uplift load section and is, also described by Fox and Cowell (1998).

Shear strength

Full-scale direct shear tests performed on 762 mm diameter *Geopier* elements and small-scale laboratory triaxial tests performed on reconstituted samples demonstrate that the angle of internal friction for *Geopier* aggregate ranges from 49° to 52°, depending on gradation. Results obtained from the full-scale direct shear tests performed on *Geopier* elements are shown in Figure 2. *Geopier* elements constructed using both well-graded base coarse stone and open-graded (#57) stone were tested.

Small-scale laboratory triaxial tests were performed at Iowa State University on reconstituted samples of well-graded *Geopier* aggregate compacted to densities consistent with those measured for installed *Geopier* elements. Test results, illustrated in Figure 3, indicate an angle of internal

friction of 51°. The high friction angles measured in the field and laboratory tests are attributed to the high density and the dilatant behavior of the aggregate produced during the high energy ramming of the crushed stone used in *Geopier* elements.

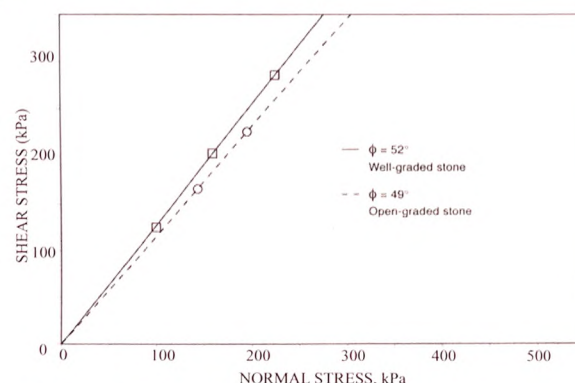


Figure 2: Results of full-scale direct shear testing performed at the tops of *Geopier* elements (Fox and Cowell, 1998)

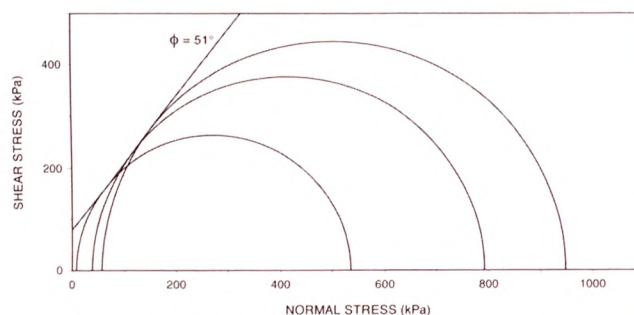


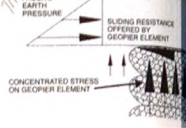
Figure 3: Results of triaxial testing of compacted *Geopier* aggregate (White, 2001)

Lateral load resistance

Lateral loads transmitted to shallow foundations are resisted by sliding resistance along the base of footings and by passive earth pressure that develops at the front of the footing as it is pushed into the adjacent soils (Figure 4). Although additional lateral load resistance is offered by the bending of the vertical bars in elements outfitted with uplift anchors, this additional resistance is small in comparison with other resistances at small values of lateral deflection. Computations indicate that the component of lateral loading resistance provided by sliding resistance is typically much greater than the component provided by passive earth pressure.

Sliding resistance at the base of the footings

As shown in Figure 4, sliding resistance at the base of *Geopier*-supported footings may be divided into sliding



resistance between the footing and the tops of the *Geopier* elements and sliding resistance between the footing and the matrix soil.

The resistance to sliding provided by the *Geopier* elements F_g is computed as the product of the normal (downward) stress on the element q_g , the tangent of the *Geopier* angle of internal friction ϕ'_g , and the cross sectional area of the *Geopier* elements A_g :

$$F_g = q_g \tan \phi'_g A_g \quad (1)$$

For footings constructed of concrete poured in place directly on top of *Geopier* elements, no reduction in friction angle ϕ'_g is required because of the rough interface between the concrete and the angular aggregate.

As described in the literature (Lawton and Fox 1994; Lawton *et al.* 1994; Fox and Cowell 1998; Wissmann *et al.* 2000, Wissmann and Fox 2000), the normal stress on the *Geopier* elements q_g depends on the average footing bearing pressure q , the stiffness ratio R_s between the *Geopier* elements and the matrix soil, and the ratio of the sum of the *Geopier* element cross-sectional areas to the footing bottom area R_a :

$$q_g = \frac{qR_s}{R_a R_s + 1 - R_a} \quad (2)$$

The stress on the *Geopier* q_g elements is significantly greater than the stress on the surrounding matrix soil because the *Geopier* elements exhibit a greater stiffness than the matrix soils. Values of the stiffness ratio R_s was presented by Lawton (2000) to range between 30 and 45 at a soft soil site in Salt Lake City, Utah. As a result of the high normal stresses and the high internal angle of friction exhibited by the rammed *Geopier* aggregate, most of the lateral load resistance offered by *Geopier* supported footings is attributed to the sliding resistance at the tops of the *Geopier* elements.

The resistance to sliding provided by the matrix soil F_m depends on the product of the normal (downward) stress on the matrix soil q_s , the tangent of the angle of internal friction of the matrix soil ϕ'_m , the matrix soil area A_m and the cohesion intercept of the matrix soil c_m :

$$F_m = q_s \tan \phi'_m A_m + c_m A_m \quad (3)$$

The matrix soil area is the difference between the foundation footprint area and the sum of the *Geopier* element cross-sectional areas. For footings constructed of concrete poured in place directly on top of prepared excavations, no reduction in the friction angle ϕ'_m is required because of the rough interface between the concrete and the soil. The stress on the matrix soil is computed as the stress on the *Geopier* elements divided by the stiffness ratio between the *Geopier* elements and the matrix soil (Fox and Cowell 1998):

$$q_s = \frac{q_g}{R_s} \quad (4)$$

The total resistance to sliding along the base of the footing F_t is computed by adding the resistance to sliding at the tops of the *Geopier* elements F_g and the resistance to sliding at the foundation/matrix soil interface F_m :

$$F_t = F_g + F_m \quad (5)$$

Composite unit friction coefficient

The allowable composite unit friction coefficient f_{all} is often used by structural engineers to determine footing resistance to lateral loads. The allowable composite unit friction coefficient f_{all} for any footing is simply computed as the ratio of the allowable lateral sliding resistance F_{all} to the downward dead load applied to the footing P :

$$f_{all} = \frac{F_{all}}{P} \quad (6)$$

where F_{all} is computed as the quotient of the ultimate resistance to sliding F_t and a factor of safety FS :

$$F_{all} = \frac{F_t}{FS} \quad (7)$$

A factor of safety of 1.5 to 2 is typically used in conjunction with equation (7). When dynamic loads are considered, the allowable load resistance is typically increased by a factor of 1/3 or more.

The composite unit friction coefficient for *Geopier* supported footings may be expressed by combining equations (1) through (7):

$$f_{all} = \frac{R_s R_a \tan \phi'_g + (1 - R_a) \tan \phi'_s}{R_a R_s + 1 - R_a} \left(\frac{1}{FS} \right) \quad (8)$$

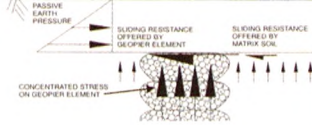


Table 1 presents typical values of f_{all} for various soil types.

Table 1: Typical composite unit friction coefficient values

SOIL CLASSIFICATION	TYPICAL ϕ'	f_{all}^*
sand and gravel	28° - 45°	0.52 - 0.55
silt and clay	20° - 30°	0.51 - 0.52

*Values computed for $R_s = 15$, $R_a = 33\%$, and $FS = 2$

Passive earth pressure

Passive earth pressure develops within the matrix soil at the front of footings as the footings push laterally into the adjacent soils. The passive force F_p that resists lateral movement depends on the foundation width B , unit weight of the soil γ , the footing embedment depth D_p , the Rankine passive earth pressure coefficient K_p and the cohesion intercept of the matrix soil c_m as shown in equation (9) (Terzaghi and Peck 1967):

$$F_p = \frac{1}{2} B K_p \gamma D_f^2 + 2c \sqrt{K_p} B D_f \quad (9)$$

where the Rankine passive earth pressure coefficient depends on the friction angle of the adjacent matrix soil ϕ'_m :

$$K_p = \frac{\tan^2(45^\circ + \phi'_m / 2)}{FS} \quad (10)$$

A factor of safety FS of 2 is typically used in conjunction with equation (10) to avoid appreciable lateral deformations. When dynamic loads are considered, the allowable load resistance is typically increased by a factor of 1/3 or more.

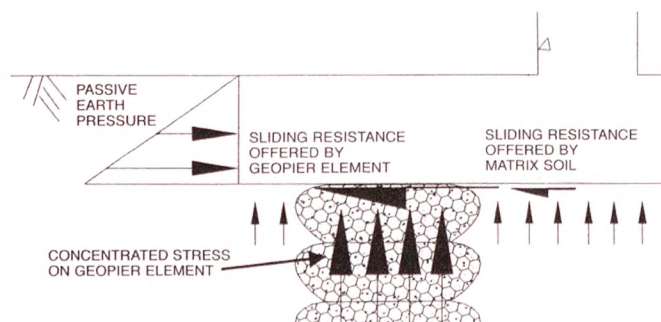


Figure 4: Stresses and lateral resistance under the footing

Analysis of a case

The analysis of a case for estimating the sliding resistance of two footings, one supported by unreinforced matrix soil and one supported by *Geopier* elements are shown in Figures 5a and 5b. Both footings are subjected to a downward load of 890 kN. To maintain simplicity in the calculations, it is assumed that neither footing is embedded in the matrix soil (no passive resistance will be developed).

For the same vertical load, the *Geopier*-supported footing resists 505 kN (allowable), compared to only 200 kN for the non-reinforced soils. The *Geopier* supported footing resists more than two and a half times the lateral load even though the footprint area of the footing is only 40 percent of the footprint area of the footing not supported by *Geopier* elements.

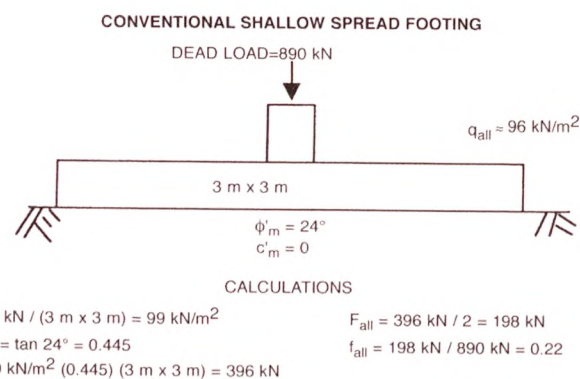
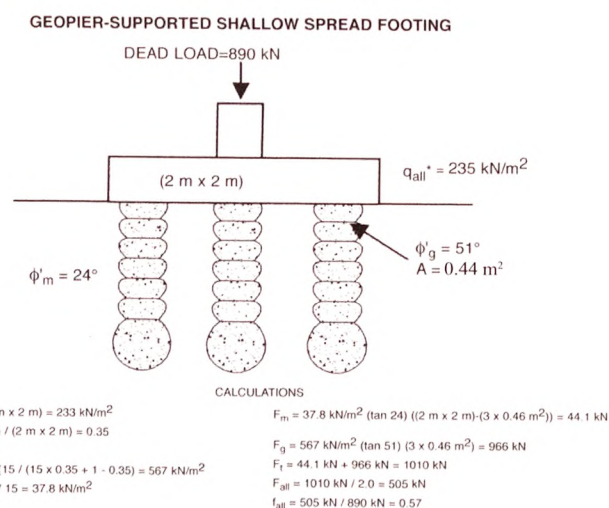
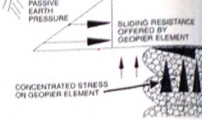


Figure 5a: Sliding resistance calculations for footing supported by unreinforced Soil



* q_{all} is greater for *Geopier*-supported footings than for conventional footings because of the increased shearing strength afforded by the *Geopier* elements.

Figure 5b: Sliding resistance calculations for *Geopier*-supported footing.



Full-scale footing lateral load tests

In 1998, researchers at the University of Utah under the auspices of the Utah Department of Transportation (Utah DOT) tested a full-scale elevated bridge bent to evaluate the response of bridge bents to simulate seismic loads induced by a $M = 7.5$ earthquake (Lawton 2000). The testing required the construction of a reaction frame subjected to large cyclic lateral loads. The reaction frame was supported by Geopier elements. The testing program provided researchers with an opportunity to verify the load resistance mechanisms described in this paper.

The large reaction frame, shown in Figure 6, was required for the application of the cyclic loads to the elevated bridge bent. The reaction frame incorporated two footings supported by Geopier elements. Because the footings were placed on the ground surface and not embedded, passive earth pressure resistance could not be developed and lateral resistance was developed exclusively by sliding at the base of the footing.

Each of the reaction frame footings measured 7.47 m long by 2.54 m wide and 1.14 m thick. Ten 0.91 m diameter Geopier elements drilled to 4.6 m and fitted with uplift anchors were used to support each of the two reaction frame footings. The subsurface conditions underlying the footings consisted of Canyon outwash and Lake Bonneville deposits, comprised of soft to moderately stiff, low plasticity silt and clay soils with interbedded layers of sand. The groundwater table at the site varied between 1.2 m to 2.1 m below grade.

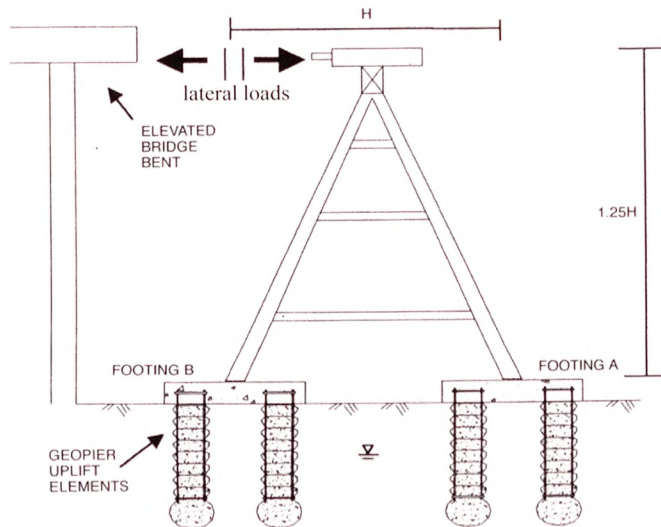


Figure 6: Outline of the reaction frame and foundation system.

When lateral loads were applied to the reaction frame, the inclined members transmitted both vertical and lateral forces to the footings. When the load was applied to the bridge by the frame in the direction shown in Figure 6, both footings were subject to lateral loads. Footing A was also subject to downward compression loads while Footing B was also subject to uplift loads. The geometry of the frame resulted in a ratio of applied vertical load to applied horizontal load of 1.25. The dead weight from the reaction frame and the dead weight of each footing resulted in a net dead load of 445 kN on each footing. Table 2 presents the total vertical load acting on each of the reaction footings at increasing applied horizontal loads.

As the applied horizontal load increased, the compressive load on Footing A also increased. At the same time, Footing B was subjected to an increasing amount of uplift load. When the uplift force applied to Footing B was greater than the dead load acting on the footing, the footing no longer applied compressive stress to the underlying soil and Geopier elements and no further lateral load resistance was offered by this footing. However, lateral load resistance continued to be developed by Footing A. The factor of safety against sliding, computed from equations (1) and (7), is also shown in Table 2.

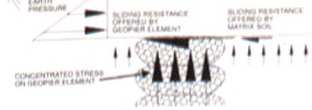
Table 2: Factors of safety corresponding to increasing lateral loads

Horizontal Load kN	Compressive load Footing A, kN	Compressive load Footing B*, kN	Factor of safety against Sliding**
0	445	445	—
178	667	222	4.17
356	890	0	2.78
534	1112	0*	2.32
890	1557	0*	1.95
1780	2669	0*	1.67

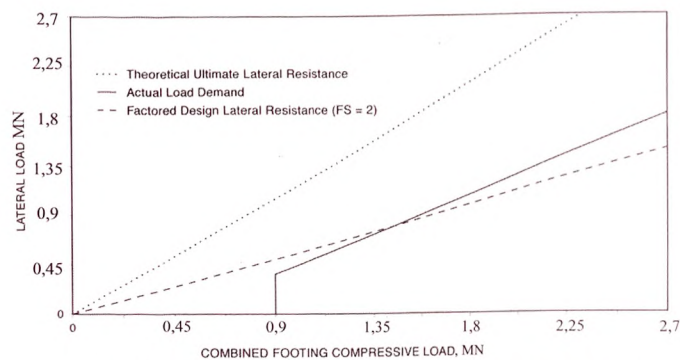
*Indicates net uplift force on the footing. As result, no lateral resistance is offered by footing.

**Neglects additional lateral load resistance provided by uplift bars installed in Geopier elements.

During the testing, a maximum horizontal load of 1779 kN was applied to the bridge bent. At the maximum value of lateral load, Footing A was subjected to a downward vertical load of 2669 kN and Footing B was subjected to an uplift load of 1779 kN that was resisted by the uplift anchors. The combined footing system was subjected to a net vertical load of 2669 kN available for the development of lateral load resistance. Figure 7 presents a plot of the



development of system compressive load as a result of applied lateral load. Figure 7 also illustrates envelopes of the theoretical ultimate lateral load resistance and the allowable lateral load resistance (factor of safety of 2.0). The research results presented in Figure 7 indicate that the lateral resistance provided by the *Geopier*-supported system is greater than the factored design lateral load resistance.



Note: Figure neglects additional lateral load resistance provided by steel uplift bars

Figure 7: Lateral load demand on reaction frame

Uplift load demands

Buildings are subject to applied lateral loads during windstorms and seismic events. The applied lateral loads create an overturning moment that must be resisted by compression and tensile forces at the building foundation (Figure 8). If the applied tensile force on the foundation is greater than the static downward force, the footing may lift off the ground and lead to structural instability. *Geopier* uplift elements are designed to resist these tensile loads. The appropriate factor of safety used in the design of uplift elements depends on a variety of factors including:

- 1) whether or not a load test is performed at the site
- 2) the rate of anticipated loading applied to the structure
- 3) the directionality of loading

Based on the *Geopier* element uplift test, which is usually performed at locations that exhibit the weakest soil conditions, a factor of safety of 2.0 is usually considered appropriate for the resistance of sustained uplift loads. If the elements are used to resist seismic loadings, lower factors of safety may be used because the dynamic resistance of the anchors is greater than the static (tested) resistance of the anchors and because loading directions reverse over short time periods thereby minimizing the possibility of sustained uplift.

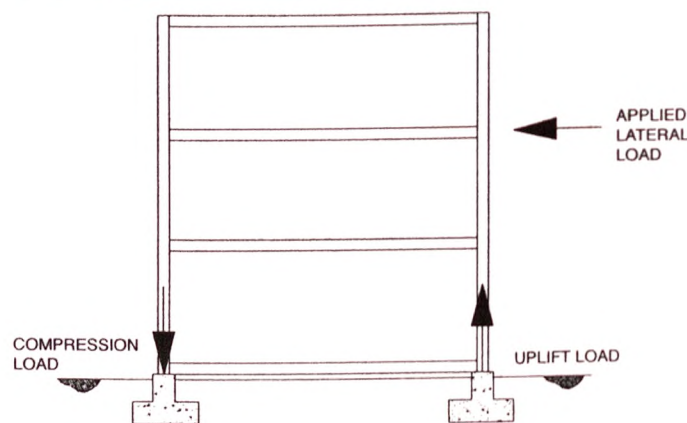


Figure 8: Building subjected to lateral loads

Construction

A constructed *Geopier* uplift element with matrix soil stress response is shown on Figure 9. *Geopier* element shafts are excavated to the required drill depth and the bottom bulb is constructed with open-graded stone. An uplift harness is then lowered into the hole to the top of the densified bottom bulb. The anchor consists of a round or rectangular steel plate with tie rods connected at the outer edge of the plate. Typical assemblies incorporate either two or four uplift rods. After the uplift harness is installed, the remainder of the *Geopier* element is constructed by ramming aggregate in thin lifts with a beveled tamper. The uplift rods must be spaced sufficiently far apart so that the tamper can fit between the rods as the pier is constructed. The uplift rods are connected to the overlying footing via standard hooks and other structural connections.

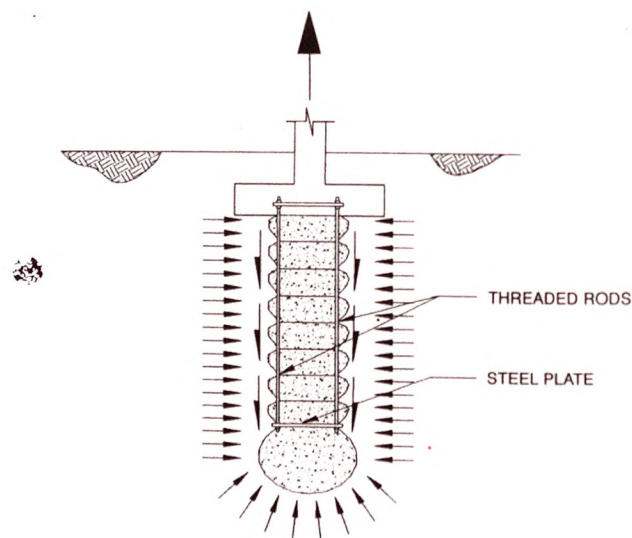
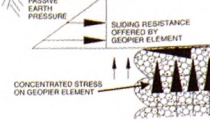


Figure 9: *Geopier* uplift element



Previous research

A significant amount of research effort has been focused on the orientation of the failure surfaces that develop during pullout of conventional embedded anchors (Meyerhof and Adams, 1968; Kulhawy *et al.*, 1979; Charlie *et al.*, 1985; Ghaly *et al.*, 1991). Field observations for conventional embedded anchors indicate that the rupture surface corresponds to either 1) an upright cylinder with a perimeter defined by the footprint of the embedded anchor or 2) a surface that, at the ground surface, is larger than the perimeter of the anchor. Kulhawy (1985) suggests that as upward forces are applied, shear stresses develop along inclined shearing planes that satisfy Coulomb failure criterion (Figure 10). With additional movement, vertical displacement shearing surfaces develop, resulting in continuing upward displacements. This shear pattern will propagate very close to the interface, essentially defining the perimeter of the uplift anchor.

When anchors with small aspect ratios are installed in soils with relatively high strength, the inclined shearing surfaces may appear at the ground surface. This failure mechanism results in a conical failure surface. The failure surface then is represented by a cylindrical surface at depth transitioning into a conical surface that appears at some distances from the perimeter of the element. Although Kulhawy (1985) has developed solutions for the conical failure surface, solutions for a continuous cylindrical surface provide nearly the same uplift resistance.

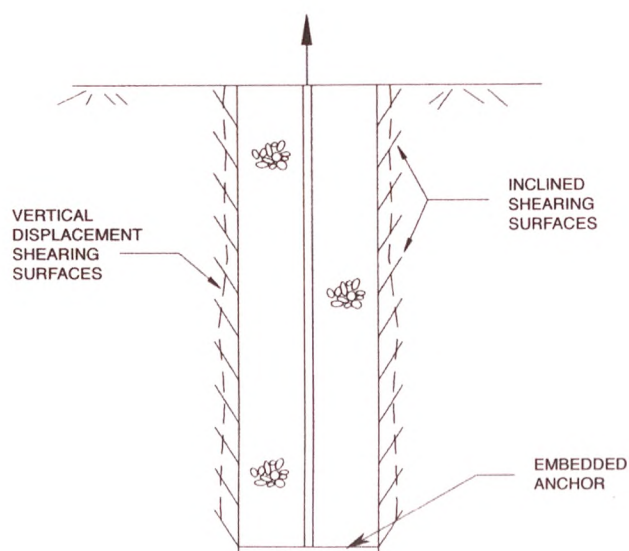


Figure 10: Shearing stresses on embedded anchor (after Kulhawy, 1985)

Pullout resistance of individual uplift elements

Observations of *Geopier* elements that have been pulled completely out of the ground during uplift indicate that the critical shearing surface is cylindrical and occurs at the perimeter of the installed element (Figures 9 and 11). Prior to complete pullout failure, radial and circumferential cracks are often observed at the ground surface. These cracking patterns are consistent with the near surface inverted conical failure surfaces described in the literature for embedded anchors loaded in tension (Kulhawy, 1985). The approach used to compute the pullout resistance of individual *Geopier* elements is presented in Figure 11. When *Geopier* elements are subjected to extreme uplift loads, a cylindrical failure surface forms around the elements. The ultimate pullout resistance Q_{ult} is computed as the sum of the weight of the *Geopier* element W and the side resistance. The ultimate side resistance is computed as the product of the unit pullout resistance f_s and the area of the sheared cylinder A_s :

$$Q_{ult} = W + f_s A_s = W + f_s \pi d H_s \quad (11)$$

where W is the buoyant weight of the *Geopier* element, d is the effective diameter of the *Geopier* element, and H_s is the shaft length of the element (Figure 11). The effective *Geopier* diameter is generally greater than the drilled diameter as a result of ramming the *Geopier* aggregate.

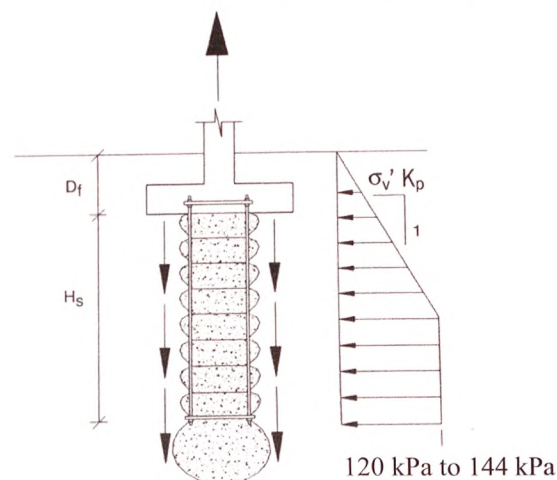
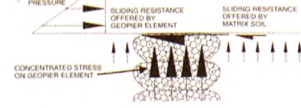


Figure 11: Individual *Geopier* pullout resistance

Cohesionless soils

For *Geopier* elements installed in cohesionless soils, the rate of drainage is typically faster than the net increases in uplift during cumulative cycles of loading.



The uplift loading resistance of individual *Geopier* elements is therefore computed using drained geotechnical analysis procedures. The unit friction f_s is computed as the sum of the drained cohesion intercept c and the product of the effective lateral pressure in the soil surrounding the *Geopier* elements σ'_h and the tangent of the angle of internal friction of the matrix soils ϕ'_m :

$$f_s = c + \sigma'_h \tan \phi'_m \quad (12)$$

The drained cohesion intercept c is often considered to be zero for clean sands and gravels. The ramming action inherent in *Geopier* construction increases the lateral earth pressure in the matrix soils surrounding the *Geopier* elements. The increase in lateral stress is dependent upon soil type, drainage, overconsolidation ratio, and confinement offered by adjacent *Geopier* elements. Post-construction lateral earth pressure is typically computed as the product of the geostatic effective vertical stress in the matrix soils σ'_v and the Rankine passive earth pressure coefficient K_p :

$$\sigma'_h = \sigma'_v K_p \quad (13)$$

where:

$$K_p = \tan^2 (45^\circ + \phi'_m/2) \quad (14)$$

As shown in Figure 11, the applied lateral earth pressure is limited by a value ranging between approximately 120 kPa to 144 kPa to conservatively account for the maximum energy that is typically imparted by the *Geopier* hammer to the surrounding soils (Hsu 2000).

Cohesive soils

When *Geopier* elements are installed in cohesive soils, the rate of uplift loading may or may not be less than the rate of drainage. Therefore, the unit friction f_s is computed as the smaller of 1) the undrained shear strength s_u of the matrix soils and 2) the drained unit friction of the matrix soils using equation (12). The ultimate uplift capacity Q_{ult} thus becomes the smaller of:

$$Q_{ult} = (c + \sigma'_n \tan \phi'_m) \pi d H_s + W \quad (15)$$

and

$$Q_{ult} = s_u \pi d H_s + W \quad (16)$$

Design of uplift rods

High strength, threaded steel rods, such as those produced by Dywidag or Williams, are typically used within the

uplift harnesses. The allowable tension load for each rod Q_{rod} is computed as the product of the allowable tensile stress of the steel F_{all} and the bar cross-sectional area A_{rod} :

$$Q_{rod} = F_{all} A_{rod} = F_{all} \pi d_{rod}^2 / 4 \quad (17)$$

where d_{rod} is the uplift rod diameter. It is suggested that the allowable tensile stress should not exceed 60% of the steel yield strength:

$$F_{all} = 0.60 F_y \quad (18)$$

The design of the uplift bars should consider corrosion. Bars may be galvanized, epoxy-coated or designed with sufficient sacrificial steel to account for corrosion over the design life of the structure.

The design methods described above and selected design parameter values should be verified with an uplift load test if the elements are used to resist significant tension loads. It is worth mentioning that since the design methods do not provide expressions to estimate displacements due to uplift loads, load tests become necessary not only to determine displacements but also ranges of stiffness.

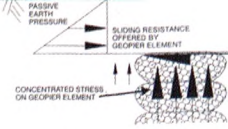
Group effects

The uplift capacity of groups of closely-spaced *Geopier* elements is computed as the smaller of: 1) the uplift capacity of a single element multiplied by the total number of elements, and 2) the uplift capacity of a soil block subject to tension (Figure 12). For *Geopier* elements installed in cohesionless soils, the volume of the block is defined by the footprint of the overlying footing and sloping sidewalls as shown in Figure 12a. The inclination angle of the sidewalls of the block β depends on the matrix soil angle of internal friction and on the lateral earth pressure induced by the construction of the *Geopier* element. Values ranging from 15° to 20° degrees are often used for β in the design calculations. The uplift resistance is computed as the buoyant weight of the soil contained within the block:

$$Q_{block} = W_{block} \quad (19)$$

For *Geopier* elements installed in cohesive soils, the volume of the block is defined by an area greater than the footing footprint and vertical sidewalls as shown in Figure 12b. The uplift resistance is computed by summing the total weight of the soil within the block and the undrained shearing resistance along the edges of the block:

$$Q_{block} = W_{block} + s_u (2B' + 2L') H_s \quad (20)$$



where B' and L' are the dimensions of the footprint of the soil block.

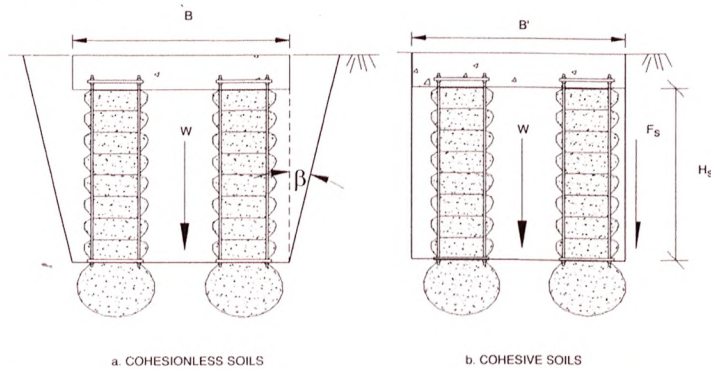


Figure 12: Group uplift capacity for closely-spaced Geopier elements

Uplift load tests

Uplift load tests are often performed on trial Geopier elements. Again the tests are typically located at an area of the site containing the weakest identified soil conditions. A typical test setup is shown in Figure 13. The uplift rods are connected to a cross-member on top of the test reaction beam. During testing, a jack extends the distance between the cross-member and the reaction beam thus pulling on the uplift rods and applying tensile loads to the Geopier element. Load testing is typically performed in general agreement with ASTM D-1144 specifications and from one to four days after the test pier installation to allow time for the dissipation of matrix soil excess pore water pressures. Uplift load tests are used to verify the design uplift capacity.

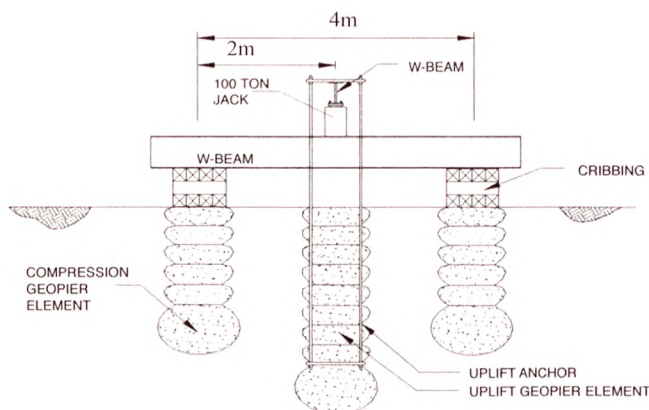


Figure 13: Typical uplift load test setup

Uplift load - deflection results

Figure 14 illustrates a characteristic plot of uplift tests. The results typically consist of three parts which can

be considered as three straight-line segments. The first segment corresponds to the seating of the uplift plate and rearrangement of aggregate particles within the lower part of the Geopier element. The slope of this line, designated m_1 , is generally small. The second segment represents upward deflection of the bottom plate caused by bulging of the lower portion of the pier and movement along the cylindrical sides of the element. This segment trends at a slope m_2 , until shearing failure occurs. The third segment is vertical or near-vertical and represents conditions at which excessive deflections occur with no or minimal additional application of loads. The ultimate uplift capacity is interpreted to occur at the intersection of the second and third line segments.

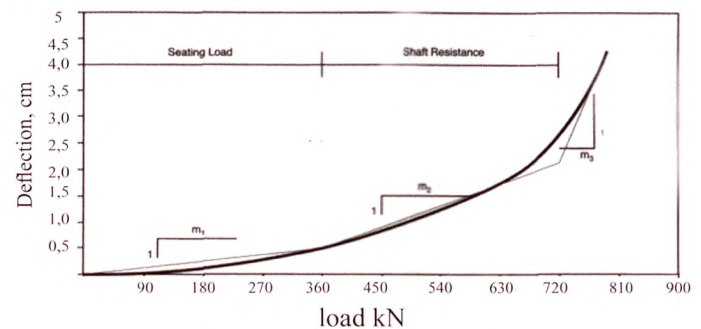


Figure 14: Uplift test results

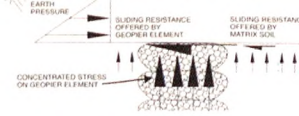
Typical deflections

Uplift deflection control is often important to maintain structural performance. Table 3 presents a summary of deflections measured for 762 mm diameter elements during uplift load tests conducted within gravel, sand, and silt/clay deposits. The deflection values include the elastic elongation of the uplift rods. Table 3 may be used to aid in predicting upward deflections for various levels of applied uplift loads. In general, uplift deflections increase with decreasing matrix soil grain size.

Table 3: Summary of uplift load test deflection

Soil	Load at seating [average of values] kN	Seating deflection [average of values] mm/kN	Skin friction deflection [average of values] mm/kN
Gravel	180-405 [293]	0.028-0.034 [0.028]	0.028-0.034 [0.034]
Sand	135-248 [192]	0.023-0.4 [0.028]	0.023-0.4 [0.073]
Silt and Clay	90-270 [180]	0.023-0.05 [0.034]	0.05-0.186 [0.085]

If the steel uplift plates are installed in soft clay or silt that exhibit the tendency to bulge outward during uplift load applications (Figure 15), the deflections of uplift elements may be greater than those presented in Table 3. Methods



used to estimate bulging potential are provided by Wissmann (2007). The ultimate uplift capacity may be estimated by the product of the limiting radial stress $\sigma'_{r,lim}$, the Rankine passive earth pressure coefficient of the *Geopier* aggregate material, and the cross-sectional area of the element:

$$Q_{ult} = \sigma'_{r,lim} \tan^2 (45 + \phi'_g/2) \pi d^2/4 \quad (21)$$

The limiting radial stress is computed as:

$$Q_{ult} = 2\sigma'_v + 5.2 s_u \quad (22)$$

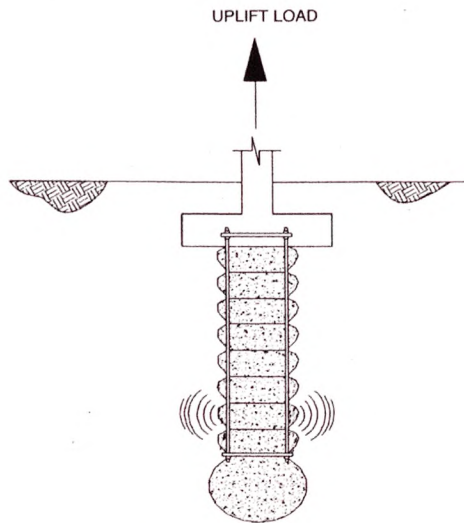


Figure 15: Outward bulging element

Summary

Geopier-supported shallow foundations provide resistance to lateral loads using the mechanisms identical to those of conventional shallow footings. Lateral loads are resisted by passive pressures at the leading face of the footing and sliding resistance at the base of the footing. The use of *Geopier* elements increases the resistance to lateral loads by increasing the available sliding resistance on the base of the footing. The sliding resistance is increased because of stress concentration to the tops of the *Geopier* elements and the high shear strength (high angle of internal friction) and the dilatant behavior of the rammed *Geopier* aggregate.

Geopier uplift elements resist applied uplift loads by developing resistance between the perimeter of the elements and the surrounding matrix soils. The elements are particularly efficient because of the increase in matrix soil lateral stress that occurs during construction. The elements are used to provide stability to shallow spread footings that are subjected to tensile loads.

Further research is needed to include the effect of applied moment and the assessment of corresponding rotational displacements.

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