# SETTLEMENTS AND STRENGTHENING OF SOFT CLAY ACCELERATED BY SAND DRAINS

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The 24-acre south portal island for the second Hampton Roads Crossing in Virginia was surcharged and the compression and strengthening of underlying soft clay were accelerated by jetted sand drains to meet the requirements of an economical construction scheme, to minimize post-construction settlements, and to maintain stability during construction. The island was extensively instrumented, and the paper presents analyses and conclusions from selected data. Both laboratory and field data indicate a much higher ratio of undrained shear strength to overburden pressure than would be expected from well-known correlations between this ratio and plasticity indexes. Laboratory values of the horizontal and vertical consolidation coefficients were lower than indicated by measured settlement rates; values derived from field permeability tests proved more reliable. Horizontal displacements in the soft clay, estimated by finite-element analyses and measured by inclinometers, proved to be useful indicators of the overall safety of the slopes. The effect of unknown factors and variable soil parameters was minimized by construction documents written with a specific bid item for "delay time," a device that proved financially beneficial. The solutions to the problems of this site can be used for similar sand drain projects where there is substantial uncertainty and variation of the soil parameters. The soil parameters reported are, of course, applicable only to similar extensive nonstratified clay deposits.

•CONSTRUCTION of the second Hampton Roads Bridge-Tunnel Crossing, connecting Norfolk with Hampton in Virginia, began in 1970 and is expected to be finished in 1975. It will consist of a 6,900-ft (2,100-m) long two-lane sunken-tube tunnel between two man-made islands and two trestles connecting the islands with the mainland. The first Hampton Roads Crossing, which was opened in 1957 (1, 2), parallels the new crossing at a distance of 250 ft (76 m).

Of the two man-made islands, the North Island is founded on sands and silty sands and presents no substantial settlement or stability problems. At the South Island, however, about 80 ft of normally consolidated clay overlies sandy soils, and sand drains and programmed surcharge were required to construct a stable island with a minimum of residual settlements. The design of the South Island was described by Rafaeli (3), and the present paper will discuss the results of an extensive instrumentation program, the behavior of the clay, and the performance of the sand drains.

The new South Island is approximately 1,950 ft long and 570 ft wide (600 x 175 m) and covers an area of 24 acres (0.1 km$^2$). The average water depth was 15 ft (5 m), and the final island elevation is +11 ft (3.4 m). The surcharge was built up to a maximum elevation of 37 ft (11.3 m), and a total volume of 1,130,000 yd$^3$ (860,000 m$^3$) of sand fill and surcharge was placed.

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of the new programs is an inability of old ones to accommodate the desired level of physical complexity.


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**Figure 7. Illustration problem 1.**

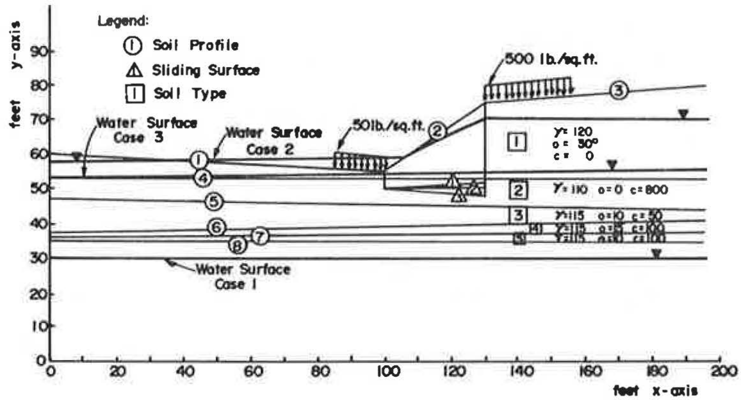



**Figure 8. Illustration problem 2.**

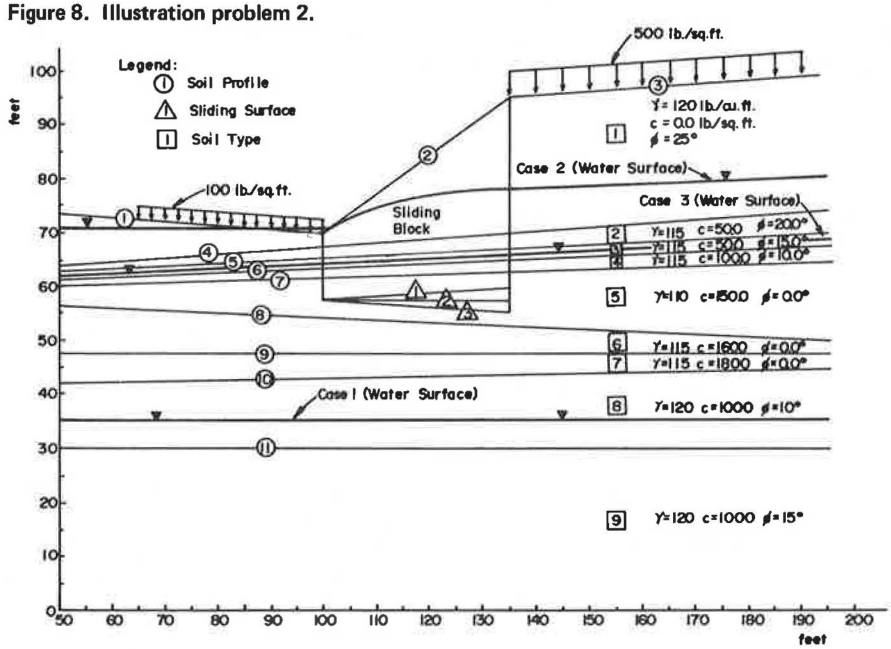



**Table 1. Summary of results for illustration problems.**

| Case Analyzed | Number and Slope of Sliding Surface | Factor of Safety | |
|---|---|---|---|
| | | Problem 1 | Problem 2 |
| 1 | 1, $\theta^+$ | 1.94 | 2.07 |
| | 2, $\theta = 0$ | 1.87 | 2.24 |
| | 3, $\theta^-$ | 1.83 | 2.42 |
| 2 | 1, $\theta^+$ | 1.66 | 1.83 |
| | 2, $\theta = 0$ | 1.65 | 1.98 |
| | 3, $\theta^-$ | 1.64 | 2.12 |
| 3 | 1, $\theta^+$ | 1.84 | 1.84 |
| | 2, $\theta = 0$ | 1.52 | 1.81 |
| | 3, $\theta^-$ | 1.66 | 1.75 |

2. Top ground surface is made up of three slopes and well-defined toe and crest points;
3. Soil properties are defined by $\gamma$, c, and $\phi$ (c or $\phi$ can equal zero);
4. Multiple (up to 10) uniform strip loads are on ground surface of the upper or lower slopes or both;
5. Water surface is anywhere in the problem space (the water surface is defined by continuous straight lines or a nonlinear surface defined by seven or fewer known coordinates or both); and
6. Multiple trial sliding surfaces are at the bottom of the central block and can be at any inclination (as many as 10 can be analyzed in a single run).

Specific trial surfaces are input for analysis. No searching technique (for identification of a minimum FS) is recommended although some ideas on this are contained elsewhere (4).

The active and passive force subroutines are potentially valuable in the solution of lateral earth pressure problems.

## ILLUSTRATION PROBLEMS

The purpose of the illustration problems is threefold: to demonstrate the use of the computer program, to show the versatility and several options of the program, and to serve as a check for duplicated decks. Two separate hypothetical problems are chosen for this purpose.

### Problem 1

The first illustration problem involves a simple soil profile shown in Figure 7. Solutions are obtained for three central block sliding surfaces and for three locations of the water surface for each sliding surface (Table 1).

### Problem 2

The second problem is more complex and is shown in Figure 8. This problem is also solved for three slopes of sliding surfaces in combination with three locations of the water surface for each sliding surface (Table 1).

## CONCLUSIONS AND RECOMMENDATIONS

The primary objective of this research was the development of a computer-assisted system for rapid prediction of the factor of safety of slopes where the mode of failure is a sliding block. The resulting program is sufficiently versatile to accommodate a three-slope ground surface and a subsurface profile with spatial variations in material properties, a steady-state flow domain, and uniform strip ground surface loadings. Up to 10 trial sliding surfaces can be analyzed concurrently, with the base of the central wedge at any inclination in any selected soil layer. The program automatically sequences the trial sliding surfaces, computing a factor of safety for each. Because many sliding surfaces may have to be examined (i.e., there is no systematic search technique that ensures identification of a minimum), this is a most important feature.

It was desired to develop a system that would be used on smaller computers. Consequently, the program uses a small storage and short computation time.

Hopefully, the program will enable a designer to check against this mode of instability for any slope where there is reason to suspect that it may occur. Such suspicion would ordinarily result from study of boring logs and profiles. Sliding blocks can be based in any soil stratum of below-average strength. Where there is no evidence of weak layers, it is likely that some common form of the circular or rotational slump analysis will be employed. In questionable cases, both types of analysis may be undertaken and factors of safety compared.

Any computer program should be tested for reliability by generation of solutions to common problems through different programs or manual calculations. Unfortunately, this is usually possible for only simple examples because the motivation for development

**Figure 4. Analysis of forces on passive wedge (case 2).**

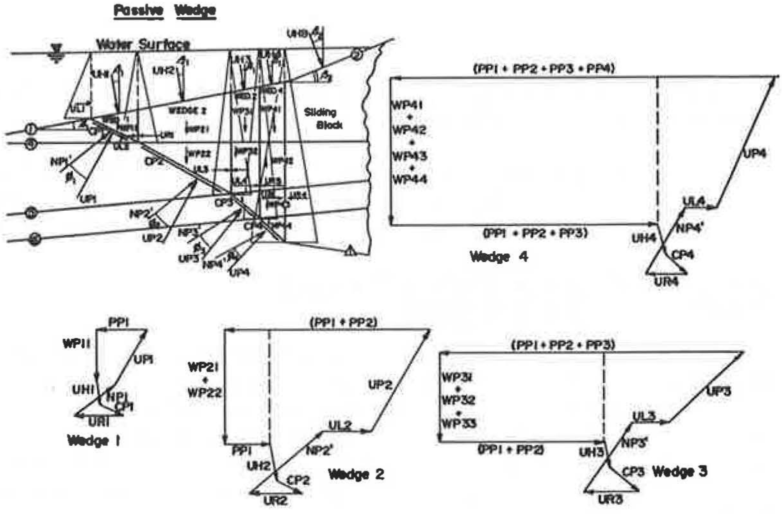



**Figure 5. Forces on sliding block (case 2).**

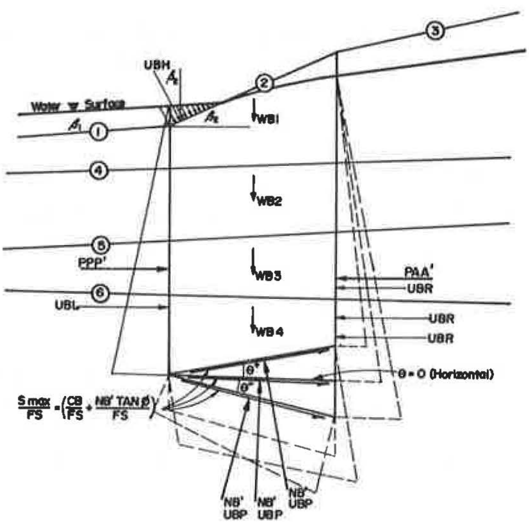
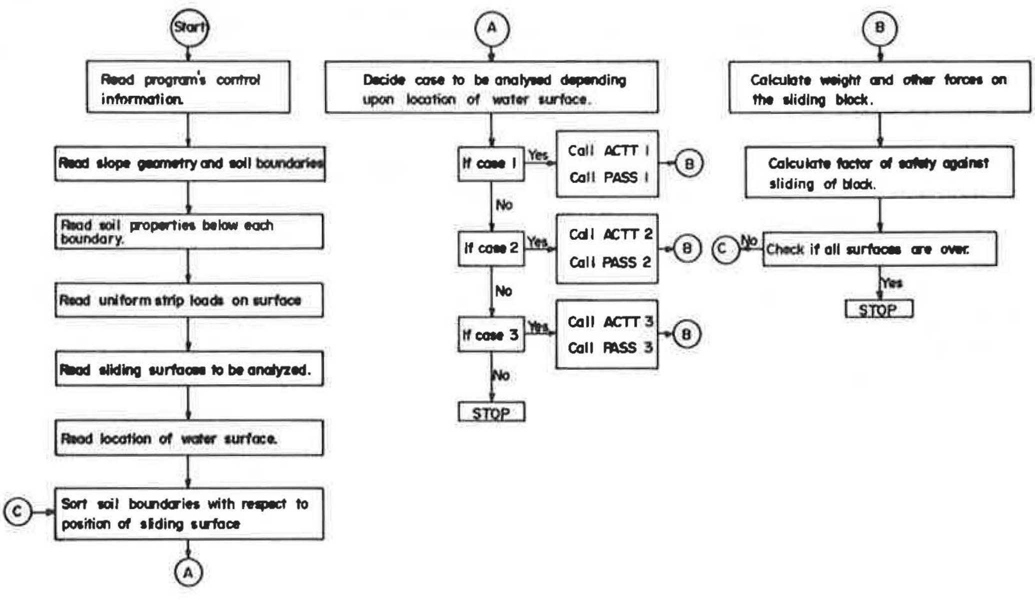



**Figure 6. Flow chart of block program.**



Elimination of NP′n from Eqs. 4 and 5 yields an expression for the incremental passive force for the n th wedge.

$$\begin{aligned} PPn = {} & WPn \tan(45 + \phi n/2) + 2\, CPn \cos(45 - \phi n/2) \\ & + U\beta_n [\sin\beta_1 + \cos\beta_1 \tan(45 + \phi n/2)] + (ULn - URn) \\ & + UPn [\cos(45 + \phi n/2) - \sin(45 + \phi n/2)\tan(45 + \phi n/2)] \end{aligned} \tag{6}$$

## Analysis of Forces on Central Block and Calculation of Factor of Safety

Figure 5 shows the appropriate free body (Fig. 2). The factor is commonly called the factor of safety (FS), although it is better interpreted as a strength reduction factor; i.e., if the real strength were divided by this factor, a reduced strength would obtain at which failure would impend. Note that the base sliding surface can be inclined up ($\theta^+$) or down ($\theta^-$) with respect to the horizontal, or may be horizontal ($\theta = 0$).

For $\theta^-$ and where forces are summed normal (N) and tangential ($\theta$) to the sliding surface, for $\Sigma F_N = 0$

$$\begin{aligned} NB' + UBP = {} & PAA \sin\theta - PPP \sin\theta + WB \cos\theta + UBH \cos\beta_2 \cos\theta \\ & - UBL \sin\theta + UBR \sin\theta - UBH \sin\beta_2 \sin\theta \end{aligned} \tag{7}$$

and $\Sigma F_\theta = 0$

$$\begin{aligned} \frac{CB}{FS} + \frac{NB' \tan\phi}{FS} = {} & PAA \cos\theta - PPP \cos\theta - WB \sin\theta \\ & - UBH \cos\beta_2 \sin\theta - UBH \sin\beta_2 \cos\theta \\ & - UBL \cos\theta + UBR \cos\theta \end{aligned} \tag{8}$$

Elimination of NB′ from Eqs. 7 and 8 yields an expression for the factor of safety for a particular trial sliding surface,

$$FS = \frac{\begin{aligned} & CB + (PAA \sin\theta - PPP \sin\theta + WB \cos\theta + UBH \cos\beta_2 \cos\theta \\ & -UBL \sin\theta + UBR \sin\theta - UBP - UBH \sin\beta_2 \sin\theta) \tan\phi \end{aligned}}{\begin{aligned} & (PAA - PPP) \cos\theta - WB \sin\theta - UBH \cos\beta_2 \sin\theta \\ & -UBH \sin\beta_2 \cos\theta - UBL \cos\theta + UBR \cos\theta \end{aligned}} \tag{9}$$

For $\theta^+$,

$$FS = \frac{\begin{aligned} & CB + (PPP \sin\theta - PAA \sin\theta + WB \cos\theta + UBH \cos\beta_2 \cos\theta \\ & + UBL \sin\theta - UBR \sin\theta - UBP + UBH \sin\beta_2 \sin\theta) \tan\phi \end{aligned}}{\begin{aligned} & (PAA - PPP) \cos\theta + WB \sin\theta + UBH \cos\beta_2 \sin\theta \\ & - UBH \sin\beta_2 \cos\theta - UBL \cos\theta + UBR \cos\theta \end{aligned}} \tag{10}$$

For $\theta = 0$ (horizontal slope),

$$FS = \frac{CB + (WB - UBP + UBH \cos\beta_2) \tan\phi}{(PAA - PPP) - UBH \sin\beta_2 - UBL + UBR} \tag{11}$$

## THE COMPUTER PROGRAM AND ITS CAPABILITIES

The flow chart for the program is shown in Figure 6. The program has been written in FORTRAN IV language, and at present it is workable on the CDC 6500 computer. It is made up of a main program and six supporting subroutines. The program makes use of common storage to optimize use of high-speed core and minimum computation time.

The program is capable of handling the following variables:

1. Multiple (up to 11) continuous soil layers are at any inclination, and layer boundaries are straight;

After homogenization, stresses and settlements are then calculated as if the rigid circular foundation with its load V were placed at the surface (fictitious ground surface) of the equivalent layer. The $\bar{\sigma}_z$ stress curve is then plotted along the centerline from the fictitious ground surface. Then the same $\bar{\sigma}_z$ stress curve for layers $h_1$ and $E_1$ is plotted from the real ground surface. Within layer $h_1$, these two $\bar{\sigma}_z$ stress curves are then connected by a smooth transition curve to obtain the resultant vertical $\bar{\sigma}_z$ stress distribution curve in the real two-layered system.

The settlements should be calculated for the thickness $z_a$ of the compressible active layer. Calculations may become difficult when there exist between the individual layers courses of soil having very large differences in moduli of elasticity. In such cases the equivalent height $h_e$ may become very large with the angle $\alpha = \text{arc cot}(z/R_o)$ approaching a value close to zero. Also, there may arise a difficulty in interpretation of results and hence the use of the charts when the lower layer has a very great value $(E \rightarrow \infty)$ of the modulus of elasticity, which in the process of homogenization gives an equivalent height $h_e$ whose value is zero.

## LIMITATIONS OF CHARTS

The assumptions made in developing the theoretical settlement influence-value chart for a central symmetrically loaded, rigid circular foundation also point out the chart's limitations. Thus, the main limitations pertain to an idealized soil, smooth base, smooth interface contact, weightless elastic medium, rigid circular foundation laid at the ground surface, displacement differences, elastic settlements are calculated for a homogeneous, semi-infinite elastic hemispace. Hence, the theoretical settlement influence values give approximative values only as compared with the real conditions in a soil. In reality, rock, gravel, sand, clay, or mixtures thereof are frequently encountered in single layers, as well as forming multilayered soil systems in various sequences of their stiffness. Such situations necessitate approximations and simplifications in pertinent calculations. The approximative nature of these calculations may also be seen in the approximative values of the elastic modulus E and Poisson's number m assumed for use in these theory-of-elasticity calculations.

### Depth of Foundation

Although these charts do not include the embedment depth effects on stress and settlement distribution in soil, the settlement influence-value chart may, nevertheless, also be used if the foundation is laid t-units below the ground surface, rendering approximative results. In such a case, the $\bar{\sigma}_z$ stress may be calculated by substituting $(\sigma_o - \gamma t)$ for $\sigma_o$ in Eq. 6. Approximative settlement s is calculated by substituting $(V - \pi R^2 \gamma t)$ for V in Eqs. 9, 10, 11, and 12.

### Smooth and Rough Bases

Carrier and Christian (17) have shown that there is essentially no difference between a rough rigid plate and a smooth rigid one when $n = 0$ or $n = 1$ for $\mu = 0.5$, where n is a power in the E-equation defining a hemispace where E varies with depth: $E = E_o\left(\frac{z}{2R_o}\right)^n$. To quote these authors: "It is probable, therefore, that roughness has no influence for intermediate values of n for a rigid or uniform circular load when $\mu$ = 0.5." The same opinion has been expressed by Gibson (18) and Schiffman (19).

### Smooth Soil Interfaces and Rough Soil Interfaces

In the absence of reliable stress and settlement measurement data relative to interface roughness or smoothness for two-layered and multilayered soil systems under externally loaded, rigid circular foundations, it is, unfortunately, impossible to say anything about the probable accuracy of the settlement determination method by this chart when it is applied to rigid-soft or soft-rigid soil systems.

## Elastic Layer of Finite Thickness Overlying a Rigid Deposit

Generally, stress distribution in a two-layered system differs from that in a homogeneous, semi-infinite medium only in cases where there are sharp differences in elasticity characteristics among the various single deformable layers of soil. Therefore, on determination of the stressed condition in the soil, consideration of nonuniformity should be given only when deformation characteristic values of the various nonuniform soils, composing nonuniform natural earth masses, differ sharply one from another. In such a case, the theory used in this study for the development of tables and charts for approximative stress and settlement evaluation is applicable as for a nearly homogeneous monolayer.

If in a two-layered soil system there is immediately below the base of a rigid circular footing an elastic compressible layer of finite thickness overlying a practically incompressible rigid deposit such as rock, then stress distribution in the upper compressible layer depends primarily on the ratio of the thickness of the compressible layer ($z = h$) to the diameter ($2R_o$) of the rigid footing. Figure 1 shows that, beginning with a relative depth of $z/R_o = 4$ (or from a depth of two diameters and down), the vertical stresses $\bar{\sigma}_z$ for all Poisson's numbers are practically the same and are small. Hence, it is believed that, within the zone of thickness of two (or three) diameters, the chart is applicable as for a homogeneous soil of infinite depth (monolayer).

Figure 1 also shows that, within a depth zone of one diameter ($z/R_o = 2$), there exist large stresses in soil below the footing. Thus, if the massive rock is located at a depth shallower than two diameters, the soil-rock system should be treated as a two-layered system by means of the method, say, of the equivalent layer.

## Rigid Layer of Finite Thickness Overlying an Elastic Deposit

If in a two-layered soil system a rigid layer overlies a compressible one, the vertical stresses distribute through the upper, rigid layer on the surface of the compressible layer over a larger area than the size of the circle on the ground surface. Hence, on the interface plane between the rigid and soft layers, the $\bar{\sigma}_z$ stress is smaller than that in a uniform monolayer mass. As in almost every theoretical settlement calculation, so here the calculated deflections tend to be larger than the actual ones.

## CONCLUSIONS

Based on the foregoing discussion, some general conclusions relative to the practical application of the settlement influence-value chart may be made as follows:

1. Regardless of some of the limitations imposed in developing this settlement influence-value chart, and in its application to real soils, the chart is nonetheless a very useful one in practical soil mechanics and foundation engineering for calculating approximative settlements in homogeneous monolayers as well as in multilayered soil systems. The chart is easy to use.
2. Because it influences the magnitude of the vertical stress distribution and that of the settlement, the real modulus of elasticity E must be known and used in these calculations and must be determined from appropriate tests.
3. Figure 1 shows that Poisson's number m has a remarkable effect on the vertical, spatial stresses in and settlement of an elastic layer.
4. For a compressible, homogeneous monolayer soil, settlement calculations can be practically based on the thickness of the active zone.
5. If in a multilayered soil system the soil elasticity characteristics of the individual layers do not differ greatly, the chart may be used for approximative settlement evaluation in the same way as for a nearly uniform, homogeneous monolayer. Otherwise, the multilayered soil system should be homogenized into an equivalent, homogeneous monolayer.
6. In a two-layered soil system that is elastic-rigid, within the depth zone, the thickness of which is about two diameters below the base of the footing, the chart is applicable as for a homogeneous soil of infinite depth (monolayer). In a two-layered

soil system of rigid-elastic consistency, the $\bar{\sigma}_z$ stresses on the rigid-elastic interface are smaller than in a uniform monolayer mass, but the settlement here tends to become smaller.

7. The uniform, elastic settlement influence-value chart derived for a rigid centrically loaded circular foundation may also be used effectively for approximating the settlements imposed by a rigid square foundation whose area is equivalent to that of the circle:

$$4A^2 = \pi R_o^2;\ R_o = 2A\sqrt{1/\pi} = 1.128A$$

where

2A = length of side of the square, and
$R_o$ = radius of circle.

This approximation is thought valid for side ratios of up to 1 to 5 only, according to Schleicher (14).

8. Because the bearing capacity of a pier (rigid die) and/or of a massive pile depends on their tolerable settlement, the settlement influence-value chart may also be used with some limitations for evaluation of the bearing capacity of circular (and quadratic in cross section) rigid piers and piles where no mantle resistance (skin friction) applies (in water, for example).

In general, the chart would tend to give conservative answers because the theory neglects depth effect on stress distribution and also neglects skin friction on the sides of the piers, which would both tend to reduce the observed amount of settlement as compared with the calculated ones.

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