of permafrost design (see Clause 6.18). Where frost protection is marginal or becomes deficient, for example, in the case where a roadway grade adjacent to an existing structure is lowered, consideration should be given to the use of insulation to provide the required frost protection.

Soil or earth borrow with at least 50% of its particles by mass between 5 and 75 µm in size should be considered frost susceptible. For frost susceptible soils, consideration should be given to excavation of frost susceptible material to below frost depth and replacement with non-frost susceptible material.

C6.10.1.6 Scour, erosion, and undermining

Requirements for scour are provided in Clause 1.9.

The prediction of scour for bridge foundations requires a scour analysis, typically undertaken by a hydraulics engineer using a hydraulic model. FHWA has published guidelines HEC-18 (2012) and HEC-23 (2009) that present the state of knowledge and practice for the evaluation, design, and inspection of bridges for scour. FHWA has produced a Technical Brief on Hydraulic Considerations for Shallow Abutment Foundations (FHWA, 2018). It describes how scour may impact shallow abutment foundations at bridges over waterways and provides technical considerations for evaluating and mitigating abutment scour at bridges with shallow foundations.

Scour design is a multi-disciplinary exercise that involves the hydraulics engineer, structural engineer, and geotechnical engineer working as a team. The role of the geotechnical engineer in scour analysis is to determine the appropriate physical, mechanical, and erodability properties of the soil and include them in the geotechnical report (Clause 6.7) for use in the scour analysis. Conceptual scour counter-measures should also be provided in the geotechnical report with the provision that the hydraulics engineer is responsible for conducting the scour analysis and selecting a preferred counter-measure in collaboration with the structural and geotechnical engineers.

Observational methods for scour have demonstrated that the modelling predictions of scour are conservative. More research is required to improve modelling techniques and predictions.

C6.10.2 Ultimate geotechnical bearing resistance

Calculation procedures and appropriate resistance factors are given in Clause 6.9.

The geotechnical ULS resistance of a foundation may be calculated by a variety of techniques, which include evaluating the shear strength of the soil and applying the results of in-situ tests. A more detailed description of the effect of spatial variability in the ground on bearing capacity can be found in Fenton et al. (2008).

The resistance of a foundation is derived from the geotechnical strength parameters, the depth of embedment of the foundation, and the weight of the soil within the failure zone. Direct and simple shear tests and conventional triaxial tests, with or without pore pressure measurements, of relatively undisturbed soil samples can be used to obtain strength parameters. The strength parameters may also be obtained from the results of in-situ tests such as field vane shear, standard penetration test (SPT), static cone penetration (CPT/CPTU), and pressuremeter tests. These in-situ tests offer the advantage of minimizing the detrimental effects associated with disturbance caused by the drilling and sampling operations, as well as by transporting the samples. The field vane test is commonly used in soft to stiff clays. However, as described by Bjerrum (1972), Holtz and Wennerstrand (1972), Aas et al. (1986), and ASTM (1988), corrections to the measured shear strength might be required. Such corrections need to be applied with caution and calibrated against local experience using engineering judgement.

Mathematical models for bearing resistance are available in Terzaghi and Peck (1967), Sokolovski (1965), Harr (1966), Chen (1975), Meyerhof (1951, 1953, and 1957), Brinch Hansen (1970), and Vesic (1973, 1975). Models should include the influence of footing geometry, footing embedment, proximity to slopes, and load inclination.

The bearing coefficients, N_c (cohesion term), N_q (surcharge term), and N_γ (weight term) are dimensionless and depend only on the value of the internal friction angle. N_c and N_q were developed for weightless soils as early as 1921 by Prandl (1921). The factor N_γ includes the effect of the foundation soil weight. Expressions for these coefficients were derived by Terzaghi (1943) and by Lundgren and Mortensen (1953). Several others have also proposed expressions for these factors. The following expressions in Clause 6.10.2 have been taken from the CFEM (2006):

$$N_q = e^{\pi \tan\phi'} \tan^2\left(45° + \frac{\phi'}{2}\right)$$

$$N_c = (N_q - 1) \cot \phi'$$

$$N_\gamma \simeq 0.1054 e^{0.1675\phi'} \approx 2(N_q + 1) \tan\phi'$$

The approximation for N_γ above is for the case of cast-in-place ("'rough"') footing because most spread footings for bridge shallow foundations are cast-in-place. The following precast ("'smooth'") interface equation for N_γ is also taken from CFEM (2006):

$$N_\gamma \approx 0.0663 e^{0.1623\phi'}$$

It is noted that these bearing resistance coefficients involve exponentials that are functions of ϕ', making the calculation of the bearing resistance sensitive to small variations in ϕ', as shown in Figure 6.3. The accurate determination of the friction angle is difficult and engineering experience and judgment must be used in its selection.

The resistance equation, as given in the Clause 6.10.2, is intended for use in an effective stress analysis. The values c', γ', and ϕ' are effective values. When evaluating the ULS resistance of soils, both the short-term (undrained or total) and long-term (drained or effective) conditions should be checked. The ultimate bearing resistance is the lesser of the two values. For clays, the short-term condition generally governs.

For the short-term or undrained condition in clay, ϕ' is zero and the value c' corresponds to undrained shear strength, s_u. For undrained conditions, in terms of total stresses, when $\phi' = 0$, then $N_c = 5.14$, $N_q = 1.00$, and $N_\gamma = 0.00$. For this case, the gross ultimate bearing resistance equation for clay becomes

$$q_u = 5.14\, s_u\, s_c\, i_c + \gamma' D_f\, s_q\, i_q$$

where s_u is the undrained shear strength and D_f is the minimum depth of footing below the ground surface. The term $\gamma' D_f$ corresponds to the total overburden pressure at the base of the footing (i.e., it represents $q' N_q$).

Based on the assumption that the pore pressure above the groundwater table is zero, the total stress is the effective stress. When the distance of the groundwater table to the base of the footing is less than the footing width, B, the value of γ' might have to be proportioned to account for the effect on unit weight. Above the water table, the effective weight is the total unit weight, while below the water table, it is the submerged unit weight. The value used for γ' must also reflect the presence of any hydraulic gradient in the soil. When an artesian pore pressure exists below the footing or the eccentricity is large, the expression for ultimate resistance, q_u, can become unreliable (DGI *Code of Practice for Foundation Engineering*).

Typically, the geotechnical resistance at ULS of a footing is calculated by assuming that the foundation is located on uniform ground with a level surface in which the depth of the supporting stratum is sufficient to develop the failure mechanism assumed in the calculation. Otherwise, the bearing resistance might require modification to account for the influence of sloping ground and non uniform or layered soils according to NAVFAC DM-7.01 (1986), Winterkorn and Fang (1986), Brinch Hansen (1970), Vesic (1973 and 1975), and Meyerhof (1953 and 1957).

Where retrieval of soil samples is expected to introduce significant sampling disturbance, the shear strength and bearing resistance of the soils can be calculated from the results of in situ tests, which include the standard penetration test (SPT), static cone penetration test (CPT), and pressuremeter test (PMT).

Standard penetration test (SPT)

The SPT is generally carried out in accordance with ASTM D1586M. Some limitations on the use of SPT test results for calculating bearing resistances are given in the CFEM (2006), Wroth (1988), Mitchell (1988), and ISUPT (1988). Many correlations have been developed between SPT N-value and various geotechnical parameters. Correlation between SPT N values and vertical settlement is established for a known stratigraphy and forms the basis for design charts relating bearing resistance, N, and width for a vertical settlement of 25 mm as shown in NAVFAC DM-7.01(1986), Peck, Hansen and Thornburn (1974) and Terzaghi and Peck (1967).

Static cone penetration test (CPT/CPTU)

A static cone penetrometer test (see ASTM D23441) is carried out by pushing the point of a cone into the soil, while recording the cone point stress and the side resistance along a short length of the straight portion of the cone immediately behind the cone point. The pore pressure induced by the cone penetration is measured by a piezo cone. The static cone results, and in particular the piezo cone data, are useful in determining the soil profile and variations in its density and strength. Theoretical and empirical functions have been proposed for relating the results of other in situ tests. The use of the static cone results in design remains largely empirical.

The advantages and limitations on the use of results derived from static cone penetration tests for the calculation of various bearing resistances are discussed in the CFEM (2006), Robertson and Campanella (1984), Schmertmann (1970, 1977), ISUPT (1988), and Lunne et al. (1997).

Pressuremeter test (PMT)

A procedure for estimating the bearing resistance of a foundation from results of a pressuremeter test (see ASTM D4719) was proposed by Menard (1975). The limitations on the use of results derived from pressuremeter tests for the calculation of geotechnical resistance at ULS are discussed in the CFEM (2006), Menard (1975), and Baguelin, Jezequel, and Shields (1978).

Further information on subsurface investigations for geotechnical site characterization is found in FHWA (2002) and CFEM (2006), Chapter 4.

Assessed values

For shallow foundations, assessed values for vertical geotechnical resistances at ULS and SLS may be used when suitable geotechnical data and detailed stratigraphy have been obtained from the project site. These assessed values should be based on knowledge of the local site area and the results of observations obtained from sites having similar stratigraphy. Assessed values for vertical geotechnical resistance at ULS for sound level bedrock or non-yielding soil (e.g., very dense or hard glacial till) may be determined from CFEM (2006).

Foundation design and structural design

In many cases, the geotechnical aspects of the foundation design (bearing capacity, settlement, sliding) are being evaluated before all the loading details are available from the structural engineer. As such, load inclination and/or eccentricity cannot be directly accounted for by the geotechnical engineer. Most geotechnical engineers do not use the load inclination factors for the initial design of shallow foundations mainly due to a lack of knowledge of the vertical and horizontal loads at the time of geotechnical investigations and preparation of geotechnical resistance recommendations for the geotechnical report.

In most projects, the geotechnical engineer will start the task of preparing the geotechnical report (Clause 6.7) before the structural engineer has completed the structural design. The applied forces on the footing and their angle of inclination would be unknown at this point in time. Consequently, i_c and δ_f cannot be calculated by the geotechnical engineer yet. In this case, the geotechnical engineer should assume, $\delta_f = 0$, i.e., zero load inclination, and consequently i_c would be equal to 1.0 in the geotechnical bearing resistance equation. The geotechnical engineer should then provide in the geotechnical report the geotechnical bearing resistance corresponding to zero load inclination. The geotechnical engineer should also clearly state that the value given does not account for load inclination and when more information about actual applied forces becomes available from the structural design, this value of the geotechnical bearing resistance should be revised to account for any applied load inclination.

The preferred method for considering the effect of inclined load on geotechnical ULS resistance is through the general bearing resistance equations given in Clause 6.10.2 using known geotechnical parameters.

Effective area

The concept of an effective area loaded by an equivalent uniform pressure is an approximation made to account for eccentric loading and was first proposed by Meyerhof (1953). The concept of equivalent footing and “Meyerhof pressure” is used for geotechnical analysis during sizing of the footing — bearing capacity and settlement analysis.

For spread footings subjected to eccentric loading, a uniform distribution of soil pressure acting over the effective area of the footing should be assumed for the geotechnical analysis of the foundation.

For eccentricity in two directions, the effective area can be determined in such a way that the resultant of the factored load passes through the centroid of the effective area. Clause 6.10.2, based on the centroid of pressure, defines the shape of the contact pressure to be used for design.

C6.10.3 Serviceability geotechnical resistance

The design of spread footings is commonly controlled by movement at the SLS, i.e., tolerable settlement, so it is advantageous to proportion spread footings at SLS and then check for adequate design at ultimate limit state (see, e.g., AASHTO, 2018 — Spread Footings).

The type of load or the load characteristics may have a significant effect on deformation of a spread

footing. The following factors should be considered in the estimation of footing deformation (AASHTO, 2018):
- ratio of sustained load to total load;
- duration of sustained loads; and
- time interval over which settlement or lateral displacement occurs.

Consolidation settlements in cohesive soils are time-dependent and short-term transient loads typically have negligible effect. However, in cases where the transient loads are only slowly changing (such as snow loads), or where the mean live load is relatively constant (such as truck loads on a major route bridge), some portion of the transient loads may be appropriate to include in a consolidation settlement prediction. In non-cohesive soils, where the permeability is usually relatively high, elastic deformation of the supporting soil due to transient load can occur. Also in non-cohesive soils, the majority of the deformation usually occurs during construction, while the loads are being applied, such that the post-construction deformation experienced by the structure is reduced, depending on the type of structure and construction method (AASHTO, 2018).

Elastic (immediate) settlement is the instantaneous deformation of the soil mass that occurs as the soil is loaded. The magnitude of elastic settlement is estimated as a function of the applied stress beneath a footing. For structures where settlement is critical, the elastic settlement is the most important deformation consideration in non-cohesive soil deposits. For footings located on over-consolidated clays, the magnitude of elastic settlement is not necessarily small and should be checked (AASHTO, 2018).

In a saturated cohesive soil, consolidation settlement occurs and is the most important deformation consideration in cohesive soils.

Further information on settlement of shallow foundations may be found in CFEM (2006), Section 11, AASHTO (2018), and French (1999).

Use of Equation 6.2 in Clause 6.4.1.1 for the design of a shallow foundation against an excessive SLS settlement is described in C6.4.1.1.

C6.10.4 Ultimate geotechnical horizontal resistance

Sliding failure of a shallow foundation will occur if the horizontal load component exceeds the more critical of either the shear resistance of the soil or the interface shear resistance between the soil and footing. For a footing on non-cohesive soil, sliding resistance depends on roughness of the interface between the footing and the soil. A rough footing base usually occurs when the footing is cast-in-place. A precast footing usually has a smooth base.

The zone in front of the footing might not contribute to sliding resistance, due either to the possibility of being removed during the design life or lack of compaction of fill during construction, or due to freeze-thaw cycles of movement over the design life. Caution is thus advised in the calculation of passive resistance to sliding. Only in situations where passive resistance in front of the footing is to be included in the horizontal resistance, the designer should also check that sufficient allowance for the movement necessary to develop the passive resistance is provided.

Sliding failure at the interface between the footing and the soil or rock or through various layers within the resisting soil or rock below the footing should be considered. Clause 6.10.4 includes two equations to be used to estimate the horizontal resistance to shear failure within the soil itself and sliding at the interface between the foundation and the soil. The horizontal resistance is the lesser of the two calculations.

The interface friction between the foundation and the soil or rock depends on the method of construction of the structural footing. If the horizontal loads are high enough to cause sliding failure, the failure would typically take place within the soil just below the surface of the footing if the footing is cast-in-place. If the footing is precast concrete or steel, with a surface that has a relatively low friction angle, the failure surface is usually at the interface of between the supporting soil or rock and the footing.

The horizontal shear resistance of the soil or rock can be supplemented by passive resistance at the toe, shear keys, dowels, or anchors preloading the footing on to the bearing material. Passive resistance at the toe should only be considered if there is assurance that it will not be reduced by future excavation. Shear keys are useful when shear resistance is required. The keys might require special construction considerations, and these should be taken into account in design. If special considerations are required during construction to ensure the development of the horizontal resistance, these should be clearly identified on the plans, e.g., protection of shale against deterioration through exposure.

Reference is made in Clause 6.10.4 to the possibility of a detailed analysis. This applies to cases in which the equilibrium equations are not used or where the soils are highly variable and an important structure is being founded. A finite element analysis is an example of such a detailed analysis. The horizontal resistance is the lower of two components. The first is the factored resistance due to cohesion (0.8 $A'c'$). For over-consolidated clays, this value of cohesion should take into account the effects of remoulding. The second is the frictional resistance due to the normal force caused by the dead load of cast-in-place concrete and of backfill. The density of the backfill appears on both sides of the sliding resistance equation as part of the action and of the resistance. A load factor of 1.25 is applied to the horizontal force and a load factor of 1.0 to the vertical force (since the vertical force is favourable with respect to horizontal resistance).

C6.10.5 Structural design of shallow foundations

As noted above, after the detailed structural design becomes complete enough to know the angle of inclination of the applied forces on the footing, the structural engineer should consult with the geotechnical engineer with regard to revising the given value of the geotechnical bearing resistance in the geotechnical report to account for any load inclination. This revision of the geotechnical bearing resistance after consideration of the load inclination would require the structural engineer to revise the structural design of the footing accordingly in terms of dimensions and reinforcement.

C6.10.5.1 Pressure distribution at the ULS

For structural design of an eccentrically loaded foundation, a triangular or trapezoidal contact stress distribution, based on factored loads, is assumed to act under the footing. It is usually assumed that the bearing stress varies linearly across the bottom of the footing to satisfy force equilibrium.

As noted above, the concept of an effective area loaded by an equivalent uniform pressure is an approximation made to account for eccentric loading and was first proposed by Meyerhof (1953). The concept of equivalent footing and “Meyerhof pressure” is used for geotechnical analysis during sizing of the footing (bearing capacity and settlement analysis). However, the structural design of a footing should be performed using the actual trapezoidal or triangular pressure distributions that model the pressure distribution under an eccentrically loaded footing more conservatively. A comparison of these two loading distributions is shown in Figure 6.5.

As shown in Figure 6.5, in the structural design of a footing, the designer needs to consider two cases of contact pressure distribution at the ULS. The first case assumes a contact pressure distribution due to a yielding soil, Clause 6.10.5.1 a), which assumes a uniform pressure distribution over the effective area *A′* in Figure C6.10. The second case, Clause 6.10.5.1 b), assumes a contact pressure distribution due to an elastic non yielding soil. This is a linear pressure distribution, assuming a linear elastic soil response and a nearly rigid footing in Figure C6.11. The pressure distribution for the yielding soil is shown in plan in Figure C6.11 for double eccentricity.

The distribution of soil stress should be consistent with properties of the soil or rock and the structure. For footings on rock or dense glacial till (non-yielding), the contact pressure at ULS is more closely represented by a linear distribution.

Figure C6.10
Shallow foundation, effective contact area — Uniform pressure distribution
(See Clause C6.10.5.1.)

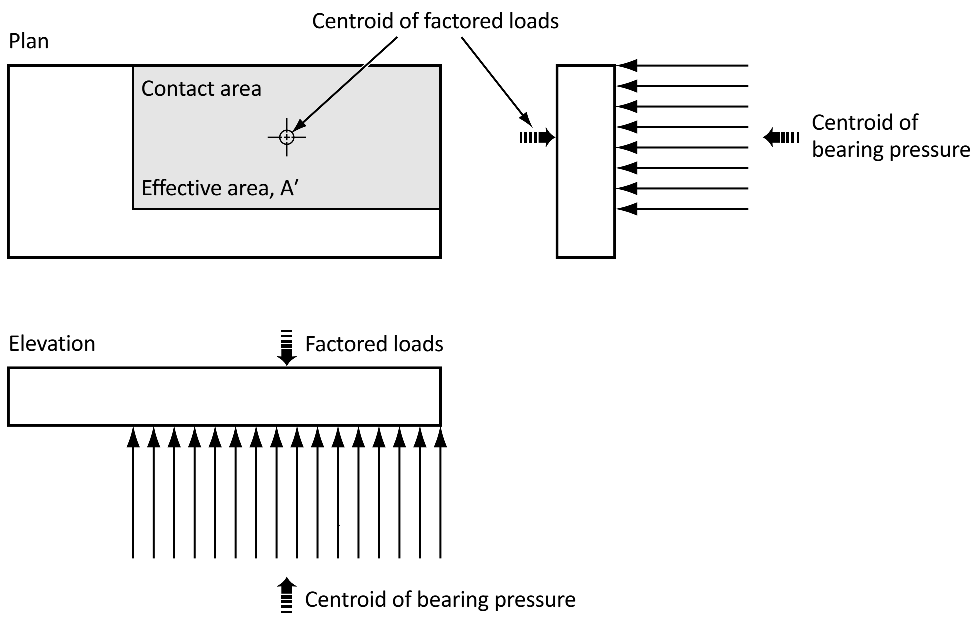
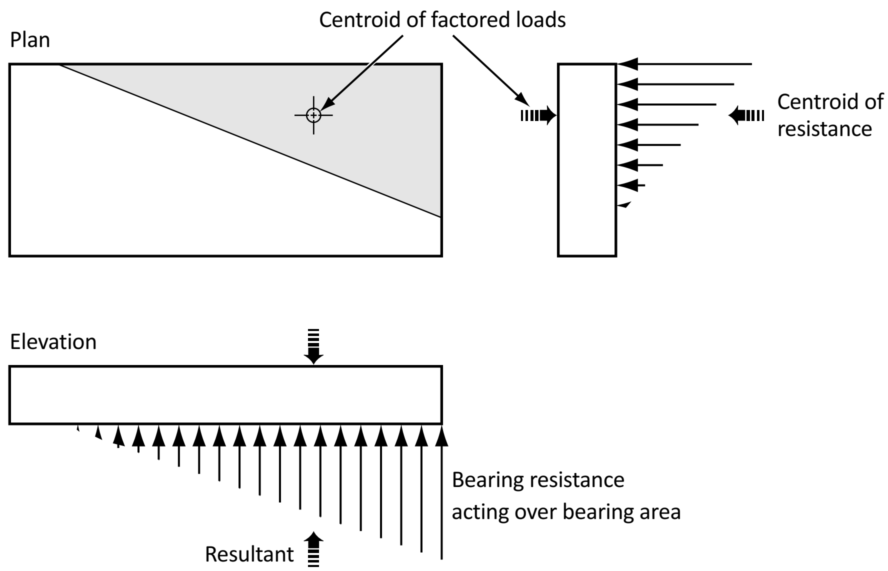


Figure C6.11
Shallow foundation, effective contact area — Linear pressure distribution
(See Clause C6.10.5.1.)



C6.10.5.2 Pressure distribution at the SLS

Under the SLS, materials are assumed to respond elastically to load. Hence, a linear distribution of stress is used in analysis. Figure 6.6 indicates the vertical contact pressure distribution that can be assumed for bearing resistance at the SLS. For those cases where the rotation and horizontal movements are to be calculated, Bowles (1988), Barker et al. (1991), and Poulos and Davis (1974) provide design data.

Normally, shallow footings on sound bedrock or non-yielding soil need not be checked for settlement. The loads required to produce detrimental settlement of the structure for such cases will be larger than the factored bearing resistance at the ULS. However, when bedrock contains, for example, either compressible clay seams at shallow depth or solution cavities, a settlement analysis should be carried out.

C6.10.5.3 Eccentricity limit

Limiting eccentricities are defined to ensure that negative contact pressures do not occur at any point beneath the footing. Footings founded on soil should be designed such that the eccentricity in any direction is less than one-sixth (1/6) of the actual footing dimension in the same direction. If eccentricity exceeds these limits, the footing size should be increased.

The eccentricity limitation given in Clause 6.10.5.3 applies to static loading. The 30% eccentricity for geotechnical proportioning was chosen in order to limit local concentration of bearing stresses so as to avoid the possibility of a bearing failure towards the edge of the footing and to avoid overturning. The limit is not related to the middle third rule employed in elastic analysis, where an eccentricity in the direction of the load of more than 1/6 results in tension at the heel. Corrections should be made to the calculated bearing resistances for eccentricity values greater than 0.30*B* (DGI *Code of Practice for Foundation Engineering*). For a footing with a non rectangular geometry, the 0.30 rule will also apply.

C6.11 Deep foundations

C6.11.1 General

General papers dealing with the reliability-based design of deep foundations at ultimate limit states can be found in Naghibi and Fenton (2011) and Fenton and Naghibi (2011). The reliability-based serviceability design of deep foundations is complicated by the fact that pile settlement calculations usually involve their own conservatism. At the moment, the resistance factors suggested in the code for SLS are merely calibrated against past practice and not based on probability theory.

The terminology for directions of loads and resistances are presented either in relation to the orientation of the pile or in relation to its position relative to the horizon depending on the situation. For individual piles, loads and resistances are generally considered to act either in the axial (parallel to the axis) and the lateral (perpendicular to the axis) direction relative to the pile. Due to the varying orientations of individual piles within piles groups, the loads and resistances are generally considered to act in either the vertical or horizontal direction relative to the horizon.

Deep foundation units comprising driven piles or drilled shafts are used in a variety of conditions, which include the following:

- where a competent soil or rock is located at an impractical depth for shallow foundations;
- where a foundation at a shallow depth may be subject to scour;
- where the use of shallow foundations would result in unacceptable settlements;
- where conditions are adverse for constructing shallow foundations, e.g., challenges related to groundwater control;
- where liquefaction potential exists; and
- where the construction of shallow foundations would require the removal of contaminated soils.

Piles are vertical or inclined foundation structural members having small cross-sections relative to their lengths. The load transfer mechanism including shaft and tip interactions with soil should be considered when determining whether a foundation is shallow or deep. Deep foundations are generally assumed to be foundation units with depth to width ratios three or greater but generally of five or greater depending on the specifics of the installation. The terminology for direction of load and resistance vectors is presented either in relation to the orientation of the pile or relative to the orientation of the horizon — depending on the context. For load and resistance vectors referring to individual piles, loads and resistances are considered to act in either the axial or lateral direction relative to the pile. For load and resistance vectors referring to the foundation system in totality, loads and resistances are considered to act in either the vertical or horizontal directions relative to the horizon.

Piles are generally categorized as driven piles or drilled shafts depending on the method of installation.

The displacement of adjacent soil during installation is reduced with drilled shaft construction. Drilled shafts are generally constructed by placing concrete and reinforcement in an excavated hole. Drilled shafts may be cast with or without a liner or by the continuous bentonite circulation method depending on the subsurface conditions.

Driven piles are typically made of wood, concrete, or steel.

Wood piles are generally limited to approximately 15 m in length due to availability constraints with timber. Concrete piles may be precast or cast-in-place, with common sections of circular, square, hexagonal, and octagonal shapes. Precast concrete piles are normally reinforced or pre-stressed to resist handling and driving stresses. Larger diameter precast concrete piles (600 mm or greater) may have hollow cores. Steel piles are usually H-piles or pipe piles.

Piles are generally designed to perform as a group connected by a pile cap. The distribution of forces transferred through the pile cap to the individual piles is a function of the stiffness of the pile cap, the geometry and stiffness of the individual piles, and the ground conditions.

A quality assurance program is required to verify that the construction procedures and site conditions agree with the design assumptions.

C6.11.1.2 Load transfer

Load sharing between a pile cap and deep foundation units depends on a number of factors, such as the stiffness and embedment of the pile cap, pile spacing, and the soil immediately below the pile cap, and is usually not to be taken into account due to the risk that the contribution might not be permanent and the pile cap itself can't be depended on for permanent support.

C6.11.1.3 Selection of deep foundation units

a) suitability of the type of pile

A technical and economic feasibility analysis should be performed. The type, size, and depth of the foundation unit selected depend on the amount of vertical and horizontal deformation that can be tolerated by the structure.

Piles develop their geotechnical resistance from a combination of shaft and tip resistance. When deep foundation units are driven adjacent to existing structures or services, precautions should be taken to avoid damage to the existing installations from heave, vibration effects, or ground displacements and compaction. This type of damage can be minimized by the use of non-displacement types of piles and selective pre-augering during installation.

Construction difficulties resulting from the presence of groundwater such as, piping and loss of ground should be considered, particularly in the case of drilled shafts.

A driven pile should be adequately proportioned to satisfy both the static and dynamic stresses imposed during installation and driving. The wave equation, combined with dynamic measurements, can be used to calculate driving stresses. There is some diversity of opinion as to what the maximum permissible driving stress should be. Usually driving stress should be limited to 50% to 60% of the specified compressive strength of concrete piles, and to 60% to 70% of the ultimate strength of wood piles in order to avoid splits and other damages during driving. Driving stresses for steel piles should be limited to about 90% of the specified yield strength.

For drilled shafts, the reliability of end bearing can be affected by the degree to which the bottom of the drilled shaft excavation is cleaned. Inspection of the bottom and sides of the excavation provides the most reliability. However, it might not be practical for such inspection as in the case of smaller diameter drilled shafts or flooded excavations. Remote inspection by camera would be an option if the turbidity of any standing water does not preclude this method. Another option is to machine clean the bottom and install the drilled shaft reinforcing cage, then pour the concrete by tremie techniques, either in the dry or in a flooded excavation. In this case, the potential loss of tip bearing would have to be more carefully considered, although it may often be concluded that the drilled shaft concrete has high strength and that any debris left at the base of the drilled shaft excavation would be mixed or compressed by the column of drilled shaft concrete to provide a strength that is at least comparable to the natural ground foundation below the base.

Tensile stresses for concrete piles should be based on the area of reinforcing steel only and should generally be limited to no more than 70% of the yield strength of the reinforcement.