CHAPTER II

LITERATURE REVIEW

2.1 Behaviour at Working Loads

McCLELLAND and FOCHT (1958) found that the load-deflection relationship of laterally loaded piles driven into cohesive soils is similar to the stress-strain relationships as obtained from consolidated undrained tests. BROMS (1964a) assumed that at working loads (loads less than one-half to one-third the ultimate lateral resistance of the pile), the deflection of a single pile or of a pile group can be considered to increase approximately linearly with the applied load. A linear analysis is of limited validity since the actual load-deflection behaviour of laterally loaded piles is very markedly non-linear. Attempts have been made to carry out non-linear analyses, either by using a stress-dependent modulus, (KUBO (1965) with subgrade reaction theory), or by an analysis assuming elasto-plastic soil behaviour (e.g. MADHAV et al (1971) with subgrade reaction theory, and SPILLERS and STOLL (1964) and POULOS (1971a, 1972, 1973) with elastic theory).

BROMS (1964a) assumed that the lateral deflections and the distribution of bending moments and shear forces can be calculated at working loads by means of the theory of subgrade reaction. The theory of subgrade reaction is basically the same as the theory of the beam on elastic foundation (HETÉNYI, 1946; TIMOSHENKO, 1966).

2.1.1 Theory of Beam on Elastic Foundation

(a) Assumptions proposed by HETÉNYI (1946) -

i) The soil is homogeneous and isotropic.

ii) Neglect the frictional forces originating along the surface where the beam is in contact with the foundation.

iii) The soil properties follow Hooke's law that the continuous reaction of the foundation is proportional to the deflection (WINKLER, 1867).

(b) Differential equation of the elastic line - BROMS (1964a) assumed that the unit soil reaction q(in kg/cm^2) acting on a laterally loaded pile increases in proportion to the lateral deflection y(in cm.) expressed by the equation

$$q = k y , \qquad \qquad ....(1)$$

where the coefficient k(in kg/cm^3) is defined as the coefficient of subgrade reaction. The numerical value of the coefficient of subgrade reaction varies with the width of the loaded area and the load distribution, as well as with the distance from the ground surface (BIOT, 1937; DEBEER, 1948; TERZAGHI, 1955; VESIĆ, 1961a, 1961b). The corresponding soil reaction per unit length Q (in kg/cm) can be evaluated from

$$Q = kDy, \qquad \qquad ....(2)$$

in which D is the diameter or width of the laterally loaded pile. If kD is denoted K (in kg/cm^2), then

$$Q = Ky . \qquad \qquad ....(3)$$

The governing equation of the problem of beam on elastic foundation is

$$\frac{d^4y}{dx^4} + \frac{K\,y}{EI} = \frac{P(x)}{EI} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(4)$$

Let $4\beta^4 = \frac{K}{EI}$.

The general deflection equation for beam on elastic foundation is

$$y_{(x)} = e^{+\beta x}(A\cos\beta x + B\sin\beta x) + e^{-\beta x}(C\cos\beta x + D\sin\beta x) \ldots(5)$$

where A,B,C and D are constants of integration to be determined from the boundary condition.

(c) Infinite beam subjected to a concentrated load - The general equations are

$$y_{(x)} = \frac{P}{8\beta^3 EI} e^{-\beta x}(\cos\beta x + \sin\beta x) \quad \ldots\ldots(6a)$$

$$y'_{(x)} = -\frac{P}{4\beta^2 EI} e^{-\beta x}\sin\beta x \quad \ldots\ldots(6b)$$

$$M_{(x)} = \frac{P}{4\beta} e^{-\beta x}(\cos\beta x - \sin\beta x) \quad \ldots\ldots(6c)$$

$$V_{(x)} = -\frac{P}{2} e^{-\beta x}\cos\beta x \quad \ldots\ldots(6d)$$

where P is a concentrated load.

2.1.2 Theory of Subgrade Reaction BROMS (1964a) assumed that the coefficient of subgrade reaction is constant within the significant depth. (The significant depth is defined as the depth in which a change of the subgrade reaction will not affect the lateral deflection at the ground surface or the maximum bending moment by more than 10%). The coefficient of subgrade reaction frequently varies with depth. SKEMPTON (1951) has shown that the coefficient of subgrade reaction for cohesive soils is approximately proportion to the unconfined compressive strength of the soil.

As the unconfined compressive strength of normally consolidated clays and silts increase approximately linear with depth, so BROMS (1964a) suggested that the coefficient of subgrade reaction can be estimated to increase in a similar manner as indicated by field data obtained by PARRACK (1952) and by PECK & DAVISSON (1962). BROMS (1964a) stated that the unconfined compressive strength of overconsolidated clays may be approximately constant with depth such as the overconsolidation of the soil has been caused by glaciation while the unconfined compressive strength may decrease with depth if the overconsolidation has been caused by desiccation. Thus, the coefficient of subgrade reaction may, for an overconsolidated clay, be either approximately constant or decrease as a function of depth.

The evaluation methods of the coefficient K for piles driven into cohesive soils have been discussed by TERZAGHI (1955), however, the numerical value of this coefficient is affected by consolidation and creep.

BROMS (1964a) suggested that the lateral deflections can be predicted at the ground surface when the coefficient of subgrade reaction increases with depth if this coefficient is assumed to be constant and if its numerical value is taken as the average within a depth equal to $0.8 \beta L$.

(a) Lateral deflections - When the coefficient of subgrade reaction is constant with depth, the distribution of lateral deflection, soil reactions, bending moments and soil rotations can be calculated analytically (HETENYI, 1946; TIMOSHENKO, 1966; BROMS, 1964a; POULOS, 1971a), numerically (NEWMARK, 1943; GLESER, 1953; HOWE, 1955; BOWLES, 1968), by finite element (BOWLES, 1974), or by model tests (THOMS, 1957; POULOS, 1973). For the case of a laterally loaded pile in a layered system consisting of an upper stiff crust and a lower layer of soft clays, the distribution of lateral deflections, bending moments and soil reactions can be calculated (DAVISSON & GILL, 1963; KHADILKAR et al 1973; REESE & WELCH, 1975).

The deflections, bending moments and soil reactions depend primarily on the dimensionless length β L, and CHANG (1937) calculated β from the equation

$$\beta = \sqrt[4]{\frac{kD}{4E_pI_p}} , \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (7)$$

where E_pI_p = the stiffness of the pile section.

BROMS (1964a) has shown that the lateral deflection y_o at the ground surface can be expressed as a function of the dimensionless quantity $y_o kD/P$. This quantity is plotted as a function of the dimensionless pile length β L in the chart. The lateral deflections have been calculated for the two cases when the pile is fully free or fully fixed at the ground surface.

Generally most of laterally loaded piles are only partly restrained and the lateral deflections at the ground surface or other behaviors will attain values between those corresponding to fully fixed or fully free conditions.

<u>Classification of pile according to stiffness</u> - It can be seen that the βL quantity characterizes the relative stiffness of a pile in an elastic half-space. This βL quantity determines the magnitude of the curvature of the elastic line and defines the rate at which the effect of a loading force dies out in the form of a damped wave along the length of the pile.

According to these βL values, BROMS (1964a) classified piles into three groups:

<u>Free headed pile</u>

(i) Short pile, $\beta L < 1.5$

(ii) Pile of medium length, $1.5 < \beta L < 2.5$

(iii) Long pile, $\beta L > 2.5$

<u>Restrained pile</u>

(i) Short pile, $\beta L < 0.5$

(ii) Pile of medium length, $0.5 < \beta L < 1.5$

(iii) Long pile, $\beta L > 1.5$

For short pile, the bending deformation of the pile can be neglected in most practical problems because this deformation will be so small to be negligible compared with the deformation produced in the foundation.

For medium length pile, its characteristic is that a force acting at one end of the pile has a finite, and not negligible, effect at the other end.

For long pile, it has a βL value such that the countereffect of the end-conditioning forces on each other is a diminishing one. HETÉNYI (1946) assumed that the other end is infinitely far away when investigating one end of the pile. Forces applied at one end will have a negligible effect at the other.

BROMS (1964a) calculated the lateral deflections at the ground surface for short and long free headed piles as follows

(1) <u>Short piles</u> $(\beta L < 1.5)$

$$y_o = \frac{4P\,(1+1.5\,\frac{e}{L})}{kDL} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (8)$$

(ii) <u>Long piles</u> $(\beta L > 2.5)$

$$y_o = \frac{2P\beta\,(e\beta + 1)}{k_\infty D} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (9)$$

where k_∞ is the coefficient of subgrade reaction corresponding to an infinitely long pile.

(iii) <u>Medium length piles</u> $(1.5 < \beta L < 2.5)$ - The lateral deflection at the ground surface of a free-headed pile can be calculated from

$$\begin{aligned}
y_o = {} & \frac{P}{8\beta^3 EI} + \frac{P}{16\beta^3 EI} E_I \left\{ [1+C(L)][1-A(L)] + 2[1+D(L)]^2 \right\} [1+A(L)] \\
& + \frac{P}{16\beta^3 EI} E_{II} \left\{ [(1-C(L)][1+A(L)] + 2[1-\ldots L)]^2 \right\} [1-A(L)] \\
& - \frac{P\,B(L)}{4\beta^3 EI} \left\{ E_I [1+C(L)] - E_{II} [1-C(L)] \right\} + \frac{M}{4\beta^2 EI} E_I [1-A^2(L)] \\
& \frac{-M\,B(L)}{8\beta^2 EI} E_I \left\{ [1+C(L)][1-A(L)] + 2[1-D(L)]^2 \right\} \\
& \frac{+M}{4\beta^2 EI} E_{II} [1-A^2(L)] \frac{+M\,B(L)}{8\beta^2 EI} E_{II} \left\{ [1-C(L)][1+A(L)] + 2[1+D(L)]^2 \right\}
\end{aligned}$$

$$\ldots\ldots\ldots\ldots (10)$$

HETÉNYI (1946) simplified this equation in the form:

$$y_o = \frac{2P\beta}{K}\left\{\frac{\text{Sinh}\,\beta L.\ \text{Cosh}\,\beta L - \sin\beta L\,\cos\beta L}{\text{Sinh}^2\beta L - \sin^2\beta L}\right\} + \frac{2M\beta^2}{K}\left\{\frac{\text{Sinh}^2\beta L + \sin^2\beta L}{\text{Sinh}^2\beta L - \sin^2\beta L}\right\},$$

..............................(11)

where M = concentrated moment acting at the ground surface;

$$E_I = \frac{e^{\beta L}}{2(\text{Sinh}\,\beta L + \sin\beta L)};$$

$$E_{II} = \frac{e^{\beta L}}{2(\text{Sinh}\,\beta L - \sin\beta L)}.$$

It should be noted that derivations of these free-headed pile and the restrained pile equations are shown in Appendix A.

(b) Coefficient of subgrade reaction - VESIĆ (1961a) has shown that the coefficient of subgrade reaction can be evaluated assuming that the pile length is large when the dimensionless length βL is larger than 2.25. In the case when the dimensionless length of the pile βL is less than 2.25, the coefficient of subgrade reaction depends primarily on the diameter of the test pile and on the penetration depth (BIOT, 1937; DEEER, 1948; TERZAGHI, 1955; VESIĆ, 1961a, 1961b).

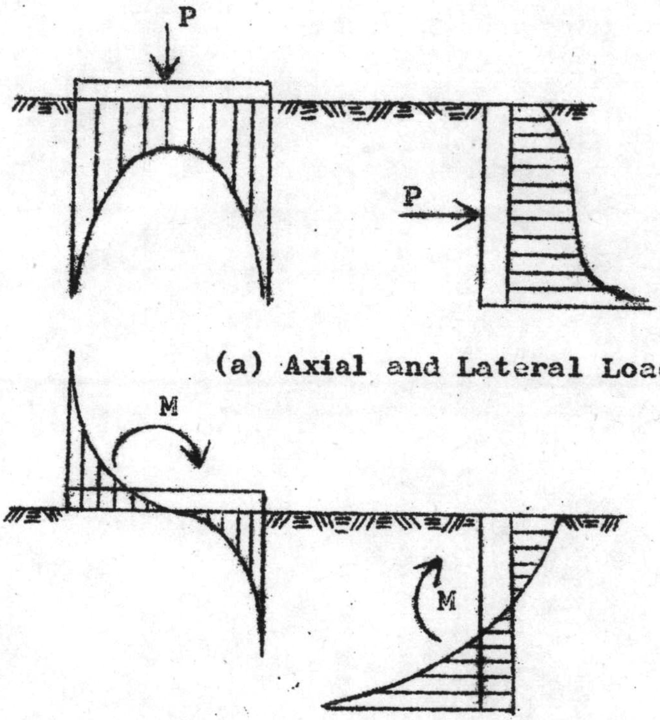
Based on research by VESIĆ (1961a), BROMS (1964a) a general expression for the modulus of subgrade reaction can be obtained from:

$$K = 0.65 \sqrt[12]{\frac{E_s D^4}{E_p I_p}} \times \frac{E_s}{1 - \mu_s^2} \quad , \ldots\ldots\ldots\ldots (12)$$

which should be double, because the medium extends on both sides of the pile, thus obtaining

$$K = 1.30 \sqrt[12]{\frac{E_s D^4}{E_p I_p}} \times \frac{E_s}{1-\mu_s^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (13)$$

BROMS (1964a) assumed that the distribution of bending moments, shear forces, soil reactions, and deflections are the same for the horizontal and the vertical members as shown in Fig. 1.



(a) Axial and Lateral Loads

(b) Overturning Moment

Fig. 1 - Distribution of Soil Reactions

(i) <u>Long piles ($\beta L > 2.25$)</u> - VESIĆ (1961a, 1961b) has shown that the coefficient of subgrade reaction k, for an infinitely long strip with the width D, (such as a wall footing founded on the surface of a semi-infinite, .. ideal elastic body) is proportional to the factor α and the coefficient of subgrade reaction K_o for a square plate, with the length equal to unity. The coefficient k_∞ can be evaluated from

$$k_{\infty} = \frac{\alpha K_o}{D} \quad , \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (14)$$

where $$\alpha = 0.52 \sqrt[12]{\frac{K_o D^4}{E_p I_p}} \; . \; \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (15)$$

This coefficient can be used for the determination of the distribution of bending moments, shear forces and deflections in laterally loaded piles.

Numberical calculations by BROMS (1964a) have indicated that the coefficient α can only vary between narrow limits for steel, concrete or timber piles. It can be determined approximately from the expression

$$\alpha = n_1 \, n_2 \quad , \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (16)$$

in which n_1 and n_2 are functions of the unconfined compressive strength of the supporting soil and of the pile material, respectively. The coefficient n_1 and n_2 are tabulated in Tables 1 and 2 (BROMS, 1964a). The coefficient α has been evaluated for steel pipe and H-piles as well as for cast-in-place or precast concrete piles with cylindrical cross sections.

Table 1 - Evaluation of the Coefficient n_1

Unconfined Compressive Strength q_u (ksc.)	Coefficient n_1
Less than 0.54	0.32
0.54 to 2.15	0.36
Larger than 2.15	0.40

Table 2 - Evaluation of the Coefficient n_2

Pile Material	Coefficient n_2
Steel	1.00
Concrete	1.15
Wood	1.30

Table 3 - Numerical Value of Coefficient m

Ratio, L/D	1.0	1.5	2	3	5	10	100
Coefficient, m	0.95	0.94	0.92	0.88	0.82	0.71	0.37

For the case when the coefficient of subgrade reaction decreases with depth, the method developed by DAVISSION & GILL (1963) can be used.

(ii) <u>Short piles</u> ($\beta L < 2.25$) - The coefficient of subgrade reaction for laterally loaded short piles with a length βL less than 2.25 may be calculated approximately by the following method.

BROMS (1964a) proposed an approximate method of calculating the coefficient of subgrade reaction and the deflection at ground surface (y_o) for a short pile as shown in Fig. 2.

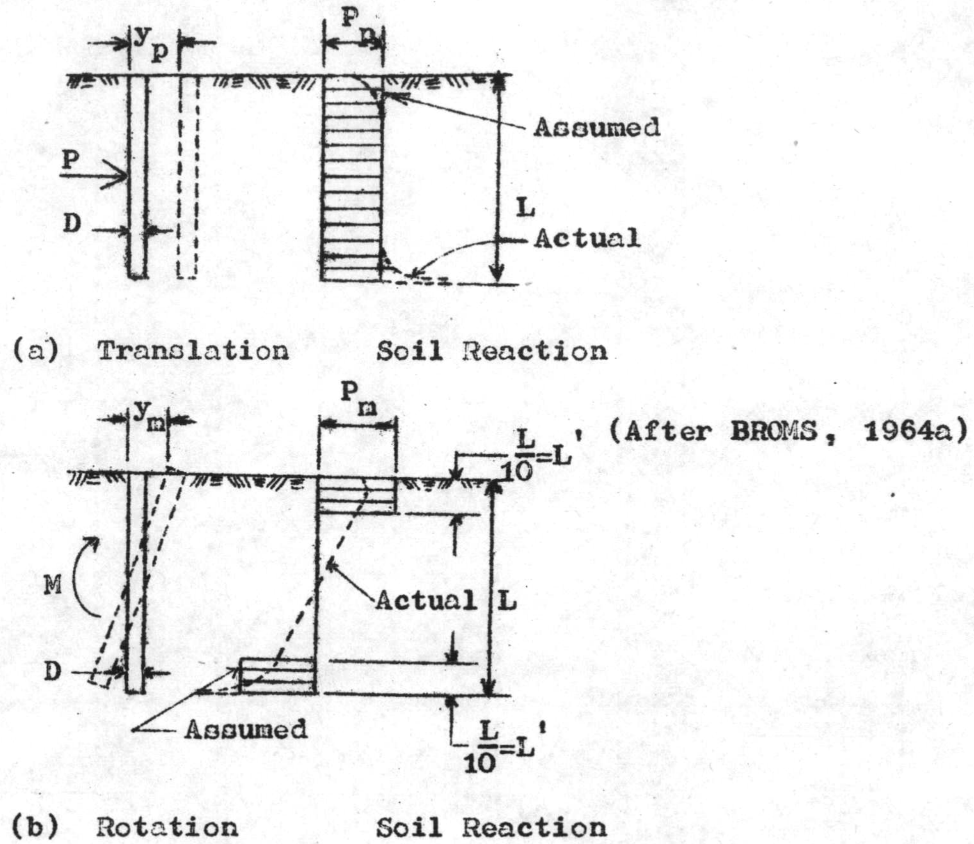


Fig. 2 - Calculation of Lateral Deflections for a Short Pile

BROMS (1964a) assumed that a lateral load P acting at mid height will cause a pure translation of the pile as shown in Fig. 2(a) while a moment M acting at mid-height of the pile will result in a pure rotation with respect to the center of the pile and the distribution of lateral earth pressures will be approximately triangular as shown in Fig. 2(b) (assuming a constant coefficient of subgrade reaction). It can be seen that any force system acting on a pile can be resolved into a single lateral force and a moment acting at the center of the embedded section of the loaded pile.

The coefficient k_p governs the lateral deflections caused by the lateral load P. The numerical value of the coefficient k_p depends in its turn on the shape of the loaded area and can be calculated from elasticity theory

(TIMOSHENKO, 1951), at low load levels when the deflections are proportional to the applied load by the equation

$$k_p = \frac{E_s}{m(1 - \mu_s^2)\sqrt{LD}} , \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(17)$$

where LD = the projected area of the pile ;

m = a numerical factor which depends on the shape of the loaded area.

The coefficient m was tabulated in Table 3 as a function of the ratio L/D (BROMS, 1964a).

The coefficient k_m governs the lateral deflections caused by the moment M. It is the coefficient of subgrade reaction corresponding to the shape and size of the two equivalent rectangular areas.

WEISSMANN & WHITE (1961) assumed that the soil reactions are uniformly distributed along 1/10 the total length of the member (Fig. 2b) and that the coefficient of subgrade reaction k_m is governed by the shape and size of the reduced area. The coefficient k_m can be evaluated from the equation

$$k_m = \frac{E_s}{m(1 - \mu_s^2)\sqrt{L'D}} , \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(18)$$

where L' = 1/10th the total length of the pile (L)

$= \frac{L}{10}$.

(iii) <u>Lateral deflection of short pile evaluated by BROMS (1964a)</u> - The lateral deflection at the ground surface can be calculated by the principle of superposition with the aid of the coefficients of subgrade reaction k_p and k_m.

The lateral deflection y_p caused by the lateral load P acting at mid-height of the laterally loaded pile, depends on the projected area LD and k_p. The lateral deflections have been calculated by the equation

$$y_p = \frac{P}{DL\,k_p} \quad . \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (19)$$

The lateral deflection y_m at the ground surface caused by a moment M acting at mid-height of a laterally loaded pile has been assumed to be the same as the edge deflection of a plate located on the ground surface and loaded by the same moment M as shown in Fig. 2(b).

WEISSMANN & WHITE (1961) proposed a method for calculating the deflections of stiff plate with an arbitrary shape and size located on the ground surface, and BROMS (1964a) applied this method for predicted lateral deflections at the ground surface of piles. From Fig. 2(b), the deflection at the center of each of the equivalent areas is equal to p_m/k_m, where p_m is the equivalent uniformly distributed pressure acting at top and bottom of the pile. Because the internal moment arm is 0.9L and the soil reactions are distributed over $\frac{L}{10}$, the equivalent pressure p_m is $\frac{10M}{0.9DL^2}$ and the lateral deflections y_m is calculated by equation

$$y_m = \frac{12.35M}{DL^2\,k_m} \quad . \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (20)$$

The coefficient of subgrade reaction increase frequently with depth. BROMS (1964a) suggested that the lateral deflections at the ground surface can be calculated assuming a constant value of the coefficient of subgrade reaction if its numerical value is taken as that corresponding to a depth of 0.25L and 0.50L for free headed and restrained short piles, respectively.

(c) Modulus of elasticity of soil - The initial modulus of elasticity of the soil varies frequently with the direction of loading. WARD et al (1959) have found for the heavily overconsolidated London clay that the initial modulus of elasticity in the lateral directions exceeded the initial modulus in the vertical direction of about 1.6. SKEMPTON (1951) has found that the initial modulus of elasticity for a cohesive soil is approximately proportional to its unconfined compressive strength. Generally the shearing strength of a normally consolidated clay increases with depth while the shearing strength of an overconsolidated clay may increase or decrease with depth, so the initial modulus of elasticity may also increase or decrease with depth.

BROMS (1964a) stated that "remolding of the soil (as a result of pile driving) cause a decrease of the initial modulus and the secant modulus to a distance of approximately one pile diameter from the surface of the pile. Consolidation also causes a substantial increase with time of the shearing strength, of the initial and of the secant moduli for normally or lightly overconsolidated clays, but the shearing strength and the secant modulus for heavily overconsolidated clays may decrease with time".

SKEMPTON (1951) observed that the deflections at working loads (approximately one-half to one-third the ultimate bearing capacity) are proportional to the secant modulus of the soil when the modulus is determined at loads corresponding to between one-half and one-third the ultimate strength of the soil. TERZAGHI & PECK (1948) stated that the secant modulus may be considerably less than the initial tangent modulus of elasticity of the soil.

POULOS (1972) proposed an empirical formula for predicted modulus of elasticity as follows

$$E_s = 400\ c_u \quad , \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (21)$$

where c_u = the undrained shear strength of the clay determined from undrained triaxial, direct shear or vane tests.

(d) <u>Plate load test</u> - The modulus of elasticity E_s and the coefficient of subgrade reaction k_o may be evaluated approximately by plate load tests. WARD et al (1959) found that the results of plate loading tests may underestimate the initial modulus of elasticity of the soil and the coefficient of lateral subgrade reaction for a heavily overconsolidated clay.

It is, in general, assumed for cohesive soils that the E_s is a constant in the analysis of plate load test. For this assumption, the modulus of elasticity will be overestimated if the E_s decreases with depth, but the modulus of elasticity will be underestimated if the E_s increases with depth because the deflections for the plate load test depend mainly on the E_s within a depth about two plate diameter below the ground surface.

TIMOSHENKO & GOODIER (1951) calculated the deflection d_o of a circular plate from the equation

$$d_o = \frac{0.8\ Bq\ (1 - \mu_s^2)}{E_{50}}, \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (22)$$

where B = the diameter of the loaded area ;

q = the intensity of the applied load ;

q/d_o = the coefficient of subgrade reaction (k_o).

BROMS (1964a) substituted $\mu_s = 0.5$ and K_o (defined as k_oB) into Eq. 22 yields

$$K_o = 1.67\ E_{50}\ . \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (23)$$

SKEMPTON (1951) has found that the secant modulus E_{50} is approximately equal to 25 to 100 times the unconfined compressive strength of a cohesive soil. Analysis of test data reported by PECK & DAVISSON (1962) on the behavior of a laterally loaded H-pile driven into a normally consolidated, highly organic silt indicates, at the maximum applied load, that the secant modulus load is approximately equal to 100 times the cohesive strength as measured by field vane tests (50 times the unconfined compressive strength of the soil).

Using a value of E_{50} equal to 25 to 100 times the unconfined compressive strength, the coefficient K_o can be expressed in terms of the unconfined compressive q_u as

$$K_o = (40 - 160)\, q_u \quad \text{.........................(24)}$$

The secant modulus as predicted from Eq. 23 can be used for calculation of the coefficient of lateral subgrade reaction of short piles ($\beta L < 2.25$).

The coefficient of subgrade reaction K_o calculated from plate load tests can also be used directly in the case of long piles ($\beta L > 2.25$) to calculate the corresponding coefficient of subgrade reaction (Eq. 14).

(e) Lateral load tests - A large number of lateral load tests have been carried out on piles driven into cohesive soils (e.g. TERZAGHI, 1943; SHILTS et al, 1948; EVANS, 1953; McCAMMON & ASCHERMAN, 1953; WAGNER, 1953; MATLOCK & RIPPEGER, 1956; BERGFELT, 1957; GAUL, 1958; PECK & IRELAND, 1961; LAZARD & GALLERRAND, 1961; MOHAN & SHRIVASTAVA, 1971; ADAMS & RADHAKRISHNA, 1973; BOTEA et al, 1973; FRANKE, 1973; REESE et al, 1975).

In many cases the available data are difficult to interpret. In general, load tests have been carried out for the purpose of proving to the satisfaction of the owner or the design engineer that the load carrying capacity of a pile or a pile group is sufficiently large to resist a prescribed lateral design load under a specific condition. Generally sufficient data are not available concerning the strength and deformation properties, the average unconfined compressive strength of the cohesive soil.

BROMS (1964a) suggested that coefficients of lateral subgrade reaction can be determined also from lateral load tests on long piles ($\beta L > 2.25$). The coefficient of lateral subgrade reaction (assuming a constant value of this coefficient within the significant depth) can be determined from Eqs. 8 and 9.

McCLELLAND & FOCHT (1852) observed the modulus of elasticity of soil from lateral load tests on long piles.

(f) Deformations caused by consolidation - An increase of the lateral deflections and a redistribution of soil reactions will occur with time because of a result of consolidation and creep of the soil surrounding a laterally loaded pile. The deformations caused by consolidation depend on the nature of the applied load, on the compressibility of the soil, on the redistribution of soil reaction along the pile, on the stress increase in the soil to a distance of soil reaction along the pile (BROMS, 1964a).

BROMS (1964a) assumed that the increase of deflections of a laterally loaded pile caused by consolidation is the same as the increase of deflections (settlements) which take place with time for spread footings and rafts founded at the ground surface or at some depth below the ground surface. SKEMPTON & BJERRUM (1957) has found that the total settlement (the sum of the initial compression and consolidation) of footings and rafts located on stiff to very stiff clays is approximately equal to two to four times the initial settlements caused by shear deformations of the soils. BROMS assumed that the apparent coefficient of subgrade reaction for the soil which governs the long-time lateral deflections and the long-time distribution of lateral earth pressures should be taken as ½ to ¼ the initial coefficient of subgrade reaction.

SKEMPTON & BJERRUM (1957) found that the total settlements for normally consolidated clays are approximately three to six times the initial settlements which take place at the time of loading. The corresponding apparent coefficient of subgrade reaction governing the long-time pressure distribution of piles driven into soft and very soft clays may be taken as 1/3 to 1/6 the initial value (Eq. 24).

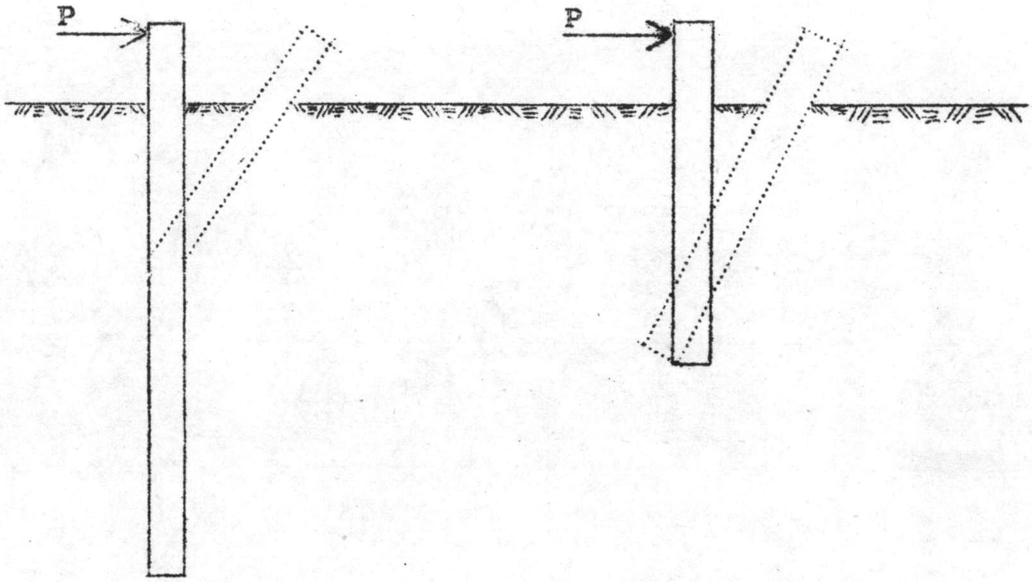
BROMS (1964a) concluded that the increase of lateral deflection in the case of long pile ($\beta L > 2.5$) caused by consolidation and creep is less than that of a short pile ($\beta L < 1.5$).

The increase in lateral deflections caused by consolidation may also be calculated by means of a settlement analysis based on the assumption proposed by BROMS (1964a) that the distribution of soil reactions along the laterally loaded piles is governed by a reduced coefficient of lateral soil reaction, that the distribution of the soil pressure within the soil located in front of the laterally loaded pile can be calculated, for example, by the 2:1 method or by any other suitable method and that the compressibility of the soil can be evaluated by consolidation tests or from empirical relationships. The 2:1 method assumes that the applied load is distributed over an area which increases in proportion to the distance to the applied load. This method closely approximates the stress distribution calculated by the theory of elasticity along the axis of loading. It should be noted that Brom's proposed methods of calculating lateral deflections have not been substantiated by test data, they should be used with caution.

2.2 Ultimate Lateral Resistance

2.2.1 General The deflections of a laterally loaded pile increase approximately linearly with the applied load at low load levels, and the lateral deflections increase very rapidly with increasing applied load when the ultimate capacity is reached. Failure of free-headed piles may take place by any of the failure mechanisms shown in Fig. 3. These failure modes are proposed by BROMS (1964a) and discussed below

(After BROMS, 1964a)

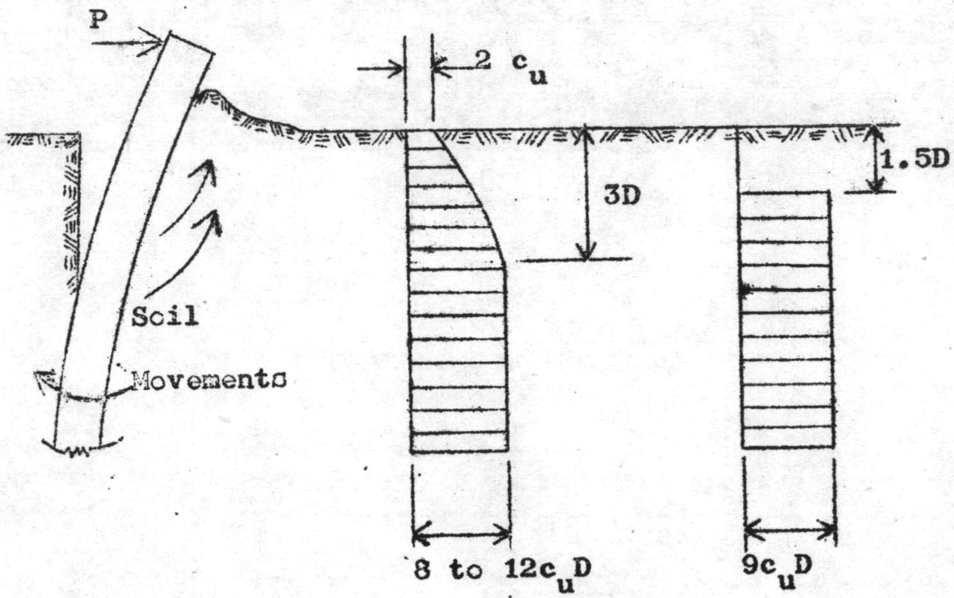


(a) Long Pile (b) Short Pile

Fig. 3 - Failure Modes for Free-Headed Piles

2.2.2 Unrestrained Piles The resulting distribution of lateral earth pressures and the failure mechanism along a laterally loaded free-headed pile driven into a cohesive soil is shown in Fig. 4. BROMS (1964a) assumed that the soil located in front of the loaded pile close to the ground surface move upwards in the direction of least resistance, while the soil located at some depth below the ground surface moves in a lateral direction from the front to the back side of the pile, and it has been observed that the soil separates from the pile on its back side down to a certain depth below the ground surface.

(After BROMS, 1964a)



(a) Deflections and Soil Movements

(b) Probable Distribution of Soil Reactions

(c) Assumed Distribution of Soil Reactions

Fig. 4 - Distribution of Lateral Earth Pressures

Based on the assumption that the shape of a circular section can be approximated by that of a square. BRINCH-HANSEN (1948) has shown that the ultimate soil reaction against a laterally loaded pile driven into a cohesive material varies between 8.3 c_u and 11.4 c_u, where the cohesive strength c_u is equal to half the unconfined compressive strength of the soil.

REESE (1958) has shown that the ultimate soil reaction increases at failure from approximately 2 c_u at the ground surface to 12 c_u at a depth of approximately three pile diameters below the ground surface. McKENZIE (1955) has found from experiments that the maximum lateral resistance is equal to approximately 8 c_u, while DASTIDAR (1956) used a value of 8.5 c_u when calculating the restraining effects of piles driven into a cohesive soil.

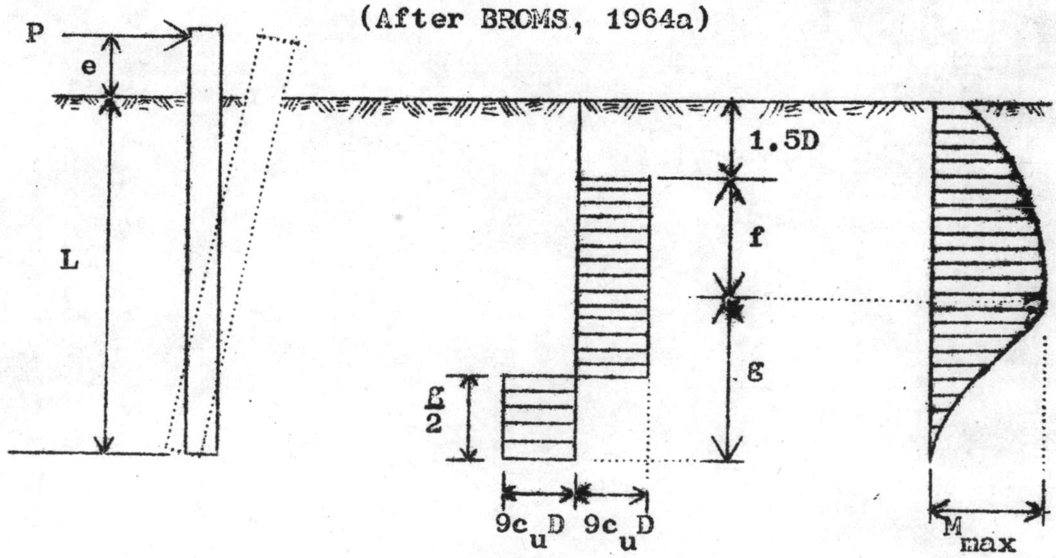
Repeating loads, such as those caused by wind forces or wave forces, cause a gradual decrease of the shear strength of the soil located close to the loaded piles. In the case of over-consolidated soil, the applied lateral load may cause a decrease of the pore pressure and as a result, gradual swelling and loss in shear strength may take place as water is absorbed from any available source. The testing data collected by BROMS (1964a) suggested that repeated loading could decrease the ultimate lateral resistance of the soil approximately one-half its initial value.

The ultimate lateral resistance of a pile group may be considerably less than the ultimate lateral resistance calculated as the sum of the ultimate resistances of the individual piles. DONOVAN (1959) found that the piles and the soil located within the pile group behaved as a unit when the piles were closer than about two pile diameters, while no reduction in lateral resistance when the pile spacing exceeded four pile diameters.

The probable distribution of lateral soil reactions is shown in Fig. 4(b). On the basis of the measured and calculated lateral resistances, BROMS (1964a) proposed the rectangular distribution substitute for the probable distribution as shown in Fig. 4(c). It has been assumed that the lateral soil reaction is equal to zero to a depth of 1½ pile diameters and equal to 9.0 $c_u D$ below this depth. The predicted maximum bending moment and required penetration depth by assuming the rectangular distribution of lateral earth pressures as shown in Fig. 4(c), will be larger than that corresponding to the probable pressure distribution at failure, however it will yield results which are on the safe side.

(a) Short piles - The distribution of soil reactions and bending moments along short pile at failure is shown in Fig. 5.

(After BROMS, 1964a)



(a) Deflection (b) Soil Reaction Distribution (c) Bending Moment Distribution

Fig. 5 - Deflection, Soil Reaction and Bending Moment Distribution for a Short Free-Headed Pile

Failure takes place when the soil yields along the total length of the pile, and the pile rotates as a unit around a point located at some depth below the ground surface. The maximum moment occurs at the level where the total shear force in the pile is equal to zero at a depth (f + 1.5D) below the ground surface. The distance f and the maximum bending moment M^{pos}_{max} can then be calculated by a method proposed by BROMS (1964a) with the two equilibrium equations:

$$f = \frac{P}{9\ c_u\ D} \qquad \text{.........................(25)}$$

$$M^{pos}_{max} = P\ (e + 1.5D + 0.5f), \qquad \text{....................(26)}$$

where e = the eccentricity of the applied load.

The lower part of the pile with length g resists the bending moment M^{pos}_{max}. Then from equilibrium equation

$$M^{pos}_{max} = 2.25\ c_u\ Dg^2, \qquad \text{.........................(27)}$$

where g = length of pile located below the point of maximum bending moment;

$$L = 1.5D + f + g. \qquad \text{...........................(28)}$$

Thus, the ultimate lateral resistance of a short pile driven into a cohesive soil can be calculated from Eqs. 25, 26, 27 and 28. The ultimate lateral resistance can also be determined directly from chart proposed by BROMS (1964a) where the dimensionless ultimate lateral resistance $P_{ult}/c_u\ D^2$ has been plotted as a function of the dimensionless embedment length L/D (Appendix B). BROMS (1964a) assumed in this analysis that the corresponding maximum bending moment M_{max}^{pos} calculated from Eqs. 26 and 27 is less than the ultimate or yield moment resistance of the pile section M_{yield}.

(b) Long piles - The mechanism of failure for a long pile when a plastic hinge forms at the location of the maximum bending moment is shown in Fig. 6(a).

(After BROMS, 1964a)

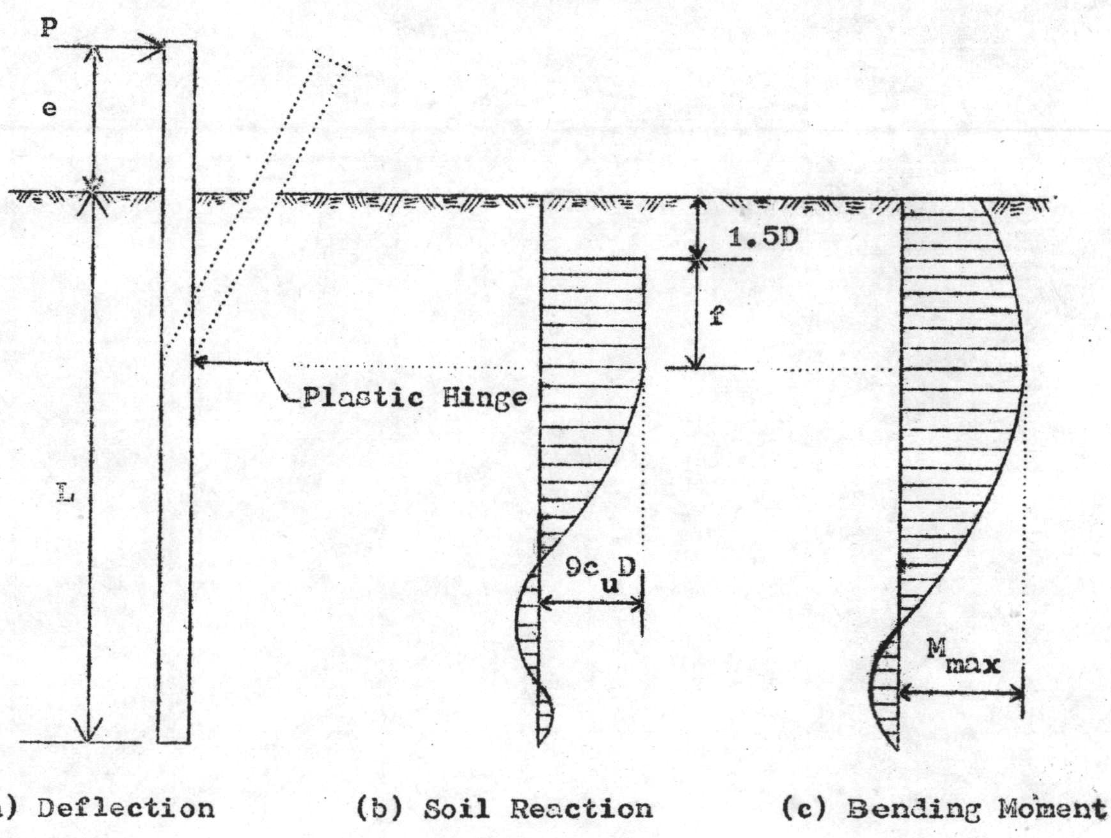


Fig. 6 - Deflection, Soil Reaction and Bending Moment Distribution for a Long Free-Headed Pile

BROMS (1964a) proposed that failure takes place when the maximum bending moment as calculated from Eq. 26 is equal to the moment resistance of the pile section. The assumed distribution of lateral earth pressures and bending moments is shown in Figs. 6(b) and 6(c). BROMS (1964a) assumed that the lateral deflections are large enough to develop the full passive resistance of the soil down to the depth corresponding to the location of the maximum bending moment in the pile. The corresponding dimensionless ultimate lateral resistance $P_{ult}/c_u D^2$ has been plotted by BROMS (1964a) as a function of dimensionless moment resistance of the pile section $M_{yield}/c_u D^3$ as shown in Appendix B,

where M_{yield} = the yield moment of the pile section.

2.3 Lateral Load Tests

2.3.1 Types of Load Test

(a) Maintained load test (ML) - The maintained load test employed for testing piles, and by far is the most common. The procedure adopted is to apply static loads in increments of the anticipated working load. Increment of 0,25,50,75,100, 0,100,125,150,175 and 200% of the working load are often employed. Each load is maintained until the deflection has ceased or has diminished to an acceptable rate or until a certain time period. The working load and twice the working load are maintained on the pile for 24 hours or sometimes longer. If the load is increased to failure, this is done by reducing the increments where failure is imminent so that ultimate load capacity can be accurately measured.

(b) Constant rate of deflection test (CRD) - This method was first suggested by WHITAKER (1963). In this test the pile is made to deflect the soil at the ground surface at a constant speed from its position, the force applied at the pile to maintain the rate of deflection being continuously measured. The time to reach ultimate bearing capacity should be approximately the same as the time would be taken in making a "quick" shear test of the soil in the laboratory (in the unconfined compression test 0.0012 in./min.) but the rate does not significantly affect the ultimate load

(WHITAKER, 1963). In practice a duration of test of about 10 minutes was found suitable for piles in clay. As the ultimate load capacity is approached very little increase in load is required to maintain a constant rate of deflection, and the ultimate bearing capacity is reached when the continuous lateral movements result in no increase in the deflection resistance.

It should be noted that the CRD is used for laterally loaded pile but for vertically loaded pile the constant rate of penetration test (CRP) is used.

Good agreement has been found to exist between the ultimate loads measured by the ML and CRP tests however has been voiced by ELLISON et al (1971) on the grounds that it does not represent the type of loading to which a pile is subjected during its working life. They also reported that this test tended to overestimate the ultimate load capacity of bored piles in London clay.

It is obvious that the deflection recorded for a given applied load in the CRD test will always be lower than the comparative deflection for the ML test, because no time is permitted for plastic deflection under sustained load this is a disadvantage of the CRD test. Otherwise, the CRD test has the great advantage that it can be carried out very quickly. As long as sufficient experience is gathered with the CRD test in the prevailing subsoil condition and if initial correlation are made the CRD test is generally to be preferred to the ML test.

(c) Quick maintained load test (Quick test) - The CRD test calls for records of time and jacking force to be made at equal interval of movements of the pile head with the rate of jacking being adjusted so that reading occur at equal intervals of time. For convenience and simplicity, the CRD test was modified by the Texas Highway Department to produce the quick test method (FULLER & HOY, 1970; FELLENIUS, 1975). Essentially, it requires that loads be added in increments of 5 or 10 tons with gross deflection readings, loads and other data recorded immediatly before and after the application of each increment of load. Each increment is held for 2½ minutes, and the next increment is then applied.

When the load-deflection curve obtained from the test data shows that the pile is definitely being failed (i.e. the load at the pile can be held only by constant pumping of the hydraulic jack and the pile deflects laterally) pumping is stopped. Gross deflection readings, loads and other data are recorded immediately after pumping has ceased and again after intervals of 2½ minutes and 5 minutes. The load in the case of constant pumping is called plunging failure load. Then all load is removed, and the pile is allowed to recover. Net deflection reading are made immediately after all load has been removed and at intervals of 2½ minutes for a total period of 5 minutes.

All test loads are carried to plunging failure or to the capacity of the equipment. The maximum proven design load is considered to be 50% of the ultimate bearing capacity, which is indicated by the intersection of lines drawn tangent to the 2 basic portions of the load-deflection curve.

(d) Repeated loading test - In the loading tests performed by the Department of Public Work Amsterdam every load was repeatedly applied after unloading. In the repetitive tests, test piles were subjected to some repetitions of loads. These loads were applied as single increments except for some numbers of applications where in the loads were applied in two or three increments.

For each number of repeated load the deflections at ground surface at elapsed time of 0, 2, 4, 6 minutes are recorded and the deflections after rebound are recorded at elapsed time of 0, 2, 4, 6 minutes.

2.3.2 Criteria of Failure In order to measure, specify or discuss the ultimate load capacity of a pile it is necessary to establish what is to be understood by "failure" where a maximum load is reached which either drops or is sustained as the pile deflections increased, the definition of failure presents no problem as long as the deflection at which this state is reached, is tolerable. For many piles this ideal failure criterian annot be applied (BRAND et al, 1972) and it becomes necessary to define failure in terms of some rather arbitrary value of the pile deflection. It is impossible to establish one maximum permissible deflection for all piles under all circumstances, and the many existing criteria of failure based on allowable deflection have generally been established to take account of the worst combination of circumstances.

a) Ultimate load criteria - The following recommendations are used as guide line for determining the ultimate load on the pile.

1. Draw tangent lines to the general slopes of the upper and lower portions of the curve, observe the load at their intersection.

2. Observe the load at which is produced an increase in deflection disproportionate to the increase in load. (Los Angeles Bldg. Code.)

3. Observe the failure load considered as somewhere in the vicinity of the break in the curve showing increased deflection per unit of load added. (Bethlehem Steel Co.)

4. Observe the maximum test load in a case where deflection is not excessive and where load and deflection were proportionate and the curve remained a straight line. (U.S. Steel Co.)

5. Observe the point at which the gross deflection begins to exceed 0.03 in. per ton of additional load. (W. H. Rabe, Bureau of Bridges, State of Ohio.)

6. Observe the point at which the gross deflection begins to exceed 0.05 in. per ton of additional load, or at which the plastic deformation begins to exceed 0.03 in. per ton of additional load. (Dr. R. L. Nordlund, Raymond Concrete Pile Co.)

7. Maximum load which causes a net deflection not exceeding 0.01 in. per ton of test load. (Building laws of the City of New York.)

8. Observe the point at which the plastic curve breaks sharply.

b) Deflection criteria —

For the deflection criteria the following recommendations are used.

9. McNULTY (1956) studied a number of piles and arrived at the allowable lateral-pile loads. It was concluded that lateral-pile movements should be limited to not over ¼ in. for buildings. Other structures might tolerate a somewhat larger movement.

10. TERZAGHI (1943) suggested that the criterion of failure of a single pile should be taken as a deflection of 0.1 D. This will lead to extremely large deflections however, for large diameter piles under their design loads. Such a criterion also has the disadvantage that it does not differentiate between elastic and plastic deflection.