**TC250/SC7/EG9: Water Pressures**

**(draft) Final Report** **2014**

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## Proposal of changes and amendments for the next version of EC 7-1

used colors:

existing Code, phrases concerning water pressure which have been under discussion

agreed proposals for new paragraphs and new wording in the next version of EC 7

~~agreed proposals for text of existing code to be discarded in the next version of EC 7~~

# Changes in relation to the existing code

Section 2 Basis of geotechnical design

**2.4 Geotechnical design by calculation**

2.4.2 Actions

(9)P Actions in which ground- and free-water forces predominate shall be identified for special consideration with regard to deformations, fissuring, variable permeability, drainage properties and erosion.

NOTE Unfavourable (or destabilising) and favourable (or stabilising) ~~permanent~~ actions may in some situations be considered as coming from a single source. If they are considered so, a single partial factor may be applied to the sum of these actions or to the sum of their effects.

2.4.4 Geometrical data

(1)P The level and slope of the ground surface, water levels, levels of interfaces between strata, excavation levels and the dimensions of the geotechnical structure shall be treated as geometrical data.

2.4.5 Characteristic values

2.4.5.3 Characteristic values of geometrical data

(1)P Characteristic values of the levels of ground and ground-water or free water shall be measured, nominal or estimated upper or lower levels.

(1a)P The characteristic piezometric water levels and accordingly the characteristic values of water pressures shall correspond to a return period at least equal to the duration of the design situation for which they are assessed. This has to be done by considering hydrological, hydrogeological and environmental information together with statistical analysis, if suitable data is available.

NOTE 1: The duration of persistent design situations will typically be the design life of the structure (e.g. 100 years); the duration of a transient design situation will typically be shorter, e.g. 24 months.

NOTE 2: A return period equal to the duration of the design is in rough estimation related to a probability of 63 % (valid with large T) for at least one occurrence within the duration of the design situation. This estimation is based on the probability being 1 - (1 - 1/N)\*\*T  that in a given number T of time periods (a period may, for example, be a month or a year) there is an occurrence of an event which randomly occurs once in N periods.

**2.4.6 Design values**

**2.4.6.1 Design values of actions**

~~(6)P When dealing with ground-water pressures for limit states with severe consequences (generally ultimate limit states), design values shall represent the most unfavourable values that could occur during the design lifetime of the structure. For limit states with less severe consequences (generally serviceability limit states), design values shall be the most unfavourable values which could occur in normal circumstances.~~

(6)P The ultimate limit state design value of piezometric water levels and accordingly the design values of water pressures shall represent a specified rare probability ~~in~~ of exceedance within the duration of the design situation (see EN 1990 1.5.2.2 - 1.5.2.5)

It may be derived either

* by direct assessment or
* by adding a margin to the characteristic piezometric water level.

NOTE 1: The value of the specified probability may be set by the national annex.

NOTE 2: The recommended value of the probability of exceedance during the duration of the design situation of the structure is 1 %.

NOTE 3: design situation could refer to transient, persistent or other situations

NOTE 4: if the water table is restricted by overflowing the appropriate level defines the design value of water pressure

(6a) In cases where water pressures are not hydrostatic, the calculation of ultimate limit state design water pressures shall use the worst credible combination of heterogeneity and anisotropy of permeability, including the effects of layering, fissuring and other heterogeneity, and shall take account of any features of geometry that could cause pressures to concentrate, such as in corners of excavations.

(6b) Ultimate limit state design values for STR- and GEO-limit states may also be based on application of partial factors to structural effects of characteristic water pressures. In this case it should be checked if the ultimate limit state design water pressure could lead to more unfavourable structural effects.

NOTE 1: Values of partial factors may be set by the national annex.

NOTE 2: The recommended values of partial factors to be applied on effects of water pressure are given in A.3 and table A.3.

(6c)P Partial factors to structural effects of characteristic water pressures shall correspond to those for permanent actions although the water pressure might be variable.

(7) In some cases extreme piezometric water levels and the related water pressures complying with 1.5.3.5 of EN 1990:2002, may be treated as accidental actions.

(7a) Accidental situations may be defined using special partial factors for situations where engineered systems fail such as deficiency of a sealing or a barrage. Extreme natural events with a probability to occur less than that of the ultimate limit design value may be classified as accidental.

NOTE 1: The values of special partial factors for accidental situations may be set by the national annex.

~~(8) Design values of ground-water pressures may be derived either by applying partial factors to characteristic water pressures or by applying a safety margin to the characteristic water level in accordance with 2.4.4(1)P and 2.4.5.3(1)P.~~

(9) The following features, which may affect the water pressures should be considered:

* the level of the free water surface or the ground-water table;
* the favourable or unfavourable effects of drainage, both natural and artificial, taking account of its future maintenance;
* the supply of water by rain, flood, burst water mains or other means;
* changes of water pressures due to the growth or removal of vegetation.

(10) Consideration should be given to unfavourable water levels that may be caused by changes in the water catchment and reduced drainage due to blockage, freezing or other causes.

(11) Unless the adequacy of the drainage system can be demonstrated and its maintenance ensured, the design ground-water table should be taken as the maximum possible level with the drainage not functioning, which may be the ground surface. This may be treated as an accidental situation.

2.4.7 Ultimate Limit States

2.4.7.3.2 Design effects of actions

(2) In some design situations, the application of partial factors to actions coming from or through the soil (such as earth or water pressures) could lead to design values~~, which~~ that are unreasonable or even physically impossible. In such ~~these~~ situations, the factors may be applied directly to the effects of actions derived from representative values of the actions, see also 2.4.6.1 (6b).

#### 2.4.7.4 Verification procedure and partial factors for uplift

(1)P Verification for uplift (UPL) shall be carried out by checking that the sum of the design values of the combination of destabilising permanent and variable vertical actions (*V*dst;d) is less than or equal to the sum of the design values of the stabilising permanent vertical actions (*G*stb;d) and of the design value of any additional resistance to uplift (*R*d):

*V*dst,d≤ *G*stb;d + *R*d (2.8)

where

*V*dst,d *= G*dst;d +*Q*dst;d

*G*stb;d: Design value of stabilising actions

*R*d : Design value of resistance due, for example, to shearing and anchoring

*G*dst;d : Resultant of buoyancy force due to ultimate limit state design value of permanent and variable water pressure and design values of permanent destabilising actions not caused by water pressure

*Q*dst,d: Design value of variable unfavourable destabilising actions other~~s~~ than water pressure

~~(2) Additional resistance to uplift may also be treated as a stabilising permanent vertical action (G~~~~stb;d~~~~)~~.

(3)P The partial factors defined in A.4(1)P and A.4(2)P shall be used to derive design values by multiplication with characteristic values for actions not caused by water pressure, *G*dst;d *Q*dst;d, *G*stb;d, and for resistance *R*d to be used in inequality (2.8).

NOTE 1: The values of the partial factors may be set by the National annex. Tables A.15 and A.16 give the recommended values.

NOTE 2: The weight of water, including that in the pores of soil may contribute to stabilizing actions, e.g. in layers 6 and 8 in fig 10.1. In this case it is subjected to a partial factor.

2.4.7.5 Verification of resistance to failure by heave due to seepage of water in the ground (HYD)

~~(1)P When considering a limit state of failure due to heave by seepage of water in the ground (HYD, see 10.3), it shall be verified, for every relevant soil column, that the design value of the destabilising total pore water pressure (~~*~~u~~*~~dst;d ) at the bottom of the column, or the design value of the seepage force (~~*~~S~~*~~dst;d) in the column is less than or equal to the stabilising total vertical stress (~~*~~~~*~~stb;d) at the bottom of the column, or the submerged weight (~~*~~G~~*~~´stb;d) of the same column:~~

*~~u~~*~~dst;d ~~*~~~~*~~stb;d (2.9a)~~

*~~S~~*~~dst;d ~~*~~G~~*~~´stb;d (2.9b)~~

~~(2)P The partial factors for~~ *~~u~~*~~dst;d,~~ *~~~~*~~stb;d,~~ *~~S~~*~~dst;d and~~ *~~G~~*~~´stb;d for persistent and transient situations defined in A.5(1)P shall be used in equations 2.9a and 2.9b.~~

~~NOTE The values of the partial factors may be set by the National annex. Table A.17 gives the recommended values.~~

(1)P When considering a limit state of failure due to heave by upwards seepage of water in the ground (HYD, see 10.3), it shall be verified that vertical equilibrium is maintained in the ground by the combination of design values of soil weight, water pressures (or derived seepage forces) and any available shear resistance in the ground.

**Section 3**  **Geotechnical data**

**3.2 Geotechnical investigations**

3.2.3 Design investigations

(7)P The existing ground-water levels shall be established during the investigation. Any free water levels observed during the investigation shall be recorded (see EN 1997-2).

(8)P The ~~extreme~~ possible range of water levels of any water source~~, which~~ that might influence the ground-water pressures shall ~~should~~ be established.

(9)P The location and capacities of any dewatering or water abstraction wells in the vicinity of the site shall be established.

**3.3 Evaluation of geotechnical parameters**

3.3.10 Geotechnical parameters from field tests

3.3.10.2 Standard penetration and dynamic probing test

(1)P In assessing blow counts, the following features shall be considered:

1. type of test;
2. detailed description of ~~the~~ test procedure;
3. ground-water conditions;
4. ~~the~~ influence of the overburden pressure;
5. ~~the~~ nature of the ground, particularly if cobbles or coarse gravel are encountered.

3.3.10.7 Compactibility tests

(1)P In assessing the compactibility of a fill material, the following features shall be taken into account:

1. type of soil or rock;
2. grain size distribution;
3. grain shape;
4. ~~the~~ heterogeneity of the material;
5. ~~the~~ degree of saturation ~~or water content~~;
6. ~~the~~ water content;
7. type of plant to be used.

**3.4 Ground Investigation Report**

### 3.4.3 Evaluation of geotechnical information

(2) In addition, the evaluation of the geotechnical data should include the following, if relevant:

* tabulation and graphical presentation of the results of the field and laboratory work in relation to the requirements of the project and, if deemed necessary,
* histograms illustrating the range of values of the most relevant data and their distribution;
* level ~~depth~~ of the ground-water table and its seasonal fluctuations;

**Section 4** **Supervision of construction, monitoring and maintenance**

**4.3** **Checking ground conditions**

4.3.2 Ground-water

(1)P As appropriate, the ground-water levels, pore-water pressures and ground-water chemistry encountered during execution shall be compared with those assumed in the design.

#### Section 9 Retaining structures

9.4 Design and construction considerations

9.4.2 Drainage systems

(1)P If the safety and serviceability of the designed structure depend on the successful performance of a drainage system, the consequences of its failure shall be considered, having regard for both safety and cost of repair. One of the following conditions (or a combination of them) shall apply:

1. a maintenance programme for the drainage system shall be specified and the design shall allow access for this purpose;
2. it shall be demonstrated both by comparable experience and by assessment of any water discharge, that the drainage system will operate adequately without maintenance.

(2) The quantities, pressures and ~~eventual~~ chemical content of any water discharge should be taken into account.

9.6 Water pressures

(1)P Determination of characteristic and design water pressures shall take account of water levels both above and in the ground.

(2)P When checking the ultimate and serviceability limit states, water pressures shall be accounted for in the combinations of actions in accordance with 2.4.5.3 and 2.4.6.1, considering the possible risks indicated in 9.4.1(5).

(3)P For structures retaining earth of medium or low permeability (silts and clays), particular attention should be given to 2.4.6.1 (11).~~water pressures shall be assumed to act behind the wall. Unless a reliable drainage system is installed (9.4.2(1)P), or infiltration is prevented, the values of water pressures shall correspond to a water table at the surface of the retained material.~~

(3a) For retaining structures with shallow foundations subjected to lateral water pressure the eccentricity of the characteristic resultant effective forces in the base of the construction is restricted to a third of the width of the base. This restriction often corresponds to a model in which compressive effective stress is maintained between the structure and the ground over at least half of the base area of the construction. In the remaining gap between the structure and ground the possibility of the full water pressure shall be considered. An example is given in Figure 9.X.

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| **Figure 9.X — Example of water pressure related to**  **eccentric loading to the base of a structure** |

(4)P Where sudden changes in a free water level may occur, both the non-steady condition occurring immediately after the change and the steady condition shall be examined.

(5)P Where no special drainage or flow prevention measures are taken, the possible effects of water-filled tension or shrinkage cracks shall be considered.

(5a) A reduction or limitation of water pressures by means of building measures, i.e. seals, drainage or wells may only be considered, if

1. their function can be maintained during the whole lifetime of the construction or
2. their function will be regularly monitored and can be repaired without serious restrictions to the usage of the construction or if
3. additional measures of flooding or ballasting will be available when needed.

Already during the design it has to be described in detail how the function can be repaired or how the flooding or ballasting can be proceeded automatically.

(5b) For the case that regular monitoring of the function is chosen a verification is necessary that ultimate limit states UPL, HYD, STR and GEO with partial factors for accidental situations are not reached if the technical system of water pressure control fails.

#### Section 10 Hydraulic failure

10.1 General

(1)P The provisions of this Section apply to three ~~four~~ modes of ground failure induced by pore-water pressure or pore-water seepage, which shall be checked, as relevant:

1. failure by uplift (buoyancy);
2. failure by heave (upwards seepage);
3. failure by internal erosion and piping.~~;~~
4. ~~failure by piping.~~

NOTE 1 Uplift ~~Buoyancy~~ occurs when pore-water pressure under a structure or a low permeability ground layer becomes larger than the mean overburden pressure (derived from ~~due to~~ the structure and/or the overlying ground layer).

NOTE 2 Failure by heave occurs when upwards seepage forces act against the weight of the soil, reducing the vertical effective stress to zero. Soil particles are then lifted away by the vertical water flow and failure occurs (boiling).

NOTE 3 Failure by internal erosion is produced by the transport of soil particles within a soil stratum, at the interface of soil strata, or at the interface between the soil and a structure. This may finally result in regressive erosion, leading to collapse of the soil structure.

NOTE 4 Failure by piping is a particular form of failure by internal erosion, for example of a reservoir, ~~by internal erosion~~, where erosion begins at the surface, then regresses until a pipe-shaped discharge tunnel is formed in the soil mass or between the soil and a foundation or at the interface between cohesive and non-cohesive soil strata. At the final failure state ~~Failure occurs as soon as~~ the upstream end of the eroded tunnel reaches the bottom of the reservoir.

NOTE 5 The conditions for hydraulic failure of the ground by uplift and heave can be expressed in terms of total stress and pore-water pressure or in terms of effective stresses and hydraulic gradient. Total stress analysis is applied to failure by uplift. For failure by hydraulic heave, both total and effective stresses may be used ~~are~~ ~~applied Conditions are put on hydraulic gradients in order to control internal erosion and piping.~~ In order to avoid failure by ~~control~~ internal erosion and piping, hydraulic gradients and flow velocity should be restricted ~~in relation to allowable hydraulic gradients~~.

(2) ~~In situations where the pore-water pressure is hydrostatic (negligible hydraulic gradient) it is not required to check other than failure by uplift.~~ In situations of negligible seepage it may be not necessary to consider HYD. Time considerations of developing seepage may get important.

NOTE Figure 10.A shows an example of a situation in which HYD verification may not be necessary

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| **Figure 10.A — Example of a situation where**  **uplift and hydraulic heave might both be critical** |

(3)P The determination of hydraulic gradients, pore-water pressures or seepage forces shall take account of:

1. ~~the variation of soil permeability in time and space;~~
2. inhomogeneity and anisotropy of the soil permeability
3. variations in water levels and pore-water pressure in time;
4. any modification of the boundary conditions (e.g. downstream excavation).

(4) It should be considered that the relevant soil stratification may be different for different failure mechanisms.

(5)P When hydraulic heave, piping or internal erosion are significant dangers to the integrity of a geotechnical structure, measures shall be taken to decrease the hydraulic gradient or seepage velocity.

(6)P ~~The measures most commonly adopted to reduce erosion or to avoid hydraulic failure are:~~ If necessary measures should be adopted to reduce erosion or to avoid hydraulic failure, such as:

1. modifications of the project in order to resist the pressures or gradients;
2. seepage control;
3. protective filters;
4. avoidance of dispersive clays without adequate filters;
5. slope revetments;
6. inverted filters;
7. relief wells;
8. reduction of hydraulic gradient and seepage velocity.

10.2 Failure by uplift (UPL)

(1)P The stability of a structure or of a low permeability ground layer against uplift shall be checked by comparing ~~the permanent stabilising actions (for example, weight and side friction) to the permanent and variable destabilising actions from water and, possibly, other sources.~~ the permanent and variable destabilising actions from water and, possibly, other sources to the permanent stabilising actions (for example, weight) and resistances (for example side friction and anchoring). Examples of situations where uplift stability shall be checked are given in Figure 7.1 and Figure 10.1.

(2)P The design shall be checked against failure by uplift using inequality (2.8) of 2.4.7.4. In this inequality, the design value of the vertical component of the stabilising permanent actions (Gstb;d) is, for example, the weight of the structure and of ground layers, while the design resistance (Rd) is the sum of, for example, any friction forces~~, (T~~~~d~~~~),~~ and any anchor forces~~, (P)~~. ~~Resistance to uplift by friction or anchor forces may also be treated as a stabilising permanent vertical action (~~*~~G~~*~~stb;d)~~. The design value of the vertical component of the destabilising permanent and variable actions, (*V*dst;d ), is the sum of the design values of the water pressures applied under the structure (permanent and variable parts) and any other upward forces.

(3) In simple cases, the check of ~~equation~~ inequality (2.8) in terms of forces may be replaced by a check in terms of total stresses and pore-water pressures.

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| a) ~~Uplift of a~~ buried hollow structure  1 groundwater table  2 water tight surface | b) ~~Uplift of a~~ lightweight embankment during flood  1 groundwater table  2 water tight surface  3 light weight embankment material |

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| --- | --- |
| change 7 to 6  Gstb;d | The hatching of layer 5 (3 times) should be the same as in  fig c and e (sand) |
| c) ~~Uplift of the~~ bottom of an excavation  1 groundwater table  4 former ground surface  5 sand 6 clay 7 gravel  12 piezometric level at the base of the clay layer | d) ~~Execution of a~~ slab below water level  1 groundwater table  8 injected sand  10 ground water level before the excavation  11 ground water level in the excavation |

|  |
| --- |
| change Td to Rd (friction)  change P to Rd (anchoring) |
| e) Anchored structure ~~anchored to resist uplift~~  9 anchorage  **Figure 10.1 — Examples of situations where uplift might be critical** |

(4) ~~The measures most commonly adopted to resist failure by uplift are:~~ If necessary measures should be adopted to resist failure by uplift, such as:

1. increasing the weight of the structure;
2. decreasing the water pressure below the structure by drainage;
3. anchoring the structure in the underlying strata.

(5)P Where piles or anchorages are used to provide resistance against failure by uplift, the design shall be checked according to 7.6.3 or 8.5, respectively. For this purpose, design values of actions and action effects shall be derived in accordance with paragraph (2) above and, in a separate calculation, also using the partial factors assigned to the limit states STR and GEO, applied to the resultant forces in the piles or anchors.

*(internal Note: This will require careful review in the light of future changes to 7.6.3 and 8.5.)*

10.3 Failure by heave (HYD)

~~(1)P The stability of soil against heave shall be checked by verifying either equation (2.9a) or equation (2.9b) for every relevant soil column. Equation (2.9a) expresses the condition for stability in terms of pore-water pressures and total stresses. Equation (2.9b) expresses the same condition in terms of seepage forces and submerged weights. An example of a situations where heave shall be checked is given in Figure 10.2.~~

(1)P The stability of soil against heave shall be checked in terms of seepage forces and buoyant weights or in terms of stresses and pore-water pressures.

NOTE: Examples of situations where heave shall be checked is given in Figure 10.2.

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| Figure 10.2 — Examples of situations where heave might be critical: groundwater flow to an enclosed excavation a: homogeneous aquifer b: stratified (layered) aquifer | |

(1a) If the stability of soil against heave is checked in terms of seepage forces and buoyant weights of soil blocks, the design value of the buoyant weight of a soil block G'd shall be calculated using the design value of the buoyant weight density of the ground 'd

'd = 'k∙γG;stb (10.Xa)

(1b) The value of the partial factor G;stb is given in A.5

NOTE: The value of the partial factor G;stb may be set by the National annex. Table A.17 gives the recommended value.

(1c)P The stability of soil against heave may be checked by verifying for every relevant soil body:

Sdst;d  G´stb;d + Rd (10.Xb)

with

Sdst;d : seepage force due to ultimate limit state design value of ground water pressure

Rd design value of shear resistance at the sides of an investigated soil column; only components of shear resistance that are independent of Sdst may be used.

NOTE: In many cases involving homogeneous soils, this means that only effective cohesive strength of the ground, c', may be used in calculating Rd. More complicated considerations could apply where soils of high and low permeability are interlayered.

(1d) As a conservative simplification Rd may be omitted from calculations.

(1e) A sample method for HYD verifications based on Terzaghi's block is given in Annex HY.1 and may be used in cohesionless soils in conditions of steady state seepage .

(2)P The determination of the design pore-water pressure distribution for ultimate limit state design shall take into account all possible unfavourable conditions, see 2.4.6.1 (6a), such as:

* thin layers of soil of low permeability and anisotropic or inhomogeneous distribution of permeability;
* spatial effects ~~due to~~ such as enclosed excavations below water level, including the variations in seepage gradient at different plan locations around the perimeter of excavation supports.

~~NOTE 1 Where the soil has a significant cohesive shear resistance, the mode of failure changes from failure by heave to failure by uplift. The stability is then checked by using the provisions of 10.2 where additional resisting forces may be added to the weight.~~

NOTE: ~~2~~ Stability against heave will not necessarily prevent internal erosion, which should be checked independently, when relevant.

(3) ~~The measures most commonly adopted to resist failure by heave are:~~ If necessary measures should be adopted to resist failure by heave, such as:

1. decreasing the water pressure below the soil mass subjected to heave;
2. increasing the resisting weight by adding a further layer of permeable soil.

###### 10.4 Internal erosion

(1)P Filter criteria shall be used to limit the danger of material transport by internal erosion.

(2)P Where an ultimate limit state due to internal erosion can occur, measures such as filter protection shall be applied at the free surface of the ground.

(3) Filter protection should generally be provided by use of ~~non-cohesive~~ soil that fulfills adequate design criteria for filter materials. In some cases, more than one filter layer may be necessary to ensure that the particle size distribution changes in a stepwise fashion to obtain sufficient protection both for the soil and the filter layers.

(4) Alternatively, artificial filter sheets such as geotextiles may be used provided it can be established that they sufficiently prevent transport of fines.

(5)P If the filter criteria are not satisfied, it shall be verified that the design value of the ~~hydraulic gradient~~ seepage velocity is well below the critical ~~hydraulic gradient~~ seepage velocity at which soil particles begin to move.

(6)P The critical ~~hydraulic gradient~~ seepage velocity for internal erosion shall be established taking into consideration at least the following aspects:

1. direction of flow;
2. grain size distribution and shape of grains;
3. stratification of the soil.

###### 10.5 Failure by piping

(1)P Where prevailing hydraulic and soil conditions can lead to the occurrence of piping (see figure 10.3), and where piping endangers the stability or serviceability of the hydraulic structure, prescriptive measures shall be taken to prevent the onset of the piping process, either by the application of filters or by taking structural measures to control or to block the ground-water flow.

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|  |
| 1 free water level 2 piezometric level directly under the low permeable soil 3 low permeable soil 4 permeable subsoil 5 pipe caused by soil material transport  6 seal which could have leakages  Figure 10.3 — Example of piping |

NOTE: Suitable structural measures are:

* application of berms on the land side of a retaining embankment, thus displacing the possible starting point of piping farther away from the structure and decreasing the seepage velocity ~~hydraulic gradient~~ at this point;
* application of impermeable screens below the base of the hydraulic structure by which the ground-water flow is either blocked or the seepage path is increased, thereby decreasing the hydraulic gradient and seepage velocity to a safe value.

(2)P During periods of extremely unfavourable hydraulic conditions such as floods, areas susceptible to piping shall be inspected regularly so that necessary mitigating measures can be taken without delay. Materials for such measures shall be stored in the vicinity.

(3)P Failure by piping shall be prevented by providing sufficient resistance against internal soil erosion in the areas where water outflow may occur.

(4) Such failure can be prevented by providing:

1. sufficient safety against failure by heave where the ground surface is horizontal;
2. sufficient stability of the surface layers in sloping ground (local slope stability).

(5)P When determining the outflow hydraulic conditions for the verification of failure by heave or of local slope stability, account shall be taken of the fact that joints or interfaces between the structure and the ground can become preferred seepage paths.

(6) A sample method for piping verifications is given in Annex HY.2

# Section 12 Embankments

## "12.3" Actions and design situations

~~(6) For shore embankments, the most unfavourable hydraulic conditions should be considered. These are normally steady seepage for the highest possible ground-water level and rapid draw-down of the free water level.~~

(6)P For embankments subjected to variations in water level rapid drawdown of the free water level shall be considered.

A.4 Partial factors for uplift limit state (UPL) verifications

(1)P For the verification of uplift limit state (UPL) the following partial factors on actions (*γ*F) shall be applied:

* *~~~~*~~G;dst~~ ~~on destabilising unfavourable permanent actions;~~
* **G;dst on destabilising unfavourable permanent actions not caused by water pressure;   
   there is no partial factor on the destabilising unfavourable permanent action of   
   water pressure, for which design values are derived directly;
* **G;stb on stabilising favourable permanent actions not caused by water pressure;
* **Q;dst on destabilising unfavourable variable actions not caused by water pressure.

NOTE The values to be ascribed to *~~~~*~~G;dst~~ ~~,~~ **G;dst , **G;stb and **Q;dst for use in a country may be found in its National annex to this standard. The recommended values are given in Table A.15.

Table A.15 - Partial factors on actions (*γ*F)

|  |  |  |
| --- | --- | --- |
| **Action** | **Symbol** | **Value** |
| Permanent  Unfavourablea  Favourableb | *γ*G;dst  *γ*G;stb | ~~1,0~~  -  1,0  0,9 |
| Variable  Unfavourablea | *γ*Q;dst | 1,5 |
| a Destabilising;  b Stabilising | | |

(2)P For the verification of uplift limit state (UPL) the following partial factors shall be applied when including resistances:

1. *γ*ϕ’ on the tangent of the angle of shearing resistance;
2. *γ*c’ on effective cohesion;

* *γ*cu on undrained shear strength;
* *γ*s;t on tensile pile resistance;
* *γ*a on anchorage resistance.

NOTE The values to be ascribed to *γ*ϕ’, **c’, **cu, **s;t, and **a for use in a country may be found in its National annex to this standard. The recommended values are given in Table A.16.

Table A.16 - Partial factors for soil parameters and resistances

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| --- | --- | --- |
| **Soil parameter / resistances** | **Symbol** | **Value** |
| Angle of shearing resistancea | *γ*ϕ’ | 1,25 |
| Effective cohesion | *γc*’ | 1,25 |
| Undrained shear strength | *γ*cu | 1,40 |
| Tensile pile resistance | *γ*s;t | 1,40 |
| Anchorage resistance | *γ*a | 1,40 |
| a This factor is applied to tan *ϕ*' | | |

A.5 Partial factors for hydraulic heave limit state (HYD) verification

(1)P For the verification of hydraulic heave limit state (HYD) the following partial factors on actions (*γ*F) shall be applied:

* *~~~~*~~G;dst~~ ~~on destabilising unfavourable permanent actions;~~
* **G;dst there is no partial factor on the destabilising unfavourable permanent action of   
   seepage force, for which design values are derived directly;;
* **G;stb on stabilising favourable permanent actions;
* **Q;dst on destabilising unfavourable variable actions.

NOTE: The values to be ascribed to *~~~~*~~G;dst~~ ~~,~~ **G;stb  and **Q;dst for use in a country may be found in its National annex to EN 1990:2002. The recommended values are given in Table A.17.

Table A.17 - Partial factors on actions (*γ*F)

|  |  |  |
| --- | --- | --- |
| **Action** | **Symbol** | **Value** |
| Permanent  Unfavourablea  Favourableb | *γ*G;dst  *γ*G;stb | ~~1,35~~ -  0,90 |
| Variable  Unfavourablea | *γ*Q;dst | 1,50 |
| a Destabilising  b Stabilising | | |

(2)P For the verification of hydraulic heave limit state (HYD) the following partial factors shall be applied when including resistances:

1. *γ*ϕ’ on the tangent of the angle of shearing resistance;
2. *γ*c’ on effective cohesion;

* *γ*cu on undrained shear strength;

NOTE The values to be ascribed to *γ*ϕ’, **c’and **cu for use in a country may be found in its National annex to this standard. The recommended values are given in Table A.17a.

Table A.17a - Partial factors for soil parameters

|  |  |  |
| --- | --- | --- |
| **Soil parameter** | **Symbol** | **Value** |
| Angle of shearing resistancea | *γ*ϕ’ | 1,25 |
| Effective cohesion | *γc*’ | 1,25 |
| Undrained shear strength | *γ*cu | 1,40 |
| a This factor is applied to tan *ϕ*' | | |

**Annex HY (informative)**

**HY.1 Hydraulic heave and Terzaghi's block**

(1) For cohesionless soils in conditions of steady state seepage, HYD ultimate limit state verification according to 10.3 (1c) may be satisfied by checking only a single soil column as, for example, “Terzaghi’s block” as shown in Figure HY.1. In this case, Rd should be omitted and a model factor γG,hyd shall be applied to ensure robustness.

In this case the design value of the buoyant weight of a soil block G'd shall be

G'stb;d = G,hyd∙G'stb;k∙γG;stb (HY.1)

NOTE: The values to be ascribed to G,hyd for use in a country may be found in its National annex to this standard. The recommended values are given in Table HY.1.

|  |
| --- |
|  |
| **Figure HY.1 — hydraulic heave and "Terzaghi's" block** |

Table HY.1 – model factor to be applied with HYD and Terzaghi's block

|  |  |  |
| --- | --- | --- |
| **model factor related to** | **Symbol** | **Value** |
| effective weight of soil subjected to vertical seepage using Terzaghi's block model | G,hyd’ | 0.6 |

**HY.2 Piping**

(1) When seepage may occur in a slope, or through the bottom of an excavation in soil susceptible to erosion, it shall be demonstrated that “piping” cannot lead to failure or intolerable displacements.

(2) The hydraulic gradient (i) at the location of exit of water from a slope shall be smaller than or equal to the prescribed safe critical value or special filter constructions shall be incorporated in the design in order to exclude the possibility of “piping”.

(3) A prescribed safe critical value ic shall be used for horizontal soil surface and in slopes a lower value depending on the slope angle and the embankment material.

NOTE: The value of the critical hydraulic gradient ic may be set by the National annex. The recommended value is 0.5.

(4) Furthermore, registrations of the phreatic surface and the quantity of seepage should be made to ensure that the slope or the bottom of an excavation behaves as predicted.

NOTE 1: Design rules of “Bligh and Lane” can be used to evaluate the possibility of piping.

NOTE 2: Measures in order to prevent “piping”are:

* Regulating seepage
* Limiting of the hydraulic gradient
* Installation of protective filters

(5) The check of the mechanism “piping” in case of a retaining wall may be made by:

1. Groundwater flow analysis to compute the hydraulic gradient at the exit point. A check shall be made that the characteristic value of the critical hydraulic gradient at the exit point ikrit;rep is less than ic.
2. Check of the seepage length according to Lane to prevent piping, see figure 10.d. In this case it shall be ensured that:

L1 + L2 ≥ γpiping \* CL \* ΔH

Where:

L1 = design value of the length of the sheetpile wall below the groundwater table in m.

L2 = design value of the length of the sheetpile below the lowest ground surface in m.

ΔH = difference in waterlevel between “the high side and the low side” of the

sheetpile wall in m.

γpiping = partial factor for “piping” form Table HY.2; this factor is for compensation

of the decrease in passive soil pressure due to upward groundwater flow.

CL = constant of Lane, see Table HY.3.

Table HY.2 – Partial factor for piping γpiping

|  |  |
| --- | --- |
| **Consequence Class** | **γpiping** |
| CC1 | 1.5 |
| CC2 | 1.75 |
| CC3 | 2.0 |

Table HY.3 – Lane Factor CL

|  |  |
| --- | --- |
| **Soil type** | **CL** |
| Very fine sand or silt | 8.5 |
| Fine sand | 7.0 |
| Medium coarse sand | 6.0 |
| Coarse sand | 5.0 |
| Fine gravel | 4.0 |
| Medium coarse gravel | 3.5 |
| Coarse gravel | 3.0 |
| Stones | 2.5 |

|  |
| --- |
|  |
| **Figure HY.2 data for check of the seepage length according to Lane to prevent piping** |

**HY.3 References**

**Terzaghi, K. (1922):** Der Grundbruch an Stauwerken und seine Verhütung. In: Die Wasserkraft, Zeitschrift für die gesamte Wasserwirtschaft, 17. Jahrgang 1922.

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