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Execution of special geotechnical works — Vertical drainage

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ICS 93.020

British Standards

National foreword

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Execution of special geotechnical works - Vertical drainage

Exécution des travaux géotechniques spéciaux - Drains verticaux

Ausführung von besonderen geotechnischen Arbeiten (Spezialtiefbau) - Vertikaldräns

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CEN members are bound to comply with the CEN/CENELEC Internal Regulations which stipulate the conditions for giving this European Standard the status of a national standard without any alteration. Up-to-date lists and bibliographical references concerning such national standards may be obtained on application to the CEN Management Centre or to any CEN member.

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Foreword

This document (EN 15237:2007) has been prepared by Technical Committee CEN/TC 288 “Execution of special geotechnical works”, the secretariat of which is held by AFNOR.

This European Standard shall be given the status of a national standard, either by publication of an identical text or by endorsement, at the latest by August 2007, and conflicting national standards shall be withdrawn at the latest by August 2007.

The document has been prepared to stand alongside EN 1997–1, *Eurocode 7: Geotechnical design — Part 1: General rules*, and EN 1997–2 *Eurocode 7: Geotechnical design — Part 2: Ground investigation and testing*. This standard expands on design only where necessary, but provides full coverage of the construction and supervision requirements.

This document was drafted by a working group comprising delegates from 10 European countries. Experts from Japan have taken part in the meetings of the working group and contributed to the formulation of the final draft. The working group commenced work in March 2002.

According to the CEN/CENELEC Internal Regulations, the national standards organizations of the following countries are bound to implement this European Standard: Austria, Belgium, Bulgaria, Cyprus, Czech Republic, Denmark, Estonia, Finland, France, Germany, Greece, Hungary, Iceland, Ireland, Italy, Latvia, Lithuania, Luxembourg, Malta, Netherlands, Norway, Poland, Portugal, Romania, Slovakia, Slovenia, Spain, Sweden, Switzerland and United Kingdom.

1 Scope

This European Standard establishes general principles for the execution, testing, supervision and monitoring of vertical drain projects.

This European Standard includes the application of prefabricated vertical drains and sand drains and deals with requirements to be placed on design, drain material and installation methods. This European Standard applies to the improvement of low-permeability, highly compressible soils by vertical drainage and preloading. Information regarding loading (embankment, vacuum or ground water lowering) and preloading is given in informative Annexes A and B.

Vertical drainage is used both in on land and in marine constructions for the following purposes:

— (pre-)consolidation and reduction of post-construction settlements;

— speeding up the consolidation process by decreasing the path lengths for pore water dissipation;

— increase of stability (by increasing effective stresses in the soil);

— groundwater lowering;

— mitigation of liquefaction effects.

In each case there is an overall treatment of the soil (the volume of the drains is small in relation to the soil volume treated).

This European Standard does not include soil improvement by means of wells, gravel and stone columns, large-diameter geotextile enclosed columns or reinforcing elements.

Vertical drainage can also be combined with other foundation or ground improvement methods, e.g. electro-osmosis, piles and compacted sand piles, dynamic compaction and deep mixing.

Guidance on practical aspects of vertical drainage, such as investigation of drain properties, execution procedures and equipment, is given in Annex A. Investigation of soil characteristics and assessment of design parameters, which are affected by drain properties and execution, are presented in Annex B.

2 Normative references

The following referenced documents are indispensable for the application of this document. For dated references, only the edition cited applies. For undated references, the latest edition of the referenced document (including any amendments) applies.

EN 1997–1, *Eurocode 7: Geotechnical design — Part 1: General rules*

EN 1997–2, *Eurocode 7: Geotechnical design — Part 2: Ground investigation and testing*

EN 13252:2000, *Geotextiles and geotextile-related products — Characteristics required for use in drainage systems*

EN ISO 9862, *Geosynthetics — Sampling and preparation of test specimens (ISO 9862:2005)*

EN ISO 10319, *Geotextiles — Wide-width tensile test (ISO 10319:1993)*

EN ISO 10320, *Geotextiles and geotextile-related products — Identification on site (ISO 10320:1999)*

EN ISO 10321, *Geotextiles — Tensile test for joints/seams by wide-width method (ISO 10321:1992)*

EN ISO 11058, *Geotextiles and geotextile-related products — Determination of water permeability characteristics normal to the plane, without load (ISO 11058:1999)*

EN ISO 12956, *Geotextiles and geotextile-related products — Determination of the characteristic opening size (ISO 12956:1999)*

EN ISO 12958:1999, *Geotextiles and geotextile-related products — Determination of water flow capacity in their plane (ISO 12958:1999)*

EN ISO 14688 (all parts), *Geotechnical investigation and testing*

3 Terms and definitions

For the purposes of this document, the following terms and definitions apply.

3.1
auger installation method
fr: installation à la tarière
de: Schneckeneinbauverfahren
installation method of sand drains by means of screw type auger or continuous flight hollow stem auger

3.2
band drain[1)]
fr: drain plat
de: Streifendrän
prefabricated drain with a rectangular cross-section, usually consisting of a central core with a channel system surrounded by a filter sleeve

3.3
cylindrical prefabricated drain
fr: drain tubulaire
de: zylindrischer Drän
drain consisting of an annular-corrugated and perforated open core, surrounded by a filter sock

3.4
discharge capacity q_w
fr: capacité de décharge
de: Durchflusskapazität
discharge capacity of a drain well is equal to the cross-sectional area of the drain multiplied by its overall permeability in longitudinal direction (the volume of water which flows out of the drain per time unit under a hydraulic gradient equal to unity)

3.5
displacement installation method
fr: installation par fonçage refoulant
de: Verdrängungseinbauverfahren
installation method of drains by means of a closed-end steel tube/mandrel

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1) The terms “wick drains” and “prefabricated vertical drains” (PVD) are also used.

3.6
drain anchor
fr: sabot d'ancrage
de: Dränschuh
anchor fixed at the end of band drains before installation and which prevents soil from intruding into the mandrel during installation and the band drain from being dragged up when the mandrel is withdrawn

3.7
drainage blanket
fr: tapis drainant
de: Drändecke
upper high-permeability drainage layer, which has good contact with the drains and prevents the creation of backpressure in the drains

3.8
dynamic installation method
fr: méthode d'installation par battage ou vibrofonçage
de: dynamische Dräneinbauweise
drain installation method using dynamic action (impact or vibratory hammer)

3.9
geotextile enclosed sand drain
fr: drains de sable dans une gaine de géotextile
de: geotextilumschlossener Sanddrän
sand drains enclosed in some type of filter fabric made of geotextile

3.10
jet installation method
fr: méthode d´installation par lançage
de: Einbau mittels Hochdruckinjektion
installation method of sand drains by means of internal jetting or rotary jet

3.11
sand drain
fr: drain de sable
de: Sanddrän
drain with circular cross-section, built up of granular material (sand, gravel) with high permeability

3.12
static installation method
fr: méthode d´installation par fonçage
de: statisches Einbauverfahren
drain installation method by means of static load (pushing)

3.13
vibro installation method
fr: méthode d´installation par vibrofonçage
de: Vibrationseinbauverfahren
installation method of sand drains by means of a top vibrator mounted on a hollow mandrel or by a depth vibrator

3.14
working platform
fr: plateforme de travail
de: Arbeitsebene
platform created for access of the drain installation machines to the location of the vertical drains

4 Information needed for the execution of the work

4.1 Prior to the execution of the work, all necessary information shall be available.

4.2 This information shall include:

a) relevant information regarding the site conditions;

b) the location of main grid lines for setting out;

c) drawings with the location and length of the drains;

d) any legal or statutory restrictions;

e) method statement for the vertical drain installation (see 8.1);

f) characteristics of the drains (physical and hydraulic characteristics), accompanying document for CE-marking;

g) specification for the drains and other materials to be used (see Clause 6) and the schedule of any testing and acceptance procedures for materials incorporated in the works;

h) description of a suitable quality management system, including supervision and monitoring.

4.3 The information regarding the site conditions shall cover, where relevant:

a) the geometry of the site (boundary conditions, topography, access, slopes, headroom restrictions etc.);

b) the ground properties of the site which may influence the execution of the vertical drains:

- soil description (soil type, soil stratification and existence and frequency of sand and silt layers, hard layers);
- penetration resistance (e.g. results of penetration tests);
- composition, lateral extent, thickness and firmness of the surface stratum, tree roots, fill etc.;
- presence of cobbles or boulders or cemented layers that can cause difficulties for the execution or can require special installation equipment;

c) the following climatic and environmental information:

- weather information in areas with extreme climatic conditions;
- marine conditions (currents, tidal movements, wave heights etc.);
- environmental hazards (any water and subsoil contamination, which could affect the execution method, the safety of the work or the discharge of excavation material from the site shall be documented, including the presence of hazardous gas and occurrence of unexploded ammunition);

d) the existing underground structures, services, known contamination and archaeological constraints;

e) the environmental restrictions, including noise, vibration and pollution;

f) planned or ongoing construction activities, such as dewatering, tunnelling and deep excavations;

g) previous experience from drain installation work adjacent to the site;

h) the characteristics of the working platform and the drainage layer (physical and hydraulic properties);

i) the conditions of structures, roads, services etc. adjacent to the work.

4.4 For the loading operation the following information shall be available:

a) construction programme for loading;

b) preloading (temporary and permanent loading);

c) time schedule for loading and possible preloading;

d) unit weight of fill used for preloading;

e) notice of any restrictions such as construction phasing required in the design;

f) monitoring programme.

4.5 The following instructions shall also be given:

a) reporting procedure for unforeseen circumstances, or conditions revealed that appear to be different from those assumed in the design;

b) reporting procedure, if an observational method of design is adopted.

4.6 Any additional or deviating requirements falling within the permission clauses given in this European Standard shall be established and agreed upon before the commencement of the works.

5 Geotechnical investigations

5.1 General

5.1.1 The geotechnical investigations carried out for the design of vertical drainage work following the requirements of EN 1997 shall provide information for the installation of the vertical drains, see 4.3 b).

5.1.2 The geotechnical investigation report shall be available in sufficient time to allow for planning and execution of the vertical drainage works.

5.1.3 If the geotechnical investigations carried out are deemed insufficient, a supplementary investigation should be conducted.

5.2 Specific requirements

5.2.1 Apart from the general geological description and the details listed in EN 1997-1, the site investigation report shall contain information regarding ground conditions for the execution of vertical drain installations and for loading, see Annex A and Annex B.

5.2.2 Information about ground conditions shall comprise:

a) piezometric levels of groundwater, its variation and possible deviation from hydrostatic pressure conditions;

b) undrained shear strength.

5.2.3 The ground level and location at any point of investigation or testing should be established relative to the recognised national datum or to a fixed reference point.

6 Materials and product

6.1 General

6.1.1 Vertical drainage involves the use of prefabricated drains and/or sand drains.

NOTE A variety of prefabricated drains exists on the market. Most of them consist of the core and a geotextile filter.

6.1.2 Prefabricated drains consisting of a core and a geotextile filter are subjected to CE marking. The following specific properties shall be given.

a) tensile strength, in kN according to EN ISO 10319;

b) elongation at maximum tensile force, in % according to EN ISO 10319;

c) tensile strength of filter, in kN/m according to EN ISO 10319;

d) tensile strength of seams and joints, in kN/m according to EN ISO 10321;

e) velocity index of filter (v_{h50}), in mm/s according to EN ISO 11058;

f) characteristic opening size of filter (O_{90}), in µm according to EN ISO 12956;

g) discharge capacity of the drain, in m^3/year according to EN ISO 12958 (see also Annex A);

h) durability, in years (EN 13252:2000, Annex B).

NOTE The dynamic perforation resistance (cone drop test, EN 918) as mentioned in EN 13252 is not compulsory for this application.

6.2 Raw materials of prefabricated drains

6.2.1 A prefabricated drain usually comprises a geotextile or geotextile-related product.

6.2.2 The product is used in applications that do not involve reinforcement of soil and where long-term strength is not a significant parameter and in natural soils with a pH-value between 4 and 9 and at a soil temperature less than 25 °C according to EN 13252. A service lifetime of up to 5 years is applicable.

6.2.3 The use of 5 % internal regenerate (e.g. raw material made out of unused core) for the production of the core is permitted. The composition should be known and the material be processed in the same way as the original product. The use of recycled material is only permitted provided that it can be verified that it is not causing pollution of the soil or ground water.

6.2.4 Materials used to manufacture the drains shall not cause pollution of the soil or the groundwater.

6.2.5 Biodegradable drains can be used if they fulfil the drainage requirements during the lifetime of the project.

6.3 Band drains

6.3.1 Shape and structure of band drains

6.3.1.1 The band drain is a prefabricated drain with a rectangular cross-section, usually consisting of a central core with a channel system surrounded by a filter. The width of the core of the band drains is typically 100 mm and the thickness is between 2 mm and 10 mm.

6.3.1.2 The core should consist of a profiled strip, with or without perforation, or a profiled mat with an open or closed structure. It should have a structure that provides regular hydraulic flow capacity.

6.3.1.3 Tears and/or other defects shall not be allowed to occur. Visual inspections for damage shall be made regularly as part of the production quality control.

6.3.2 Measurements

6.3.2.1 The roll length, width and thickness of the core at any given place should comply with the dimensions given by the manufacturer (within allowable deviations given by the manufacturer). See accompanying document for CE-marking (EN 13252).

6.3.2.2 The measurements should comply with EN ISO 9863-1.

6.3.3 Durability

6.3.3.1 The durability of the drain shall comply with the durability aspects given in EN 13252 (weathering, required service lives up to five years or more when the drains are installed to mitigate soil liquefaction).

6.3.3.2 The drains should be protected against weathering during storage on the site.

6.3.3.3 The product shall not be exposed longer than the time announced by the producer for CE Marking (EN 13252) unless the product is protected by a wrapping material or stored in-house. The recommendations of the supplier shall be followed.

6.3.4 Tensile strength and elongation

6.3.4.1 The required tensile strength of the band drain is very dependent upon the type of installation machine, installation technique and depth of the drain. Tensile strength of the band drains in the longitudinal direction shall be high enough to prevent breakage during and after installation.

6.3.4.2 Testing of tensile strength and elongation of the band drain should be made in accordance with EN ISO 10319 (to be modified with regard to the width of the product).

6.3.4.3 The following characteristics of a band drain are recommended:

a) elongation ≥ 2 % at failure of the weakest element;

b) elongation ≤ 10 % at a tensile force of 0,5 kN (20 % if exposed to frost);

c) minimum tensile force > 1,5 kN at failure of the weakest element. The seam should not fail during the test.

NOTE These values depend on the installation equipment and procedure and may need to be adjusted accordingly.

6.3.4.4 The strength of the seam, measured according to EN ISO 10321 in a range of temperatures, which apply to the project site, shall be at least 1 kN/m.

6.3.5 Discharge capacity

6.3.5.1 The discharge capacity and the filtration characteristics are the most important properties. The following factors influence the discharge capacity:

a) due to increasing lateral effective pressure during the consolidation process the filter is squeezed into the channel system of the core, which reduces the channel area;

b) the vertical compression of the soil taking place during the consolidation process may lead to buckling of the relatively incompressible band drains, which may reduce the channel area;

c) fines may intrude through the filter into the core and cause blocking of the channel system;

d) soil temperature has an influence on the compression resistance and creep of the drains and thus on the discharge capacity.

6.3.5.2 The required discharge capacity of the band drain is largely dependent upon the purpose of ground improvement, the consolidation parameters of the soil, the drain spacing and the depth of drain installation (see Annex B).

6.3.5.3 The discharge capacity shall be high enough to satisfy the design requirements.

6.3.5.4 The recommended value of the discharge capacity is given in Annex B.

6.3.5.5 The discharge capacity test should be carried out in accordance with EN 12958 with the modifications given in A.4.1.2 in Annex A.

6.3.5.6 For usual applications, the discharge capacity test should be performed at the laboratory temperature and then the test report should be referred to a temperature of 20 °C. For applications in tropical environment, the discharge capacity test should be performed at a temperature corresponding to the soil temperature at the place of drain installation and then the test report should be referred to that specific temperature.

6.3.5.7 The test period should be long enough to yield a constant value of discharge capacity, preferably at least two days at the maximum static pressure stipulated by the designer.

6.3.6 Filter of band drains

6.3.6.1 The filter sleeve should be composed of a non-woven material consisting of fibres that are mechanically, chemically or thermally bonded.

6.3.6.2 The filter sleeve should have a regular structure.

6.3.6.3 The occurrence of creases, tears, holes and/or other defects shall not be allowed. The seams of the filter sleeve shall be constructed in such a way that fines cannot intrude into the core of the band drain.

6.3.6.4 Visual inspections for damage shall be made regularly during production in accordance with the factory production control plan.

6.3.7 Tensile strength per unit width of filter

6.3.7.1 The tensile strength of the filter shall be sufficient to prevent breakage during and after installation.

6.3.7.2 Testing should be carried out in accordance with EN ISO 10319. The average of the individually measured values for the tensile strength should not be lower than 3 kN/m in the longitudinal direction. For installations deeper than 25 m or in difficult soil conditions, a minimum tensile strength of 6 kN/m in the longitudinal direction is recommended.

6.3.8 Velocity index of filter

Testing should be carried out in accordance with EN ISO 11058. The average of the individually measured values of the velocity index (v_{h50}) should be higher than 1 mm/s. In case of drain installation for liquefaction problems, the filter pore size should be adapted to ensure adequate permeability of the filter for this application, see 6.4.

6.3.9 Pore size of filter

6.3.9.1 The pore size of filter shall be selected to ensure sufficient discharge capacity and avoid serious loss of discharge capacity due to clogging of the filter and/or the core by soil particles. The seams of the filter shall not have an opening size bigger than that of the geotextile filter.

6.3.9.2 Primarily, the requirements for the filter sleeve characteristics shall be given in the project, considering the soil properties at the site and the installation conditions (dry or wetland, offshore).

6.3.9.3 The value of the characteristic opening size O_{90}, measured in accordance with EN ISO 12956 should not be higher than 80 μm.

NOTE The value of O_{90} may be influenced by project-specific requirements and higher values may be acceptable.

6.3.9.4 In silty soils and silt the characteristic opening size O_{90} of the filter should be adapted to the soil conditions according to the following criteria:

a) $< d_{85,\text{soil}}$ in silty soils, which are problematic from the point of view of filtering technique;

b) $< 1{,}5\ d_{50,\text{soil}}$ to $2{,}8\ d_{50,\text{soil}}$ in soils, which are difficult from the point of view of filtering technique, mainly medium and coarse silt.

6.3.10 Quality control

6.3.10.1 The band drain shall comply with all European requirements and conformity assessment procedures that apply to it. The properties shall be within the limits announced in the accompanying document accredited by a Notifying Body.

6.3.10.2 The filter and drain characteristics and corresponding testing methods, as well as the proposed testing frequency, are given in Table 1, adapted from EN 13252. The sampling procedure for the different testing methods shall comply with EN ISO 9862.

NOTE The on-site testing frequency should be decided between the parties involved.

Table 1 — Proposed testing frequency for fabrication control

Property	Proposed test frequency	Required standard
Filter:		
Thickness	25 000 m^2	EN 9863-1
Mass per unit area	25 000 m^2	EN 9864
Pore size	200 000 m^2	EN 12956
Velocity index	200 000 m^2	EN 11058
Tensile strength in the longitudinal direction	200 000 m^2	EN 10319
Tensile strength in the cross direction	200 000 m^2	EN 10319
Drain composite:		
Width and thickness	25 000 m	EN 9863-1
Mass per unit length	25 000 m	EN 9864
Tensile strength in the longitudinal direction	100 000 m	EN 10319
Elongation at maximum tensile force	100 000 m	EN 10319
Discharge capacity straight	500 000 m	Annex A
Discharge capacity buckled	500 000 m	Annex A
Tensile strength of filter seam	100 000 m	EN 10321
Durability	500 000 m	EN 13252

6.4 Prefabricated cylindrical drain

6.4.1 Shape and structure of cylindrical drains

6.4.1.1 A cylindrical drain consists of an annular-corrugated and perforated open core, surrounded by a filter sock. The drain diameter is typically 50 mm in outer diameter and 45 mm in inner diameter.

6.4.1.2 Tears and/or other defects shall not be allowed to occur. Visual inspections for damage shall be made regularly as part of the production quality control.

6.4.2 Measurements

The diameter and thickness of the core should comply with the dimensions given by the manufacturer (within allowable deviations given by the manufacturer).

6.4.3 Durability

6.4.3.1 The durability of the drain shall comply with the durability aspects given in EN 13252 (Annex B, B.1 weathering, and B.2, required service lives up to five years or more when the drains are installed to mitigate soil liquefaction).

6.4.3.2 The drains should be protected against weathering during storage on the site.

6.4.3.3 The product shall not be exposed longer than the time announced by the producer for CE Marking (EN 13252) unless the product is protected by a wrapping material or stored in-house. The recommendations of the supplier shall be followed.

6.4.4 Tensile strength and elongation

6.4.4.1 The required tensile strength of the cylindrical drain is very dependent upon the type of installation machine, installation technique and depth of the drain. Tensile strength of the drains in the longitudinal direction shall be high enough to prevent breakage during and after installation.

6.4.4.2 Testing of tensile strength and elongation of the cylindrical drain should be made in accordance with the standard tensile test with modified clamps.

6.4.4.3 The strength of the seam, measured according to EN ISO 10321 in a range of temperatures which apply to the project site, shall be at least 1 kN/m.

6.4.5 Discharge capacity and filtration characteristics

6.4.5.1 The discharge capacity of the cylindrical drains is usually much larger than is required for soil consolidation. It may decrease if the cylindrical core is crushed due to an increase of lateral effective pressure during the consolidation process and/or buckling.

6.4.5.2 The perforation of the pipe (core) shall not be sealed due to compression of the filter sleeve.

6.4.5.3 The pipe and filter of drains used for liquefaction mitigation shall resist the effect of ageing during the design lifetime of the structure.

6.4.6 Filter of cylindrical drains

6.4.6.1 The filter sleeve should be composed of an arrangement of fibres that is mechanically, chemically or thermally bonded.

6.4.6.2 The filter sleeve should have a regular structure.

6.4.6.3 The occurrence of creases, tears, holes and/or other defects shall not be allowed. The seams of the filter sleeve shall be constructed in such a way that fines cannot intrude into the core of the band drain.

6.4.6.4 Visual inspections for damage shall be made regularly during production in accordance with the factory production control plan.

6.4.7 Tensile strength per unit width of filter

6.4.7.1 The tensile strength of the filter shall be sufficient to prevent breakage during and after installation.

6.4.7.2 Testing should be carried out in accordance with EN ISO 10319. The average of the individually measured values for the tensile strength should not be lower than 3 kN/m in the longitudinal direction. For installations deeper than 25 m or in difficult soil conditions, a minimum tensile strength of 6 kN/m in the longitudinal direction is recommended shall be compliant with the recommended value.

6.4.8 Velocity index of filter

Testing should be carried out in accordance with EN ISO 11058. The average of the individually measured values of the velocity index (v_{h50}) should be higher than 1 mm/s. The average of the individually measured values shall comply with the recommended value. In case of drain installation for liquefaction problems, the filter pore size should be adapted to ensure adequate permeability of the filter for this application.

6.4.9 Pore size of filter

The requirements are identical to 6.3.9.

6.4.10 Quality control

The prefabricated drain product shall comply with all European requirements and conformity assessment procedures that apply to it. The properties shall be within the limits announced in the accompanying document accredited by a Notifying Body.

6.5 Sand drains

6.5.1 The sand drain has a circular cross-section, built up of granular material with high permeability. The diameter of the sand drain can vary typically from 150 mm to 500 mm.

6.5.2 The grain size distribution of the material used for sand drains should preferably fall within the limits given in Annex A.

6.5.3 The permeability of the sand should be high enough to avoid significant well resistance. The permeability requirements depend on the permeability of the surrounding soil and the depth of drain installation (see Annex B) and should primarily be defined by the designer.

6.5.4 Material used for sand drains shall not cause pollution of soil or groundwater when installed.

7 Considerations related to design

7.1 Field trials

7.1.1 Test areas with various drain spacing and/or various drain types may be required as a basis for the final design of the drain installation work.

7.1.2 The process of consolidation in the test areas should be monitored by settlement measurement in combination with pore pressure measurement, preferably by means of settlement gauges and piezometers placed at various depths. The strength increase due to consolidation may be assessed by laboratory tests and/or *in-situ* tests.

7.1.3 When relevant, the horizontal displacements along the periphery of the test area may be measured by means of inclinometer.

7.1.4 In case of test areas with partially penetrating drains the influence on the consolidation process of underlying untreated soil layers should be taken into account.

8 Execution

8.1 Method statement

8.1.1 A method statement shall be prepared, which details the vertical drainage works. The method statement shall detail the location, drain grid/pattern, equipment and method of installation, possible restrictions during the construction phase and any hazards associated with the execution of the work.

8.1.2 The equipment and installation method chosen by the contractor shall be assessed and approved.

8.1.3 The following method statement shall be delivered as a minimum (see Clause 4):

a) objective and scope of drain installation;

b) site installation and working areas;

c) plant and equipment;

d) control procedures;

e) procedures regarding possible interruptions during drain installation and/or preloading;

f) verification testing methods;

g) working documents (layout, drawings, reports);

h) measures to avoid puncturing of artesian ground water;

i) safety and environmental risk assessment.

8.1.4 If judged necessary, trial installations should be carried out to confirm the installation machine suitability for the site conditions and the drain material utilised.

8.2 Preparation of the site

8.2.1 The preparation shall be carried out in accordance with the design specifications and the specific site conditions. This shall include suitable access for plant and machinery, levelling of the working platform, providing adequate ground bearing capacity for equipment and installation of a drainage blanket. The drainage blanket may be installed after the installation of the vertical drains in on land constructions.

8.2.2 All materials and products for vertical drainage delivered to the site shall be identified and checked against the materials and products specifications.

8.2.3 When the drainage blanket serves as a working platform and is installed directly on the ground surface, it should have a minimum thickness of 0,5 m with an initially horizontal upper surface. When the drainage layer is installed on top of the working platform, made of ordinary fill material, the thickness of the working platform is usually at least 0,5 m for on land construction. The drainage blanket is then at least 0,3 m thick. It should consist of gravelly sand or sandy gravel, containing less than 5 % of material with grain size < 0,06 mm. It should be protected from ingress of fine-grained material and frost that can detrimentally affect its permeability. It can also consist of an appropriate draining system of geotextile or geotextile-related products.

8.2.4 When the drainage blanket consists of a layer of granular material, instructions should be given regarding the methods and frequency of checking the grain size distribution and permeability of the drainage blanket material.

8.2.5 The drainage blanket should be protected against freezing to ensure that its permeability is not seriously reduced.

8.2.6 In the case of marine drain installation, the drainage blanket should consist of granular material, preferably sandy gravel, with a minimum thickness of 0,5 m. The drainage blanket should be placed before the drain installation.

8.3 Drain installation

8.3.1 The surface location of installation of each drain shall deviate less than 0,15 m from the specified location. The verticality of the leader shall not be less than 50 (vertical):1 (horizontal), unless obstructions make it impossible.

8.3.2 Where it is impossible to install a drain as a result of obstructions, another drain shall be installed as close as possible.

8.3.3 If the presence of fill or dense soil at the surface makes installation of drains by conventional methods difficult, hard layers should be penetrated by pre-drilling or other appropriate methods prior to drain installation.

8.3.4 Drains shall be installed to the depth specified in the design (within a tolerance of 0,15 m). In soft soil deposits with varying thickness, this depth may be defined as that of the underlying more resistant layer.

8.3.5 For each installed drain, information shall be provided regarding installation date and installation depth.

8.3.6 During installation of sand drains the amount of sand filled into the drains shall be monitored to check whether necking or failure of the drains takes place or not.

8.3.7 For band drain installation, the mandrel should leave a free inside space for the drain and should be constructed in a way to limit soil disturbance. Further information is given in Annex A.

8.3.8 The installation rig for band drains should preferably be provided with a fully automatic recorder. The following parameters should be recorded:

a) drain identification number;

b) date and time;

c) depth of installation;

d) accumulated amount of installed drain length;

e) verticality and location.

8.3.9 Splicing of drains is permitted as long as the drainage path is not obstructed and the tensile strength of the splice complies with the requirements of 6.3.4 and 6.3.7.

8.4 Special aspects

8.4.1 The method of drain installation should not threaten the site stability. In particular, attention should be paid to excess pore water pressure being built up during the installation process by dynamic methods and in strain sensitive clays (quick clays) by displacement methods.

8.4.2 The execution of sand drain installation shall be performed by personnel who have previous experience of drain installation methods (closed-end casing installation, open-end mandrel installation, jet installation etc.).

8.4.3 Before band drains are installed into the soil they should be provided with an anchor, which keeps the drain in place when the mandrel is withdrawn from the soil. The soil should be prevented from intruding into the mandrel during installation (closed-end mandrel) (see Annex A).

8.4.4 After the mandrel is withdrawn the band drains shall be cut so that the drains are in adequate hydraulic connection with the drainage blanket, preferably 0,2 m to 0,25 m above the surface of the working platform.

8.4.5 For fully penetrating drains connected to a lower high-permeability layer, the penetration into this layer shall be sufficient to ensure that the drains are in adequate hydraulic connection with it.

9 Supervision and monitoring

9.1 Supervision

9.1.1 In order to check that construction complies with the design and other contract documents, suitably qualified personnel, experienced in the technique, shall be in charge of supervising the execution work.

9.1.2 Where unforeseen conditions are encountered or new information about ground conditions become available, they shall be reported immediately to those responsible for the design.

9.1.3 The specific procedures for verification, control and acceptance shall be established before the commencement of the work.

9.1.4 The actual frequency and method of control should be stated.

9.1.5 Identification of prefabricated drains on site shall be carried out according to EN ISO 10320, or according to similar procedures in case of drains with specific characteristics.

9.2 Monitoring

9.2.1 The extent and procedures of monitoring shall be specified by the design.

9.2.2 The extent of the monitoring system should account for the type of loading (e.g. step-wise loading, vacuum, groundwater lowering), for the choice of drain type and for previous experience of results achieved under similar soil and loading conditions and with similar types of drains.

9.2.3 The construction process shall be controlled and information concerning the ground conditions and construction tolerances shall be monitored during execution.

9.2.4 The consolidation process shall be monitored by appropriate settlement observations. The final primary consolidation settlement can be estimated with good accuracy from the time-related settlement observations (see Annex A).

9.2.5 The consolidation process should also be verified by appropriate methods of pore pressure observations, especially in the case of stability problems or when the observational method of design is used.

9.2.6 Where relevant, lateral time-related movements along the outer boundaries of the loaded area shall be monitored. Appropriate methods shall be used to evaluate these movements, e.g. by inclinometers.

9.2.7 The frequency of settlement and pore pressure observations should be adjusted to make a realistic interpretation of the consolidation process possible.

9.2.8 Monitoring instruments should be installed early enough to have stable reference values before the start of the loading process.

9.2.9 When relevant, the strength increase of the ground should be confirmed by means of laboratory tests on sampled specimens and/or *in-situ* tests.

10 Records

10.1 Records during construction

Records shall be made of relevant aspects of drain installation, tests and observations as described in Clauses 8 and 9 and these shall be available at the site.

10.2 Records at the completion of the work

Records shall be made of the as-built works, including:

a) records as in 10.1;

b) information detailing the completed drain installation, including test results, and any changes from the design drawings and specifications;

c) details of materials and products used;

d) details of relevant geotechnical soil conditions.

11 Special requirements

11.1 General

11.1.1 Only those aspects of site safety and protection of the environment that are specific to vertical drainage are considered in this clause.

11.1.2 All relevant European Standards and national standards, specifications and statutory requirements regarding safety and environment during execution of the work shall be respected.

11.1.3 The drain installation shall not damage existing underground utilities.

11.2 Safety

11.2.1 The installation equipment shall comply with European and/or national safety rules regarding construction and stability. Documents regarding operation, maintenance and safety shall be present in the installation equipment. Workers shall be trained according to national safety regulations regarding cranes and tall plant.

11.2.2 The drain installation rig should be equipped with a roll bar to protect the cabin in the case of machine overturning, caused by local, insufficient bearing capacity of the working platform.

11.2.3 The working platform shall have sufficient bearing capacity to carry the load of the installation rig.

11.2.4 To confirm that the working platform has sufficient bearing capacity to carry the load of the installation rig, the total area should be tested with a fully loaded shovel or a dumper for determining weak spots in the working platform.

11.2.5 During the drain installation works, in case two or more installation rigs are working on the same working platform, they should be separated horizontally by a distance larger than the total height of the rigs.

11.2.6. In the case of drain installation close to existing power lines or underground utilities, attention should be paid to relevant safety regulations.

11.3 Environmental protection

11.3.1 Construction shall identify and take into account environmental restrictions such as noise, vibrations, water pollution and impact on adjacent structures.

11.3.2 If contaminated groundwater is squeezed out of the soil through the drains during the consolidation process, it shall be treated.

11.3.3 At certain locations, the installation of drains may create a connection between aquifers, which can be detrimental to the environment. This should be assessed and the design modified if necessary.

11.4 Impact on adjacent structures

Where sensitive structures or unstable slopes are present in the vicinity of the site or the possible sphere of influence of the works, their condition should be carefully observed and documented prior to and during the works.

Annex A
(informative)

Practical aspects of vertical drainage

A.1 Introduction

In cases where external loading of low-permeability soils, such as clay, gyttja[2)], decomposed peat etc., causes a stress increase exceeding the pre-consolidation pressure of the soil, excess pore water pressure will be induced, followed by a consolidation process in which pore water is squeezed out of the soil. The volume decrease of the soil caused thereby is accompanied by a gradual increase in effective stress and a corresponding decrease in excess pore water pressure. The consolidation process will continue until the excess pore water pressure has completely dissipated and the load is carried by effective stresses, a process whose duration depends on the consolidation characteristics of the soil and the drainage paths (the longer the drainage paths, the longer the consolidation process). The aim of vertical drain installation is to shorten the drainage paths and the time required for the excess pore water pressure, induced by the loading operation, to dissipate. The time of excess pore water pressure dissipation (the consolidation time) will be shorter the closer the drains are installed.

A.2 Fields of application

As mentioned in A.1 the installation of vertical drains is carried out as a means of speeding up long-term consolidation settlements caused by loading. Another objective is to improve stability conditions by an overall increase in shear strength. In seismic regions vertical drainage can also be used for the purpose of mitigating liquefaction phenomena.

Examples of areas where this technique has generally been applied are:

- embankments for roads and railroads;
- construction and reinforcements of dikes;
- embankments for construction sites of housing estates, industrial estates, terminals etc.;
- preloading for landfills;
- marine constructions and near-shore applications;
- land reclamation, ports and airports.

Vertical drains have also been used as a means of electro-osmotic dewatering. In this case electrodes are inserted into the prefabricated band drains and connected to a voltage gradient [5] and [27]. The rate of consolidation thus achieved will be influenced by the voltage gradient and the electro-osmotic permeability coefficient.

A growing area of application is in the environmental field, remediation of contaminated ground. Contaminated water squeezed out through the drains may have to be treated before disposal.

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2) Decomposed plant and animal remains; may contain inorganic constituents (EN ISO 14688-1).

The required life span of vertical drains is normally limited to a maximum of about 5 years, with the exception of drains used for liquefaction mitigation where the lifetime needs to be significantly longer.

A.3 Execution of vertical drainage

The functional requirements of the project form the basis for the geotechnical design of vertical drainage. The execution of a vertical drainage system is shown in Figure A.1. It includes the creation of a working platform, the placement of a drainage blanket, positioning of the drain pattern and installation of the drains, followed by the loading operation and monitoring.

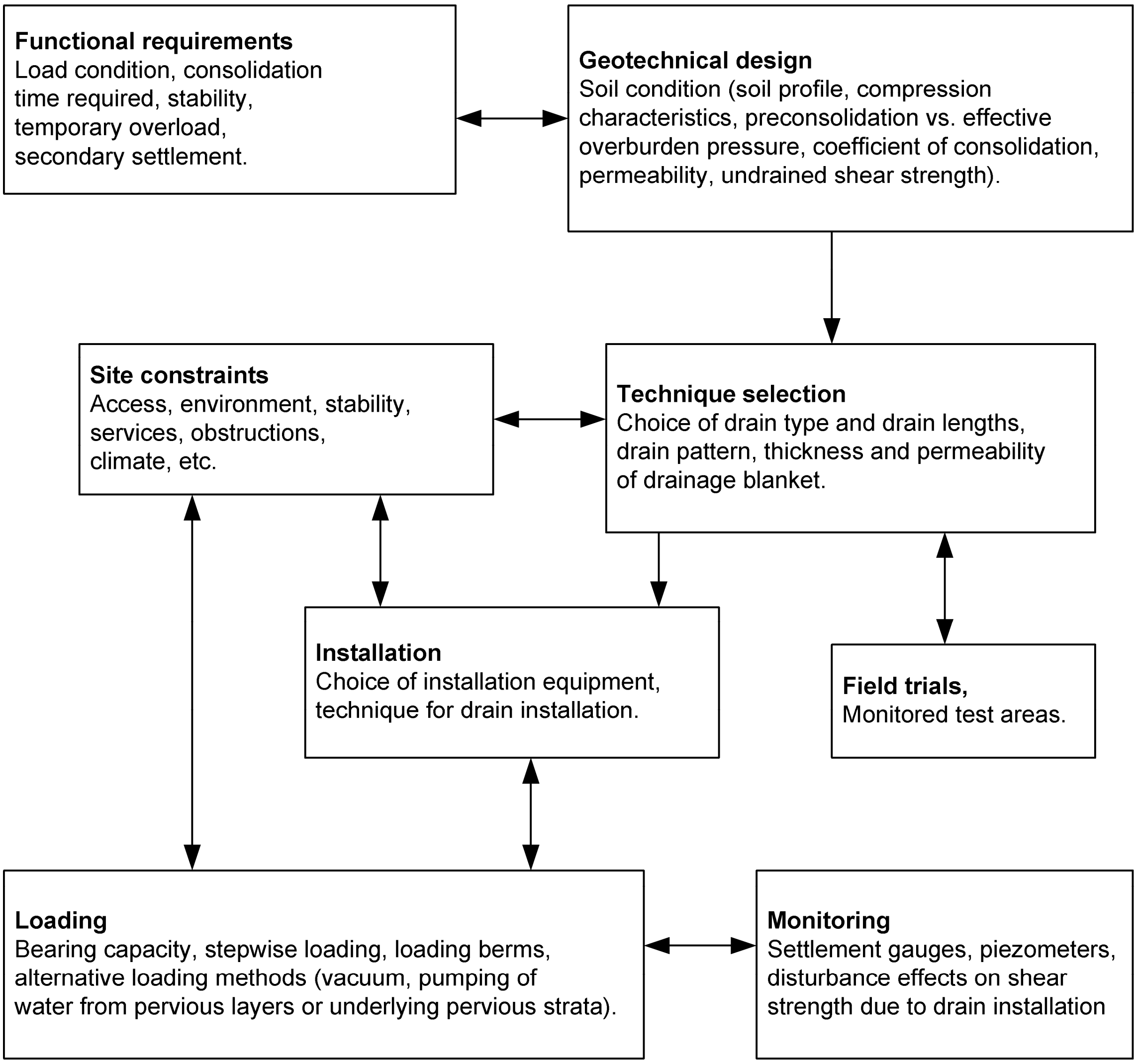
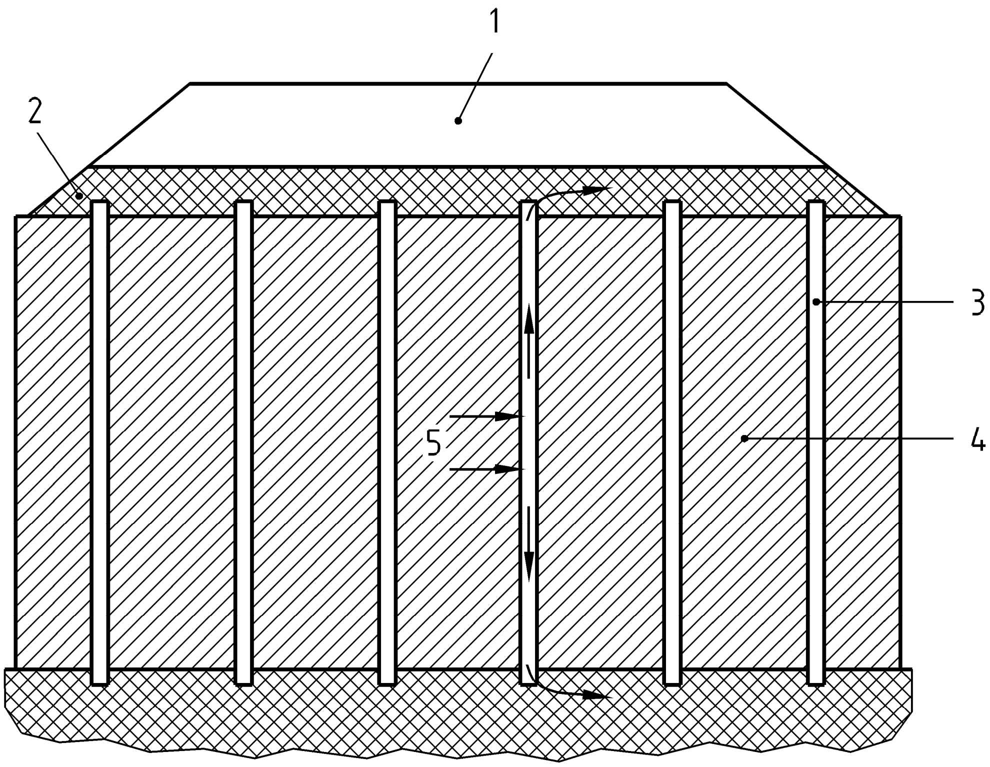


Figure A.1 — Chart of execution of vertical drainage

Prefabricated drain types have gradually replaced sand drains, which previously were frequently used. The installation of vertical drains may detrimentally affect the original properties of the soil (e.g. decrease the shear strength and coefficient of consolidation). A possible decrease in shear strength has to be taken into account in cases where stability under loading conditions may be threatened. Vertical drainage and preloading are illustrated in Figure A.2. Due to the excess pore water pressure created by loading, pore water is squeezed out of the soil in the horizontal direction towards the drains and thereafter in the vertical direction through the drains. A generally smaller amount of water is also squeezed out of the soil in the vertical direction between the drains (contributory effect of one-dimensional consolidation).



Key

1 surcharge load
2 drainage blanket
3 vertical drains
4 clay layer
5 pore water flow

Figure A.2 — Sketch showing fully penetrating drains (drains in contact with drainage layers at top and bottom), drainage blanket and surcharge load

Depending upon the installation method and procedure used, the installation of vertical drains may affect the original properties of the soil (e.g. decrease the shear strength and coefficient of consolidation). This should be considered in the design.

A.4 Drain types

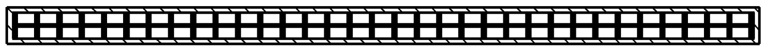
A.4.1 Band drains

A.4.1.1 General

Prefabricated band drains consist typically of a central core surrounded by a filter sleeve, Figure A.3. The width of the band drains is typically 100 mm.

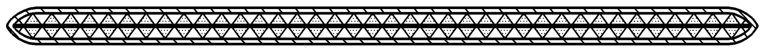
A.4.1.2 Types of drains

a) Channel-shaped core with glued filter



b) Channel-shaped core with wrapped filter

c) Geo-mat with edge-sealed filter



d) Cusp-shaped core with wrapped filter

Figure A.3 — Examples of band drains

A.4.1.3 Methods of installation

Band drains are installed inside a hollow mandrel with rectangular, rhomboid or circular cross-section. The size of the mandrel is normally adapted to leave a free inside space for the band drain during installation. Moreover, the bending rigidity of the mandrel needs to be high enough to ensure verticality of the drain installed.

An anchor, which is fixed to the drain tip before installation, prevents the drain from being dragged up when the mandrel is withdrawn, Figure A.4. During installation the soil should be prevented from intruding between the inside surface of the mandrel and the drain. Otherwise, the drain will be subjected to high tensile forces upon withdrawal. The shape of the mandrel and the anchor needs to be fitted to prevent soil intrusion into the mandrel.

The penetration of the mandrel is either performed by means of a static load or by dynamic action, using a vibratory or impact hammer. Static installation is preferable in soils sensitive to disturbance.

After withdrawal of the mandrel, the drains should be cut in a way to ascertain good contact with the drainage blanket, preferably about 25 cm above the working platform.

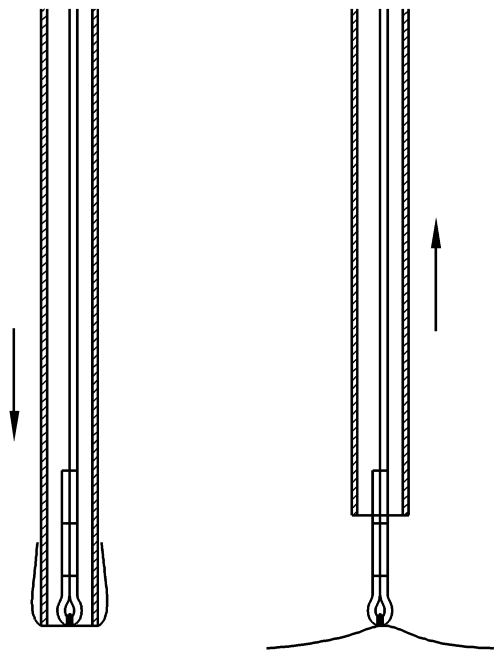


Figure A.4 — Example of band drain anchor

A.4.1.4 Precautions for the drain installation

The tensile strength of the band drain needs to be high enough to prevent breakage of the drains during and after installation. The required tensile strength depends upon the type of execution equipment, installation technique, soil conditions and depth of the drain.

If possible, the mandrel should be filled with water during installation to avoid the band drain from becoming surrounded by air when the mandrel is withdrawn. The presence of air will reduce the filter permeability and the horizontal permeability of the soil surrounding the drain as well as the discharge capacity. A hydrophilic finish on the filter surface improves the affinity for water.

Static installation is preferable to dynamic installation in soils sensitive to disturbance.

The drain installation produces a zone of smear around the mandrel in which the permeability in the horizontal direction for certain types of soil, particularly fine-grained soils with coarser layers, may be considerably reduced.

Nevertheless, in some cases the un-drained shear strength of the soil may be high enough to resist a collapse of the hole created by the mandrel and thus leave an open space between the drain and the soil when the mandrel is withdrawn. This makes it difficult to estimate the effect of smear as well as the nominal drain diameter to be used in the design.

A.4.1.5 Factors influencing the band drain efficiency

Discharge capacity

It is important that the discharge capacity of the drains installed (the amount of water flow per time unit in the vertical direction through the drain under a hydraulic gradient equal to one) is sufficient to achieve the required degree of consolidation in accordance with the design.

The required discharge capacity (see Annex B) depends on the depth of drain installation, the drain spacing (higher with increasing depth of installation and decreasing drain spacing) and the consolidation characteristics of the soil (higher with increasing permeability and compressibility).

The actual discharge capacity of the drains installed in the soil is influenced by the band drain properties, by the drain installation method (including the effects of smear zone, the hole created by the mandrel and the presence of air in the drain) and by the interaction between the soil and the drain (lateral earth pressure against the drain, possible clogging of the filter and/or the core and effect of buckling).

In highly compressible soil (e.g. peat and gyttja) the relative compression, taking place during the consolidation process, may cause buckling or kinking of the drains, which may seriously reduce their discharge capacity, see Figure A.5. Buckling usually takes place in the upper part of the soil. However, the extreme buckling conditions shown in Figure A.5 can be expected only in very deformable soils with vertical strains of the order of 5 0%. This is not the case in ordinary soil and loading conditions, where the vertical strains are typically 10 % to 15 % and buckling phenomena have no influence on the discharge capacity.

Figure A.5 — Buckling and kinking of drain due to very large relative compression of peat

A.4.1.6 Drainage blanket

For the efficiency of the vertical drainage system, an appropriate drainage blanket (a layer of granular material of appropriate thickness and/or an appropriate drainage system of geotextile or geotextile-related products) needs to be installed to eliminate the risk of a build-up of backpressure in the drains by the water squeezed out through the drains (see A.4.5). Backpressure in the drains reduces the hydraulic gradient created between the soil and the drains and prolongs the consolidation process.

The drainage blanket should be protected from frost effects when used in cold regions.

A.4.1.7 Determination of band drain discharge capacity

The discharge capacity of band drains depends on the drain structure and its constituents. It may be determined at the end of the fabrication process by means of tests which account for the main factors influencing the discharge capacity, i.e. lateral pressure against the drain which causes intrusion of the filter into the channels of the core, intrusion of fine soil particles into the channels through the filter, possible clogging of the channels, effects of buckling on the channel area and effect of temperature. These tests are normally included in a quality control procedure and they do not need to be remade for each band drain installation job. The discharge capacity characteristics should be used by the designer and referred to in the drain installation statement (Clause 8).

Discharge capacity of straight band drains

The discharge capacity can be derived from the flow capacity measured according to EN ISO 12958.

NOTE The discharge capacity q_w is the in-plane flow capacity q_p multiplied by the drain width b and divided by the hydraulic gradient i. For common applications, the in-plane flow capacity[3] at the temperature 20 °C can be obtained by applying a correction factor R_T as described in EN ISO 12958.

For applications where higher soil temperatures occur like landfills, dredging sludge depots and in tropical areas, the tests should be executed at the highest soil temperature at the concerning location.

The duration of the test should also be taken into account and a correction factor f_{cr} should be applied to the value of q_p.

The discharge capacity of a drain q_w (m^3/year) at 20 °C is calculated as:

$$q_w = \frac{q_p b R_T}{i f_{cr}} = \frac{\theta b R_T}{f_{cr}}$$

where

q_p in-plane flow capacity (m2/year);

b drain width (m);

i hydraulic gradient;

$R_T = 1,763 / \left(1 + 0,03771T + 0,00022T^2\right)$ where

T temperature in °C;

θ transmissivity[4] (m2/year);

f_{cr} creep factor.

Two types of testing devices for determining the discharge capacity according to EN ISO 12958 are presented in Figure A.6. In apparatus number 1, the specimen is covered on both sides by closed-cell foam rubber with a thickness of 10 mm. The membrane in apparatus number 2 is made of latex with a maximum thickness of 0,35 mm.

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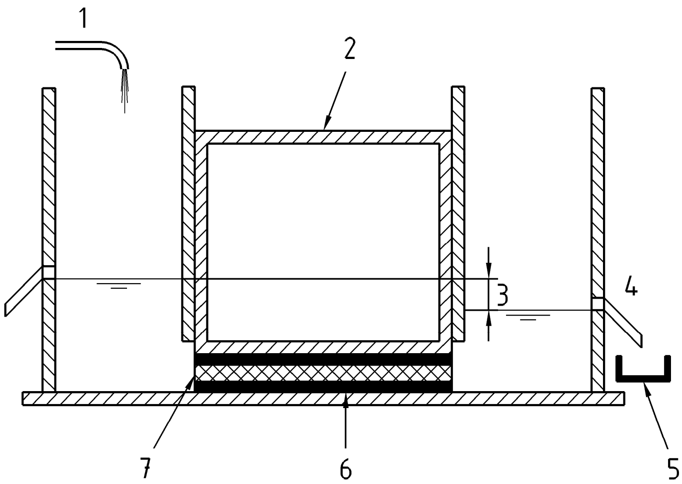
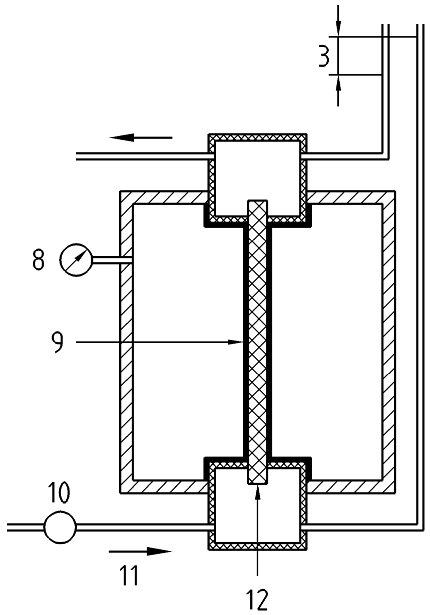
[3] The volumetric flow rate of water and/or liquids per unit width of the drain at defined gradients in the plane of the drain.

[4] In-plane laminar water flow capacity of a drain expressed at a hydraulic gradient equal to 1.



a) Apparatus number 1



b) Apparatus number 2

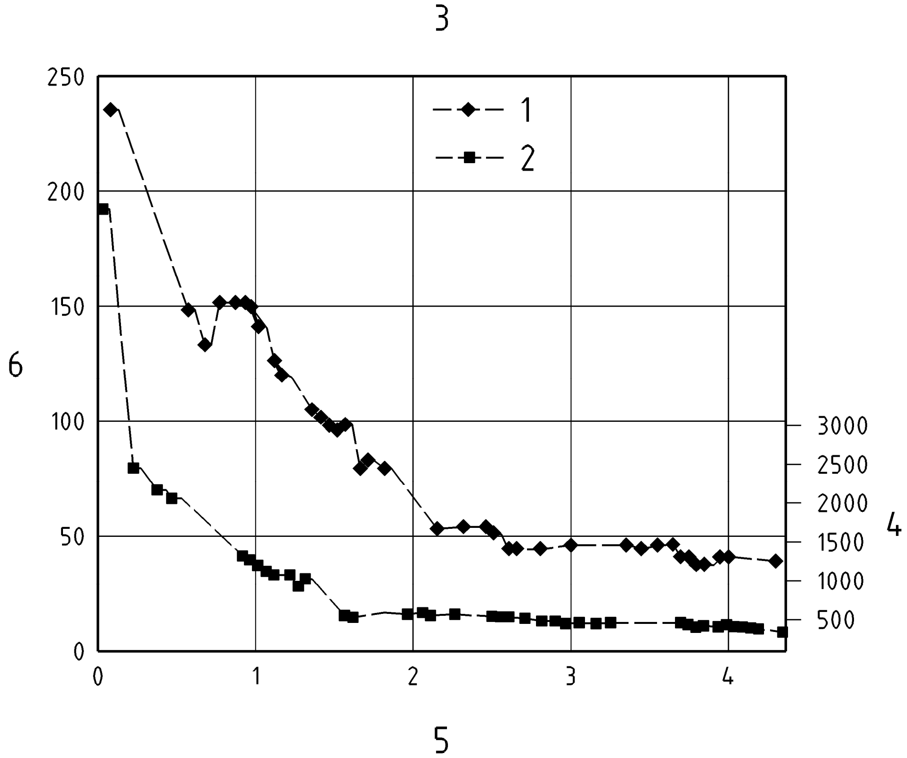
Key

1 water supply
2 load
3 head loss
4 over flow
5 water collection
6 foam
7 sample length 300 mm
8 manometer
9 rubber membrane
10 flow meter
11 flow direction
12 sample height 350 mm

Figure A.6 — Testing devices for determination of discharge capacity [(Figure A.6 a) apparatus number 1 and (Figure A.6 b) apparatus number 2, according to EN ISO 12958]

The duration of the discharge capacity test will influence the in-plane water flow capacity due to creep of the filter, which causes an intrusion of the filter into the channel system, thereby reducing the discharge capacity, see Figure A.7. The creep factor f_{cr} mention above is used to estimate the value of the stabilized discharge capacity from the result of a test of shorter duration. It depends on the testing apparatus and should be determined or checked for each testing device.

The variations of discharge capacity of a band drain with time, as measured in the two different testing devices, are presented in Figure A.7.



Key

1 Apparatus 1 (ASTM)

2 Apparatus 2 (Delft)

3 Discharge capacity straight at 30 °C, 500 kPa

4 Discharge capacity, m^3/year

5 Time, weeks

6 Discharge capacity. cm^3/s

Figure A.7 — Creep effect on discharge capacity observed in the duration of a discharge capacity test [6]

Based on experience, the creep factors given in Table A.1 are proposed for the two testing apparatuses shown in Figure A.6. If other types of testing device are used, creep factors should be determined based on measured data similar to those shown in Figure A.7.

Table A.1 — Creep factors (values if historical data are missing)

Testing period Days	**Creep factor f_{cr}**	
	Apparatus 1	Apparatus 2
2	10	5
7	8	3
30	3	1

Where appropriate, a 30 days in-plane flow test can be made to determine the creep factor for the discharge capacity of each type of drain. For common applications, the values based on the tests required for CE-marking can be used. If the tests are carried out with apparatus number 1 the measured creep factors always have to be multiplied by 3.

The discharge capacity tests should be performed with a hydraulic gradient of 0,1 under, respectively, the static pressures 20 kPa, 100 kPa and 200 kPa, possibly also under higher static pressure with regard to the specific design conditions.

These specific design conditions depend on depth of drain installation, load of respectively fill, temporary surcharge and/or vacuum. The testing pressure (kPa) can be calculated from the relation:

$$\sigma_{\mathrm{t}} = f_{\mathrm{m}} K_{\mathrm{o}} \sigma'_{\mathrm{v}}$$

where

σ_t external applied pressure during testing;

f_m partial factor for testing pressure (1,2, see Annex B, B.4.1.3);

K_o coefficient of earth pressure at rest (0,65 to 0,75 for soils with high plasticity index);

σ'_v the in-situ effective overburden pressure at the depth of installation plus the vertical stress increase caused by fill, temporary surcharge and/or vacuum at the depth of installation.

Discharge capacity of buckled band drain

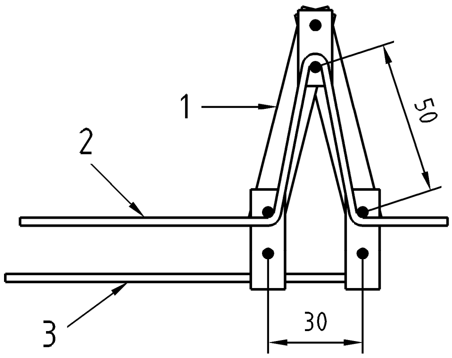
The influence of buckling on the discharge capacity of a given band drain should be considered in the design when the estimated vertical strain of the soil around the drain is high (typically more than 20 %).

The discharge capacity test on buckled drains should be performed with a hydraulic gradient of 0,1 under, respectively, the static pressures 20 kPa, 60 kPa and 120 kPa, possibly also under higher static pressure with regard to the specific design conditions. This can be done, for example, by means of the apparatus shown in Figure A.8, which is suitable for device number 2, Figure A.6.

The test report should contain the information given in Clause 9 of EN ISO 12958:1999. Moreover, it is recommended to present the discharge capacity test results as shown in Table A.2, both for straight and buckled drains.

NOTE Since the test is made with a very sharp angle of the band drain, without measurements of the intermediate angles, the discharge capacity of the buckled drain serves as an index of the influence of vertical strains on the discharge capacity and should be used as such by the designer.

Dimensions in millimetres



Key

1 rod A

2 drain sample

3 guide rod

Figure A.8 — Example of apparatus to test discharge capacity of buckled drain [12]

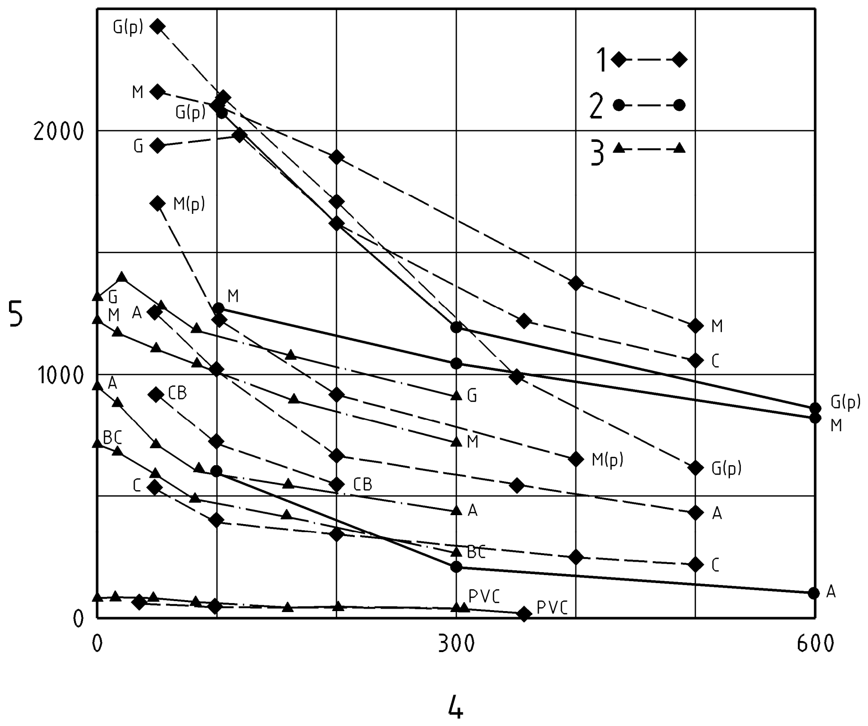
Table A.2 — Discharge capacity q_w and q_{wb} (in m^3/year) for various static pressures (in kPa) and a hydraulic gradient i = 0,1

Specimen	Straight drain				Buckled drain			
	$q_{w(20/0,1)}$	$q_{w(100/0,1)}$	$q_{w(200/0,1)}$	$q_{w(XXX/0,1)}$	$q_{wb(20/0,1)}$	$q_{wb(60/0,1)}$	$q_{wb(120/0,1)}$	$q_{wb(XXX/0,1)}$
1								
2								
3								
Average								

Discharge capacity of band drains in contact with the soil

The discharge capacity values obtained in the kind of test apparatuses shown in Figures A.6 and A.7 may differ from those obtained if the drain is surrounded by the soil into which it is installed. Therefore, the values obtained in the testing chamber serve as an index of what can be estimated in the field. Obviously, the discharge capacity will be progressively reduced by increasing filter intrusion into the channels of the core due to increasing effective lateral soil pressure during the consolidation process.

Discharge capacity tests on band drains installed in soil on a laboratory scale and subjected to increasing effective lateral stress have resulted in the values presented in Figure A.9. In the Italian tests [24] the drains were tested at full scale. In the Swedish tests [19] and in the Japanese tests [26] the drains were tested with reduced width (40 mm and 30 mm, respectively).

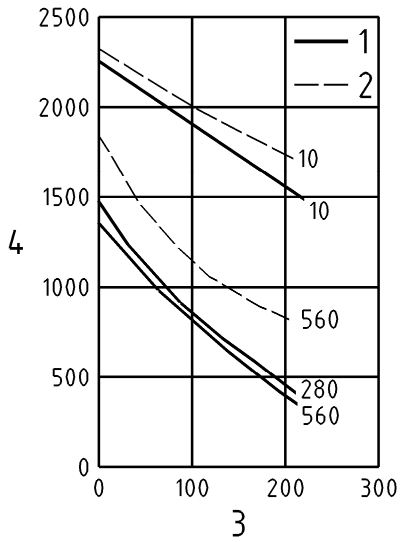


Key

1 small-scale tests (Sweden)	BC = Bando Chemical
2 large-scale tests (Italy)	C = Colbond
3 small-scale tests (Japan)	CB = Castle Board
4 effective lateral pressure, kPa	G = Geodrain
5 discharge capacity, m^3/year	M = Mebradrain
	PVC = PVC Drain
A = Alidrain	(p) indicates filter sleeve of specially prepared paper

Figure A.9 — Results of discharge capacity tests for different band drains carried out on a laboratory scale. Drains enclosed in soil [19][24][26]

Ageing of the filter in the soil can be expected due to bacteriological activity or fungi attacks. The result of an investigation of ageing effects on discharge capacity reported by [29] is shown in Figure A.10. The tests were carried out on sampled drains pulled out of peat and gyttja after different lengths of time after installation.



Key

1 peat

2 gyttja

3 effective lateral pressure, kPa

4 discharge capacity, m^3/year

Figure A.10 — Influence on discharge capacity of time-dependent filter deterioration [29]. The number of days that the drains were left in the soil after installation is shown against each curve. Full lines represent drains installed in peat, broken lines drains installed in gyttja

The clogging of the filter and/or core by fine clayey or colloidal particles is usually prevented by imposing a maximum value of the characteristic opening size O_{90} of the filter (as defined for a geotextile by EN ISO 12956), which is based on experience and laboratory tests and should be adapted to the particle size distribution of the soil.

A.4.2 Prefabricated cylindrical drains

A.4.2.1 Types of drains

A prefabricated drain consists of a tubular core, typically 50 mm in outer diameter and 45 mm in inner diameter, made of annular-corrugated perforated plastic, resistant to crushing, shocks, rapid tension and ageing, surrounded by a filter sock made of non-woven geotextile.

A.4.2.2 Method of installation

The prefabricated cylindrical drains are installed inside a hollow, cylindrical mandrel with an external diameter of typically 100 mm. The mandrel, which is normally pushed into the soil by static loading, needs to have sufficient rigidity. An anchor plate is fixed to the drain tip before installation and prevents soil from intruding into the mandrel during installation.

Upon withdrawal of the mandrel the drains are cut in a way to ascertain good contact with the drainage layer, preferably 25 cm above the working blanket.

A.4.2.3 Factors influencing drain efficiency

Cylindrical drains are deemed to have sufficient discharge capacity for any vertical drain project. The only recognised factor that may limit their efficiency is the existence of a smear zone around the drain, created by the insertion of the mandrel. According to experience, the consolidation process can be analysed theoretically, disregarding the effect of mandrel installation and smear, by assuming the drain diameter equal to 50 mm.

A.4.2.4 Other fields of application

Annular-corrugated perforated cylindrical drains consisting of a pipe of high-density polyethylene, surrounded by a non-woven geotextile filter, have been developed in USA for reduction of liquefaction potential in earthquake regions [44].

A.4.3 Sand drains

A.4.3.1 Types of drains

Sand drains usually consist of sand columns, 18 cm to 50 cm in diameter, which are installed into the soil and are in direct contact with the soil.

The sand used for sand drains should preferably fall within the grain size limits shown with cross-ruled area in Figure A.11. However, there are many case histories where sand drains have functioned successfully having wider grain size distributions, falling outside the limits of the cross-ruled area. The grain size distribution of the sand used in these case histories falls within the limits given by the outer unbroken lines in Figure A.11.

A.4.3.2 Methods of installation

Sand drains are either installed by so-called non-displacement methods or by so-called displacement methods.

The *non-displacement methods* comprise shell and auger drilling, powered auger drilling, water jetting, flight augering and wash boring. The *auger method* consists in screwing the auger down to the required depth, then pulling it upwards while sand is transferred to the hole below the auger tip through the axis. The hollow auger method consists of screwing the auger down to the required depth and then pulling it upwards while sand is transferred to the hole below the auger tip through its hollow axis. In the *water jetting method*, the hole, which will be filled with sand, is first created by water jetting at a pressure and flow adjusted to the soil condition. Sand is then poured into the hole without compaction.

The *displacement methods* comprise mandrel or vibro installation methods. In the *mandrel method* a hollow mandrel with a flap at its lower end is driven into the ground. As the mandrel is withdrawn, the flap opens and water-saturated sand filled into the mandrel thereby creates the sand drain. In the *vibro installation method*, a mandrel with or without a flap on its lower end is inserted into the soil to the required depth by means of a top vibrator mounted on the mandrel. After installation the vibrator is continuously pulled upwards without compacting the sand fill exerting from the lower end of the mandrel.

Alternatively, the drains are installed by means of a depth vibrator, which after installation is continuously pulled upwards without compacting the sand fill.

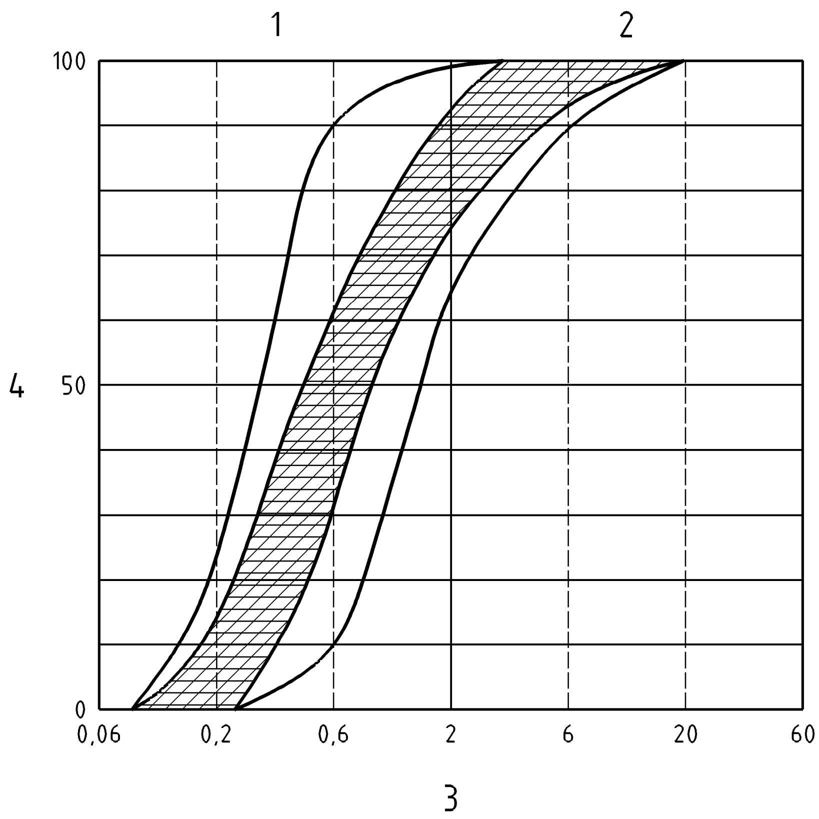
A.4.3.3 Efficiency of sand drains

<u>Continuity and diameter</u>

The continuity of the sand drains is of paramount importance and the diameter of the drains installed should agree with the design requirements. Continuity may be at risk when the hole is first created and thereafter filled with sand. This is the case in the water jetting method and in the non-hollow auger method. Continuity and fairly constant drain diameter are assured in the methods where sand is poured into a tube driven into the soil and in the hollow auger method.

Discharge capacity

The discharge capacity of sand drains with the preferable grain size distribution given in Figure A.11 will vary from about 800 m^3/year (25 cm^3/s) for sand drains with diameter 0,18 m to about 4 000 m^3 /year (127 cm^3/s) for sand drains with diameter 0,4 m. The permeability requirements of the sand drains depend on the permeability of the surrounding soil and the depth of drain installation (see Annex B, Figures B.2 to B.5). These values are higher than is required at vertical drainage sites. Therefore, sand drains can normally be considered to be unaffected by well resistance.



Key

1 sand

2 gravel

3 grain size d, mm

4 content of grains < d in wt, % of total mass

Figure A.11 — Grain size limits of granular material to be used in sand drains

The values of discharge capacity described are based on the condition that the sand used for the sand drains is water-saturated during the installation process. The intrusion of air into the sand strongly reduces the discharge capacity.

Interaction with surrounding soil

Closed-end tube (mandrel) installation causes lateral displacement of the soil around the tube accompanied by an overall disturbance effect and by a zone of smear where horizontal layers with higher permeability are distorted vertically. The installation may also create vertical cracks in the soil surrounding the drain, which become filled with sand [37].

Installation by the jetting method causes a minimum of soil disturbance due to installation. It generally creates a hole larger than the nominal diameter and is therefore considered as particularly efficient. Jetting pressure and water flow need to be appropriate to the soil conditions.

During installation the risk of necking or possible failure of the drain because of insufficient support from the surrounding soil should be monitored. This can be done by continuously checking the amount of sand filled into the drains. Necking can be a serious problem in quick clays.

A.4.3.4 Geotextile enclosed sand drains

The risk of necking of sand drains can be avoided by enclosing the drains in geotextile. Originally this was done only for small-diameter drains but nowadays the method is also used for drains with larger diameter, as mentioned above.

The discharge capacity of sand drains with the preferable grain size distribution given in Figure A.11 will be about 350 m^3/year (11 cm^3/s) for sand drains with diameter 0,12 m. The material to be used in small-diameter fabric drains, for example sand wicks, should be coarser than shown in Figure A.11 in order that the requirements on discharge capacity will be fulfilled.

A.5 Drainage blanket and working platform

For the efficiency of the vertical drainage system an appropriate drainage blanket (a layer of granular material of appropriate thickness and/or a geotextile or geotextile-related products) should be installed. The consolidation settlement causes a depression of the central part of the drainage blanket. Temporary wells for removing drained water from the drainage blanket may therefore be required, especially in cases where the width of the drainage blanket is large. Protection of the drainage blanket against frost effects should be considered when relevant.

The permeability of the drainage blanket shall be high enough not to cause backpressure in the drains in the way shown in Figure A.12.

Figure A.12 — Example of drainage blanket of granular material with insufficient permeability, showing water trapped in the drainage blanket, implying backpressure in the drain

The execution of a vertical drainage project requires the presence of a working platform with an upper surface suitable to facilitate the vertical installation of the drains. The working platform needs to be capable of carrying the installation equipment. The presence of pockets and lenses of soft soil in the working platform can significantly reduce the local bearing capacity and result in overturning of the installation rig. The placement of a geotextile separation layer underneath the working platform may be a way of avoiding the risk of heterogeneities in the working platform.

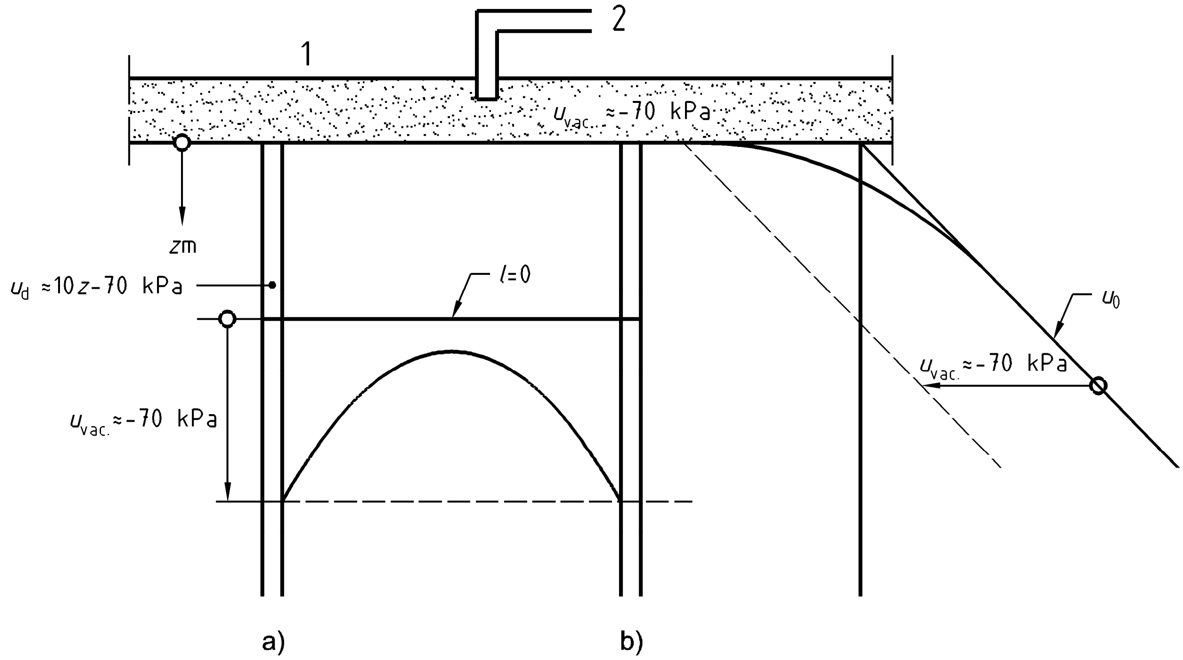
A.6 Loading

The loading operation usually consists of placing a surface load on top of the drainage blanket. This is a critical phase of vertical drainage projects. Loading needs to be carried out in such a way that the stability of the ground is not endangered. Therefore, the unit weight of the fill used for loading has to be defined and controlled. The un-drained shear strength of the soil may be detrimentally affected, not only by the drain installation in itself, but also by the loading operation if carried out with heavy equipment. In most cases, it is important that the filling operation is monitored by settlement and pore pressure observations.

If the shear strength of the soil is too low to permit placement of the fill to full height, loading berms are required. Alternatively, loading has to be carried out stepwise, followed by investigation of the gain in shear strength and dissipation of excess pore water pressure during the consolidation process, required to permit the placement of the next load-step, and so on. In the case of stepwise loading the specified thickness of each embankment layer need to be checked in order to avoid excess loading and consequential failure.

Groundwater lowering in permeable strata in connection with the drains can also be used as an alternative to, or in combination with, external loading.

At sites of drain installation where the stability conditions are unsatisfactory, the surface load can be replaced or augmented by the vacuum method, Figure A.13. In this case the drainage blanket is overlain by an airtight cover and sealed hermetically along its outer borders. The drainage blanket is connected to a vacuum pump, which produces under-pressure in the drains in relation to the pore water pressure in the soil and results in consolidation [9] [10]. The under-pressure achieved by the vacuum method in this case is maximum 70 kPa to 80 kPa.



Key

u_{d}. = pore water pressure in the drains

u_{vac} = under-pressure (assumed equal to a vacuum of 70 % of atmospheric pressure)

a) pore pressure dissipation caused by the drains

b) pore pressure dissipation without drains

1 airtight cover

2 to vacuum pump

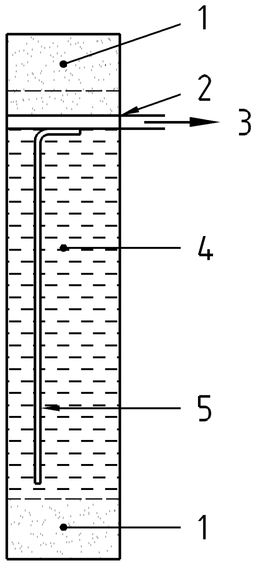
Figure A.13 — Sketch of the vacuum method and its effect on pore water pressure, both for horizontal pore water flow towards the drains (a) and for vertical pore water flow between the drains (b)

Another method to achieve vacuum [40] is shown in Figure A.14. In this system, the band drains are cut at the bottom of ditches, excavated to a depth of 1 m below the bottom of the working platform along each row of vertical drains. Each row of band drains is then connected to a horizontal circular drain, which is covered with a strip of liner. The cylindrical drains are connected to a vacuum pump and the under-pressure thus achieved in the cylindrical drains is transferred to the vertical drains.

An advantage of this system is that an airtight cover over the total area is not needed as in the conventional system. A disadvantage is that no under-pressure is achieved in the upper 1 m layer. In this case the maximum under-pressure achieved is about 50 kPa.

A.14a

A.14b



A.14c

Key

1 sand
2 liner strip
3 to vacuum pump
4 clay
5 vertical drains

Figure A.14 — Installation of horizontal cylindrical drain (left) and its connection to the vertical drains

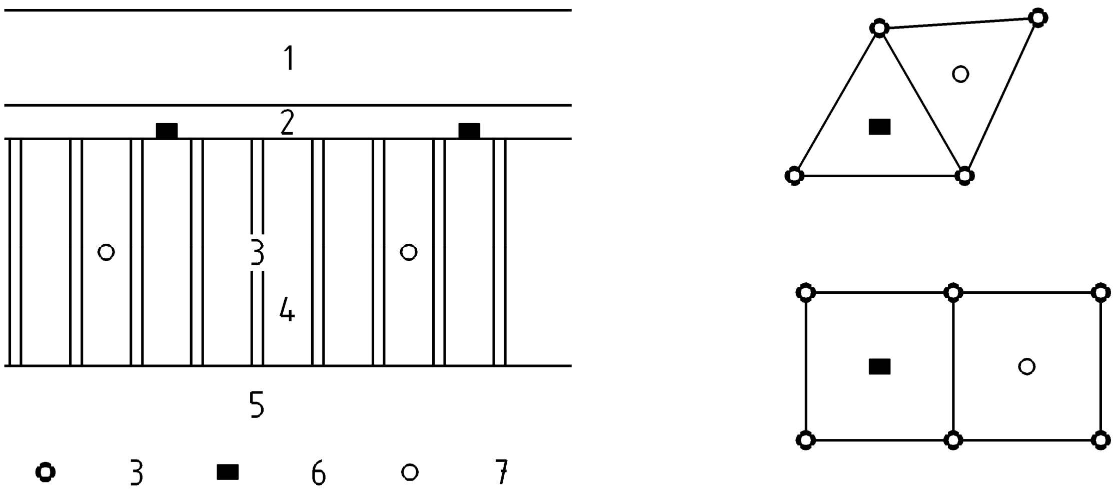
A.7 Monitoring

The effect of vertical drainage should be monitored by both settlement and pore pressure measurements. The measured values are used to check the actual rate of consolidation and the assumptions made in the design. It is important that the monitoring system is installed in due time before the installation of the drains, both with regard to the effect of drain installation itself (excess pore pressure due to disturbance caused by drain installation and its possible negative influence on stability) and with regard to the interpretation of the results of observation subsequently achieved.

The aim of soil improvement by vertical drainage is generally to prevent unacceptable settlement from taking place. Therefore, settlement observations are a necessary ingredient in the monitoring system.

Excess pore pressure observations by means of piezometers installed at different depths is doubtless the most appropriate way of checking that the degree of consolidation has reached the set level according to the design. The piezometers should be placed in the centre between the drains where the rate of consolidation is a minimum. However, the interpretation of the results of pore pressure measurements can be quite intricate. The results will depend on the position of the piezometer in relation to the drain (which may differ from intended position), the piezometer (the filter tip) will move downwards in the course of consolidation, the results may be affected by pore back pressure from the surroundings, gas evolution may give erroneous results etc. Moreover, the pore pressure situation after completed consolidation may not revert to its original equilibrium condition. In spite of the problems, pore pressure measurements are an important part of the monitoring system and the conclusion to be drawn about the result of soil improvement achieved should be based on both settlement and pore pressure observations.

Typical locations for observations of settlement and pore pressures for a case with homogeneous ground of limited thickness are shown in Figure A.15 and for a case with stratified ground in Figure A.16. The number of measurement profiles depends on the extent of the site and the thickness and layering of the compressible layers that are treated by vertical drainage.



Key

1 embankment

2 drainage blanket and working platform

3 vertical drain

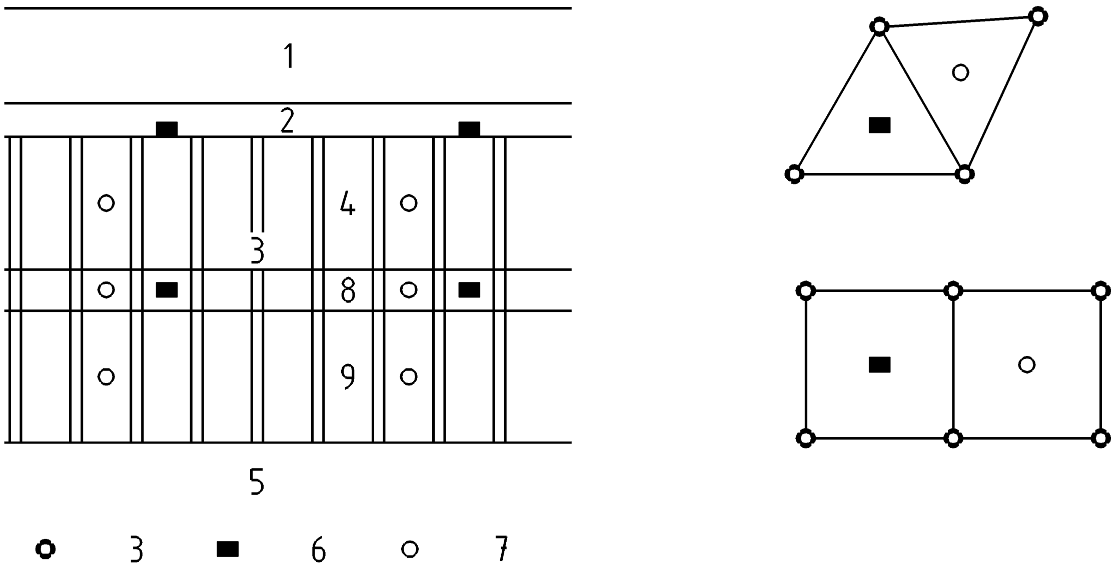
4 compressible soil

5 underlying permeable layer

6 settlement gauge

7 piezometer

Figure A.15 — Typical instrumentation for monitoring the efficiency of vertical drainage (simple case)



Key

1 embankment
2 drainage blanket and working platform
3 vertical drain
4 compressible soil
5 underlying permeable layer
6 settlement gauge
7 piezometer
8 permeable sand layer
9 compressible soil

Figure A.16 — Typical instrumentation for monitoring the efficiency of vertical drainage (site with different layers)

In practice, one needs to consider the degree of consolidation achieved in the soil layers having the lowest coefficient of consolidation (usually having also the most unfavourable compression characteristics). In homogeneous soil condition, the lowest degree of consolidation is achieved where the effect of vertical one-dimensional consolidation is minimal, i.e. in the middle of the clay layer. If the discharge capacity of the drains is too low this will strongly influence the degree of consolidation achieved with increasing depth of installation. Using only surface settlement observations as a means of checking the degree of consolidation achieved throughout the soil layer may consequently lead to wrong conclusions.

Annex B
(informative)

Aspects of design

B.1 General

This annex covers some specific aspects of the design of vertical drainage systems, including the evaluation of soil characteristics and influence of drain characteristics, drain pattern and depth of drain installation. It does not cover the detailed principles or methods of geotechnical design, for which reference should be made to EN 1997-1 and EN 1997-2.

The scope of the application of vertical drainage is to handle and solve problems associated with the following aspects:

1) consolidation settlement in low-permeability soils (resulting from surface loading or groundwater lowering);

2) stability (of structures and embankments).

As a result of soil improvement by vertical drainage, the effects of dynamic and cyclic loading (e.g. in seismic regions) can be reduced as well as the effects of vibrations on structures and human beings. Vertical drainage can also be used for remediation of contaminated ground and for mitigation of liquefaction potential.

Vertical drainage design encompasses two phases, functional design and process design:

1) in the first phase, the need for vertical drainage needs to be quantified. This phase of functional design defines the loading and drain spacing which will produce the desired effects on rate of consolidation and settlements, and eventually on the un-drained shear strength of the soil. The objectives are linked up with improving the ground by preloading and enabling staged construction of an embankment, and also with creating satisfactory drainage paths for pore water in the case of liquefaction;

2) In the second phase the method of drain installation and their functioning in practice has to be designed. This phase of process design accounts for effects of drain installation on the ground, for the geometry, the nature and the dimensions of the drains, for possible buckling in case of excessive strains in some soil layers etc.

B.2 Design process

Vertical drainage may be used for different purposes. However, the process of designing vertical drainage always includes the operations listed in Figure B.1: the objective (design basis) and the ground properties (first row of boxes) interact with the settlement and stability analyses to satisfy the requirements put on the effect of the drains, i.e. to reach a given degree of global and/or local consolidation within a specific period of time.

Ground treatment by vertical drainage and the associated loading shall be designed and executed in such a manner that the structure, supported by the treated ground, during its intended life and with an appropriate degree of reliability and cost-effectiveness, will remain fit for the intended use and sustain all actions and influences that are likely to occur. This requires that the serviceability and ultimate limit states are satisfied.

The requirements for the serviceability and ultimate limit states shall be specified by the client. The design shall be in accordance with the requirements put forward in EN 1997-1. The observational method, which involves adapting the design in a planned manner, can be an important part of the design.

The design shall take into account the combinations of loads that could occur during construction and service. It shall account for the known effect of the drain installation on the properties of the ground.

The installation of vertical drains may induce excess pore water pressure and cause a short-term reduction of the un-drained shear strength.

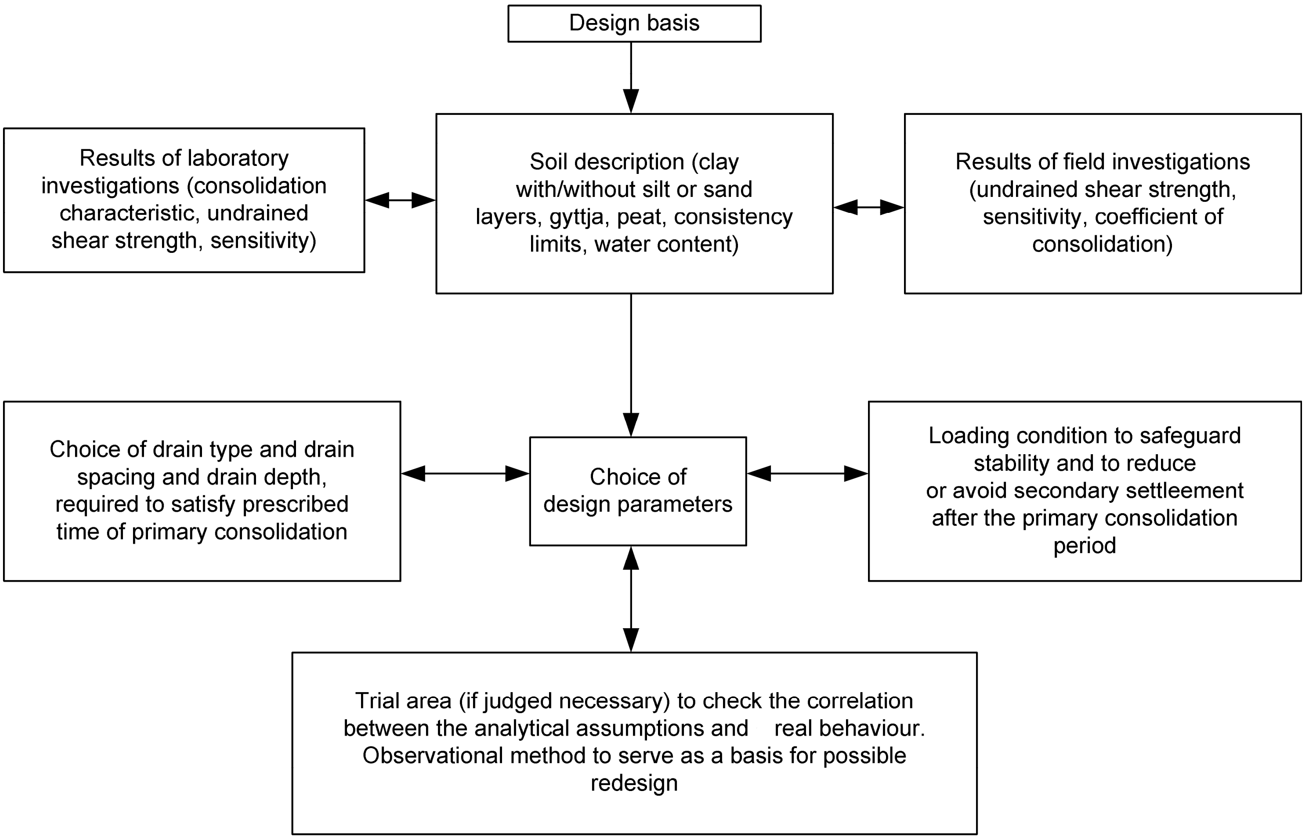


Figure B.1 — Chart of design process, including laboratory and field investigations, functional design and field trials

B.3 Investigations for vertical drainage

B.3.1 General

The subsoil characteristics are usually determined by means of field investigation methods (e.g. cone penetration tests, field vane tests and pore pressure observations at various depths) in combination with sampling for laboratory analysis. The pore water pressure distribution with depth forms the basis for evaluation of the effective overburden pressure distribution with depth. This information is required to determine whether or not the soil is overconsolidated or normally consolidated. However, one needs to realise that the pore water pressure may vary considerably with time of the year and amount of precipitation. Occasional high pore water pressure, which reduces the magnitude of effective overburden pressure, can give a false impression of overconsolidated soil.

The testing should be carried out in compliance with EN 1997-2. The soil identification and classification, which is based on the results of the soil investigation, shall comply with EN ISO 14688.

The penetration resistance of the soil should be investigated to provide information for selecting the capacity of installation rigs.

B.3.2 Laboratory investigations

The consolidation and settlement parameters are conventionally determined by oedometer tests on undisturbed soil samples, taken by means of high-quality piston samplers. The results of conventional oedometer tests yield values of the compression modulus, the preconsolidation pressure and the coefficient of consolidation in vertical pore water flow. For determination of the coefficient of consolidation in horizontal pore water flow by oedometer tests, allowance for radial drainage needs to be made.

Laboratory testing also includes determination of the un-drained shear strength and sensitivity of the soil as well as unit weight, water content and index testing.

B.3.3 Field investigations

Field investigations normally comprise determination of the un-drained shear strength by field vane tests and/or cone penetration tests. The coefficient of consolidation and the permeability in horizontal pore water flow can be evaluated from cone penetration tests with a pore pressure device (CPTU). This is done by intermittent sounding accompanied by a study of the excess pore pressure dissipation caused by the sounding operation [50] [51] [49] [33].

Possible contamination of the pore water can be investigated by sampling of pore water at various depths [44].

B.4 Aspects of design

B.4.1 Settlement

B.4.1.1 Total settlement

The design related to the soil deformations caused by the loading operation shall be in accordance with EN 1997-1.

The question of whether the soil is normally consolidated or overconsolidated is of great importance for the correctness of the settlement analysis and for whether the use of vertical drainage is adequate or not. A correct determination of the preconsolidation pressure is of paramount importance. The use of vertical drains in a case where the effective stresses induced by the loading operation are below the preconsolidation pressure of the soil is counter-productive since the installation of the drains may cause disturbance effects that result in an increase in settlement. Thus, vertical drainage should only be utilised in cases where the preconsolidation pressure will be exceeded by the stresses induced by the loading operation.

The soil deformations caused by external loading include both vertical and horizontal displacements, whose relative magnitudes depend on the loading condition, the shear strength of the soil and the ratio of the width of loading to the depth of the soil layer. Especially if test areas are used as a basis for design, the widths of which are small in comparison with the depth of the soil layer, horizontal displacements may contribute considerably to the vertical settlement observed. In such cases vertical inclinometers, placed along the borders of the test area, provide information about the influence on the vertical settlement of horizontal deformations.

In the analysis of the total settlement obtained after completed consolidation, the influence on soil deformation properties of possible disturbance effects caused by drain installation should be considered. The disturbance effects depend very much on the method of drain installation, the size and shape of the mandrel and the structural features and un-drained shear strength of the soil. To ensure the accuracy of the settlement analysis it is important that the average unit weight of any fill material used for loading is given in the specification. It is also necessary to take into account the load reduction due to buoyancy effects if part of a surcharge becomes submerged during the consolidation process.

The total primary consolidation settlement can be estimated from the settlement gradually developed during the consolidation process. For example, according to Asaoka [1] [2], the relation established between the settlements observed at equal time intervals Δt can be used to assess the final primary consolidation settlement.

The settlement achieved by the use of the vacuum method (see Annex A) is governed by the effectiveness of the sealing system. Normally, a maximum of 70 % to 80 % vacuum can be achieved, resulting in an effective stress increase of 70 kPa – 80 kPa. The ratio of effective vertical stress increase to horizontal effective stress increase will differ from the corresponding ratio obtained by external loading. In consequence, the increase in shear strength during the consolidation process will differ from that obtained by external loading.

Temporary overloading can reduce secondary creep settlement following upon the primary consolidation period. The required temporary overloading depends on the deformation characteristics of the soil and on the secondary consolidation settlement requirements. A temporary overload of at least 0,25 to 0,35 times the permanent design load, maintained until termination of primary consolidation, is usually enough to significantly decrease creep settlement after the temporary load is removed [14] [20].

B.4.1.2 Rate of consolidation settlement

<u>*Design assumptions*</u>

For the analysis of the rate of consolidation settlement, the drainage characteristics have to be identified (diameter D of hypothetical soil cylinder dewatered by each drain, drain diameter d_w, diameter of zone of smear d_s, discharge capacity q_w) as well as the soil consolidation parameters (coefficient of consolidation c_h, permeability in horizontal pore water flow in undisturbed soil k_h and in the zone of smear k_s), see Figure B.2.

As can be seen from Figure B.2, the value of D depends on the drain installation pattern (1,05 times the drain spacing for drains placed in equilateral triangular pattern; 1,13 times the drain spacing for drains placed in equilateral square pattern). The drain diameter d_w for a band drain can be assumed equal to that of a cylindrical drain with the same circumference as the band drain, i.e. $d_\mathrm{w} = 2\ (\mathrm{b} + \mathrm{t})/\pi$ where b is the width and t is the thickness of the band drain [17][5].

The consolidation parameters of the soil are usually based on the results of oedometer tests where excess pore water dissipation takes place in the vertical direction. This differs from the real case with vertical drains where excess pore water dissipation mainly takes place in the horizontal direction. The difference between the oedometer case and reality becomes important where seams or layers exist with higher permeability than the main body of the soil. For the determination of the coefficient of consolidation in horizontal pore water flow, oedometer tests provided with radial drainage, or CPTU tests as described in B.3.3, can be used.

In some site conditions, the drain installation procedure may increase the soil compressibility and/or decrease the coefficient of consolidation and the permeability of the soil. It may also create excess pore water pressure in the soil. Such perturbations of the initial soil conditions should be considered in the design. When relevant, it is important that monitoring equipment is installed in due time before the drains are installed so that the disturbance effects can be registered and duly considered.

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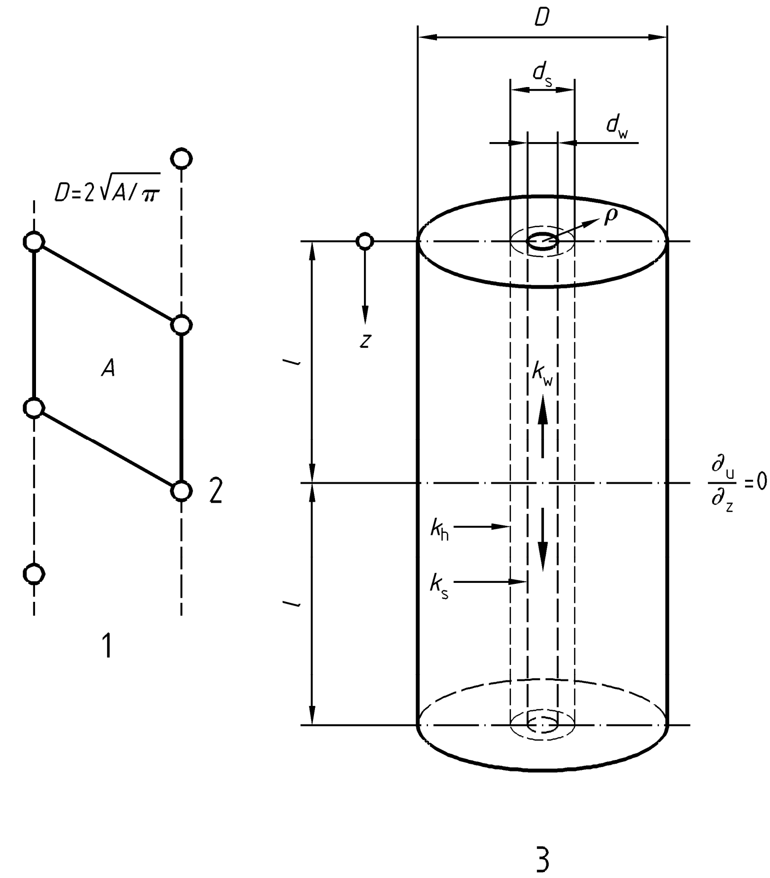
5) According to [45] the value of the equivalent diameter should be $d_{w,\mathrm{eq.}} = (b+t)/2$.

The insertion of the mandrel into the soil during drain installation also creates a zone of smear where horizontal layers are distorted in the vertical direction, followed by a reduction in horizontal soil permeability. The width and the characteristics of the zone of smear are a function of the installation method. The influence of the smear zone should also be considered with due account of the hole created by the mandrel during drain installation. The dimensions of the mandrel are temporarily much larger than those of the drains.

The mandrel used for band drain installation is not usually filled with water during the installation process. In consequence a cushion of air may be left between the drain and the surrounding soil after the mandrel is withdrawn. A cushion of air causes a negative effect on the consolidation process, similar to that of smear. It is taken into account in the choice of smear zone parameters based on experience. The installation may also cause vertical cracks around the mandrel, which in the case of sand drains become filled with sand intruding from the drains [37]. In cases where the un-drained shear strength of the soil is high, the installation may leave an open hole around the drain, which has a favourable effect on the discharge capacity.

Investigations of the characteristics and extent of the zone of smear caused by drain installation have been performed by e.g. [4], [8], [23], [34] and [42].



Key

1 plan

2 drain

3 perspective

Figure B.2 — Soil cylinder dewatered by a drain

Method of analysis

Theoretically, whatever pattern is used, each drain is considered to dewater a hypothetical soil cylinder whose cross-sectional area equals the cross-sectional area enclosed by four neighbouring drains, Figure B.2. The most efficient way of utilising the capacity of vertical drains for the purpose of speeding up the consolidation process is to install the drains in an equilateral triangular pattern. The consolidation process is mainly governed by pore water flow in the radial direction towards the drain and to a lesser extent by pore water flow in the vertical direction between the drains. Two methods of analysis exist, the so-called “free strain analysis” and the “equal strain analysis”. As shown by Barron [3] the difference in results regarding average consolidation process obtained between the two methods of analysis is negligible. Therefore, because of its simplicity the equal strain analysis, Equation (1), has become routine [18], [28], [32], [35], [52].

In the methods of analysis used for determination of the influence of well resistance (limited discharge capacity), the consolidation characteristics of the soil are generally assumed to be constant throughout the soil layer. The influence of layers with different consolidation characteristics has been analysed by Onoue [41].

Another conventional assumption in analysis is the validity of Darcy’s law. Experience from a number of field tests [16], [21] and [46] and from laboratory tests on permeability [16] and [13] has shown that there is a deviation from Darcy’s law at small hydraulic gradients. Consolidation equations valid for both Darcian and non-Darcian flow have been developed [22].

The basic theory of vertical drainage used in routine analysis of most of vertical drainage projects was published by Hansbo [18] as an extension of Barron’s theory [3] for the case of drains with limited discharge capacity. Accordingly, the rate of consolidation follows the relation:

$$\bar{U}_{\text{h}} = 1 - \exp\left(\frac{8c_{\text{h}}t}{\mu D^2}\right)$$

where, omitting terms of minor significance,

$$\mu = \frac{D^2}{D^2 - d_{\text{w}}^2}\left[\ln\left(\frac{D}{d_{\text{s}}}\right) + \frac{k_{\text{h}}}{k_{\text{s}}}\ln\left(\frac{d_{\text{s}}}{d_{\text{w}}}\right) - \frac{3}{4}\right] + \frac{k_{\text{h}}}{q_{\text{w}}}\pi z[2l - z]$$

An important parameter in vertical drain analysis is the discharge capacity of the drains q_{w}, i.e. the amount of water flow per time unit that can take place in the vertical direction through the drain at a hydraulic gradient equal to one. (In EN 10318, the discharge capacity is equal to the transmissivity times the width of the drain.) Drains have appeared on the market with insufficient discharge capacity when installed to great depth. If the drains have insufficient discharge capacity, the degree of consolidation obtained by drain installation in homogeneous soil decreases with depth of installation.

The ratio of the time of consolidation t, considering the effect of well resistance (limited discharge capacity), and the time of consolidation t_1, neglecting the effect of well resistance, can be expressed by the relation $t = t_1$ $(1 + \Delta t)$, where the delay Δt in time of consolidation follows the relation:

$$\Delta t = \frac{\pi z(2l - z)k_{\text{h}}\left(D^2 - d_{\text{w}}^2\right)}{q_{\text{w}}D^2\left[\ln(D/d_{\text{s}}) + (k_{\text{n}}/k_{\text{s}})\ln(d_{\text{s}}/d_{\text{w}}) - 3/4\right]}$$

The most unfavourable case with regard to discharge capacity requirements is obtained when $k_s = k_h$ which yields:

$$\Delta t = \frac{\pi z(2l - z)k_h\left(D^2 - d_w^2\right)}{q_w D^2\left[\ln\left(D/d_w\right) - 3/4\right]}$$

The average Δt value becomes equal to two thirds of the value obtained at depth $z = l$.

The effect of well resistance (discharge capacity) depends on the depth of drain installation, the drain spacing and whether the drains are penetrating or not, Figure B.3. In the case shown in Figure B.3 the delay Δt at 30 m depth becomes 1,46 (146 %) and the average Δt value 0,97.

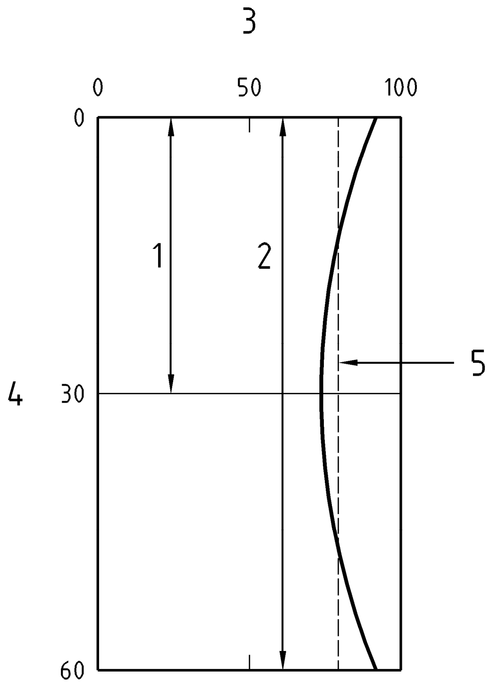
Provided that an increase of 10 % in the time of consolidation due to well resistance at the tip of partially penetrating drains ($z = l$, Figures B.2 and B.3) can be permitted relative to that obtained by using fully efficient drains, a conservative estimate of the required discharge capacity with regard to soil permeability and depth of installation is exemplified in Figure B.4. For penetrating drains (efficient drainage at top and bottom) the delay in consolidation takes place at mid-depth, see Figure B.3, and hence the depth values in Figure B.4 are doubled. The drain spacing in Figure B.4 is assumed equal to 0,9 m (drains placed in equilateral triangular pattern, i.e. D = 0,945 m, see Figure B.3) and the equivalent drain diameter d_w = 0,065 m.

The required discharge capacity according to Figure B.4 for partially penetrating drains, installed to a depth of 15 m in silty clay with a permeability of 0,25 m/year (0,8 $\times$ 10^{-8} m/s) becomes 1000 m^3/year, while the required discharge capacity for partially penetrating drains, installed to a depth of 15 m in clay with a permeability of 0,03 m/year (0,95 $\times$ 10^{-9} m/s) becomes 110 m^3/year.

The discharge capacity requirements decrease with increasing drain spacing. For a band drain spacing of, for example, 1,5 m (D = 1,575 m) and 2 m (D = 2,1 m), respectively, the required discharge capacities are 80 % and 70 %, respectively, of those presented in Figure B.4.

If the admissible delay in the time of consolidation is reduced to 5 %, the required discharge capacity given in Figure B.4 is doubled.

For penetrating drains, the depth values l given in Figure B.4 refer to half the depth of drain installation, see Figures B.2 and B.3.



Key

1 Partially penetrating drain (l = 30 m)

2 Penetrating drain ($2l$ = 60 m)

3 Degree of consolidation $\overline{U}_h$ %

4 Depth of drain installation, m

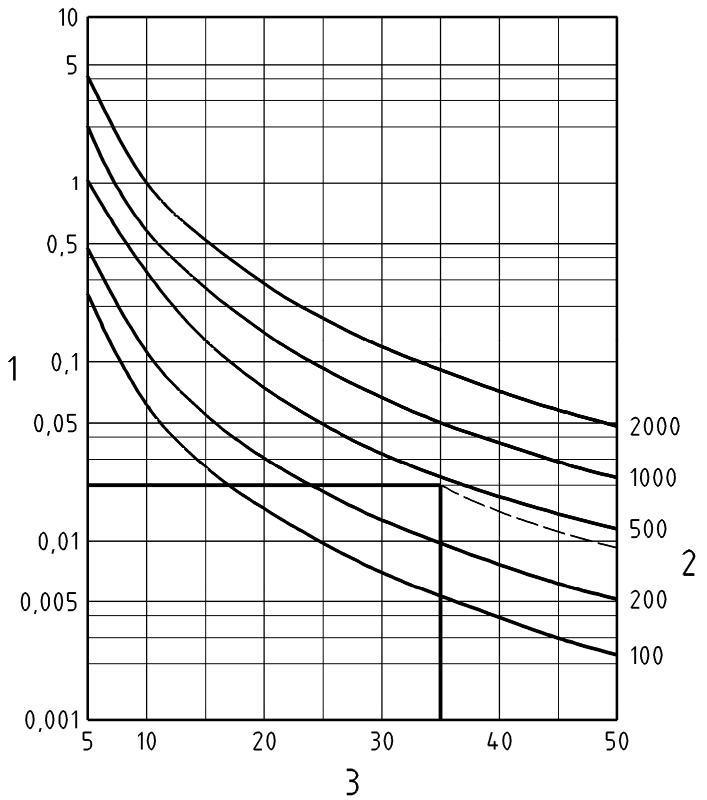
5 $\overline{U}_{h,\text{average}}$

Consolidation parameters: q_w = 100 m^3/year (≈ 3,2 cm^3/s), c_h = 1,0 m^2/year (≈ 3,2 × 10^{-8} m^2/s), $K_s = k_h$ = 0,1 m/year (≈ 3,2 × 10^{-9} m/s), time of consolidation t = 0,5 year. Drain spacing 0,9 m (equilateral triangular pattern; D = 0,945 m), drain diameter d_w = 0,065 m.

Figure B.3 — Example of the influence of well resistance on the degree of consolidation for partially penetrating and penetrating drains installed to depths 30 m and 60 m, respectively

With time a certain deterioration of the filter can be expected due to bacteriological activity or fungi attacks (see Annex A, Figure A.10). Deterioration generally reduces the discharge capacity towards the end of the consolidation process. Therefore, it has a relatively small influence on the rate of consolidation.

In highly compressible soils, the relative compression taking place during the consolidation process can lead to buckling or kinking of the drains (see Annex A, Figure A.5), which may seriously reduce the discharge capacity of certain types of drains [31].



Key

1 permeability ($k_s = k_h$), m/year (1 m/year = 3,17 x 10^{-8} m/s)

2 q_w, m^3/year

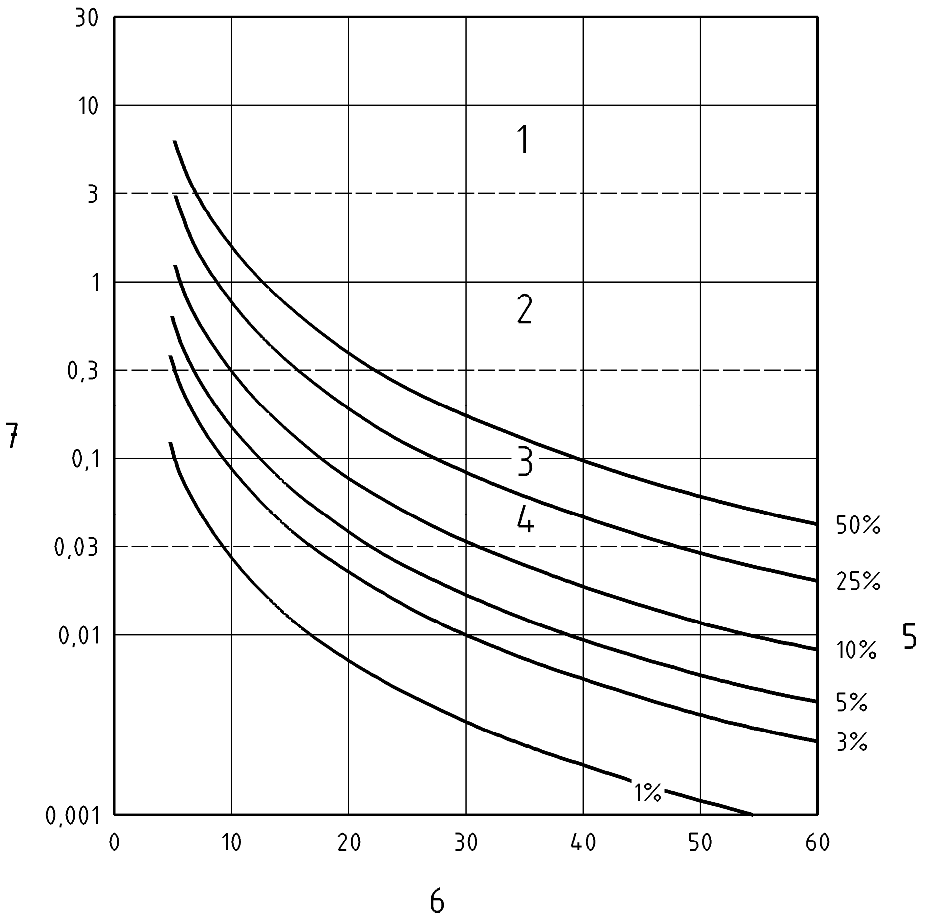
3 depth of installation, m

Drain spacing 0,9 m (equilateral triangular pattern; D = 0,945 m), drain diameter d_W = 0,065 m.

Figure B.4 — Requirements on discharge capacity q_w with regard to coefficient of permeability of the soil for a prolongation in time of consolidation of 10 % at depth l of drain installation (see Figures B.2 and B.3)

Table B.1 – Examples of minimum discharge capacities by consolidation analysis

Values of discharge capacity q_w in m^3/year for a delay in time of consolidation at depth $z = l$ of Δt = 10%									
Soil permeability	D/d_w = 10 (band drains)			D/d_w = 15 (band drains)			D/d_w = 5 (sand drains)		
	l = 10m	l = 20m	l = 30m	l = 10m	l = 20m	l = 30m	l = 10m	l = 20m	l = 30m
$k_s = k_h$= 0,315 m/year	630 m^3/year	2 525 m^3/year	5 690 m^3/year	505 m^3/year	2 010 m^3/year	4 530 m^3/year	1 105 m^3/year	4 420 m^3/year	9 950 m^3/year
$k_s = k_h$= 0,031 5 m/year	63 m^3/year	253 m^3/year	569 m^3/year	50 m^3/year	201 m^3/year	453 m^3/year	110 m^3/year	442 m^3/year	995 m^3/year



Key

1 peat

2 peat/silt

3 silt/clay

4 clay

5 delay

6 depth l of drain installation, m

7 permeability, m/year

Drain spacing 0,9 m (equilateral triangular pattern; D = 0,945 m), drain diameter d_w = 0,065 m, $k_s = k_h$.

Figure B.5 — Delay in time of consolidation at depth *l* of drain installation (see Figure B.2 and B.3) for drains with a discharge capacity of 500 m^3/year (16 cm^3/s)

B.4.1.3 Safety factors for prefabricated band drains

With regard to possible negative effects on the discharge capacity of prefabricated band drains, consideration has to be taken to the influence of effective lateral soil pressure against the drains, of soil temperature and of long-term biological and chemical activities. In order to guarantee the efficiency of the drains, testing of the discharge capacity of the drains (see Annex A) should be carried out with due reference to the expected maximum effective lateral pressure against the drains and the temperature condition in the actual project multiplied by certain required safety factors, [36]. How this should be done is exemplified in Annex A.

B.4.2 Stability

Stability analysis is very important when soil improvement is undertaken by vertical drain installation and preloading. In the stability analysis of the embankment load placed on the ground surface, the reinforcing effect of the vertical drains themselves (e.g. sand drains) is not taken into account. However, estimation and a follow-up of the strength increase produced during consolidation, particularly when stepwise loading is used, is an important part of the analysis.

The un-drained shear strength, determined in the field (e.g. by field vane tests or cone penetration tests) or by laboratory tests (e.g. fall-cone test, triaxial test or unconfined compression test), should be adjusted with regard to the consistency limits of the soil and to the shearing direction [7]. If the placement of the external load involves stability problems, the load has to be placed stepwise. After each load-step, the gain in shear strength achieved during the consolidation process has to be investigated before the placement of the following load-step, in order that the stability condition is not jeopardised.

A possibility of estimating the strength gain in each load-step is to utilise empirical correlations, e.g. between liquid limit, un-drained shear strength and preconsolidation pressure [15] or between plasticity index, un-drained shear strength and preconsolidation pressure [48]. If there is no change in liquid limit or plasticity index during the consolidation process, the relative change in un-drained shear strength can be assumed equal to the relative change in preconsolidation pressure. Valuable empirical correlations for estimating the strength gain have also been presented by Mesri [38] and Ladd [30]. Since the preconsolidation pressure increases with effective stress increase in the ground, it depends directly on the degree of consolidation, which characterises both settlement and excess pore water pressure decrease. Therefore, pore pressure monitoring should be part of the prescriptions for vertical drainage projects, as described in Annex A.

Stability problems can be avoided by exchanging external loading by the vacuum method or by pumping water from underlying pervious soil (see Annex A). Normally, 70 % vacuum can be achieved, which results in an effective stress increase similar to that produced by a surface load of 70 kN/m^2. However, the ratio of vertical to horizontal effective stress increase in the two cases will be different. This will have a different effect on the increase of un-drained shear strength caused by consolidation than the increase caused by surface loading. If the vacuum method is utilised in near-shore installations, the resulting effective stress increase will also include the overburden pressure of the seawater.

Annex C
(informative)

Degree of obligation of the specifications

The provisions are marked corresponding to their degree of obligation:

— *(REQ) :* Requirement;

— *(REC) :* Recommendation;

— *(PER) :* Permission;

— (*ST*) *:* Statement.

4.1 (*REQ*)	**6.3.2.2** (*REC*)	**6.3.9.2** (*ST*)	**6.4.8** (*REC*)	**8.3.3** (*REC*)
4.2 (*REQ*)	**6.3.3.1** (*REQ*)	**6.3.9.3** (*REC*)	**6.4.9** (*ST*)	**8.3.4** (*REQ*)
4.3 (*REQ*)	**6.3.3.2** (*REC*)	**6.3.9.4** (*REC*)	**6.4.10** (*REQ*)	**8.3.5** (*REQ*)
4.4 (*REQ*)	**6.3.3.3** (*REQ*)	**6.3.10.1** (*REQ*)	**6.5.1** (*ST*)	**8.3.6** (*REQ*)
4.5 (*REQ*)	**6.3.4.1** (*REQ*)	**6.3.10.2** (*REQ*)	**6.5.2** (*REC*)	**8.3.7** (*REC*)
4.6 (*REQ*)	**6.3.4.2** (*REC*)	**6.4.1.1** (*ST*)	**6.5.3** (*REC*)	**8.3.8** (*REC*)
5.1.1 (*REQ*)	**6.3.4.3** (*REC*)	**6.4.1.2** (*REQ*)	**6.5.4** (*REQ*)	**8.3.9** (*REQ*)
5.1.2 (*REQ*)	**6.3.4.4** (*REQ*)	**6.4.2.1** (*REC*)	**7.1.1** (*ST*)	**8.4.1** (*REC*)
5.1.3 (*REC*)	**6.3.5.1** (*ST*)	**6.4.3.1** (*REQ*)	**7.1.2** (*REC*)	**8.4.2** (*REQ*)
5.2.1 (*REQ*)	**6.3.5.2** (*ST*)	**6.4.3.2** (*REC*)	**7.1.3** (*ST*)	**8.4.3** (*REC*)
5.2.2 (*REQ*)	**6.3.5.3** (*REQ*)	**6.4.3.3** (*REQ*)	**7.1.4** (*REC*)	**8.4.4** (*REQ*)
5.2.3 (*REC*)	**6.3.5.4** (*REC*)	**6.4.4.1** (*REQ*)	**8.1.1** (*REQ*)	**8.4.5** (*REQ*)
6.1.1 (*ST*)	**6.3.5.5** (*REC*)	**6.4.4.2** (*REC*)	**8.1.2** (*REQ*)	**9.1.1** (*REQ*)
6.1.2 (*REQ*)	**6.3.5.6** (*REC*)	**6.4.4.3** (*REQ*)	**8.1.3** (*REQ*)	**9.1.2** (*REQ*)
6.2.1 (*ST*)	**6.3.5.7** (*REC*)	**6.4.5.1** (*ST*)	**8.1.4** (*REC*)	**9.1.3** (*REQ*)
6.2.2 (*ST*)	**6.3.6.1** (*REC*)	**6.4.5.2** (*REQ*)	**8.2.1** (*REQ*)	**9.1.4** (*REC*)
6.2.3 (*PER*)	**6.3.6.2** (*REC*)	**6.4.5.3** (*REQ*)	**8.2.2** (*REQ*)	**9.1.5** (*REQ*)
6.2.4 (*REQ*)	**6.3.6.3** (*REQ*)	**6.4.6.1** (*REC*)	**8.2.3** (*REC*)	**9.2.1** (*REQ*)
6.2.5 (*ST*)	**6.3.6.4** (*REQ*)	**6.4.6.2** (*REC*)	**8.2.4** (*REC*)	**9.2.2** (*REC*)
6.3.1.1 (*ST*)	**6.3.7.1** (*REQ*)	**6.4.6.3** (*REQ*)	**8.2.5** (*REC*)	**9.2.3** (*REQ*)
6.3.1.2 (*REC*)	**6.3.7.2** (*REC*)	**6.4.6.4** (*REQ*)	**8.2.6** (*REC*)	**9.2.4** (*REQ*)
6.3.1.3 (*REQ*)	**6.3.8.1** (*REC*)	**6.4.7.1** (*REQ*)	**8.3.1** (*REQ*)	**9.2.5** (*REC*)
6.3.2.1 (*REC*)	**6.3.9.1** (*REQ*)	**6.4.7.2** (*REC*)	**8.3.2** (*REQ*)	**9.2.6** (*REQ*)

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— *(REC) :* Recommendation;

— *(PER) :* Permission;

— (*ST*) *:* Statement.

9.2.7 (*REC*)	**11.1.1** (*ST*)	**11.2.2** (*REC*)	**11.3.1** (*REQ*)
9.2.8 (*REC*)	**11.1.2** (*REQ*)	**11.2.3** (*REQ*)	**11.3.2** (*REQ*)
9.2.9 (*REC*)	**11.1.3** (*REQ*)	**11.2.4** (*REC*)	**11.3.3** (*REQ*)
10.1 (*REQ*)	**11.2.1** (*REQ*)	**11.2.5** (*REC*)	**11.4** (*REC*)
10.2 (*REQ*)			

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