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Foundation Design

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Foundations to Resist Tilting Moments Imposed on Upright Cantilevers Supporting Highway Signs

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To have an upright cantilever structurally strong enough and at the same time as economical as possible, requires careful analysis of the supporting foundation. However, neither time nor money permit foundations to be put in that way. It is usually much cheaper to make a foundation enough larger to include expected factors of ignorance and safety. Nevertheless, some engineering must be used and the solution contained in this paper is a good beginning point.

Certain observations about horizontal soil resistances seemed to be established enough to use as a basis to start. The first is that for a vertical cantilever, a slim deep foundation is the most economical for a given load with limits of the strength of the foundation as an efficient structure. The soil has horizontal resistance to movement depending on its cohesive or granular makeup, or both. The resistance varies with the depth or the amount of overlying earth above. There is some relation between movement and resistance. In this analysis it is assumed that (a) within the limits used the soil is an elastic body, (b) the strength of a foundation varies with its projected area, (c) no part of the soil shall be stressed above its ability to withstand the load, and (d) in empirical solution errors shall be kept on the side of over-design.

In the solution itself, several empirical methods were used. It was impractical to design a foundation in a given soil for a given load at a given height, but if one took a foundation of given dimensions in a given soil and chose a neutral axis, then by integrating the soil resistance one finds both shear and moments imposed. A family of curves can then be drawn, from which generalizations and empirical solutions follow.

This type of analysis seems to be justified from model studies and tests run in 1945, then on some 12 years of usage covering many thousands of poles from 20 to 200 ft high supporting any type of load, and latest on the tests run by the Research Department of the Ohio State Highway Department.

● THE RESISTANCE of the soil to the tilting forces on a deep, slim foundation cannot be exactly evaluated. There is, however, much need to determine approximate solutions and keep errors in assumption or solution on the side of safety and also to try to suggest factors of safety as great as factors of ignorance.

Using the best references available, it seems that certain approximations can be made. As in many other empirical solutions, it is sometimes necessary to find the limiting relationships and stay within those limits in order to keep the solution from being too cumbersome. Acknowledgment is made to Terzaghi (1) and Hogentogler (2).

It will develop that the most economical foundations for this purpose are slender and of some depth, and that their strength against tilting is such that such forces as bearing and uplift should be set aside as negligible.

DEFINITION OF TERMS

Units are feet and pounds on unit (1-ft) width of foundation.

R_p = passive horizontal soil resistance
R_a = active horizontal soil pressure
R_t = net resistance of soil to horizontal movement at depth "Z" as a maximum allowable pressure
R_d = net resistance of soil to horizontal movement due to its deflection
R = actual net resistance of soil to horizontal movement at depth "Z"
M = moment of horizontal load P at L distance from neutral axis of the foundation; this moment is solved for a foundation of unit width
M_z = moment imposed on foundation of depth D, by the soil, integrated around neutral axis
M_f = bending moment in foundation
V_f = horizontal shear in foundation
D = depth of foundation
Z = any depth
Z_1 = depth of neutral axis of foundation
dZ = increment depth
C = coefficient of soil cohesion
ϕ = angle of internal friction
G = weight of soil
L = vertical distance from load "P" to neutral axis of foundation
h = distance of load above groundline (not used except to find degree of error, use "L" instead in calculations)
A = Anderson's constant of cohesive resistance for soil in question
B = Anderson's constant of internal friction resistance of soil in question

DEVELOPMENT OF THEORY

Under the elastic theory it is accepted that resistance to motion is directly proportional to deflection (Fig. 1). Again in the study of soils it is generally assumed that resistance to unit deflection varies with depth. This solution is based on the assumption that these two relationships are straight line functions:

The net resistance of a soil to horizontal movement of something in it is the difference of the pressures on its two sides or passive resistance less active pressure. Thus $R_t = R_p - R_a$. Terzaghi (1) gives

$$R_p = 2C \tan (45^\circ + \phi/2) + GZ \tan^2 (45^\circ + \phi/2)$$

$$R_a = -2C \cot (45^\circ + \phi/2) + GZ \cot^2 (45^\circ + \phi/2)$$

or

$$R_t = 2C \left[\tan(45^o + \phi/2) + \cot(45^o + \phi/2)\right] + GZ\left[\tan^2(45^o + \phi/2) - \cot^2(45^o + \phi/2)\right]$$

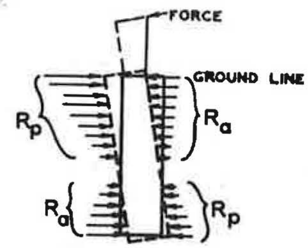

Figure 1.

For simplicity let $a = 2C \tan(45^o + \phi/2) + \cot(45^o + \phi/2)$ and $b = G \tan^2(45^o + \phi/2) - \cot^2(45^o + \phi/2)$

Thus $R_t = a + bZ$

From theory thus far:

R is proportional to R_t and to R_d.

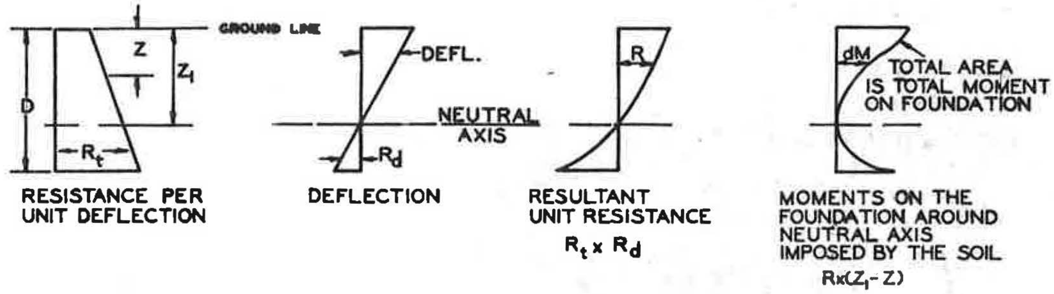

Figure 2. Development of forces and moments.

However, a deflection cannot give a resultant resistance greater than R_t, which is the capability of the soil to resist.

Thus $R = R_t \cdot R_d \cdot$ constant

Now R_d is proportional to the distance from the neutral axis; that is, $R_d = K(Z_1 - Z)$ (Fig. 2).

or, substituting, $R = (a + bZ)\,K\,(Z_1 - Z)$ (including both constants in "K")

But R must not exceed $a + bZ$.

That is, $R = a + bZ = (a + bZ)\,K\,(Z_1 - Z)$.

From inspection, the groundline will be the weak point for a cylindrical foundation.

Thus, at 0 depth, $Z = 0$, $R = a = K\,a\,Z_1$

or $K = 1/Z_1$ and,

$$R = (a + bZ)(1 - Z/Z_1) = a - \left(\frac{a}{Z_1} - b\right) Z - \frac{b}{Z_1} Z^2 \text{ (Fig. 2)}$$

Integrating the moments from the bottom of the foundation by means of integrating shear gives answers too critical of height of load for any use. But integrating the resisting turning moments imposed on the foundation by the soil and around the neutral axis, gives the first useful approach.

$dM = R \cdot$ lever arm, or $R(Z_1 - Z)$

or $dM = (a - aZ/Z_1 + bZ - bZ^2/Z_1) \quad (Z_1 - Z)$

$$= a\, Z_1 - (2a - bZ_1)\, Z + (a/Z_1 - 2b)\, Z^2 + \frac{bZ^3}{Z_1}$$

$$M = \int dM_z d_z = a\, Z_1 Z - \left(\frac{2a - bZ_1}{2}\right) Z^2 + \left(\frac{a}{3Z_1} - \frac{2b}{3}\right) Z^3 + \frac{b}{4Z_1} Z^4 + C_1$$

but if $Z = D$, $M = 0$; then at $Z = 0$, $M = C_1$

$$\text{Thus } -C_1 = aZ_1D - \left(\frac{2a-bZ_1}{2}\right) D^2 + \left(\frac{a}{3Z_1} - \frac{2b}{3}\right) D^3 + \frac{b}{4Z_1} D^4$$

Since all moments total zero, the moment imposed on the foundation from the force above the ground = C_1 (all about neutral axis) then external moment

$$M = -aZ_1D + \left(\frac{2a - bZ_1}{2}\right) D^2 - \left(\frac{a}{3Z_1} - \frac{2b}{3}\right) D^3 - \frac{b}{4Z_1} D^4$$

(about the neutral axis)

$$\text{and } V_f = az - \frac{1}{2}\left(\frac{a}{Z_1} - b\right) Z^2 - \frac{b}{3Z_1} Z^3 - a\,D + \frac{1}{2}\left(\frac{a}{Z_1} - b\right) D^2 + \frac{b}{3Z_1} D^3$$

If $Z = 0$ $V_f = - a\, D + \frac{1}{2}\left(\frac{a}{Z_1} - b\right) D^2 + \frac{b}{3Z_1} D^3$ which represents the shear imposed from the structure above the groundline. Integrating V_f, the moment in the foundation is

$$M_f = \int V_f\, dZ = \frac{a}{2} Z^2 - \frac{1}{6}\left(\frac{a}{Z_1} - b\right) Z^3 - \frac{b}{12Z_1} Z^4$$

$$- aDZ + \frac{1}{2}\left(\frac{a}{Z_1} - b\right) D^2Z + \frac{b}{3Z_1} D^3Z + C_2$$

$Z = D$, $M = 0$

$$- C_2 = \frac{a}{2} D^2 - \frac{1}{6}\left(\frac{a}{Z_1} - b\right) D^3 - \frac{b}{12Z_1} D^4 - aD^2 + \frac{1}{2}\left(\frac{a}{Z_1} - b\right) D^3 + \frac{b}{3Z_1} D^4$$

$$C_2 = \frac{a}{2} D^2 - \frac{1}{3} \left(\frac{a}{Z_1} - b\right) D^3 - \frac{b}{4} D^4$$

$$M_f = \frac{a}{2} Z^2 - \frac{1}{6}\left(\frac{a}{Z_1} - b\right) Z^3 - \frac{b}{12Z_1} Z^4 - aDZ + \frac{1}{2}\left(\frac{a}{Z_1} - b\right) D^2Z + \frac{b}{3Z_1} D^3Z$$

$$+ \frac{a}{2} D^2 - \frac{1}{3}\left(\frac{a}{Z_1} - b\right) D^3 - \frac{b}{4} D^4$$

To study stresses in the foundation itself, the net soil forces are integrated on the foundation, from the bottom to the top, to give the shear V_f.

$$V_f = R\, dz = a - \left(\frac{a}{Z_1} - b\right) Z - \frac{b}{Z_1} Z^2\, dZ$$

$$= aZ - \frac{1}{2}\left(\frac{a}{Z_1} - b\right) Z^2 - \frac{b}{3Z_1} Z^3 + C_2$$

$Z = D$, $V_f = 0$, and $V_f = C_2$, or when $Z = 0$,

$$V_f = -aD + 1/2 \left(\frac{a}{Z_1} - b\right) D^2 + \frac{b}{3Z_1} D^3$$

Taking various values of Z_1/D, a, and b, the moment and shear at D = o can be found, and from these two values height of load, in this case from the groundline up, can be determined. However, these values are very critical and complex and set aside as unwieldy, in favor of the following, which is a simple, adequate development for resisting moment of the foundation. However, internal stresses in the foundation are developed further in a later section.

It was found that M_z (about the neutral axis) varies only three percent to five percent over the range of heights of loads from one times foundation depth to 20 times foundation depth. Consequently, the analysis will use M_z (around the neutral axis). It was also found that if a value of $Z_1 = 2/3$ D was assumed, the errors would be on the safe side. Incidentally, this depth of neutral axis $Z_1 = 2/3$ D was observed in model studies of both granular and plastic soils.

Substituting then, the 2/3 value for Z_1/D, we get:

$M = 1/6\ aD^2 + 1/24\ bD^3$ (basic formula). Remember again that M is ft-lb allowable, around the neutral axis per unit width of foundation. As developed later, the practical formula will be $M = AD^2 + BD^3$ with values given for A and B for various soils.

CERTAIN VALUES

Hogentogler (2) gives basic values of certain soils, which can be used with the basic formulas of Terzaghi (1) to arrive at values of a and b in the basic formula. The arbitrary weight of soil is 100 pcf, for certain soils. The terms are fairly broad but certain additional factors of safety which are later mentioned are beneficial.

TABLE 1

Types of Soil	Coef. of Cohesion	Angle of Internal Friction (deg)	a	b
Silt, wet	0	10	0	73
Sand, wet or dry	0	34	0	326
Clay, very soft	200	2	800	27
Clay, medium	1000	6	4000	42
Clay, very stiff	2000	12	8120	87
Cemented sand and gravel	500 to 1000 say 750	34	3600	326
Sandy clay	1000	34	4800	326
Silty clay	200	14	800	102

FACTORS OF SAFETY

Certain additional strengths of the foundation may well be mentioned. The preceding theory takes into account only the cylindrical projection of the foundation. It is most certain that there is some conical effect, particularly in sandy soils. Again there is no consideration given for skin friction, which has considerable effect, particularly if the foundation has rough vertical and bottom surfaces. The soil can withstand deflections at certain depths beyond the assumed solution and continue to exert full pressure, and hence allow other depths to take greater loads, after the plastic theory. Soil also has certain ability to withstand short-time loads greater than the limits assumed. Housel (3) suggests an overload factor of 2.5 where some small movement is not dangerous.

It should be remembered that foundations should be extended below the frost line to resist "heaving." Also, freshly disturbed soil such as trenches, etc., may subtract from the above-mentioned factors of safety.

SPECIAL SHAPES

Since in all soils seen thus far, the weakest point was at the groundline, the design should be balanced by increasing the diameter of the top 1/3, letting the bottom diameter of the foundation remain at unity.

The top diameter is increased to where R_t at the bottom is as great as allowable. The top diameter is increased to N, keeping the bottom diameter as unity. At bottom R was $\frac{a+bD}{2}$

Bottom R is increased to a + bD or increased by $\frac{a + bD}{2}$.

Assuming again a neutral axis 2/3 down, the top or groundline resistance must be increased by the same amount, times the respective lever arms of the top and bottom. Top increase is (N-1)a at the groundline and since there is twice the lever arm as at the bottom, $2\ (N-1)a = \frac{a + bD}{2}$, from which $N = \frac{bD}{4a} + \frac{5}{4}$, which is the multiplier of the top diameter for most efficient value for a bottom diameter of unity. A much more involved solution gives the ideal increase in top diameter, but values remain nearly the same. This is an efficient shape, volume-wise, and it normally takes the top third below the frost line.

Increasing the top 1/3 by the ratio N, or to N times unity,

$$M = \int_0^D M\, dZ + (N-1) \int_0^{D/3} M_Z dZ$$

$$= (.1296N + .037)\ a\ D^2 + (.017N + .0247)\ bD^3$$

which is abbreviated by substitution to $AD^2 + BD^3$ (Table 2).

NON-HOMOGENEOUS SOILS

Where soils are in layers of different types, and this is more usual than not, there is a different type of problem. Usually the top layer is of one type and the subsoil of an entirely different composition. These cases are solved by assuming the top third to be of one type soil and the bottom 2/3 of another. Actually the portion of the foundation around the neutral axis has little value and therefore the last assumption does not introduce practical errors. More than two kinds of soil may affect the solution but main interest is in the soil at the top 1/3 and bottom 1/6 of the foundation.

The solution for the most efficient foundation will include the top width greater than the bottom width by the ratio "N"

SOLUTION FOR TWO TYPES OF SOIL

The most practical solution for this problem is to assume a straight

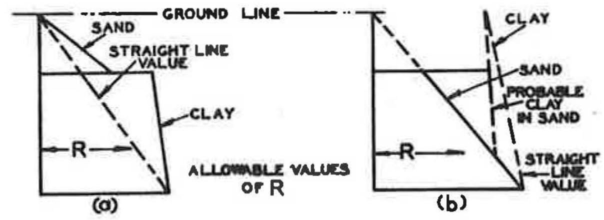

Figure 3.

line characteristic for the two soils. Soil net resistance values may be as shown in Figure 3.

Where there is a sandy type soil on top and a clay soil below, a straight line curve is less than the actual values. Where the clay type overlies the sand, the weakest part might be at the top of the sand, but this usually has sufficient penetration of clay to give it cohesive characteristics. Again, this point is near enough the neutral axis that there is not enough deflection to run up the force to a dangerous point.

Therefore, there is a new substitute or empirical soil whose "a^1" value is "a_1", and whose "b^1" value is

$$b^1 = \frac{a_2 + b_2 D - a_1}{D}$$ (where a_1 and b_1 are values for soil at top and a_2 and b_2 are values for soil at bottom)

This might possibly give negative values for "b," where a shallow foundation lies in a good hard clay top and a sandy bottom, but the equations and formulas still hold true within practical limits. Rather than use negative values the "a" value is reduced to give zero value of "b".

The "N" values are solved the same way as before. However, "N" is limited by practical dimensions to about five. Values for specific soils are given in Table 2 in which "A" and "B" values are developed.

SPECIAL SHAPES

If the direction of maximum moment is known, and it generally is, it is most efficient to increase the width of the top third at right angles, to the direction of force. Certain variables of this general shape are shown in Figure 4.

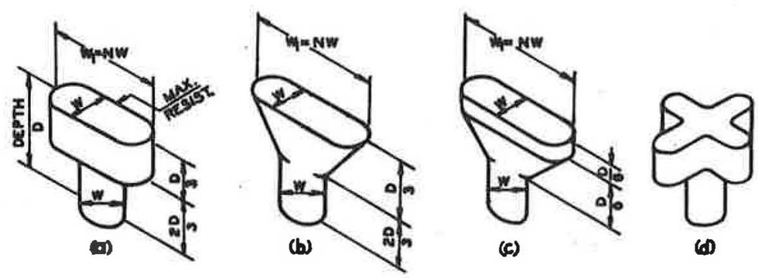

Figure 4.

Shape (a) (Fig. 4) is that used in the general formula, and shapes (b) and (c), which might be easier to dig, have about 90 percent of the strength of the formula. It might be noted that shape (c) is the most efficient foundation on a tilting resistance per unit volume for a given over-all depth that the author has investigated. Shape (d) is used where the direction of maximum force is not known. A pinching at the middle or

TABLE 2

Values for various soil combinations - contemplated depths between 5' and 1

Where $M = AD^2 + BD^2$, wide upper portions by ratio of "N"

M = safe tilting moment around neutral axis, per foot bottom width of foundation.

Soil Symbols

S Cl - Sandy clay
S - Loose sand or loose sand and gravel
Cem S - Cemented sand and gravel
Sf Cl - Very soft clay
M Cl - Medium clay
H Cl - Hard clay
St - Silt
St Cl - Silty clay - loamy clay

Upper Soil	Lower Soil	a_1	b_1	a_2	b_2	a or a^1	b or b^1	N	.1296N +.037	.017N +.0247	A	B
S Cl	S Cl					4800	325	1.4	.218	.049	1040	15.9
S Cl	S	4800	325	0	320	3200	0	1.25	.199	.046	1470	0
S Cl	Cem S	4800	325	3600	325	4800	205	1.25	.199	.046	950	9.4
S Cl	H Cl	4800	325	8100	85	4800	305	1.4	.218	.049	1040	14.9
S	S					0	320	Say 5	.683	.110	0	35.2
S	S Cl	0	320	4800	325	0	645	Say 5	.683	.110	0	71.0
S	Cem S	0	320	3600	325	0	500	Say 5	.683	.110	0	55.0
Sf Cl	S Cl	800	27	4800	325	800	590	3	.425	.076	340	44.8
Sf Cl	M Cl	800	27	4000	40	800	250	2	.296	.059	230	14.7
Sf Cl	H Cl	800	27	8100	85	800	570	3	.425	.076	340	43.3
M Cl	S	4000	40	0	325	3250	0	1.25	.199	.046	645	0
M Cl	Cem S	4000	40	3600	325	4000	285	1.4	.218	.049	870	14.0
M Cl	M Cl					4000	40	1.25	.199	.046	800	1.8
M Cl	H Cl	4000	40	8100	85	4000	350	1.4	.218	.049	870	17.1
H Cl	H Cl					8100	85	1.25	.199	.046	1610	3.9
St Cl	St Cl					800	100	1.5	.231	.051	180	5.1
St Cl	M Cl	800	100	4000	40	800	250	2	.296	.059	235	14.7
St Cl	H Cl	800	100	8100	85	800	570	3	.425	.076	340	43.3
St Cl	S Cl	800	100	4800	325	800	590	3	.425	.076	340	44.8
St Cl	Cem S	800	100	3600	325	800	510	3	.425	.076	340	38.8

increasing bottom and top diameters is theoretically more efficient, but not practical due to small diameters used at bottom.

INTERNAL STRESSES IN THE FOUNDATION

Integrating R_t from the bottom of the foundation upward, gives the internal moment in the foundation. Effective "a" and "b" values are from Table 2. Thus a general equation for the stresses in the foundation can be derived:

$$R = a + bZ - \frac{aZ}{Z_1} - \frac{bZ^2}{Z_1}$$

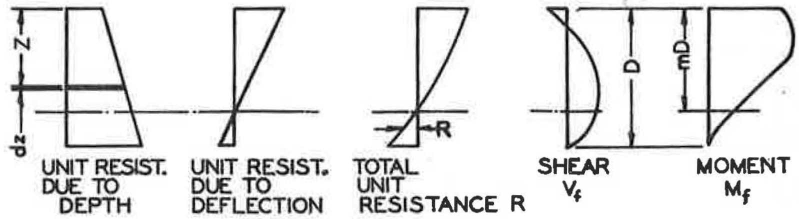

Figure 5.

Integrating, where V_p = shear at base of pole

$$V_f = aZ - \frac{a}{2Z_1} Z^2 + \frac{b}{2} Z^2 - \frac{b}{3Z_1} Z^3 + C \text{ (since at } Z = 0,\ V_f = V_p = C)$$

$$Z = D,\ V_f = 0, \text{ thus } C = -aD + \frac{1}{2}\left(\frac{a}{Z_1} - b\right) D^2 + \frac{b}{3Z_1} D^3$$

$$V_f = aZ - \frac{a}{2Z_1} Z^2 + \frac{b}{2} Z^2 - \frac{b}{3Z_1} Z^3 - aD + \frac{1}{2}\left(\frac{a}{Z_1} - b\right) D^2 + \frac{b}{3Z_1} D^3$$

Now it is found that the distribution of the shear in the foundation varies with the ratio of a to b (that is, cohesiveness versus granular nature of the soil) and that the a-to-b ratio effect is modified by total depth. To simplify the solution, certain ratios are taken as follows:

$$X = \frac{Z}{D} \text{ or } Z = XD;\ m = \frac{Z_1}{D} \text{ or } Z_1 = mD; \text{ and } b = \frac{na}{D} \text{ or } n = \frac{bD}{a}$$

Substituting, since $C = V_p$

$$V_p = aD\left(-1 + \frac{1}{2}m - \frac{n}{2} + \frac{n}{3m}\right)$$

$$V_f = aD\left(X - \frac{x^2}{2m} + \frac{nx^2}{2} - \frac{nx^3}{3m} - 1 + \frac{1}{2m} - \frac{n}{2} + \frac{n}{3m}\right)$$

Similarly, where M_p = moment at base of pole

$$M_f = \int V_f dZ = D\int V_f dx$$

$$-M_p = aD^2\left(-\frac{1}{2} + \frac{1}{3m} - \frac{n}{3} + \frac{n}{4m}\right)$$

$$M_f = aD^2\left(\frac{x^2}{2} - \frac{x^3}{6m} + \frac{nx^3}{6} - \frac{nx^4}{12m} + \frac{x}{2m} - \frac{nx}{2} + \frac{nx}{3m} + \frac{1}{2} - \frac{1}{3m} + \frac{n}{3} - \frac{n}{4m}\right)$$

Therefore,

$$\frac{h}{D} = \frac{4 - 6m + 3n - 4mn}{12m - 6 + 6mn - 4n}$$

The most encountered values are $\frac{h}{D} = 4$, and since they represent slightly higher values of foundation stresses, they are used for the following proportionate values of shear, moment and depth in the foundation.

It is suggested that the foundation be designed in size for the poorest soil in the class studied, but that the stresses in the foundation be studied as well for the strongest of the soils considered, since shears

TABLE 3

Values for m h/D	n = 0	n = .1	n = 1	n = 10	n = ∞
2	.533	.540	.587	.663	.688
4	.519	.526	.573	.653	.678
6	.514	.521	.568	.649	.675
10	.508	.516	.563	.645	.672

and moments may be greater in the foundation for the later condition, even for the same groundline moments.

TABLE 4

$\left(\frac{h}{D} = 4\right)$

	n = 0		n = .1		n = 1.0		n = 10		n = ∞	
x	V_f/aD	M_f/aD^2	V_f/aD	M_f/aD^2	V_f/aD	M_f/aD^2	V_f/aD	M_f/aD^2	V_f/bD^2	M_f/bD^2
.0	-.035	-.143	-.036	-.148	-.0431	-.186	-.130	-.505	-.0084	-.0354
.1	.055	-.142	.055	-.147	.0326	-.186	.007	-.512	-.0034	-.0360
.2	.126	-.133	.126	-.137	.137	-.173	.198	-.502	.0077	-.0360
.4	.211	-.098	.211	-.099	.256	-.135	.620	-.420	.0401	-.0312
.6	.218	-.054	.219	-.057	.296	-.078	.891	-.263	.0654	-.0203
.8	.147	-.016	.152	-.017	.219	-.024	.766	-.090	.0599	-.0070
.9	.085	-.004	.089	-.003	.129	-.007	.478	-.025	.0382	-.0021
1.0	.0	0	0	0	0	0	0	0	0	0

TABLE 5

$\left(\frac{h}{D} = 2\right)$

	n = 0		n = .1		n = 1.0		n = 10		n = ∞	
x	V_f/aD	M_f/aD^2	V_f/aD	M_f/aD^2	V_f/aD	M_f/aD^2	V_f/aD	M_f/aD^2	V_f/bD^2	M_f/bD^3
.0	-.062	-.125	-.066	-.127	-.080	-.160	-.218	-.440	-.015	-.0301
.1	+.037	-.126	+.025	-.129	+.016	-.164	-.018	-.456	-.011	-.0315
.2	+.098	-.119	+.099	-.123	+.101	-.158	+.111	-.444	+.001	-.0321
.4	+.188	-.090	+.190	-.093	+.227	-.124	+.539	-.389	+.036	-.0287
.5	+.204	-.073	+.210	-.073	+.261	-.099	+.715	-.326	+.049	-.0247
.6	+.200	-.049	+.206	-.056	+.270	-.072	+.824	-.248	+.060	-.0191
.8	+.138	-.015	+.141	-.016	+.204	-.023	+.752	-.089	+.056	-.0067
.9	+.078	-.003	+.081	-.004	+.121	-.006	+.456	-.025	+.036	-.0021
1.0	+ 0	0	0	0	0	0	0	0	0	0

Relative moment curves for various n values are shown in Figures 6 and 7.

Where a foundation is the shape of an inverted frustrum of a cone, it is seen that the relative value of "b" diminishes. This geometry is as follows:

$$R = N_a + \left[b - \frac{(N-1)}{D} a\right] Z$$

Thus where the top of a foundation has been increased to N times the bottom diameter, one may assume

$$R = Na + b^1 Z,$$

$$\text{Where } b^1 = b - \frac{(N-1)a}{D}$$

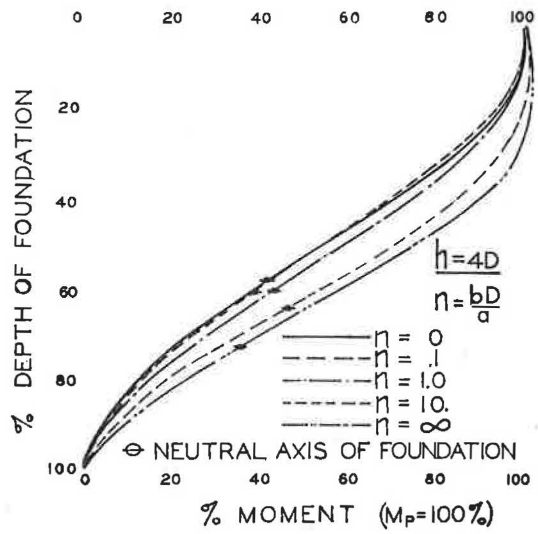

Figure 6.

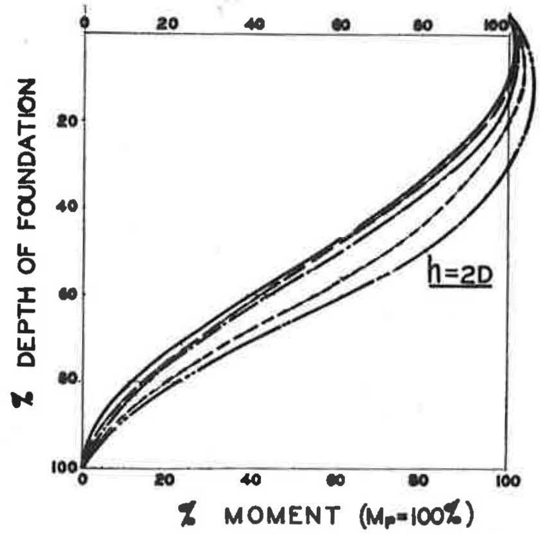

Figure 7.

Example - Say a = 900, b = 400, d = 12', N = 3, a^1 = 2700, b^1 =

$$400 - \frac{2.900}{12} = 250$$

and R = 2700 + 250Z, and the foundation can be computed by using a^1 and b^1 in place of a and b in the formulas. It should be noted the width of the foundation is the bottom width.

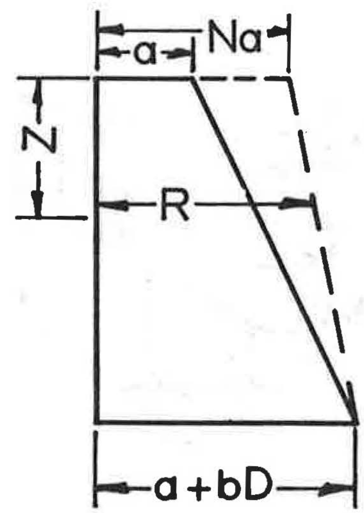

Figure 8.

SUMMARY

1. In general the most efficient foundation to resist tilting moment is slim and deep; its slimness only limited by practical limitations such as internal strength and means of digging.

2. If moments are taken around an assumed neutral axis, general equations can be used whose errors are on the safe side and whose accuracies are within about 5 percent, assuming soil values to be absolute.

3. Special shapes are so much more efficient that general formulas incorporate the basic general shape (Fig. 4a).

4. The allowable tilting resistance of a foundation in ft-lb per unit bottom width is $M = AD^2 + BD^3$ (see Table 2 for values of "A" and "B").

5. Foundation strengths (for reinforcing) may be checked according to formula in that section, but if foundation bolts go near the bottom of a foundation, further reinforcing is probably unnecessary.

EXAMPLES

1. Find practical dimensions for a foundation for a dead end pole with the following:

Five thousand-lb design load at 30-ft height. Constant load about $\frac{1}{2}$ this value.

Soil - 8 in. of top soil, 4 ft sandy clay, then hard clay bottom soil.

From Table 2, $M = 1040D^2 + 14.9D^3$ and $N = 1.4$; at the surface moment = 150,000 ft-lb (not used).

Ignore the top .67 ft as being too liable to be disturbed.

Assume a depth of not over 9 ft which gives a neutral axis 6 ft deep.

Then M = 5000 x 36.67 = 183,333 ft-lb.

Contemplate a bottom width of 24 in.

Then M per unit width = 91,667 or $91{,}667 = 1040D^2 + 14.9D^3$.

D	D^2	D^3	$1040D^2$	$14.9D^3$	Allow. M
8'	64	512	66,500	7,600	74,100
9'	81	729	84,300	10,800	95,100

(which is sufficient)

The total depth is 9 ft 8 in. after adding the top soil.

Thus a foundation (Fig. 9).

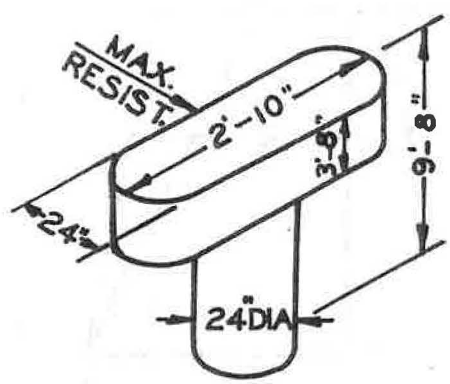

Figure 9.

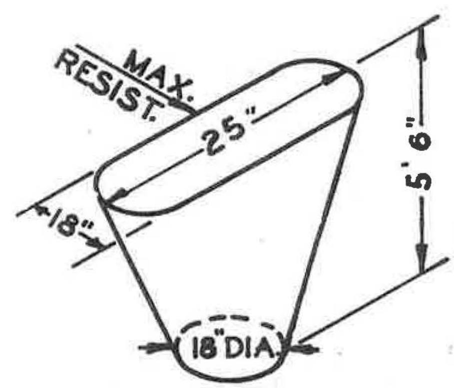

Figure 10.

2. Foundation for 1200-lb occasional load, direction known at 26 ft.

Soil - medium clay overlying cemented sand and gravel.

From Table 2 A = 870, B = 14.0, N = 1.4

Assume a depth of 6 ft, neutral axis = 4 ft.

Moment around neutral axis = 1200 x 30 = 36,000. If bottom = 18 in. dia., $M = 24{,}000 = 870D^2 + 14D^3$

D	D^2	D^3	$870D^2$	$14D^3$	Allow. M
5'	25	125	21,700	1750	23,450
6'	36	216	31,300	3020	34,320
5.1'	26	132	22,600	1850	24,450

Use 5 ft 6 in. for even half feet. In order to get away from heaving a sloped foundation was used (Fig. 10).

REFERENCES

1. Terzaghi, "Theoretical Soil Mechanics." Wiley and Sons, Inc., Arts. 12 and 127.

2. Hogentogler, "Engineering Properties of Soil." McGraw-Hill, Table 22, p. 216 and Table 23, p. 220.

3. Housel. Proceedings, ASCE, Part 2, p. 1056 (October 1943).

Tests of Tilting Moment Resistance of Cylindrical Reinforced Concrete Foundations for Overhead Sign Supports

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● THE HIGHWAY industry, as a result of the Federal-Aid Highway Acts of 1956 and 1958 authorizing 41,000 miles of interstate system, is faced with the necessity of building large numbers of overhead signs, sign bridges and other pole-mounted traffic control devices. The Ohio Department of Highways is concerned about the small amount of experimental data which is available in existing literature on the problem of foundations for pole-mounted structures. When approached by the Subcommittee on Supports for Traffic Control Devices of the Committee on Traffic Control Devices, Highway Research Board, the Department recognized the need for such information and undertook this foundation test project.

The objectives were to establish some preliminary strength data on foundations to resist tilting moments:

1. In shapes giving indication of good economy.
2. Which can be dug with generally available mechanical equipment.
3. In several easily recognized soils; namely, plastic, granular and organic.
4. Simulating conditions met in practice insofar as practical.

Foundations for poles must be designed for enough strength to prevent structural failure and yet for the sake of economy should not be too greatly overdesigned. The problem is complicated by the fact that a given sign installation does not usually justify very much soil investigation and engineering for the design of a foundation.

The principal structural requirement of a sign foundation is to resist the overturning moments due to horizontal wind loads on sign areas supported some heights above the ground. The utility industries have long used slender and deep foundations which take advantage of the horizontal resistance of the soil. This design was used for the test foundations because it required no concrete form work and is quite economical of labor and materials.

The scope of this project was the construction and testing of cylindrical foundations of reinforced concrete approximately 32 in. in diameter, 8 and 12 ft deep in the three soil types. In the tests, measurements were made of the movement of each foundation caused by known applied overturning moments in both short-term overload tests and long-term fixed load tests.

SELECTION OF TEST SITES

Test sites had to meet several conditions and considerable time was spent in a search. Most important was finding the desired soil types with some uniformity for depths up to 12 ft. This was difficult in the time available, and the sites finally selected were the best compromise that could be made. Other requirements for the sites were that they be on state-owned land, have sufficient space available for construction and testing of the foundations, and be in reasonable proximity to several interested parties.

Tentative selection of a number of possible sites was made on the basis of soil profiles available at the laboratory. Three sites were finally selected by making a visual classification of soil samples taken with a power auger. Plastic soil was found in the state highway maintenance yard at Mt. Gilead, Morrow County; granular soil was found on right-of-way on SR 76 0.6 mi south of Holmesville, Holmes County; and organic soil on new right-of-way on relocated US 30, just west of SR 13 near Mansfield, Richland County. The soils used differ considerably from one another and are very common in Ohio.

SOIL STUDIES

Additional samples were taken and soil studies made to determine accurately the character of the soils in each test site. A standard sampler, 2-in. OD, 1 3/8-in. ID driven by a 140-lb hammer in free fall of 30 in. was used and blows per foot of penetration were recorded. Where possible, a pressed tube sampler was used to obtain undisturbed samples for shear tests.

Laboratory tests determined the mechanical analysis, liquid limit, plastic limit, plasticity index and moisture content of the samples. Based on these tests, the soil types were determined by the Ohio classification system which is a modification of the Highway Research Board system, and also by the Unified Soil Classification system. The soils data are summarized in Table 1. The soil profiles shown in Figure 1 are based on visual examination of the excavated soil at construction using the previously determined soil classifications.

The plastic soil was found to contain more granular material and silt than was desired originally; hence is not, strictly speaking, "plastic". The soil ranged from brown sandy silt A-4a to brown sandy clay A-6a. The 12-ft foundation when constructed was in brown sand and gravel at depths from 9 to 12 ft. Average wet density was 138 pcf. Penetration resistance of the standard driven sampler ranged from 14 to 132 blows per foot. An attempt to obtain undisturbed samples for shear tests was unsuccessful because the pressed sampler would not penetrate the soil.

The granular soil ranged from brown gravel A-1-a to brown sand A-3-2, with an average wet density of 127 pcf. Penetration resistance ranged from 16 to 70 blows per foot. No attempt was made to obtain undisturbed samples of this soil.

The organic soil was dark gray organic elastic clay A-7-5 and A-7-6. Some of the samples were fibrous. Two wet weight determinations were 77 and 100 pcf. Moisture contents ranged from 29 to 81 percent. Loss on ignition averaged 12 percent. Shear tests on undisturbed samples resulted in coefficients of cohesion ranging from 0 to 0.23 tons per sq ft and angles of internal friction from 0 to 19 deg. This soil was clearly of little value for foundation purposes, but was used in order to gain some data on admittedly poor soil.

DESIGN OF FOUNDATIONS

The experimental foundations were so designed that they would be simple and economical to construct and require no concrete form work or

TABLE 1

SOIL TEST DATA

Lab. No. So.-	Represents Depth ft	Physical Characteristics: Mechanical Analysis: Agg. %	C Sand %	F Sand %	Silt %	Clay %	L.L.	P.I.	Water %	Soil Classification: Ohio	Unified	Penetration* Blows/6 in	Remarks
8 ft.		Plastic					Mt. Gilead					Wet density:	137 lb/cu ft
73125	1-2	9	9	17	35	30	25	11	13	A-6a	CL	-	Brown Sandy Clay Unconf. Comp. q_u= 1.67
73126	3-4	7	9	16	36	32	26	11	12	A-6a	CL	45/48	Brown Sandy Clay Unconf. Comp. q_u= 1.66
73127	5-6	9	8	13	38	32	26	7	15	A-4a	CL-ML	66/66	Brown Sandy Silt
73123	7-8	5	8	13	36	38	27	11	16	A-6a	CL	8/13	Brown Sandy Clay
73124	9-10	32	19	26	16	7	NP	NP	18	A-1-b	SM	10/11	Gray Sand & Gravel
12 ft.		Plastic					Mt. Gilead					Wet density:	140 lb/cu ft
73096	2-4	7	8	16	36	33	26	6	13	A-4a	CL-ML	13/20	Brown Sandy Silt
73097	4-5	5	7	12	42	34	26	8	14	A-4a	CL	12/18	Brown Sandy Silt
73098	6-7	5	8	13	37	37	28	11	16	A-6a	CL	13/24	Brown Sandy Clay
73099	8-9	13	8	14	38	27	22	6	13	A-4a	CL-ML	7/13	Brown Sandy Silt
73100	10-11	9	9	14	40	28	19	5	13	A-4a	CL-ML	5/9	Gray Sandy Silt
73101	12-13	12	8	14	37	29	20	6	14	A-4a	CL-ML	5/10	Gray Sandy Silt
73102	14-15	1	2	57	25	15	NP	NP	18	A-4a	SM	6/11	Gray Sandy Silt
73103	16-17	0	1	72	18	9	NP	NP	19	A-3a	SM	12/18	Gray Silty Sand
73104	18-19	9	4	60	22	5	NP	NP	17	A-3a	SM	11/23	Gray Silty Sand
8 ft		Granular					Holmesville					Wet density:	127 lb/cu ft
73108	2-3	50	26	11	9	4	NP	NP	5	A-1-a	GM	8/9	Brown Sand & Gravel
73109	4-5		Stone Fragments						3	-	GP	17/18	Brown Gravel
73110	6-7	0	21	25	51	3	NP	NP	16	A-4b	ML	10/20	Brown Sandy Silt
73111	8-9	71	10	9	9	1	NP	NP	2	A-1-a	GM	33/37	Brown Sandy Gravel
73112	10-11	63	18	14	3	2	NP	NP	9	A-1-a	GP	22/32	Brown Sandy Gravel
73113	12-13	42	40	9	8	1	NP	NP	10	A-1-b	SP	15/25	Brown Gravelly Sand
73114	14-15	32	42	14	11	1	NP	NP	11	A-1-b	SM	27/38	Brown Gravelly Sand
12 ft		Granular					Holmesville					Wet density:	127 lb/cu ft
73115	1-2	24	11	6	25	34	34	13	13	A-6a	CL	6/10	Brown Sandy Clay
73116	3-4	43	8	17	20	12	18	3	11	A-2-4	GM	9/12	Brown Silty Sandy Gravel
73117	5-6	75	11	8	4	2	NP	NP	8	A-1-a	GP	14/20	Brown Sandy Gravel
73118	7-8	76	10	8	5	1	NP	NP	13	A-1-a	GP	14/24	Brown Sandy Gravel
73119	9-10	66	21	10	1	2	NP	NP	14	A-1-a	GP	20/24	Brown Sandy Gravel
73120	11-12	27	44	18	8	3	NP	NP	16	A-1-b	SM	10/16	Brown Silty Gravelly Sand
73121	13-14	57	26	10	5	2	NP	NP	12	A-1-a	GP	28/28	Brown Sandy Gravel
73122	15-16	8	58	26	5	3	NP	NP	17	A-3a	SP	19/18	Brown Sand
8 ft		Organic					Mansfield					Wet density:	77 lb/cu ft
72663	2-3	0	0	4	52	44	47	20	53	A-7-6	CL	2/2	Mottled Brown & Gray Clay
72664	4-5	0	1	13	32	54	41	17	46	A-7-6	CL	1/2	Gray Silty Clay, sl. Organic C=0.16 ϕ=19°39'
72665	6-7	0	1	6	40	53	45	21	58	A-7-6	CL	1/2	Gray Organic Clay
72666	8-9	1	1	9	34	55	136	66	77	A-7-5	OH	1/1	Dark Gray Organic Clay w/marl
-	10-11	-	-	-	-	-	-	-	-	-	-	0/1	Dark Gray Organic Clay w/marl (visual only)
72667	$12\frac{1}{2}$-$13\frac{1}{2}$	0	1	7	49	43	39	13	44	A-6a	ML	1/1	Gray Silt & Clay, sl. Organic
72668	14-15	0	1	7	51	41	44	16	72	A-7-6	ML	1/2	Gray Silty Clay, sl. Organic
12 ft		Organic					Mansfield					Wet density:	100 lb/cu ft
72854	0-2	0	1	4	58	37	37	14	29	A-6a	CL	-	Mottled Bro. & Gr. Silt & Clay C=0.23 ϕ=19°00'
72855	3-4	0	1	1	28	70	92	46	79	A-7-5	OH	-	Gray Organic Clay C=0.21 ϕ=0°00'
72856	4-6	0	0	5	30	65	66	31	57	A-7-5	OH	-	Gray Organic Clay C=0.03 ϕ=6°50'
72857	6-8	0	2	2	44	52	75	42	74	A-7-5	CH	-	Gray Organic Clay
72858	8-10	0	1	1	32	66	70	10	62	A-5	OH	-	Gray Elastic Silt & Clay w/organic mat'l
72859	10-12	0	1	18	51	30	63	34	61	A-7-6	CH	-	Gray Organic Clay C=0.09 ϕ=2°00'
72860	12-14	2	1	2	45	50	76	48	69	A-7-6	CH	-	Gray Organic Clay C=0.00 ϕ=9°20'
72861	14-16	0	1	5	36	58	92	49	81	A-7-5	OH	-	Gray Organic Clay C=0.00 ϕ=7°30'

*Split tube sampler 2-in OD, 1-3/8-in ID driven by 140 lb hammer in free fall of 30 inches. Blows are recorded separately for first and second halves of a 1-ft penetration.

backfilling with disturbed soil. The cylindrical shape offered the best control of dimensions and was readily obtained by excavating with a power auger.

Diameter was that obtained by the use of a 30-in. diameter auger, usually about 32 in. Two depths were used in each soil: 8 and 12 ft.

Steel reinforcement consisted of four $2\frac{1}{4}$-in. anchor rods which extended 5 ft into the concrete and also served to mount the pole. In addition there were placed, in the tension side only for the sake of economy in the test foundations, 14 No. 4 deformed round reinforcing bars which lapped the anchor rods 3 ft and extended to the bottom of the foundation. Details of the test foundation are shown in Figure 2.

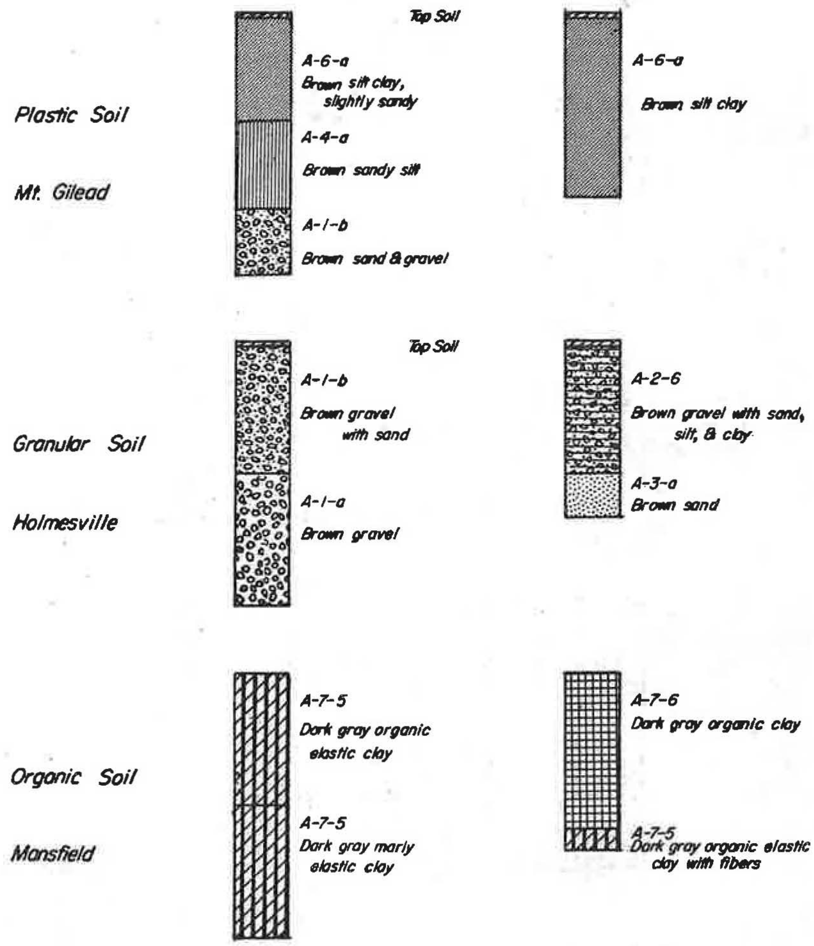

Figure 1. Soil profiles.

CONSTRUCTION

Construction of the test foundations was a relatively simple procedure. Excavation was performed by a 30-in. diameter auger mounted on a Williams rig. Diameter of the holes was about 32 in. for the plastic and organic soils. In the granular soil, boulders were encountered initially and then fine sand which tended to cave, causing irregular shapes and resulting in average diameters of approximately 36 in. Time required for excavation with the auger ranged from 10 to 15 min per hole except where caving occurred in the granular soil when up to 30 min were required. Excavation is illustrated in Figure 3.

The anchor rods were accurately positioned by means of wood templets constructed of 2- by 6-in. lumber. Concrete used was Ohio Class E, a $5\frac{1}{2}$-bag mix which developed 3,000 to 3,500 psi compressive strength at 28 days. The holes were partially filled with concrete, the No. 4 reinforcing bars were inserted, and the remainder of the concrete was placed. The completed foundation is shown in Figure 4.

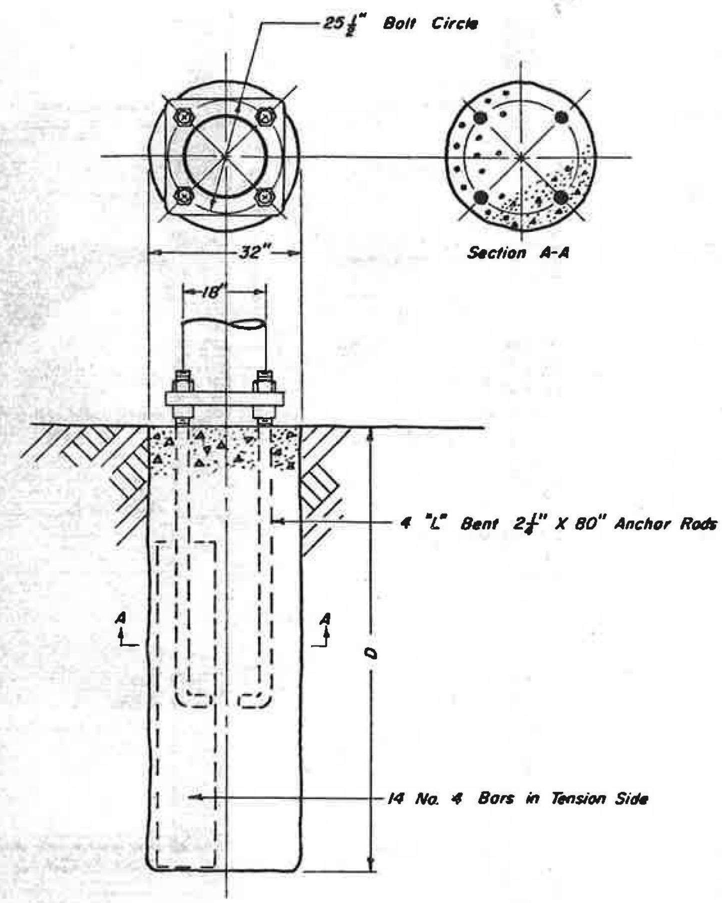

Figure 2. Test foundation details.

The amount of concrete required for the 8-ft holes was about $1\frac{1}{2}$ cu yd and for the 12-ft holes about $2\frac{1}{2}$ cu yd. For the holes in granular soil which were oversize due to caving, the concrete required was about 2 cu yd and $3\frac{1}{2}$ cu yd for the 8- and 12-ft foundations, respectively.

After the concrete had cured, the steel poles which were 26 ft long, 18 in. in diameter and weighed about 1,800 lb each were mounted with the aid of a truck-mounted crane. The pole base plates rested on square leveling nuts on the anchor rods. Hex nuts were used to tighten down the base plate. Erection of the pole is shown in Figure 5.

SHORT-TERM TESTS

Short-term loads were applied by means of the arrangement shown in Figure 6. The loading cable was $\frac{1}{2}$-in. steel wire rope attached to the pole about 25 ft above the groundline and anchored to expanding deadman anchors buried 6 ft about 125 ft away from the pole foundations. The cables were put in tension either with a 6-ton chain hoist and yoke arrangement or by means of a 7-part block and tackle system powered by a truck-mounted winch. The block and tackle system was found to be superior because it provided greater travel of the moving block and was also

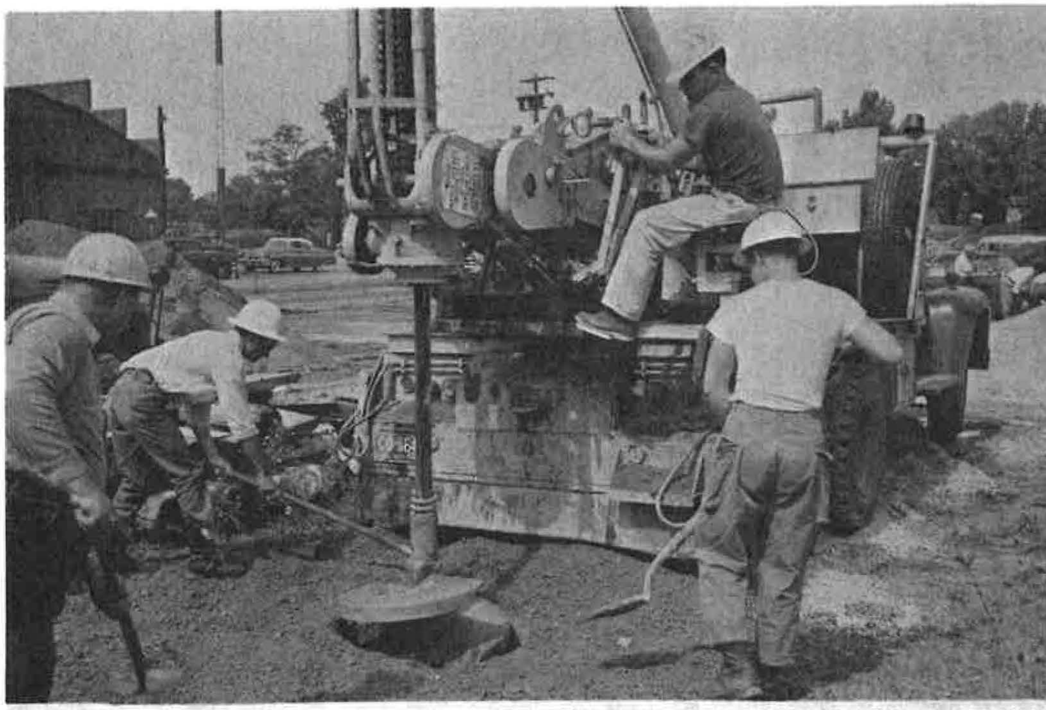

Figure 3. Excavation with 30-in. auger.

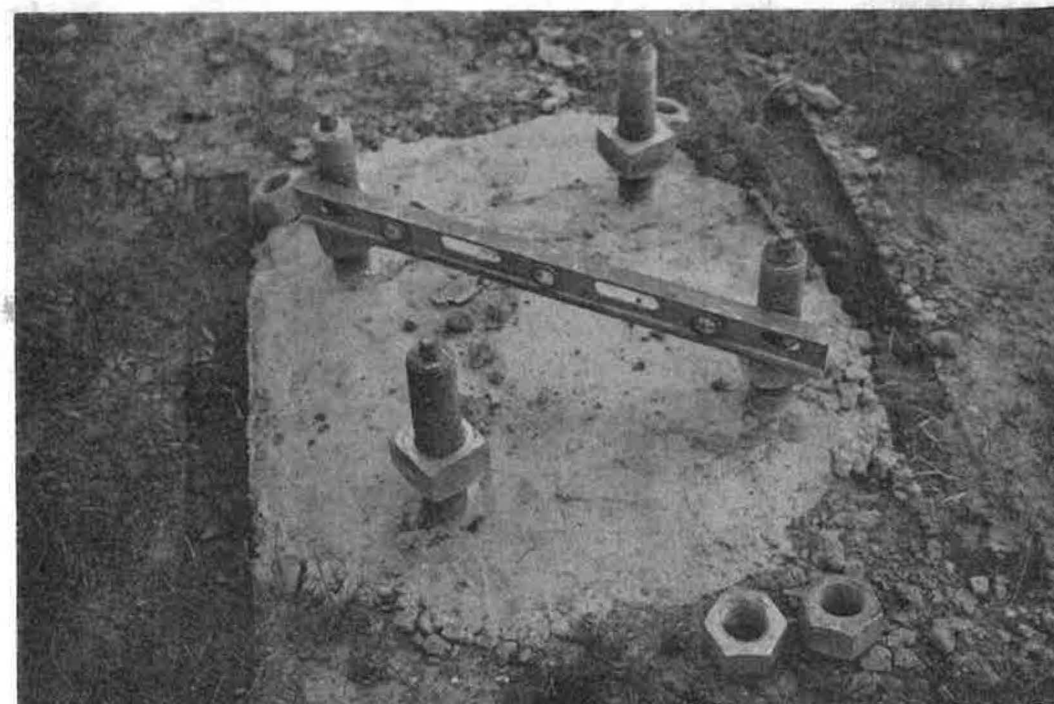

Figure 4. Completed foundation.

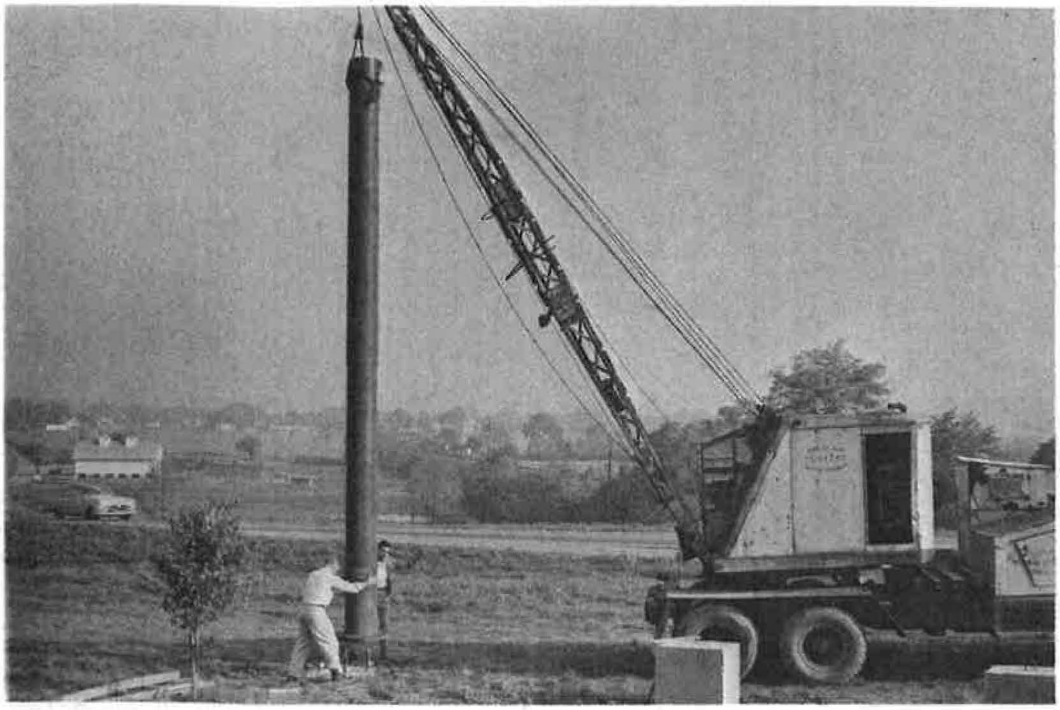

Figure 5. Mounting steel pole.

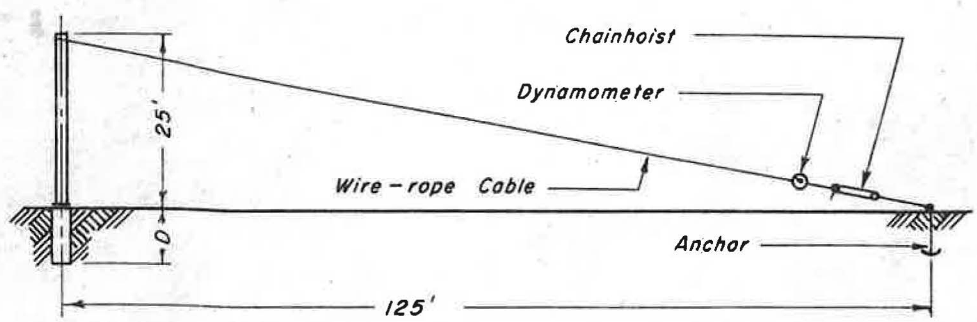

Figure 6. Variable loading for short-term tests.

faster. Tension in the cable was measured by means of 10,000-lb capacity Chattilon dynamometers. The horizontal load or thrust applied to the pole was the measured cable tension corrected for slope. This horizontal load, multiplied by height above groundline, was considered the applied overturning moment in pound-feet.

When the cable tension exceeded the capacity of the dynamometers used, two were used in parallel between steel yokes placed in the cable, two deadman anchors were used and the wire rope was doubled. The short-term test is illustrated in Figure 7. The use of chain hoist, dynamometers, and steel yokes for loading is illustrated in Figure 8.

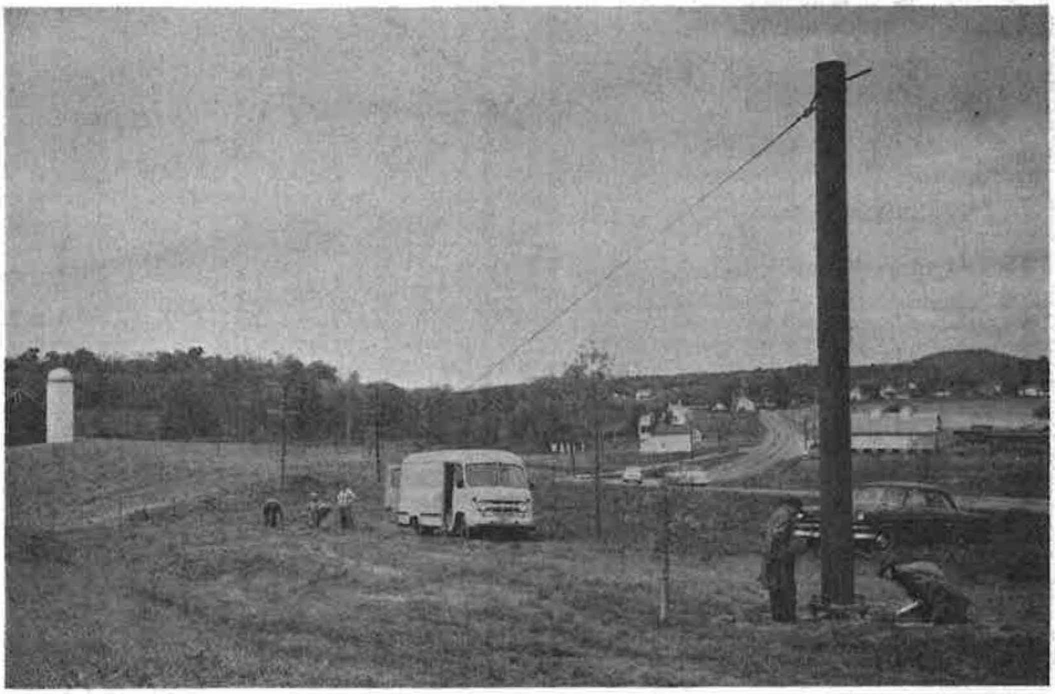

Figure 7. Short-term test.

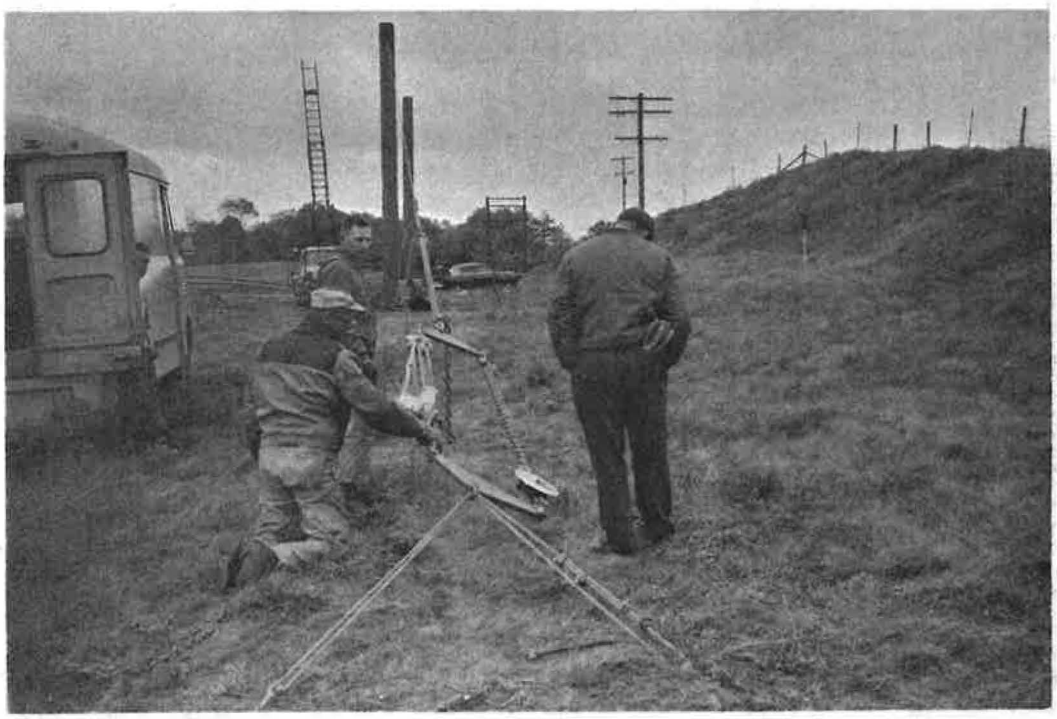

Figure 8. Chain hoist and dynamometers in yokes.

Tilt of the foundation was measured by means of an improvised clinometer as illustrated in Figure 9. It consisted of a steel bar mounted on leveling screws which carried an accurate 1-min Starrett mechanics' level and a 0- to 1-in. Ames dial indicator. As the tilt of the foundation increased, the clinometer was leveled by means of the adjusting screws and the dial indicator measured the change in elevation of one end of the clinometer. This reading divided by the 10-in. base yielded the tangent of the tilt angle directly.

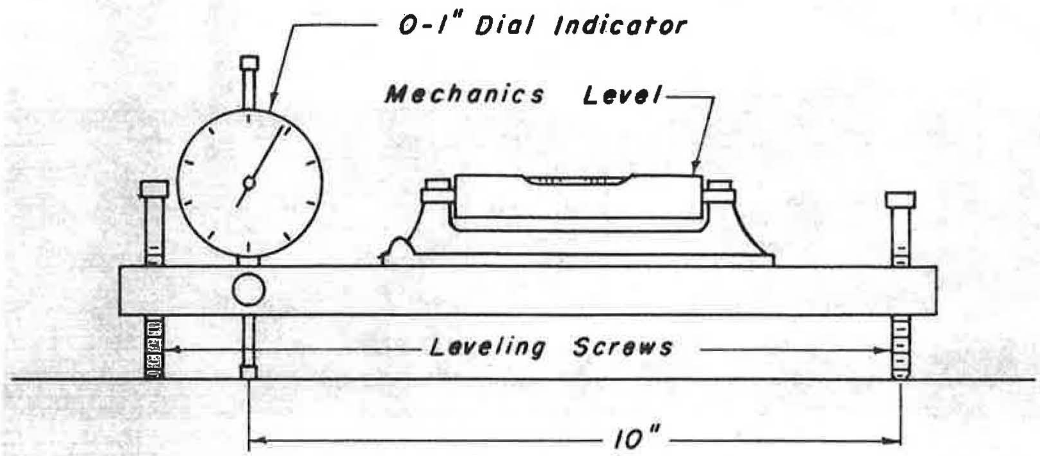

Figure 9. Clinometer details.

For the short-term tests, the clinometer was initially set with a small "seating" load of about 12,500 lb-ft on the foundation. The load was increased by increments and the tilt of the foundation was measured for each load. Deflection of the top of the pole was measured by means of transit readings on an attached scale. After each load increment, the load was reduced back to the seating load and measurements were made. This procedure obtained information on recovery characteristics of each soil.

In addition, measurements were made of the horizontal movement of the top of the foundation by means of an Ames dial indicator. These measurements made possible the computation of depth of the neutral axis or center of rotation of the foundation.

In the short-term tests the maximum loads were applied and tests completed within 3 hr.

LONG-TERM TESTS

After completion of the short-term test on each foundation, a constant load was applied so that the movement of the foundation could be observed over a long period of time. The magnitude of load used was roughly one-half the maximum load applied in the short-term test. The arrangement is shown in Figure 10. A concrete cube weighing about one ton was suspended from the wire rope at a point between the pole and the anchor so as to produce the desired horizontal component of tension

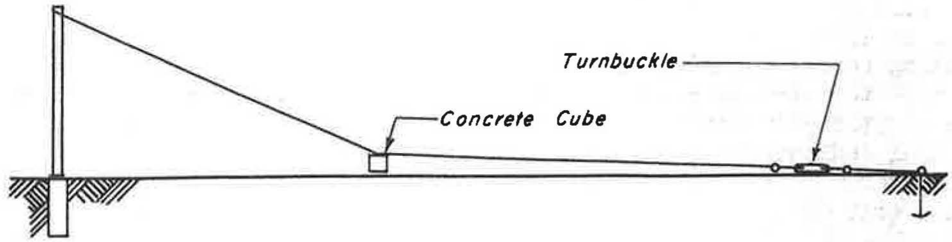

Figure 10. Fixed loading for long-term tests.

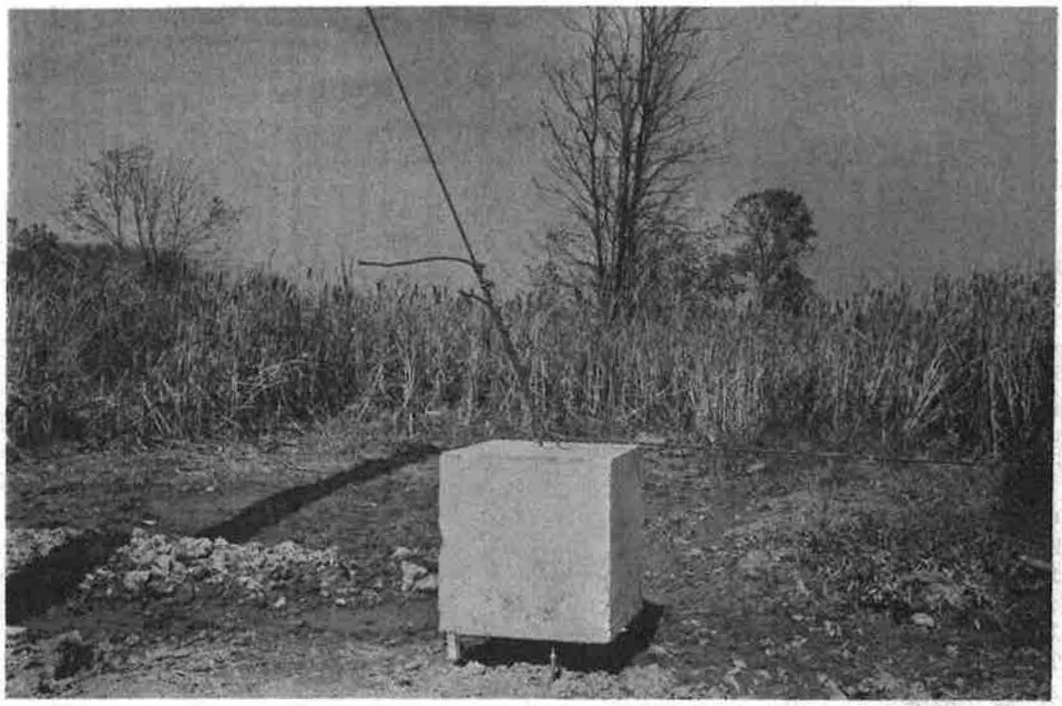

Figure 11. Concrete cube for long-term loading.

in the cable. One of the cubes is shown in Figure 11. With this arrangement, movement of the pole or anchor would cause the weight to drop slightly but the horizontal force on the pole would not change significantly.

Measurements of movement of the foundations were made at intervals of 1 to 2 months.

Tilt of each foundation was determined by measuring periodically the slope of the surface of the foundation with the clinometer direct and reversed. Changes in this slope were considered to be tilt of the foundation.

A turnbuckle in the cable was used to compensate for movement of either top of pole or anchorage and to restore the weight to its original elevation.

TABLE 2

SHORT-TERM TEST DATA FOR 8-FOOT FOUNDATION IN PLASTIC SOIL

Depth of foundation: 8.2 ft
Height of load: 24.4 ft
Horizontal load factor: 0.985

November 12-13,1957

Rdg. No.	Time: E.S.T.	Time: E-lapsed min	Load: Dynamometer Readings East	Load: Dynamometer Readings West	Load: Cable Tension lb	Load: Horiz. Comp. lb	Load: Moment at Ground-line lb-ft	Horizontal Movement of Fdn. at Ground-line in	Horizontal Movement of Top of Pole in	Tilt of Top of Found-ation radians	Depth of Center of Rotation ft	Remarks
	pm											
1	3:17	0	-	-	500	490	11940	0.000	0.00	0.0000	-	Nov. 12
2	3:20	3	-	-	1000	980	24000	0.001	0.19	-0.0001	-	
3	3:24	7	-	-	500	490	11940	0.000	0.00	-0.0002	-	
4	3:28	11	-	-	2000	1970	48100	0.010	0.66	+0.0001	-	
5	3:32	15	-	-	500	490	11940	0.003	0.06	-0.0001	-	
6	3:36	19	-	-	3000	2960	72100	0.025	1.25	+0.0004	5.2	
7	3:39	22	-	-	500	490	11940	0.007	0.12	0.0001	5.8	
8	3:45	28	-	-	4000	3940	96100	0.046	1.81	0.0007	5.5	
9	3:50	33	-	-	500	490	11940	0.019	0.28	0.0002	8.0	
10	3:54	37	-	-	5000	4930	120200	0.077	2.53	0.0011	5.8	
11	3:58	41	-	-	500	490	11940	0.029	0.41	0.0004	6.0	
12	4:02	45	-	-	6000	5910	144200	0.110	3.25	0.0015	6.1	
13	4:06	49	-	-	500	490	11940	0.042	0.59	0.0005	7.0	
14	4:11	54	-	-	7000	6900	168300	0.142	4.03	0.0019	6.2	
15	4:15	58	-	-	500	490	11940	0.055	0.81	0.0007	6.5	
16	4:20	63	-	-	8000	7880	192200	0.183	4.84	0.0025	6.1	
17	4:25	68	-	-	500	490	11940	0.073	1.12	0.0010	6.1	
18	4:30	73	-	-	9000	8860	216000	0.222	5.88	0.0031	6.0	
19	4:35	78	-	-	500	490	11940	0.092	1.56	0.0010	7.7	
20	4:40	83	-	-	10000	9850	240000	0.277	7.12	0.0037	6.2	
21	4:45	88	-	-	500	490	11940	0.118	2.19	0.0013	7.6	
	am											
22	9:40	1103	100	400	500	490	11940	0.000	2.03	0.0007	-	Nov. 13
23	10:45	1168	4200	5800	10000	9850	240000	0.207	7.28	0.0057	7.3	
24	11:00	1183	100	400	500	490	11940	0.048	2.50	0.0032	6.5	
25	11:24	1207	5300	5700	11000	10830	264000	0.256	8.56	0.0065	7.4	
26	11:28	1211	5500	6500	12000	11820	289000	0.296	10.25	0.0071	7.5	
27	11:33	1216	5200	7800	13000	12810	313000	0.419	14.88	0.0093	7.7	
28	11:40	1223	100	400	500	490	11940	0.196	8.41	0.0059	7.4	

TEST RESULTS

Results of the short-term tests indicate that the plastic and granular soils were similar in their strength characteristics as measured by resistance of the foundations to overturning. Test results are given in Tables 2 to 7 inclusive and curves of overturning moment versus angular tilt are plotted in Figure 12. Although the curves for plastic and granular soils appear similar, the granular soil is slightly weaker because these foundations were oversize. The curve for the 8-ft foundation in plastic soil shows a discontinuity because a repetition of the same overturning load caused an increased tilt. As anticipated, the overturning resistance of the organic soil compared very poorly with the other soils.

The total angular tilt observed in these tests was quite small. For the plastic and granular soils, the maximum tilts were about $\frac{1}{4}$ deg. For these, the tests were halted because either the deadman anchors began to yield or the foundations themselves began to show signs of distress as

TABLE 3

SHORT-TERM TEST DATA FOR 12-FOOT FOUNDATION IN PLASTIC SOIL

Depth of foundation: 12.0 ft
Height of load: 24.4 ft
Horizontal load factor: 0.984

Mt. Gilead
November 13, 1957

Rdg. No.	Time E.S.T.	Time E-lapsed min	Load: Dynamometer Readings East	Load: Dynamometer Readings West	Load: Cable Tension lb	Load: Horiz. Comp. lb	Load: Moment at Ground-line lb-ft	Horizontal Movement of Fdn. at Ground-line in	Horizontal Movement of Top of Pole in	Tilt of Top of Foundation radians	Depth of Center of Rotation ft	Remarks
	pm											
1	3:04	0	-	-	950	940	22900	0.000	0.09	0.0001	-	
2	3:08	4	900	1100	2000	1970	48100	0.004	0.72	0.0001	3.3	
3	3:12	8	100	400	500	490	12000	0.000	0.00	0.0000	-	
4	3:18	14	2100	1900	4000	3940	96200	0.016	2.34	0.0004	3.3	
5	3:23	19	100	400	500	490	12000	0.003	0.16	0.0000	-	
6	3:31	27	3400	2600	6000	5910	144000	0.038	3.91	0.0006	5.3	
7	3:39	35	400	100	500	490	12000	0.010	0.47	0.0001	8.3	
8	4:09	65	3100	4900	8000	7870	192000	0.074	5.65	0.0012	5.1	
9	4:15	71	400	100	500	490	12000	0.021	0.84	0.0003	5.8	
10	4:25	81	3400	6600	10000	9840	240000	0.109	7.59	0.0018	5.0	
11	4:43	99	600	100	700	690	16800	0.036	1.65	0.0006	5.1	
12	4:51	107	5000	7000	12000	11810	288000	0.166	10.41	0.0026	5.3	
13	5:03	119	500	100	600	590	14400	0.054	2.78	0.0010	4.5	
14	5:28	144	5200	7800	13500	13290	324000	0.319	16	0.0057	4.7	Dead man anchors yielding
15	5:35	151	400	100	500	490	12000	0.172	-	0.0034	4.2	Too dark for transit readings

TABLE 4

SHORT-TERM TEST DATA FOR 8-FOOT FOUNDATION IN GRANULAR SOIL

Depth of foundation : 8.0 ft
Height of load : 24.4 ft
Horizontal load factor : .975

Holmesville
December 3, 1957

Rdg. No.	Time E.S.T.	Time E-lapsed min	Load: Dynamometer Readings East	Load: Dynamometer Readings West	Load: Cable Tension lb	Load: Horiz. Comp. lb	Load: Moment at Ground-line lb-ft	Horizontal Movement of Fdn. at Ground-line in	Horizontal Movement of Top of Pole in	Tilt of Top of Foundation radians	Depth of Center of Rotation ft	Remarks
	pm											
1	12:55	0	100	400	500	490	11960	0.000	0.00	0.0000	-	
2	1:02	7	950	1050	2000	1950	47600	0.012	1.44	0.0004	2.5	
3	1:15	20	100	400	500	490	11960	0.004	0.19	0.0001	3.3	
4	2:40	105	800	2200	4000	3900	95200	0.048	4.00	0.0010	4.0	
5	2:48	113	100	400	500	490	11960	0.021	0.75	0.0004	4.4	
6	3:07	132	2600	3400	6000	5850	142800	0.102	5.69	0.0019	4.5	
7	3:20	145	100	400	500	490	11960	0.047	1.25	0.0010	3.9	
8	3:40	165	3800	4200	8000	7800	190300	0.175	7.75	0.0030	4.9	
9	3:46	171	200	800	1000	980	23900	0.100	2.50	0.0019	4.4	
10	4:00	185	4000	6000	10000	9750	238000	0.296	10.06	0.0049	5.0	
11	4:03	188	4000	6000	10000	9750	238000	0.301	10.13	0.0049	5.1	
12	4:15	200	100	400	500	490	11960	0.178	3.19	0.0029	5.1	
13	4:21	206	0	0	0	0	0	0.157	-	0.0026	5.1	

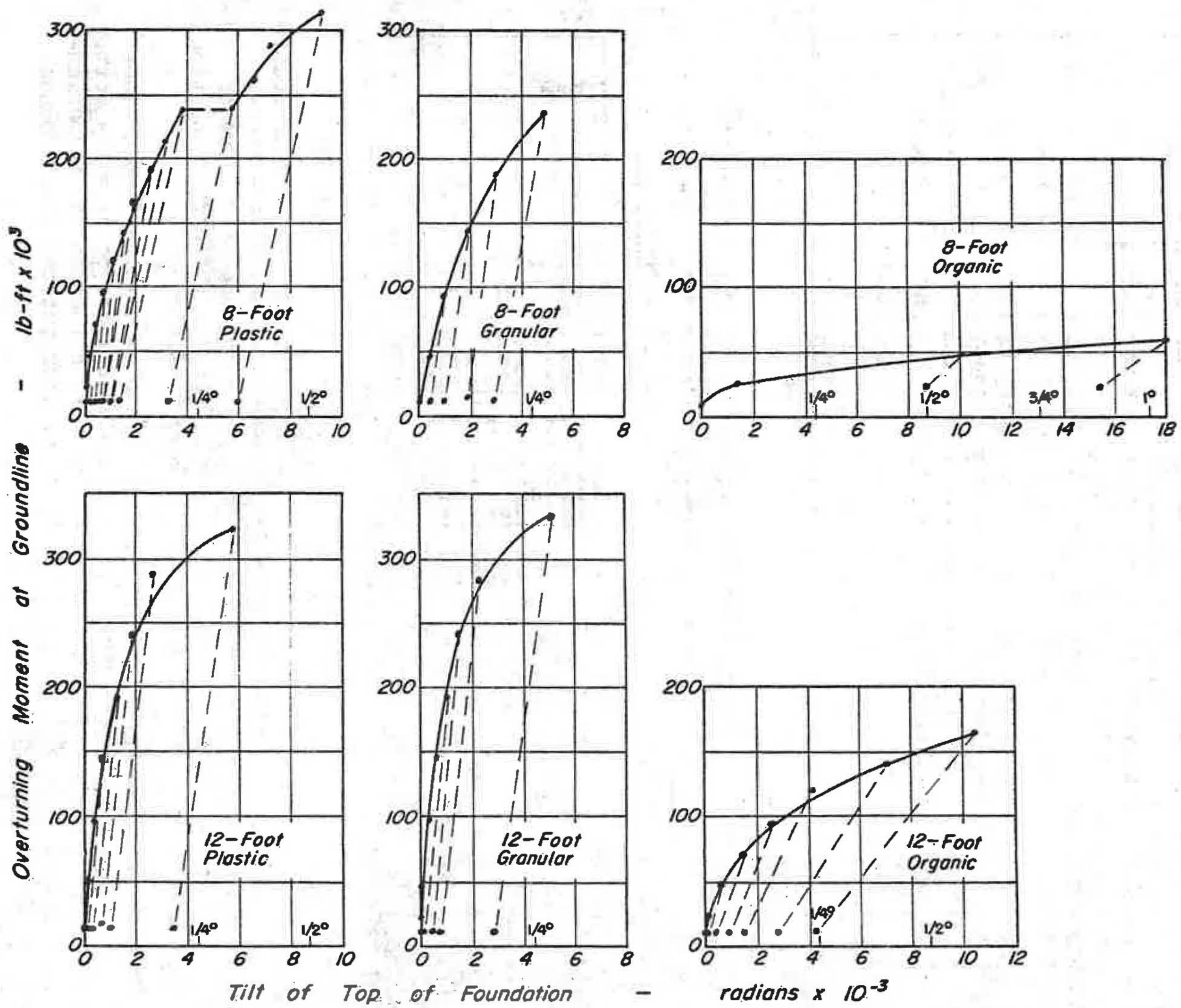

Figure 12. Tilt of foundations in short-term tests.

TABLE 5

SHORT TERM TEST DATA FOR 12-FOOT FOUNDATION IN GRANULAR SOIL

Depth of foundation : 12.3 ft
Height of load : 24.3 ft
Horizontal load factor : 0.974

Holmesville
December 16, 1957

Rdg. No.	Time E.S.T.	Time E-lapsed min	Load: Dynamometer Readings East	Load: Dynamometer Readings West	Load: Cable Tension lb	Load: Horiz. Comp. lb	Load: Moment at Ground-line lb-ft	Horizontal Movement of Fdn. at Ground-line in	Horizontal Movement of Top of Pole in	Tilt of Top of Found-ation radians	Depth of Center of Rotation ft	Remarks
	pm											
1	1:35	0	250	250	500	490	11830	0.000	0.00	0.0000	-	
2	1:37	2	500	500	1000	970	23700	0.003	0.28	0.0000	-	
3	1:40	5	250	250	500	490	11830	0.001	-0.03	0.0000	-	
4	1:43	8	1025	1025	2050	2000	48600	0.010	0.94	-0.0001	-	
5	1:45	10	250	250	500	490	11830	0.003	0.10	-0.0001	-	
6	1:47	12	2050	2100	4150	4040	98400	0.033	2.44	+0.0003	9	
7	1:49	14	250	250	500	490	11830	0.009	0.22	-0.0001	-	
8	1:52	17	3050	3100	6150	5990	145800	0.068	3.19	+0.0006	9	
9	1:55	20	250	250	500	490	11830	0.020	0.56	0.0000	-	
10	2:01	26	4100	4100	8200	7990	194200	0.107	5.88	0.0010	9	
11	2:05	30	250	250	500	490	11830	0.034	1.00	0.0001	-	
12	2:10	35	5100	5100	10200	9940	242000	0.161	8.19	0.0015	9	
13	2:12	37	250	250	500	490	11830	0.053	1.91	0.0004	10	
14	2:15	40	6000	6000	12000	11700	284000	0.223	11.41	0.0023	8.1	
15	2:17	42	250	250	500	490	11830	0.077	3.72	0.0007	9.2	
16	2:25	50	7000	7000	14000	13630	332000	0.336	20.44	0.0051	5.5	
17	2:27	52	250	250	500	490	11830	0.233	11.28	0.0028	6.9	

TABLE 6

SHORT-TERM TEST DATA FOR 8-FOOT FOUNDATION IN ORGANIC SOIL

Depth of foundation: 7.9 ft
Height of load: 24.1 ft
Horizontal load factor: 0.975

Mansfield
October 9, 1957

Rdg. No.	Time E.S.T.	Time E-lapsed min	Load: Dynamometer Readings East	Load: Dynamometer Readings West	Load: Cable Tension lb	Load: Horiz. Comp. lb	Load: Moment at Ground-line lb-ft	Horizontal Movement of Fdn. at Ground-line in	Horizontal Movement of Top of Pole in	Tilt of Top of Found-ation radians	Depth of Center of Rotation ft	Remarks
	p.m.											
1	2:13	0	-	-	400	390	9400	0.000	0.00	0.0000	-	
2	2:17	4	-	-	1000	970	23400	0.025	0.66	0.0013	1.9	
3	2:21	8	-	-	2100	2050	49400	0.314	3.82	0.0100	3.2	
4	2:25	12	-	-	1000	970	23400	0.131	2.69	0.0087	3.0	
5	2:31	18	-	-	2600	2540	61200	0.392	6.69	0.0180	2.9	
6	2:35	22	-	-	1000	970	23400	0.278	4.72	0.0154	2.7	

evidenced by spalling or cracking of the concrete around the anchor rods. The strength limit of these soils was not reached in any of the tests. The tests in organic soil, however, were stopped because of failure of the soil. Here a tilt of over 1 deg was observed for the 8-ft foundation and the load-deflection curve had become very flat.

Recovery characteristics of the soils are shown by the dashed lines plotted in Figure 12. In general, the plastic soil showed slightly better recovery characteristics than the granular. The situation for the organic soil is not clear; the 8-ft foundation was poor but the 12-ft foundation exhibited good recovery.

The influence of foundation depth is clearly evident in the slopes of the load-tilt curves if not in the load maximums attained. For the plastic soil tilts of the 8-ft foundation are about double those of the 12-ft foundations, although the ratio of depths is only 1.5 to 1. For the granular soil the ratio of tilts is 3 to 1. For the organic soil, the curve for the 8-ft foundation is so flat that no direct comparison is possible. It appears that for the plastic soil, the strength developed is a function of depth approximately squared, and for granular soil approximately cubed.

The computations for depth of neutral axis or center of rotation are based on the equation s = re in which s is the observed lateral movement of the top of the foundation in feet, e is the observed angular tilt in radians, and r is the radius of the rotating system in feet. In the light

TABLE 7

SHORT-TERM TEST DATA FOR 12-FOOT FOUNDATION IN ORGANIC SOIL

Depth of foundation: 12.0 ft
Height of load: 24.2 ft
Horizontal load factor: 0.982

Mansfield
December 17, 1957

Rdg. No.	Time: E.S.T.	Time: E-lapsed min	Load: Dynamometer Readings East	Load: Dynamometer Readings West	Load: Cable Tension lb	Load: Horiz. Comp. lb	Load: Moment at Ground-line lb-ft	Horizontal Movement of Fdn. at Ground-line in	Horizontal Movement of Top of Pole in	Tilt of Top of Found-ation radians	Depth of Center of Rotation ft	Remarks
	pm											
1	12:51	0	-	-	500	490	11900	0.000	0.00	0.0000	-	
2	12:55	4	-	-	1000	980	23700	0.007	0.63	0.0001	5.8	
3	12:57	6	-	-	500	490	11900	0.002	0.06	0.0000	-	
4	12:59	8	-	-	2050	2010	48600	0.039	1.50	0.0006	5.4	
5	1:00	9	-	-	500	490	11900	0.020	0.19	0.0001	-	
6	1:03	12	-	-	3000	2940	71100	0.109	2.50	0.0015	6.1	
7	1:05	14	-	-	500	490	11900	0.045	0.32	0.0004	9.4	
8	1:09	18	-	-	4000	3920	94900	0.215	3.69	0.0026	6.9	
9	1:10	19	-	-	500	490	11900	0.090	0.69	0.0009	8.3	
10	1:13	22	-	-	5100	5010	121000	0.358	5.19	0.0042	7.1	
11	1:16	25	-	-	500	490	11900	0.145	0.94	0.0015	8.1	
12	1:18	27	-	-	5900	5790	140000	0.594	7.19	0.0067	7.4	
13	1:20	29	-	-	6000	5890	142000	0.635	7.44	0.0071	7.5	
14	1:22	31	-	-	500	490	11900	0.260	1.69	0.0028	7.7	
15	1:27	36	-	-	7000	6860	166000	0.923	9.82	0.0105	7.3	
16	1;29	38	-	-	500	490	11900	0.413	-	0.0043	8.0	

TABLE 8

LONG-TERM TEST DATA FOR 8-FOOT FOUNDATION IN PLASTIC SOIL

Depth of foundation: 8.2 ft
Weight of concrete cube: 2,230 lb
Horizontal component of load: 6,000 lb
Height of load: 24.4 ft
Overturning moment at groundline: 146,400 lb-ft
Depth of center of rotation: 6.0 ft
Moment arm to center of rotation: 30.4 ft

Mt. Gilead

Date	E-lapsed Time days	Height of Cube ft	Horiz. Move. of Top of Fdn. ft	Tilt of Foundation: By Clinometer on Foundation: Dial N in	Dial S in	Diff/2 in	Rotation radians	Tilt of Foundation: By Movement of Top of Pole: Transit Rdg. in	Horiz. Move. in	Rotation radians	Remarks
Nov. 14, 57	0	2.68	0.00	-	-	0.012	0.0000	5.56	0.00	0.0000	
Dec. 18, 57	34	2.44	-0.01	-	-	-	-	5.31	0.25	0.0007	Ice on foundation
		2.71	-0.01	-	-	-	-	5.25	0.31	0.0009	
Jan. 28, 58	75	2.60	0.00	0.647	0.629	0.009	-0.0003	4.81	0.75	0.0021	
		2.72	0.00	0.645	0.633	0.006	-0.0006	4.81	0.75	0.0021	
Mar. 7, 58	113	2.45	0.01	0.385	0.338	0.023	+0.0011	4.81	0.75	0.0021	
		2.70	0.01	0.403	0.338	0.033	0.0021	4.62	0.94	0.0026	
Apr. 23, 58	160	2.44	0.02	0.703	0.647	0.028	0.0016	4.38	1.18	0.0032	
		2.69	0.02	0.703	0.647	0.028	0.0016	4.31	1.25	0.0034	
May 26, 58	193	2.63	0.02	0.385	0.340	0.023	0.0011	4.38	1.18	0.0032	
		2.70	0.02	0.384	0.340	0.022	0.0011	4.31	1.25	0.0034	
July 22, 58	250	2.59	0.02	0.387	0.345	0.021	0.0009	4.25	1.31	0.0036	
		2.70	0.02	0.388	0.345	0.022	0.0010	4.19	1.37	0.0038	

TABLE 9

LONG-TERM TEST DATA FOR 12-FOOT FOUNDATION IN PLASTIC SOIL

Mt. Gilead

Depth of foundation:	12.0 ft
Weight of concrete cube:	2,240 lb
Horizontal component of load:	8,000 lb
Height of load:	24.4 ft
Overturning moment at groundline:	195,000 lb-ft
Depth of center of rotation:	5.0 ft
Moment arm to center of rotation:	29.4 ft

Date	E-lapsed Time	Height of Cube	Horiz. Move. of Top of Fdn.	Tilt of Foundation							Remarks
				By Clinometer on Foundation				By Movement of Top of Pole			
				Dial N	Dial S	Diff/2	Rotation	Transit Rdg.	Horiz. Move.	Rotation	
	days	ft	ft	in	in	in	radians	in	in	radians	
Nov. 14, 57	0	2.87	0.00	-	-	0.101	0.0000	2.69	0.00	0.0000	
Dec. 18, 57	34	2.09	0.00	-	-	-	-	2.94	-0.25	-0.0007	Ice on foundation
		2.94	0.00	-	-	-	-	2.31	0.38	0.0011	
Jan. 28, 58	75	2.53	0.00	0.746	0.541	0.102	0.0001	2.38	0.31	0.0009	
		2.93	0.00	0.757	0.534	0.112	0.0011	2.06	0.63	0.0018	
Mar. 7, 58	113	2.56	0.01	0.486	0.259	0.114	0.0013	2.25	0.44	0.0012	
		2.91	0.01	0.485	0.259	0.113	0.0012	2.00	0.69	0.0020	
Apr. 23, 58	160	2.53	0.01	0.735	0.512	0.112	0.0011	1.88	0.71	0.0020	
		2.89	0.01	0.735	0.512	0.112	0.0011	1.62	1.07	0.0030	
May 26, 58	193	2.79	0.01	0.475	0.246	0.115	0.0014	1.75	0.94	0.0027	
		2.91	0.01	0.475	0.242	0.117	0.0016	1.56	1.13	0.0032	
July 22, 58	250	2.74	0.00	0.478	0.252	0.113	0.0012	1.44	1.25	0.0035	
		2.89	0.00	0.478	0.252	0.113	0.0012	1.31	1.38	0.0039	

TABLE 10

LONG-TERM TEST DATA FOR 8-FOOT FOUNDATION IN GRANULAR SOIL

Holmesville

Depth of foundation:	8.0 ft
Weight of concrete cube:	2,240 lb
Horizontal component of load:	6,000 lb
Height of load:	24.4 ft
Overturning moment at groundline:	146,000 lb-ft
Depth of center of rotation:	4.5 ft
Moment arm to center of rotation:	28.9 ft

Date	E-lapsed Time	Height of Cube	Horiz. Move. of Top of Fdn.	Tilt of Foundation							Remarks
				By Clinometer on Foundation				By Movement of Top of Pole			
				Dial N	Dial S	Diff/2	Rotation	Transit Rdg.	Horiz. Move.	Rotation	
	days	ft	ft	in	in	in	radians	in	in	radians	
Dec. 16, 57	0	2.50	0.00	0.205	0.543	0.169	0.0000	-	-	-	
Dec. 18, 57	2	2.26	0.00	-	-	-	-	10.88	0.00	0.0000	Ice on foundation
		2.51	0.00	-	-	-	-	11.00	0.12	0.0003	
Jan. 28, 58	43	2.18	0.00	0.406	0.779	0.186	0.0017	11.00	0.12	0.0003	
		2.50	0.00	0.404	0.776	0.186	0.0017	10.88	0.00	0.0000	
Mar. 6, 58	80	2.22	0.01	0.178	0.572	0.197	0.0028	11.81	0.93	0.0027	
		2.52	0.01	0.231	0.629	0.199	0.0030	11.88	1.00	0.0029	
Apr. 23, 58	128	2.27	0.02	0.168	0.575	0.203	0.0034	12.25	1.37	0.0040	
		2.53	0.02	0.168	0.574	0.203	0.0034	12.31	1.43	0.0041	
May 27, 58	162	2.39	0.02	0.159	0.566	0.204	0.0035	12.22	1.34	0.0039	
		2.50	0.02	0.159	0.565	0.203	0.0034	12.81	1.93	0.0056	
Jul. 22, 58	218	2.44	0.03	0.150	0.560	0.205	0.0036	12.44	1.56	0.0045	
		2.50	0.03	0.149	0.560	0.205	0.0036	12.50	1.62	0.0047	

of the usual assumption of center of rotation being 2/3 of the depth, the results obtained are somewhat puzzling. The 8-ft plastic, 8-ft granular and 12-ft organic, with 6.0 ft, 5.0 and 8.0 ft, respectively, were about true to form. The computed depth of center of rotation of the 12-ft plastic and 12-ft granular foundations, however, were 4.7 and 5.5 ft, respectively. This departure from the 2/3 depth rule of thumb suggests that the foundations did not rotate as rigid bodies, but rather that there was bending of the slender foundations under the applied overturning moment.

In the case of the 8-ft foundation in organic soil, the computed depth of rotation was 2.9 ft; because the magnitude of the applied load was insufficient to cause bending, the cause was undoubtedly nonuniformity of the soil. The soil samples indicated greater strength for the top 3 ft than for the fottom 5 ft. This foundation probably rotated about this surface layer of slightly stronger soil.

Results of the long-term tests indicate that fixed loads about one-half as great as the maximum loads used in short-term tests produce about

TABLE 11

LONG-TERM TEST DATA FOR 12-FOOT FOUNDATION IN GRANULAR SOIL

Holmesville

Depth of foundation: 12.3 ft
Weight of concrete cube: 2,250 lb
Horizontal component of load: 8,000 lb
Height of load: 24.3 ft
Overturning moment at groundline: 194,000 lb-ft
Depth of center of rotation: 9.0 ft
Moment arm to center of rotation: 33.3 ft

Date	E-lapsed Time	Height of Cube	Horiz. Move. of Top of Fdn.	Tilt of Foundation							Remarks
				By Clinometer on Foundation				By Movement of Top of Pole			
				Dial N	Dial S	Diff/2	Rotation	Transit Rdg.	Horiz. Move.	Rotation	
	days	ft	ft	in	in	in	radians	in	in	radians	
Dec. 16, 57	0	2.90	0.00	0.500	0.716	0.108	0.0000	8.88	-	-	
Dec. 18, 57	2	2.37	0.00	-	-	-	-	8.50	0.00	0.0000	Ice on foundation
		2.90	0.00	-	-	-	-	8.81	0.31	0.0008	
Jan. 28, 58	43	2.27	0.00	0.534	0.750	0.113	0.0005	8.56	0.06	0.0002	
		2.89	0.00	0.355	0.506	0.126	0.0018	8.88	0.38	0.0009	
Mar. 6, 58	80	2.55	0.00	0.362	0.513	-	-	8.81	0.31	0.0008	
		2.55	0.00	0.352	0.513	-	-	8.81	0.31	0.0008	No adjustment of load made
Apr. 23, 58	128	2.30	0.00	0.250	0.493	0.122	0.0014	8.81	0.31	0.0008	
		2.96	0.00	0.249	0.492	0.122	0.0014	9.25	0.75	0.0019	
May 27, 58	162	2.74	0.00	0.258	0.485	0.113	0.0005	9.12	0.62	0.0016	
		2.93	0.00	0.248	0.482	0.117	0.0009	9.38	0.88	0.0022	
July 22, 58	218	2.84	0.00	0.269	0.518	0.125	0.0017	9.19	0.69	0.0017	
		2.91	0.00	0.269	0.518	0.125	0.0017	9.38	0.88	0.0022	

TABLE 12

LONG-TERM TEST DATA FOR 8-FOOT FOUNDATION IN ORGANIC SOIL

Mansfield

Depth of foundation: 7.9 ft
Weight of concrete cube: 2,250 lb
Horizontal component of load: 1,000 lb
Height of load: 24.1 ft
Overturning moment at groundline: 24,100 lb-ft
Depth of center of rotation: 3.0 ft
Moment arm to center of rotation: 27.1 ft

Date	E-lapsed Time	Height of Cube	Horiz. Move. of Top of Fdn.	Tilt of Foundation							Remarks
				By Clinometer on Foundation				By Movement of Top of Pole			
				Dial E	Dial W	Diff/2	Rotation	Transit Rdg.	Horiz. Move.	Rotation	
	days	ft	ft	in	in	in	radians	in	in	radians	
Oct. 9, 57	0	2.50	-	-	-	0.024	0.0000	33.28	0.00	0.0000	
Oct. 14, 57	5	2.42	-	-	-	-	-	32.88	0.40	0.0012	
		2.50	-	-	-	-	-	32.75	0.53	0.0016	
Dec. 17, 57	69	-	Foundation under water.				No measurements or adjustments made.				
Jan. 28, 58	111	2.18	-	0.586	0.703	0.058	0.0034	32.50	0.78	0.0024	
		2.50	-	0.597	0.704	0.054	0.0030	31.25	2.03	0.0062	
Mar. 6, 58	148	2.11	-	0.356	0.694	0.169	0.0145	27.62	5.66	0.0174	
		2.50	-	0.353	0.698	0.172	0.0148	27.56	5.72	0.0176	
Apr. 23, 58	196	2.39	-	0.170	0.546	0.188	0.0164	26.38	6.90	0.0212	
		2.50	-	0.170	0.548	0.189	0.0165	26.88	6.90	0.0212	
May 27, 58	230	2.49	-	0.161	0.551	0.195	0.0171	26.50	6.78	0.0208	No adjustment necessary
July 22, 58	286	2.41	-	0.044	0.467	0.212	0.0188	25.94	7.34	0.0226	
		2.49	-	0.044	0.467	0.212	0.0188	25.88	7.40	0.0228	

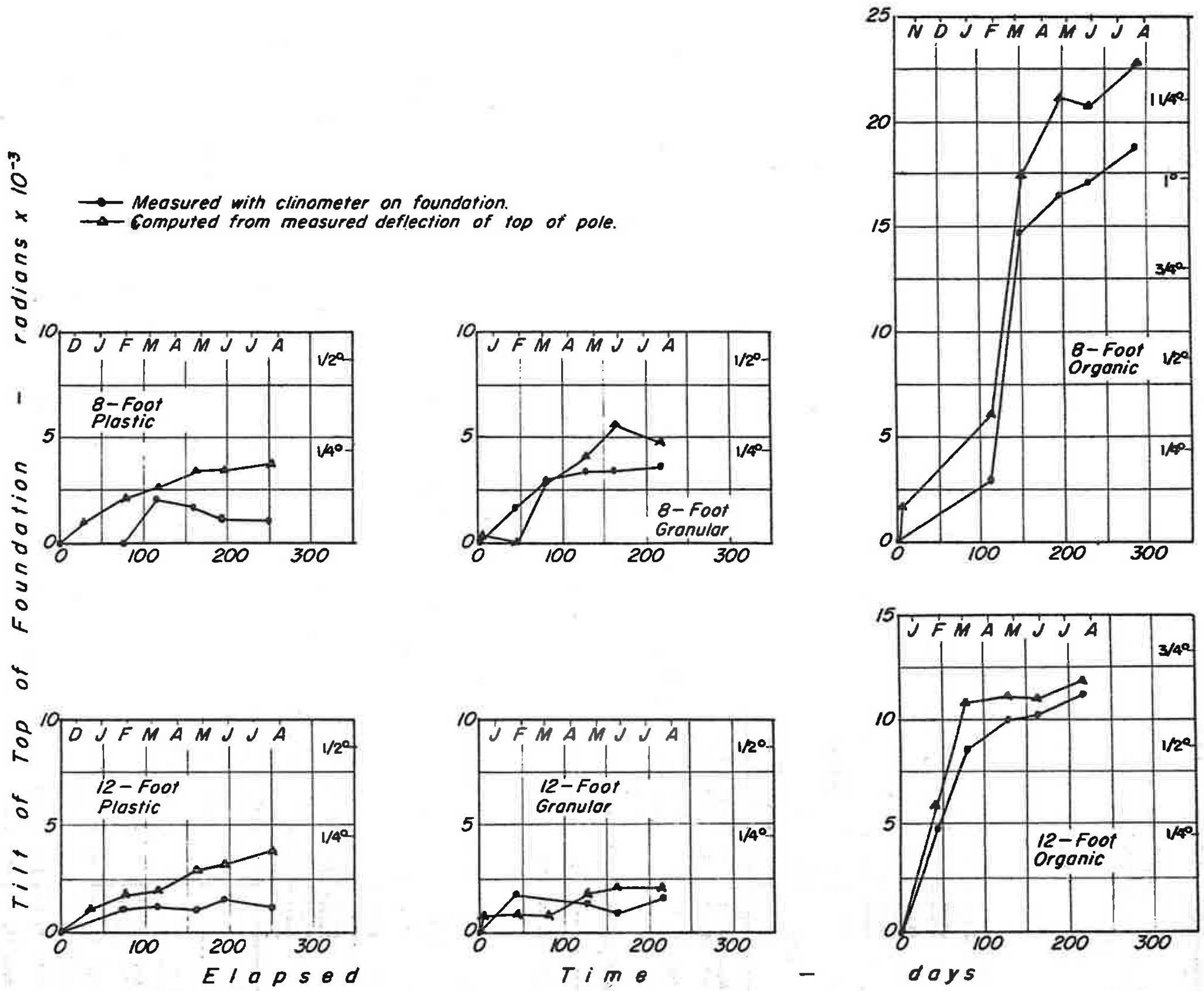

Figure 13. Tilt of foundations in long-term tests.

TABLE 13

LONG-TERM TEST DATA FOR 12-FOOT FOUNDATION IN ORGANIC SOIL

Depth of foundation: 12.0 ft
Weight of concrete cube: 2,220 lb
Horizontal component of load: 3,000 lb
Height of load: 24.2 ft
Overturning moment at groundline: 72,600 lb-ft
Depth of center of rotation: 6.0 ft
Moment arm to center of rotation: 30.2 ft

Mansfield

Date	E-lapsed Time	Height of Cube	Horiz. Move. of Top of Fdn.	Tilt of Foundation							Remarks
				By Clinometer on Foundation				By Movement of Top of Pole			
	days	ft	ft	Dial E in	Dial W in	Diff 2 in	Rotation radians	Transit Rdg. in	Horiz. Move. in	Rotation radians	
Dec. 17, 57	0	2.50	-	0.562	0.696	0.067	0.0000	93.19	0.00	0.0000	
Jan. 28, 58	42	2.04	-	0.534	0.762	0.114	0.0041	91.12	2.07	0.0057	
		2.50	-	0.531	0.762	0.116	0.0049	91.00	2.19	0.0060	
Mar. 6, 58	79	2.04	-	0.472	0.778	0.153	0.0086	89.25	3.94	0.0108	No load adjustment made.
Apr. 23, 58	127	2.04	-	0.193	0.522	0.164	0.0097	89.25	3.94	0.0108	
		2.48	-	0.192	0.526	0.167	0.0100	89.12	4.07	0.0112	
May 27, 58	161	2.40	-	0.199	0.536	0.168	0.0101	89.25	3.94	0.0108	Water on foundation
		2.50	-	0.197	0.535	0.169	0.0102	89.19	4.00	0.0110	
July 22, 58	217	2.37	-	0.185	0.540	0.178	0.0111	88.88	4.31	0.0118	
		2.50	-	0.181	0.537	0.178	0.0111	88.88	4.31	0.0118	

the same amount of tilt in a period of a year. The test results are given in Tables 8 to 13 inclusive. Tilt is plotted against time in Figure 13. Tilt as measured by means of the clinometer did not in every case agree with tilt as measured by deflection of the top of the pole. However, the angles measured are small and subject to some error of measurement. The results again show plastic and granular soils similar and organic soil with far less strength. It may be noted that during the test period, the greatest increases in tilt occurred during February and March.

SUMMARY

The cylindrical test foundations were very simple to construct and were very economical, yet they withstood with small angular deflection overturning moments of considerable magnitudes. The maximum loads applied were greatly in excess of design live wind loads for ordinary traffic sign structures. It appears that highway engineers who must design supports for very large traffic signs can learn something about foundations from the experience of the utility industry with deep, slender foundations.

Wind loads are inherently intermittent or transient in nature, hence it would seem that of the two series of tests, the short-term test results would be more appropriate for use in establishing design criteria for foundations. These results indicate that for a given angular deflection, slender and deep foundations will resist much greater short-term loads than long-term loads. This means that a sign structure with foundation designed for a reasonable wind load should successfully withstand considerable overloads due to relatively infrequent occurrences of high-velocity wind.

Of the three soil types tested, the plastic and granular soils demonstrated strengths which were very similar. Overturning moments as high as 300,000 lb-ft produced angular deflections less than $\frac{1}{2}$ deg. The maximum test loads applied were limited by capacity of the testing equipment and not by failure of the soil to resist overturning. The organic soil developed far less strength; even so the 12-ft foundation resisted a moment of 150,000 lb-ft at a deflection of $\frac{1}{2}$ deg. The data indicate that this type of foundation, constructed in undisturbed soil, should resist considerable overturning loads in what is normally considered a very poor soil for foundation purposes.

The effect of foundation depth is clearly evident in the slopes of the short-term load-deflection curves. The data indicate that resistance of a slender, deep foundation to overturning varies between the square and cube of depth.

The computed depth of center of rotation in several cases suggests that they did not rotate as rigid bodies, but that there may have been bending or beam action in these slender foundations. Their design must therefore take into account the bending moment expected and sufficient reinforcing steel must be provided to prevent failure in bending.

The 30-in. auger has been used extensively by the Ohio Department of Highways for excavating sign foundations, and has proven very satisfactory. It is fast, works well in most soils except in large boulders or loose sand and eliminates the need for concrete forms. Its use results in a concrete foundation supported by undisturbed soil, thereby obtaining the maximum possible soil strength.

The data presented here are hardly sufficient for a basis for establishing design criteria for foundations, but they should be useful to any engineer faced with the necessity of determining the size of a foundation for a sign. Further work should be done to develop a wider base of observed load test data, using embedded as well as anchor mounted poles, other soil types and foundations of other diameters and depths.

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A Device for Evaluating Horizontal Soil Resistance for Overhead Sign Supports

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● THE PROBLEM of estimating lateral strength of soils is complicated by such unpredictable factors as slope, adjacent disturbances, changing soil conditions, and even questions as to adequacy of established soil constants. Again, usually when a hole is to be dug for a foundation, the

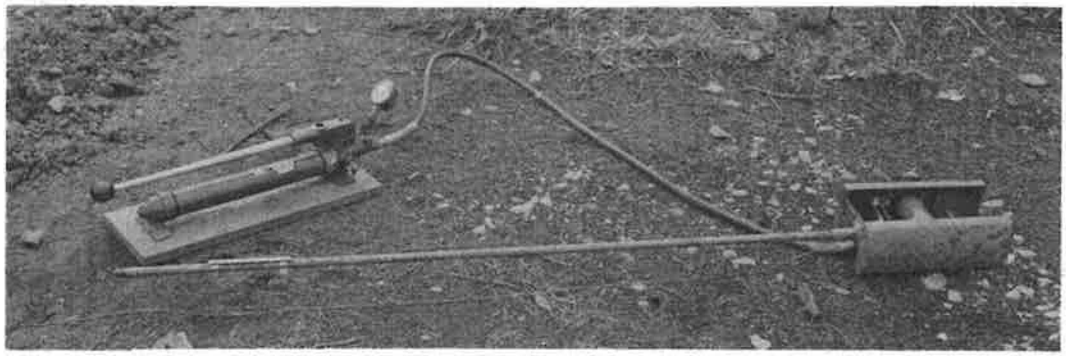

Figure 1. Device for evaluating horizontal soil resistance.

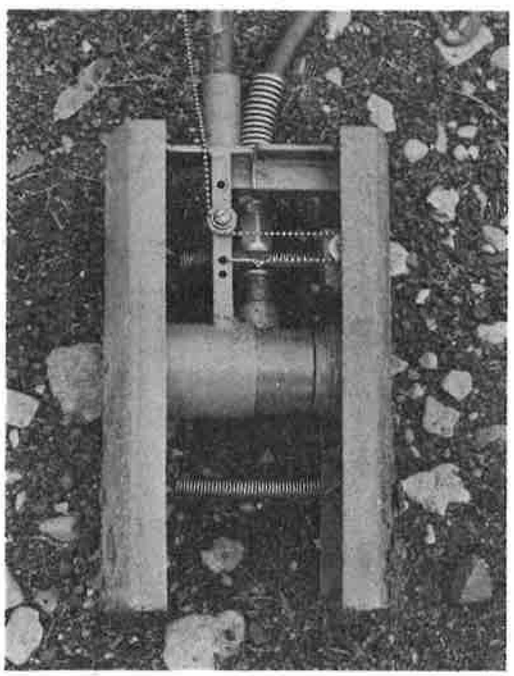

Figure 2. Detail of loading head, showing chain deflection gage.

answer is required immediately. Such values can be determined by a simple device (Fig. 1) consisting of two partial sections of a cylinder, each 12 in. long and 4 in. wide, with a 6-ton hydraulic jack between (Fig. 2). The jack is fed through a rubber hose connected to a pump above ground. The cylinder sections retract together by means of two springs. The jack is attached to $\frac{1}{2}$-in. pipe, which can be extended as needed in the field. An indicator consisting of a ball chain over two pulleys, which doubles the movement of the chain compared to the distance the cylinder sections move apart, is provided. Force is read with an oxygen-type gage attached to the high-pressure pump end of the hose (Fig. 3).

To use the device, an 8-in. diameter hole is bored to the required depth. The device is lowered to the test depth and an in-

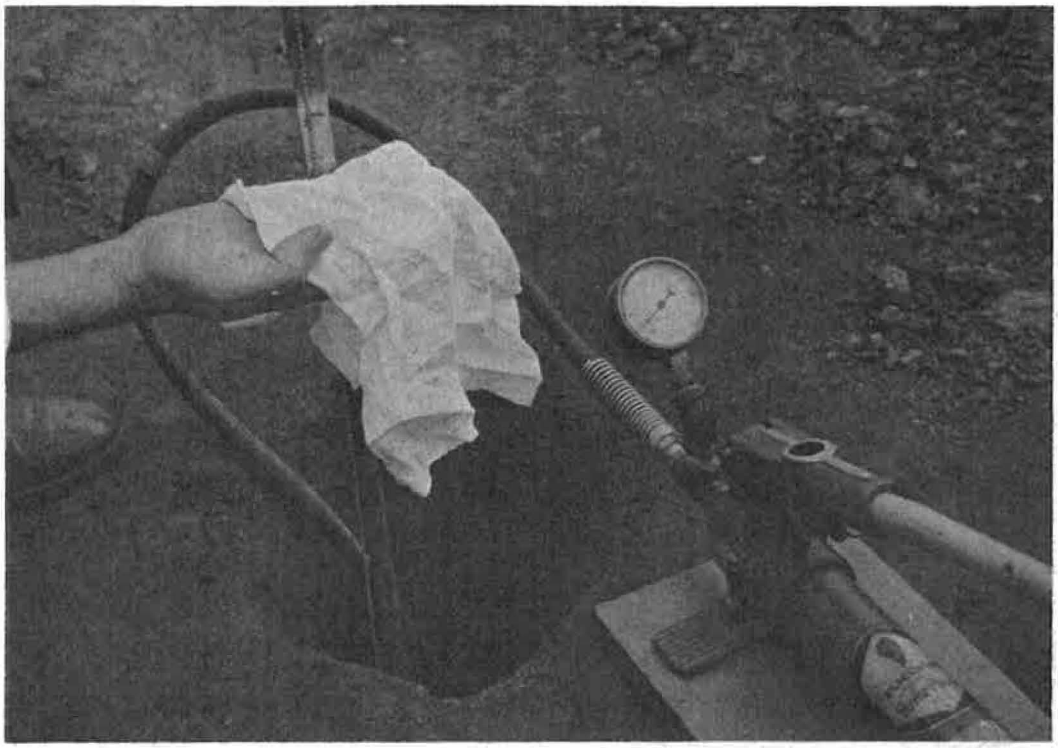

Figure 3. Device installed and under preload deflection.

itial side pressure, or preload, is applied. Then additional pressure is applied until the desired deflection is reached. The jack is then collapsed and the process repeated at different levels.

Specifically, the device has a projected area of 0.375 sq ft and a 1.48-to-1 pressure ratio factor. Thus, 1,000 psi equals 4,400 psf.

A preload of 100 psi gage or 440 psf, and a deflection of 3/4 in. total, or 3/8 in. on each half-cylinder, is suggested. This is multiplied by two at the scale and is fairly easy to read.

The device could be used by field crews who make the installations, simply referring to tables giving depth of foundation for various pressures read, perhaps modified for certain types of soils and a few notes covering adjacent slopes, drainage conditions, etc.

If sufficient interest in this device is evidenced, it probably can be made available, either through plans furnished to the highway departments, or through manufacturers, depending on the quantity required.