CHAPTER TWO

DESIGN CRITERIA

BASIC DESIGN CONSIDERATIONS

The designer of a deep foundation must possess a variety of skills, much experience, and considerable knowledge of engineering sciences. No set of simple rules and procedures (such as those developed in some areas of structural design) can be expected to cover the variety of conditions and forms of instability that can affect a deep foundation. The following discussion outlines some basic design criteria that a design engineer may find useful in meeting the basic requirements of safety, dependability, functionality, and economy.

As in the case of shallow foundations, principal dimensions of deep foundations are determined so as to satisfy two basic requirements of safety: (a) the foundation must possess sufficient safety against failure and (b) the foundation should not undergo excessive displacements under working loads. Thus, the nominal design load (Q_a) should not exceed a specified fraction of the ultimate load (Q_o), so that at any time

$$Q_a \leq Q_o/F_s \qquad (1)$$

in which F_s is a safety factor. At the same time, the settlements and horizontal displacements of the foundation under working loads should not exceed specified limits set by the usage requirements and structural tolerances of the supported structure. (For details on safety factors, working loads for settlement analysis, and settlement tolerances, see Refs. *1*, *18*, *21*, *22*.)

The ultimate load (Q_o) is the load that can cause either the structural failure of the foundation itself or the bearing-capacity failure of the soil. Excluding buckling and bending under the action of lateral loads and failure caused by excessive stresses during pile driving, which is discussed later, structural failure is assumed to occur when the maximum axial stress in the foundation shaft equals the critical stress for the shaft material (yield stress for steel, compressive or tensile strength for concrete or timber). This condition may govern design where pile points penetrate into very dense sand or rock. In most other situations, the ultimate load is determined from considerations of bearing-capacity failure of the soil.

THE ULTIMATE-LOAD CRITERION

Although the mode of shear failure of soil under a shallow foundation varies with the soil type, rate of loading, and other factors (cf. Ref. *1*), experience shows that soil under a deep foundation always fails in the same manner; i.e., in punching shear under the foundation point, accompanied or preceded by direct-shear failure of the soil along the foundation shaft (Fig. 2). As in the case of punching shear of shallow foundations, the ultimate load is rarely well defined; in many cases there is no visible collapse of the foundation and no clearly defined peak load (Fig. 3). To decide, on the basis of visual examination alone, on the magnitude of ultimate load in such cases can be quite deceiving. Figure 4 (*23*) shows the same load-to-settlement relationship of a test pile drawn in two different scales. Although the upper diagram may suggest that an "ultimate load" of 100 tons (90 metric tons) is reached, the lower indicates that the pile still has unused capacity at that load. Thus an unambiguous criterion is needed to establish the nominal magnitude of the "ultimate load."

Various ultimate-load criteria, all empirical in nature, have been proposed and used by different researchers and design organizations (*19*). As given in Table 2, such criteria are most often based on considerations of plastic (irrecoverable) or total (plastic plus elastic) settlements of the pile under the test load. A comparison of ultimate loads

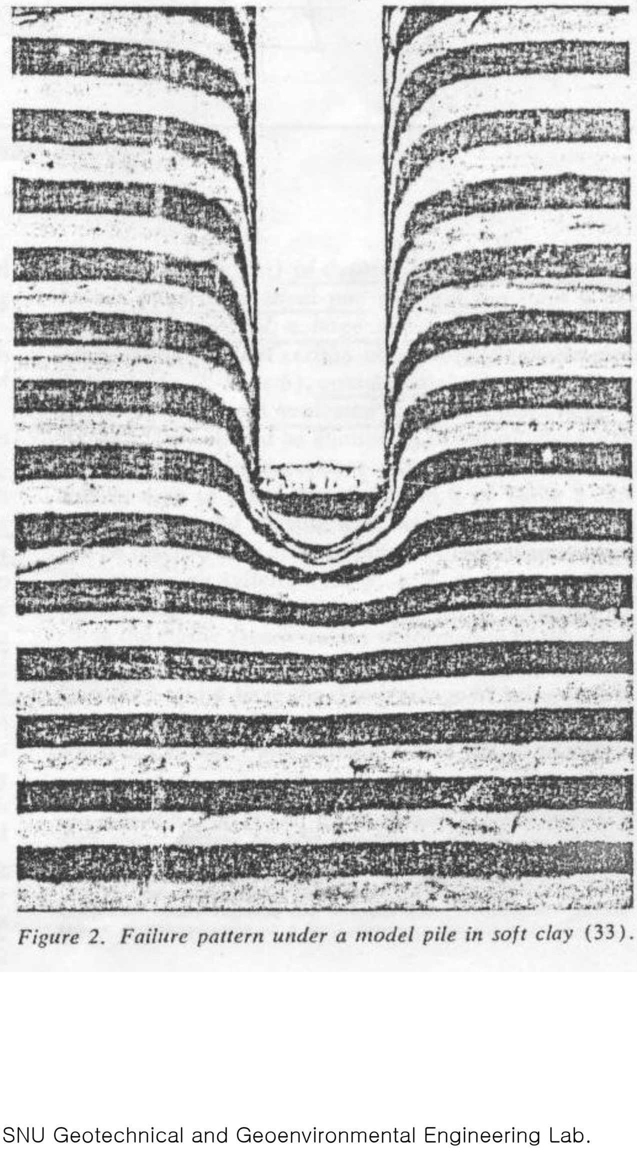

Figure 2. Failure pattern under a model pile in soft clay (33).

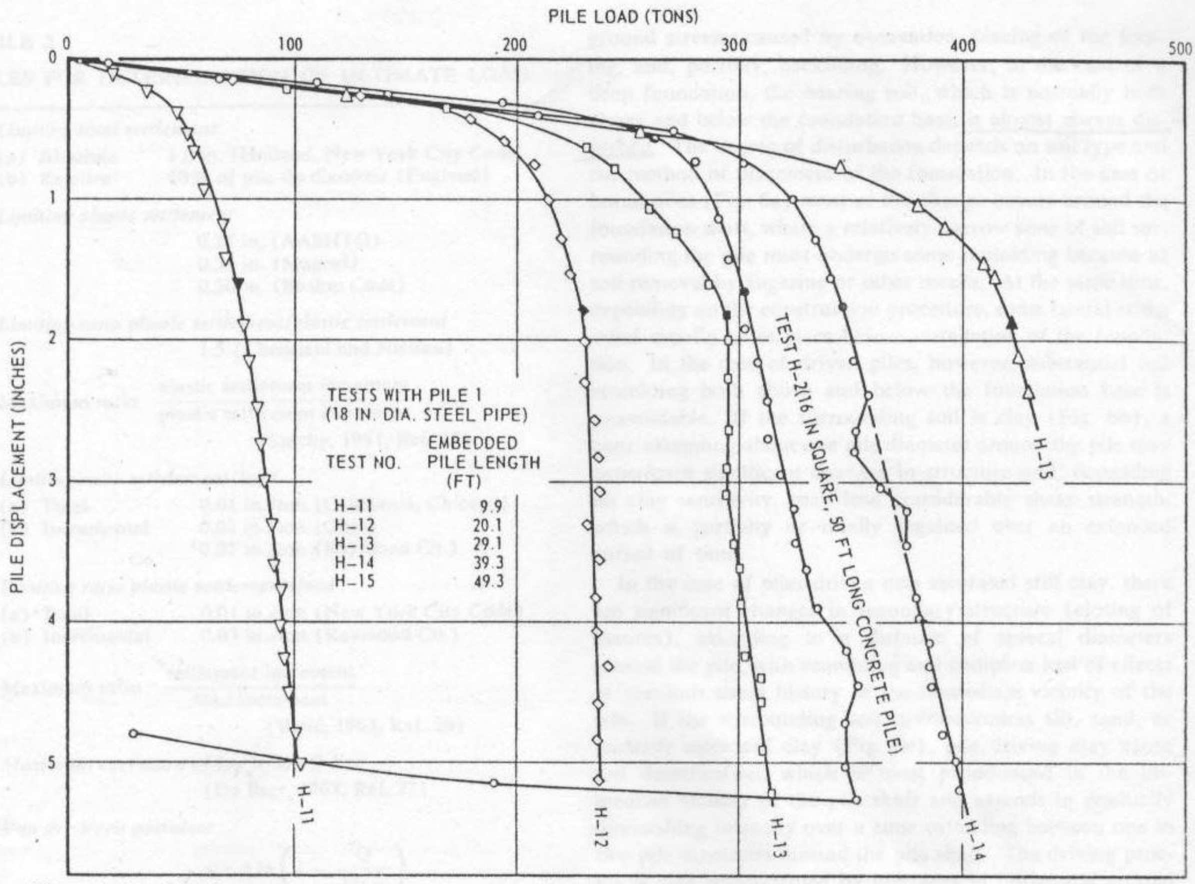

Figure 3. Load-displacement diagrams for series of test piles in sand (44).

obtained by applying these criteria to results of actual load tests shows relatively little difference (±10 percent) as long as the piles are not greater than 12 in. (300 mm) in diameter. However, substantial differences between ultimate loads obtained by various criteria can be found from results of load tests of large-diameter or very long piles (*221*).

To properly interpret these differences, it is essential to recall some basic facts about the mechanism of load transfer between a pile and surrounding soil. Modern research on pile behavior has established that full mobilization of skin resistance requires a relative displacement between the pile shaft and surrounding soil of 0.25 to 0.40 in. (6 to 10 mm), regardless of pile size and length (*24*). At the same time, mobilization of ultimate point resistance of a pile requires a displacement of approximately 10 percent of pile-tip diameter for driven piles and as much as 30 percent of the pile-tip diameter for bored piles. With these facts it is not difficult to understand that 1 in. (25 mm) of total settlement or 0.5 in. (13 mm) of plastic settlement may indeed nearly mobilize the ultimate load of a 6-in. (50-mm)-diameter pile but only a fraction of the ultimate load of a 96-in. (2 400-mm)-diameter bored pile. A simple calculation based on knowledge of basic load-settlement relationships of loaded areas demonstrates that 0.03 in./ton (0.8 mm/metric ton) of deformation may be indicative of failure stage for a small pile and still represent a normal deformation rate of a large pile in the safe-load range. Thus, it follows that certain ultimate-load criteria given in Table 2 (1a, 2, 5, or 6), containing absolute magnitudes of plastic or total limit settlement can not generally be valid. As such, they should be eliminated or substituted by analogous pile-diameter-dependent criteria.* It should be also equally easy to prove that Criterion 9 of Table 2 can not be generally valid because it assumes that the ultimate load can be reached only after an infinitely large displacement.** The remaining criteria (1b, 3, 4, 7, and 8) appear to be equally dependable, particularly if 1b is corrected to exclude the elastic (recoverable) deformation of the pile shaft from the total settlement. From the practical point of view, Criteria 3 and 4 have the disadvantage of being tied to the traditional time-consuming maintained-load testing method whereas Criteria 7 and 8 require load testing to very large

* The widely used AASHTO criterion of defining failure load as plastic settlement of 0.25 in. (6 mm) may be in order for small piles; it is definitely overconservative for piles exceeding 12 in. (300 mm) in diameter. A good substitute criterion would be limiting plastic settlement to, perhaps, 2 percent of the pile diameter.

** This criterion can offer some service in situations where load test was terminated before reaching the ultimate load; it offers a consistent approach to extrapolation of a load-settlement diagram toward failure.

TABLE 2

RULES FOR DETERMINATION OF ULTIMATE LOAD

1. *Limiting total settlement*
   (a) Absolute 1.0 in. (Holland, New York City Code)
   (b) Relative 10% of pile tip diameter (England)

2. *Limiting plastic settlement*
   0.25 in. (AASHTO)
   0.33 in. (Magnel)
   0.50 in. (Boston Code)

3. *Limiting ratio plastic settlement/elastic settlement*
   1.5 (Christiani and Nielsen)

4. Maximum ratio $\frac{\text{elastic settlement increment}}{\text{plastic settlement increment}}$
   (Széchy, 1961, Ref. 25)

5. *Limiting ratio settlement/load*
   (a) Total 0.01 in./ton (California, Chicago)
   (b) Incremental 0.03 in./ton (Ohio)
   0.05 in./ton (Raymond Co.)

6. *Limiting ratio plastic settlement/load*
   (a) Total 0.01 in./ton (New York City Code)
   (b) Incremental 0.03 in./ton (Raymond Co.)

7. Maximum ratio $\frac{\text{settlement increment}}{\text{load increment}}$
   (Vesić, 1963, Ref. 26)

8. *Maximum curvature of log w/log Q line*
   (De Beer, 1967, Ref. 27)

9. *Van der Veen postulate*

$$w = \beta \ln\left(1 - \frac{Q}{Q_{max}}\right)$$

(Van der Veen, 1953, Ref. 23)

displacements, normally not less than one half of the pile-tip diameter. Thus, Criterion 1b is probably the most acceptable for general engineering practice. It should be used in the following corrected form: Unless the load-settlement curve of a pile shows a definite peak load, the ultimate load is defined as the load causing total pile settlement equal to 10 percent of the point diameter for driven piles and 25 percent of the point diameter for bored piles.

COMPUTATION OF THE ULTIMATE LOAD

The basic problem of computation of ultimate load of a deep foundation can be formulated as follows: A cylindrical shaft of diameter B (Fig. 5) is placed to depth D inside a soil mass of known physical properties. A static, vertical, central load (Q) is applied at the top and increased until a shear failure in the soil is produced. The problem is to determine the ultimate load (Q_o) that this foundation can support.

Although an obvious similarity exists between this problem and the analogous problem for a shallow foundation, there are some distinct differences that must be kept in mind from the outset. In the case of a shallow foundation, the bearing soil, which is under the foundation base, has normally not been disturbed, except for changes in effective ground stresses caused by excavation, placing of the footing, and, possibly, backfilling. However, in the case of a deep foundation, the bearing soil, which is normally both above and below the foundation base, is almost always disturbed. The degree of disturbance depends on soil type and the method of placement of the foundation. In the case of bored piles (Fig. 6a) most of the change occurs around the foundation shaft, where a relatively narrow zone of soil surrounding the pile must undergo some remolding because of soil removal by augering or other means. At the same time, depending on the construction procedure, some lateral-stress relief usually takes place before installation of the foundation. In the case of driven piles, however, substantial soil remolding both above and below the foundation base is unavoidable. If the surrounding soil is clay (Fig. 6b), a zone extending about one pile diameter around the pile may experience significant changes in structure and, depending on clay sensitivity, may lose considerable shear strength, which is partially or totally regained over an extended period of time.

In the case of piles driven into saturated stiff clay, there are significant changes in secondary structure (closing of fissures), extending to a distance of several diameters around the pile, with remolding and complete loss of effects of previous stress history in the immediate vicinity of the pile. If the surrounding soil is cohesionless silt, sand, or partially saturated clay (Fig. 6c), pile driving may cause soil densification, which is most pronounced in the immediate vicinity of the pile shaft and extends in gradually diminishing intensity over a zone extending between one to two pile diameters around the pile shaft. The driving process is also accompanied by increases in horizontal ground stress and changes in vertical stress in the pile vicinity, some or all of which can be lost by relaxation in creep-prone soils. In dense, cohesionless soils (such as sand or gravel), loosening may take place in some zones, along with substantial grain crushing and densification in the immediate vicinity of the pile. [According to Kérisel and Adam (*28*), some of the test piles in dense sand were excavated and pulled out with a hull of highly densified, crushed material that resembled a fine-grained sandstone.] In such soils there are permanent changes in horizontal as well as in vertical ground stress that can be highly pronounced. Hard driving can leave large residual stresses in both the pile and the soil, consideration of which may be essential for understanding the behavior of the pile-soil system (Fig. 6c). Because piles are often designed in groups, the situation is further complicated by the complex and not always well-understood effect of placing of adjacent piles. For these and other reasons the problem under consideration poses difficulties unparalleled in other common soil mechanics problems. A general solution to the problem is not yet available and will be difficult to formulate.

For design purposes the ultimate load is conventionally separated into two components, the shaft or skin load (Q_s) and the base or point load (Q_p), which are superimposed as follows:

$$Q_o = Q_p + Q_s = q_o A_p + f_s A_s \quad (2)$$

A_p and A_s represent, respectively, the bearing areas of the