**CHAPTER**

# 4    Footing Foundations

## 4-1 Introduction

Footing foundations, also known as spread, combined, or mat footings, transmit design loads into the underlying soil mass through direct contact with the soil immediately beneath the footing. In contrast, pile-supported foundations transmit design loads into the adjacent soil mass through pile friction, end bearing, or both. This chapter addresses footing foundations. Pile foundations are covered in Chapter 5, *Pile Foundations-General*.

Each individual footing foundation must be sized so that the maximum soil-bearing pressure does not exceed the allowable soil bearing capacity of the underlying soil mass. As the load-bearing capacity of most soils is relatively low (2 to 5 Tons per Square Foot (TSF)), the result is footing areas that can be large in relation to the cross section of the supported member. This is particularly true when the supported member is a bridge column.

In addition to bearing capacity considerations, footing settlement also must be considered and must not exceed tolerable limits established for differential and total settlement. Each footing foundation also must be structurally capable of spreading design loads laterally over the entire footing area.

Since the foundation is supported only by the supporting soil mass, the quality of the soil is extremely important. The contract specifications[1] allow the Engineer to revise the footing foundation elevations to ensure that they are on quality material. Refer to Chapter 3, *Contract Administration,* for information on the responsibility of the Engineer as it applies to footing foundations.

## 4-2 Types

Footing foundations can be classified into two general categories:
1. Footings that support a single structural member, frequently referred to as "spread footings."

---

[1] 2010 SS, Section 19-3.04, *Payment* or 2006 SS, Section 19-3.07, *Measurement.*

2. Footings that support two or more structural members, referred to as "combined footings."

Typically, columns are located at the center of spread footings, whereas retaining walls are eccentrically located in relation to the centerline of a continuous footing. Locating a load away from the centroid (center) of the footing creates an eccentricity that changes the distribution of loads in the soil and may result in a bearing pressure that exceeds the allowable bearing capacity. These undesirable loading conditions increase the further the column is placed from the centroid or as the eccentricity increases. The worst of these cases is an edge-loaded footing where the edge of the column is placed at the edge of the footing. The major consideration for these footings is excessive settlement and/or footing rotation on the eccentrically loaded portion of the footing. The effect of column eccentricity on footing rotation and soil-bearing pressures is similar to a centrally loaded footing with a moment. This also will cause an unbalanced load transfer into the soil as shown in Figure 4-1.
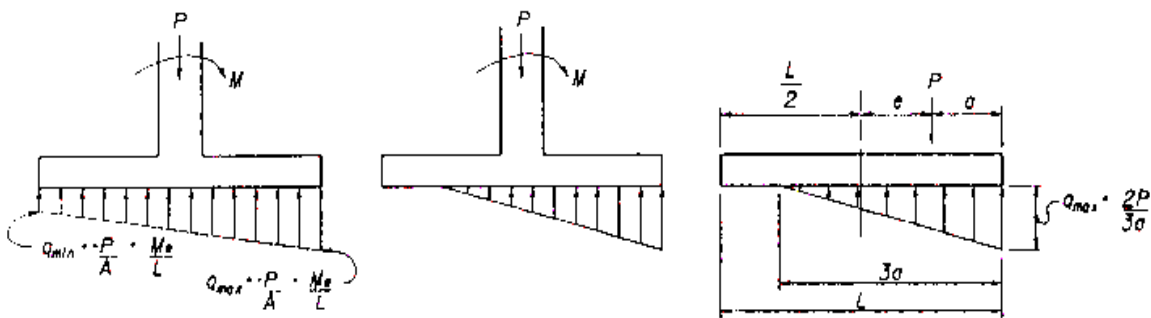


a) Resultant Load in Kern                b) Resultant Load Outside of Kern

**Figure 4-1.  Loaded Footing with Moment.**

In Figure 4-1, the moment (M) may come from a loading condition that needs to be transferred into the soil mass or may be the resultant of the length of the eccentricity multiplied by the load (P). The phrase "outside the kern" refers to a situation when the eccentricity is so great that there is no compression or, worse, there is tension on one side of the footing.

Problems resulting from eccentricities can be addressed by combining two or more columns onto a single footing. This usually is accomplished by one of two methods. In the first method, a single rectangular or trapezoidal footing supports two columns (combined footing). In the other method, a narrow concrete beam structurally connects two spread footings. This type is a cantilever or strap footing.

Combined footings generally are required when loading conditions (magnitude and location of load) are such that single-column footings create undesirable loading conditions, are impractical, or uneconomical. Combined footings also may be required

when column spacing is such that the distance between footings is small or when columns are so numerous that footings cover most of the available foundation area. Generally, economics will determine whether these footings should be combined or remain as individual footings. A single footing that supports numerous columns and/or walls is referred to as a mat footing, and is commonly seen in building work.

Caltrans performed seismic retrofits of spread footings extensively throughout the 1990's. Although this is not a separate category, it is important to understand that foundation work sometimes entails modifications of an existing structure. While the retrofit program is, for the most part, complete there still are structures that may need upgrades either for seismic concerns, scour, or bridge widening. Details of previous footing retrofit strategies are shown in Appendix C, *Footing Foundations*.

Footing foundations encountered in bridge construction almost always support a single structural member (column, pier, or wall) and invariably are referred to as spread footings. Although closely spaced columns do occur in multiple column bents, they are rarely supported on a combined footing. However, recent seismic and scour retrofit projects have incorporated designs that joined together the adjacent footings.

## 4-3  Bearing Capacity

The ultimate bearing capacity of a soil mass supporting a footing foundation is the maximum pressure that can be applied without causing shear failure or excessive settlement. Ultimate bearing capacity solutions are based primarily on the Theory of Plasticity; that is, the soil mass is assumed to be incompressible (does not deform) prior to shear failure. After failure, deformation of the soil mass occurs with no increase in shear (plastic flow).

The implication of the previous statements is that theoretical predictions can only be applied to soils that are homogeneous and incompressible. However, most soils are neither homogeneous nor incompressible. Consequently, known theoretical solutions used in bearing capacity analyses have been modified to provide for variations in soil characteristics. These modifications primarily are based on empirical data obtained through small and, more recently, large-scale testing.

The ultimate soil strength is referred to as Gross Ultimate Bearing Resistance ($q_n$) in Load Resistance Factor Design (LRFD) and Ultimate Gross Bearing Capacity ($q_{ult}$) when working with Working Stress Design (WSD). Once $q_n$ and $q_{ult}$ are calculated, the value is reduced by a factor of safety. The revised value is referred to as Allowable Bearing Capacity ($q_{all}$).

### 4-3.1  Failure Modes
The mode of failure for soils with bearing capacity overloads is shear failure of the soil mass that supports the footing foundation. It will occur in one of three modes:
1.  General shear.
2.  Punching shear.
3.  Local shear.

The Theory of Plasticity describes the general shear failure mode. The other two failure modes: punching and local shear, have no theoretical solutions.

A general shear failure is shown in Figure 4-2 and can be described as follows: The soil wedge immediately beneath the footing (an active Rankine zone acting as part of the footing) pushes Zone II laterally. This horizontal displacement of Zone II causes Zone III (a passive Rankine zone) to move upward.
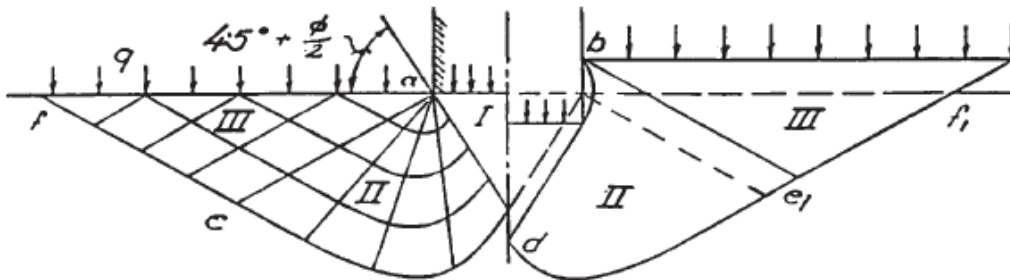


**Figure 4-2  General Shear Failure Concept.**

General shear failure is a brittle failure and usually is sudden and catastrophic. Although ground surface bulging may be observed on both sides of the footing after failure, the failure usually occurs on one side of the footing. Two examples of this failure are:
1.  An isolated structure may tilt substantially or completely overturn.

2.  A footing restrained from rotation by the structure will see increased stresses in the footing and column portions of the structure, which may lead to excessive settlement or collapse.

A punching shear failure (Figure 4-3) presents little, if any, ground surface evidence of failure, since the failure occurs primarily in soil compression immediately beneath the footing. This compression is accompanied by vertical movement of the footing and may or may not be observed, i.e., movement may be occurring in small increments. Footing stability usually is maintained throughout failure (no rotation).
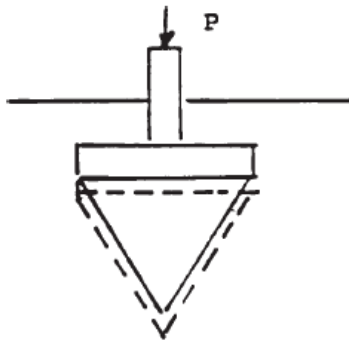
**Figure 4-3.  Punching Shear Failure.**

Local shear failure (Figure 4-4) may exhibit both general and punching shear characteristics, soil compression beneath the footing, and possible ground surface bulging.
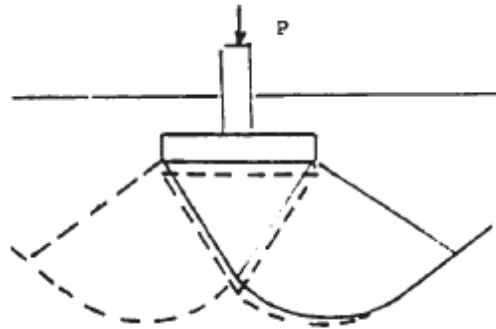


**Figure 4-4.  Local Shear Failure.**

Refer to Figure 4-5 for photographs of actual test failures using a small steel rectangular plate (about 6 inches wide) and sand of different densities.
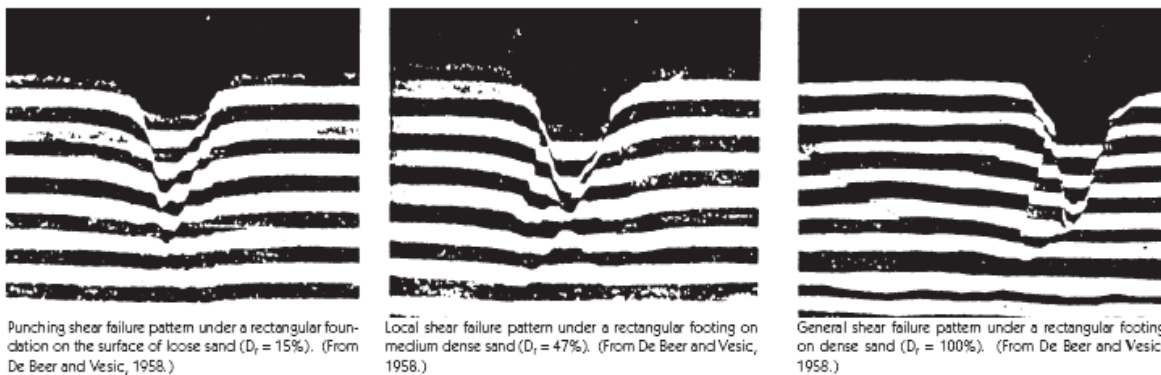


Punching shear failure pattern under a rectangular foundation on the surface of loose sand ($D_r$ = 15%).  (From De Beer and Vesic, 1958.)

Local shear failure pattern under a rectangular footing on medium dense sand ($D_r$ = 47%).  (From De Beer and Vesic, 1958.)

General shear failure pattern under a rectangular footing on dense sand ($D_r$ = 100%).  (From De Beer and Vesic, 1958.)

**Figure 4-5.  Failure Modes.**

The failure mode of a given soil profile cannot be predicted. However, it can be said that the mode of failure depends substantially on the compressibility or incompressibility (Relative Density) of the soil mass. This is not to imply that the soil type of the underlying

material alone determines failure mode. For example, a shallow footing supported on very dense sand will usually fail in general shear, but the same footing supported on very dense sand that is underlain by a soft clay layer may fail in punching shear.

The ultimate bearing capacity of a given soil mass under spread footings usually is determined by one of the variations of the general bearing capacity equation, which was derived by Terzaghi and later modified by Mererhof. It can be used to compute the ultimate bearing capacity as follows:

$$q_{ult} = \frac{\gamma B}{2} N_\gamma + cN_c + \gamma D_f N_q \quad \text{(Terzaghi)}$$

Where: $q_{ult}$ = ultimate bearing capacity

$\Gamma$ = soil unit weight

B = foundation width

$D_f$ = depth to the bottom of the footing below final grade

c = soil cohesion, which for the un-drained condition equals:

$$c = s = \frac{1}{2} q_u$$

Where: s = soil shear strength

$q_u$ = the unconfined compressive strength

In the above equation, $N_\gamma$, $N_c$, and $N_q$ are dimensionless bearing capacity factors that are functions of the angle of internal friction. The term containing factor $N_\gamma$ shows the influence of soil weight and foundation width. The term containing factor $N_c$ shows the influence of the soil cohesion, and that of $N_q$ shows the influence of the surcharge.

**4-3.2  Factors Affecting Bearing Capacity**
Several factors can affect the bearing capacity of a particular soil. They include soil type, relative density or consolidation, soil saturation and location of the water table, and surcharge loads. These factors can act individually or in concert with each other to increase or decrease the bearing capacity of the underlying soil.

When the supporting soil is a cohesionless material (sands), the most important soil characteristic in determining the bearing capacity is the relative density of the material. An increase in relative density is accompanied by an increase in the bearing capacity. Relative density is a function of both ø and γ; the angle of internal friction and unit weight, respectively. In cohesive soils (clays), the unconfined compressive strength ($q_u$,) is the soil

characteristic that affects bearing capacity. The unconfined compressive strength ($q_u$) is a function of clay consistency. The bearing capacity increases with an increase in $q_u$ values. The bearing capacity of both sands and clays are influenced by the location of the water table with respect to the bottom of the footing. When the distance to the water table from the bottom of the footing is greater than or equal to the width of the footing B, (Refer to Figure 4-6), the soil unit weight is used in the general bearing capacity formula. At these depths, the bearing capacity is only marginally affected by the presence of water and can be disregarded. When the water table is at or below the base of the footing, a ratio between the unit weight of the soil above the water table and the submerged unit weight is used in the first term of the bearing capacity equation. The impact of the water table on the bearing capacity of the soil beneath the bottom of the footing is substantial as it effectively reduces the first term of the equation by approximately 50%. The submerged unit weight $\gamma'$ or $\gamma_{sub,}$ as it is sometimes called, is determined as follows:

$$\gamma' = \gamma_{sat} - \gamma_w$$

Where:   $\gamma'$ = Submerged unit weight

$\gamma_m$ = Saturated unit weight (Sometimes shown at $\gamma_{sat}$)

$\gamma_w$ = Unit weight of water
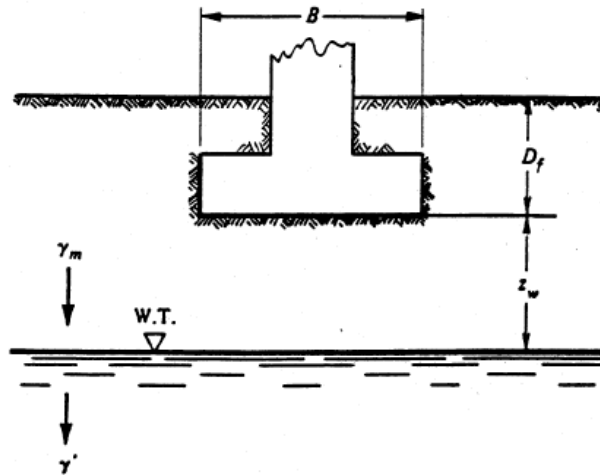


for $z_w \geq B$ : use $\gamma = \gamma_m$  (no effect)
for $z_w < B$ : use $\gamma = \gamma' + (z_w/B)*(\gamma_m - \gamma')$
for $z_w \leq B$ : use $\gamma = \gamma$

**Figure 4-6.  Influence of Groundwater Table on Bearing Capacity.**

It is apparent that bearing capacity of both cohesionless and cohesive soils will be reduced as the water table gets closer to the bottom of footings. This is validated by the general bearing capacity formula, as lower capacities will occur when the lighter submerged unit weight of soil is substituted for the dry unit weight. Therefore, the effects of the water table on the bearing capacity of the footing soil mass must be considered at all times during construction.
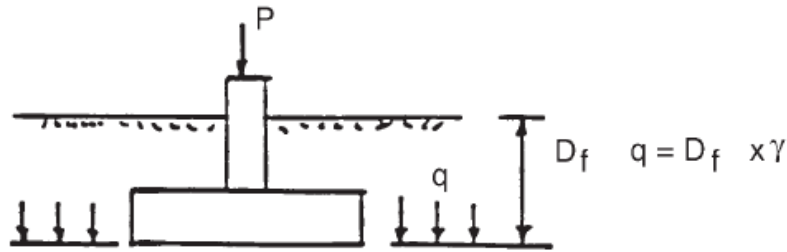
**Figure 4-7. Surcharge Load on Soil.**

The depth of the footing below original ground or future finished grade is yet another factor that affects the bearing capacity of the soil beneath the foundation. The term $D_f$ is used in determining the overburden, or surcharge load, acting on the soil at the plane of the bottom of footing (Figure 4-7). This surcharge load has the net effect of increasing the bearing capacity of the soil by restraining the vertical movement of the soil outside the footing limits.
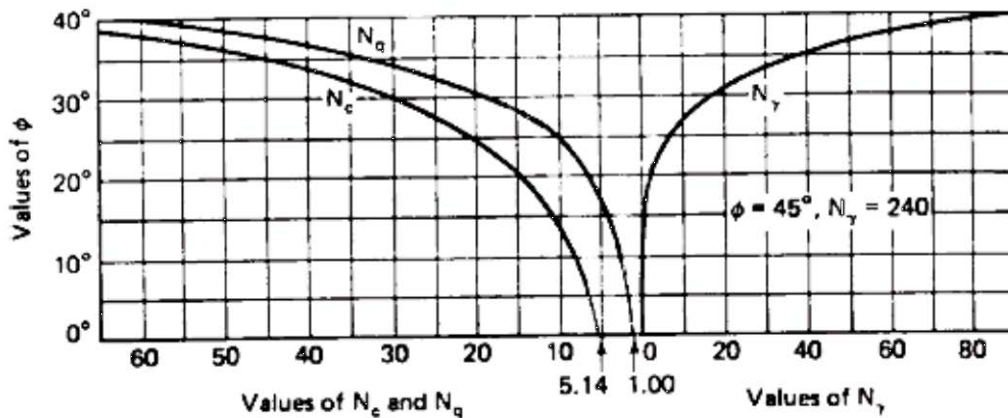


**Figure 4-8. Relationship Between ø and Bearing Capacity Factors.**

Lastly, the shape of the footing foundation affects the bearing capacity of the soil. Theoretical solutions for ultimate bearing capacity are limited to continuous footings (length/width$\geq$10). Shape factors for footings (other than continuous footings) have been determined primarily through semi-empirical methods. In general, the ultimate bearing capacity of a foundation material supporting a square or rectangular footing is greater than the capacity of a continuous footing when the supporting material is cohesive (clay), and less than the bearing capacity of a continuous footing when the supporting material is cohesionless (sand).
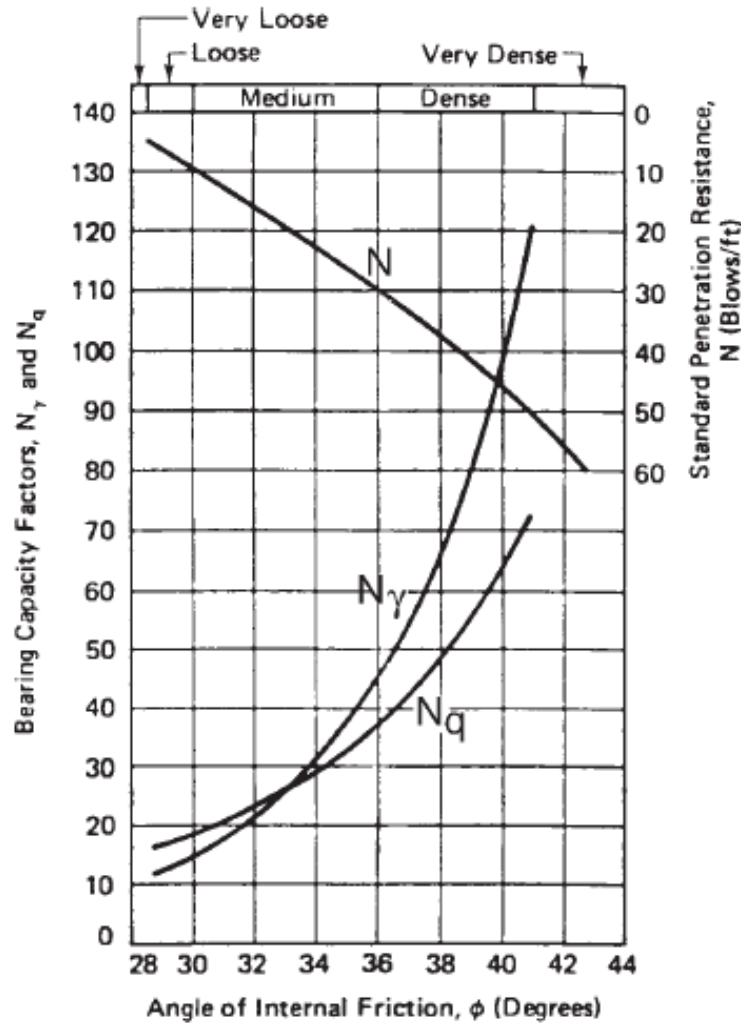
**Figure 4-9. Relationship of Bearing Capacity Factors to ø and N (Standard Penetration Resistance) for Cohesionless Soils.**

The general bearing capacity equation also can be used to give a field estimate of the ultimate bearing capacity of temporary footings, such as falsework pads. For cohesionless soils, a relationship between the standard penetration resistance, N, and the bearing capacity factors, $N_\gamma$ and $N_q$, is shown in Figure 4-9. The relationship between N and the angle of internal friction, ø, also can be determined from Figure 4-9. When soils are known to have some cohesion, the value of ø determined from Figure 4-9 then can be used in the chart shown in Figure 4-8 to determine the bearing capacity factors, $N_\gamma$, $N_c$, and $N_q$. Values for ø, $q_u$, N, and γ can be found on the *Log of Test Borings (LOTB)* or can be approximated by using the tables for granular and cohesive soils shown in Appendix A, *Foundation Investigations*.

## 4-4 Settlement

Footing foundations will settle over time as the soil densifies from the additional weight it is required to support. Caltrans's current practice is to limit total permissible settlement to:

- One inch for a shallow footing for multi-span structures with continuous spans or multi-column bents.
- One inch for single span structures with diaphragm abutments.
- Two inches for single span structures with seat abutments.

To achieve this, allowable bearing pressures generally are reduced to 25% to 33% of the ultimate bearing capacity as determined by the general bearing capacity formula. This reduction essentially places a factor of safety on the ultimate bearing capacity and is in line with the reductions discussed above to obtain allowable and nominal bearing capacities.

Cohesionless soils will densify under the pressure of the foundation as the individual soil particles are pushed together, effectively compacting it. In general, soils with low relative densities will see more settlement than well-compacted soils that have higher relative densities. Settlement is immediate in cohesionless materials. Cohesive soils, however, consolidate over time as the pressure of the overlying foundation forces water from the soil, relieving excess pore water pressures.

## 4-5 Ground Improvement/Soil Modification

Bridges frequently need to be constructed at locations where the *in situ* material is not suitable for the intended purpose. Instead of utilizing a pile foundation, Geotechnical Services specifies ground modification of the foundation area to "engineer" it for its intended use. Economics, soil type, and engineering loads drive the decision to use ground modification and avoid the additional cost of a pile foundation.

Ground modification techniques are used to increase the bearing capacity of the foundation material by increasing the relative compaction of the material either through densification or the introduction of grouts to compress and bind the soils. Ground modification techniques generally lend themselves to cohesionless materials. These techniques can include the following: settlement periods, vibro-compaction, jet grouting, stone columns, dynamic compaction, and wick drains, among others. These modification techniques improve the bearing capacity of the soil by increasing the relative density of the soil through external means, or by adding materials such as a cement or chemical grout to achieve a similar result. Modification of cohesive soils can be achieved; however, these methods often are time-consuming and limited to wick drains and settlement periods. As discussed later in this chapter, the replacement of poor quality soils by over-excavation and replacement with competent material may be appropriate.

Some modification techniques involve a settlement period where the underlying foundation is preloaded with a surcharge for a specified length of time prior to the foundation construction. The loading typically consists of an embankment constructed to specified limits. Geotechnical Services determines the need to preload the foundation area, specifies the limits of the embankment, and sets forth the duration of the settlement period in the contract *Special Provisions*.

When settlement periods are less than 60 days, the Engineer should install settlement hubs in the top of the bridge embankments and monitor (survey) and record changes to the original elevations. The Engineer is responsible for terminating a settlement period. Data from the hub elevation surveys are used to determine when this should take place. If settlement is still taking place at the end of the 60-day period, then the settlement period should be extended until the settlement has ceased. However, if no settlement occurred during the last week or two of the settlement period, the settlement period should be terminated at the end of the 60-day period or to shorten the length of the settlement period. The Contractor should be notified of this decision in writing.

Settlement platforms usually are required when settlement periods greater than 60 days are specified. Geotechnical Services has a Geotechnical Instrumentation Branch that provides advice for the installation of the settlement platforms[2] (Refer to Appendix C, *Footing Foundations,* for California Test 112 - *Method for Installation* and *Use of Embankment Settlement Devices*). Unless this work is outlined in the *Special Provisions*, the Engineer needs to write a change order to compensate the Contractor for the initial installation of the settlement platforms.

## 4-6  Construction and Inspection

As discussed in Chapter 3, *Contract Administration*, the Engineer should have a complete understanding of all contract documents as early as practical in the construction process. This ensures that potential impact, with regard to the foundations, is identified early and paths to resolution are begun before actual construction begins.

The Engineer should write a letter reminding the Contractor of the provisions stated in the contract specifications[3] (Refer to Chapter 3-3, *Change Orders,* of this Manual and Appendix C, *Footing Foundations,* for sample letter). This reminds the Contractor that footing elevations and seal courses shown on the contract plans are approximate only and foundation modifications may be required[4].

---

[2] Bridge Construction Memo 130-13.0, *CIDH Pile Information Submittal.*
[3] 2010 SS, Section 51-1.03C(1), *Preparation, General,* or 2006 SS, Section 51-1.03, *Depth of Footing.*
[4] Bridge Construction Memo 2-9.0, *Footing and Seal Course Revisions.*

The Engineer should review and become familiar with the following documents as described in Chapter 3, *Contract Administration*. This table identifies specific sections of the *Standard Specifications* to consider for footing foundations.

**Table 4-1.  Standard Specifications Sections for Footing Foundations.**

| ISSUE | SPECIFICATION |
|---|---|
| Acceptable methods for water control and foundation treatment. | 2010 SS, Section 19-3.03D, *Irrigation Sleeves,* or 2006 SS, Section 19-3.04, *Water Control and Foundation Treatment.* |
| Contractor notification to Engineer when excavation is substantially complete and ready for inspection (prior to Engineer's authorization to pour concrete). | 2010 SS, Section 19-3.03B(1), *Structure Excavation, General,* or 2006 SS, Section 19-3.05, *Inspection.* |
| Excavation measurements limits, and addressing revisions to excavation limits required to meet Designer's intent. | 2010 SS, Section 19-3.04, *Payment,* or 2006 SS, Section 19-3.07, *Measurement.* |
| Relative compaction (of not less than 95% is required for embankments within 150 feet of bridge abutments or retaining wall footings not supported on piles). | 2010 SS, Section 19-5.03B, *Relative Compaction (95%)*, or 2006 SS, Section 19-5.03, *Relative Compaction (95%).* |
| Bridge footings constructed into an embankment (embankment constructed to the elevation of the grading plane, and finished slope extended to the grading plane before excavating for the footings). | 2010 SS, Section 19-6.03A, *Construction, General,* or 2006 SS, Section 19-6.01, *Placing.* |
| Surcharge and settlement period specifications and embankment minimum limits in the *Special Provision*s (embankment shall remain in place for the required period before excavating for footings). | 2010 SS, Section 19-6.03D, *Settlement Periods and Surcharges,* or 2006 SS, Section 19-6.025, *Settlement Period.* |
| Specifications for plan footing elevations and seal courses, and Engineer change orders to footing elevations (Bridge Construction Memo 2-9.0, *Footing and Seal Course Revisions*). | 2010 SS, Section 51-1.03C(1), *Preparation, General,* or 2006 SS, Section 51-1.03, *Depth of Footing.* |
| Groundwater pumping from foundation enclosures that prevents removal of any concrete material. | 2010 SS, Section 51-1.03C(1), *Preparation, General,* or 2006 SS, Section 51-1.04, *Pumping.* |
| Concrete placing, vibrating, and screeding procedures for footings. | 2010 SS, Section 51-1.03D(1), *Placing Concrete, General,* or 2006 SS, Section 51-1.09, *Placing Concrete.* |

## 4-7  Excavations

Excavation and trenching are inherent when constructing foundation elements, such as footing foundations. The *Caltrans Trenching and Shoring Manual* provides information about the complete process for administering, designing and reviewing excavation work and plans. What follows is a brief description of what to consider before excavation.

### 4-7.1 Open Excavations

The open excavation, or trench, is a potentially dangerous area at a construction site. Worker safety must be considered and addressed during excavation operations and/or shoring construction. The Division of Occupational Safety and Health (DOSH), better known as CalOSHA, requires each employee in an excavation be protected from cave-ins by an adequate protective system. The protective system can consist of metal or timber shoring, a shield system, a sloping system, or a sloping and benching system. When a sloping or sloping and benching system is substituted for shoring or other protective systems, and the excavation is less than 20-feet deep, DOSH requirements can be selected by the Contractor in accordance with the requirements of Section 1541.1(b) of the *Construction Safety Orders*[5]. Section 1541.1(b)(1) allows slopes to be constructed (without first classifying the soil) in accordance with the requirements for a Type C soil (1½:1 maximum). Section 1541.1(b)(2) requires the Contractor's "competent person" to first classify the soil as either a Type A, B, or C soil, or stable rock, before selecting the appropriate slope configuration. Section 1541.1(b)(3) allows the use of tabulated data under certain conditions; and Section 1541.1(b)(4) addresses engineered plans. The Engineer should refer to the *Caltrans Trenching and Shoring Manual* or go directly to the Cal/OSHA website[6] when reviewing a Contractor's excavation safety plan for compliance with the construction safety orders.

Surcharge loads from materials, equipment, or excavation spoils must be located a sufficient distance back from the edge of excavations to maintain slope stability. For sloped excavations, the minimum setback can be determined from Figure 4-10 on the following page.

---

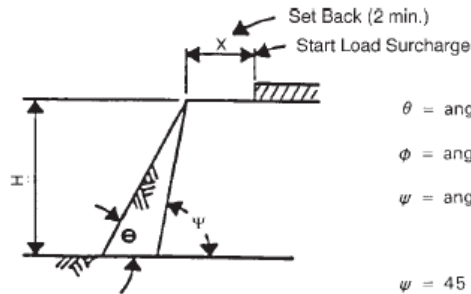[5] http://www.dir.ca.gov/title8/1541_1.html
[6] http://www.dir.ca.gov/samples/search/query.htm

Example:
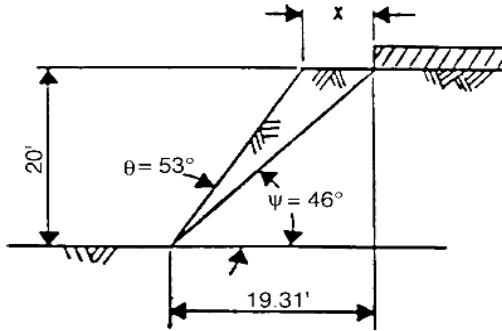


$H = 20'$

$\theta = 53$ degrees (3/4:1)

$\psi = 46$ degrees
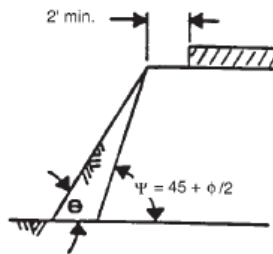
$20'/\tan(46) = 19.31'$

$20'/\tan(53) = 15.07'$

$X = 19.31' - 15.07' = 4.24'$



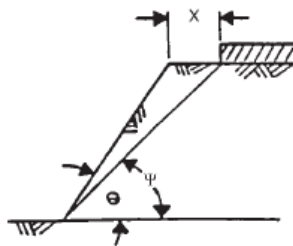$\theta$ = angle of slope

$\phi$ = angle of internal friction

$\psi$ = angle of the failure plane

$\psi = 45 + (\phi/2)$



If $\theta \le \psi$ then the surcharge will not affect the stability of the slope, and X may be the OSHA minimum of 2 feet.



If $\theta > \psi$ then X must be calculated by geometry and $X \ge 2$ feet.

**Figure 4-10.  Slope Setback for Open Excavations/Trenches.**

### 4-7.2 Cofferdams or Shored Excavations

Cofferdams and/or shored excavations require an engineered plan stamped by a registered Civil Engineer. The Contractor is responsible for designing these elements and the Engineer is responsible for review and authorization. The *Caltrans Trenching and Shoring Manual* provides procedures for reviewing and authorizing these plans. An important consideration in shored excavations is the minimum setback for a surcharge when on level ground. The setback usually is equal to the depth of the excavation unless specific surcharge loads are considered in the shoring design. The "Boussinesq" strip load formula is recommended for calculating the lateral pressures due to surcharge. (Figure 4-11). For example, no minimum setback of the surcharge load would be required if the earth support system is designed for the summation of lateral pressures due to the surcharge and earth pressures. However, a barrier should be provided to prevent material from entering the excavation. The *Caltrans Trenching and Shoring Manual* includes several examples of how this formula is used, and the SC website has a spreadsheet that can be used to calculate the pressures[7].
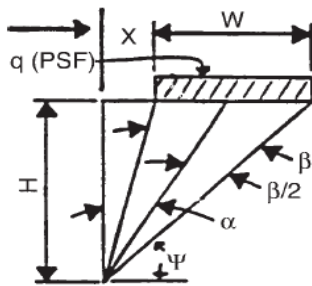


To calculate lateral pressures due to surcharge, the "Bousineaq" strip load formula is recommended.

At Depth "H",

$$\sigma_h(PSF) = \frac{2q}{\pi}(\beta_r - \sin\beta\cos 2\alpha)$$

where $\beta_r$ is in radians

At full height H,

$$\alpha + \frac{\beta}{2} \leq \psi$$

$$\alpha = \arctan\frac{X}{H} + \frac{\beta}{2}$$

$$\beta = \arctan\frac{X+W}{H} - \arctan\frac{X}{H}$$

$$W_{max} = \tan\psi H - \tan\left(\arctan\frac{X}{H}\right)H$$

**Figure 4-11. Effect of Surcharge Loads for Shored Excavations.**

---

[7] http://onramp.dot.ca.gov/hq/oscnet, Software, Bossinesq Strip Loading Program.

If the earth support system is not designed for lateral pressures due to surcharge, then a setback distance must be used. It can be calculated as shown in Figure 4-12. Setback information should be shown on the authorized shoring plans and clearly designated in the field. Refer to the *Caltrans Trenching and Shoring Manual* for information regarding shoring design and construction.



Let X = setback of surcharge load
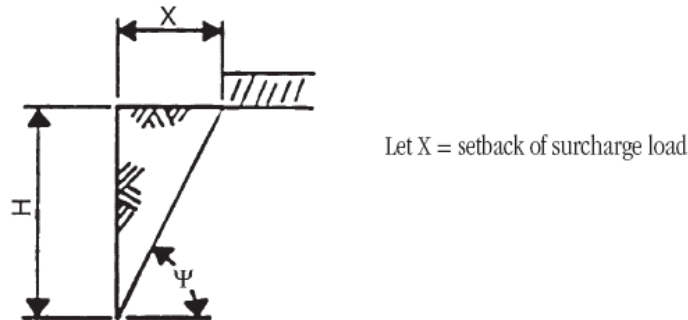
$$X = \frac{H}{\tan\left(45 + \frac{\phi}{2}\right)}$$

$\phi = \angle$ of internal friction

$\psi = \angle$ of failure plane $= 45 + \frac{\phi}{2}$

For most soils, $\psi$ is about 55°

**Figure 4-12. Setback Calculation for Shored Excavations When Surcharges are not Considered in the Shoring Design.**

### 4-7.3 Wet Excavations
The contract specifications[8] describe methods to be utilized when water is encountered in excavations and seal courses are not shown on the contract plans. The means and methods used to control groundwater are at the option of the Contractor and need to be clearly understood, as there are environmental considerations when dealing with groundwater control. Contract specifications[9] address the control and disposal of ground water. All SC employees have the responsibility to inspect structure work for compliance with environmental regulations; as such, these operations should be discussed with the Resident Engineer to ensure that the environmental considerations are addressed before any work begins.

[8] 2010 SS, Section 19-3.03D, *Water Control and Foundation Treatment,* or 2006 SS, Section 19-3.04, *Water Control and Foundation Treatment.*
[9] 2010 SS, Section 13-4.03G, *Dewatering,* or Special Provisions for contracts using 2006 SS.

Sump pumps frequently are used to remove surface water and minor infiltrations of groundwater that enter an excavation. The sumps and any connecting interceptor ditches should be located well outside of the footing area, and below the bottom of footings so that groundwater will not disturb the foundation's bearing surface.

In cohesionless (granular) soils, it is important to make sure that the fine particles within the soil mass are not carried away by the pumping operation. Loss of fines may impair the bearing capacity of the soil for the foundation under construction, and also may cause existing structures adjacent to the operations to settle. The amount of soil particles carried away can be determined by periodically collecting discharged water in a container and observing the amount of sediment. If there is a large flow of groundwater and/or prolonged pumping is required, the sump(s) should be lined with a filter material to prevent or minimize the loss of fines.

In some excavations, the use of sumps may not be sufficient to address the infiltration of groundwater into the excavation. When this is the case, cofferdams generally are used; however, some contractors will opt to lower the groundwater table. One commonly used method to achieve this is with the single-stage well point system (Figure 4-13).
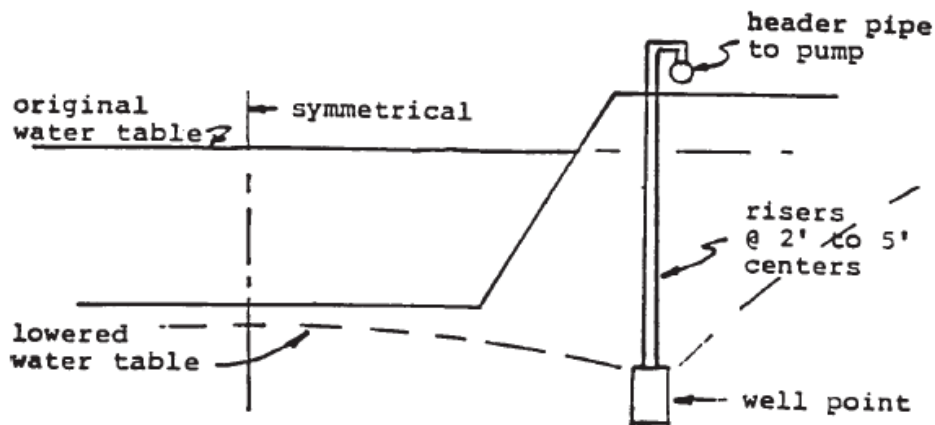


**Figure 4-13.  Single Stage Well Point System.**

A well point is a section of perforated pipe about 2 to 3 inches in diameter and 2 to 4 feet in length. Perforations are covered with a screen and the end of the pipe is equipped with a driving head and/or holes for jetting. Several well points are installed around the excavation perimeter, generally spaced at 2 to 5-foot centers. They are connected to 2 to 3-inch diameter riser pipes and are inserted into the ground by driving and/or jetting. The riser pipes are connected to a header pipe that is connected to a pump. A single stage well point system can lower the water table 15 to 18 feet below the elevation of the header pipe. For greater depths, a multiple stage system must be used. A single or multiple stage well point system is effective in fine to medium granular soils or soils containing seams

of such material. In stratified clay soils, vertical sand drains (auger holes backfilled with sand) may be required to draw water down from above the well points.

Another system for lowering the water table is a deep well. Deep wells consist of either a submersible pump, turbine, or water ejector at the bottom of 6 to 24-inch diameter casings, either slotted or perforated. Units are screened, but filter material should be provided in the well to prevent clogging and loss of fines.

Deep wells can be spaced 25 to 120 feet apart and are capable of lowering a large head of water. They can be located a considerable distance from the excavation and are less expensive than the multiple stage well point system for dewatering large areas; however, they are only appropriate in certain soils.

If a soft clay strata overlying sand is encountered and dewatering is contemplated, lowering the water table by pumping from underlying layers of sand may not be a preferred option because it will cause large, progressive settlement of the clay strata in the surrounding area. By lowering the water table in the sand lens, the condition in the clay lens switches from an un-drained condition to a drained condition. This allows excess pore water pressures to be dissipated more quickly and to a greater extent than it would have been had the water table not been lowered. Essentially, there is an increase in the effective pressure acting on the saturated clay, i.e., density of clay above the lowered water table will increase from a submerged unit weight to a saturated unit weight, an increase of 62.4 Pounds per Cubic Foot (PCF) (Figure 4-14).
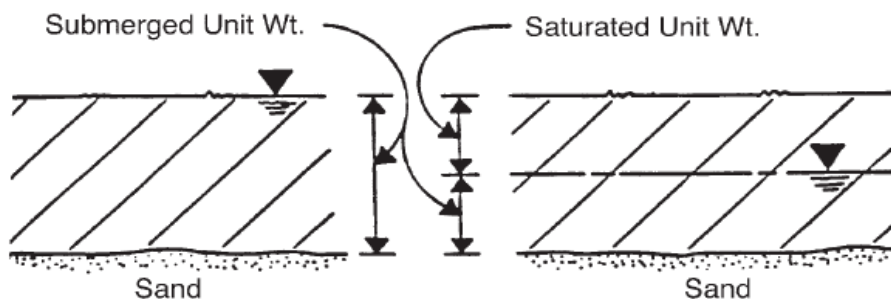


Figure 4-14.  Saturated vs. Submerged Unit Weight.

**4-7.4  Bottom of Excavation Stability**
The control of groundwater is essential to the stability of a shoring system and the underlying soil intended to support the new foundations. In addition to controlling groundwater to facilitate construction operations, the Engineer also must consider soil heave and piping as they relate to the stability of the bottom of the excavation.

Heave is the phenomenon whereby the static or hydraulic pressures (head) of the surrounding material cause the upward movement of the material in the bottom of the excavation. This corresponds with settlement of the surrounding material. Heave

generally occurs in soft clays when the hydrostatic head, 62.4(h + z), is greater than the weight of the overburden at the bottom of the excavation, $\gamma z$ (Figure 4-15).
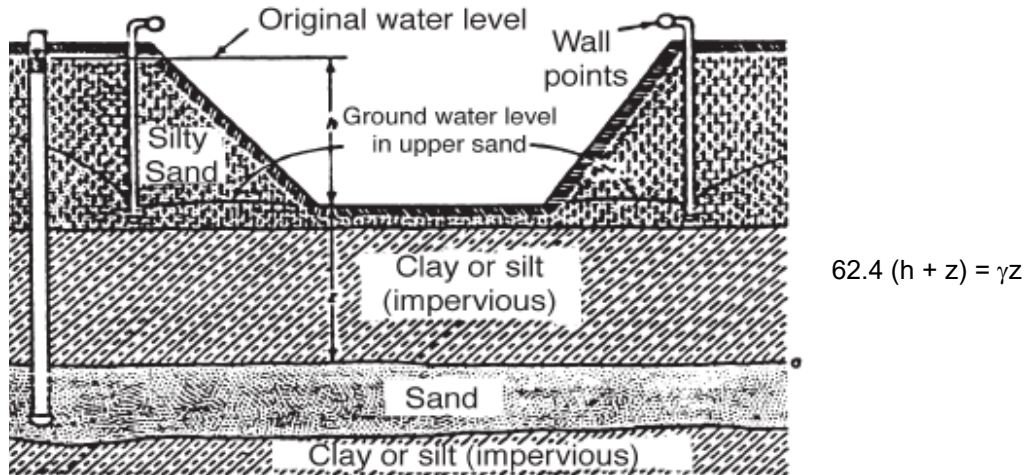


62.4 (h + z) = $\gamma z$

**Figure 4-15. Bottom of Excavation Stability Problems Due to Excess Hydrostatic Head Against an Impervious Layer.**

Piping is associated with pervious materials and can occur when an unbalanced hydrostatic head exists. This unbalanced head may cause large upward flows of water into the excavation, transporting material in the process, and may result in settlement of the surrounding area. Review the *Caltrans Trenching and Shoring Manual* if instability problems are expected at the bottom of excavations.

### 4-7.5  Foundation Inspection & Construction Considerations
A *Footing Foundation Construction Checklist* is presented in Appendix K-4 to assist field personnel in preparing documents and inspecting field work to ensure compliance with contract requirements.

Inspection should confirm the following:
- Stability of slopes and sides of excavations conform to CalOSHA requirements.
- Foundation materials conform to the information shown on the *LOTB* (allowance should be made for some non-uniformity such as small pockets and lenses of material having somewhat different properties).
- Condition of the foundation-bearing surface is undisturbed by excavation operations and uncontaminated by sloughing and/or entrance of water.
- Proximity of structures, highways, railroads, and other facilities that may require shoring or underpinning,(checked prior to excavation) conform to guidelines.
- Foundation element forms conform to layout, depth, dimensions, and construction grade shown on the contract plans. Forms are mortar-tight.

- Reinforcing steel is firmly and securely tied in place; shear steel is hooked to both top and bottom rebar mats and securely tied; and there is a proper concrete cover over the top rebar mat.
- Concrete has the proper mix number; adequate truck revolutions; concrete temperature and back-up alarm; wet down rebar and forms; does not drop over 8 feet; and has been reconsolidated and the top one foot of concrete finished no sooner than 15 minutes after initial screeding before it is cured.
- A sufficient bench width around the excavation to prevent sloughing or cave-in and provide for access and for work area.

The footing forms are either built out of timber or consist of prefabricated panels. The forms generally are secured at the bottom by stakes, horizontal kickers, or ties, and are externally braced, tied or strapped at the top. If the forms extend above the top of the footing elevation, a pour strip or similar device must be attached to the forms to designate the top-of-footing elevation.

Footings for shored excavations often are excavated and placed "neat," which means that the excavation limits essentially are the footing limits. Concrete is placed against the sides of the excavation, eliminating the need for footing forms. Top-of-footing grades must be clearly delineated with stakes or flagged spikes driven into the sides of the excavation. "Neat" excavations must conform to the planned footing dimensions. If they vary, the exact, as-constructed footing dimensions must be placed on the "as-built" drawings. (Previous seismic retrofit projects and footing widenings were not "as-built" properly and costly contract change orders were required to address these undocumented overpours. Care should be taken to make sure that the footing concrete is not damaged during shoring removal operations.)

Whether footings are formed or excavated "neat," a template should be constructed to ensure that the position of the vertical reinforcing steel is maintained during concrete placement. All reinforcing steel must be securely blocked and tied to prevent vertical and/or lateral displacement during concrete placement. Reinforcing steel must not be hung or suspended from the formwork or templates, as the weight of suspended rebar can cause settlement in the form panels and affect pour grades and displace during concrete placement. Top reinforcing steel mats that are supported must be blocked to the forms or sides of the excavation. The bottom reinforcing steel mat that supports the vertical column steel must be adequately blocked to prevent any settlement. In addition, reinforcing steel dowels must be tied in place before concrete placement and not "wet-set" during or after concrete placement.

The effective depth of reinforcing steel is critical and always must be verified. For a footing supporting a single column, pier or wall, the effective depth is the distance from the centroid of the reinforcing steel to the top of the concrete footing. The bottom mat must be located at the design depth, even for over-excavated footings, since the bottom mat supports the vertical column reinforcement and the location of the top mat is tied to

the bottom mat by the shear hooks. Lowering the bottom mat is not recommended, as it would require longer vertical steel, longer shear hooks, and may require mechanical or welded splices on the longitudinal bars. It should be noted that the additional concrete placed below the bottom steel mat in over-excavated footings does not increase the design depth of the footing but should be noted on the as-built plan sheets.

Footing inspections should occur as the work progresses so that deviations and non-compliant issues can be addressed in a timely manner. However, it is important to inspect the footing just prior to concrete placement to ensure that nothing has changed. All material that has sloughed into the excavation must be removed prior to placing concrete. To verify that settlement of the rebar cage has not occurred, the minimum clearances between the bottom of the excavation and the bottom reinforcing steel mat must be re-inspected. The foundation material should be wet down but not saturated. The ends of the concrete pour chutes should be equipped to prevent free fall of concrete in excess of 8 feet. This will prevent segregation of the concrete and may include a hopper and/or length of tremie tube.

## 4-8  Foundation Problems and Solutions

The excavated surface at the planned footing elevation must be inspected after the excavation is completed.  The Engineer must conduct a thorough physical inspection of the foundation material to determine if the foundation is suitable, disturbed and/or contaminated, or unsuitable. Addressing contaminated material is the responsibility of the Contractor, while unsuitable material is the responsibility of Caltrans. The phrase "contaminated material" as used here should not be confused with materials contaminated with lead, hydrocarbons, heavy metals, etc. Information about environmentally contaminated materials is addressed in the contract plans and *Special Provisions*.

### 4-8.1  Disturbed and/or Contaminated Material
Disturbed and/or contaminated foundation material encountered at the planned bottom of footing elevation is unacceptable and must be corrected even if the material itself is suitable. Disturbance of the foundation-bearing surface usually is caused by the excavation means and methods, including excavating below the footing elevation or disturbing the grade with the teeth on the excavator bucket. Contamination usually is due to the presence of water (typically uncontrolled) or sloughing. All disturbed or contaminated material must be removed to expose a suitable foundation surface. The foundation must then be restored by the Contractor, at the Contractor's expense, to a condition at least equal to the undisturbed foundation as determined by the Engineer.

The following precautionary measures can be taken during excavation and construction to avoid or minimize the disturbance and/or contamination of the foundation surface:

- Under-excavate with mechanical equipment and excavate to bottom of footings by hand or by using a cleanup bucket
- Divert surface water away from the excavation
- Minimize exposure of the foundation material to the elements by constructing footings as soon as possible after excavation.

## 4-8.2  Unsuitable Foundation Material

The importance of confirming suitable foundation material cannot be overstated. The Engineer is responsible for determining the foundation suitability as it relates to the design intent. That is, the foundation material has to have the minimum material properties required for the structure to behave as the Designer intended. Simple tests can be performed in the field to determine the bearing capacity and verify the suitability of the foundation material. They are discussed in the *Caltrans Soil and Rock Logging, Classification, and Presentation Manua*l and include:

- Penetration tests – granular soils.
- Finger tests – cohesive soils.
- Pocket penetrometer – cohesive soils.

Note that these simple and expeditious tests only give an approximate evaluation of the soil at or immediately below the surface.

The *LOTB* should be reviewed when the Engineer determines that the undisturbed original material encountered at planned footing elevation is either unsuitable or of a questionable nature. It may be that the anticipated suitable material may well be just below the excavated surface. If the Engineer is certain that the material encountered at the planned footing elevation is unsuitable, then hand-excavating a small exploratory hole to determine the limits of the unsuitable material may be appropriate. The questionable material, related concerns, and possible resolutions should be discussed with Geotechnical Services and the Designer.

## 4.8-3  Modifications Due to Disturbed, Contaminated or Unsuitable Material

Corrective action is required whenever changes in the bottom of footing elevations are made to address disturbed, contaminated, or unsuitable material. The Contractor is responsible for corrective actions to address disturbed or contaminated material. The Engineer is responsible for actions addressing unsuitable material. The corrective actions are similar in either situation. They fall into two categories:

1. Replacement of the original foundation material to achieve the original bottom-of footing-elevation.
2. Revisions to the structure to address a different bottom-of-footing elevation.

There are engineering/design considerations in either of these actions and it is important to discuss considerations and consequences with the Designer, Geotechnical Services, and the Contractor, and work toward a solution that fulfills the design intent and keeps the project moving forward.

Options for restoring the foundation material at the bottom-of-footing elevation to its specified elevation after removal of unsuitable or contaminated material are as follows:

- Excavate to a stratum that has sufficient bearing capacity, replace the removed, unsuitable material with concrete, and then construct the footing at the planned footing elevation.
- Excavate to a stratum that has sufficient bearing capacity, replace the removed unsuitable material with aggregate base or structure backfill to 95% compaction, and then construct the footing at the planned footing elevation.

Revisions to address a different bottom-of-footing elevation or a lower-than-anticipated bearing capacity should be discussed with the Designer. Revisions may require a major structural redesign. The following are possible options, however, they may not be the best alternatives in real construction situations.

- Maintain top-of-footing as planned and overform footing depth. The rebar cage will remain at the theoretical elevation shown on the contract plans; however, the depth between the bottom-of-footing and the bottom mat of the rebar cage will be increased by the amount of over-excavation. This option is similar to previously described methods. It essentially exchanges the use of larger/taller footing forms for a reduction in the number of concrete pours. This option may well be the preferred option for minor revisions to bottom-of-footing elevations.
- Excavate down to a stratum that has sufficient bearing capacity and increase the height of the column or wall. This method may not be acceptable if the increase in height necessitates redesign of the column or wall. This decision should be discussed with the Designer.
- Increase the footing size so that the bearing pressure does not exceed the allowable bearing capacity of the foundation material encountered at the planned footing elevation. Settlement also must be considered, as it cannot exceed tolerable limits. This decision should be discussed with the Designer and Geotechnical Services.

Although footing revisions are contemplated by the contract documents, footing revisions made necessary due to unsuitable material encountered at the planned footing elevation will require a change order. Impacts to the construction schedule also must be considered when making these decisions. The Resident Engineer should be kept aware of these issues. The preferred method for compensating the Contractor for the cost of the corrective work is by adjustment of contract items at contract unit prices, and is included in the contract specifications[10] as the method of payment for the following revisions:

- Raising the bottom of a spread footing above the elevation shown on the contract plans.
- Lowering the bottom of a spread footing 2 feet or less below the elevation shown on the contract plans.

---

[10] 2010 SS, Section 19-3.04, *Payment,* or 2006 SS, Section 19-3.07, *Measurement.*

For other revisions, agreed price or force account methods should be used when the Engineer determines that the above method is unsatisfactory or does not address changes to the character of the work as a result of the revisions.

## 4-9  Safety

Excavations are a potentially dangerous construction activity. CalOSHA has requirements that must be followed before beginning any excavation that is 5 feet or more in depth, into which a person is required to descend. This information is fully described in the *Caltrans Trenching and Shoring Manual*; however, a brief overview is provided below.

Prior to the start of excavation work, the Contractor is required to:
- Obtain a CalOSHA excavation permit.
- Identify a "competent person" responsible for the excavations.
- Provide an excavation plan to the Engineer for review and authorization prior to starting excavation as required by the contract specifications.[11]
- Provide an engineered system stamped by an Engineer who is registered in the State of California for any engineered shoring system.
- Obtain a stamp for any sloping or benching system that is greater than 20 feet from an Engineer who is registered in the State of California.

Once authorized, the excavation needs to be inspected to ensure compliance with the authorized plan and CalOSHA requirements. Daily inspections (prior to the beginning of a shift and after any hazard-increasing occurrence, such as rain) of excavations or protective systems must be made by the Contractor's "competent person" for evidence of any condition that could result in cave-ins, failure of a protective system, hazardous atmospheres, or any other hazardous condition. When any evidence of a situation is found that could result in a hazardous condition, exposed employees must be removed until the necessary precautions have been taken to ensure their safety.

Safety railings must be located around the excavation perimeter, preferably attached to the shoring that extends above the surrounding ground surface. If the shoring does not extend above the ground, then the railing must be located a sufficient distance back from the excavation lip to adequately protect the workmen in the excavation from being injured by falling objects or debris. Locating the safety rail back from the excavation lip usually provides more stable ground to anchor the rail posts. Spoil piles must be located more than two feet away from the excavation lip for excavations deeper than 5 feet unless there is an adequate retaining device in place to prevent materials from entering the excavation.

---

[11] 2010 SS, Section 7-1.02K(6)(b), *Excavation Safety,* or 2006 SS, Section 5-1.02A, *Excavation Safety Plans.*

Although the vertical side of a non-shored excavation must be less than 5 feet in height, care must be exercised when working around the perimeter to avoid falling into the excavation because of sloughing or slip-out of the material at the excavation lip. Spoil piles must be located at least one foot away from the excavation lip for trenches that are less than 5 feet in depth.

Excavations can be considered confined spaces, as they are prone to hazardous atmospheres with limited access and egress. CalOSHA requires the Contractor to take adequate precautions to ensure that oxygen levels and atmospheric contaminants are within acceptable limits. Employees entering excavations should be trained in confined space protocols.

Whenever work is proceeding adjacent to or above the level of vertical projections of exposed rebar, workers must be protected against the hazards of impalement on the exposed ends of the rebar. The impalement hazard can be eliminated by either bending over the ends of the projecting rebar, or by use of one of the following methods:

- When work is proceeding at the same level as the exposed protruding rebar, worker protection can be provided by guarding the exposed ends of rebar with CalOSHA-approved protective covers, troughs, or caps. Approved manufactured covers, troughs, or caps will have the manufacturer's name, model number, and the CalOSHA-approval number embossed or stenciled on the cover, trough, or cap. Any manufactured protective device not so identified is not legal.
- When work is proceeding above any surface of protruding rebar, impalement protection must be provided by the use of: (1) guardrails, (2) an authorized fall protection system, or (3) authorized protective covers or troughs. Caps are prohibited for use as impalement protection for workers working above a level of 7 1/2-feet above the protruding rebar.

Protective covers used for the protection of employees working above grade must have a minimum 4 x 4-inch square surface area or 4 1/2-inches in diameter, if round. Protective covers or troughs may be job-built, provided they are designed to CalOSHA minimum standards, that the design of the cover or trough was prepared by an Engineer currently registered in the State of California, and a copy of the authorized design is on file in the job records prior to their use.