Final Report

Foundation Design Methods for Poles and Towers

by

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Introduction

The purpose of this project was to examine all the available information on the foundation design methods used for poles and towers and to make recommendations to the Connecticut Department of Transportation.

The published literature was reviewed and several utilities and an oil company were contacted for information.

The conclusions and recommendations of the authors are presented with design examples.

Background

Figures 1 through 5 illustrate the various types of foundations that are used to support towers and poles. The design methods for the foundations shown in Figures 1 through 4 have been presented in several texts on foundation engineering (9,10,11) and the reader is referred to these books for more detail. The foundation type shown in Figure 5 is widely used for the support of highway signs and lighting, but the design methods are largely emperical and sometimes inefficient.

The major effort of this project was spent in evaluating reported field data and theories concerning the behavior of rigid poles embedded in soil.

POLE FOUNDATIONS

The foundation for poles must be simple, inexpensive and adaptable to a wide range of loads and soil characteristics. The primary forces are applied relatively high up on the poles and the pole and foundation must be designed to resist large moments. In order to keep deflection within tolerable limits, the pole and its foundation structure must be stiff and can be considered rigid relative to the soil.

A common type of foundation for this type of structure is made by augering a hole 1-3 feet in diameter into the ground and filling the hole with concrete, reinforcing, and tie-down bolts that permit the attachment of a steel pole. Designing this type of foundation consists of determining the diameter, the depth of the hole and the reinforcing required.

Design Criteria

F. E. Behn (1) describes several full scale tests on steel poles supported by augured in concrete piers 30-36 inches in diameter. Typical plots of applied moment versus augular deflection for two of Behn's tests have been reproduced in Figure 6. These plots show that the soil has considerable reserve strength at deflections well above those that can be tolerated in the field. This is a very important characteristic of the behavior of rigid poles embedded in soil. As the applied moment increases, a greater amount of soil reaches its ultimate strength with a resulting increase of deflection. Abrupt failure does not occur even at very large deflections if the embedment to diameter ratio is more than five. A very good discussion of this phenomenon is given by Davison and Prakash (2). This behavior dictates that any design criteria be based on allowable deflections not the ultimate load-carrying capacity.

Design Methods

There are two general methods of designing pole foundations for limited deflection. The first uses the coefficient of horizontal subgrade reaction as described by Terzaghi (8) and elastic analysis to predict the deflection (6,2). The second assumes a pseudo-elastic stress distribution (3,5,4) and limits the maximum stress in the soil to values that have given tolerable deflections in the field. This maximum soil stress is based either on the soil type and its consistency or the unconfined compression strength for clays and the friction angle for sands. Both methods assume rigid rotation of the pile and the resulting stress distribution in the soil as shown in Figure 7. The point of rotation and depth to maximum stress are dependent on the soil characteristics and the relative values of P, e, D, and B.

EVALUATION OF DESIGN METHODS

Coefficient of Horizontal Subgrade Reaction

Of the two techniques reviewed that use the coefficient of horizontal subgrade reaction, only Broms' (6) method is simple enough to make it of practical use. Several typical situations of soil and loading were assumed. The deflections calculated by Broms' method were considerably more than deflections measured in the field according to data given by Broms (6) and Behn (1). The main source of error lies in the assumed coefficient of horizontal subgrade reaction. In addition this method shows no increase in stiffness with increasing pile diameter, a fact that is at odds with field observations by Anderson (7) and Kinney (4). These two problems led the authors to discard the horizontal subgrade reaction method as too cumbersome and conservative for general use.

Maximum Stress Method

This second method is used by Czerniak (3), Ivey and Hawkins (5), Rutledge, as discussed by Kinney (4), and also in an alternate method by Broms (6). This method assumes that a large amount of deflection has occurred when the stress in the soil in front of the pier exceeds the passive stress. In order to limit deflections of the pier under working loads, the maximum stress in the soil must not exceed some fraction of the passive stress. Based on field observations, Kinney (4), Ivey and Hawkins (5) and Czerniak (3) indicate that for piers one to three feet in diameter, a maximum stress of from one-third to one-fifth of the conventional Rankine passive stress for walls will not cause "excessive" deflection of the piers. For piers whose depths are more than about twice the diameter, Broms states (6) that the ultimate passive stress is approximately three times that for the case of a long wall.

Several typical situations of soil and loading were assumed and the depth of foundation required to limit deflection at the ground surface to 1/2" was calculated using the maximum stress methods (3,4,5,6). The results were compared to the deflection under similar situations reported by Behn (1) and Broms (6). These comparisons indicated that both Broms' and Rutledge's maximum stress methods gave reasonable results, whereas the other maximum stress methods were very conservative. All these methods are simple to use.

For cohesionless soil, Broms gives the following equation for the ultimate lateral load capacity:

$$P_{ult} = \frac{D^3 \, 1/2 \, \gamma_e B \, K_p}{e + D}$$

where D is the depth of embedment, γ_e is the effective unit weight of the soil, B is the diameter of the pier, e is the height above ground surface that the load P is applied and $K_p = \frac{1 + \sin \phi}{1 - \sin \phi}$ where ϕ is the friction angle of the soil. Broms also states that the pier rotation under this loading will be from 0.002 to 0.006 radians. If the pier is nine feet deep, this represents a movement at the ground surface of approximately 0.2 to 0.6 inches.

Rutledge's method is represented by the equation:

$$P_{all} = \frac{S_1\ BD^2}{2.4D + 2.6e}$$

where P_{all} is that load that will cause a pier movement at ground surface of 1/2", and S_1 is the lateral stress on the soil at a depth of approximately 1/4D. The maximum allowable stress S_1, that can be permitted, is a function of soil, and ranges from about 900 psf for very soft soil to 4500 psf for very hard soil. This method is applicable to both cohesionless and cohesive soils.

Broms' and Rutledge's methods yield similar results, if in Rutledge's equation it is assumed that S_1 is equal to $1/4\ D \times \gamma_e K_p \times 3$. The factor 3 is due to the fact that the ultimate passive stress against a relatively narrow pier is approximately three times the Rankine wall passive stress.

The main discrepancy between the two methods is that Broms states that at rotations of 0.002 to 0.006 radians the ultimate load capacity has been reached, whereas Rutledge feels that there is considerable additional load-carrying capacity at this deflection.

The latter opinion is supported by Behn's field tests and 25 years experience with this design method by the billboard industry.

SUGGESTED DESIGN PROCEDURE

Based on review and analysis of the existing literature, the authors suggest that the following procedure be used in the design of foundation piers for highway sign poles.

The maximum stress method of Broms or Rutledge is recommended as the best design method with modification of the allowable maximum soil stress.

In the case of cohesionless soils, the equation for the required depth of embedment takes the form of:

$$D_{req}^3 = \frac{P(D_{req} + e)}{\gamma_e B\, K_p/4}$$

For cohesive soils the equation takes the form of:

$$D_{req}^2 = \frac{P(2.4\, D_{req} + 2.6e)}{S_2\, B}$$

where S_2 is the maximum allowable soil stress and should be one-half of the stress S_1 recommended by Rutledge in order to limit deflections of the pier to 0.25" or less. S_2 will vary from 450 psf for soft soil to 2250 psf for dense glacial till. For clay, if the undrained shear strength (S_u) is known, the maximum allowable stress S_2 should be equal to 2 S_u. This is approximately one-fourth of the yield stress and limits the deflection at ground surface to approximately 0.25". It should be recognized that the stiffness and strength of the pier varies approximately as the square of the embedment depth, and where doubt exists as to the quality of the soil, the strength and stiffness can be increased considerably by increasing the depth by a few feet.

It is also important to recognize that the method of construction will affect the behavior considerably, and care must be taken to reduce disturbance of the soil adjacent to the pier or to compact the soil properly if it is disturbed.

In order to calculate the required amount of reinforcement in concrete piers, it can be assumed that the maximum moment in the pier will not exceed 150% of the moment applied at the ground surface.

SUMMARY AND CONCLUSIONS

After considerable review, both Rutledge's and Broms' maximum stress methods, with some modification, are recommended as the simplest and most accurate methods for determining the required depth of embedment for pole foundations. Very few actual full scale load tests have been reported in the literature so that some uncertainty still exists concerning the stiffness of embedded pole foundations. If additional load tests become available, these recommendations should be updated.

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APPENDIX

Design Examples

1. Given

$P = 2500$ lb
$e = 20$ ft
$B = 3$ ft

water table @ 20 ft
dense sand $\phi = 37^{\circ}$ $K_p = 4$ $\gamma_3 = 130$ pcf

$$D^3_{req} = \frac{P(e + D_{req})}{\gamma_e \, BK_p/4}$$

try D = 6 ft

$$D^3 = \frac{2500\ (20 + 6)}{130 \times 3 \times 4/4} = 167$$

D = 5.5 ft

try D = 5.4 ft

$$D^3 = \frac{2500\ (20 + 5.4)}{130 \times 3 \times 4/4} = 163$$

$D_{req} = 5.5$ ft

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2. Given

$P = 3000$ lb
$e = 30$ ft
$B = 3$ ft

clay Su = 500 psf
use $S_2 = 1000$ psf

$$D^2_{req} = \frac{P\ (2.4\ D_{req} + 2.6\ e)}{S_2\ B}$$

try D = 8 ft

$$D^2 = \frac{3000\ (2.4 \times 8 + 2.6 \times 30)}{1000 \times 3} = 97.3$$

D = 9.8 ft

try D = 10 ft

$$D^2 = \frac{3000\ (24 + 78)}{1000 \times 3} = 102$$

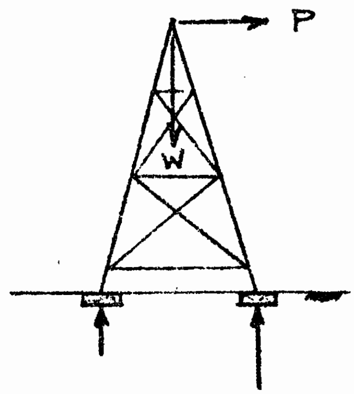
$D_{req} = 10$ ft

FIGURE 1

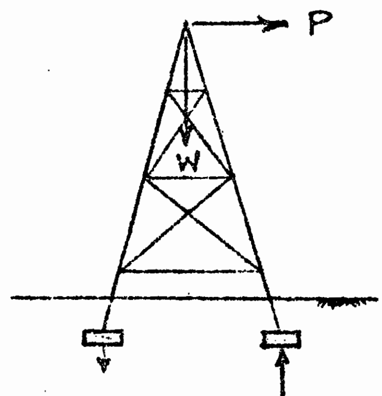
TOWER ON SPREAD FOOTINGS :

CASE 1

IN THIS DESIGN THE WEIGHT OF THE TOWER IS SUFFICIENT TO OFFSET ANY UPLIFT PRODUCED BY THE LATERAL FORCE.



CASE 2 IN THIS DESIGN THE WEIGHT OF THE TOWER CANNOT COUNTERACT THE UPLIFT PRODUCED BY THE LATERAL FORCE. THEREFORE THE WEIGHT OF THE FOOTING MUST BE INCREASED AND/OR BURIED TO PROVIDE SUFFICIENT PROTECTION AGAINST UPLIFT.



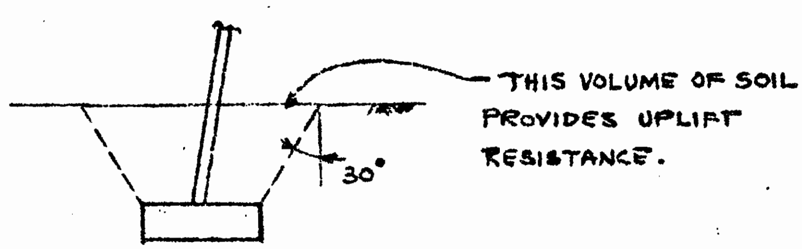


FIGURE 2

GUYED MAST :

THIS DESIGN METHOD RELIES ON THE WEIGHT OF THE ANCHOR AND SOIL RESISTANCE TO OFFSET ANY TENSION PRODUCED IN THE CABLE BY THE LATERAL FORCE.

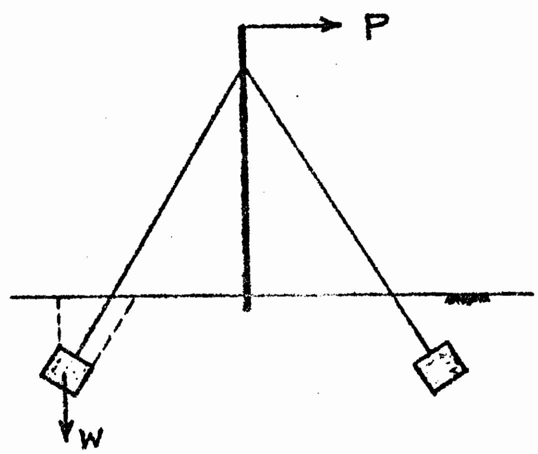


FIGURE 3

TOWER ON PILES :

THIS METHOD UTILIZES THE CAPABILITIES OF THE PILES TO RESIST ANY DOWNWARD AND UPLIFT FORCES BY SKIN FRICTION.

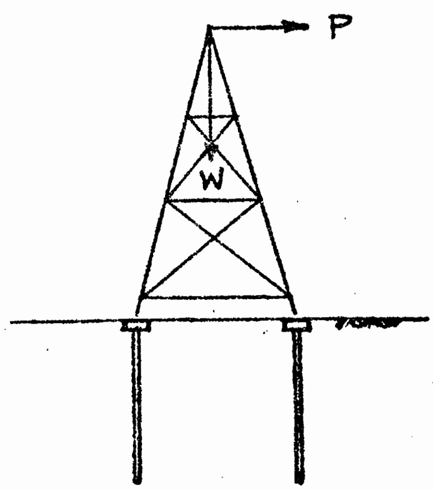


FIGURE 4

POLE EMBEDDED IN MASSIVE FOUNDATION :

THIS DESIGN RELIES ON THE AREA OR WEIGHT OF THE FOUNDATION TO OFFSET ANY OVERTURNINGING WHICH IS PRODUCED BY THE LATERAL FORCE.

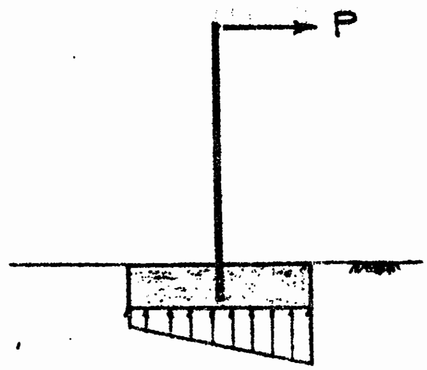
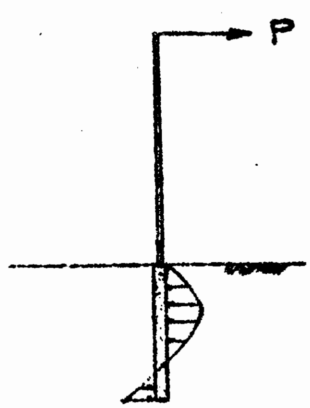


FIGURE 5

RIGID POLE EMBEDDED IN GROUND :

THIS DESIGN UTILIZES THE LATERAL STRENGTH OF THE SOIL TO RESIST OVERTURNING.



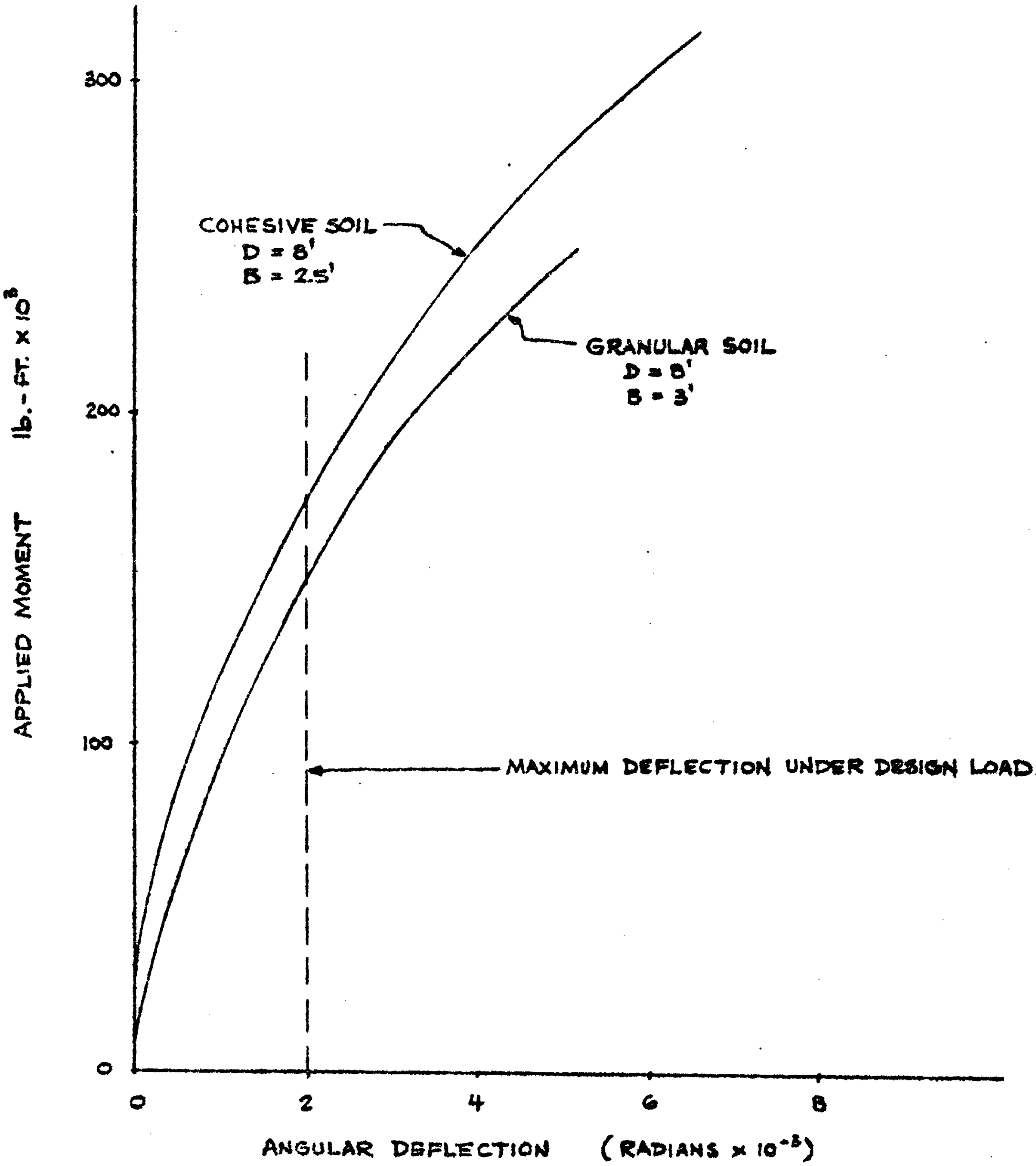


FIG. 6 TYPICAL MOMENT VS. DEFLECTION FOR RIGID PIERS. (AFTER BEHN 1959)

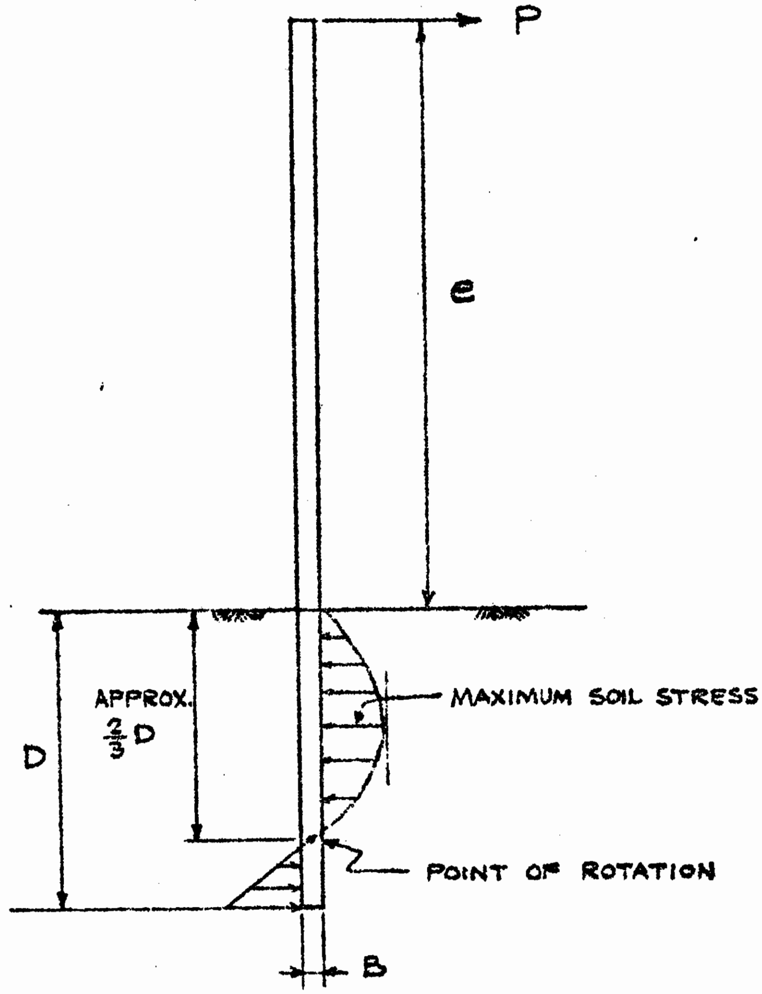


FIG. 7 SOIL STRESS DUE TO PIER ROTATION