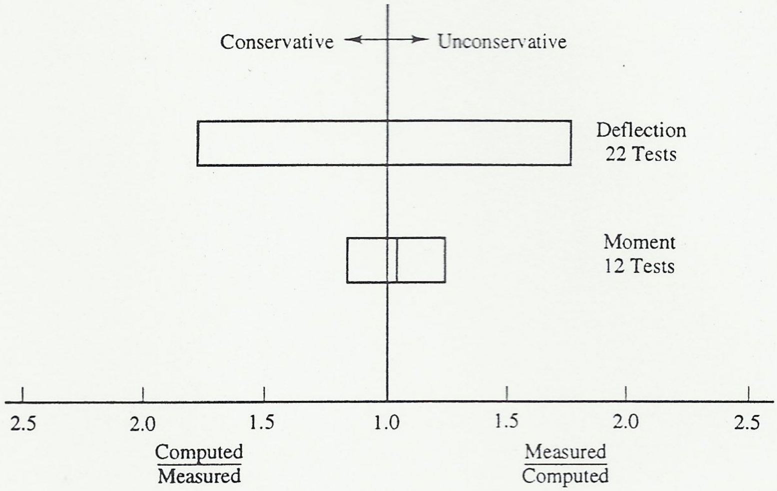

Figure 17.19 90% confidence intervals for computed lateral deflection and bending moment predictions from *p-y* analysis (based on data from Reese and Wang, 1986). The line in the middle of each bar represents the average prediction, and the number to the right is the number of data points.

Evans and Duncan's Method

Evans and Duncan (1982) developed a convenient method of expressing the lateral load-deflection behavior in chart form. They compiled these charts from a series of *p-y* method computer analyses using the computer program COM624.

The advantages of these charts include the following:

- The analyses can be performed more quickly, and they do not require the use of a computer.
- The load corresponding to a given pile deflection can be determined directly, rather than by trial.
- The load corresponding to a given maximum moment in the pile can be determined directly, rather than by trial.

These charts are also a useful way to check computer output from more sophisticated analyses.

The charts presented here are a subset of the original method and apply only to deep foundations that satisfy the following criteria:

- The stiffness, *EI*, is constant over the length of the pile.
- The shear strength of the soil, expressed either as s_u or ϕ, and the unit weight, γ, are constant with depth.

- The pile is long enough to be considered fixed at the bottom. For relatively flexible piles, such as timber piles, this corresponds to a length of at least 20 diameters. For relatively stiff piles, such as those made of steel or concrete, the length must be at least 35 diameters.

We can idealize deep foundations that deviate slightly from these criteria, such as tapered piles, by averaging the EI, s_u, ϕ, or γ values from the ground surface to a depth of 8 pile diameters.

Characteristic Load and Moment

Evans and Duncan defined the *characteristic shear load*, V_c, and *characteristic moment load*, M_c, as follows:

$$V_c = \lambda B^2 E R_I \left(\frac{\sigma_p}{E R_I}\right)^m (\varepsilon_{50})^n \tag{17.18}$$

$$M_c = \lambda B^3 E R_I \left(\frac{\sigma_p}{E R_I}\right)^m (\varepsilon_{50})^n \tag{17.19}$$

$$R_I = \frac{I}{\pi B^4/64} \tag{17.20}$$

$= 1.00$ for solid circular cross sections
$= 1.70$ for square cross sections

For plastic clay and sand:

$$\lambda = 1.00 \tag{17.21}$$

For brittle clay[1]:

$$\lambda = (0.14)^n \tag{17.22}$$

[1] A brittle clay is one with a residual strength that is much less than the peak strength.

For cohesive soils:

$$\sigma_p = 4.2\, s_u \tag{17.23}$$

For cohesionless soils:

$$\sigma_p = 2\, C_{p\phi}\, \gamma B \tan^2(45 + \phi/2) \tag{17.24}$$

Where:

V_c = characteristic shear load
M_c = characteristic moment load
λ = a dimensionless parameter dependent on the soil's stress-strain behavior
B = diameter of foundation
E = modulus of elasticity of foundation
 = 29,000,000 lb/in^2 (200,000 MPa) for steel
 = $15{,}200\, \sigma_r\, (f_c' / \sigma_r)^{0.5}$ for concrete
 = 1,600,000 lb/in^2 (11,000 MPa) for Southern Pine or Douglas Fir
f_c' = 28-day compressive strength of concrete
σ_r = reference stress = 14 lb/in^2 = 0.10 MPa
R_I = moment of inertia ratio (dimensionless)
σ_p = representative passive pressure of soil
ε_{50} = strain at which 50% of the soil strength is mobilized
m, n = exponents from Table 17.1
I = moment of inertia of foundation
 = $\pi B^4/64$ for solid circular cross-sections
 = $B^4/12$ for square cross-sections
 Also see tabulated values in Chapter 18
s_u = undrained shear strength of soil from the ground surface to a depth of 8 pile diameters
ϕ = friction angle of soil (deg) from ground surface to a depth of 8 pile diameters
$C_{p\phi}$ = passive pressure factor = $\phi/10$
γ = unit weight of soil from ground surface to a depth of 8 pile diameters. If the groundwater table is in this zone, use a weighted average of γ and γ_b, where γ_b is the buoyant unit weight in the zone below the groundwater table.

The value of ε_{50} could be determined from triaxial compression tests. Typically, $\varepsilon_{50} \approx 0.01$ for clays and $\varepsilon_{50} \approx 0.002$ for medium dense sands containing little or no mica.

TABLE 17.1 VALUES OF EXPONENTS m AND n FOR USE IN EQUATIONS 17.18 and 17.19

Soil Type	For V_c		For M_c	
	m	n	m	n
Cohesive	0.683	-0.22	0.46	-0.15
Cohesionless	0.57	-0.22	0.40	-0.15

Evans and Duncan (1982).

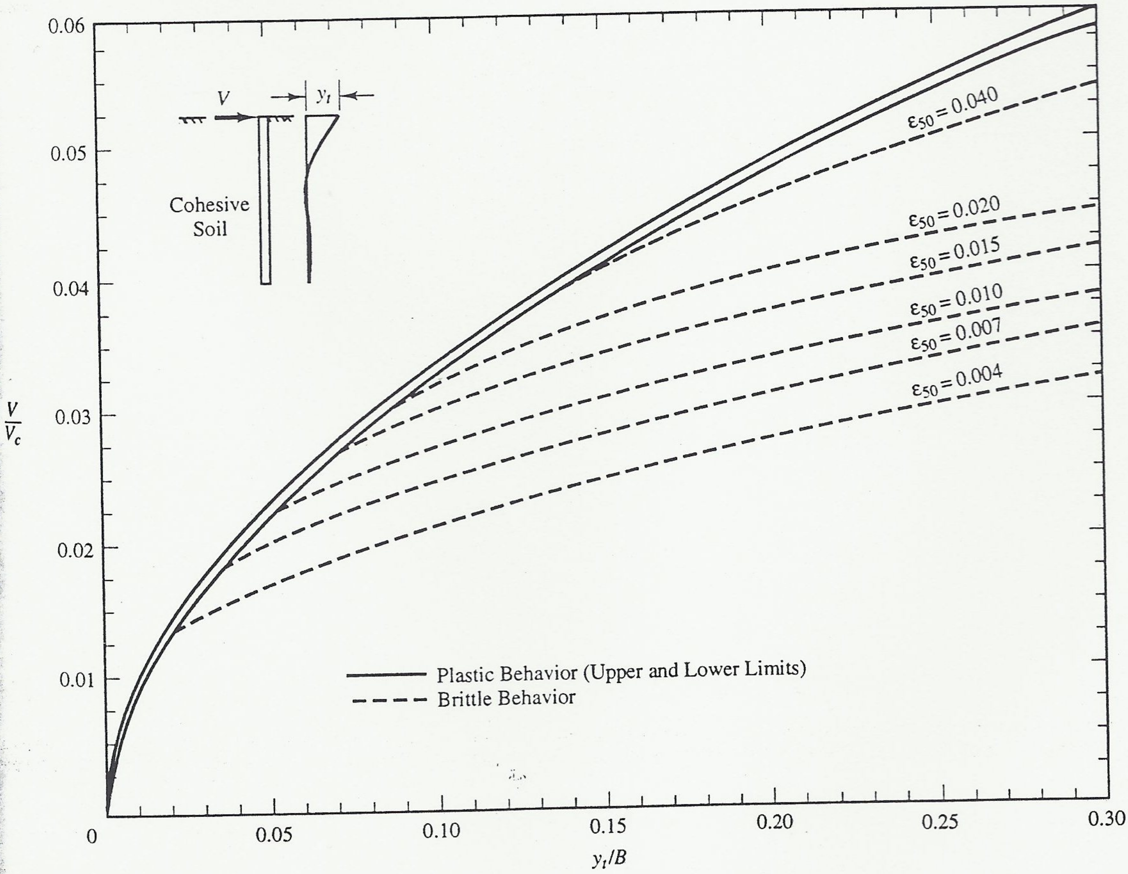

Figure 17.20 Shear load vs. lateral deflection curves for free-head conditions in cohesive soil (Evans and Duncan, 1982).

Using the Charts

The charts in Figures 17.20 to 17.29 express the relationships between the actual shear, moment and deflection, where:

V = applied shear at top of foundation

M = applied moment at top of foundation

M_{max} = maximum moment in foundation

y_t = lateral deflection at top of foundation

Some foundations are subjected to both shear and moment loads. As a first approximation, compute the lateral deflections and moments from each component separately and add them. Alternatively, use the nonlinear superposition procedure described in Evans and Duncan (1982).

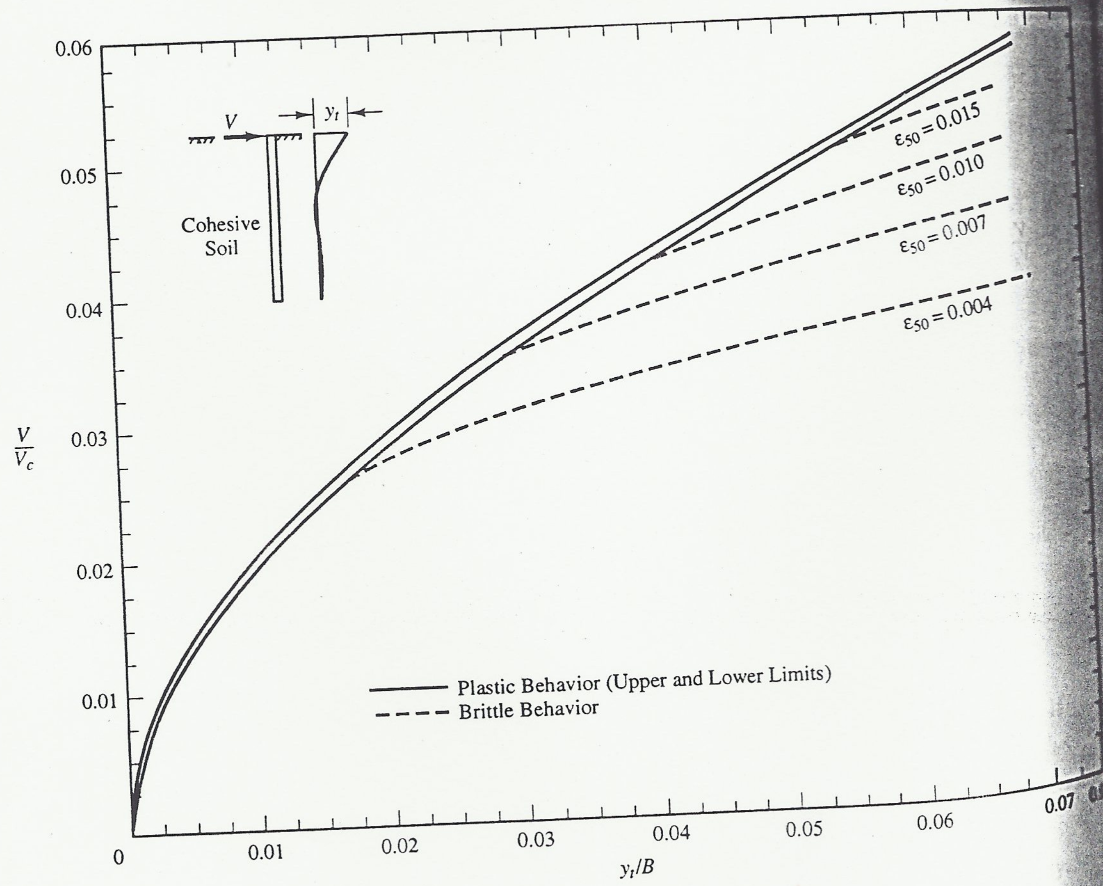

Figure 17.21 Shear load vs. lateral deflection curves for restrained-head condition in cohesive soil (Evans and Duncan, 1982).

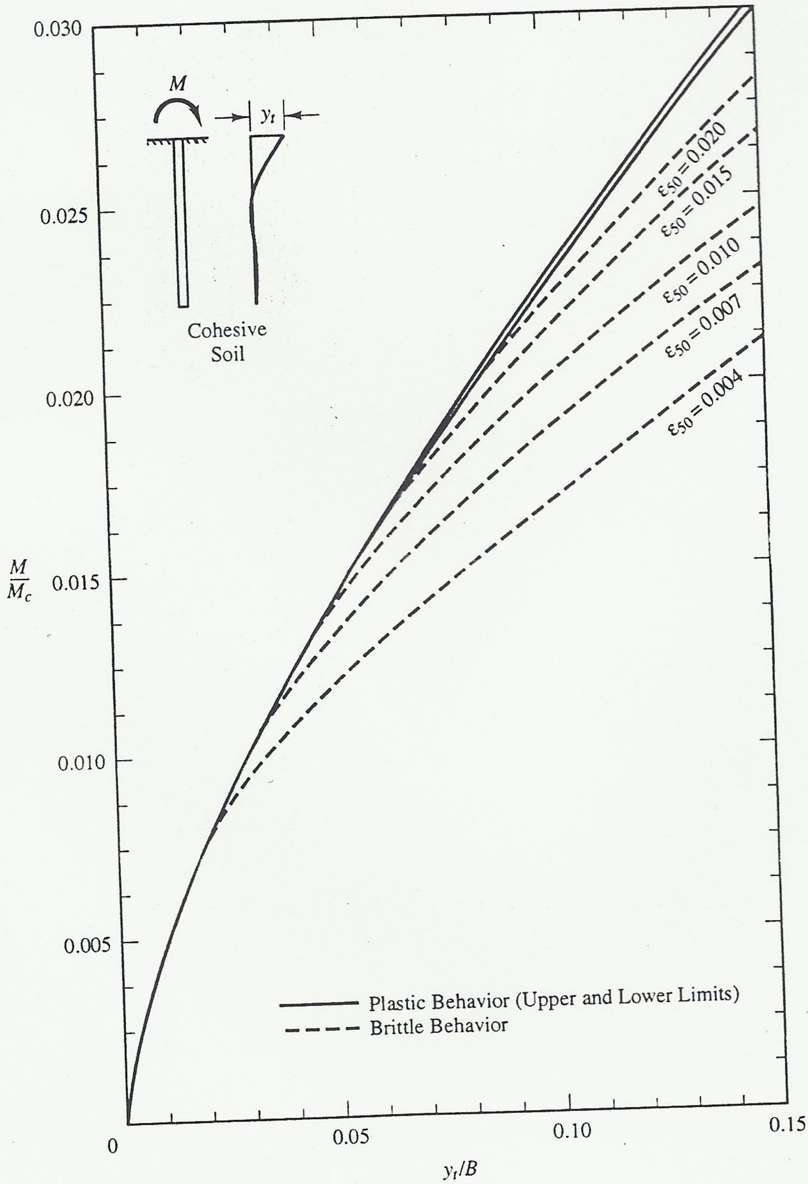

Figure 17.22 Moment load vs. lateral deflection curves for free-head condition in cohesive soil (Evans and Duncan, 1982).

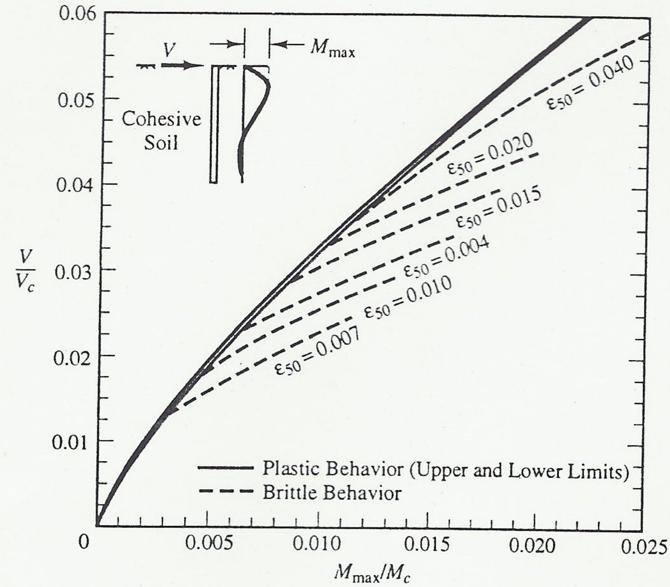

Figure 17.23 Shear load vs. maximum moment curves for free-head condition in cohesive soil (Evans and Duncan, 1982).

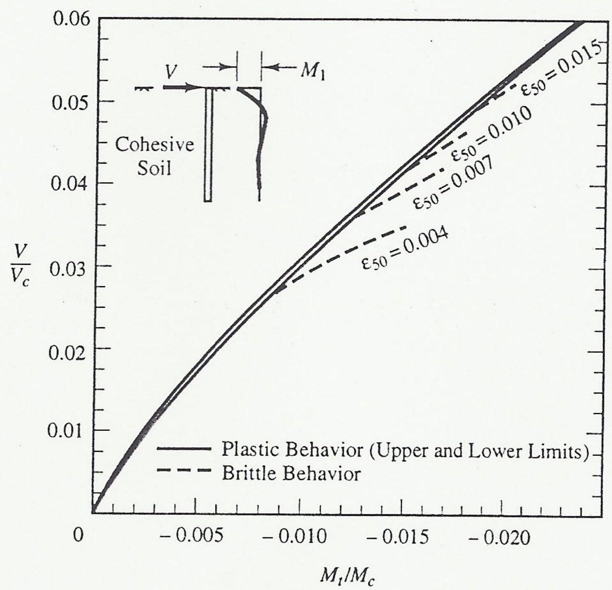

Figure 17.24 Shear load vs. maximum moment curves for restrained-head condition in cohesive soil (Evans and Duncan, 1982).

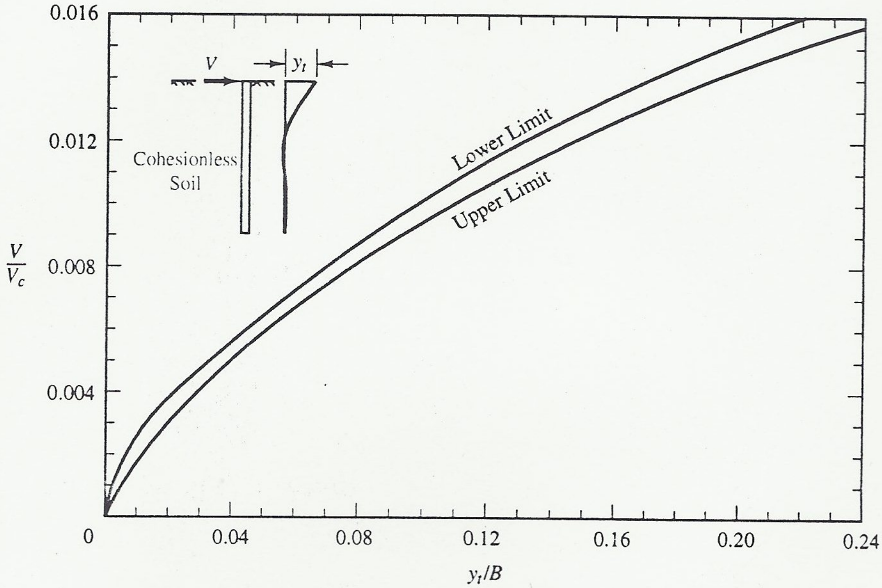

Figure 17.25 Shear load vs. lateral deflection curves for free-head condition in cohesionless soil (Evans and Duncan, 1982).

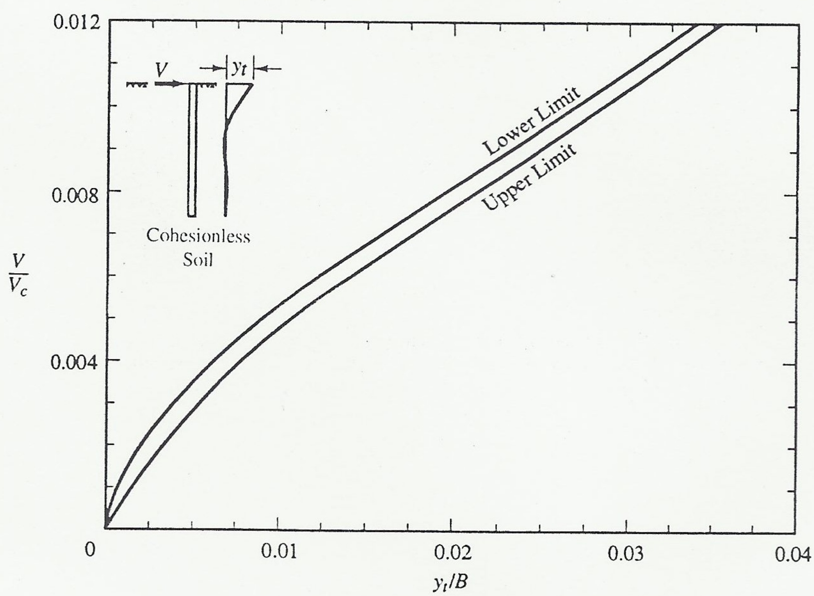

Figure 17.26 Shear load vs. lateral deflection curves for restrained-head condition in cohesionless soil (Evans and Duncan, 1982).

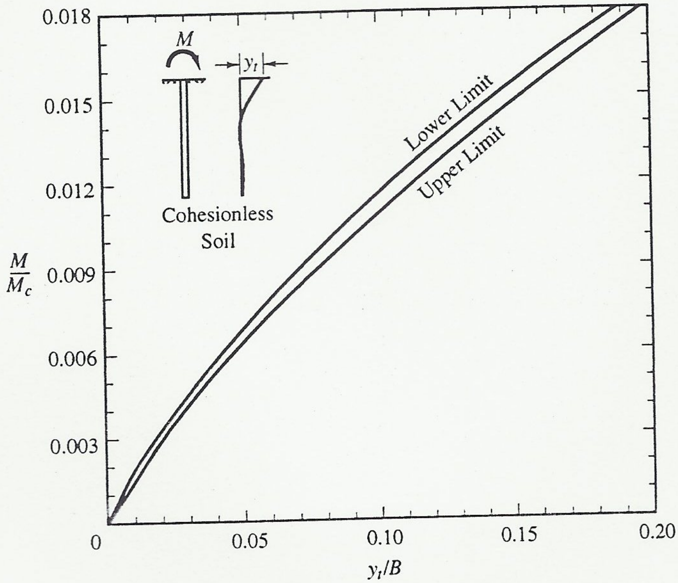

Figure 17.27 Moment load vs. lateral deflection curves for free-head condition in cohesionless soil (Evans and Duncan, 1982).

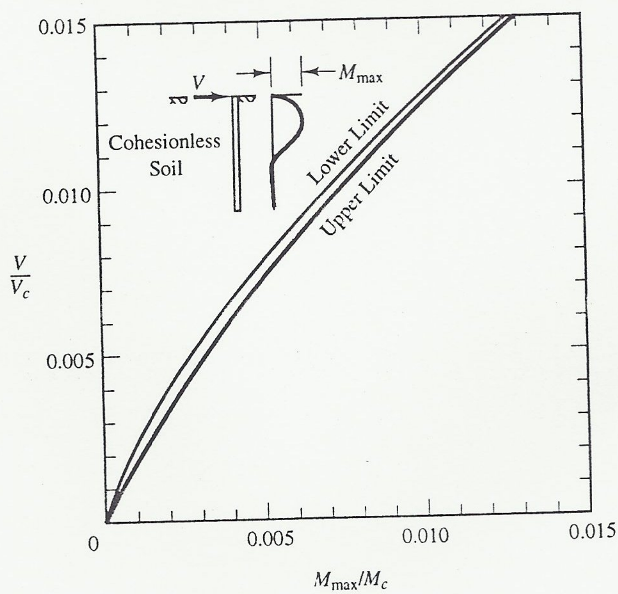

Figure 17.28 Shear load vs. maximum moment curves for free-head condition in cohesionless soils (Evans and Duncan, 1982).

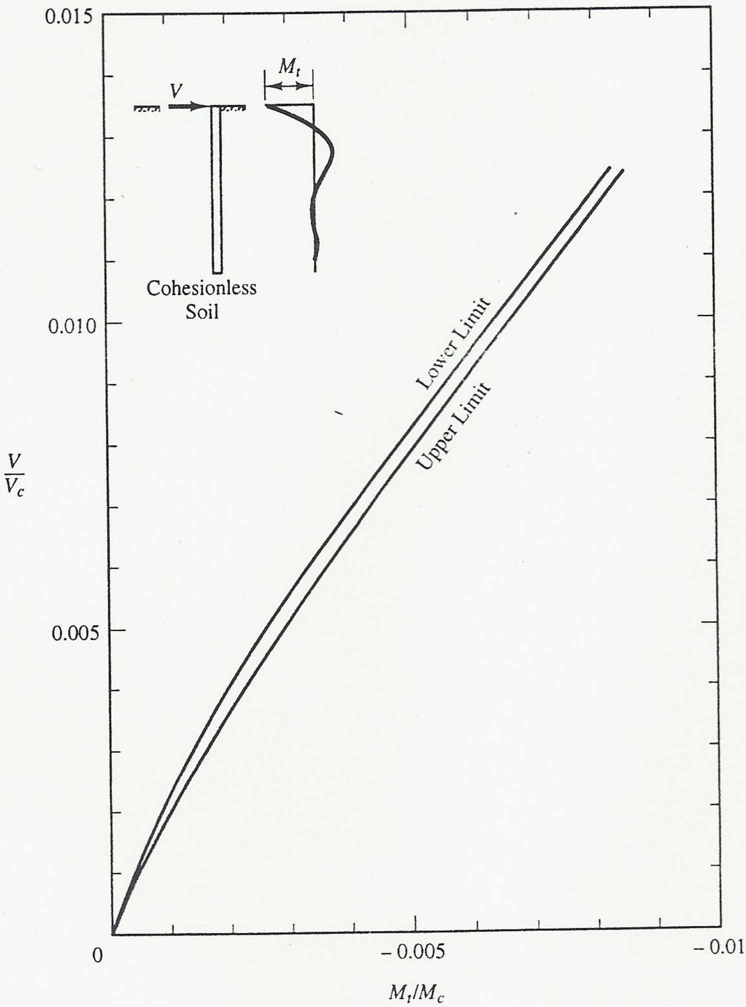

Figure 17.29 Shear load vs. maximum moment curves for restrained-head condition in cohesionless soils (Evans and Duncan, 1982).

Example 17.2

A 20 k shear load will be applied to a 12 inch square, 60 ft long restrained-head concrete pile. The soil is a sand with $\phi = 36°$ and $\gamma = 120$ lb/ft^3. The groundwater table is at a depth of 40 ft. The pile is made of concrete with a 28-day compressive strength of 6000 lb/in^2. Compute the lateral deflection at the top of this pile and the maximum moment.

Solution:

Use units of pounds and inches. $\lambda = 1.00$; $R_I = 1.70$; $\varepsilon_{50} = 0.002$:

$$C_{p\phi} = \frac{\phi}{10} = \frac{36}{10} = 3.6$$

$$\begin{aligned}\sigma_p &= 2\, C_{p\phi}\, \gamma B \tan^2(45 + \phi/2)\\ &= (2)(3.6)\left(\frac{120}{12^3}\right)(12)\tan^2(45 + 36/2)\\ &= 23.1 \text{ lb/in}^2\end{aligned}$$

$$\begin{aligned}E &= 15{,}200\, \sigma_r (f_c' / \sigma_r)^{0.5}\\ &= (15{,}200)(14)(6000/14)^{0.5}\\ &= 4{,}400{,}000 \text{ lb/in}^2\end{aligned}$$

Using Equation 17.18:

$$\begin{aligned}V_c &= \lambda B^2 E R_I \left(\frac{\sigma_p}{E R_I}\right)^m (\varepsilon_{50})^n\\ &= (1.00)(12)^2(4{,}400{,}000)(1.70)\left(\frac{23.1}{(4{,}400{,}000)(1.70)}\right)^{0.57}(0.002)^{-0.22}\\ &= 3{,}056{,}000 \text{ lb}\end{aligned}$$

$$V/V_c = 20{,}000/3{,}056{,}000 = 0.0065$$

From Figure 17.26: $y_t/B = 0.0150$

$y_t = (0.0150)(12) = \mathbf{0.18\ in}$ $\Leftarrow$ *Answer*

Using Equation 17.19:

$$\begin{aligned}M_c &= \lambda B^3 E R_I \left(\frac{\sigma_p}{E R_I}\right)^m (\varepsilon_{50})^n\\ &= (1.00)(12)^3(4{,}400{,}000)(1.70)\left(\frac{23.1}{(4{,}400{,}000)(1.70)}\right)^{0.40}(0.002)^{-0.15}\\ &= 205{,}200{,}000 \text{ in–lb}\end{aligned}$$

From Figure 17.28: $M_{max}/M_c = 0.0041$

$$M_{max} = (0.0041)(205{,}200{,}000) = \mathbf{841{,}000\ in\text{-}lb} \qquad \Leftarrow \textit{Answer}$$

17.6 GROUP EFFECTS

The analysis of lateral loads becomes more complex when we consider pile groups. There are two basic questions:

- How are the applied loads distributed among the piles in the group?
- How does the ultimate capacity and load-deflection behavior of the group compare to that of a single isolated pile?

Unfortunately, both are difficult to evaluate, largely because of the many factors that influence group behavior (O'Neill, 1983). These factors include the following:

- The number, size, spacing, orientation, and arrangement of the piles
- The soil type
- The type of connection at the top (fortunately, group piles are connected with a pile cap, so only the restrained-head condition need be considered)
- The interaction between the cap and the piles
- The vertical contact force between the cap and the soil
- The lateral resistance developed between the side of the cap and the soil
- The differences between the *p-y* curves for the inner piles and those for the leading row of piles
- The method and sequence of pile installation
- The as-built inclination of the piles (although the design drawings may show them perfectly plumb, in reality they will have some accidental batter)

Theories have been developed that consider some of these factors (e.g., Poulos, 1979), but no comprehensive method has yet been proposed. The limited load test data reported in the literature also have not produced a clear picture.

An important characteristic to define is *pile - soil - pile interaction (PSPI)* (O'Neill, 1983). This mechanism works as follows: The lateral movement of a pile relieves some of the stress on the soil behind it. This soil, in turn, provides less resistance to lateral movement of the next pile. Thus, different piles may have different *p-y* curves. This has also been called the *shadow effect*.

PSPI is most pronounced when the piles are closely spaced. Because of this effect, the leading row of piles carries more than a proportionate share of the load. Some load test data confirm this behavior (Holloway et al., 1982). The net result is that the lateral deflection of a pile group will be greater than that of a single isolated pile subjected to proportionate share of the group load. For conventionally spaced onshore pile groups, this ratio may be on the order of 2 to 3. Bogard and Matlock (1983) and Brown and Shie (1990) have suggested methods of computing group deflections that consider this effect, but both are in preliminary form.

Response to Applied Shear Loads

Because of PSPI, the leading row of piles will carry more than its share of the applied shear load. For design purposes, it may be reasonable to assume that each pile in the leading row carries twice its share of the applied shear load:

$$V \approx \frac{2\,V_g}{N} \tag{17.25}$$

Where:

V = shear load on a leading row pile
V_g = applied shear load on pile group
N = number of piles in the group

A complete analysis also might consider the lateral resistance on the side of a buried pile cap. It may be appropriate to consider a portion of the passive earth pressure acting on the leading face less the active pressure acting on the trailing face. Methods for computing these pressures are described in Chapter 23. However, the lateral deflection required to mobilize the full passive pressure may be greater than the allowable lateral deflection (see Table 23.2). It would be prudent to ignore this resistance when scour, settlement, or some other mechanism might threaten the integrity of the cap-soil contact. It is always appropriate to ignore the shear resistance along the bottom of the cap.

Response to Applied Moment Loads

A moment load applied to a pile group induces uplift and downward loads on the individual piles. To compute these loads, assume pinned connections exist between the piles and the cap and that the incremental load in each pile is proportional to its incremental axial displacement, as defined in Equations 17.26 and 17.27 and Figure 17.30. Add this incremental axial load to that from the applied axial load.

$$M_g = \sum P_i r_i \tag{17.26}$$