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Introduction

A literature review of foundation design methods for poles and towers was conducted in the Department of Civil Engineering for the Connecticut Department of Transportation in 1973. The final report summarized the approaches published to date and observed that most previous work showed theoretical solutions, but little field data. Design approaches appeared conservative (1) and there was little information showing the effect of foundation diameter on the resistance of the soil displacement under lateral load.

The present project was (74-1) originally intended for gathering field data by measuring the motion and rotation of span pole foundations as the span wire was tensioned. This appeared to be an attractive method since it fitted into a standard CONN DOT operation. However, only a few span poles were installed in the district around U Conn. during the Summer of 1974 and little data was obtained.

During the Fall of 1974 and Spring of 1975 the JNCAU authorized continuation of the field tests using salvaged steel poles directly embedded in soil. While the field program was being formulated contacts with Northeast Utilities indicated their interest in the same type of experimental program, especially the use of processed backfill to embed poles instead of more expensive concrete. A cooperative effort was developed for the field work with a program to evaluate the resistance to lateral loads of poles set in augered holes and surrounded by well graded backfill and the effect of pole diameter on strength and stiffness. The participation of Northeast Utilities through their own personnel and the efforts of the field crews of their affiliate, Connecticut Light and Power (CL & P) enhanced the field work.
Theoretical

1. General

Many articles have been published on both the displacement and ultimate resistance of poles to lateral loads. The list of references and bibliography at the end of the text contain the papers reviewed in connection with this, and the previous work (1). Most of the papers contain limited experimental data. The theories that showed the most promise for interpreting experimental results are those of Reese and Matlock (2)(3) and Broms (4)(5)(6). Reese and Matlock (2) derived theoretical relations for displacements of soil and piles based on the differential equations describing a beam against an elastic foundation. Broms developed expressions for the ultimate resistance of the soil and showed some applications with the ground line displacement equations. The work contained herein concerned the measurement of pole-soil interaction for directly embedded poles and evaluating the soil properties with the aid of the appropriate theory. Only the theory required for these tests is described herein and the reader is directed to the original papers for more detailed treatment.

2. Soil-Pole Interaction

The theory of Reese and Matlock (2)(3) indicates that poles can be categorized as rigid, intermediate and flexible. The difference among these categories depends on the relative stiffness of the pole and soil and the embedment depth. At shallow depth the pole behaves as a rigid member and the displacement of the pole from its original position depends on the stress strain properties of the soil alone. At sufficient depth of embedment the pole behaves as a flexible member in that the soil reaction against the lower portion pole is sufficient to create an inflection point on its elastic curve.
As the depth of embedment increases the displacement of the pole within the soil is less dependent on the soil stiffness. At intermediate depths of embedment no inflection point is developed in the pole and the stiffness of both the pole and the soil are important to the displacements. The majority of poles tested in this program were shallow enough to be considered rigid. Three of the tests were with poles buried deep enough to be considered intermediate. None were buried deep enough to develop an inflection point in its elastic curve.

To solve the theoretical equations some assumptions must be made as to the nature of the resistance supplied to the pole by the soil and the manner in which it varies with depth. The two most common cases treated are:

1. The modulus and resistance of the soil increase linearly with depth.

2. The modulus and resistance of the soil remains constant with depth.

Case 1 describes conditions often found in cohesionless soils and normally consolidated clays. Case 2 can be applied to over-consolidated clays.

3. Poles in Cohesionless Soil

The fundamental parameter used to describe the stress-strain properties of cohesionless soil is the coefficient of horizontal subgrade reaction k in lb/in. This parameter was originally defined by Terzaghi (7) and can be considered as the modulus of the soil at a depth of one foot. The modulus at any point equals k times depth. The strength of the soil is described by the angle of internal friction φ.

a. Ultimate resistance of the soil

The ultimate resistance of a soil to a laterally loaded pole can
be predicted by the expression: (5)

\[ P_{\text{max}} = \frac{0.5 \gamma B L^2 K}{(e+L)} \]

where: \( \gamma \) = unit weight of the soil capable of developing friction, below the ground water table; the buoyant unit weight should be used; \( B \) = diameter or width of the pole; \( L \) = embedded length of the pole; \( e \) = vertical moment arm from the ground surface to the point of load; \( K = \frac{L + \sin \phi}{P} \) and is known as the coefficient of passive resistance; \( \phi \) = angle of internal friction of the soil.

The dimensions in Eq. 1 are illustrated in Fig. 1. The maximum moment in the pole can be calculated by: (5)

\[ M_{\text{max}} = P_{\text{max}} \left[ e + 0.5 h \left( \frac{P}{\gamma B L} \right)^{1/2} \right] \]

b. Ground line displacements

The theory of Reese and Matlock was used to backfigure the coefficient of subgrade reactions for the soils surrounding the embedded poles. To distinguish among rigid, intermediate or flexible poles the criterion noted by Broms was used. (5) This criterion involves the dimensionless embedment length \( Z_{\text{max}} = nL \). Where \( n = \left( \frac{B}{E I} \right)^{1/5} \); \( E \) = Young's modulus of the pole and \( I \) = moment of inertia. The poles can then be classed as:

<table>
<thead>
<tr>
<th>( nL )</th>
<th>Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt;2.0</td>
<td>Rigid</td>
</tr>
<tr>
<td>2.0 to 4.0</td>
<td>Intermediate</td>
</tr>
<tr>
<td>&gt;4.0</td>
<td>Flexible</td>
</tr>
</tbody>
</table>

For a rigid pole the ground line displacement \( (\gamma_k) \) can be calculated or \( k \) can be backfigured from the equation:
Fig. 1  Illustration of pole dimensions used in the text
\[ y_0 = \frac{18F (1 + 1.33 \frac{\pi}{6})}{L^3 k} \]

For intermediate poles the equation derived by Matlock and Reese (3) must be used. This equation is:

\[ y_0 = \frac{P}{EI} \eta^3 A_y + \frac{P a}{EI} \eta^2 B_y \]

where: \( A_y \) and \( B_y \) are parameters dependent on \( Z_{\text{max}} = nL \). Substituting for \( \eta \) into equation 4 yields:

\[ y_0 = \frac{PL^3}{EI} \left[ \frac{A_y}{(Z_{\text{max}})^3} + \frac{B_y}{L(Z_{\text{max}})^2} \right] \]

The corresponding values for \( A_y \) and \( B_y \) were extracted from Reese and Matlock (3):

<table>
<thead>
<tr>
<th>( n_{\text{max}} )</th>
<th>( A_y )</th>
<th>( B_y )</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>4.75</td>
<td>3.40</td>
</tr>
<tr>
<td>3</td>
<td>2.70</td>
<td>1.77</td>
</tr>
<tr>
<td>4</td>
<td>2.46</td>
<td>1.63</td>
</tr>
<tr>
<td>5</td>
<td>2.40</td>
<td>1.63</td>
</tr>
</tbody>
</table>

Using the values in the table, values of \( \frac{y_0EI}{PL^3} \) can be plotted against \( Z_{\text{max}} \) for a family of curves, each curve representing one value of \( \frac{n}{L} \). A plot is shown in Fig. 2. The curves in Figure 2 can be used to backfigure \( k \) for intermediate poles. The field data yield the value on the vertical axis to enter the chart. From there proceed to the appropriate \( \frac{n}{L} \) curve then vertically to read \( Z_{\text{max}} \). Use \( Z_{\text{max}} \) to compute \( k \).

For flexible poles the ground line displacements can be calculated from:
Fig. 2  Chart for analyzing poles of intermediate stiffness in cohesionless soils.
\[ y_0 = \frac{0.03 P}{k^{1/5} (2\pi)^{2/5}} \]

4. Poles in Precompressed Clays

The properties in precompressed clays are assumed constant with depth. A modulus of subgrade reaction \( k \), in \( F/L^2 \) defines the stress-strain properties of the clay. The strength parameter used in determining the ultimate resistance of the soil to lateral load is the undrained strength, \( S_u \), also assumed constant with depth.

a. Ultimate resistance of the soil

The approach of Broms (4) assumes that the top of the soil to a depth of 1.5 \( d \) makes a negligible contribution to the ultimate resistance. The maximum resistance per foot of depth that the remaining depth can develop is \( 9 S_u \). The embedment depth \( L \) required to resist \( P_{\text{max}} \) can be determined by solving the following equations:

\[
L = 2x + f + 1.5B
\]

\[
x = f - \left[ \frac{e + 1.5B + \frac{1}{10} \frac{P_{\text{max}}}{S_uB}}{9S_uB} \right]
\]

\[
f = \frac{P_{\text{max}}}{9S_uB}
\]

The Chart by Broms (4) for solving Eq. 7, 8, and 9 is shown in Fig. 3.

The maximum moment that occurs in the pole under \( P_{\text{max}} \) is:

\[
M_{\text{max}} = \frac{P_{\text{max}}}{9S_uB} \left[ e + 1.5B + \frac{1}{10} \frac{P_{\text{max}}}{S_uB} \right]
\]

b. Ground line displacements

Poles in cohesive soils can also be categorized as rigid, intermediate or flexible depending on a dimensionless depth parameter. The depth
Fig. 3 Chart showing the embedment depth to attain a desired ultimate lateral resistance in precompressed clay. C. F. Broms (4)
parameter is \( SL \) where \( S = \left( \frac{K_o}{4 \pi \frac{L}{H}} \right)^{1/4} \). Where \( K_o \) = modulus of subgrade reaction in force/unit area. When \( SL \) is less than 1.5 the pole behaves as a rigid member and the ground line displacements can be computed from:

\[
y_o = \frac{4P (1 + 1.5 \frac{S}{L})}{K_o L}
\]

The equations for intermediate and flexible poles were not needed for this study and were therefore not worked out.

5. Non-homogeneous soil conditions

The equations presented above are for soils having a single \( k \) or a single \( K_o \). The soils within the top twenty (20) feet of a deposit will seldom be homogeneous enough to satisfy this condition. In most cases the soils could probably be treated as three distinct layers for computing ground line displacements. Three layer equations for rigid poles based on the approach of Matlock and Reese (2) have been derived and are shown in the appendix.

6. Discussion of Theoretical Assumptions

A basic assumption of the Reese & Matlock theory, as with many others, is that the soil behaves elastically. This results in workable equations but predicts the same deflection for all pole diameters. Another difficulty arises with poles placed in soft soil. It is common practice to excavate a hole much larger than the pole and replace the soft soil with a high grade sand and gravel. The theory does not indicate the amount of excavation and backfill required to achieve the required strength and stiffness.

The description of the variation of soil stiffness with depth appears to be reasonable using a coefficient of subgrade reaction \( k \) for cohesionless soils and a modulus of subgrade reaction for stiff clays. The modulus of subgrade reaction represents the stress-strain relation for the soil. In
cohesionless soil, the modulus of subgrade reaction at any depth can be obtained by multiplying the coefficient $k$ by the depth to the point of interest.

The theory accounts for both size of load and distance above the ground surface for application of the load.
Method of Testing

1. General

Six poles were used in the tests: three wood and three steel. Each pole was set and pulled several times. The description and properties of these poles are listed in Table I. The methods of installation and testing were essentially the same for all poles at all locations and are described below. A test consisted of applying a lateral load, 20 to 30 feet above the ground to a directly embedded pole. The load was applied in increments. After each load increment was applied, measurements of displacements and rotation were made. The loading was continued until the ground line displacements became excessive or the ultimate resistance of the soil was reached.

2. The Loading System

The load was applied to the pole through a cable looped around the pole and secured at the desired height. The load was developed by pulling the cable by means of a hand operated winch attached to a buried anchor. A dynamometer or load cell was connected between the winch and cable to measure the applied load. A schematic sketch of the method for loading the poles is shown in Fig. 4. Figure 5 shows a close-up of one of the arrangements of winches and load cell.

3. Setting the Poles and Anchors

The Connecticut Light and Power Co. (CL&P) supplied some logistical support for these tests and the services of a field crew to set, extract and move the poles. The field crew had a truck-mounted motorized auger (16 inches in diam.) and a small crane boom. This equipment is shown in Fig. 6.

At each site at least one anchor was installed. The anchor assembly consisted of a rod about 10 feet long and 5/8 inch in diameter having an
# TABLE I

Properties of Poles Used in Tests

<table>
<thead>
<tr>
<th>Pole No.</th>
<th>End Diam. (large)</th>
<th>End Diam. (small)</th>
<th>Length ft.</th>
<th>Approx. EI*</th>
<th>Material</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>ft.</td>
<td>ft.</td>
<td></td>
<td>ft²-lb x 10⁷</td>
<td></td>
</tr>
<tr>
<td>A</td>
<td>(29 cm.) 0.95</td>
<td>(22 cm.) 0.72</td>
<td>40</td>
<td>0.5</td>
<td>Wood</td>
</tr>
<tr>
<td><strong>B</strong></td>
<td>(37 cm.) 1.21</td>
<td>(20 cm.) 0.66</td>
<td>44.7</td>
<td>2.5</td>
<td>Wood</td>
</tr>
<tr>
<td>B'</td>
<td>1.24</td>
<td>0.82</td>
<td>48.8'</td>
<td>1.1</td>
<td>Wood</td>
</tr>
<tr>
<td>C</td>
<td>(46.5 cm) 1.53</td>
<td>(36 cm.) 1.18</td>
<td>39</td>
<td>3.0</td>
<td>Wood</td>
</tr>
<tr>
<td>D</td>
<td>1.25</td>
<td>1.00</td>
<td>30.25</td>
<td>1.7</td>
<td>Steel</td>
</tr>
<tr>
<td>E</td>
<td>1.25</td>
<td>1.00</td>
<td>30.25</td>
<td>1.7</td>
<td>Steel</td>
</tr>
<tr>
<td>F</td>
<td>1.0</td>
<td>0.75</td>
<td>21.94</td>
<td>1.5</td>
<td>Steel</td>
</tr>
</tbody>
</table>

* EI reported as approximate value at ground line when buried 6 ft.

** Pole B' broke during testing, B' substituted
Fig. 5  Load cell and four hand operated winches in parallel.
Fig. 6 Truck mounted motorized auger used in embedding poles
"eye" on one end and threaded length on the other end. A 1 foot square plate was fastened by nuts to the threaded end. The anchor was installed by augering a hole 8 feet deep, setting in the anchor assembly and backfilling with rocks and excavated soil.

The poles were set far enough away from the anchor so that the cable made a small angle with the horizontal. The distances between pole and anchor varied between 120 and 200 feet. A hole was augered as shown in Fig. 6 to the desired depth. The pole was set into the hole as shown in Fig. 7 using the crane boom. The pole was then plumbed and the hole around the pole was backfilled, tamping the backfill by hand in six inch lifts until the backfill was approximately level with the surrounding soil as shown in Fig. 8.

4. Measurements

At the first location (Mansfield Landfill) the poles were instrumented with a meter stick attached horizontally at four elevations above ground. Readings were made with a Wild T1 5 sec Theodolite set forty feet away from the pole and perpendicular to the line of pull. At each applied load the displacement and rotation of each meter stick was read and recorded. Analysis of the displacement and rotation at the various elevations allowed the stiffness of each pole as represented by the Young's Modulus (E) and moment of inertia (I) to be backfigured.

The displacement of the poles at the ground line was included in these measurements. The groundline displacements were used to backfigure the coefficient of horizontal subgrade reaction. Several poles were taken to the maximum load the soil could resist. This load was evidenced by the continual rotation of the pole as this load was attempted to be maintained.

When enough tests to determine the EI were made, use of the meter sticks and transit was discontinued and groundline displacements were measured with a dial gage. When using the dial gage a horizontal platform was strapped to the base of the pole and a tiltmeter, shown in Fig. 9, was set on the platform to measure pole rotation.
Fig. 7 A pole being set in augered hole

Fig. 8 Placing and compacting backfill
Tests in Dense Sand

1. Description of the Site

The first tests were made in sand because its engineering properties are most easily defined. Site investigations indicated that the area within the boundaries of the Town of Mansfield Landfill contains a deposit of dense, coarse to medium sand of sufficient depth for these tests. The actual location selected was to the east of the access road and south of the area actively used for sanitary landfilling in 1975. The area was flat. The layout for the tests is shown in Fig. 10. The ground water table is below the elevation of the bottom of the poles.

The firm of Clarence Welti Associates, Inc., from Glastonbury, Connecticut was hired to do machine borings, standard penetration tests and pressuremeter tests. The results of these are shown in the appendix. In-place density tests were also made. Samples of sand recovered from the in-place density tests were returned to the laboratory where tests to determine the particle size distribution and the maximum and minimum densities were conducted. A typical soil profile is shown in Fig. 11. Particle size distributions are shown in Fig. 12. All data indicate that the sand in this area is dense. Most of the area is covered by a stiff crust, probably compacted from truck traffic. Tests were run with this crust in place as well as with this crust stripped off.

2. The Test in Sand

A total of seventeen tests were conducted on the poles in the dense sand. Table II shows the schedule of tests with and without the stiff
Tight gravely medium fine sand with some cobbles and silt

Dense, poorly graded medium coarse sand with some fine sand and a trace of gravel. (variation to mostly fine sand noted.)

Tight sandy gravel

Dense medium coarse sand

Dense, uniform fine sand with some medium to coarse sand.

Dense, well graded sand and gravel. (Variation to poorly graded medium coarse sand with some fine sand noted)

O - Soil samples taken for laboratory analysis.

Fig. 11
* REFER TO FIG.11 FOR SAMPLE LOCATIONS

FIG. 12  PARTICLE SIZE DISTRIBUTIONS FROM SAMPLES COLLECTED FROM MANSFIELD LANDFILL
### TABLE II

Testing Schedule of Poles in Sand

(Tests with Stiff Crust in Place)

<table>
<thead>
<tr>
<th>Nominal Depth (ft)</th>
<th>A</th>
<th>B</th>
<th>C</th>
<th>D</th>
<th>E</th>
<th>F</th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td></td>
<td>x</td>
<td>x</td>
<td></td>
<td></td>
<td>x</td>
</tr>
<tr>
<td>6</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
<td>x</td>
</tr>
<tr>
<td>8</td>
<td></td>
<td>x</td>
<td>x</td>
<td>x</td>
<td></td>
<td></td>
</tr>
<tr>
<td>10</td>
<td></td>
<td>x</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

(Tests with Stiff Crust Removed)

| 4 |   | x | x | x |   | x |
crust. Each pole was embedded and tested several times. The sides of
the hole collapsed slightly during augering, resulting in a hole slightly
larger than the pole as shown in Fig. 13. The hole was cleaned of loose
sand before the pole was placed. The backfill material was the sand removed
by augering. The backfill was tamped in six inch lifts until firm enough
to cause the tamping tool to yield a ringing sound. The annular space around
the pole was too small to run a standard field density test. Proper backfill
and some simple compaction criterion is required for normal pole placement.

The tests listed in Table II were short duration tests. Each test was
conducted in a period of several hours. One long term test was conducted
for six months to determine if significant creep occurs in sand.

3. Results

a. Coefficient of horizontal subgrade reaction

The relative stiffness of the pole and soil as well as the embedment
depth determined the method of analysis to be used to determine k. The
stiffness of the poles was backfigured from the horizontal meter stick
readings using the method of moment area from strength of materials. (8)
The computed values of k are shown in Table I.

Each test yielded a load-ground line displacement curve. Examples of
these curves are shown in Fig. 14 and 15. The curves shown in Fig. 14 are
for test with the stiff crust in place. As can be seen from Fig. 14 the
ground line displacement under a given load decreases with increasing depth
of embedment. Figure 15 shows similar data in the area without the stiff
crust. The tests shown in Fig. 15 were conducted with poles of differing
diameters embedded approximately the same depth. The results shown in Fig. 15
indicate an increased stiffness with increasing diameter.
FIG. 13  Pole placed and ready for backfilling at Mansfield Landfill Site
Fig. 14  Load vs. Ground Line Displacement Curves for Poles in Area with Stiff Crust
Fig. 15  LOAD VS. GROUND LINE DISPLACEMENTS CURVES FOR POLES IN THE AREA WITHOUT THE STIFF CRUST
The greatest difference between the horizontal load and the cable load was 2%, due to the slope of the cable, but the accuracy of the readings was about ± 3%. No adjustment was therefore made to the cable readings in the computations. The values of the coefficient of subgrade reaction were backfigured using the cable loads and ground line displacements in the appropriate equations listed in the theoretical section. (Eq. 3 and Eq. 5.) Ground line displacements were approximately proportional to applied loads up to a displacement of about 0.5 in. All k were computed with \( y_0 = 0.5 \) inches. The results are shown in Table III. As can be seen from Table III the values of \( k \) with the crust in place tend to be greater than those without the crust. Also with the crust in place the apparent \( k \) tends to decrease with greater embedment. The stiff crust tends to reduce ground line displacements and is more effective at shallow embedment depths. The \( k \) determined by the pressuremeter tests ranged from 20 to 56 zips/ft\(^3\) for the sand below the stiff crust.

b. Ultimate Capacity

As the loads increased the load-displacement curve became non-linear. At a ground line displacement somewhat larger than three inches, in these tests, the soil reached its ultimate resistance, evident from the inability of the soil to resist any increase in load. As the pole rotated under this maximum load a radial crack pattern and heave first formed in front of the pole. As the rotation increased a similar pattern formed behind the pole.

The ultimate load carrying capacity of poles as governed by the soil is predicted by Eq. (1). Several poles were loaded to their ultimate load carrying capacity. The values of the coefficient of passive resistance \( K_p \) backfigured from the test data are shown in Table IV. As can be seen from Table IV the backfigures values of \( K_p \) are fairly consistent except for the two thinnest poles A and F.
### TABLE III

Backfigured Coefficient of Subgrade Reaction \((k)\) in Kips/ft\(^3\) Based on 0.5 Inch Ground Line Displacement

**Tests Without Stiff Crust (All Poles Buried \(\frac{4}{3}\) ft)**

<table>
<thead>
<tr>
<th>Pole No.</th>
<th>(F)</th>
<th>(A)</th>
<th>(D)</th>
<th>(C)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter (ft.)</td>
<td>0.83</td>
<td>0.96</td>
<td>1.25</td>
<td>1.50</td>
</tr>
<tr>
<td>(k) kips/ft(^3)</td>
<td>79.0</td>
<td>99.0</td>
<td>124.0</td>
<td>123.0</td>
</tr>
</tbody>
</table>

**Tests with Stiff Crust - Poles Behaved as Rigid Members**

<table>
<thead>
<tr>
<th>Pole No.</th>
<th>(A)</th>
<th>(B)</th>
<th>(B)</th>
<th>(C)</th>
<th>(D)</th>
<th>(E)</th>
<th>(F)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth of Embedment</td>
<td>4.25 ft.</td>
<td>4 ft.</td>
<td>5.83 ft.</td>
<td>5.83 ft.</td>
<td>8 ft.</td>
<td>6 ft.</td>
<td>4.5 ft.</td>
</tr>
<tr>
<td>(k) kips/ft(^3)</td>
<td>199.8</td>
<td>191.0</td>
<td>134.6</td>
<td>218</td>
<td>112.4</td>
<td>140.2</td>
<td>102.8</td>
</tr>
</tbody>
</table>

**Tests with Stiff Crust - Poles Behaved as Intermediate Members**

<table>
<thead>
<tr>
<th>Pole No.</th>
<th>(A)</th>
<th>(B)</th>
<th>(C)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth of Embedment ft.</td>
<td>6</td>
<td>8</td>
<td>8</td>
</tr>
<tr>
<td>(k) kips/ft(^3)</td>
<td>68.4</td>
<td>120.0</td>
<td>188.6</td>
</tr>
</tbody>
</table>
### TABLE IV

Summary of Backfigured Values of $K_p$

from Tests in Sand

<table>
<thead>
<tr>
<th>Pole</th>
<th>A</th>
<th>B</th>
<th>B</th>
<th>C</th>
<th>C</th>
<th>E</th>
<th>F</th>
<th>F</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth ft.</td>
<td>4.25</td>
<td>4.0</td>
<td>5.83</td>
<td>6.0</td>
<td>5.83</td>
<td>5.75</td>
<td>6</td>
<td>4.5</td>
</tr>
<tr>
<td>$K_p$</td>
<td>13.5</td>
<td>7.6</td>
<td>8.2</td>
<td>8.7</td>
<td>8.1</td>
<td>8.0</td>
<td>10.5</td>
<td>9.4</td>
</tr>
</tbody>
</table>
Tests in Glacial Till

1. Description of the Sites

The poles were tested at two sites where the soil is best described as a glacial till. Both sites are on the UConn Campus. One site located in a meadow north of the microwave towers is shown in Fig. 16 and will be referred to as MM site. The till in this site is known to be disturbed. The MM site was previously used for agricultural experiments of growing corn over a sanitary landfill that required removal and replacement of the soil to a considerable depth. Some organic soil content was noted at a depth of six feet. The soil in this area did not show the usual layering found in glacial till. The in-place density of the soil in this area was 98 lb/ft³. The water content was 15%. The ground water table in this area was below the bottom of the poles.

The other site is west of a pond behind "W" Parking Lot off Rte. 395 and is in an area where the soil is undisturbed. This site is shown in Fig. 17 and will be referred to as the Pond Site. There has been some construction close to the Pond Site. The soil profile as observed in the holes showed the usual thin lenses normally found in glacial till. The in-place density in this area was 112 lb/ft³. The water content was about 15%. The ground water table was near the bottom of the poles for most tests at the Pond Site. For one of the tests (Pole A) the ground water table was near the surface.

The data from both sites is presented together because they show similar effects. The schedule of poles tested at these sites is shown in Table V.
## TABLE V

Schedule of Poles Tested at MM and Pond Sites

### Poles Tested at MM Site

<table>
<thead>
<tr>
<th>Pole</th>
<th>B</th>
<th>C</th>
<th>D</th>
<th>E</th>
<th>F</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth ft.</td>
<td>7.7</td>
<td>5.54</td>
<td>4.33</td>
<td>6</td>
<td>6</td>
</tr>
</tbody>
</table>

### Poles Tested at Pond Site

<table>
<thead>
<tr>
<th>Pole</th>
<th>B</th>
<th>C</th>
<th>D</th>
<th>F</th>
<th>A</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth ft.</td>
<td>6</td>
<td>5.25</td>
<td>5.5</td>
<td>4.3</td>
<td>5.3</td>
</tr>
</tbody>
</table>
2. The Tests

The same type auger was used to prepare holes to the various depths in the glacial till. At the NW site one pole (B) was backfilled with the glacial till, the others were backfilled with sand from Mansfield Landfill. At the Pond Site the poles were backfilled with well graded crushed stone supplied by Northeast Utilities (NU). This backfill is the type normally used by NU for direct embedment. Northeast Utility specifications for the processed stone are shown in the appendix.

The augered holes in the till were smaller in diameter than those in the sand because the till has more apparent cohesion than the sand. Water was added to the backfill at the Pond Site to aid in compaction. The backfills at both sites were tamped by hand in six inch lifts until the tamper striking the backfill made a slight ringing sound.

At the NW site the anchor was embedded in higher ground than the poles. At the Pond Site two anchors were embedded so that two poles could be tested simultaneously.

During several pole installations at the Pond site a ground vibration, sensed by standing in the area, could be felt several feet away from the auger. Glacial till normally contains a number of cobble sized stones. This vibration was probably caused by the auger pushing the cobbles against the sides of the holes. Pole D at the Pond Site was placed in a hole augered in two-foot increments. After each two-foot penetration the auger was removed and cleaned, until the desired depth was reached. This was an attempt to decrease the disturbance to the surrounding soil due to augering.
The instrumentation for all but one of these tests consisted of a dial gage for measuring ground line displacements and the platform and rotation gage for measuring pole rotation at the ground line. Pole C at the MM site was instrumented with the meter sticks, dial gage and rotation gage. Pole C was tested first and a comparison between the dial gage and transit readings on the ground line meter stick were made. These values compared and since the elastic properties of each pole had already been determined the use of the meter sticks and transit was discontinued.

The first tests at the MM site showed a significant rate of strain under all loads. Each load increment at these sites was kept constant until rate of increase in ground line displacement decreased to approximately 0.0005 inches per minute or less. During two tests loads were held fairly constant on the pole for several days. One duration was 5 and the other was 7 days. For these loads a hanger with concrete blocks was placed on the cable to aid in keeping the load constant as the pole rotated. The variation in the cable load overnight was normally less than fifty pounds.

3. Results

The behavior of the poles at both the MM and Pond Sites was similar in that both showed a significant time-dependent behavior. At each load, ground line displacements increased with time. An example of these time dependent strains is shown in Fig. 18. A summary of strain rate behavior for all poles is plotted in Fig. 19. Although Fig. 19 shows some scatter, the behavior of the poles at the Pond Site is similar to the behavior at the MM site indicating that the auger causes disturbance to the surrounding soil, reducing its stiffness.
FIG. 18 INCREASE OF GROUND LINE DISPLACEMENTS WITH TIME FOR POLE B (MM SITE)

POLE B
MM SITE
L = 7.7'

225 lbs.

575

750

GROUND LINE DISPLACEMENT (in.)

1.2

0.17

0.02

0.03

0.04

0.12

0.13

0.18

0.19

1 2 3 4 5 6 7 8 9 10 20 30 40 60 80 100

TIME (min.)
Fig. 19  SUMMARY OF THE RATE OF INCREASE OF GROUND LINE DISPLACEMENTS AT VARIOUS LOADS (Meadow and Pond Sites)
The data was analyzed assuming that the subgrade reaction increases linearly with depth. Due to the time effect the coefficient of subgrade reaction was computed for instantaneous loading (i.e. w/o creep), instantaneous loading plus 1 year of creep and instantaneous loading plus 100 years of creep. For these computations it was assumed that the ground line displacements followed a linear log time plot. An example of the resulting curves is shown in Fig. 20. The complete results are shown in Table VI for the MM site and in Table VII for the Pond Site. As can be seen from these tables the values of k are approximately the same. The instantaneous values are of the same order of magnitude as the sand. One pole at the Pond Site (Pole A) could resist only a small load when the ground water table had reached a shallow depth.

The one difficulty with the approach of using 1 year and 100 year curves is allowing for the effect of rainfall. During several of the long term tests rain fell on the area. As the rain water percolated through the soil the rate of ground line displacement increased. After the rain ceased, the rate of displacement decreased below the rate before the rain. In devising the 100 year curve it was assumed that the strains would be the same over a long period whether or not they were temporarily accelerated by rainfall.
Fig. 20 Example graph of the ground line displacement curves calculated to include time effects
## TABLE VI

Summary of Data from MM Site
Backfigured Coefficients of Subgrade
Reactions k Based on Cohesionless Soils kips/ft^3
Based on 0.5 in Groundline Displacement

<table>
<thead>
<tr>
<th>Pole</th>
<th>B</th>
<th>C</th>
<th>D</th>
<th>E</th>
<th>F</th>
</tr>
</thead>
<tbody>
<tr>
<td>inst.</td>
<td>64</td>
<td>116</td>
<td>36</td>
<td>134</td>
<td>90</td>
</tr>
<tr>
<td>1 yr.</td>
<td>42</td>
<td>**</td>
<td>**</td>
<td>48</td>
<td>42</td>
</tr>
<tr>
<td>100 yr.</td>
<td>36</td>
<td>**</td>
<td>**</td>
<td>36</td>
<td>36</td>
</tr>
</tbody>
</table>

* Pole D showed a lower modulus but less creep
** Pole C was tested rapidly

Backfigured Values of $K_p$
from Loads Carried to Failure at MM

<table>
<thead>
<tr>
<th>Pole</th>
<th>D</th>
<th>E</th>
</tr>
</thead>
<tbody>
<tr>
<td>$K_p$</td>
<td>19</td>
<td>7</td>
</tr>
</tbody>
</table>
## TABLE VII

Summary of Data from Pond Site

Backfigured Coefficient of Subgrade

Reaction $k$ in kips/ft$^3$ Based on Cohesionless Soils Using

0.5 Inch Ground Line Displacement

<table>
<thead>
<tr>
<th>Pole</th>
<th>A</th>
<th>B</th>
<th>C</th>
<th>D</th>
<th>F</th>
</tr>
</thead>
<tbody>
<tr>
<td>inst.</td>
<td>*</td>
<td>122</td>
<td>266</td>
<td>114</td>
<td>120</td>
</tr>
<tr>
<td>1 yr.</td>
<td>--</td>
<td>82</td>
<td>132</td>
<td>64</td>
<td>68</td>
</tr>
<tr>
<td>100 yr.</td>
<td>--</td>
<td>78</td>
<td>136</td>
<td>48</td>
<td>60</td>
</tr>
</tbody>
</table>

* Pole A was tested after the water table rose to surface of the soil and showed no resistance to overturning. It could not sustain a load of 200 lb. @ 30 ft. off the ground.
Tests in Clayey Silt

1. Description of the Site

This site is located about 900 feet west of Route 5 and 300 feet south of the American Legion Post in Enfield, Connecticut. The area is shown schematically in Fig. 21. The soil in this area is a stiff clayey silt covered by 16 to 24 inches of miscellaneous fill and the area is flat. The ground water table was at the surface of the soil during the tests. The water content of the soil was 17%. The dry density was 116 lb/ft$^3$.

2. The Tests

Only two poles (S & E) were used at this site. The technique of augering was the same as previously described. The backfill was a bank run gravel containing a trace of silt. Pole S was embedded 7.5 feet and pulled at a height of 22.7 feet. Pole E was embedded 7 feet and pulled at a height of 20 feet. The poles were loaded from two anchors buried 140 feet to the west of the poles.

3. Results

The poles were loaded to about 400 lb each. The ground line displacements showed a time dependence. It was originally planned to have each pole sustain this load for about a month. However after the test was in progress for about five days, the site was vandalized. As a result the tests had to be terminated. The collected data were analyzed assuming that the coefficient of horizontal subgrade reaction is constant with depth (cohesive soil). The resulting coefficients are shown in Table VIII for instantaneous loading (w/o creep), instantaneous loading plus 1 year of creep, instantaneous loading plus 100 years of creep.

Pole E was loaded to failure, $P_{max} = 2500$ lb. Assuming the failure occurred in the soil backfigured $S_u = 2280$ lb/ft$^2$. 
## TABLE VIII

Modulus of Subgrade Reaction $K_0$ for Poles Buried in Clayey Silt in Enfield ($K_0$ assumed constant with depth)

<table>
<thead>
<tr>
<th>Pole</th>
<th>B</th>
<th>E</th>
</tr>
</thead>
<tbody>
<tr>
<td>Inst</td>
<td>44</td>
<td>74</td>
</tr>
<tr>
<td>1 yr.</td>
<td>21</td>
<td>20</td>
</tr>
<tr>
<td>100 yr.</td>
<td>16</td>
<td>18</td>
</tr>
</tbody>
</table>
Discussion of Field Test Results

1. General

The results of the field tests indicate that directly embedded poles are capable of developing the full resistance of the surrounding soil. There were some unexpected time effects from the glacial till. Undisturbed glacial till should be capable of developing as much resistance to strain as sands. The behavior of poles buried by different excavation techniques in glacial till requires further study.

a. Results of Analysis

The embedded poles used in this study were surrounded by only a few inches of compacted backfill. The testing program showed the directly embedded pole capable of developing the full resistance of the surrounding soil to displacement and failure. Values for the coefficient of subgrade reaction previously published for sands are shown in Table IX. The backfigured values from the tests at the Mansfield Landfill compare with the values for dense sand listed in Table IX. The values in Table IX are appropriate for a pole width of 1 foot displacing one inch.

The tests in glacial till showed the importance of placement methods. The undisturbed till at the Pond Site should be capable of developing as much resistance as dense sand. Since the poles at this site showed the same creep behavior as the NN site the augering must have disturbed the surrounding soil. Embedment of poles in glacial till by backhoe would probably lead to less disturbance and a more resilient soil surrounding the backfill and will probably show less creep. From these tests it appears that when designing a pole to be embedded in a soil subject to creep the short term coefficient of subgrade
**TABLE IX**

Coefficients of Horizontal Subgrade Reaction for Cohesionless Soils

<table>
<thead>
<tr>
<th>Blow Count per Foot Std. Pen. Test</th>
<th>Relative Density Designation</th>
<th>( k ) in Kips/ft(^3)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Tersaggi (7)</td>
</tr>
<tr>
<td>0-4</td>
<td>V. loose</td>
<td>---</td>
</tr>
<tr>
<td>7</td>
<td>Loose</td>
<td>14</td>
</tr>
<tr>
<td>20</td>
<td>Medium</td>
<td>42</td>
</tr>
<tr>
<td>40</td>
<td>Dense</td>
<td>112</td>
</tr>
<tr>
<td>60</td>
<td>V. Dense</td>
<td>---</td>
</tr>
<tr>
<td>80</td>
<td></td>
<td>---</td>
</tr>
</tbody>
</table>
reaction should be reduced by a factor of three.

The poles buried in the clayey silt also behaved as expected from the surrounding soil. The modulus of horizontal Subgrade Reaction backfigured from the field test agrees with previously published data shown in Table X. The instantaneous modulus was slightly lower than that shown in Table X for a soil having an $S_u$ of about 2 kips/ft².

Published "k" values for normally consolidated clay are shown in Table XI. The resistance of normally consolidated clay increases with depth so that the analyses for these soils is similar to cohesionless.

3. Width Effect in Sands

The derivation of the equations for ground-line displacement assumes a linear stress-strain relation for the soil. This model implies that making the pole wider will not reduce ground line displacements under a given load. Previous studies (12) (13) (14) indicate that enlarging or stiffening poles near the ground surface reduces the ground line displacements by 50%. The tests in sand reported herein indicate that the effect of widening the pole may be greater.

Data from the Mansfield Landfill Area show an interesting trend of "k" with width of loaded area. Fig. 22 shows the results from the pole tests w/o the stiff crust. Also included in this figure are the results of the pressure meter tests. As can be seen from Fig. 22 the backfigured coefficient of subgrade reaction increases with increasing width of loaded area from $\frac{1}{4}$ to $\frac{3}{4}$ inches. Data was also collected from other sources reporting similar behavior in sands. As a first approximation it appears reasonable when designing pole foundations several feet in diameter to increase the values shown in Table IX my multiplying by $\sqrt{B}$, where B is the diameter of the foundation in feet.
TABLE X

Modulus of Horizontal Subgrade Reaction for Precompressed Clays

<table>
<thead>
<tr>
<th>Slow Count per Foot Std. Pen. Test</th>
<th>Approx. $S_u$ (Kips/ft$^2$)</th>
<th>Designation</th>
<th>$K$ in Kips/ft$^2$</th>
<th>Remarks</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>8-15</td>
<td>1.2 - 2.0</td>
<td>Stiff</td>
<td>100</td>
<td></td>
<td></td>
</tr>
<tr>
<td>15-30</td>
<td>2.0 - 4.0</td>
<td>Very Stiff</td>
<td>200</td>
<td>Considered pre-Compressed</td>
<td>Davisson and Prakash (12)</td>
</tr>
<tr>
<td>&gt;30</td>
<td>74.0</td>
<td>Hard</td>
<td>400</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Blow Count per Foot Std. Pen. Test</td>
<td>Approx. $S_u$ Kips/ft$^2$</td>
<td>Designation</td>
<td>$k$ in Kips/ft$^3$</td>
<td>Remarks</td>
<td>Reference</td>
</tr>
<tr>
<td>-----------------------------------</td>
<td>--------------------------</td>
<td>-------------</td>
<td>------------------</td>
<td>---------</td>
<td>-----------</td>
</tr>
<tr>
<td>2-4</td>
<td>0.25 - 0.50</td>
<td>Soft</td>
<td>8</td>
<td>Considered Normally Consolidated</td>
<td>Singh et al (10)</td>
</tr>
<tr>
<td>4-8</td>
<td>0.5 - 1.0</td>
<td>Medium Stiff</td>
<td>16</td>
<td></td>
<td></td>
</tr>
<tr>
<td>---</td>
<td>---</td>
<td>Very Soft Organic Silt</td>
<td>0.8</td>
<td></td>
<td>Davisson and Prakash (12)</td>
</tr>
</tbody>
</table>
Fig. 22 Backfigured K values plotted against pole diameter from Mansfield landfill tests.
Model Tests in Sand

1. Description

The field tests indicated that the load-strain relation experienced by poles depends on the soil into which they are embedded and on the method of excavation. The characteristics of the soil surrounding the backfill to a thickness of three times the diameter appears to be most critical. (7) Direct embedment of a pole in an area of marginal soil requires either very deep embedment or removal of the marginal soil and replacement with a well compacted, well graded backfill. In areas containing poor soils, NU and other utilities usually excavate large holes, replacing the poor soil with compacted gravel on crushed stone.

Model tests were conducted in sand to study the behavior of poles surrounded by dense backfill in a loose sand.

Tests were conducted in a steel tank 4 ft on a side and 3 ft deep containing a medium size sand. Water could be made to flow upward through the sand. The sand was made loose by flowing the water fast enough to turn the sand quick then shutting off the flow. The sand was made dense by vibrating it with a concrete vibrator while keeping the sand saturated. All tests were done with the free water just at the top of the sand.

Pipes of various sizes were tested in the tank by pulling horizontally 37.5 inches above the top of the sand. Ground line displacements were measured with a dial gage.

2. The Tests

Several sizes of poles were first tested in loose then in dense sand. Horizontal load and ground line displacements were recorded as well as
strains in the surrounding soil. When these tests were complete a series
of tests were conducted in which the pipe was surrounded by a compacted well
graded backfill with the sand surrounding the backfill in a loose state.

3. Results

The tests with the compacted backfill surrounded by loose sand were
compared with the tests in which the pipe was surrounded by sand. The
results are shown in Fig. 23. All of the tests shown in Fig. 23 used a
pipe 1-5/8 inch in diameter embedded 8 inches. In two of the tests, the
compacted backfill was cylindrical in shape, 8 inches in diameter and
8 inches deep. In the other test the backfill was 8 inches in diameter
at the top and tapered to two inches in diameter at the base of the pipe.
As can be seen from Fig. 23 the pipe surrounded by compacted backfill is
always as stiff to lateral loads as dense sand. The results presented
in Fig. 23 also indicate that the ultimate strength should be at least
as great as dense sand alone. The lowest value involved the use of a
cone shaped backfill. In this case the resistance was approximately equal
to the dense sand alone. In each test using compacted backfill about 80% of
the strain occurred within the backfill. This phenomenon has also been
observed in field tests. (15)

These results lead to a tentative design criterion for directly embedding
poles in soft or weak soil. The backfill in these areas should be as large
in diameter as it is deep and the backfill surrounding the pole should have
a minimum horizontal thickness at the surface of three times the pole
diameter. These criteria should be checked with full scale tests in the
field. A pole directly embedded in a soft deposit on a tangent transmission
line could be pulled until the ground line displacement reached several
inches. After the test the pole could be plumbed and the backfill recompacted.
Fig. 23 Comparison of results from model tests in sand using various embedment techniques and materials.
The effect of width of pipe was also investigated in the model tests. In Fig. 24 the load required to cause a ground line displacement of 1 mm is plotted against the diameter of pipe. As can be seen from Fig. 24 the load required to cause this displacement increases with increasing pipe diameter.
Fig. 24  Plot of cable load at 1 mm G. L. displacement vs. diameter of pole from model tests in sand
Design Considerations

Design can be approached in many ways. The general aspects considered in design will be discussed here. The first criterion that must be satisfied is that the pole and foundation must have an adequate factor of safety with respect to the ultimate load. Failure can result if the applied loads exceed either the shear and moment capacity of the pole, or the ultimate resistance of the soil. Other criterion can and usually are imposed on the design such as limiting the ground line displacement or rotation of the pole.

Factors of safety can be applied to the loads or to the strength properties of pole and soil. Judgment will be required in selecting the appropriate soil properties. The factors of safety for the overall system should be evaluated so that it is about the same for all modes of failure. Suggested values for factor of safety are 1.5 for dead loads and 2.0 for live loads. (6) Limiting the displacements and rotation may of course increase the final factor of safety with respect to ultimate loads especially in granular soil.

Depending on soil type, the coefficient of subgrade reaction may be different for dead loads that will be applied constantly over the life of the structure and live loads due to wind which will be applied for short intervals. In the case of soils showing time dependent behavior ground line displacement and rotation should be computed on the basis of long term parameters for dead loads and on the basis of the estimated time of loading for the live loads.

The coefficient and modulus of subgrade reaction can be estimated from the standard penetration test as illustrated in Tables IX, X and XI. Pressuremeter tests are also excellent for estimating subgrade reaction parameter but are relatively expensive. The measured values of subgrade stiffness
should be corrected for size. An example of the relation between pressure-meter data and standard penetration tests for sands is shown in Fig. 25. Figure 25 was constructed from data available through this project and shows a good correlation.

In the case of directly embedding a pole in soft soil, the following procedure can be used. Design the pole for ground line displacements and rotation based on dense sand. After the embedment depth is determined call for an excavation with a width equal to the depth. Check ultimate resistance based on the strength of the average resistance between dense sand and the weaker soil. In the weaker soil bearing may also be a problem. Supply sufficient depth below the base of the pole to reduce the stresses on the soft soil. The sides of the excavation may be vertical as shown in Fig. 26. They can also be sloped as shown in Fig. 27. The backfill should be well graded to insure interlocking during compaction. Suggested gradation limits are shown in Fig. 28. The limits shown in Fig. 28 are more liberal than that of Northeast Utilities. All backfill should be compacted to about 95% of maximum dry density determined by the standard proctor method. (16)
Fig. 25  Plot of subgrade modulus against standard penetration test results at depth of 8 - 10 feet
FIG. 26  BACKFILL CONFIGURATION FOR EMBEIDMENT IN POOR SOIL
Fig. 27 ALTERNATE BACKFILL CONFIGURATION FOR EMBEDMENT IN POOR SOIL.
Design Example

Ground Line Moment = 780 kip-ft

Lateral Force = 12 kips P.S. = 2. \( P_{\text{max}} = 2 \times 12 \text{ kips} = 24 \text{ kips} \)

Avg. Blow Count "H" = 20 (Cohesionless Soil)

Ground Water Table at Surface

Allowable Ground Line Displacement = 5 inches = 0.42 ft.

Assumed Width of Base in Soil = 3 ft.

from Blow Count

a.) est. \( \phi = 33^\circ \)

b.) est. \( k = 36 \text{ kips/ft}^3 \) (see Table IX)

c.) est. \( \gamma_0 = 65 \text{ lb/ft}^3 \) (submerged)

Req'd: Depth of Embedment "L"

\[
e = \frac{780}{12} = 65 \text{ ft.}
\]

\[
\gamma_0 = \frac{18 P (1 + 1.33 \left( \frac{9}{12} \right))}{kL^2}
\]

modify \( k \) for width and design displacement

(see appendix)

\[ k_d = 36 \text{ kips/ft}^3 \sqrt{\frac{1}{3} \cdot \frac{1}{L}} \]

= 28 kips/ft^3

Check Ultimate (Eq. 1)  Try \( L = 12 \) ft.

\[ P_{\text{max}} = 0.5 \times 65 \text{ lb/ft}^3 \times 3 \text{ ft} \times (12^3) \frac{3.39}{(65 + 12) \times 1000} \]

= 7.4 kips too small

Try 18 ft. \( P_{\text{max}} = 23.2 \text{ kips} \)

by interpolation need 18.5 ft.

20 ft. \( P_{\text{max}} = 31.1 \text{ kips} \)

Check displacement
Actual $y_o = \frac{18(12)(1+1.33 - 69)}{28 (18.5)^2} - \frac{18.5}{28 (18.5)^2}$

$= 0.13 \text{ ft} = 1.5 \text{ inches}$

Note: in this case the ultimate load controls
Conclusions

1. Directly embedding poles is, in many cases, an economical method of setting the poles to resist lateral forces.

2. Behavior of directly embedded poles depends on the stress-strain characteristics of the soil surrounding the pole to a distance of approximately three pole diameters.

3. The stiffness of glacial till can be reduced by the method of excavating the hole to receive the pole.

4. The stress-strain properties of clays and disturbed glacial till are time dependent which must be considered in the design.
Recommendations

1. Backfill should be selected according to Fig. 28.

2. The excavation to receive the pole in good soil should allow a minimum clear space around the pole approximately twice the width of the tamping tool.

3. In glacial till the excavation should be made with a backhoe since auguring appears to disturb the surrounding soil.

4. Full scale tests should be conducted on poles directly embedded in soft or loose soil using the enlarged backfill technique shown in Fig. 26 and 27.

5. More tests should be conducted on glacial tills in both the natural state and in compacted fills.
References


Bibliography

Additional Articles Reviewed


Meldiv, I. M., "Finding Depth of Footing for a Pole Subject to Lateral Load," Civil Engineering, American Society of Civil Engineers, March 1958, pg. 66.

Patterson, D., "Pole Embedment to Resist Lateral Load," Civil Engineering, American Society of Civil Engineers, July 1958, pg. 69.

Reed, Paul, "Embedment Depth for Poles II," Civil Engineering, American Society of Civil Engineers, March 1975, pg. 77.
Robbins, W. G., "Piers Supported by Passive Earth Pressure," Civil Engineering, American Society of Civil Engineers, April 1957.


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"X" BR. FINE-CRS. SAND & GRAVEL

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**Fig. 29** Boring logs from Mansfield Landfill Area
Fig. 30  Example of pressuremeter data from Mansfield Landfill Site

\[ M_v = 2.5 \frac{(AP)}{\Delta V/V} \]

Balloon Pressure (T.S.F.)

Total Volume (C.C.)

2800  3000  3200  3400  3600
## Northeast Utilities Specification on Processed Backfill

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Derivation of Ground Line Displacement

Equations for Rigid Pole in Three Layer System

The equations of equilibrium for a pole under a lateral load can be written:

\[
P = \gamma_0 \int_0^L E_s \, dx + S \int_0^L x E_s \, dx
\]

\[
M = -\gamma_0 \int_0^L x E_s \, dx - S \int_0^L x^2 E_s \, dx
\]

where: \( P \) = lateral load; \( M \) = ground line moment; \( \gamma_0 \) = ground line displacement; \( S \) = slope of the rigid pole; \( x \) = vertical coordinate; \( E_s \) = modulus at any depth.

Sketches

**Cohesionless Case**

**Cohesive Case**
For cohesionless soil $E_a = K \eta$ (3) assuming $\eta = 1$

where $E_a$ is the modulus of subgrade reaction, $K$ is the coefficient of subgrade reaction.

For 3 layered system as shown above

\[
F_t = \gamma_t \left\{ \int_0^{L_1} k_1 x \, dx + \int_{L_1}^{L_2} (k_1 L_1 + k_2 x) \, dx + \int_{L_2}^{L_3} [k_1 L_1 + k_2 (L_2 - L_1) + k_3 x] \, dx \right\} +
\]

\[
S = \gamma_t \left\{ \int_0^{L_1} k_1 x^2 \, dx + \int_{L_1}^{L_2} (k_1 L_1 + k_2 x) x \, dx + \int_{L_2}^{L_3} [k_1 L_1 + k_2 (L_2 - L_1) + k_3 x] x \, dx \right\}
\]

\[
M = -\gamma_t \left\{ \int_0^{L_1} k_1 x^2 \, dx + \int_{L_1}^{L_2} (k_1 L_1 + k_2 x) x \, dx + \int_{L_2}^{L_3} [k_1 L_1 + k_2 (L_2 - L_1) + k_3 x] x \, dx \right\} +
\]

\[
- \gamma_t \left\{ \int_0^{L_1} k_1 x^2 \, dx + \int_{L_1}^{L_2} (k_1 L_1 + k_2 x) x^2 \, dx + \int_{L_2}^{L_3} [k_1 L_1 + k_2 (L_2 - L_1) + k_3 x] x^2 \, dx \right\}
\]

\[
P_t = \gamma_t k_1 L_1^2 A + Sk_1 L_1^2 B
\]

\[
M_t = -\gamma_t k_1 L_1^3 B - Sk_1 L_1^4 C
\]

Solving for $S$ and equating

\[
P_t L_1^2 C + M_t B = \gamma_t k_1 L_1^3 (AC-B^2)
\]
where (for 3 layers)

\[ A = \frac{\beta_3}{2} - \frac{1}{2} + a_2 \left( \frac{\beta_2 - 1}{2} \right) + a_3 \left( \frac{\beta_3^2 - \beta_2^2}{2} \right) \]  

\[ B = \frac{\beta_2^2}{2} - \frac{1}{6} + a_2 \left( \frac{\beta_2^2 - 1}{3} \right) + a_3 \left( \frac{\beta_2^3 - \beta_3^3}{3} \right) \]  

\[ C = \frac{1}{4} + \left( \frac{\beta_1^2 - 1}{3} \right) + a_2 \left( \frac{\beta_1^2 - \beta_2}{3} \right) + a_3 \left( \frac{\beta_1^3 - \beta_2^3}{2} \right) \]  

\[ a_3 \left( \frac{\beta_2^3 - \beta_2^4}{4} \right) \]  

where \( \beta_2 = \frac{L_2}{L_1}, \beta_3 = \frac{L_3}{L_1}, a_2 = \frac{k_2}{k_1}, a_3 = \frac{k_3}{k_1} \)

for 2 layers \( (\beta_2 = \beta_3, a_2 = a_3) \)

\[ A = \frac{1}{2} + (\beta_2 - 1) + a_2 \left( \frac{\beta_2^2 - 1}{2} \right) \]  

\[ B = \frac{1}{3} + \frac{\beta_2^2 - 1}{2} + a_2 \left( \frac{\beta_3 - 1}{3} \right) \]  

\[ C = \frac{1}{4} + \frac{\beta_2^2 - 1}{3} + a_2 \left( \frac{\beta_2^3 - 1}{4} \right) \]
Cohesive Case.

Substituting the values for $R_s$ as shown above in Figure (31) into equations (1) and (2) and integrating one obtains

$$P_t = Y_t \left( K_1 l_1 + K_2 (l_2 - l_1) + K_3 (l_3 - l_2) \right) - S \left( \frac{K_1 L_t^2}{2} + \frac{K_2}{2} (L_2^2 - L_1^2) + \frac{K_3}{2} (L_3^2 - L_2^2) \right)$$

and

$$M_t = -Y_t \left( \frac{K_1 L_t^2}{2} + \frac{K_2}{2} (L_2^2 - L_1^2) + K_3 (L_3^2 - L_2^2) \right) - S \left( \frac{K_1 L_t^2}{3} + \frac{K_2}{3} (L_2^3 - L_1^3) + \frac{K_3}{3} (L_3^3 - L_2^3) \right)$$

Solving for $S$ and equating

$$2 \gamma_t L_t C + 3M_t \beta = Y_t k_1 L_t^2 \left( 2 AC - \frac{3}{2} \beta^2 \right)$$

where

$$A = 1 + \lambda_2 (\beta_2 - 1) + \lambda_3 (\beta_3 - \beta_2)$$

$$B = 1 + \lambda_2 (\beta_2^2 - 1) + \lambda_3 (\beta_3^2 - \beta_2^2)$$

$$C = 1 + \lambda_2 (\beta_2^3 - 1) + \lambda_3 (\beta_3^3 - \beta_2^3)$$

$$\beta$$ is defined as before $\lambda_4 = \frac{K_4}{K_1}$
Pole B at Mansfield Landfill

Ground Line Deflection for 1 kp load @ 30'

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**Subgrade Reaction $\frac{kips}{ft^2}$**

Case 1: $k=80$, $N=30$

Case 2: $k=62$, $N=24$
Suggested corrections for $k$ and $K$

\[ k_d = k_r \sqrt{\frac{y_r}{y_d}} \cdot \sqrt{\frac{B}{1}} \]

where $k_r$ = reference coefficient of subgrade reaction for pole 1 foot in diameter.

$k_d$ = design $D_0$

$y_r$ = ground line displacement at which $k_r$ was determined

$y_d$ = design ground line displacement

$B$ = diameter of pole foundation in feet where 1 ft $\leq B \leq 8$ ft.