Seminar
Nieuwe Eurocode 7 – Geotechniek

Welkom!

KIVI Engineering Society

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Chairman of the day

Adriaan van Seters

Thanks to our sponsors

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Volg instructies van het personeel

Verlaat het pand zo snel mogelijk via de dichtstbijzijnde nooduitgang

AED's aanwezig: op de 4e etage – bij de receptie – buiten bij de hoofdingang

Brandblusapparatuur op alle etages en elke vloer

Program

09.30 Welcome

09.40 The new generation of Eurocodes – Eurocode EN1990 'Basis of Design' and the link to Eurocode 7

10.05 Eurocode 7 – Part 1 - Geotechnical Design – Safety concept, ULS- and SLS-design

10.30 EC7-Part 1 from a Dutch perspective

11.00 Coffee break

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Welcome

Adriaan van Seters

KIVI Engineering Society

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Seminar on Eurocode 7 Introduction

Adriaan van Seters
Chair TC250/SC7
NEN-Geotechniek

Development of Eurocode 7

2002 – 2007 – Introduction present Eurocodes

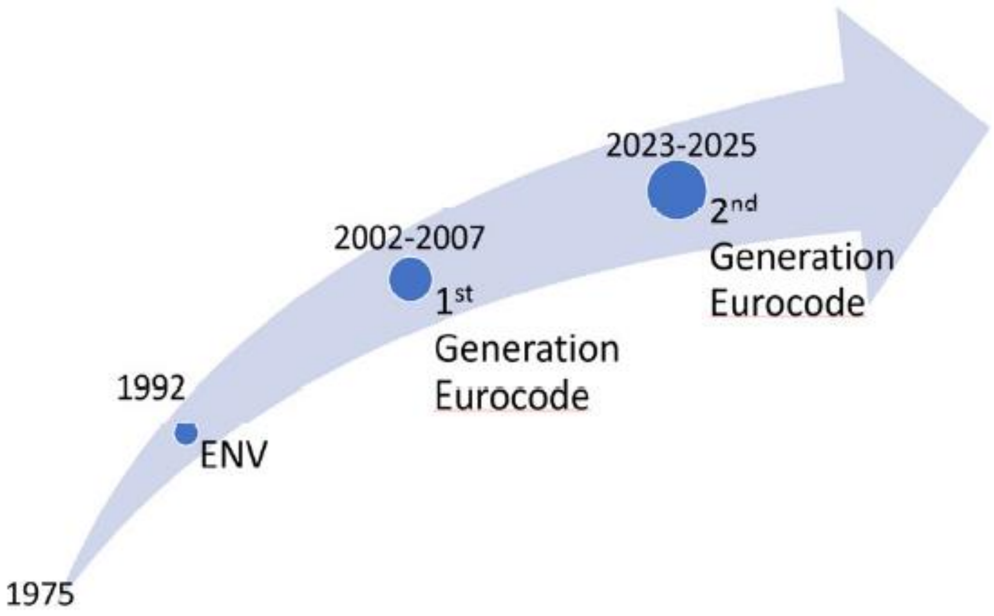
2011 – 2016 – Evolution Groups studing topics for revision of Eurocode 7

2015 – 2024 – Drafting of 2nd Generation Eurocode 7 by 6 project teams, 13 Review TaskGroups

2035 – 2040 – Next Generation of EC7??

Large Geotechnical Community involved:

- 30 in Project teams
- 200 in Taskgroups
- hundreds in National Mirror Committees



Main objectives for Second Generation EC7

CEN/TC250: Standard suitable for all common design cases without demanding disproportinate levels of effort

<u>Other objectives:</u>

Ease of use

- Clear language, same structure in all Eurocodes,
- Avoid alternative rules
- No rules of little practical use, no “textbook”

Harmonisation

- More common design rules,
- Less Nationally Determined Parameters
- Rules for Rock Engineering

Future developments

- Finite Element Method
- Probabilistic design

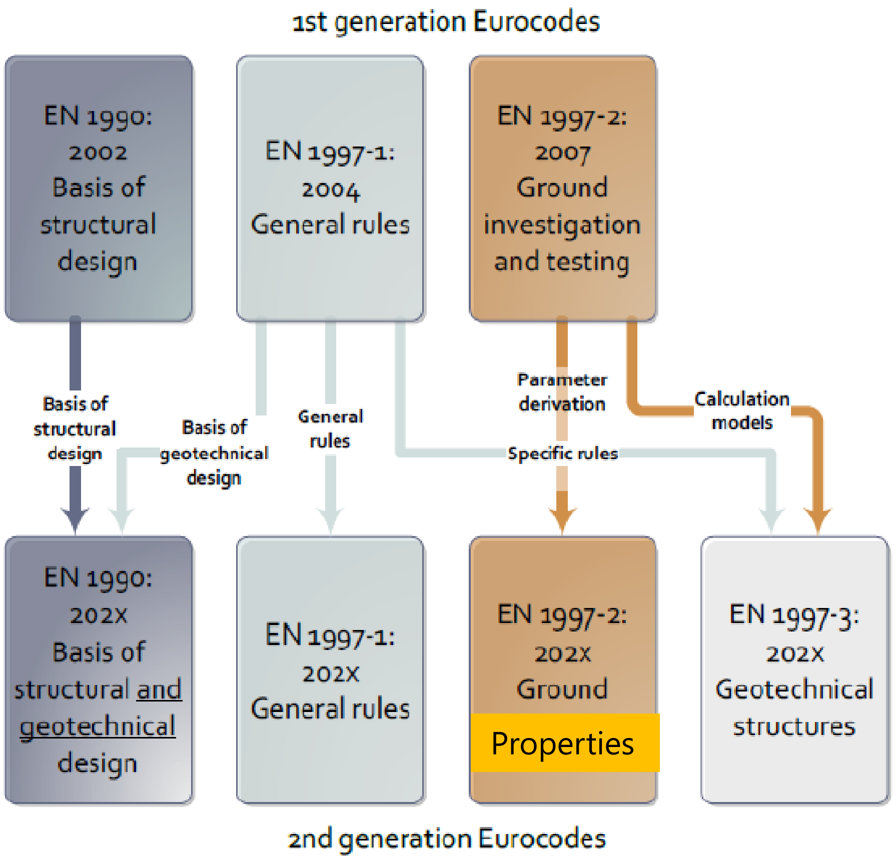
Main changes in Eurocode 7

Old Eurocode (3 parts):

1. Basis of design – EN1990
2. EC7 Part 1 – Geotechnical rules
3. EC7 Part 2 – Testing and derivation of parameters

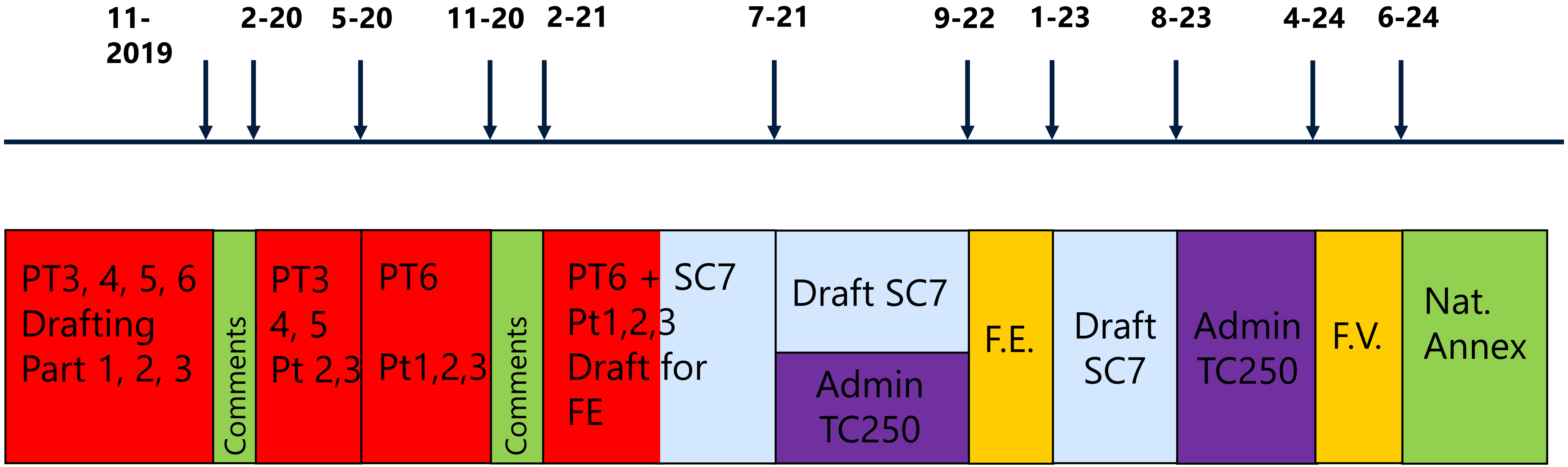
New Eurocode (4 parts!):

1. EN1990 – Basis of design – also geotechnical!
2. EC7 Part 1 – General rules for all structures, safety, characteristic values
3. EC7 Part 2 – Ground Properties and how to derive them from tests
4. EC7 Part 3 – Rules for specific geotechnical structures, many calculation models in Annexes



Bond (2019)

Development of Eurocode 7



F.E. – Formal Enquiry – Comments from Member States
F.V. – Formal Vote – Yes/No Vote from Member States

How can you contribute?

Participation in Eurocode 7:

Review of the Eurocode text:

- Per November: drafts of all 3 parts of Eurocode 7 are available
- Review by countries until End of January 2020

In The Netherlands:

- Get digital copy of the text from Carloes Pollemans - NEN: carloes.pollemans@nen.nl
- Use the WORD-commenting form
- Send your comments before January 15th 2020 to Carloes.
- Dutch Mirror-group = NEN-committee Geotechniek et al.

FUGRO

The new generation of Eurocodes – Eurocode EN1990 'Basis of Design' and the link to Eurocode 7

Andrew Bond

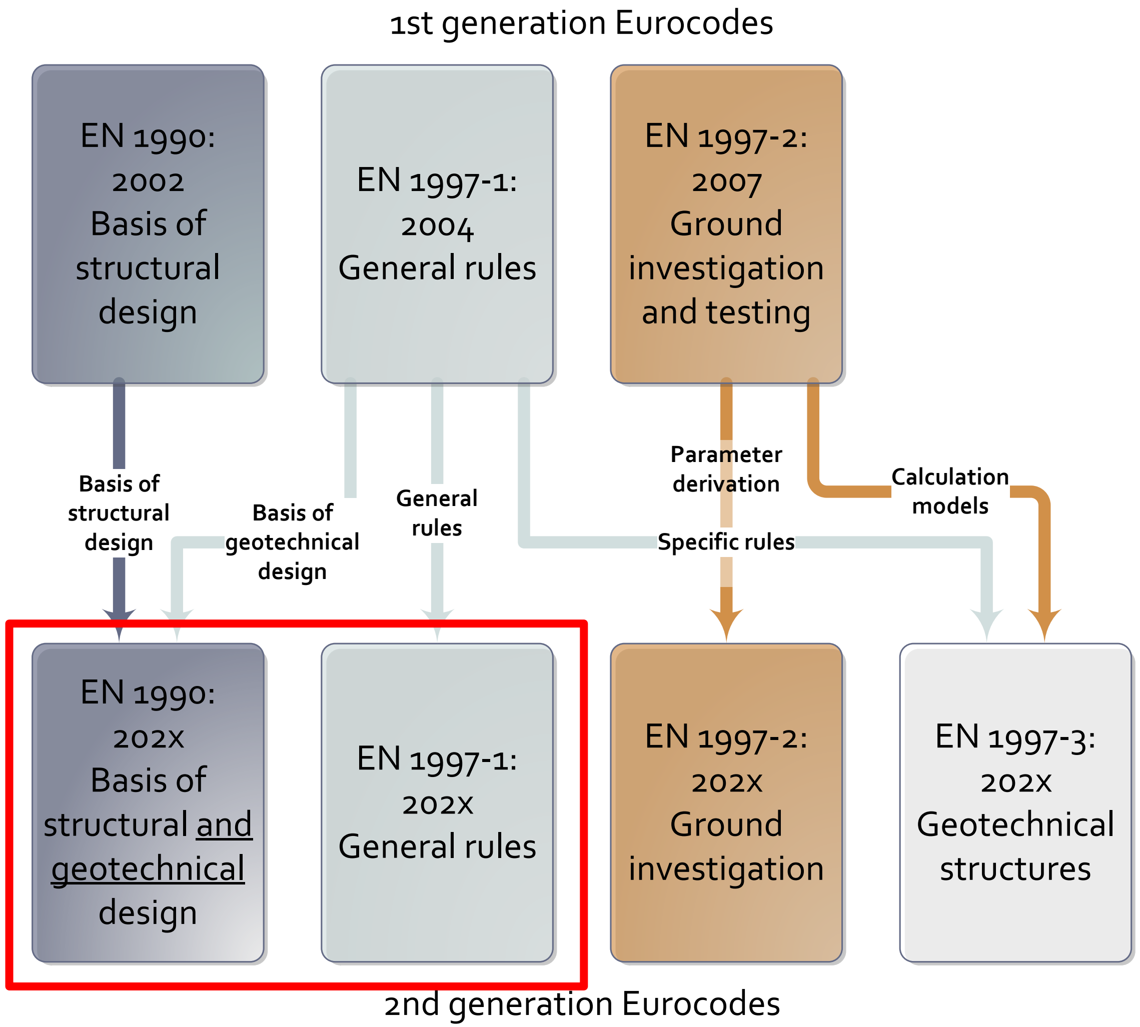
KIVI Engineering Society

NEN

New generation of Eurocodes ENs 1990 & 1997: “Basis of design” and “Geotechnical design”

Dr Andrew BOND, Geocentrix (Past-Chairman TC250/SC7)

Redistribution of topics between Eurocode 7 and EN 1990



Improvements in 2nd generation EN 1990

New generation of Eurocodes ENs 1990 & 1997

1st generation of EN 1990 and 1997-1 Verification of ultimate limit states

Loss of static equilibrium (limit state 'EQU') is verified using:

$$E_{d,dst} \leq E_{d,stb}$$

Rupture and excessive deformation of a section, member, or connection ('STR' and/or 'GEO') are verified using:

$$E_d \leq R_d$$

In EN 1997-1, uplift ('UPL') is verified using:

$$V_{dst,d} \leq G_{stb,d} + R_d \; \left(\equiv E_{d,dst} \leq E_{d,stb} + R_d\right)$$

This expression caters for combined loss of equilibrium <u>and</u> rupture, which is only mentioned in NOTE 2 to Table A1.2(A) of EN 1990

1st generation of EN 1990
Design effects-of-actions and resistance

In EN 1990:2002, design values of effects-of-actions E_d can be calculat… from:

$$E_\mathrm{d} = \overbrace{\gamma_\mathrm{Sd} E\{\gamma_{\mathrm{f},i} F_{rep,i}; a \ldots\}}^{material\ strength\ missing}$$

goo…

In EN 1990,:2002, desi… …n:

Equations need to be generalized to cater for non-linear behaviour and geotechnical design

$$\ldots\left\{\eta_i \frac{X_{\mathrm{k,i}}}{\gamma_{\mathrm{m,i}}}; a_\mathrm{d}\right\} = R\left\{\eta_i \frac{X_{\mathrm{k,i}}}{\gamma_{M,\mathrm{i}}}; a_\mathrm{d}\right\}$$

$$-or-$$

$$\underbrace{R_k/\gamma_\mathrm{R}}_{factors\ on\ resistance}$$

2nd generation of EN 1990
ULS verification including non-linear behaviour

Ultimate limit states must be verified using:

$$E_d \leq R_d$$

Factor may be applied to actions:

Factored actions (DC1-3)

$\gamma_F = \gamma_{Sd} \times \gamma_f$

Factors may be applied to materials:

Material factor approach (MFA)

$\gamma_M = \gamma_{Rd} \times \gamma_m$

or to effects:

$E_d = \boxed{\gamma_F} E\{\Sigma(\psi F_k); a_d; X_{Rd}\}$

Factored effects (DC4)

or to resistance:

$R\{\eta X_k; a_d; \Sigma F_{Ed}\}$

Resistance factor approach (RFA)

$\gamma_R = \gamma_M = \gamma_{Rd} \times \gamma_m$

2nd generation of EN 1990
Design values of the effects of actions

The design effect of actions E_d <u>should</u> be ca

Effects now depend on material properties

$$E_d = \gamma_{Sd} E\left\{\Sigma(\gamma_f \psi F_k); a_d; \boxed{X_{Rd}}\right\}$$

For **linear structural systems** and **certain geotechnical structures**, E_d <u>may</u> be calculated from:

$$E_d = E\left\{\overbrace{\boxed{\Sigma F_d}}^{F_d=\gamma_F \psi F_k}; a_d; X_{Rd}\right\} = \underbrace{E\left\{\Sigma\left(\boxed{\gamma_F}\psi F_k\right); a_d; X_{Rd}\right\}}_{\gamma_F=\gamma_{Sd}\times\gamma_f}$$

Factors applied to actions

For **non-linear structural systems** and **certain geotechnical structures**, E_d <u>may</u> be calculated from:

$$E_d = \gamma_E E\left\{\overbrace{\boxed{\Sigma F_{rep}}}^{F_{rep}=\psi F_k}; a_d; X_{Rd}\right\} = \ldots$$

$or \{\ldots X_{rep}\}$

Factors applied to effects

$\gamma_E = \gamma_{Sd} \times \gamma_f$

EN 1997 specifies the geotechnical structures for which these apply

2nd generation of EN 1990
'Design cases' replace Sets A, B, and C

design case

set of partial factors applied to actions or effects of actions for verification of a specific limit state

Design cases first appear here:

Annex A (normative) **Application rules**

A.1 General application and application for buildings

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

Similar tables will appear for other structural types:

- for general application and for buildings, in Annex A.1;
- for bridges, in Annex A.2;
- for towers, masts and chimneys, in Annex A.3;
- for silos and tanks, in Annex A.4;
- for structures supporting cranes and other machineries in Annex A.5;
- for marine coastal structures, in Annex A.6.

2nd generation of EN 1990

Partial factors for buildings/geotechnical structures

Action or effect				Partial factors γ_F & γ_E for Design Cases 1-4				
Type	Group	Symbol	Resulting effect	Struct-ural	Static equilibrium and uplift*		Geotechnical design	
				DC1	DC2(a)	DC2(b)	DC3	DC4
Permanent action (G_k)	All	γ_G	unfavourable/ destabilizing	1.35 K_F	1.35 K_F	1.0	1.0	G_k is not factored
	Water	$\gamma_{G,w}$		1.2 K_F	1.2 K_F			
	All	$\gamma_{G,stb}$	stabilizing	not used	1.15		not used	
	Water	$\gamma_{G,w,stb}$			1.0			
	(All)	$\gamma_{G,fav}$	favourable	1.0	1.0		1.0	
Prestress (P_k)		γ_P		See other relevant Eurocodes				
Variable action (Q_k)	All	γ_Q	unfavourable	1.5 K_F	1.5 K_F		1.3	1.1
	Water	$\gamma_{Q,w}$		1.35 K_F	1.35 K_F		1.15	1.0
	(All)	$\gamma_{Q,fav}$	favourable	0				
Effects-of-actions (E)		γ_E	unfavourable	effects are not factored				1.35 K_F
		$\gamma_{E,fav}$	favourable					1.0

Set 'B'
DA 1-1

Set 'A'
Table A1.2(A) NOTE 2

Set 'C'
DA 1-2

DA2 *

*worse outcome of (a) and (b) applies

2^{nd} generation of EN 1990
The 'single-source principle'

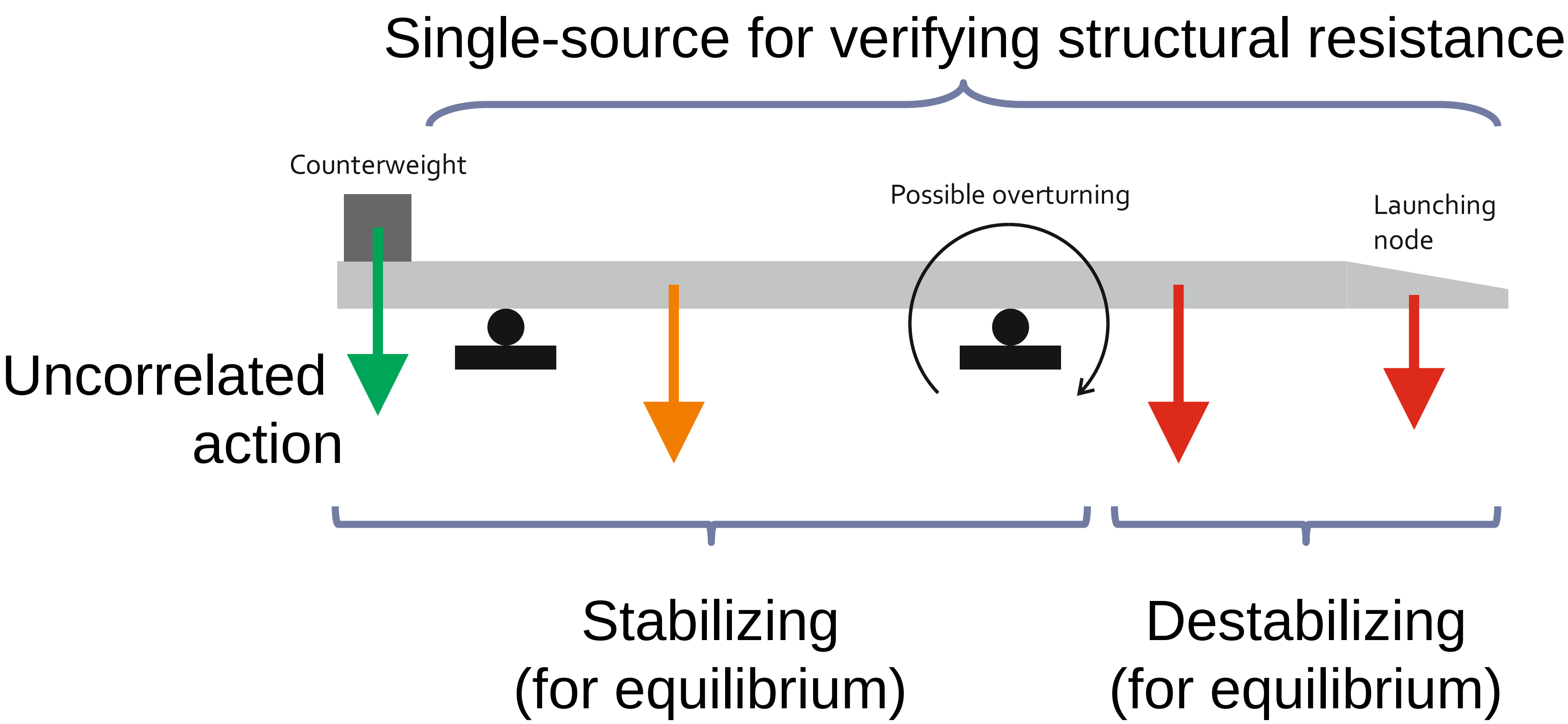
Actions from a single source that, owing to physical reasons, induce effects that are strongly correlated with one another may be treated as a single action, even when they originate in, or act on, different parts of the structure, or originate from different materials.

NOTE 1 This rule is commonly known as the 'single-source principle'.

NOTE 2 The single-source principle typically applies to the self-weight of the structure or the ground and of components made of composite materials as well as for water pressures acting on both sides of a structure with flow passing around or underneath.

When verifying loss of static equilibrium, variations in the magnitude or spatial distribution of permanent actions from a single-source should be considered.

2nd generation of EN 1990
Applying single-source/variation from it



2nd generation of EN 1990
Specification of variable water actions

The representative value of a variable water action ($Q_{w,rep}$) is given by:

$$Q_{w,rep} = G_{w,rep} + \underbrace{\overbrace{Q_{w,k}}^{\substack{depending\ on \\ design\ situation}}}_{=Q_{w,k}|Q_{w,comb}|Q_{w,freq}|Q_{w,qper}}$$

Value of variable water action	Symbol	Probability of exceedance	Return period (years)
Characteristic	$Q_{w,k}$	2% per annum	50
Combination	$Q_{w,comb}$	5% per annum	20
Frequent	$Q_{w,freq}$	1% during design service life	-
Quasi-permanent	$Q_{w,qper}$	50% during design service life	-
Accidental	$A_{w,rep}$	0.1% per annum	1000

2^{nd} generation of Eurocode 7
Specification of groundwater pressures

Representative groundwater pressure ($F_{w,rep}$) is given by:

Uncertainty in static water pressure

$$F_{w,rep} = \begin{cases} \overbrace{G_{w,k,sup} \text{ or } G_{w,k,inf}}^{\textit{whichever is more adverse}} \\ -or- \\ G_{w,k,mean} + \underbrace{Q_{w,rep}}_{=\boxed{Q_{w,k}}|Q_{w,comb}|Q_{w,freq}|Q_{w,qper}} \end{cases}$$

$$\textit{depending on design situation}$$

Variability in dynamic water pressure

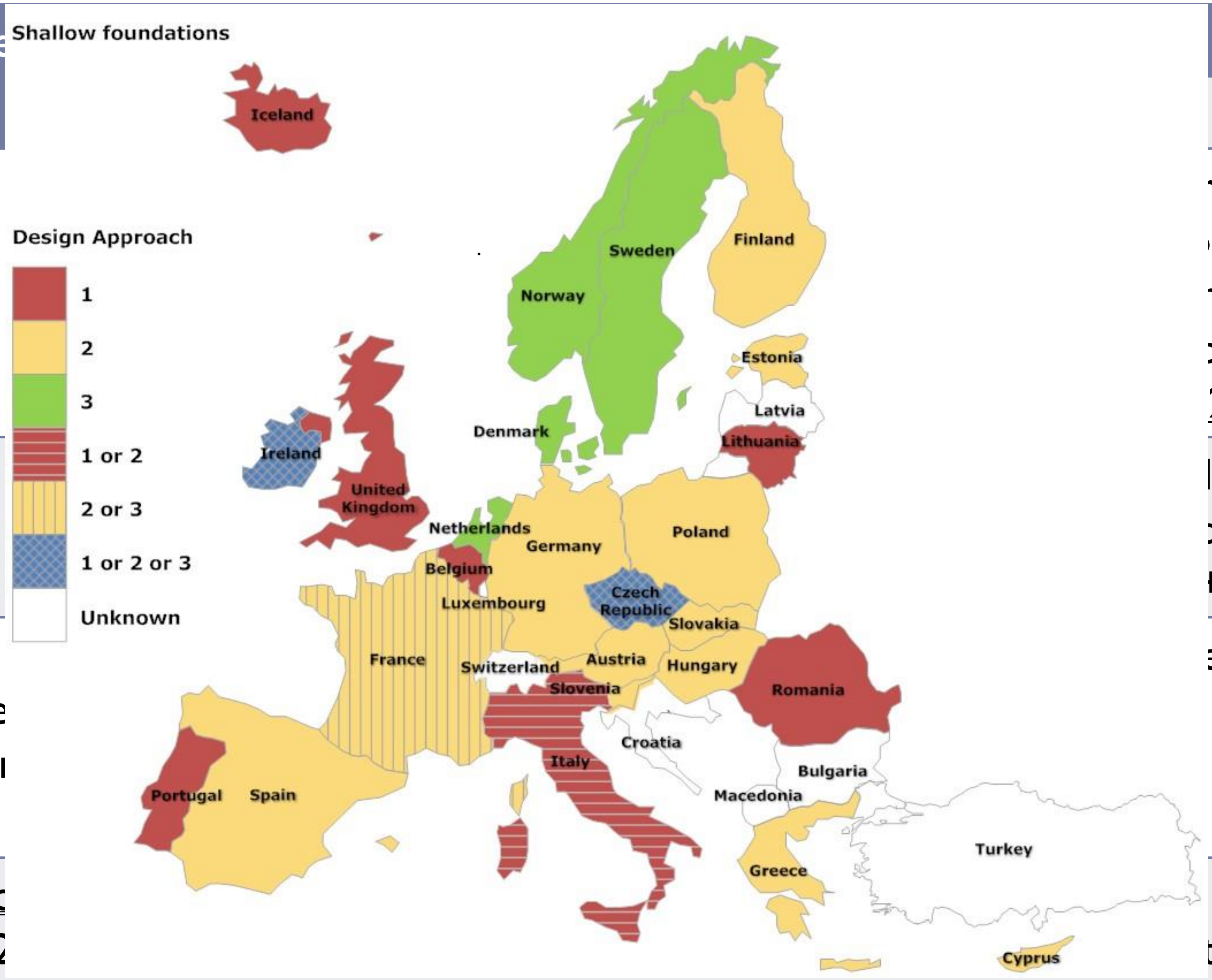
If there is insufficient data to derive values on the basis of annual probability of exceedance, … $Q_{w,k}$ and $Q_{w,comb}$ should be selected as a cautious estimate of the worst value likely to occur during the design situation

Improvements in 2nd generation EN 1997

New generation of Eurocodes ENs 1990 & 1997

1st generation of Eurocode 7 Complexity of Design Approaches (Bond & Harris, 2008)

Structure	3
General structures	ructural ns/effects material perties 2+M2+R3
Slopes	laterial perties +M2+R3
Axially loaded pile and anchor	es
(Major) fac Sets A1-A2	tances



2nd generation of EN 1990
Design values of resistance

The design resistance R_d should be calculat

Resistance now depends on actions

$$R_d = \overbrace{\frac{1}{\gamma_{Rd}} R\left\{\frac{\eta X_k}{\gamma_m}; a_d; \boxed{\Sigma F_{Ed}}\right\}}$$

R_d may be calculated from (the '**material factor approach**

Factors applied to strength

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

R_d may be calculated from (the '**resistance factor approac**

Factors applied to resistance

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

2nd generation of Eurocode 7
Partial factors for ULS (Bond et al., 2019)

Verification of	Partial factor on		Material factor approach*		Resistance factor approach
			a	b	
Overall stability of slopes	Actions/effects	γ_F/γ_E	DC3 γ_G = 1.0, γ_Q = 1.3		Not permitted
	Ground properties	γ_M	$\gamma_{\tan\varphi}$ [illegible]		
	Earth resistance	γ_{Re}	Not factored		
Spread foundations	Actions/effects	γ_F/γ_E	DC1 $\gamma_G = 1.35\ K_F$ $\gamma_Q = 1.5\ K_F$	DC3 γ_G = 1.0 γ_Q = 1.3	DC4 γ_Q = 1.1 $\gamma_E = 1.35\ K_F$
	Ground properties	γ_M	γ_t [illegible] γ [illegible]	[illegible]	[illegible]red
	Bearing resistance	γ_{Rv}	Not permitted		1.4
	Sliding resistance	γ_{Rh}			1.1
*Where two cases (a and b) are given, verify both					

Harmonized choice (MFA only)

National choice via NDP (MFA or RFA)

2nd generation of EN 1997
Introducing 'representative' values of material properties

- Design value of a material property X_d should be calculated from:

$$X_{\mathrm{d}} = \frac{\boxed{X_{\mathrm{rep}}}}{\gamma_{\mathrm{M}}} = \frac{\boxed{\eta}\, X_{\mathrm{k}}}{\gamma_{\mathrm{M}}}$$

- Example, for concrete:

$$\overbrace{f_{\mathrm{c,d}} = \alpha_{cc}\frac{f_{\mathrm{c,k}}}{\gamma_{\mathrm{C}}}}^{\mathrm{EN\ 1992-1-1:2004}} \equiv \frac{\overbrace{(\eta_{cc}k_{tc})f_{\mathrm{c,k}}}^{\mathrm{prEN\ 1992-1-1}}}{\gamma_{\mathrm{C}}} \Rightarrow f_{\mathrm{c,rep}} = (\eta_{cc}k_{tc})f_{\mathrm{c,k}}$$

- For ground properties:

$$X_{\mathrm{rep}} = \begin{cases} \eta X_{\mathrm{k}} & \textit{based on statistics (mostly, 50\% fractiles)} \\ X_{nom} & \textit{based on judgement} \text{ ("cautious estimate")} \end{cases}$$

1st generation of Eurocode 7
Geotechnical Categories are confused!


(14) Geotechnical Category 1 should
ely simple structures:

— for which it is possible to ensure that the fundamental requirements will be satisfied on the basis of experience and qualitative geotechnical investigations;

(15) Geotechnical Category 1 procedures should
there is negligible risk in
terms of overall stability or
s, which are known from
comparable local experienc
ese cases the procedures
may consist of routine meth
struction.

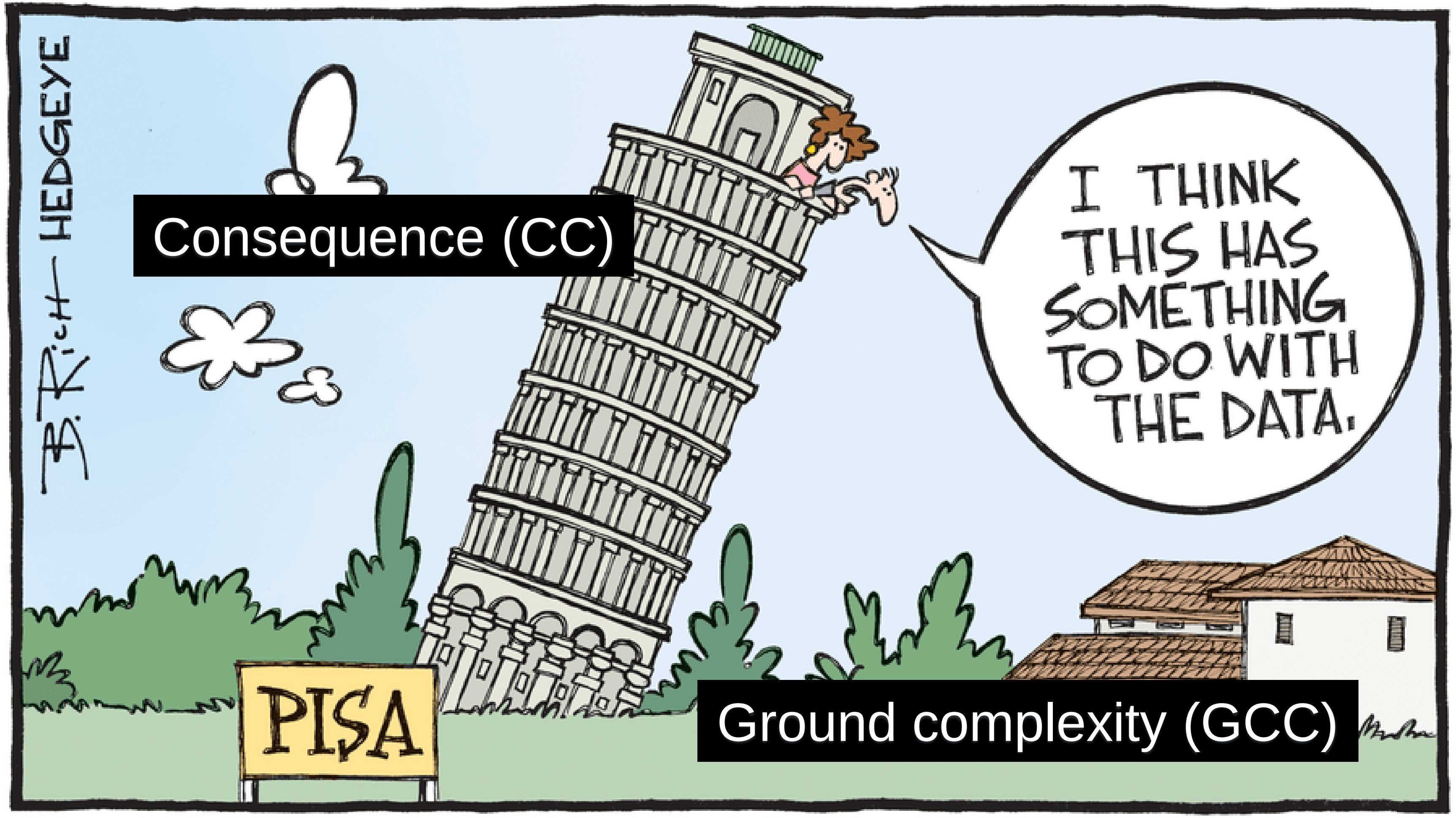
(16) Geotechnical Catego
sed only if there is no excavation below
the water table or if c
indicates that a proposed excavation below
the water table will

(17) Geotechnical Cate
include conventional types
with no exceptional risk
oading cond

(18) Designs for structures in Geotechnical Category 2 should normally include quantitative geotechnical data and analysis to ensure that the fundamental requirements are satisfied.

(19) Routine procedures for field and laboratory testing and for design and execution may be used for Geotechnical Category 2 designs.

2^{nd} generation of Eurocode 7
Separation of consequence and complexity



2nd generation of EN 1990
Consequence classes, examples, and factors

Consequence class/ Description		Loss of human life*	Economic, social or environ-mental*	Examples of buildings	Factor K_F
CC4	Highest	Extreme	Huge	Additional provisions can be needed	
CC3	Higher	High	Very great	Grandstands, large buildings, e.g. a concert hall	1.1
CC2	Normal	Medium	Considerable	Residential and office buildings, small buildings	1.0
CC1	Lower	Low	Small	Agricultural buildings, buildings where people do not normally enter, such as storage buildings, etc.	0.9
CC0	Lowest	Very low	Negligible	Alternative provisions may be used	
*CC is chosen based on the more severe of these two columns					

2nd generation of EN 1990 vs EN 1997
Minimum 'quality levels' based on consequence class

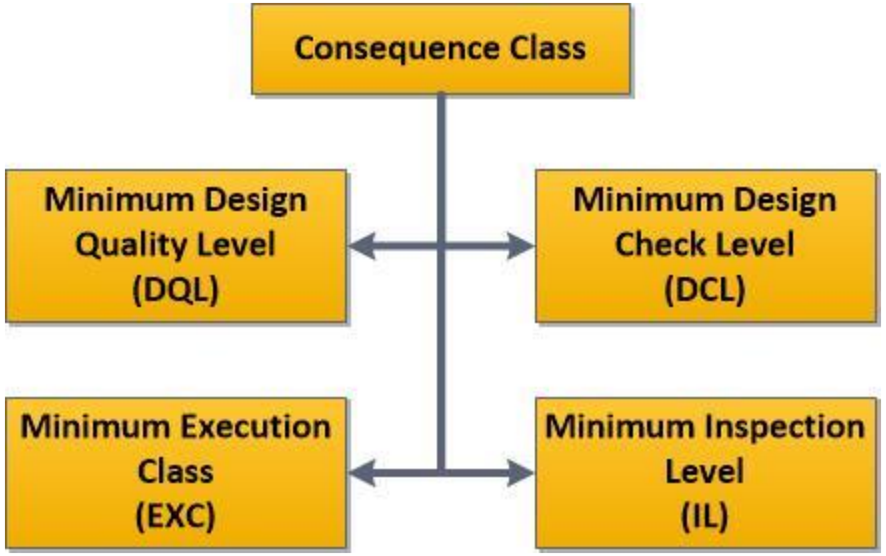
Consequ-ence class	Minimum design quality level	Minimum design check level	Minimum execution class	Minimum inspection level
Higher (CC3)	DQL3	DCL3	See relevant execution and product standards	IL3
Normal (CC2)	DQL2	DCL2		IL2
Lower (CC1)	DQL1	DCL1		IL1

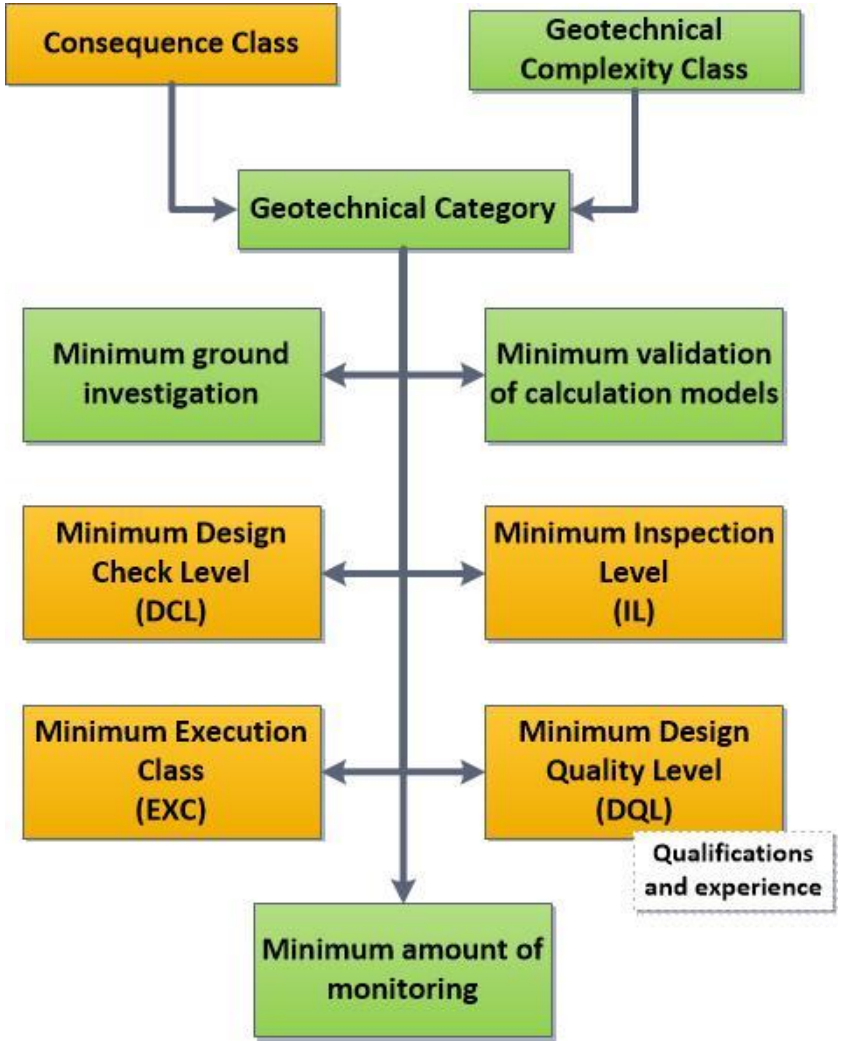
Consequence Class (CC)	Geotechnical Complexity Class (GCC)		
	Lower (GCC1)	Normal (GCC2)	Higher (GCC3)
Higher (CC3)			GC3
Medium (CC2)		GC2	
Lower (CC1)	GC1		

2nd generation of Eurocode 7
Geotechnical complexity classes

Complexity		General features
GCC3	Higher	Any of the following applies • difficult soils • difficult geomorphologies • significant thickness of r Bad und • sliding ground • steep soil slopes • significant geometric variability • significant sensitivity to groundwater conditions • significant complexity of the ground-structure interaction • little experience with calculation models for the current situation
GCC2	Normal	Covers everything not contained in GCC1 or GCC3
GCC1	Lower	All the following conditions apply • uniform ground conditions and standard construction technique • isolated shallow foundatic Good tically applied in the zone • well established design m able for the local conditions and the planned construction technique • low complexity of the ground-structure-interaction

2nd generation of EN 1990 vs EN 1997 Outcomes based on CC or GC





Conclusion

New generation of Eurocodes ENs 1990 & 1997

Improvements in 2nd generation of EN 1990

- Simplification of EQU, STR, and GEO
  - Improves treatment of combined ultimate limit states
- Catering for non-linearity and coupling
  - Incorporates basis of geotechnical design into EN 1990
  - Better treatment of non-linear structural design
- Design cases
  - Simple packaging of complicated loading conditions
- Simpler presentation of combinations of actions
  - Greater clarity in the text
- Water actions
  - Clear specification of probabilities of exceedance
- Management measures to achieve the intended structural reliability
  - Flexible system that caters for national preferences

Improvements in 2nd generation of EN 1997

- Organizational changes to Eurocode 7
  - Clearer layout aids ease-of-navigation
  - Greater consistency with EN 1990 aids ease-of-use
- No more Design Approaches!
  - Simpler (but not simple) choice of partial factors
- Catering for different groundwater conditions
  - Better specification of groundwater pressures
- Separating consequence from hazard
  - Clear distinction between consequence of failure and complexity of the ground
  - Geotechnical Categories now drive meaningful decisions

Thank you for your attention

Dr Andrew BOND, Geocentrix (Past-Chairman TC250/SC7)

Eurocode 7 – Part 1 - Geotechnical Design – Safety concept, ULS- and SLS-design

Sebastian Burlon

KIVI Engineering Society

NEN

Dutch Eurocode 7 Seminar
Eurocode 7 – Part 1 – Geotechnical Design

Sébastien Burlon
(PT6 Leader)

Evolution of EN 1997-1: Collective Work

- *Contributions from PT1*
  (November 2015- October 2017)

- *Contributions from PT2 (in charge of the first draft of EN 1997-1)*
  (November 2015-April 2018)

- PT6: S.Burlon, J.Estaire, G.Franzen, G.Nuijten and G.Scarpelli
  \+ A.Bond (Immediate Past-Chair) and A. Van Seters (Chair)
  (November 2018-April 2021)

- New draft of EN 1997-1:
  Some changes based on WG comments
  rock engineering and dynamic and cyclic loadings

- In April 2020, PT6 will have to harmonize the three parts of EN 1997

Content of EN 1997-1

- 12 Sections + 9 Annexes (normative and informative) – 110 pages

- Sections:

1. Scope
2. Normative References
3. Terms, definitions and symbols
4. Basis of design
5. Materials
6. Groundwater
7. Geotechnical analysis
8. ULS
9. SLS
10. Execution
11. Testing
12. Reporting

- Annexes:

A. Partial factors for ground properties (N)
B. Representative value assessment procedures (I)
C. Limiting values of strutural deformation and ground movement (I)
D. Checklist for construction supervision and performance (I)
E. Additional requirements and recommendations for reporting
F. Ground properties (N)
G. Qualification and professional experience (I)
H. Observational Method (I)
I. Bibliography (I)

Some important issues

- 12 Sections + 9 Annexes (normative and informative) – 110 pages

- Sections:

1. Scope
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12. Reporting

- Rock engineering
- Reliability management: Ground Model, Geotechnical Design Model, Representative values
- ULS/SLS – MFA/RFA
- Numerical modelling
- Groundwater pressures, uplift and hydraulic verifications
- Dynamic and cyclic loading

Rock engineering

- Soils and rocks are addressed at the same level: ground

EN 1997-1 is almost ready to be used for rock engineering

<u>EN 1997-1</u>

- Clause 3: Definitions
- Clause 4: Reliability (GCC, CC, GC),

Observational Method, Verification by partial factors and other methods

- Clause 5 : Ground/Soils/Rocks

<u>EN 1997-2</u>

- Clause 3: *main ground properties* are described

<u>EN 1997-3</u>

- Provide approaches and calculation methods to include rock engineering for the design of slopes, shallow foundations, deep foundations, retaining walls, anchors, reiforced ground and ground improvement

Reliability

- Many tools have been introduced in order to address reliability issues :

Consequence classes (from CC0 to **CC4**)

Geotechnical complexity classes (from GC1 to GC3)

↓

Geotechnical categories
(from GC1 to GC3)
(Minimum amount of ground investigation)

↓

Design check level
Design Qualification Level and Experience Level
Inspection level

Verification of geotechnical structures

- Four procedures are presented at the same level:
    - calculations using partial factor method (4.4) or other reliability-based methods;
    - prescriptive measures (4.5);
    - testing (4.6) – see also Clause 11;
    - the Observational Method (4.7) – see also Annex I (modifications are still needed).

    - + Design assisted by testing (4.8).

Representative values

- The concept of characteristic values has been updated.

- Representative values include now two complementary concepts that can be used in parallel based on derived values presented in the GIR:
  - "*Nominal*" values: cautious estimate of ground properties based on the experience and the avalaible information
  - Characterisitc values: statistical analysis of the available data (average with a reliabilty of 95%)

- Annex B presents the main equations to assess characteristic values (effect of the depth not considered).

$$X_k = X_{mean} - k_N\sigma = X_{mean}(1 - k_N.V)$$

$$X_k = \mathrm{e}^{Y_{mean} - \mathrm{k_N}\sigma} = \mathrm{e}^{Y_{mean}(1-k_N.V)} \quad (Y = lnX)$$

$$k_N = \frac{t_{95,N-1}}{\sqrt{N}} \qquad k_N = \frac{N_{95}}{\sqrt{N}}$$

- Values of typical coefficient of variations are provided : $V = \dfrac{\sigma}{X_{mean}}$

ULS

- **RFA (Resistance Factor Approach) / MFA (Material Factor Approach)**

Many alternatives depending on : geotechnical structures, calculation methods (analytical, semi-empirical, numeric modelling, , etc.)

→ ***The specific choices are presented in EN 1997-3***

		Type of geotechnical limit states			
		Rupture and Excessive deformation		*Static equilibrium and uplift*	*Hydraulic failure*
Design Cases		MFA	RFA	MFA and/or RFA	
DC1	$\gamma_Q > \gamma_G > 1.0$	**X (old DA1-1)**	**X (old DA2)**		Specific verifications: - total stresses - hydraulic gradient
DC2	$\gamma_Q > \gamma_G > 1.0$ $\gamma_G = 1.0$; $\gamma_Q > 1.0$			**X**	
DC3	$\gamma_G = 1.0$;$\gamma_Q > 1.0$	**X (old DA1-2)**			
DC4	$\gamma_E > 1.0$; $\gamma_Q > 1.0$		**X (old DA2*) EFA**		

- The old DA3 does not exist anymore but many alternatives are existing : **MFA+DC3 for slopes, MFA+DC3 and DC4 for retaining walls, MFA+DC1 for shallow foundations, etc.**

Numerical modelling - Coupling DC4 (EFA) and MFA (DC3)

Calculation with representative values of the actions and the ground properties for each phase

DC4 (EFA)		MFA (DC3)
$F_d=1.35F_k$ $R_{mob} \leq R_{ult}/\gamma_R$	← Phase 1 →	$R_f>1.25$ F_d
$F_d=1.35F_k$ $R_{mob} \leq R_{ult}/\gamma_R$	← Phase 2 →	$R_f>1.25$ F_d
$F_d=1.35F_k$ $R_{mob} \leq R_{ult}/\gamma_R$	← Phase n →	$R_f>1.25$ F_d

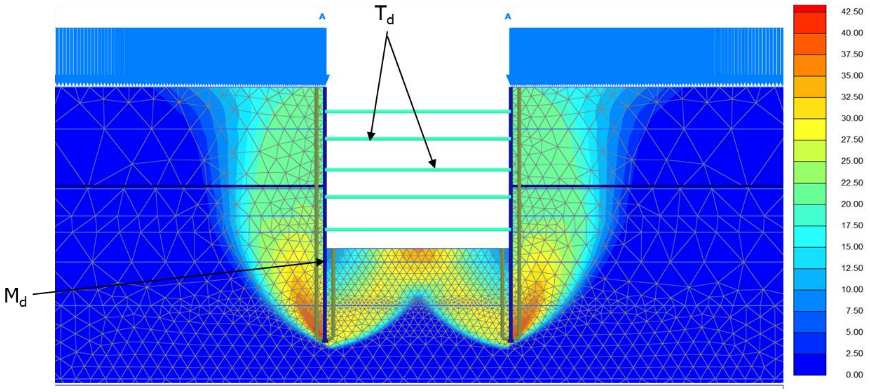
DC4 (EFA)
→ **Structural ULS**
→ **Geotechnical ULS**
(need of specific approach)

MFA (DC3)
→ **Structural ULS**
→ **Geotechnical ULS**

Numerical modelling – MFA (DC3)

During the shear strength reduction procedure, Eurocode 7 recommends:

- (i) to reach the value 1.25 to check the geotechnical verifications,
- (ii) to check the structural verifications with the structural forces that are obtained from this calculation.



Calculation with representative values of the loads and the ground properties for each phase

Phase 1 → Shear strength reduction procedure (MFA) → R_f>1.25, F_d

Phase 2 → Shear strength reduction procedure (MFA) → R_f>1.25, F_d

Phase n → Shear strength reduction procedure (MFA) → R_f>1.25, F_d

Numerical modelling – EFA (DC4)

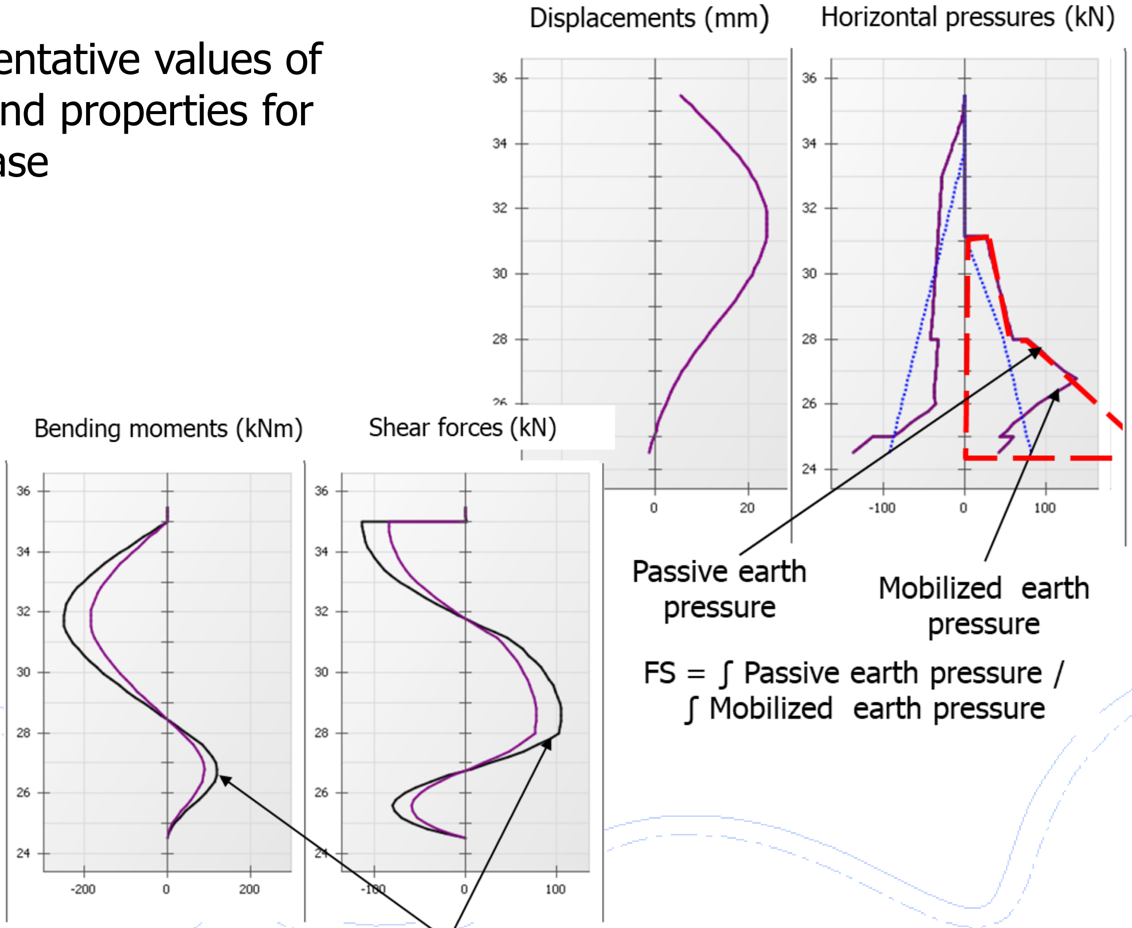
Calculation with representative values of the loads and the ground properties for each phase

ULS verifications

$F_d = 1.35F_k$
$R_{mob} \leq R_{ult}/\gamma_R$ ← Phase 1

$F_d = 1.35F_k$
$R_{mob} \leq R_{ult}/\gamma_R$ ← Phase 2

$F_d = 1.35F_k$
$R_{mob} \leq R_{ult}/\gamma_R$ ← Phase n



FS = ∫ Passive earth pressure / ∫ Mobilized earth pressure

Partial factors are applied of the effet of the actions:
$M_d=1.35M_k$
$T_d=1.35T_k$

terrasol setec

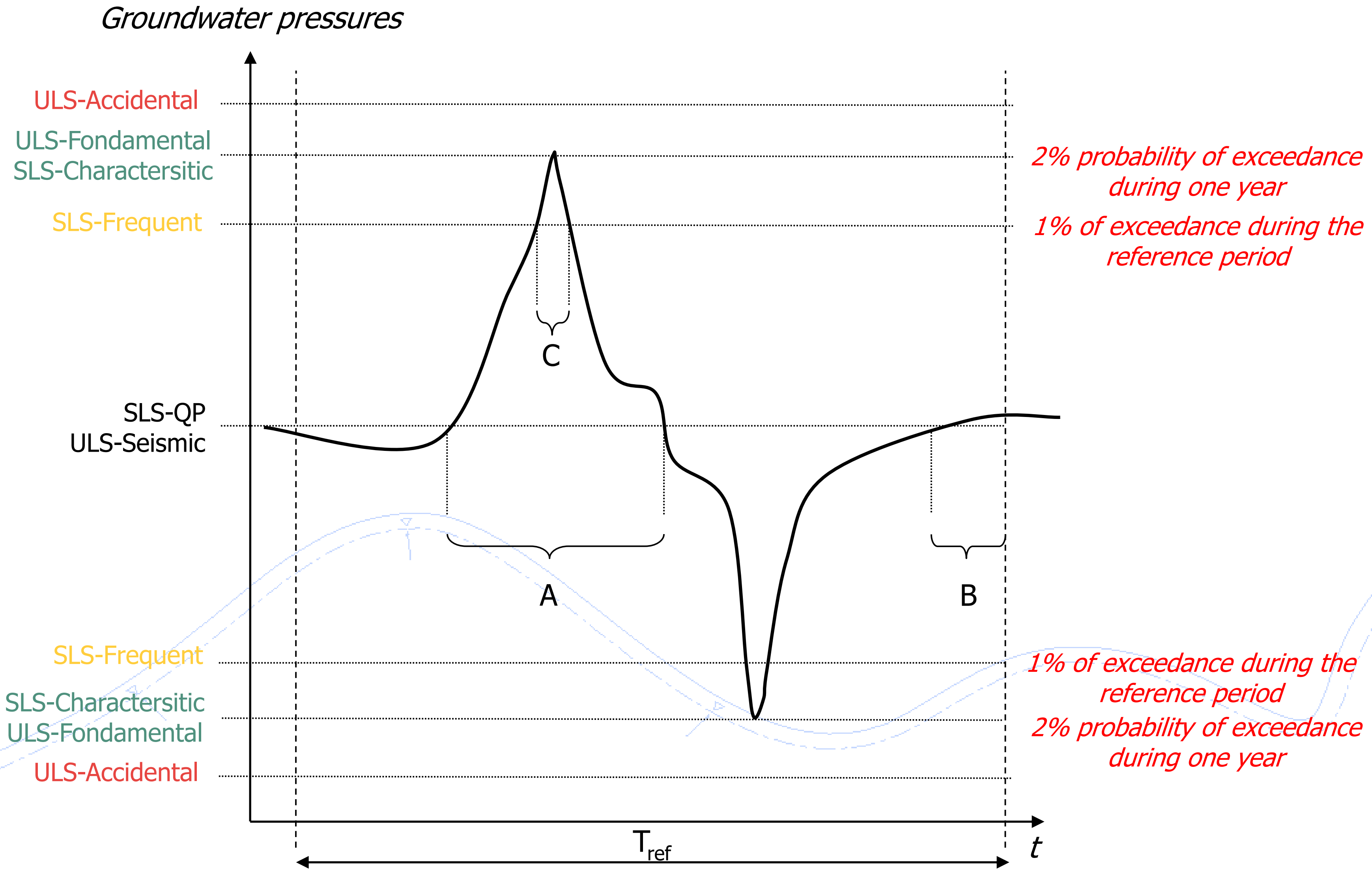
Groundwater pressures

- The text introduces piezometric levels and groundwater pressures:

$$h_w=p_w/\gamma_w+z$$

- Representative values of piezometric levels and groundwater pressures can be selected.
- Only **design values** of groundwater pressures can be defined.
- Modifications of the representative values of piezometric levels or groundwater pressures are possible.

Selection of groundwater pressures



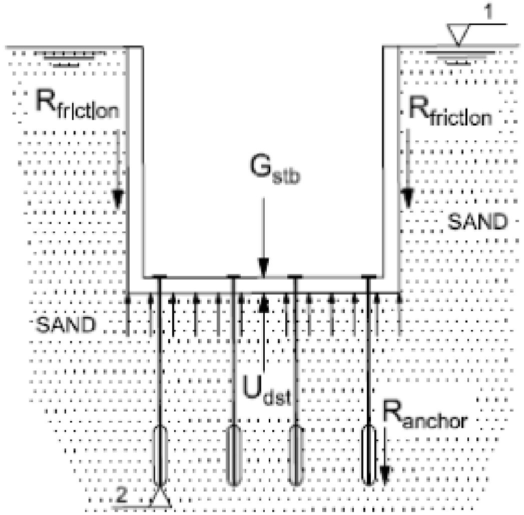
Uplift

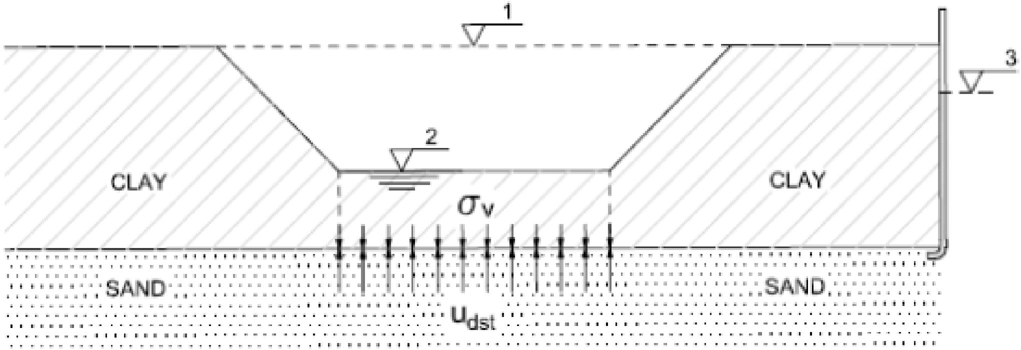
Rigid body

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

No rigid body

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





Hydraulic verifications

➢The two main verifications are the following:

➢ $i_{\mathrm{d}} \leq i_{\mathrm{cd}}$

(verification in terms of hydraulic gradient – local verification)

➢ $u_{\mathrm{d}} \leq \gamma_{\mathrm{w,k}}(z + h_{\mathrm{w}}) + \gamma_{\mathrm{HYD}}\left(\gamma_{\mathrm{k}} - \gamma_{\mathrm{w,k}}\right)z$

(verification that the vertical effective stress is not too much reduced - global verification)

$u_{\mathrm{d}} \leq u_0 + \gamma_{\mathrm{HYD}}\,\sigma'_{\mathrm{v;0}}$ ⇒ $\sigma'_{\mathrm{v;}d} \geq \sigma'_{\mathrm{v;0}}(1 - \gamma_{\mathrm{HYD}})$ ⇒ $\dfrac{\sigma'_{\mathrm{v,0}} - \sigma'_{\mathrm{v,d}}}{\sigma'_{\mathrm{v,0}}} \leq \gamma_{\mathrm{HYD}}$

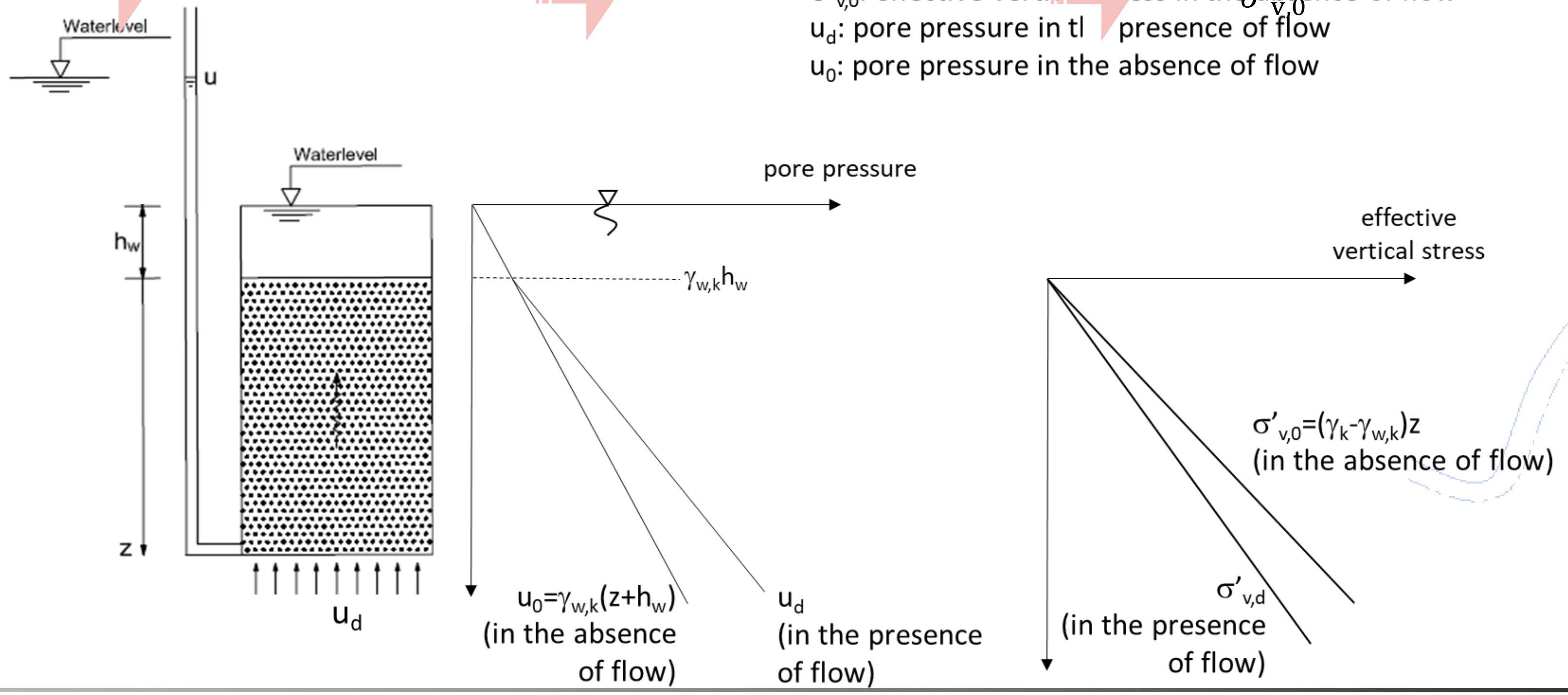
Legend:

$\sigma'_{v,d}$: effective vertical stress in the presence of flow

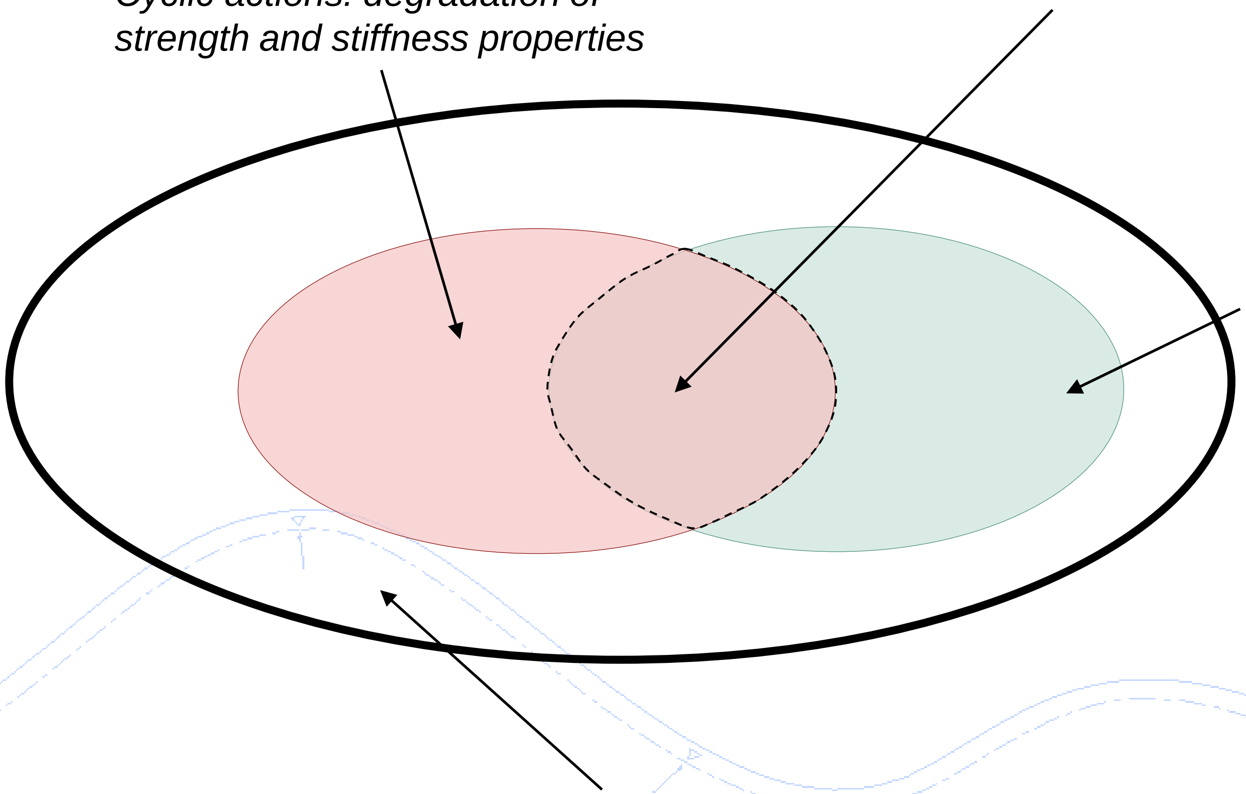
$\sigma'_{v,0}$: effective vertical stress in the absence of flow

u_d: pore pressure in the presence of flow

u_0: pore pressure in the absence of flow



Dynamic and cyclic loading



Dynamic and cyclic loading

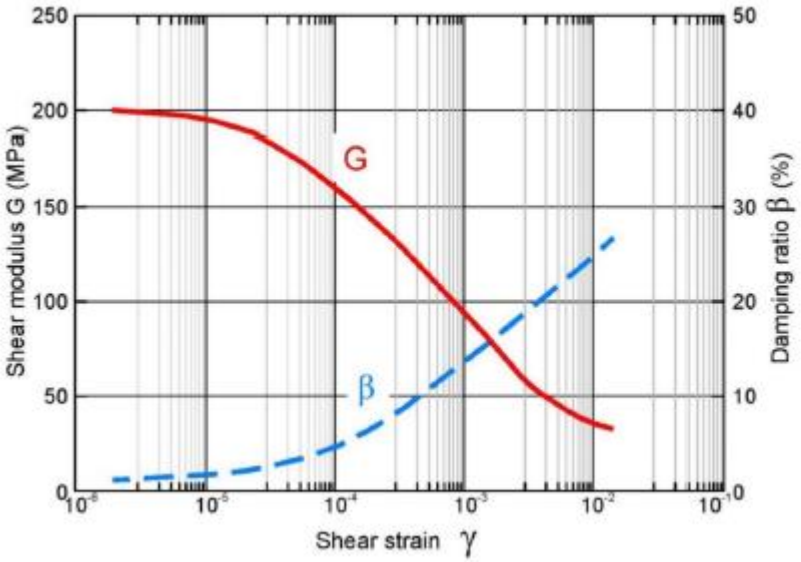
- Dynamic actions :
  - both for ULS and SLS
  - Use of appropriate tools (pseudo-static conditions, full dynamic analysis with material and radiative damping especially for high frequency levels)

- Cyclic actions :
  - Effects of cyclic loading are assessed using the characteristic combinations of SLS (amplitude of the variable load) and the frequent combinations of SLS (effect of cycle number)
  - If established, degradation of stiffness and strength properties are used for ULS and SLS combinations

- Cyclic and Dynamic actions: consider both cyclic and dynamic aspects

Dynamic and cyclic loading

- Interaction with EN 1997-2: all the parameters needed have been presented:
    - Secant shear modulus – Accounting for strain level (« S » shaped curves)
    - Damping ratio – Accounting for strain level



- Cyclic shear strength

Dynamic and cyclic loading

- Interaction with EN 1997-3 (next steps):

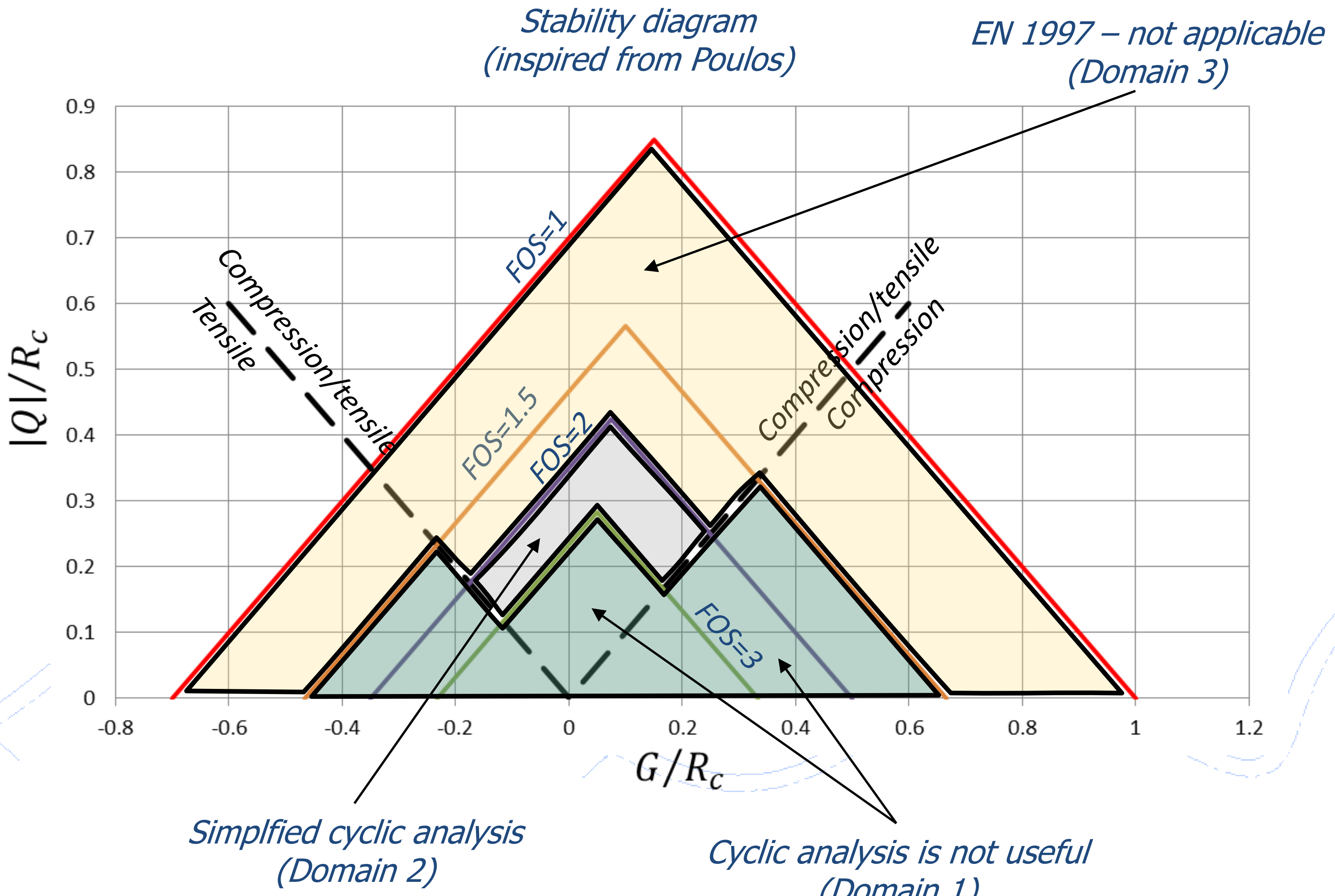
  - *Dynamic loading:*

    - Accepted concepts only for foundations (footings and piles).
    - Gazetas'charts for dynamic design of shallow and deep foundations (interaction with EN 1998-5)

  - *Cyclic loading:*

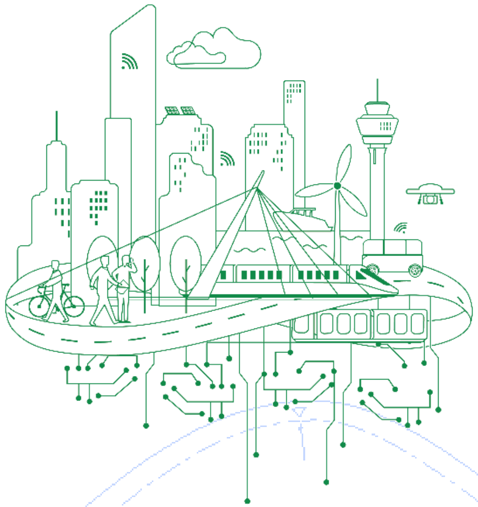
    - Cyclic effects seem to be taken into account only for the design on foundations
    - Criteria for cyclic effects and simplified rules for shallow and deep foundations:
      - Domain 1: cyclic analysis is not useful;
      - Domain 2: simplified cyclic analysis (degradation of soil properties – adjust model factors);
      - Domain 3: specific methods (not described in EN 1997).

Dynamic and cyclic loading



Conclusions

- General principles of geotechnical design are presented
- Reliability tools
- Very flexible code to manage ULS/SLS
- New topics: groundwater pressures, dynamic and cyclic loading, rock engineering
- Your comments are welcome !



THANK YOU
FOR YOUR ATTENTION

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EC7-Part 1 from a Dutch perspective

Hans Brinkman

KIVI Engineering Society

NEN

Geotechnical Design – go Dutch (economic optimisation & freedom)

Enabling Delta Life

Hans Brinkman

New Generation of Eurocodes and EC7-Part 1 from a Dutch perspective

Topics

- Design approaches – a comparison
- Groundwater – Extreme value statistics
- (re)use of information – Bayesian updating
- Decrease with time of annual failure probability – Take the survived construction phase in account
- Increase with time of annual failure probability – How to deal with degradation?
- Probabilistic reliability analysis – Standard option in software how about EN 1997?

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Design approaches – a comparison with current NL-situation

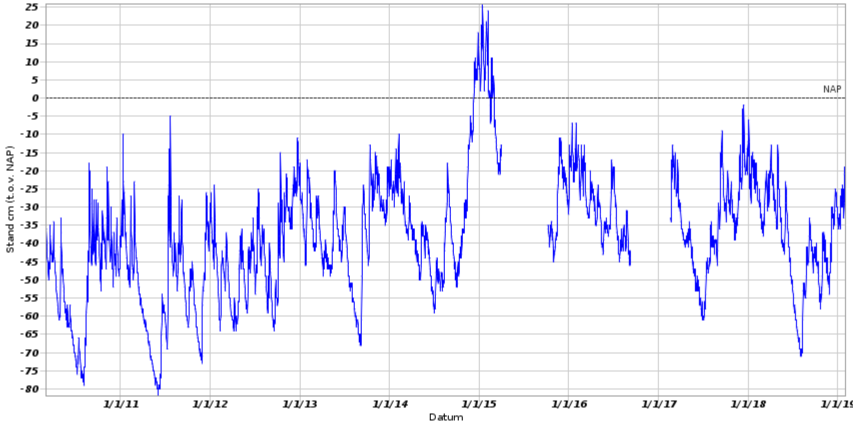
		MFA (a)	MFA (b)	MFA (c)	RFA (a)	RFA (b)
4	Slopes, cuttings, and embankments	**DC3 + M3**				
5	Spread foundations	DC1 + M1	DC3 + M3	**DC1 + M3**	DC1 + $\gamma_{R;i}$	
6	Piled foundations, Axial				**DC1 + $\gamma_{R;i;a}$**	DC3+ $\gamma_{R;i;b}$
	Piled foundations, Transversal	DC4 + M1	DC3 + M3			
	Pile groups and piled rafts	DC4 + M1	DC3 + M3			
7	Retaining structures	**DC4 + M1**	**DC3 + M3**		DC4 + $\gamma_{R;I}$	
8	Anchors				**DC1 + $\gamma_{R;i;1}$**	DC1 + $\gamma_{R; i;3}$
9	Reinforced ground	DC4 + M1	DC3 + M3		DC4 + $\gamma_{R;i}$	
10	Ground improvement, Axial diffused ground improvement	DC1+M1	DC3+M1			
	Ground improvement, Axial discrete rigid Inclusions				DC1 + $\gamma_{R;i;a}$	DC3 + $\gamma_{R;i;b}$
	Ground improvement, Transverse	DC4 + M1	DC4+M1			

at present NL

~at present NL

Groundwater – Extreme value statistics

- prEN 1997-1:2019 art 6.3.3. (2) <REQ> When assessing design groundwater pressures directly or by applying a deviation to the representative piezometric level or groundwater pressure, design values of groundwater pressures for ultimate limit states shall have a **probability of exceedance** as specified in EN1990
- Groundwater is often the key risk driver
- In need of good guideline how statistics work for these Extreme value distributions and the combination of long-term regional and short-term local observations
- Now in prEN1997-1:2019 appendix B only detailed guidelines for the determination of characteristic values soil parameters
- Groundwater should be added to appendix B



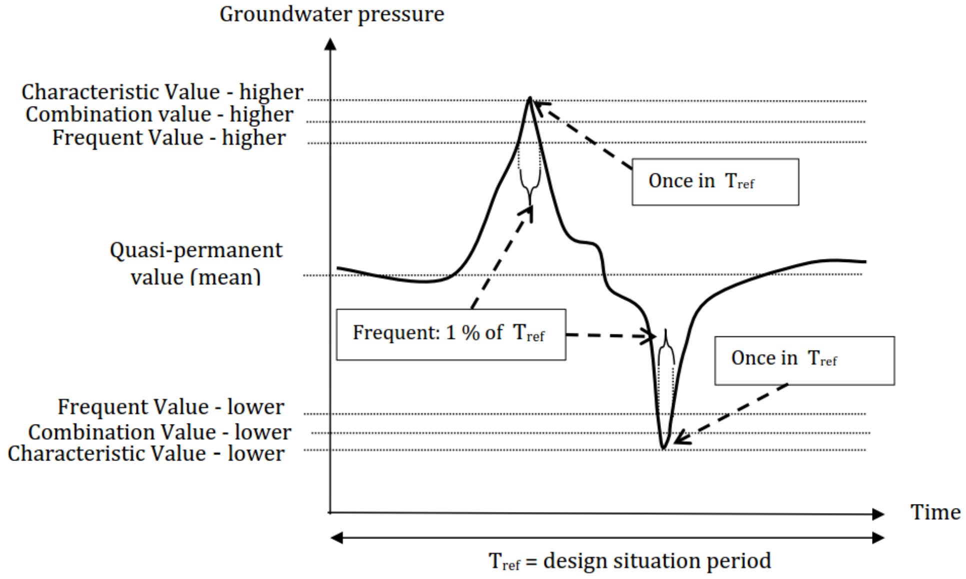


Figure 6.1 Representative values of groundwater pressures – illustration of characteristic, combination, frequent, and quasi-permanent values

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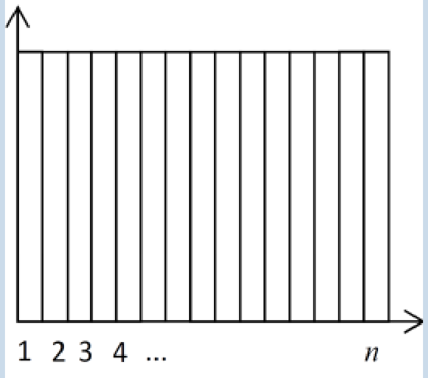
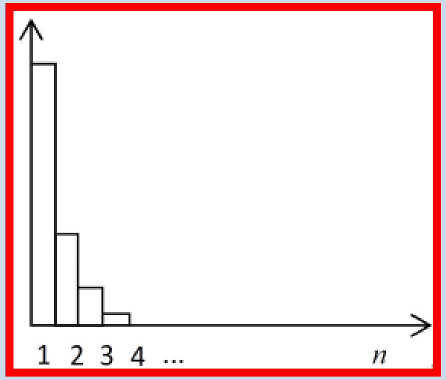
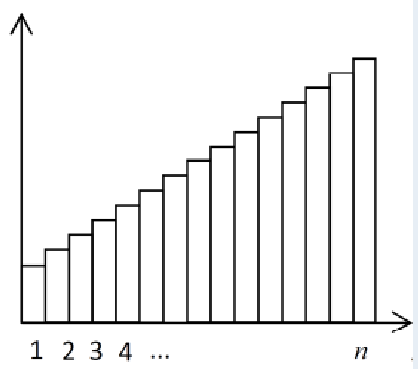
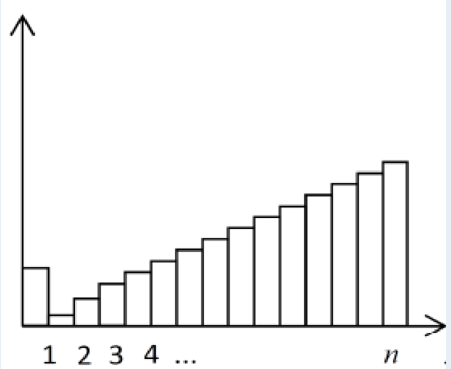
Guidelines how to (re)use information

- Regional/Local parameter sets ➔ Add Bayesian updating to 1997-1 appendix B

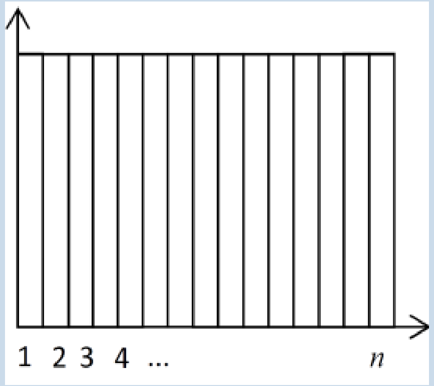
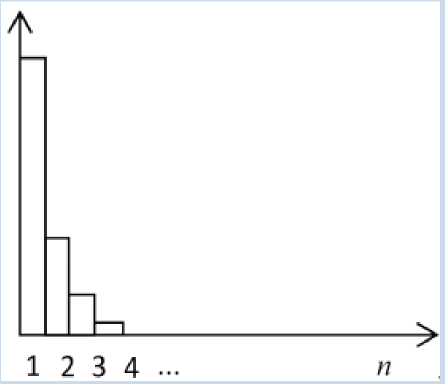
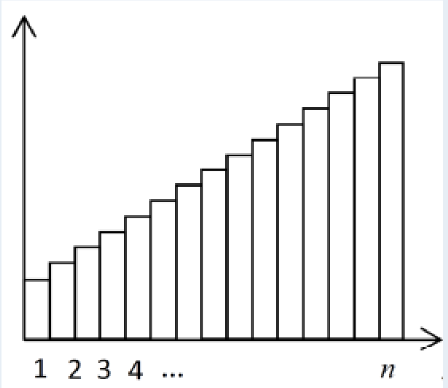
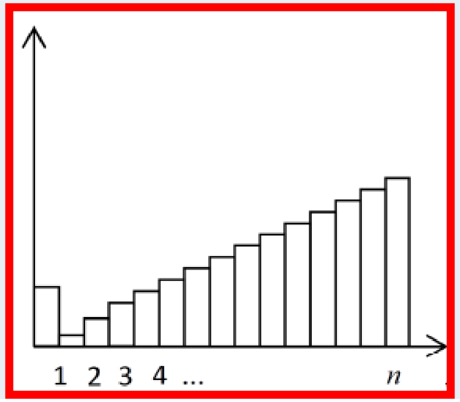
$$\mu_{1+2} = \frac{\frac{\mu_1 \sigma_2^{\,2}}{n} + \mu_2 \sigma_1^{\,2}}{\sigma_1^{\,2} + \frac{\sigma_2^{\,2}}{n}} \qquad \sigma_{1+2} = \sqrt{\frac{\frac{\sigma_1^{\,2} \sigma_2^{\,2}}{n}}{\sigma_1^{\,2} + \frac{\sigma_2^{\,2}}{n}}}$$

Decrease with time of annual failure probability
Take the survived construction phase in account for reliability of service life

- For many geotechnical structures influence variable load not dominant
- Construction phase critical and strength increases with time
- CC during construction phase lower than in Service life

	A. No correlation between years	B. Correlation between years large
C. No increase of load and/or degradation (stationary)		
D. Increase of load and/or degradation (non-stationary)		

Increase with time of annual failure probability
How to deal with degradation?

	A. No correlation between years	B. Correlation between years large
C. No increase of load and/or degradation (stationary)		
D. Increase of load and/or degradation (non-stationary)		

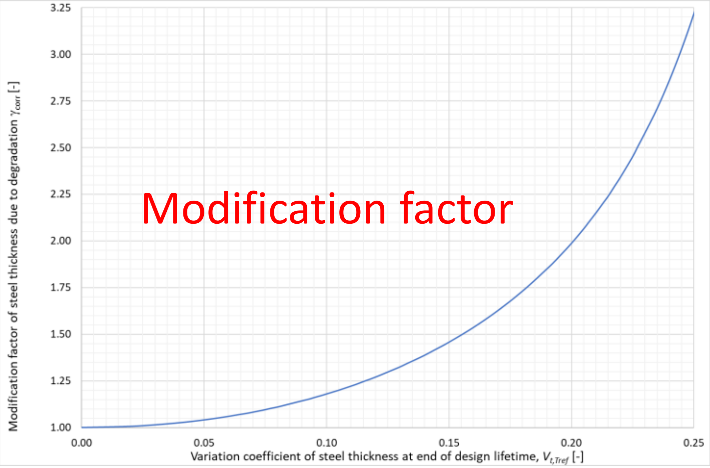
Example: the design of the degrading sheet pile structure is sensitive to deviations in a geometrical parameter

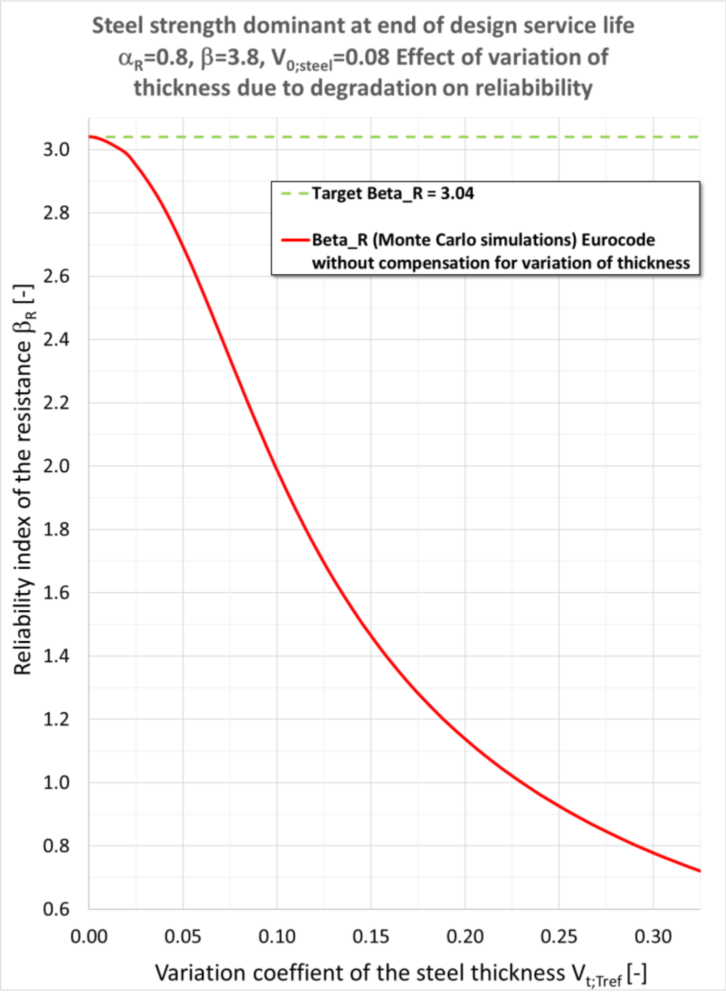
Design values of geometrical data EN1990 art 8.3.7

- High variation coefficient $V_{\bar{c}_{T\,ref}}$ of horizontal average thickness reduction $\bar{c}_{T\,ref}$
- Regional variation $V_{\bar{c}_{T\,ref}}$ = ~0.65 (Amsterdam-Rijnkanaal and Twentekanalen)
- The variation of horizontal average thickness at T_{ref} depends on t_0

$$V_{\bar{t}_{T_{ref}}} = \frac{V_{\bar{c}_{T_{ref}}} \cdot \bar{c}_{m;T_{ref}}}{t_0 - \bar{c}_{m;T_{ref}}}$$

- In EN 1993-5 only nominal degradation presented
- Guideline needed for sheet piles
  - Compulsory inspection at T_{ref} – 10 years
  - Modification factor





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Probabilistic reliability analysis
Standard option in software

- EN1997 ~100% semi-probabilistic approach ?

Deltares

Coffee break

Vervolg programma

11.30 Eurocode 7 – Part 2 – Geotechnical Investigation and testing – Derived values for geotechnical parameters
12.00 EC7-Part 2 from a Dutch perspective
12.30 Lunch

Eurocode 7 – Part 2 – Geotechnical Investigation and testing - Derived values for geotechnical parameters

David Norbury

KIVI
Engineering Society

NEN

Tomorrow’s geotechnical toolbox:
EN 1997-2:202x
Ground properties

La boîte à outils géotechnique de demain :
EN1997-2:202x
Propriétés des terrains

INTRODUCTION TO OCTOBER 2019 DRAFT

Presentation

Introduction to October 2019 draft for comment

3	Definitions	ALL - largely populated
4	Ground Model	David Norbury
5	Sequence of ground investigations	David Norbury
6	Ground investigations: type and extent	David Norbury
7	Physical and chemical properties	Philippe Reiffsteck
8	Strength properties	Marcos Arroyo
9	Stiffness and consolidation properties	Philippe Reiffsteck
10	Cyclic response and seismic parameters	Sebastiano Foti
11	Groundwater and hydraulic conductivity	Håkan Garin
12	Thermal properties	Håkan Garin
13	Reporting	David Norbury

Structural changes since 2007

- Change of title to reflect contents – Ground Properties
- Inclusion of Ground Model
- 90^0 turn around
    - lists of tests giving parameters have become
    - list of parameters and tests that can be used to measure them
- Inclusion of:
    - Investigation and testing of rock
    - Use of geophysics

4 Ground Model

- A ground model shall be formed of the conditions at, under and around the site
- Progressively developed by means of the Desk Study, Site Inspection, Ground Investigations
- The Ground Model should progress from a simple textual description of ground conditions to a two-dimensional graphic description, to a three-dimensional model to a multidisciplinary information model
- The coverage of and detail within the Ground Model shall be consistent with the zone(s) of influence of the structure(s) and the Geotechnical Category

5.1 Planning Ground Investigations:

- shall be planned so that the necessary geotechnical information in all the geotechnical units influencing the anticipated design situations is collected
- should be carried out in phases to identify and progressively reduce uncertainties and increase reliability of the information about the ground
- should investigate the anisotropy of the ground when appropriate
- should identify the soil and rock materials through rockhead
- shall identify ground or groundwater conditions that may change during execution or in the service life of the structure
- identify disposition of any anthropogenic ground with respect to the natural ground
- where contaminated or aggressive ground or groundwater conditions are likely to be encountered which can affect the site investigators appropriate safety measures shall be taken

5 Sequence of ground investigation

5.2 Desk Study

5.3 Site Inspection

5.4 Preliminary ground investigations

5.5 Ground investigation for design and execution

5.6 Groundwater investigations

5.7 Conformity testing

5.8 Geotechnical monitoring

5.9 Personnel for ground investigations

6.1 Ground investigations

- The type, extent and density of the ground investigations shall be based on the anticipated type and design of the structure, the GC and the zone of influence
- Results from the programme should be kept under continual review and the work adjusted as necessary

6.4 Disposition of GI

- The depth and lateral extent of ground investigation shall be sufficient to identify the distribution of geotechnical units and their properties
- The spacing and depth of ground investigation points for specific structures should conform to the requirements given in EN 1997-3 or below.
- Spacing – 6.3.2

6.3.2 Spacing and depth of ground investigation points

(1) <REQ> Ground investigation points should be located no greater than X_{max} apart in plan.

NOTE 1. The value of X_{max} is given in Table 6.1 (NDP) unless the National Annex gives different values.

Table 6.1. (NDP) Maximum spacing and minimum number of ground investigation points

Structures		Geotechnical Category 1		Geotechnical Category 2	
		X_{max}	N_{min}	X_{max}	N_{min}
Low-rise structures		35 m	3 (1?)	30 m	3
High-rise structures	4-10 storeys	30 m	2	25 m	3 (4?)
	11-20 storeys	25 m	3	20 m	3 (5?)
	>20 storeys	20 m	3 (4?)	15 m	3 (6?)
Estate roads, parking areas and pavements		[a]	[a]	40 m	2
Power lines, wind turbines		[a]	[a]	1 per pylon	
Wind turbines		[a]	[a]	2 per turbine	
Linear structures	< 3 m high	[a]	[a]	100 m	-
	≥ 3 m high	[a]	[a]	50 m	-
Silos and tanks		[a]	[a]	15 m	3
Bridges piers		[a]	[a]	1 per pier/base	
Surface excavations		[a]	[a]	25 m	3

[a]Where no spacing or number of points is given this should be assessed on a project-specific basis.

(2) <RCM> The number of ground investigation points should be no less than N_{min}.

NOTE 1. The value of N_{min} is given in Table 6.1 (NDP) unless the National Annex gives different values.

6.4 Disposition of GI

- The depth and lateral extent of ground investigation shall be sufficient to identify the distribution of geotechnical units and their properties
- The spacing and depth of ground investigation points for specific structures should conform to the requirements given in EN 1997-3 or below.
- Spacing – 6.3.2
- Depth investigated shall cover:
  - the zone of influence of the structure;
  - the effects of unloading of the ground;
  - the depth of effect of any dewatering works on groundwater conditions;
  - the presence of any destabilising features in the ground on or around the site

6.6 Derived values

- Derived values should be determined using theory, correlation or empiricism
- Derived values should be established from data obtained in the Desk Study, Site Inspection, field and laboratory, testing ground investigation and the monitoring.
- Derived values should be reviewed together and compared for consistency and critically reviewed where there are differences
- The information given for each correlation should specify the applicable ground types, the database that supports the model, the estimated transformation errors
- It should be verified that the results of field or laboratory tests are at a scale, rate and with boundary conditions appropriate to the design situation(s)

Sections on parameters (7 – 12)

- Definitions given
- Failure/ behaviour models provided and permission given for their use; guidance given on how and when to use, and caveats as to when not appropriate
- Direct measurement by testing – field and laboratory
- Indirect assessment via derivation with guidance on correlations (in annexes)

7 Physical and chemical properties

The physical properties of soils, rock, and water are controlled by the nature and proportions of the particles, water and air present. The properties which might need to be measured are listed with the tests appropriate for their measurement.

7.1 Classification

7.2 Intrinsic physical properties

7.3 State properties

7.4 Density Index

7.5 Degree of compaction

7.6 Ground chemistry

7.7 Groundwater properties

7.1 Classification

Main objective is classification according to

- soil for civil engineering purposes should conform to EN ISO 14688-2.
- materials for earthworks should conform to EN 16907-2.
- rock for civil engineering purposes should conform to EN ISO 14689.
- site for seismic purposes should conform to EN 1998-1.

7.2 Intrinsic physical properties

7.2.1 Particle density

7.2.2 Maximum and minimum void ratio

7.2.3 Particle size analysis

7.2.4 Particle shape

7.2.5 Consistency (Atterberg) limits

7.2.6 Organic content

7.2.7 Soil dispersibility and rock stability

7.3 State properties

7.3.1 Bulk density

7.3.2 Water content

7.3.3 Porosity

7.3.4 In situ stress state

7.3.5 Saturation

7.4 Density Index

7.5 Degree of compaction

Chemistry

7.6 Ground Chemistry

- 7.6.2 Mineralogy
- 7.6.3 Carbonate content
- 7.6.4 Sulfate content
- 7.6.5 pH value
- 7.6.6 Chloride content

7.7 Groundwater chemistry

- 7.7.2 Density
- 7.7.3 Chemistry

8 Strength

- Ground strength shall be described using strength envelopes.
  - Strength envelopes can describe one or various failure modes. A specific failure mode or combination thereof is dominant in most practical applications
- Strength envelopes may be defined in terms of total or effective stress
- The stress range of application should be indicated when a ground strength envelope is specified for design
- It should be indicated if a ground strength envelope applies to a:
  - peak strength condition; or
  - constant volume shearing strength condition; or
  - residual strength shearing condition

8.2 Strength parameters

8.2.1 Mohr-Coulomb envelopes

8.2.2 Hoek-Brown envelopes

8.2.3 Other models

- Alternative strength envelopes to those defined above may also be employed.
  - More elaborate descriptions of the effect of intermediate principal stress on shear strength than those provided by Mohr-Coulomb and Hoek-Brown models are sometimes necessary.
- Strength envelopes shall be considered as calculation models and validated according to 1997-1, 7.1.1.

Evaluation of strength parameters

8.3 Evaluation of strength parameters

Direct and Indirect determinations

- Test standards, parameters obtained and interpretation guidelines given

8.4 Evaluation of rock (and rock mass) strength parameters

Direct and Indirect determinations

8.5 Evaluation of rock mass strength parameters – TBD

8.6 Rock joint strengths

8.7 Interface strengths

9 Stiffness

- Ground stiffness should be described by a stress-strain curve over the expected stress and strain ranges for the anticipated design situation.
- Ground stiffness may be approximated by one or more elastic moduli, each modulus limited to a particular stress or strain range.
  - Relevant moduli include tangent moduli, such as the initial Young's modulus of elasticity (E0), and secant moduli, such as Young's modulus at 50 % of the maximum shear stress (E50).

9.1.2 Determination of stiffness

- Ground stiffness properties should be determined directly (from test results), according to 9.1.2.
- For structures in Geotechnical Categories 1 or 2, ground stiffness properties may be determined indirectly (using appropriate transformation models), according to 9.1.3.
- For structures in Geotechnical Category 1, ground stiffness properties may be estimated using empirical models, according to 9.1.4.

Compression or swelling

9.2 One dimensional compression or swelling

– by direct, indirect or empirical methods

10 Cyclic response and seismic parameters

- Ground investigations of the mechanical response to dynamic loads shall provide the relevant information for:
  - seismic design;
  - design for cyclic loadings;
  - design for vibrations induced by human activities.
- Ground investigations for dynamic loading should provide the relevant information on:
  - stress-strain response to cyclic loads, including small strain elastic moduli;
  - development of excess pore pressures under cyclic loads;
  - cyclic shear strength .
  - post cyclic behaviour in terms of post-cyclic shear strength, consolidation of cyclic-induced pore water pressure and post-cyclic creep
- The pre-failure stress-strain response to cyclic loading may be described in terms of variation of the secant elastic modulus and damping ratio vs cyclic strain.

Evaluation
– direct and indirect methods using field (with geophysics) and laboratory testing

10.2 Measurement of cyclic response

10.3 Secant modulus and damping ratio curves

10.4 Small strain moduli and seismic velocities

10.5 Excess pore pressure

10.6 Cyclic shear strength

10.7 Additional parameters for seismic site response evaluation

11 Groundwater and hydraulic conductivity

- Groundwater investigations shall provide all relevant information on groundwater needed for geotechnical design and construction.
- Groundwater investigations should provide information on:
  - the depth, thickness, extent and conductivity of water-bearing strata in the ground;
  - joint systems in the rock;
  - the permeability or hydraulic conductivity of each geotechnical unit;
  - the piezometric head of aquifers and their variation over time;
  - actual piezometric heads including possible extreme levels and their periods of recurrence;
  - the piezometric pressure distribution;
  - the chemical composition and temperature of groundwater.
- Groundwater measurements shall be planned, conducted and reported in accordance with EN ISO 18674-4.

11.2 Piezometric pressure and piezometric head

- Piezometric pressure should be measured using:
  - open systems (open standpipe and open pipe with an inner hose)
  - closed systems.
- The type of equipment to be used for piezometric measurements shall be selected according to:
  - the type and conductivity of the ground;
  - the purpose of the measurements;
  - the required observation period;
  - the expected groundwater fluctuations;
  - the response time of the equipment and ground.
- Direct (eg standpipe, piezocone) or indirect (eg CPTU)

11.3 Hydraulic conductivity

- The evaluation of hydraulic conductivity should assess:
  - the extent to which the boundary conditions (degree of saturation, the direction of flow, hydraulic gradient, stress conditions, density and layering, side leakage and head loss in filter and tubing) affect the test results;
  - how well these conditions match the situation in the field.
- The following items shall be considered when determining the coefficient of conductivity of a geotechnical unit:
  - the preferred test type for conductivity determination;
  - the orientation of the test;
  - the need for additional classification tests.

Direct by field tests (eg EN 22282) or indirect (eg pumping, DPT)

12 Thermal properties

- Ground investigations of thermal properties shall provide relevant information needed for geothermal design and construction.
- Ground investigations for thermal engineering should provide information on:
  - geological conditions;
  - hydrogeological conditions;
  - geotechnical conditions;
  - hydrochemical conditions;
  - geothermal conditions.

12 Thermal properties (cont)

12.2 Frost susceptibility

12.3 Thermal conductivity

12.4 Heat capacity

12.5 Thermal diffusivity

13 Reporting

- The results of a geotechnical investigation shall be compiled in a Ground Investigation Report
- The Ground Investigation Report shall consist of:
  - the Ground Model;
  - a factual account of all investigation activities carried out
  - a presentation of all appropriate geotechnical information including geological features and relevant data;
  - a geotechnical evaluation of the information, stating the assumptions made in the interpretation of the test results.
- The contents of the Ground Investigation Report should include the headings listed in Annex L
- The GIR should include derived values
- The GIR shall state known gaps in the knowledge and limitations of the results

Annexes (all Informative but with NDPs)

A Suitability of test methods

Proposed structures/ engineering works and ground information needed (H = most necessary)		Ground information needed (EN 1997-2 clauses) (content to act as linkage between Parts 2 and 3) →	4.2.1 Desk study - History of site	4.2.2 Site reconnaissance - Ground features and geomorphology	4.2.2 Sequence of strata / ground profile	4.2.5 Description of soils and rocks	4.2.6 Description of discontinuities in soils and rocks	7 Physical properties	7 Chemical properties	8 Material strengths	9 Material stiffnesses	10 Dynamic properties	11 Groundwater conditions	11 Ground permeability	12 Thermal properties	Presence of voids (natural or man-made)	Properties for materials re-use	Contaminated ground	Chemical properties/ aggressive ground
	EN 1997-3 clauses (content to end up in Part 3 clauses?)	4 Slopes	M	H	H	H	H	H	L	H	L	L	H	H	L	M	L	L	L
		4 Cuttings	M	H	H	]	H	H	L	H	M	L	H	H	L	H	H	L	L
		4 Embankments	H	H	H	H	H	H	L	H	H	H	M	M	L	M	M	L	L
		5 Spread foundations	H	M	H	H	M	H	M	M	H	L	M	L	L	H	L	M	L
		6 Piled foundations	H	H	H	H	M	H	M	M	H	L	M	L	L	H	M	M	M
		7 Retaining structures	H	H	H	H	H	H	L	H	H	M	H	H	L	M	L	M	H
		8 Anchors	M	L	H	H	H	H	H	H	H	M	H	H	L	H	L	M	H
		9 Reinforced soil structures	M	M	H	M	M	H	H	H	H	M	H	H	L	M	M	M	H
		10 Ground improvement	H	H	H	M	M	H	M	H	H	H	H	H	M	H	L	H	H
	EN 1997-2 miscellaneous clauses (existing Annex B)	Linear - roads	M	M	M	M	M	H	L	M	M	M	M	L	L	M	H	L	M
		Linear pipelines	M	M	M	H	M	H	H	M	M	L	H	M	M	M	H	L	M
		Linear - tunnels	L	H	H	H	H	H	M	H	M	L	H	L	L	H	M	M	H
		Underground openings	L	H	H	H	H	H	M	H	M	L	H	L	L	H	M	M	M
		Dams and weirs	H	H	H	H	H	H	L	M	M	M	H	L	L	H	M	L	M
		Construction materials	L	L	M	H	M	H	H	M	M	L	H	L	L	L	H	H	H
		Ground source heat installations	M	M	H	M	H	L	L	L	L	L	H	H	H	M	L	L	L

Ground information needed (EN 1997-2 clauses) (content to act as linkage between Parts 2 and 3)			4.2.1 Desk study - History of site	4.2.2 Site reconnaissance - Ground features and geomorphology	4.2.2 Sequence of strata / ground profile	4.2.5 Description of soils and rocks	4.2.6 Description of discontinuities in soils and rocks	7 Physical properties	7 Chemical properties	8 Material strengths	9 Material stiffnesses	10 Dynamic properties	11 Groundwater conditions	11 Ground permeability	12 Thermal properties	Presence of voids (natural or man-made)	Properties for materials re-use	Contaminated ground	Chemical properties/ aggressive ground
Appropriate methods of investigations (H = most and most relevant information)	EN 1997-2 Clauses (content to end up in Part 2 clauses)	4.2.1 Desk study		H	H	H	H	M	M	H	H	H	H	H	H	H	H	H	H
		4.2.2 Site reconnaissance	H		H	H	H	L	L	M	M	L	H	M	M	H	M	H	M
		4.2.2 Surface mapping (geomorphological, geological)	H		H	H	H	L	L	L	L	L	H	M	L	H	M	H	L
		4.2.2 Ground mapping - geophysical survey	M	H	H	N/A	N/A	L	L	M	M	H	H	M	L	H	N/A	H	L
		6 Boreholes without samples	M	H	H	N/A	N/A	M	L	H	H	H	H	H	M	H	N/A	L	N/A
		6 Trial pits	H	H	H	H	H	H	M	H	H	M	H	H	M	H	M	M	L
		6.1 Boreholes with sampling	H	H	H	H	H	H	H	H	M	M	L	M	M	H	M	H	L
		6.2 Probes (DP, CPT)	N/A	N/A	H	N/A	N/A	H	L	H	H	M	M	M	L	H	L	L	N/A
		6.2 Field testing	N/A	N/A	N	N/A	N/A	H	M	H	H	H	H	H	H	H	M	H	M
		6.2 Geophysics	L	L	H	N/A	N/A	H	L	H	H	H	M	M	M	H	L	M	L
		6.2 Laboratory testing	N/A	N/A	N/A	M	M	H	H	H	H	H	N	M	H	N/A	H	L	H
		6.3 Instrumentation and monitoring	N/A	L	N/A	N/A	N/A	L	L	M	M	M	H	H	M	N/A	N/A	N/A	L

Annexes

All Informative providing “additional guidance”

National Choice as whether to adopt (in NA)

A Suitability of test methods

B Desk study and site inspection

C Information to be obtained from ground investigation

D Methods for evaluating strength properties

E Methods for evaluating stiffness and consolidation properties

F Indirect methods for evaluating cyclic and seismic parameters

G Ground Investigation report

New coverage

- Rock investigation
- Geophysics

Rock investigation

- As with 1997-1, text considers that ‘ground’ includes ‘soil and rock’ wherever possible
- Current complete lack of EN test standards
  - Being addressed in TC 182 / 396 but will not be available for this draft
  - Relying on ISRM suggested methods or national standards

Geophysics

- Included as a method of investigation and measurement wherever possible.
  - Hampered by lack of standard documents elsewhere despite some early progress in TC ISO 182.

References to geophysics include:

- 5.3 use in site inspection
- 6.1 use in field investigation
- 6.2 classification of ground using
- <u>6.4.2</u> Geophysical testing may be used to identify:
  - Stratigraphy
  - Cavities
  - Buried objects
  - Relevant ground properties
  - Weathering
  - Groundwater conditions
- 9.1, 10.4 input to small strain moduli
- 10.4.2 determination of elastic velocities
- 10.7.1 depth to seismic bedrock
- 13.1.3 measurements in reporting

PT programme going forward

• ENQUIRY to NSBs	underway
• PT Response to Enquiry	February – April
• PT deliver final document	end April (2020)

EC7-Part 2 from a Dutch perspective

Bas Vos

KIVI Engineering Society

NEN

Boskalis

EUROCODE 7 PART 2: NOTED CHANGES

BAS VOS

DELFT

DEC 4TH 2019

Boskalis

CHANGES IN EC7: PART 2

- EXISTING EC7 PART 2 AND 3: FOCUS ON TEST DESCRIPTIONS TO FIND PROPERTIES WITH LAB TESTING (PART 2) AND FIELD TESTING (PART 3)
- NEW EC7 PART 2 DIFFERENT FOCUS: WHAT SOIL PROPERTY IS NEEDED AND WHAT SAMPLE + TEST REQUIRED TO DETERMINE THIS?
- TEST DETAILS NOT IN EC, BUT RELEVANT EN-ISO (+BS, ASTM) STANDARDS GIVEN
- CORRELATIONS AND FORMULAS IN APPENDICES PART 2 (WAS PART 3)
- QUANTIFY UNCERTAINTY IN TESTS AND CORRELATIONS: REPORT THIS

Boskalis

PARAMETER VS REQUIRED TESTING

Table 7.9 — Laboratory and in situ tests for determination of stress properties

Property	Test	Test standard	MQC	Comments on suitability and interpretation
In situ stress state: horizontal effective stress p'_0, K_0	Flat jack	ISRM suggested method	-	measured response of the rock mass in a stress-disturbed zone (e.g. the wall of a tunnel)
	Self-boring pressuremeter	EN ISO 22476-06	-	
	Pre-bored pressuremeter	EN ISO 22476-05	-	Pre-bored expansion test Specific procedure is used
	Full displacement pressuremeter	EN ISO 22476-08	-	Insertion by full displacement Specific procedure is used
	Marchetti dilatometer	EN ISO 22476-11	-	Insertion by full displacement Choice of correlation depending of soil type
	Total pressure cells	EN ISO 18674-05	-	Insertion by full displacement
Earth pressure coefficient at rest K_0	Triaxial	EN 17892-09	1	Specific procedure shall be used
In situ stress state: minimum/maximum horizontal stresses and orientation/components of the stress tensor	Hydraulic fracturing /hydraulic tests on pre-existing fractures	ISRM suggested methods	-	Vertical axis often considered as one principal direction and vertical stress magnitude equals weigth of the overdurden
In situ stress state: six components of the stress tensor	Overcoring in a borehole	ISRM suggested methods	-	Elastic parameters required
Initial pore pressure	Piezometers	EN ISO 18674-04	-	
Pre-consolidation state: σ'_p, OCR	Incremental loading oedometer test	EN 17892-05	1	Specific apparatus and procedure shall be used
	Constant rate of stress oedometer test	ASTM D4186	1	

MINIMUM NO OF TESTS

6.3.2 Spacing and depth of ground investigation points

(1) <REQ> Ground investigation points should be located no greater than X_{max} apart in plan.

NOTE 1. The value of X_{max} is given in Table 6.1 (NDP) unless the National Annex gives different values.

Table 6.1. (NDP) Maximum spacing and minimum number of ground investigation points

Structures		**Geotechnical Category 1**		**Geotechnical Category 2**	
		X_{max}	N_{min}	X_{max}	N_{min}
Low-rise structures		35 m	3 (1?)	30 m	3
High-rise structures	4-10 storeys	30 m	2	25 m	3 (4?)
	11-20 storeys	25 m	3	20 m	3 (5?)
	>20 storeys	20 m	3 (4?)	15 m	3 (6?)
Estate roads, parking areas and pavements		[a]	[a]	40 m	2
Power lines, wind turbines		[a]	[a]	1 per pylon	
Wind turbines		[a]	[a]	2 per turbine	
Linear structures	< 3 m high	[a]	[a]	100 m	-
	≥ 3 m high	[a]	[a]	50 m	-
Silos and tanks		[a]	[a]	15 m	3
Bridges piers		[a]	[a]	1 per pier/base	
Surface excavations		[a]	[a]	25 m	3
[a]Where no spacing or number of points is given this should be assessed on a project-specific basis.					

(2) <RCM> The number of ground investigation points should be no less than N_{min}.

NOTE 1. The value of N_{min} is given in Table 6.1 (NDP) unless the National Annex gives different values.

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STRENGTH PROPERTIES

- CHAPTER 8: STRENGTH PROPERTIES

- PARAMETERS / MODELS DEFINED:

  -MOHR COULOMB
  -HOEK-BROWN
  -ROCK JOINT STRENGTH
  -”OTHER MODELS”

- BIT MEAGRE ON SOIL MODEL SIDE?

- WHAT ABOUT SHANSHEP MODEL OR SOFT SOIL/HARDENING SOIL MODELS?

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GROUND INVESTIGATION REPORT

- 13.1.1 GENERAL

(1) <REQ> *The results of a ground investigation shall be compiled in a ground investigation report*

(2) <REQ> The Ground Investigation Report shall consist of: (….)

- a geotechnical evaluation of the information, stating the assumptions made in the derivation of values from the test results.

- 13.1.4 EVALUATION OF GEOTECHNICAL INFORMATION

(1) <REQ> *The geotechnical information shall be evaluated and reviewed, including: (…)*
- for each geotechnical unit, the geotechnical properties according to 7 to 12;
- the derived values of geotechnical parameters;
- apparently anomalous or outlier results for a parameter;
- any limitations or gaps in the data;
- the uncertainties in the data.

Lunch (atrium / ground floor)

Vervolg programma

13.30 Eurocode 7 – Part 3 – Slopes, Raft and Pile foundations, Ground Improvement

14.00 EC7-Part 3 from a Dutch perspective

14.30 Pauze

KIVI Engineering Society

NEN

Eurocode 7 – Part 3 – Slopes, Raft and Pile foundations, Ground Improvement

Christian Moormann

KIVI Engineering Society

NEN

Eurocode 7 – Part 3
Slopes, spread and pile foundations, ground improvement

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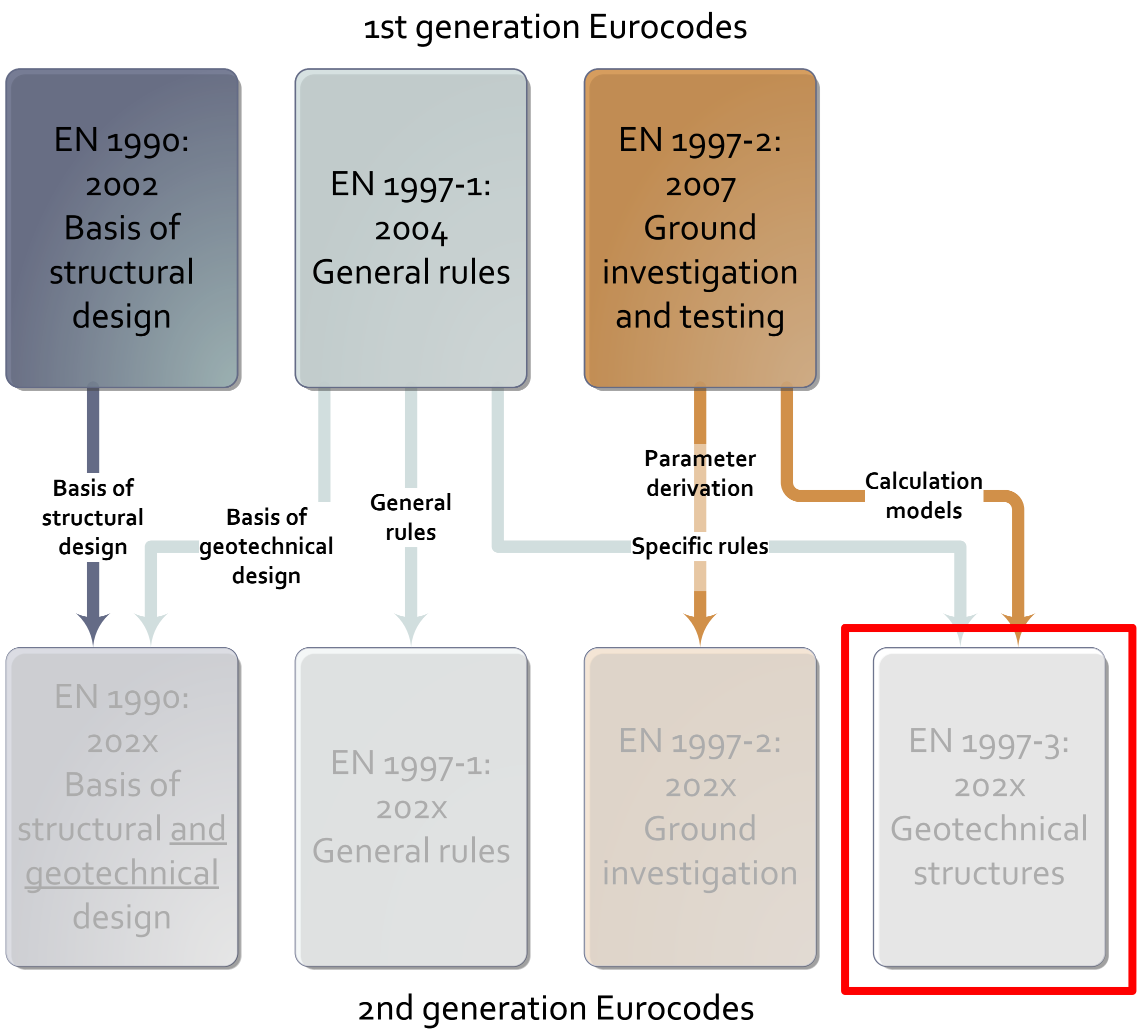
EN 1997-3:202x: Input from EN 1997-1:2004 and EN 1997-2:2007



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EN 1997-3: Geotechnical Structures - Contents

prEN 1997-3:202x	prEN 1997-1:2004
1. Scope	
2. Normative references	
3. Terms, definitions, and symbols	
4. Slopes, cuttings, and embankments	← Sections 11+12
5. Spread foundations	← Section 6
6. Piled foundations	← Section 7
7. Retaining structures	← Section 9
8. Anchors	← Section 8
9. Reinforced ground	← new
10. Ground improvement	← new (Section 5.5)

- Annexes A-G (corresponding to Clauses 4-10)

EN 1997-3: Geotechnical Structures - Contents

prEN 1997-3:202x	**prEN 1997-1:2004**
1. Scope	
2. Normative references	
3. Terms, definitions, and symbols	
4. Slopes, cuttings, and embankments	← Sections 11+12
5. Spread foundations	← Section 6
6. Piled foundations	← Section 7
7. Retaining structures	← Section 9
8. Anchors	← Section 8
9. Reinforced ground	← new
10. Ground improvement	← new (Section 5.5)

- Annexes A-G (corresponding to Clauses 4-10)

EN 1997-3: Geotechnical Structures - Contents

Clauses 4-10 each follow a common sub-structure:

1. Scope
2. Basis of design
3. Materials ← new
4. Groundwater ← new
5. Geotechnical analysis
6. Ultimate limit states
7. Serviceability limit states
8. Execution ← new
9. Testing ← new
10. Reporting ← new

EN 1997-3: New sub-clauses on materials, groundwater, execution, testing, and reporting

x.3 Materials

- primary source of information about ground properties: Part 2 *Ground investigation*
- Part 3 covers materials outside scope of other Eurocodes (e.g. geosynthetics)

x.4 Groundwater

- primary source of information: Part 1 *General rules*
- Part 3 adds detailed recommendations for specific geotechnical structures

x.8 Execution

- primary source of information: geotechnical execution standards (TC 288)
- Part 3 caters for ‘missing’ standards, e.g. for slopes, spread foundations, gravity walls

x.9 Testing

- important in Clauses 6 *Pile foundations* and 8 *Anchors*
- defers to external testing standards for test procedures

x.10 Reporting

- primary source of information: Part 1 *General rules*
- Part 3 adds detailed recommendations for specific geotechnical structures

EN 1997-3: Sub-clauses x.5 on Geotechnical Analysis

- ´pure geotechnical´ sub-clauses related to basis of analysis
  (→ i.e. no mention of verification, partial factors etc.)
- commonly-used formulae moved from old Annexes into sub-clauses x.5
- Examples:

Clause 5 *Spread foundations* gives for bearing capacity:

$$R_N = A' \left(c' N_c\, b_c\, d_c\, g_c\, i_c\, s_c + q' N_q\, b_q\, d_q\, g_q\, i_q\, s_q + 0.5\, \gamma' N_\gamma b_\gamma\, d_\gamma\, g_\gamma\, i_\gamma\, s_\gamma\right)$$

… where N_x, b_x, etc. are given in Annex B (and are subject to national determination)

Clause 7 *Retaining structures* gives for active earth pressure:

$$p'_a = K_{a\gamma}(\overline{\gamma_a} \times z_a - u) - K_{ac}c' + K_{aq}q$$

… where $K_{a\gamma}$, K_{ac}, K_{aq} are given in Annex D (and are subject to national determination)

Members of PT 4

Leader	**Christian**	**Moormann**	**D**	**Coordination, reviewing, reporting, harmonization, interdependencies, …**	
Member	**Gary**	**Axelsson**	**S**	**Clause 4:**	**Slopes, cuttings and embankments**
Member	**Trevor**	**Orr**	**IRL**	**Clause 5:**	**Spread Foundations**
Member	**Chris**	**Raison**	**GB**	**Clause 6:**	**Piled Foundations**
Member	**Bob**	**Essler**	**GB**	**Clause 10:**	**Ground Improvement**
(Ex-officio)	**Bond**	**Andrew**	**GB**	Past Chairman EC 7 and Project Director	
(Ex-officio)	**Adriaan**	**van Seeters**	**NL**	Chairman EC 7	

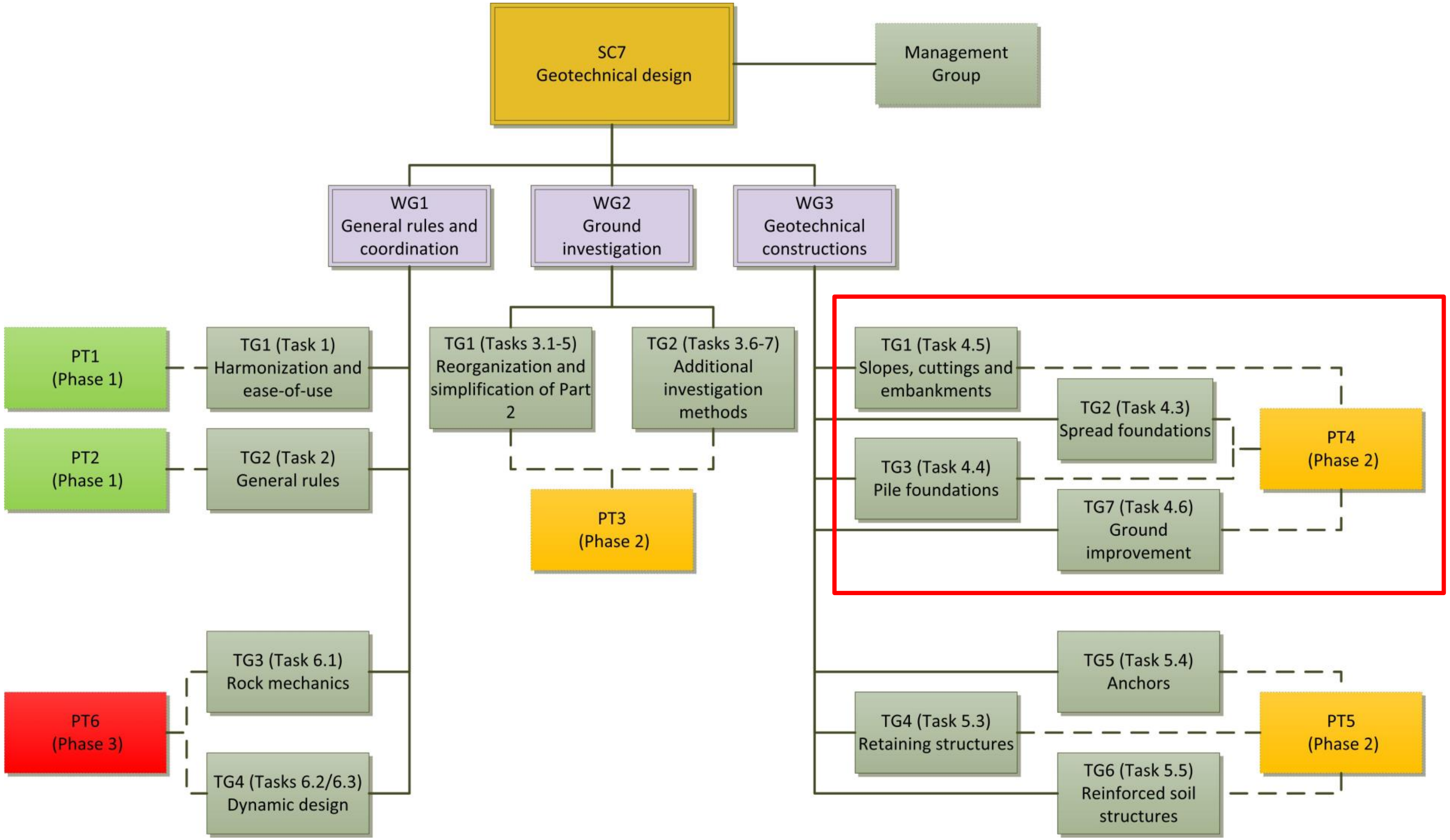
Mandate for PT4

- ➢ **To enhance ease of use**
  - identify common technical r
  - focus on basic rules relevar
  - consider only rules which a
  - simplification and improvem
  - include calculation models
- ➢ **To harmonize practice acros**
  → reduction in number of N
  - Compilation of Design Approaches nationally used
  - Compilation of NDPs nationally used, example: pile design

Piles — Rev 2 — 01.11.2015

	Country	NSB	EN 1997-1	Ultimate Load Test	Proof Load Test	Table A.6 Resistance factors - driven γ_b	γ_s	γ_t	γ_{st}	Table A.7 Resistance factors - bored γ_b	γ_s	γ_t	γ_{st}	Table A.8 Resistance factors - CF γ_b	γ_s	γ_t
			Factor Set R1			1,0	1,0	1,0	1,25	1,25	1,0	1,15	1,25	1,1	1,0	1,1
			R2			1,1	1,1	1,1	1,15	1,1	1,1	1,1	1,15	1,1	1,1	1,1
			R3			1,0	1,0	1,0	1,0	1,0	1,0	1,0	1,0	1,0	1,0	1,0
			R4			1,3	1,3	1,3	1,6	1,6	1,3	1,5	1,6	1,45	1,3	1,4
GBR	UK Chris Raison Confirmed 09 Jun 15	BSI	DA1-1(R1)	N		1,0	1,0	1,0	1,0	1,0	1,0	1,0	1,0	1,0	1,0	1,0
				Y		1,0	1,0	1,0	1,0	1,0	1,0	1,0	1,0	1,0	1,0	1,0
			DA1-2(R4)	N	N	1,6	1,5	1,6	2,0	2,0	1,6	2,0	2,2	2,0	1,6	2,0
				N	Y	1,4	1,3	1,4	1,7	1,7	1,4	1,7	1,7	1,7	1,4	1,7
				Y	N	1,6	1,5	1,6	2,0	2,0	1,6	2,0	2,2	2,0	1,6	2,0
				Y	Y	1,4	1,3	1,4	1,7	1,7	1,4	1,7	1,7	1,7	1,4	1,7
DEU	Germany Christian Moormann E-mail 09 Jun 15	DIN	DA2(R2)			1,1	1,1	1,1	1,15	1,1	1,1	1,1	1,15	1,1	1,1	1,1
			Experience			1,4	1,4	1,4	1,5	1,4	1,4	1,4	1,5	1,4	1,4	1,4
FRA	France Sebastien Burlon E-mail 08 Jun 15	AFNOR	ELU fond			1,1	1,1	1,1	1,15	1,1	1,1	1,1	1,15	1,1	1,1	1,1
			ELU acc			1,0	1,0	1,0	1,05	1,0	1,0	1,0	1,05	1,0	1,0	1,0
POL	Poland Boleslaw Klosinski E-mail 08 Jun 15	PKN	DA2(R2)			1,1	1,1	1,1	1,15	1,1	1,1	1,1	1,15	1,1	1,1	1,1
BEL	Belgium Monika De Vos E-mail 08 Jun 15	NBN	DA1-1(R1)			axially loaded piles : only DA1-1 (revision not yet published) ; γst = γs (different installation factor for tension)										
			normal QC			1,0	1,0	1,0	1,0	1,2	1,0	1,1	1,0	1,1	1,0	1,05
			higher degree of QC			1,0	1,0	1,0	1,0	1,0	1,0	1,0	1,0	1,0	1,0	1,0
			DA1-2(R4)													
			normal QC			1,35	1,35	1,35	1,35	1,65	1,35	1,50	1,35	1,50	1,35	1,43
			higher degree of QC			1,35	1,35	1,35	1,35	1,35	1,35	1,35	1,35	1,35	1,35	1,35
														Table A.8bis Resistance factors - Screw		
			DA1-1(R1)													
			normal QC											1,07	1,00	1,00
			higher degree of QC											1,00	1,00	1,00
			DA1-2(R4)													
			normal QC											1,45	1,35	1,40
			higher degree of QC											1,35	1,35	1,35
CYP	Cyprus Panicos Papadopoulos E-mail 08 Jun 15	CYS	DA2(R2)			1,1	1,1	1,1	1,15	1,1	1,1	1,1	1,15	1,1	1,1	1,1
DNK	Denmark Ole Møller E-mail 12 Jun 15	DS	LC1 (6.10a), GEO/STR			1,3	1,3	1,3	1,3	1,3	1,3	1,3	1,3	1,3	1,3	1,3
			LC2 (6.10b), GEO/STR			1,3	1,3	1,3	1,3	1,3	1,3	1,3	1,3	1,3	1,3	1,3
			LC3 (6.10a), GEO/STR			1,3 K_{FI}	1,3 K_{FI}	1,3 K_{FI}	1,3 K_{FI}	1,3 K_{FI}	1,3 K_{FI}	1,3 K_{FI}	1,3 K_{FI}	1,3 K_{FI}	1,3 K_{FI}	1,3 K_{FI}
			LC4 (6.10b), GEO/STR			1,3 K_{FI}	1,3 K_{FI}	1,3 K_{FI}	1,3 K_{FI}	1,3 K_{FI}	1,3 K_{FI}	1,3 K_{FI}	1,3 K_{FI}	1,3 K_{FI}	1,3 K_{FI}	1,3 K_{FI}
			LC5 (6.10a), STR *			1,0*	1,0*	1,0*	1,0*	1,0*	1,0*	1,0*	1,0*	1,0*	1,0*	1,0*
						* in LC5 (STR) the partial coefficients on the structural materials are increased (multiplied) by a factor γ_0 = 1,2 K_{FI} Thus the geotechnical resistance is not relevant (or decisive) in LC5										
SWE	Sweden Gary Axelsson E-mail 08 Jun 15	SIS	DA2(R2)			1,2	1,2	1,2	1,3	1,3	1,3	1,3	1,4	1,3	1,3	1,3
ITA	Italy Alessandro Mandolini E-mail 08 Jun 15	UNI	R1			1,0	1,0	1,0	1,0	1,0	1,0	1,0	1,0	1,0	1,0	1,0
			R2			1,45	1,45	1,45	1,6	1,7	1,45	1,6	1,6	1,6	1,45	1,55
			R3			1,15	1,15	1,15	1,25	1,35	1,15	1,3	1,25	1,3	1,15	1,25
FIN	Finland buildings Finland bridges Veli-Matti Uotinen E-mail 04 Jun 15	SFS	DA2(R2)			1,2	1,2	1,2	1.35/1.5	1,2	1,2	1,2	1.35/1.5	1,2	1,2	1,2
ESP	Spain José Estaire E-mail 12 Jun 15	AENOR	R2			1,25	1,05	1,15	1,05	1,35	1,10	1,25	1,10	1,45	1,15	1,30
MKD	Macedonia Josif Josifovski E-mail 15 Jun 15	ISRM	Any			no change				no change				no change		
NLD	Netherlands Mandy Korff E-mail 26 Jun 15	NEN	R3 - no inv. / use of inst records			1,4	1,4	1,4								
			R3 - no investigaation			1,8	1,8	1,8		1,8	1,8	1,8		1,8	1,8	1,8
			R3 - based on tests/CPT			1,2	1,2	1,2	1,35	1,2	1,2	1,2	1,35	1,2	1,2	1,2
			R3- based on pile load test on specific piles			1,15	1,15	1,15	1,25	1,15	1,15	1,15	1,25	1,15	1,15	1,15
AUT	Austria Schremser Roman E-mail 02 Jul 15	ON	DA2(R2)			1,1	1,1	1,1	1,15	1,1	1,1	1,1	1,15	1,1	1,1	1,1
NOR	Norway	SN	DA2(R2)			1,1	1,1	1,1	1,2	1,3	1,3	1,3	1,4	1,2	1,2	1,2
EST	Estonia	EVS	DA2(R2)			1,1	1,1	1,1	1,2	1,3	1,3	1,3	1,35	1,25	1,15	1,2

PT4 shadowed by 4 Task Groups of WG3



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Main modifications with regard to current EN 1997-1

- **General:**
  - Sections 11 and 12 in EN 1997-1 were merged
  - Similar clauses were deleted or rephrased
  - Lots of textbook like clauses were deleted, e.g.:
    - *“When determining the weight of the embankment from the weight density of fill (see 3.3.3), care should be taken to include fill particles of size > 20 mm to 60 mm in the density tests. They are often not included but can have a considerable effect on the weight density.”*
  - Basic requirements are found in EN 1997-1 and EN 1990
  - Execution standard EN 16907 Earthworks

- **4.1 Scope of clause 4:**

(1) <REQ> This clause shall apply to the **design of cuttings, embankments and existing slopes within the zone of influence of construction works and activities**.

(2) <REQ> This clause shall also apply to **overall stability, local stability**, and displacement of nearby structures and infrastructure within the zone of influence.

(3) <REQ> This clause shall also apply to **dams and levees** but excludes the verification of water retention of those structures

Main modifications with regard to current EN 1997-1

▪ 4.2.6 Recommendations for ground investigation

4.2.6 (2) <RCM> The **minimum number of ground profiles and their maximum plan spacing** shall conform to **EN 1997-2**, depending on the Geotechnical Category

4.2.6 (5) <RCM> The **depth of the ground investigation (z_a)**, see Figure 4.1, should be selected considering the following:

- the maximum depth of the excavation/cutting (h), of the embankment unless a stratum of high shear resistance is identified;
- 1,5 times the maximum height (h), of the embankment unless a stratum of high shear resistance is identified;
- the depth of any possible failure surface;
- for embankments, at least down to the bottom of the deepest fine soil layer (or layer of high compressibility) that could undergo consolidation settlement, depending on the depth of influence.

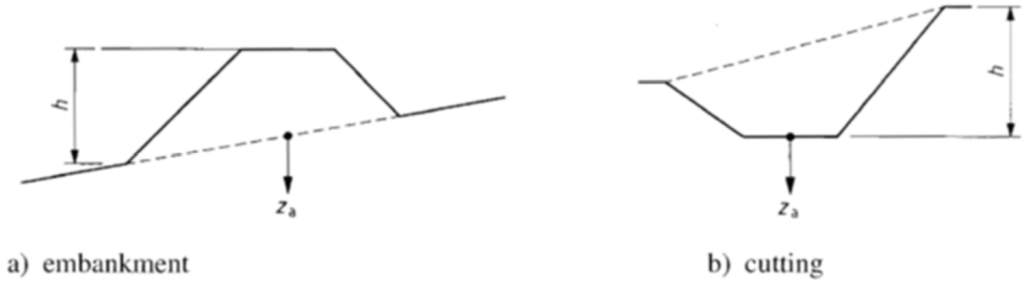

Figure 4.1 – Reference level for measuring the minimum depth of investigation

Main modifications with regard to current EN 1997-1

- **4.2.7 Geotechnical reliability**

  Guidance provided for selecting Geotechnical Complexity Class (GCC) for slopes

  → **Consequence Class and Geotechnical Category acc. to EN 1997-1**

  → **detailed criteria for GCC1**

Table 4.1 (NDP) – Selection of Geotechnical Complexity Class for slopes, cuttings, and embankments founded in or on fill or soil

Geotechnical Complexity Class	Complexity	General features causing uncertainty
GCC 3	Higher	Considerable uncertainty regarding any of the following: • soils with very high sensitivity to disturbance/deformation (St>30); • possible progressive failure; • continuously moving ground of slopes; • potential presence of pre-existing failure surfaces; • difficult[a] deep excavation below groundwater level; • high hydraulic gradient with significant[a] seepage forces and /or significant[a] adverse effects of internal erosion or piping; • exposure to significant[a] erosion or scour that could lead to failure; • significant ongoing settlement that could lead to failure; • significant dynamic, cyclic, or seismic loads that could have adverse effects on the structure.
GCC 2	Normal	GCC2 should be selected if GCC1 and GCC3 are not applicable. Some of the following could apply: • ongoing ground settlement; • significant influence of frost or thawing period; • possible erosion or scour; • artesian groundwater level or pressure; • structures close to cuttings or slopes with limited risk of adverse effects.
GCC 1	Lower	Negligible risk of overall stability and damaging settlements. The following conditions apply for cuttings: • above the groundwater level and; • less than 1,0 m depth in fine-grained soils of very low undrained shear strength (c_u = 10-20 kPa) or; • less than 2,0 m depth in fine-grained soils of low undrained shear strength (c_u =20-40 kPa) or; • less than 3,0 m depth in coarse soil or fine-grained soils of at least medium undrained shear strength (c_u > 40 kPa) and; • slope inclination, vertical to horizontal, less than 1:2; • maximum 2 kPa external load[b] within 1,0 m of the slope crest and all other loads[b] are limited to 15 kPa or equivalent; • close to level ground (< 1:10) within the zone of influence of the cutting/excavation. All of the following conditions apply for embankments: • low embankment height (< 3,0 m) on competent ground; • close to level ground (< 1:10) within the zone of influence of the embankment

[a]the terms ‘difficult’ and ‘significant’ are relative to any comparable experience that exists for the particular geotechnical structure and design situation; [b]representative value

Main modifications with regard to current EN 1997-1

4.3 Materials

Specification of ground properties needed as input for calculation

→ link to EN 1997-2

→ volume friction angle and residual strength considered

4.3 Materials

4.3.1 Ground properties

4.3.1.1 General

(1) <REQ> Ground properties shall be determined according to EN 1997-1, 5.1-5.4, and EN 1997-2.

4.3.1.2 Properties of soil and fill

(1) <PER> In accordance with EN 1997-1, 4.2.2(4), drained or undrained soil or fill parameters (or a combination of both) may be used in the design of slopes, depending on the soil's hydraulic conductivity and the duration of any loading or unloading.

(2) <RCM> Potential reduction in strength caused by weather conditions during or after execution, in particular exposure and saturation of the ground and thawing of frozen ground, should be considered. <proposed for transfer to Part 1>

(3) <RCM> The following soil and fill parameters and field measurements should be considered as input for calculations of both overall and local stability:

- undrained shear strength of fine soils;
- effective shear strength;
- grain-size distribution;
- internal friction angle (peak, constant volume, or residual);
- weight density (dry, saturated, moist);
- groundwater pressure (groundwater level in coarse soil);
- sensitivity of fine soils;
- Atterberg limits of fine soils.

<add reference to EN 1997-2 where relevant under (3) and (4)>

(4) <RCM> The following soil and fill parameters and field measurements should be considered as input for calculations of settlement:

- pre-consolidation pressure in fine-grained soil;
- weight density (dry, saturated, moist);
- groundwater pressure (groundwater level in coarse soil);
- compressibility parameters;
- hydraulic conductivity;
- secondary compression index (creep).

Main modifications with regard to current EN 1997-1

4.6.7 Partial Factors

- **Material factor approach (MFA)** for slopes using DC 3 with M3 according to EN 1997-1
- **Partial factors**
  → for persistent and transient
  → for accidental design situations
  → reduced for constant-volume conditions and for residual slip surface
- allows for reduction of γ_M by $K_{M,tr} \leq 1.0$ for transient design situations (NDP), default 1.0

Table 4.2 (NDP) – Partial factors for the verification of ground resistance of slopes, cuttings, and embankments for fundamental (persistent and transient) design situations

Verification of	Partial factor on	Symbol	Material factor approach (MFA)[1, 2]
Overall stability	Actions and effects-of-actions	γ_F and γ_E	DC3
	Ground properties (other than those given below)	γ_M	M3[3]
	Coefficient of internal friction under constant-volume conditions	$\gamma_{\tan\varphi,cv}$	1,1 K_M
	Coefficient of friction along a residual slip surface	$\gamma_{\tan\varphi,res}$	1,1 K_M
Bearing resistance	see Clause 5		

[1]Values of the partial factors for Design Case 3 (DC3) are given in EN 1990 Annex A.
[2]Values of the partial factors for Set M3 are given in EN 1997-1 Annex A.
[3]Also includes ground properties of diffused ground improvement.

Table 4.3 (NDP) – Partial factors for the verification of ground resistance of slopes and embankments for accidental design situations

Verification of	Partial factor on	Symbol	Material factor approach (MFA)[1]
Overall stability	Actions and effects-of-actions	γ_F and γ_E	Not factored
	Ground properties (other than those given below)	γ_M	M3[2]
	Coefficient of internal friction under constant-volume conditions	$\gamma_{\tan\varphi,c}$	max(1,0K_M; 1.0)
	Coefficient of friction along a residual slip surface	$\gamma_{\tan\varphi,res}$	max(1,0K_M; 1.0)
Bearing resistance	see Clause 5		

[1]Values of the partial factors for Set M3 are given in EN 1997-1 Annex A.
[2]Also includes ground properties of diffused ground improvement.

Aspects being currently discussed with WG3/TG1

- Annex A (informative) with overview on widely accepted calculation methods for stability of soil slopes

  → *still too generic?*
  → *more guidance?*
  → *or to be deleted?*

Table A.1 – Calculation methods for analysing the stability of soil slopes

No[c]	Method	Type of method[a,b]	Special design conditions/limitations	Comments, assumptions
1	Bishop (simplified and rigorous)	Slices, circular arc	Not recommended with external horizontal loads	Simplified ignores interslice shear forces with interslice forces are horizontal
2	Generalized limit equilibrium	Slices, any shape of surface	Applicable with all slope geometries and soil profiles	
3	Janbu generalized (modified)	Slices, circular arc, non-circular, polyline		Location of interslice normal force is assumed by a line of thrust
4	Morgenstern-Price			Direction of interslice forces by variable user function
5	Spencer			Constant interslice forces function
6	Sarma	Slices, polyline	Seismic loading, critical acceleration. Static conditions: horizontal load set to zero	Can include non-vertical slices and multi-wedge failure mechanisms
7	Block/wedge method	Multiple body, polyline	Pre-defined planar failure surface. Divided into three segments	Earth-pressure can be used as driving and resisting force. No moment equilibrium
8	Multiple wedge method	Multiple body, plane surfaces, blocks, wedges		No moment equilibrium.
9	Infinite slope	Single body, plane surface	Long shallow slopes	
10	Culmann, finite slope		Steep slopes, drained analysis	
11	Logarithmic spiral	Single body; logarithmic spiral	Homogeneous soil, drained analysis	No moment equlibrium

[a]Where ground or embankment material is relatively homogeneous and isotropic, circular failure surfaces can normally be assumed, except when high external loads are present
[b]polyline includes interconnected plane surfaces
[c]references to the methods are listed in the Bibliography

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Concept / Strategy

- to update and revise the existing Section 6 of EN 1997-1 → evolution
- to clarify the text and make it more understandable
  to remove textbook material
- to adjust to new structure of EN 1997-3
- to consider basic requirements found in EN 1997-1:2018 and EN 1990:2018

- Scope of Clause 5

5.1 Scope

(1) <REQ> This clause shall apply to the design of spread foundations, including pad, strip, and raft foundations.

(2) <REQ> This clause shall also apply to the design of working platforms and unreinforced load transfer platforms.

(3) <PER> This clause may be applied to the design of deep foundations, including caissons, that behave as spread foundations.

Main modifications with regard to current EN 1997-1

5.2.2 Geomterical data

´Zone of influence´ for spread foundations

(5) <RCM> In homogeneous ground or where the ground properties increase with depth, the minimum depth of the zone of influence should be determined as the larger of:

- the depth at which the increase in vertical effective stress due to the applied load is 20% of the applied load; and
- the depth at which the increase in vertical effective stress due to the applied load is 20% of the initial in situ vertical effective stress, $\sigma'_{v,init}$ as shown in Figure 5.1, i.e. $\Delta\sigma'_v = 0.2 \cdot \sigma'_{v,init}$.

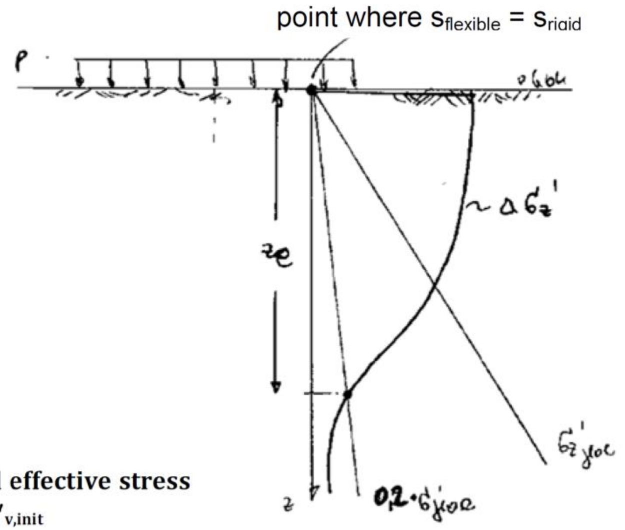


Figure 5.1 – Depth of influence zone given by depth at which increase in vertical effective stress due to the applied load, $\Delta\sigma'_v$ is 20% of the initial in situ vertical effective stress, $\sigma'_{v,init}$

Main modifications with regard to current EN 1997-1

5.2.6.2 Recommendations on ground investigation

from existing EN 1997-2, slightly modified

→ Minimum depth of investigation

5.2.6.2 Minimum depth of investigation

(1) <RCM> The depth of the ground investigation should be sufficient to determine the ground conditions within the zone of influence of the structure according to 5.2.2.1(5) and (6).

(2) <RCM> For low-rise buildings in Geotechnical Category 1, the minimum depth of investigation below the planned base of a spread foundation z_a should be 2 m.

(3) <RCM> For low-rise buildings in Geotechnical Category 2, the minimum depth of investigation below the planned base of a spread foundation z_a should conform to Formula (5.1):

$$\boxed{z_a \geq \max(3b_F; 3m)} \tag{5.1}$$

where:

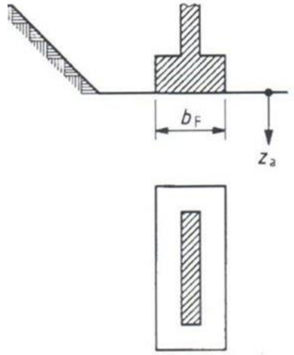
b_F is the smaller side length of the foundation (on plan) – see Figure 5.2a.

(4) <RCM> For high-rise structures and industrial structures, the minimum depth of investigation below the planned base of a spread foundation z_a should conform to Formula (5.2):

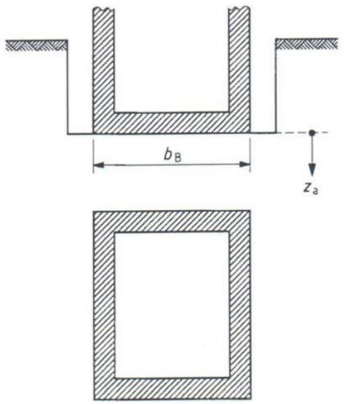
$$\boxed{z_a \geq \max(3b_F; 6m)} \tag{5.2}$$

where:

b_F is the is the smaller side length of the foundation (on plan) – see Figure 5.2a.



a) foundation



b) structure

Main modifications with regard to current EN 1997-1

- **5.2.7 Geotechnical reliability**

  Guidance provided for selecting Geotechnical Complexity Class (GCC) for spread foundations

Table 5.1 (NDP) Selection of Geotechnical Complexity Class for spread foundations

Geotechnical Complexity Class	Complexity	Examples of general features causing uncertainty
GCC 3	Higher	Considerable uncertainty regarding any of the following: • ground with weak[a] layers • ground with persistent movement • areas of probably site instability Further examples with high[a] complexity: • structures with high[a] concentrated loads • foundations with relevant non-monotonic loading • foundations for tower structures like chimneys, pylons etc. • extended raft foundations on variable ground • spread foundations with significantly different foundation levels • spread foundations subject to significant dynamic, cyclic, or seismic loading that might affect the structure
GCC 2	Normal	GCC2 should be selected if GCC1 and GCC3 are not applicable. Some of the following could apply: • frost heave • uplift or settlement due to presence or removal of nearby trees • potential erosion
GCC 1	Lower	Negligible risk of the occurrence of an ultimate or serviceability limit state The following conditions apply for spread foundations: • pad and strip foundations in combination with ground conditions which are simple and the properties of which are known from comparable experience • negligible[a] risk of ground movements • no excavation below the groundwater level or such excavation is straightforward[a]
[a]the terms 'weak', 'high', 'negligible', and 'straightforward' are relative to any comparable experience that exists for the particular design situation		

Main modifications with regard to current EN 1997-1

- **5.5 Geotechnical analysis**

Formula provided for design by calculation as part of normative text:

→ **bearing resistance** (depth factors included for bearing resistance)

→ **sliding resistance** → **punching failure**

Example: Drained bearing resistance of a spread foundation using soil parameters:

$$R_{\mathrm{Nu}} = A'\left(c_{\mathrm{u}}N_{\mathrm{cu}}b_{\mathrm{cu}}d_{\mathrm{cu}}g_{\mathrm{cu}}i_{\mathrm{cu}}s_{\mathrm{cu}} + q + 0.5\gamma B' N_{\gamma\mathrm{u}}\right) \quad (5.3)$$

where:

A' is the effective plan area of the foundation;

B' is the effective foundation width shown in Figure 5.3;

N_{cu} is non-dimensional bearing resistance factor for undrained conditions;

$N_{\gamma\mathrm{u}}$ is a non-dimensional bearing resistance factor for the influence of the soil weight density ($N_{\gamma\mathrm{u}}$ is zero for undrained conditions except when the ground surface slopes downwards away from the foundation when it is negative)

c_{u} is the soil's undrained shear strength (assuming that $\varphi_{\mathrm{u}} = 0^{\circ}$)

q is the overburden pressure applied to the ground outside the foundation;

b_{cu}, d_{cu}, g_{cu}, i_{cu}, and s_{cu}, are non-dimensional factors to account for the effects of base inclination, embedment depth, ground surface inclination, load inclination, and foundation shape.

NOTE 1. Equations for bcu, dcu,, gcu, icu, and scu are given in Annex B.4b.

(2) <RCM> The effective plan area of the foundation (A') should be calculated from Formula (5.4):

$$A' = B' \times L' = (B - 2e_{\mathrm{B}})(L - 2e_{\mathrm{L}}) \quad (5.4)$$

where:

B' is the effective foundation width shown in Figure 5.3;

L' is the effective foundation length shown in Figure 5.3;

B is the actual foundation width shown in Figure 5.3;

L is the actual foundation length shown in Figure 5.3;

e_{B} is the eccentricity of the applied load in the direction of B;

e_{L} is the eccentricity of the applied load in the direction of L.

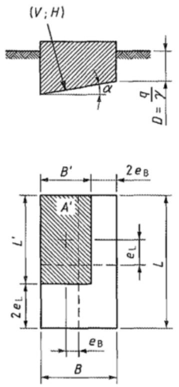


Figure 5.3 – Notation for a spread foundation with an inclined base and eccentric load

Main modifications with regard to current EN 1997-1

- **Annex B.13:**
  New tables added providing presumed bearing resistances for spread foundations in Geotechnical Category 1 with different sizes resting on different types of soil.

Table B.8 – Presumed bearing resistance pressure (in kPa) for strip foundations on clay soils

Embedment depth (m)	Average soil consistency		
	Stiff (I_C = 0,75–1,0)	Very stiff ($I_C \approx$ 1,0–1,25)	Hard (I_C > 1,25)
0,5	90	140	200
1,0	110	180	240
1,5	130	210	270
2,0	150	230	300
$q_{u,rep}$ (kPa)[a]	120 to 300	300 to 700	> 700
[a]Mean representative unconfined compressive strength			

→ **Criteria ´bearing failure´ and ´acceptable settlements´ covered**

Main modifications with regard to current EN 1997-1

Spread foundations subject to loads with large eccentricities:

- 5.6.5 Rotational failure (ULS)

(1) <REQ> The stability against rotational failure of spread foundations subject to loads with large eccentricities shall be verified by checking that the destabilizing design moments about the assumed point of rotation are less than or equal to the stabilizing design moments.

(2) <REQ> The design stabilising and destabilising moments shall be calculated using the DC1 partial factors.

(3) <RCM> The eccentricity of loading on a spread foundation should be limited to the values given in Table 5.2.

Table 5.2 Limits to the load eccentricity e in the case of ULS design

	Strip foundation	**Circular foundation**	**Rectangular foundation**
Fundamental (persistent and transient) situations	$e \leq \left(\frac{7}{15}\right) B$	$e \leq \left(\frac{37}{80}\right) D$	$e \leq \left(1 - 2\frac{e_B}{B}\right)\left(1 - 2\frac{e_L}{L}\right) \geq \frac{1}{15}$

- 5.7.4 Tilting (ULS)

(1) <REQ> For spread foundations subject to eccentric loading, it shall be verified that differential settlement of the foundation will not result in the occurrence of a serviceability limit state due to unacceptable tilting of the supported structure.

(2) <RCM> To avoid the occurrence of a serviceability limit state, the eccentricity of the loading on a spread foundation should be limited to the values given in Table 5.5.

Table 5.5 Limits to the load eccentricity in the case of SLS design

Design situation	**Strip foundation**	**Circular foundation**	**Rectangular foundation**
SLS	$e \leq \frac{D}{6}$	$e \leq \frac{D}{8}$	$e \leq \left(1 - 2\frac{e_B}{B}\right)\left(1 - 2\frac{e_L}{L}\right) \geq \frac{2}{3}$

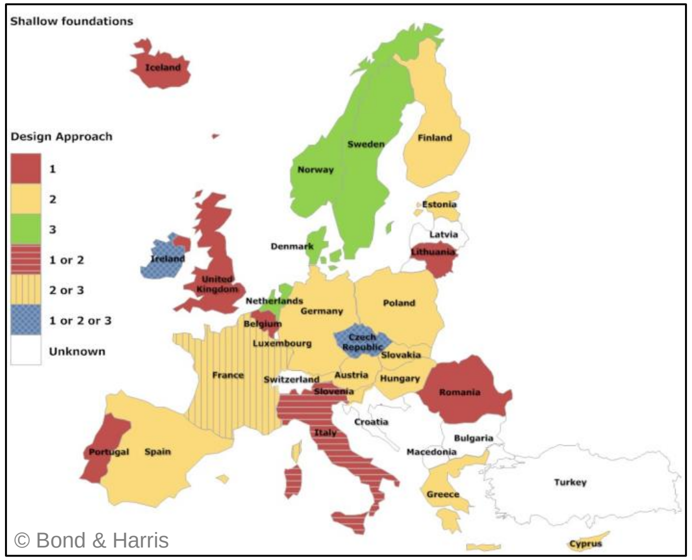
Main modifications with regard to current EN 1997-1

- **Material factor approach (MFA) or Resistance factor approach (RFA) for spread foundations**
- **Partial factors**
  - → **for persistent and transient**
  - → **for accidental design situations**
- allows for reduction of γ_R by $K_{R,tr} \leq 1.0$ for transient design situations (NDP), default 1.0

Table 5.2 (NDP) – Partial factors for the verification of ground resistance of spread foundations for fundamental (persistent and transient) design situations

Verification of	Partial factor on	Symbol	Material factor approach (MFA)			Resistance factor approach (RFA)
			(a)	(b)	(c)	
Overall stability	See Clause 4					
Bearing and sliding resistance	Actions and effects-of-actions	γ_F and γ_E	DC1[1]	DC3[1]	DC1[1]	DC1[1]
	Ground properties	γ_M	M1[2]	M3[2]	M3[2]	Not factored
	Bearing resistance	γ_{Rv}	Not factored			1,4
	Sliding resistance	γ_{Rh}	Not factored			1,1

[1]Values of the partial factors for Design Cases (DCs) 1 and 3 are given in EN 1990 Annex A, Table A.1.8.
[2]Values of the partial factors for Sets M1 and M3 are given in EN 1997-1 Annex A, Table A.1.8.



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Main modifications with

- **Scope and classification of**

Table C.1 – Examples of pile types in different classes

Pile type	Class	Example pile types
Displacement piles	High	Driven cast-in-place concrete piles Solid section precast concrete piles Closed-ended tubular steel piles Closed-ended tubular precast concrete piles Open-ended tubular steel piles when plugged Open-ended tubular precast concrete piles when plugged Timber piles
	Low	Continuous helical displacement piles (also known as displacement auger piles) Cast-in-place concrete screw piles Open-ended tubular steel piles Steel sheet piles Steel H-section piles
Replacement piles	CFA	Bored cast-in-place piles installed using continuous flight auger Micropiles
	Bored	Bored cast-in-place concrete piles with or without temporary casing Caissons excavated by hand or by machine Barrettes Diaphragm walls
Other	High	Steel helical piles

6.1 Scope

(1) <REQ> This Clause shall apply to the de

(2) <RCM> Piles should be classified in acc

NOTE 1. The pile class is used to determin

NOTE 2. Examples of piles in different clas

Table 6.1 – Classification of piles

Pile type	Description	Class
Displacement pile	Pile installed in the ground without excavation of material, causing the ground to be displaced radially as well as vertically	High displacement
		Low displacement
Replacement pile	Pile installed in the ground after the excavation of material	Continuous flight auger
		Bored
Pile not listed above		Unclassified

<Drafting note regarding Table 6.1: PT4 needs guidance whether two classes are needed for replacement piles [as given in the current published EN 1997-1] or whether a single class is sufficient. If two classes are suggested, TG3 is asked for proposals, suitable names and a specification for these two classes.>

Main modifications with regard to current EN 1997-1

- **6.2.7.1 Geotechnical Complexity Class**

Table 6.2 (NDP) – Selection of Geotechnical Complexity Class for piled foundations

Geotechnical Complexity Class	Complexity	General features causing uncertainty
GCC 3	Higher	Considerable uncertainty regarding ground conditions or any of the following apply unless there is comparable experience or evidence of previous successful use: – difficult ground conditions – friction piles in very low strength ground – vertical or horizontal ground movements – site instability – significant cyclic, dynamic or repeated loading
GCC 2	Normal	GCC2 should be selected if GCC1 and GCC3 are not relevant
GCC 1	Lower	All of the following conditions apply: – negligible uncertainty regarding the ground conditions – no ground movements

Main modifications with regard to current EN 1997-1

- **6.3 Materials:**

  more information about pile materials included

6.3.3 Steel reinforcement

(1) <REQ> Reinforcement for concrete piles and grout and mortar micropiles shall conform to EN 10080 and EN 1992-1-1.

(2) <REQ> Hollow steel reinforcement bars used as reinforcing elements shall conform to EN 10210, EN ISO 683-1 and EN ISO 683-2.

6.3.4 Ductile cast iron

(1) <REQ> Ductile cast iron for piles or piled foundations and the values of cast iron parameters shall conform to EN 1561.

6.3.5 Plain and reinforced concrete

(1) <REQ> Concrete for piled foundations shall be specified in accordance with and conform to EN 1992-1-1 and EN 206.

(2) <REQ> Exposure classes for concrete shall conform to EN 206 and concrete cover requirements to EN 1992-1-1.

NOTE 1. For the majority of reinforced concrete piles or piled foundations constructed in natural ground, the exposure class will be classified as XA1, XA2 or XA3. Currently EN 1992-1-1 does not provide guidance for the cover allowance for durability for these exposure classes.

(3) <REQ> In the absence of guidance for durability purposes, the minimum cover required for environmental conditions $c_{min,dur}$ shall be 25 mm for reinforced concrete used for both precast and cast-in-place piles.

(4) <REQ> The allowance for deviation Δc_{dev} shall be 50 mm for concrete cast against the ground and 10 mm for precast piles.

(5) <PER> The value for Δc_{dev} for precast piles may be reduced in accordance with EN 1992-1-1, 6.4.3(3) when fabrication is subject to a quality assurance system with measurement of concrete cover.

6.3.6 Plain and reinforced grout and mortar

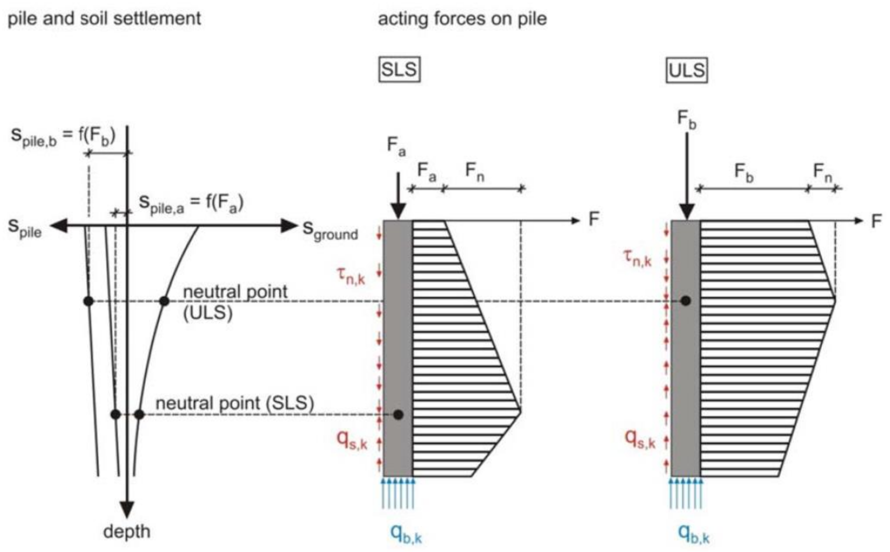
(1) <REQ> Grout and mortar used for small diameter minipiles and micropiles shall be specified in accordance with and conform to EN 1992-1-1, EN 206, EN 445 and EN 447 as appropriate.

(2) <REQ> Exposure classes for grout and mortar and rules for durability and cover shall conform to 6.3.5(2) to (5).

Main modifications with regard to current EN 1997-1

- **6.5.2 Effect of ground displacement:** more guidance provided especially for downdrag (negative skin friction)

C.11 **Calculation model for downdrag due to vertical ground movements**



6.5.2.2 Downdrag (negative shaft friction)

(1) <REQ> The verification of limit states shall take account of downdrag caused by moving ground and shall determine whether the drag settlement results in a serviceability limit state in the overall structure.

(2) <REQ> The adverse effect of the drag force shall be included in the structural design of the pile for both serviceability and ultimate limit states.

(3) <RCM> The effect of the downdrag or negative shaft friction should be modelled by carrying out an interaction analysis to determine the depth of the neutral plane L_{dd} corresponding to the point where the pile settlement s_{pile} equals the ground settlement s_{ground}.

NOTE 1. This also marks the boundary between negative shaft friction above, and positive shaft friction below the neutral plane.

(4) <RCM> The interaction analysis should provide force, displacement and strain profiles for the full depth of the pile to enable the representative drag force D_{rep} acting on the pile shaft above the neutral plane to be determined.

(5) <PER> For simple cases, approximate assumptions may be adopted to identify the level of the neutral plane allowing the ground displacement to be treated as an equivalent drag force.

(6) <PER> If the pile settlement is greater than the settlement of the surrounding ground, the neutral plane may be assumed to be located at the ground surface.

(7) <PER> If the pile settlement is much smaller than the settlement of the surrounding ground, the neutral plane may be assumed to be located at the base of the settling layer.

(8) <RCM> The equivalent drag force D_{rep} should be determined from Formula (6.3):

$$D_{rep} = \pi D \int_0^{L_{dd}} \tau_s \cdot \mathrm{d}z \qquad (6.3)$$

where:

D is the diameter of the pile for circular piles or equivalent diameter for non-circular piles;

τ_s is the unit shaft friction at depth z;

L_{dd} is the depth to the neutral plane.

NOTE 1. Calculation models for downdrag are included in Annex C.

(9) <RCM> The value selected for the unit shaft friction should be based on upper (superior) ground parameters, in order to provide a cautious estimate of the downdrag force.

Main modifications with regard to current EN 1997-1

- **Desing of axially loaded single piles**

6.5.3 Design of axially loaded single piles

6.5.3.1 Design by calculation

(1) <PER> The axial resistance of a single pile may be based on the results of field and laboratory testing or comparable experience.

(2) <REQ> The axial resistance of a single pile designed by calculation shall be determined by one of the following methods:

– using derived ground properties determined for the various geotechnical units based on evaluation of all results of field and laboratory tests (Method A, the Ground Model Method); or

– using derived ground properties or by direct correlations with individual profiles of field or laboratory tests (Method B, the Model Pile Method).

Main modifications with regard to current EN 1997-1

- **Design of pile groups**

6.5.5 Design of pile groups

(1) <REQ> Verification of limit states shall be carried out by numerical, analytical, or empirical calculation methods, based on the observed performance of comparable pile group foundations.

(2) <REQ> Pile group design shall take into account that the resistance and load-displacement behaviour of individual piles in a group might show significant variation compared to the behaviour of single piles.

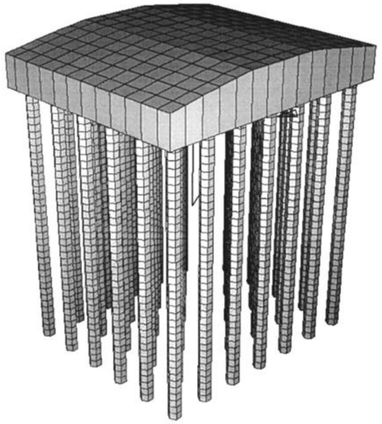
(3) <RCM> Calculation of pile group effects should take into account the potential changes in stress and density of the ground resulting from pile installation together with the effects of group behaviour due to the structural loads.

(5) <REQ> The ultimate resistance of a pile group shall be taken as the lower of:

- the sum of the resistances of the individual piles in the group;
- the resistance of the block of ground bounded by the perimeter of the pile group.

(6) <RCM> The ultimate vertical compressive resistance of a pile group R_{group} should be determined from Formula (6.8):

$$R_{group} = \min\left\{\sum_{i}^{n} R_{c,i}\,;R_{block}\right\} \tag{6.8}$$



Main modifications with regard to current EN 1997-1

- **Design of piled rafts**

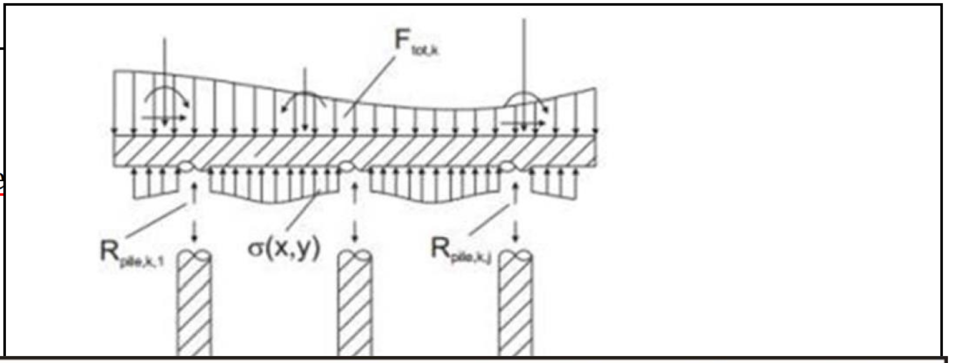
6.5.6 Design of piled rafts

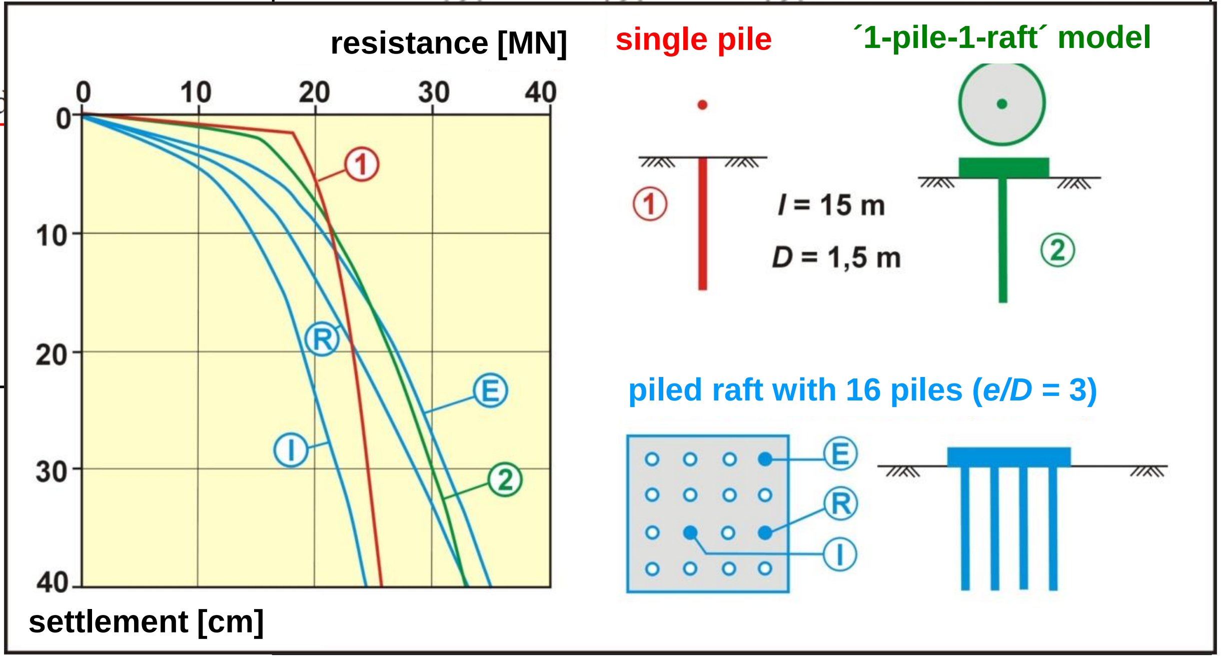
(1) <RCM> The ultimate vertical compressive resistance
Formula (6.9):

$$R_{\text{group}} = \left(\sum_{i}^{n} R_{\text{c,i}} + R_{\text{raft}} \right)$$

(2) <RCM> The design of piled rafts should
6.1:

- pile-soil interaction;
- pile-pile interaction;
- raft-soil interaction;
- pile-raft interaction.





Main modifications with regard to current EN 1997-1

- **Validation of pile design by site-specific load testing or comparable experience**

6.5.8 Validation of pile design by site-specific load testing or comparable experience

(1) <RCM> Pile design should be validated using site-specific sta
parameter values, verify compressive or tensile resistance, a
serviceability limit state conditions.

(2) <PER> Pile design against compression loading may also be
rapid load tests provided that these tests have been validate

(3) <PER> Site-specific load testing may be omitted where there
evidence of previous successful use for the same type of pile
similar ground conditions.

NOTE 1. A classification of additional information for validation o

Table 6.4 (NDP) – Minimum quantity of load testing for validation of pile design

Type of load test	Validation of design by	
	Investigation Tests	Control Tests
Static load test	1 or ≥ 0.5% N	2 or ≥ 1% N
Rapid load test	2 or ≥ 1.0% N	4 or ≥ 2% N
Dynamic impact load test	3 or ≥ 2.5% N	6 or ≥ 5% N
N = total number of working piles for a reference area of 2,500m²		

Table 6.3 – Classification of additional information used to validate pile design

Classification[a]	Pile load tests on same site	Comparable experience[b]
Comprehensive	Investigation tests as specified in Table 6.4 (NDP)	Extensive comparable experience or database
Limited	Control tests as specified in Table 6.4 (NDP)	Limited comparable experience or database
Minimum	No pile load tests	Minimum comparable experience or no database

[a]Classification based on the higher of the two columns
[b]Comparable experience is defined in EN 1997-1, 3.1.1.17. For piled foundations, this includes documented data from different sites for similar pile types under similar ground and loading conditions such as historical pile load test data, research or evidence of successful use based on measurements or observations of pile performance.

(5) <RCM> The number of site-specific pile loads tests n_{test} to conform to Table 6.3 should be selected according to the type or purpose of the load test.

NOTE 1. Values of n_{test} are given in Table 6.4 (NDP) unless the National Annex gives different values.

Main modifications with regard to current EN 1997-1

- **6.6 Ultimate limit states**

Model factors

(2) <REQ> The design axial compressive resistance R_{cd} be determined from Formula (6.11):

$$R_{cd} = \frac{R_{c,rep}}{\gamma_{Rc} \cdot \gamma_{Rd}} \quad or \quad \left(\frac{R_{b,rep}}{\gamma_{Rb} \cdot \gamma_{Rd}} + \frac{R_{s,rep}}{\gamma_{Rs} \cdot \gamma_{Rd}}\right)$$

Table 6.5 (NDP) – Model factor γ_{Rd} for verification of axial pile resistance by calculation

Verification by		Model factor γ_{Rd}
Calculation Method A Ground Model Method	Comprehensive additional information[1]	1.3
	Limited additional information[1]	1.55
	Minimum additional information[1]	1.8
Calculation Method B Model Pile Method	Ménard Pressuremeter test	1.15
	Cone penetration test	1.2
	CPT with comprehensive comparable experience	1.0
[1]Classification of additional information is given in Table 6.3.		

Table 6.6 (NDP) – Model factor γ_{Rd} for verification of axial pile resistance by testing

Verification by		Model factor γ_{Rd}			
		Fine soils	Coarse soils	Rock	Competent Rock
Static load tests		1.0	1.0	1.0	1.0
Dynamic impact and rapid load tests (closed form solutions)[a]	Shaft bearing	Not permitted	Signal matching	Signal matching	1.2
	End bearing		1.3	1.3	1.2
Dynamic impact and rapid load tests (signal matching) [a]	Shaft bearing	1.5	1.1	1.2	1.1
	End bearing	1.4	1.2	1.2	1.1
Wave equation analysis		Not permitted	1.6	1.5	1.4
Pile driving formulae			1.8	1.7	1.5

[a]When dynamic impact and rapid load tests are not calibrated by site-specific static load testing, but by comparable experience only, the values for γ_{Rd} are increased as follows:
+0.1 when calibration is based on comprehensive additional information, as defined in Table 6.3
+0.25 when calibration is based on limited additional information, as defined in Table 6.3

<Drafting NOTE>The model factors in Table 6.6 (NDP) have been increased (provisionally) compared to current EN 1997 so that they are > 1.0 (so that the model factor correctly reflects uncertainty in the model). The values are still under review by PT4 and once agreed the correlation factors for testing will be re-calibrated to the new model factors>

Main modifications with regard to current EN 1997-1

- **6.6.1.7 Calculation of representative resistances**

Method A

(1) <REQ> For design by calculation using Method A, the representative resistance of a single pile R_{rep} shall be determined from Formula (6.16):

$$R_{rep} = R_{calc} \tag{6.16}$$

Method B

(2) <REQ> For design by calculation using Method B and for design assisted by testing, the representative resistance of a single pile R_{rep} shall be determined from Formula (6.17):

$$R_{rep} = min\left\{\frac{(R_m)_{mean}}{\xi_{m,mean}}; \frac{(R_m)_{min}}{\xi_{m,min}}\right\} \tag{6.17}$$

(5) <REQ> The values of the correlation factors $\xi_{m,mean}$ and $\xi_{m,min}$ for Method B shall be determined based on the number of profiles in the single data set and the coefficient of variation CoV calculated in (4).

NOTE 1. Values of $\xi_{m,mean}$ and $\xi_{m,min}$ for verification by calculation using Method B are given in Table 6.7 (NDP) unless the National Annex gives different values.

NOTE 2. The correlation factors given in in Table 6.7 (NDP) assume profiles arranged on a grid with maximum spacing of 30 m.

Table 6.7 (NDP) – Correlation factors for pile design by calculation (Method B)

Correlation factor	Coefficient of variation	Number of profiles						
		1	2	3	4	5	7	10
$\xi_{m,mean}$	≤ 12%	Use $\xi_{m,min}$ alone		1.30	1.28	1.28	1.27	1.26
	15%			1.40	1.39	1.38	1.37	1.36
	20%			1.67	1.64	1.63	1.61	1.60
	≥ 25%			1.98	1.95	1.93	1.90	1.89
$\xi_{m,min}$	All	1.4	1.27	1.23	1.20	1.15	1.12	1.08

Main modifications with regard to current EN 1997-1

- **6.6.1.7 Calculation of representative resistances**

Correlation factors

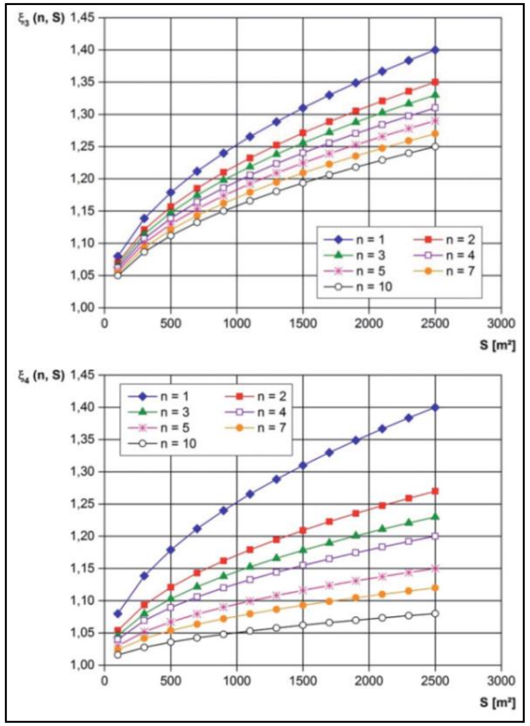


Table 6.8 (NDP) – Correlation factors for pile design based on results of static load tests

Correlation factor	Number of static load tests				
	1	2	3	4	5
$\xi_{m,mean}$	1.4	1.3	1.2	1.1	1.0
$\xi_{m,min}$	1.4	1.2	1.05	1.0	1.0
Numbers of static load tests are per a reference area of 2,500 m²					

Table 6.9 (NDP) – Correlation factors for pile design based on rapid load tests

Correlation factor	Number of rapid load tests					
	2	3	5	10	20	>50
$\xi_{m,mean}$	1.6	1.55	1.5	1.45	1.4	1.3
$\xi_{m,min}$	1.5	1.45	1.35	1.3	1.25	1.2
<Drafting NOTE> PT4 requests guidance on suitable correlation factors for > 50 tests						

Table 6.10 (NDP) – Correlation factors for pile design based on dynamic impact tests

Correlation factor	Number of dynamic impact tests					
	3	5	10	20	>50	All
$\xi_{m,mean}$	1.55	1.5	1.45	1.4	1.3	1.25
$\xi_{m,min}$	1.45	1.35	1.3	1.25	1.2	1.15
<Drafting NOTE> PT4 requests guidance on suitable correlation factors for > 50 tests						

Main modifications with regard to current EN 1997-1

- **6.6.4 Partial factors for single piles**

Table 6.11 (NDP). Partial factors for the verification of ultimate resistance of single piles for fundamental (persistent and transient) design situations

Verification of	Partial factor on	Symbol	Material factor approach	Resistance factor approach (RFA)			
				Pile class	(c)		(d)	
				All	DC1		DC3	
					1.15		1.0	
					Not factored			
				High displacement	1.2	1.0	1.3	1.3
				Low displacement	1.2	1.0	1.35	1.3
				CFA	1.1	1.1	1.45	1.3
				Bored	1.1	1.1	1.6	1.3
				Unclassified	1.35	1.25	1.9	1.5
				High displacement	1.1		1.3	
				Low displacement			1.35	
				CFA			1.4	
				Bored			1.5	
				Unclassified	1.3		1.75	
				All	DC1		DC3	
					Not factored			
				High displacement	1.15		1.6	
				Low displacement			1.6	
				CFA			1.6	
				Bored			1.6	
				Unclassified	1.6		1.9	
				Not Used				
				All	1.1		1.4	

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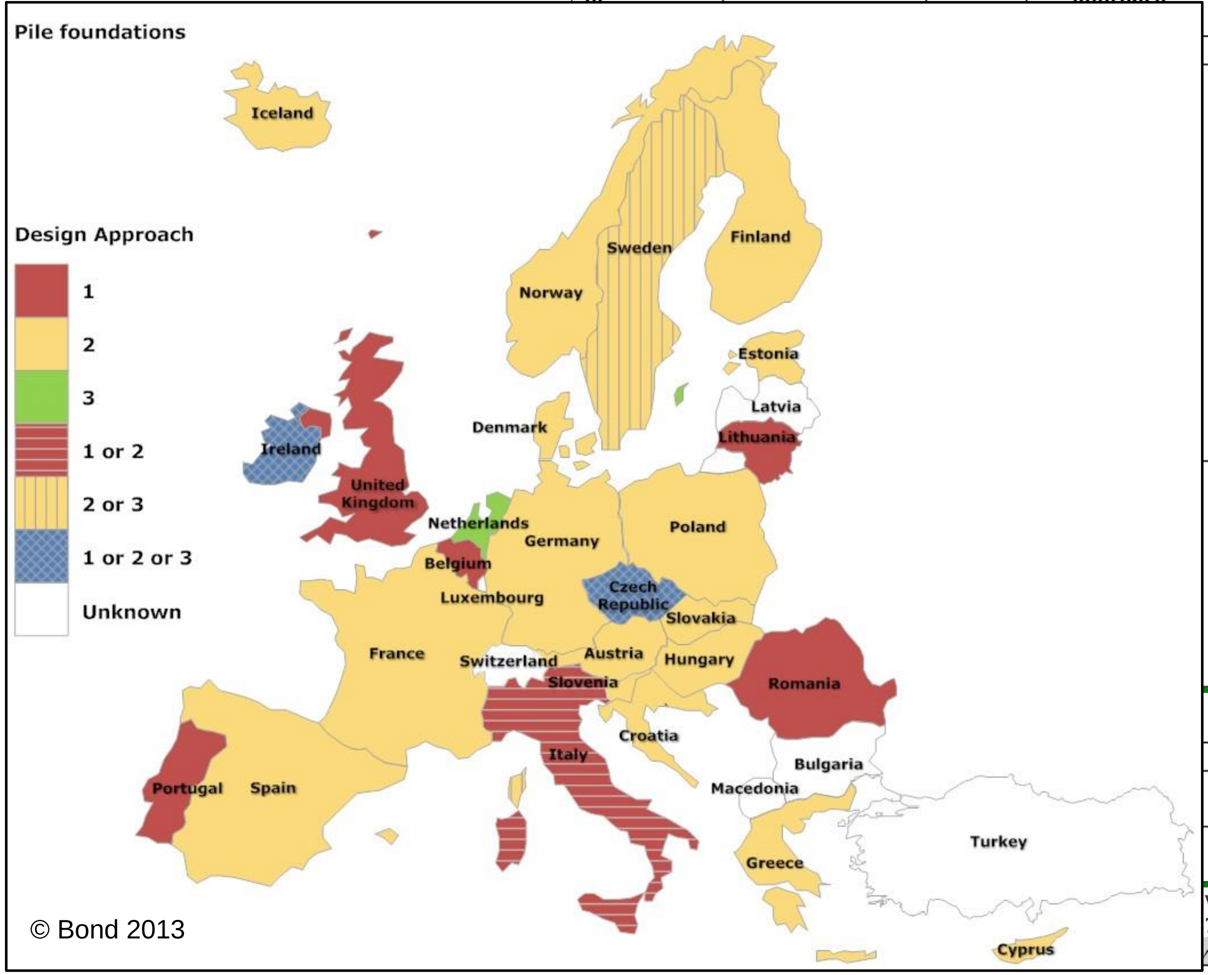


Main modifications with regard to current EN 1997-1

- **6.6.4 Partial factors for pile groups and piled rafts**

  - MFA or RFA for axial resistance of pile groups & piled rafts
  - MFA for transverse resistance of pile groups & piled rafts

Table 6.12 (NDP). Partial factors for the verification of ultimate resistance of pile groups and piled rafts for fundamental (persistent and transient) design situations

Verification of	Partial factor on	Symbol	Material factor approach (MFA) (a)	Material factor approach (MFA) (b)	Resistance factor approach (RFA)
Axial resistance	Actions and effects-of-actions[1]	γ_F and γ_E	DC4	DC3	Use partial factors from Table 6.11 (NDP)
	Ground properties[2]	γ_M	M1	M3	
	Resistance	γ_R	Not factored		1.4
Transverse resistance	Actions and effects-of-actions[1]	γ_F and γ_E	DC4	DC3	Not Used
	Ground properties[2]	γ_M	M1	M3	
	Combined axial and transverse	γ_{Re}	Not factored		
Combined axial and transverse resistance	Same as for transverse resistance				

[1]Values of the partial factors for Design Cases (DCs) 3 and 4 are given in EN 1990 Annex A.
[2]Values of the partial factors for Sets M1 and M3 are given in EN 1997-1 Annex A.

$$F_{\mathrm{d,group}} \leq R_{\mathrm{d,group}}$$

$$R_{\mathrm{d,group}} = \frac{R_{\mathrm{rep,group}}}{\gamma_{\mathrm{R}}} \; or \left(\frac{\sum_i^n R_{\mathrm{c,rep,i}}}{\gamma_{\mathrm{Rt}}} + \frac{R_{\mathrm{rep,raft}}}{\gamma_{\mathrm{R,raft}}} \right)$$

Main modifications with regard to current EN 1997-1

- **6.9 Testing**

6.9.6 Acceptance tests

6.9.6.1 Rig monitoring and instrumentation

(1) <REQ> For continuous flight auger and displacement auger piles, the piling rig shall be fitted with a suitable automated instrumentation and monitoring system capable of measuring the execution metrics throughout the boring and concreting of the pile.

(2) <RCM> Piling rigs used to install driven displacement piles should be fitted with a suitable automated instrumentation and monitoring system capable of measuring the execution metrics throughout the pile driving process.

6.9.6.2 Non-destructive integrity tests

(1) <RCM> All cast-in-place or precast concrete piles shall be subject to non-destructive integrity testing to verify the pile does not include any defects within the shaft and has not been damaged during installation.

(2) <PER> The method for integrity testing may be chosen from the following:

– low strain Pile Integrity Test;
– thermal integrity profiling;
– cross-hole sonic logging method;
– distributed fibre optic sensing method.

0| Contents

Concept / strategy

- **Effectively a new clause**

3.1.8.5 discrete ground improvement

ground improvement zone comprising inclusions created in the ground with properties differing from the surrounding ground

3.1.8.6 diffused ground improvement

ground improvement where the ground improvement zone can be modelled with a single set of parameters

- **Scope and families of ground improvement**

10.1 Scope

(1) <REQ> This Clause shall apply to all forms of ground improvement used with the following geotechnical structures and applications:

– slopes, cuttings, and embankments (see also Clause 4);
– spread foundations (see also Clause 5);
– retaining structures (see also Clause 7);
– water control.

(2) <REQ> Ground improvement shall be classified according to Table 10.1 and divided into two families:

– diffused ground improvement; or
– discrete ground improvement.

NOTE 1. Examples of ground improvement in these two families are given in Table 10.2.

NOTE 2. Details of example techniques listed in Table 10.2 are given in Annex G.

Table 10.2 – Examples of ground improvement in different classes and families

Class	Family	
	Diffused	Discrete
I – Improved ground	Compactive Methods Soil Replacement Consolidation Methods	Granular Columns
II – Modified Ground	Grouting Methods Mixing Methods Other Methods	Grouting/Mixing Methods Steel/Wood Columns Concrete/Grout Columns
III – Groundwater control	Fissure grouting in rock Grouting methods	Cut-off walls Drains

Concept / strategy

- **Families of ground improve**

Table 10.1 – Classification of ground im

Class		Family	
		Diffused	Discrete
I	Improved ground	... having increased shear capacity and/or reduced permeability compared to the surrounding ground but can be classified as improved ground	... containing inclusions with increased shear capacity and stiffness compared to the surrounding ground
II	Modified Ground	... having measurable unconfined compressive strength and is significantly stiffer than the surrounding ground and/or of reduced permeability and comprises a composite of a binder and ground. It usually behaves as a structural zone	... containing rigid inclusions with measurable unconfined compressive strength and is significantly stiffer than the surrounding ground, may be an engineered material such as timber, concrete/grout or steel or a composite of a binder and ground
III	Groundwater control	... having reduced or increased permeability with a primary function to control groundwater pressures or flows	... provides either a barrier to groundwater flow or elements to increase drainage

Concept / strategy

- **Annex G.3**
  Examples of discrete and diffused ground improvement techniques

Table G.2 – Examples of discrete ground improvement techniques

Method	Technique	Class	Description	Execution Standard
Mixing Methods	Dry methods	II&III	Mechanical disaggregation of soils while introducing a dry binder pneumatically and commonly cement. Most usually executed in soft to very soft clays and silts. Land and marine based rigs available to considerable depths.	EN 14679
	Wet methods	II&III	Mechanical disaggregation of soils while introducing a fluid binder. Generally more powerful system than the dry system and can be executed in various type of soils. Land and marine based rigs available to considerable depths.	EN 14679
	Jet grouting	II&III	Hydraulic disaggregation of soils using high velocity jets of fluid binder combined or not with either water or water and air. Suitable for most soils and available for land or marine use to considerable depths.	EN 12716
Granular Columns	Stone columns/ Vibro-replacement	II	Compacted stone columns are created in the ground to form a composite ground with the surrounding soil. For cohesive soil, this increases the shear strength, and at the same time, it stabilises early settlement to reduce the consolidation settlement rate. For granular soils, it increases the relative density, thus enhances the shear strength. Land and marine based rigs available to considerable depths.	EN 14713
	Sand columns/ Sand compaction piles	II	Compacted sand columns are created in the ground to form a composite ground with the surrounding soil. For cohesive soil, this increases the shear strength, and at the same time, it stabilises early settlement to reduce the consolidation settlement rate. For granular soils, it increases the relative density, thus enhances the shear strength. Land and marine based rigs available to considerable depths.	EN 14713
	Dynamic replacement	II	The use of dynamic compaction to drive bulbs of granular material into soft soils thereby both improving the soil by the dynamic compaction and the introduction of competent granular piers. Most often used in soft cohesive soils to improve strength and accelerate drainage. Land and marine based rigs available.	None
	Geosynthetics encased columns	II	Stone or sand columns, encased in a geotextile casing, formed in very soft soils where the lateral restraint is too small to prevent very significant column bulging. The geotextile casing provides support to the columns and prevents excessive bulging under load. Land and marine based rigs available to significant depths.	None
Steel/Wood	Vibrated	II	Rigid columns of steel or wood are vibrated into the ground, causing some densification, to form a composite ground with	None

Concept / strategy

- **Definition of terms**

3.1.8.4 rigid inclusion

inclusions with higher stiffness and a measurable unconfined compressive strength

(7) <REQ> The design differentiation between a pile and a rigid inclusion shall be based on:

- the physical or structural connection or contact detail (if any) with the foundation;
- whether the foundation support design includes any load contribution from the ground other than direct shaft or other friction applied to the inclusion.

(6) <REQ> An inclusion placed in the ground in isolation and acting as a single element shall be designed as a piled foundation in accordance with Clause 6.

Main modifications with regard to current EN 1997-1

- **Geotechnical Complexity Class (GCC) for ground improvement**

Table 10.5 (NDP) – Selection of Geotechnical Complexity Class for ground improvement

Geotechnical Complexity Class	Complexity	General features causing uncertainty
GCC 3	Higher	Considerable uncertainty regarding the following apply: • difficult ground or groundwater conditions • difficult geomorphologies or complex geological conditions • significant complexity of the ground-structure interaction • unusual application of ground improvement with no comparable experience • unusual performance requirements for the ground improvement out with documented comparable experience
GCC 2	Normal	Some of the following apply: • some uncertainty regarding the ground or ground water conditions • some ground-structure interaction • ground improvement application with previous design experience • performance requirements within previously achieved limits
GCC 1	Lower	Not applicable to Ground Improvement

[a] The terms 'difficult', 'significant', etc. are relative to any comparable experience that exists for the particular geotechnical structure and design situation

Main modifications with regard to current EN 1997-1

▪ 10.3.2 Improved ground properties

10.3.2.3 Diffused or Discrete Ground Improvement – Class II

(1) <REQ> The characteristic value of the unconfined compressive strength of the improved ground $q_{uk,imp}$ shall be determined from Formula 10.4:

$$q_{uk,imp} = \exp\left(m_y - k_n\{P\} \cdot s_y\right) \tag{10.4}$$

where:

m_y mean of the measured values of $\log(q_{u,field})$;

s_y standard deviation of the measured values of $\log(q_{u,field})$;

$k_n\{P\}$ acceptance value for the sample distribution in terms of *P;*

$\log(q_{u,field})$ logarithm of the unconfined compressive strength measured in unconfined compressive tests on field samples;

P percentage of test results passing the required characteristic value.

NOTE 1. Table 10.1 gives values of k_n for varying passing percentages.

NOTE 2. The value of P is 10% unless the National Annex gives a different value.

Table 10.1 – Values of k_n to be used with Formula 10.4

Percent Passing, *P* (%)	5%	10%	15%	20%	25%	30%
Acceptance value, k_n	1.64	1.28	1.04	0.84	0.69	0.53

Main modifications with regard to current EN 1997-1

- **10.5.3 Geotechnical Analysis – Discrete ground improvement**

(1) <REQ>Where discrete ground improvement is utilised as part of a system to support or retain a structure an interaction calculation method shall include:

- the evaluation of the interaction effects between the ground, discrete inclusions, and the overlying structure, embankment, or load transfer platform similar as for piled rafts (see 6.5.6);
- the derivation of the neutral plane corresponding to the point where the inclusion settlement equals the ground settlement (see Figure 10.3);
- the derivation of the distribution ratio to determine the proportion of the load applied to individual discrete inclusions;
- a verification of the structural resistance of the individual discrete inclusions;
- a verification of buckling resistance.

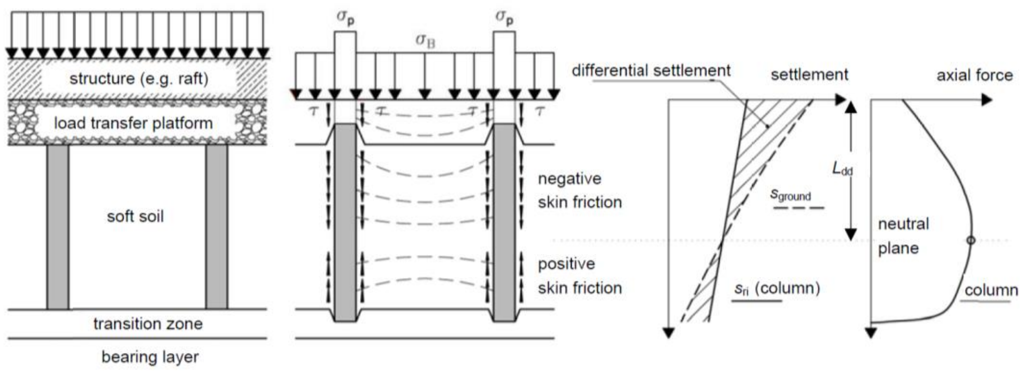


Figure 10.3 – Interaction effects of ground improvement with rigid inclusions

(2) <REQ> For discrete ground improvement the load distribution and the depth of the neutral plane L_{dd} shall be calculated according to Figure 10.3 using representative material parameters.

(3) <RCM> The total compressive resistance R_{sys} of a ground improvement system with rigid inclusions should be determined from Formula (10.8):

$$R_{sys} = \sum_{i=1}^{n} R_{ri,i} + R_g \qquad (10.8)$$

Main modifications with regard to current EN 1997-1

- **Ultimate limit state**

10.6.1 General

(1) <REQ> Verification of design shall be by the appropriate method as set out in EN 1997-1, E5.3.

(2) <REQ> For all ground improvement, verification shall determine that:

- the design improved ground properties have been achieved;
- external/geotechnical stability of the overall system and internal/structural stability of the ground improvement is achieved; and
- installed inclusions conform geometrically to the requirements of the design.

(3) <RCM> The design resistance of ground improvement system with rigid inclusions $R_{\mathrm{sys,d}}$ should be determined from Formula (10.10):

$$\boxed{R_{\mathrm{sys,d}} = \frac{R_{\mathrm{sys,rep}}}{\gamma_{\mathrm{R,sys}}\gamma_{\mathrm{Rd}}}} \qquad (10.10)$$

Main modifications with regard to current EN 1997-1

- **Ultimate limit state – partial material factors for improved ground**

Table 10.6 (NDP) – Partial factors for the verification of ultimate resistance of ground improvement for fundamental (persistent and transient) design situations

<table>
<tr><th rowspan="2">Verification of</th><th rowspan="2">Partial factor on</th><th rowspan="2">Symbol</th><th colspan="2">Material factor approach (MFA)</th><th colspan="2">Resistance factor approach (RFA)</th></tr>
<tr><th>(a)</th><th>(b)</th><th>(c)</th><th>(d)</th></tr>
<tr><td rowspan="3">Axial compressive resistance of diffused ground improvement</td><td>Actions and effects-of-actions[1]</td><td>γ_F and γ_E</td><td>DC1</td><td>DC3</td><td colspan="2" rowspan="3">Refer to other clauses as appropriate</td></tr>
<tr><td>Ground properties[2]</td><td>γ_M</td><td colspan="2">Not factored</td></tr>
<tr><td>Total resistance</td><td>γ_t</td><td colspan="2">Not factored</td></tr>
<tr><td rowspan="4">Axial compressive resistance of discrete rigid inclusions</td><td>Actions and effects-of-actions[1]</td><td>γ_F and γ_E</td><td colspan="2" rowspan="4">Not Used</td><td>DC1</td><td>DC3</td></tr>
<tr><td>Ground properties[2]</td><td>γ_M</td><td colspan="2">Not factored</td></tr>
<tr><td>Bearing resistance of LTP</td><td>γ_R</td><td colspan="2">Refer to Clauses 5 and 9</td></tr>
<tr><td>Overall system resistance</td><td>$\gamma_{R,sys}$</td><td>1.2</td><td>1.4 K_R</td></tr>
<tr><td rowspan="3">Transverse resistance of discrete and diffused ground improvement</td><td>Actions and effects-of-actions[1]</td><td>γ_F and γ_E</td><td>DC4</td><td>DC3</td><td colspan="2" rowspan="3">Not Used</td></tr>
<tr><td>Ground properties[2]</td><td>γ_M</td><td>M1</td><td>M3</td></tr>
<tr><td>Transverse resistance</td><td>γ_{Re}</td><td colspan="2">Not factored</td></tr>
<tr><td colspan="7">[1]Values of the partial factors for Design Cases (DCs) 1, 3, and 4 are in EN 1990 Annex A.
[2]Values of the partial factors for Sets M1 and M3 are in EN 1997-1 Annex A.</td></tr>
</table>

Main modifications with regard to current EN 1997-1

- **10.9 Testing**

Table 10.7 – Testing frequency for ground improvement

Ground Improvement Class	Area of ground improvement zone (m^2)	No. of tests*	
I and III	<900	Minimum 5	
	>900	Minimum 5 + 1 test per 625m^2	
II	**No of Inclusions**	Type A+	Type B+
	1 to 600	1 in 75	1 in 150
	601 to 2000	8 + 1 additional per 150 (maximum 16)	4 + 1 additional per 300 (maximum 8)
	>2000	16 + 1 additional per 250	8 + 1 additional per 500

*Type of control testing as required by the relevant execution standard or as specified by the relevant authority or for a specific project with the relevant parties.
+ Type A – Inclusions required for ULS, Type B Inclusions required only for SLS

Subjects of discussion

- **Should flow charts included as example for progressive design of ground improvement?**

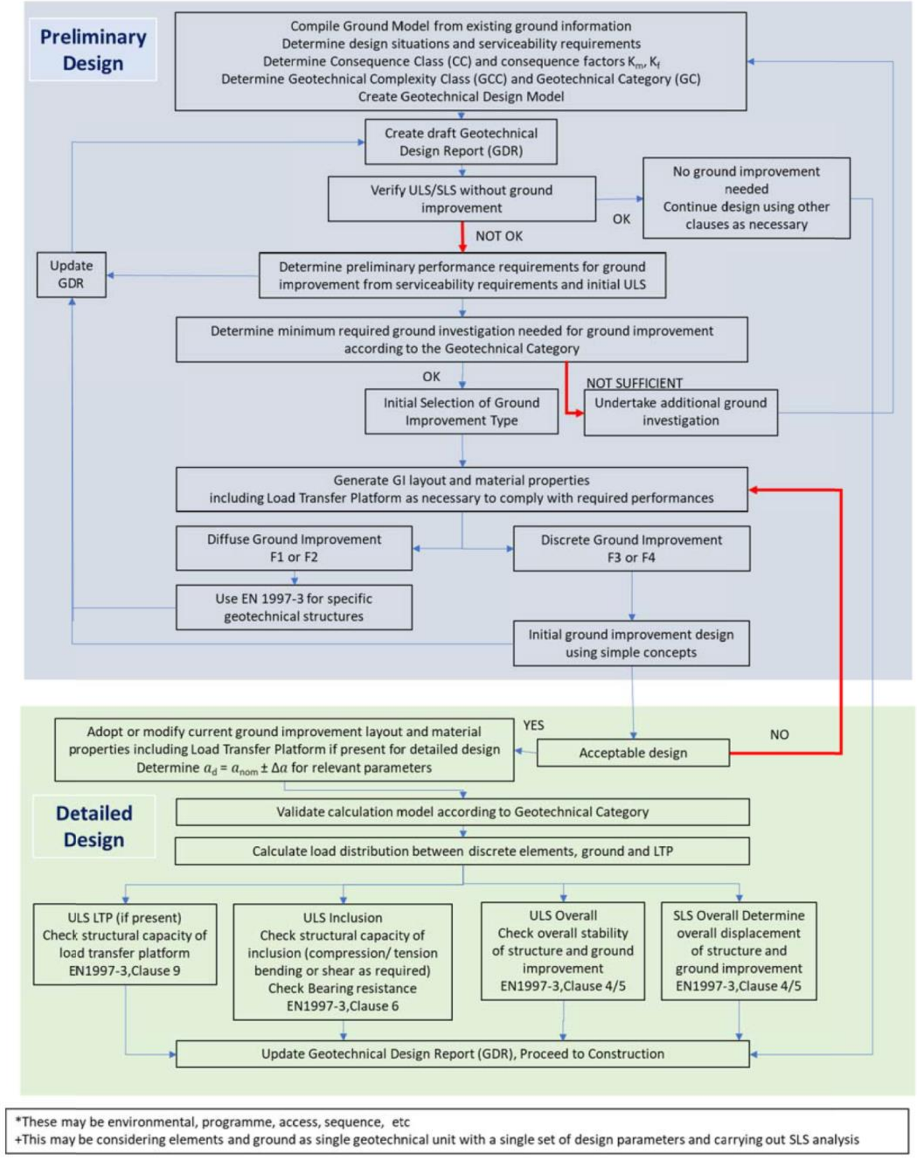
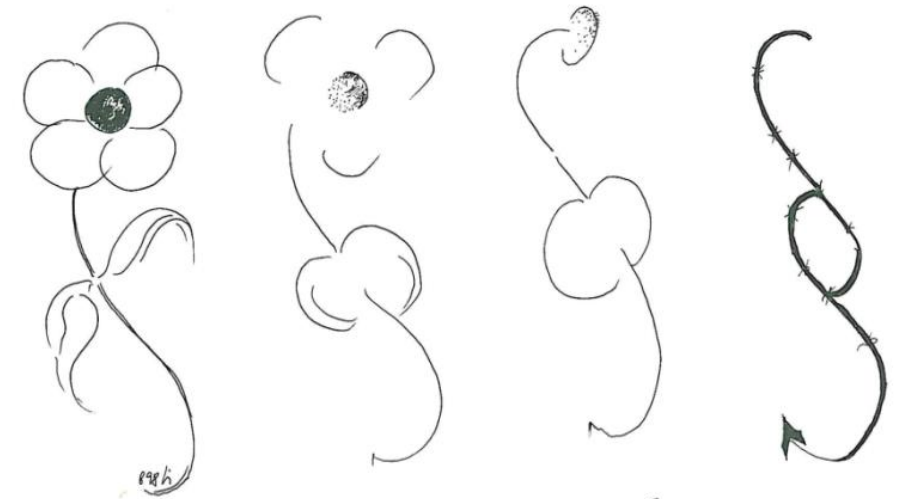


Figure G.1 – Flow chart showing preliminary and detailed design of ground improvement

Sincere thanks to

- all members of PT 4
- all members of WG3 / TG 1, TG 2, TG 3 and TG 7
- Andrew Bond
- Adriaan von Seeters



… about evolution of standards …

© Stiglat

…and you for your kind attention!

EC7-Part 3 from a Dutch perspective

Mandy Korff

KIVI Engineering Society

NEN

Part 3 – Slopes, Shallow and Pile foundations, Ground Improvement

Introduction and discussion

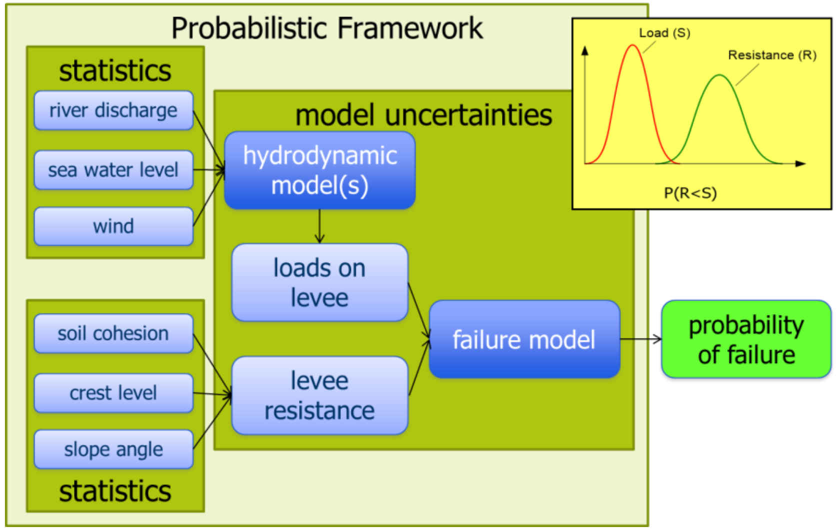
Dr. ir. Mandy Korff

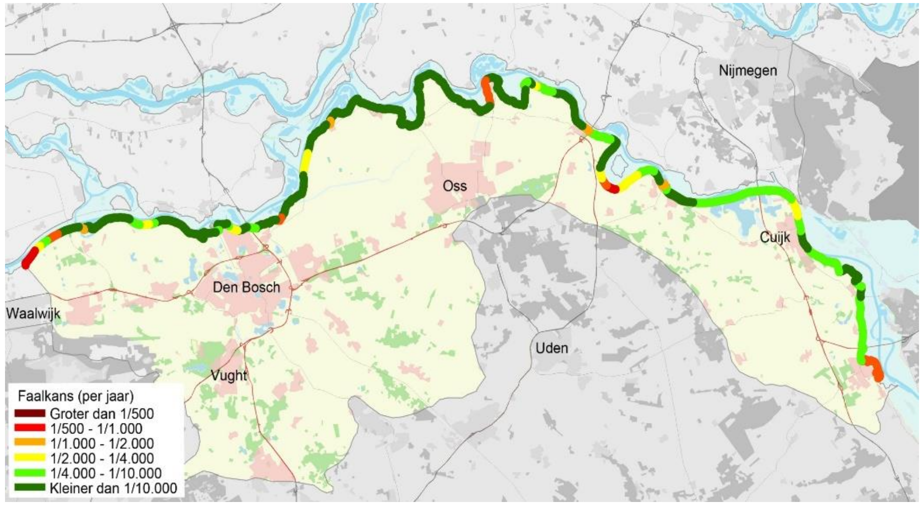
28 februari 2020

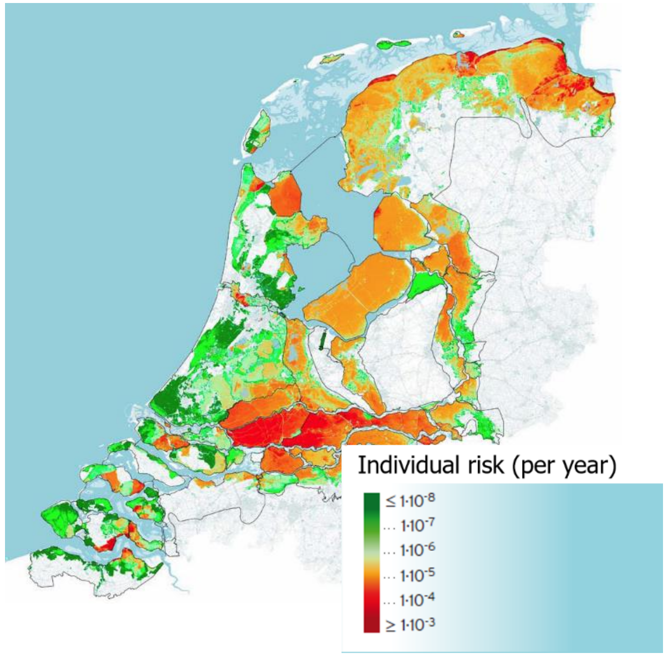
Slopes - levees

In the Netherlands levees are not designed with Eurocode, except for SLS conditions if the levees are part of the primary or secundary defense system.

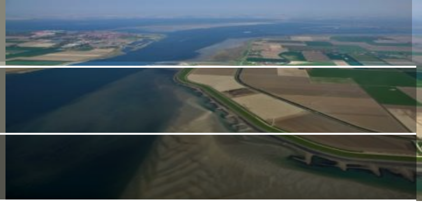
Levees are usually designed based on probabilistic soil scenarios.







Deltares



Slopes and Embankments

For embankments Eurocode 7 is used (2019)

Tekst new:

4.1 Scope

(1) <REQ> This clause shall apply to the design of cuttings, embankments and existing slopes within the zone of influence of construction works and activities.

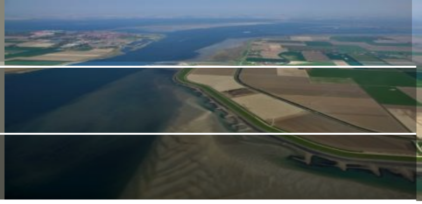
NOTE 1. EN 16907 applies to the construction of earthworks, including cuttings and embankments.

(2) <REQ> This clause shall also apply to overall stability, local stability, and displacement of nearby structures and infrastructure within the zone of influence.

(3) <REQ> This clause shall also apply to dams and levees but excludes the verification of water retention of those structures.

Deltares

Shallow foundation



Checking of existing foundations is currently the main issue in the Netherlands

Observational method?

5.5.5 Verification by the Observational Method

(1) <PER> The Observational Method may be used to verify the limit states of a spread foundation if any of the following conditions apply:

– it is not possible to verify by calculation, testing or prescriptive measures that the occurrence of the limit states referred to in 4.2.4 are sufficiently unlikely;
– the assumptions made in the calculations are not based on reliable data.

Piles

Many developments ongoing in The Netherlands at the moment

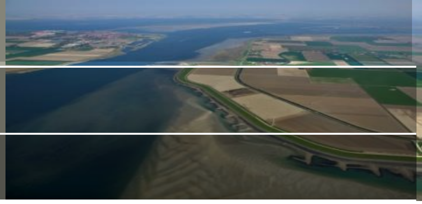
– pile load testing (static / RLT) Piles

- development of new pile design method (updating Dutch method/Koppejan)

Common topics for Dutch conditions:

- Negative skin friction
- Group effects (positive and negative)
- Variable loads

Piles



Will these clauses lead to new checks?

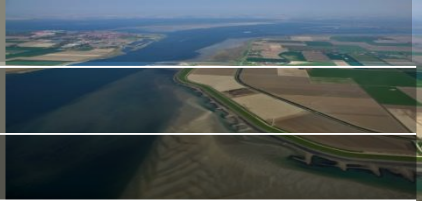
6.2.2.2 Pile geometry

(1) <REQ> Pile dimensions shall be selected according to the pile type and method of execution, the stability of the ground, and the susceptibility of the ground to changes caused by pile installation, taking into account potential bulging of the pile and oversized or undersized bores.

(2) <REQ> The adverse effect of pile imperfections (including positional and verticality tolerances or curvature of the pile shaft) that affect pile behaviour shall be taken into account in the verification of limit states.

6.2.5 Robustness

(1) <REQ> The design of piled foundations shall be modified to account for any significant variations from the expected pile behaviour encountered during driving or variations from expected ground conditions revealed during boring.



Piles

(3) <REQ> For piled foundations in soils or very weak and weak rock masses, the minimum depth of investigation below the anticipated base of the piled foundation d_{min} shall be determined from Formula (6.1):

$$d_{min} = max(5\ m; 3D; p_{group}) \tag{6.1}$$

where:

D is the base diameter (for circular piles) or one-third of the perimeter (for non-circular piles) of the pile with the largest base;

p_{group} is the smaller dimension of a rectangle circumscribing the group of piles forming the foundation, limited to a maximum of 25 m

Is this realistic for pile groups? Depth of CPTs to 25 m under the tip?

Deltares

Piles

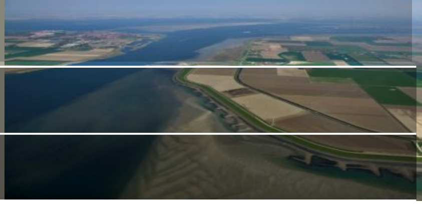
Table 6.2 (NDP) – Selection of Geotechnical Complexity Class for piled foundations

Geotechnical Complexity Class	Complexity	General features causing uncertainty
GCC 3	Higher	Considerable uncertainty regarding ground conditions or any of the following apply unless there is comparable experience or evidence of previous successful use: difficult ground conditions friction piles in very low strength ground vertical or horizontal ground movements site instability significant cyclic, dynamic or repeated loading
GCC 2	Normal	GCC2 should be selected if GCC1 and GCC3 are not relevant
GCC 1	Lower	All of the following conditions apply: negligible uncertainty regarding the ground conditions no ground movements

<Drafting note: PT6 to advise on what constitutes significant cyclic, dynamic or repeated loading>

What do you think?

Piles negative skin friction



(7) <PER> If the pile settlement is much smaller than the settlement of the surrounding ground, the neutral plane may be assumed to be located at the base of the settling layer.

(8) <RCM> The equivalent drag force D_{rep} should be determined from Formula (6.3):

$$D_{\text{rep}} = \pi D \int_0^{L_{\text{dd}}} \tau_{\text{s}} \cdot \text{dz} \qquad (6.3)$$

where:

D is the diameter of the pile for circular piles or equivalent diameter for non-circular piles;

τ_s is the unit shaft friction at depth z;

L_{dd} is the depth to the neutral plane.

NOTE 1. Calculation models for downdrag are included in Annex C.

No group effect in NSF?

Deltares

Piles partial factors

Values are NDPs!

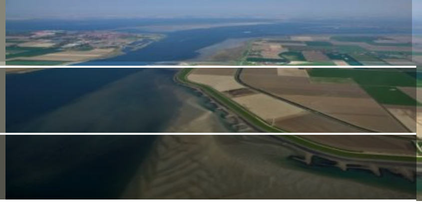
Table 6.11 (NDP). Partial factors for the verification of ultimate resistance of single piles for fundamental (persistent and transient) design situations

Verification of	Partial factor on	Symbol	Material factor approach (MFA)		Resistance factor approach (RFA)				
			(a)	(b)	Pile class	(c)		(d)	
Axial compressive resistance	Actions and effects-of-actions[1]	γ_F and γ_E	Not Used		All	DC1		DC3	
	Downdrag	$\gamma_{F,drag}$				1.15		1.0	
	Ground properties[2]	γ_M				Not factored			
	Base and shaft resistance in compression	γ_b | γ_s			High displacement	1.2	1.0	1.3	1.3
					Low displacement	1.2	1.0	1.35	1.3
					CFA	1.1	1.1	1.45	1.3
					Bored	1.1	1.1	1.6	1.3
					Unclassified	1.35	1.25	1.9	1.5
	Total resistance in compression	γ_t			High displacement	1.1		1.3	
					Low displacement			1.35	
					CFA			1.4	
					Bored			1.5	
					Unclassified	1.3		1.75	

[2]Values of the partial factors for Sets M1 and M3 are given in EN 1997-1 Annex A.

<Drafting NOTE: PT5 requests feedback on the proposed values for $\gamma_{F,drag}$>

Pile monitoring



(4) <RCM> Monitoring of pile execution should be carried out for all piles over the full depth of each pile and should include:

– piling rig monitoring and instrumentation records;
– drive blow records for driven piles;
– visual inspection of spoil and observations of ground conditions for auger bored and drilled piles.

6.9.6.1 Rig monitoring and instrumentation

(1) <REQ> For continuous flight auger and displacement auger piles, the piling rig shall be fitted with a suitable automated instrumentation and monitoring system capable of measuring the execution metrics throughout the boring and concreting of the pile.

(2) <RCM> Piling rigs used to install driven displacement piles should be fitted with a suitable automated instrumentation and monitoring system capable of measuring the execution metrics throughout the pile driving process.

Trial / test piles

(4) <REQ> Execution of the trial pile shall be performed in an identical manner to that proposed for the working piles and shall conform to the relevant execution standards given in 6.8.1.

(5) <PER> In cases where it is impractical to install or construct full-size large diameter trial piles, a smaller diameter trial pile can be installed provided that:

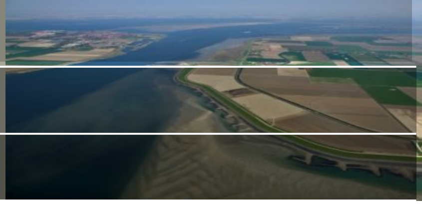
- the ratio of the trial pile to working pile diameter is not less than 0.5;
- the trial pile is constructed or installed in an identical manner to the proposed working piles;
- the trial pile is instrumented to allow separation of the base and shaft resistance during any test

6.9.4 Static load tests

(1) <REQ> The execution of the test pile shall be carried out in an identical manner to that proposed for the working piles and shall conform to the execution standards given in 6.8.1.

Deltares

Reinforced ground structures



Extensive of EC Part 3

Table 9.1 (NDP) – Selection of Geotechnical Complexity Class for reinforced ground structures

Geotechnical Complexity Class	Complexity	Examples of general features causing uncertainty
GCC 3	Higher	Considerable uncertainty regarding any of the following:
		known from comparable experience • negligible[a] risk of ground movements • low excavation below the groundwater level or such excavation is straightforward[a] • low height vertical or steep slope structures (< 3 m)
[a]the terms 'weak', 'high', 'low', 'negligible', and 'straightforward' are relative to any comparable experience that exists for the particular design situation		

d. Concentric arches method

(1) <REQ> In the concentric arches method, the surcharge on the load transfer platform shall be assumed to have a shape of inverse triangle.

NOTE 1. The detailed calculation procedure can be found in the Dutch Design Guideline CUR226.

Deltares

Ground improvement

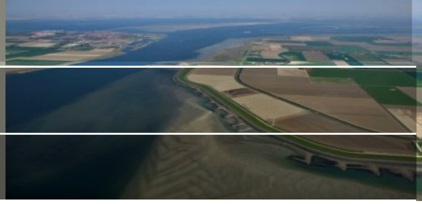


Table 10.1 – Classification of ground improvement techniques

Class		Family	
		Diffused	**Discrete**
I	Improved ground	... having increased shear capacity and/or reduced permeability compared to the surrounding ground but can be classified as improved ground	... containing inclusions with increased shear capacity and stiffness compared to the surrounding ground
II	Modified Ground	... having measurable unconfined compressive strength and is significantly stiffer than the surrounding ground and/or of reduced permeability and comprises a composite of a binder and ground. It usually behaves as a structural zone	... containing rigid inclusions with measurable unconfined compressive strength and is significantly stiffer than the surrounding ground, may be an engineered material such as timber, concrete/grout or steel or a composite of a binder and ground
III	Groundwater control	... having reduced or increased permeability with a primary function to control groundwater pressures or flows	... provides either a barrier to groundwater flow or elements to increase drainage

Family	
Diffused	**Discrete**
Compactive Methods Soil Replacement Consolidation Methods	Granular Columns
Grouting Methods Mixing Methods Other Methods	Grouting/Mixing Methods Steel/Wood Columns Concrete/Grout Columns
Fissure grouting in rock Grouting methods	Cut-off walls Drains

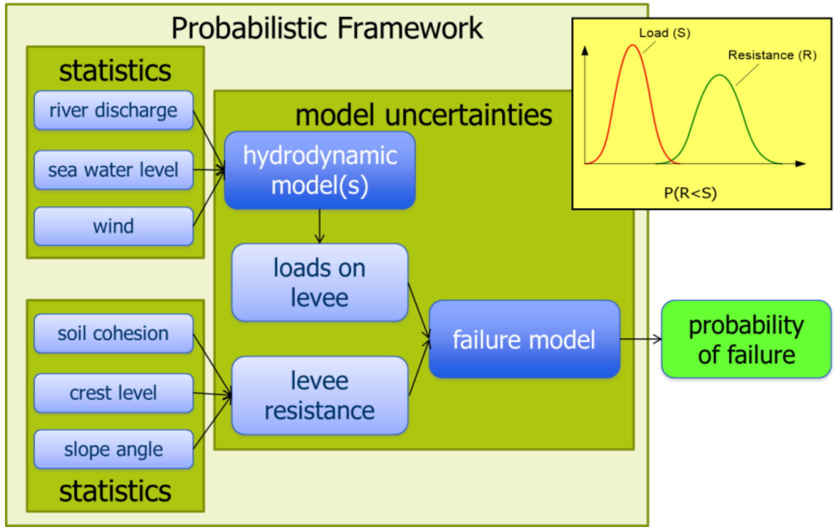
Much more context in nw EC7 – part 3 for ground improvement

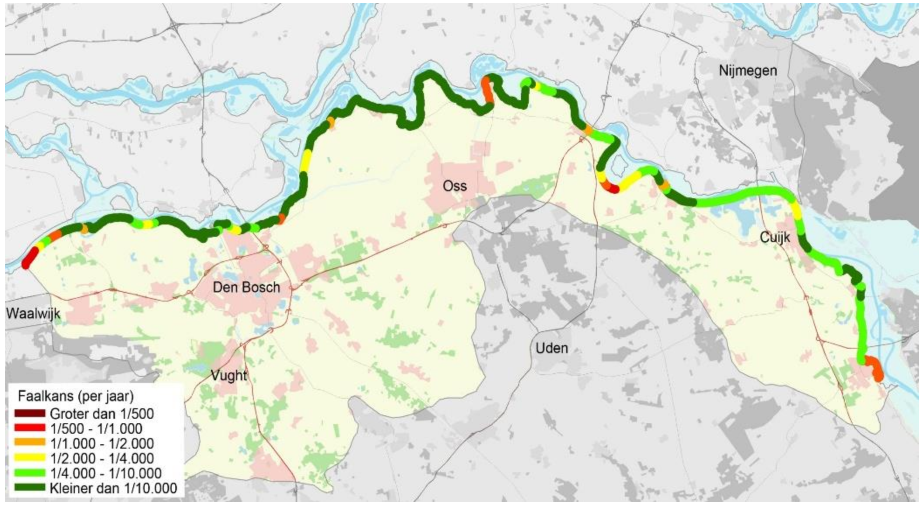
Starting points for discussion

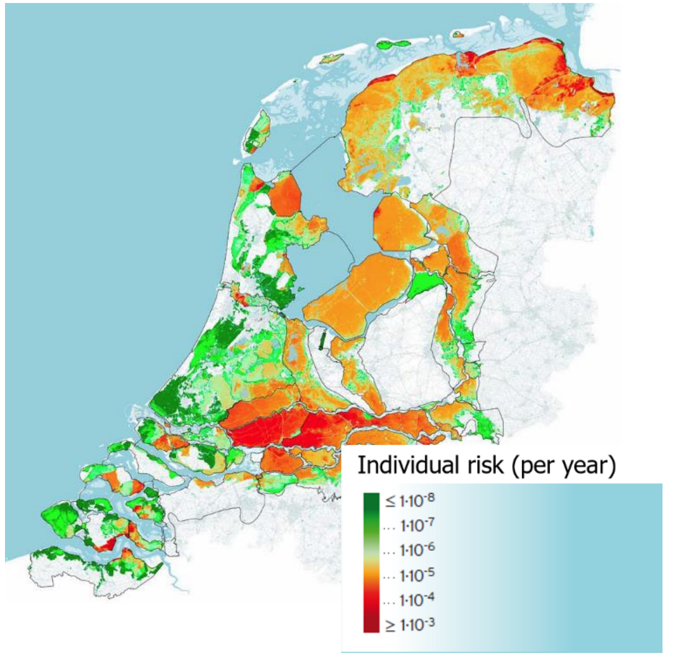
Slopes - levees

In the Netherlands levees are not designed with Eurocode, except for SLS conditions if the levees are part of the primary or secundary defense system.

Levees are usually designed based on probabilistic soil scenarios.







Deltares

Piles

Many developments ongoing in The Netherlands at the moment

– pile load testing (static / RLT) Piles

- development of new pile design method (updating Dutch method/Koppejan)

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- Negative skin friction
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- Variable loads

Deltares

Piles

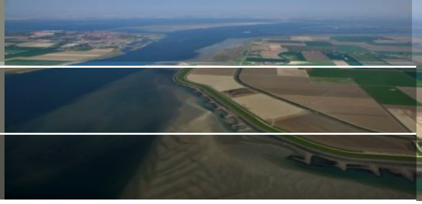

Table 6.2 (NDP) – Selection of Geotechnical Complexity Class for piled foundations

Geotechnical Complexity Class	Complexity	General features causing uncertainty
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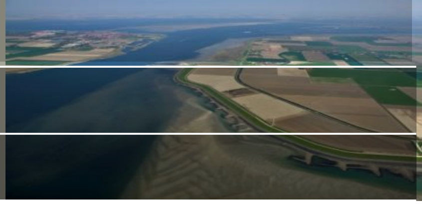
What do you think?

<Drafting note: PT6 to advise on what constitutes significant cyclic, dynamic or repeated loading>

Deltares



Piles

(3) <REQ> For piled foundations in soils or very weak and weak rock masses, the minimum depth of investigation below the anticipated base of the piled foundation d_{min} shall be determined from Formula (6.1):

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Is this realistic for pile groups? Depth of CPTs to 25 m under the tip?

Coffee break

Vervolg programma

15.00 Eurocode 7 – Part 3 – Retaining structures, Anchors and Reinforced Soil
15.30 EC7-Part 3 from a Dutch perspective perspective
16.00 Pile Load Testing in The Netherlands. What happens in practice?
16.30 Afsluiting

KIVI Engineering Society

NEN

Eurocode 7 – Part 3 – Retaining structures, Anchors and Reinforced Soil

Pierre Schmitt

KIVI
Engineering Society

NEN

Eurocode 7 – Part 3

Retaining structures, anchors, and reinforced ground

PT5

Chris Jenner

Martin Vanicek

Klaus Dietz

Christos Vrettos

Pierre Schmitt

- Scope of PT5
  - Clause 7 : Retaining structures
  - Clause 8 : Anchors
  - Clause 9 : Reinforced ground
- The status of these Clauses were very different in practice :
  - Clause 7 : Existing Clause 9 (now Clause 7) in previous EC7 was already consistent, but had not been modified after 2005
  - Clause 8 : Clause 8 had already been re-written in 2014
  - Clause 9 : This Clause did not exist at all in previous EC7

Eurocode 7 – Part 3

Clause 7 : Retaining structures

- Consistency with previous EC7:
  - Scope = embedded structures, gravity walls, composite structures
  - General recommendations relative to earth pressure calculation in the main text, earth pressure coefficients provided in Annex.
  - In practice, Clause 9 in previous EC7 included wise requirements that we did not delete (reduce and simplify when necessary).
  - Unchanged list of limit states to verify.
  - Reference to Clause 5 (Spread foundations) for the geotechnical resistance under gravity walls.
  - Reference to Clause 4 (Overall stability).
  - Reference to Clause 8 (Geotechnical resistance of anchors).

- Main evolutions compared with previous EC7:

  - SC7 requirements : provide more guidance about calculation models, promote cost-efficiency, look for harmonization, have ease-of-use in mind, try to be understood outside Europe as well. Not so easy to reconcile in practice…

  - Consensus on detailed calculation models not easy to achieve, but we tried to review main existing models and to provide guidance on their application ranges.

  - Basal heave addressed in Annex D, as a complement to hydraulic failure (hydraulic heave, piping, uplift) adressed in EN 1997-1.

  - Annex D also includes recommendations on compaction effects, vertical stability of embedded structures, and interaction between anchors and retaining structures.

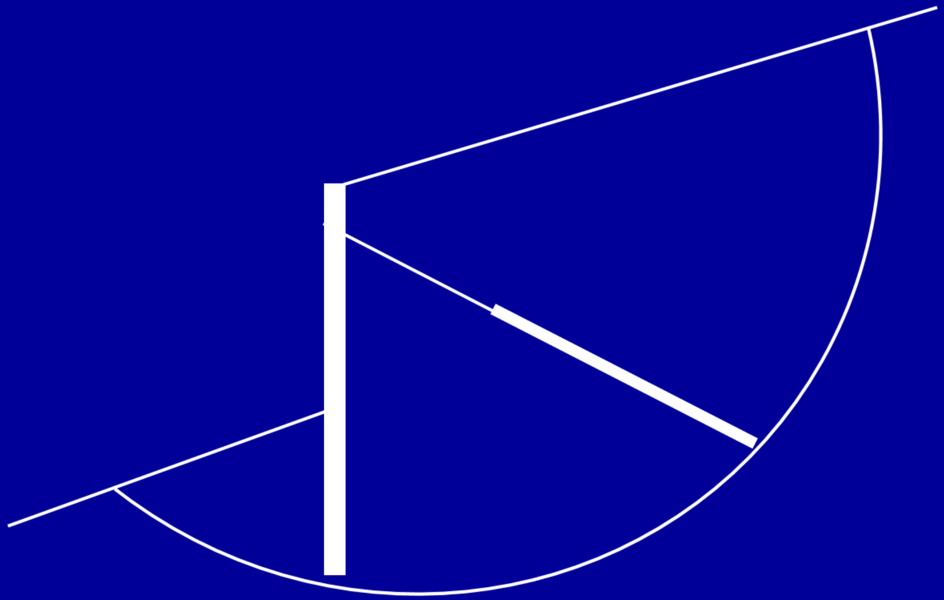
- Observational method:
  - Previous EC7 promoted OM as a design method, recommended to be used when geotechnical behaviour is complex and difficult to predict by calculation only.
  - First drafts of EN 1997-1 rather introduced OM as an option to increase cost-efficiency in specific conditions.
  - As retaining structures are an essential field of application for OM, it was decided to develop it within Clause 7, as part of Robustness, and Execution as well to insist and regulate interfaces between Execution and Design.
  - Use of OM is either a <PER>, when the purpose is cost-efficiency, or a <REC>, when the purpose is to increase robustness and deal with geotechnical hazard.

- MFA / RFA status :

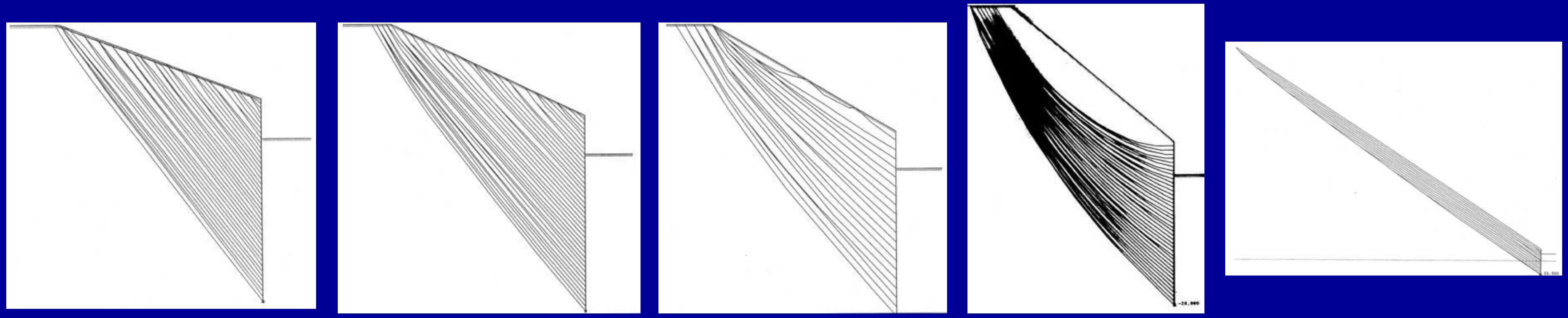
  - MFA is now mandatory for overall stability.

  - Meanwhile, regarding the assessment of structural forces applied to retaining structures, MFA can be performed through various calculation models (analytic-Bishop, FEM) that do not provide the same results.

  - MFA or RFA still may be used to justify rotational stability, although MFA implicitly applies if FEM has been used to justify overall stability.



- MFA / RFA proposal :
  - Countries that use RFA to calculate the required embedded length with respect to rotational stability generally concentrate partial factors on the passive earth resistance.
  - Such approach may not be safe in the presence of overall displacements, that may increase earth pressures acting on the wall, more especially if safety factors are low with respect to overall stability, and even more if the retaining structure itself may play a part in the stabilization process.



- MFA / RFA proposal :
  - Countries that use RFA to calculate the required embedded length with respect to rotational stability generally concentrate partial factors on the passive earth resistance.
  - Such approach may not be safe in the presence of overall displacements, that may increase earth pressures acting on the wall, more especially if safety factors are low with respect to overall stability, and even more if the retaining structure itself may play a part in the stabilization process.
  - It is thus recommended to use higher safety factors on (external) overall stability when RFA is used to verify rotational stability.
  - That makes a link with previous EC7 and national practices, while allowing both approaches.

- Unsolved issues :
  - Minimum earth pressure : every one agrees on the necessity to specify a minimum pressure, but not on its value.
  - Intermediate values of earth pressure : Annex D provides guidance to assess relevant orders of magnitude of the subgrade reaction coefficient, but there is a requirement to limit the number of clauses allowed to traditional methods compared with numerical models.
  - This needs to be discussed, more especially as SC7 required some guidance about soil reaction models applicable to the specific case of bridges abutments.

Eurocode 7 – Part 3

Clause 8 : Anchors

- Consistency with previous versions :
  - Clause 8 applies to anchors with a free length only.
  - Geotechnical resistance cannot derive from calculations only (this was allowed in the 2005 version, but tests were mandatory in the 2014 version).
  - As a consequence, MFA is not applicable to assess the design geotechnical resistance of anchors.
  - For RFA, the partial factor to be used is 1.1 as in previous versions.

- Main evolutions compared with previous versions (1) :
  - Clarification of terminology : « grouted anchors » instead of « ground anchors ». EN 1537 to be revised accordingly.
  - Investigation and suitability tests are still mandatory, but NA may authorize designs based on comparable experience.
  - The design anchor force is clearly defined compared with the 2014 version, it explicitly refers to the calculation of the retained structure or slope.
  - The expression $F_{serv,k}$ is replaced by $F_{SLS,k}$.

- Main evolutions compared with previous versions (2) :
  - Acceptance tests are mandatory for all anchors, and not only for grouted anchors as in the 2014 version.
  - Only Test Methods 1 and 3 are now applicable.
  - Annex E provides guidance for the layout of anchors in stratified ground and staggering of anchors in case of a protuding wall corner.

- Design by testing (1) :

  - Evolution of terminology : the « pull-out resistance » used in previous versions is the maximum force that can be taken by the anchor. This implicitly refers to an asymptotic load-displacement curve, that may not be systematically obtained in practice.

  - As test methods described in EN-ISO 22477-5 refer to creep rate rather than displacement evolution with load, it has been decided to exclusively refer to criteria based on measured creep rates, and accordingly define a « geotechnical ultimate limit state resistance » that may generally be considered as conservative compared with the properly-so-called pull-out resistance.

- Design by testing (2) :

  - As Test Method 2 is now abandoned, only 2 design philosophies now subsist in Europe, that is a significant step towards harmonization…

  - Test Method 1 (Cyclic Load Test, CLT) applies a severe criterion to derive the Geotechnical ULS resistance ($\alpha 1$ = 2mm), so that no explicit verification is needed with respect to SLS.

  - Test Method 3 (Maintained Load Test, MLT) applies a larger criterion ($\alpha 3$ = 5mm), unless the pull-out resistance itself is met during the test, but requires an explicit verification for SLS, that consists in checking that the service load is lower than the Critical Creep Load defined in EN-ISO 22477-5.

- Design by testing (3):

  - Partial factors applied over the Critical Creep Load when Test Method 3 is used are 1.2 for permanent anchors (design life > 2 years) and 1.1 for temporary anchors. In practice, the value of the Critical Creep Load must be assessed based on test results.

  - When checking that SLS is not met during suitability or acceptance tests, the service load is increased by a factor 1.25 for permanent anchors or 1.15 for temporary anchors, and an additional $\alpha 3$ criterion is used.

  - A significant difference between Test Methods 1 and 3 is that proof loads applied during suitability or acceptance tests refer to the ULS design load only for the former, and to the SLS load only for the latter.

  - In both cases, correlation factors may be taken as 1 since all anchors are tested.

- Unsolved issues:
  - Still two Test Methods TM1 and TM3 (although it is likely that associated designs are not so different in practice).

Eurocode 7 – Part 3

Clause 9 : Reinforced ground

- Scope of Clause 9, compared with Clause 10:
  - Reinforced ground (Clause 9) is not Improved ground (Clause 10)
  - Clause 9 does not include rigid inclusions
  - Inclusions covered by Clause 9 are not vertical, but horizontal or subhorizontal.
  - Reinforced ground member elements covered by Clause 9 are not supposed to act as a direct foundation, but used to retain ground.

- Clause 9 covers a large variety of materials:
  - Reinforced ground (e.g. Soil nailed structures)
  - Reinforced fill (e.g. Reinforced Earth)
    - Steel reinforcements
    - Geosynthetics reinforcements
  - Rock bolts

- Clause 9 covers a large variety of structures:
  - Reinforced fill (slopes/embankments, walls, bridge abutments)
  - Soil nailed structures (slopes/cuttings, walls)
  - Bolt structures (rock bolts)
  - Basal reinforcement for embankments (with or without rigid inclusions)
  - Voids overbridging
  - Veneer reinforcement
  - Reinforcement under shallow foundations
  - Geosynthetic encased columns

- Consequences for design recommendations:
  - Clause 9 essentially focuses on reinforcements themselves, in terms of resistance and durability
  - For the evaluation of actions, it refers to other Clauses (4, 7, 10) whenever relevant, with additional recommendations specific to reinforcements themselves.
  - Specific calculation models are proposed in Annex F for checking that geotechnical structures involving reinforced ground elements are safely designed.

- Specific calculation models:
  - Coherent gravity method (reinforced fill structures)
  - Tie-back wedge method (reinforced fill structures)
  - Two-part wedge method (reinforced fill structures and nailed structures)
  - Resistances to transverse sliding and extrusion (reinforced embankment bases)
  - Hewlett and Randolph method (embankments over rigid inclusions)
  - EBGEO method (embankments on rigid inclusions)
  - Concentric arches method (embankments on rigid inclusions)
  - Voids overbridging
  - Veneer stability

- Specific failure mechanisms for reinforced fill or soil nailed structures:
  - Geotechnical resistance entirely outside the structure – External stability – reference to other Clauses (4, 5, 7)
  - Geotechnical resistance partly outside partly inside the structure – Compound stability – reference mainly to Clause 4
  - Structural resistance (internal stability) : tensile and shear resistances of reinforcing elements (structural and geotechnical), resistances of connections, resistances of facing elements – generally covered internally in Clause 9

- Design resistance (strength) of reinforcing elements
  - Geosynthetics
    - material Eurocode does not exist
    - ISO TR 20432
    - National guidance / codes
    - ISO TR + EBGEO selected as basis for EC7
  - Steel for fill applications
    - EC 3 as material Eurocode exists
    - Durability for tension elements not covered by EC3
    - Specific research data available for tension steel fill reinforcing elements
    - How to combine EC3 and specific data – task for this meeting
  - Soil nails
    - Steel tendons without grout cover – similar issues as above
    - Steel tendons with grout cover – approach similar to anchors / tension piles assumed
    - Non steel tendons – not enough data available – allowed only after specific study

- Factoring approaches:
  - For external stability
    - Reference to clauses 4 and 5 / 7 are made
  - For compound stability
    - Reference to clause 4 is applied
  - For internal stability
    - Both MFA and RFA are allowed
  - For reinforcing materials
    - The available strength in element is reduced by material resistance factor
    - Material resistance factor varies according to MFA or RFA approach used during the analysis – point for discussion during this meeting
    - Calibration exercises are still under way to confirm the actual values
  - For ground × reinforcing element interface
    - Strength / resistance of the interface can be determined by both testing and calculation
    - Both MFA and RFA are allowed for calculation determination
    - RFA is used for testing determination
    - Calibration exercises are still under way to confirm the actual values of partial factors

EC7-Part 3 from a Dutch perspective

Jan van Dalen

KIVI Engineering Society

NEN

EC7- 3 from a dutch perspective
Retaining walls, anchors

Jan van Dalen

Gemeente Rotterdam TUDelft Delft University of Technology DAED INGENIEURS

Contents

Retaining walls

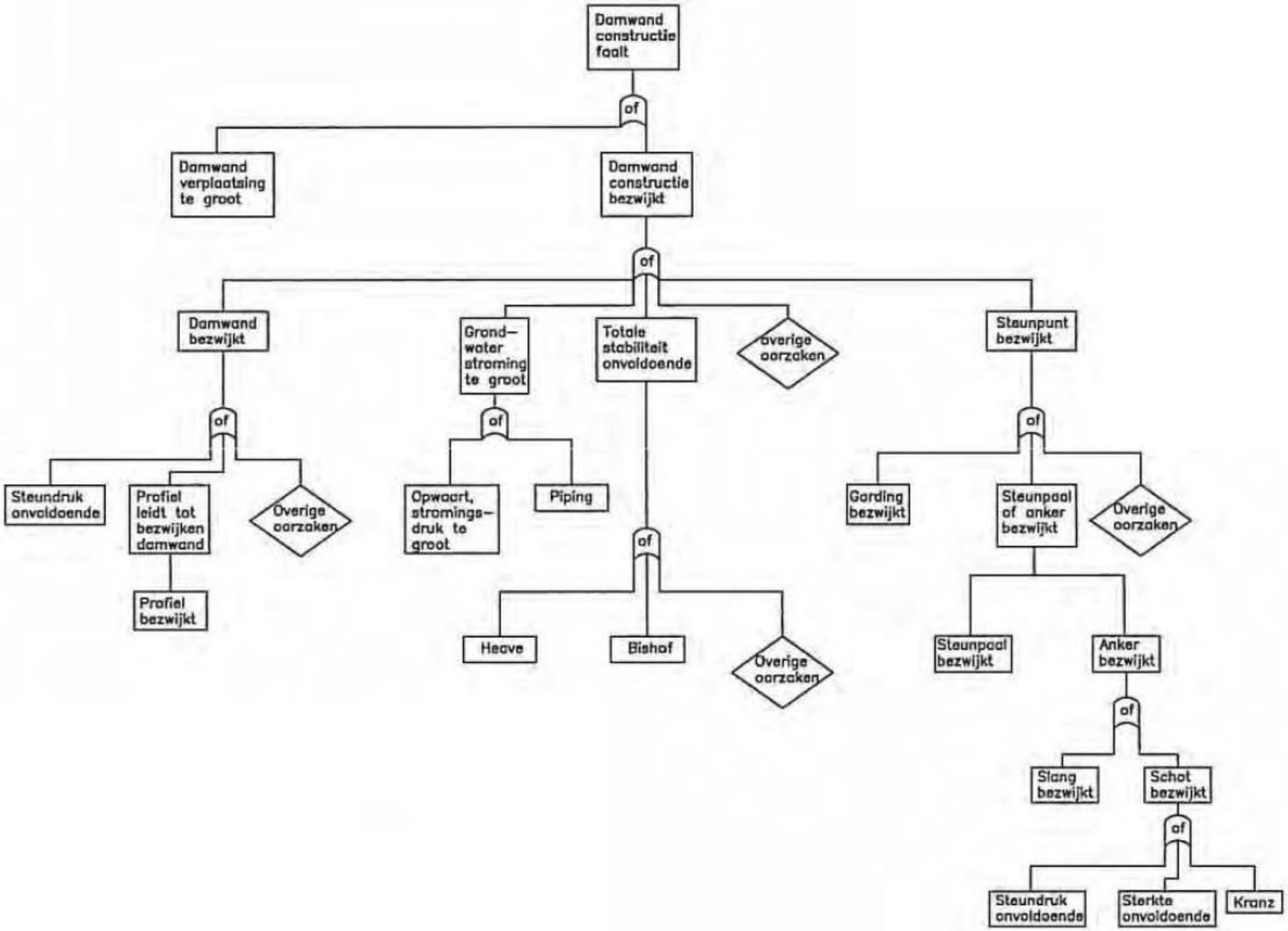
- Dutch approach of the safety factors and design calculations
- Method of determination of passive effective stresses
- Diaphragm walls

Anchors and corrosion

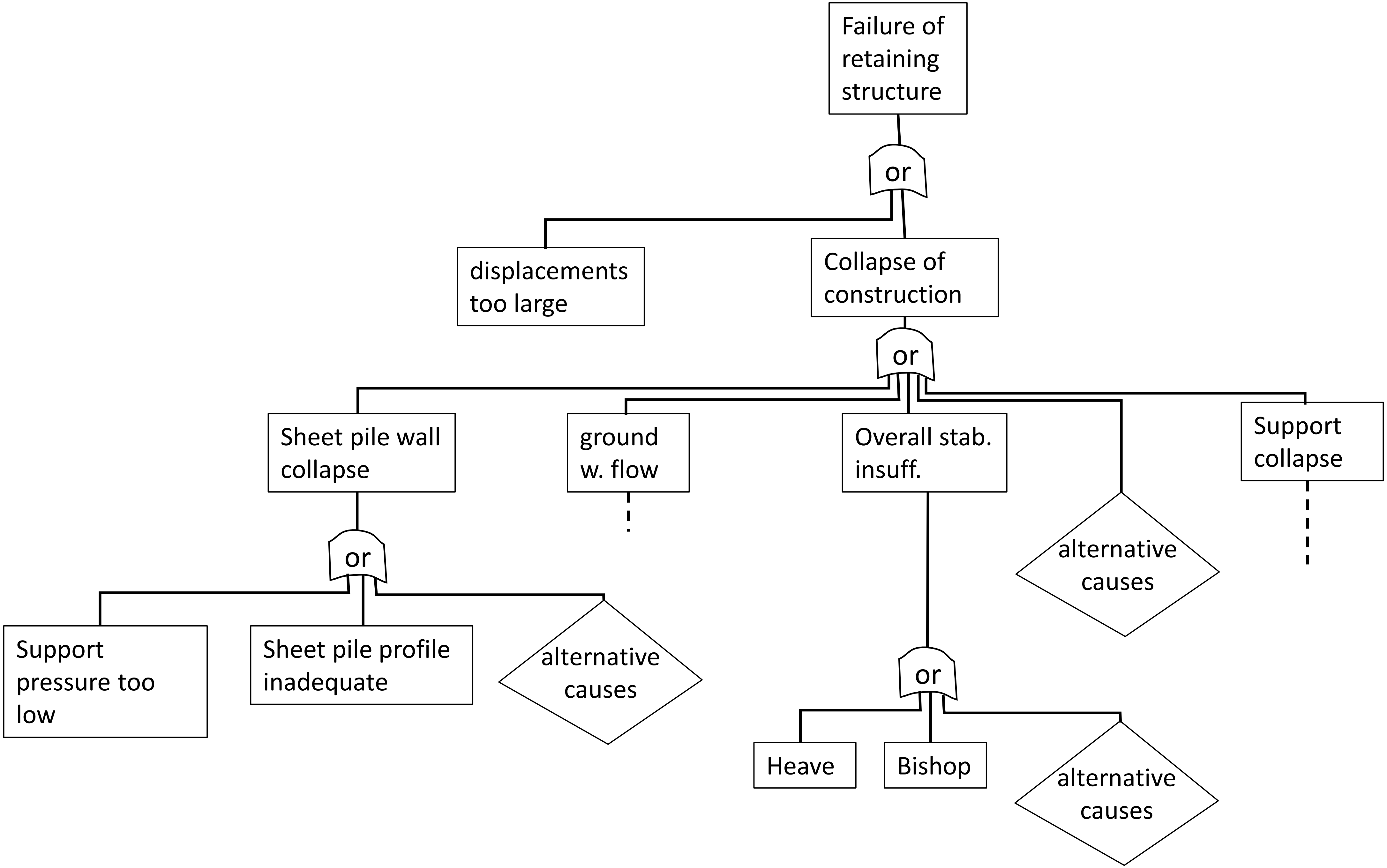
- Dealing with anchor types used in the Netherlands

Dutch approach to retaining wall calculations

- ULS with design parameters; material factors based on fault-tree and reliability analyses



Damwand constructie faalt
of
Damwand verplaatsing te groot
Damwand constructie bezwijkt
of
Damwand bezwijkt
Grond-water stroming te groot
Totale stabiliteit onvoldoende
overige oorzaken
Steunpunt bezwijkt
of
Steundruk onvoldoende
Profiel leidt tot bezwijken damwand
Overige oorzaken
Profiel bezwijkt
of
Opwaart, stromings-druk te groot
Piping
of
Heave
Bishof
Overige oorzaken
of
Gording bezwijkt
Steunpaal of anker bezwijkt
Overige oorzaken
Steunpaal bezwijkt
Anker bezwijkt
of
Slang bezwijkt
Schot bezwijkt
of
Steundruk onvoldoende
Sterkte onvoldoende
Kranz


Failure of retaining structure
or
displacements too large
Collapse of construction
or
Sheet pile wall collapse
ground w. flow
Overall stab. insuff.
Support collapse
alternative causes
or
Support pressure too low
Sheet pile profile inadequate
alternative causes
or
Heave
Bishop
alternative causes

Determination of partial factors

- Factors on:
  - Strength parameters of soil
  - Loads
  - Prescribed variations in surface levels

- Factors determined by
  - Based on Monte Carlo simulations on different types of constructions
  - Determination of influence factors per parameter

Dutch approach to retaining wall calculations

Some parameters can have a positive or a negative effect:

- Groundwater level at the passive side
- Horizontal Stiffness of the soil, prior to active or passive situation (subgrade reaction coeff.)

Dutch ULS check with sensitivity analyses

Calculation nr	Limit state	Subgrade react. coeff.	Design value freatic level on the passive side
1 [b]	ULS	Low	High $\max(\mu + \gamma \times \sigma \,;\, \mu + \Delta)$
2 [b c]	ULS	High	High $\max(\mu + \gamma \times \sigma \,;\, \mu + \Delta)$
3	ULS	Low	Low $\min(\mu - \gamma \times \sigma \,;\, \mu - \Delta)$
4 [c]	ULS	High	Low $\min(\mu - \gamma \times \sigma \,;\, \mu - \Delta)$

Dutch ULS check with sensitivity analyses

Calculation nr	Limit state	Subgrade react. coeff.	Design value freatic level on the passive side
1 [b]	ULS	Low	High $\max(\mu + \gamma \times \sigma ; \mu + \Delta)$
2 [b c]	ULS	High	High $\max(\mu + \gamma \times \sigma ; \mu + \Delta)$
3	ULS	Low	Low $\min(\mu - \gamma \times \sigma ; \mu - \Delta)$
4 [c]	ULS	High	Low $\min(\mu - \gamma \times \sigma ; \mu - \Delta)$
5 [d]	SLS	Low	Low Characteristic value

Factors, to be applied on the results of nr. 5 (SLS calculation):

RC1, RC2	$M_{s;d} = 1{,}2 \times M_{s;k}$	$D_{s;d} = 1{,}2 \times D_{s;k}$	$P_{max} = 1{,}2 \times P_k$
RC3	$M_{s;d} = 1{,}35 \times M_{s;k}$	$D_{s;d} = 1{,}35 \times D_{s;k}$	$P_{max} = 1{,}35 \times P_k$

Subgrade reaction coefficient

Text in EC:

D.7 Beam-on-spring models

(1) <PER> Intermediate values of earth pressures may be calculated by use of the subgrade reaction coefficient, $k = \Delta\sigma / \Delta y$, where $\Delta\sigma$ is the variation of earth pressure associated with a variation of horizontal wall displacement Δy.

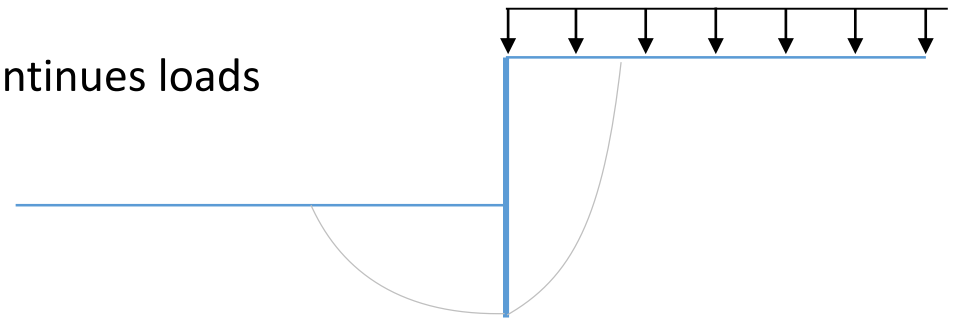
NOTE 1. This is a simplification that assimilates the ground to independent springs.

NOTE 2. Due to its empirical nature, assessed values of the coefficient of subgrade reaction should always be derived from comparable experience in similar conditions. Guidance is provided in Annex D.8.

Horizontal effective stresses
(Beam on springs and analytical models)

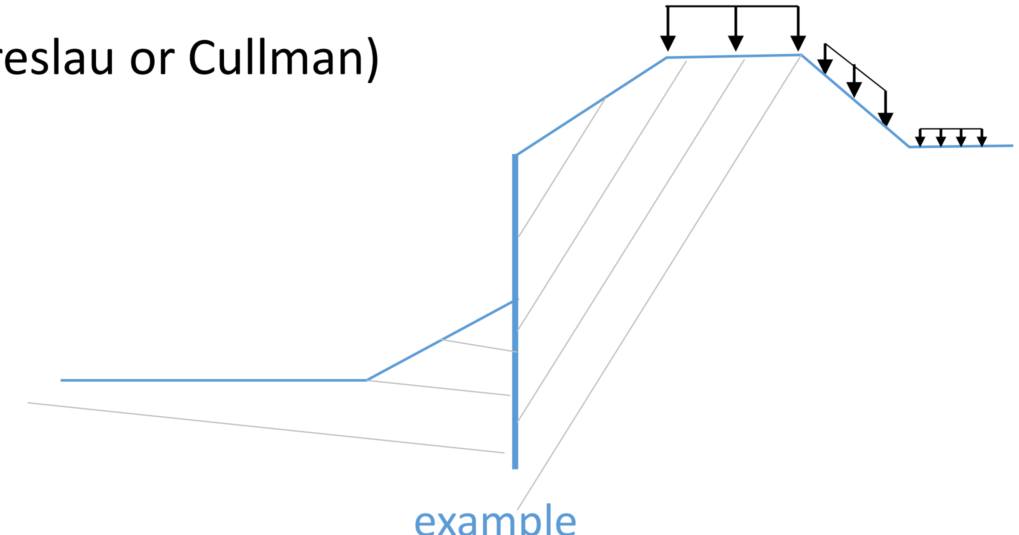
Dutch method, 2 alternatives:

- Curved slip surfaces, (most used method: Kötter)
  - Used in most cases with straight ground level and continues loads



example

- Straight slip surfaces: (Most used method: Müller-Breslau or Cullman)
  - Mostly used for irregular ground levels and/or loads



example

Literature hor. soil pressures

Culmann, K., 1866. “Die Graphische Statik.” Zürich.

Kötter, F., 1903. “Die Bestimmung des Druckes an gekrümmten Gleitflächen.” Sitzungsbericht Kön. Preu. Ak. d. Wissenschaften, Berlin.

Müller-Breslau, H., 1906. “Erddruck auf Stützmauern.” Verlag Kröner, Stuttgart.

Current NAD, Earth pressures and wall friction

Naming of the wall roughness	Definition of the wall roughness	Wall friction (δ)	
		Straight slip surface	Curved slip surface
Toothed	$> 10\ d_{50}$	$0,67\ \varphi'_k$	$\leq \varphi'_k$
Coarse	$0,5\ d_{50} – 10\ d_{50}$	$0,67\ \varphi'_k$	$\leq \varphi'_k – 2.5^o$ with a maximum of 27.5^o
Half coarse	$0,1\ d_{50} – 0,5\ d_{50}$	$0,33\ \varphi'_k$	$0,5\ \varphi'_k$
Smooth	$< 0,1\ d_{50}$	$0°$	$0°$

Current NAD, Earth pressures and wall friction

Interpretation in case of steel, prefab concrete, plastic or wood

	Naming of the wall roughness	Definition of the wall roughness	Wall friction (δ)	
			Straight slip surface	Curved slip surface
	Toothed	$> 10\ d_{50}$	$0{,}67\ \varphi'_k$	$\leq \varphi'_k$
$d_{50} < 2$mm	Coarse	$0{,}5\ d_{50} - 10\ d_{50}$	$0{,}67\ \varphi'_k$	$\leq \varphi'_k - 2.5^{o}$ with a maximum of 27.5^{o}
$2 \leq d_{50} \leq 8$mm	Half coarse	$0{,}1\ d_{50} - 0{,}5\ d_{50}$	$0{,}33\ \varphi'_k$	$0{,}5\ \varphi'_k$
$d_{50} > 8$mm	Smooth	$< 0{,}1\ d_{50}$	$0°$	$0°$

Based on experience by Rijkswaterstaat

Earth pressures and wall friction

7.5 Geotechnical analysis

7.5.1 Determination of earth pressures

(5) <REQ> The amount of shear stress that can be mobilised at the interface between the ground and the structure shall be determined by the ground/structure interface coefficient (tan δ), where δ is the inclination of stresses applied to the interface.

(6) <REQ> The value of the ground/structure interface coefficient (tan δ) shall satisfy Formula 7.1:

$$\tan\delta \leq k_{\delta} \times \tan\varphi \tag{7.1}$$

where:

tan φ is the value of the soil's coefficient of internal friction;

k_{δ} is a constant depending on the roughness of the ground structure interface and local disturbance during execution.

(7) <REQ> The value of k_{δ} shall not exceed 1.0.

(8) <PER> A value of k_{δ} = 1,0 may be assumed for concrete cast directly against soil and for stone infill or backfill used for crib walls and gabions.

(9) <RCM> The value of k_{δ} should not exceed 2/3 for retaining structures formed with smooth surfaces.

Earth pressures and wall friction

7.5.4 Values of passive earth pressure

(5) <REQ> If limiting values of passive earth pressure are calculated by assuming planar failure surfaces, the ground/structure interface coefficient (tan δ) in Formula (7.1) shall be taken as 0.

In our view too conservative for a requirement!

Wall friction Diaphragm walls

7.5 Geotechnical analysis

7.5.1 Determination of earth pressures

(5) <REQ> The amount of shear stress that can be mobilised at the interface between the ground and the structure shall be determined by the ground/structure interface coefficient (tan δ), where δ is the inclination of stresses applied to the interface.

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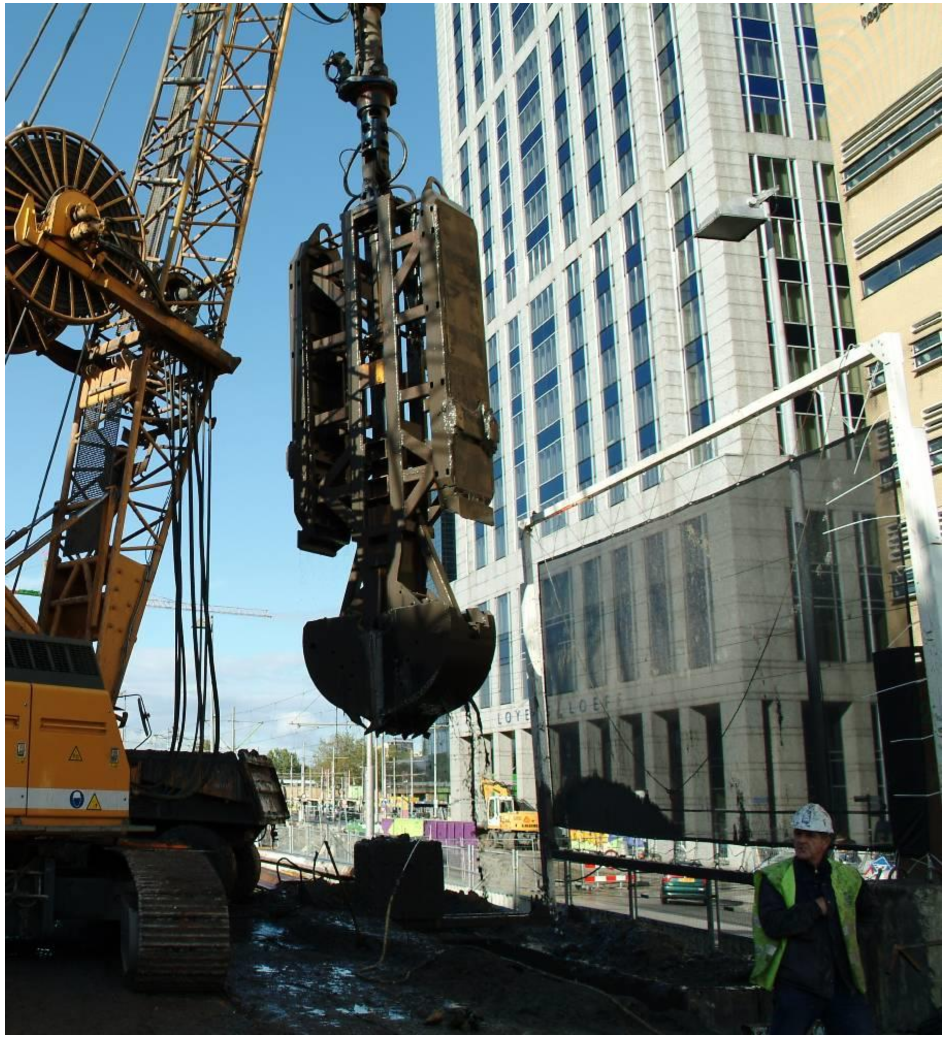
Installation of Diaphragm walls

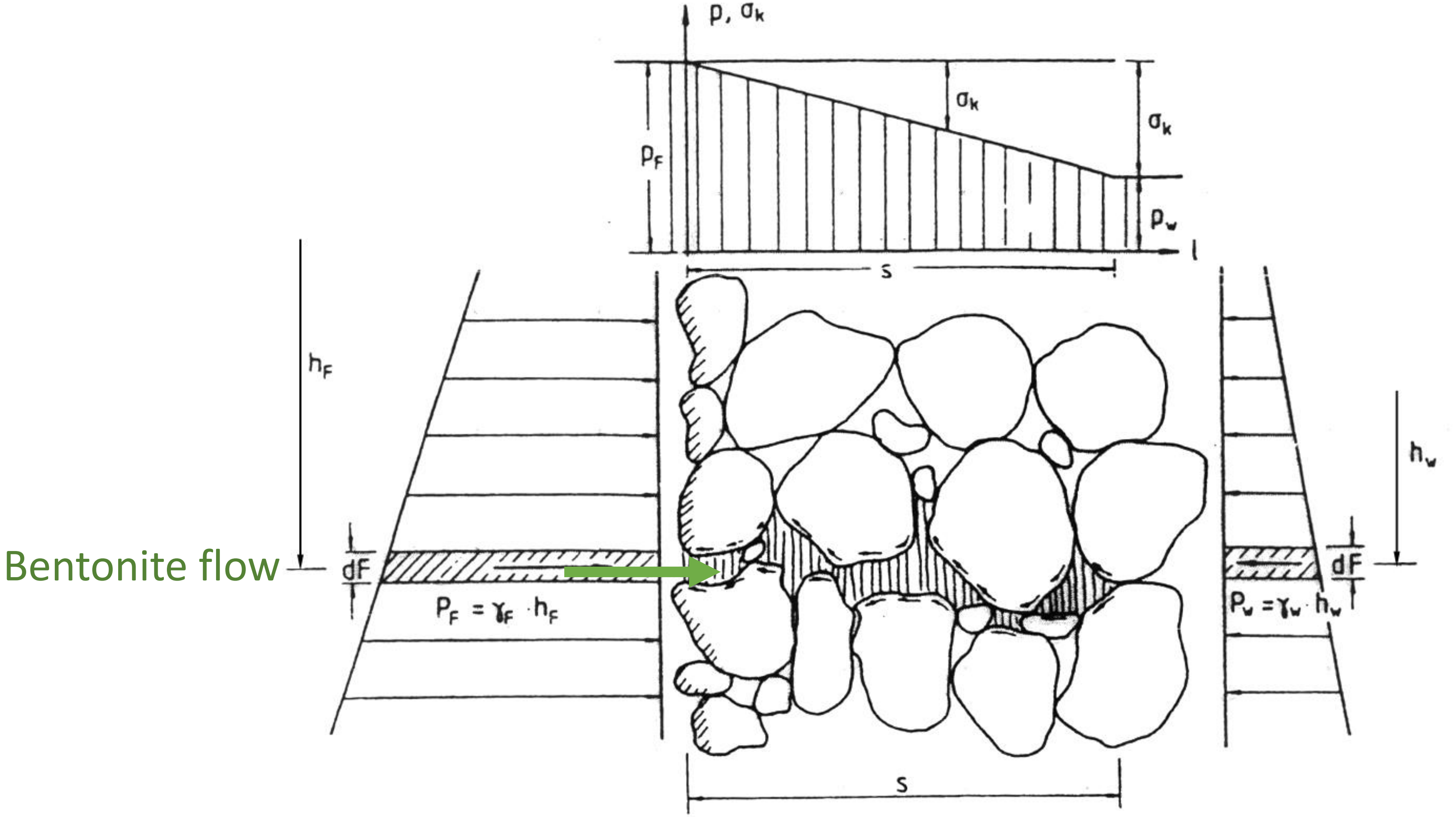
Use of support fluid



Bentonite in the excavated trench

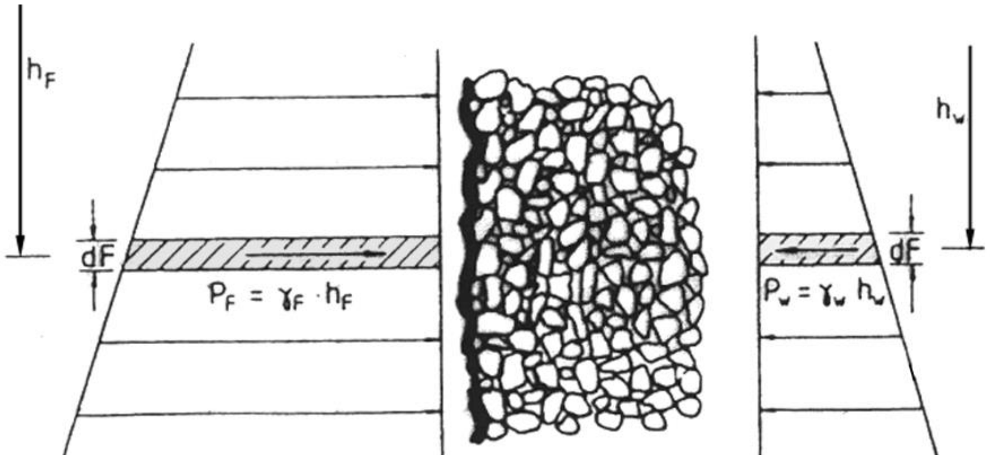
Trench stability: Microstability

filtercake in case of coarse material:

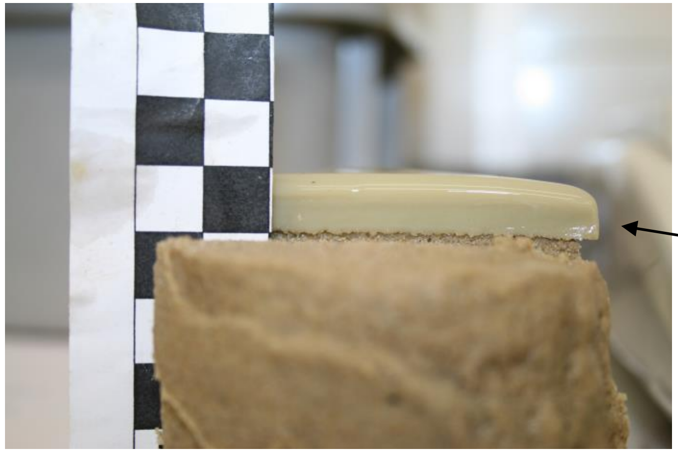


a. Detail korrelstructuur met verloop vloeistofspanningen.

Trench stability: Micro stability



forming of filtercake in case of sand ($d_{10} < 0.2$mm)



Filtercake forming under laboratory circumstances

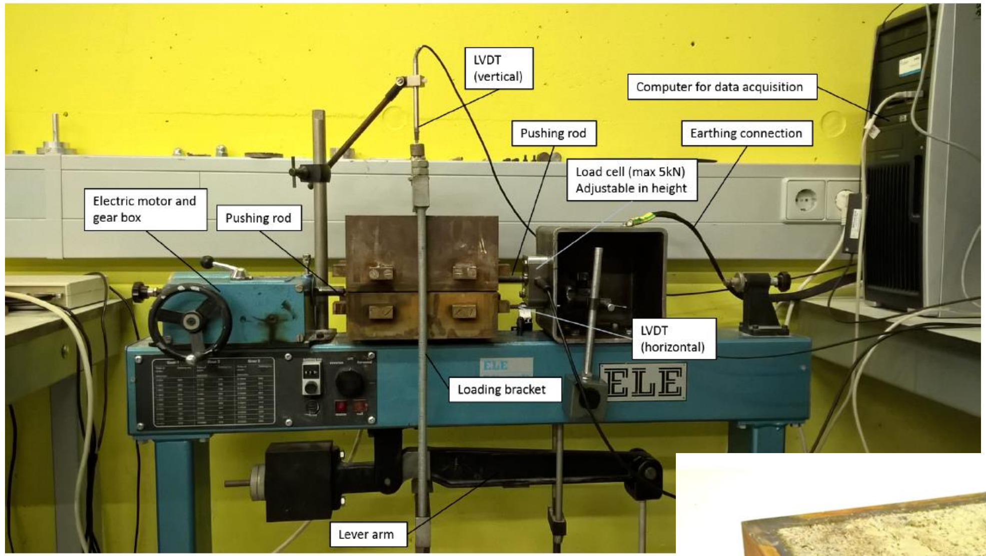
Filtercake, as peeled from a D-wall

MSc. Thesis: SKIN FRICTION OF DIAPHRAGM WALLS

Jip de Wolf

Soil/D-wall interface: layered system

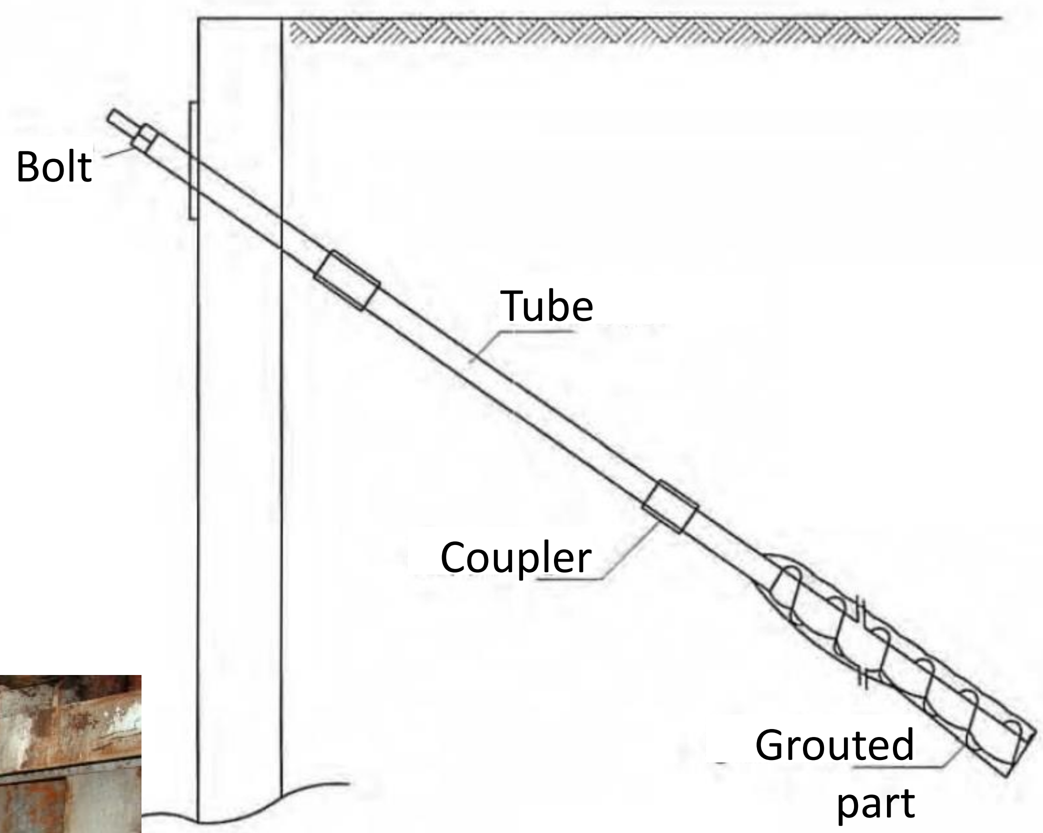
Experimental study: Direct-shear tests on sand/filter cake/concrete samples



Conclusions for Diaphragm walls in sand
(d_{10} < 0.2mm)

- Curved slip surfaces: $\delta'_k = \varphi'_k$ with a maximum of 20º
- Straight slip surfaces: $\delta'_k = 2/3\ \varphi'_k$ with a maximum of 13.3º

Anchors and corrosion



Anchors

8.3 Materials

(4) <REQ> For grouted anchors, corrosion protection shall be in accordance with EN 1537.

EN 1537 focusses on grouted anchors with high tensile strength:

- For permanent anchors, full corrosion protection is required
- This is needed if steel vulnerable to forms of pit corrosion => brittle failure

Anchors of steel with lower tensile strength ($f_y \leq 500$ to $600\ N/mm^2$)

- In the Netherlands those anchors are often used
- These steels are not vulnerable for forms of pit corrosion, only general corrosion
- For these anchors a corrosion rate / residual cross section end of life span is formulated

Concluding statements for discussion

Retaining walls

- Sensitivity analyses should be required
  - for groundwater level at the passive side
  - for subgrade reaction coefficients
- $\delta = 0$ for all methods with planar failure surfaces is too conservative
- For Diaphragm walls in sand $\delta = \varphi$ is too optimistic

Anchors

- Demanding EN1537 corrosion protection for all grouted anchors is too conservative and rules out many used anchors in the Netherlands!

Pile Load Testing in The Netherlands. What happens in practice?

Bartho Admiraal

KIVI Engineering Society

Volker Staal en Funderingen

Pile load testing: Dutch practice

Seminar New Eurocode

Ir. Bartho Admiraal

Volker Staal en Funderingen

NVAF 1947 70 JAAR 2017

EFFC

Comparison Event: EC 7 Calculations

DEJÀ VU

10th International Conference on

Piling and Deep Foundations

Amsterdam, 31st May – 2nd June 2006

Session Comparison Event: EC 7 Calculations

Maurice Bottiau,
Group Commercial Director, Franki Geotechnics B

Wolf-R. Linder and Björn Böhle
Brückner Grundbau, Essen
Chairman and member EFFC, Technical Working Group

Frits van Tol
Techn. University of Delft, GeoDelft

10th Int'l Conf. Piling & Deep Foundations: Amsterdam
Session: Comparison Event: EC 7 Calculations

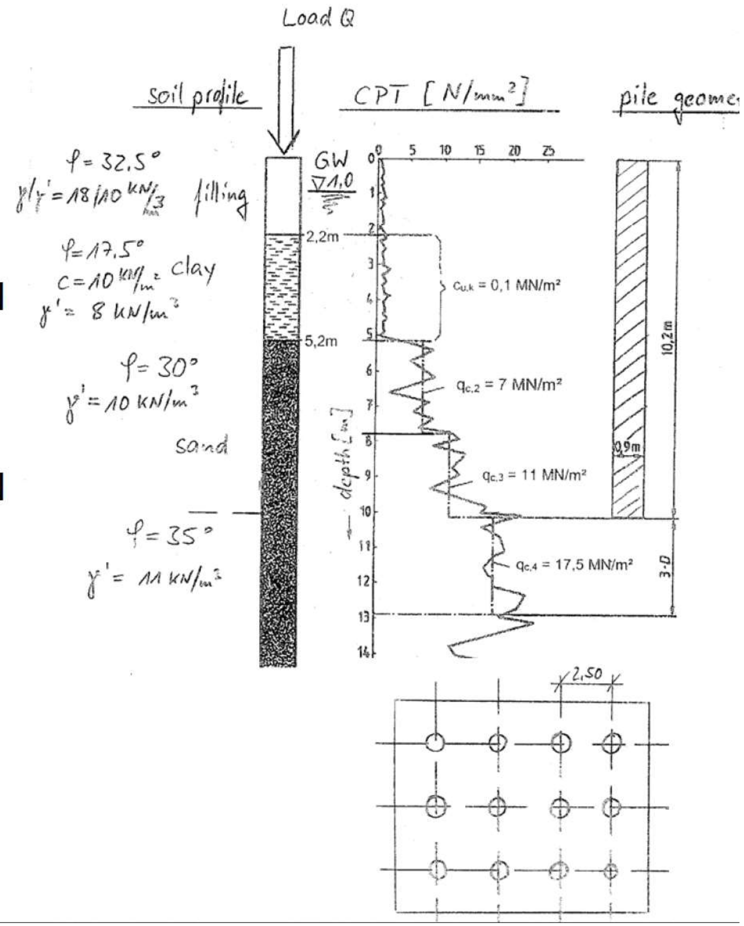
Example 1: bored pile / pile group

single pile capacity (ULS):

min/max: Q = 667 ... 4 182 kN

capacity of group (12 piles, ULS):

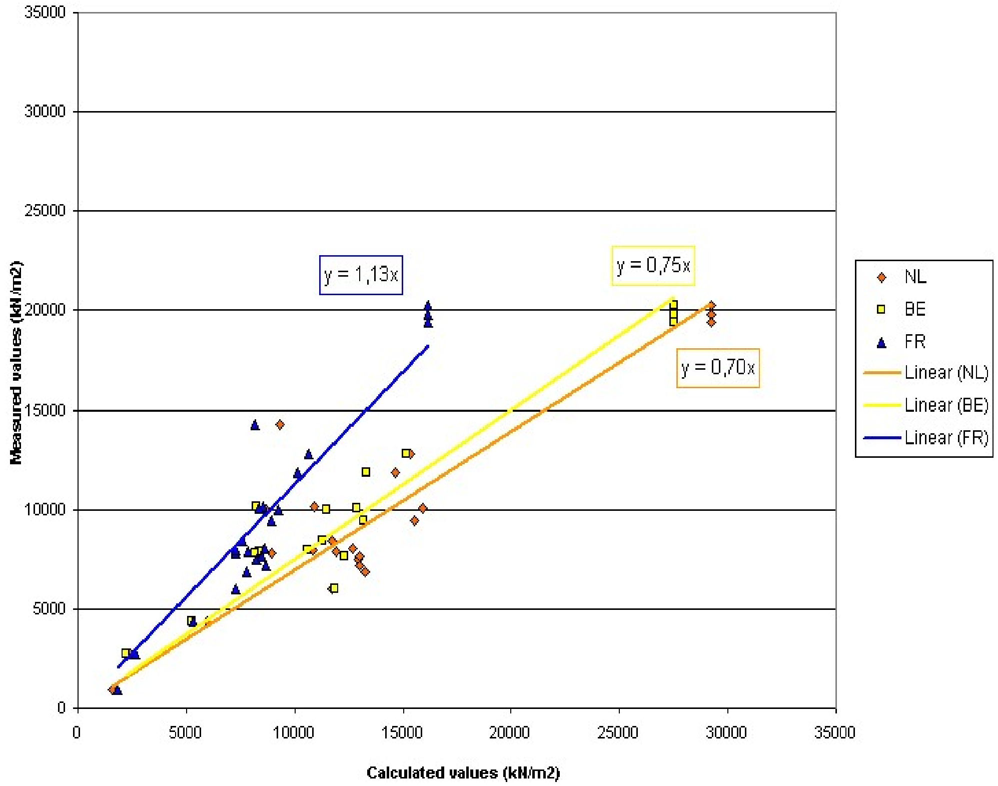
min/max: Q = 8004 ... 48 553 kN



But times have changed

- NEN-EN 1997-1+C1+A1:2016 => NEN 9997-1+C2: 2017
- α_p -/- 30%
  - ➢ more / larger piles
  - ➢ **+** installation energy
  - ➢ **-** feasability
- NPR 7201: 2017
  - Pile testing culture for α_p >?

CUR-report 229 –
“Axial bearing capacity of piles”



NPR- Axial pile load tests

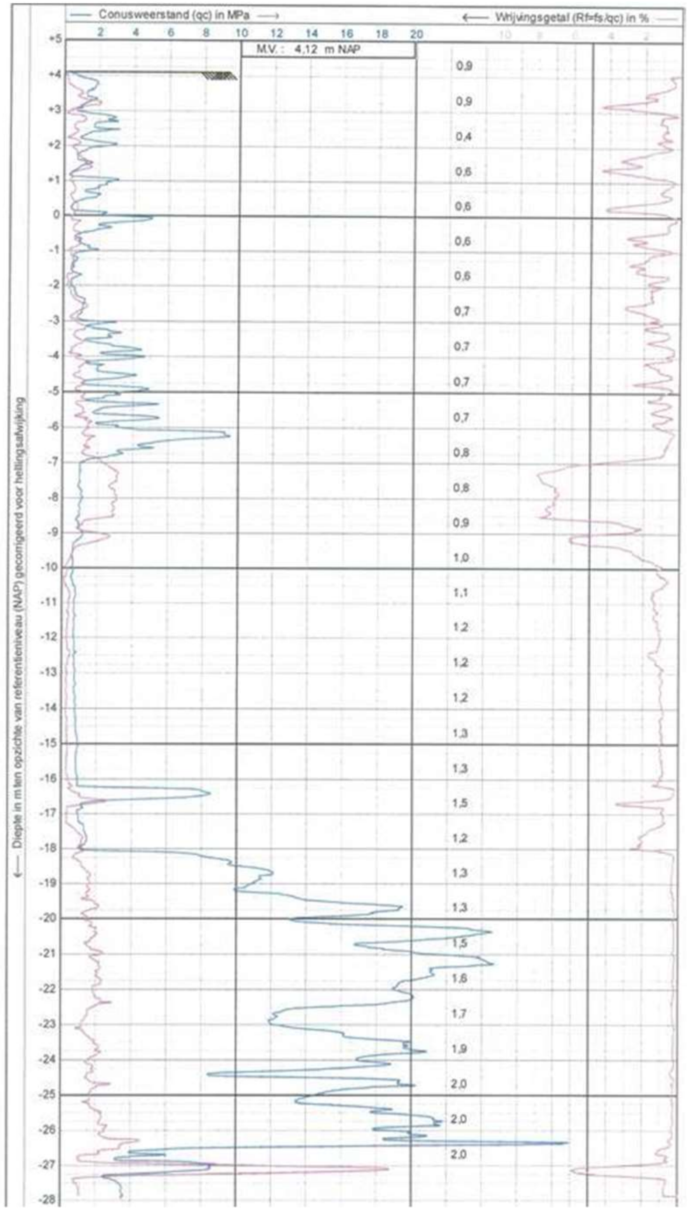
4 Classes

Class	Testload	Method	Measurement	Result
A1	Failure	SLT + sensors	Separate shaft / tip	α-factors (NL or local)
A2	Failure	SLT / RLT + sensors	Separate shaft / tip	Pile capacity project
B	Failure	SLT / RLT	Displacement top	Pile capacity project
C	1,37 – 1,67 F_d	SLT / RLT	Displacement top	Acceptance pile design
D	1,0 F_d	SLT / RLT	Displacement top	Acceptance specific pile

….3 years later

- Pile tests at projects
- National α_s, α_p en α_t -factors
- R&D

Projects: Port of Rotterdam



Motivation

- New set of pile base factors results significant increase in project costs.
- No foundation pile failure are known in Rotterdam
- Much higher risk of damage during installation

Scope

- 4 precast concrete piles # 450x450
- Steel test frame, 8 grout anchors
- Test load 9,000 kN

Full scale field test 2017 Waalhaven Rotterdam

Project costs

- €450,000

Results

- Failure load: 6,500 kN
- 25% less foundation piles

Benefits

- 5% reduction of project costs
- ≈ €550,000

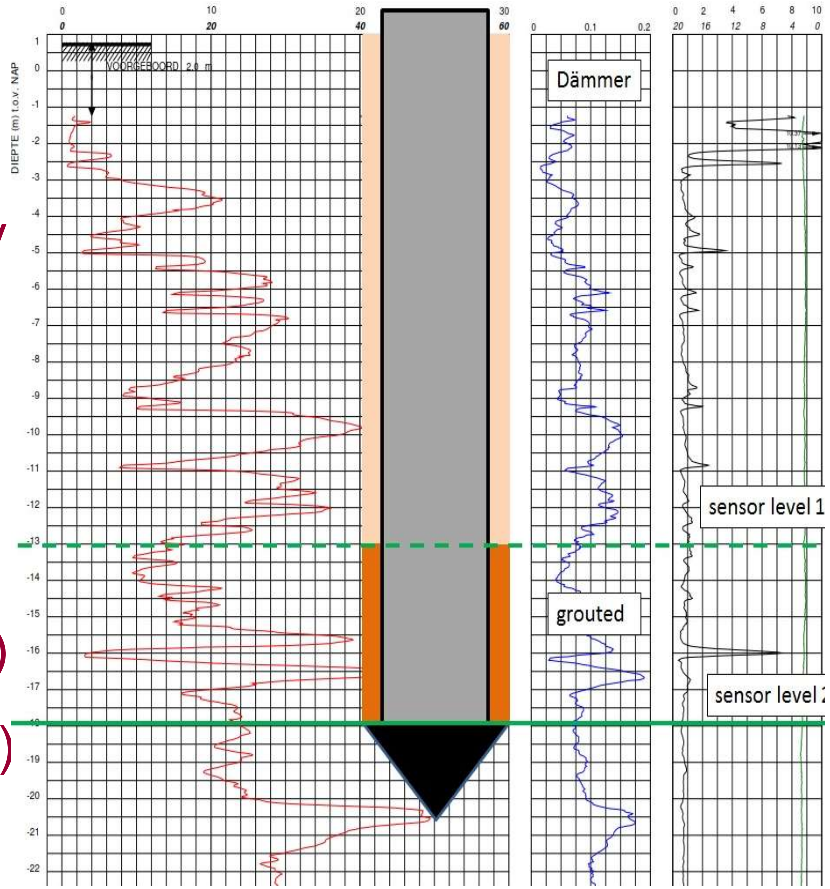
Projects: van Rossum, Crux, Van 't Hek

Motivation

- High resistance sand layer below NAP -/- 25m
- Risk of low production and so delay on schedule
- Much higher risk of non withdrawal of casings

Scope

- 1 suitability pile SI Ø 762/950 (DDI)
- 3 load test piles SI Ø 609/850 (DDI)



Full scale field test 2018
Sonate The Hague

RLT: StatRapid, 40 ton

Results

- Failure load: 7,000 kN
- α_s = 0.009, α_p = 0.84 [x q_c]
- PTL NAP -/- 25m
- 10% less foundation piles

National α_s, α_p: contractors

Situation

Motivation

- Procedures
  - Effect of soil conditions
  - Method Statement documented
  - α-factors **only** valid for concerning supplier

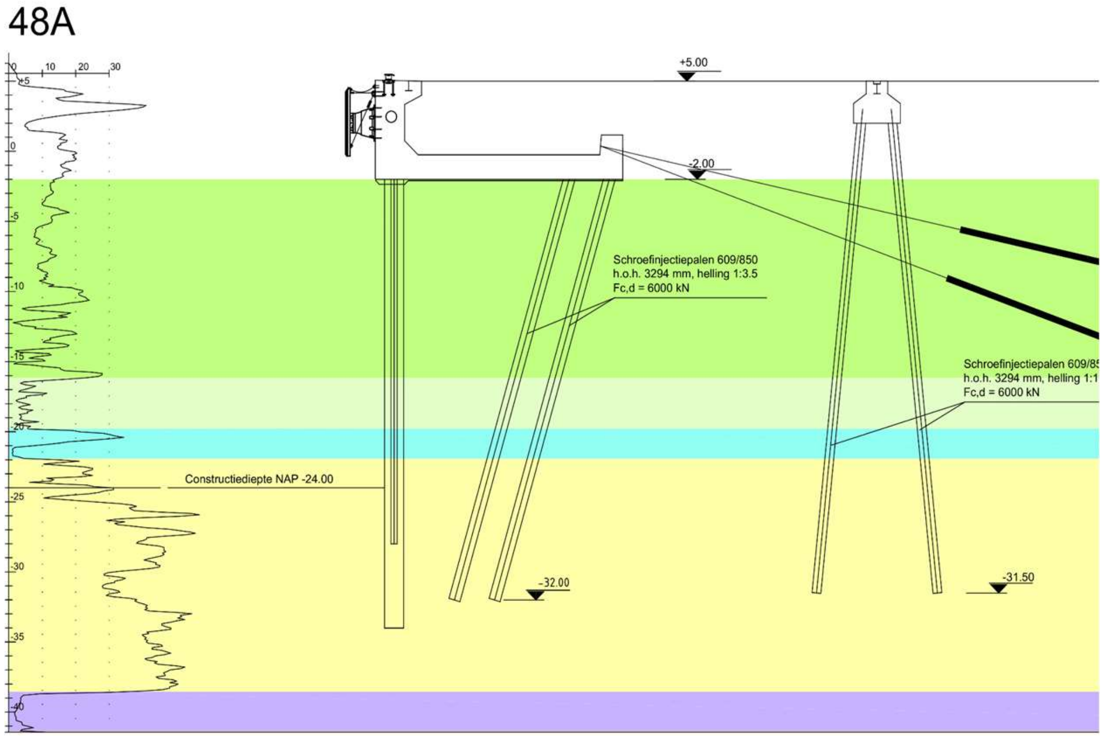
- Costs

National α_s, α_p: Port of Rotterdam

Motivation

- Reduce installation risks
- Optimize project costs (change SI-piles into vibro piles??)
- CO_2 reduction
- Derive appropriate values for α_p and α_s and limiting value=> A1 test.
- Verify existing design methods (Koppejan)

3 km new deep sea quay wall



Full scale field test 2019/2020 MV2, Rotterdam

Scope

- 4 precast concrete piles # 400x400.
- 4 SI piles Ø 609/850
- 4 Vibro piles Ø 356/480
- Spider-shaped test frame
- 100 SI-anchors.

- Project costs ≈ €2,500,000

⇒ Test load **25,000 kN** !!!

⇒ Planning: Nov 2019 – Jan 2020

R&D

TU Delft: Prof. Dr. Ken Gavin

- Installation effects on σ'_{gr}
  => α_p
- Installation effects on α_s
  => aging / set up
- Length / depth effects on α_s

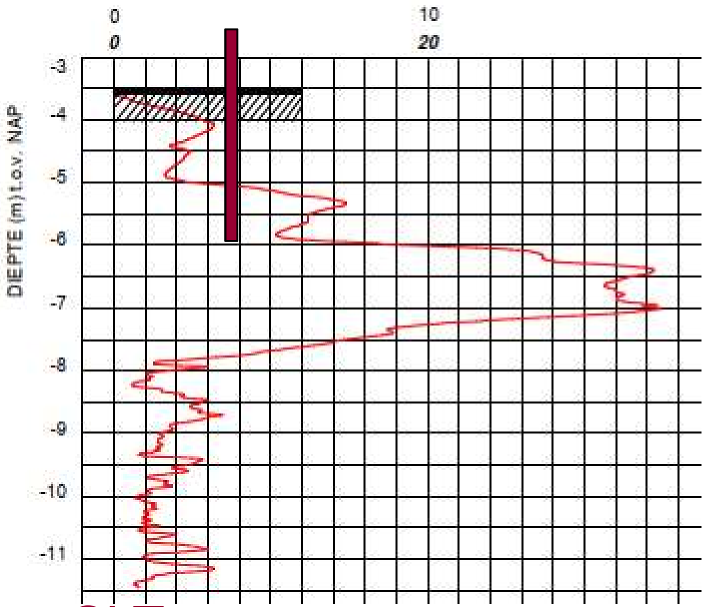
NVAF

- Installation variables SI-piles

R&D: NVAF

Installation variables

- Shape pile tip
  - Flat tip
  - Conical tip



- SLT

Volker Staal en Funderingen

R&D: NVAF

Results

- **SI piles**

Pile	Tip shape	α_p	α_s
3	flat	0.31	0.007
7	flat	0.35	0.008
	mean	**0.33**	**0.008**
2	conical	0.40	0.008
6	conical	0.39	0.008
	mean	**0.40**	**0.008**
	Δ	17.5 %	0 %

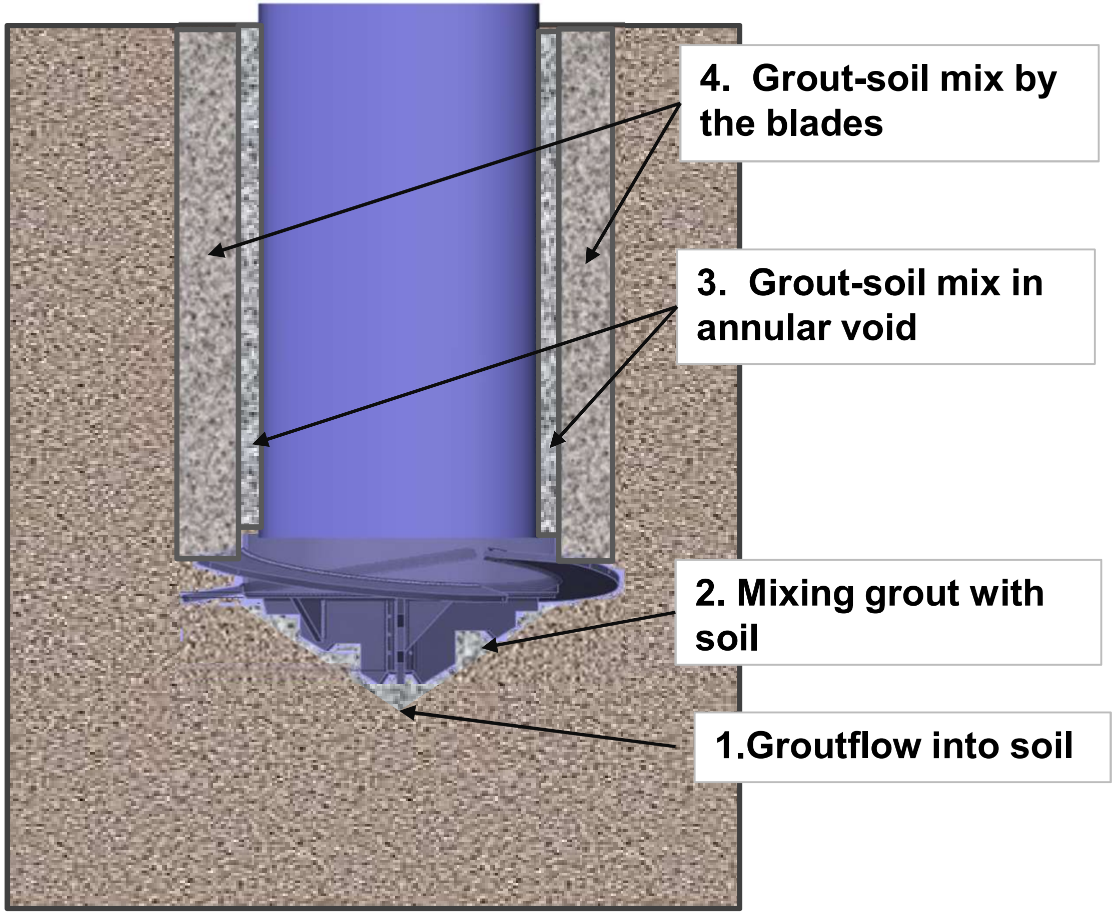
- **Driven piles**

Pile	Tip shape	α_p	α_s
1	flat	0.48	0.007

R&D: NVAF

Installation variables

- Shape pile tip
- Grouting parameters
  - Flow [l/min]
  - W.B.R.
  - Tip grouting



Pile load testing: Dutch practice

Hopeful development

- Contractors
- Owners
- Science
- NEN

It is not about nescessity of testing, but about benefit of testing

Closing & drinks

Dank voor uw aanwezigheid!