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Analysis of Piles Under Dynamic Loading

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SYNOPSIS This paper presents a method of analyzing piles for transmission poles. The various de-
sign loads are summarized and the application of various types of load factors is described.
Methods of determining required pier embedment and design shear stresses are recommended. Con-
struction factors which might impact cost are also discussed.

INTRODUCTION

The need for a method of designing concrete
piles for transmission poles has increased in
recent years because of a tendency for environ-
mental groups to favor this type of structure
verses lattice towers. The single tubular
shaft is a more graceful structure, less no-
ticeable at a distance, and requires less land
area. The single pole foundation, however, is
subject to much greater overturning moments
than the typical tower footing, and older
methods of analysis of piers subject to large
moments have provided footings of excessive
size and cost.

In this paper a method of analysis and design
reported recently by this author (1979) to
provide economical pole foundations for 230KV
and 500KV transmission lines, is presented in
detail.

DESIGN LOADS

The loads applied by the pole to the top of
the pile are the result of:

A. Horizontal pull of the conductors
B. Wind on the pole and conductors
C. Unbalanced loading from broken conduct-
    ors.
D. Construction loads (while stringing con-
    ductors.
E. Gravity loads from conductors, pole,
    insulators, ice, etc.

National Electrical Safety Code (NESC) provides
wind, snow, and ice loads to be used for de-
signs in each geographical area. Use of this
criteria, however, needs to be tempered by the
designers judgement, as the transmission line
may often traverse "pockets" of unusual clima-
tological conditions not accounted for in NESC.

The NESC also provides load factors to be ap-
plied to the design wind, conductor, gravity,
and construction loads. The load factors are
used to establish the "Ultimate" or "Yield"
loading on the tubular arms and shaft of the
transmission pole. The code does not, however,
address load factors to be used in establishing
the ultimate resistance of the pier to rotation
in soil. These should stem from a mutual agree-
ment of the designer and geotechnical engineer
and are usually a reflection of the consultant's
confidence in his own laboratory and field
work. The consultant should provide separate
load factors, too, for each of the five load
types A,B,C,D,&E, above, utilizing greater
load factors where the maximum design load is
subject to the vagaries of weather.

Since conductor tension will relax substanti-
ally after initial construction, the consultant
and designer may chose to use initial line
tensions for A & C in tandem with reduced load
factors, or line tension after one year with
increased load factors. Load factors to be
applied to design loads in designing rebar and
concrete should be in accordance with the code
of the American Concrete Institute (ACI). Again,
there are no provisions in that code for the
relaxation of applied loads, but the economy
to be gained from such a phenomenon should be
considered in design, particularly for angle
poles.

FIELD INVESTIGATIONS AND TESTS

On typical transmission line projects,the geo-
technical engineer provides the pile designer
graphical representations of soil depth verses
ultimate passive pressure as shown in figure I.

As shown in the example, allowable pressure
typically decreases with an increase in pier
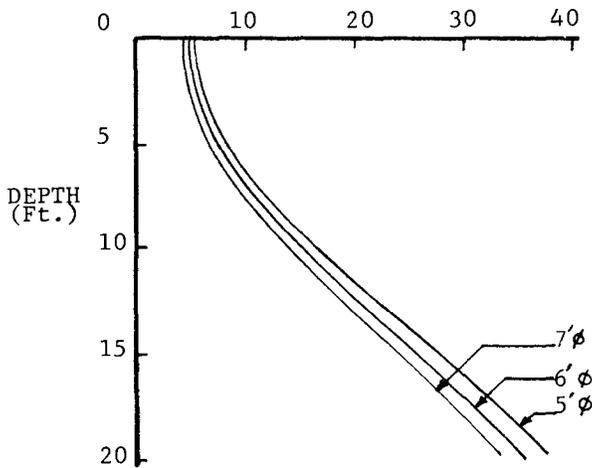diameter.

Ultimate Passive Resistance (Kips/S.F.)

Figure I

This is often the result of his analysis of:
  A. Standard penetration tests by a test
      boring rig.
  B. Shear tests of "Undisturbed"soil samples
  C. Pressure meter tests
  D. Field compression tests
  E. Full scale load tests
After several iterations of tests C,D,&E in a
specific geological mass, these would no longer
be required for similar masses as the tests of
A&B would enable the geotechnical engineer to
properly identify or classify the soil and re-
late it to load capacity values assigned pre-
viously to that mass.

For sites where compression, meter, or load
tests aren't available, the ultimate lateral
resistance of piles against transverse loads
may be determined by a method presented by
J. Brinch Hansen (1961). The method enables
a direct determination of the horizontal pres-
sures on the pile as a function of soil friction
angle. For the resultant pressure per unit
area of the pile face, Hansen writes the gener-
al expression:

$$e = \bar{q}\ k_q + C\ K_c$$

Where $\bar{q}$ = The effective overburden pressure

  K_q = Earth pressure coefficient for over-
  burden pressure

  K_c = Earth pressure coefficient for
  cohesion

  C = Soil cohesion

Hansen also presents charts which enable a
direct determination of the coefficients k_q
and K_c from the soil friction angle.

The total transmission line should be divided
into several geotechnical zones, and ultimate
lateral resistance curves (see figure I) pre-
pared by the consultant to represent the mini-
mum soil characteristics to be expected in each.
While standard penetration tests by a heavy
boring rig aren't justified at all pole loca-
tions, they usually are at large angle points
in the transmission line. A test hole by a
small truck-mounted auger should be considered
at all pole locations between the sites of
major drill holes to at least verify strata
depths and thicknesses.

Utilizing the load factors provided by the
geotechnical consultant, the designer factors
the design vertical load, shear, and moments
to determine the "Ultimate" load applied by
the pole to the top of the pile. The method
of statics presented by Hansen (1961) can be
used to verify that an embeded pile of assumed
dimensions has an ultimate restraining moment
at least equivalent to the applied, "Factored"
one. The calculation proceeds in the following
way, as shown in figure II:
  A. Estimate the required pier depth
  B. Calculate the resisting pressure on the
      pier from the passive pressure diagram
      provided by the geotechnical consultant
      (see fig. I)
  C. Determine the point of zero shear at L_1
      ($V_{passive}$ = H)

  D. Calculate the maximum applied moment at L_1

  E. Select L_2 so that shear between points
      L_1 and L_2 equals shear between points L_2
      and L_3.

  F. Determine the resisting moment at L_1

  G. If the resisting moment at L_1 is less
      than the applied moment, increase L_3
      and repeat steps C thru F.

Figure II

The bending moments to be utilized for rebar design should also be determined from the ultimate soil resistance diagram, adjusted as required to utilize the correct ACI load factors. Wang and Salmon (1973) presents a detailed analysis of flexural design for circular piers by the "strength" method. All rebar aren't usually required continuous to the bottom of the pier, but those extending beyond a cut-off point need to be sufficient in number to safisfy embedment bond.

DETERMINATION OF PIER SHEAR STRESSES

An analysis of shear stresses in a concrete pile requires practical consideration of equal and opposite (or nearly opposite) reactions on a pile. The ACI code permits the designer to disregard applied forces within a distance d/2 of the face of the support. Equating this condition to a rotating pile, applied forces within a distance d/2 of the rotation point (see figure III) can be similarly ignored.
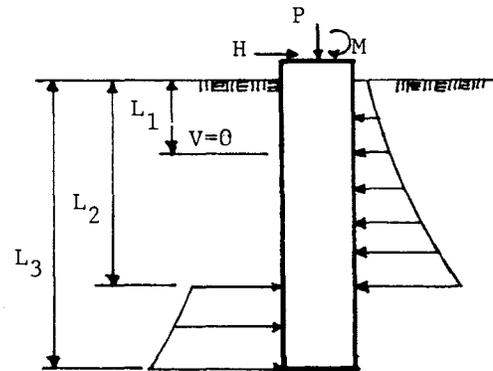
While a large part of the applied forces can be negated thus when the pier is loaded by the ultimate stress diagram, the design shear stress is likely to be critical prior to "Ultimate" loading, that is, in the "working" stress range.

For this reason concrete piles should be checked for shear stress assuming the lateral force diagram that would prevail prior to "Ultimate" loading. For this calculation a method presented by Davisson and Prakash (1963) can be used.
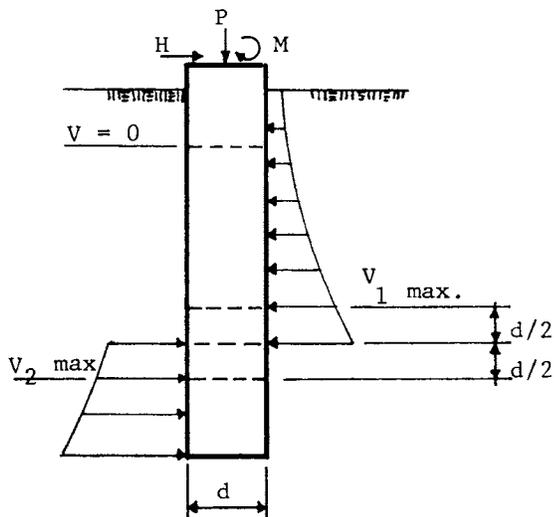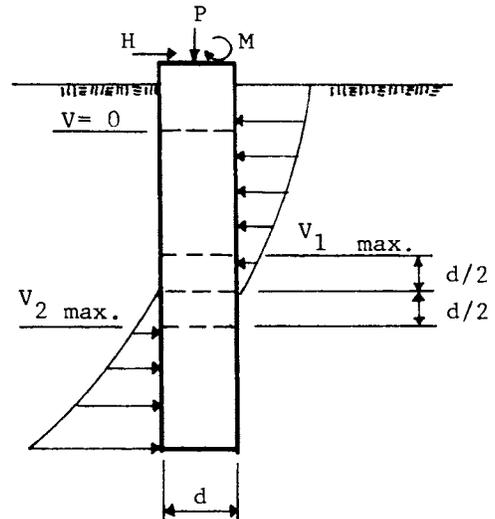
In that technical paper, piles are considered as essentially rigid members and a criteron is given for determining when the flexural member that serves as the pile is stiff enough to be considered rigid. Given the ground line moment and shear, the authors present simple equations which permit the designers to draw a passive pressure diagram as shown in figure IV.

The method requires first that the geotechnical engineer provide a value of the subgrade modulus (k_x) to define the soil stiffness.

MAXIMUM SHEAR FOR PIER DESIGN
(Working Loads Applied To Pier)

Figure IV

PRACTICAL CONSIDERATIONS DURING CONSTRUCTION

1. The pier shaft should not be larger in diameter than required to accomodate the anchor bolt cage, with 4 inches of concrete cover. Although only 3 inches are required in accordance with the ACI code, an additional inch of clearance should be provided all around in the design to allow for augered excavations which may be skewed slightly to the required vertical alignment of the anchor bolts. An enlarged, monolithic pier cap may be built when greater area is required to accomodate

MAXIMUM SHEAR FOR PIER DESIGN
(Ultimate Loads Applied To Pier)

Figure III

the base plate.

2. The inclination of the augered pier isn't usually a problem unless the skew is severe, say 1 in 20. This is because the virtual eccentricity of the applied loads at the top of the pier is usually much greater by comparison, for high voltage transmission poles. When the eccentricity is relatively large, it may be combined with the eccentricity of the applied load to determine the maximum total design moment.

3. Although it is good construction practice to make the bottom of the pile excavation as dense as possible, extra compaction effort for deep transmission pole foundations may not be justified. Davisson and Prakash (1963) have indicated that shear resistance to rotation at the base of the pile may or may not be present. In providing the passive pressure verses depth graphs, the geotechnical consultant and design engineer should weight the additional resistance possible, verses the cost of compacting the bases of deep footings. Even assuming twelve inches of loose soil at the base and 50% eventual consolidation, it isn't likely that six inches of settlement would have an impact on the design or operation of the transmission line.

4. After free-falling concrete from great heights for many years in the construction of dams, it has been proven that such free-fall is not detrimental to the quality of the concrete provided:
    A. The fall is uninterrupted by rebar, anchor bolts, or other embedments.
    B. The concussion from the impact does not cause the sides of the excavation to cave.

For this reason, elephant trunks or drop pipes to the bottom of the pile aren't usually required. The concussion, and vibration resulting from it, should be sufficient to preclude the need for mechanical vibrators. This equipment may be required for the top eight to ten feet of the pile.

CONCLUSIONS

Although effort has escalated in recent years to find alternative concepts to the concrete pile for the support of high voltage transmission poles, the method is still quite efficient provided the designer applies reasonable load factors, initiates a comprehensive geotechnical investigation, utilizes sound pile analysis, and doesn't specify costly, unnecessary construction requirements. The method permits the job superintendent to auger a hole, set the anchor bolt cage, place concrete, and complete all work to grade that same day. In addition to the hole digger, only a small boom or crane is required to

handle the anchor bolt cage. The operation requires a minimum amount of labor and isn't required to coordinate with another trade, as is often the case when backfill is required around a direct embedded pole.

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