SOIL REINFORCEMENT USED TO ARREST BEARING CAPACITY FAILURE AT A STEEL MILL

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Bearing capacity failure of shallow foundations is rare as a result of the success of geotechnical engineering approaches in the past 70 years. Recently, however, foundation rotational displacements, consistent with bearing capacity failure surfaces, were observed in a steel mill in northern Nebraska, USA. The area of the mill experiencing failures was being used to stage stacks of steel billets. The steel stacks measured 6 m square in plan dimension, reached to heights of up to 10 m, and exerted contact pressures of up to 824 kN/m^2 (17,200 psf). Subsequent geotechnical investigations revealed that the site was underlain by slag fill and weak native clay. The weak subsurface conditions, combined with observations of footing rotations, indicated that bearing capacity failure was occurring. The integrity of the steel mill was at risk if nothing was done to stabilize the structure.

A preliminary solution incorporating driven piles supporting a concrete floor was considered to transfer the applied loads to stronger soil layers. This option was discarded because of the high cost of construction. The selected option consisted of installing 9-meter (30 ft) long *Rammed Aggregate Pier* elements to stabilize the foundation soils and increase the factor of safety against foundation bearing capacity failure.

The design presented significant challenges because conventional bearing capacity solutions do not account for vertically reinforced soil deposits and for stress concentration to the relatively stiff piers. This paper describes the methods used to design the aggregate piers for the stabilization of the stacks of steel. Expressions are developed for estimating the limit equilibrium bearing capacity of vertically reinforced deposits and for incorporating the concept of stress concentration on the foundation stabilization system. This paper is of particular significance because it provides guidance for the simplification of a complicated design issue as it applies to an economical and effective soil reinforcement system.

1.0 Introduction

Bearing capacity failure of shallow foundations is rare. However, rotational foundation displacements have occurred at a steel mill in northern Nebraska, USA. Structural repairs, as shown in the photograph in Figure 1, were required at a number of building columns to compensate for the movement of the foundations. Survey measurements confirmed that the building was "racking" with superstructure displacements explainable only by the rotation of the shallow spread footings (Figure 2). Because of on-going displacements, the structural and geotechnical design consultants judged that the integrity of the steel mill was at risk if nothing was done to stabilize the structure.

The rotated foundations are located in an area of the mill being used to stage large stacks of steel billets that exert high downward pressures of up to 824 kN/m^2 (17,200 psf). The billet stacks were immediately identified as the source of the problem. After performing a geotechnical investigation and evaluating deep foundation alternatives, the design team recommended that the soil below the billets be stabilized by using *Geopier®* soil reinforcing elements to increase the soil shear strength below the billets and arrest further movements. It was soon discovered, however, that traditional methods of calculating foundation bearing capacity do not provide solutions for vertically reinforced materials and do not account for stress concentration to the stiff reinforcing elements. This paper discusses the solution to the steel mill bearing capacity problem and presents a design approach developed for analyzing vertically reinforced soils.

2.0 Foundation Rotations

The Nebraska steel mill is housed by an 18 to 20 m tall steel-framed metal clad structure. The steel columns are typically supported by 4.5 m by 9 m spread footings, embedded to a depth of 3 m. To facilitate lifting of the steel billets, two overhead cranes run along rails mounted on the building columns. The rotation of the spread footings and consequent racking of the building columns (Figure 2) caused the crane rails to become misaligned, requiring numerous structural repairs.

3.0 Subsurface Conditions

The site is located in northeastern Nebraska, USA, an area where the natural soils generally consist of unconsolidated Quaternary sediments that are comprised of eolian silts (loess) and sands. Geotechnical investigations performed after the distress was noted revealed that the site is underlain by 3.5 m of slag fill, underlain by 5.5 m of weak native clay. The weak clay is underlain by medium dense silty sand (Figure 3). Geotechnical design parameter values for each layer were established from field and laboratory tests and are presented in Table 1.

Table 1: Geotechnical design parameter values

Parameter Value	Slag Fill	Weak Clay	Silty Sand
Depth to layer bottom (m)	3.5	9	*
Soil moisture content (%)	12	30	19
Total unit weight (kN/m^3)	18.8	18.1	19.6
Undrained shear strength (kN/m^2)	0	18.8	0
Angle of internal friction (degrees)	33	0	35

* Exceeds boring termination depth

4.0 Bearing Capacity of Unreinforced Soils

Bearing capacity analyses of the billet stacks performed using classical expressions developed by Terzaghi and others [1] in conjunction with the parameter values listed in Table 1 resulted in a factor of safety of near unity. These analyses supported the conclusion that the heavy billets induced rotational displacements in the soil profile leading to the rotation of the building columns (Figure 2).

5.0 Construction Alternatives

A temporary solution was to reduce the height of the billet stacks. A long-term solution was still needed, however, because the full-height stacks were needed for efficient steel milling. A long-term solution of installing deep pile foundations and supporting the billets on top of a concrete floor was considered. This option was discarded because of the extremely high cost of the work and long construction schedules. *Geopier* soil reinforcement was then considered and later selected because of cost and schedule advantages. The *Geopier* solution was especially attractive because plant production could continue during installations with only brief interruptions.

6.0 Geopier Soil Reinforcement

Geopier soil reinforcing elements were installed by drilling 760 mm (30 inch) diameter holes to depths of 9.1 m (30 feet) below grade, placing controlled lifts of aggregate stone within the cavities, and compacting the aggregate using a specially designed high-energy beveled impact tamper (Figure 4). The first lift consists of clean stone and is rammed into the soil to form a bottom bulb below the excavated shaft. The piers are completed by placing additional 0.3 m (one-foot) thick lifts of well-graded aggregate over the bottom bulb and densifying the aggregate with the beveled tamper. During densification, the beveled shape of the tamper forces stone laterally into the sidewall of the excavated cavity. This action increases the lateral stress in the matrix soil thus providing additional stiffening. The piers exhibit high friction angles as exhibited by the results of full-scale top-of-pier direct shear tests (Figure 5).

7.0 Interesting Design Challenges

Upper-bound solutions for estimating the limit equilibrium bearing capacity of shallow spread footings are plentiful in the literature for isotropic materials [1], [2], [3]. These solutions may also be implemented for horizontally stratified deposits simply by calculating the weighted average of the soil parameter values in the zone of anticipated foundation failure [4]. For vertically reinforced deposits, however, such as soils reinforced by *Geopier* elements (Figure 6) installed directly below but not adjacent to footings, a simple weighted average procedure is not applicable. This is because the soils directly below the footing have a larger influence on the bearing capacity solution than do the soils adjacent to the footing as a result of the additional vertical stress applied below the footing.

Bearing capacity solutions for *Geopier*-supported foundations become even more difficult because of stress concentration that occurs to the tops of the piers in the reinforced zone of soil. The Geopier elements are stiffer than the matrix soil and attract a higher proportion of the foundation stress than do the matrix soils. The combination of this stress concentration and the high angle of internal friction of the elements increases the composite resistance to internal shearing.

For relatively low levels of top-of-pier stress (less than approximately 628 kPa (30 ksf)), the amount of stress concentration can be readily obtained from comparisons of the stiffness modulus of the Geopier elements and the stiffness modulus of the matrix soil [5], [6]. However, applications of high stresses to the tops of the Geopier elements causes the elements to deform by bulging into the matrix soils. When bulging occurs, the matrix soils attract an incrementally higher proportion of the foundation stress, thereby also providing an incremental increase in the matrix soil confining pressure, an effect that reduces the propensity for Geopier bulging effectively "stiffening" the element.

For an efficient design, each of these mechanisms: 1.) presence of vertically reinforced deposit, 2.) stress concentration to the relatively stiff high friction angle Geopier elements, and 3.) Geopier-matrix soil bulging interactions, must be accounted for.

8.0 Design Approach

The following steps describe the design approach used to provide an effective solution:

1. The relative stiffness of the reinforcing elements and matrix soils was evaluated with a finite difference computer code that simulated the bulging response of the piers and the "hardening" of the piers as additional confining stress was applied by the matrix soils.
2. The composite angle of internal friction of the reinforced zone below the billet stacks (Figure 6) was then obtained by calculating the weighted average of the shear strength available in the reinforcing elements and the shear strength available in the matrix soils.
3. A lower bound solution for bearing capacity, which allows for the incorporation of vertically reinforced deposits, was then used to estimate lower bound bearing capacity of the soils.
4. Lastly, a correlation between lower bound and upper bound solutions was used to estimate the bearing capacity of the supported foundations.

The design steps are described in more detail below.

8.1 Modeling *Geopier* stiffness

Figure 7 presents a typical modulus test result for a *Geopier* element installed in cohesive soils. A bilinear response is noted with the first portion of the curve corresponding to stiff behavior prior to the onset of pier bulging. Bulging is initiated at the inflection point and the pier exhibits a decrease in stiffness at stresses exceeding the inflection point stress. The stress-deflection curve shown in Figure 7 may be modeled using the incremental tangent modulus approach suggested by Kondner [7] and Kondner and Zelasko [8] and made popular by Duncan and his co-workers [9], [10]. Using these popular methods, the incremental tangent stiffness modulus, $k_{g,t}$, may be written as:

$$k_{g,t} = k_{g,i} (1-R_f SL)^2 , \qquad (1)$$

where $k_{g,i}$ is the initial stiffness modulus value established from a modulus test performed on an installed *Geopier* element, R_f is a constant representing the hyperbolic reduction factor, and SL is stress level. For the subject analysis, stress level, SL, is defined as the ratio of applied top-of-pier stress (q_g) to the top-of-pier stress that corresponds to the onset of plastic radial bulging ($q_{bulging}$). The top-of-pier stress corresponding to the onset of plastic radial bulging may be evaluated as the product of the limiting radial stress $\sigma'_{r,lim}$) and the Rankine passive earth pressure coefficient for the *Geopier* aggregate. The limiting radial stress for cohesive soils is developed from cavity expansion theory [11]:

$$\sigma'_{r,lim} = \sigma'_v + 4\,c \ , \qquad (2)$$

where σ'_v is the effective overburden stress and c is the undrained shear strength of the matrix soil. Combining the expressions described above, stress level (SL) may be expressed as:

$$SL = q_g / [\,(\tan^2(45+\phi'_g/2))\,(\sigma'_v + 4\,c)] \ . \qquad (3)$$

Equations 1 and 3 allow for the generation of the stiffness modulus backbone curve by applying small incremental levels of top-of-pier stress, solving for $k_{g,t}$, and calculating pier settlement for that stress increment.

8.2 Evaluating stress concentration

Because the billets are stiff relative to the foundation materials, it is assumed that the applied billet loads result in a uniform deflection of the *Geopier* elements and the matrix soils between the elements. The stress that the billets apply to the tops of the piers and the stress that the billets apply to the matrix soils thus depend only on the relative stiffness (R_s) of the two materials and on the area replacement ratio (R_a) of the Geopier elements. The stress applied to the tops of the Geopier elements (q_g) may be estimated with the expression [6]:

$$q_g = q\,R_s / (R_s R_a - R_a + 1) \ , \qquad (4)$$

where q is the stress applied by the billets, R_a is the ratio of the sum of the cross-sectional areas of the *Geopier* elements to the total footprint area of the supported billets, and R_s is the stiffness ratio. The stiffness ratio is the ratio of the stiffness modulus of the *Geopier* elements, which can be represented using Equation 1, to the stiffness modulus of the matrix soils, which can be estimated by the ratio of the applied stress to the settlement of the soil calculated using consolidation settlement expressions. Because the stiffness modulus of the *Geopier* elements depends on the proximity to the stress corresponding to plastic radial bulging, and because the stress corresponding to plastic radial bulging depends on the overburden stress in the matrix soil which, in turn depends on the relative stiffness between the Geopier elements and the matrix soil, the solution may be achieved using an incremental stepping procedure described in the following steps.

1. A small increment of billet pressure is applied to the system. Initially it is assumed that only 10 percent of this stress is applied to the matrix soil.
2. The initial stiffness of the matrix soil is established by calculating the ratio of the applied stress on the matrix soil to the consolidation settlement estimated for that stress increment.
3. The stiffness modulus of the Geopier element is established using Equations 1 and 3.
4. The stiffness ratio, R_s is established from the results of Steps 2 and 3.
5. The stress applied to the *Geopier* elements is estimated using Equation 4.

6. The incremental settlement of the *Geopier* elements (and of the matrix soil) is calculated as the quotient of incremental *Geopier* stress to incremental *Geopier* tangent modulus, $k_{g,i}$.

7. Additional step of stress are applied and the system is solved incrementally.

8. At the end of the analysis, the complete billet load is applied and the calculated top-of-Geopier stress is used to establish the composite shear strength of the reinforced soil mass.

8.3 Rankine lower bound solution

The Rankine solution, which is a lower bound solution, solves the bearing capacity question by equilibrating average stresses acting within two blocks of slipping soil (Figure 8). The depth of influence is determined by the angle of internal friction within the triangular block of soil directly under the foundation (Block A). This triangular block of soil underneath the foundation is restrained from moving by the triangular block of soil adjacent to the foundation (Block B). The width of Block B is determined by the angle of internal friction of the soil adjacent to the foundation. Because the depth of Block A and width of Block B are dependent on the friction angles in each region, a solution may be provided for vertically reinforced materials using the expression:

$$q_{ult,lb} = (B/2)\gamma_u C_1^3 C_2^2 + 2c_u C_1^2 C_2 + C_1 (2c_r - \gamma_r (B/2)), \qquad (5)$$

where B is the foundation width, γ_u is the composite unit weight in the unreinforced zone, γ_r is the composite unit weight in the reinforced zone, c_u is the composite cohesion in the unreinforced zone, c_r is the composite cohesion in the reinforced zone, C_1 and C_2 are equal to the following equations:

$$C_1 = \tan (45 + \phi'_r/2) \qquad (6)$$

$$C_2 = \tan (45 + \phi'_u/2) \qquad (7)$$

where ϕ'_r is the composite friction angle in the reinforced zone and ϕ'_u is the composite friction angle in the unreinforced zone.

8.4 Terzaghi's upper bound solution

Despite advantages in simplicity and in allowing for solutions to be developed for vertically reinforced materials, Rankine's solution is not often used in practice because it provides overly conservative results. It is possible, however, to develop independent solutions for Terzaghi's upper bound solution and for the Rankine solution for isotropic materials. For the matrix soils at the site, the ratio between the Terzaghi and Rankine solutions is approximately 1.5. Thus, this correction factor may be applied to Rankine's solution for vertically reinforced deposits to obtain a reasonable solution.

9.0 Design and Construction Solution

Using the approach outlined above, a required area replacement ratio of 12 percent is required to achieve a factor of safety of 1.05 for ultimate billet bearing pressures of 824 MN/m^2 (17.2 ksf) and a factor of safety of 1.15 for maximum conceivable billet bearing pressures of 630 MN/m^2 (13.2 ksf). The required area replacement ratio results in pier spacings of 2 m (6.5 feet) on-center throughout the billet laydown area.

Modulus test results performed at the site are provided in Figure 9. At a maximum stress of 1,755 kPa (36.7 ksf), the deflection of the *Geopier* element was measured to be 43 mm (1.7 inches), which results in a stiffness modulus value of 40.5 MN/m^3. The stiffness modulus value was approximately 1.8 times greater than predicted at the design stress level, indicating acceptable element performance.

Construction of the elements incorporated two installation crews and necessitated the use of a high-torque drilling tool to get through buried concrete obstructions. A total of 612 elements were installed in 35 days, meeting the rigorous project schedule. The Geopier soil reinforcing elements resulted in a significant cost savings relative to the driven pile solutions. Because of the compatibility between the reinforcing elements and the matrix soils, no concrete pad was required resulting in an additional cost savings.

10.0 Conclusions

At a site in northern Nebraska, USA, high bearing pressures applied by heavy steel billets were causing rotation displacements below footings supporting a steel mill. Geopier soil reinforcing elements were used to support the billets and arrest the rotational shearing displacements. Traditional methods used to evaluate foundation bearing capacity were found to be inadequate for this application because they do not account for vertically reinforced soil deposits and do not explicitly handle the positive effects of stress concentration to the stiff Geopier elements. A simple analytical approach was developed to accommodate the strain hardening behavior of the pier as additional confinement is added and to evaluate the long-term top-of-pier stress below the billets. Accordingly, the composite shear strength parameter values of the reinforced soil were established. Vertical reinforcement was treated using Rankine's bearing capacity expressions to separately evaluate the effects of greater shear strength parameter values in the reinforced soils below the billets as compared to the unreinforced matrix soils adjacent to the billets. Factors of safety computed using Rankine's solution were scaled to establish more reasonable Terzaghi upper bound method safety factors. The installation of the aggregate piers has stopped the rotational movements at a significant cost savings to the owner.

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Figure 1: Structural repairs required as a result of foundation movements

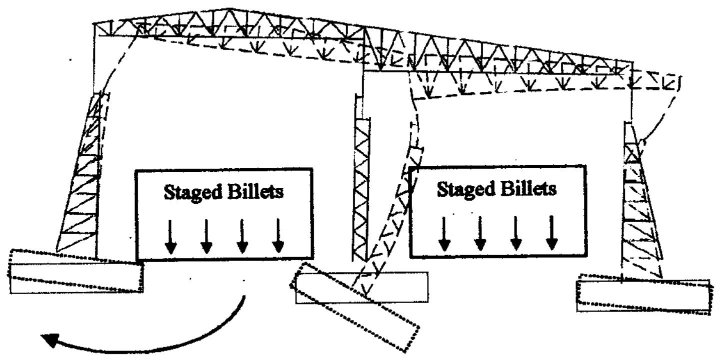

Figure 2: Results of survey measurements suggesting foundation rotations (Exaggerated)

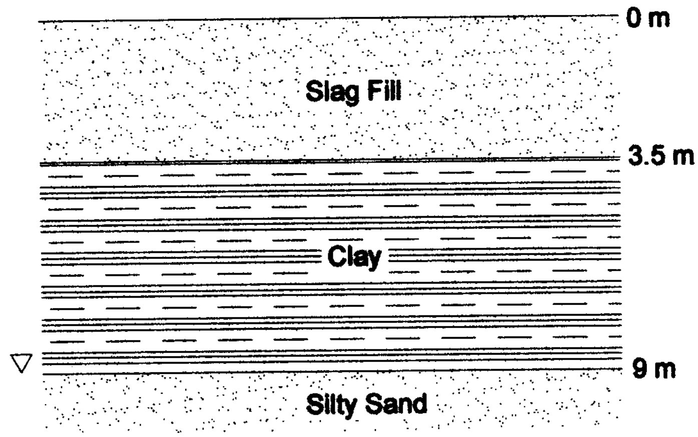

Figure 3: Subsurface conditions

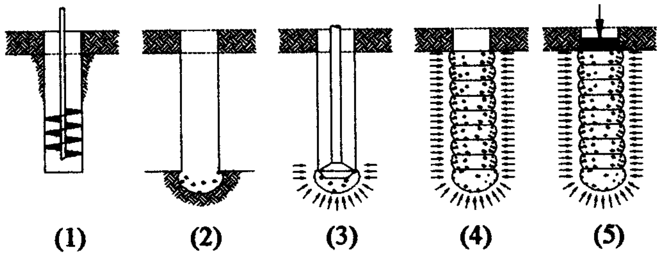

Figure 4: Construction Process

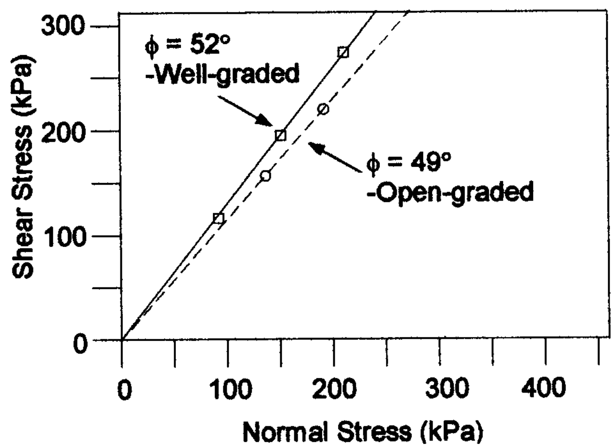

Figure 5: Results of full-scale direct shear tests

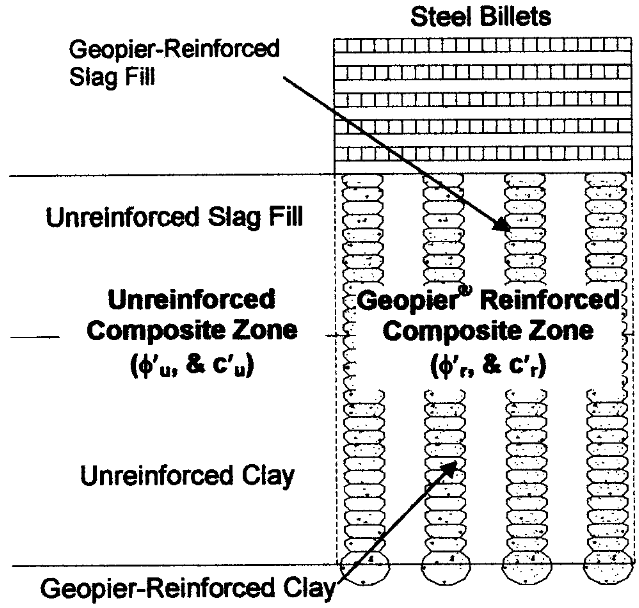

Figure 6: Vertically reinforced soil conditions

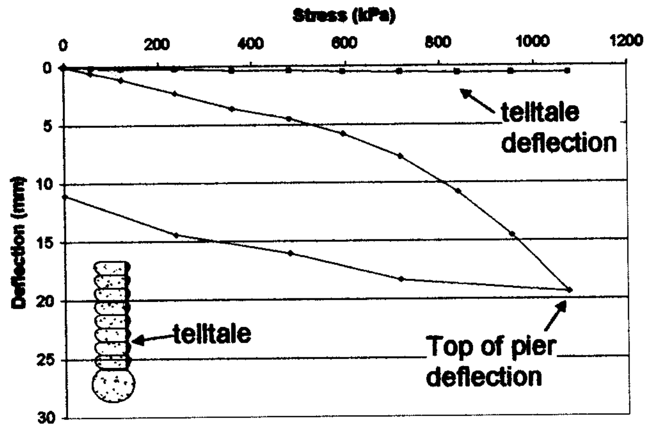

Figure 7: Typical results of modulus test

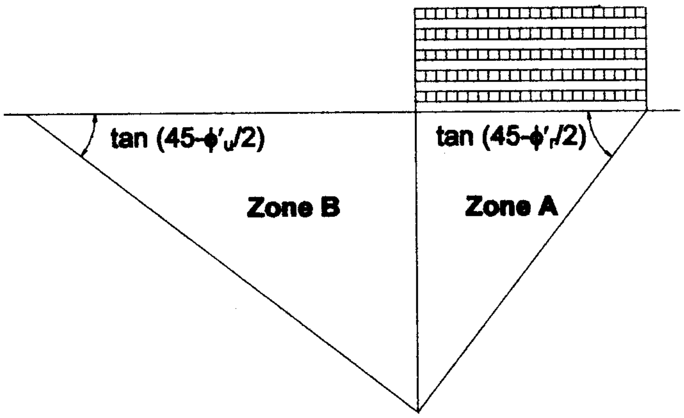

Figure 8: Rankine's lower-bound bearing capacity solution

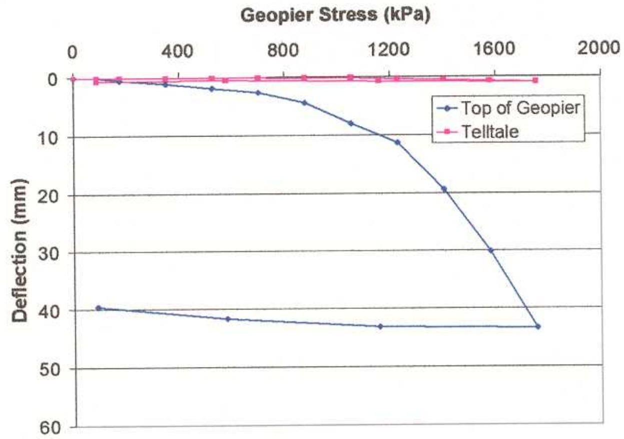

Figure 9: Modulus test results