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# 32

# Deep Foundations




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## 32.1 Introduction

A bridge foundation is part of the bridge substructure connecting the bridge to the ground. A foundation consists of man-made structural elements that are constructed either on top of or within existing geologic materials. The function of a foundation is to provide support for the bridge and to transfer loads or energy between the bridge structure and the ground.

A deep foundation is a type of foundation where the embedment is larger than its maximum plane dimension. The foundation is designed to be supported on deeper geologic materials because either the soil or rock near the ground surface is not competent enough to take the design loads or it is more economical to do so.

The merit of a deep foundation over a shallow foundation is manifold. By involving deeper geologic materials, a deep foundation occupies a relatively smaller area of the ground surface. Deep foundations can usually take larger loads than shallow foundations that occupy the same area of the ground surface. Deep foundations can reach deeper competent layers of bearing soil or rock, whereas shallow foundations cannot. Deep foundations can also take large uplift and lateral loads, whereas shallow foundations usually cannot.

The purpose of this chapter is to give a brief but comprehensive review to the design procedure of deep foundations for structural engineers and other bridge design engineers. Considerations of selection of foundation types and various design issues are first discussed. Typical procedures to calculate the axial and lateral capacities of an individual pile are then presented. Typical procedures to analyze pile groups are also discussed. A brief discussion regarding seismic design is also presented for its uniqueness and importance in the foundation design.

## 32.2 Classification and Selection

### 32.2.1 Typical Foundations

Typical foundations are shown on Figure 32.1 and are listed as follows:

A *pile* usually represents a slender structural element that is driven into the ground. However, a pile is often used as a generic term to represent all types of deep foundations, including a (driven) pile, (drilled) shaft, caisson, or an anchor. A *pile group* is used to represent various grouped deep foundations.

A *shaft* is a type of foundation that is constructed with cast-in-place concrete after a hole is first drilled or excavated. A *rock socket* is a shaft foundation installed in rock. A shaft foundation also is called a *drilled pier* foundation.

A *caisson* is a type of large foundation that is constructed by lowering preconstructed foundation elements through excavation of soil or rock at the bottom of the foundation. The bottom of the caisson is usually sealed with concrete after the construction is completed.

An *anchor* is a type of foundation designed to take tensile loading. An anchor is a slender, small-diameter element consisting of a reinforcement bar that is fixed in a drilled hole by grout concrete. Multistrain high-strength cables are often used as reinforcement for large-capacity anchors. An *anchor for suspension bridge* is, however, a foundation that sustains the pulling loads located at the ends of a bridge; the foundation can be a deadman, a massive tunnel, or a composite foundation system including normal anchors, piles, and drilled shafts.

A *spread footing* is a type of foundation that the embedment is usually less than its smallest width. Normal spread footing foundation is discussed in detail in Chapter 31.

### 32.2.2 Typical Bridge Foundations

Bridge foundations can be individual, grouped, or combination foundations. Individual bridge foundations usually include individual footings, large-diameter drilled shafts, caissons, rock sockets, and deadman foundations. Grouped foundations include groups of caissons, driven piles, drilled shafts, and rock sockets. Combination foundations include caisson with driven piles, caisson with drilled shafts, large-diameter pipe piles with rock socket, spread footings with anchors, deadman with piles and anchors, etc.

For small bridges, small-scale foundations such as individual footings or drilled shaft foundations, or a small group of driven piles may be sufficient. For larger bridges, large-diameter shaft foundations, grouped foundations, caissons, or combination foundations may be required. Caissons, large-diameter steel pipe pile foundations, or other types of foundations constructed by using the cofferdam method may be necessary for foundations constructed over water.

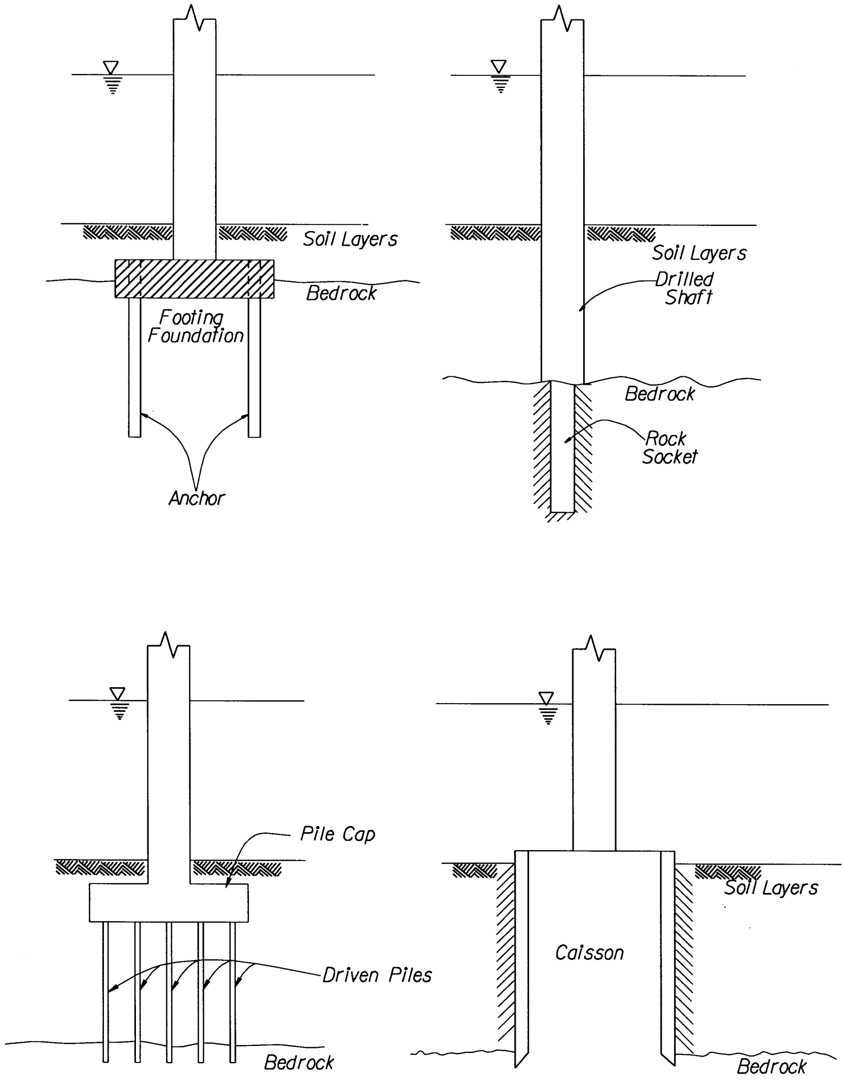


**FIGURE 32.1** Typical foundations.

Bridge foundations are often constructed in difficult ground conditions such as landslide areas, liquefiable soil, collapsible soil, soft and highly compressible soil, swelling soil, coral deposits, and underground caves. Special foundation types and designs may be needed under these circumstances.

## 32.2.3 Classification

Deep foundations are of many different types and are classified according to different aspects of a foundation as listed below:

*Geologic conditions* — Geologic materials surrounding the foundations can be soil and rock. Soil can be fine grained or coarse grained; from soft to stiff and hard for fine-grained soil, or from loose to dense and very dense for coarse-grained soil. Rock can be sedimentary, igneous, or metamorphic; and from very soft to medium strong and hard. Soil and rock mass may possess predefined weaknesses and

**TABLE 32.1** Range of Maximum Capacity of Individual Deep Foundations

| Type of Foundation | Size of Cross Section | Maximum Compressive Working Capacity |
|---|---|---|
| Driven concrete piles | Up to 45 cm | 100 to 250 tons (900 to 2200 kN) |
| Driven steel pipe piles | Up to 45 cm | 50 to 250 tons (450 to 2200 kN) |
| Driven steel H-piles | Up to 45 cm | 50 to 250 tons (450 to 2200 kN) |
| Drilled shafts | Up to 60 cm | Up to 400 tons (3500 kN) |
| Large steel pipe piles, concrete-filled; large-diameter drilled shafts; rock rocket | 0.6 to 3 m | 300 to 5,000 tons or more (2700 to 45000 kN) |

discontinuities, such as rock joints, beddings, sliding planes, and faults. Water conditions can be different, including over river, lake, bay, ocean, or land with groundwater. Ice or wave action may be of concern in some regions.

*Installation methods* — Installation methods can be piles (driven, cast-in-place, vibrated, torqued, and jacked); shafts (excavated, drilled and cast-in-drilled-hole); anchor (drilled); caissons (Chicago, shored, benoto, open, pneumatic, floating, closed-box, Potomac, etc.); cofferdams (sheet pile, sand or gravel island, slurry wall, deep mixing wall, etc.); or combined.

*Structural materials* — Materials for foundations can be timber, precast concrete, cast-in-place concrete, compacted dry concrete, grouted concrete, post-tension steel, H-beam steel, steel pipe, composite, etc.

*Ground effect* — Depending on disturbance to the surrounding ground, piles can be displacement piles, low displacement, or nondisplacement piles. Driven precast concrete piles and steel pipes with end plugs are displacement piles; H-beam and unplugged steel pipes are low-displacement piles; and drilled shafts are nondisplacement piles.

*Function* — Depending on the portion of load carried by the side, toe, or a combination of the side and toe, piles are classified as frictional, end bearing, and combination piles, respectively.

*Embedment and relative rigidity* — Piles can be divided into long piles and short piles. A long pile, simply called a pile, is embedded deep enough that fixity at its bottom is established, and the pile is treated as a slender and flexible element. A short pile is a relatively rigid element that the bottom of the pile moves significantly. A caisson is often a short pile because of its large cross section and stiffness. An extreme case for short piles is a spread-footing foundation.

*Cross section* — The cross section of a pile can be square, rectangular, circular, hexagonal, octagonal, H-section; either hollow or solid. A pile cap is usually square, rectangular, circular, or bell-shaped. Piles can have different cross sections at different depths, such as uniform, uniform taper, step-taper, or enlarged end (either grouted or excavated).

*Size* — Depending on the diameter of a pile, piles are classified as pin piles and anchors (100 to 300 mm), normal-size piles and shafts (250 to 600 mm), large-diameter piles and shafts (600 to 3000 mm), caissons (600 mm and up to 3000 mm or larger), and cofferdams or other shoring construction method (very large).

*Loading* — Loads applied to foundations are compression, tension, moment, and lateral loads. Depending on time characteristics, loads are further classified as static, cyclic, and transient loads. The magnitude and type of loading also are major factors in determining the size and type of a foundation (Table 32.1).

*Isolation* — Piles can be isolated at a certain depth to avoid loading utility lines or other construction, or to avoid being loaded by them.

*Inclination* — Piles can be vertical or inclined. Inclined piles are often called battered or raked piles.

*Multiple Piles* — Foundation can be an individual pile, or a pile group. Within a pile group, piles can be of uniform or different sizes and types. The connection between the piles and the pile cap can be fixed, pinned, or restrained.

### 32.2.4 Advantages/Disadvantages of Different Types of Foundations

Different types of foundations have their unique features and are more applicable to certain conditions than others. The advantages and disadvantages for different types of foundations are listed as follows.

#### Driven Precast Concrete Pile Foundations

Driven concrete pile foundations are applicable under most ground conditions. Concrete piles are usually inexpensive compared with other types of deep foundations. The procedure of pile installation is straightforward; piles can be produced in mass production either on site or in a manufacture factory, and the cost for materials is usually much less than steel piles. Proxy coating can be applied to reduce negative skin friction along the pile. Pile driving can densify loose sand and reduce liquefaction potential within a range of up to three diameters surrounding the pile.

However, driven concrete piles are not suitable if boulders exist below the ground surface where piles may break easily and pile penetration may be terminated prematurely. Piles in dense sand, dense gravel, or bedrock usually have limited penetration; consequently, the uplift capacity of this type of piles is very small.

Pile driving produces noise pollution and causes disturbance to the adjacent structures. Driving of concrete piles also requires large overhead space. Piles may break during driving and impose a safety hazard. Piles that break underground cannot take their design loads, and will cause damage to the structures if the broken pile is not detected and replaced. Piles could often be driven out of their designed alignment and inclination and, as a result, additional piles may be needed. A special hardened steel shoe is often required to prevent pile tips from being smashed when encountering hard rock. End-bearing capacity of a pile is not reliable if the end of a pile is smashed.

Driven piles may not be a good option when subsurface conditions are unclear or vary considerably over the site. Splicing and cutting of piles are necessary when the estimated length is different from the manufactured length. Splicing is usually difficult and time-consuming for concrete piles. Cutting of a pile would change the pattern of reinforcement along the pile, especially where extra reinforcement is needed at the top of a pile for lateral capacity. A pilot program is usually needed to determine the length and capacity prior to mass production and installation of production piles.

The maximum pile length is usually up to 36 to 38 m because of restrictions during transportation on highways. Although longer piles can be produced on site, slender and long piles may buckle easily during handling and driving. Precast concrete piles with diameters greater than 45 cm are rarely used.

#### Driven Steel Piles

Driven steel piles, such as steel pipe and H-beam piles, are extensively used as bridge foundations, especially in seismic retrofit projects. Having the advantage and disadvantage of driven piles as discussed above, driven steel piles have their uniqueness.

Steel piles are usually more expensive than concrete piles. They are more ductile and flexible and can be spliced more conveniently. The required overhead is much smaller compared with driven concrete piles. Pipe piles with an open end can penetrate through layers of dense sand. If necessary, the soil inside the pipe can be taken out before further driving; small boulders may also be crushed and taken out. H-piles with a pointed tip can usually penetrate onto soft bedrock and establish enough end-bearing capacity.

#### Large-Diameter Driven, Vibrated, or Torqued Steel Pipe Piles

Large-diameter pipe piles are widely used as foundations for large bridges. The advantage of this type of foundation is manifold. Large-diameter pipe piles can be built over water from a barge, a trestle, or a temporary island. They can be used in almost all ground conditions and penetrate to a great depth to reach bedrock. Length of the pile can be adjusted by welding. Large-diameter pipe piles can also be used as casings to support soil above bedrock from caving in; rock sockets or rock anchors can then be constructed below the tip of the pipe. Concrete or reinforced concrete can be placed inside the pipe after it is cleaned. Another advantage is that no workers are required to work below water or the ground surface. Construction is usually safer and faster than other types of foundations, such as caissons or cofferdam construction.

Large-diameter pipe piles can be installed by methods of driving, vibrating, or torque. Driven piles usually have higher capacity than piles installed through vibration or torque. However, driven piles are hard to control in terms of location and inclination of the piles. Moreover, once a pile is out of location or installed with unwanted inclination, no corrective measures can be applied. Piles installed with vibration or torque, on the other hand, can be controlled more easily. If a pile is out of position or inclination, the pile can even be lifted up and reinstalled.

### Drilled Shaft Foundations

Drilled shaft foundations are the most versatile types of foundations. The length and size of the foundations can be tailored easily. Disturbance to the nearby structures is small compared with other types of deep foundations. Drilled shafts can be constructed very close to existing structures and can be constructed under low overhead conditions. Therefore, drilled shafts are often used in many seismic retrofit projects. However, drilled shafts may be difficult to install under certain ground conditions such as soft soil, loose sand, sand under water, and soils with boulders. Drilled shafts will generate a large volume of soil cuttings and fluid and can be a mess. Disposal of the cuttings is usually a concern for sites with contaminated soils.

Drilled shaft foundations are usually comparable with or more expensive than driven piles. For large bridge foundations, their cost is at the same level of caisson foundations and spread footing foundations combined with cofferdam construction. Drilled shaft foundations can be constructed very rapidly under normal conditions compared with caisson and cofferdam construction.

### Anchors

Anchors are special foundation elements that are designed to take uplift loads. Anchors can be added if an existing foundation lacks uplift capacity, and competent layers of soil or rock are shallow and easy to reach. Anchors, however, cannot take lateral loads and may be sheared off if combined lateral capacity of a foundation is not enough.

Anchors are in many cases pretensioned in order to limit the deformation to activate the anchor. The anchor system is therefore very stiff. Structural failure resulting from anchor rupture often occurs very quickly and catastrophically. Pretension may also be lost over time because of creep in some types of rock and soil. Anchors should be tested carefully for their design capacity and creep performance.

### Caissons

Caissons are large structures that are mainly used for construction of large bridge foundations. Caisson foundations can take large compressive and lateral loads. They are used primarily for over-water construction and sometimes used in soft or loose soil conditions, with a purpose to sink or excavate down to a depth where bedrock or firm soil can be reached. During construction, large boulders can be removed.

Caisson construction requires special techniques and experience. Caisson foundations are usually very costly, and comparable to the cost of cofferdam construction. Therefore, caissons are usually not the first option unless other types of foundation are not favored.

### Cofferdam and Shoring

Cofferdams or other types of shoring systems are a method of foundation construction to retain water and soil. A dry bottom deep into water or ground can be created as a working platform. Foundations of essentially any of the types discussed above can be built from the platform on top of firm soil or rock at a great depth, which otherwise can only be reached by deep foundations.

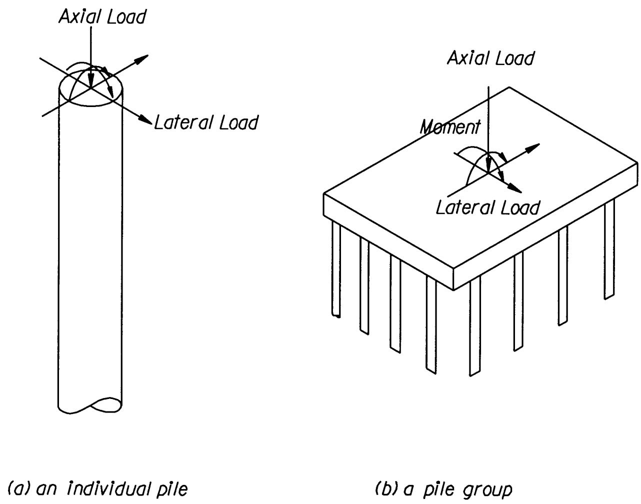


**FIGURE 32.2** Acting loads on top of a pile or a pile group. (a) Individual pile; (b) pile group.

A spread footing type of foundation can be built from the platform. Pile foundations also can be constructed from the platform, and the pile length can be reduced substantially. Without cofferdam or shoring, a foundation may not be possible if constructed from the water or ground surface, or it may be too costly.

Cofferdam construction is often very expensive and should only be chosen if it is favorable compared with other foundation options in terms of cost and construction conditions.

### 32.2.5 Characteristics of Different Types of Foundations

In this section, the mechanisms of resistance of an individual foundation and a pile group are discussed. The function of different types of foundations is also addressed.

Complex loadings on top of a foundation from the bridge structures above can be simplified into forces and moments in the longitudinal, transverse, and vertical directions, respectively (Figure 32.2). Longitudinal and transverse loads are also called horizontal loads; longitudinal and transverse moments are called overturning moments, moment about the vertical axis is called torsional moment. The resistance provided by an individual foundation is categorized in the following (also see Figure 32.3).

*End-bearing*: Vertical compressive resistance at the base of a foundation; distributed end-bearing pressures can provide resistance to overturning moments;
*Base shear*: Horizontal resistance of friction and cohesion at the base of a foundation;
*Side resistance*: Shear resistance from friction and cohesion along the side of a foundation;
*Earth pressure*: Mainly horizontal resistance from lateral Earth pressures perpendicular to the side of the foundation;
*Self-weight*: Effective weight of the foundation.

Both base shear and lateral earth pressures provide lateral resistance of a foundation, and the contribution of lateral earth pressures decreases as the embedment of a pile increases. For long piles, lateral earth pressures are the main source of lateral resistance. For short piles, base shear and end-bearing pressures can also contribute part of the lateral resistance. Table 32.2 lists various types of resistance of an individual pile.

For a pile group, through the action of the pile cap, the coupled axial compressive and uplift resistance of individual piles provides the majority of the resistance to the overturning moment loading. Horizontal (or lateral) resistance can at the same time provide torsional moment resistance.

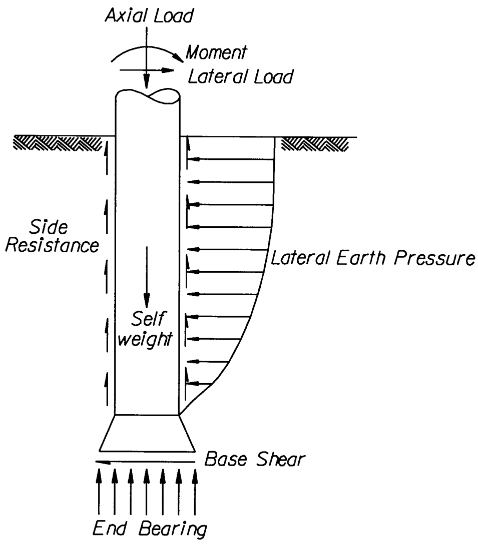


**FIGURE 32.3** Resistances of an individual foundation.

**TABLE 32.2** Resistance of an Individual Foundation

| Type of Foundation | Type of Resistance: Vertical Compressive Load (Axial) | Vertical Uplift Load (Axial) | Horizontal Load (Lateral) | Overturning Moment (Lateral) | Torsional Moment (Torsional) |
|---|---|---|---|---|---|
| Spread footing (also see Chapter 31) | End bearing | — | Base shear, lateral earth pressure | End bearing, lateral earth pressure | Base shear, lateral earth pressure |
| Individual short pile foundation | End bearing; side friction | Side friction | Lateral earth pressure, base shear | Lateral earth pressure, end bearing | Side friction, lateral earth pressure, base shear |
| Individual end-bearing long pile foundation | End bearing | — | Lateral earth pressure | Lateral earth pressure | — |
| Individual frictional long pile foundation | Side friction | Side friction | Lateral earth pressure | Lateral earth pressure | Side friction |
| Individual long pile foundation | End bearing; side friction | Side friction | Lateral earth pressure | Lateral earth pressure | Side friction |
| Anchor | — | Side friction | — | — | — |

**TABLE 32.3** Additional Functions of Pile Group Foundations

| Type of Foundation | Type of Resistance: Overturning moment (Lateral) | Torsional moment (Torsional) |
|---|---|---|
| Grouped spread footings | Vertical compressive resistance | Horizontal resistance |
| Grouped piles, foundations | Vertical compressive and uplift resistance | Horizontal resistance |
| Grouped anchors | Vertical uplift resistance | — |

A pile group is more efficient in resisting overturning and torsional moment than an individual foundation. Table 32.3 summarizes functions of a pile group in addition to those of individual piles.

### 32.2.6 Selection of Foundations

The two predominant factors in determining the type of foundations are bridge types and ground conditions.

The bridge type, including dimensions, type of bridge, and construction materials, dictates the design magnitude of loads and the allowable displacements and other performance criteria for the foundations, and therefore determines the dimensions and type of its foundations. For example, a suspension bridge requires large lateral capacity for its end anchorage which can be a huge deadman, a high capacity soil or rock anchor system, a group of driven piles, or a group of large-diameter drilled shafts. Tower foundations of an over-water bridge require large compressive, uplift, lateral, and overturning moment capacities. The likely foundations are deep, large-size footings using cofferdam construction, caissons, groups of large-diameter drilled shafts, or groups of a large number of steel piles.

Surface and subsurface geologic and geotechnical conditions are another main factor in determining the type of bridge foundations. Subsurface conditions, especially the depths to the load-bearing soil layer or bedrock, are the most crucial factor. Seismicity over the region usually dictates the design level of seismic loads, which is often the critical and dominant loading condition. A bridge that crosses a deep valley or river certainly determines the minimum span required. Over-water bridges have limited options to chose in terms of the type of foundations.

The final choice of the type of foundation usually depends on cost after considering some other factors, such as construction conditions, space and overhead conditions, local practice, environmental conditions, schedule constraints, etc. In the process of selection, several types of foundations would be evaluated as candidates once the type of bridge and the preliminary ground conditions are known. Certain types of foundations are excluded in the early stage of study. For example, from the geotechnical point of view, shallow foundations are not an acceptable option if a thick layer of soft clay or liquefiable sand is near the ground surface. Deep foundations are used in cases where shallow foundations would be excessively large and costly. From a constructibility point of view, driven pile foundations are not suitable if boulders exist at depths above the intended firm bearing soil/rock layer.

For small bridges such as roadway overpasses, for example, foundations with driven concrete or steel piles, drilled shafts, or shallow spread footing foundations may be the suitable choices. For large over-water bridge foundations, single or grouped large-diameter pipe piles, large-diameter rock sockets, large-diameter drilled shafts, caissons, or foundations constructed with cofferdams are the most likely choice. Caissons or cofferdam construction with a large number of driven pile groups were widely used in the past. Large-diameter pipe piles or drilled shafts, in combination with rock sockets, have been preferred for bridge foundations recently.

Deformation compatibility of the foundations and bridge structure is an important consideration. Different types of foundation may behave differently; therefore, the same type of foundations should be used for one section of bridge structure. Diameters of the piles and inclined piles are two important factors to considere in terms of deformation compatibility and are discussed in the following.

Small-diameter piles are more “brittle” in the sense that the ultimate settlement and lateral deflection are relatively small compared with large-diameter piles. For example, 20 small piles can have the same ultimate load capacity as two large-diameter piles. However, the small piles reach the ultimate state at a lateral deflection of 50 mm, whereas the large piles do at 150 mm. The smaller piles would have failed before the larger piles are activated to a substantial degree. In other words, larger piles will be more flexible and ductile than smaller piles before reaching the ultimate state. Since ductility usually provides more seismic safety, larger-diameter piles are preferred from the point of view of seismic design.

Inclined or battered piles should not be used together with vertical piles unless the inclined piles alone have enough lateral capacity. Inclined piles provide partial lateral resistance from their axial capacity, and, since the stiffness in the axial direction of a pile is much larger than in the perpendicular directions, inclined piles tend to attract most of the lateral seismic loading. Inclined piles will fail or reach their ultimate axial capacity before the vertical piles are activated to take substantial lateral loads.

# 32.3 Design Considerations

## 32.3.1 Design Concept

The current practice of foundation design mainly employs two types of design concepts, i.e., the permissible stress approach and the limit state approach.

By using the permissible stress approach, both the demanded stresses from loading and the ultimate stress capacity of the foundation are evaluated. The foundation is considered to be safe as long as the demanded stresses are less than the ultimate stress capacity of the foundation. A factor of safety of 2 to 3 is usually applied to the ultimate capacity to obtain various allowable levels of loading in order to limit the displacements of a foundation. A separate displacement analysis is usually performed to determine the allowable displacements for a foundation, and for the bridge structures. Design based on the permissible concept is still the most popular practice in foundation design.

Starting to be adopted in the design of large critical bridges, the limit state approach requires that the foundation and its supported bridge should not fail to meet performance requirements when exceeding various limit states. Collapse of the bridge is the ultimate limit state, and design is aimed at applying various factors to loading and resistance to ensure that this state is highly improbable. A design needs to ensure the structural integrity of the critical foundations before reaching the ultimate limit state, such that the bridge can be repaired a relatively short time after a major loading incident without reconstruction of the time-consuming foundations.

## 32.3.2 Design Procedures

Under normal conditions, the design procedures of a bridge foundation should involve the following steps:

1. Evaluate the site and subsurface geologic and geotechnical conditions, perform borings or other field exploratory programs, and conduct field and laboratory tests to obtain design parameters for subsurface materials;
2. Review the foundation requirements including design loads and allowable displacements, regulatory provisions, space, or other constraints;
3. Evaluate the anticipated construction conditions and procedures;
4. Select appropriate foundation type(s);
5. Determine the allowable and ultimate axial and lateral foundation design capacity, load vs. deflection relationship, and load vs. settlement relationship;
6. Design various elements of the foundation structure; and
7. Specify requirements for construction inspection and/or load test procedures, and incorporate the requirements into construction specifications.

## 32.3.3 Design Capacities

### Capacity in Long-Term and Short-Term Conditions

Depending on the loading types, foundations are designed for two different stress conditions. Capacity in total stress is used where loading is relatively quick and corresponds to an undrained condition. Capacity in effective stress is adopted where loading is slow and corresponds to a drained condition. For many types of granular soil, such as clean gravel and sand, drained capacity is very close to undrained capacity under most loading conditions. Pile capacity under seismic loading is usually taken 30% higher than capacity under static loading.

#### Axial, Lateral, and Moment Capacity

Deep foundations can provide lateral resistance to overturning moment and lateral loads and axial resistance to axial loads. Part or most of the moment capacity of a pile group are provided by the axial capacity of individual piles through pile cap action. The moment capacity depends on the axial capacity of the individual piles, the geometry arrangement of the piles, the rigidity of the pile cap, and the rigidity of the connection between the piles and the pile cap. Design and analysis is often concentrated on the axial and lateral capacity of individual piles. Axial capacity of an individual pile will be addressed in detail in Section 32.4 and lateral capacity in Section 32.5. Pile groups will be addressed in Section 32.6.

#### Structural Capacity

Deep foundations may fail because of structural failure of the foundation elements. These elements should be designed to take moment, shear, column action or buckling, corrosion, fatigue, etc. under various design loading and environmental conditions.

#### Determination of Capacities

In the previous sections, the general procedure and concept for the design of deep foundations are discussed. Detailed design includes the determination of axial and lateral capacity of individual foundations, and capacity of pile groups. Many methods are available to estimate these capacities, and they can be categorized into three types of methodology as listed in the following:

- Theoretical analysis utilizing soil or rock strength;
- Empirical methods including empirical analysis utilizing standard field tests, code requirements, and local experience; and
- Load tests, including full-scale load tests, and dynamic driving and restriking resistance analysis.

The choice of methods depends on the availability of data, economy, and other constraints. Usually, several methods are used; the capacity of the foundation is then obtained through a comprehensive evaluation and judgment.

In applying the above methods, the designers need to keep in mind that the capacity of a foundation is the sum of capacities of all elements. Deformation should be compatible in the foundation elements, in the surrounding soil, and in the soil–foundation interface. Settlement or other movements of a foundation should be restricted within an acceptable range and usually is a controlling factor for large foundations.

### 32.3.4 Summary of Design Methods

Table 32.4 presents a partial list of design methods available in the literature.

### 32.3.5 Other Design Issues

Proper foundation design should consider many factors regarding the environmental conditions, type of loading conditions, soil and rock conditions, construction, and engineering analyses, including:

- Various loading and loading combinations, including the impact loads of ships or vehicles
- Earthquake shaking
- Liquefaction

**TABLE 32.4** Summary of Design Methods for Deep Foundations

| Type | Design For | Soil Condition | Method and Author |
|---|---|---|---|
| Driven pile | End bearing | Clay | $N_c$ method [67] |
| | | | $N_c$ method [23] |
| | | | CPT methods [37,59,63] |
| | | | CPT [8,10] |
| | | Sand | $N_q$ method with critical depth concept [38] |
| | | | $N_q$ method [3] |
| | | | $N_q$ method [23] |
| | | | $N_q$ by others [26,71,76] |
| | | | Limiting $N_q$ values [1,13] |
| | | | Value of $\phi$ [27,30,39] |
| | | | SPT [37,38] |
| | | | CPT methods [37,59,63] |
| | | | CPT [8,10] |
| | | Rock | [10] |
| | Side resistance | Clay | $\alpha$-method [72,73] |
| | | | $\alpha$-method [1] |
| | | | $\beta$-method [23] |
| | | | $\lambda$-method [28,80] |
| | | | CPT methods [37,59,63] |
| | | | CPT [8,10] |
| | | | SPT [14] |
| | | Sand | $\alpha$-method [72,73] |
| | | | $\beta$-method [7] |
| | | | $\beta$-method [23] |
| | | | CPT method [37,59,63] |
| | | | CPT [8,10] |
| | | | SPT [37,38] |
| | Side and end | All | Load test: ASTM D 1143, static axial compressive test |
| | | | Load test: ASTM D 3689, static axial tensile test |
| | | | Sanders' pile driving formula (1850) [50] |
| | | | Danish pile driving formula [68] |
| | | | *Engineering News* formula (Wellingotn, 1988) |
| | | | Dynamic formula — WEAP Analysis |
| | | | Strike and restrike dynamic analysis |
| | | | Interlayer influence [38] |
| | | | No critical depth [20,31] |
| | Load-settlement | Sand | [77] |
| | | | [41,81] |
| | | All | Theory of elasticity, Mindlin's solutions [50] |
| | | | Finite-element method [15] |
| | | | Load test: ASTM D 1143, static axial compressive test |
| | | | Load test: ASTM D 3689, static axial tensile test |
| Drilled shaft | End bearing | Clay | $N_c$ method [66] |
| | | | Large base [45,57] |
| | | | CPT [8,10] |
| | | Sand | [74] |
| | | | [38] |
| | | | [55] |
| | | | [52] |
| | | | [37,38] |
| | | | [8,10] |
| | | Rock | [10] |
| | | Rock | Pressure meter [10] |

**TABLE 32.4 (continued)** Summary of Design Methods for Deep Foundations

| Type | Design For | Soil Condition | Method and Author |
|---|---|---|---|
| | Side resistance | Clay | α-method [52] |
| | | | α-method [67] |
| | | | α-method [83] |
| | | | CPT [8,10] |
| | | Sand | [74] |
| | | | [38] |
| | | | [55] |
| | | | β-method [44,52] |
| | | | SPT [52] |
| | | | CPT [8,10] |
| | | Rock | Coulombic [34] |
| | | | Coulombic [75] |
| | | | SPT [12] |
| | | | [24] |
| | | | [58] |
| | | | [11,32] |
| | | | [25] |
| | Side and end | Rock | [46] |
| | | | [84] |
| | | | [60] |
| | | | [48] |
| | | | [61,62] |
| | | | FHWA [57] |
| | | All | Load test [47] |
| | Load-settlement | Sand | [57] |
| | | Clay | [57] |
| | | | [85] |
| | | All | Load test [47] |
| All | Lateral resistance | Clay | Broms' method [5] |
| | | Sand | Broms' method [6] |
| | | All | *p–y* method [56] |
| | | Clay | *p–y* response [35] |
| | | Clay (w/water) | *p–y* response [53] |
| | | Clay (w/o water) | *p–y* response [82] |
| | | Sand | *p–y* response [53] |
| | | All | *p–y* response [1] |
| | | | *p–y* response for inclined piles [2,29] |
| | | | *p–y* response in layered soil |
| | | | *p–y* response [42] |
| | | Rock | *p–y* response [86] |
| | Load-settlement | All | Theory of elasticity method [50] |
| | | | Finite-difference method [64] |
| | | | General finite-element method (FEM) |
| | | | FEM dynamic |
| | End bearing | | Pressure meter method [36,78] |
| | Lateral resistance | | Pressure meter method [36] |
| | | | Load test: ASTM D 3966 |
| Group | Theory | | Elasticity approach [50] |
| | | | Elasticity approach [21] |
| | | | Two-dimensional group [51] |
| | | | Three-dimensional group [52] |
| | Lateral g-factor | | [10] |
| | | | [16] |

- Rupture of active fault and shear zone
- Landslide or ground instability
- Difficult ground conditions such as underlying weak and compressible soils
- Debris flow
- Scour and erosion
- Chemical corrosion of foundation materials
- Weathering and strength reduction of foundation materials
- Freezing
- Water conditions including flooding, water table change, dewatering
- Environmental change due to construction of the bridge
- Site contamination condition of hazardous materials
- Effects of human or animal activities
- Influence upon and by nearby structures
- Governmental and community regulatory requirements
- Local practice

### 32.3.6 Uncertainty of Foundation Design

Foundation design is as much an art as a science. Although most foundation structures are man-made, the surrounding geomaterials are created, deposited, and altered in nature over the geologic times. The composition and engineering properties of engineering materials such as steel and concrete are well controlled within a variation of uncertainty of between 5 to 30%. However, the uncertainty of engineering properties for natural geomaterials can be up to several times, even within relatively uniform layers and formations. The introduction of faults and other discontinuities make generalization of material properties very hard, if not impossible.

Detailed geologic and geotechnical information is usually difficult and expensive to obtain. Foundation engineers constantly face the challenge of making engineering judgments based on limited and insufficient data of ground conditions and engineering properties of geomaterials.

It was reported that under almost identical conditions, variation of pile capacities of up to 50% could be expected within a pile cap footprint under normal circumstances. For example, piles within a nine-pile group had different restruck capacities of 110, 89, 87, 96, 86, 102, 103, 74, and 117 kips (1 kip = 4.45 kN) respectively [19].

Conservatism in foundation design, however, is not necessarily always the solution. Under seismic loading, heavier and stiffer foundations may tend to attract more seismic energy and produce larger loads; therefore, massive foundations may not guarantee a safe bridge performance.

It could be advantageous that piles, steel pipes, caisson segments, or reinforcement steel bars are tailored to exact lengths. However, variation of depth and length of foundations should always be expected. Indicator programs, such as indicator piles and pilot exploratory borings, are usually a good investment.

## 32.4 Axial Capacity and Settlement — Individual Foundation

### 32.4.1 General

The axial resistance of a deep foundation includes the tip resistance ($Q_{end}$), side or shaft resistance ($Q_{side}$), and the effective weight of the foundation ($W_{pile}$). Tip resistance, also called end bearing, is the compressive resistance of soil near or under the tip. Side resistance consists of friction, cohesion, and keyed bearing along the shaft of the foundation. Weight of the foundation is usually ignored under compression because it is nearly the same as the weight of the soil displaced, but is usually accounted for under uplift loading condition.

At any loading instance, the resistance of an individual deep foundation (or pile) can be expressed as follows:

$$Q = Q_{\text{end}} + \Sigma Q_{\text{side}} \pm W_{\text{pile}} \tag{32.1}$$

The contribution of each component in the above equation depends on the stress–strain behavior and stiffness of the pile and the surrounding soil and rock. The maximum capacity of a pile can be expressed as

$$Q^c{}_{\text{max}} \le Q^c{}_{\text{end\_max}} + \Sigma Q^c{}_{\text{side\_max}} - W_{\text{pile}} \quad \text{(in compression)} \tag{32.2}$$

$$Q^t{}_{\text{max}} \le Q^t{}_{\text{end\_max}} + \Sigma Q^t{}_{\text{side\_max}} + W_{\text{pile}} \quad \text{(in uplift)} \tag{32.3}$$

and is less than the sum of all the maximum values of resistance. The ultimate capacity of a pile undergoing a large settlement or upward movement can be expressed as

$$Q^c{}_{\text{ult}} = Q^c{}_{\text{end\_ult}} + \Sigma Q^c{}_{\text{side\_ult}} - W_{\text{pile}} \le Q^c_{\text{max}} \tag{32.4}$$

$$Q^t{}_{\text{ult}} = Q^t{}_{\text{end\_ult}} + \Sigma Q^t{}_{\text{side\_ult}} + W_{\text{pile}} \le Q^t_{\text{max}} \tag{32.5}$$

Side- and end-bearing resistances are related to displacement of a pile. Maximum end bearing capacity can be mobilized only after a substantial downward movement of the pile, whereas side resistance reaches its maximum capacity at a relatively smaller downward movement. Therefore, the components of the maximum capacities ( $Q_{\text{max}}$ ) indicated in Eqs. (32.2) and (32.3) may not be realized at the same time at the tip and along the shaft. For a drilled shaft, the end bearing is usually ignored if the bottom of the borehole is not cleared and inspected during construction. Voids or compressible materials may exist at the bottom after concrete is poured; as a result, end bearing will be activated only after a substantial displacement.

Axial displacements along a pile are larger near the top than toward the tip. Side resistance depends on the amount of displacement and is usually not uniform along the pile. If a pile is very long, maximum side resistance may not occur at the same time along the entire length of the pile. Certain types of geomaterials, such as most rocks and some stiff clay and dense sand, exhibit strain softening behavior for their side resistance, where the side resistance first increases to reach its maximum, then drops to a much smaller residual value with further displacement. Consequently, only a fixed length of the pile segment may maintain high resistance values and this segment migrates downward to behave in a pattern of a progressive failure. Therefore, the capacity of a pile or drilled shaft may not increase infinitely with its length.

For design using the permissible stress approach, allowable capacity of a pile is the design capacity under service or routine loading. The allowable capacity ( $Q_{\text{all}}$ ) is obtained by dividing ultimate capacity ( $Q_{\text{ult}}$ ) by a factor of safety (FS) to limit the level of settlement of the pile and to account for uncertainties involving material, installation, loads calculation, and other aspects. In many cases, the ultimate capacity ( $Q_{\text{ult}}$ ) is assumed to be the maximum capacity ( $Q_{\text{max}}$ ). The factor of safety is usually between 2 to 3 for deep foundations depending on the reliability of the ultimate capacity estimated. With a field full-scale loading test program, the factor of safety is usually 2.

**TABLE 32.5** Typical Values of Bearing Capacity Factor $N_q$

| | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| $\varphi^a$ (degrees) | 26 | 28 | 30 | 31 | 32 | 33 | 34 | 35 | 36 | 37 | 38 | 39 | 40 |
| $N_q$ (driven pile displacement) | 10 | 15 | 21 | 24 | 29 | 35 | 42 | 50 | 62 | 77 | 86 | 120 | 145 |
| $N_q{}^b$ (drilled piers) | 5 | 8 | 10 | 12 | 14 | 17 | 21 | 25 | 30 | 38 | 43 | 60 | 72 |

[a] Limit φ to 28° if jetting is used.

[b] 1. In case a bailer of grab bucket is used below the groundwater table, calculate end bearing based on φ not exceeding 28°.

2. For piers greater than 24-in. diameter, settlement rather than bearing capacity usually controls the design. For estimating settlement, take 50% of the settlement for an equivalent footing resting on the surface of comparable granular soils (Chapter 5, DM-7.01).

*Source*: NAVFAC [42].

## 32.4.2 End Bearing

End bearing is part of the axial compressive resistance provided at the bottom of a pile by the underlying soil or rock. The resistance depends on the type and strength of the soil or rock and on the stress conditions near the tip. Piles deriving their capacity mostly from end bearing are called end bearing piles. End bearing in rock and certain types of soil such as dense sand and gravel is usually large enough to support the designed loads. However, these types of soil or rock cannot be easily penetrated through driving. No or limited uplift resistance is provided from the pile tips; therefore, end-bearing piles have low resistance against uplift loading.

The end bearing of a pile can be expressed as:

$$Q_{\text{end\_max}} = \begin{cases} cN_c A_{\text{pile}} & \text{for clay} \\ \sigma'_v N_q A_{\text{pile}} & \text{for sand} \\ \dfrac{U_c}{2} N_k A_{\text{pile}} & \text{for rock} \end{cases} \tag{32.6}$$

where

$Q_{\text{end\_max}}$ = the maximum end bearing of a pile
$A_{\text{pile}}$ = the area of the pile tip or base
$N_c$, $N_q$, $N_k$ = the bearing capacity factors for clay, sand, and rock
$c$ = the cohesion of clay
$\sigma'_v$ = the effective overburden pressure
$U_c$ = the unconfined compressive strength of rock and $\frac{U_c}{2} = S_u$, the equivalent shear strength of rock

### Clay

The bearing capacity factor $N_c$ for clay can be expressed as

$$N_c = 6.0\left(1 + 0.2\frac{L}{D}\right) \le 9 \tag{32.7}$$

where $L$ is the embedment depth of the pile tip and $D$ is the diameter of the pile.

### Sand

The bearing capacity factor $N_q$ generally depends on the friction angle $\phi$ of the sand and can be estimated by using Table 32.5 or the Meyerhof equation below.

$$N_q = e^{\pi \tan\varphi} \tan^2\left(45 + \frac{\varphi}{2}\right) \tag{32.8}$$

The capacity of end bearing in sand reaches a maximum cutoff after a certain critical embedment depth. This critical depth is related to $\phi$ and $D$ and for design purposes is listed as follows:

$$L_c = 7D, \qquad \phi = 30^o \text{ for loose sand}$$

$$L_c = 10D, \qquad \phi = 34^o \text{ for medium dense sand}$$

$$L_c = 14D, \qquad \phi = 38^o \text{ for dense sand}$$

$$L_c = 22D, \qquad \phi = 45^o \text{ for very dense sand}$$

The validity of the concept of critical depth has been challenged by some people; however, the practice to limit the maximum ultimate end bearing capacity in sand will result in conservative design and is often recommended.

### Rock

The bearing capacity factor $N_k$ depends on the quality of the rock mass, intact rock properties, fracture or joint properties, embedment, and other factors. Because of the complex nature of the rock mass and the usually high value for design bearing capacity, care should be taken to estimate $N_k$. For hard fresh massive rock without open or filled fractures, $N_k$ can be taken as high as 6. $N_k$ decreases with increasing presence and dominance of fractures or joints and can be as low as 1. Rock should be treated as soil when rock is highly fractured and weathered or in-fill weak materials control the behavior of the rock mass. Bearing capacity on rock also depends on the stability of the rock mass. Rock slope stability analysis should be performed where the foundation is based on a slope. A higher factor of safety, 3 to as high as 10 to 20, is usually applied in estimating allowable bearing capacity for rocks using the $N_k$ approach.

The soil or rock parameters used in design should be taken from averaged properties of soil or rock below the pile tip within the influence zone. The influence zone is usually taken as deep as three to five diameters of the pile. Separate analyses should be conducted where weak layers exist below the tip and excessive settlement or punch failure might occur.

### Empirical Methods

Empirical methods are based on information of the type of soil/rock and field tests or index properties. The standard penetration test (SPT) for sand and cone penetration test (CPT) for soil are often used.

Meyerhof [38] recommended a simple formula for piles driven into sand. The ultimate tip bearing pressure is expressed as

$$q_{\text{end\_max}} \leq 4N_{\text{SPT}} \qquad \text{in tsf (1 tsf = 8.9 kN)} \tag{32.9}$$

where $N_{\text{SPT}}$ is the blow count of SPT just below the tip of the driven pile and $q_{\text{end\_max}} = Q_{\text{end\_max}} / A_{\text{pile}}$. Although the formula is developed for piles in sand, it also is used for piles in weathered rock for preliminary estimate of pile capacity.

Schmertmann [63] recommended a method to estimate pile capacity by using the CPT test:

$$q_{\text{end\_max}} = q_b = \frac{q_{c1} + q_{c2}}{2} \tag{32.10}$$

where

$q_{c1}$ = averaged cone tip resistance over a depth of 0.7 to 4 diameters of the pile below tip of the pile
$q_{c2}$ = the averaged cone tip resistance over a depth of 8 diameters of the pile above the tip of the pile

Chapter 31 presents recommended allowable bearing pressures for various soil and rock types for spread footing foundations and can be used as a conservative estimate of end-bearing capacity for end-bearing piles.

**TABLE 32.6** Typical Values of $\alpha$ and $f_s$

| Range of Shear Strength, $S_u$ ksf | Formula to Estimate $\alpha$ | Range of $\alpha$ | Range of $f_s$ ksf[a] | Description |
|---|---|---|---|---|
| 0 to 0.600 | $\alpha = 1.0$ | 1 | 0–0.6 | Soft clay |
| 0.600 to 3 | $\alpha = 0.375\left(1+\frac{1}{S_u}\right),$ | 1–0.5 | 0.6–1.5 | Medium stiff clay to very stiff clay |
| 3 to 11 | $\alpha = 0.375\left(1+\frac{1}{S_u}\right),$ | 0.5–0.41 | 1.5–4.5 | Hard clay to very soft rock |
| 11 to 576 (76 psi to 4000 psi) | $\alpha = \frac{5}{\sqrt{2S_u}}$, $S_u$ in psi, | 0.41–0.056 | 4.5–32 (31–220 psi) | Soft rock to hard rock |

*Note*: 1 ksf = 1000 psf; 1 psi = 144 psf; 1 psf = 0.048 kPa; 1 psi = 6.9 kPa

[a] For concrete driven piles and for drilled piers without buildup of mud cakes along the shaft. (Verify if fs ≥ 3 ksf.)

## 32.4.3 Side Resistance

Side resistance usually consists of friction and cohesion between the pile and the surrounding soil or rock along the shaft of a pile. Piles that derive their resistance mainly from side resistance are termed *frictional piles*. Most piles in clayey soil are frictional piles, which can take substantial uplift loads.

The maximum side resistance of a pile $Q_{\text{side\_max}}$ can be expressed as

$$Q_{\text{side\_max}} = \sum f_s A_{\text{side}} \tag{32.11}$$

$$f_s = K_s \sigma'_v \tan\delta + c_a \tag{32.12}$$

$$c_a = \alpha S_u \tag{32.13}$$

where

$\sum$ = the sum for all layers of soil and rock along the pile
$A_{\text{side}}$ = the shaft side area
$f_s$ = the maximum frictional resistance on the side of the shaft
$K_s$ = the lateral earth pressure factor along the shaft
$\sigma'_v$ = the effective vertical stress along the side of the shaft
$\delta$ = the friction angle between the pile and the surrounding soil; for clayey soil under quick loading, $\delta$ is very small and usually omitted
$c_a$ = the adhesion between pile and surrounding soil and rock
$\alpha$ = a strength factor, and
$S_u$ = the cohesion of the soil or rock

**TABLE 32.7** Typical Values Cohesion and Adhesion $f_s$

| Pile Type | Consistency of Soil | Cohesion, $S_u$ psf | Adhesion, $f_s$ psf |
|---|---|---|---|
| Timber and concrete | Very soft | 0–250 | 0–250 |
| | Soft | 250–500 | 250–480 |
| | Medium stiff | 500–1000 | 480–750 |
| | Stiff | 1000–2000 | 750–950 |
| | Very stiff | 2000–4000 | 950–1300 |
| Steel | Very soft | 0–250 | 0–250 |
| | Soft | 250–500 | 250–460 |
| | Medium stiff | 500–1000 | 480–700 |
| | Stiff | 1000–2000 | 700–720 |
| | Very stiff | 2000–4000 | 720–750 |

1 psf = 0.048 kPa.
*Source*: NAVFAC [42].

**TABLE 32.8** Typical Values of Bond Stress of Rock Anchors for Selected Rock

| Rock Type (Sound, Nondecayed) | Ultimate Bond Stresses between Rock and Anchor Plus ($\delta_{skin}$), psi |
|---|---|
| Granite and basalt | 250–450 |
| Limestone (competent) | 300–400 |
| Dolomitic limestone | 200–300 |
| Soft limestone | 150–220 |
| Slates and hard shales | 120–200 |
| Soft shales | 30–120 |
| Sandstone | 120–150 |
| Chalk (variable properties) | 30–150 |
| Marl (stiff, friable, fissured) | 25–36 |

*Note*: It is not generally recommended that design bond stresses exceed 200 psi even in the most competent rocks. 1 psi = 6.9 kPa.
*Source*: NAVFAC [42].

**TABLE 32.9** Typical Values of earth Pressure Coefficient $K_s$

| | Earth Pressure Coefficients $K_s$ | | |
|---|---|---|---|
| Pile Type | $K_s$ [a] (compression) | $K_s$ [a] (tension) | $K_s$ [b] |
| Driven single H-pile | 0.5–1.0 | 0.3–0.5 | — |
| Driven single displacement pile | 1.0–1.5 | 0.6–1.0 | 0.7–3.0 |
| Driven single displacement tapered pile | 1.5–2.0 | 1.0–1.3 | — |
| Driven jetted pile | 0.4–0.9 | 0.3–0.6 | — |
| Drilled pile (less than 24-in. diameter) | 0.7 | 0.4 | — |
| Insert pile | — | — | 0.7 (compression)<br>0.5 (tension) |
| Driven with predrilled hole | — | — | 0.4–0.7 |
| Drilled pier | — | — | 0.1–0.4 |

[a] From NAVFAC [42].
[b] From Le Tirant (1979), $K_s$ increases with OCR or $D_R$.

**TABLE 32.10** Typical Value of Pile-Soil Friction Angles $\delta$

| Pile Type | $\delta$, ° | Alternate for $\delta$ |
|---|---|---|
| Concrete[a] | — | $\delta = \frac{3}{4}\varphi$ |
| Concrete (rough, cast-in-place)[b] | 33 | $\delta = 0.85\varphi$ |
| Concrete (smooth)[b] | 30 | $\delta = 0.70\varphi$ |
| Steel[a] | 20 | — |
| Steel (corrugated) | 33 | $\delta = \varphi$ |
| Steel (smooth)[c] | — | $\delta = \varphi - 5°$ |
| Timber[a] | — | $\delta = \frac{3}{4}\varphi$ |

[a] NAVFAC [42].
[b] Woodward et al. [85]
[c] API [1] and de Ruiter and Beringen [13]

Typical values of $\alpha$, $f_s$, $K_s$, $\delta$ are shown in Tables 32.6 through 32.10. For design purposes, side resistance $f_s$ in sand is limited to a cutoff value at the critical depth, which is equal to about 10$B$ for loose sand and 20$B$ for dense sand.

Meyerhof [38] recommended a simple formula for driven piles in sand. The ultimate side adhesion is expressed as

$$f_s \leq \frac{N_{SPT}}{50} \quad \text{in tsf (1 tsf = 8.9 kN)} \tag{32.14}$$

where $N_{SPT}$ is the averaged blow count of SPT along the pile.

Meyerhof [38] also recommended a formula to calculate the ultimate side adhesion based on CPT results as shown in the following.

*For full displacement piles*:

$$f_s = \frac{q_c}{200} \leq 1.0 \quad \text{in tsf} \tag{32.15}$$

or

$$f_s = 2 f_c \leq 1.0 \tag{32.16}$$

*For nondisplacement piles*:

$$f_s = \frac{q_c}{400} \leq 0.5 \quad \text{in tsf} \tag{32.17}$$

or

$$f_s = f_c \leq 0.5 \tag{32.18}$$

in which

$q_c$, $f_c$ = the cone tip and side resistance measured from CPT; averaged values should be used along the pile

### Downdrag

For piles in soft soil, another deformation-related issue should be noted. When the soil surrounding the pile settles relative to a pile, the side friction, also called the negative skin friction, should be considered when there exists underlying compressible clayey soil layers and liquefiable loose sand layers. Downdrag can also happen when ground settles because of poor construction of caissons in sand. On the other hand, updrag should also be considered in cases where heave occurs around the piles for uplift loading condition, especially during installation of piles and in expansive soils.

## 32.4.4 Settlement of Individual Pile, *t–z*, *Q–z* Curves

Besides bearing capacity, the allowable settlement is another controlling factor in determining the allowable capacity of a pile foundation, especially if layers of highly compressible soil are close to or below the tip of a pile.

Settlement of a small pile (diameter less than 350 mm) is usually kept within an acceptable range (usually less than 10 mm) when a factor of safety of 2 to 3 is applied to the ultimate capacity to obtain the allowable capacity. However, in the design of large-diameter piles or caissons, a separate settlement analysis should always be performed.

The total settlement at the top of a pile consists of immediate settlement and long-term settlement. The immediate settlement occurs during or shortly after the loads are applied, which includes elastic compression of the pile and deformation of the soil surrounding the pile under undrained loading conditions. The long-term settlement takes place during the period after the loads are applied, which includes creep deformation and consolidation deformation of the soil under drained loading conditions.

Consolidation settlement is usually significant in soft to medium stiff clayey soils. Creep settlement occurs most significantly in overconsolidated (OC) clays under large sustained loads, and can be estimated by using the method developed by Booker and Poulos (1976). In principle, however, long-term settlement can be included in the calculation of ultimate settlement if the design parameters of soil used in the calculation reflect the long-term behavior.

Presented in the following sections are three methods that are often used:

- Method of solving ultimate settlement by using special solutions from the theory of elasticity [50,85]. Settlement is estimated based on equivalent elasticity in which all deformation of soil is assumed to be linear elastic.
- Empirical method [79].
- Method using localized springs, or the so called *t–z* and *Q–z* method [52a].

### Method from Elasticity Solutions

The total elastic settlement $S$ can be separated into three components:

$$S = S_b + S_s + S_{sh} \tag{32.19}$$

where $S_b$ is part of the settlement at the tip or bottom of a pile caused by compression of soil layers below the pile under a point load at the pile tip, and is expressed as

$$S_b = \frac{p_b D_b I_{bb}}{E_s} \tag{32.20}$$

$S_s$ is part of the settlement at the tip of a pile caused by compression of soil layers below the pile under the loading of the distributed side friction along the shaft of the pile, and can be expressed as

$$S_s = \sum_i \frac{(f_{si} l_i \Delta z_i) I_{bs}}{E_s} \tag{32.21}$$

and $S_{sh}$ is the shortening of the pile itself, and can be expressed as

$$S_{sh} = \sum_i \frac{(f_{si} l_i \Delta z_i) + p_b A_b (\Delta z_i)}{E_c (A_i)} \tag{32.22}$$

where

$p_b$ = averaged loading pressure at pile tip
$A_b$ = cross section area of a pile at pile tip; $A_b p_b$ is the total load at the tip
$D_b$ = diameter of pile at the pile tip
$i$ = subscript for $i$th segment of the pile
$l$ = perimeter of a segment of the pile
$\Delta z$ = axial length of a segment of the pile; $L = \sum_i \Delta z_i$ is the total length of the pile.
$f_s$ = unit friction along side of shaft; $f_{si} l_i \Delta z_i$ is the side frictional force for segment $i$ of the pile
$E_s$ = Young's modulus of uniform and isotropic soil
$E_c$ = Young's modulus of the pile
$I_{bb}$ = base settlement influence factor, from load at the pile tip (Figure 32.4)
$I_{bs}$ = base settlement influence factor, from load along the pile shaft (Figure 32.4)

Because of the assumptions of linear elasticity, uniformity, and isotropy for soil, this method is usually used for preliminary estimate purposes.

### Method by Vesic [79]

The settlement $S$ at the top of a pile can be broken down into three components, i.e.,

$$S = S_b + S_s + S_{sh} \tag{32.23}$$

Settlement due to shortening of a pile is

$$S_{sh} = (Q_p + \alpha_s Q_s) \frac{L}{AE_c} \tag{32.24}$$

where

$Q_p$ = point load transmitted to the pile tip in the working stress range
$Q_s$ = shaft friction load transmitted by the pile in the working stress range (in force units)
$\alpha_s$ = 0.5 for parabolic or uniform distribution of shaft friction, 0.67 for triangular distribution of shaft friction starting from zero friction at pile head to a maximum value at pile tip, 0.33 for triangular distribution of shaft friction starting from a maximum at pile head to zero at the pile tip
$L$ = pile length
$A$ = pile cross-sectional area
$E_c$ = modulus of elasticity of the pile

Settlement of the pile tip caused by load transmitted at the pile tip is

$$S_b = \frac{C_p Q_p}{D q_o} \tag{32.25}$$

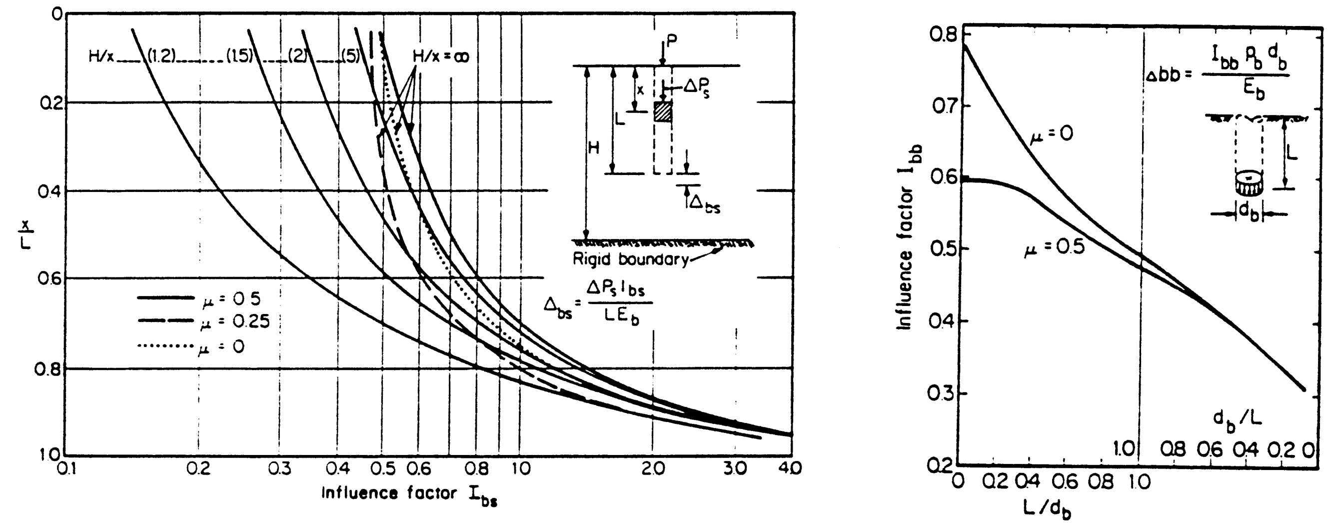


**FIGURE 32.4** Influence factors $I_{bb}$ and $I_{bs}$ . [From Woodward, Gardner and Greer (1972)[85], used with permission of McGraw-Hill Book Company]

where

$C_p$ = empirical coefficient depending on soil type and method of construction, see Table 32.11.
$D$ = pile diameter
$q_o$ = ultimate end bearing capacity

and settlement of the pile tip caused by load transmitted along the pile shaft is

$$S_s = \frac{C_s Q_s}{h q_o} \tag{32.26}$$

where

$C_s = (0.93 + 0.16 D / B) C_p$
$h$ = embedded length

**TABLE 32.11** Typical Values of $C_p$ for Estimating Settlement of a Single Pile

| Soil Type | Driven Piles | Bored Piles |
|---|---|---|
| Sand (dense to loose) | 0.02–0.04 | 0.09–0.18 |
| Clay (stiff to soft) | 0.02–0.03 | 0.03–0.06 |
| Silt (dense to loose) | 0.03–0.05 | 0.09–0.12 |

*Note:* Bearing stratum under pile tip assumed to extend at least 10 pile diameters below tip and soil below tip is of comparable or higher stiffness.

## Method Using Localized Springs: The *t–z* and *Q–z* method

In this method, the reaction of soil surrounding the pile is modeled as localized springs: a series of springs along the shaft (the *t–z* curves) and the spring attached to the tip or bottom of a pile (the *Q–z* curve). *t* is the load transfer or unit friction force along the shaft, *Q* is the tip resistance of the pile, and *z* is the settlement of soil at the location of a spring. The pile itself is also represented as a series of springs for each segment. A mechanical model is shown on Figure 32.5. The procedure to obtain the settlement of a pile is as follows:

- Assume a pile tip movement *zb*_1; obtain a corresponding tip resistance *Q*_1 from the *Q–z* curve.
- Divide the pile into number of segments, and start calculation from the bottom segment. Iterations:
  1. Assume an averaged movement of the segment *zs*_1; obtain the averaged side friction along the bottom segment *ts*_1 by using the *t–z* curve at that location.
  2. Calculate the movement at middle of the segment from elastic shortening of the pile under axial loading *zs*_2. The axial load is the tip resistance *Q*_1 plus the added side friction *ts*_1.
  3. Iteration should continue until the difference between *zs*_1 and *zs*_2 is within an acceptable tolerance.

  Iteration continues for all the segments from bottom to top of the pile.
- A settlement at top of pile *zt*_1 corresponding to a top axial load *Qt*_1 is established.
- Select another pile tip movement *zb*_2 and calculate *zt*_2 and *Qt*_2 until a relationship curve of load vs. pile top settlement is found.

The *t–z* and *Q–z* curves are established from test data by many authors. Figure 32.6 shows the *t–z* and *Q–z* curves for cohesive soil and cohesionless soil by Reese and O'Neil [57].

Although the method of *t–z* and *Q–z* curves employs localized springs, the calculated settlements are usually within a reasonable range since the curves are backfitted directly from the test results. Factors of nonlinear behavior of soil, complicated stress conditions around the pile, and partial

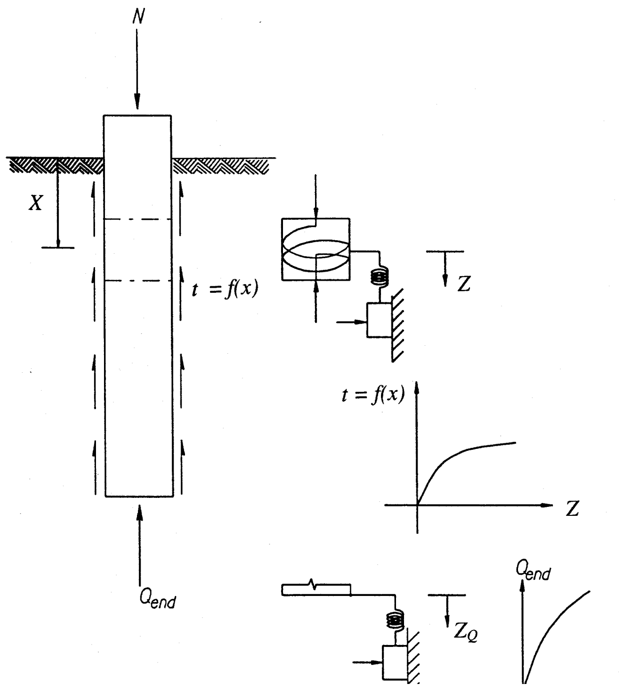


**FIGURE 32.5** Analytical model for pile under axial loading with *t–z* and *Q–z* curves.

corrections to the Winkler's assumption are embedded in this methodology. Besides, settlement of a pile can be estimated for complicated conditions such as varying pile geometry, different pile materials, and different soil layers.

## 32.5 Lateral Capacity and Deflection — Individual Foundation

### 32.5.1 General

Lateral capacity of a foundation is the capacity to resist lateral deflection caused by horizontal forces and overturning moments acted on the top of the foundation. For an individual foundation, lateral resistance comes from three sources: lateral earth pressures, base shear, and nonuniformly distributed end-bearing pressures. Lateral earth pressure is the primary lateral resistance for long piles. Base shear and distributed end-bearing pressures are discussed in Chapter 31.

### 32.5.2 Broms' Method

Broms [5] developed a method to estimate the ultimate lateral capacity of a pile. The pile is assumed to be short and rigid. Only rigid translation and rotation movements are considered and only ultimate lateral capacity of a pile is calculated. The method assumes distributions of ultimate lateral pressures for cohesive and cohesionless soils; the lateral capacity of piles with different top fixity conditions are calculated based on the assumed lateral pressure as illustrated on Figures 32.7 and 32.8. Restricted by the assumptions, the Broms' method is usually used only for preliminary estimates of the ultimate lateral capacity of piles.

#### Ultimate Lateral Pressure

The ultimate lateral pressure $q_{h,u}$ along a pile is calculated as follows:

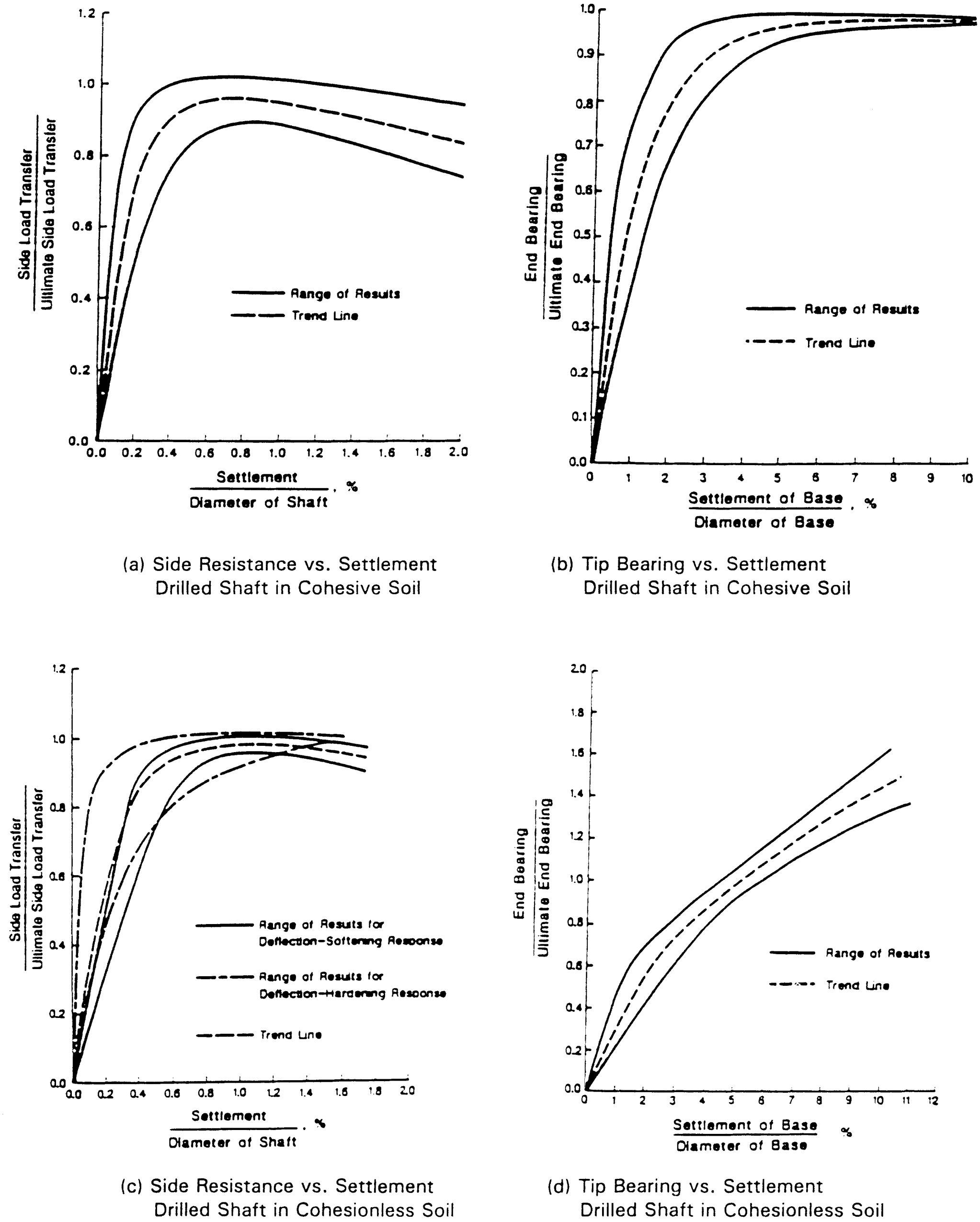


**FIGURE 32.6** Load transfer for side resistance ($t$–$z$) and tip bearing ($Q$–$z$). (a) Side resistance vs. settlement, drilled shaft in cohesive soil; (b) tip bearing vs. settlement, drilled shaft in cohesive soil; (c) side resistance vs. settlement, drilled shaft in cohesionless soil; (d) tip bearing vs. settlement, drilled shaft in cohesionless soil. (From AASHTO LRFD Bridge Design Specifications, First Edition, coyyright 1996 by the American Association of State Highway and Transportation officials, Washington, D.C. Used by permission.)

$$q_{h,u} = \begin{cases} 9c_u & \text{for cohesive soil} \\ 3K_p p_0' & \text{for cohesionless soil} \end{cases} \tag{32.27}$$

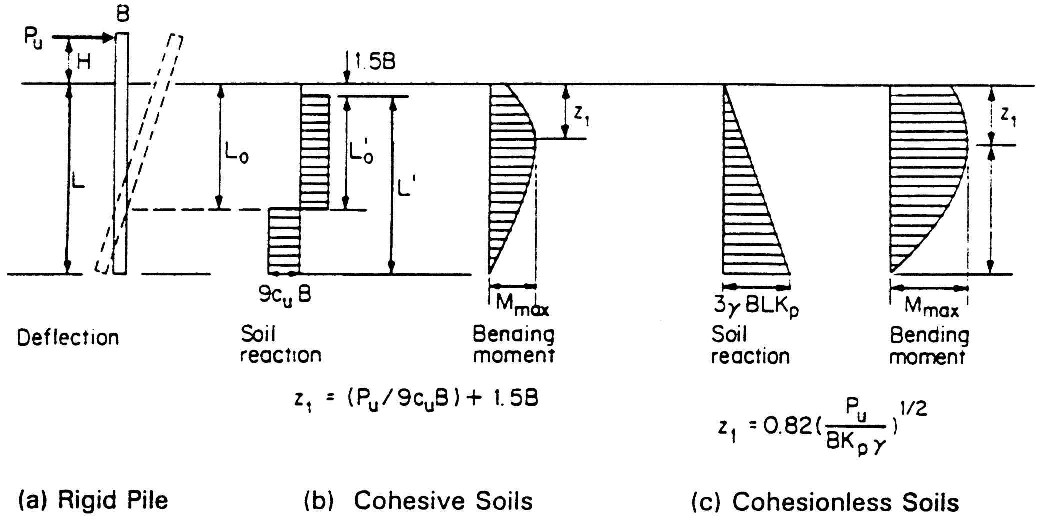


**FIGURE 32.7** Free-head, short rigid piles — ultimate load conditions. (a) Rigid pile; (b) cohesive soils; (c) cohesionless soils. [After Broms (1964)[5,6]]

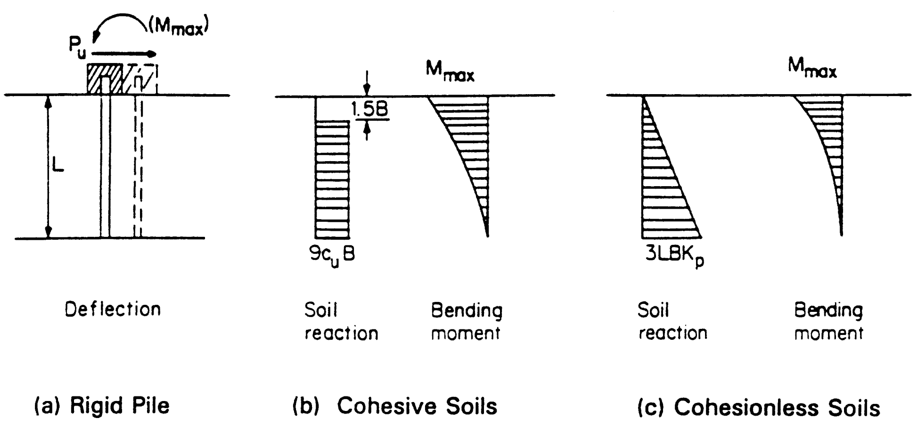


**FIGURE 32.8** Fixed-head, short rigid piles — ultimate load conditions. (a) Rigid pile; (b) cohesive soils; (c) cohesionless soils. [After Broms (1964)[5,6]]

where

$c_u$ = shear strength of the soil

$K_p$ = coefficient of passive earth pressure, $K_p = \tan^2(45^o + \varphi/2)$ and $\varphi$ is the friction angle of cohesionless soils (or sand and gravel)

$p_0'$ = effective overburden pressure, $p_0' = \gamma_z'$ at a depth of $z$ from the ground surface, where $\gamma'$ is the effective unit weight of the soil

## Ultimate Lateral Capacity for the Free-Head Condition

The ultimate lateral capacity $P_u$ of a pile under the free-head condition is calculated by using the following formula:

$$P_u = \begin{cases} \left(\dfrac{L_0'^2 - 2L'L_0' + 0.5L'^2}{L' + H + 1.5B}\right)(9c_u B) & \text{for cohesive soil} \\ \dfrac{0.5BL^3K_p\gamma'}{H+L} & \text{for cohesionless soil} \end{cases} \tag{32.28}$$

where

$L$ = embedded length of pile

$H$ = distance of resultant lateral force above ground surface

$B$ = pile diameter

$L'$ = embedded pile length measured from a depth of $1.5B$ below the ground surface, or $L' = L - 1.5B$

$L_0$ = depth to center of rotation, and $L_0 = (H + 23L)/(2H + L)$

$L_0'$ = depth to center of rotation measured from a depth of $1.5B$ below the ground surface, or $L_0' = L_0 - 1.5B$

### Ultimate Lateral Capacity for the Fixed-Head Condition

The ultimate lateral capacity $P_u$ of a pile under the fixed-head condition is calculated by using the following formula:

$$P_u = \begin{cases} 9c_u B(L - 1.5B) & \text{for cohesive soil} \\ 1.5\gamma' BL^2 K_p & \text{for cohesionless soil} \end{cases} \tag{32.29}$$

## 32.5.3 Lateral Capacity and Deflection — *p–y* Method

One of the most commonly used methods for analyzing laterally loaded piles is the *p–y* method, in which soil reactions to the lateral deflections of a pile are treated as localized nonlinear springs based on the Winkler's assumption. The pile is modeled as an elastic beam that is supported on a deformable subgrade.

The *p–y* method is versatile and can be used to solve problems including different soil types, layered soils, nonlinear soil behavior; different pile materials, cross sections; and different pile head connection conditions.

### Analytical Model and Basic Equation

An analytical model for pile under lateral loading with *p–y* curves is shown on Figure 32.9. The basic equation for the beam-on-a-deformable-subgrade problem can be expressed as

$$EI\frac{d^4y}{dx^4} - P_x\frac{d^2y}{dx^2} + p + q = 0 \tag{32.30}$$

where

$y$ = lateral deflection at point $x$ along the pile

$EI$ = bending stiffness or flexural rigidity of the pile

$P_x$ = axial force in beam column

$p$ = soil reaction per unit length, and $p = -E_s y$; where $E_s$ is the secant modulus of soil reaction.

$q$ = lateral distributed loads

The following relationships are also used in developing boundary conditions:

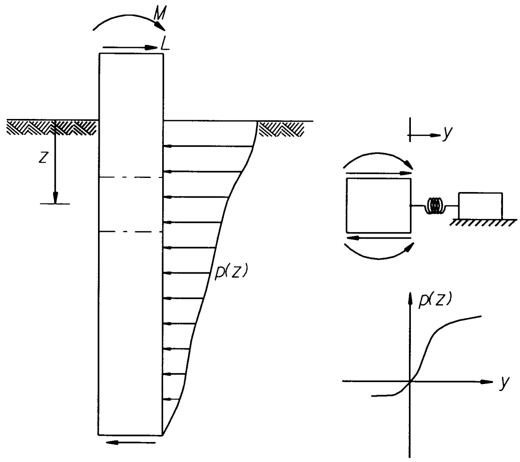


**FIGURE 32.9** Analytical model for pile under lateral loading with *p–y* curves.

$$M = -EI\frac{d^4y}{dx^4} \tag{32.31}$$

$$Q = -\frac{dM}{dx} + P_x\frac{dy}{dx} \tag{32.32}$$

$$\theta = \frac{dy}{dx} \tag{32.33}$$

where $M$ is the bending moment, $Q$ is the shear force in the beam column, $\theta$ is the rotation of the pile.

The *p–y* method is a valuable tool in analyzing laterally loaded piles. Reasonable results are usually obtained. A computer program is usually required because of the complexity and iteration needed to solve the above equations using the finite-difference method or other methods. It should be noted that Winkler's assumption ignores the global effect of a continuum. Normally, if soil behaves like a continuum, the deflection at one point will affect the deflections at other points under loading. There is no explicit expression in the *p–y* method since localized springs are assumed. Although *p–y* curves are developed directly from results of load tests and the influence of global interaction is included implicitly, there are cases where unexpected outcomes resulted. For example, excessively large shear forces will be predicted for large piles in rock by using the *p–y* method approach, where the effects of the continuum and the shear stiffness of the surrounding rock are ignored. The accuracy of the *p–y* method depends on the number of tests and the variety of tested parameters, such as geometry and stiffness of pile, layers of soil, strength and stiffness of soil, and loading conditions. One should be careful to extrapolate *p–y* curves to conditions where tests were not yet performed in similar situations.

## Generation of *p–y* Curves

A *p–y* curve, or the lateral soil resistance *p* expressed as a function of lateral soil movement *y*, is based on backcalculations from test results of laterally loaded piles. The empirical formulations of *p–y* curves are different for different types of soil. *p–y* curves also depend on the diameter of the pile, the strength and stiffness of the soil, the confining overburden pressures, and the loading conditions. The effects of layered soil, battered piles, piles on a slope, and closely spaced piles are also usually considered. Formulation for soft clay, sand, and rock is provided in the following.

## *p–y* Curves for Soft Clay

Matlock [35] proposed a method to calculate *p–y* curves for soft clays as shown on Figure 32.10. The lateral soil resistance *p* is expressed as

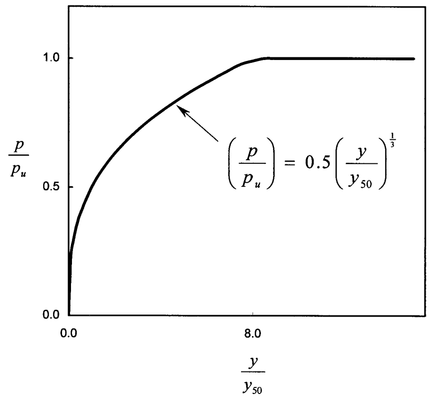


**FIGURE 32.10** Characteristic shape of *p–y* curve for soft clay. [After Matlock, (1970)[35]]

$$p = \begin{cases} 0.5\left(\frac{y}{y_{50}}\right)^{1/3} p_u & y < y_p = 8y_{50} \\ p_u & y \geq y_p \end{cases} \tag{32.34}$$

in which

$p_u$ = ultimate lateral soil resistance corresponding to ultimate shear stress of soil
$y_{50}$ = lateral movement of soil corresponding to 50% of ultimate lateral soil resistance
$y$ = lateral movement of soil

The ultimate lateral soil resistance $p_u$ is calculated as

$$\mathrm{p_u} = \begin{cases} \left(3 + \frac{\gamma' x}{c} + J\frac{x}{B}\right) cB & x < x_r = (6B)/\left(\frac{\gamma' B}{c} + J\right) \\ 9cB & x \geq x_r \end{cases} \tag{32.35}$$

where $\gamma'$ is the effective unit weight, $x$ is the depth from ground surface, $c$ is the undrained shear strength of the clay, and $J$ is a constant frequently taken as 0.5.

The lateral movement of soil corresponding to 50% of ultimate lateral soil resistance $y_{50}$ is calculated as

$$y_{50} = 2.5\varepsilon_{50} B \tag{32.36}$$

where $\varepsilon_{50}$ is the strain of soil corresponding to half of the maximum deviator stress. Table 32.12 shows the representative values of $\varepsilon_{50}$.

## *p–y* Curves for Sands

Reese et al. [53] proposed a method for developing *p–y* curves for sandy materials. As shown on Figure 32.11, a typical *p–y* curve usually consists of the following four segments:

| Segment | Curve type | Range of $y$ | Range of $p$ | $p$–$y$ curve |
|---|---|---|---|---|
| 1 | Linear | 0 to $y_k$ | 0 to $p_k$ | $p = (kx)y$ |
| 2 | Parabolic | $y_k$ to $y_m$ | $p_k$ to $p_m$ | $p = p_m\left(\frac{y}{y_m}\right)^n$ |
| 3 | Linear | $y_m$ to $y_u$ | $p_m$ to $p_u$ | $p = p_m + \frac{p_u - p_m}{y_u - y_m}(y - y_m)$ |
| 4 | Linear | $\geq y_u$ | $p_u$ | $p = p_u$ |

**TABLE 32.12** Representative Values of $\varepsilon_{50}$

| Consistency of Clay | Undrained Shear Strength, psf | $\varepsilon_{50}$ |
|---|---|---|
| Soft | 0–400 | 0.020 |
| Medium stiff | 400–1000 | 0.010 |
| Stiff | 1000–2000 | 0.007 |
| Very stiff | 2000–4000 | 0.005 |
| Hard | 4000–8000 | 0.004 |

1 psf = 0.048 kPa.

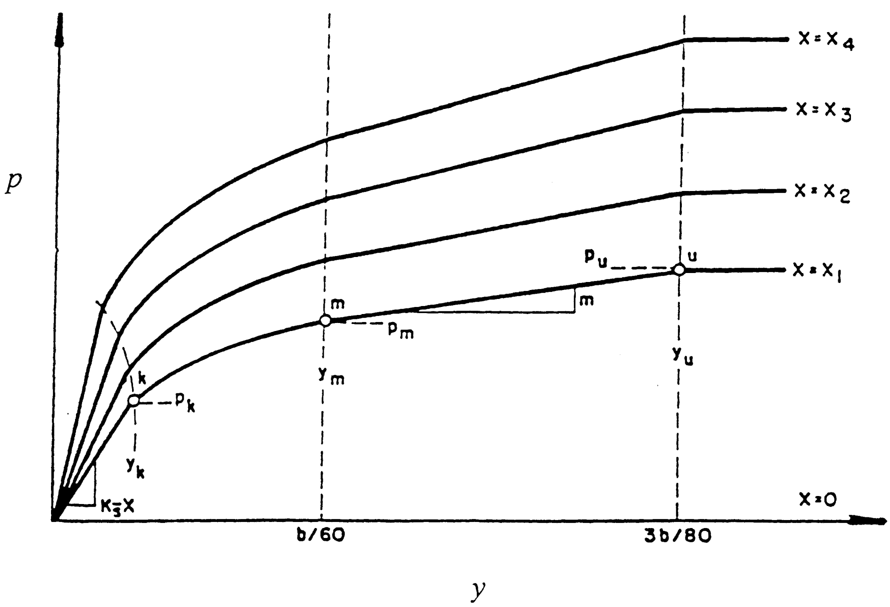


**FIGURE 32.11** Characteristic shape of $p$–$y$ curves for sand. [After Reese, et al. (1974)[53]]

where $y_m$, $y_u$, $p_m$, and $p_u$ can be determined directly from soil parameters. The parabolic form of Segment 2, and the intersection with Segment 1 ($y_k$ and $p_k$) can be determined based on $y_m$, $y_u$, $p_m$, and $p_u$ as shown below.

Segment 1 starts with a straight line with an initial slope of $kx$, where $x$ is the depth from the ground surface to the point where the $p$–$y$ curve is calculated. $k$ is a parameter to be determined based on relative density and is different whether above or below water table. Representative values of $k$ are shown in Table 32.13.

**TABLE 32.13** Friction Angle and Consistency

| Relative to Water Table | Friction Angle and Consistency 29°–30° (Loose) | 30°–36° (Medium Dense) | 36°–40° (Dense) |
|---|---|---|---|
| Above | 20 pci | 60 pci | 125 pci |
| Below | 25 pci | 90 pci | 225 pci |

1 pci = 272 kPa/m.

Segment 2 is parabolic and starts from end of Segment 1 at

$$y_k = \left[\frac{p_m / y_m}{(kx)^n}\right]^{1/(n-1)}$$

and $p_k = (kx)y_k$, the power of the parabolic

$$n = \frac{y_m}{p_m}\left(\frac{p_u - p_m}{y_u - y_m}\right)$$

Segments 3 and 4 are straight lines. $y_m$, $y_u$, $p_m$, and $p_u$ are expressed as

$$y_m = \frac{b}{60} \tag{32.37}$$

$$y_u = \frac{3b}{80} \tag{32.38}$$

$$p_m = B_s p_s \tag{32.39}$$

$$p_u = A_s p_s \tag{32.40}$$

where $b$ is the diameter of a pile; $A_s$ and $B_s$ are coefficients that can be determined from Figures 32.12 and 32.13, depending on either static or cyclic loading conditions; $p_s$ is equal to the minimum of $p_{st}$ and $p_{sd}$, as

$$p_{st} = \gamma x\left[\begin{array}{l}\dfrac{K_o x \tan\varphi \sin\beta}{\tan(\beta-\varphi)\cos\alpha} + \dfrac{\tan\beta}{\tan(\beta-\varphi)}(b + x\tan\beta\tan\alpha) \\ +K_o x \tan\beta(\tan\phi\tan\varphi - \tan\alpha) - K_a b\end{array}\right] \tag{32.41}$$

$$p_{sd} = K_a b x\gamma[\tan^8\beta - 1] + K_o b\gamma x \tan\phi \tan^4\beta \tag{32.42}$$

$$p = \min(p_{st}, p_{sd}) \tag{32.43}$$

in which $\varphi$ is the friction angle of soil; $\alpha$ is taken as $\varphi/2$; $\beta$ is equal to $45^o + \varphi/2$; $K_o$ is the coefficient of the earth pressure at rest and is usually assumed to be 0.4; and $K_a$ is the coefficient of the active earth pressure and equals to $\tan^2(45^o - \varphi/2)$.

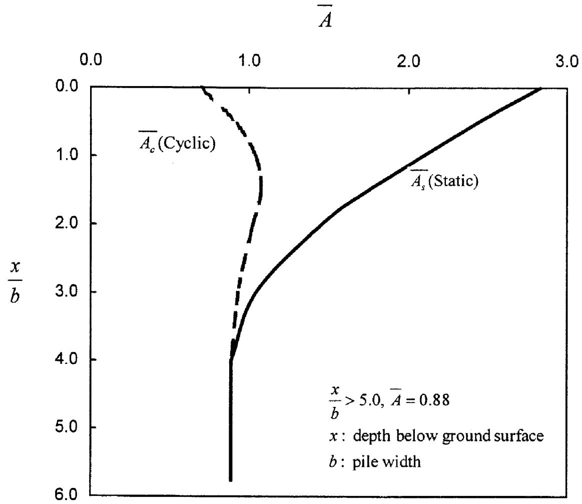


**FIGURE 32.12** Variation of $A_s$ with depth for sand. [After Reese, et al. (1974)[53]]

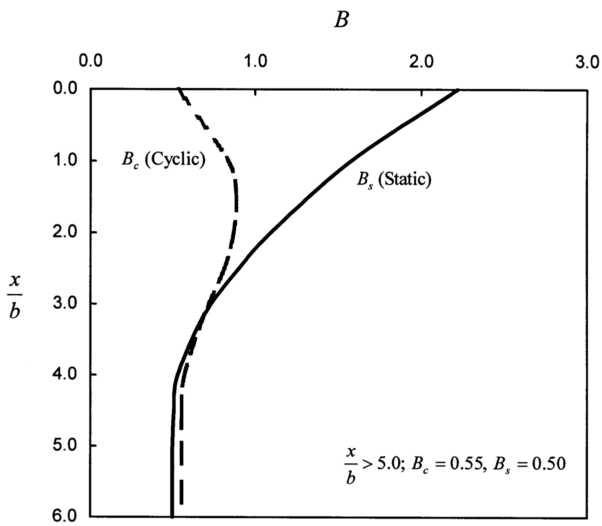


**FIGURE 32.13** Variation of $B_s$ with depth for sand. [After Reese, et al. (1974)[53]]

## 32.5.4 Lateral Spring: *p–y* Curves for Rock

Reese[86] proposed a procedure to calculate *p–y* curves for rock using basic rock and rock mass properties such as compressive strength of intact rock $q_{ur}$, rock quality designation (RQD), and initial modulus of rock $E_{ir}$. A description of the procedure is presented in the following.

A *p–y* curve consists of three segments:

$$
\begin{aligned}
&\text{Segment 1: } p = K_{ir} y && \text{for} \quad y \le y_a \\
&\text{Segment 2: } p = \frac{p_{ur}}{2}\left(\frac{y}{y_{rm}}\right)^{0.25} && \text{for} \quad y_a < y < 16 y_{rm} \qquad (32.44) \\
&\text{Segment 3: } p = p_{ur} && \text{for} \quad y \ge 16 y_{rm}
\end{aligned}
$$

where $p$ is the lateral force per unit pile length and $y$ is the lateral deflection.

$K_{ir}$ is the initial slope and is expressed as

$$K_{ir} = k_{ir} E_{ir} \tag{32.45}$$

$k_{ir}$ is a dimensionless constant and is determined by

$$k_{ir} = \begin{cases} \left(100 + \dfrac{400 x_r}{3b}\right) & \text{for} \quad 0 \le x_r \le 3b \\ 500 & \text{for} \quad x_r > 3b \end{cases} \tag{32.46}$$

$x_r$ = depth below bedrock surface, $b$ is the width of the rock socket
$E_{ir}$ = initial modulus of rock.

$y_a$ is the lateral deflection separating Segment 1 and 2, and

$$y_a = \left( \frac{p_{ur}}{2 y_{rm}^{0.25} K_{ir}} \right)^{1.333} \tag{32.47}$$

where

$$y_{rm} = k_{rm} b \tag{32.48}$$

$k_{rm}$ is a constant, ranging from 0.0005 to 0.00005.

$p_{ur}$ is the ultimate resistance and can be determined by

$$P_{ur} = \begin{cases} a_r q_{ur} b \left(1 + 1.4 \dfrac{x_r}{b}\right) & \text{for} \quad 0 \le x_r \le 3b \\ 5.2 a_r q_{ur} b & \text{for} \quad x_r > 3b \end{cases} \tag{32.49}$$

where

$q_{ur}$ = compressive strength of rock and $\alpha_r$ is a strength reduction factor determined by

$$\alpha_r = 1 - \frac{\mathrm{RQD}}{150} \qquad 0 \le \mathrm{RQD} \le 100 \tag{32.50}$$

RQD = rock quality designation for rock.

## 32.6 Grouped Foundations

### 32.6.1 General

Although a pile group is composed of a number of individual piles, the behavior of a pile group is not equivalent to the sum of all the piles as if they were separate individual piles. The behavior of a pile group is more complex than an individual pile because of the effect of the combination of piles, interactions between the piles in the group, and the effect of the pile cap. For example, stresses in soil from the loading of an individual pile will be insignificant at a certain depth below the pile tip. However, the stresses superimposed from all neighboring piles may increase the level of stress at that depth and result in considerable settlements or a bearing capacity failure, especially if there exists an underlying weak soil layer. The interaction and influence between piles usually diminish for piles spaced at approximately 7 to 8 diameters.

The axial and lateral capacity and the corresponding settlement and lateral deflection of a pile group will be discussed in the following sections.

## 32.6.2 Axial Capacity of Pile Group

The axial capacity of a pile group is the combination of piles in the group, with consideration of interaction between the piles. One way to account for the interaction is to use the group efficiency factor $\eta_a$, which is expressed as:

$$\eta_a = \frac{P_{\text{Group}}}{\sum_i P_{\text{Single\_Pile},i}} \tag{32.51}$$

where $P_{\text{Group}}$ is the axial capacity of a pile group. $\sum_i P_{\text{Single\_Pile},i}$ is the sum of the axial capacity of all the individual piles. Individual piles are discussed in detail in Section 32.4. The group efficiency for axial capacity depends on many factors, such as the installation method, ground conditions, and the function of piles, which are presented in Table 32.14.

**TABLE 32.14** Group Efficiency Factor for Axial Capacity

| Pile Installation Method | Function | Ground Conditions | Expected Group Efficiency | Design Group Efficiency (with minimum spacing equal to 2.5 pile diameter) |
|---|---|---|---|---|
| Driven Pile | End bearing | Sand | 1.0 | 1.0 |
| | Side friction | Loose to medium dense sand | >1.0, up to 2.0 | 1.0, or increase with load test |
| | Side friction | Dense sand | May be ≧ 1.0 | 1.0 |
| Drilled shaft | All | Sand | <1.0 | 0.67–1.0 |
| Driven pile and drilled shaft | Side friction | Soft to medium stiff clay | <1.0 | 0.67–1.0 |
| | End bearing | Soft to medium stiff clay | <1.0 | 0.67–1.0 |
| | Side friction | Stiff clay | 1.0 | 1.0 |
| | End bearing | Stiff clay | 1.0 | 1.0 |
| | Side friction | Clay | <1.0 | Also use "Group Block" |
| | End bearing | Clay, or underlying clay layers | <1.0 | Also use "Group Block" |

At close spacings, driven piles in loose to medium dense sand may densify the sand and consequently increase the lateral stresses and frictions along the piles. However, driven piles in dense sand may cause dilation of the sand and consequently cause heave and damage to other piles. The influence of spacing to the end bearing for sand is usually limited and the group efficiency factor $\eta_a$ is taken as 1.0, under normal conditions.

For drilled piers in loose to medium dense sand, no densification of sand is made. The group efficiency factor $\eta_a$ is usually less than 1.0 because of the influence of other close piles.

For driven piles in stiff to very stiff clay, the piles in a pile group tend to form a "group block" that behaves like a giant, short pile. The size of the group block is the extent of soil enclosed by the piles, including the perimeter piles as shown on Figure 32.14. The group efficiency factor $\eta_a$ is usually equal to 1.0. For piles in soft to medium stiff clay, the group efficiency factor $\eta_a$ is usually less than 1.0 because the shear stress levels are increased by loading from adjacent piles.

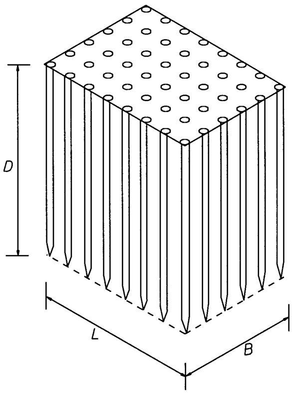


**FIGURE 32.14** Block failure model for pile group in clay.

The group block method is also often used to check the bearing capacity of a pile group. The group block is treated as a large deep spread footing foundation and the assumed bottom level of the footing is different depending on whether the pile is end bearing or frictional. For end-bearing piles, the capacity of the group block is examined by assuming the bottom of the footing is at the tip of the piles. For frictional piles, the capacity of the group pile is checked by assuming that the bottom of the footing is located at ⅓ of the total embedded length above the tip. The bearing capacity of the underlying weaker layers is then estimated by using methods discussed in Chapter 31. The smaller capacity, by using the group efficiency approach, the group block approach, and the group block approach with underlying weaker layers, is selected as the capacity of the pile group.

### 32.6.3 Settlement of a Pile Group

The superimposed stresses from neighboring piles will raise the stress level below the tip of a pile substantially, whereas the stress level is much smaller for an individual pile. The raised stress level has two effects on the settlement of a pile group. The magnitude of the settlement will be larger for a pile group and the influence zone of a pile group will be much greater. The settlement of a pile group will be much larger in the presence of underlying highly compressible layers that would not be stressed under the loading of an individual pile.

The group block method is often used to estimate the settlement of a group. The pile group is simplified to an equivalent massive spread footing foundation except that the bottom of the footing is much deeper. The plane dimensions of the equivalent footing are outlined by the perimeter piles of the pile group. The method to calculate settlement of spread footings is discussed in Chapter 31. The assumed bottom level of the footing block is different depending on either end bearing or frictional piles. For end-bearing piles, the bottom of the footing is at the tip of the piles. For frictional piles, the bottom of the footing is located at ⅓ of total embedded length above the tip. In many cases, settlement requirement also is an important factor in the design of a pile group.

Vesic [79] introduced a method to calculate settlement of a pile group in sand which is expressed as

$$S_g = S_s \sqrt{\frac{B_g}{B_s}} \tag{32.52}$$

where
$S_g$ = the settlement of a pile group
$S_s$ = the settlement of an individual pile

$B_g$ = the smallest dimension of the group block
$B_s$ = the diameter of an individual pile

### 32.6.4 Lateral Capacity and Deflection of a Pile Group

The behavior of a pile group under lateral loading is not well defined. As discussed in the sections above, the lateral moment capacity is greater than the sum of all the piles in a group because piles would form couples resulting from their axial resistance through the action of the pile cap. However, the capacity of a pile group to resist lateral loads is usually smaller than the sum of separate, individual piles because of the interaction between piles.

The approach used by the University of Texas at Austin (Reese, O'Neil, and co-workers) provides a comprehensive and practical method to analyze a pile group under lateral loading. The finite-difference method is used to model the structural behavior of the foundation elements. Piles are connected through a rigid pile cap. Deformations of all the piles, in axial and lateral directions, and force and moment equilibrium are established. The reactions of soil are represented by a series of localized nonlinear axial and lateral springs. The theory and procedures to calculate axial and lateral capacity of individual piles are discussed in detail in Sections 32.4 and 32.5. A computer program is usually required to analyze a pile group because of the complexity and iteration procedure involving nonlinear soil springs.

The interaction of piles is represented by the lateral group efficiency factors, which is multiplied to the *p–y* curves for individual piles to reduce the lateral soil resistance and stiffness. Dunnavant and O'Neil [16] proposed a procedure to calculate the lateral group factors. For a particular pile *i*, the group factor is the product of influence factors from all neighboring piles *j*, as

$$\beta_i = \beta_0 \prod_{\substack{j=1 \\ j \neq i}}^{n} \beta_{ij} \tag{32.53}$$

where $\beta_i$ is the group factor for pile *i*, $\beta_0$ is a total reduction factor and equals 0.85, $\beta_{ij}$ is the influence factor from a neighboring pile *j*, and *n* is the total number of piles. Depending on the location of the piles *i* and *j* in relation to the direction of loading, $\beta_{ij}$ is calculated as follows:

*i* is leading, or directly ahead of *j* ( θ = 0°) $$\beta_l = \beta_{ij} = 0.69 + 0.5 \log_{10}\left(\frac{S_{ij}}{B}\right) \le 1 \tag{32.54}$$

*i* is trailing, or directly behead of *j* (θ = 180°) $$\beta_t = \beta_{ij} = 0.48 + 0.6 \log_{10}\left(\frac{S_{ij}}{B}\right) \le 1 \tag{32.55}$$

*i* and *j* are abreast, or side-by-side (θ = 90°) $$\beta_s = \beta_{ij} = 0.78 + 0.36 \log_{10}\left(\frac{S_{ij}}{B}\right) \le 1 \tag{32.56}$$

where $S_{ij}$ is the center-to-center distance between *i* and *j*, $B$ is the diameter of the piles *i* and *j*, and $\theta$ is the angle between the loading direction and the connection vector from *i* to *j*. When the piles *i* and *j* are at other angles to the direction of loading, $\beta_{ij}$ is computed by interpolation, as

$$0° < \theta < 90° \qquad \beta_{\theta 1} = \beta_{ij} = \beta_l + (\beta_s - \beta_l)\frac{\theta}{90} \tag{32.57}$$

$$90° < \theta < 180° \qquad \beta_{\theta 2} = \beta_{ij} = \beta_t + (\beta_s - \beta_t)\frac{\theta - 90}{90} \tag{32.58}$$

In cases that the diameters of the piles *i* and *j* are different, we propose to use the diameter of pile *j*. To avoid an abrupt change of $\beta_0$ from 0.85 to 1.0, we propose to use:

$$\beta_o = \begin{cases} 0.85 & \text{for} \quad \dfrac{S_{ij}}{B_j} \le 3 \\ 0.85 + 0.0375\left(\dfrac{S_{ij}}{B_j} - 3\right) & \text{for} \quad 3 < \dfrac{S_{ij}}{B_j} < 7 \\ 1.0 & \text{for} \quad \dfrac{S_{ij}}{B_j} \ge 7 \end{cases}$$

## 32.7 Seismic Design

Seismic design of deep bridge foundations is a broad issue. Design procedures and emphases vary with different types of foundations. Since pile groups, including driven piles and drilled cast-in-place shafts, are the most popular types of deep bridge foundations, following discussion will concentrate on the design issues for pile group foundations only.

In most circumstances, seismic design of pile groups is performed to satisfy one or more of the following objectives:

- Determine the capacity and deflection of the foundation under the action of the seismic lateral load;
- Provide the foundation stiffness parameters for dynamic analysis of the overall bridge structures; and
- Ensure integrity of the pile group against liquefaction and slope instability induced ground movement.

### 32.7.1 Seismic Lateral Capacity Design of Pile Groups

In current practice, seismic lateral capacity design of pile groups is often taken as the same as conventional lateral capacity design (see Section 32.5). The seismic lateral force and the seismic moment from the upper structure are first evaluated for each pile group foundation based on the tributary mass of the bridge structure above the foundation level, the location of the center of gravity, and the intensity of the ground surface acceleration. The seismic force and moment are then applied on the pile cap as if they were static forces, and the deflections of the piles and the maximum stresses in each pile are calculated and checked against the allowable design values. Since seismic forces are of transient nature, the factor of safety required for resistance of seismic load can be less than those required for static load. For example, in the Caltrans specification, it is stipulated that the design seismic capacity can be 33% higher than the static capacity [9].

It should be noted that in essence the above procedure is pseudostatic, only the seismic forces from the upper structure are considered, and the effect of seismic ground motion on the behavior of pile group is ignored. The response of a pile group during an earthquake is different from its response to a static lateral loading. As seismic waves pass through the soil layers and cause the soil layers to move laterally, the piles are forced to move along with the surrounding media. Except for the case of very short piles, the pile cap and the pile tip at any moment may move in different directions. This movement induces additional bending moments and stresses in the piles.

Depending on the intensity of the seismic ground motion and the characteristics of the soil strata, this effect can be more critical to the structural integrity of the pile than the lateral load from the upper structure.

Field measurements (e.g., Tazoh et al. [70]), post-earthquake investigation (e.g., Seismic Advisory Committee, [65]), and laboratory model tests (e.g., Nomura et al., [43]) all confirm that seismic ground movements dictate the maximum responses of the piles. The more critical situation is when the soil profile consists of soft layer(s) sandwiched by stiff layers, and the modulus contrast among the layers is large. In this case, local seismic moments and stresses in the pile section close to the soft layer/hard layer interface may very well be much higher than the moments and stresses caused by the lateral seismic loads from the upper structure. If the site investigation reveals that the underground soil profile is of this type and the bridge is of critical importance, it is desirable that a comprehensive dynamic analysis be performed using one of more sophisticated computer programs capable of modeling the dynamic interaction between the soil and the pile system, e.g., SASSI [33]. Results of such dynamic analysis can provide a better understanding of the seismic responses of a pile group.

## 32.7.2 Determination of Pile Group Spring Constants

An important aspect in bridge seismic design is to determine, through dynamic analysis, the magnitude and distribution of seismic forces and moments in the bridge structure. To accomplish this goal, the characteristics of the bridge foundation must be considered appropriately in an analytical model.

At the current design practice, the force–displacement relationships of a pile foundation are commonly simplified in an analytical model as a stiffness matrix, or a set of translational and rotational springs. The characteristics of the springs depend on the stiffness at pile head for individual piles and the geometric configuration of piles in the group. For a pile group consisting of vertical piles, the spring constants can be determined by the following steps:

- The vertical and lateral stiffnesses at the pile head of a single pile, $K_{vv}$ and $K_{hh}$, are first evaluated based on the pile geometry and the soil profile. These values are determined by calculating the displacement at the pile head corresponding to a unit force. For many bridge foundations, a rigid pile cap can be assumed. Design charts are available for uniform soil profiles (e.g., NAVFAC [42]). For most practical soil profiles, however, it is convenient to use computer programs, such as APILE [18] and LPILE [17], to determine the single pile stiffness values. It should be noted that the force–deformation behavior of a pile is highly nonlinear. In evaluating the stiffness values, it is desirable to use the secant modulus in the calculated pile-head force–displacement relationship compatible to the level of pile-head displacement to be developed in the foundation. This is often an iterative process.

  In calculating the lateral stiffness values, it is common practice to introduce a group factor $\eta$, $\eta \leq 1.0$, to account for the effect of the other piles in the same group. The group factor depends on the relative spacing $S/D$ in the pile group, where $S$ is the spacing between two piles and $D$ is the diameter of the individual pile. There are studies reported in the literature about the dynamic group factors for pile groups of different configurations. However, in the current design practice, static group factors are used in calculation of the spring constants. Two different approaches exist in determining the group factor: one is based on reduction of the subgrade reaction moduli; the other is based on the measurement of plastic deformation of the pile group. Since the foundation deformations in the analysis cases involving the spring constants are mostly in the small-strain range, the group factors based on subgrade reaction reduction should be used (e.g., NAVFAC [42]).
- The spring constants of the pile group can be calculated using the following formulas:

$$K_{G,x} = \sum_{i=1}^{N} K_{hh,i} \tag{32.59}$$

$$K_{G,y} = \sum_{i=1}^{N} K_{hh,i} \tag{32.60}$$

$$K_{G,z} = \sum_{i=1}^{N} K_{vv,i} \tag{32.61}$$

$$K_{G,yy} = \sum_{i=1}^{N} K_{vv,i} \cdot x_i^2 \tag{32.62}$$

$$K_{G,xx} = \sum_{i=1}^{N} K_{vv,i} \cdot y_i^2 \tag{32.63}$$

where $K_{G,x}$, $K_{G,y}$, $K_{G,z}$ are the group translational spring constants, $K_{G,yy}$, $K_{G,xx}$ are the group rotational spring constants with respect to the center of the pile cap. All springs are calculated at the center of the pile cap; $K_{vv,i}$ and $K_{hh,i}$ are the lateral and vertical stiffness values at pile head of the $i$th pile; $x_i$, $y_i$ are the coordinates of the $i$th pile in the group; and $N$ is the total number of piles in the group.

In the above formulas, the bending stiffness of a single pile at the pile top and the off-diagonal stiffness terms are ignored. For most bridge pile foundations, these ignored items have only minor significance. Reasonable results can be obtained using the above simplified formulas.

It should be emphasized that the behavior of the soil–pile system is greatly simplified in the concept of "spring constant." The responses of a soil–pile structure system are complicated and highly nonlinear, frequency dependent, and are affected by the inertia/stiffness distribution of the structure above ground. Therefore, for critical structures, it is advisable that analytical models including the entire soil–pile structure system should be used in the design analysis.

### 32.7.3 Design of Pile Foundations against Soil Liquefaction

Liquefaction of loose soil layers during an earthquake poses a serious hazard to pile group foundations. Field observations and experimental studies (e.g., Nomura et al. [43], Miyamoto et al. [40], Tazoh and Gazetas [69], Boulanger et al. [4]) indicate that soil liquefaction during an earthquake has significant impacts on the behavior of pile groups and superstructures. The impacts are largely affected by the intensity of liquefaction-inducing earthquakes and the relative locations of the liquefiable loose soil layers. If a loose layer is close to the ground surface and the earthquake intensity is moderate, the major effect of liquefaction of the loose layer is to increase the fundamental period of the foundation–structure system, causing significant lateral deflection of the pile group and superstructure. For high-intensity earthquakes, and especially if the loose soil layer is sandwiched in hard soil layers, liquefaction of the loose layer often causes cracking and breakage of the piles and complete loss of capacity of the foundation, thus the collapse of the superstructure.

There are several approaches proposed in the literature for calculation of the dynamic responses of a pile or a pile group in a liquefied soil deposit. In current engineering practice, however, more emphasis is on taking proper countermeasures to mitigate the adverse effect of the liquefaction hazard. These mitigation methods include

- Densify the loose, liquefiable soil layer. A stone column is often satisfactory if the loose layer is mostly sand. Other approaches, such as jet grouting, deep soil mixing with cementing agents, and *in situ* vibratory densification, can all be used. If the liquefiable soil layer is close to the ground surface, a complete excavation and replacement with compacted engineering fill is sometimes also feasible.
- Isolate the pile group from the surrounding soil layers. This is often accomplished by installing some types of isolation structures, such as sheet piles, diaphragm walls, soil-mixing piles, etc., around the foundation to form an enclosure. In essence, this approach creates a huge block surrounding the piles with increased lateral stiffness and resistance to shear deformation while limiting the lateral movement of the soil close to the piles.
- Increase the number and dimension of the piles in a foundation and therefore increase the lateral resistance to withstand the forces induced by liquefied soil layers. An example is 10 ft (3.3 m) diameter cast-in-steel shell piles used in bridge seismic retrofit projects in the San Francisco Bay Area following the 1989 Loma Prieta earthquake.

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