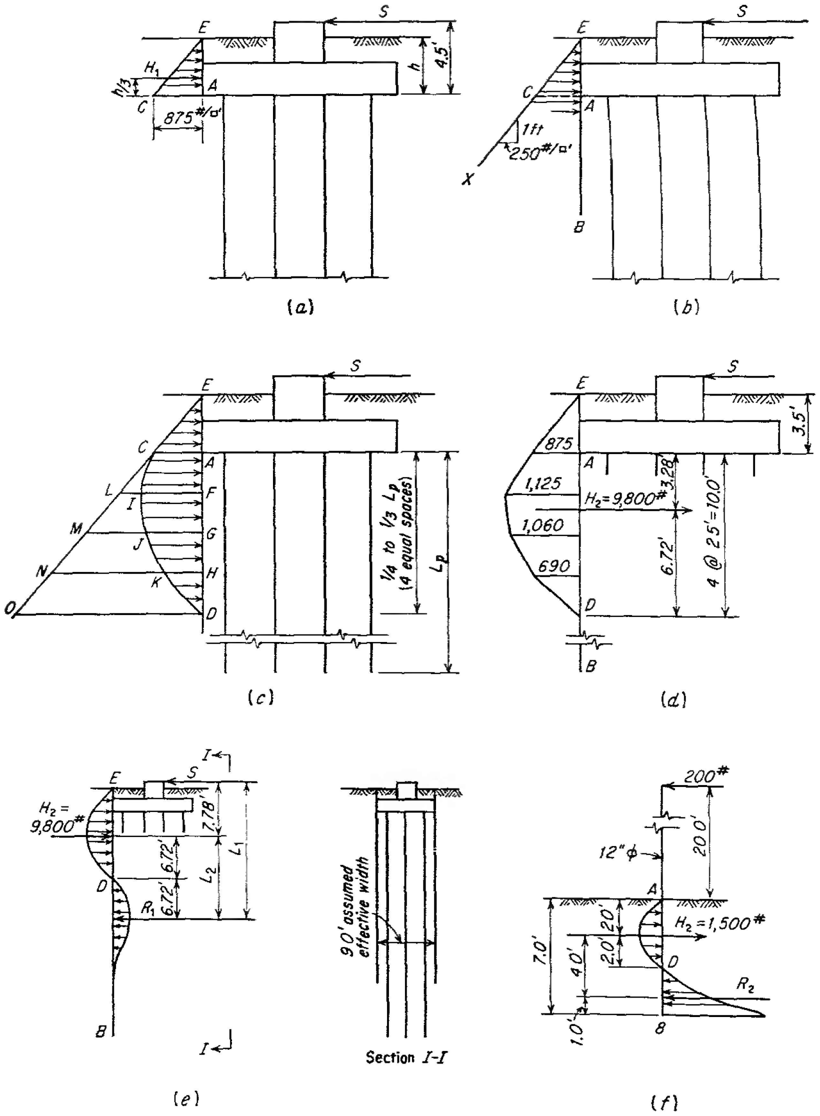

FIG. 10-25A. A study of lateral resistance of piles.

puted value at CA, Sketch (c), as follows:

Top of pile: CA = full computed value at point A
Top quarter point: $IF = 0.75LF$
Middle quarter point: $JG = 0.50MG$
Bottom quarter point: $KH = 0.25NH$
Bottom point: D: pressure = 0

Compute the total magnitude of the pressure diagram for a vertical slice 1 ft. wide and find the location of the resultant, as pictured in (*d*), assuming the top 10 ft. of the piles to be helpful in this case. Then the maximum possible passive resistance is about 9,800 lb. per ft. of width applied 3.28 ft. below the bottom of the footing. If the top 15 ft. of the pile is assumed to be bearing on the soil (length AD), the similarly computed force $H_2 = 16{,}900$ lb. and is located approximately 5.28 ft. below the bottom of the footing.

5. Next, referring to Sketch (*d*), assume that the line of action of H_2 is the fulcrum about which the footing and piles tend to rotate. On this assumption, the moment to be used for computing the overturning pressures on the piles might be assumed to be $M = S(4.5 + 3.28) = 40 \times 7.78$ ft.-kips instead of the 40×8.5 assumed in Art. 10-6 for the footing shown in Fig. 10-7(*a*). For $AD = 15$ ft., the lever arm would be 10.22 ft. A lever arm of about 8 ft. seems to be acceptable.

6. Since $S/9 = 4.44$ kips per ft. of width, the abutting resistance H_2 of Fig. 10-25A(*d*) is larger than necessary. However, assume that the resistance R_1 in Sketch (*e*) is as far below D as H_2 is above it. Then the required magnitude of the passive pressure for equilibrium is

$$H_2 = \frac{SL_1}{L_2} = \frac{4.44 \times 21.22}{13.44} = 7 \text{ kips (approx)}$$

The computed safety factor is

$$\text{S.F.} = 9.8/7 = 1.4$$

7. When the passive pressure against the footing as computed for H_1, Sketch (*a*), is sufficient to counteract the force S, it seems to be desirable to take moments about the tops of the piles when computing the vertical pressure on the piles. When H_1 is not sufficient, assume the lever arm of S to be located as computed in item 5 above.

8. For single piles or poles, one might estimate the resisting moment due to embedment in a somewhat similar manner, using AD as the top one-half to two-thirds or three-quarters of the embedded length when the member is relatively stiff and strong, where A is the surface of the ground and where the assumed resisting width is twice the diameter of the pole. For example, assume the pole shown in Sketch (*f*). If the

passive pressure is 300 p.s.f. per ft. of depth and the bottom pressure diagram is triangular, $H_2 = 1{,}500$ lb. (approx) and the safety factor is

$$\text{S.F.} = \frac{1{,}500 \times 4}{200 \times 22} = 1.36$$

As far as the footing of Fig. 10-7 is concerned, and as a result of the preceding computations and when the soils are too weak, it seems to be advisable to provide some better way of resisting the large force S. Batter piles might be used as explained in the next article.

Figure 10-25B is a nomograph[1] which is very convenient for use in determining the required embedment for poles to hold a given load at a stated height. This is based upon an assumed yielding of ½ in. at the top of the ground. It is suitable for designing telephone poles, billboard supports, and similar structures where appreciable movement is not harmful. It yields much larger safe loads than does the method described in connection with Fig. 10-25$A(f)$, which is intended for use in the design of structures where practically no yielding is permissible.

To use Fig. 10-25B, try the following example, as stated by Kinney: $P = 3{,}000$ lb., $H = 20$ ft., soil = average (2,500 p.s.f. allowable), and width $b = 14$ in.

1. Draw a line from $S_1 = 2{,}500$ through $P = 3{,}000$ and extend to find $C = 1.2$.
2. From $C = 1.2$, draw a line through $b = 14$ in. to L, the line of depth coefficients ($L = 1.0$ in this case).
3. From $L = 1.0$, draw a horizontal line to the curve for $H = 20$ ft., then draw vertically downward to the bottom of the diagram and find the required depth of embedment, 8.5 ft. in this instance.

If the horizontal forces acting on a group of piles are caused by the tendency of the surrounding soil itself to move laterally, the piles can offer little or no resistance since they depend upon the supporting soil for their own strength. If the piles go through a deep layer of unstable soil into a stable sand stratum, they will usually tilt or be broken by the deformation caused by any lateral movement of the top layer.

A good example of the fact that piles embedded in a deep plastic soil will merely move with that soil is the case illustrated in Fig. 10-26. When this viaduct was planned, it was considered desirable to terminate the superstructure near the flare in the roadways shown. This required an extensive fill a little over 30 ft. deep above the original ground, but it would avoid the use of complicated and costly structures. The soil was a layer of peat 8 to 10 ft. thick over a very deep stratum of fairly soft clay.

[1] Edwin E. Kinney, Correct Embedment for Pole Structures, Wood Preserving News, October, 1959. (Chart prepared for the Outdoor Advertising Association of America, Inc. by P. C. Rutledge.)

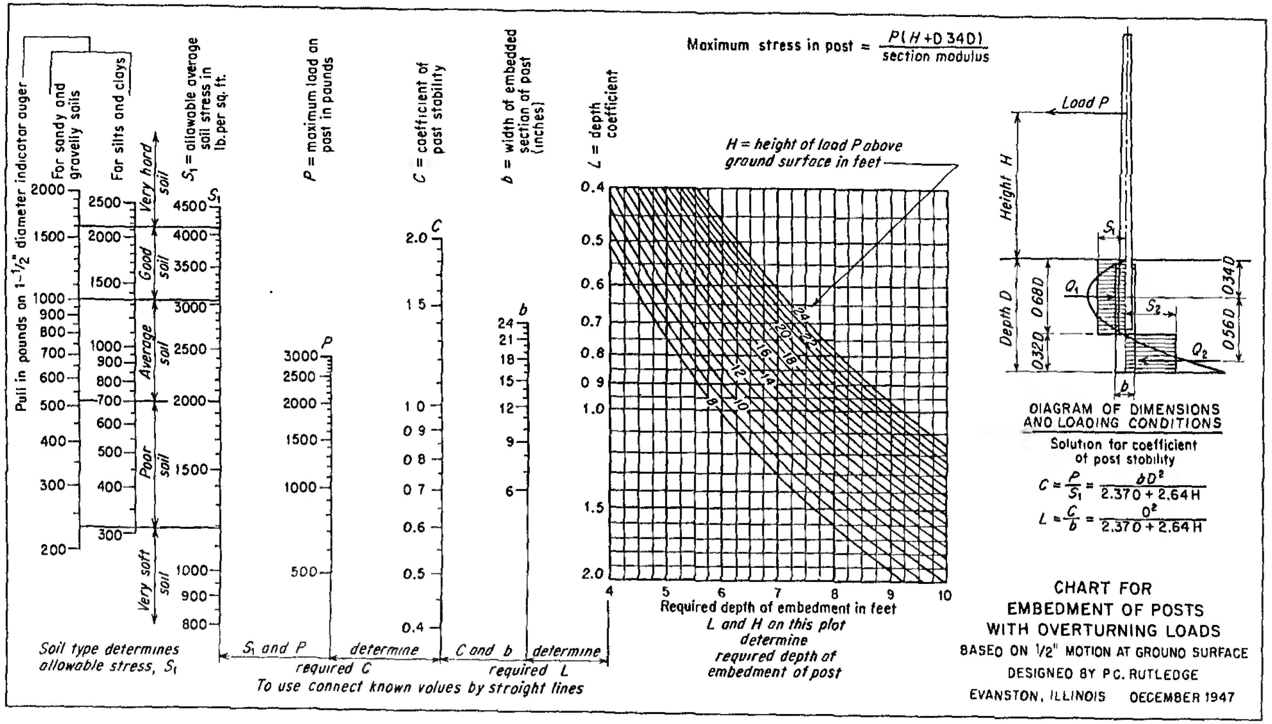

FIG. 10-25B. Nomograph for determining embedment of poles.