Second generation of Eurocode 7

Basis of design and how the influence of Groundwater is incorporated in the new code

22nd of February 2023 Online 15:00 – 17:00 CEST

WELCOME

Basis of design and how the influence of Groundwater is incorporated in the new code

Second generation of Eurocode 7

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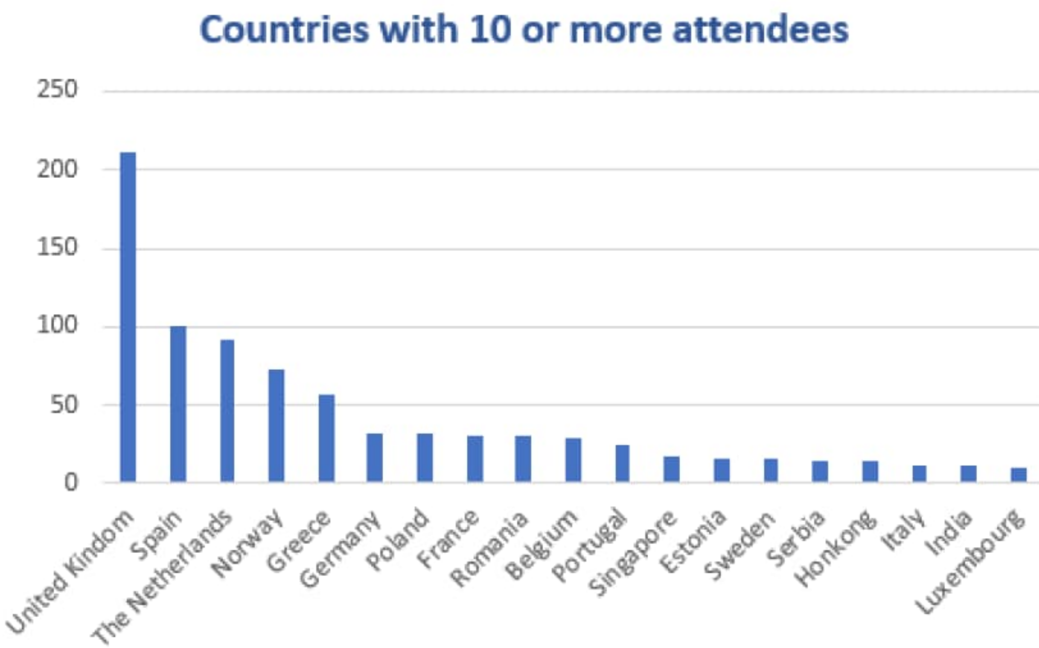
Preliminary report on 20-2-2023 : 957 registered attendees!



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Spain	Australia	Turkey
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Romania	Argentina	Haiti
Belgium	Cambodia	Indonesia
Portugal	Croatia	Iraq
China	Finland	Israel
Singapore	Iran	Lebanon
Estonia	Perú	Lithuania
Serbia	Senegal	Malaysia
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Ireland	Chile	UKraine
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Denmark	Czech Republic	Zaragoza
Canada	Kosovo	Zimbabwe
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About this webinar

- Please mute your microphone
- Only use your screen on when talking.
- Questions via Q&A (no ‘raise hand’)

(You are able to upvote a question)

- We don’t have time to answer all your questions, we will answer them afterwards
- This webinar will be recorded.
- The Presentations, the Q&A and a link to the recording of the webinar are sent afterwards.

Chairman of the Day

5.1.2.e

[redacted]

Director at COWI and member of ISSMGE ERTC10, UK

[redacted] is a geotechnical engineer with over 30 years of practising experience. He was the lead of the Numerical Skills Team at Arup Geotechnics and is currently responsible for the Excellence Team of numerical modelling at COWI

He has published several papers on the application of limit states design under the current Design Approach 1 of Eurocode 7 and is active in the working groups involving the numerical modelling and retaining wall clauses/chapters in the Next Generation of Eurocode 7

Programme

15.00	Introduction	[redacted]
15.05	prEN1990:202x Eurocode Basis of Stuctural and Geotechnical Design	[redacted]
15:30	Additional aspects of basis of design in prEN1997-1 General rules	[redacted] 5.1.2.e
15:50	Groundwater in prEN1997-1 202x	[redacted]
16:10	Groundwater related limit states – examples	[redacted]
16.30	Q&A session	[redacted]
17.00	Closure	

Basis of Design and the influence of Groundwater in the Next Generation of EC7

[redacted] 5.1.2.e

Director COWI UK Ltd.

Member of ISSMGE ERTC10, UK

Objectives of ERTC10 include:

- Dissemination of information about EC7 and changes introduced in its second generation
- Providing guidance and recommendations related to application of EC7 in practical geotechnical design
- Assistance with organization and participation in international conferences and activities related to evaluation and application of EC7
- Providing link between ISSMGE, academia, industry and standardization bodies to foster development and implementation of EC7

Programme

- prEN1990:202x Eurocode Basis of Structural and Geotechnical Design (25mins)
- Additional aspects of basis of design in prEN1997-1 General rules (20mins)
- Grounwater in prEN1997-1 202x (20mins)
- Groundwater related limit states – examples (20mins)
- Q&A (30mins)
- Closure

Speakers – Basis of Design

prEN1990:202x Eurocode Basis of Structural and Geotechnical Design

[redacted]

5.1.2.e

[redacted] is Director of Geocentrix Ltd, which provides consulting services to large and small contractors, consultants, and clients. He wrote the software programs Geocentrix ReWaRD® (retaining wall design) and Geocentrix Repute® (piled foundations) that are used in many countries throughout the world.

As Chairman of the Eurocode 7 committee between 2010 and 2019, [redacted] led the work developing the 2nd Generation of EC7. He was also a member of the project team that wrote the **2nd Generation of EN 1990, Basis of structural and geotechnical design**, ensuring that geotechnical engineering get proper treatment in the head Eurocode.

[redacted] is co-author of the highly regarded book Decoding Eurocode 7 and has written numerous other papers and guidance notes on the practical use of the Eurocodes.

Additional aspects of basis of design in prEN1997-1 General rules

[redacted].

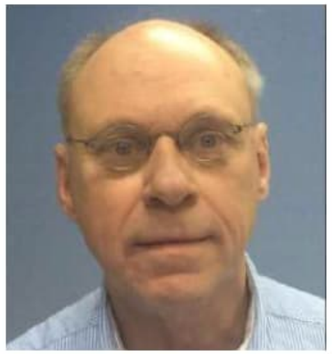
5.1.2.e

[redacted] is the Vice Chair of TC250 Subcommittee 7 - Eurocode 7 – Geotechnical Design
Senior Geotechnical Engineer at GeoVerkstan, Sweden

Speakers – Groundwater

Grounwater in prEN1997-1

[redacted]



5.1.2.e

[redacted] is the Chair of CEN/TC250 Subcommittee "Eurocode 7" and a Principal Consultant at Fugro Netherlands.

Groundwater related limit states – examples

[redacted]

5.1.2.e

[redacted] is a Geotechnical Specialist with more than 25 yrs of experience and a part-time lecturer of Geotechnics at the University of Malaga, in Spain, and lead geotechnical consultant for the Whitearth company. Since 2018, he has been a World Bank consultant on disaster risk management and resilient infrastructure development.

He led the TG B2 for the design examples analysis of hydraulic ultimate limit state.

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Basis of structural and geotechnical design

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nen

Basis of design and how the influence of groundwater is incorporated in the new code

Basis of structural and geotechnical design

Dr [redacted] 5.1.2.e
Geocentrix Ltd 5.1.2.e
Past-Chair, TC250/SC7 5.1.2.e

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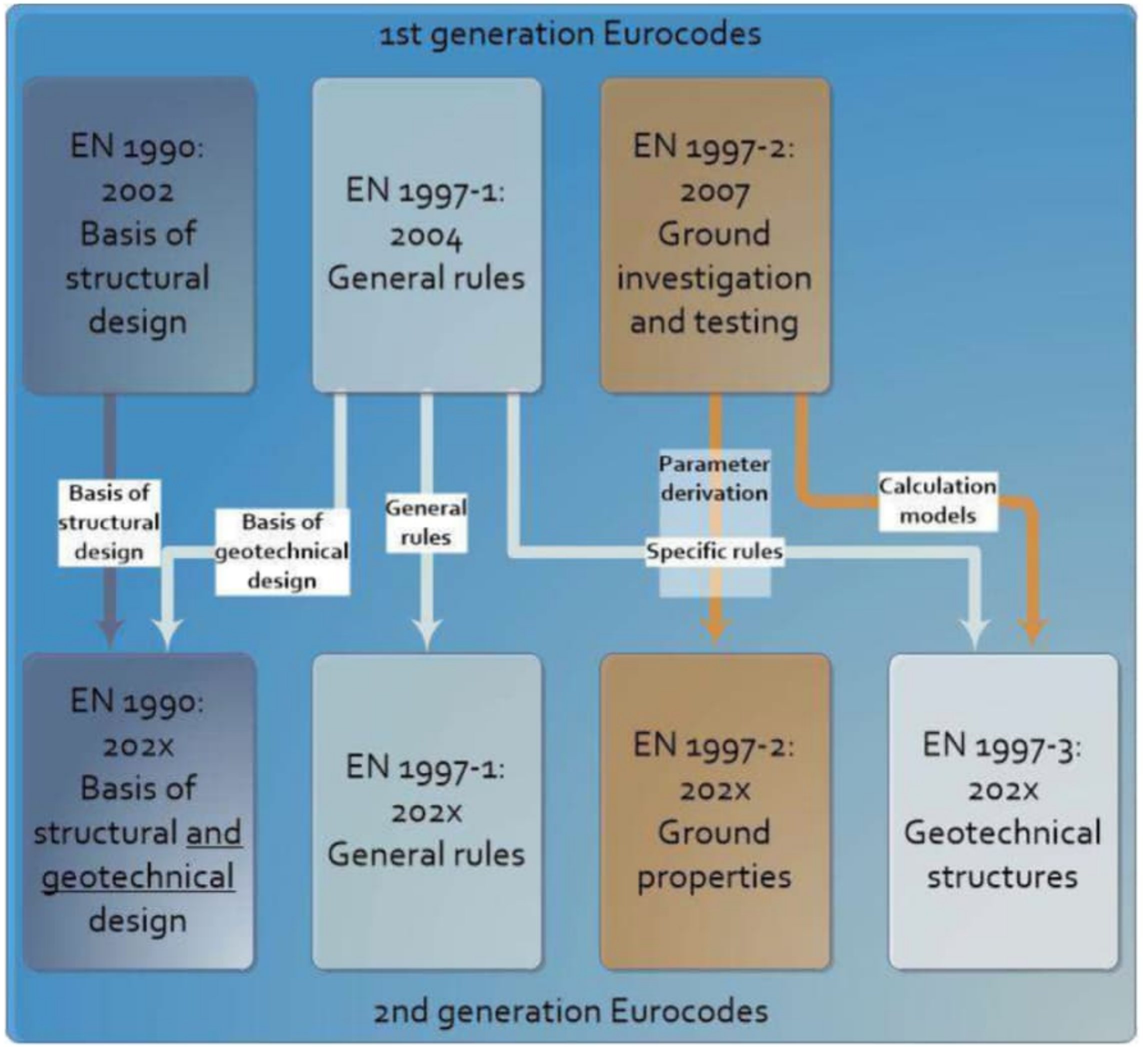
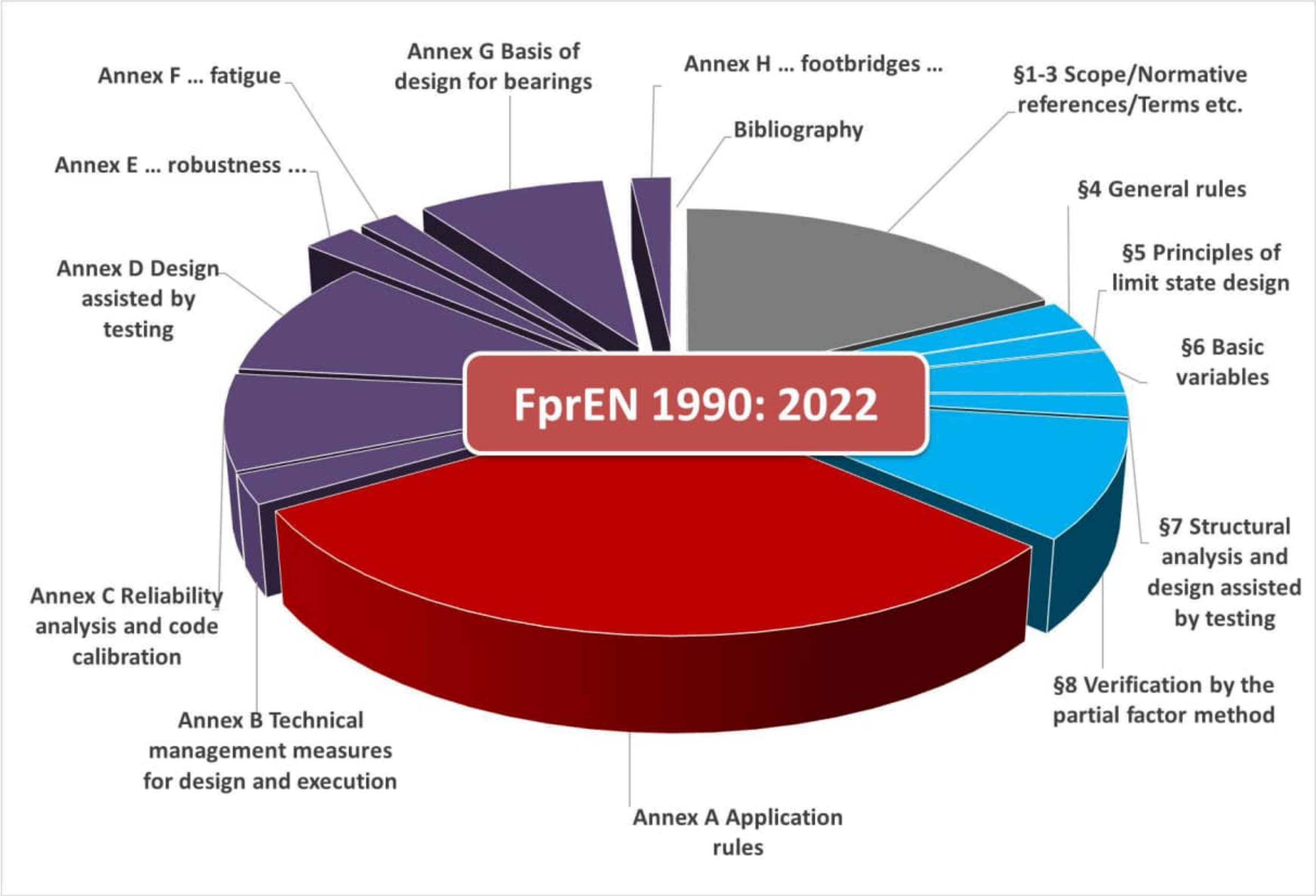
Transformation of Eurocode 7 into 3 Parts



Transformation of Eurocode 7 into 3 Parts



Scope of the Eurocode

*"[The Eurocode] establishes principles and **requirements for the safety, serviceability, robustness and durability of structures, including geotechnical structures**, appropriate to the consequences of failure.*

*"[It] is intended to be used in conjunction with the other Eurocodes for the **design of buildings and civil engineering works, including temporary structures**"*

FprEN 1990:2022

The Eurocode:

- describes the basis for structural and geotechnical design and verification according to the **limit state principle**
- gives verification methods based primarily on the **partial factor method**

EN 1990 is also applicable for:

- structural assessment of existing structures
- developing the design of repairs, improvements and alterations
- assessing changes of use
- the design of structures where materials or actions outside the scope are involved

Level of reliability

The choice of an appropriate **level of reliability for the structure** should take account of the following:

- possible **consequences of failure** in terms of risk to life, injury, and potential economic losses
- the possible **cause and mode of attaining a limit state** (e.g. failure modes with or without warnings, e.g. ductile or brittle failure)
- **public aversion to failure**
- the **expense** and procedures necessary **to reduce the risk of failure**

Parameter	Reference period (years)	Symbol	Consequence class		
			CC1	**CC2**	CC3
Probability of failure	50	$P_{f,50}$	~ 10^{-3}	**~ 10^{-4}**	~ 10^{-5}
Beta index	50	β_{50}	3.3	**3.8**	4.3
Beta index	1	β_1	4.2	**4.7**	5.2

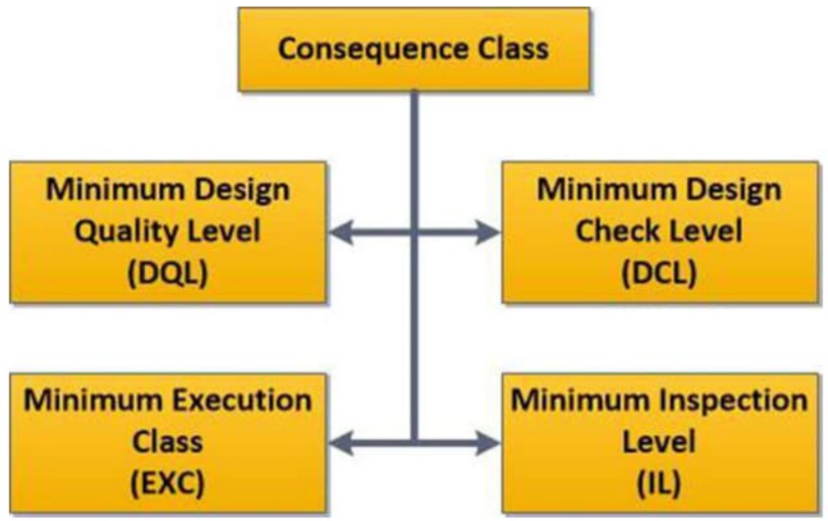
Consequences classes in EN 1990

New

Consequence class/ Description		Loss of human life*	Economic, social or environ-mental*	Failure prob-ability, $P_{f,50}$	Factor k_F		
					Buildings	Bridges**	
CC4	Highest	Extreme	Huge	Additional provisions can be needed			
CC3	Higher	High	Very great	$\sim 10^{-5}$	1.1	CC3b	1.1
						CC3a	1.0
CC2	Normal	Medium	Considerable	$\sim 10^{-4}$	1.0	CC2	1.0
CC1	Lower	Low	Small	$\sim 10^{-3}$	0.9	CC1	0.9
CC0	Lowest	Very low	Insignificant	Alternative provisions may be used			

*CC is chosen based on the more severe of these two columns
**For bridges, CC3 is further divided into CC3b (upper class) and CC3a (lower class)

Technical management measures



CC	Level	Minimum design quality level (DQL)*	Minimum design check level (DCL)	Minimum execution class	Minimum inspection level (IL)
CC3	3	Complex	Extended independent	See relevant execution and product standards	Extended independent
CC2	2	Advanced	Normal independent		Normal independent
CC1	3	Simple	Self-checking		Self-checking

*Have at least the same level of design qualification and experience to that required to perform … design

Verification of ultimate limit states

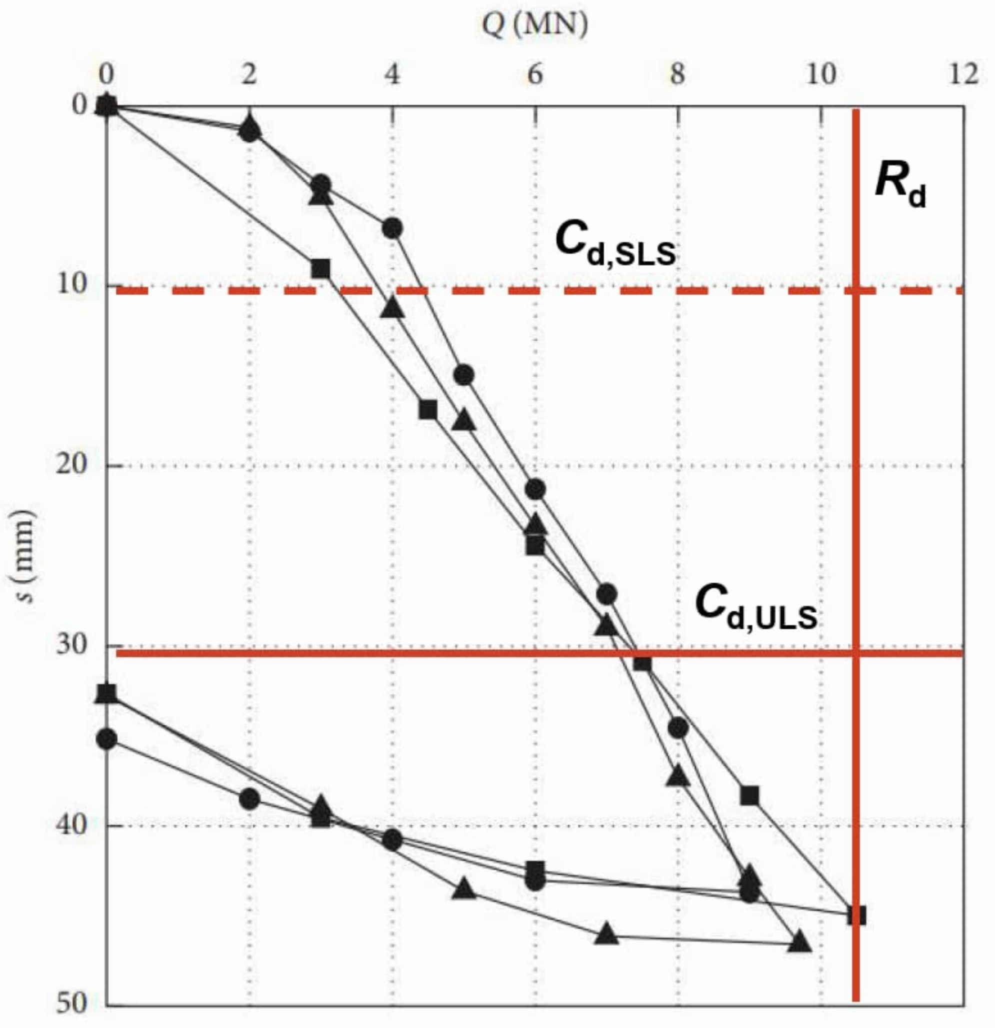
Ultimate limit states caused by **rupture, mechanisms, buckling, hydraulic gradients, fatigue, vibrations, or time-dependent effects** are verified using:

$$\underbrace{\overbrace{E_{\mathrm{d}}}^{\substack{design\ effects\\ of\ actions}} \leq \overbrace{R_{\mathrm{d}}}^{\substack{design\\ resistance}}}_{forces\ or\ stresses}$$

Ultimate limit states caused by **excessive deformation** are verified using:

$$\underbrace{\overbrace{E_{\mathrm{d}}}^{\substack{design\ effects\\ of\ actions}} \leq \overbrace{C_{\mathrm{d,ULS}}}^{\substack{limiting\ design\\ value\ of\ excessive\\ deformation}}}_{displacements\ or\ strains}$$

Excessive deformation vs serviceability constraint



Factors on actions or on action-effects?

Partial factors may be applied to actions (Verification Cases 1 to 3):

$$E_{\mathrm{d}} = \underbrace{E\left\{\Sigma\left(\boxed{\gamma_{\mathrm{F}}}\psi F_{\mathrm{k}}\right); a_{\mathrm{d}}; X_{\mathrm{Rd}}\right\}}_{\gamma_{\mathrm{F}}=\gamma_{\mathrm{Sd}}\times\gamma_{\mathrm{f}}}$$

or to action-effects (Verification Case 4):

$$E_{\mathrm{d}} = \underbrace{\boxed{\gamma_{\mathrm{E}}}E\{\Sigma(\psi F_{\mathrm{k}}); a_{\mathrm{d}}; X_{\mathrm{Rd}}\}}_{\gamma_{\mathrm{E}}=\gamma_{\mathrm{Sd}}\times\gamma_{\mathrm{f}}}$$

VC1 is used for verification of structural resistance (and for geotechnical design)

VC2 is used for static equilibrium and uplift

VC3 and VC4 are used for geotechnical design

Partial factors for fundamental design situations (general application)

<table>
<tr><th colspan="4">Action or effect</th><th colspan="5">Partial factors γ_F and γ_E for Verification Cases 1-4</th></tr>
<tr><th rowspan="2">Type</th><th rowspan="2">Group</th><th rowspan="2">Symbol</th><th rowspan="2">Resulting effect</th><th>Structural*</th><th colspan="2">Static equilibrium and uplift**</th><th colspan="2">Geotechnical design</th></tr>
<tr><th>VC1</th><th>VC2(a)</th><th>VC2(b)</th><th>VC3</th><th>VC4</th></tr>
<tr><td rowspan="5">Permanent action (G_k)</td><td>All</td><td>γ_G</td><td rowspan="2">unfavourable/ destabilizing</td><td rowspan="9">Set ‘B’
STR/GEO
DA 1-1
DA 2
DA 3</td><td colspan="2" rowspan="9">Set ‘A’
EQU</td><td rowspan="9">Set ‘C’
STR/GEO
DA 1-2
DA 3</td><td rowspan="5">G_k is not factored</td></tr>
<tr><td>Water</td><td>$\gamma_{G,w}$</td></tr>
<tr><td>All</td><td>$\gamma_{G,stb}$</td><td rowspan="2">stabilizing</td></tr>
<tr><td>Water</td><td>$\gamma_{Gw,stb}$</td></tr>
<tr><td>(All)</td><td>$\gamma_{G,fav}$</td><td>favourable</td></tr>
<tr><td colspan="2">Prestressing (P_k)</td><td>γ_P</td><td></td><td rowspan="6">-
STR/GEO
DA 2*</td></tr>
<tr><td rowspan="3">Variable action (Q_k)</td><td>All</td><td>γ_Q</td><td rowspan="2">unfavourable</td></tr>
<tr><td>Water</td><td>γ_{Qw}</td></tr>
<tr><td>(All)</td><td>$\gamma_{Q,fav}$</td><td>favourable</td></tr>
<tr><td colspan="2" rowspan="2">Effects-of-actions (E)</td><td>γ_E</td><td>unfavourable</td><td colspan="4" rowspan="2">γ_E is not applied</td></tr>
<tr><td>$\gamma_{E,fav}$</td><td>favourable</td></tr>
<tr><td colspan="9">*Also used for geotechnical design; **Less favourable outcome of (a) and (b) applies</td></tr>
</table>

Why are partial factors on water actions less than on other actions?

Partial factors applied to water actions are smaller than in EN 1990:2002 and less than applied to other actions:

$$\gamma_{Gw} < \gamma_G \text{ and } \gamma_{Qw} < \gamma_Q$$

Partial factors are made up of two components:

$$\gamma_F = \overbrace{\underbrace{\gamma_{Sd}}_{\substack{uncertainty\\ of\ the\ model}}}^{model\ factor} \times \overbrace{\underbrace{\gamma_f}_{\substack{deviation\ of\\ the\ action}}}^{partial\ factor}$$

FprEN 1990 considers potential deviations of water actions to be less than that of other types of actions. The same thinking applies in the current Eurocodes:

"If the maximum depth of liquid and the unit weight of the heaviest stored liquid are defined, the value of the partial factor γ_F [i.e. γ_Q] may be reduced from 1,50 to 1,35"

EN 1991-4:2006, A.2.1(2)

Permanent actions

The representative value of a permanent action G_{rep} is one of:

- a **single characteristic value** G_k (equal to the **mean** value, G_{mean})
  - when the uncertainty in G is small*
- **upper or lower characteristic values** ($G_{k,sup}$ and $G_{k,inf}$)
  - when the uncertainty in G is not small
  - (or) the structure is sensitive to variations in its value or spatial distribution
- a **nominal** value (G_{nom})

*For most structural members, small is ≤ 5 % for loss of static equilibrium or uplift; otherwise ≤ 10 %. For the ground, see EN 1997-1

Variable actions

The representative value of a variable action Q_{rep} is one of:

- its **characteristic** value Q_k
- its **combination** value Q_{comb}
- its **frequent** value Q_{freq}
- its **quasi-permanent** value Q_{qper}

The characteristic value of a variable action Q_k is one of:

- an **upper** value with a specified probability of exceedance*
- a **lower** value with a specified probability of exceedance*
- a **nominal** value (when the statistical distribution of Q is unknown)

*during a specific reference period

Probabilities of exceedance of variable actions

*"The **combination, frequent and quasi-permanent values** should be determined by **multiplying the characteristic values … by combination factors**"*

FprEN 1990:2022, 6.1.2.3(3)

$$Q_{\mathrm{comb}}|Q_{\mathrm{freq}}|Q_{\mathrm{qper}} = (\psi_0|\psi_1|\psi_2) \times Q_{\mathrm{k}}$$

Value of variable action	Symbol	Probability of exceedance	Return period (years)	Combination factor
Characteristic	Q_k	2% per annum	50	-
Combination	Q_{comb}	?	?	ψ_0
Frequent	Q_{freq}	Fraction of time exceeded = 1%		ψ_1
Quasi-permanent	Q_{qper}	Fraction of time exceeded = 50%		ψ_2

Factors on resistance or on material properties?

Partial factors may be applied to material properties (the *material factor approach*, MFA):

$$R_d = \underbrace{R\left\{\frac{\eta X_k}{\boxed{\gamma_M}}; a_d; \sum F_{Ed}\right\}}_{\gamma_M = \gamma_{Rd} \times \gamma_m}$$

or **to resistance** (the *resistance factor approach*, RFA):

$$R_d = \underbrace{\frac{R\{\eta X_k; a_d; \sum F_{Ed}\}}{\boxed{\gamma_R}}}_{\gamma_R = \gamma_M = \gamma_{Rd} \times \gamma_m}$$

MFA and RFA are used for verification of structural resistance

Both MFA and RFA are used for geotechnical design

Partial factors for fundamental design situations (ground properties)

Ground property (of soil, except noted)	Symbol	M1	M2
Soil properties			
Shear strength in effective stress analysis (τ_f)	$\gamma_{\tau f}$	1.0	1.25 k_M
Coefficient of peak friction (tan φ'_p)	$\gamma_{\tan\varphi,p}$		
Peak effective cohesion (c'_p)	$\gamma_{c,p}$		
Coefficient of friction at critical state (tan φ'_{cs})	$\gamma_{\tan\varphi,cs}$		1.1 k_M
Coefficient of residual friction (tan φ'_r)	$\gamma_{\tan\varphi,r}$		
Shear strength in total stress analysis (c_u)	γ_{cu}		1.4 k_M
Rock properties			
Unconfined compressive strength (q_u)	γ_{qu}	Same as γ_{cu}	
Shear strength of rock (τ_r)	$\gamma_{\tau r}$	1.0	1.25 k_M
Unconfined compressive strength of rock (q_u)	γ_{qu}		1.4 k_M
Properties of discontinuities			
Shear strength of rock discontinuities (τ_{dis})	$\gamma_{\tau dis}$	1.0	1.25 k_M
Coefficient of residual friction (tan $\varphi'_{dis,r}$)	$\gamma_{\tan\varphi,dis,r}$		1.1 k_M

In the 2nd Generation Eurocodes:

- The basis of geotechnical design has been moved to EN 1990
- EN 1990 has been generalized to cater for non-linear materials
- Factoring action-effects now has equal status with factoring actions
- There is a clear distinction between the Material Factor Approach (MFA) and the Resistance Factor Approach (RFA)
- Water actions are fully specified and attract smaller partial factors than before

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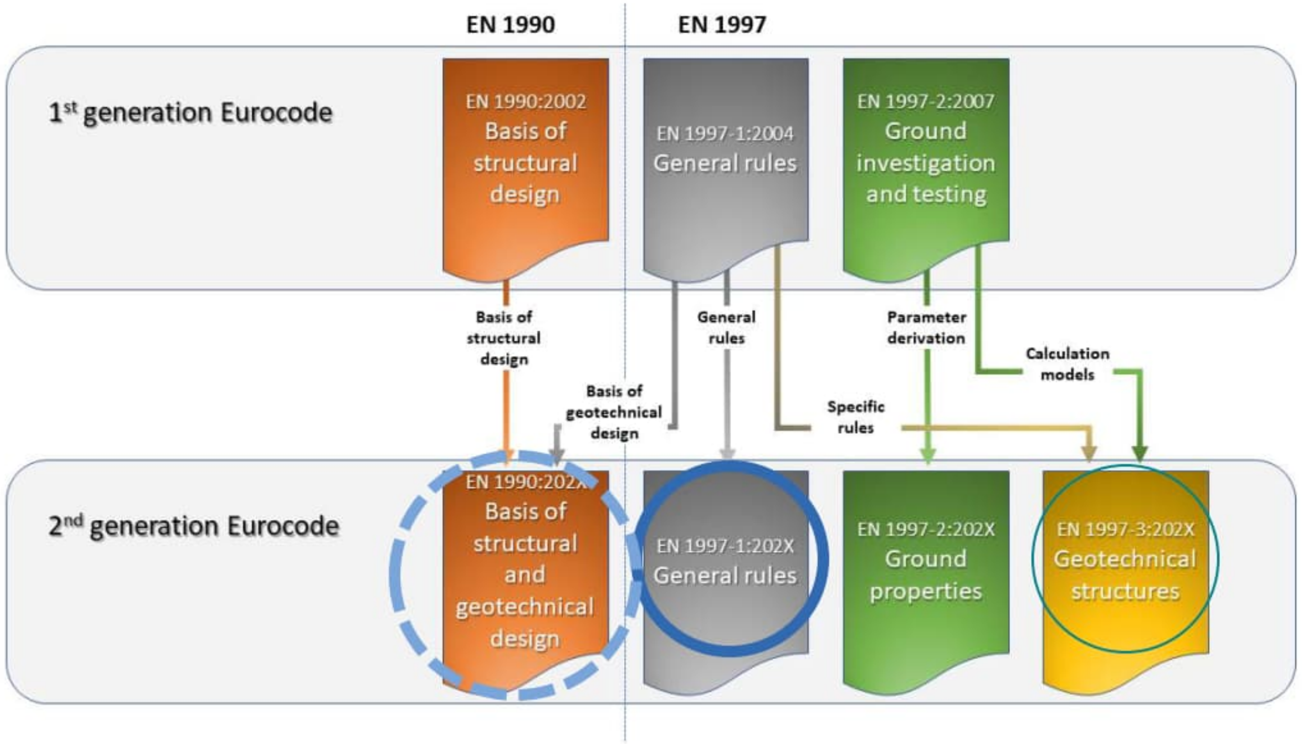
Additional aspects of basis of design in prEN1997-1 General rules

[redacted], GeoVerkstan 5.1.2.e5.1.2.e
Vice chair TC250 SC7 Geotechnical Design

SIMSG ISSMGE

Additional aspects of basis of design



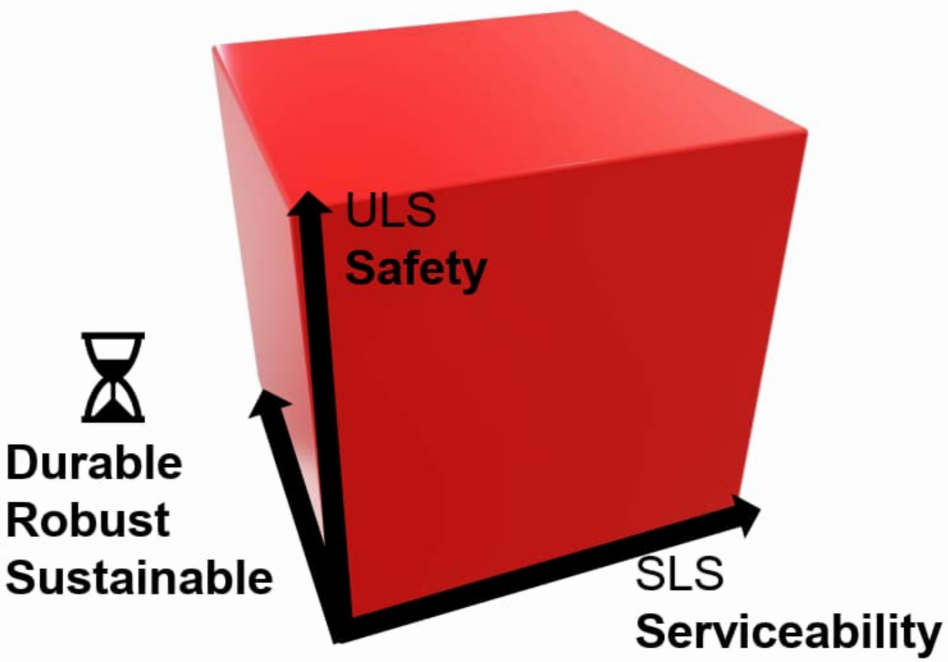
2nd generation of Eurocode

Basis of design for all structures -> EN1990
+
Basis of design for all **geotechnical structures** -> EN 1997-1
+
Specific rules for a geotechnical structure -> EN 1997-3

5.1.2.e


EN 1997-1 applicable for all
Geotechnical Structures
in all Ground and
Groundwater conditions
5.1.2.e

A toolbox for verification that your **geotechnical structure** is within the cube.



Additional aspects of basis of design

Geotechnical Design Model, GDM

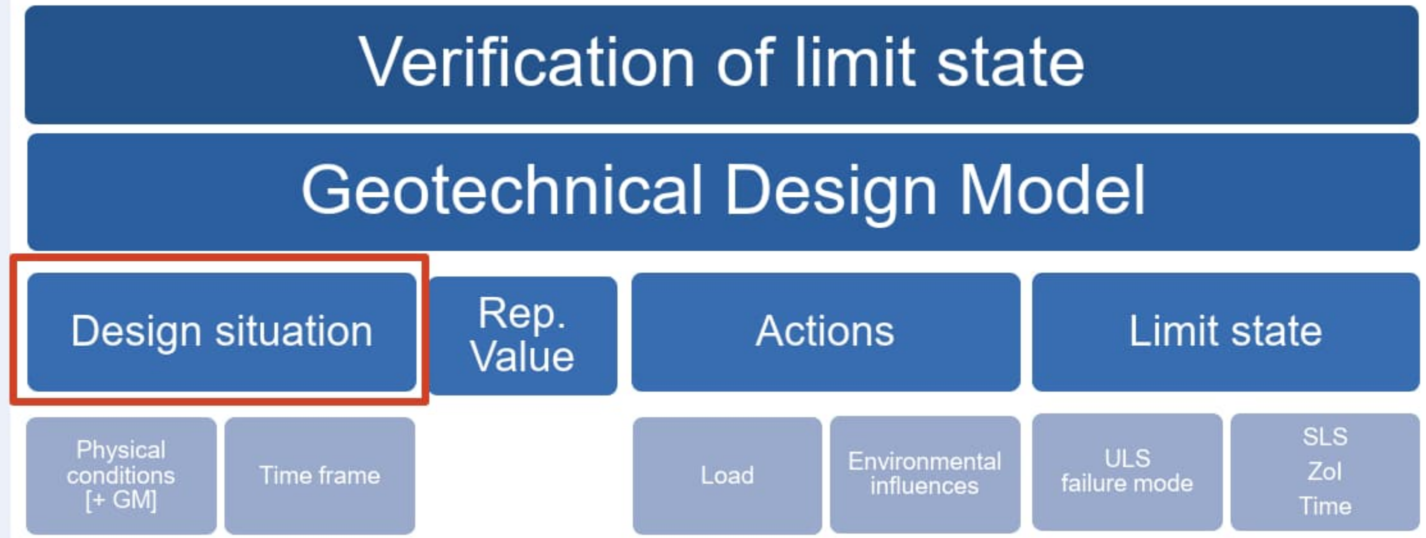
[EN 1997-1, 4.2.3.1]

The basis of design is compiled in a GDM.

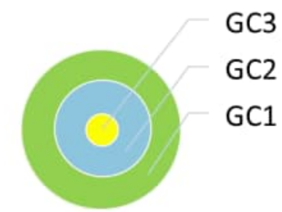
“GDM a carrier of the needed information for your verification”

GDM shall

- Be developed for each design situation.
- Include corresponding combination of actions.
- Include associated relevant limit states.
- Based on a validated Ground model.
- Include representative values for geotechnical units in the zone-of-influence



Apropriate reliability



5.1.2.e

Additional aspects of basis of design

Design situation

EN 1990, 5.2, Table 5.1

Design situation	Conditions
Persistent	Normal use and exposure
Transient	Temporary use and exposure during a period much shorter than the design service life of the structure
Accidental	Exceptional conditions or exposure
Seismic	Exceptional conditions during a seismic event
Fatigue	Conditions caused by fatigue actions

Definition [EN 1990, 3.1.2.2]

Design situation

physical conditions expected to occur during a certain time period for which it is to be demonstrated,

with sufficient reliability, that relevant limit states are not exceeded

Physical conditions [EN 1997-1, 4.2.2(2)]

Geometrical properties of the structure and the site.

Geometrical and material properties of the ground and groundwater.

Ground Model

Environmental influences on the structure, the ground, the groundwater.

Also this [EN 1997-1, 4.2.2(3)]

- Phases: Execution to maintenance
- Impact from execution on geometry/material properties
- Practicability and buildability
- Transient or permanent changes to ground/**groundwater** during service life

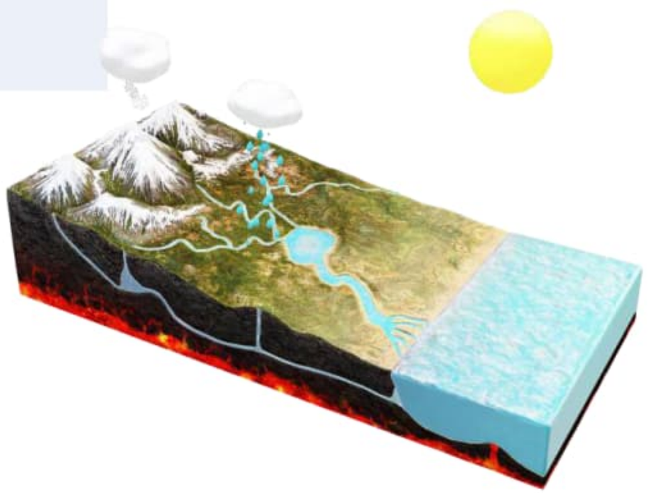
Additional aspects of basis of design

Ground Model

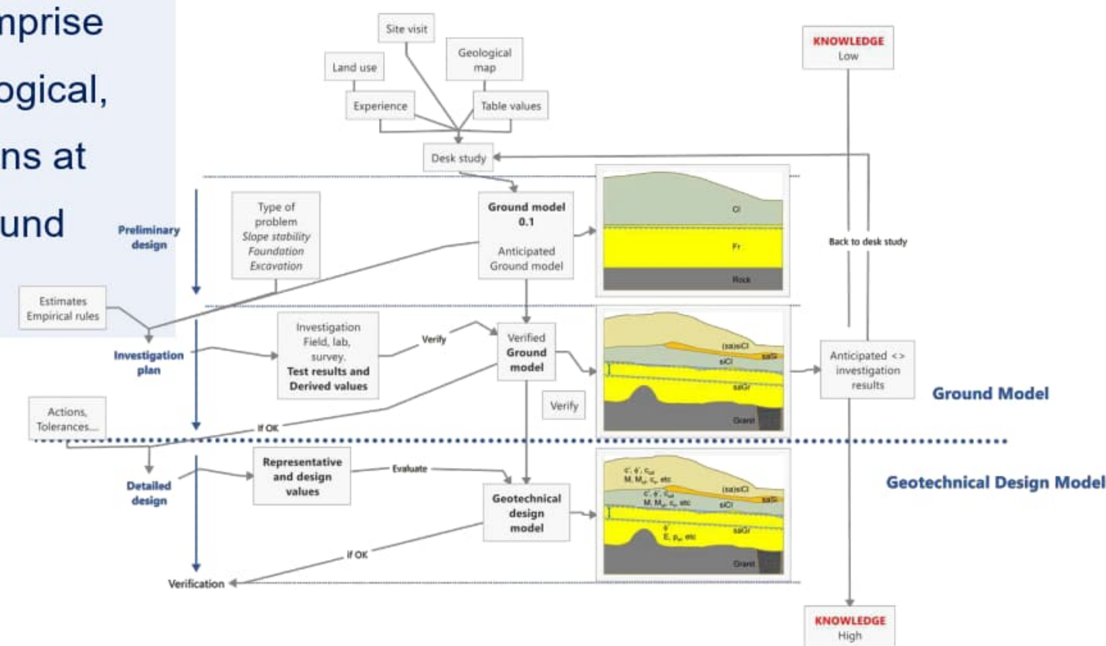
EN 1997-2, 4.1(4)
The Ground Model shall be progressively developed and updated based on potential new information.

Ground Model
EN 1997-1, 3.1.6.6
site specific outline of the disposition and character of the ground and groundwater based on results from ground investigations and other available data.

EN 1997-2, 4.1(1)
A Ground Model shall comprise the geological, hydrogeological, and geotechnical conditions at the site, based on the ground investigation results.



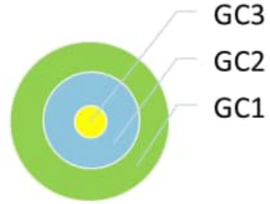
Illustration



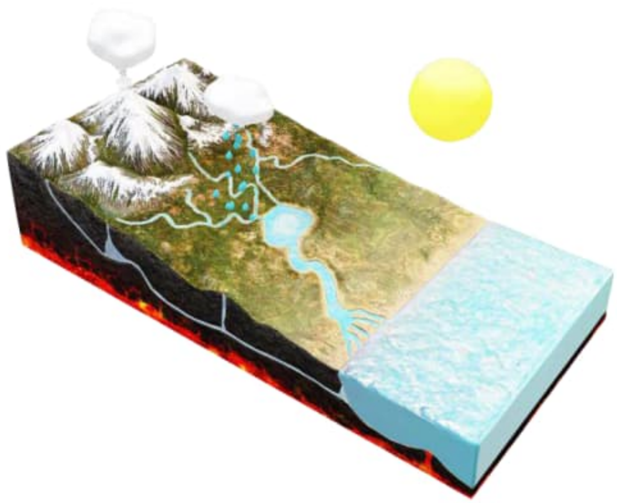
5.1.2.e

Additional aspects of basis of design

Ground Model



EN 1997-2, 4.1(3)
The detail and the extent of the Ground Model shall be consistent with the **Geotechnical Category** and the **zone of influence** of the structure.



Geotechnical Reliability
To ensure appropriate reliability that the Geotechnical structure during its design service life will:
sustain all foreseeable actions,
meet specified serviceability requirement, and
meet specified durability requirement.

Consequence Class (CC)	Geotechnical Complexity Class (GCC)		
	Low (GCC1)	Normal (GCC2)	High (GCC3)
High (CC3)	GC2	GC3	GC3
Normal (CC2)	GC2	GC2	GC3
Low (CC1)	GC1	GC2	GC2

GC1


GC2


GC3


Additional aspects of basis of design

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[EN 1997-1, 4.2.3.1]

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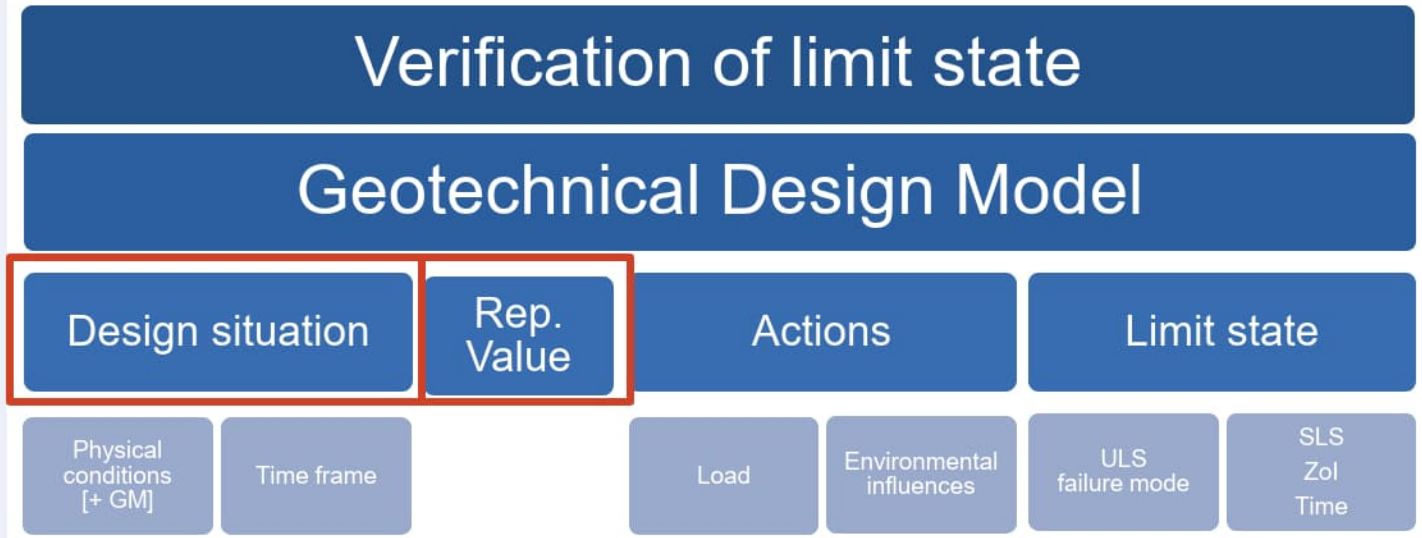


Additional aspects of basis of design

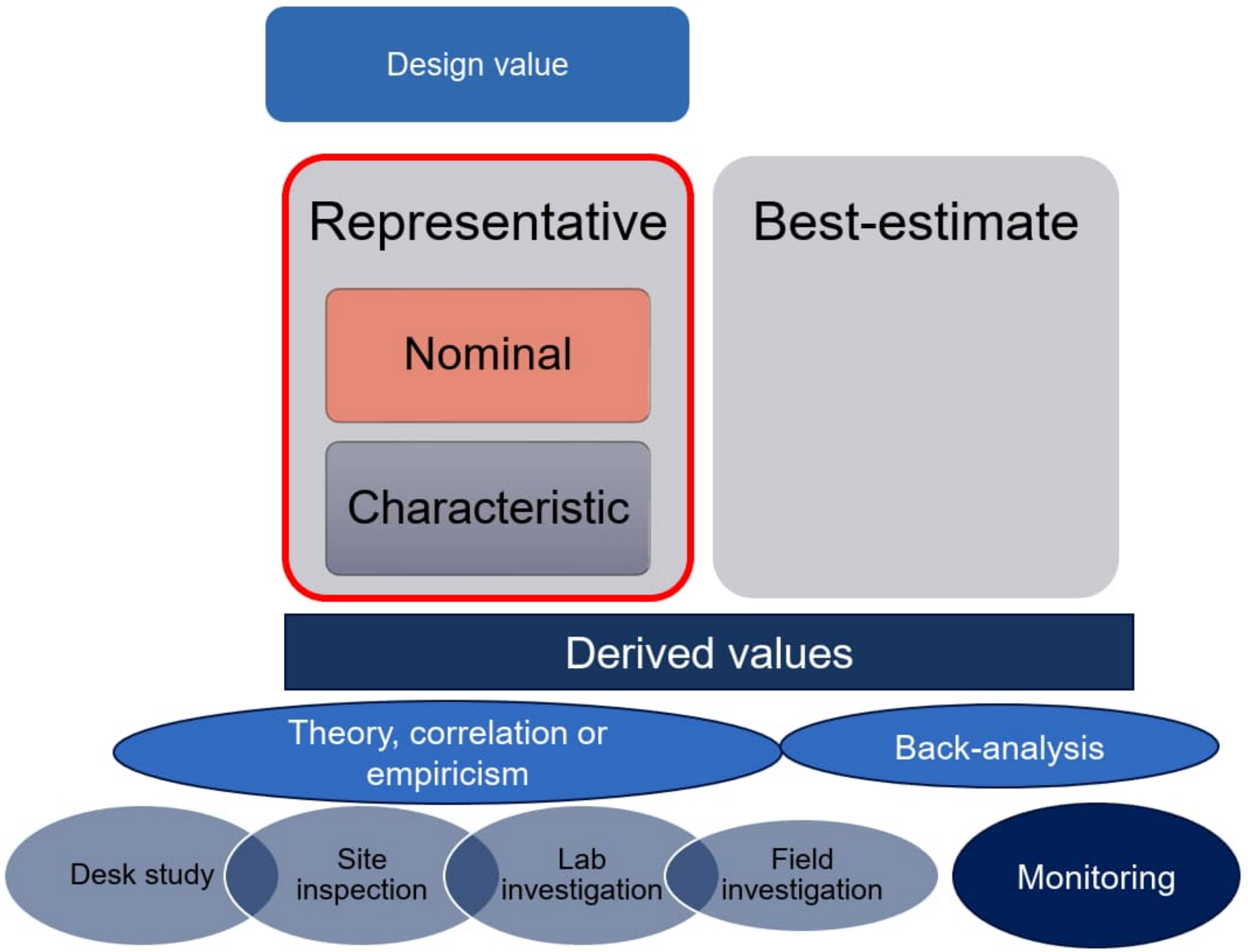
Representative value

EN 1997-1, 4.3.2

Representative value of ground properties

Characteristic – statistical evaluated
Nominal – selected cautious estimate

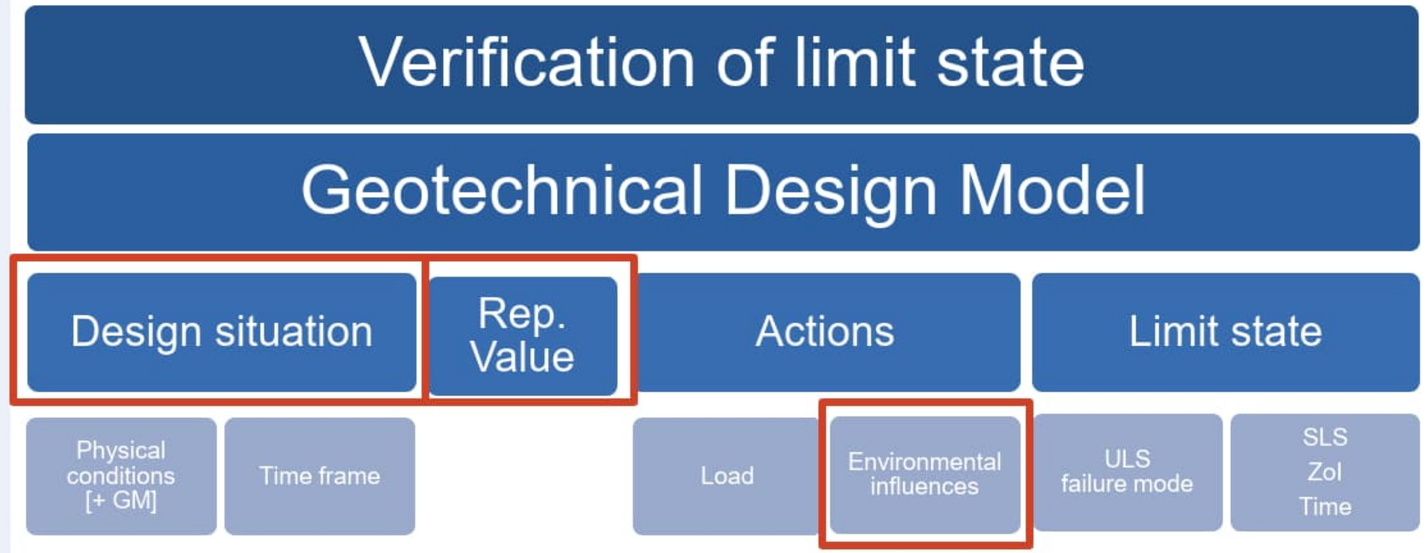
Best-estimate – for prognosis



Additional aspects of basis of design

Geotechnical Design Model, GDM

[EN 1997-1, 4.2.3.1]



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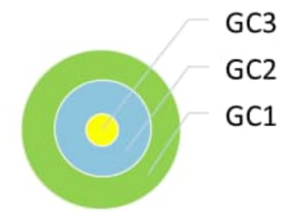
- Based on a validated Ground model.
- Include representative values for geotechnical units in the zone-of-influence

Acceptable reliability



Actions – environmental influences

EN1997-1 4.3.1.4

Action

The adverse effect:

- Temperature, climate variation
- Freezing/thawing
- Execution:
  - Mass displacement
  - Increase groundwater pressure
- Biological activity

Design situation

The adverse effect:

- Underground spaces
- Freezing/thawing
- Dewatering activities
- Climate change effects
- …….

Durability

The adverse effect:

- Temperature, climate change
- Electro-chemical composition
- Contamination
- Mineralogical composition
- ….

Ground properties

The adverse effect:

- Temperature, climate change
- Freezing/thawing
- Biological activity
- …..

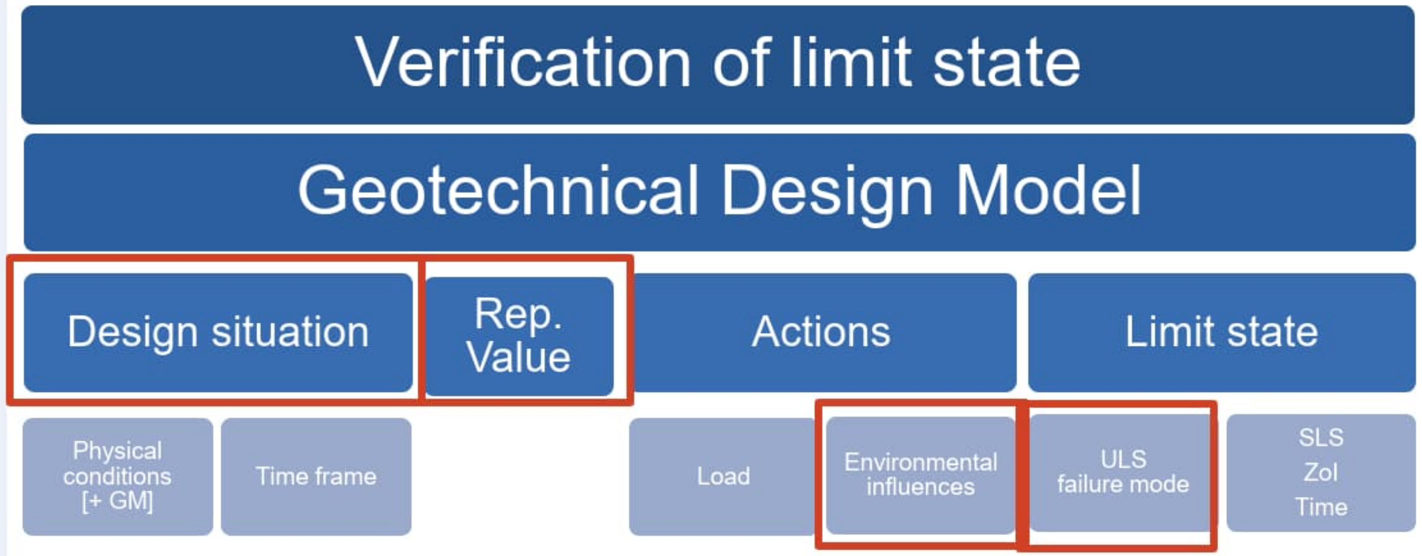
5.1.2.e

Additional aspects of basis of design

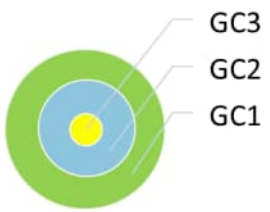
Geotechnical Design Model, GDM
[EN 1997-1, 4.2.3.1]

The basis of design is compiled in a GDM.
"GDM a carrier of the needed information for your verification"

GDM shall
- Be developed for each design situation.
- Include corresponding combination of actions.
- Include associated relevant limit states.
- Based on a validated Ground model.
- Include representative values for geotechnical units in the zone-of-influence



Acceptable reliability



Ultimate limit state

EN1997-1, 8

Rupture
[EN 1997-1, 8.1.1]

- Rupture in ground
- Translation or rotational failure
- Bearing capacity
- Loss of geotechnical resistance of element in the ground

Excessive deformation
[EN 1997-1, 8.1.2]

- Failure in structural element due to deformation in ground
- Failure of existing structural element due to execution of another structure

Loss of static equilibrium
EN 1997-1, 8.1.3

- Loss of rotational equilibrium
- Loss of vertical equilibrium (uplift)

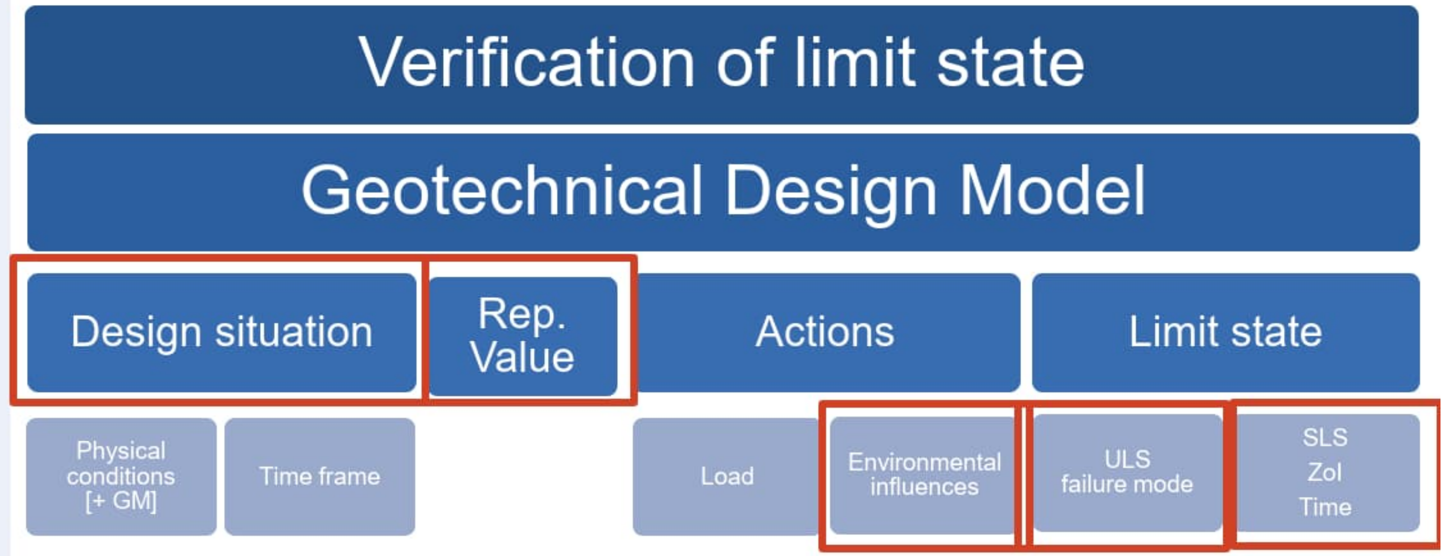
Hydraulic failure
EN 1997-1, 8.1.4

- Hydraulic heave
- Internal erosion and piping

Additional aspects of basis of design

Geotechnical Design Model, GDM

[EN 1997-1, 4.2.3.1]



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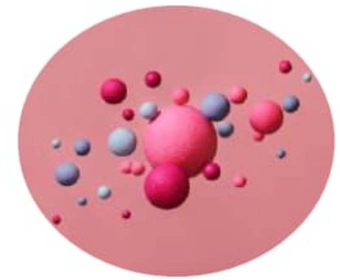
5.1.2.e

Serviceability limit state
EN1997-1, 9



Impact within zone-of-influence
EN 1997-1, 4.2.5

Ground movements [EN 1997-1, 9.3]

Account for

- Loading distribution
- Consolidation creep
- Changes in groundwater
- Degradation – cyclic effects
- Changes in zone of influence
- ……..

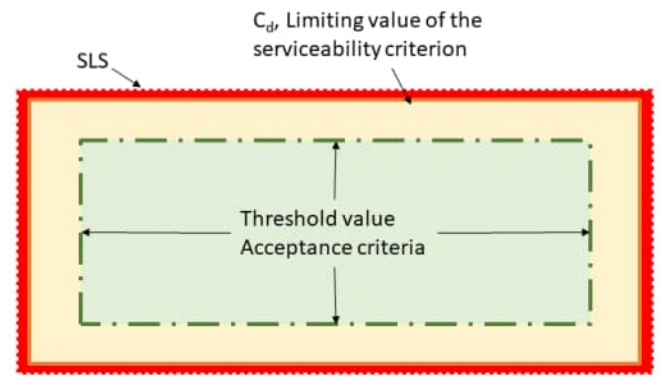
Structural aspects [EN 1997-1, 9.4]

Account for

- Occurrence of ground movement
- Relative movement within structure
- Acceptable value of movements (confidence in)
- ……

Hydraulic aspects EN 1997-1, 9.5

Serviceability criterion to avoid failure of the structure or server impact within the zone of influence



Serviceability criteria
EN 1997-1, 9.2

Robust, Durable and Sustainable

EN1997-1, 4.1.4, 4.1.6, 4.1.7

Robustness
[EN 1997-1, 4.1.4]

Definition EN 1990, 3.1.2.30

ability of a structure to withstand **unforeseen adverse** events without being damaged to an extent disproportionate to the original cause

Durability
[EN 1997-1, 4.1.6]

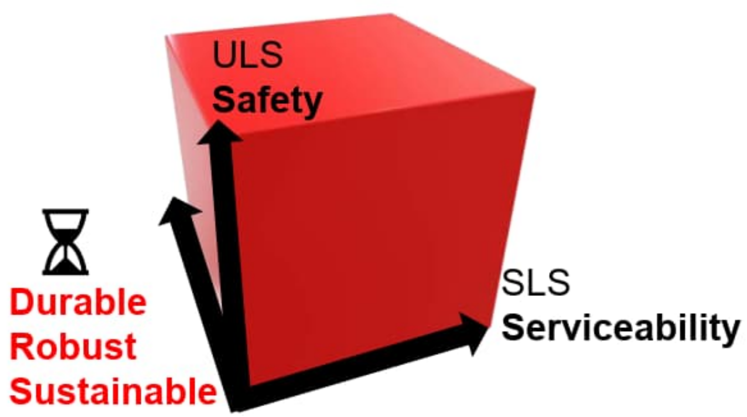
Definition EN 1990, 3.1.2.31

ability of a structure or structural member to satisfy, **with planned maintenance**, its design performance **requirements over the design service life**

Sustainability
EN 1997-1, 4.1.7

Definition EN 1990, 3.1.2.32

ability to **minimize the adverse impact** of the construction works on non-renewable resources in the environment, on society, and on economy **during their entire life cycle**



Additional aspects of basis of design

Geotechnical Design Model, GDM
[EN 1997-1, 4.2.3.1]

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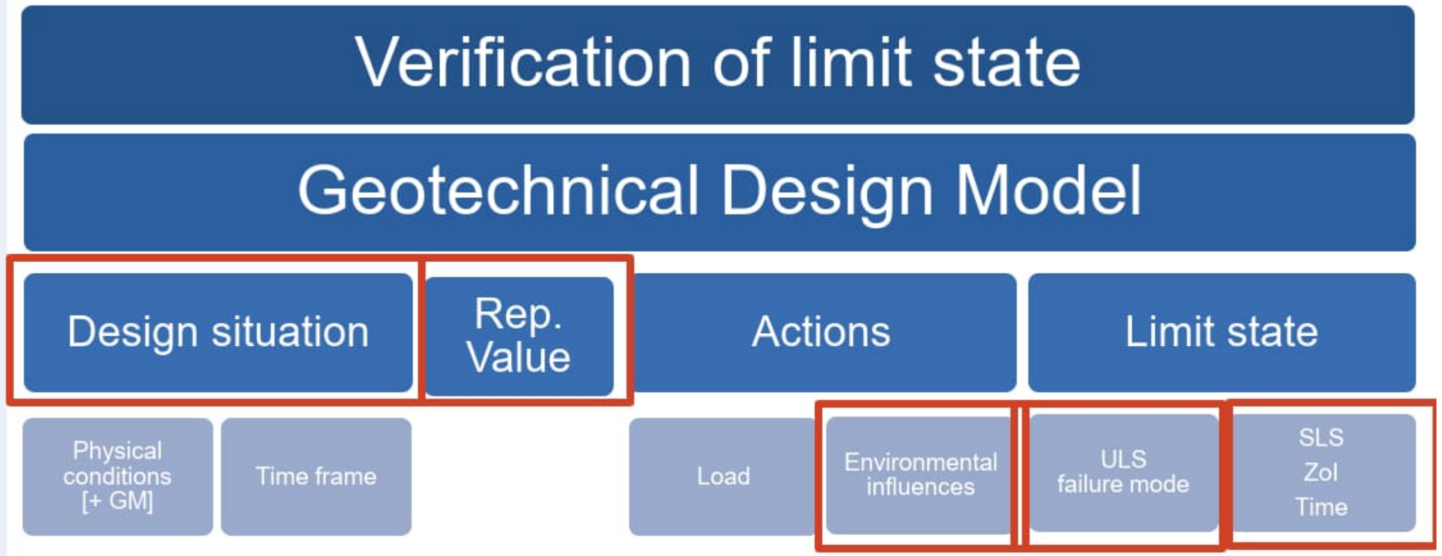
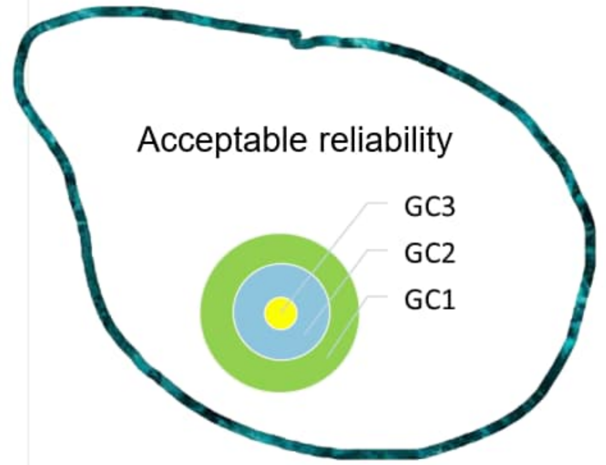




Additional aspects of basis of design

Validation of GDM

EN1997-1, 4.2.3.2

Validate the information in the Ground Model (EN 1997-1, 4.2.4)

Ground Investigation Report

Validate your Geotechnical Design Model (EN 1997-1, 4.2.3.2)

Do you reach the reliability needed for your structure (GC)?

If not – additional information/investigation is needed!

The validation give you information about the remaining uncertainties in your GDM. Account for these in your verification!

Acceptable reliability

Consequence Class (CC)	Geotechnical Complexity Class (GCC)		
	Low (GCC1)	Normal (GCC2)	High (GCC3)
High (CC3)	GC2	GC3	GC3
Normal (CC2)	GC2	GC2	GC3
Low (CC1)	GC1	GC2	GC2

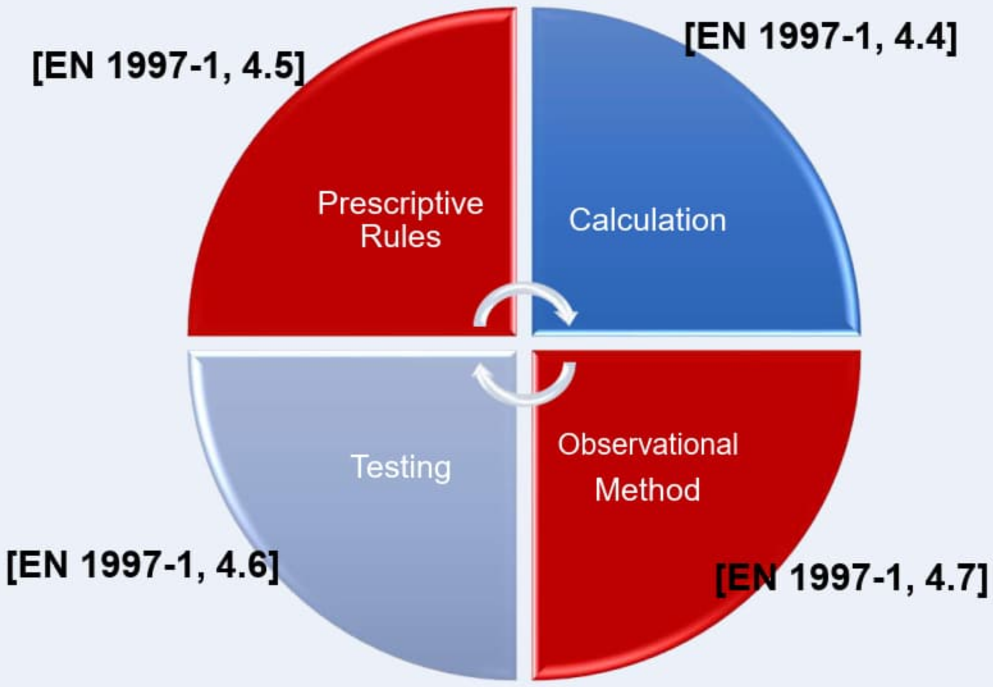
Table 4.5(NDP) — Measures to validate the Geotechnical Design Model

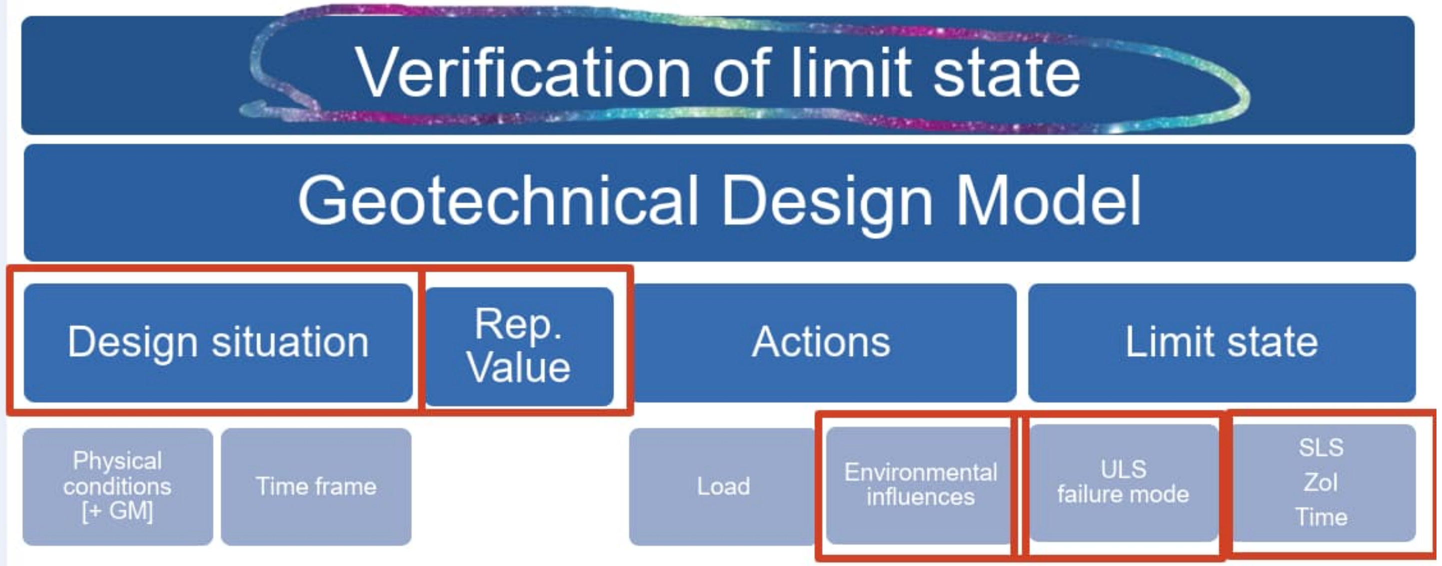
Geotechnical Category	Measures
GC3	All items given for GC2 and, in addition: — sensitivity analyses of key ground properties for the design to identify need of additional information to cover all anticipated design situations; — sensitivity analyses of key geometrical properties for the design to identify need of additional measures; — check that the information available is sufficient to determine the variability of the ground properties and groundwater conditions.
GC2	All items given for GC1 and, in addition: — comparison of derived values from different sources within each geotechnical unit to determine representative values of ground properties with appropriate level of confidence; — check that GDM includes all ground properties and groundwater conditions affecting the design situation; — check that GDM is appropriate and compatible with the considered ultimate limit states (failure modes) and serviceability limit states; — check that the ground properties are determined for a time frame compatible with the considered limit states and design situation.
GC1	All items given below: — check the consistency of assumed geotechnical units and geotechnical properties with available information from the desk study and comparable experience; — confirmation of the Geotechnical Design Model with information from site inspection.

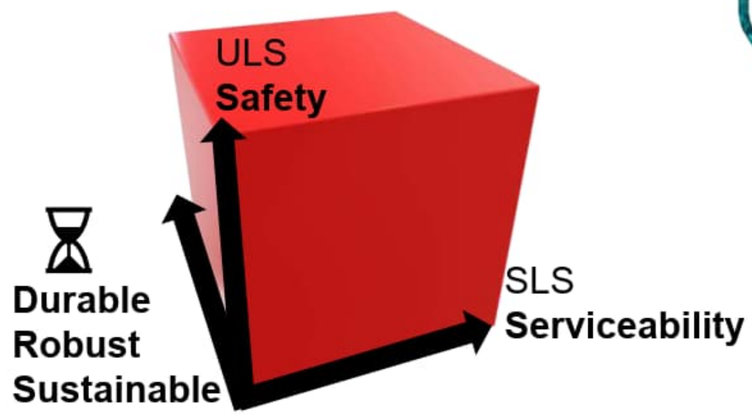
5.1.2.e

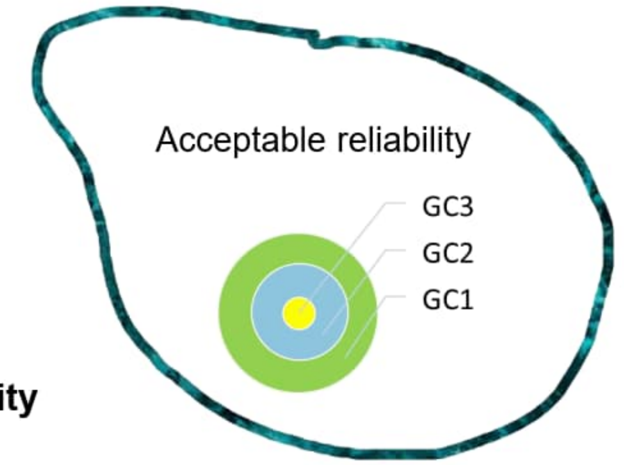
Additional aspects of basis of design

Verification
[EN 1997-1, 4.4, 4.5, 4.6, 4.7]

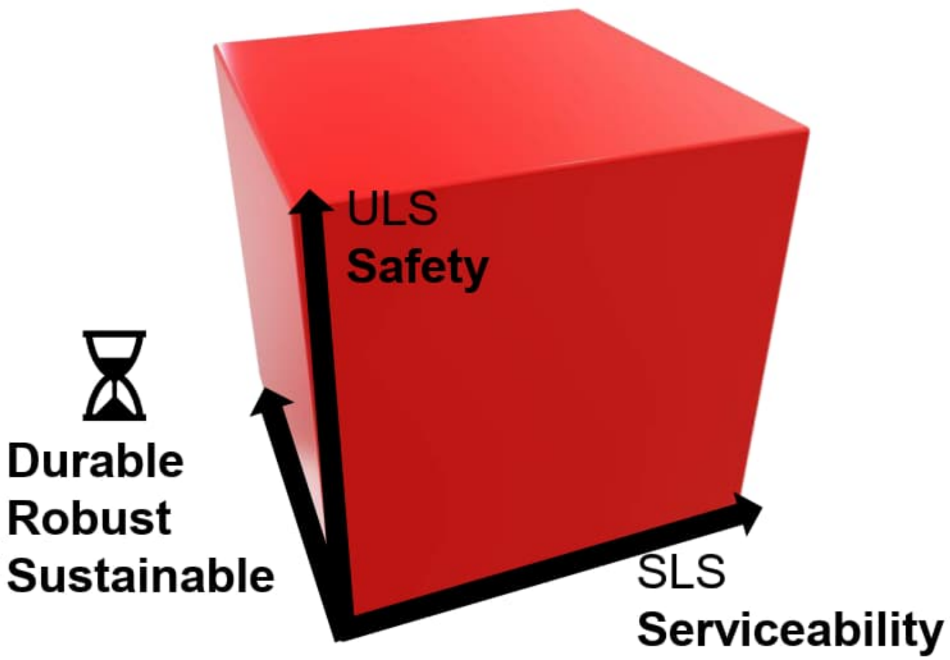








SC7 hope you will be successful
using the updated toolbox to verify that your
geotechnical structure is within the cube.

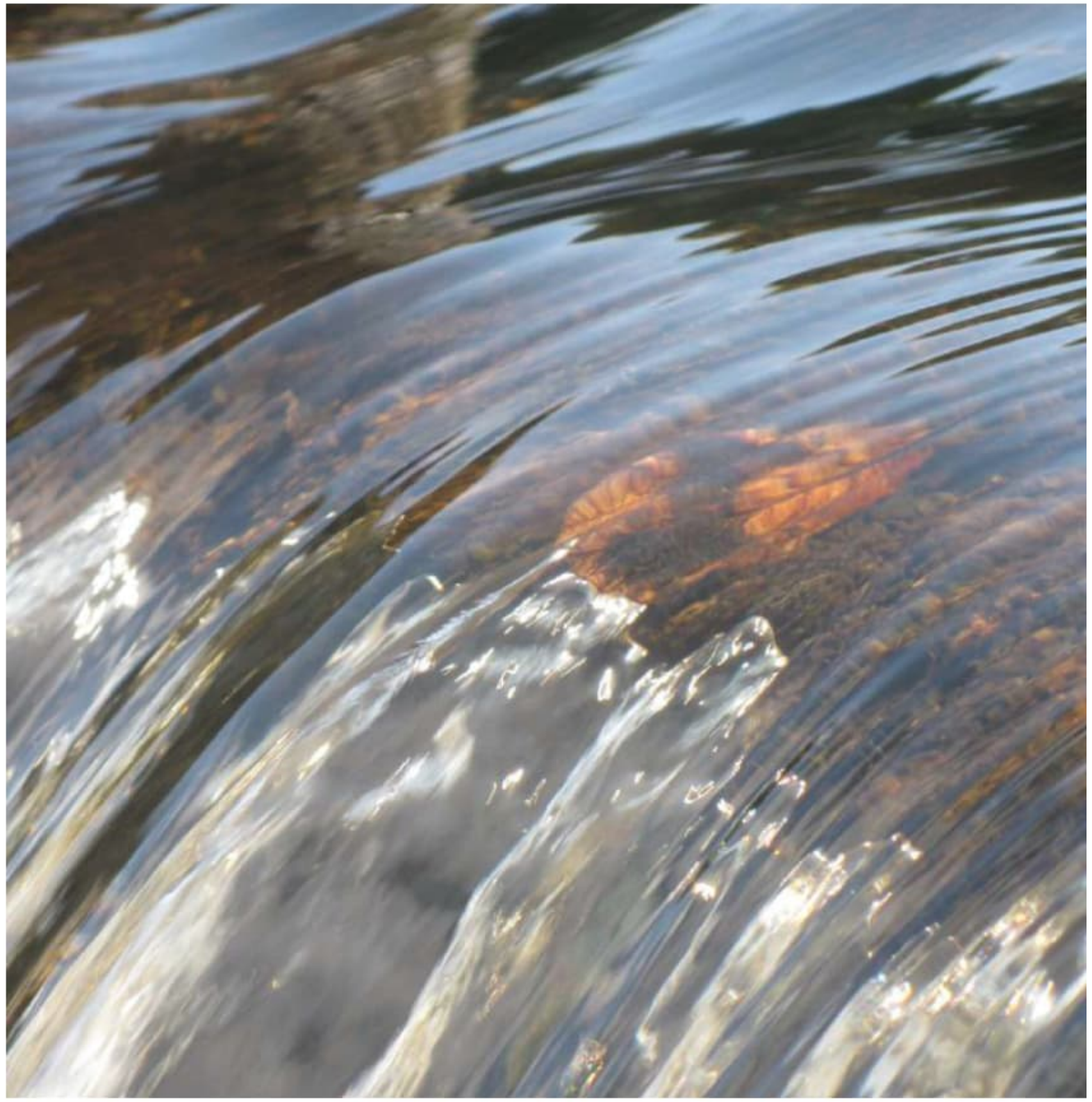


Thanks for listening!

Gunilla Franzén

Vice chair TC250 SC7 Geotechnical Design

Mail: gunilla.franzen@geoverkstan.se



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5.1.2.e

Second generation of Eurocode 7

Basis of design and how the influence of Groundwater is incorporated in the new code

22th of February 2023 Online 15:00 – 17:00 CEST

Groundwater aspects

5.1.2.e

Contents

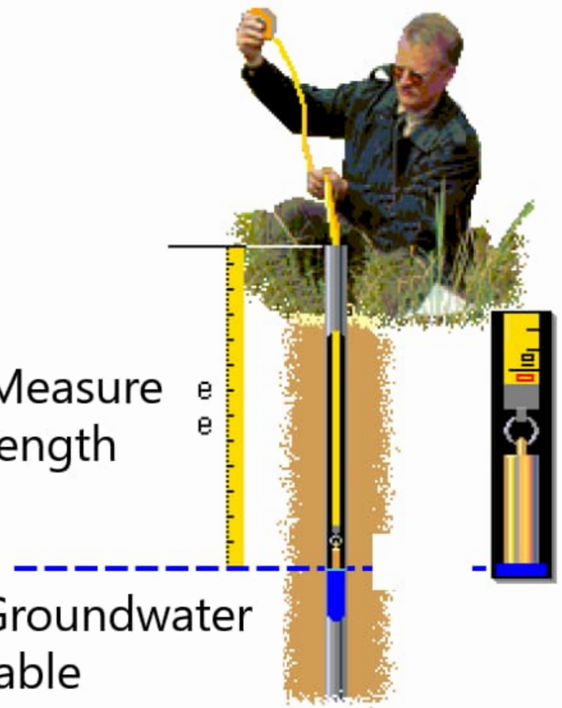
2nd Generation Eurocode 7 – focus on groundwater

1. **Groundwater levels and pressures**
2. **Limit states in groundwater design**
3. **Obtaining geo-hydraulic parameters**
4. **Groundwater control measures**

See also: examples – Joaquin Perez

Groundwater levels and pressures

Measurement of Groundwater levels



Hand readings

Automatic

Groundwater levels and pressures

Definitions in EN1997-1:

Groundwater level = level of water surface in the ground

Piezometric level h_w = level of water in a standpipe

Groundwater pressure u:

$$u = (h_w - z)\,\gamma_w$$

z = elevation, where u is measured (+ upwards)

γ_w = density of water

Representative values of groundwater pressures / levels (1)

6.4 Representative values of groundwater pressures

(1) If there is sufficient data to derive its value on the basis of the annual probability of exceedance, the **representative value of groundwater pressure $F_{w,rep}$** should be selected as either:

- a **single permanent value**, equal to the **characteristic upper $G_{wk;sup}$ or lower $G_{wk;inf}$** value of groundwater pressure (whichever is more adverse according to the considered limit state);
- the **combination** of:
  - a **permanent value G_{wk}**, equal to the **mean value** of groundwater pressure, and
  - a **variable value,** equal to the **representative value $Q_{w,rep}$** of the variation in groundwater pressure.

NOTE 1 The values of $G_{wk,sup}$ **and** $G_{wk,inf}$ are based on an annual **probability of exceedance of 2 %** (which corresponds to a return period of 50 years), unless the National Annex gives a different value.

(2), (6): If **not** sufficient data → **nominal values, cautious estimates**

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Representative values of groundwater pressures / levels (2)

6.4 (3) The representative value $Q_{w,rep}$ of the amplitude of the variation in groundwater pressure shall be selected as one of the following, depending on the design situation:

- the characteristic value Q_{wk} — 2 % probability of exceedance (1/50 years)
- the combination value $Q_{w,comb}$ — 5 – 10 % probability of exceedance (1/20 or 1/10 years)
- the frequent value $Q_{w,freq}$ — 1 % of the reference period
- the quasi-permanent value $Q_{w,qper}$. — mean

NOTE The values of Q_{wk}, $Q_{w;comb}$, $Q_{w;freq}$, and $Q_{w;qper}$ are based on the probabilities of exceedance given in prEN 1990:2021, 6.1.3.2.

Accidental loading: *Aw;d* → 0.1 % probability of exceedance (1/1000 years)

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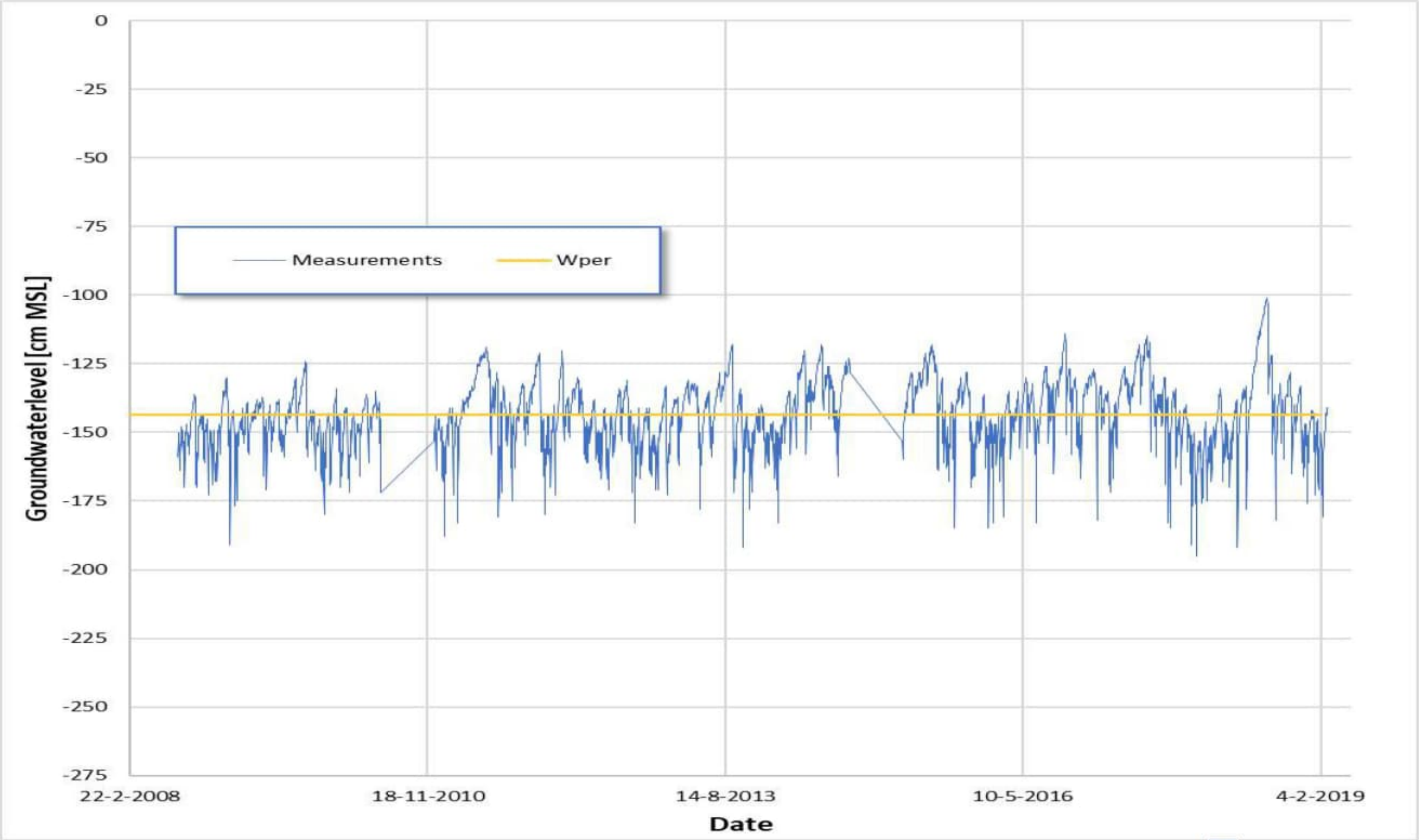
Determining representative values

How does it work?

Piezometer readings

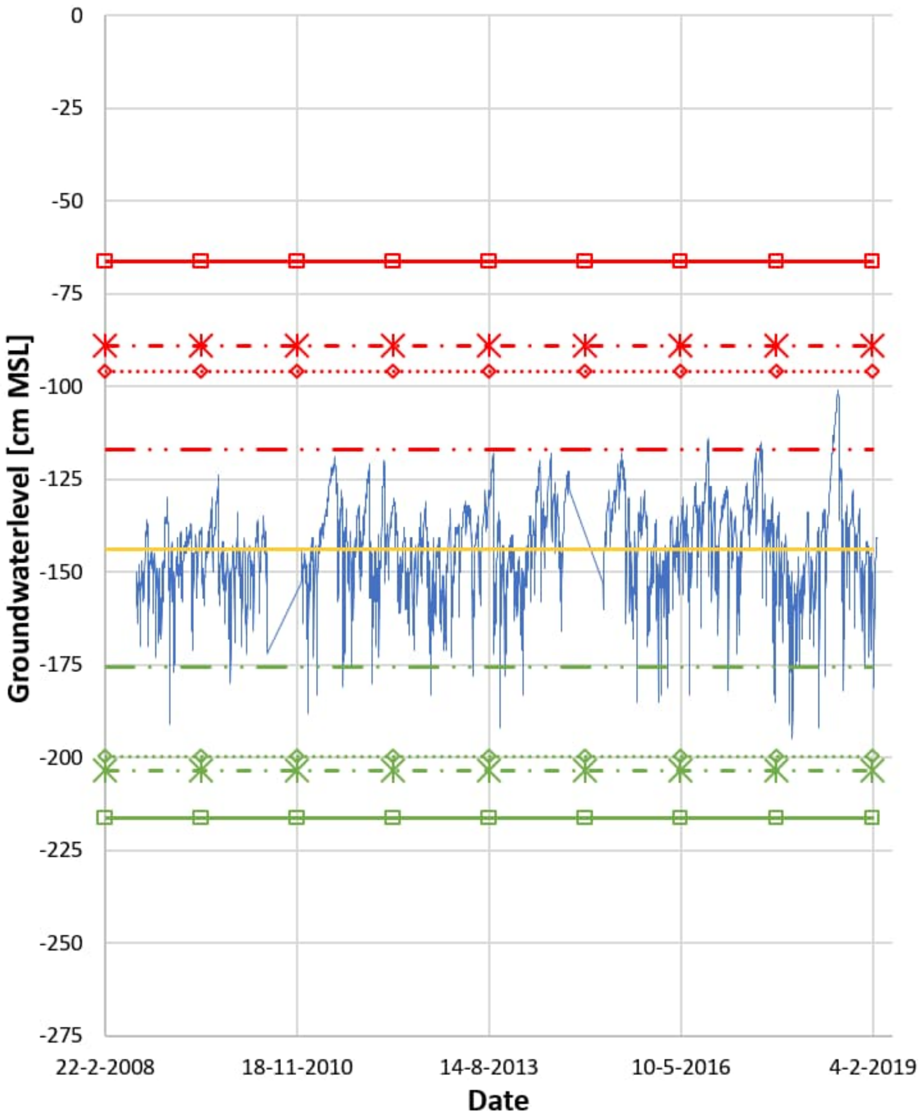


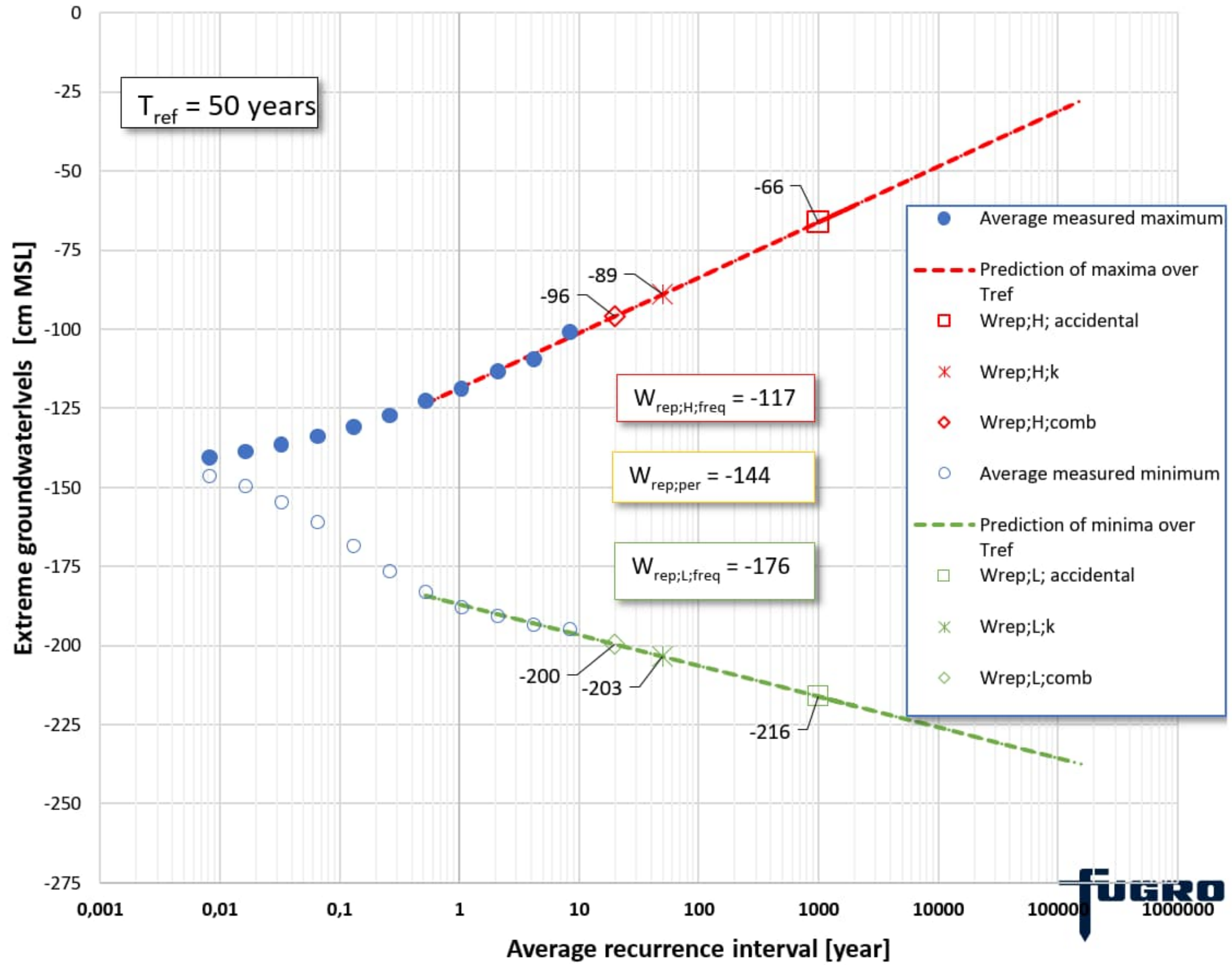
Many thanks to

Hans Brinkman

DELTARES

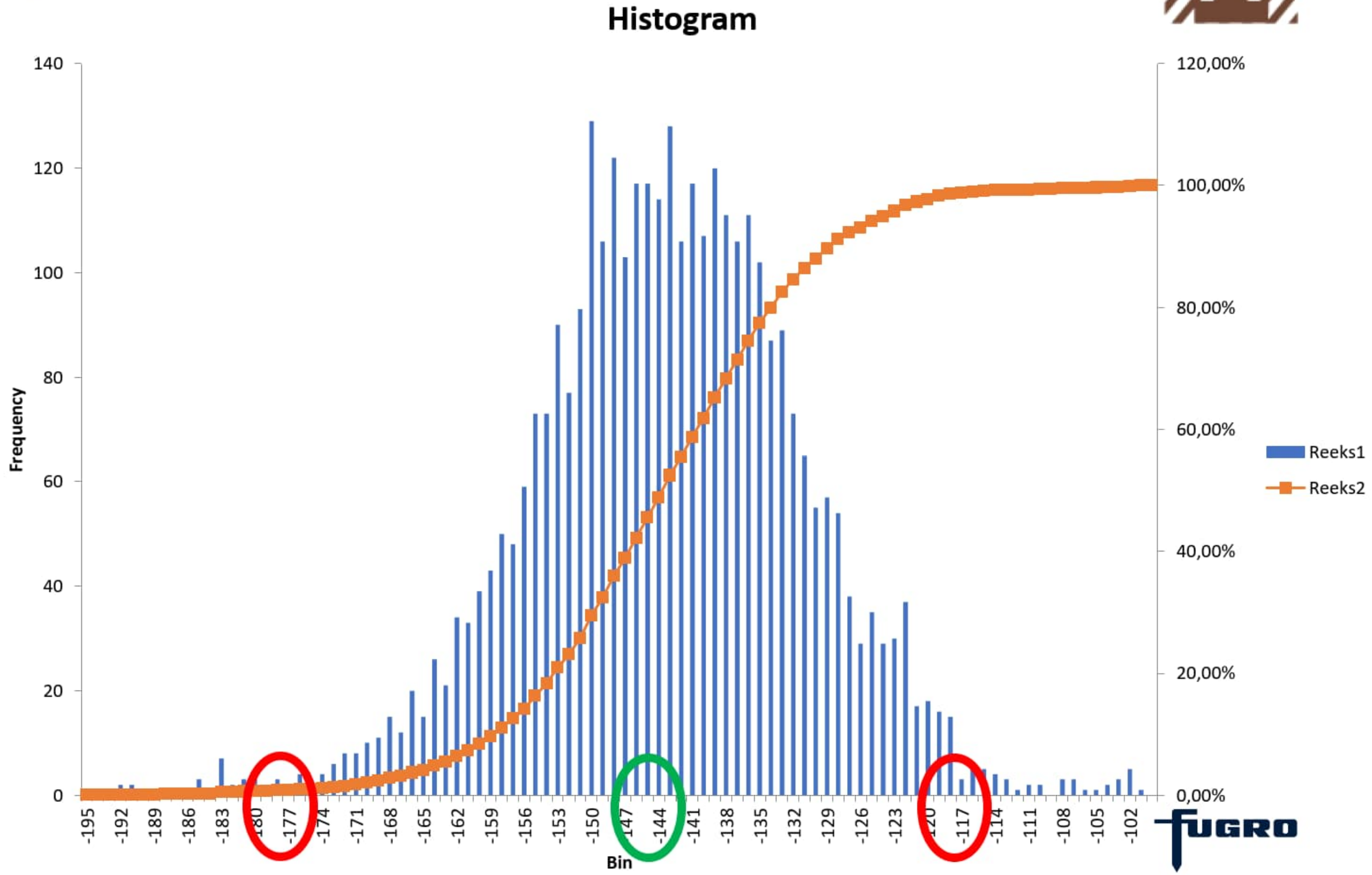
Interpretation of Groundwater levels





Frequent value

Groundwater level is exceeded during 1 % of the period



Representative high values of waterlevels (Mean Sea Level)

	Permanent G_w (m MSL)	Variable Q_w (m MSL)
Average G_{wk}	- 1.44	
Characteristic	- 0.89	0.55
Combination	- 0.96	0.48
Frequent	- 1.17	0.27
Quasi-permanent	- 1.44	0.00
Accidental	- 0.66	0.78

6.5 Design values

From Representative to Design values for ULS:

6.5.1 (1) Design values of groundwater pressures in ultimate limit states shall be determined by one of the following methods:

- **direct assessment;** or
- applying a **deviation** to the representative **piezometric level** or to the representative **groundwater pressure**; or
- applying a **partial factor** to the representative groundwater **pressures** or to their action **effects.**

NOTE 1 Methods that involve direct assessment or application of a deviation are usually suitable in cases where groundwater pressures are used to calculate shear strength from effective stresses (e.g. overall stability analyses or retaining wall design). Application of a partial factor is usually suitable in cases where groundwater pressures are used to calculate forces and bending moments on structural elements.

NOTE 2 The value of the partial factor is given in prEN 1990:2021, Table A.1.8.

Design values (2)

Dutch National Annex EN1990 – Surface + Groundwater

Method 1 – Statistics

Method 2 – Estimate highest design value $G_{w;d}$:

$G_{w;d}$ = $G_{wk;sup}$ + k * ($G_{wk;sup}$ – G_{wk}) . k = 1.0 (general), k = 0.5 (tidal areas)

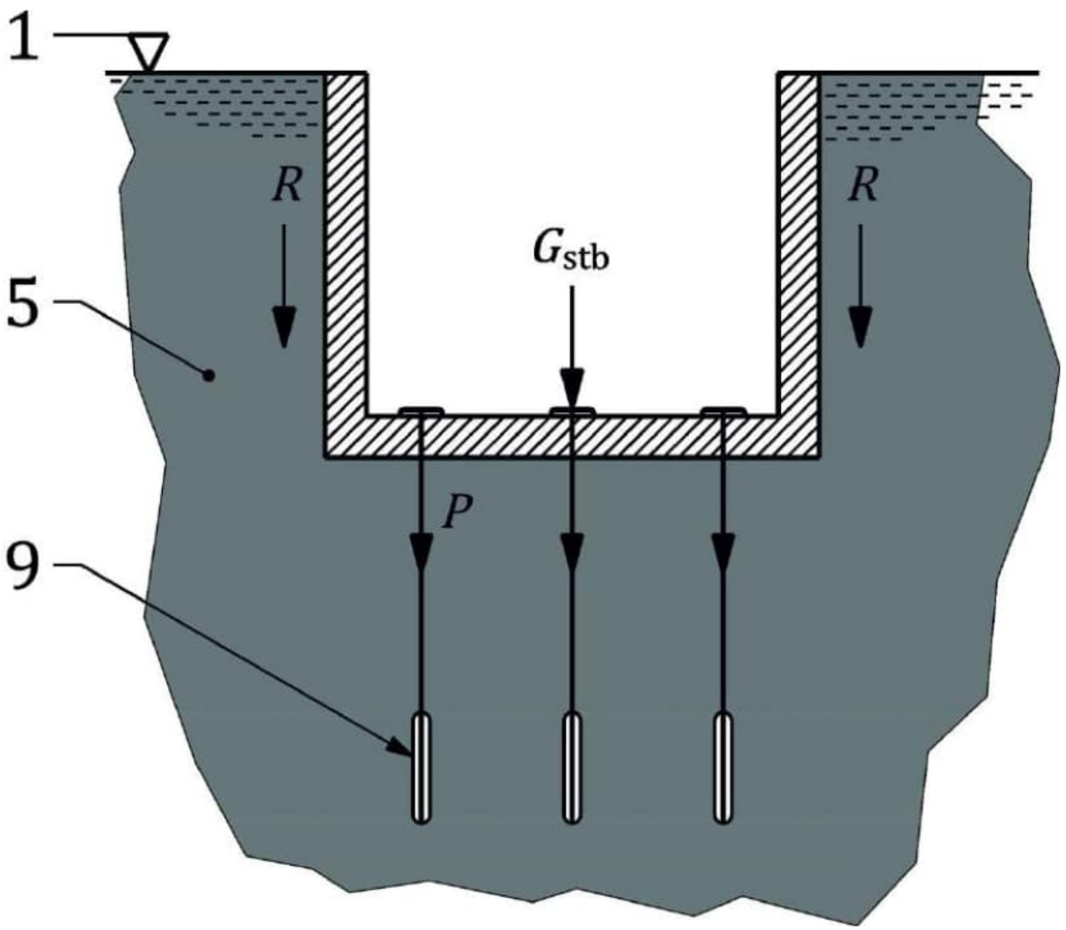
$G_{wk;sup}$ = highest characteristic value

G_{wk} = mean value

Minimum: $G_{w;d}$ = $G_{wk;sup}$ + 0.3 m

Check on Uplift – EN1997-1 8.1.3.2 (2) – Rigid structure

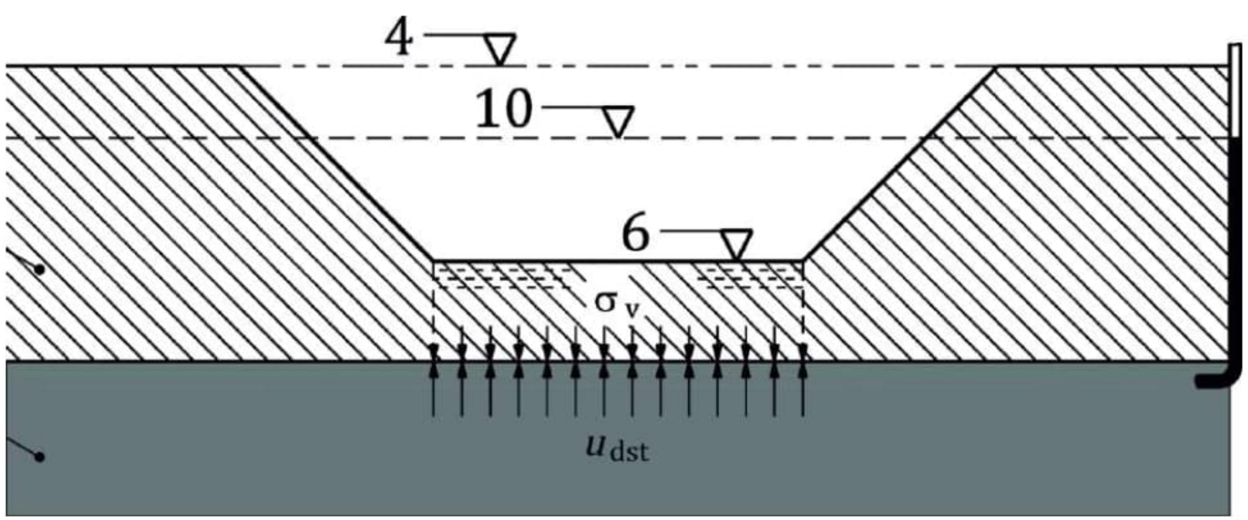
$$U_{d,dst} + G_{d,dst} + Q_{d,dst} - G_{stb} \leq R_d$$



$U_{d,dst}$	destabilising (uplift) force due to groundwater;
$G_{d,dst}$	any permanent destabilizing force (upwards) not caused by groundwater;
$Q_{d,dst}$	any variable destabilizing force (upwards) not caused by groundwater pressures;
$G_{d,stb}$	stabilizing (downward) forces;
R_d	any resistance to uplift.

Partial action factors VC2 (a) and VC2 (b)			
U_{dst}	1.2 / 1.0	Q_{dst}	1.5 / 1.5
G_{dst}	1.35 / 1.0	G_{stb}	1.15 / 1.0

Check on Uplift – EN1997-1 8.1.3.2 (2) – Non-Rigid



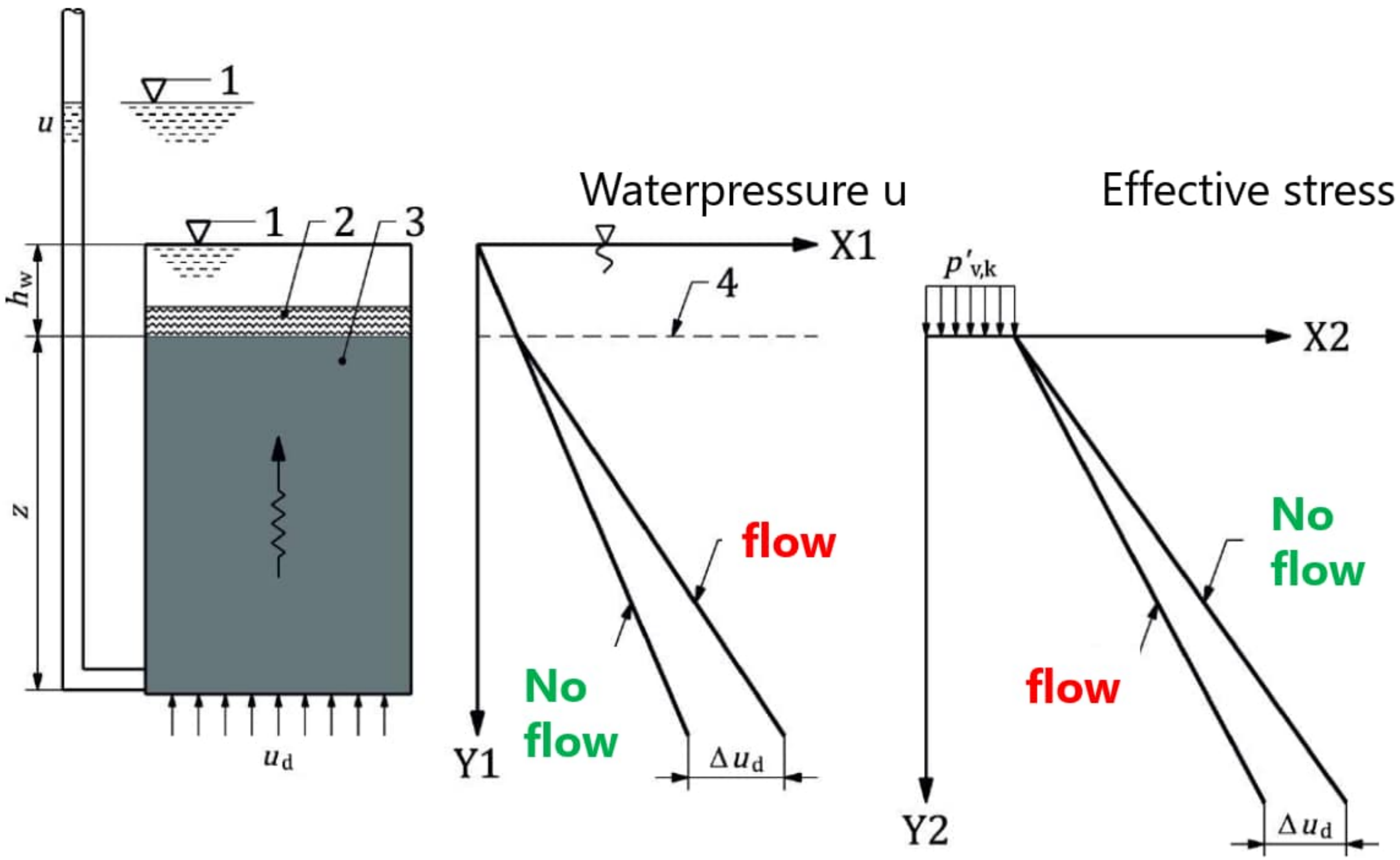
$$u_{d;dst} - \sigma_{v;d} \leq 0$$

$u_{d,dst}$ destabilizing (uplift) groundwater pressures;

$\sigma_{v;d}$ (stabilizing) vertical total stress at the base of the layer that is subject to uplift.

Partial action factors VC2 (a) and VC2 (b)	
u_{dst}	1.2 / 1.0
σ_v	1.0 / 1.0

Hydraulic failure – EN1997-1 8.1.4.2



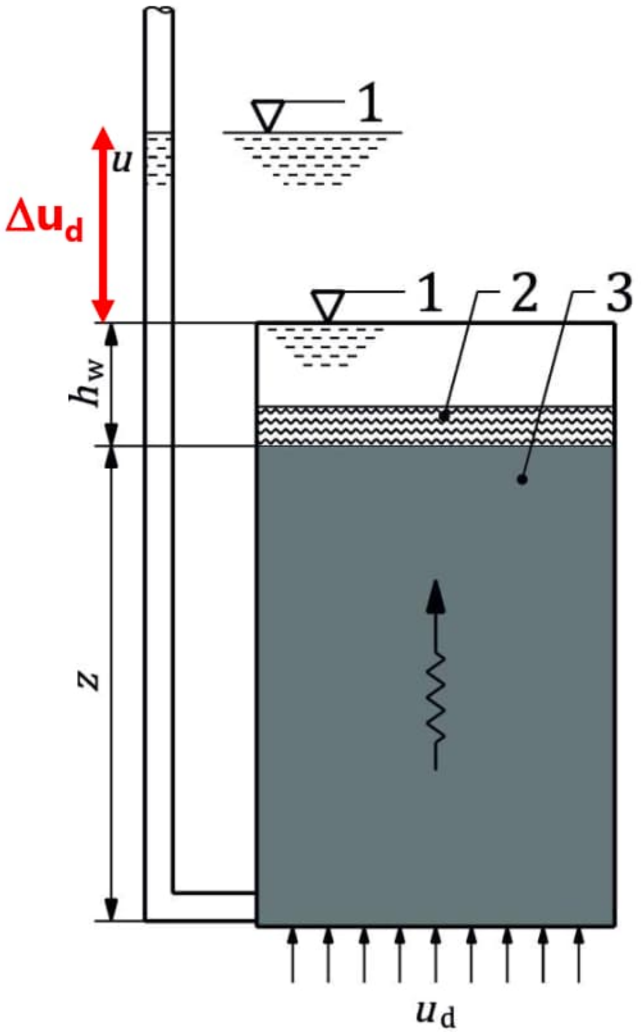
Heave

Upward flow

Effective stress > 0.33 * original value

Δu_d is excess groundwater pressure

Hydraulic failure – EN1997-1 8.1.4.2 Heave



$$\Delta u_d \leq \gamma_{HYD}(\gamma_{rep} - \gamma_{w,rep})z + \gamma_{pv}\, p'_{v;rep}$$

Δu_d	excess groundwater pressure = $u_d - u_0$;
u_d	groundwater pressure in the presence of flow;
u_0	groundwater pressure in the absence of flow (hydrostatic);
$\gamma_{w,rep}$	weight density of the groundwater;
z	vertical distance of the point in the ground below the ground surface (not including any overlying fill);
h_w	vertical distance from the surface water level to the ground surface;
γ_{rep}	weight density of the ground;
$p'_{v,rep}$	value of any effective overburden pressure at the ground surface;
γ_{HYD}	partial factor for hydraulic heave = 0.67;
γ_{pv}	partial factor on the effective overburden pressure = 0.67.

Hydraulic failure – EN1997-1 8.1.4.3 (3)

Internal Erosion and Piping

$$i_d \leq i_{cd}$$

i_{d} hydraulic gradient;

i_{cd} critical hydraulic gradient.

(4) The critical hydraulic gradient for internal erosion and piping should consider:

- the direction of flow;
- the grain size distribution and shape of grains;
- layering of the ground.

NOTE 1 Values of i_{cd} depend on particle size and soil grading. Typical values between 0.3 and 0.9.

NOTE 2 Methods to determine i_{cd} are given in The International Levee Handbook, CIRIA Report C731 (2013).

EN1997-2 Clause 11
Groundwater and geohydraulic properties

11.2 Measurement of groundwater pressure and pressure head

11.3 Determination of Geohydraulic properties – Hydraulic Conductivity K:

- constant and faling head laboratory tests
- testing in borehole – open and closed systems
- water pressure tests in rock mass
- pumping tests
- infiltrometer tests
- CPT – porewater dissipation tests

Groundwater control measures – EN1997-3 Clause 12.1

(4) Groundwater control measures should be classified according to Table 12.1.

Table 12.1 Classification of groundwater control measures

Class	Measures	Objective	Examples
1	Adjustment of hydraulic conductivity	Reduce leakage through ground	Grouting, soil mixing, leakage prevention using natural clay layer
2	Dewatering/infiltration	Control groundwater and/or surface water level	Drains, wells
3	Impermeable barriers	Prevent (i.e. cut off) the flow of groundwater	Sheet pile with jet grouting, plastic and geo-synthetic barriers, ground freezing.

Groundwater control measures – EN1997-3 Clause 12.7

SERVICEABILITY LIMIT STATE

(2) It shall be verified, throughout the zone of influence, that the groundwater control measures fulfil the inequality given by Formula (12.1):

$$C_{d,SLS,min} \leq E_d \leq C_{d,SLS,max}$$

E_d	effect of actions caused by the groundwater, after application of groundwater control measure
$C_{d,SLS,min}$	minimum design value of the relevant serviceability criterion for the considered geotechnical structure within the zone of influence; and
$C_{d,SLS,max}$	maximum design value of the relevant serviceability criterion for the considered geotechnical structure within the zone of influence

NOTE 1. E_d can be expressed as, for example:

- groundwater or surface water pressure;
- hydraulic conductivity;
- rate of flow of water.

NOTE 2. $C_{d,SLS}$ can be expressed as, for example:

- minimum or maximum groundwater/surface water pressure;
- minimum or maximum hydraulic conductivity;
- minimum or maximum rate of flow of water.

Closure – Eurocode 7 gives the rules….

2nd Generation Eurocode 7 - focus on Groundwater (Part 1):

Selection of representative groundwater levels and pressures

Design groundwater levels and pressures

Ultimate Limit States for:

- Uplift, Hydraulic Heave, Hydraulic gradient

Determination of groundwater pressures and hydraulic conductivity (Part 2)

Groundwater control measures (Part 3)

- Seviceability Limit State

To avoid……

Thank you

(Chair-ec7@fugro.com)

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Unlocking **Insights**
from **Geo-data**

Second generation of Eurocode 7

Basis of design and how the influence of Groundwater is incorporated in the new code

22nd of February 2023 Online 15:00 – 17:00 CEST

Groundwater related limit states – examples

SIMSG ISSMGE

WEBINAR – 22 February 2023

Basis of design and how the influence of Groundwater is incorporated in the new code

Groundwater related limit states – examples

[redacted] 5.1.2.e

University of Malaga (Spain)

uma.es

Standaard voor vooruitgang nen

Scope:

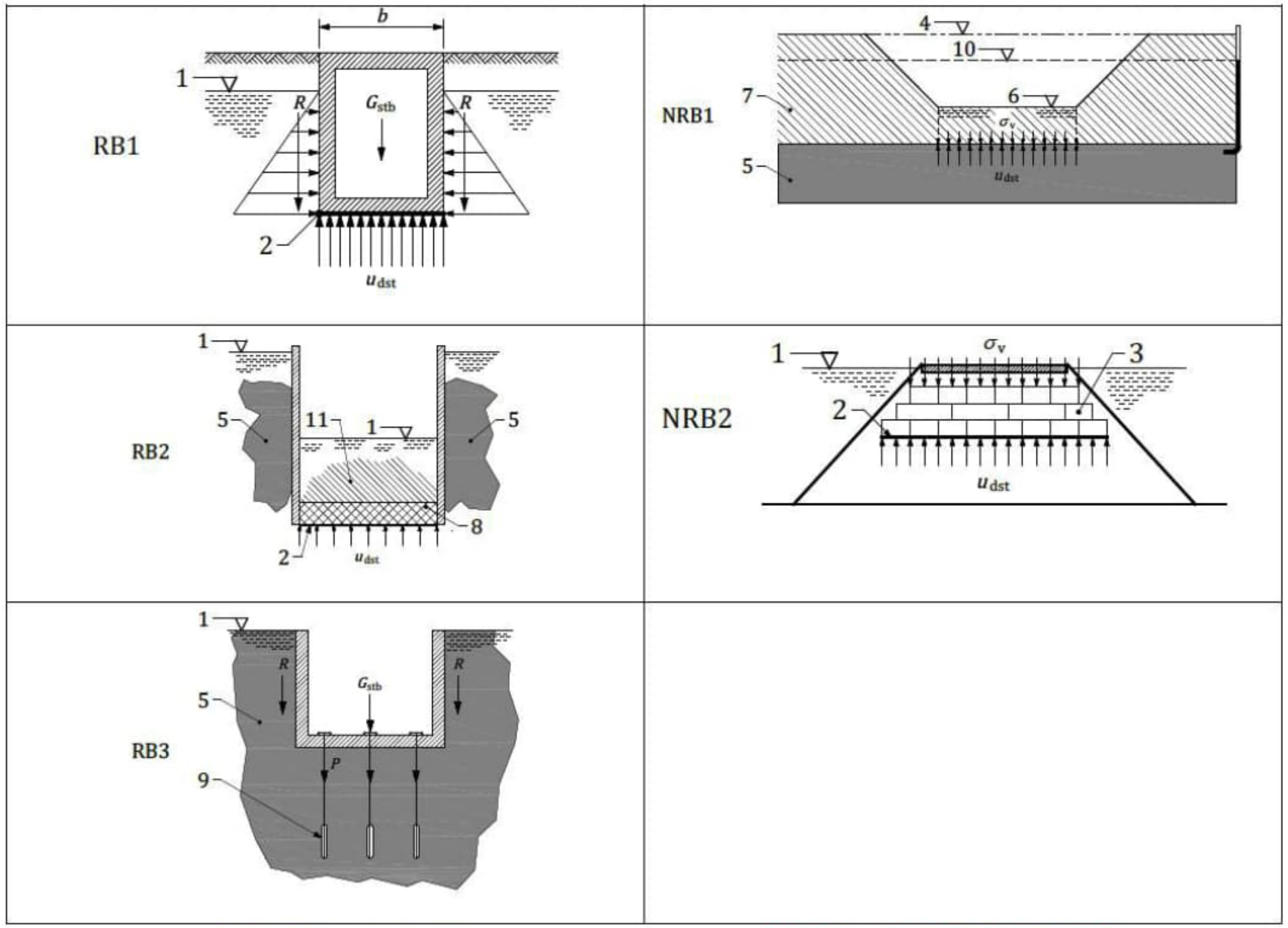
Examples on groundwater related ULS:

- Loss of vertical equilibrium due to uplift (2)

- Hydraulic heave (1)

Loss of vertical equilibrium due to uplift

Examples of design situations where uplift might be critical (§ 8.1.3.2)



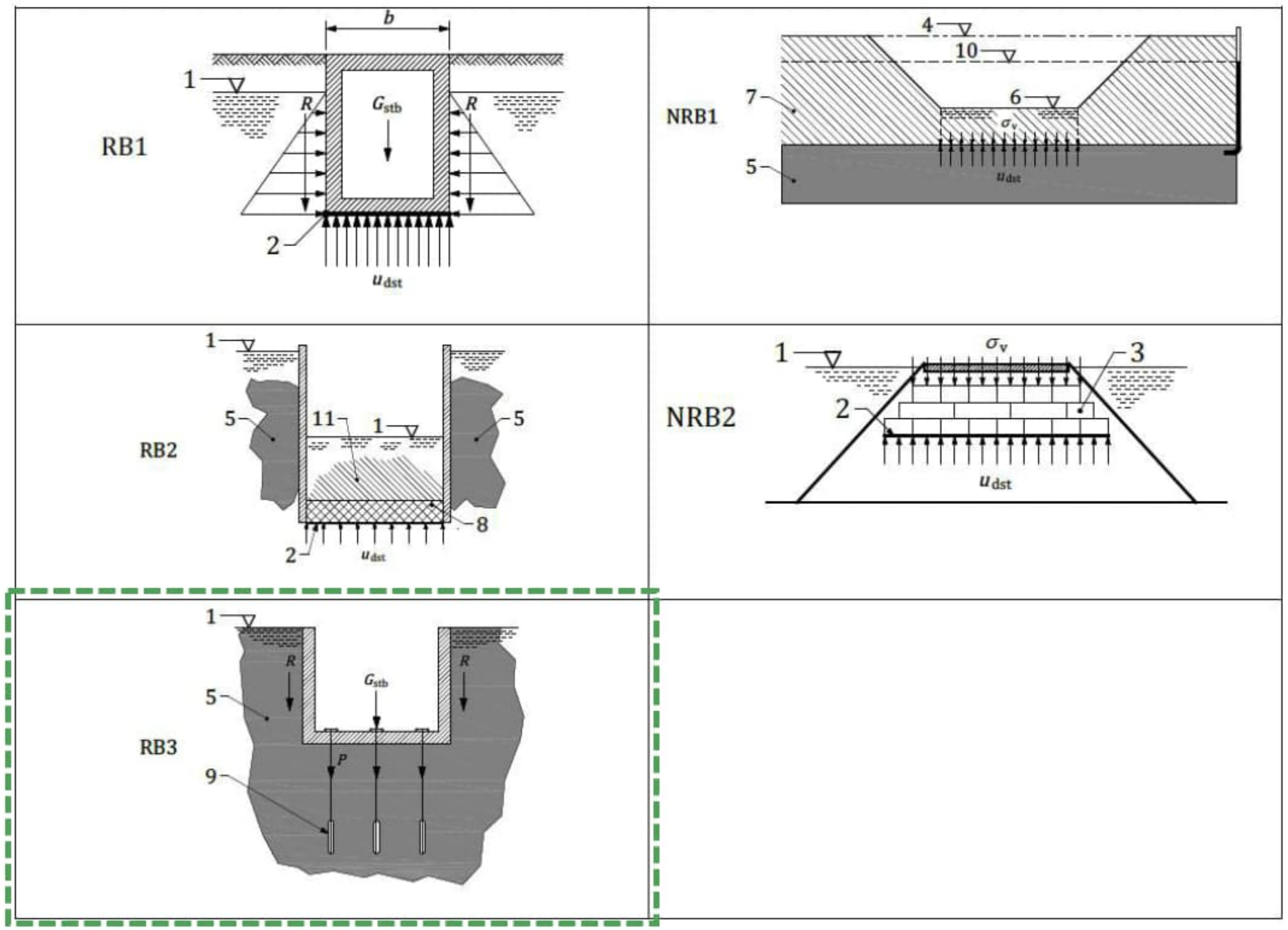
RIGID BODY **NON - RIGID BODY**

Key

	Rigid body		Non-rigid body
1	Groundwater table	1	Groundwater table
2	Water tight surface	2	Water tight surface
5	Sand	3	Light-weight material
8	Injected sand	4	Former ground surface
9	Anchor	5	Gravel
11	Sand	6	Groundwater table bottom excavtion
		7	Clay
		10	Groundwater table in gravel

Loss of vertical equilibrium due to uplift

Examples of design situations where uplift might be critical (§ 8.1.3.2)



RIGID BODY **NON - RIGID BODY**

Key

	Rigid body
1	Groundwater table
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5	Sand
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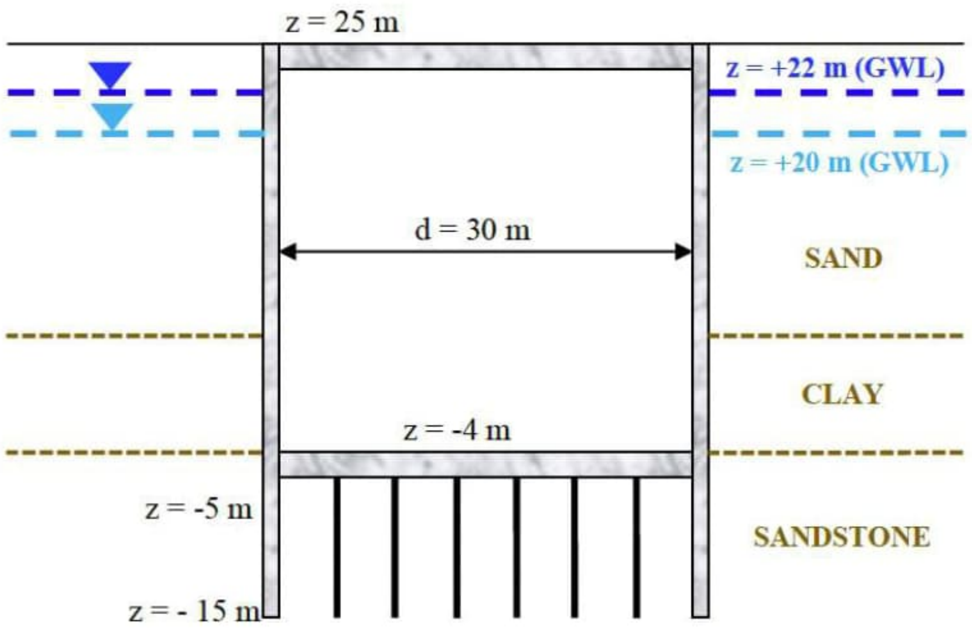
11	Sand

	Non-rigid body
1	Groundwater table
2	Water tight surface
3	Light-weight material
4	Former ground surface
5	Gravel
6	Groundwater table bottom excavtion
7	Clay
10	Groundwater table in gravel

Loss of vertical equilibrium due to uplift

Example 1 – RIGID BODY



Cylindrical tank

Inner diameter 30 m

Side diaphragm wall is 40 m deep and 1 m thick

Thickness of the top and bottom slabs 1 m

The slab is anchored by micropiles resisting the uplift action induced by the ground water

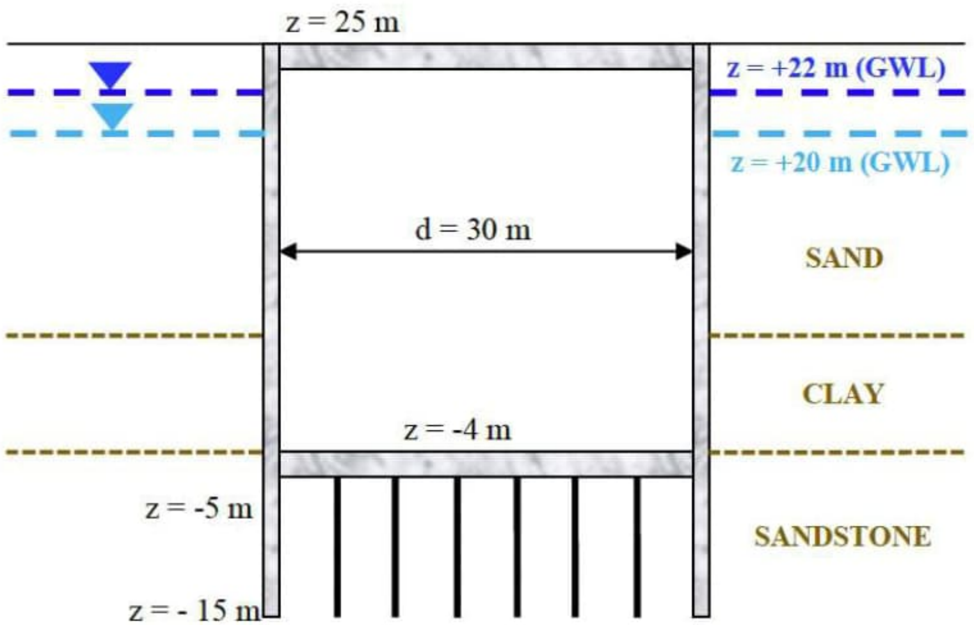
Levels of the ground water table

+ 20 m above sea level (masl) during construction

+ 22 masl during service life

Loss of vertical equilibrium due to uplift

Example 1 – RIGID BODY



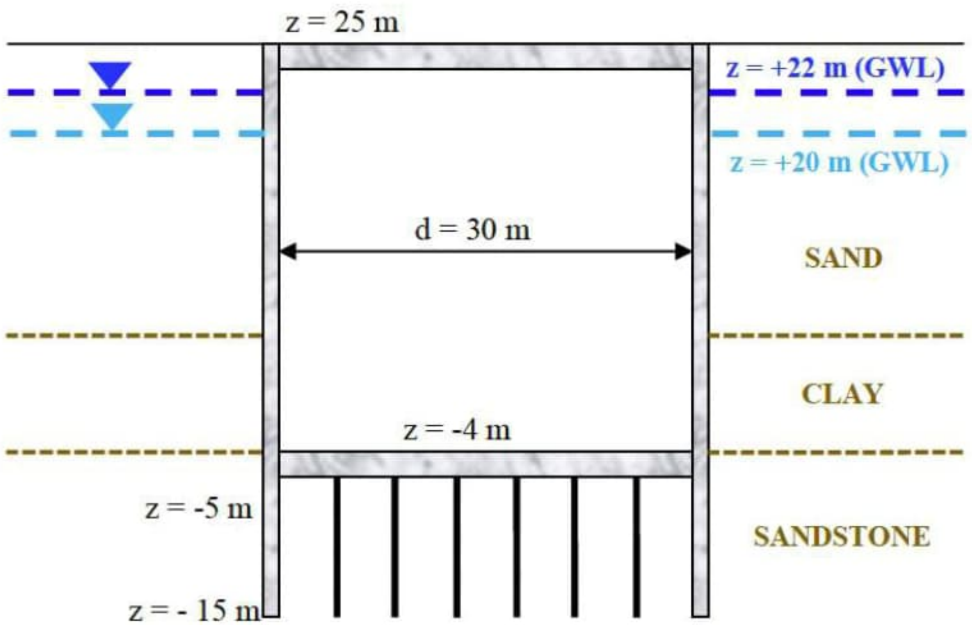
Ground profile

Ground types	From (top) (masl)	To (bottom) (masl)	Saturated weight density γ_{sat} (kN/m^3)
Sand	25	5	19
Clay	5	-4	18
Sandstone	-4	-30	20

Specific weight of water γ_w assumed to be 10 kN/m^3

Loss of vertical equilibrium due to uplift / Example 1 – RIGID BODY

Exercise 1: Compute the required design value of the resistance [Rd] in order to comply with the SC7 requirements with respect to uplift ULS as a rigid body



$$U_{d,dst} + G_{d,dst} + Q_{d,dst} - G_{d,stb} \leq R_d$$

(2) <REQ> If the structure or ground layer acts as a rigid body (Figure 8.1a), the inequality given by Formula (8.1) shall be verified:

$$U_{d,dst} + G_{d,dst} + Q_{d,dst} - G_{d,stb} \leq R_d \tag{8.1}$$

where

$U_{d,dst}$ is the design value of destabilising (uplift) force due to groundwater pressures;

$G_{d,dst}$ is the design value of any permanent destabilizing force (upwards) not caused by groundwater pressures;

$Q_{d,dst}$ is the design value of any variable destabilizing force (upwards) not caused by groundwater pressures;

$G_{d,stb}$ is the design value of the stabilizing (downward) forces;

R_d is design value of any resistance to uplift.

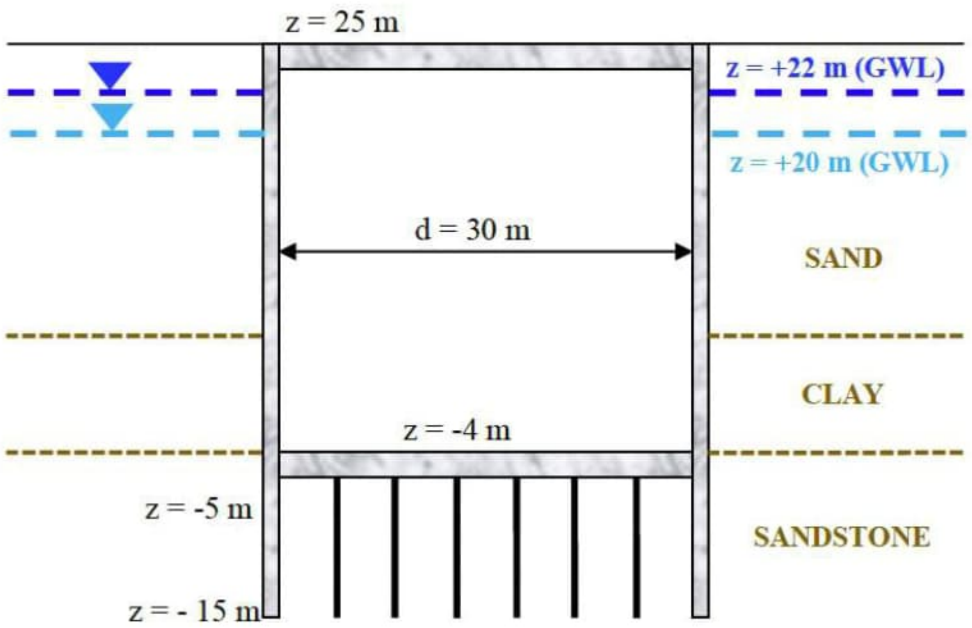
NOTE 1 Partial factors for actions are given in EN 1990-1, Annex A; for ground properties in Table 4.7(NDP); and for resistances in EN 1997-3.

NOTE 2 The contribution of piles, anchors, etc. to R_d is determined according to EN 1997-3.

Note: it is only asked to calculate the necessary value of Rd, not to estimate its value or to assess whether the resulting value seems reasonable (out of scope)

Loss of vertical equilibrium due to uplift

Example 1 – RIGID BODY



Methodology and assumptions

Following *§6.4 of EC7-1 May 2021 Draft* the more adverse value between characteristic upper and lower groundwater level should be considered when calculating the representative value of groundwater pressure, thus GWL = + 22 masl

For the partial factors, please see next page

Table 4.7(NDP) – Partial factors on ground properties for persistent, transient, and accidental design situations

Ground property	Symbol	M1[1]	M2[1]	
		Persistent Transient Accidental	Persistent Transient	Accidental
Soil and Fill parameters				
Shear strength in effective stress analysis[2] (τ_f)	$\gamma_{\tau f}$	1,0	1,25 K_M	1,1
Coefficient of peak friction (tan φ'_p)[4]	$\gamma_{\tan\varphi,p}$	1,0	1,25 K_M	1,1
Peak effective cohesion (c'_p)	$\gamma_{c,p}$	1,0	1,25 K_M	1,1
Coefficient of friction at critical state (tan φ'_{cs})[4]	$\gamma_{\tan\varphi,cs}$	1,0	1,1 K_M	1,0
Coefficient of residual friction (tan φ'_r)[4]	$\gamma_{\tan\varphi,r}$	1,0	1,1 K_M	1,0
Residual effective cohesion (c'_r)	$\gamma_{c,r}$	1,0	1,1 K_M	1,0
Shear strength in total stress analysis[2] (c_u)	γ_{cu}	1,0	1,4 K_M	1,2
Unconfined compressive strength (q_u)	γ_{qu}	Same as γ_{cu}		
Rock parameters				
Shear strength[2] (τ_r)	$\gamma_{\tau r}$	1,0	1,4 K_M	1,2
Coefficient friction along discontinuity (tan φ'_{dis})[4]	$\gamma_{\tan\varphi dis}$	1,0	1,4 K_M	1,2
Unconfined compressive strength[3] (q_u)	γ_{qu}	1,0	1,4 K_M	1,2
Interface parameters				
Coefficient of ground/structure interface friction (tan δ)	$\gamma_{\tan\delta}$	1,0	1,25 K_M	1,1

[1] M1, and M2 are alternative sets of material factors. EN 1997-3 specifies which set to use for specific geotechnical structures.
[2] Intended to be used for numerical models and non-Mohr-Coulomb strength criteria.
[3] Used for foundation purposes only.
[4] the partial factor is applied to the tanφ

Loss of vertical equilibrium due to uplift

Example 1 – RIGID BODY

Methodology and assumptions

With respect to the partial factors, the left table taken, from the EC7-1 draft May 2021, shows the partial factors for ground properties for any design situation.

As can be seen, the weight density of ground (or water) is not factored although actions arising from ground weight density (or water) can be factored.

Table A.1.8 (NDP) — Partial factors on actions and effects for fundamental (persistent and transient) design situations

Action or effect				Partial factors γ_F and γ_E for Design Cases 1 to 4				
Type	Group	Symbol	Resulting effect	Structural resistance	Static equilibrium and uplift		Geotechnical design	
Design case				DC1[a]	DC2(a)[b]	DC2(b)[b]	DC3[c]	DC4[d]
Formula				(8.4)	(8.4)		(8.4)	(8.5)
Permanent action (G_k)	All[f]	γ_G	unfavourable /destabilizing	1,35K_F	1,35K_F	1,0	1,0	G_k is not factored
	Water	$\gamma_{G,w}$		1,2K_F	1,2K_F	1,0	1,0	
	All[f]	$\gamma_{G,stb}$	stabilizing[g]	not used	1,15[e]	1,0	not used	
	Water[l]	$\gamma_{G,w,stb}$			1,0[e]	1,0		
	All	$\gamma_{G,fav}$	favourable[h]	1,0	1,0	1,0	1,0	
Prestress (P_k)		γ_P[k]						
Variable action (Q_k)	All[f]	γ_Q	unfavourable	1,5K_F	1,5K_F	1,5K_F	1,3	$\gamma_{Q,1}/\gamma_{G,1}$[j]
	Water[l]	$\gamma_{Q,w}$		1,35K_F	1,35K_F	1,35K_F	1,15	1,0
	All	$\gamma_{Q,fav}$	favourable	0				
Effects of actions (E)		γ_E	unfavourable	Effects are not factored				1,35K_F
		$\gamma_{E,fav}$	favourable					1,0

[a] Design Case 1 (DC1) is used both for structural and geotechnical design.

[b] Design Case 2 (DC2) is used for the combined verification of strength and static equilibrium, when the structure is sensitive to variations in permanent action arising from a single-source. Values of γ_F are taken from columns (a) or (b), whichever gives the less favourable outcome.

[c] Design Case 3 (DC3) is typically used for the design of slopes and embankments, spread foundations, and gravity retaining structures. See EN 1997 for details.

[d] Design Case 4 (DC4) is typically used for the design of transversally loaded piles and embedded retaining walls and (in some countries) gravity retaining structures. See EN 1997 for details.

[e] The values of $\gamma_{G,stb}$ = 1.15 and 1.0 are based on $\gamma_{G,inf}$ = 1,35 ρ and 1,2 ρ with ρ = 0.85.

[f] Applied to all actions except water pressures.

[g] Applied to the stabilizing part of an action originating from a single source.

[h] Applied to actions whose entire effect is favourable and independent of the unfavourable action.

[j] $\gamma_{Q,1}$ = corresponding value of γ_Q from DC1 and $\gamma_{G,1}$ = corresponding value of γ_G from DC1.

[k] See other relevant Eurocodes for the definition of γ_P where γ_P is materially dependent.

[l] For water actions induced by waves and currents, see Annex A.6.

Loss of vertical equilibrium due to uplift

Example 1 – RIGID BODY

Methodology and assumptions

As per indications of EC0, the design cases *DC2(a)* and *DC2(b)* are to be used for the combined verification of strength and static equilibrium, as the structure is sensitive to variations in permanent action arising from a single-source, i. e., the ground water sub-pressure applied at the bottom of the tank. Values of partial factors on actions γ_F will be taken from columns (a) or (b), whichever gives the less favourable outcome.

Loss of vertical equilibrium due to uplift

Example 1 – RIGID BODY

Methodology and assumptions

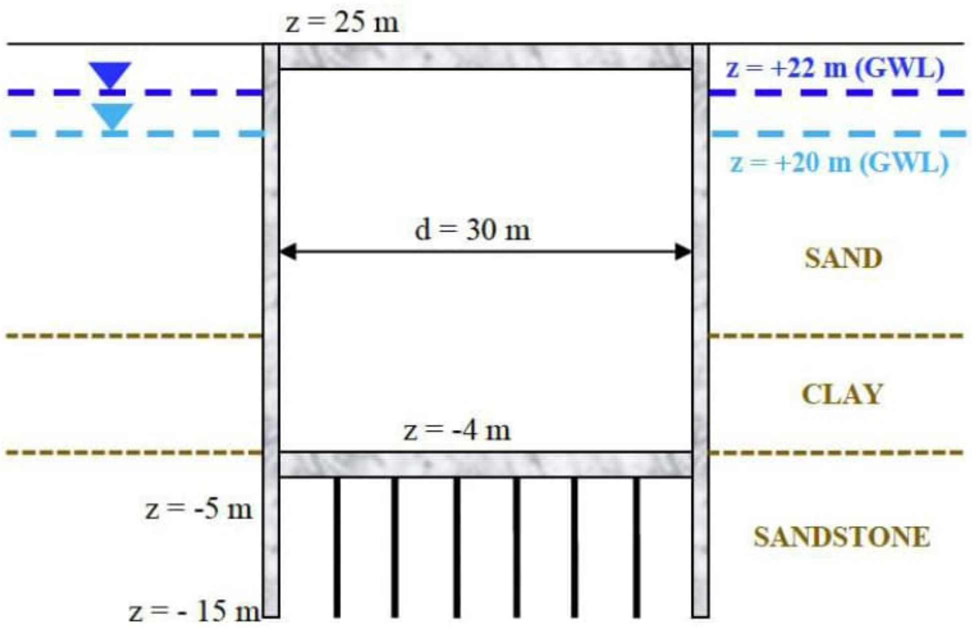
Table A.1.9 (NDP) — Consequence factors for buildings

Consequence class (CC)	Description of consequences	Consequence factor K_F
CC3	Higher	1,1
CC2	Normal	1,0
CC1	Lower	0,9

As per indications of EC0, values of consequence factor [K_F] for different consequence classes in Table A.1.8 (NDP) are given in Table A.1.9 (NDP), unless the National Annex fives different values for use in a country.

Loss of vertical equilibrium due to uplift

Example 1 – RIGID BODY



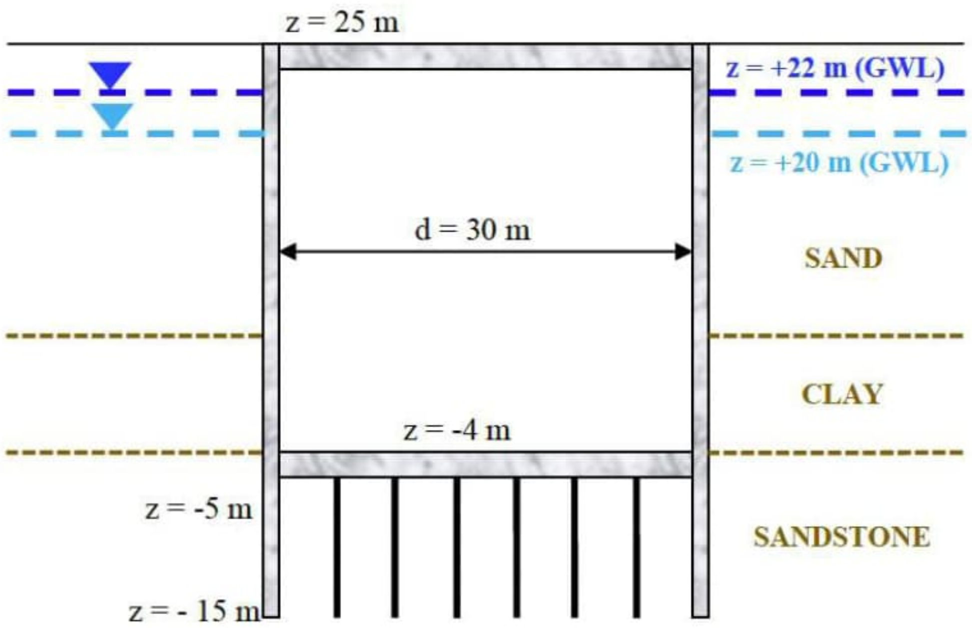
Methodology and assumptions

In this exercise, the resistance [Rd] would be provided by the micropiles and the friction resistance along the sidewall of the structure

After the completion of structural works and end of dewatering within the structure, the verification of UPL is as follows (please, see next page)

Loss of vertical equilibrium due to uplift

Example 1 – RIGID BODY – exercise 1



Methodology and assumptions

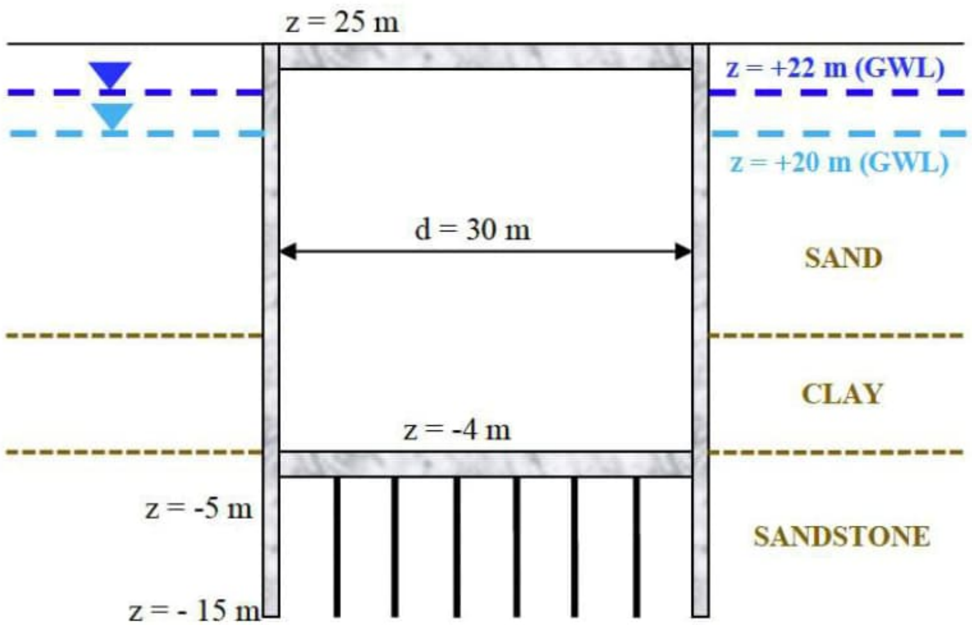
$$U_{d,dst} + G_{d,dst} + Q_{d,dst} - G_{d,stb} \leq R_d$$

U_{dst} is the value of destabilising (uplift) force due to groundwater pressures – applied below bottom slab and below toe level of d-walls:

U_{dst} = [π x (15 m)² x (22 m – (-5 m)) x 10 kN/m2] + [1 x π x 30.5 m x (22 m – (-15 m)) x 10 kN/m2] = 226300 kN

Loss of vertical equilibrium due to uplift

Example 1 – RIGID BODY – exercise 1



Methodology and assumptions

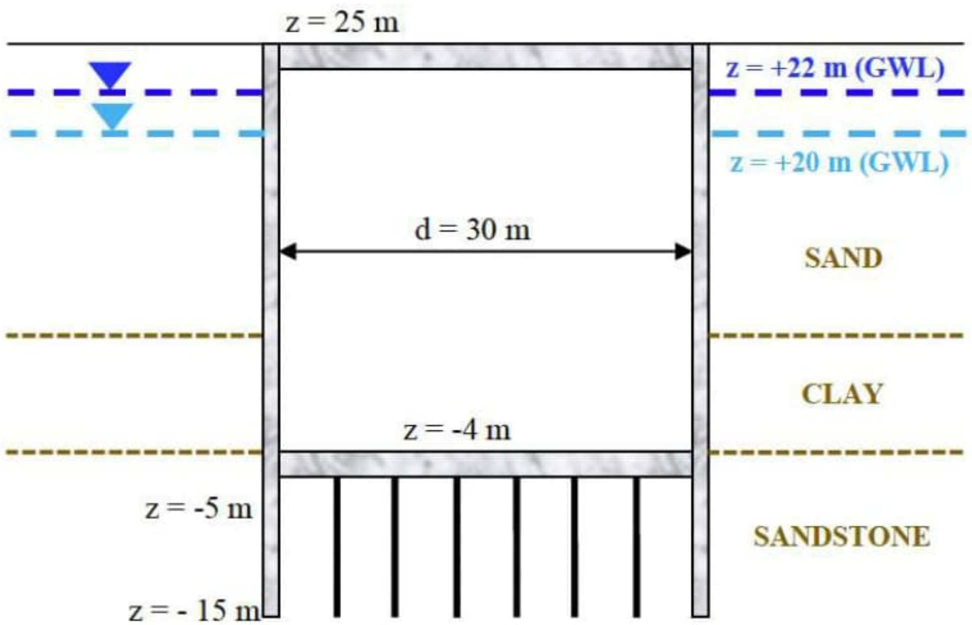
$$U_{d,dst} + G_{d,dst} + Q_{d,dst} - G_{d,stb} \leq R_d$$

G_{dst} is the value of permanent destabilizing force (upwards) not caused by groundwater pressures, and it is not applicable for this example.

Q_{dst} is the value of variable destabilizing force (upwards) not caused by groundwater pressures, and it is not applicable for this example.

Loss of vertical equilibrium due to uplift

Example 1 – RIGID BODY – exercise 1



Methodology and assumptions

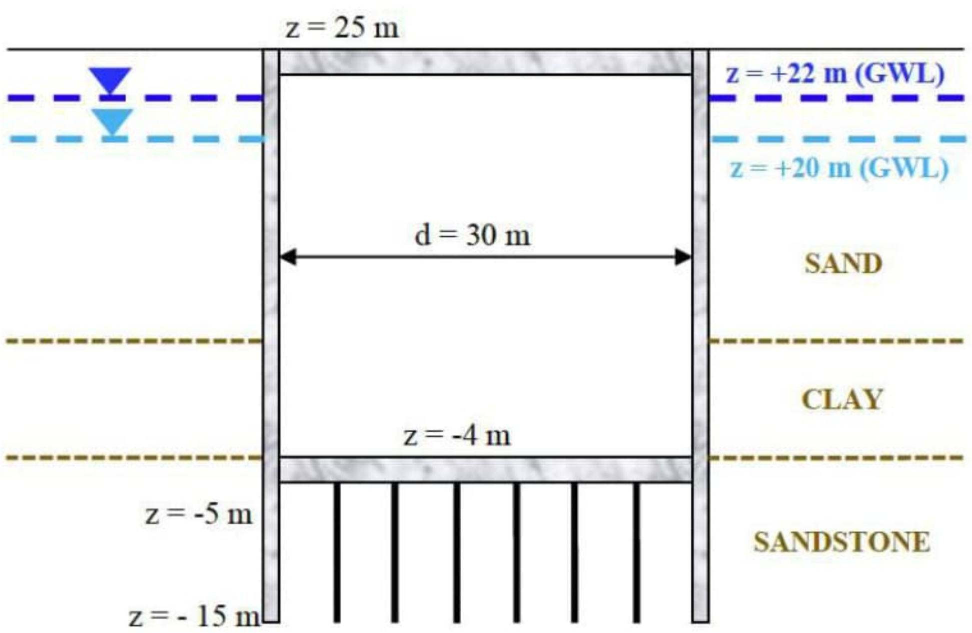
$$U_{\mathrm{d,dst}} + G_{\mathrm{d,dst}} + Q_{\mathrm{d,dst}} - G_{\mathrm{d,stb}} \leq R_d$$

G_{stb} is the value of stabilising (downward) forces, that is limited to self-weight:

G_{stb} = [1 x π x 30.5 m x (25 m – (-15 m)) x 25 kN/m3] + 2 x [π x (15 m)^2 x 1 m x 25 kN/m3] = 131160 kN

Loss of vertical equilibrium due to uplift

Example 1 – RIGID BODY – exercise 1



DC2(a) approach

$$U_{d,dst} + G_{d,dst} + Q_{d,dst} - G_{d,stb} \leq R_d$$

The following inequality should be verified for each consequence class:

$$1.2 \times k_F \times U_{dst} + 1.35 \times k_F \times G_{dst} + 1.5 \times k_F \times Q_{dst} - 1.15 \times G_{stb} \leq R_d$$

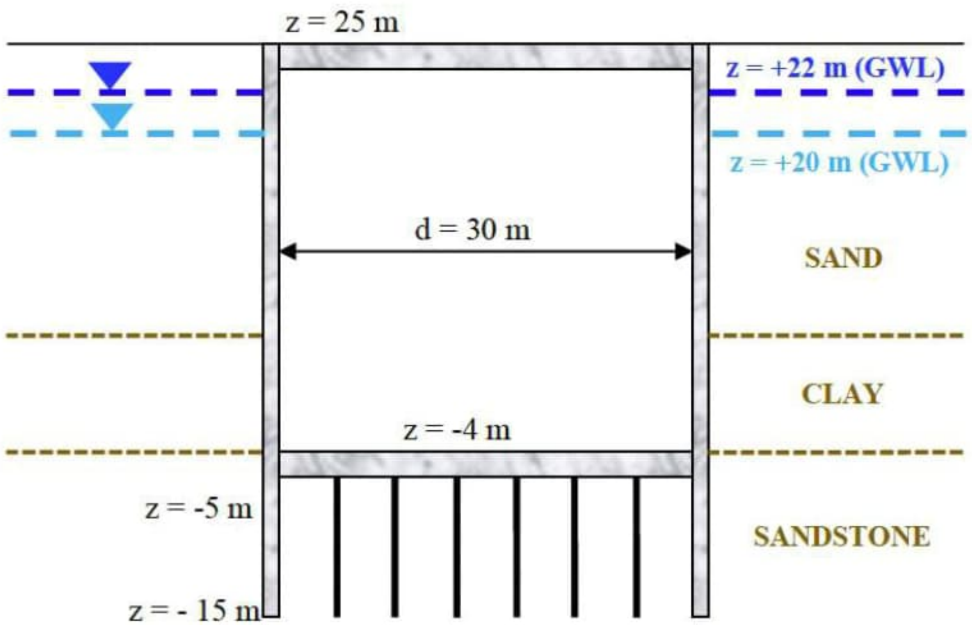
Considering the DC2(a) approach, for the different consequence classes the following values are obtained (please see next page)

Loss of vertical equilibrium due to uplift

Example 1 – RIGID BODY – exercise 1

DC2(a) approach

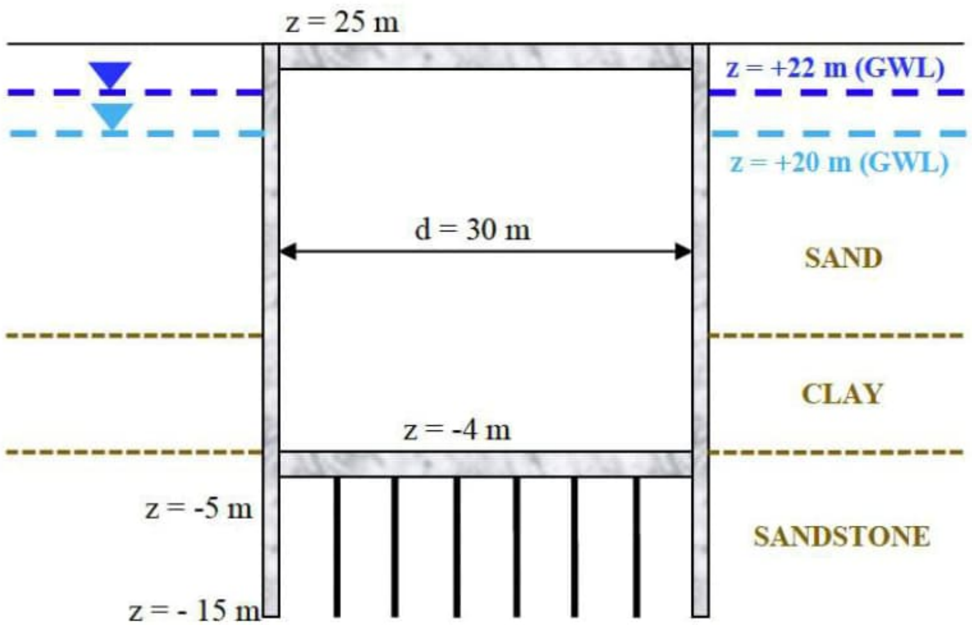
$$U_{d,dst} + G_{d,dst} + Q_{d,dst} - G_{d,stb} \leq R_d$$



DC2(a) approach	Consequence Class 1	Consequence Class 2	Consequence Class 3
k_F =	0.9	1.0	1.1
$U_{d,dst} = k_F \times 1.2 \times U_{dst}$	$1.08 \times U_{dst}$	$1.20 \times U_{dst}$	$1.32 \times U_{dst}$
$U_{d,dst}$ (value)	244400 kN	271560 kN	298720 kN
$G_{d,dst} = k_F \times 1.35 \times G_{dst}$	$1.215 \times G_{dst}$	$1.35 \times G_{dst}$	$1.485 \times G_{dst}$
$G_{d,dst}$ (value)	0 kN	0 kN	0 kN
$Q_{d,dst} = k_F \times 1.5 \times Q_{dst}$	$1.35 \times Q_{dst}$	$1.50 \times Q_{dst}$	$1.65 \times Q_{dst}$
$Q_{d,dst}$ (value)	0 kN	0 kN	0 kN
$G_{d,stb} = 1.15 \times G_{stb}$	$1.15 \times G_{stb}$	$1.15 \times G_{stb}$	$1.15 \times G_{stb}$
$G_{d,stb}$ (value)	150830 kN	150830 kN	150830 kN

Loss of vertical equilibrium due to uplift

Example 1 – RIGID BODY – exercise 1



DC2(a) approach

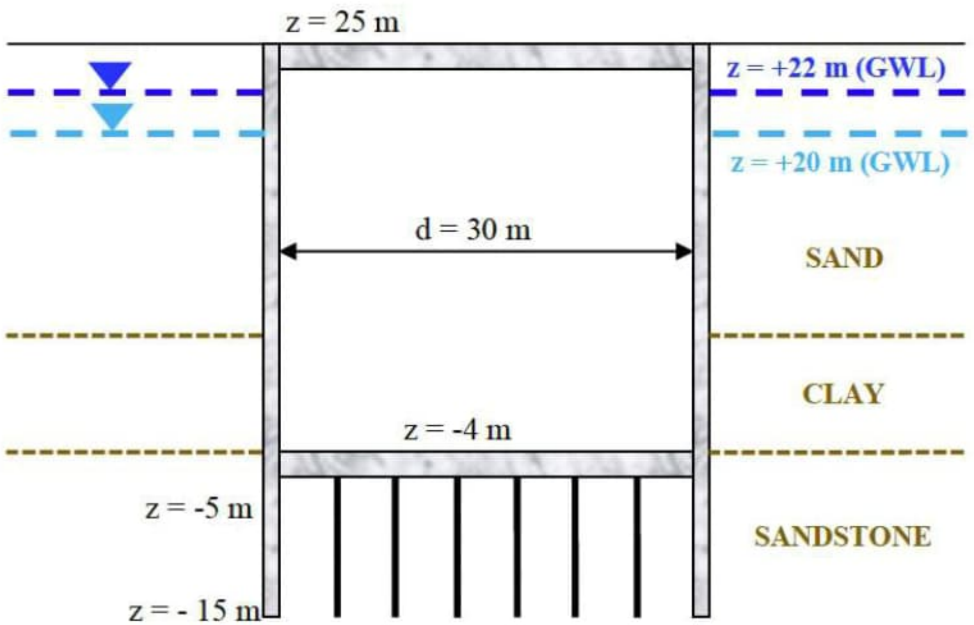
$$U_{d,dst} + G_{d,dst} + Q_{d,dst} - G_{d,stb} \leq R_d$$

Hence, the following values of required resistance are derived:

DC2(a) approach	Consequence Class 1	Consequence Class 2	Consequence Class 3
$R_d \geq$	93570 kN	120730 kN	147890 kN

Loss of vertical equilibrium due to uplift

Example 1 – RIGID BODY – exercise 1



DC2(b) approach

$$U_{d,dst} + G_{d,dst} + Q_{d,dst} - G_{d,stb} \leq R_d$$

The inequality for this approach is:

$$U_{dst} + G_{dst} + 1.5 \times k_F \times Q_{dst} - G_{stb} \leq R_d$$

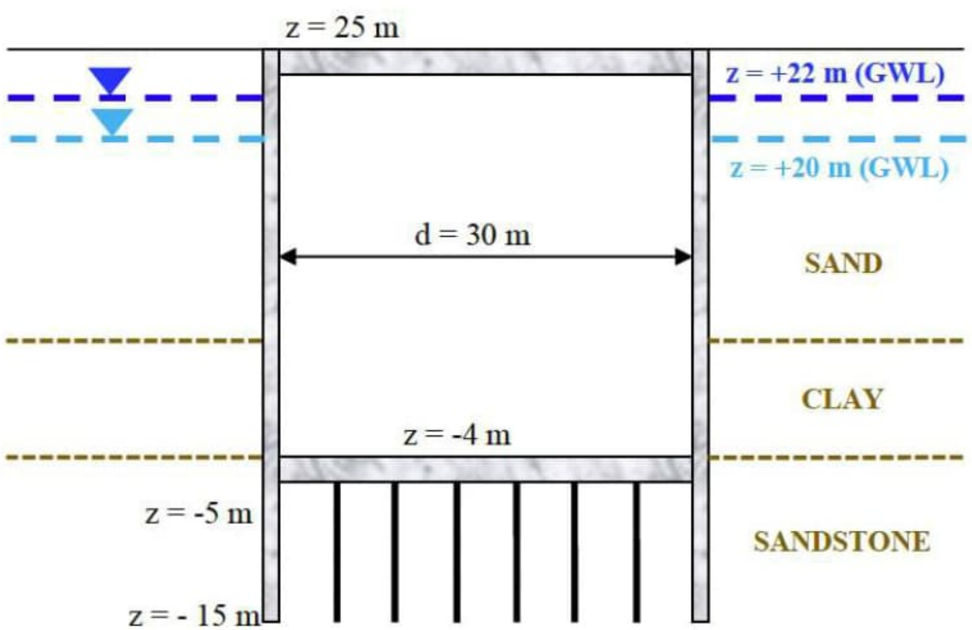
Similarly, if the DC2(b) approach is considered, the following values are deduced (please see next page)

Loss of vertical equilibrium due to uplift

Example 1 – RIGID BODY – exercise 1

DC2(b) approach

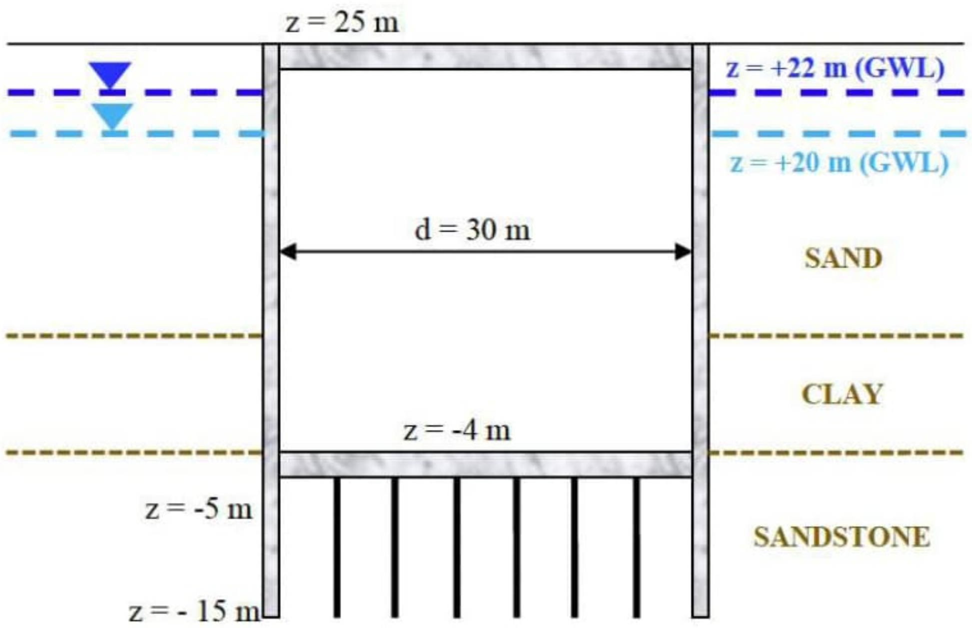
$$U_{d,dst} + G_{d,dst} + Q_{d,dst} - G_{d,stb} \leq R_d$$



DC2(b) approach	Consequence Class 1	Consequence Class 2	Consequence Class 3
k_F =	0.9	1.0	1.1
$U_{d,dst} = U_{dst}$	1.00 x U_{dst}	1.00 x U_{dst}	1.00 x U_{dst}
$U_{d,dst}$ (value)	226300 kN	226300 kN	226300 kN
$G_{d,dst} = G_{dst}$	1.00 x G_{dst}	1.00 x G_{dst}	1.00 x G_{dst}
$G_{d,dst}$ (value)	0 kN	0 kN	0 kN
$Q_{d,dst} = k_F$ x 1.5 x Q_{dst}	1.35 x Q_{dst}	1.50 x Q_{dst}	1.65 x Q_{dst}
$Q_{d,dst}$ (value)	0 kN	0 kN	0 kN
$G_{d,dst} = G_{stb}$	1.00 x G_{stb}	1.00 x G_{stb}	1.00 x G_{stb}
$G_{d,stb}$ (value)	131160 kN	131160 kN	131160 kN

Loss of vertical equilibrium due to uplift

Example 1 – RIGID BODY – exercise 1



DC2(b) approach

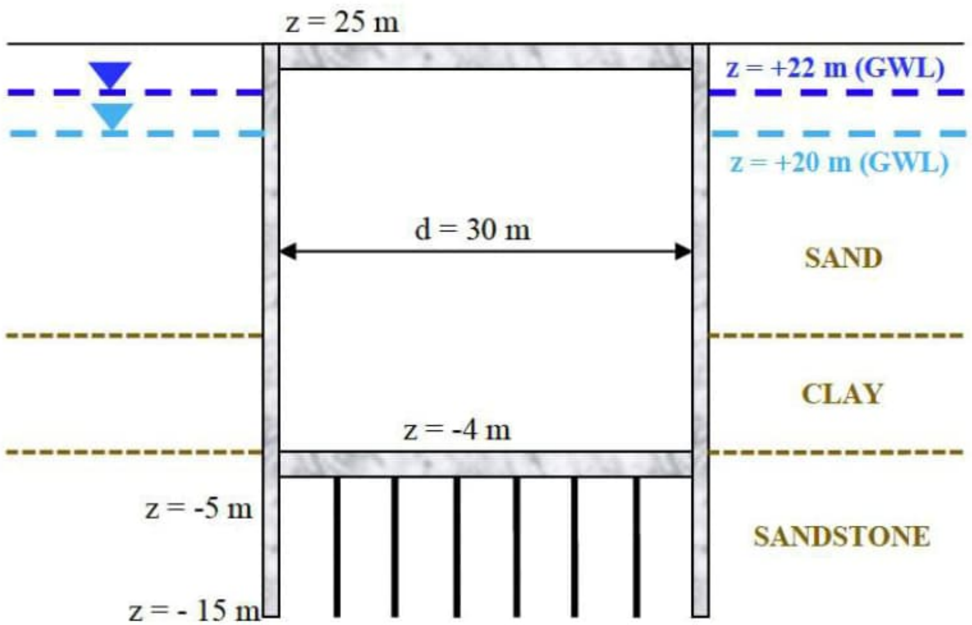
$$U_{d,dst} + G_{d,dst} + Q_{d,dst} - G_{d,stb} \leq R_d$$

Hence, the following values of required resistance are derived:

DC2(b) approach	Consequence Class 1	Consequence Class 2	Consequence Class 3
$R_d \geq$	95140 kN	95140 kN	95140 kN

Loss of vertical equilibrium due to uplift

Example 1 – RIGID BODY – exercise 1



$$U_{d,dst} + G_{d,dst} + Q_{d,dst} - G_{d,stb} \leq R_d$$

DC2(a) approach – required resistance [Rd]

DC2(a) approach	Consequence Class 1	Consequence Class 2	Consequence Class 3
$R_d \geq$	93570 kN	120730 kN	147890 kN

DC2(b) approach – required resistance [Rd]

DC2(b) approach	Consequence Class 1	Consequence Class 2	Consequence Class 3
$R_d \geq$	95140 kN	95140 kN	95140 kN

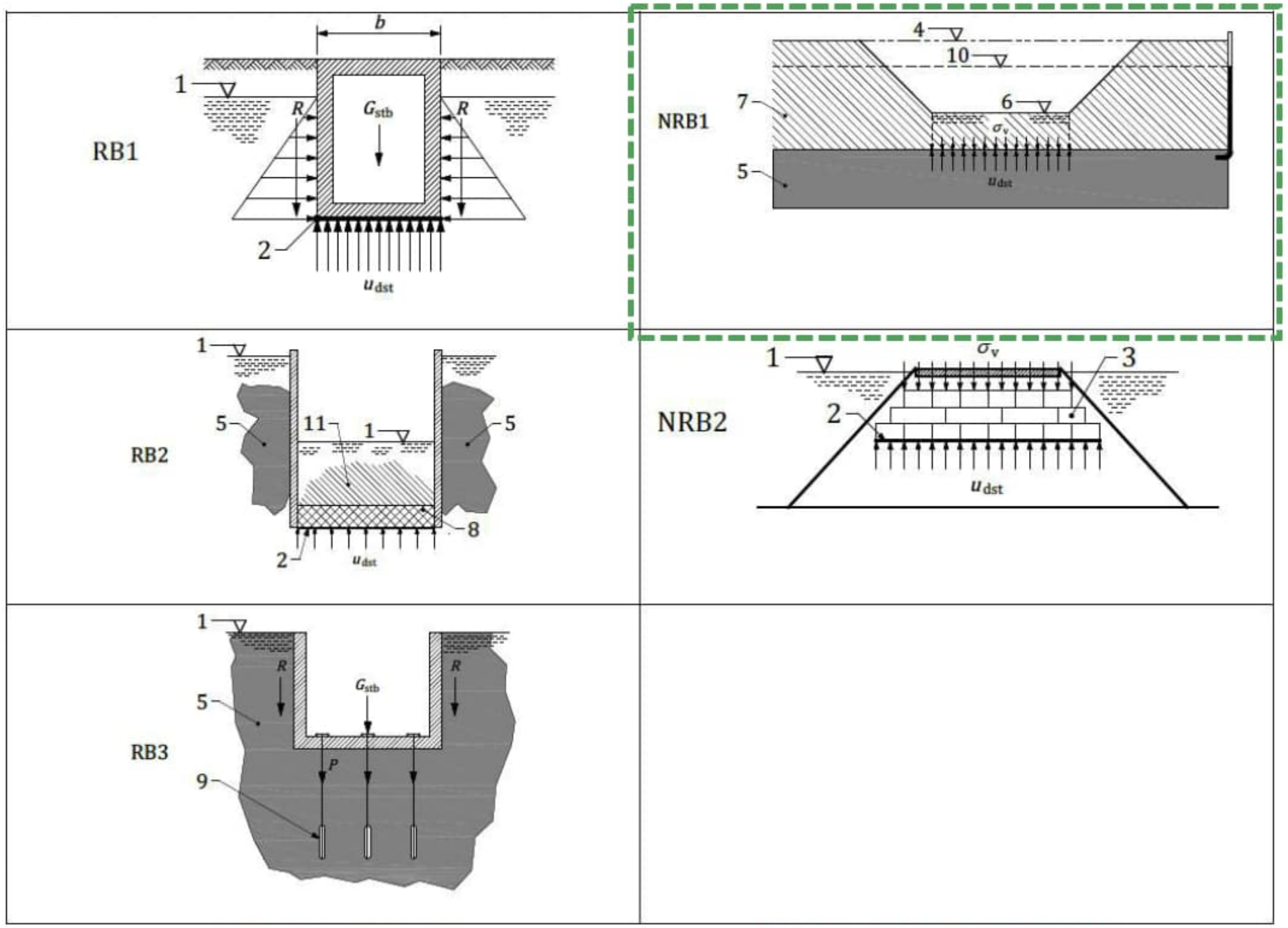
Final required resistance [Rd]

Finally, according to note b at table A.1.8 (NDP) presented above, the less favourable outcome must be considered, as indicated below:

Final required resistance	Consequence Class 1	Consequence Class 2	Consequence Class 3
$R_d \geq$	95140 kN	120730 kN	147890 kN

Loss of vertical equilibrium due to uplift

Examples of design situations where uplift might be critical (§ 8.1.3.2)



RIGID BODY | **NON - RIGID BODY**

Key

Rigid body

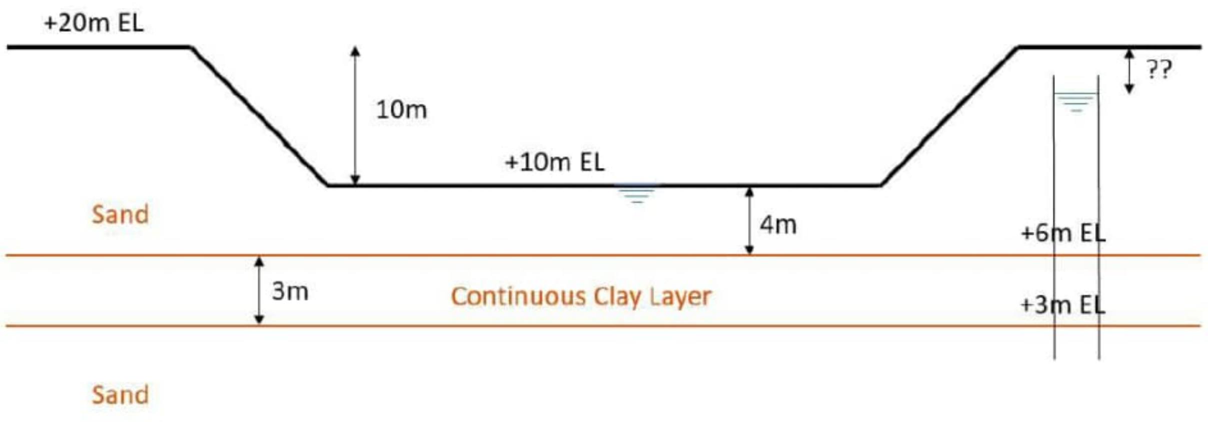
1 Groundwater table
2 Water tight surface
5 Sand
8 Injected sand
9 Anchor
11 Sand

Non-rigid body

1 Groundwater table
2 Water tight surface
3 Light-weight material
4 Former ground surface
5 Gravel
6 Groundwater table bottom excavtion
7 Clay
10 Groundwater table in gravel

Loss of vertical equilibrium due to uplift

Example 2 – NON - RIGID BODY



The saturated weight density for the sand is γ_{sat} = 20 kN/m³ and for the clay γ_{sat} = 18 kN/m³.

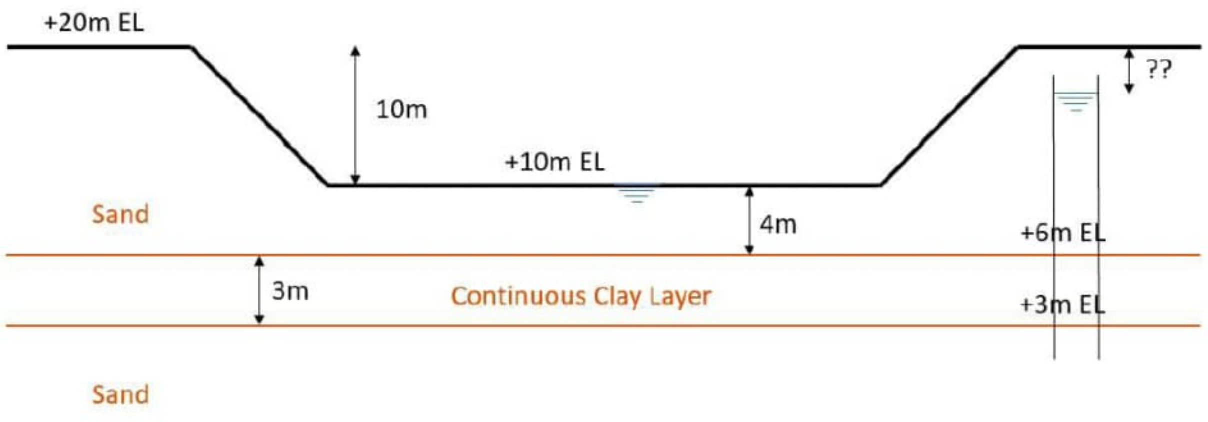
Failure of an open cut excavation

The example deals with an open cut excavation of 10 m depth. The underlying ground is composed by sand and a continuous clay layer of 3m thickness located between 4 m and 7 m depth below the bottom of the excavation.

Before the excavation is made, the piezometric water level is the same in both sand layers with the clay layer being an impervious barrier. Within the open cut the water level is lowered down to the bottom of the excavation.

Loss of vertical equilibrium due to uplift

Example 2 – NON - RIGID BODY – exercise 1



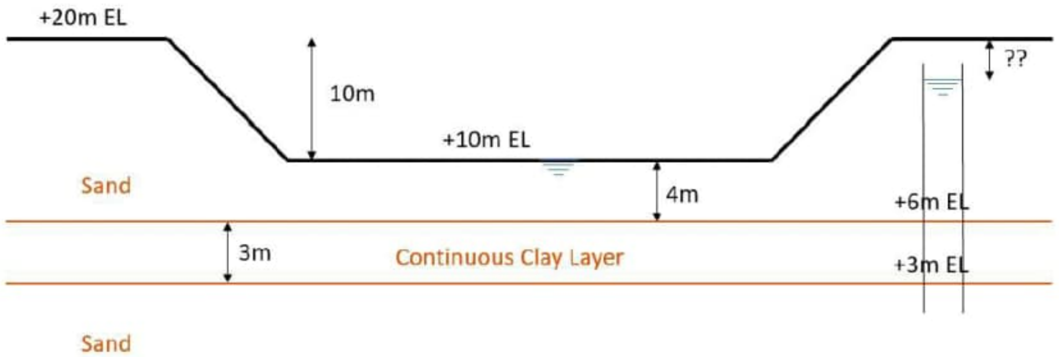
The saturated weight density for the sand is γ_{sat} = 20 kN/m³ and for the clay γ_{sat} = 18 kN/m³.

Failure of an open cut excavation

Assess the maximum piezometric level in the lower level of sand for which the requirements stated in EC7-1 May 2021 draft against uplift UPL for non-rigid bodies are fulfilled.

Loss of vertical equilibrium due to uplift

Example 2 – NON - RIGID BODY – exercise 1



Methodology and assumptions

§8.1.3.2 (3) of EC7-1 May 2021 Draft (see below)

The vertical total pressure at the bottom of the clay layer will be:

$$\sigma_v = 4 \text{ m x } 20 \text{ kN/m}^3 + 3 \text{ m x } 18 \text{ kN/m}^3 = 134 \text{ kPa}$$

(3) <REQ> If the structure or ground layer does not act as a rigid body (Figure 8.1b), the inequality given by Formula (8.2) shall be verified:

$$u_{d;dst} - \sigma_{v;d} \leq 0 \quad (8.2)$$

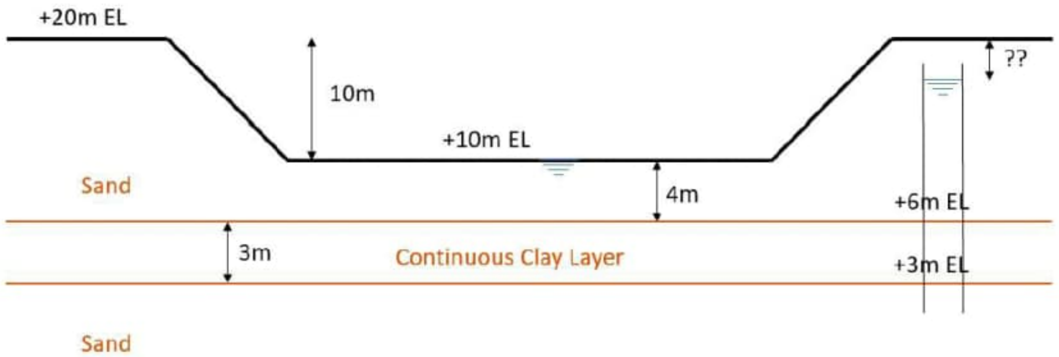
where

$u_{d,dst}$ is the design value of destabilizing (uplift) groundwater pressures;

$\sigma_{v;d}$ is the design value of the (stabilizing) vertical total stress at the base of the layer that is subject to uplift.

Loss of vertical equilibrium due to uplift

Example 2 – NON - RIGID BODY – exercise 1



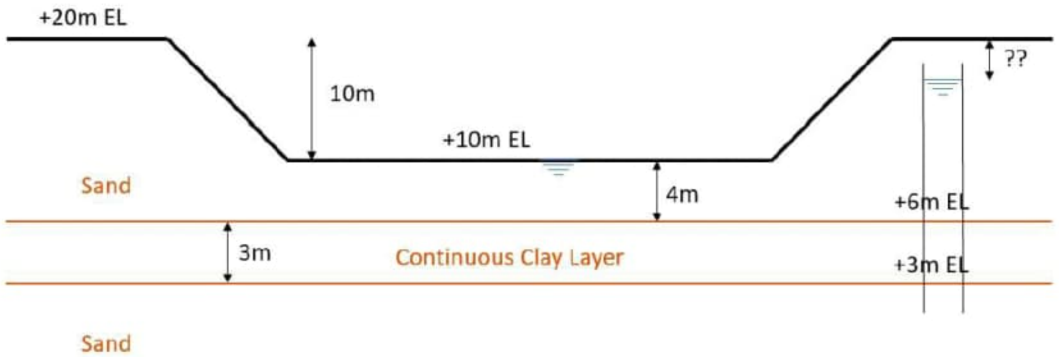
DC2(a) approach	Consequence Class 1	Consequence Class 2	Consequence Class 3
k_F =	0.9	1.0	1.1
$u_{d,dst} = k_F \times 1.2 \times u_{dst}$	$1.08 \times u_{dst}$	$1.20 \times u_{dst}$	$1.32 \times u_{dst}$
$u_{d,dst}$ (value)	154 kPa	154 kPa	154 kPa
$\sigma_{v,d} = 1.15 \times \sigma_v$	$1.15 \times \sigma_v$	$1.15 \times \sigma_v$	$1.15 \times \sigma_v$
$\sigma_{v,d}$ (value)	154 kPa	154 kPa	154 kPa
Maximum piezometric level	17,3 m EL	15,8 m EL	14,7 m EL

DC2(a) approach

The maximum piezometric level allowed in the lower level of sand would vary between 17.3 m EL and 14.7 m EL depending on the consequence class.

Loss of vertical equilibrium due to uplift

Example 2 – NON - RIGID BODY – exercise 1



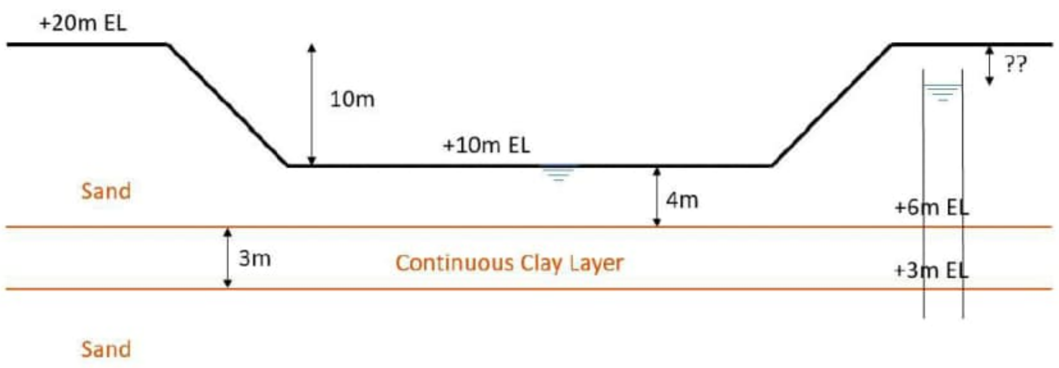
DC2(b) approach	Consequence Class 1	Consequence Class 2	Consequence Class 3
k_F =	0.9	1.0	1.1
$u_{d,dst} = u_{dst}$	1.0 x u_{dst}	1.0 x u_{dst}	1.0 x u_{dst}
$u_{d,dst}$ (value)	134 kPa	134 kPa	134 kPa
$\sigma_{v,d} = 1.15 \times \sigma_v$	1.0 x σ_v	1.0 x σ_v	1.0 x σ_v
$\sigma_{v,d}$ (value)	134 kPa	134 kPa	134 kPa
Maximum piezometric level	16,4 m EL	16,4 m EL	16,4 m EL

DC2(b) approach

The maximum piezometric level allowed in the lower level of sand would be 16,4 m EL for all consequence classes.

Loss of vertical equilibrium due to uplift

Example 2 – NON - RIGID BODY – exercise 1



	Consequence Class 1	Consequence Class 2	Consequence Class 3
Maximum piezometric level	+16.4 m EL	+15.8 m EL	+14.7 m EL

Final result:

According to note b of table A.1.8 (NDP) of EC0 the less favourable outcome must be considered.

Scope:

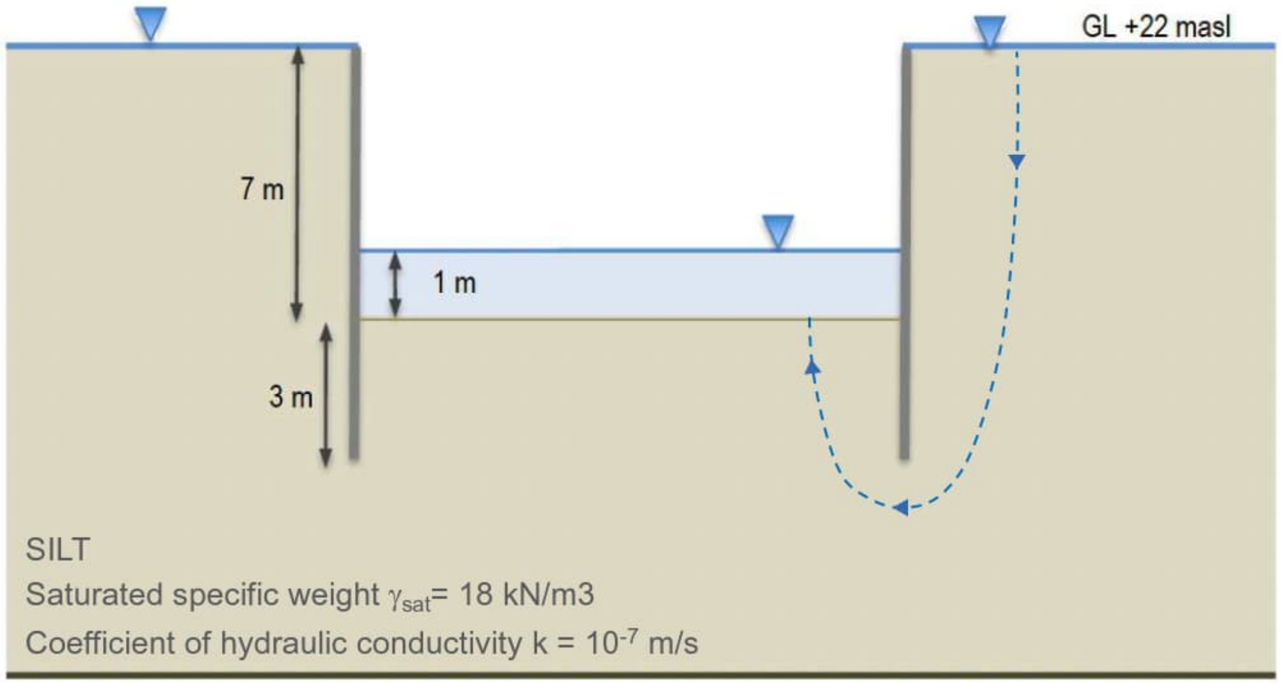
Examples on groundwater related ULS:

- Loss of vertical equilibrium due to uplift (2)

- **Hydraulic heave (1)**

Hydraulic heave

Seepage under diaphragm wall (§ 8.1.4.2)



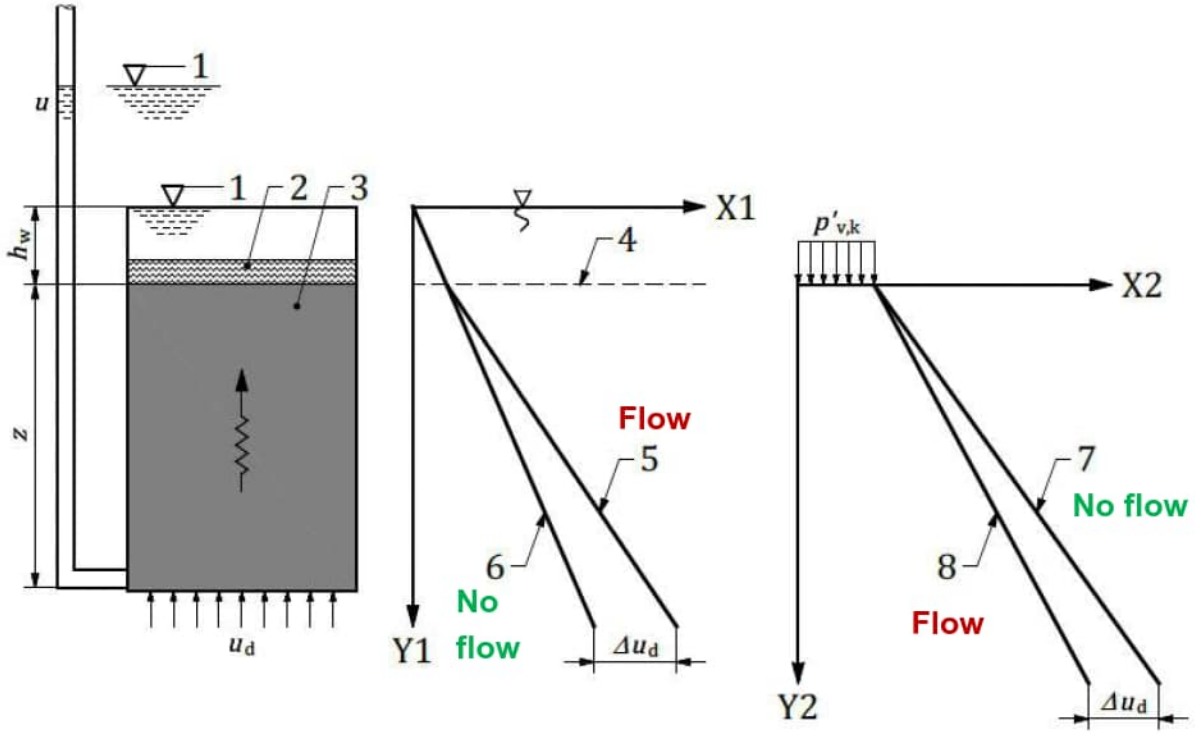
The example deals with the seepage under a diaphragm wall that has been built for an open excavation in silt. The ground surface is located at 22 masl, and the excavation is 7 m deep. The foot of the wall is at 12 masl, hence the embedment depth of the wall is z=3 m. In plan the excavation has a rectangular shape 32 m width and 120 m long, thus allowing for a 2D analysis of the seepage of water and the study of heave risk.

The regional GWL is located at the ground surface, and the water level within the pit is kept 1 m above the bottom of the excavation depth.

Hydraulic heave
New code approach

<REQ> To prevent an ultimate limit state of hydraulic heave (Figure 8.2), the inequality given in Formula (8.3) shall be verified:

$$\Delta u_{\mathrm{d}} \leq \gamma_{\mathrm{HYD}} \left(\gamma_{\mathrm{rep}} - \gamma_{\mathrm{w,rep}}\right) z + \gamma_{\mathrm{pv}} p'_{\mathrm{v,rep}} \qquad (8.3)$$



Key

1 Water level

2 Filter

3 Ground

4 γ_{rep} hw

5 u_d (in presence of flow)

6 u_0 (in absence of flow)

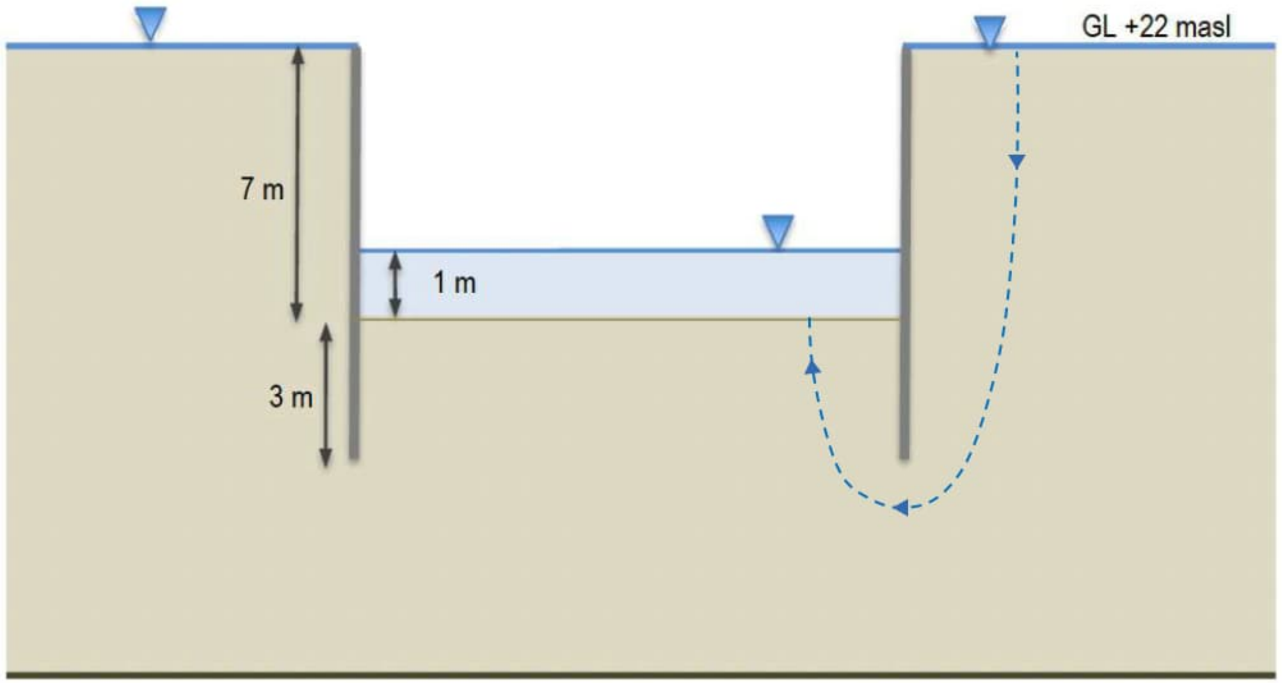
7 $\sigma'_{v,0}$ (in absence of flow)

8 $\sigma'_{v,d}$ (in prescende of flow)

Figure 8.2 – One dimensional upward flow of water

Hydraulic heave

Comparison between the current EC7 and the new draft version



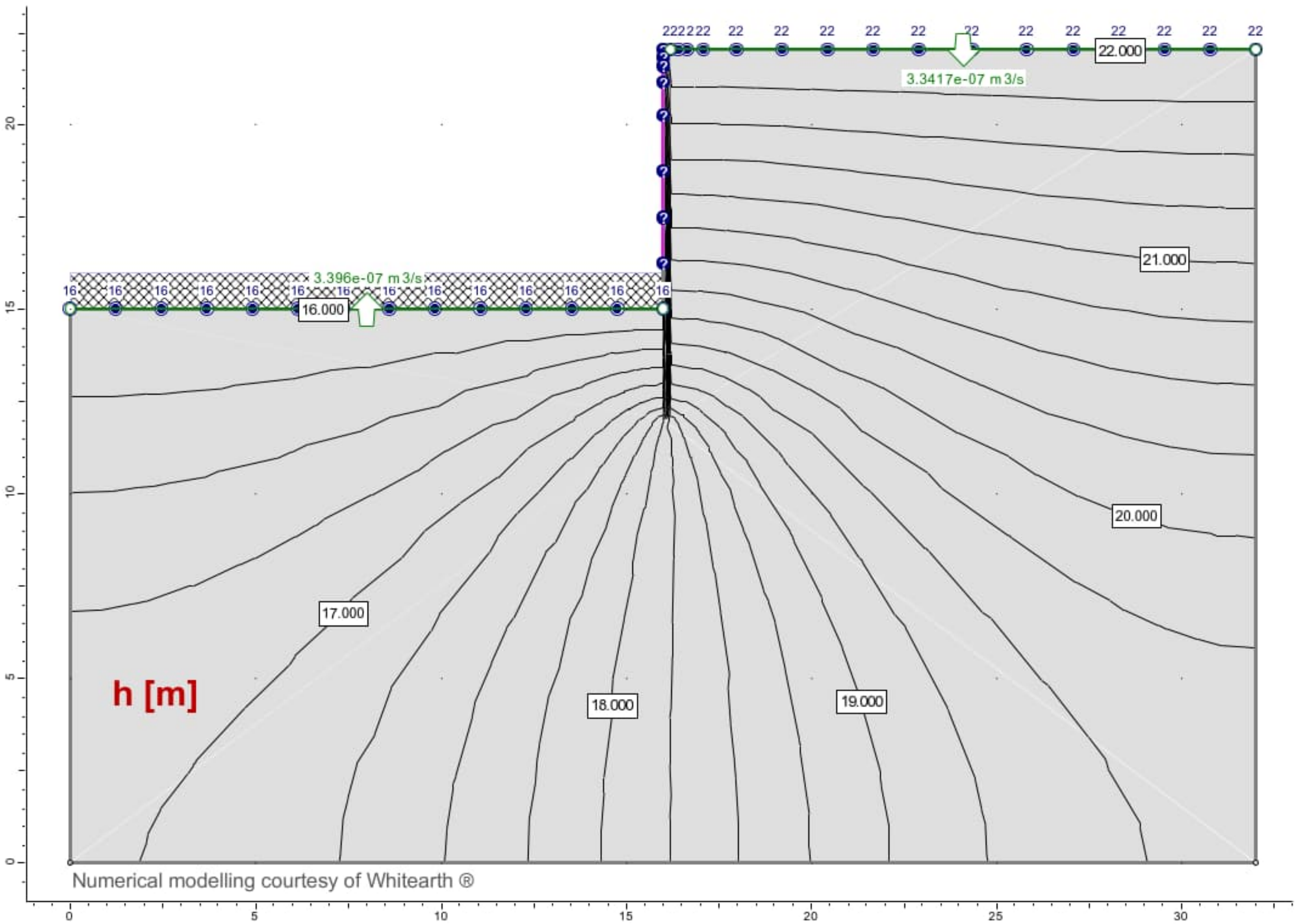
$$\Delta u_{d} \leq \gamma_{HYD}\left(\gamma_{rep} - \gamma_{w,rep}\right)z + \gamma_{pv}p'_{v,rep}$$

New code draft

With respect to the partial factors, the the purpose of this factor [γ_{HYD} = 0,67] is to ensure that the effective stress in the ground remains positive and to allow for local variations of stresses at a granular scale, relevant to hydraulic failures.

Hydraulic heave

Exercise 1 - Assess if the situation would comply with the SC7 requirements with respect to hydraulic heave ULS

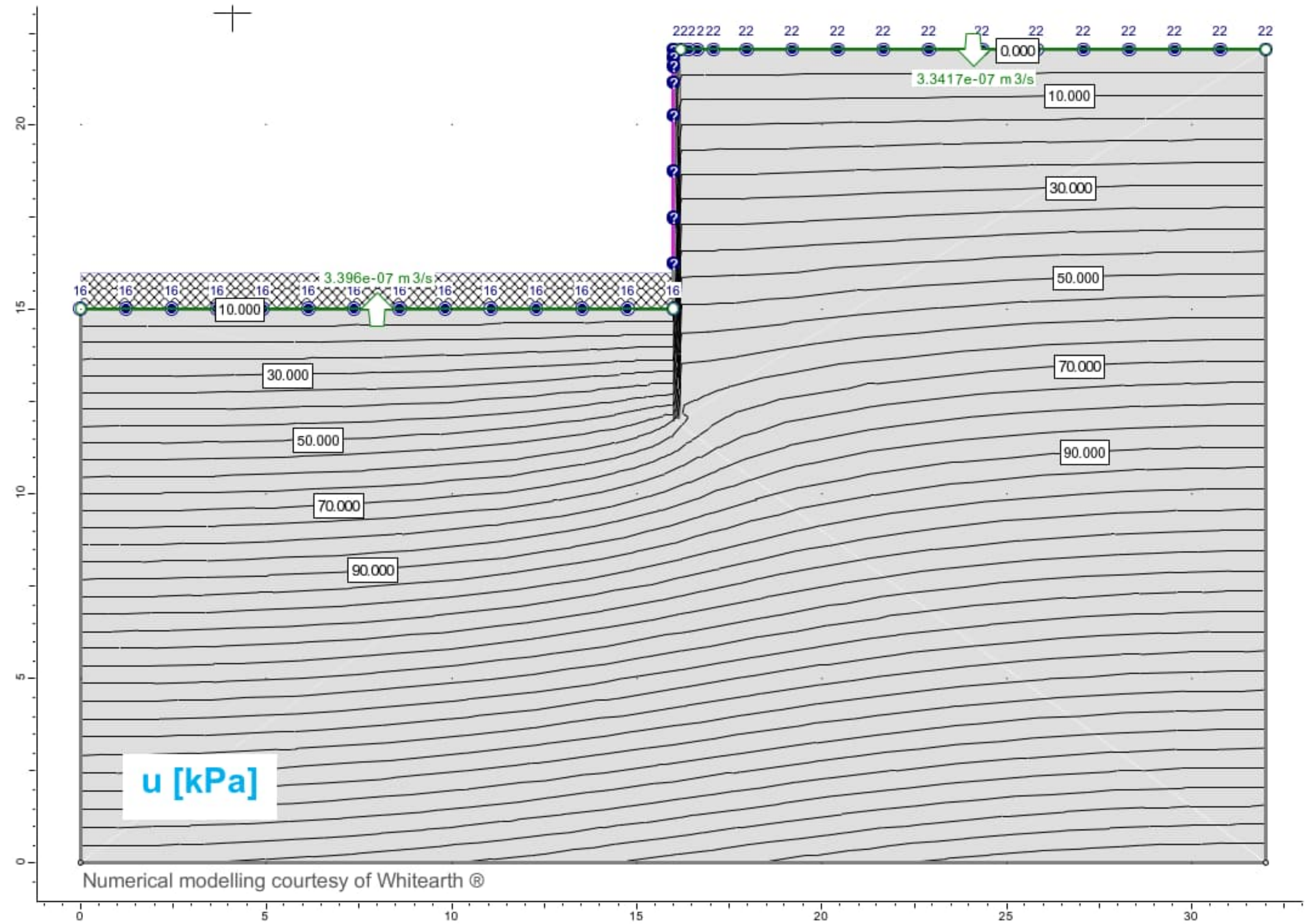


2D Laplace equation solving

The 2D flow of the water has been investigated solving Laplace's equation with numerical methods in commercial software. The left figure shows the contour lines for head [h (m)] around the diaphragm wall, note that $\Delta h = 0.25$ m. For simplicity, the specific weight of water has been assumed to be $\gamma_w = 10$ kN/m³.

Hydraulic heave

Exercise 1 - Assess if the situation would comply with the SC7 requirements with respect to hydraulic heave ULS

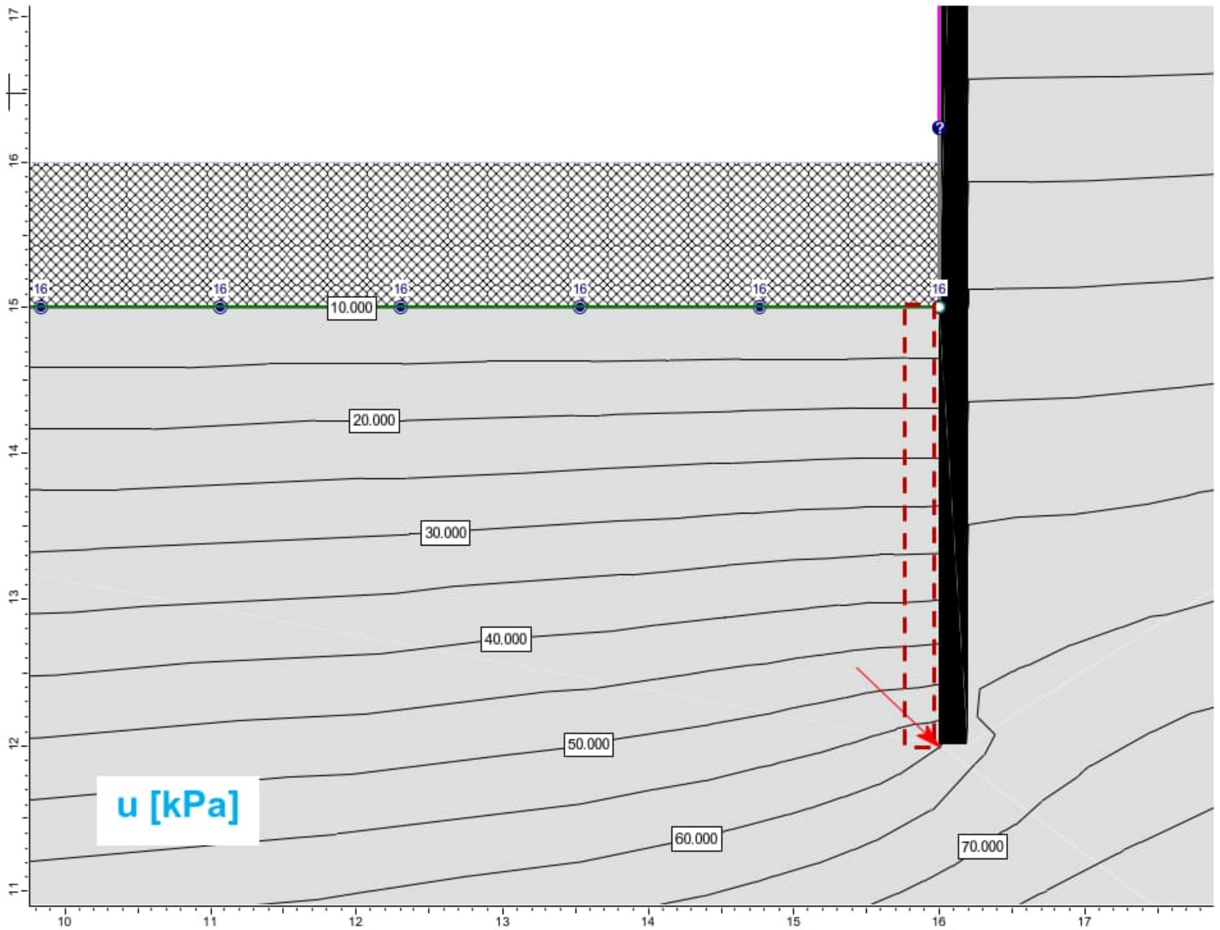


2D Laplace equation solving

The left picture shows the contour of the pore water pressure [u] in the presence of flow. The contour lines have been drawn following $\Delta u = 5$ kPa.

Hydraulic heave

Exercise 1 - Assess if the situation would comply with the SC7 requirements with respect to hydraulic heave ULS



2D Laplace equation solving

This picture shows a detailed view of the area surrounding the wall and close to the bottom of the excavation. The most critical relevant column is that adjacent to the wall [1xD].

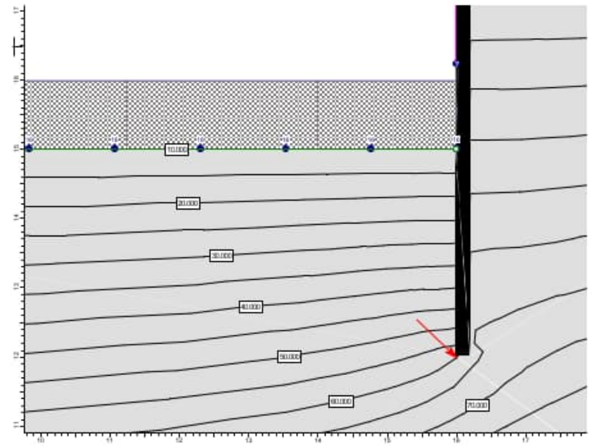
The values of stress/pore water pressure present at the left corner of the base of the wall (marked with an arrow) are shown in next page

Hydraulic heave

Exercise 1 - Assess if the situation would comply with the SC7 requirements with respect to hydraulic heave ULS

The following values of stress/pore water pressure are present at the left corner of the base of the wall (marked with an arrow):

- Pore water pressure in presence of flow [u_d] 60 kPa (from numerical model).
- Pore water pressure in absence of flow [u_0] equal to (16 m – 12 m)*γ_w = 40 kPa.
- Resulting design pore water pressure in the presence of flow [Δu] equal to 20 kPa.
- Total vertical pressure [σ_v] equal to 3 m*(γ_{sat} - γ_w) + 4 m*γ_w = 64 kPa.
- Effective vertical pressure in presence of flow [$\sigma'_{v,d}$] equal to 64 kPa-60 kPa = 4 kPa.
- Effective vertical pressure in absence of flow [$\sigma'_{v,0}$] equal to 3 m*(γ_{sat}-γ_w) = 24 kPa.



According to clause (4), to prevent an ultimate limit state of hydraulic heave, the following inequality shall be verified:

$$\Delta u_d \leq \gamma_{HYD} \left(\gamma_{rep} - \gamma_{w,rep}\right) z + \gamma_{pv} p'_{v,rep}$$

Hence,

$$20 kN/m^2 \leq \gamma_{HYD} \, (18 \, kN/m^3 - 10 \, kN/m^3) \, 3 \, m + \gamma_{pv} \, . \, 0$$

Hydraulic heave

Exercise 1 - Assess if the situation would comply with the SC7 requirements with respect to hydraulic heave ULS

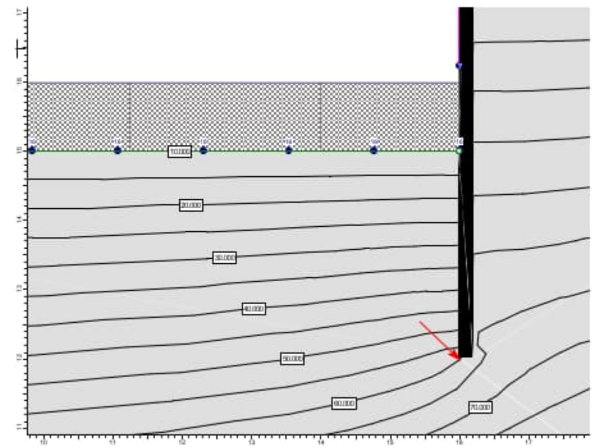
Considering NOTE 2 on the partial factors the inequality yields,

$$20\ \text{kN/m}^2 \le 0.67\ (18\ \text{kN/m}^3 - 10\ \text{kN/m}^3)\ 3\ \text{m} + 0.67\ .\ 0$$

$$20\ \text{kN/m}^2 \le 0.67\ (18\ \text{kN/m}^3 - 10\ \text{kN/m}^3)\ 3\ \text{m} + 0.67\ .\ 0$$

$$20\ \text{kN/m}^2 \le 16.08\ \text{kN/m}^2\ \text{(inequality not satisfied)}$$

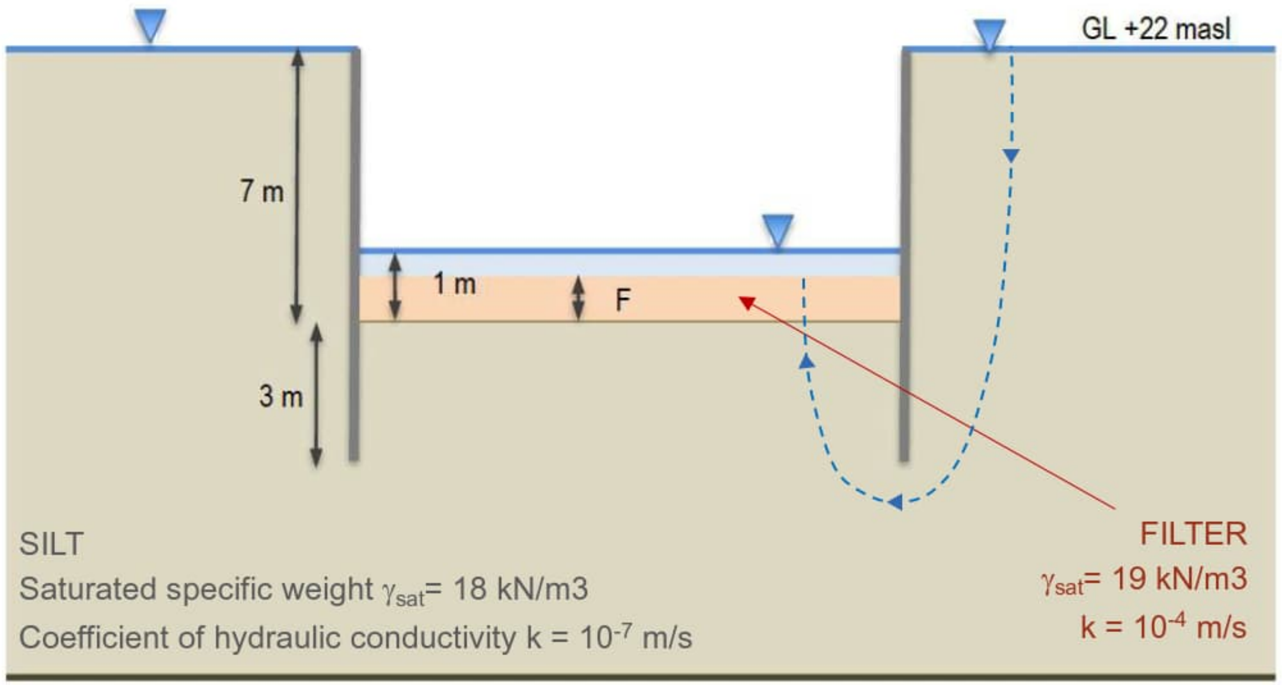
And therefore the requirements for safety against heave risk are not verified.



$$\Delta u_{\mathrm{d}} \le \gamma_{\mathrm{HYD}} (\gamma_{\mathrm{rep}} - \gamma_{\mathrm{w,rep}}) z + \gamma_{\mathrm{pv}} p'_{\mathrm{v,rep}}$$

Hydraulic heave

Exercise 2 - Find the minimum thickness [F] for a filter of coarse sand that would be required to fulfil the requirements of the code with respect to hydraulic heave ULS.



The properties of the natural ground remain the same, and the properties of the filter are [γ_{sat} = 19 kN/m³; k = 1.10^{-4} m/s]. For simplicity, it is considered that the specific weight of the filter does not change regardless its saturation ratio.

Hydraulic heave

Exercise 2 - Find the minimum thickness [F] for a filter of coarse sand that would be required to fulfil the requirements of the code with respect to hydraulic heave ULS.

In this case, the following inequality shall be verified

$$20 \leq \gamma_{HYD}\ (18\ kN/m3 - 10\ kN/m3)\ 3\ m + \gamma_{pv} \cdot p'_{v,rep}$$

Applying the partial factors, the inequality yields,

$$20 \leq 0.67\ (18\ kN/m3 - 10\ kN/m3)\ 3\ m + 0.67 \cdot p'_{v,rep}$$

Hence,

$$20 \leq 0.67\ (18\ kN/m3 - 10\ kN/m3)\ 3\ m + 0.67 \cdot p'_{v,rep}$$

$$20 \leq 16.08 + 0.67 \cdot p'_{v,rep}$$

$$5.85\ kN/m^2 \leq p'_{v,rep}$$

According to the left calculations, the minimum value that is requested for the vertical effective pressure at the bottom of the filter is 5.85 kPa. Considering the specific weight of the filter (γ_{sat} = 19 kN/m3), the *'submerged'* specific weight of the filter would be 9 kN/m3, and the minimum thickness that is requested to comply with the code is 0.65 m.

Acknowledgements

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5.1.2.e

Thanks for joining us!

The recordings and presentations of this webinar will be shared.

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