The calculated vertical bearing pressure shall take account of the turbine vertical load, the foundation and soil backfill weight, accounting for buoyancy effects in the event of potentially high ground water levels as described in 8.5.2.1. The effect of horizontal and torsion loads from the turbine shall be included in the calculation of bearing pressure and capacity. This load shall be applied over an area calculated by taking into account the eccentricity of the overturning load.

Ground rupture shall be verified in the load case:

$$q_{\mathrm{Ed}} = \frac{F_{\mathrm{zd}}}{A'} < q_{\mathrm{Rd}} \qquad 15)$$

where

q_{Ed} is the plastic (uniform) ground pressure based on eccentricity calculation;

A' is the effective foundation area around the line of action of the resultant force for F_{zd};

q_{Rd} is the design values of bearing capacity of soil in ultimate limit state, including appropriate partial safety factor for material (resistance).

q_{Rd} shall incorporate an adequate partial safety factor on material (resistance) of not less than the values given in Annex Q.

The method used shall be consistent with reference standard for the region for which the design is being applied. Method 1 shall be used where the partial safety factor is applied to the material properties. Method 2 shall be used where the partial safety factor is applied to effects of the material properties, i.e. to the resulting resistance. The difference in applied partial safety factors accounts for the non-linearity of the material parameters in the bearing capacity calculation, particularly for granular soils.

8.5.2.4 Sliding resistance

Sliding capacity shall be checked to ensure adequate resistance to horizontal loads.

The presence of any potentially weak, thin soil bands below the foundation shall be taken into account in assessing sliding capacity. Buoyancy effects shall be taken into account in the event of potentially high ground water levels as described in 8.5.2.1.

The beneficial effect of passive soil pressures against the foundation sides may be included if they can be well quantified.

Sliding shall be evaluated in the load case:

$$\tau_{\mathrm{Ed}} = \frac{H_{\mathrm{d}}}{A'} < \tau_{\mathrm{Rd}} \qquad (16)$$

where

τ_{Ed} is the design values of shear stress acting at the soil/structure interface;

H_{d} is the horizontal force acting on the soil formation including unfavourable partial safety factor on load;

A' is the effective foundation area around the line of action of the resultant force for F_{zd};

τ_{Rd} is the design values of sliding stress of soil in ultimate limit state, including appropriate partial safety factor on material and/or resistance.

Design values of sliding stress of soil, $\tau_{Rd,}$ shall incorporate an adequate partial safety factors on material and/or resistance of not less than the values given in Annex Q.

The method used shall be consistent with reference standard for the region for which the design is being applied as presented in 8.5.2.3.

8.5.2.5 Overall (slope) stability

If the foundation is situated near to a change in topography, for example on an embankment, next to a retained structure or slope, then the total geotechnical safety (overall stability) shall be evaluated by slope analyses or similar analyses.

Potential limit states resulting in loss of stability shall be assessed using accepted geotechnical principles. The resistance provided by the soil and relevant structural components shall be calculated to ensure they exceed the applied actions.

The stability assessment shall include the effect of any anticipated changes through the design lifetime of the foundation which can reasonably be anticipated, including maintenance, vegetation growth, climatic conditions and groundwater variations.

Overall stability shall be evaluated in the load case:

$$F_d < R_d \tag{17}$$

where

F_d is the design value of destabilising action including unfavourable partial safety factor on load;

R_d is the design value of stabilising action comprising gravity and soil resistance including unfavourable partial safety factor on load.

R_d shall incorporate an adequate partial safety factor on material and/or resistance of not less than the values given in Annex Q.

The method used shall be consistent with reference standard for the region for which the design is being applied as presented in 8.5.2.3.

Since the soil mass within a slope can act as a stabilising or destabilising force, it is not necessary to apply a partial safety factor on load to the soil density or self-weight gravity load. The partial safety factors given in Annex Q provide adequate safety in this case.

Buoyancy effects shall be taken into account in the event of potentially high ground water levels as described in 8.5.2.1.

8.5.3 Serviceability limit state (SLS)

8.5.3.1 Long term behaviour

Verification of the geotechnical behaviour under SLS shall be performed to ensure that the foundation satisfies the serviceability criteria over the design lifetime of the wind turbine. Serviceability criteria include:

1) compliance with the dynamic and (if specified) static rotational and lateral stiffness specified by the turbine manufacturer as the basis for the loads calculations,
2) control of maximum inclination and settlement of the foundation over the design lifetime of the foundation, and

3) prevention of degradation of the soil bearing capacity or stiffness due to repeated or cyclic loading for example accumulated generation of pore water pressures, hysteresis, creep, liquefaction or other degradation mechanism which can ultimately lead to ULS failure.

8.5.3.2 Foundation stiffness

The foundation system shall meet the required stiffness criteria as defined in Clause 5.

Dynamic foundation stiffness shall be verified based on small-strain soil modulus. The foundation stiffness is a function of contact area, and this shall be calculated for the S3 load level, and any reduction from full contact shall be accounted in the stiffness calculation.

Static foundation stiffness, if specified by the turbine manufacturer, shall be verified based on a soil modulus which makes allowance for the reduction of small strain shear stiffness as a function of actual soil strain at S1 load level. This reduction depends on the soil characteristics and degree to which soil strength has been mobilized. The foundation stiffness shall be calculated for the S1 load level including any reduction from full contact area.

Guidance on the selection of appropriate soil modulus and foundation stiffness is presented in Annex L.

8.5.3.3 Inclination and settlement

Foundation displacement due to long term settlement shall be calculated to quantify maximum inclination (rotation due to differential settlement) and absolute settlement over the design lifetime of the foundation.

The foundation shall not exceed maximum inclination criteria on which the turbine loads are calculated due to out-of-vertical of the tower. Maximum allowable foundation inclination should be specified by the turbine manufacturer in addition to any allowance for construction tolerances. In the absence of specific criteria specified by the turbine manufacturer, a value of rotation of the tower base 3 mm/m (0,17°) may be assumed due to differential settlement.

The foundation shall be limited to a maximum absolute settlement criteria (average across the foundation), which is consistent with its serviceability requirements. Settlement limits may be governed by soil strain limits, ductility of the electrical ducts where they exit the foundation, or other criteria determined by the design team. In the absence of specific criteria imposed by the design team, a value of 25 mm may be assumed for allowable total settlement.

The calculation of inclination and absolute settlement shall be performed using S3 load level and static stiffness applied over the design lifetime of the system.

Where the foundation is sited on non-uniform soil conditions, the potential for differential settlement shall be checked. Significant differences in soil or rock type may be mitigated by replacement or the incorporation of an attenuation layer for example layer of compacted structural fill. This is particularly significant for foundations located partially on fresh bedrock and the risk of any hard points shall be addressed in a similar manner.

8.5.3.4 Soil degradation under cyclic loading

Potential soil sensitivity to repetitive or cyclic loads shall be identified in the GIR. The risk of progressive or sudden degradation of the soil capacity or stiffness shall be evaluated as part of the foundation design. This risk may be addressed by fulfilling a zero ground gap criterion or by other mitigation measures outlined in 8.5.3.5.

A zero ground gap criterion can be fulfilled by proportioning the base to remain in full contact with the soil, under the S3 load level with partial safety factors for load of 1,0.

Alternative mitigation measures include limiting bearing pressures to acceptable values as recommended in the GIR or by replacement of sensitive soils.

If it can be demonstrated that the following conditions are satisfied, then it is permissible that the resulting foundation design is subject to gapping between the foundation and underlying soil formation at the S3 load level.

1) The foundation geometry is not controlled by rotational stiffness requirements or, in cases where it is, the soil modulus has been accurately determined based on in-situ measurement of shear modulus for example cone penetration test or shear wave velocity measurements.
2) The foundation stiffness calculation specifically accounts for any loss of contact area.
3) Compliance with foundation inclination and settlement criteria are not sensitive to loss of contact area.
4) The absence of high or variable ground water conditions with the potential to lead to high pore water pressure or erosion of the soil under the foundation during prolonged cyclic loading.
5) Cyclic loading is not expected to lead to a significant reduction of soil modulus such that it governs the foundation geometry.
6) The soil is identified as not susceptible to degradation of strength under repeated cyclic loading at the load levels being applied such that it governs the foundation geometry.

8.6 Piled foundations

8.6.1 General

Piled foundations consist of a pilecap connected to one or several pile shafts which derive their geotechnical resistance through a combination of shaft friction, end bearing and lateral passive resistance.

The structural resistance of the pilecap and piles shall follow the principles presented in Clauses 6 and 7. If required by the design process, the interface between the pilecap and pile shall be clearly documented. Especially where one design element is sensitive to the properties of the other for example load transfer and rotational stiffness, this shall be clearly communicated in the design documentation.

8.6.2 Pile loads

A global stability assessment shall be performed to determine characteristic axial pile loads for the extreme load cases presented in 5.4 based on the geometric arrangement. It may normally be assumed that the pilecap behaves as a rigid structure for the calculation of pile loads. The loads shall take account of permanent actions due to self-weight of the turbine, tower, pilecap foundation and any soil backfill, and a superposed variable push-pull action derived from wind loading. No capacity should normally be derived from bearing pressures on the underside of the pilecap.

Analyses shall be performed for moments applied about all axes of symmetry to ensure that the worst case orientation is considered.

Design horizontal pile loads shall be calculated by distributing the total load into the piles, taking account of any torsional loads about the vertical axis. Depending on whether piles are designed as vertical or inclined, horizontal loads are not always equal in all piles under any particular loading direction.

Design pile loads shall be derived by applying appropriate partial safety factors on load as defined in Clause 5.

The effect of ground water shall be taken into consideration, including the potential variation across the site and maximum/minimum levels through the design lifetime of the project.

8.6.3 Ultimate limit state

8.6.3.1 Pile geotechnical capacity

Ultimate vertical and horizontal pile capacity shall be derived using total or effective stress approaches based on established analysis methods taking account of soil conditions, pile type and installation method. Pile capacity shall be calculated to incorporate appropriate partial safety factor on resistance and as required by local and national standards and in a manner consistent with the geotechnical and load testing applied. Full scale static or dynamic load testing of piles, either in advance of or during the main works may be used to validate the pile design and allow reduced partial safety factors on resistance if allowed in the local standards.

Axial pile capacity shall be based on shaft friction and end bearing. The effect of negative skin friction shall be included as an additional permanent load for soft soils which demonstrate a risk of ongoing long term settlement.

Lateral capacity shall be based on passive soil capacity. The beneficial effect of a moment connection at the interface of the pilecap and pile head may be included subject to adequate structural connection details. Passive soil resistance acting on the pile cap may be considered where ground conditions allow.

The axial and lateral capacities shall take account of the pile installation method and its effect on the pile/soil interface behaviour.

The axial and lateral capacity of the piles may be considered to be independent if the pile is sufficiently long to provide resistance to axial and lateral forces in different sections of the pile. Short piles or piles with high shaft friction near the pilehead may require additional assessment to address any interaction effects.

The effect of pile spacing shall be included in the analysis, and may become important if spacing is less than 5 pile diameters.

8.6.3.2 Pile structural capacity

The structural capacity of the pile should be determined by the specialist pile designer to account for the combination of compression, tension and lateral loads. The principles provided in Clauses 6 and 7 for steel and reinforced concrete shall be applied to the structural design of piles. The pile structural design shall include for ultimate, serviceability and fatigue limit states.

The effects of pile driving during installation shall be included in the structural verification such that maximum stresses include a reduction factor and fatigue analysis including the installation stresses.

Particular attention shall be given to the interface connection details between the piles and the pilecap to ensure full transmission of loads under all limit states. In cases of tension piles, an adequate load path between the tensile face of the pile cap and the pile structure shall be provided.

Pile head bending moments shall be assessed for fatigue damage using elastic theory. Pile head bending moments shall be assessed for ultimate limit state unless the design can accommodate plastic hinges at pile heads and pile head moment is not required to be developed.

In case of any welded parts in the pile design (including tack welds used as a reinforcement cage construction aid), the fatigue assessment of the pile structure shall consider the reduced SN properties of the welded component as described in Clause 7.

8.6.4 Serviceability limit state

8.6.4.1 General

The design shall include specific consideration to ensure that the piled foundation satisfies the serviceability criteria over the design lifetime of the wind turbine. The serviceability criteria include:

1) compliance with the static and (if specified) dynamic rotational and lateral stiffness specified by the turbine manufacturer as the basis for the loads calculations,
2) control of maximum inclination and settlement of the foundation over the design lifetime of the foundation, and
3) prevention of degradation of the soil bearing capacity or stiffness due to repeated or cyclic loading for example accumulated generation of pore water pressures, hysteresis, creep, liquefaction or other degradation mechanism.

8.6.4.2 Foundation stiffness

The foundation system shall meet the required rotational and lateral stiffness as defined in the wind turbine interface document under serviceability loads.

The analysis may consist of equivalent springs applied to the underside of the pilecap at the pile positions, or may be calculated based on standard solutions taking into account the pile deflection as a function of soil stiffness.

Dynamic foundation stiffness shall be verified based on small-strain soil modulus. The foundation stiffness is a function of the relative pile and soil modulus at the S3 load level. The soil stiffness shall take account of potential reduced values if the piles experience load reversals (tension to compression) over this load range.

Static foundation stiffness, if specified by the turbine manufacturer, shall be verified based on a soil modulus which makes allowance for the reduction of small strain shear stiffness as a function of actual soil strain under S1 load level. This reduction depends on the soil characteristics and degree to which soil strength has been mobilized.

8.6.4.3 Pile deflection

Pilehead deflection shall be calculated to quantify maximum inclination (rotation) and absolute settlement of the foundation, using the same criteria as presented in 8.5.3.3.

The pile flexibility and deflection required to mobilize shaft friction, end bearing and passive resistance shall be accounted in the deflection analysis. Potential pile group effects on the development of resistance with displacement shall be taken into account.

8.6.4.4 Soil degradation under cyclic loading

The risk of progressive or sudden degradation of the pile capacity or stiffness shall be evaluated as part of the pile design.

Soil sensitivity to repetitive or cyclic loads shall be identified and mitigation provided based on the recommendations of the GIR. Suitable mitigation may be obtained by limiting the mobilised shaft friction and end bearing stress to a low proportion of the pile capacity, or by limiting or eliminating pile tension at the S3 load level.

8.7 Rock anchored foundations

8.7.1 General

Rock anchored foundations consist of a foundation connected to several post-tensioned rock anchors. This foundation derives its geotechnical resistance through bearing onto the rock surface.

Rock anchors are pre-stressed and maintain equilibrium of the foundation. The pre-stressing force from the rock anchors should be treated as an external force. Partial safety factors for load shall be applied on the pre-stressing force together with losses according to 8.7.6.

The structural resistance of the foundation and anchors shall follow the principles presented in Clauses 6 and 7. It is normal that the design responsibility is split between different designers as rock anchors are a specialist design element. If the design is split in this way, responsibilities for discharging the requirements of this code shall be clearly communicated and recorded. Where one design element is sensitive to the properties of the other, for example load transfer and rotational stiffness, this shall be clearly communicated in the design documentation.

8.7.2 Types of rock anchor foundation

8.7.2 considers two types of rock anchor foundation.

1) Conventional reinforced concrete anchor cap – reinforced concrete cap anchored into the bedrock. The anchor head shall be protected by a protective cap if it is positioned on top of the footing. The connection between the tower and concrete cap (foundation) is designed following the principle in Clause 7 using traditional anchor bolt or insert ring.
2) Steel rock adapter – transition section connecting the tower to the under lying concrete and rock. The tower is connected to the transition section by short bolts and the rock anchors connect the transition section to the rock.

Rock anchors may consist of post-tensioned multi strand tendons or threaded bars. The post tension system should have an approval or product certification according to local standards for post tensional systems.

8.7.3 Geotechnical data

The geotechnical data for the rock shall be investigated in accordance with 8.3.

Geotechnical site investigation (SI) shall include drilling of boreholes to verify the quality of the rock and determine the anchoring zone. Core drilling is recommended to be conducted at some sites to verify the quality of the rock and investigate the potential presence of fissures, crack zones and ground water. The bore holes shall be drilled to at least the same depth as the proposed length of the anchors.

The geotechnical site investigation, inspection data and quality of the rock shall be evaluated and compiled in a GIR as described in 8.3. The report shall evaluate and state the maximum allowable ground pressure in ultimate limit state and the modulus of elasticity of the rock.

An inspection of the rock at the foundation area and surrounding area shall be conducted during the foundation construction works when the rock surface is cleaned from natural soil. This inspection shall validate the conditions reported in the GIR and used in the design before and after any blasting operations and that all compressible material is removed.

8.7.4 Corrosion protection

The rock anchors shall be designed as permanent rock anchors, normally containing a double corrosion protection system. Examples of allowable systems are given in Annex M. Galvanised protection systems shall not be allowed.

Special measures shall be adopted to protect the tendons or bars below the anchor plate (bearing plate) on top of the foundation to prevent water ingress and corrosion of the anchor. This may be achieved by a steel trumpet placed at the bottom of the anchor plate together with rubber seals and corrosion protection compound. Normally, the anchor plate should be galvanized as shown in Annex M.

A protection cap filled with corrosion protection compound shall protect the anchor head on top of the anchor plate. The anchor head should allow access for inspection throughout its design life.

8.7.5 Anchor inspection and maintenance

The designer shall ensure that the rock anchor post tension force can be checked through the design lifetime and also that any post tension losses can be corrected if necessary.

The designer shall specify the inspection and maintenance requirements of the rock anchor system through its design lifetime.

8.7.6 Post tension tolerances and losses

Post tension execution tolerances shall be taken into account as a percentage tolerance and/or partial safety factors on load according to local requirements.

Losses of post tension shall be taken into account in all limit states to account for:

1) immediate losses (slip of wedges and elastic losses), and
2) time dependent losses due to concrete and rock mass creep, concrete shrinkage and tendon relaxation.

Unless a value is determined by calculation, normal losses of 20 % due to relaxation, creep and shrinkage shall be assumed.

The design value of post tension force used in the calculations shall account for positive or negative tolerances and the presence or absence of losses, depending on whether they are favourable or unfavourable for the limit state being considered.

8.7.7 Ultimate limit state

8.7.7.1 Overturning

Overturning shall be assessed as presented in 8.5.2.2, including the effect of the anchor holding down force.

8.7.7.2 Ground rupture – Rock bearing capacity

The vertical bearing capacity shall be verified (see 8.5.3.2).

$$q_{\mathrm{Ed}} = \frac{F_{\mathrm{zd}}}{A'} < q_{\mathrm{Rd}} \tag{18}$$

where

q_{Ed} is the design value of plastic ground pressure as a uniform load;

q_{Rd} is the design value of bearing capacity of the rock mass as a uniform load including the effect of fissures or other discontinuity and including appropriate partial material and/or resistance factors;

F_{zd} is the design value of vertical force;

A' is the effective foundation area around the centre of gravity for F_{zd}.

The bearing capacity of the rock is influenced of the local geometry of the rock and the magnitude of F_{zd}.

The local maximum elastic ground pressure shall also be verified.

$$q_{\text{Ed, elastic}} < q_{\text{Rd, local}} \tag{19}$$

where

$q_{\text{Ed, elastic}}$ is the design value of elastic ground pressure;

$q_{\text{Rd, local}}$ is the design value of maximum bearing capacity on a local area of the rock (unconfined compressive strength value).

8.7.7.3 Sliding

Sliding shall not occur between the rock, levelling concrete and the foundation, taking into account the beneficial effect of the rock anchor tension force.

8.7.8 Serviceability limit state

8.7.8.1 Foundation stiffness

See 8.5.3, although in practice rock anchor foundations are significantly stiffer than other types and this criterion is not normally a design driver.

8.7.8.2 Inclination and settlement

Settlements are not normally a design driver on solid rock and do not require a specific check.

8.7.8.3 No gapping

If the anchor cap is modelled as a rigid body, zero ground gapping shall occur at the S3 load level including the effect of the anchor holding down force. If the ground pressure is modelled by vertical springs to represent the contact with the rock, then the following items shall be satisfied:

1) contact pressure around the whole perimeter of the foundation to prevent water ingress for corrosion protection;
2) contact pressure at the rock anchor and 0,1 m from the outer edge of the anchor ensure load sharing between anchor and cap to improve fatigue resistance.

This verification shall ensure the corrosion protection of the anchors.

For rock adapter type foundations, the transition section shall be in contact with concrete over the whole area under S1 load level, with no loss of pressure over the perimeter or no gapping (decompression). This requirement is important to secure the rotational stiffness of the tower and fatigue resistance of the anchors.

8.7.9 Robustness check

A robustness check of the foundation shall be performed to allow for anchor failure. In this load case, a minimum of one rock anchor or 10 % of all rock anchors (whichever is greater) shall be assumed to have lost their preload. The structure shall remain stable with the remaining rock anchors, to be verified for the S1 load level.

The design of the foundation shall have a redundancy plan in case an anchor fails during construction.

8.7.10 Rock anchor design

8.7.10.1 General

Rock anchors shall be designed according to local standards and building codes. At the S1 load level, no rock anchor yielding is permissible.

Testing, supervision and monitoring of rock anchor installation shall be conducted.

During drilling of holes, the rock quality shall be logged in a drilling report, and any findings which invalidate the design assumptions shall be addressed.

Drill holes should be hydraulic tested with falling head water test to ensure that the hole is "closed".

During prestressing of rock anchors, the foundation shall be monitored to verify that there are no settlements.

The tendon or bar shall have an upper free length from the stressing point and down to the bonded zone. Over the free length, the tendon or bar is free to strain separately to the surrounding grout and rock. The free length at the top of the anchor is very important to secure the proper action of the anchor.

The free length of the rock anchor ensures:

- robustness (no brittle (fragile) failure mode),
- low stress variations in the fatigue load case, and
- minimize losses due to slip of wedges.

The required fixed anchor length (L_{fixed}), also known as bond length (L_{bond}), over which the load is transmitted to the surrounding rock, shall be verified.

$$L_{\text{fixed}} = L_{\text{bond}} = \frac{\gamma_{\text{F}} \times P_{\text{lock-off}}}{f_{\text{bd}} \times \pi \times \theta} \tag{20}$$

where

γ_{F} is the partial safety factor for load;

$P_{\text{lock-off}}$ is the lock-off load for rock anchor;

f_{bd} is the design value of bond strength between rock and cement grout;

θ is the diameter of bore hole.

L_{anchor} $= L_{\text{free}} + L_{\text{fixed}}$

For good quality rock (rock mass rating, RMR > 60), the bond strength could be calculated with this formula:

$$f_{\text{bd}} = \frac{0{,}1 \times f_{\text{cck}}}{\gamma_{\text{M3}}} \tag{21}$$

where

f_{cck} is the characteristic grout compression strength;

γ_{M3} = 3.

8.7.10.2 Fatigue of rock anchors

A fatigue assessment shall be performed for all rock anchors.

No additional detailed fatigue analysis is required if the stress range for the highest fatigue load in pre-stressing steel of multi strands is lower than $\Delta\sigma_{d,\ fatigue,\ strands} < 70$ MPa in the fatigue limit state. The stress range can be evaluated from the gapping between foundation and rock ($\delta_{Lift\ off}$) at the position of the rock anchor.

$$\varepsilon = \text{elongation} = \delta_{Lift\ off}/L_{Free}$$

where

ε is the elongation

$$\Delta\sigma = \varepsilon \times E_S$$

where

E_S is the e-modulus of rock anchor.

Fatigue of threaded bars shall be verified in accordance with the requirements of 6.7.4.

8.7.10.3 Geotechnical bearing capacity of rock anchors

The geotechnical bearing capacity of the rock anchors shall be verified and the global effect of spacing of the anchors shall be included.

$$P_{lock\text{-}off} < R_d = \frac{R_k}{\gamma_R} \tag{22}$$

where

R_d is the design resistance regarding to geotechnical bearing capacity;

γ_R = 1,35;

R_k is the weight of the mobilised rock volume, normally an inverted cone with a bottom angle of 60° to the horizontal, starting at the bottom of the rock anchor.

8.7.10.4 Length of rock anchors

The total length of the rock anchor is governed by either geotechnical bearing capacity and thickness of foundation, illustrated as H_2 in Figure 7, or free and fixed anchor length according to 8.7.10.3, illustrated as L_{FREE} and L_{FIXED} in Figure 7.