

DIAMOND PIER FOUNDATION ANALYSIS

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ABSTRACT

The Pin Foundations, Inc. Diamond Pier foundation system comprised of a concrete footing with four inclined “pins” that serve to resist vertical and lateral loads has become effective in sustainable construction technology. The current analysis method of this system used by Pin Foundations, Inc. is simplistic and considers an equivalent bearing area of soil, whereby the foundation is treated much like a shallow foundation or footing with a larger contact area. The internal reinforcement mechanics and the function of the diamond pier foundation are similar to that of a system of reticulated micropiles. This study treats the foundation system as a group of micropiles, capable of sustaining vertical and lateral loads while accounting for the group effects of closely spaced piles. A more rigorous analysis method is proposed based on engineering mechanics and is governed by the deflection limitations of the foundation system. This analysis method was used to predict the vertical load capacity of the Diamond Pier DP-50 foundation system. The proposed analysis method predicted a vertical load capacity of 2150-4700lb for a soil modulus range of 130-300psi, a range corresponding to weak soils, while the analysis method used by Pin Foundations, Inc. predicted a vertical load capacity of 2700lb and 3600lb for 1500psf and 2000psf soils, respectively, which correspond to the two weakest soils observed in the 2003 IRC. A subsequent study was conducted to observe the change in vertical and lateral capacity of the Diamond Pier DP-50 foundation corresponding to a range of batter angles between 0° and 80°. As the batter angle was increased from 0° to 80°, the proposed analysis method predicted a decreasing vertical capacity and an increasing lateral capacity, with the greatest change in these values occurring between batter angles of 30° and 60°.

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INTRODUCTION

With a heightened interest in sustainability, low impact construction techniques have been gaining popularity in new construction and renovation projects. Low impact construction techniques extend to pile foundations in the use of micropiles as a low impact foundation because the construction equipment used in their installation has a lighter footprint on the soil. One novel design concept that incorporates these micropiles is the Pin Foundations, Inc. Diamond Pier foundation system. It has been used for over 20 years as a low impact foundation alternative to traditional concrete footings.

The Diamond Pier foundation a patented foundation system for small to medium sized structures, founded on a variety of soils. The foundation system is comprised of a concrete footing with 4 batter (or inclined) “pins” that serve to resist vertical and lateral loads (see Fig. 1). Since this system makes use of the internal reinforcement mechanism provided by the combination of the pins and the surrounding soil, it has the potential to eliminate the need for larger sized concrete foundations thereby saving the need for additional materials, and minimizing the footprint of the foundation.

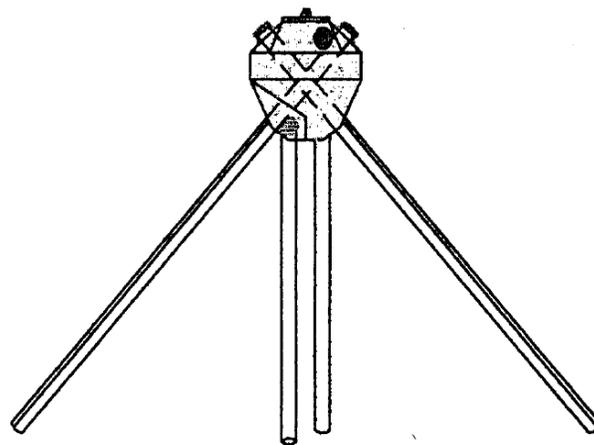


Figure 1 – Pin Foundations, Inc. Diamond Pier foundation (Pin Foundations, Inc., 2007)

The internal reinforcement mechanism provided by the combination of the pins and the surrounding soil is complex and as such the design methods of such systems are in their infancy. The current method used by Pin Foundations, Inc. is to analyze the Diamond Pier foundation system as a shallow footing. Pin Foundation, Inc. currently uses a bearing capacity analysis, which uses the A-frame shape of two pins to bear on the soil and create a “coherent soil mass...around the pins and propagates the applied load downward and radiating outward” (see fig. 3) (Pin Foundations, Inc., 2004). Additionally, an arching factor is used to describe this propagation of load by multiplying the bearing width of the pin by a factor of 2-3 (Pin Foundations, In., 2004). This analysis requires simplifications in the design process for the foundation to be analyzed as a shallow footing, such as neglecting the contribution of the micropiles due to pile friction and tip resistance.

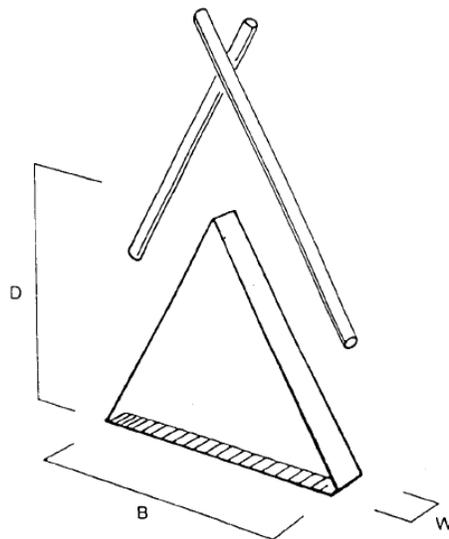


Figure 2 – Shallow Footing Analogy of Diamond Pier Foundation (Pin Foundations, Inc., 2004)

The mechanics and the function of the diamond pier foundation are reminiscent of the system of reticulated micropiles pioneered since the early 1950s by Lizzi (Lizzi, 1982; Lizzi 1985). Since then micropile technology has advanced significantly both as an effective

retrofitting tool and as a low impact foundation. The term micropile used to describe a small diameter (≤ 12 in.) pile that can either be driven or drilled into the ground (Bruce, 1999). When used in new foundations, or retrofitting of existing foundations, the construction equipment has a lighter footprint on the soil compared to traditional pile driving or drilling rigs, and thus, micropile foundations are considered low impact foundations.

The increased capacity of the micropile group due to the “knot effect” of the soil-pile system. This can be seen by the larger mobilized area of soil in Fig. 2. However, Xanthakos, et al. (1994) was sure to note that this increased capacity was not being accounted for in current design practice.

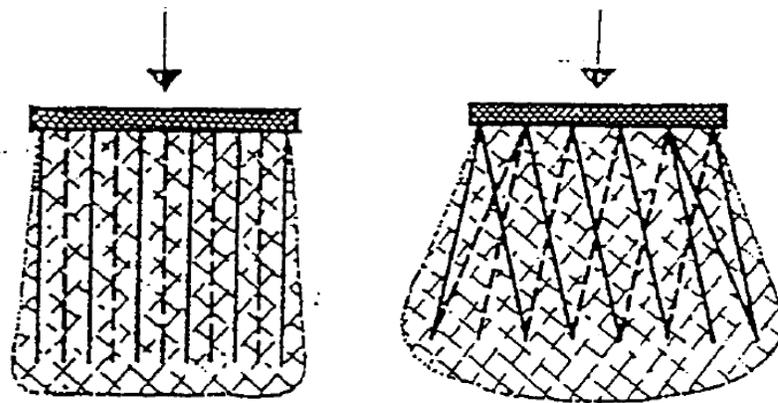


Figure 3 – Single Pile vs. Reticulated Pile Network (Muhra, 1997)

This study proposes analysis methods that consider the such foundation systems as the group action of small diameter piles but with allowance for increased strength and stiffness properties resulting from the internal reinforcement mechanism. The analysis method for micropiles will be outlined following the methods proposed for driven, ungrouted piles, as this will provide an accurate representation of micropile behavior for both vertical and lateral loads. The analysis first presents the method to analyze a single pile and then extended to a group of piles under both vertical and lateral loads. A method for determining the force distribution in a

pile group is also proposed. Finally, the results of the proposed pile group analysis are compared to the results obtained by Pin Foundations, Inc. for their Diamond Pier DP-50 foundation system.

MODEL DEVELOPMENT

The procedure for determining axial and lateral pile capacities was followed using Scott (1981), and Poulos and Davis (1980). Although variations of the equations proposed by these two methods are outlined in other texts and published papers, these variations only incrementally improve estimates of pile behavior for specific soil or loading conditions. The disagreements on values for deflections, stresses, etc. err on the conservative side for the procedure outlined, and these errors tend to decrease as the length-to-diameter ratio of the pile increases (Poulos, 1971). Scott (1981) will be used predominately in this paper because, in addition to its simplicity, it is regarded as a premier method for determining pile behavior. Because pile groups may take any shape, it is important in this paper to outline the general procedure to obtain a solution for the capacity of these pile groups.

Single Pile – Axial Loads

The axial pile analysis was represented by a Winkler soil spring model under elastic deformations of both the soil and the pile (see Fig. 4).

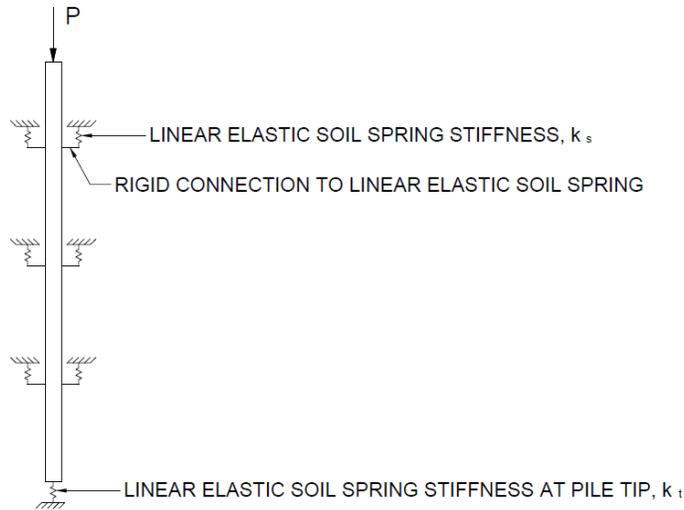


Figure 4 – Vertical Pile Winkler Spring Model

Because elastic deformations were assumed, the shearing stress along the length of the pile, τ , can be represented by the following equation:

$$\tau = k_s w \quad (1)$$

where k_s is the Winkler subgrade reaction modulus for elastic behavior, assumed to be a constant value with depth, and w is the axial displacement of the pile. Equation (1) is similar to that of other springs found in nature ($F = kx$), however, the spring constant here has units of FL^{-3} . k_s is a property of the soil that can be found through in situ testing, laboratory tests on undisturbed samples, or back calculation from pile load tests. Values of k_s may also be estimated through various forms of literature; however, this approach should be used with caution as these equations are usually applicable to specific soil or loading conditions. Scott (1981) proposed for a generic soil:

$$k_s = \frac{G_s}{4a} \quad (2)$$

where G_s is the soil shear modulus, and a is the radius of a circular pile. G_s is given by:

$$G_s = \frac{E_s}{2(1 - \nu_s)} \quad (3)$$

where ν_s is the Poisson ratio of the soil.

The assumption of elastic behavior is justified in pile design as long as the resulting deflections are small. If the deflections become large, plastic behavior of the soil will need to be included in the design. Additionally, the equations presented in this paper regarding pile groups were derived assuming an elastic response of the soil. A plastic response of the soil on a group of piles would change the interaction coefficients because some piles would experience plastic soil behavior before others. Table 1 illustrates the soil modulus for a cohesionless soil proposed by Poulos (1971), based on the density of the soil and an elastic response.

Table 1 – Soil Density vs. Soil Modulus (Poulos, 1971)

Soil Density	Range of values of E_s (psi)	Average E_s (psi)
Loose	130 – 300	250
Medium	300 – 600	500
Dense	600 – 1,400	1,000

Another assumption made is that the subgrade reaction modulus is constant with depth. This assumption is accurate for cohesive soils; however, cohesionless soils display an increase in the subgrade reaction modulus with an increase in depth. The current methods to account for an increasing soil modulus assumes the stress distribution in the pile remains unchanged, which Poulos (1971) says is questionable. The design can be conducted with an increasing subgrade reaction modulus, and the method will be the same as follows in this report. A constant subgrade reaction modulus will be used here to simplify the design procedure, and this method will provide an accurate estimate for the Diamond Pier DP-50 foundation system because the soil will experience minimal increase in soil modulus at the shallow depths to which the pins are driven.

Scott (1981) proposed the soil reaction modulus at the pile tip, k_t , to be:

$$k_t = \frac{G_s}{8a} \quad (4)$$

It is important to keep in mind there are two components to vertical piles resistance: shaft resistance and pile tip resistance. In practice, the contribution due to pile tip resistance will likely be much smaller for small diameter piles than the contribution due to shaft resistance, unless the pile tip is enlarged to increase its capacity, or it is embedded in rock or hard stratum. In most cases, the tip resistance may conservatively be omitted. Here, the pile tip resistance is included.

The equation for axial deflection, w , resulting from the axial load, P , can be derived:

$$w = \frac{P}{\lambda EA} (D_1 e^{-\lambda z} - D_2 e^{\lambda z}) \quad (5)$$

in which, E and A are the axial modulus and cross-sectional area of the pile, respectively, assumed to be constant in this analysis, and z is the depth at which the deflection is calculated. λ is the “characteristic” of the system, and is defined by:

$$\lambda = \sqrt{\frac{Sk_s}{EA}} \quad (6)$$

in which S is the perimeter of the pile. As can be seen by equation (6), λ is an important parameter in the determination of pile behavior because it includes both the soil modulus and pile modulus, k_s and E , respectively. When λ is small, the pile is stiff in relation to the soil. On the other hand, a large λ indicates a stiff soil compared to the pile. λ is in units of L^{-1} . The inverse of λ has units of L , and is called the “characteristic length” of the system.

The constants D_1 and D_2 are given by:

$$D_1 = \frac{1}{1 + \left(\frac{1 - \frac{\lambda E}{k_t}}{1 + \frac{\lambda E}{k_t}} \right) e^{-2\lambda l}} \quad (7)$$

$$D_2 = \frac{1}{1 + \left(\frac{1 + \frac{\lambda E}{k_t}}{1 - \frac{\lambda E}{k_t}} \right) e^{2\lambda l}} \quad (8)$$

The vertical deflection, w , can be differentiated with respect to z once to obtain the force, F , and twice to obtain the shearing stress, τ , at any point along the length of the pile:

$$F = P(D_1 e^{-\lambda z} + D_2 e^{\lambda z}) \quad (9)$$

$$\tau = -\frac{\lambda P}{S} (D_1 e^{-\lambda z} - D_2 e^{\lambda z}) \quad (10)$$

The contribution of the force at the pile tip, P_t , can be computed to be:

$$P_t = \frac{P A_t k_t}{\lambda E A} (D_1 e^{-\lambda l} - D_2 e^{\lambda l}) \quad (11)$$

in which A_t , represents the cross-sectional area of the pile at the tip. The area at the pile tip may be equal to the cross-sectional area of the pile, as used in calculation of the axial resistance related to shaft resistance, or it may be some other value. Here, the pile is assumed to have a constant area over its entire length.

The deflection of the pile due to uplift can be determined by considering only the shaft resistance, and neglecting the contribution of the pile tip. Scott (1981) proposed the following equation to determine the deflection without tip resistance:

$$w = \frac{P}{\lambda E A} (C_1 e^{\lambda z} + C_2 e^{-\lambda z}) \quad (12)$$

where C_1 and C_2 are constants determined by:

$$C_1 = \frac{e^{\lambda l}}{e^{\lambda l} - e^{-\lambda l}} \quad (13)$$

$$C_2 = \frac{e^{-\lambda l}}{e^{\lambda l} - e^{-\lambda l}} \quad (14)$$

From equations (5) and (9) through (12), the characteristics of pile reaction for a single pile under axial load can be determined.

Pile Group – Axial Loads

Because the axial load analysis for an individual pile was based on the displacement characteristics of the pile, the pile group, analysis will also be a function of displacement. The nine pile group used by Scott (1981) (see Fig. 5) will be used here to demonstrate how to apply the design equations for a general case.

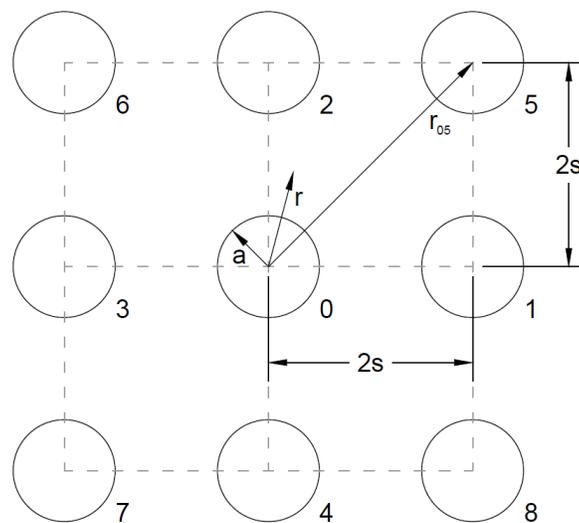


Figure 5 – Nine Pile Group

In order to determine the subgrade reaction coefficient of a group of piles, Scott (1981) proposed the following equation to compute displacements of a given pile of interest based on the resultant load on the pile and influence of nearby piles in the group:

$$w_i \left(\frac{r}{a} \right) = \frac{P}{2\pi G_s} \ln \left(\frac{\frac{b}{a}}{\frac{r}{a}} \right) \quad (15)$$

Equation (15) relates the deflection of the pile of interest, w_i , to the influence of a neighboring pile, which is spaced a dimensionless distance, r/a , away. r is the radius from the center of the pile of interest to the center of a neighboring pile, a is the radius of the pile of interest. b/a is the maximum dimensionless radius of pile influence, which Scott (1981) determined to be 50 pile radii (25 pile diameters), or $b/a = 50$, based on an axially loaded rigid disk in an elastic half-space. If,

$$\frac{r}{a} > \frac{b}{a}$$

the influence of the neighboring pile of radial distance r away on the pile of interest is zero because it is out of the radius of influence ($w_j(r/a) = 0$). If there are multiple nearby piles in the group, then equation (15) is summed to account for all piles within the radius of pile influence, $b/a = 50$. For example, pile 0 in the pile group proposed would have the following deflection equation:

$$w_0 = \frac{1}{2\pi G_s} \left(P_0 \ln \left(\frac{\frac{b}{a}}{\frac{1}{a}} \right) + P_1 \ln \left(\frac{\frac{b}{a}}{\frac{2s}{a}} \right) + P_2 \ln \left(\frac{\frac{b}{a}}{\frac{2s}{a}} \right) + P_3 \ln \left(\frac{\frac{b}{a}}{\frac{2s}{a}} \right) + P_4 \ln \left(\frac{\frac{b}{a}}{\frac{2s}{a}} \right) \right. \\ \left. + P_5 \ln \left(\frac{\frac{b}{a}}{\frac{2\sqrt{2}s}{a}} \right) + P_6 \ln \left(\frac{\frac{b}{a}}{\frac{2\sqrt{2}s}{a}} \right) + P_7 \ln \left(\frac{\frac{b}{a}}{\frac{2\sqrt{2}s}{a}} \right) + P_8 \ln \left(\frac{\frac{b}{a}}{\frac{2\sqrt{2}s}{a}} \right) \right) \quad (16)$$

Similar equations can be derived for the rest of the piles in the group. Notice that by symmetry about the pile group, piles 1, 2, 3, and 4, have the same influence on pile 0, and the same holds true for piles 5, 6, 7, and 8. Thus, the influence of pile 1 on pile 0, w_{01} , is the same as the influence of pile 2 on pile 0, w_{02} , or, $w_{01} = w_{02} = w_{03} = w_{04}$, and $w_{05} = w_{06} = w_{07} = w_{08}$. Equation (16) becomes:

$$w_0 = \frac{1}{2\pi G_s} \left(P_0 \ln \left(\frac{50}{\frac{1}{a}} \right) + 4P_1 \ln \left(\frac{50}{\frac{2s}{a}} \right) + 4P_5 \ln \left(\frac{50}{\frac{2\sqrt{2}s}{a}} \right) \right) \quad (17)$$

This technique of using symmetry in the pile group may be employed to save on computational effort; however, it is equally as valid to assess the pile deflections independently without considering symmetry. The results will be identical.

The pile cap is assumed to be rigid, and thus, the vertical deflections of the pile heads need to be identical. Equations of compatibility may be obtained by requiring the difference in vertical deflection between two piles to be zero, or:

$$w_i - w_j = 0 \quad (18)$$

where i and j are two piles in the group, for example, $i = 0$ and $j = 1$ in the nine pile group. This procedure can be carried out for the remaining piles in the group by keeping $i = 0$, and altering j from pile 1 to pile 7.

To maintain equilibrium, the individual forces on the piles, P_i , must also equal the total force applied to the system, P :

$$P = \sum_{i=1}^n P_i \quad (19)$$

It is convenient to normalize the forces by dividing the axial force, P , to obtain the proportion of the total load in each pile:

$$\sum_{i=1}^n P_i = 1 \quad (20)$$

where P_i now equals P_i/P . Using all equations similar to equation (16), and equation (20), one can obtain the normalized forces in each pile, P_i , based on the spacing and influence of neighboring piles in the group.

Because of the influence of neighboring piles, a modified subgrade reaction modulus is used to account for the decrease in the soil spring stiffness. Scott (1981) proposed the following dimensionless displacement to obtain the modified subgrade reaction modulus for an individual pile:

$$w' = \frac{2\pi G_s w}{P_N} \quad (21)$$

where G_s is the soil shear modulus, w is the deflection of the pile cap equal to the deflection of the head of any one pile (i.e. equation (16) or similar), and P is the normalized load on the pile group ($P_N = 1$).

The new modified subgrade reaction modulus value for the pile of interest, k_{si} , is:

$$k_{si} = \frac{P_i k_s}{w'} \ln \left(\frac{b}{a} \right) \quad (22)$$

where P_i is the normalized load on the pile of interest, k_s is the subgrade reaction modulus, determined by equation (2), w' is the dimensionless displacement from equation (21), and b/a is the dimensionless radius of influence ($b/a = 50$). The k_{si} values may now be used in equation (5) and equations (9) through (12) to obtain the deflection, force, and shearing stress in each pile of the pile group. Scott (1981) does not outline a procedure to modify the pile tip reaction modulus, k_t , so it is assumed that this property is not affected by pile spacing. This assumption

will be minimal for a general group of piles due to the minimal resistance provided by the pile tip compared to the pile shaft.

Single Pile – Lateral Loads

The lateral pile analysis is represented by a Winkler beam on an elastic foundation, whereby the soil is replaced by a set of infinitely closely spaced linear springs serving to resist the beam deflection (see Fig. 6). The beam in this analysis is represented by a pile of diameter, d . This method was chosen due to its simplicity and acceptable correspondence with more complicated methods or load tests. Here again, the soil and the pile are assumed to remain elastic throughout the deformation (i.e. small deflections). It is also assumed that the criteria for design is deflection governed, due to the long, slender, flexible nature of piles.

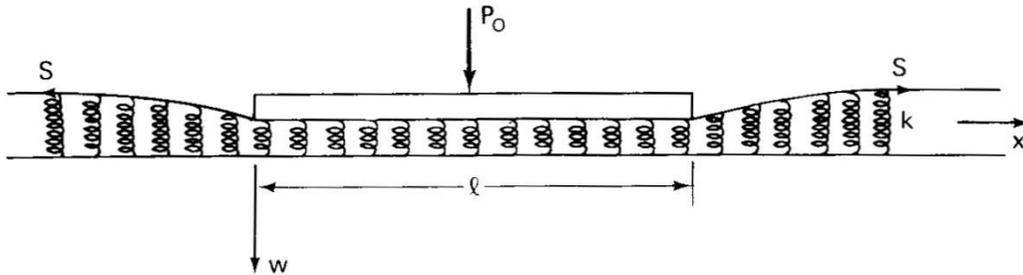


Figure 6 – Winkler Beam on Elastic Foundation (Scott, 1981)

The general solution to this type of Winkler beam is:

$$u = e^{\lambda z}(C_1 \cos(\lambda z) + C_2 \sin(\lambda z)) + e^{-\lambda z}(C_3 \cos(\lambda z) + C_4 \sin(\lambda z)) \quad (23)$$

which represents the lateral deflection, u , with respect to soil depth, z , where C_x are constants to be determined based on initial loading or boundary conditions, and:

$$\lambda = \sqrt[4]{\frac{k_h}{4EI}} \quad (24)$$

where k_h is the horizontal subgrade reaction modulus, E is the modulus of elasticity of the pile, and I is the moment of inertia of the pile. Scott (1981) proposed the following to relate k_h to the soil modulus, E_s :

$$k_h = E_s \quad (25)$$

Note that the value of λ derived in equation (24) is different than that of the axial pile load because of the different mechanisms that were assumed for soil response. In the axial load case, the pile acts like a column to resist the load by the shear stress developed around the perimeter of the pile, in addition to a minor bearing contribution by the pile tip. In the case of a laterally loaded pile, the pile acts as a beam to resist the applied load with a soil pressure acting across the width of the pile, or across the pile diameter, d , for a circular pile. Although the equation used to derive λ is different for lateral loads, it has the same implication of being a characteristic of the soil-pile interaction. Hetenyi (1946) proposed the following classification of beams on an elastic foundation:

1. Short beams: $\lambda l < \pi/4$
2. Beams of medium length: $\frac{\pi}{4} < \lambda l < \pi$
3. Long beams: $\lambda l > \pi$

The pile will be classified as a pile of finite length. Using the equations derived for a finite pile will allow piles of all lengths to be used in the analysis, including short and medium length, even though Scott (1981) states that representing piles as a long beam is generally acceptable for piles due to the long, slender nature of these members. When using a computer program, the additional difficulty in computation required for a beam of finite length is negligible.

Scott (1981) proposed the following equations to represent the deflection, u , rotation, θ , moment, M , and shear, S , for a fixed-head pile loaded by a concentrated force, H , at one end (see Fig. 7). Note that the pile shown is attached to a rigid pile cap:

$$u = \frac{H\lambda}{k_h} \left(\frac{\cosh(2\lambda l) + \cos(2\lambda l) + 2}{\sinh(2\lambda l) + \sin(2\lambda l)} \right) \quad (26)$$

$$\theta = 0 \quad (27)$$

$$M = \frac{H}{2\lambda} \left(\frac{\cosh(2\lambda l) - \cos(2\lambda l)}{\sinh(2\lambda l) + \sin(2\lambda l)} \right) \quad (28)$$

$$S = -H \quad (29)$$

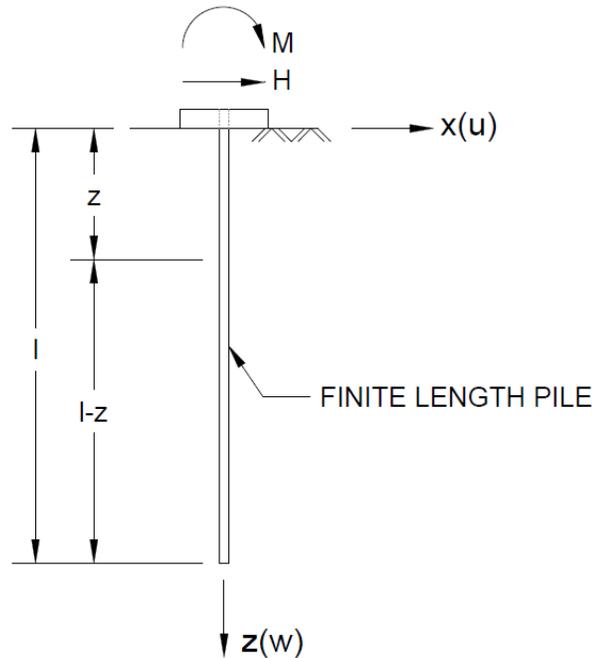


Figure 7 – Finite Length Pile

The moment here is related to the horizontal load due to the fact that the rotation at the pile head must be zero for a fixed-head pile. From equations (26) through (29), the deflection, rotation, moment, and shear may be computed and compared with the deflection requirements of the design.

Pile Group – Lateral Loads

In order to obtain the forces in the piles contained in the pile group, the procedure presented by Poulos and Davis (1980) for a fixed-head pile will be used. Because the piles are connected by a rigid pile cap, the lateral deflections of the piles will be identical, but the forces in the piles will differ. Much like in the axial group loading case, this analysis takes into account pile spacing, s , and angle between piles, β , to determine interaction coefficients for neighboring piles. For a general group of piles in an elastic soil:

$$u_j = \bar{u}_H \left[\sum_{\substack{i=1 \\ i \neq k}}^n (H_i \alpha_{uHji}) + H_j \right] \quad (30)$$

where u_j is the lateral deflection of pile of interest, pile j , \bar{u}_H is the unit reference displacement determined by applying a unit horizontal force to a fixed-head pile (equation (31)), H_i is the load on pile i , H_j is the load on pile j , and α_{uHji} is the interaction coefficient between pile j and i considering the distance and angle, β , between the piles. Note that Poulos and Davis (1980) used the symbols, ρ , to designate deflection, k , to designate the pile of interest, and j to represent the influence of pile j on pile k . These symbols have been modified here to be consistent with the other methodologies presented in this paper. \bar{u}_H is determined by the equation:

$$\bar{u}_H = \frac{I_{\rho F}}{E_s l} \quad (31)$$

where $I_{\rho F}$ is the deflection influence factor, determined by Fig. 8 using the length-to-diameter ratio, L/d , relative pile/soil stiffness, K_R :

$$K_R = \frac{EI}{E_s l^4} \quad (32)$$

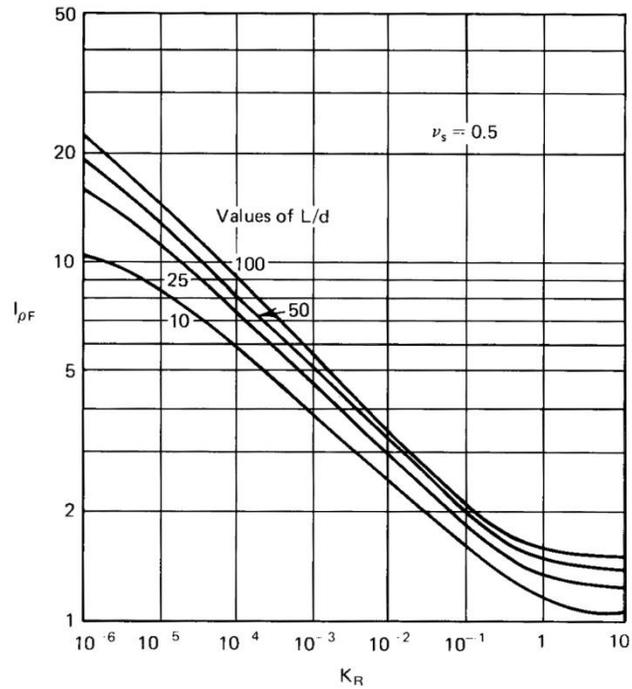


Figure 8 – $I_{\rho F}$ vs. K_R (Poulos and Davis, 1980)

α_{uHji} can be determined from Figs. 9 through 12 using the s , β , K_R , and L/d ,

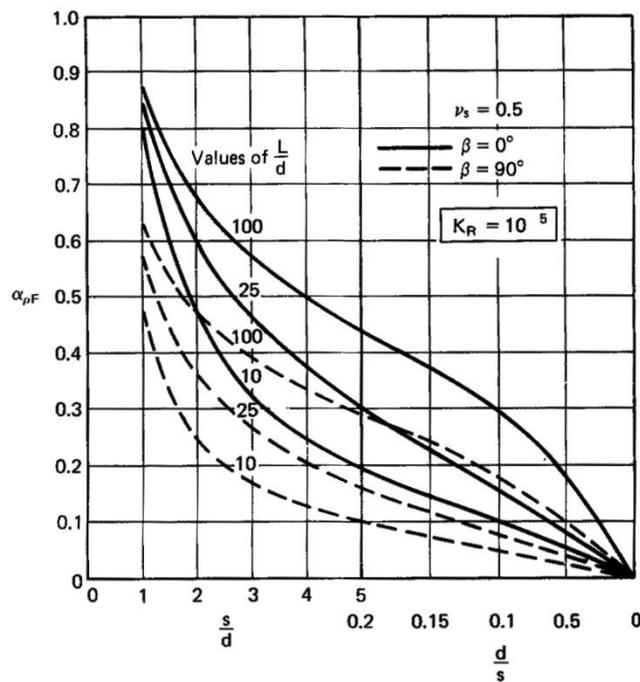


Figure 9 – α_{uHji} ($\alpha_{\rho F}$) vs. s/d for $K_R = 10^{-5}$ (Poulos and Davis, 1980)

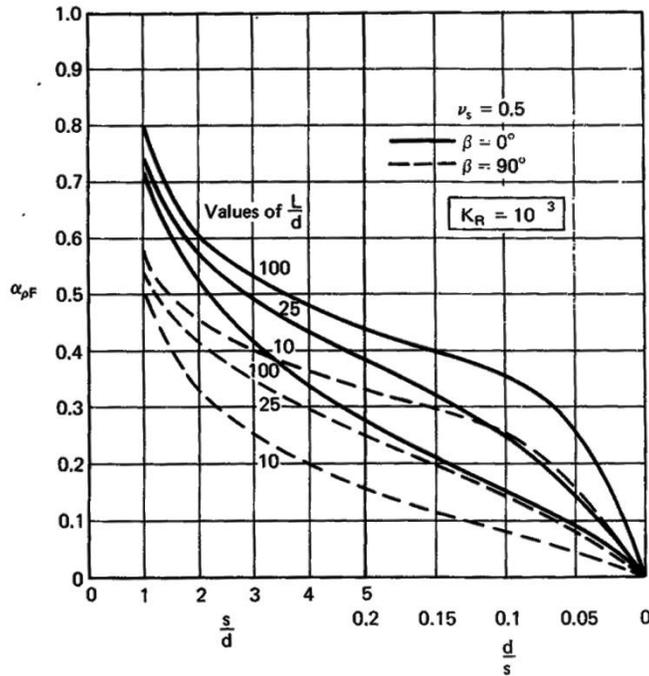


Figure 10 – α_{uHji} ($\alpha_{\rho F}$) vs. s/d for $K_R = 10^{-3}$ (Poulos and Davis, 1980)

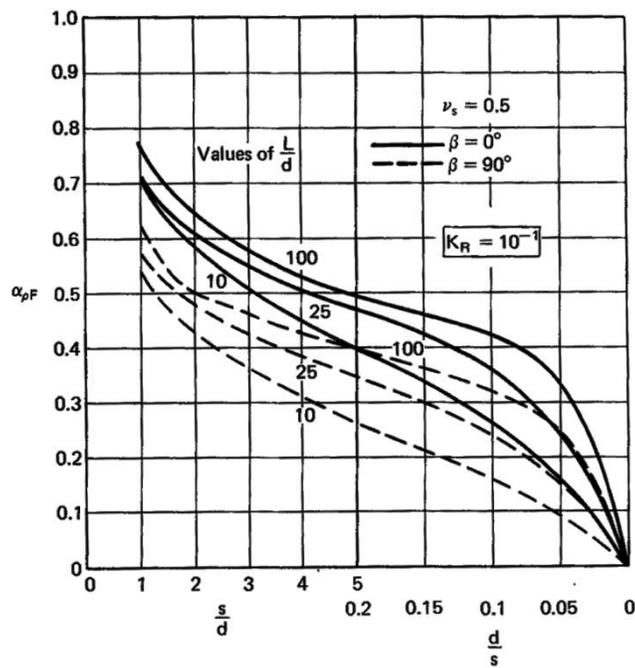


Figure 11 – α_{uHji} ($\alpha_{\rho F}$) vs. s/d for $K_R = 10^{-1}$ (Poulos and Davis, 1980)

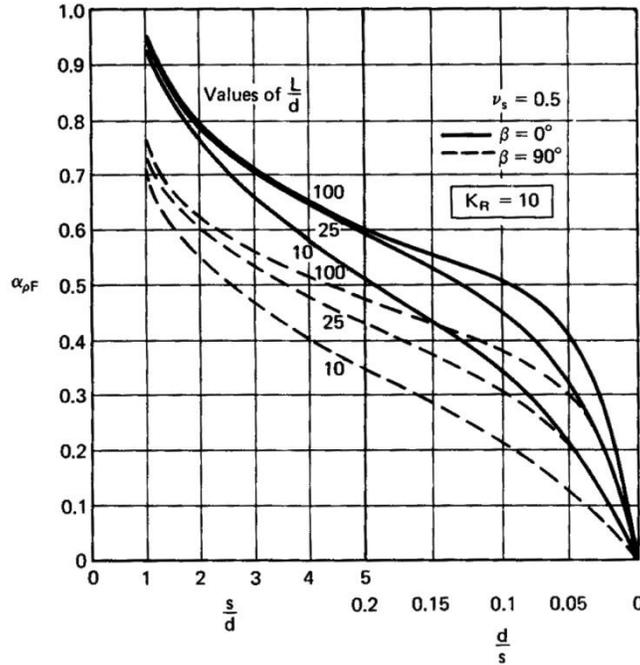


Figure 12 – α_{uHji} ($\alpha_{\rho F}$) vs. s/d for $K_R = 10$ (Poulos and Davis, 1980)

As can be seen by Figs. 9 through 12, this method for lateral group interaction coefficients assumes the Poisson ratio of the soil, ν_s , equal to 0.5. Poulos and Davis (1980) assert that ν_s has “very little influence on the interaction factors” and may be used for all soils.

The final equation required to determine the force in the piles due to the group effect is to take global force equilibrium, or:

$$\sum_{j=1}^n H_j = H \quad (33)$$

Normalizing lateral loads, as was done for the axial load case, yields:

$$\sum_{j=1}^n H_j = 1 \quad (34)$$

It should be noted again that symmetry may be accounted for in order to minimize computational effort. In order for multiple piles to be counted as symmetrical, they must be

symmetrical about the pile group, as well as symmetrical about the axis of loading. Thus, for the nine-pile group presented in the vertical pile group analysis (see Fig. 5), piles 1 and 2 are symmetrical, as are 2 and 4, and 5, 6, 7, and 8.

In order to obtain the modified horizontal subgrade reaction modulus for a laterally loaded pile group, the analysis method presented by Scott (1981) will be used, utilizing the forces and deflections calculated by the Poulos and Davis method. Scott (1981) defines a dimensionless coefficient, u' as:

$$u' = \frac{u_j G_s}{H_N} \quad (35)$$

where u_j is the displacement of the pile group (as calculated in equation (30)), and H_N is the normalized load placed on the group ($H_N = 1$). Thus, the modified subgrade reaction coefficient for pile j may be computed as:

$$k_{hj} = \frac{G_s H_j}{u'} k_h = \frac{H_j}{u_j} k_h \quad (36)$$

in which H_j is the normalized force on pile j (i.e. the load on the pile divided by the total load).

The modified horizontal subgrade reaction coefficient may now be used to compute the quantities in equation (23) and equations (26) through (29).

General Pile Groups

So far, the analysis methods have consisted of determining vertical and lateral displacements of a vertical single pile, or pile group of vertical piles. In reality, batter piles are often used to generate more resistance to the lateral displacements due to lateral loads or moments placed on the pile group.

The procedure outlined by Scott (1981) will be followed to determine the loading (axial, lateral, and moment) distribution to the piles, and rotation and deflections of the pile cap. It was assumed in this analysis that the piles are fixed to a rigid pile cap, the piles are fully embedded in the soil, and the cap is at ground level, but does not interact with the soil. Also, it is assumed that the deflection takes place at the center of the pile cap. Fig. 13 demonstrates a generic pile group with the coordinate system for this analysis.

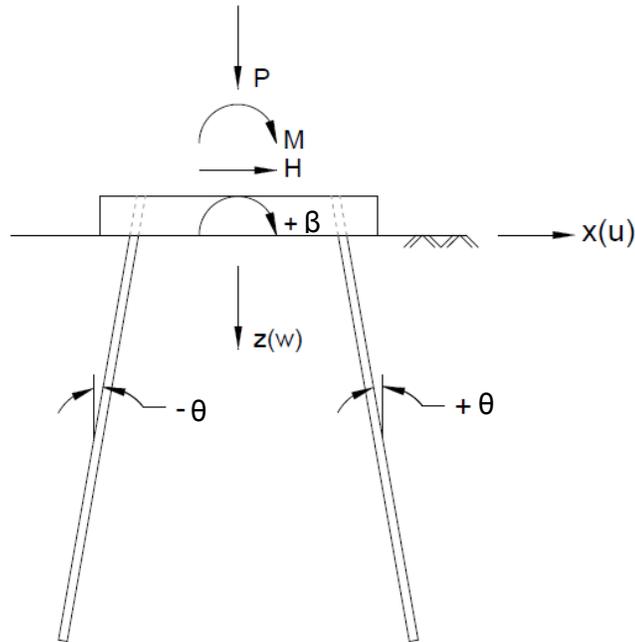


Figure 13 – General Pile Group

By taking global equilibrium of the system, the total axial deflection of pile j , e_j , is related to the horizontal pile cap displacement, u , vertical pile cap displacement, w , and pile cap rotation, θ , by:

$$e_j = e_u + e_w + e_\beta \quad (37)$$

where:

$$e_u = u \sin(\alpha_j) \quad (38)$$

$$e_w = w \cos(\alpha_j) \quad (39)$$

$$e_\beta = \beta x_j \cos(\alpha_j) \quad (40)$$

x_j = Position of pile j with respect to the x-axis

$$\beta = \theta \quad (41)$$

The forces in each pile are determined by the stiffness of the pile relative to the loads induced to the pile group. The axial load on pile j , P_j , can be related to the axial deflection, e_j , by the stiffness, K_{pej} :

$$P_j = K_{pej}e_j = K_{pej}[u \sin(\alpha_j) + w \cos(\alpha_j) + u \sin(\alpha_j)] \quad (42)$$

where:

$$K_{pej} = \frac{\lambda EA}{D_1 e^{-\lambda l} - D_2 e^{\lambda l}} \quad (43)$$

where the variables in equation (43) are for the axial load case. The modified subgrade reaction modulus, k_{si} , should be used in the calculation of all parameters to account for the interaction between neighboring piles. P_j is positive when pile is in compression.

Again, by taking global equilibrium of the system, the total lateral deflection of pile j , s_j , is related to the vertical cap displacement, w , horizontal cap displacement, u , and cap rotation, θ , by:

$$s_j = s_u - s_w - s_\beta \quad (44)$$

where:

$$s_u = u \cos(\alpha_j) \quad (45)$$

$$s_w = w \sin(\alpha_j) \quad (46)$$

$$s_\beta = \beta x_j \sin(\alpha_j) \quad (47)$$

For lateral deflection without rotation, the lateral load and moment at the pile top, H and M , respectively, were obtained from a finite beam on an elastic foundation loaded at mid span. The lateral load at the top of pile j , H_j , can be related to the lateral deflection, s_j , and rotation, β , by the stiffness, K_{Ts_j} and $K_{T\theta_j}$:

$$H_j = K_{Hs_j}s_j - K_{H\theta_j}\beta = K_{Hs_j}[u \cos(\alpha_j) - w \sin(\alpha_j) - \beta x_j \sin(\alpha_j)] - K_{H\theta_j}\beta \quad (48)$$

where:

$$K_{Hs_j} = \frac{k_h}{\lambda} \left(\frac{\sinh(2\lambda l) + \sin(2\lambda l)}{\cosh(2\lambda l) + \cos(2\lambda l) + 2} \right) \quad (49)$$

$$K_{H\theta_j} = \frac{k_h}{2\lambda^2} \left(\frac{\sinh^2(\lambda l) + \sin^2(\lambda l)}{\cosh^2(\lambda l) + \cos^2(\lambda l)} \right) \quad (50)$$

where λ in equations (49) and (50) are for the lateral load case. Here again, the modified horizontal subgrade reaction modulus, k_{hj} , should be used in the calculation of all parameters to account for the interaction between neighboring piles.

The Moment at the top of pile j , M_j , can be related to the lateral deflection, s_j , and rotation, β , by the stiffness, K_{Ms_j} and $K_{M\theta_j}$:

$$M_j = K_{Ms_j}s_j - K_{M\theta_j}\beta = K_{Ms_j}[u \cos(\alpha_j) - w \sin(\alpha_j) - \beta x_j \sin(\alpha_j)] - K_{M\theta_j}\beta \quad (51)$$

where:

$$K_{Ms_j} = \frac{k_h}{2\lambda^2} \left(\frac{\cosh(2\lambda l) - \cos(2\lambda l)}{\cosh(2\lambda l) + \cos(2\lambda l) + 2} \right) \quad (52)$$

$$K_{M\theta_j} = \frac{k_h}{2\lambda^3} \left(\frac{\sinh(\lambda l)\cosh(\lambda l) - \sin(\lambda l)\cos(\lambda l)}{\cosh^2(\lambda l) + \cos^2(\lambda l)} \right) \quad (53)$$

The pile forces, P_j and H_j , may be resolved about the global coordinate system to obtain the global forces, X_j and Z_j :

$$X_j = -P_j \sin(\alpha_j) - H_j \cos(\alpha_j) \quad (54)$$

$$Z_j = -P_j \cos(\alpha_j) + H_j \sin(\alpha_j) \quad (55)$$

Thus, global equilibrium may be satisfied by equating the cap forces and pile forces to sum to zero:

$$H + \sum_{i=1}^n X_j = 0 \quad (56)$$

$$P + \sum_{i=1}^n Z_j = 0 \quad (57)$$

$$M + \sum_{i=1}^n M_j + \sum_{i=1}^n x_j Z_j = 0 \quad (58)$$

At this point in the analysis, all the equations have been laid out to determine the deflections and rotation of the pile group, and the forces placed on the piles in the pile group. Because this is a two dimensional model, should the batter angle, θ , slope out of the plane of the figure, the batter angle shall be taken as θ when determining vertical load capacity, but shall be taken as zero for lateral load capacity (Scott, 1981). The accuracy and usefulness of this procedure will be outlined in the following illustrated example.

Case Study: Diamond Pier DP-50 Foundation System

In the illustrated example, a Pin Foundations, Inc. Diamond Pier DP-50 foundation system (see Fig. 1 for schematic) was used to rate the capacity of the pile group in different soils via a Matlab computer program. All of the information provided within this report has been retrieved from the Pin Foundations, Inc. website, and all information is available in the public

domain (www.pinfoundations.com). Information regarding this foundation system can be found in Appendix A.

The vertical capacity of the pile group in this example was determined by utilizing the pile group equations outlined in this report, governed by the limiting deflections of foundations, which here were assumed to be 0.5in. vertical (z). The pile cap was assumed to be rigid, providing fixity for the pile heads, but did not interact with the soil. The steel pins used in the foundation were assumed to be spaced 6" o.c. apart at the pile cap and the pins were angled at 45° from vertical. As discussed previously, the batter angle selected for the two middle piles sloping out of the plane of the figure was θ for vertical load capacity, and zero for lateral load capacity. Because of the reinforcing effect of the piles in the soil, as discussed in the introduction, an increase in 10% in the soil modulus was used. This increase was an assumed value for the increase in cohesion of the soil as a result of the small diameter piles. Table 2 illustrates the allowable capacity of the Diamond Pier DP-50 foundation system when used in soils of varying stiffness.

Table 2 – Vertical Load Capacity vs. Soil Modulus for Diamond Pier DP-50

Soil Modulus E_s (psi)	Vertical Load P (lb)
100	1700
200	3200
300	4700
400	6100
500	7400
600	8800
700	10100
800	11500
900	12800
1000	14100
1100	15400
1200	16700
1300	18000

1400	19300
1500	20600

As can be seen by Table 2, the capacity of the pile group is highly dependent on the soil modulus, and as the soil becomes stiffer, the vertical capacity of the foundation increases. This conclusion comes from the fact that the soil is modeled as a linear elastic spring, and as the soil spring becomes stiffer, the capacity of the soil to accommodate larger forces increases. From Table 2, there is a linear relationship between the soil modulus and the vertical load capacity of the foundation.

Table 2 may be used to predict the load capacity of the Pin Foundations, Inc. Diamond Pier DP-50 foundation system. As described in Table 1, Poulos (1971) proposed the soil modulus for a loose density soil to range between 130-300psi. This would be a conservative estimate of soil behavior as this would correspond to weak soils. Using this range of soil modulus values in conjunction with Table 2, the predicted load capacity of the Diamond Pier DP-50 foundation would be expected to vary from 2150-4700lb for vertical capacity. Pin Foundations, Inc. calculated a vertical bearing capacity of 2700lb for a 1500psf bearing capacity soil, and 3600lb for a 2000psf bearing capacity soil for the same foundation using a bearing capacity analysis (Pin Foundations, Inc., 2008). These soils were selected because they represent the worst soils observed by the 2003 IRC, which was the design code used in the analysis. As can be seen by the results, the predicted vertical capacity values obtained from pile group analysis for weak soils correspond well with the bearing capacity values obtained by Pin Foundations, Inc. This gives credibility to the application of the pile group analysis, and the assumptions contained in that analysis, to predict deflection behavior of the Diamond Pier DP-50 foundation.

One limitation in this analysis is there is no method to account for the reinforcing effect of the pile in the soil. The increase in soil modulus due to this effect was assumed here to be 10%, based on engineering judgment, but this assumption may not be accurate in all cases, or may be different when analyzing vertical loads and lateral loads. In particular, one study has noted the increase in lateral capacity for batter micropiles due to the arching effect within the soil (Kulhawy and Mason, 1996). Further research into this field would yield increased capacities for pile groups when considering the reinforcing effect of the soil-pile system.

One interesting aspect of the pile group to observe is the variation in load capacity on the pile group by manipulating the pile batter angle, θ , which was assumed in Table 2 to be 45°. The following analysis was conducted on the Diamond Pier DP-50 foundation. The limiting deflections on the foundation here were assumed to be 0.5in. horizontal (x), and 0.5in. vertical (z). The Simpson Strong Tie ABU post bracket used by the Diamond Pier foundation (Pin Foundation, Inc., 2007) provides a simple connection at the top of the pile cap; thus, only vertical and lateral loads were imposed on the foundation. These loads were placed on the foundation simultaneously to provide a worst case loading to the foundation.

Table 3 illustrates the results of this analysis for an assumed soil modulus of 500psi. This soil modulus was selected because a medium stiff soil would better illustrate the variations in allowable loads compared to a soft soil, where variations would be minimal. The values proposed in Table 3 should not be used in the design of the foundation system as a study of the site soil characteristics and final design of the foundation system should be conducted by a qualified engineer.

Table 3 – Load Capacity vs. Batter angle, θ , for DP-50, $E_s = 500psi$

Batter Angle θ (Degrees)	Vertical Load P (lb)	Lateral Load H (lb)
------------------------------------	-------------------------	------------------------

0	9400	3600
10	9200	4000
20	8700	4500
30	7700	5400
40	6400	6400
50	5000	7600
60	3600	8700
70	2400	9700
80	1600	10300
90	1300	10400

As can be seen by Table 3, the maximum vertical loads occurred when θ approached 0° . This result would be expected because piles are much stiffer under axial loads compared to lateral loads. The maximum lateral loads occurred as θ approached 90° . This result would also be expected because the angle of the piles allows the piles to axially resist lateral loadings.

The resulting changes in load capacity due to the manipulation of batter angles is non-linear, with the maximum changes occurring in the 30° - 60° range (see Fig. 14). Likewise, the smallest changes occurred at the extremities of the batter angles. The variation in the batter angles would allow an engineer to optimize the design of the foundation to accommodate the needs of the project.

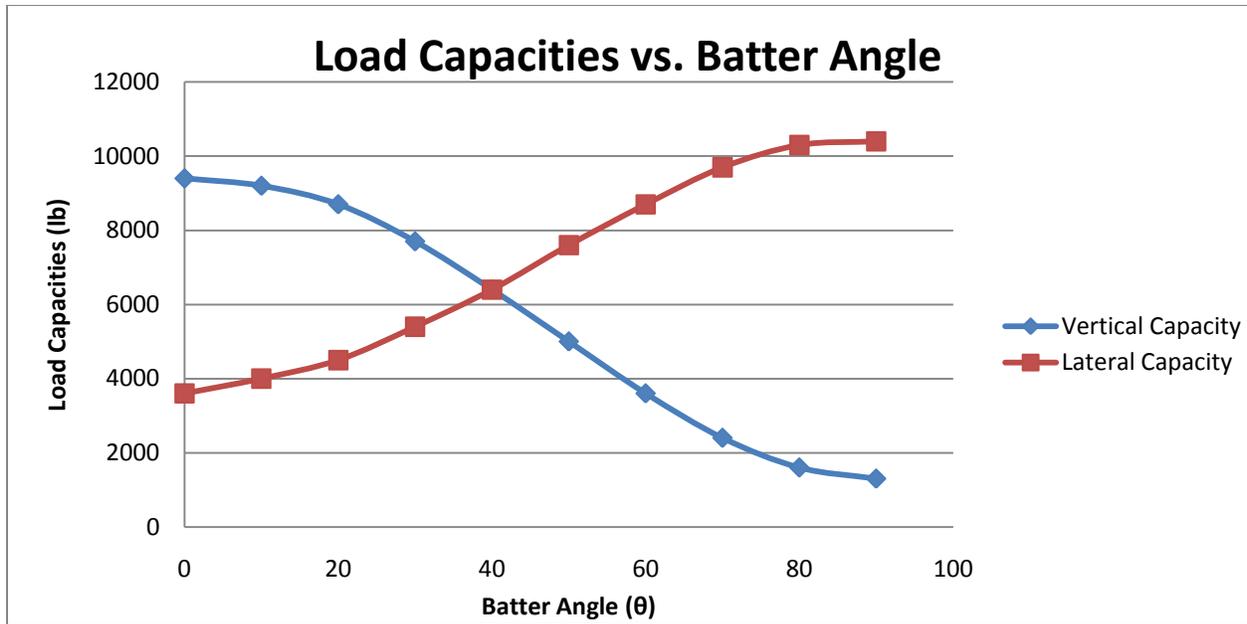


Figure 14 – Allowable Load Capacities vs. Batter Angle

SUMMARY AND CONCLUSIONS

The goal for this paper was to present a rational pile group analysis method capable of being applied to small diameter piles. This analysis method was two dimensional, with the inputs to the model being loading conditions, pile properties, pile spacing, pile batter angle, and soil modulus. The outputs obtained are pile group deflections, pile cap rotation, and pile forces.

Two case studies were conducted on the Pin Foundations, Inc. Diamond Pier DP-50 foundation system. The first involved calculating the vertical capacity of the foundation. The calculated vertical capacity using pile group analysis was expected to range from 2150-4700lb for weak soils, while Pin Foundations Inc. calculated a bearing capacity of 2700lb and 3600lb for 1500psf and 2000psf soils, respectively. As can be seen, the values obtained from the bearing capacity analysis fell within the range of expected values using the pile group analysis.

The second case study included changing the batter angle of the Diamond Pier DP-50 foundation with the simultaneous application of vertical and lateral loads. This study showed

that the vertical capacity of the foundation system decreased as the batter angle was increased, while the lateral capacity increased with increasing batter angle. This result comes from the fact that these piles are stiff in the axial direction and more compliant in the transverse direction.

There were limitations to the model as it was presented in this report. The first was the assumption of the foundation system being two dimensional. The consequence of this assumption is that loading conditions may not be considered simultaneously in multiple directions, and required piles with batter angles out of the plane to be analyzed with a batter angle of zero for lateral capacities, and the actual batter angle for vertical capacities. A rational mechanics based model is needed to analyze the three dimensional configuration of the pile group. The second assumption is a 10% increase in soil modulus due to the cohesion increase resulting from the small diameter pins being driven into the soil. A method is needed to quantify the effect of the soil-pile interaction.

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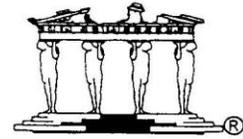
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Appendix A

Pin Foundation, Inc. Technical Sheets

DIAMOND PIER™

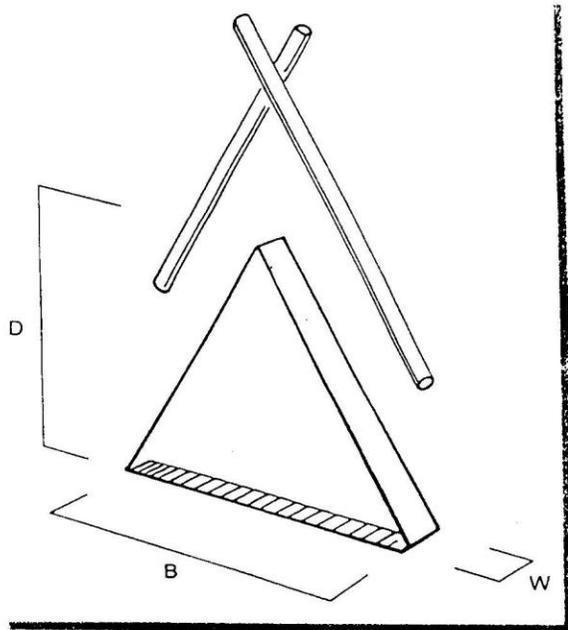
ENGINEERING — PIER CAPACITY ANALYSIS OVERVIEW



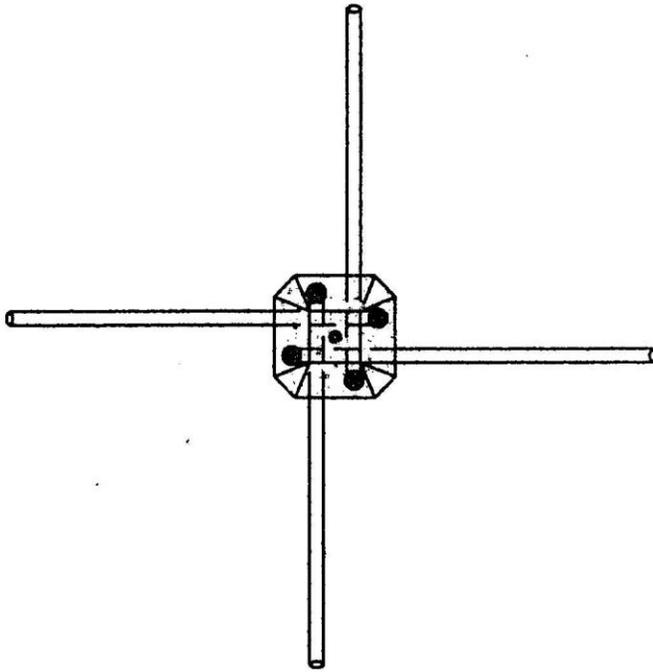
This overview presents the techniques that we use in determining the capacity for the Diamond Pier system. Our design methods are based on sound and accepted geotechnical engineering principles, and have been reviewed by qualified and competent professional engineers. The capacity calculations result from over two decades of study and testing and will continue to evolve as our knowledge and experience grows.

Pin Foundations differ from traditional vertical pile foundations in that the length of the pin is determined before construction and no driving resistance has to be achieved before installation is complete. This is possible because the mechanism a pin foundation uses to transfer load at the pile-soil interface is unique. Vertical piles rely on sliding frictional resistance along the length of the pile and point resistance at the pile tip to transfer the applied load. Pin Foundations, however, utilize the pressing action of the pin against the soil around it and along the length of the pin to transfer load. Our bearing capacity analysis works on the principle that a coherent soil mass develops around the pin and propagates the applied load downward and radiating outward; the degree of which is dependent on the soil conditions encountered. Although the pin does develop some sliding and tip resistance, those benefits are not considered, in the interest of being conservative and eliminating complexity from the calculations. The Diamond Pier uplift and lateral resistance works on the same pile-soil interface principle, but bearing capacity usually is the governing design criteria.

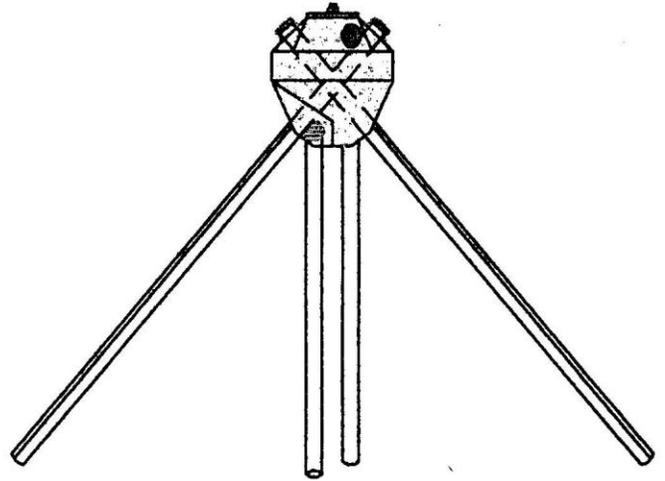
Our bearing analysis combines two pins to form a rigid A-frame, and calculates the capacity of the soil wedge between the pins. That soil wedge (shown) represents an equivalent spread footing along its base; with length (B) being the distance between the pin tips, and the width (W) defined by the arching factor. The arching factor, a function of specific soil characteristics, describes how the load radiates outward and engages soils beyond those immediately below the pin; typically 2 to 3 times the pin diameter. The depth (D) of the equivalent footing is measured vertically from the surface of the soil down to the tip of the pin. This value can be adjusted, if needed, to account for neglected soil near the surface that does not contribute to the overall bearing capacity of the system. Using these dimensions, along with the soil's phi angle, unit weight, and cohesion, we calculate the Diamond Pier's capacity using the same accepted general bearing capacity equation used to design traditional shallow foundations. The pier's design capacity is made more conservative by assigning factors of safety to the derived values.



The concrete portion of the Diamond Pier combines four pins into two rigid A-frames, and provides a flexible connection system to the supported structure. The pier has undergone significant research and development to assure it is strong and durable. The concrete has been laboratory and field tested to simulate the conditions that will be encountered in many service environments, including freeze-thaw conditions. We continuously test new concrete designs and reinforcement options in an effort to improve the pier and its maximum load rating.

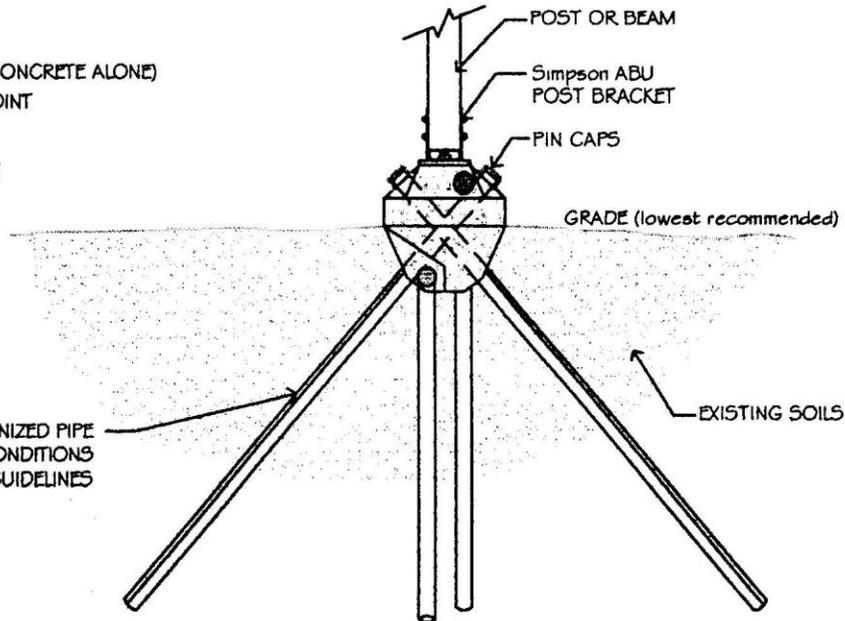


PLAN VIEW W/ 5.25 FOOT PINS



ELEVATION W/ 5.25 FOOT PINS

WEIGHT APPROX. 100 LBS. (CONCRETE ALONE)
 13 INCHES SQUARE AT MIDPOINT
 14 INCHES HIGH
 BRACKET SEAT - 5.5" SQUARE
 60° PINS SHOWN



1.5" NOM. DIAMETER GALVANIZED PIPE
 NOTE: LENGTH VARIES WITH SOIL CONDITIONS
 SEE MANUFACTURER'S CAPACITY GUIDELINES

DIAMOND PIER™ DP-100

US PAT. #5039256 & #6578333

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ICC Report ESR-1895 ICC Report ESR-1895 ICC



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no digging, no pouring, no waiting, no digging

CADD
Details
available.



What is a Diamond Pier® ?

The **Diamond Pier®** is a unique combination of pre-cast concrete and steel bearing pins. Together they form a solid foundation that reaches deep into the ground - **without having to dig or pour concrete!**

The pier is light enough to hand carry, and installs right at the surface with simple hand held tools.

It's better for the environment, and it can be used in any penetrable soil - sands, silts, clays and small rounded gravels. It is *not* intended for rocky soils, ledge, deep marine applications, loose fill, deep peats or perma-frost.

Once fixed in the ground, the foundation has an excellent bearing capacity, and the added benefit of uplift and lateral resistance. Post or beam brackets are attached to an anchor bolt pre-cast in the top of the pier, and framing can begin immediately - no digging, no waiting for the weather, no concrete curing and no soil piles to remove.

The first prototype of the Diamond Pier® was installed in 1984, supporting a deck that still stands level today. (See photo above.) The pier was patented in 1992, and has been used on public projects across the Country for more than a decade. The smaller DP-50 Pier with 36 inch Pins has now been recognized by the ICC, ESR - Report 1895, and is available with a variety of Pin lengths to builders throughout the United States.

Diamond Piers® are simple to use, and **save time, mess and money.**



First deck on prototype pier - 1984
Lawrence, New York

How are Diamond Piers® Installed?

The Piers are simple to install using small lightweight construction tools. The concrete head is simply set in a shallow hole cut into the surface soil, and the pins are driven through it and capped.

Just take care lifting the heavy piers. First, set the pier to its proper height, typically to its mid-point or deeper, depending on the structure and topography. Make sure that it is level and in its proper location, then pack the little bit of dirt removed back around the edges of the pier to fix it in place for pin driving.

Verify that there are no buried utilities and that your gloves and goggles are on. Slide pairs of pins into opposing holes, being careful not to tip the pier out of level. Hold the pin up against the top side of the concrete driving hole and sledge the Pins partway into the surrounding soil - maintaining plumb, and working around the pier on opposing pins until they begin to bite in the soil and lock the pier rigidly in place.

A small demo hammer with a pin driving bit is the optimum driving tool for finishing the Pin driving. As with the sledging, make sure the Pin is held against the top side of the driving hole as you drive it. This ensures that you will not crack or damage the concrete head.



For most structures, dead load should be applied before applying caps.

Leave the tops of the Pins protruding from the concrete approximately 3/4", apply the framing dead loads, and then seal the Pins with the rubber caps to the head with adhesive caulk. If there is enough room under the structure to drive Pins, a good part of it may be framed, squared and plumbed before finishing the Pin driving.

And the Diamond Pier is totally removable. If you put it in the wrong place, the Pins can be rotated back out and the pier relocated. The small piers should take about 10-15 minutes each to install, 20-30 minutes for the big ones - depending on Pin length and soil density. See next page for information about installing in heaving soils.

See back page for more information on removing Pins, or applying inspection plugs before Pin driving. (Visit www.pinfoundations.com for complete Installation Instructions and to view the Installation Video.)



Make sure to remove any deformations on the end of the pin, and caulk and seal the cap.



Heave Resistant

Do they work in Heave?

Yes. The spread Pin configuration works like a bell shaped footing to resist uplift, and the base of the pier is pointed to cleave shallow heaving soils. In properly drained sites, the DP-50 can absorb the strains of low to moderate heaves up to 1 inch. If in extreme conditions, some minor Pin displacement occurs, the Pins can simply be reset without affecting the structure. For the most severe soils, the larger piers, or the addition of rounded gravels around the head, may be required.

(See "Heave and Expansion" on the web site.)



Will the Pins Rust away?

No. The galvanized Pins are protected from significant corrosion because they are buried and not exposed to oxygen. Pins supplied are intended to last the life of the structure.

Some industrial sites or unique soils may require Pins with additional protection, or stainless steel.



Salt Corrosion Test

How much load can a Diamond Pier® carry?

There are currently three pier sizes. Their bearing capacities are based on Pin length and the soils they are engaged in. The DP-50, the smallest pier, is recognized by the ICC for generic load values equivalent to 1.8 square feet of bearing area. (See chart below)

The DP-75 and DP-100 pier capacities are not based on generic values, but are instead engineered for each individual site soil and loading condition. Each pier has different Pin diameters and maximum Pin lengths, and these allow for a variety of capacities in a given site soil. Their maximum capacities however are limited by the strength of the concrete head itself. For the larger piers, soils and loading information is required for a Pin Foundations engineer to provide compression, uplift and lateral capacities. (See bold text below)



DP-100, DP-75, DP-50 with corresponding Pin diameter and driving bit.

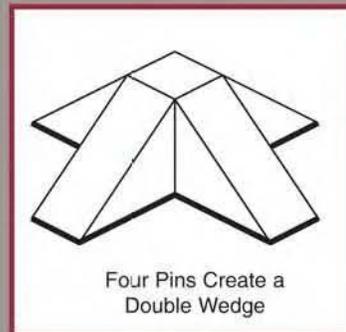
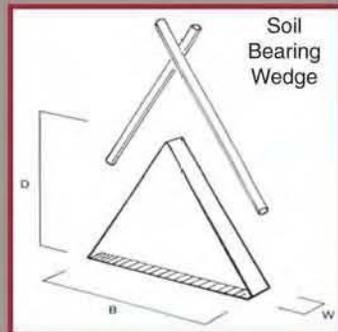


Concrete Strength Test

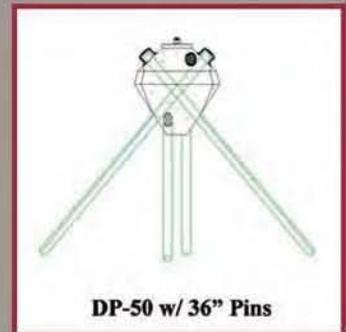
BEARING



Bearing Load Test - Pier and Pins Push against the Soil as a Single Solid Shape.



Four Pins Create a Double Wedge



DP-50 w/ 36" Pins

The Diamond Pier is a bearing system, and performs in the soil much like any flat bottom footing. Because the pins are short, very stiff, and locked in the pier under load, the entire foundation pushes against the soil as a single solid shape. Each pair of pins, acting in unison, bears on a wedge of soil immediately below it. Four pins create a double bearing wedge. In plan, this wedge has a base square footage area that can be applied to a given soil's psf just like a conventional concrete footing.

This chart shows the ICC recognized tested bearing strength of the DP-50 pier w/ 36" Pins, as it compares to conventional flat base round and square footings.

For the DP-75 and DP-100 pier, a site-specific capacity analysis may be

required. This can be obtained from our authorized engineers, if specific soils characteristics, such as Unit Weight, Angle of Internal Friction, and Cohesive Strength, are provided by a local certified geotechnical engineer.

RESIDENTIAL SOIL TYPES IRC 2003/2006, Table R401.4.1	CAPACITIES		Diamond Pier®
	CYLINDER	CUBE	
	18" dia.	16" sq.	DP-50, 36" Pins
2000 psf soil sands, silty sand, clayey sand; gravels clayey gravels, silty gravels	3540 #	3560 #	3600 #
1500 psf soil clays, silts, clayey silts, silty clays sandy clays, sandy silts	2655 # (1.77 SF)	2670 # (1.78 SF)	2700 # (1.8 SF Equiv. Bearing)

NOTES: 1. Cylinder and Cube capacity are based on 12 inch depth into natural grade, or deeper cylinders, 2 to 4 feet down, formed in waxed or plastic forms to the full depth of the hole.
2. Pins longer than 36 inches may be required in some frost or expansive soil zones.
3. Allowable loads utilize 4 Pins per Pier. Pin length includes that portion of the Pin embedded within the Pier.



Frame Foundations for Modular Structures



Direct Connect to Grade Beams for Decks



Porches, Carports



Stairways, Walkways

What can be Built with Diamond Piers® ?

Diamond Piers® can support just about any structure that connects to a post or beam bracket. Decks, porches, carports, walkways, ramps, stairs, gazebos, sheds - even houses!

Pier spacing, size and pin length depend on the soils, the weight of the structure itself and the live loads the structure is meant to carry - snow, people, hot tubs, etc.

On sloping sites, tall posts should be braced, as with any similar structure. Steep sites greater than 2:1 are not right for the Diamond Pier®, as the downhill Pin will not have enough soil to bear in.

On gentle slopes, series of piers may be set at differing heights to adjust for level. The pier may also be set completely below grade if preferred, or recessed from the outer edge of framing members in order to allow perimeter skirting to be constructed.

Brackets are structurally interchangeable on the two pier sizes, however larger brackets may hang over the top of the DP-50, requiring a heavy duty bracket with internal blocking to redistribute the load back to the top of the pier.



Boardwalks, Bridges



Heavy Decks, Big Timbers



Houses, Additions

Field Inspections?

Inspectors can verify Pin length at any time when inspection plugs are used. The plug is inserted in the lower end of the Pin before driving, and allows the inspector to slide a tape all the way down the inside of the Pin to measure its length.

Piers suffering a structural crack during Pin driving should be removed and replaced. Surface chips are not structural cracks.

Piers installed out of plumb more than 5 degrees should be reset before loading.



Inspection Plugs



Pin Removal

Trouble Installing?

A pier set in the wrong location can be moved! Just turn and lift the Pins with a pipe wrench to corkscrew them out. The pier may be also be rotated to avoid rocks or roots, and the Pins re-driven.

Most brackets have an extra large attachment hole, also allowing for horizontal adjustment.

Rocks or roots may also be dug to and removed, as long as the soil is repacked before pin driving.

Pins may be cut off if they hit deep impenetrable obstructions. See Installation Instructions for details. A list of appropriate driving hammers and bits is also on the website.

Are they Code Approved?

The Diamond Pier® DP-50 model is now recognized by the International Code Council for residential applications. The ESR report, ESR-1895, is available on the website along with the full Third Party test results. The Piers also meet all ASTM and ACI 318 standards for pre-cast concrete products, and all ASTM standards for galvanized steel hardware. The DP-100 pier must be submitted as an engineered alternate to the code.

Limited Warranty

If properly handled and installed, Pin Foundations Inc. will replace any of its manufactured components due to defects in quality or workmanship, for a period of three years from date of purchase. No other warranties are expressed or implied. PFI shall not be liable for any consequential or other damages, including injury to persons or property, arising from the use of its supplied products.

PIN FOUNDATIONS INC

1-253-858-8809

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Appendix B

Matlab Script and Sample Output

SAMPLE MATLAB OUTPUT

>> DP50Es100 **Input File***

P =

1700 **Axial Load on Pile Group**

H =

1000 **Lateral Load on Pile Group**

M =

0 **Moment on Pile Group**

Displ =

0.4458 **x-Displacement of Pile Cap**
0.47478 **z-Displacement of Pile Cap**
0.016655 **Rotation of Pile Cap (Radians)**

GlobeForces =

Axial	Shear	Moment	
-27.663	357.33	2384.1	Pile 1
445.46	186.85	912.21	Pile 2
657.64	-156.91	-2753.8	Pile 3
445.46	186.85	912.21	Pile 4

* **Bold text will not appear in Matlab output, it has been included post-analysis to describe output characteristics.**