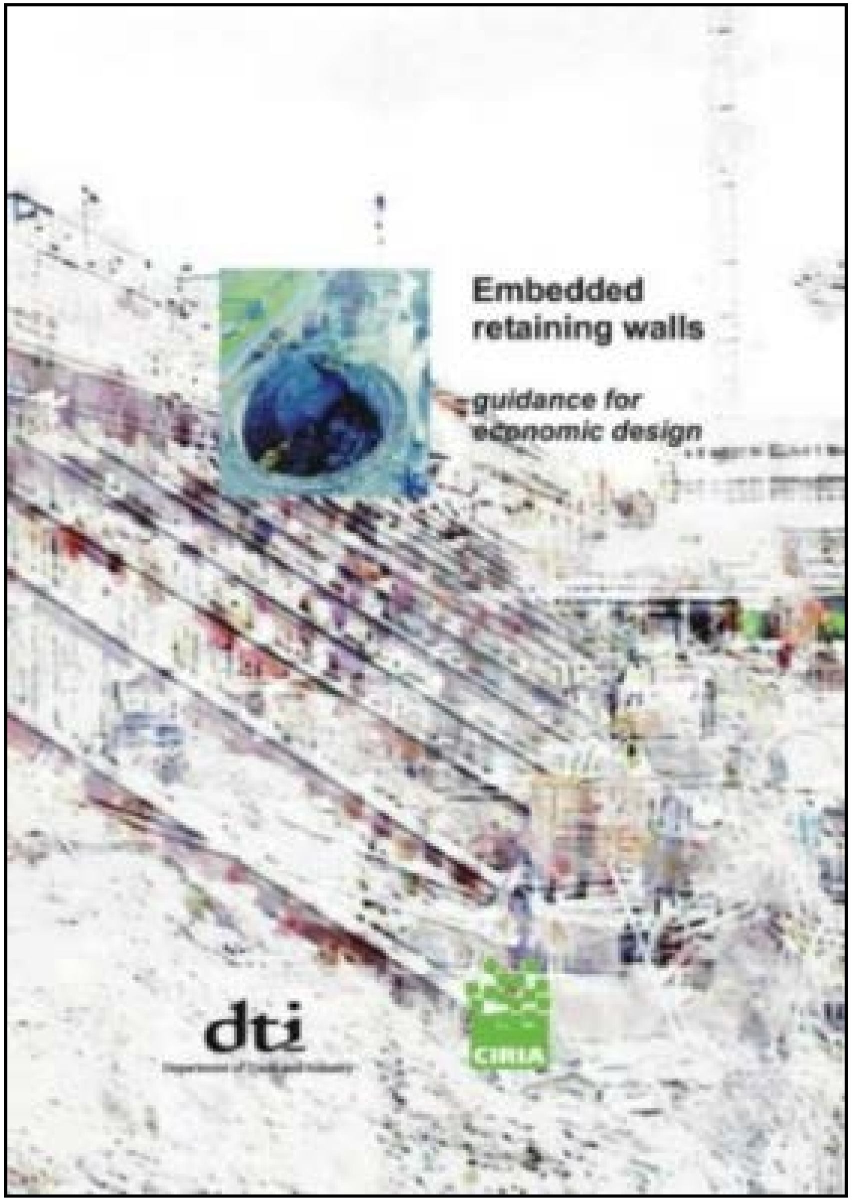
C760
Embedded Retaining walls:
guidance for design

INTRODUCTION



Embedded retaining walls

guidance for economic design

dti

CIRIA

Presentation Outline

- **Background to update**

- **Feedback on C580 publication**
  - Industry consultation
  - Where C580 is used

- **The C580 Update**
  - Scope of C760
  - Research team & Project Steering Group
  - Project status and timeline

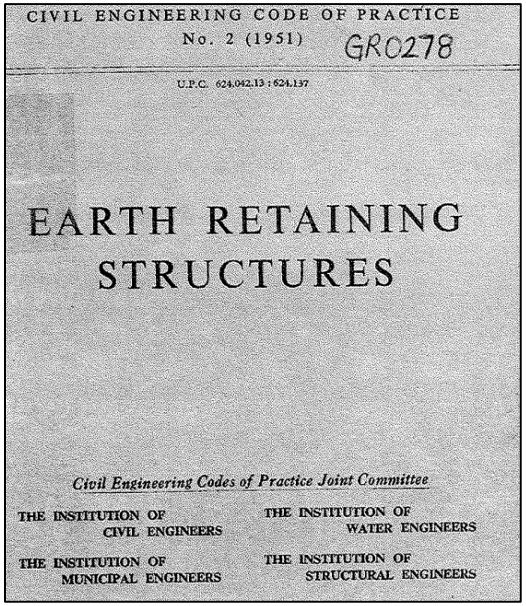
- **Fundamental considerations**
  - Key issues
  - Selection of φ’
  - How codes of practice achieve safety
  - Role of statistical analysis

- **Closing comments**

Background to the update

CP 2

- Published in 1951
- Issued by the Civil Engineering Codes of Practice Joint Committee



CIVIL ENGINEERING CODE OF PRACTICE
No. 2 (1951) GR0278

U.P.C. 624.042.13 : 624.137

EARTH RETAINING STRUCTURES

Civil Engineering Codes of Practice Joint Committee

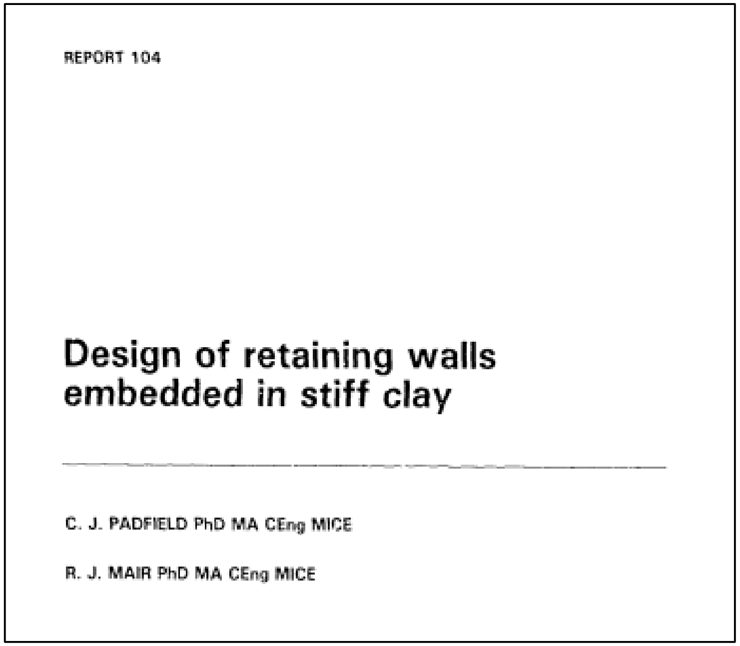
THE INSTITUTION OF CIVIL ENGINEERS
THE INSTITUTION OF WATER ENGINEERS
THE INSTITUTION OF MUNICIPAL ENGINEERS
THE INSTITUTION OF STRUCTURAL ENGINEERS

CIRIA Report R104 (Padfield & Mair, 1984)

Design of retaining walls embedded in stiff clay

Applicable to cantilever and singly propped walls embedded in stiff clay

- Highly influential
- Funded by Department of Environment
- Range of approaches presented
- Essentially targeting London Clay



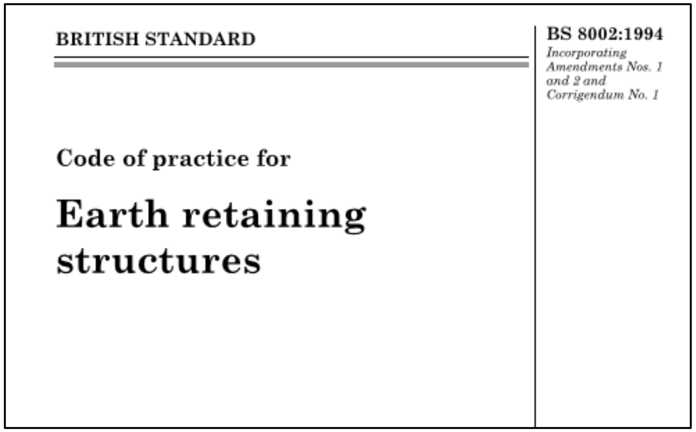
REPORT 104

Design of retaining walls embedded in stiff clay

C. J. PADFIELD PhD MA CEng MICE

R. J. MAIR PhD MA CEng MICE

Background to the update



BRITISH STANDARD

BS 8002:1994

Incorporating Amendments Nos. 1 and 2 and Corrigendum No. 1

Code of practice for

Earth retaining structures

BS 8002

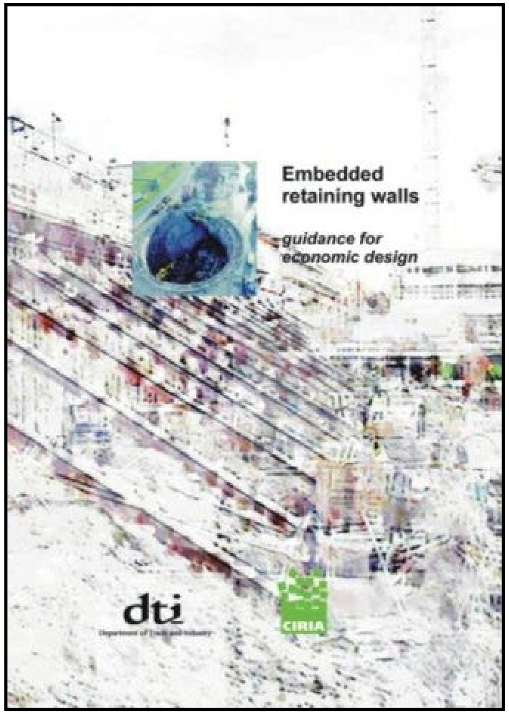
- Published in 1994
- Useful information (for example on φ’)
- Two checks

CIRIA C580 (Gaba, Simpson, Powrie & Beadman, 2003)
Embedded retaining walls – guidance for economic design

Applicable to cantilever and multi-propped walls embedded in stiff clay and other competent soils

- Hugely influential
- Funded by Department of Trade and Industry
- Three approaches A, B and C

<u>CIRIA’s best selling design guide</u>



Embedded retaining walls

guidance for economic design

Background to the update

EC7 (BS EN 1997-1:2004)

- Eurocode 7: Geotechnical design
- Replaced BS 8002 (1994)
- Three design approaches
- Approach 1 adopted in UK: two checks Combinations 1 and 2

BRITISH STANDARD

BS EN 1997-1:2004
Incorporating corrigendum February 2009

Eurocode 7: Geotechnical design —

Part 1: General rules

BS 8002

- Updated in 2015 for EC7 compatibility
- Funded by High Speed 2 Project
- Brought back some useful information
- NCCI to EC7

BS 8002:2015

BSI Standards Publication

Code of practice for earth retaining structures

CIRIA C580 update – C760 - updates and extends C580 for application in soft clays, stiff clays, coarse grained soils and weak rocks plus compatibility with Eurocodes

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Feedback on existing report - Industry consultation

Questionnaire

Online survey via Surveymonkey & distributed via:

- BGA global mailing list & those on the ICE list with a geotechnical interest
- BGA LinkedIn & Facebook
- Arup global geotechnical community & forum
- CIRIA Industry Workshop mailing list

> 260 responses from 17 different countries

Workshop

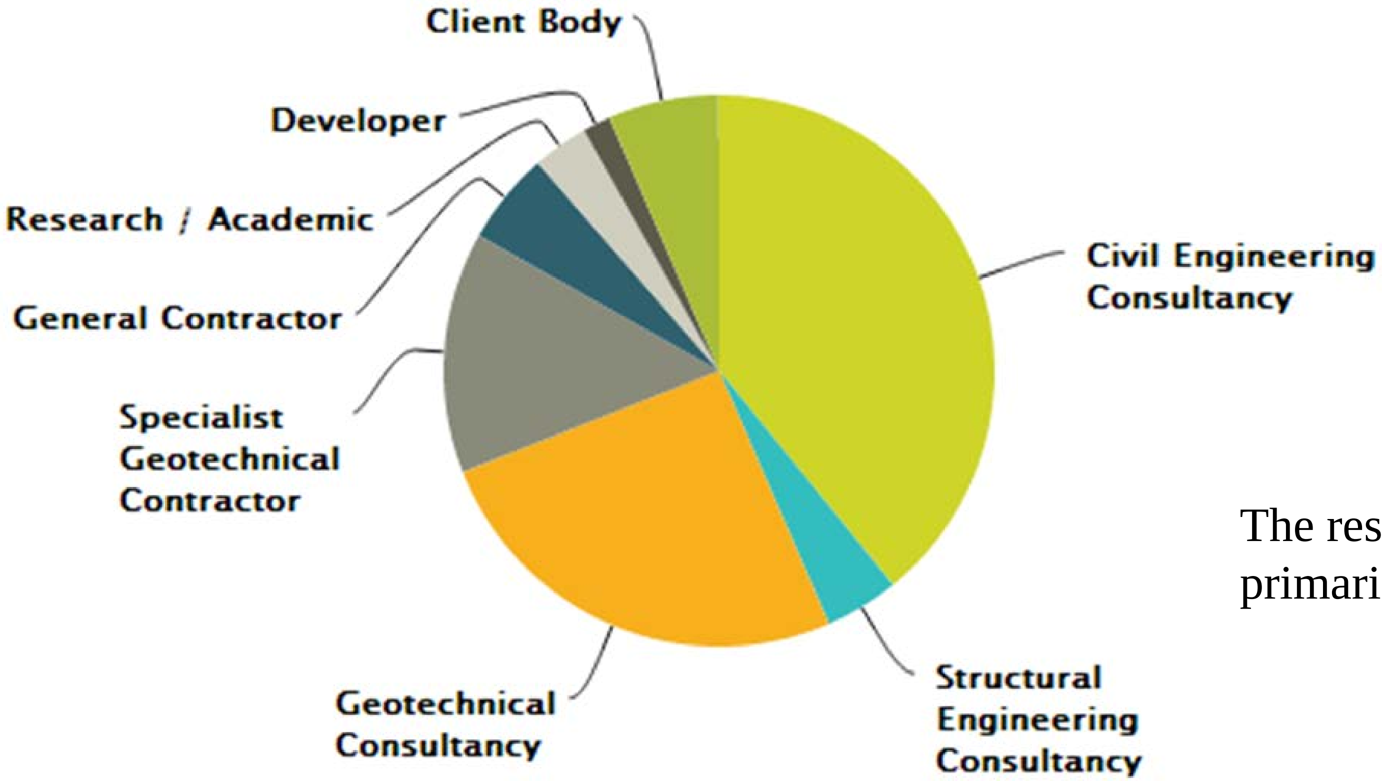
- Full day workshop: 9th May 2014
- 50+ attendees from Clients, Contractors, Designers & Academia, including 2 Rankine lecturers
- 10 facilitators

Questionnaire respondents

260 questionnaire responses from > 115 companies/organisations

Organisations: single person to > 60,000 members

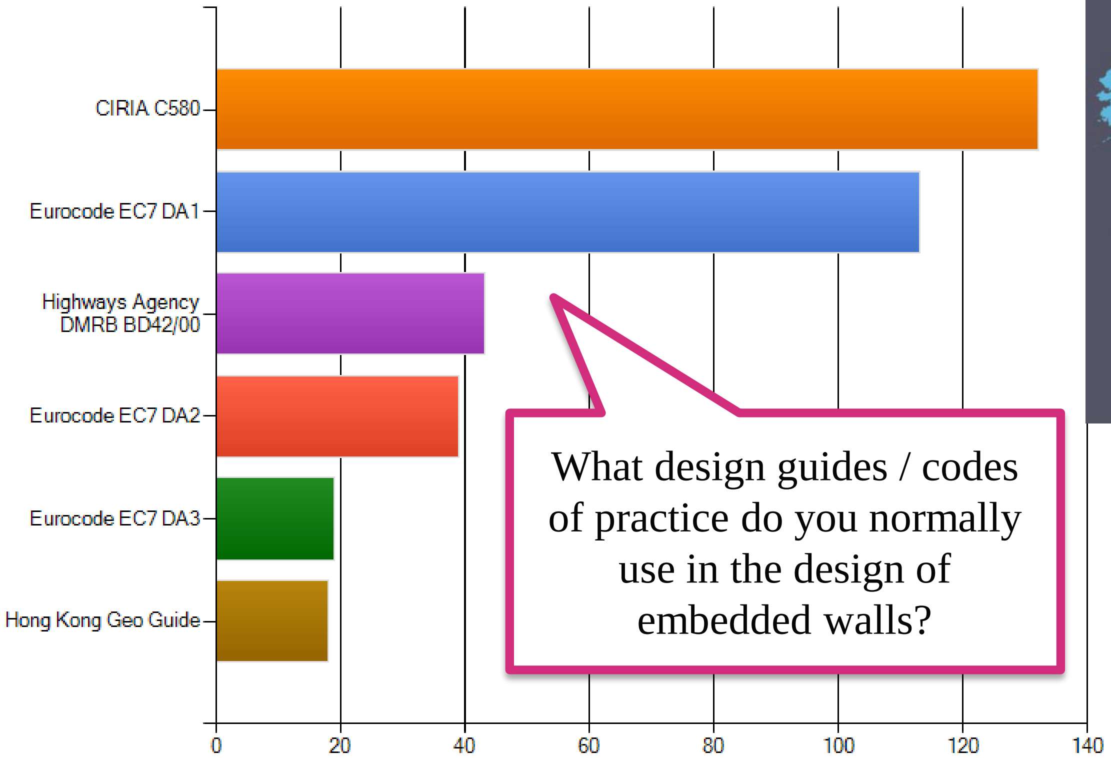
Typically, around 25% of the company staff were geotechnical specialists



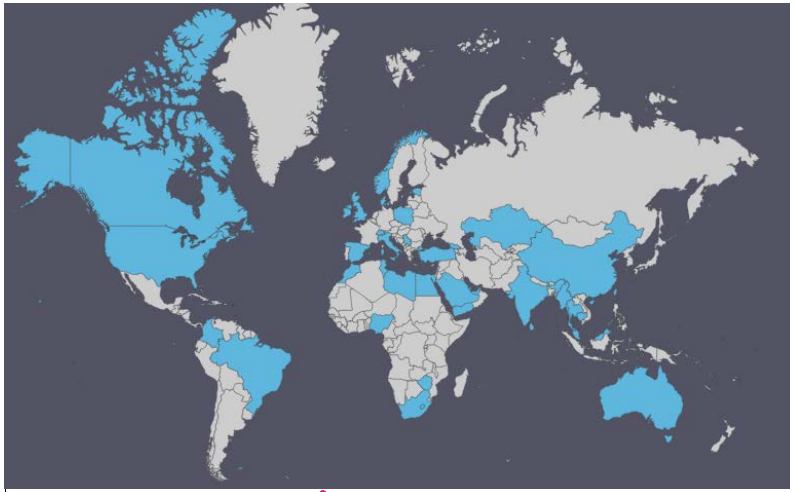
The respondents were primarily designers

Where C580 is used

- C580 used extensively for design of embedded retaining walls around the world
- 46% of respondents had used the report <u>outside</u> the UK
- Local guidance exists adopting C580 principles



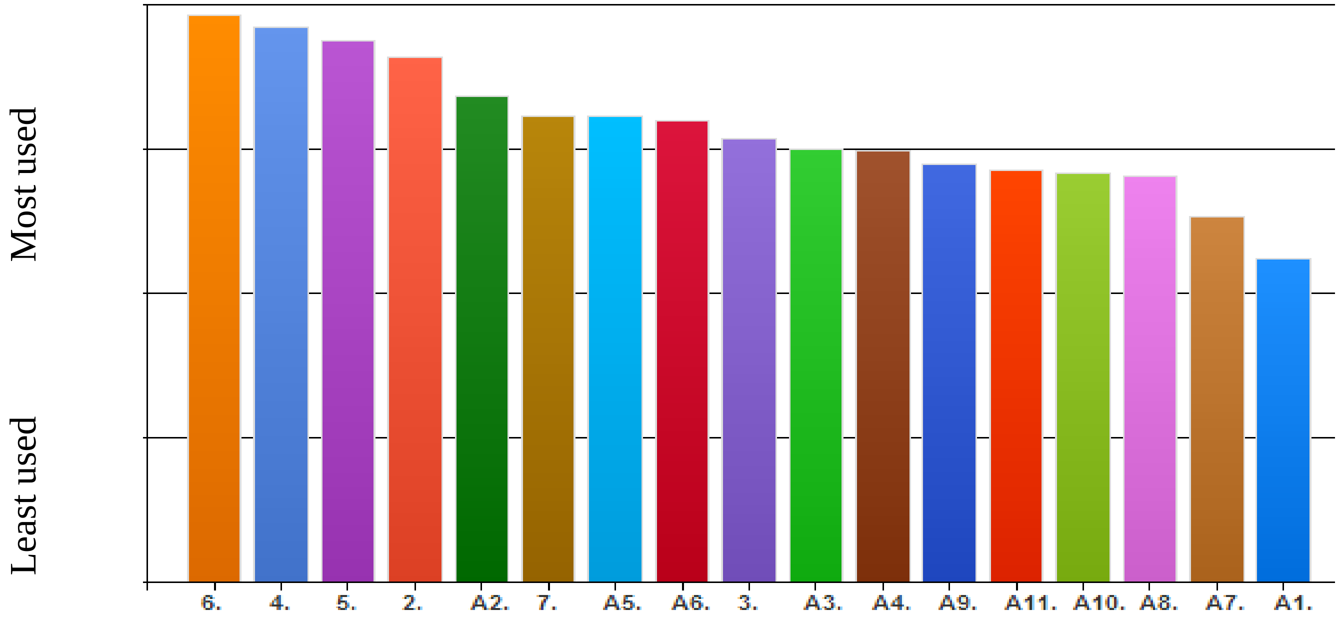
What design guides / codes of practice do you normally use in the design of embedded walls?



Where in the world have you applied the principles of C580 ?

Response to specific questions: which parts most referred to?

- All parts well referred to, but Design and Analysis most commonly used.
- Ground movement database, predictions and case histories, followed by parameter selection



6. Design of Wall	A3. Wall Types
4. Analysis	A4. Soil Mechanics
5. Determination and Selection of Parameters	A9. Analysis of Berms
2. Design Considerations	A11. Worked Example
A2. Ground Movements and Case History Data	A10. Effect of Method of Analysis
7. Design of Support Systems	A8. Review of Design Methods to BS 8002, EC7 and BD42
A5. Effects of Wall Installation	A7. Comparison of Lumped Moment Factors …
A6. Earth Pressure Coefficients	A1. Example CDM Risk Assessment Forms
3. Construction Considerations and Wall Selection	

Comparison with last survey

Is guidance in current design guides & codes of practice clear & unambiguous?

2001

2014

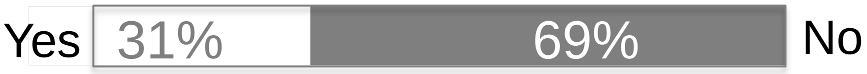
The most common reasons for a ‘No’ answer were:

Many of the issues are with the application of EC7

- Code unclear on water pressures
- Code unclear where to apply factors and what factors to apply
- Not enough guidance on selection of parameters
- Design of temporary works requires further guidance
- C580 design method should be compatible with EC7

Comparison with last survey

Does guidance in current codes of practice result in the economic design and construction of embedded walls?



2001

2014

The most common reasons given for being over conservative:

- Numerical software if used as recommended often results in overdesign
- Actual movements typically significantly less than predicted
- Variation in factors of safety – do we apply ULS correctly?
- Greater guidance is required when selecting characteristic values
- Need for mixed soil analysis can result in over conservative design

Where can greater economy be gained without sacrificing safety?

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- Fundamental considerations
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  - How codes of practice achieve safety
  - Role of statistical analysis
- Closing comments

C580 Update: scope - general

C580 is a global phenomenon: CIRIA's best selling & most downloaded design guide ever.

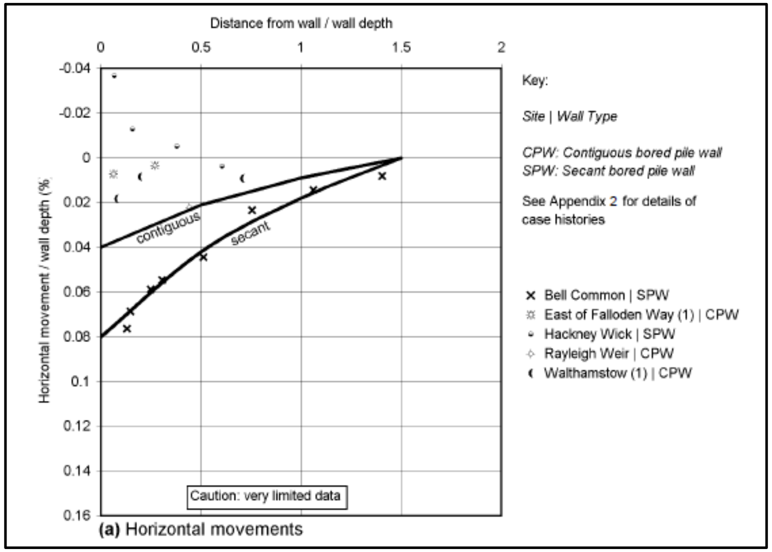
The update (C760) will be compatible with Eurocodes & other NCCI documents. It will:

- Extend application beyond stiff clays & competent soils to include soft clays & weak rocks
- Update & extend ground movements database & guidance on 3D geometrical/corner effects on movement estimation
- Clarify key issues of confusion: parameter selection/water pressures/φ'/wall friction/unplanned excavation/2D & 3D numerical modelling & analysis for ULS
- Provide a step by step approach to EC7 compliant design by calculation compatible with EC2 & EC3
- Establish a robust framework & provide step by step guidance for EC7 compliant design by the use of the Observational Method
- Provide guidance on maintenance, inspection & monitoring/geothermal piles/carbon calculation
- Influence the future evolution of EC7 in 2020

C580 Update: scope

- areas for development

- Ground movements database
- EC7 compliant design methodology
- Use of FE: 2D and 3D & application to ULS (updated worked examples)
- Weak rocks and soft soils



(a) Horizontal movements

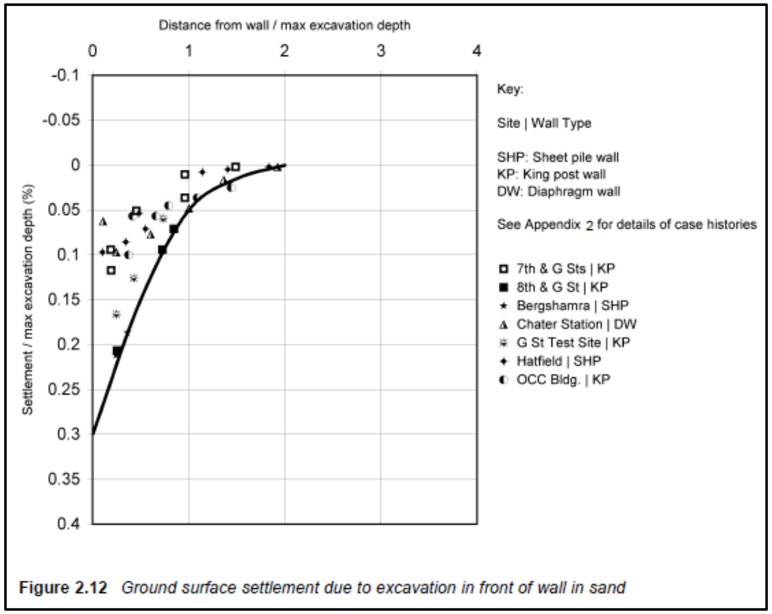
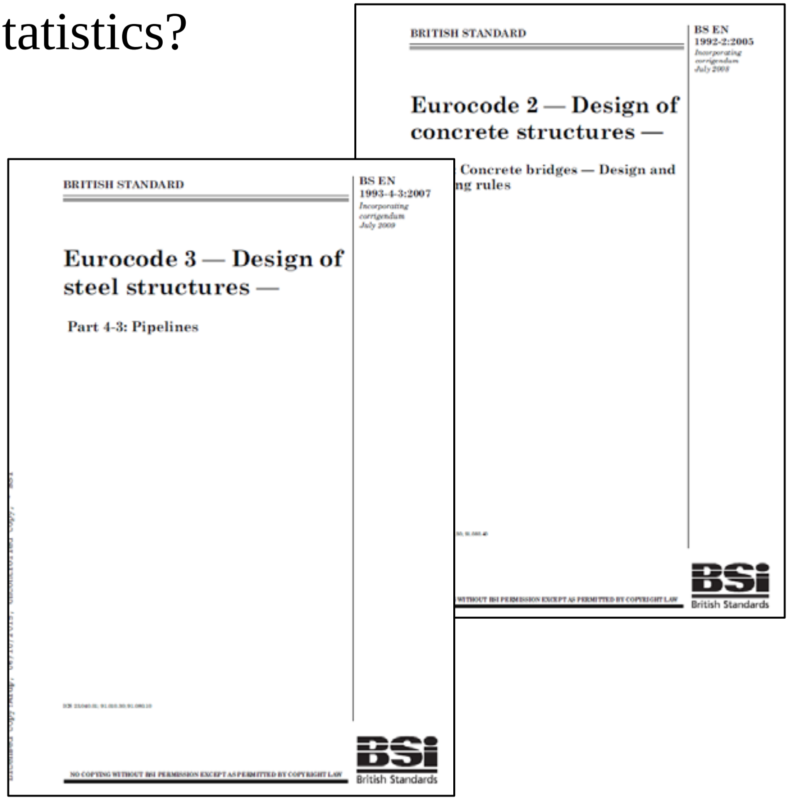


Figure 2.12 *Ground surface settlement due to excavation in front of wall in sand*

C580 Update: scope - areas for further guidance

- Interpretation of design combinations (ULS, SLS, progressive failure avoidance)
- Choice of φ’: peak, constant volume, residual – what is appropriate?
- Choosing characteristic values - the role of statistics?
- Wall friction/adhesion
- Water pressures
- Unplanned excavation
- EC2 and EC3 compatibility
- Numerical modelling



BRITISH STANDARD

BS EN 1992-2:2005
Incorporating corrigendum July 2008

Eurocode 2 — Design of concrete structures —

Concrete bridges — Design and
ng rules

British Standards

BRITISH STANDARD

BS EN 1993-4-3:2007
Incorporating corrigendum July 2009

Eurocode 3 — Design of steel structures —

Part 4-3: Pipelines

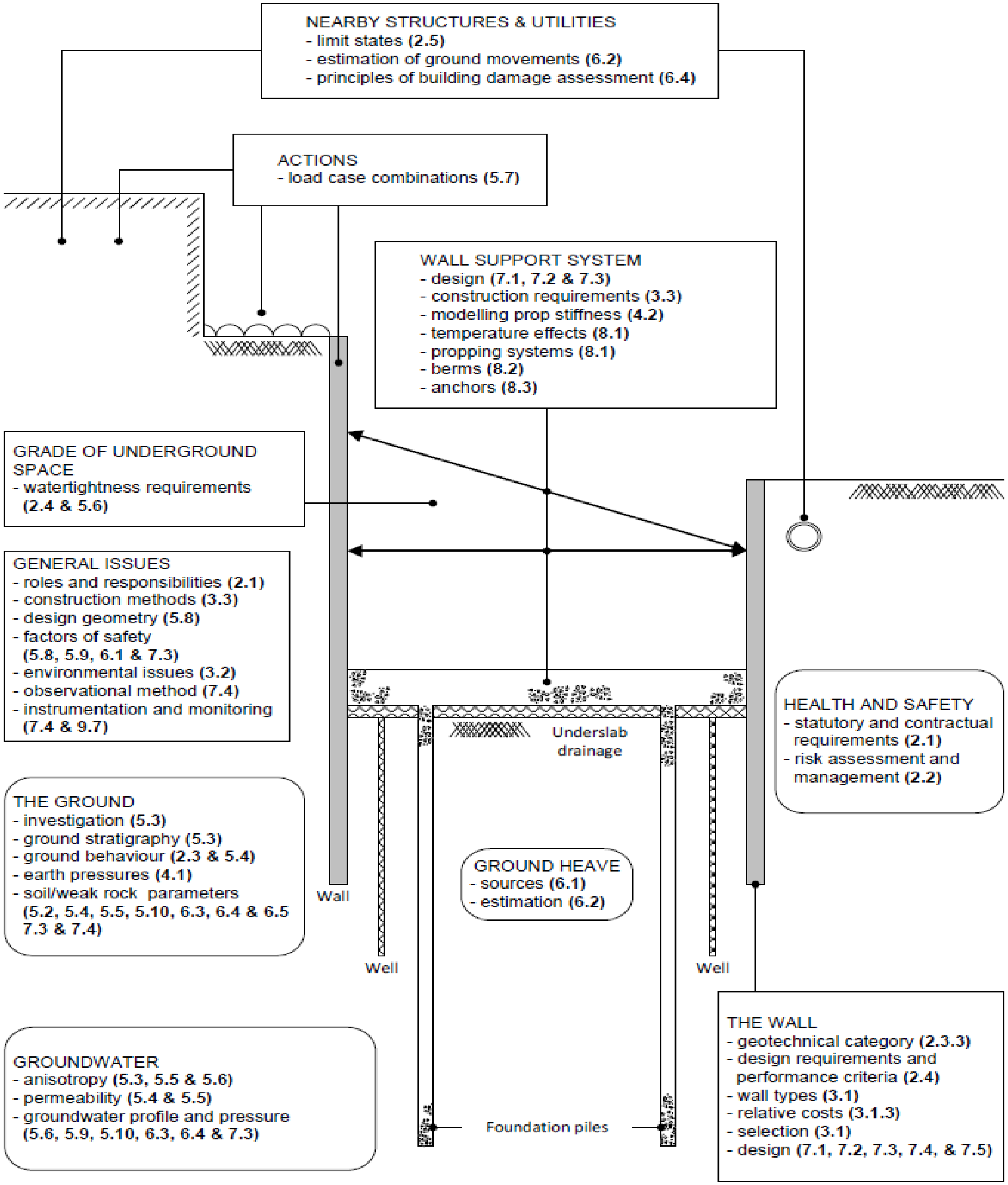
British Standards

C580 Update:

scope - contents

1. Introduction
2. Design considerations
3. Construction considerations and wall selection
4. Analysis
5. Determination and selection of parameters
6. Ground movements and building damage assessment
7. Design of wall (EC7 – DA1 compliant and application of EC2 and EC3)
8. Design of support systems (props, berms, ground anchorages)
9. Maintenance inspection and monitoring
10. Recommendations for further work and research (including proposals for possible amendments to EC7 methodology)

Plus associated appendices


NEARBY STRUCTURES & UTILITIES
- limit states (2.5)
- estimation of ground movements (6.2)
- principles of building damage assessment (6.4)

ACTIONS
- load case combinations (5.7)

WALL SUPPORT SYSTEM
- design (7.1, 7.2 & 7.3)
- construction requirements (3.3)
- modelling prop stiffness (4.2)
- temperature effects (8.1)
- propping systems (8.1)
- berms (8.2)
- anchors (8.3)

GRADE OF UNDERGROUND SPACE
- watertightness requirements (2.4 & 5.6)

GENERAL ISSUES
- roles and responsibilities (2.1)
- construction methods (3.3)
- design geometry (5.8)
- factors of safety (5.8, 5.9, 6.1 & 7.3)
- environmental issues (3.2)
- observational method (7.4)
- instrumentation and monitoring (7.4 & 9.7)

HEALTH AND SAFETY
- statutory and contractual requirements (2.1)
- risk assessment and management (2.2)

THE GROUND
- investigation (5.3)
- ground stratigraphy (5.3)
- ground behaviour (2.3 & 5.4)
- earth pressures (4.1)
- soil/weak rock parameters (5.2, 5.4, 5.5, 5.10, 6.3, 6.4 & 6.5 7.3 & 7.4)

GROUND HEAVE
- sources (6.1)
- estimation (6.2)

GROUNDWATER
- anisotropy (5.3, 5.5 & 5.6)
- permeability (5.4 & 5.5)
- groundwater profile and pressure (5.6, 5.9, 5.10, 6.3, 6.4 & 7.3)

THE WALL
- geotechnical category (2.3.3)
- design requirements and performance criteria (2.4)
- wall types (3.1)
- relative costs (3.1.3)
- selection (3.1)
- design (7.1, 7.2, 7.3, 7.4, & 7.5)

The research team

The authors

Arup
Asim Gaba
Stuart Hardy
Lauren Doughty

Southampton University
Professor William Powrie

Cementation Skanska
Dimitrios Selemetas

Project Steering Group

Clients

- Brian McGinnity (Chair) – London Underground
- Colin Rawlings – HS2
- Duncan McFadyean – HS2
- Mark Shaw – Highways Agency
- Andy Tan – Environment Agency

Contracting

- Tony Suckling – Balfour Beatty
- John Wilkinson – Kier Construction
- Toby Hayward – Laing O'Rourke
- Malcolm Corlett – BAM Nuttall
- Graham White – Arcelor Mittal
- David Preece – Bachy Soletanche

Consultancy

- Tony O'Brien – Mott Macdonald
- Mick Gavins – Atkins
- David Beadman – Byrne Looby
- Peter Scott – SKM Jacobs

Academia

- Professor David Potts – Imperial College

Funders

hs2 engine for growth

HIGHWAYS AGENCY
Safe roads, reliable journeys, informed travellers

Environment Agency

UNDERGROUND

Balfour Beatty

LAING O'ROURKE

BACHY SOLETANCHE

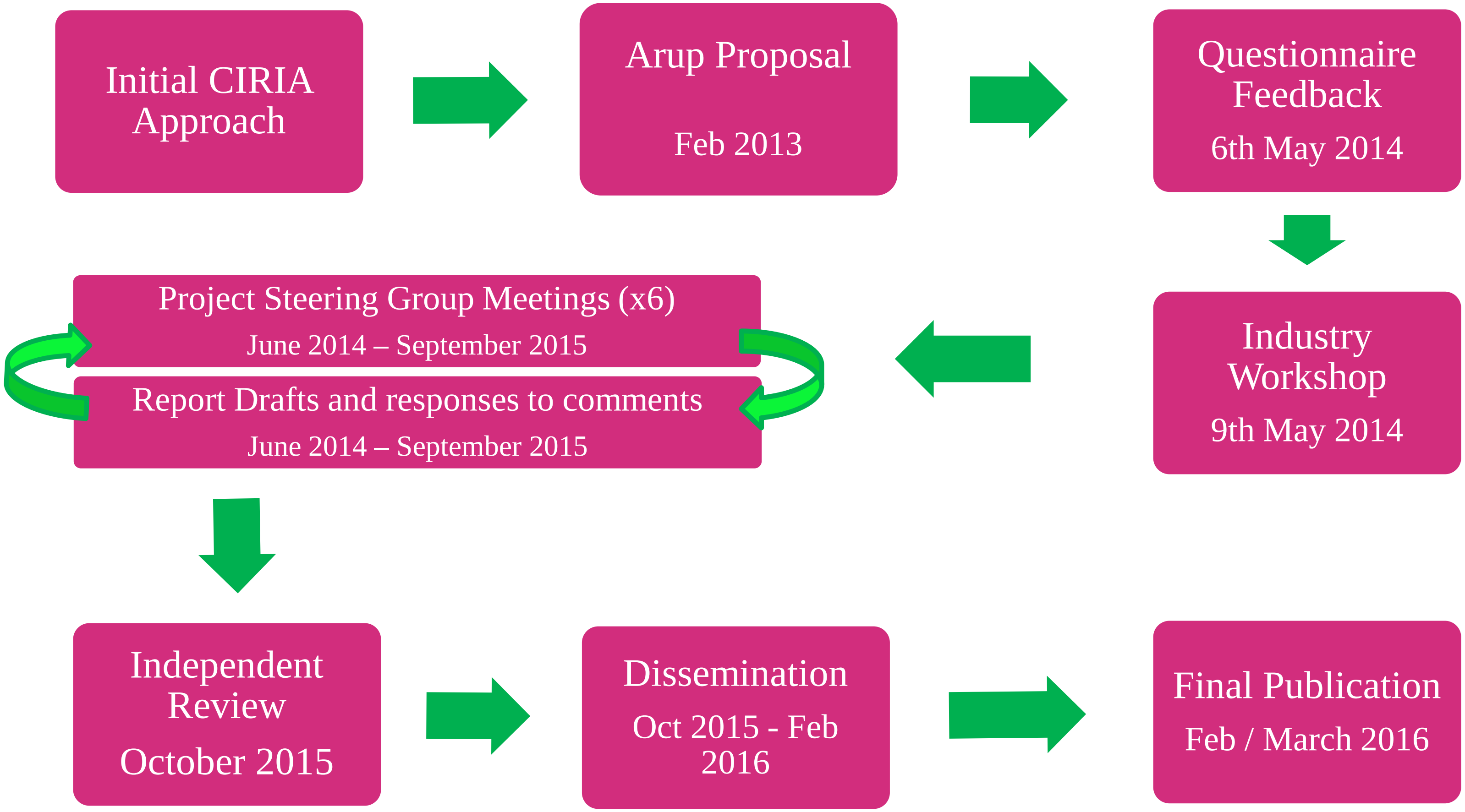
ArcelorMittal

Vp plc Groundforce Shorco
Excavation Support

SKANSKA

KIER
DEFINING TOMORROW'S ENVIRONMENTS

Key Project Stages



Initial CIRIA Approach

Arup Proposal

Feb 2013

Questionnaire Feedback

6th May 2014

Industry Workshop

9th May 2014

Project Steering Group Meetings (x6)

June 2014 – September 2015

Report Drafts and responses to comments

June 2014 – September 2015

Independent Review

October 2015

Dissemination

Oct 2015 - Feb 2016

Final Publication

Feb / March 2016

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Key issues

- Misunderstandings about DA1 – C1 & C2: thought by some to relate to SLS & ULS respectively – they are both overall ULS cases
- Designer to adopt characteristic value of ϕ' *affecting the occurrence of the limit state* under consideration and apply a partial factor of 1.25 to the tangent of that number in DA1 C2
- BS 8002 distanced walls from failure by use of a mobilisation factor applied in conjunction with indirect stipulations (e.g. unplanned excavation, mandatory minimum surcharge, conservative wall friction/adhesion) and gave guidance on ϕ'_{crit} and ϕ'_{peak}
- Treatment of extreme events?
- Characteristic values? Role of statistical analysis in geotechnical engineering?
- Key is to capture the best ideas and practice from experience gained from the application of BS 8002, C580 and EC7 (BS EN 1997:2004) to clarify misunderstandings and issues and put forward a coherent design approach that is compatible and consistent

Selected key issues

- The choice of φ’ for embedded retaining wall analysis?
- How do codes of practice achieve safety?
- The role of statistical analysis in geotechnical design?

Choice of φ’ for embedded retaining wall design

- What value of φ’ should be used in the design of an embedded retaining wall for compliance with EC7?

- Section 2.4.3 of EC7 (BS EN 1997-1:2004) states

  *“(2)(P) Values obtained from test results and other data shall be interpreted appropriately **for the limit state considered**”*

- Section 2.4.5.2 of EC7 (BS EN 1997-1:2004) states

  *“(2)(P) The characteristic value of a geotechnical parameter shall be selected as a cautious estimate of the value **affecting the occurrence of the limit state**”*

- Which limit states should we consider? What is the value of φ’ which affects the “occurrence of the limit state”?

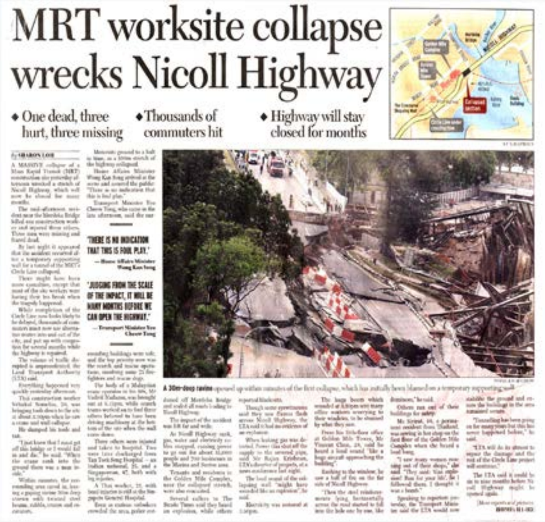
Which limit states should we consider?

- Section 2.4.7 – Ultimate Limit State (safety of people/structures)
    - EQU (loss of equilibrium of the structure and ground as a rigid body)
    - STR (internal failure or deformation of a structure)
    - GEO (failure or excessive deformation of the ground)
- Section 2.4.8 – Serviceability Limit State (function/comfort/appearance)

Ultimate Limit State

- Austria Alps - following snow melt

- Singapore - Nicoll Highway



MRT worksite collapse wrecks Nicoll Highway

- One dead, three hurt, three missing
- Thousands of commuters hit
- Highway will stay closed for months

"THERE IS NO INDICATION THAT THIS IS FOUL PLAY."
— Home Affairs Minister Wong Kan Seng

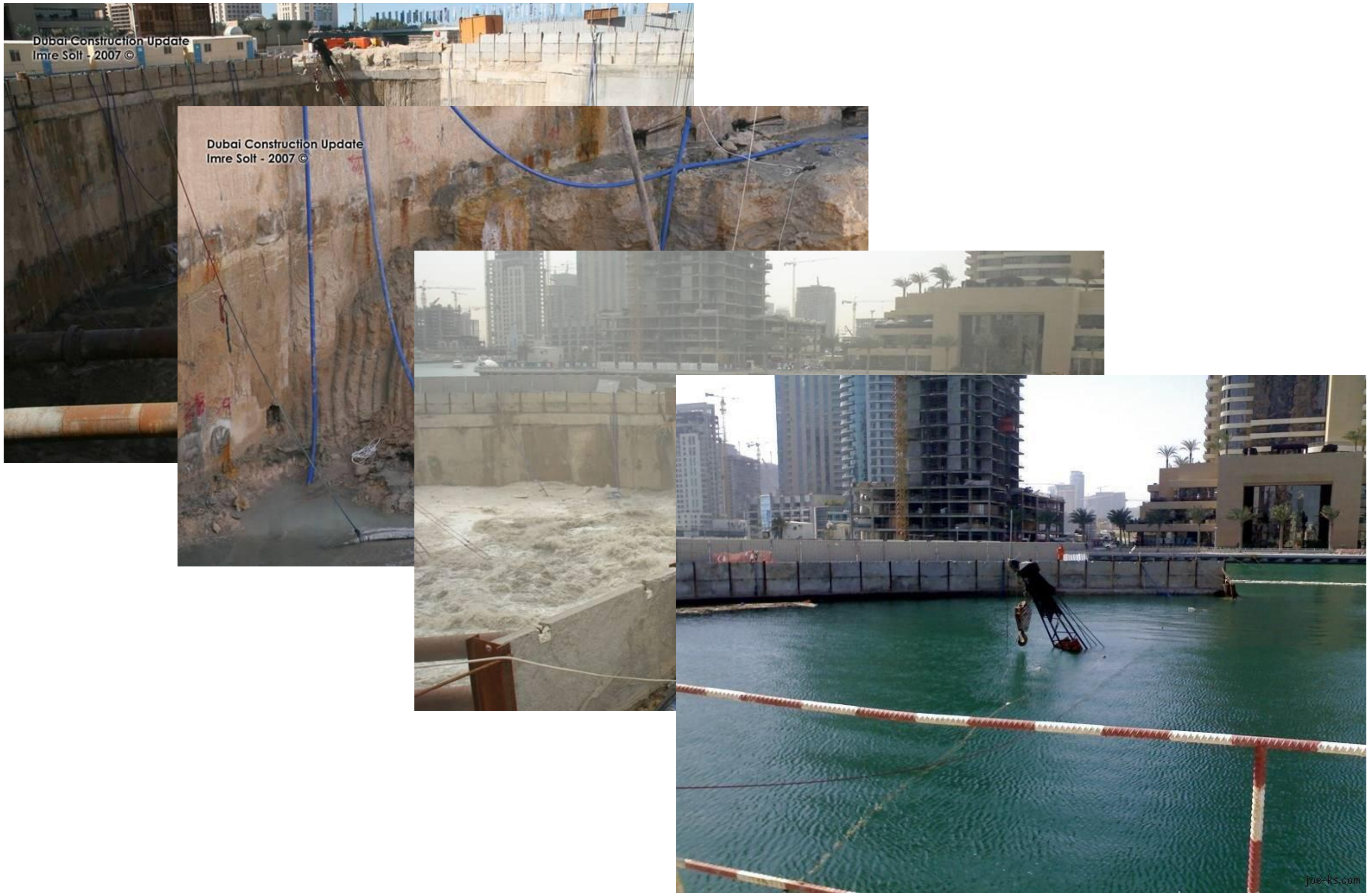
"JUDGING FROM THE SCALE OF THE IMPACT, IT WILL BE MANY MONTHS BEFORE WE CAN OPEN THE HIGHWAY."
— Transport Minister Yeo Cheow Tong

A 30m-deep ravine opened up within minutes of the first collapse, which has initially been blamed on a temporary supporting wall.

- Cyprus - villa

Ultimate Limit State

- Infinity Towers, Dubai



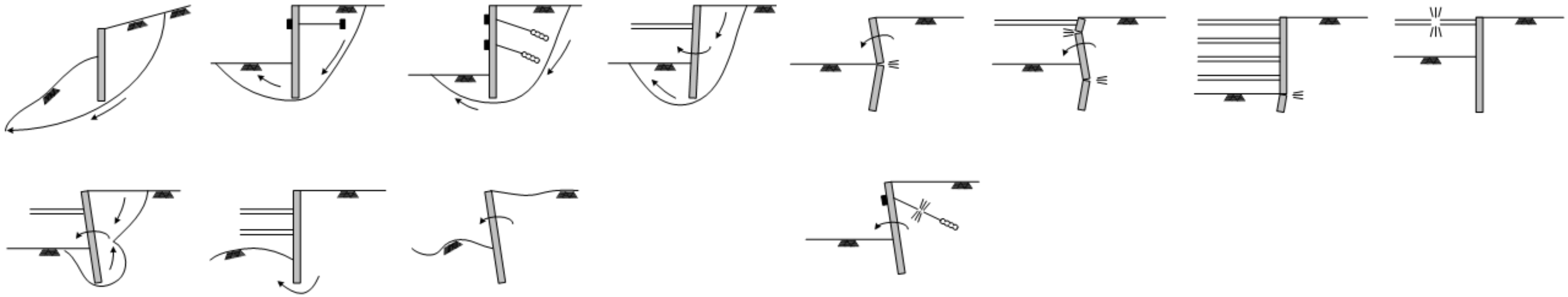
Serviceability Limit State

- Of the wall itself

- Of neighbouring structures

What φ’ applies at the Ultimate Limit State?

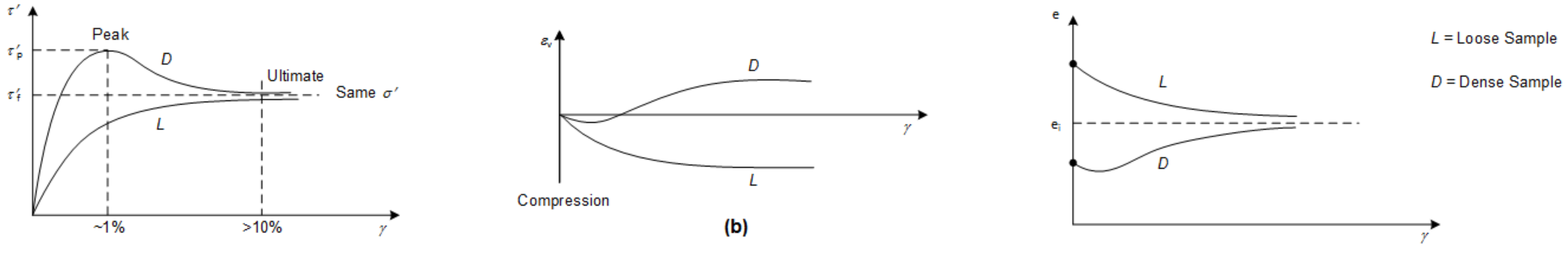
- ULS failure mechanisms for embedded retaining walls



(a) - *Rotational failure*

(b) - *Structural failure*

- Critical State Soil Mechanics



- Powrie (1996) suggested φ'_{cv} operational for some case histories

What is the intention of EC7 (BS EN 1997-1:2004)?

- Table A.4 sets γ_φ to 1.25 for DA1 Combination 2 and notes:

  "This factor is applied to tan φ' *"*

- Table A.NA.4 sets γ_φ to 1.25 for DA1 Combination 2 and notes:

  "Applied to tan φ' *and tan* φ'_{cv} *although it might be more appropriate to determine the design value of* φ'_{cv} *directly"*

How was it done before?

- **CP2** – did not apply partial factors to strength parameters
- **CIRIA Report 104 (1984)** recommended the following values for F_s on tan φ’
  - Moderately conservative Fs = 1.1 to 1.2 (temp) and Fs = 1.2 to 1.5 (perm)
  - Worst credible F_s = 1.0 (temp) and F_s = 1.2 (perm)
- **BS8002 (1994)** proposed the lesser of
  - tan φ_{peak}’ / 1.2 or
  - tan φ_{cv}’
- **C580 (2003)** select by inspection more onerous of:
  - Design Approach A: moderately conservative values $F_{s\varphi}$ = 1.2 applied to tan φ_{peak}, φ_{cv} or $\varphi_{residual}$
  - Design Approach B: worst credible values $F_{s\varphi}$ = 1.0 applied (generally) to tan φ_{cv} or $\varphi_{residual}$

Selected key issues/challenges

- The choice of φ’ for embedded retaining wall analysis
- **How do codes of practice achieve safety?**
- The role of statistical analysis in geotechnical design?

Where is safety achieved in BS 8002 (1994)?

- Designers have to ensure our structures are remote from failure

- If we use BS 8002 with φ'_{cv} and a partial factor of 1.0, in principle, our wall is on the point of failure, should movements develop as would be the case with a cantilever or single prop wall and φ'_{mob} tends towards φ'_{cv}

- But BS 8002 required *"the mobilisation factor should be used in conjunction with the "unplanned" excavation in front of the wall, the minimum surcharge loading and the water pressure regime"*

Where is safety achieved in BS 8002 (1994)?

- Minimum Surcharge?
  - 10kPa (Section 3.3.4.1)

- Minimum unplanned excavation (over-dig)?
  - 10% of retained height or below lowest prop up to maximum of 0.5m (Section 3.2.2.2)

- Ground water level?
  - 1m below ground level?
  - MEFP?
  - Water filled tension cracks?

- Choice of tan δ
  - Design tan δ = 0.75 x design tan φ (Section 3.2.6)

Where is safety achieved in EC7 (BS EN 1997:2004)?

- Minimum Surcharge?
  - No minimum surcharge (Section 9.3.1.3)

- Minimum unplanned excavation (over-dig) - maybe?
  - 10% of retained height (if cantilever) or below lowest prop up to a maximum of 0.5m in normal circumstances, although smaller values including zero, may be used (Section 9.3.2.2)

- Groundwater?
  - Characteristic values for water levels (Section 9.3.2.3(1)P)
  - Most unfavourable values for groundwater pressures (Section 2.4.6.1(6)P)

Compare BS 8002 and EC7 (BS EN 1997:2004) – propped wall

- Embedded retaining wall with:
  - $\varphi_{peak} = 35°$ $\varphi_{crit} = 30°$
  - 15m deep excavation
  - Maximum span 5m
  - 2 levels of temporary props
  - No anticipated surcharge
  - Tight site controls on excavation levels

BS 8002	**BS EN 1997:2004**
$\varphi_{design} = \tan^{-1}(\tan\varphi_{peak} / 1.2)$ or φ_{crit} $\varphi_{design} = 30°$	$\varphi_{design} = \tan^{-1}(\tan\varphi_{peak} / 1.25)$ $\varphi_{design} = 29.2°$
Minimum surcharge 10kPa	No surcharge
Unplanned excavation of 0.5m	No unplanned excavation
Onerous groundwater	most unfavourable groundwater

Compare BS 8002 and EC7 (BS EN 1997:2004) – cantilever wall

- Embedded retaining wall with:
  - $\varphi_{peak} = 45°$ $\varphi_{crit} = 36°$
  - 5m deep
  - Cantilever wall
  - 20kPa stock pile surcharge
  - No special site control

BS 8002

$\varphi_{design} = \tan^{-1} (\tan \varphi_{peak} / 1.2)$ or φ_{crit}

$\varphi_{design} = 36°$

Surcharge 20kPa

Unplanned excavation of 0.5m

Onerous groundwater

BS EN 1997:2004

$\varphi_{design} = \tan^{-1} (\tan \varphi_{crit} / 1.25)$

$\varphi_{design} = 30.2°$

Surcharge = 20 x 1.3

Surcharge = 26kPa

Unplanned excavation of 0.5m

most unfavourable groundwater

Selected key issues/challenges

- The choice of φ’ for embedded retaining wall analysis
- How do codes of practice achieve safety?
- **The role of statistical analysis in geotechnical design?**

Does statistical analysis have a role in geotechnical design?

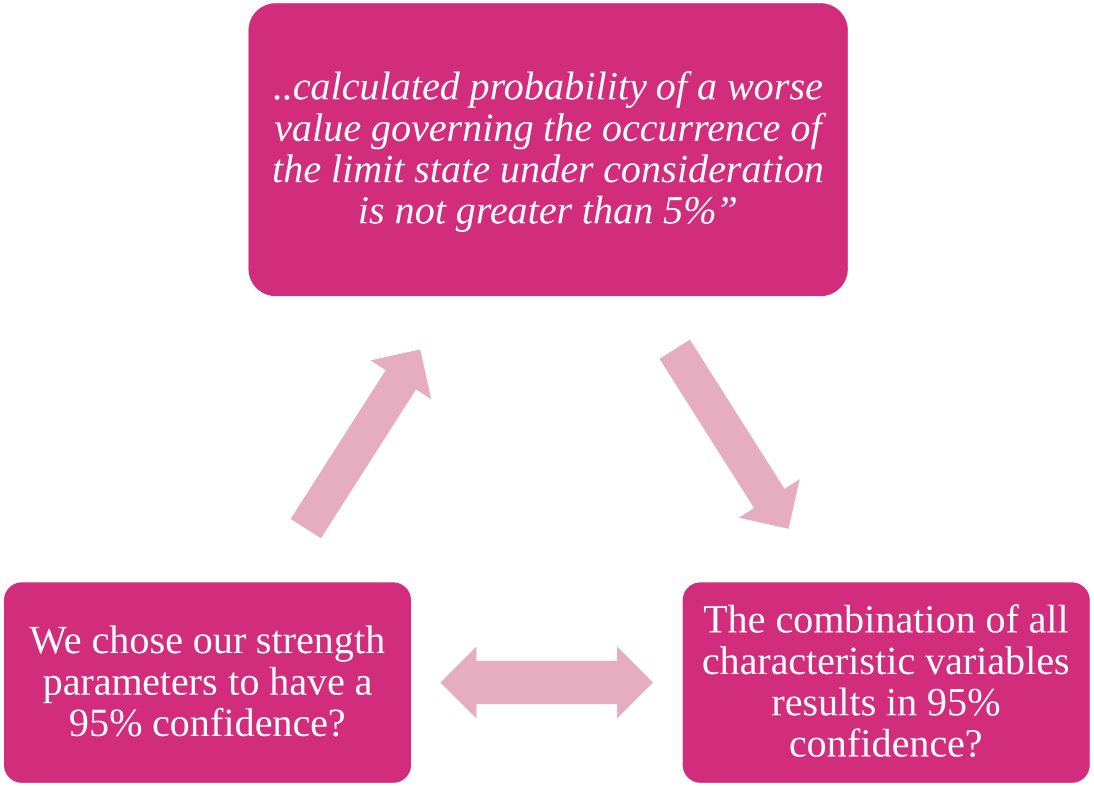
- Section 2.4.5.2 of BS EN 1997-1:2004 states

  "(2) (P) The characteristic value of a geotechnical parameter shall be selected as a cautious estimate of the value affecting the occurrence of the limit state"

  "(11) ***If*** *statistical methods are used, the characteristic value should be derived such that the calculated probability of a worse value governing the occurrence of the limit state under consideration is not greater than 5%"*

  NOTE In this respect, a cautious estimate of the mean value is a selection of the mean value of the limited set of geotechnical parameter values, with a confidence level of 95%; where local failure is concerned, a cautious estimate of the low value is a 5% fractile

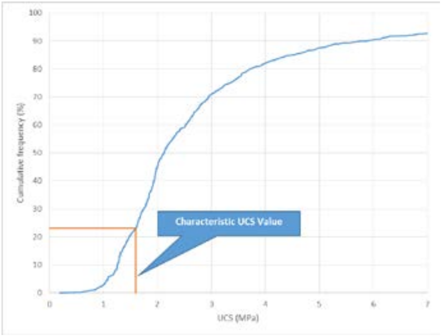
What does this mean?

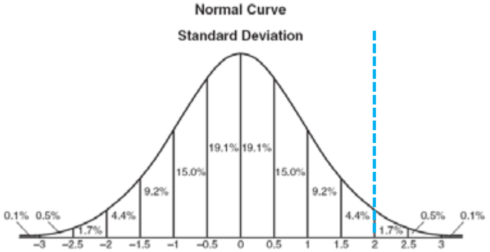


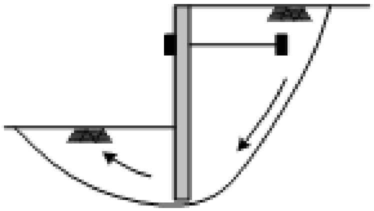
..calculated probability of a worse value governing the occurrence of the limit state under consideration is not greater than 5%”

We chose our strength parameters to have a 95% confidence?

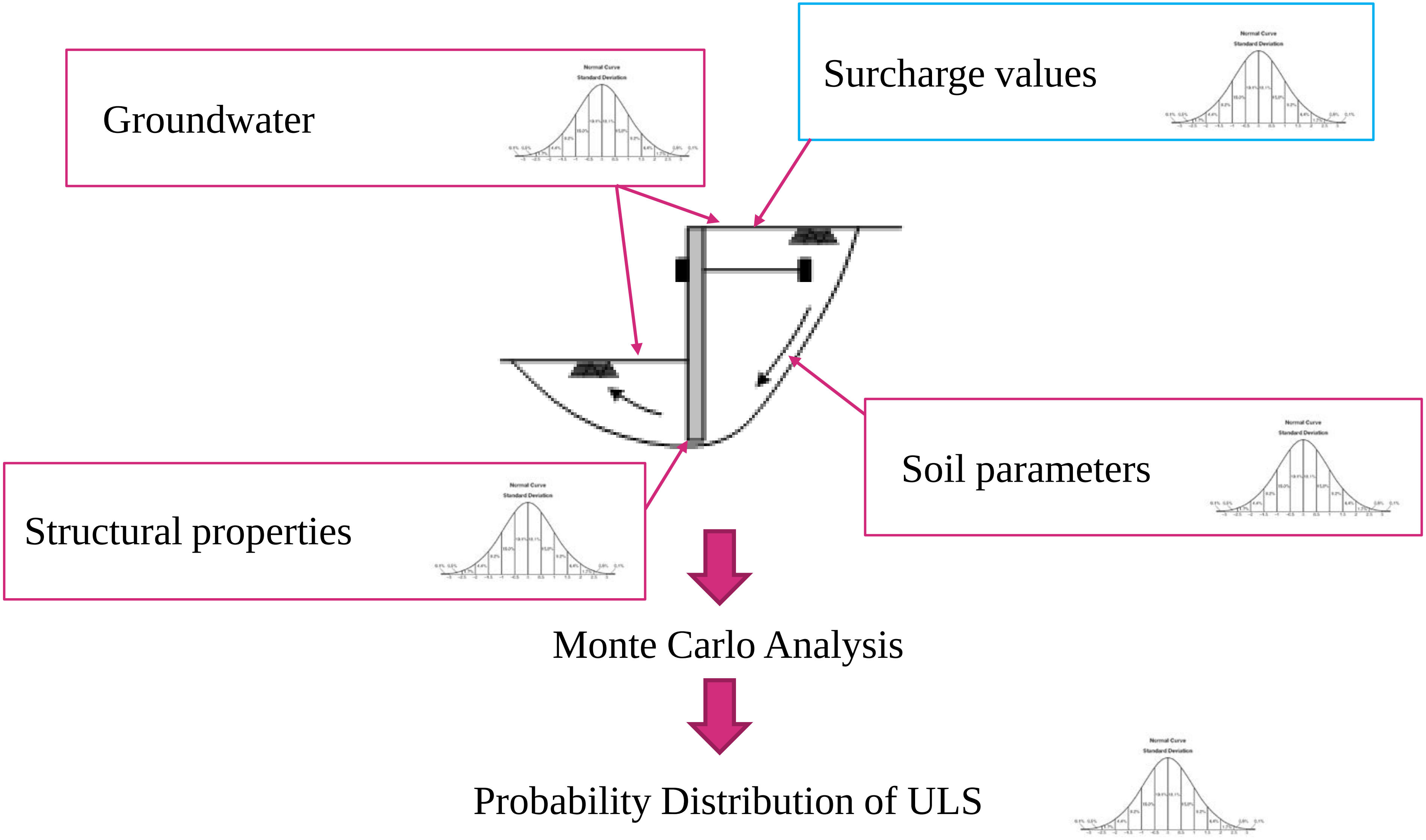
The combination of all characteristic variables results in 95% confidence?







If it is 95% confidence in the occurrence of the limit state:



Groundwater

Surcharge values

Soil parameters

Structural properties

Monte Carlo Analysis

Probability Distribution of ULS

To summarise……

- BS 8002 ensured walls were remote from ULS by indirect means (unplanned excavation, mandatory minimum surcharge, conservative wall friction)
- To some extent EC7 (BS EN 1997:2004) does this, but is less prescriptive on the indirect means used by BS 8002
- Safety ensured by partial factors on soil strength and choice of characteristic values and most unfavourable groundwater pressures
- BS 8002 gave logical guidance on φ_{crit} and φ_{peak}. No such guidance in EC7 (BS EN 1997:2004) on application of partial factors on φ_{crit} and φ_{peak}
- EC7 (BS EN 1997:2004) attempted to bring the definition of soil parameters in line with structural practice (95% percentiles) with little success
- Is this progress, or is it unhelpful?

Presentation Outline

- **Background to update**
- **Feedback on C580 publication**
  - Industry consultation
  - Where C580 is used
- **The C580 Update**
  - Scope of C760
  - Research team & Project Steering Group
  - Project status and timeline
- **Fundamental considerations**
  - Key issues
  - Selection of φ'
  - How codes of practice achieve safety
  - Role of statistical analysis
- **Closing comments**

Closing comments

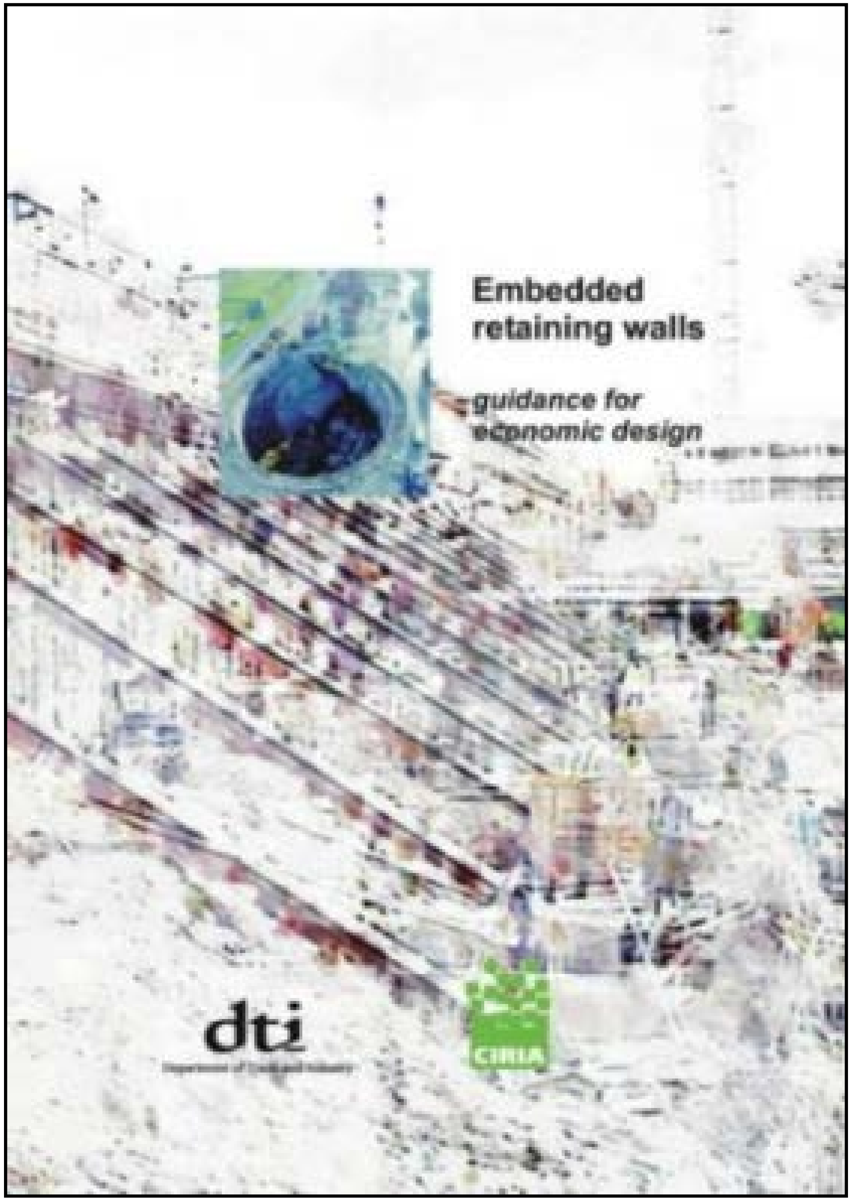
C580 is a global phenomenon: CIRIA's best selling & most downloaded design guide ever.

The update (C760) will be compatible with Eurocodes & other NCCI documents. It will:

- Extend application beyond stiff clays & competent soils to include soft clays & weak rocks
- Update & extend ground movements database & guidance on 3D geometrical/corner effects on movement estimation
- Clarify key issues of confusion: parameter selection/water pressures/φ'/wall friction/unplanned excavation/2D & 3D numerical modelling & analysis for ULS
- Provide a step by step approach to EC7 compliant design by calculation compatible with EC2 & EC3
- Establish a robust framework & provide step by step guidance for EC7 compliant design by the use of the Observational Method
- Provide guidance on maintenance, inspection & monitoring/geothermal piles/carbon calculation
- Influence the future evolution of EC7 in 2020

C760
Embedded Retaining walls:
guidance for design

MAJOR REVISIONS TO C580



Embedded retaining walls

guidance for economic design

dti

CIRIA

Presentation Outline

- **Introduction: EC7 compliant design of embedded retaining walls**

- **Design by calculation**
  - EC7 fundamentals: limit states
  - Design Approach 1: combination 1 and 2 and associated partial factors
  - Parameter selection
  - Choice of water levels/φ'/wall friction and adhesion/unplanned excavation

- **Design by The Observational Method: outline**

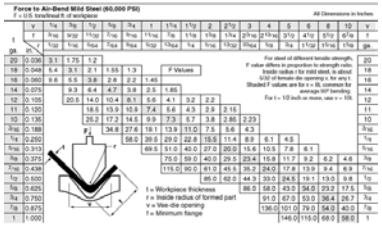
- **Other significant revisions to C580 guidance**
  - Derivation of weak rock parameters
  - Update and expansion of ground movements database
  - Choice of wall : construction tolerances, use of nomograms and typical structural details
  - Guidance on the use of finite element analysis for ULS calculations
  - EC2 and EC3 compliant structural design of wall and its support system
  - Durability & plastic design of steel walls and crack width considerations & reinforcement design of reinforced concrete walls

- **Closing comments**

Introduction: EC7 compliant design

EC7-1 Clause 2.1(4) permits design to be undertaken:

- by calculation
- by prescriptive measures
- by experimental models and load tests
- by The Observational Method



Force to Air-Bend Mild Steel (60,000 PSI)
F = U.S. tons/lineal ft. of workpiece
All Dimensions in Inches

For steel of different tensile strength, F value differs in proportion to strength ratio. Inside radius r for mild steel, is about 5/32 of female die opening v, for any t. Shaded F values are for v = 8t, common for average 90° bending. For t = 1/2 inch or more, use v = 10t.

t = Workpiece thickness
r = Inside radius of formed part
v = Vee-die opening
f = Minimum flange

Presentation Outline

- **Introduction: EC7 compliant design of embedded retaining walls**

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  - EC7 fundamentals: limit states
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  - Durability & plastic design of steel walls and crack width considerations & reinforcement design of reinforced concrete walls

- **Closing comments**

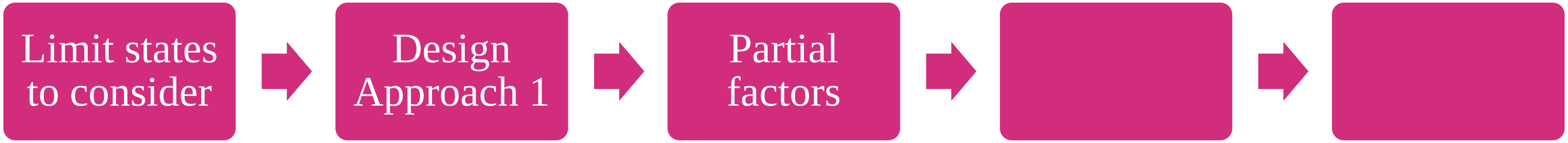
Design by calculation: EC7 fundamentals

Limit states to consider

Design by calculation: EC7 fundamentals

Limit states to consider → Design Approach 1 → → →

Design by calculation: EC7 fundamentals



Limit states to consider

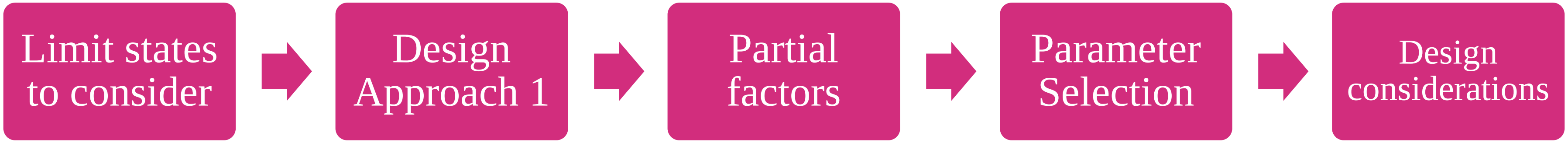
Design Approach 1

Partial factors

Design by calculation: EC7 fundamentals

- Limit states to consider
- Design Approach 1
- Partial factors
- Parameter Selection

Design by calculation: EC7 fundamentals



Limit states to consider

Design Approach 1

Partial factors

Parameter Selection

Design considerations

Design by calculation: limit states

EC7-1 Section 2.4.7 – Ultimate Limit State (safety of people/structures)

- EQU (loss of equilibrium of the structure and ground as a rigid body)
- STR (internal failure or deformation of a structure)
- GEO (failure or excessive deformation of the ground)

EC7-1 Section 2.4.8 – Serviceability Limit State (function/comfort/appearance)

Design by calculation: limit states

Austria Alps following snow melt

Singapore - Nicoll Highway

MRT worksite collapse wrecks Nicoll Highway

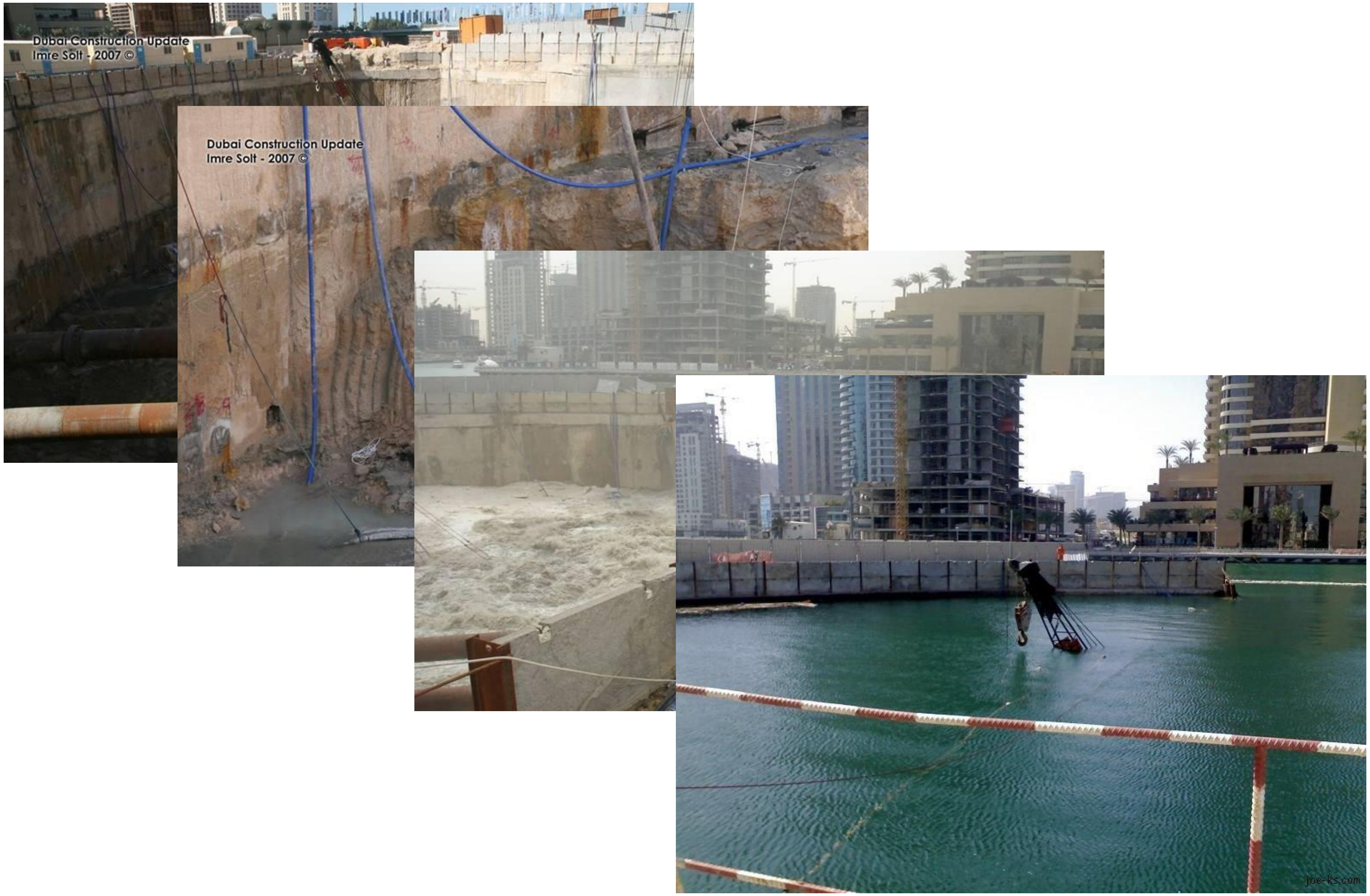
- One dead, three hurt, three missing
- Thousands of commuters hit
- Highway will stay closed for months

'THERE IS NO INDICATION THAT THIS IS FOUL PLAY.'
— Home Affairs Minister Wong Kan Seng

'JUDGING FROM THE SCALE OF THE IMPACT, IT WILL BE MANY MONTHS BEFORE WE CAN OPEN THE HIGHWAY.'
— Transport Minister Yeo Cheow Tong

Ultimate Limit State

Infinity Towers, Dubai



Serviceability Limit State

Of the wall itself

Of neighbouring structures

Design by calculation – Design Approach 1

UK adopted Design Approach 1 which requires two Ultimate Limit State checks

Both are ULS checks

Both are STR/GEO checks

SLS is a separate check

Combination 1	Combination 2
Partial factor = 1.0 applied to soil/rock parameters	Partial factors >1.0 applied to soil/rock parameters
Partial factors >1.0 applied to unfavourable variable actions	Partial factors = 1.3 on unfavourable variable actions
Partial factor = 1.35 applied to effects of actions	Partial factor = 1.0 applied to effects of actions

Design by calculation – Partial factors

$$X_d = \frac{X_k}{\gamma_M}$$

$$c'_d = c_k' / \gamma_c$$

$$\tan \varphi'_d = \tan \varphi_k' / \gamma_\varphi$$

$$c_{u,d} = c_{k,u} / \gamma_{cu}$$

$$q_{u,d} = q_{k,u} / \gamma_{qu}$$

Design Approach 1	**Soil and rock parameters**				**Unfavourable actions**	
	γ_φ	γ_c	γ_{cu}	γ_{qu}	Permanent	Variable
Combination 1 DA1C1	1.0	1.0	1.0	1.0	1.35	1.5
Combination 2 DA1C2	1.25	1.25	1.4	1.4	1.0	1.3

DA1C1 design effects of actions $E_d = \gamma_E E_k = 1.35 \times E_k$

DA1C2 design effects of actions $E_d = \gamma_E E_k = 1.00 \times E_k$

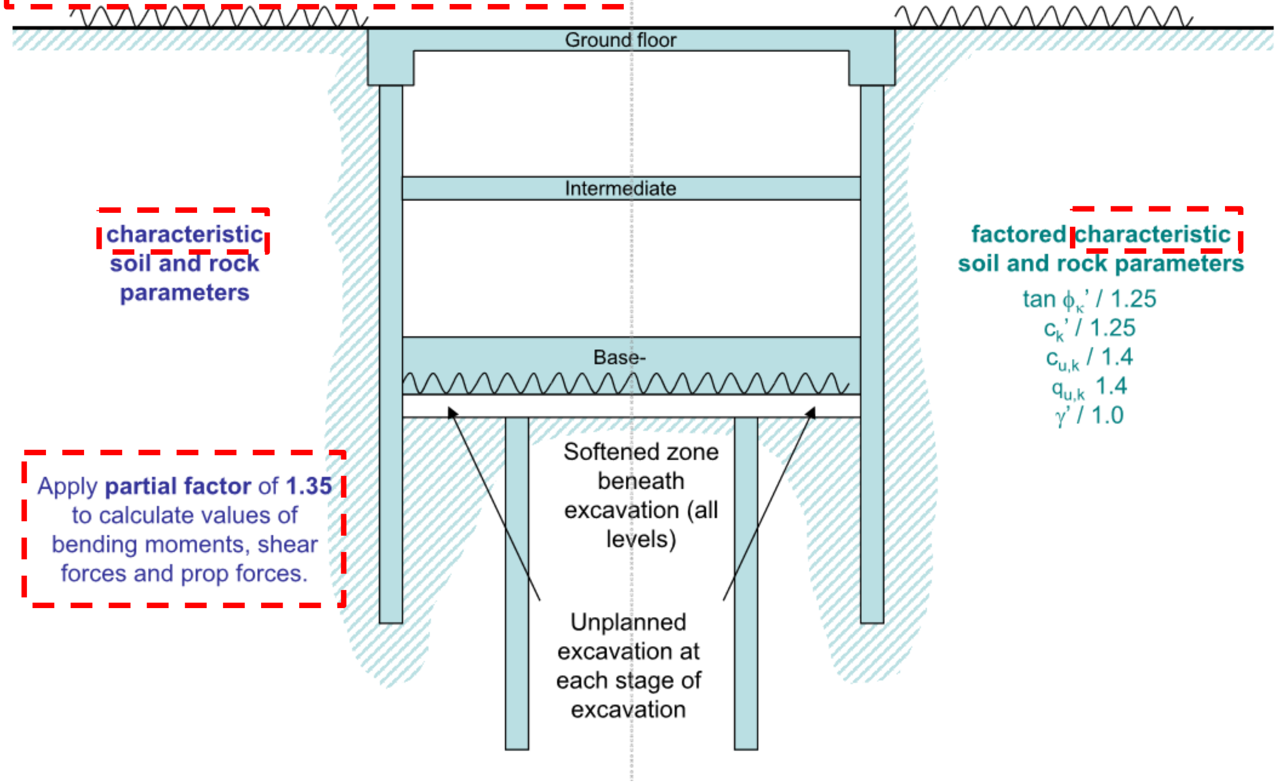
Design by calculation – Partial factors

Design Approach 1 Combination 1 (DA1C1)

γf = 1.1 (=1.5/1.35) on unfavourable variable loading
γf = 0.0 on favourable variable loading
γf = 1.0 (=1.35/1.35) on permanent non-soil loading

Design Approach 1 Combination 2 (DA1C2)

γf = 1.3 on unfavourable variable loading
γf = 0.0 on favourable variable loading
γf = 1.0 on permanent non-soil loading



characteristic soil and rock parameters

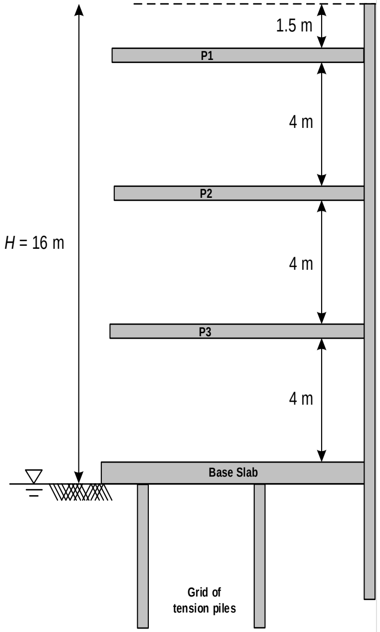
Apply **partial factor** of **1.35** to calculate values of bending moments, shear forces and prop forces.

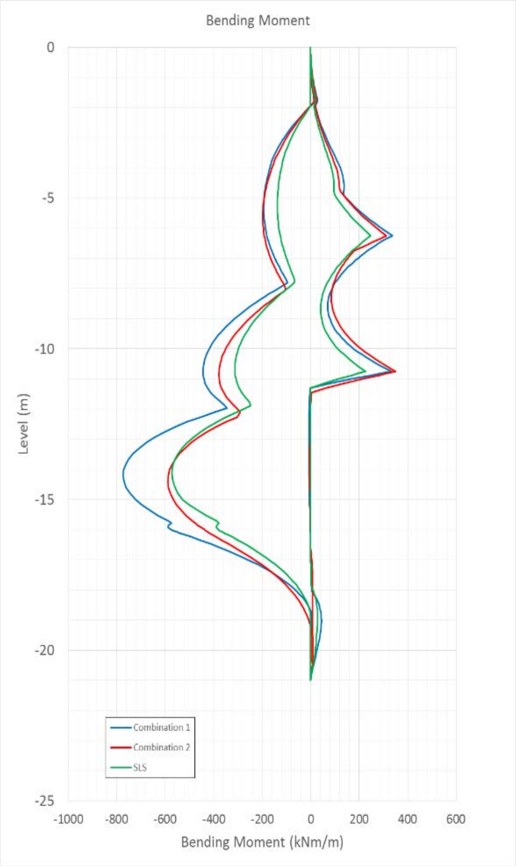
factored characteristic soil and rock parameters

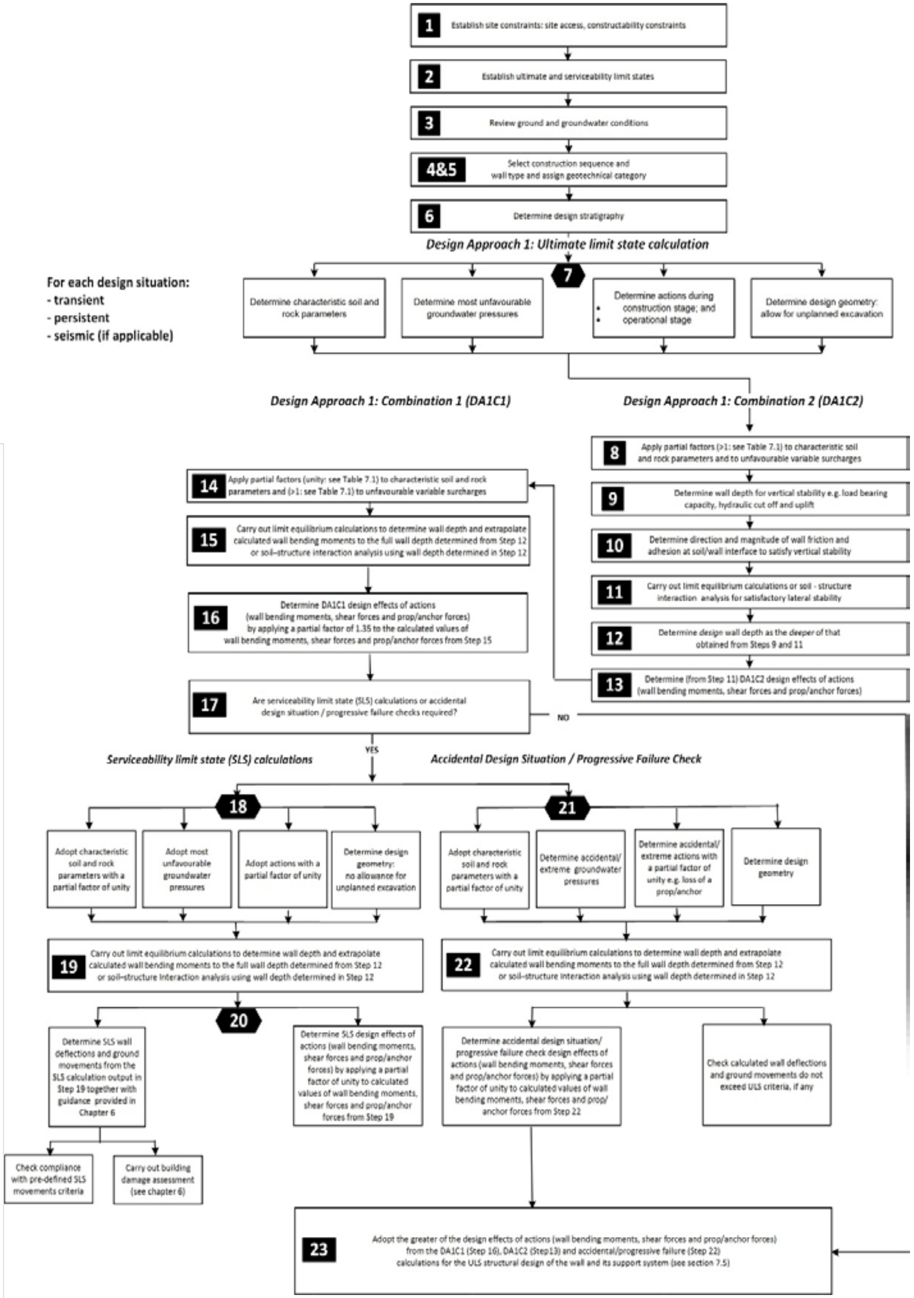
$\tan \phi_{\kappa}' / 1.25$
$c_{k}' / 1.25$
$c_{u,k} / 1.4$
$q_{u,k}$ 1.4
$\gamma' / 1.0$

Design by calculation – methodology

Detailed step by step design methodology and worked examples







1 Establish site constraints: site access, constructability constraints

2 Establish ultimate and serviceability limit states

3 Review ground and groundwater conditions

4&5 Select construction sequence and wall type and assign geotechnical category

6 Determine design stratigraphy

Design Approach 1: Ultimate limit state calculation

For each design situation:
- transient
- persistent
- seismic (if applicable)

7
- Determine characteristic soil and rock parameters
- Determine most unfavourable groundwater pressures
- Determine actions during
  - construction stage; and
  - operational stage
- Determine design geometry: allow for unplanned excavation

Design Approach 1: Combination 1 (DA1C1)

Design Approach 1: Combination 2 (DA1C2)

8 Apply partial factors (>1: see Table 7.1) to characteristic soil and rock parameters and to unfavourable variable surcharges

9 Determine wall depth for vertical stability e.g. load bearing capacity, hydraulic cut off and uplift

10 Determine direction and magnitude of wall friction and adhesion at soil/wall interface to satisfy vertical stability

11 Carry out limit equilibrium calculations or soil - structure interaction analysis for satisfactory lateral stability

12 Determine design wall depth as the deeper of that obtained from Steps 9 and 11

13 Determine (from Step 11) DA1C2 design effects of actions (wall bending moments, shear forces and prop/anchor forces)

14 Apply partial factors (unity: see Table 7.1) to characteristic soil and rock parameters and (>1: see Table 7.1) to unfavourable variable surcharges

15 Carry out limit equilibrium calculations to determine wall depth and extrapolate calculated wall bending moments to the full wall depth determined from Step 12 or soil–structure interaction analysis using wall depth determined in Step 12

16 Determine DA1C1 design effects of actions (wall bending moments, shear forces and prop/anchor forces) by applying a partial factor of 1.35 to the calculated values of wall bending moments, shear forces and prop/anchor forces from Step 15

17 Are serviceability limit state (SLS) calculations or accidental design situation / progressive failure checks required?

NO

YES

Serviceability limit state (SLS) calculations

Accidental Design Situation / Progressive Failure Check

18
- Adopt characteristic soil and rock parameters with a partial factor of unity
- Adopt most unfavourable groundwater pressures
- Adopt actions with a partial factor of unity
- Determine design geometry: no allowance for unplanned excavation

19 Carry out limit equilibrium calculations to determine wall depth and extrapolate calculated wall bending moments to the full wall depth determined from Step 12 or soil–structure Interaction analysis using wall depth determined in Step 12

20
- Determine SLS wall deflections and ground movements from the SLS calculation output in Step 19 together with guidance provided in Chapter 6
  - Check compliance with pre-defined SLS movements criteria
  - Carry out building damage assessment (see chapter 6)
- Determine SLS design effects of actions (wall bending moments, shear forces and prop/anchor forces) by applying a partial factor of unity to calculated values of wall bending moments, shear forces and prop/anchor forces from Step 19

21
- Adopt characteristic soil and rock parameters with a partial factor of unity
- Determine accidental/ extreme groundwater pressures
- Determine accidental/ extreme actions with a partial factor of unity e.g. loss of a prop/anchor
- Determine design geometry

22 Carry out limit equilibrium calculations to determine wall depth and extrapolate calculated wall bending moments to the full wall depth determined from Step 12 or soil–structure Interaction analysis using wall depth determined in Step 12

- Determine accidental design situation/ progressive failure check design effects of actions (wall bending moments, shear forces and prop/anchor forces) by applying a partial factor of unity to calculated values of wall bending moments, shear forces and prop/ anchor forces from Step 22
- Check calculated wall deflections and ground movements do not exceed ULS criteria, if any

23 Adopt the greater of the design effects of actions (wall bending moments, shear forces and prop/anchor forces) from the DA1C1 (Step 16), DA1C2 (Step13) and accidental/progressive failure (Step 22) calculations for the ULS structural design of the wall and its support system (see section 7.5)

Design by calculation – parameter selection

EC7-1 Section 2.4.5.2 states

“(2) (P) The characteristic value of a geotechnical parameter shall be selected as a cautious estimate of the value affecting the occurrence of the limit state”

What does this mean in reality?

Design by calculation – parameter selection

EC7-1 Section 2.4.5.2 states

“(2) (P) The characteristic value of a geotechnical parameter shall be selected as a cautious estimate of the value affecting the occurrence of the limit state”

What does this mean in reality?

Does statistics have a role?

Design by calculation – parameter selection

EC7-1 Section 2.4.5.2 states

“(2) (P) The characteristic value of a geotechnical parameter shall be selected as a cautious estimate of the value affecting the occurrence of the limit state”

What does this mean in reality?

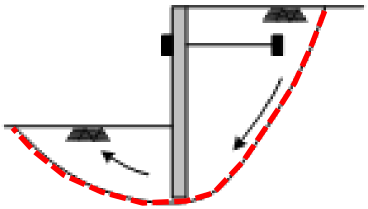
Does statistics have a role?

Is it different to values defined in CIRIA 104, BS8002 or C580?

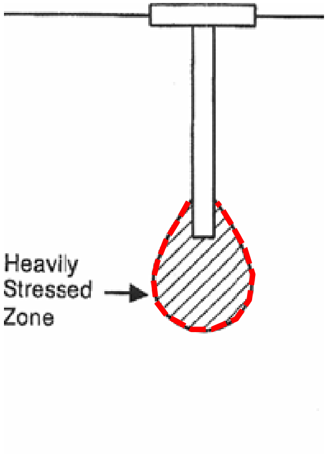
Design by calculation – parameter selection

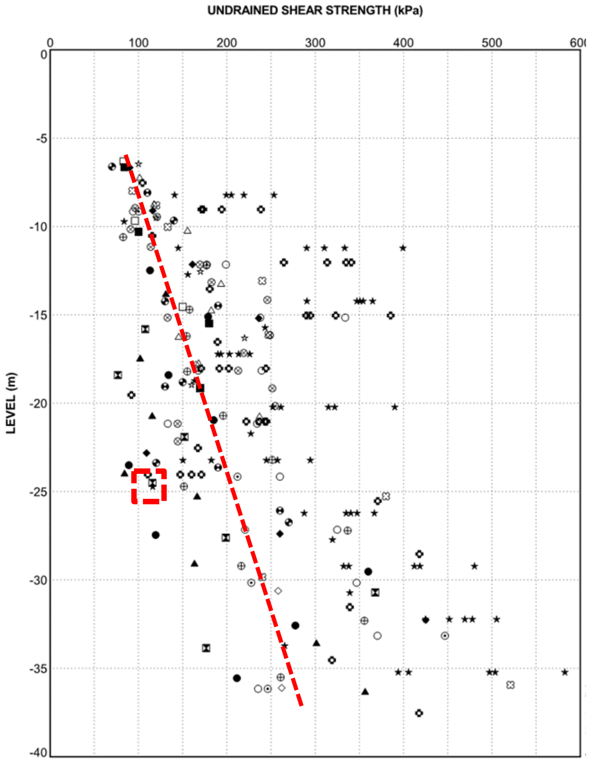
Value affecting the occurrence of the limit state

Embedded retaining wall at ULS



End bearing pile at ULS





Design by calculation – parameter selection

The characteristic value is not a “best estimate” or a “statistical mean” but is a function of the problem being considered

Cautious estimate of the mean

Moderately Conservative in CIRIA 104 & C580

Representative in BS 8002

=

Characteristic in EC7

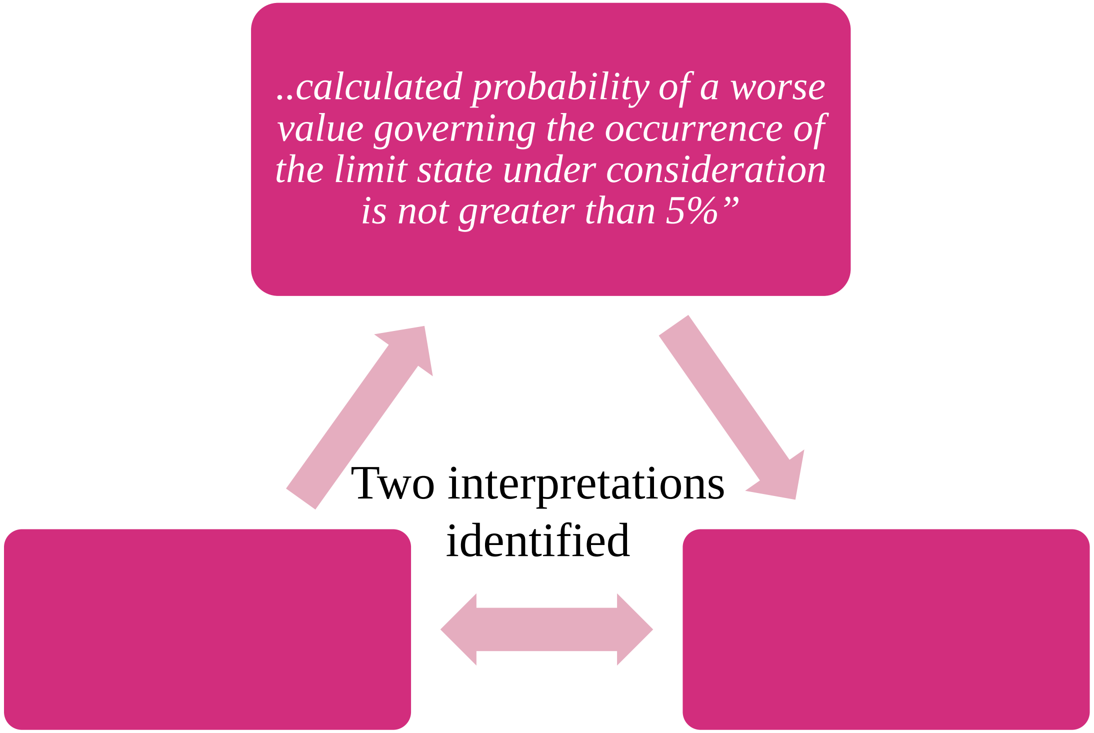
Design by calculation – parameter selection

The role of statistics:

“(11) If statistical methods are used, the characteristic value should be derived such that the calculated probability of a worse value governing the occurrence of the limit state under consideration is not greater than 5%”

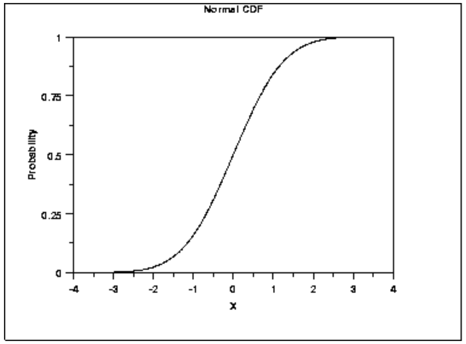
NOTE In this respect, a cautious estimate of the mean value is a selection of the mean value of the limited set of geotechnical parameter values, with a confidence level of 95%; where local failure is concerned, a cautious estimate of the low value is a 5% fractile

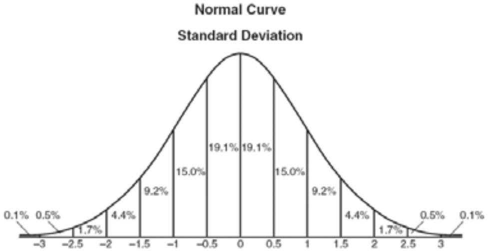
Design by calculation – parameter selection



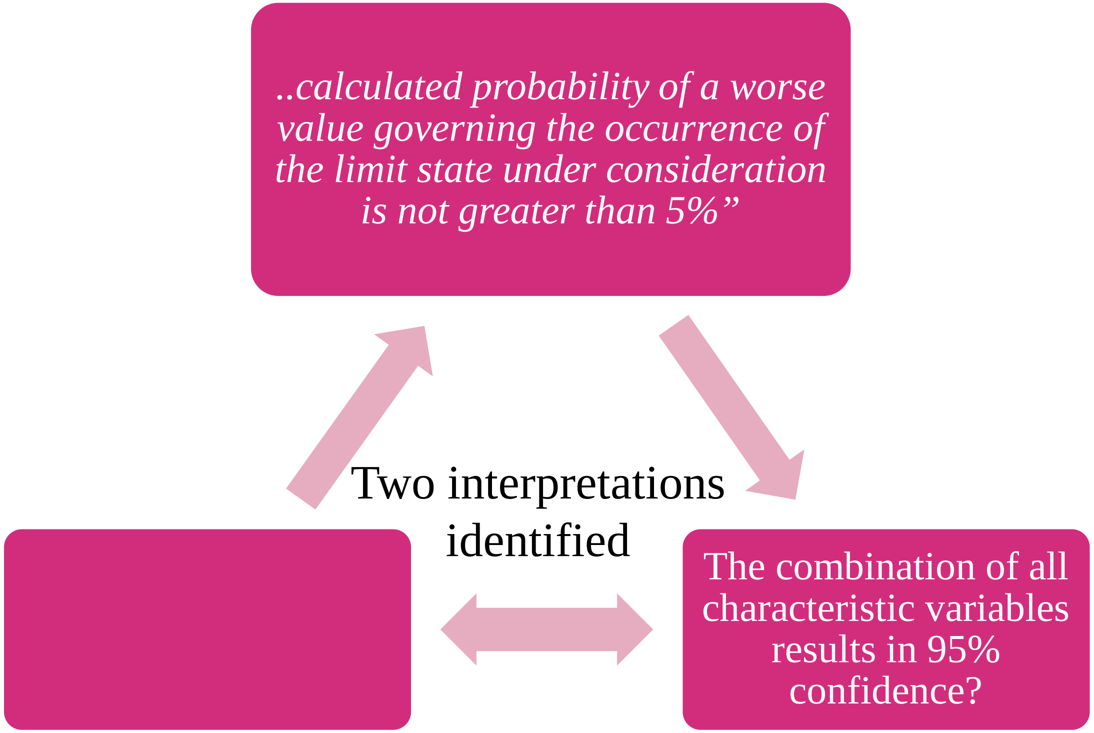
..calculated probability of a worse value governing the occurrence of the limit state under consideration is not greater than 5%”

Two interpretations identified





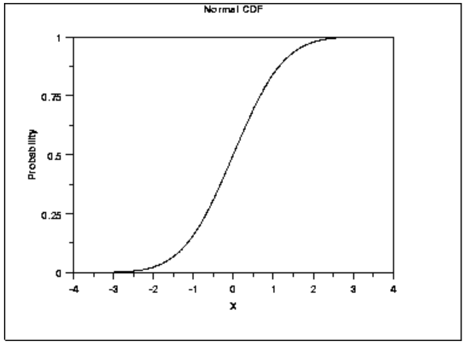
Design by calculation – parameter selection

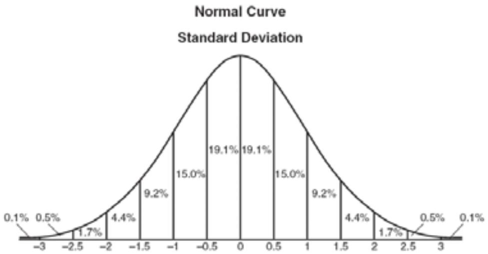


..calculated probability of a worse value governing the occurrence of the limit state under consideration is not greater than 5%”

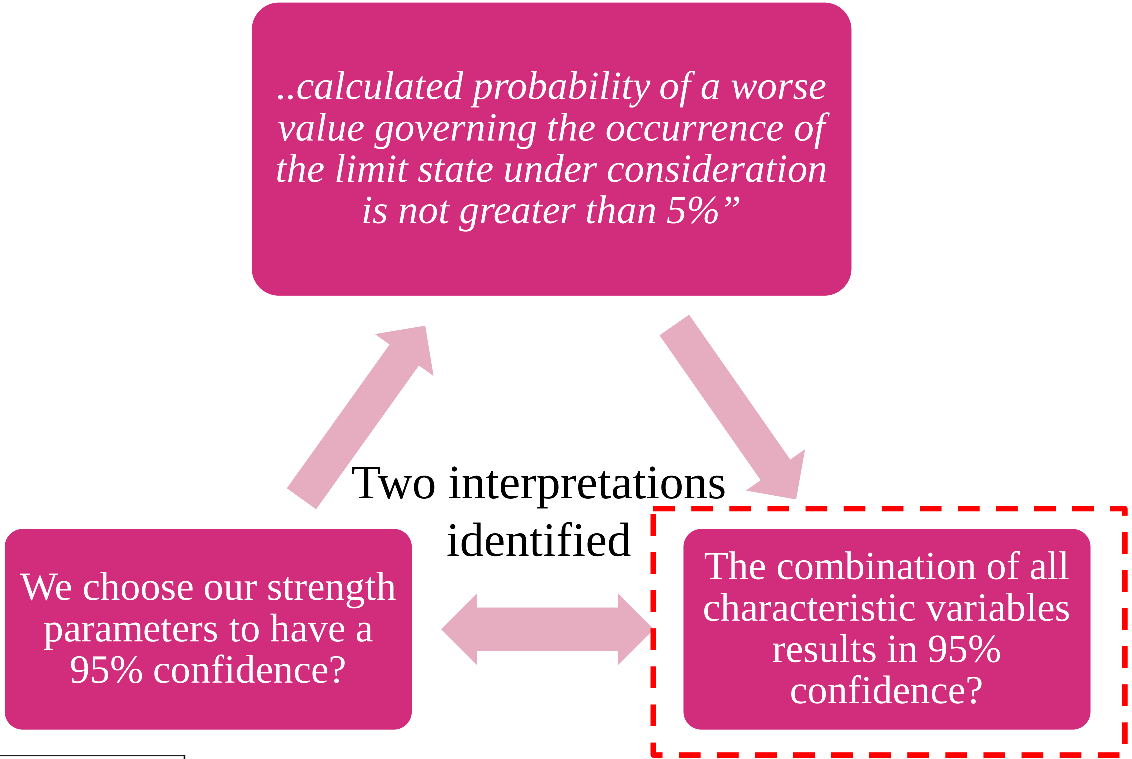
Two interpretations identified

The combination of all characteristic variables results in 95% confidence?





Design by calculation – parameter selection

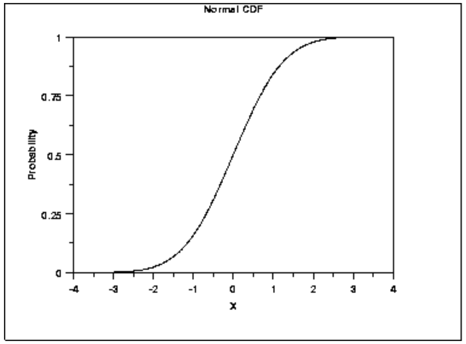


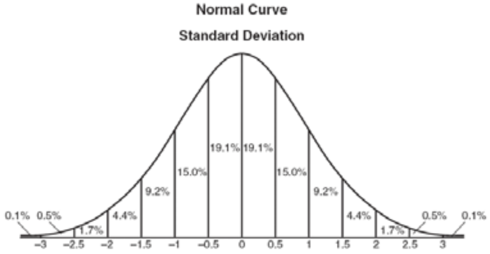
..calculated probability of a worse value governing the occurrence of the limit state under consideration is not greater than 5%"

Two interpretations identified

We choose our strength parameters to have a 95% confidence?

The combination of all characteristic variables results in 95% confidence?

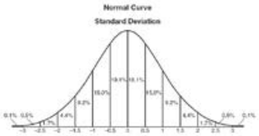




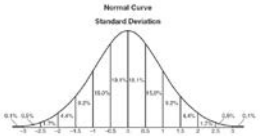
Design by calculation – parameter selection

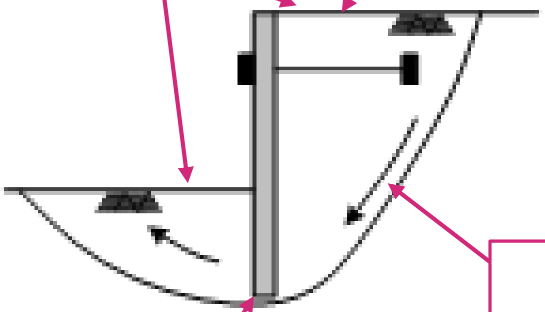
To satisfy the second…..

Ground water levels

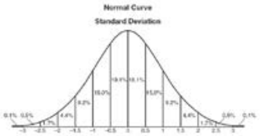


Surcharge values

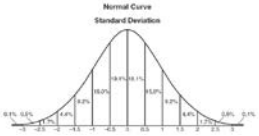




Soil parameters

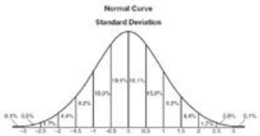


Structural properties

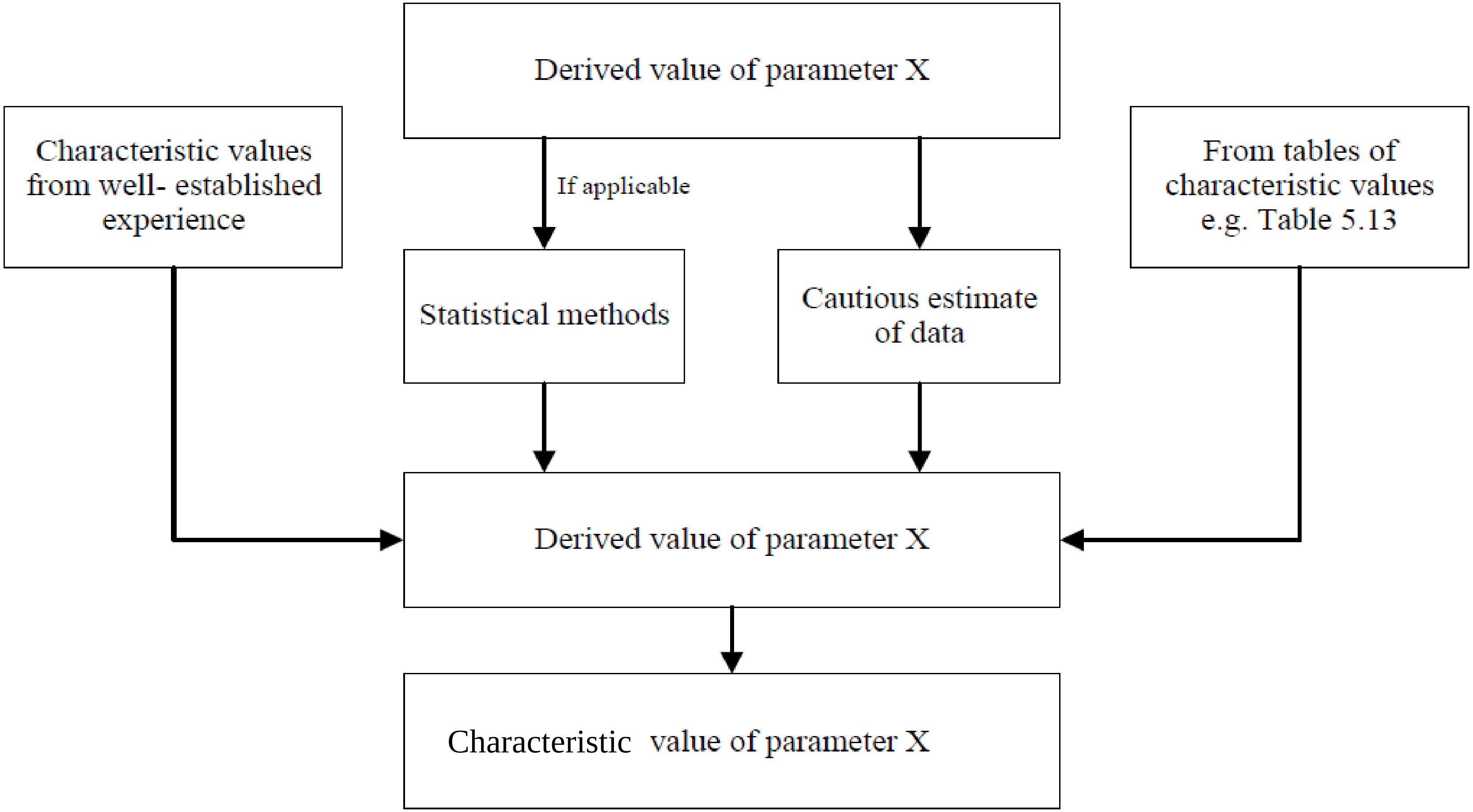


Monte Carlo Analysis

Probability Distribution of ULS



Design by calculation – parameter selection



Derived value of parameter X

If applicable

Statistical methods

Cautious estimate of data

Characteristic values from well- established experience

From tables of characteristic values e.g. Table 5.13

Derived value of parameter X

Characteristic value of parameter X

Note: There is no unique characteristic value X_k – there may be several characteristic values of X_k in relation to the limit states under consideration

Design by calculation – parameter selection

Table 5.14 *Values for φ'_{ang}, φ'_{PSD} and φ'_{dil} to obtain values of $\varphi'_{cv,k}$ and $\varphi'_{pk,k}$ for siliceous sands and gravels*

$$\varphi'_{pk,k} = \varphi'_{cv,k} + \varphi'_{dil} \qquad \varphi'_{cv,k} = 30° + \varphi'_{ang} + \varphi'_{PSD}$$

Soil Property	Determined from	Classification	Parameter[4]
Angularity of particles[1]	Visual description of soil	Rounded to well-rounded	$\varphi'_{ang} = 0°$
		Sub-angular to sub-rounded	$\varphi'_{ang} = 2°$
		Very angular to angular	$\varphi'_{ang} = 4°$
Uniformity coefficient, C_U[2]	Soil grading	$C_u < 2$ (even graded)	$\varphi'_{PSD} = 0°$
		$2 \leq C_u \leq 6$ (even graded)	$\varphi'_{PSD} = 2°$
		$C_u \geq 6$ (medium to multi-graded)	$\varphi'_{PSD} = 4°$
		Gap graded with Cu of fines[5] < 2	$\varphi'_{PSD} = 0°$
		Gap graded with Cu of fines[5] = 2 - 6	$\varphi'_{PSD} = 2°$
Density index, I_D[3]	Standard penetration test blow count corrected for energy rating and overburden pressure $(N_1)_{60}$ (as described in Section 5.1 above)	$I_D = 0\%$	$\varphi'_{dil} = 0°$
		$I_D = 25\%$	$\varphi'_{dil} = 0°$
		$I_D = 50\%$	$\varphi'_{dil} = 3°$
		$I_D = 75\%$	$\varphi'_{dil} = 6°$
		$I_D = 100\%$	$\varphi'_{dil} = 9°$

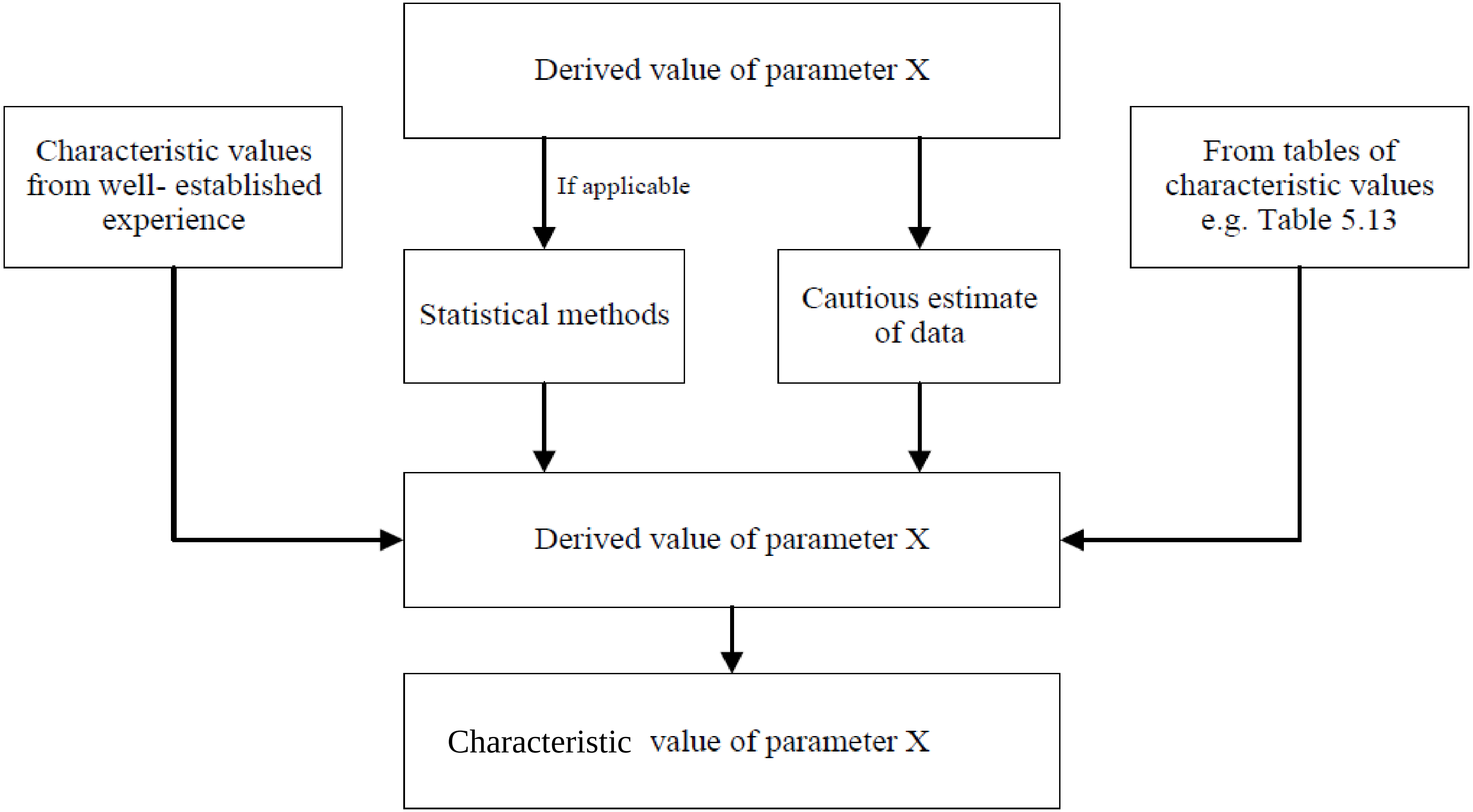
Table 5.11 *Suggested values for the characteristic weight density of fill*

Type of Material	Dry (kN/m³)		Saturated (kN/m³)	
	Loose	Dense	Loose	Dense
Gravel	16	18	20	21
Well graded sand and gravel	19	21	21.5	23
Coarse or medium sand	16.5	18.5	20	21.5
Well graded sand	18	21	20.5	22.5
Fine or silty sand	17	19	20	21.5
Rock fill and quarry waste	15	17.5	19.5	21
Brick hardcore	13	17.5	16.5	19
Slag fill	12	15	18	20
Ash fill	6.5	10	13	15
Topsoil	16	19	20	21
River mud	14.5	17.5	19	20
Silt	18		18	
Very soft clay	16		16	
Soft clay	17		17	
Firm clay	18		18	
Stiff clay	19		19	
Very stiff clay or hard clay	20.0-21.0		20.0-21.0	

Table 5.13 *$\varphi'_{cv,k}$ for clay soils*

Plasticity index (%)	$\phi'_{cv,k}$ (degrees)
15	27
30	24
50	21
80	18

Design by calculation – parameter selection



Derived value of parameter X

Characteristic values from well- established experience

If applicable

Statistical methods

Cautious estimate of data

From tables of characteristic values e.g. Table 5.13

Derived value of parameter X

Characteristic value of parameter X

Note: There is no unique characteristic value X_k – there may be several characteristic values of X_k in relation to the limit states under consideration

Design by calculation – design considerations

Water levels

Choice of φ’

Wall friction

Unplanned excavation

Design by calculation – design considerations

Water levels

Choice of φ’

Wall friction

Unplanned excavation

Design considerations – water levels

Some questions to ask yourself:

- Is the ground water level the same for combination 1 and 2?
- Should we take account of accidental cases such as a flood or burst water main in the combination 1 and 2 analyses?
- Should the water levels be the same in the short and the long term?
- In combination 1 should we be applying a partial factor of 1.35 to a most onerous water level?

Design considerations – water levels

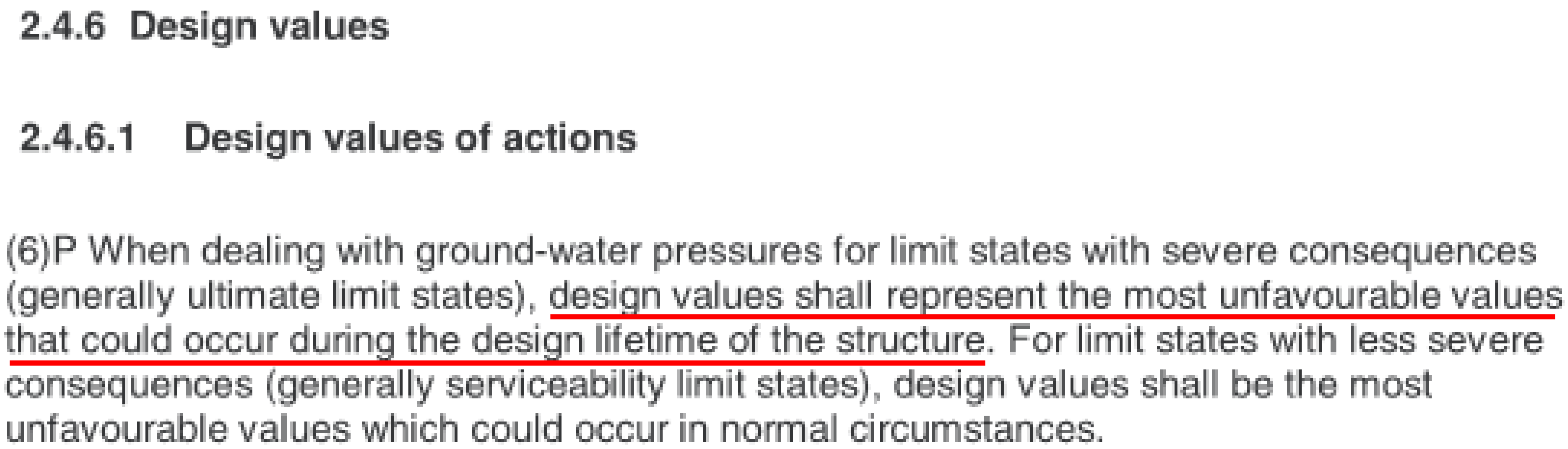
A common approach is…..

- Combination 1 – "moderately conservative" estimate of normal ground water conditions which is then factored by 1.35 with the other effects of actions (i.e. wall bending moment, shear force and prop/anchor forces).

- Combination 2 – "worst credible" level in short term and long term

Design considerations – water levels

EC7-1



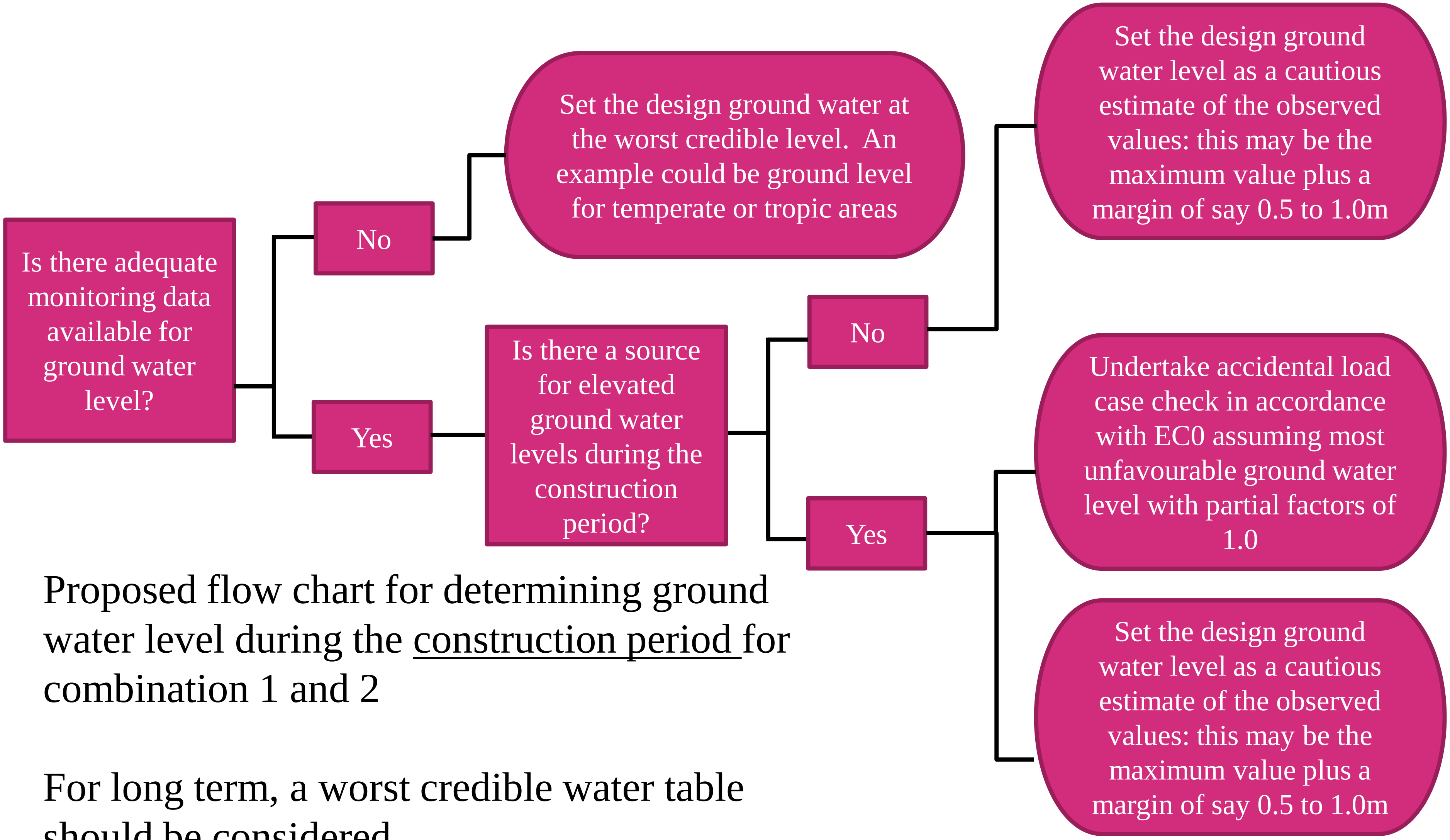
2.4.6 Design values

2.4.6.1 Design values of actions

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

- They should be the same for combination 1 and 2 as they are both ULS analyses.
- The designer can choose between factoring water pressures or adjusting water levels to derive design water pressures.
- Eurocodes (EC0) allow us to treat accidental cases differently

Design considerations – water levels



Is there adequate monitoring data available for ground water level?

No

Set the design ground water at the worst credible level. An example could be ground level for temperate or tropic areas

Yes

Is there a source for elevated ground water levels during the construction period?

No

Set the design ground water level as a cautious estimate of the observed values: this may be the maximum value plus a margin of say 0.5 to 1.0m

Yes

Undertake accidental load case check in accordance with EC0 assuming most unfavourable ground water level with partial factors of 1.0

Set the design ground water level as a cautious estimate of the observed values: this may be the maximum value plus a margin of say 0.5 to 1.0m

Proposed flow chart for determining ground water level during the construction period for combination 1 and 2

For long term, a worst credible water table should be considered

Design by calculation – design considerations

Water levels

Choice of φ’

Wall friction

Unplanned excavation

Design by calculation – design considerations

Water levels

Choice of φ’

Wall friction

Unplanned excavation

Design considerations – choice of φ’

- There are a number of φ’ values that can be chosen for a retaining wall analysis, but the most common are φ'_{peak}, φ'_{cv} and φ'_{res}. So which value to choose?

- Designer to adopt characteristic value of ϕ’ *affecting the occurrence of the limit state* under consideration

Design by calculation – design considerations

Water levels

Choice of φ’

Wall friction

Unplanned excavation

Design by calculation – design considerations

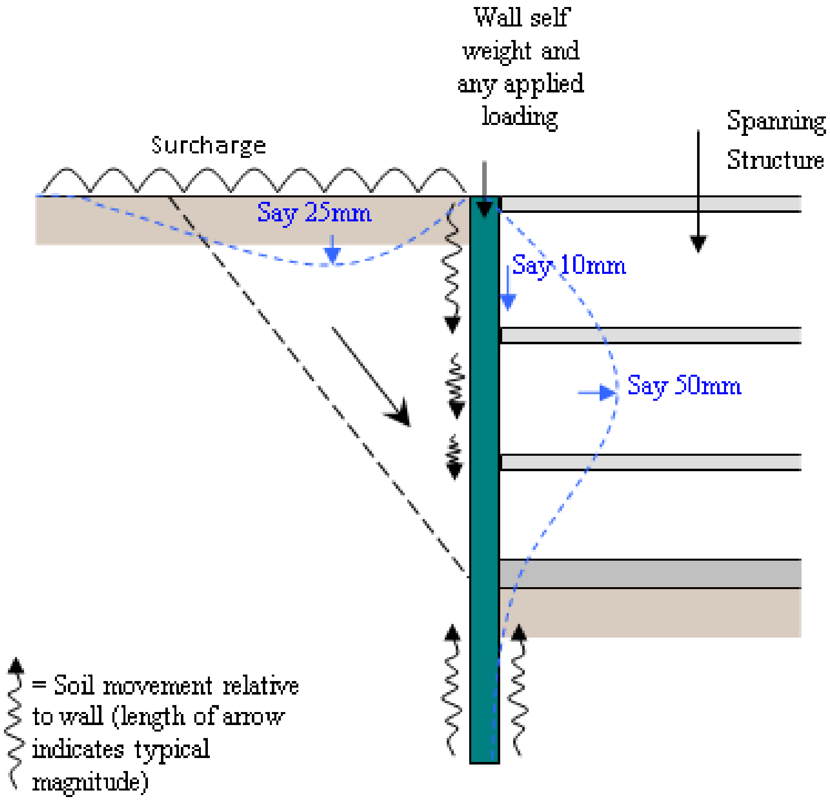
Water levels

Choice of φ’

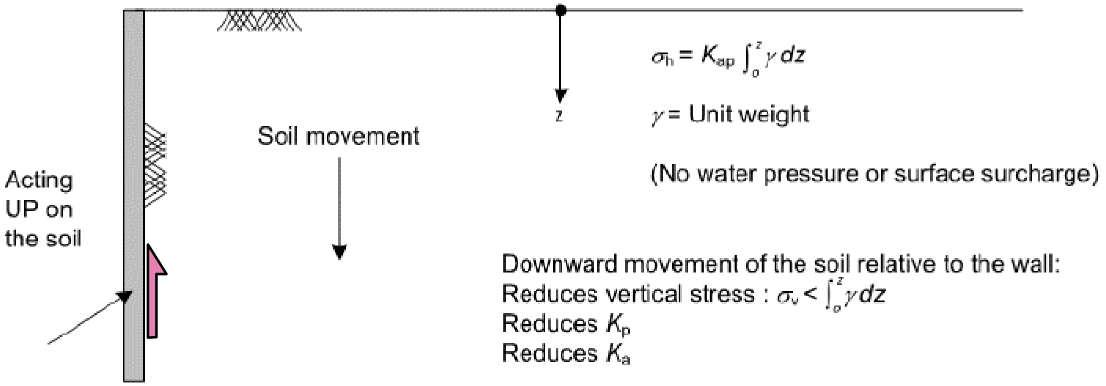
Wall friction

Unplanned excavation

Design considerations – wall friction



Design considerations – wall friction



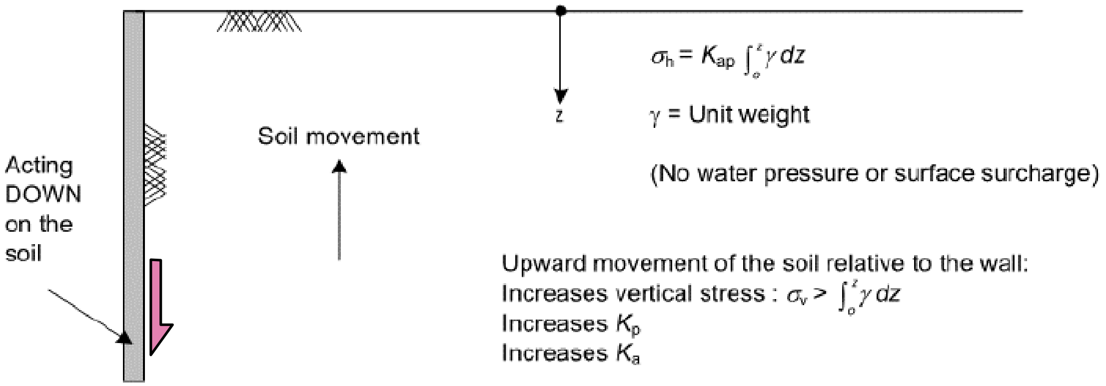
$\sigma_h = K_{ap} \int_o^z \gamma \, dz$

γ = Unit weight

(No water pressure or surface surcharge)

Downward movement of the soil relative to the wall:
Reduces vertical stress : $\sigma_v < \int_o^z \gamma dz$
Reduces K_p
Reduces K_a

(a) Wall friction: downward movement of the soil relative to the wall



$\sigma_h = K_{ap} \int_o^z \gamma \, dz$

γ = Unit weight

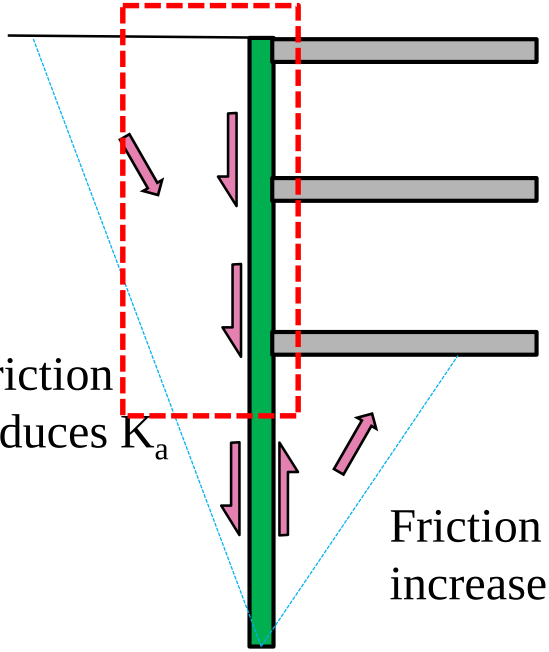
(No water pressure or surface surcharge)

Upward movement of the soil relative to the wall:
Increases vertical stress : $\sigma_v > \int_o^z \gamma \, dz$
Increases K_p
Increases K_a

(b) Wall friction: upward movement of the soil relative to the wall

Design considerations – wall friction

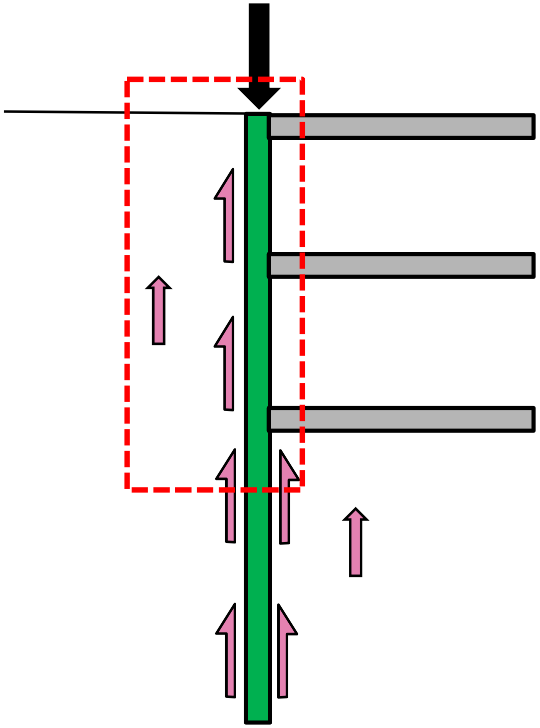
Acting as a retaining wall



Friction reduces K_a

Friction increases K_p

Acting as a foundation

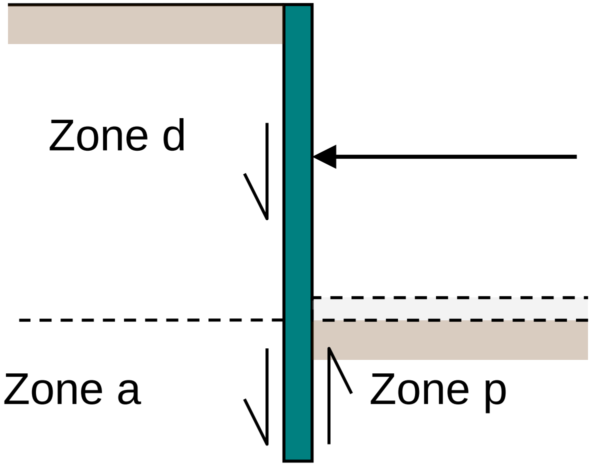


⇨ Direction of soil movement relative to wall

⇀ Friction

Design considerations – wall friction

Example 1: where retaining wall is not part of the foundation system
- wall not load bearing



<u>Zones a,d and p:</u>

for coarse grained soils
use $\delta_d = + k \varphi'_{cv,d}$

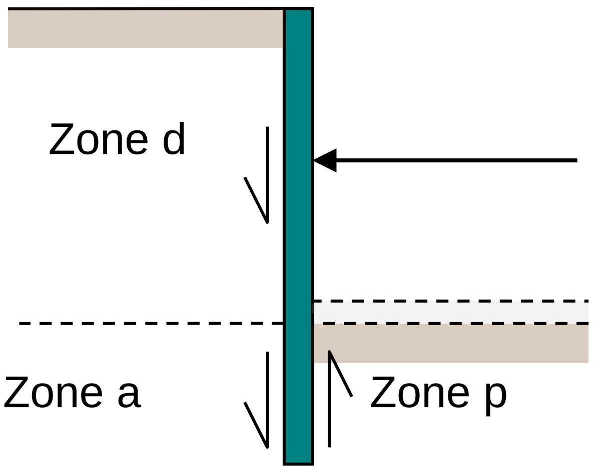
for fine grained soils
use $c_{w,d} = 0.5c_{u,d}$ (short term)
$c'_{w,d} = 0$ (long term)

The value of *k* is typically 1.0 for rough walls and 0.67 for smooth walls

Design considerations – wall friction

Example 2: for bottom up construction sequence with retaining wall forming part of foundation system – wall not load bearing in short term

Short Term



Zones a,d and p:

for coarse grained soils
use $\delta_d = + k\, \varphi'_{cv,d}$

for fine grained soils
use $c_{w,d} = 0.5c_{u,d}$

The value of *k* is typically 1.0 for rough walls and 0.67 for smooth walls

Design considerations – wall friction

Example 2: for bottom up construction sequence with retaining wall forming part of foundation system – wall load bearing in long term

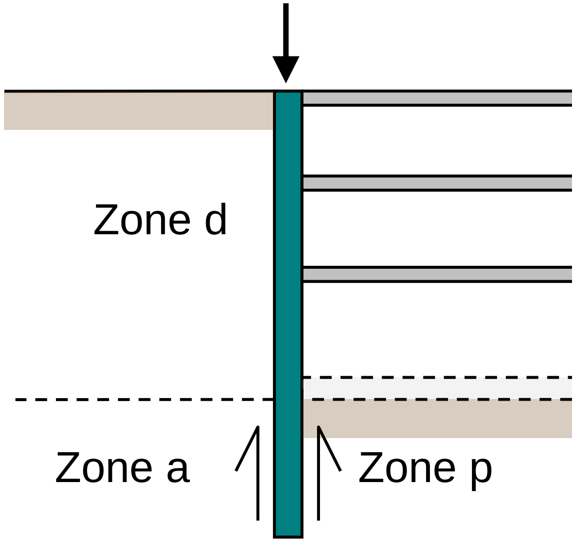
Long Term

Zone d:

for coarse grained soils
use $\delta_d = 0$

Zone a: (note opposite sign)

for coarse grained soils
use $\delta_d = - k\,\varphi'_{cv,d}$



Zones d, a and p

for fine grained soils
use $c'_{w,d} = 0$

Zone p:

for coarse grained soils
use $\delta_d = + k\,\varphi'_{cv,d}$

The value of *k* is typically 1.0 for rough walls and 0.67 for smooth walls

Design considerations – wall friction

Example 3: for top down construction with load bearing retaining wall (in short term as well as long term)

Zone d:

use $\delta_d = 0$

Zone a:

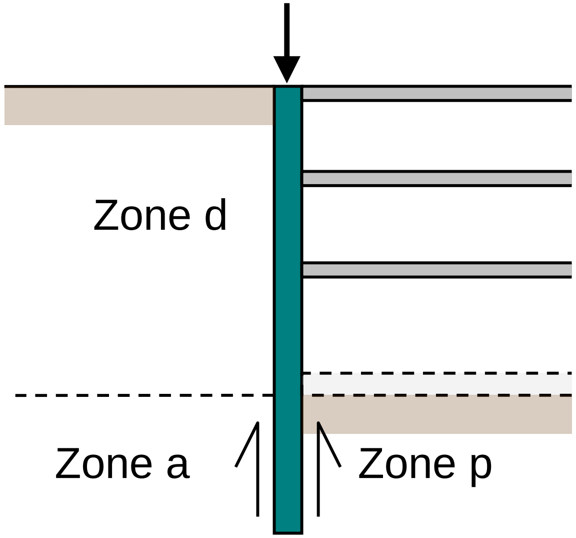
for coarse grained soils,

use $\delta_d = -k\,\varphi'_{cv,d}$

for fine grained soils

use $c_{w,d} = -0.5c_{u,d}$ (short term)

$c'_{w,d} = 0$ (long term)



Zone p:

for coarse grained soils,

use $\delta_d = +k\,\varphi'_{cv,d}$

for fine grained soils

use $c_{w,d} = +0.5c_{u,d}$ (short term)

$c'_{w,d} = 0$ (long term)

The value of *k* is typically 1.0 for rough walls and 0.67 for smooth walls

Design by calculation – design considerations

Water levels

Choice of φ’

Wall friction

Unplanned excavation

Design by calculation – design considerations

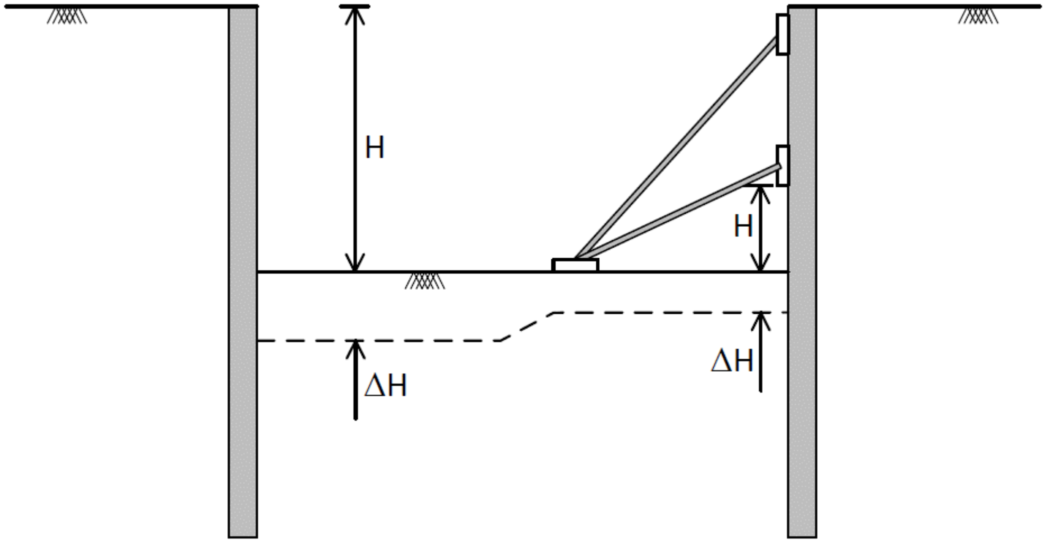
Water levels

Choice of φ’

Wall friction

Unplanned excavation

Design considerations – unplanned excavation



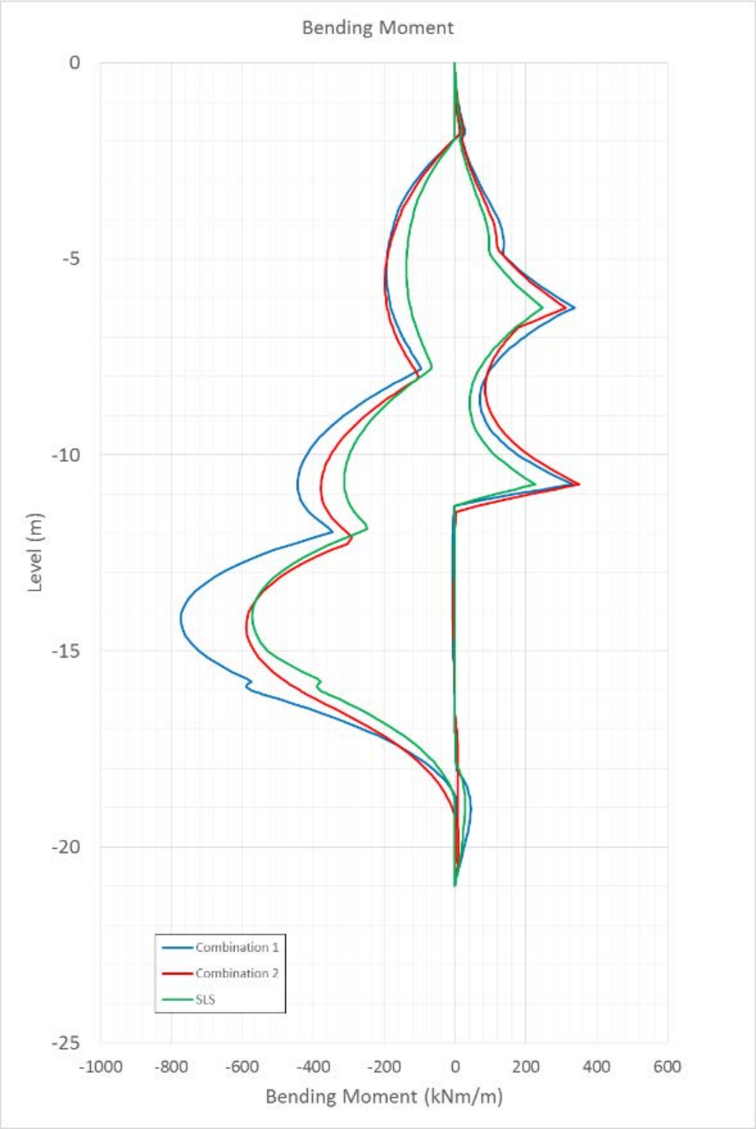
EC7-1 Section 9.3.2.2(2):

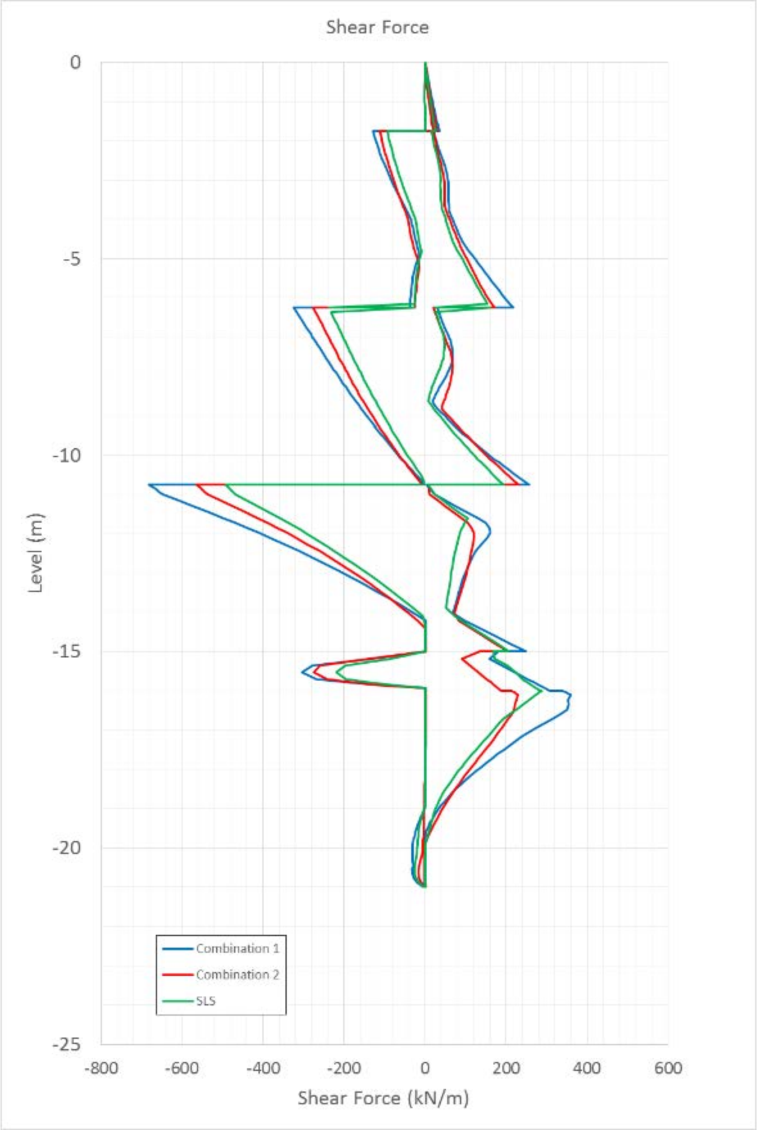
- Allowance of 0.5m, or
- 10% of the total height for cantilever walls or the height retained below the lowest support level for propped or anchored walls
- This may be reduced to zero if “adequate engineering supervision” is specified
- C760 recommends a practical minimum of 0.1m

Summary of design by calculation

- Limit states to consider
  - Ultimate limit state
  - Serviceability limit state
- Design Approach 1
  - Partial factors
  - How to apply to embedded retaining walls
- Choosing Characteristic parameters
  - “Applicable to the occurrence of the limit state”
  - Is problem dependent
- Design considerations
  - Water levels/choice of φ’/wall friction and adhesion
  - Unplanned excavation

Summary of design by calculation





Presentation Outline

- **Introduction: EC7 compliant design of embedded retaining walls**

- **Design by calculation**
  - EC7 fundamentals: limit states
  - Design Approach 1: combination 1 and 2 and associated partial factors
  - Parameter selection
  - Choice of water levels/φ'/wall friction and adhesion/unplanned excavation

- **Design by The Observational Method: outline**

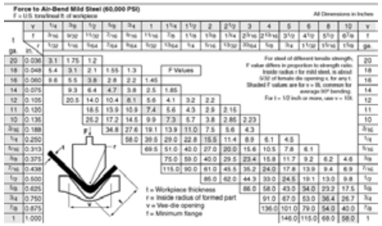
- **Other significant revisions to C580 guidance**
  - Derivation of weak rock parameters
  - Update and expansion of ground movements database
  - Choice of wall : construction tolerances, use of nomograms and typical structural details
  - Guidance on the use of finite element analysis for ULS calculations
  - EC2 and EC3 compliant structural design of wall and its support system
  - Durability & plastic design of steel walls and crack width considerations & reinforcement design of reinforced concrete walls

- **Closing comments**

Eurocode 7 Design

EC7-1 Clause 2.1(4) permits design to be undertaken:

- by calculation



- by prescriptive measures

- by experimental models and load tests

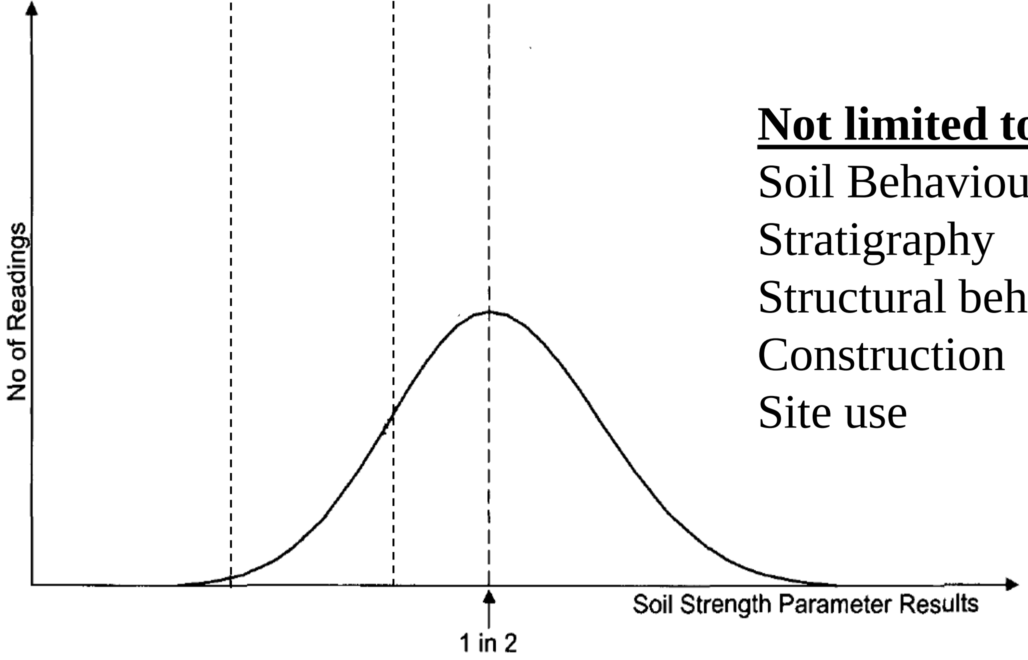
- by The Observational Method

The Observational Method

Definition (C185):

"The Observational Method in ground engineering is a continuous, managed, integrated process of design, construction control, monitoring and review that enables previously defined modifications to be incorporated during or after construction as appropriate. The objective is to achieve greater overall economy without compromising safety. The method can be adopted from the inception of a project, or later if benefits are identified"

The Observational Method: Parameter definition



Worst credible

Characteristic

Most probable

<u>Not limited to soil parameters</u>

Soil Behaviour

Stratigraphy

Structural behaviour

Construction

Site use

The Observational Method

Chapter 8 of C580 - “Areas of further work and research”:

> *“Significant cost savings can be achieved by adopting a risk based approach to design and construction through the use of the Observational method”*

The Observational Method

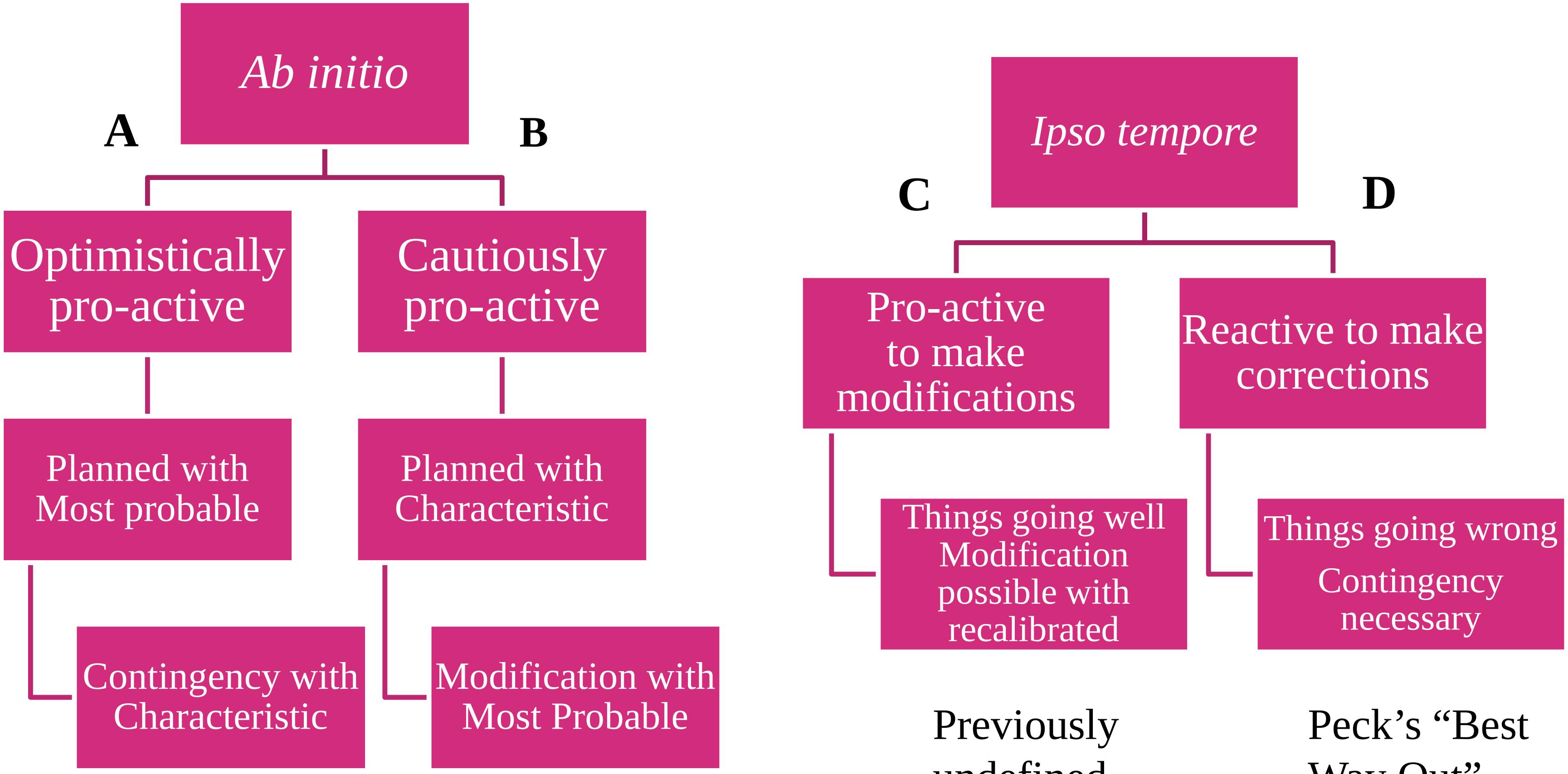
Ab Initio = From the start

Ipso Tempore = In the moment

Tempura

The Observational Method

New framework proposed in C760:



Ab initio

A

B

Optimistically pro-active

Cautiously pro-active

Planned with Most probable

Planned with Characteristic

Contingency with Characteristic

Modification with Most Probable

Ipso tempore

C

D

Pro-active to make modifications

Reactive to make corrections

Things going well Modification possible with recalibrated

Things going wrong Contingency necessary

Previously undefined

Peck’s “Best Way Out”

Ab initio in Peck 1969

Ab initio in C185

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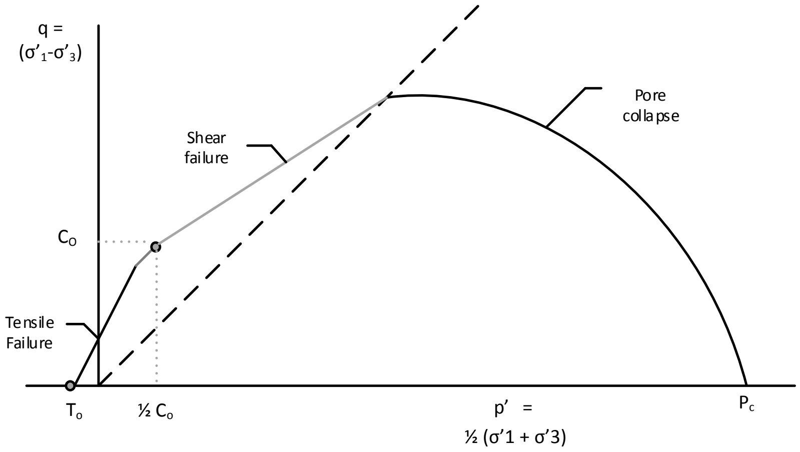
- **Closing comments**

Derivation of parameters for weak rock

- Use of the Hoek-Brown criteria

$$\sigma'_1 = \sigma'_3 + \sigma_{ci}\left(m_b \frac{\sigma'_3}{\sigma_{ci}} + s\right)^a$$



- Guidance is given on deriving equivalent Mohr-Coulomb parameters for a given stress level: worked example in Appendix 5
- K_0 generally taken as 1.0, but rapidly drops to K_a

Ground movements

Ground movements are caused by :

- *Wall installation*
- *Stress relief in the ground* due to excavation in front of the wall
- *Wall deflection due to excavation* in front of the wall (effects of excavation geometry, construction sequence, type and stiffness of wall and its support system, ground stiffness and strength, etc.)
- *Other causes* e.g. changes in groundwater pressure, loss of ground by water flow, impact of nearby ground improvement installations such as grouting (permeation and compensation), installation of shallow excavations (e.g. for drains), etc.
- *Quality of workmanship and construction control*

Any one of these factors may control movement: typically the result of complex interaction of all of the above in 3 dimensions

Ground movements

Ground movements cannot be predicted exactly.

They can be estimated by:

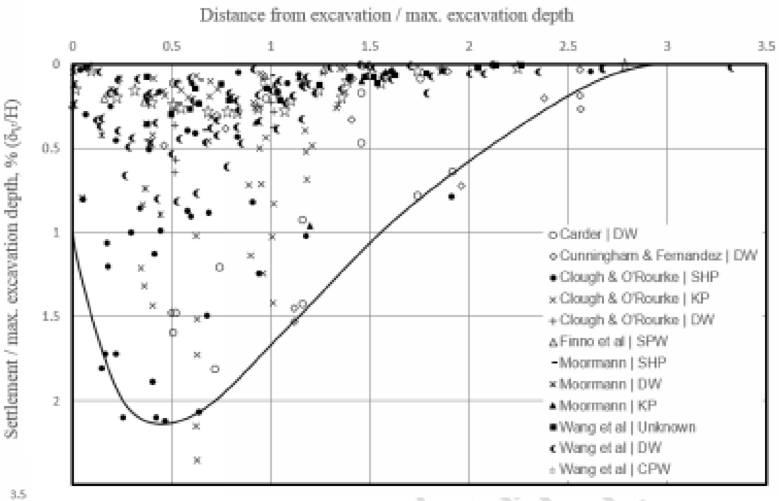
- *Empirical relationships* e.g. relating wall deflections to excavation depth and support type
- *Semi-empirical methods* e.g. pseudo-finite element methods
- *Numerical methods* analysing ground and structure with appropriate constitutive models (constant stiffness with no strain dependency will not yield reasonable predictions)
- *Other methods* – e.g. velocity fields, mobilised strength method, etc.

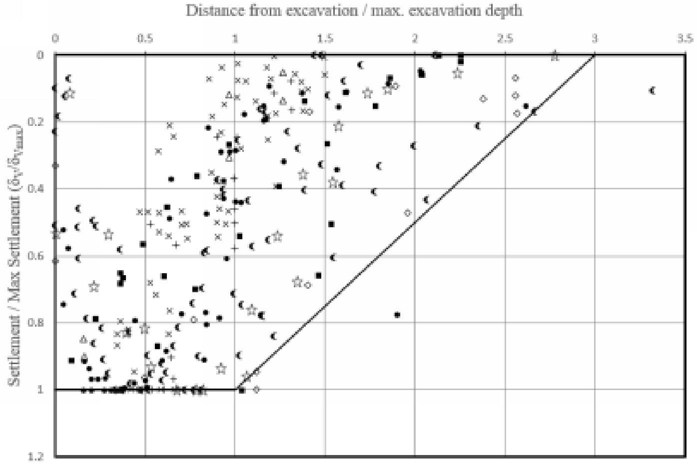
Essential to make optimum use of precedence in comparable conditions via good quality reliable case history data.

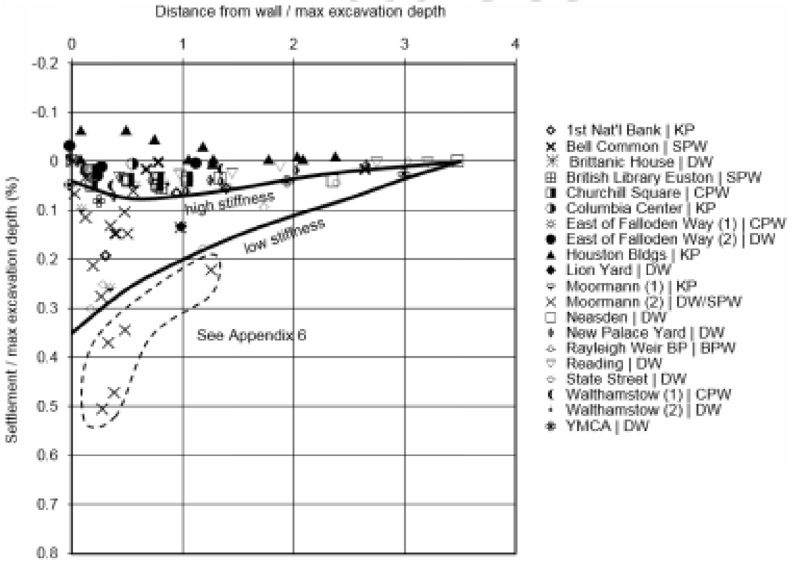
The designer must make every effort to understand the sequence and methods of construction (especially details of any local construction problems experienced)

Ground movement database

Additional case histories for soft soils



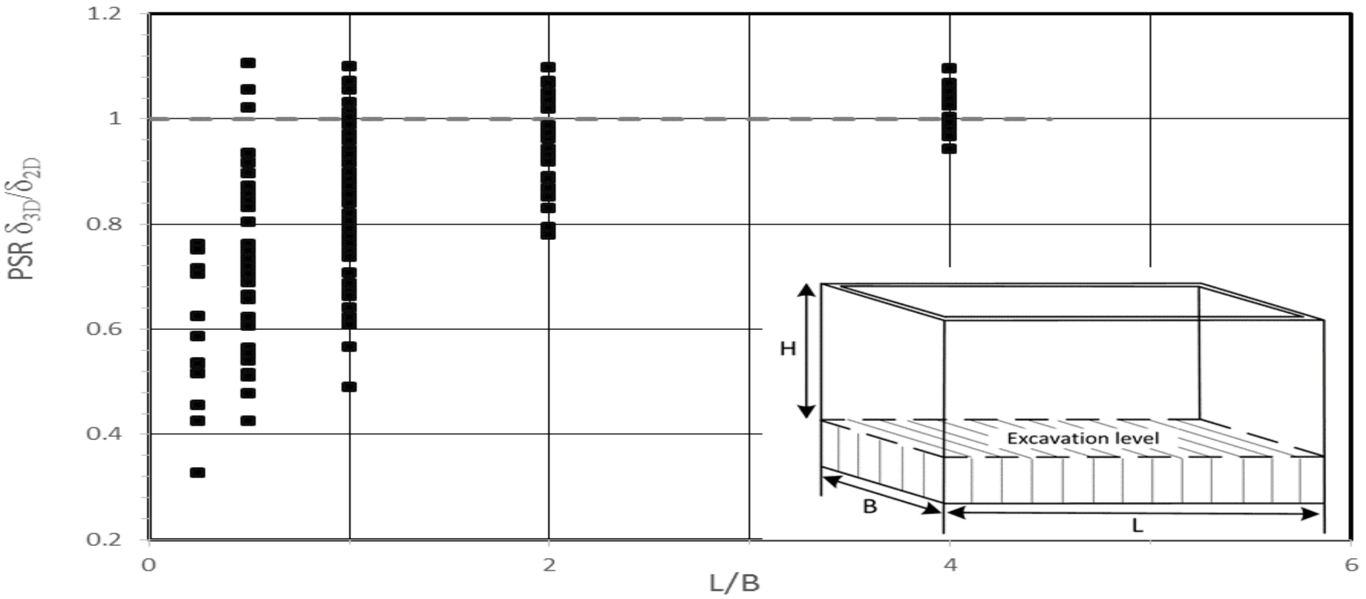


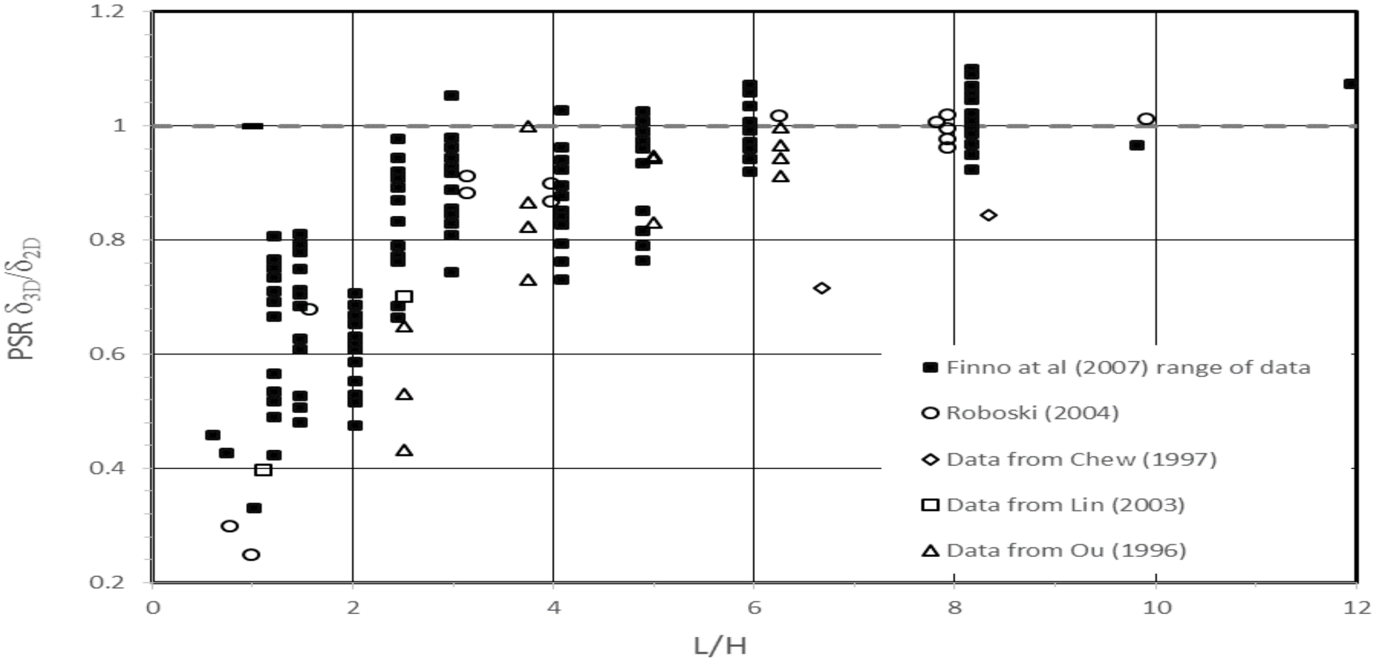


Additional data to augment existing graphs

Ground movement database

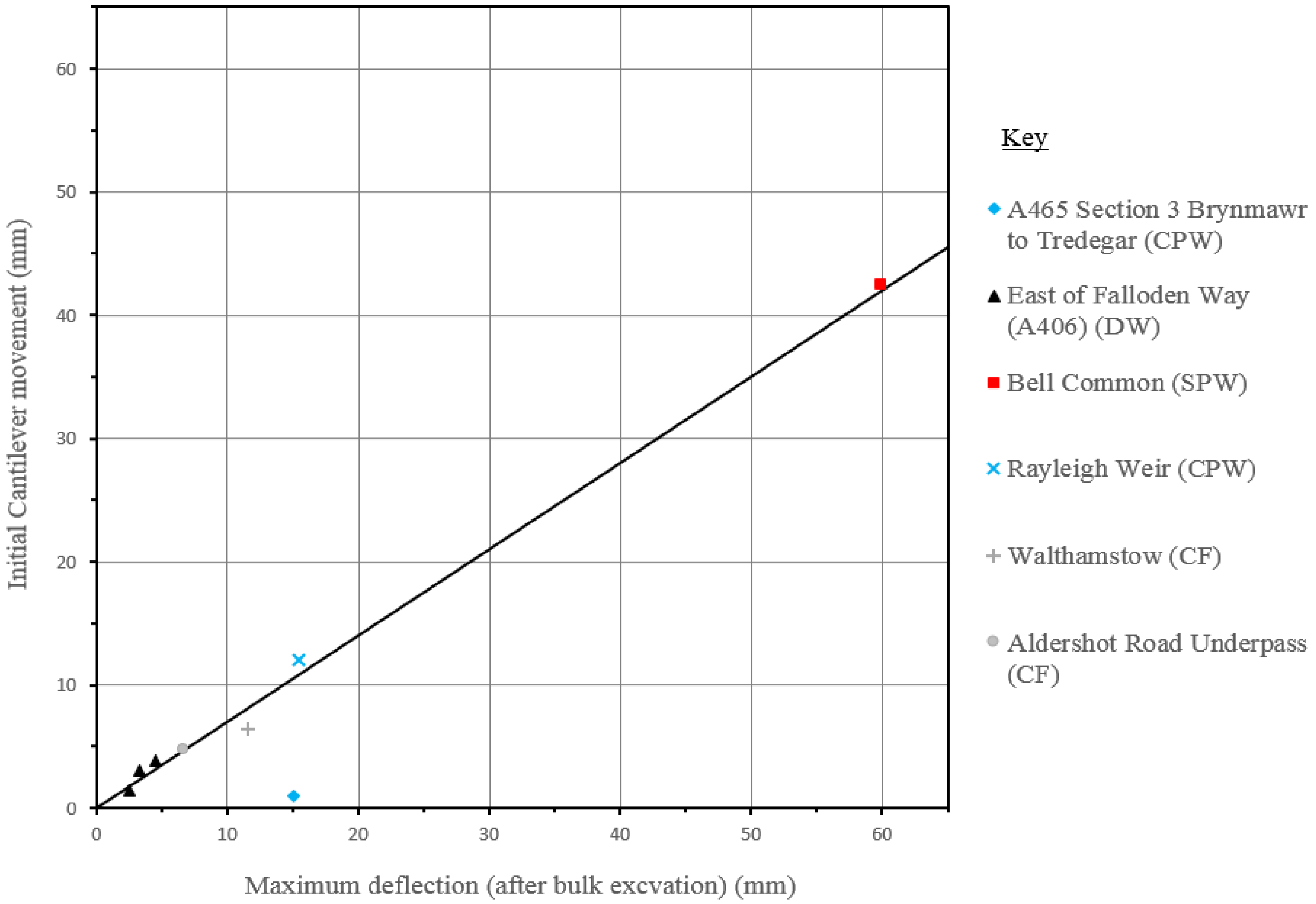
3D effects





Ground movement database

The initial deflection before first prop is installed can be the largest component of overall wall deflection – of the order of 70%



Control of ground movements

- Good workmanship essential. Supports should be tight to the wall
- The early installation of a stiff first prop with a shallow first stage excavation is one of the best ways to reduce wall deflections
- Ensure wall has adequate embedment in stiff strata
- Avoid unplanned over-excavation and minimise dig beyond support levels
- Minimise delays to the construction of the wall and its support system
- Prevent deterioration of lateral support e.g. from a clay berm
- Minimise removal of fines during dewatering
- Minimise drawdown outside excavation

Movements vs wall type

- Displacement flexibility number, $\Delta = EI / h^5$ (Addenbrooke *et al.*, 2000)

  where,

  EI is the Young's modulus multiplied by the second moment of area of the wall section per metre length, and

  h is the average vertical prop spacing of a multi-propped support system

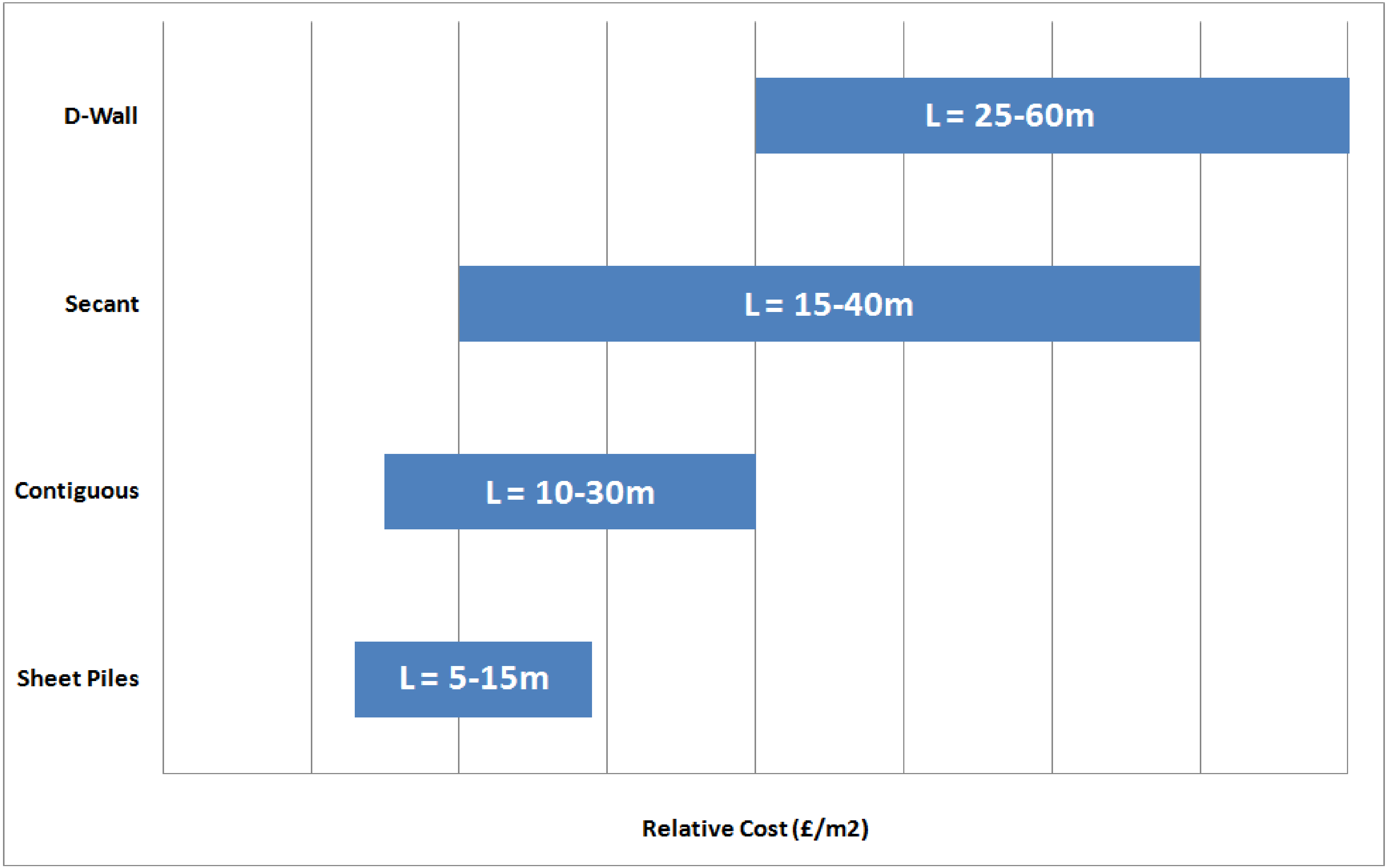
  By keeping Δ constant, various wall types can be considered for the same absolute displacement

- Flexible walls with many props (smaller h) will give similar displacement to stiff walls (greater EI) and fewer props (longer h)

Guidance on choice of wall type: updated typical tolerances

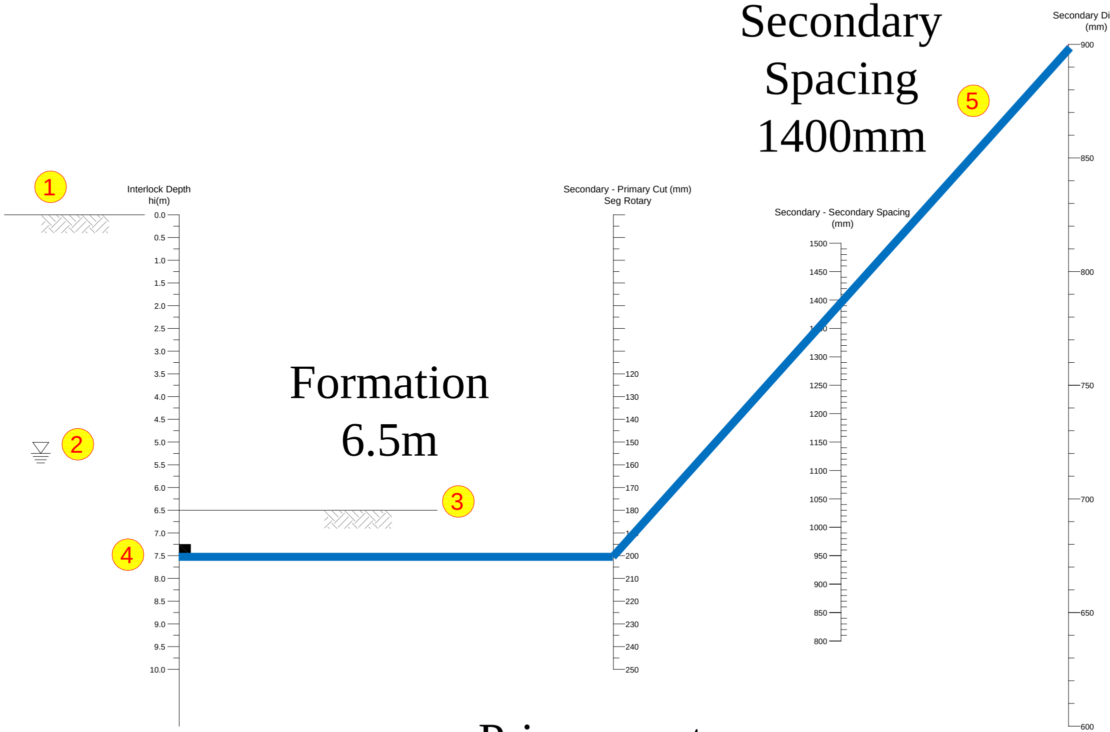
Wall Type	**Technique**	**Range of retained height**		**Verticality**		**Groundwater Control**	
		Cantilever	Propped	Typical	Best achievable	Temp.	Perm.
Sheet piles	Driven	≤ 5m	5-10m	1:75	1:100	Yes	Yes
King post	Conventional rotary bored or driven	≤ 4m	5-10m	1:75	1:75	No	No
Contiguous pile	CFA	≤ 8m	5-16m	1:75	1:100	No	No
	Conventional rotary bored	≤ 8m	5-25m	1:75	1:125		
Hard/soft secant	CFA	≤ 8m	5-16m	1:75	1:100	Yes	No
	Conventional rotary bored	≤ 8m	5-25m	1:75	1:125		
Hard/firm secant	CFA	≤ 8m	5-16m	1:75	1:100	Yes	Yes
Hard/firm & Hard/Hard secant	Cased rotary or cased-CFA using thick-wall casing	≤ 8m	5-25m	1:150	1:200	Yes	Yes
Diaphragm wall	Rope grab	≤ 8m	5-30m	1:100	1:150	Yes	Yes
	Hydraulic grab		5-40m	1:150	1:200		
	Mill		20-50m+	1:200	1:400		

Choice of wall type: relative costs



Guidance on choice of wall type

- Introduction of nomograms



Secondary
diameter
900mm

Secondary
Spacing
1400mm

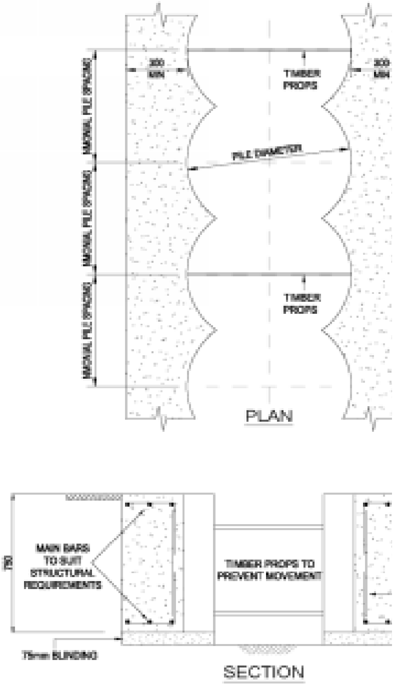
Formation
6.5m

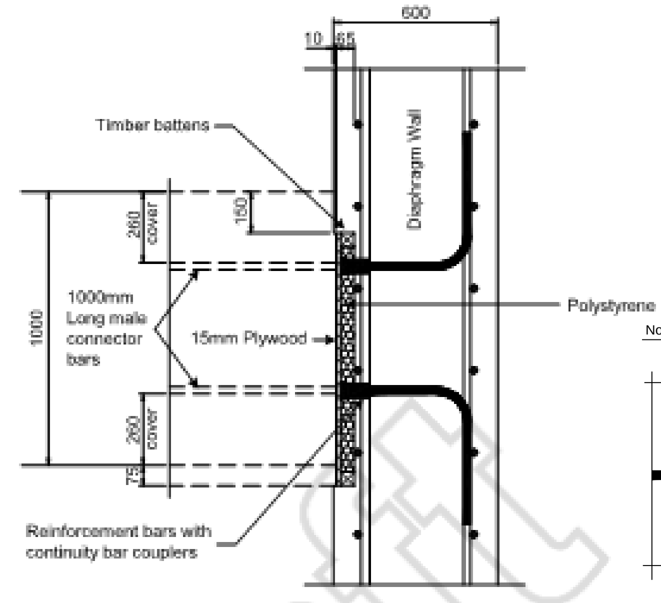
Interlock
depth = 7.5m

Primary cut =
200mm

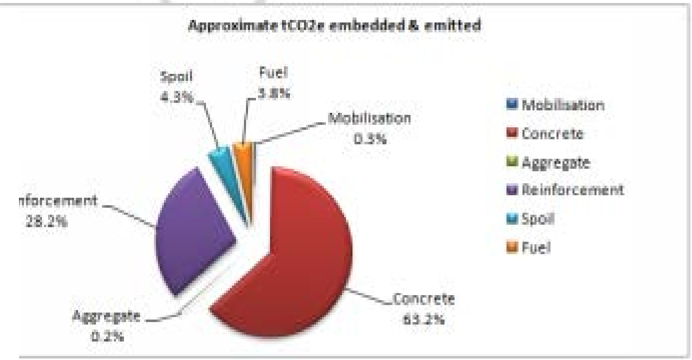
Guidance on choice of wall type

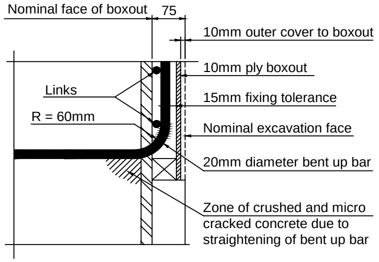
- Guidance on geo-thermal piles
- Guidance on carbon calculation
- Typical details for guide walls
- Typical details for connection details

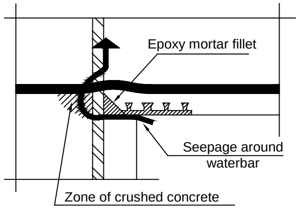




Sectional elevation through diaphragm wall showing the future slab connection

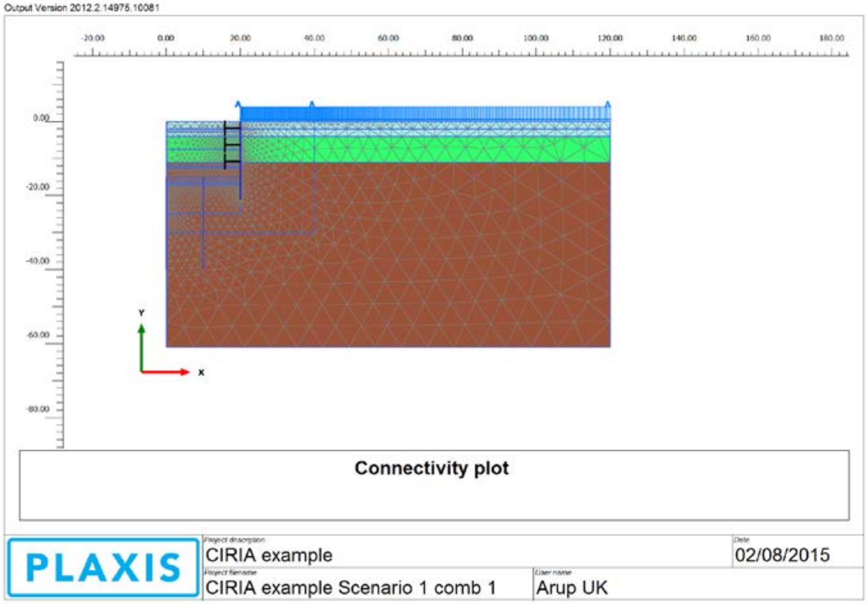






Guidance on the use of finite element analysis

- Fully worked exampled using PLAXIS
- Guidance on how to apply partial factors for Combination 2 analyses
  - Strategy 1
  - Strategy 2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Connectivity plot

Project description: CIRIA example
Date: 02/08/2015
Project filename: CIRIA example Scenario 1 comb 1
User name: Arup UK

- Ground movements predictions and building damage assessments

Guidance on the use of finite element analysis:

DA1 Combination 2

<u>Strategy 1</u>

- Carry out whole analysis with the soil strength already reduced by the requisite partial factors

<u>Strategy 2</u>

- Start and continue the analysis to each key stage with the full soil strength and then reduce soil strength by the required partial factor to investigate the effect on each stage separately

Either approach is acceptable – emerging preference for Strategy 2

Strategy 2 adopted in design example in C760

Structural design of wall and support system

- Alignment EC2, 3 and 7

- Prop design compatible with BS8002 (2015)

- Ground anchor design compatible with BS8081 (2015)

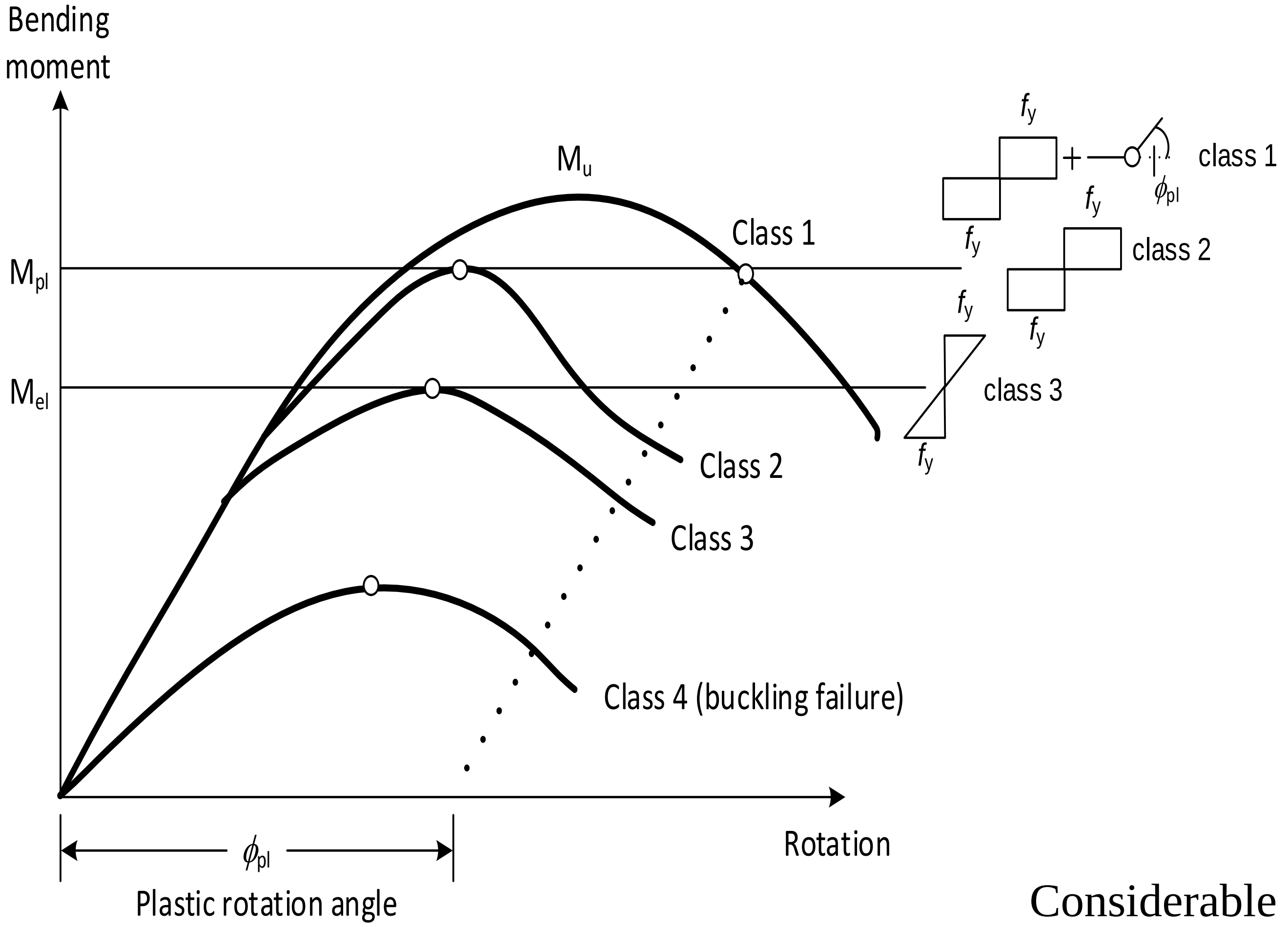
Durability of steel sheet pile walls: corrosion rates

Loss of thickness (mm) due to corrosion for steel piles in soils
– after Piling Handbook (Arcellor Mittal, 2008) and EC3-5

Required design working life	5 years	25 years	50 years	75 years	120 years
Undisturbed natural soils (sand, silt, clay, schist)	0.00	0.30	0.60	0.90	1.20
Polluted natural soils and industrial sites	0.15	0.75	1.50	2.25	3.00
Aggressive natural soils (swamp, marsh, peat)	0.20	1.00	1.75	2.50	3.25
Non-compacted and non-aggressive fills (clay, schist, sand, silt)	0.18	0.70	1.20	1.70	2.20
Non-compacted and aggressive fills (ashes, slag)	0.50	2.00	3.25	4.50	5.75

For steel piles exposed to fresh or seawater – see Table 4.3 EC3-5

Structural design of steel sheet pile walls to EC3-5



Class 1 and 2 sections:

$$M_{c,Rd} = \beta_B W_{pl} f_y / \gamma_{mo}$$

Class 3 sections:

$$M_{c,Rd} = \beta_B W_{el} f_y / \gamma_{mo}$$

Considerable material use reductions:

- Class 2 design : 15%
- Class 1 design : 25%

Structural design of reinforced concrete walls to EC2

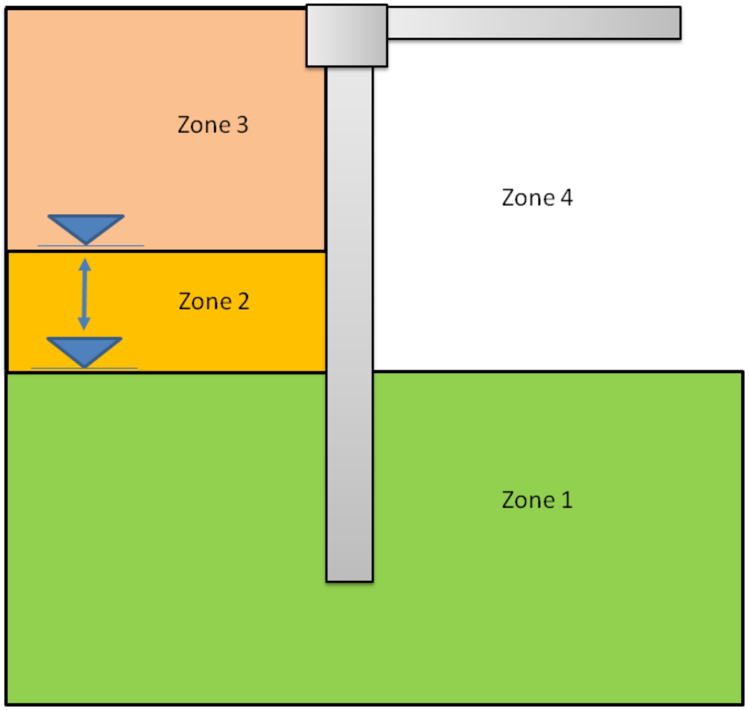
- reinforcement design

- Execution Standards BS EN 1536:2015 and BS EN 1538:2010 - minimum spacing between vertical reinforcement bars 100mm, can be reduced to 80mm provided maximum aggregate size < 20mm. Minimum spacing between longitudinal bars may be reduced to 80mm over the lap length.
- A lap length of 40 x main bar diameter away from high stress locations. In high stress areas, lap length should be calculated assuming good bond conditions provided cover to the main bar is 2 x main bar diameter and the wall is constructed in accordance with BS EN 1536:2015 or BS EN 1538:2010.
- Diaphragm wall panels: minimum clear spacing of 500mm at tremie locations to allow tremie pipe to be installed and withdrawn without snagging on the reinforcement.
- Where the embedded retaining wall supports vertical loads from the superstructure or basement slabs, the wall designer should consider the combinations of actions as described in EC0 to establish the ultimate actions under DA1C1 and DA1C2 combinations.
- Crack width calculations should be based on the bending moment distribution derived from an SLS calculation assuming the quasi-permanent combination of actions as described in EC0.

Structural design of reinforced concrete walls to EC2

- rationalisation of crack width considerations

- Zone 1 = wet
- Zone 2 = wet and dry
- Zone 3 = dry
- Zone 4 = atmospheric



Structural design of reinforced concrete walls to EC2

- rationalisation of crack width considerations

- Pragmatic approach to crack width control: savings possible.
- Specification of maximum crack width is a serviceability consideration, arises from concerns about durability, watertightness and aesthetics.
- Before making an assessment of what is an acceptable crack width, the designer should consider:
  - ground conditions (particularly permeability)
  - bending moment distribution (whether the section will be in tension or compression)
  - exposure class and corrosion conditions at a given crack location
  - crack widths as a function of time as well as depth (e.g. it may be acceptable to have a larger crack width during the transient design situation and a tighter limit in the persistent design situation)
- Location and orientation of crack more important than size: aligned vs transverse to main reinforcement
- Autogenous healing of cracks: generally accepted that a crack width of 0.2mm or less is likely to be self healing due to formation of calcium carbonate crystals, provided pressure gradient across section < 5.

Structural design of wall support system

- props

- Design prop load will depend on the analysis method adopted in calculation. Prop loads calculated from LE may be un-conservative as effects of SSI not included – the calculated prop loads should be increased by 15% to allow for stress redistribution and arching behind the wall in the case of multiple props. Where the wall is propped by a single prop, the LE calculated prop load should be increased by 30%

- The ULS structural prop actions derived from wall design calculations should be the greater of:

  $E_d = P_{ULS,d} \cdot \gamma_{Sd}$

  $E_d = \gamma_G \cdot P_{SLS} \cdot \gamma_{Sd}$

  $P_{ULS,d}$ is the design effects of actions determined from DA1C1 and DA1C2 calculations

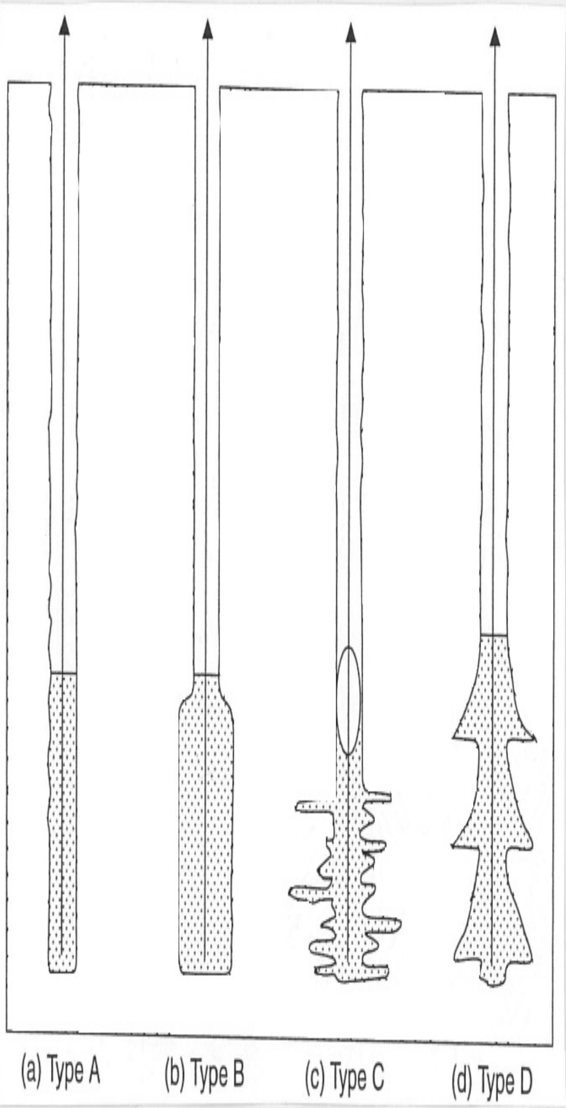
  γ_{Sd} = model factor = 1.0 for SSI analysis; 1.15 (or 1.30) for LE calculation for multi-propped (or singly propped) wall

  $\gamma_G = 1.35$

  P_{SLS} = calculated SLS effect of action

- Temperature effects: $Q_{k,temp} = \alpha . \delta t \, . \, E \, . \, A \, . \, (\beta/100)$ where, $\beta=0.3$ (flexible wall) $\beta=0.5$ (stiff wall)

Structural design of wall support system - ground anchorages

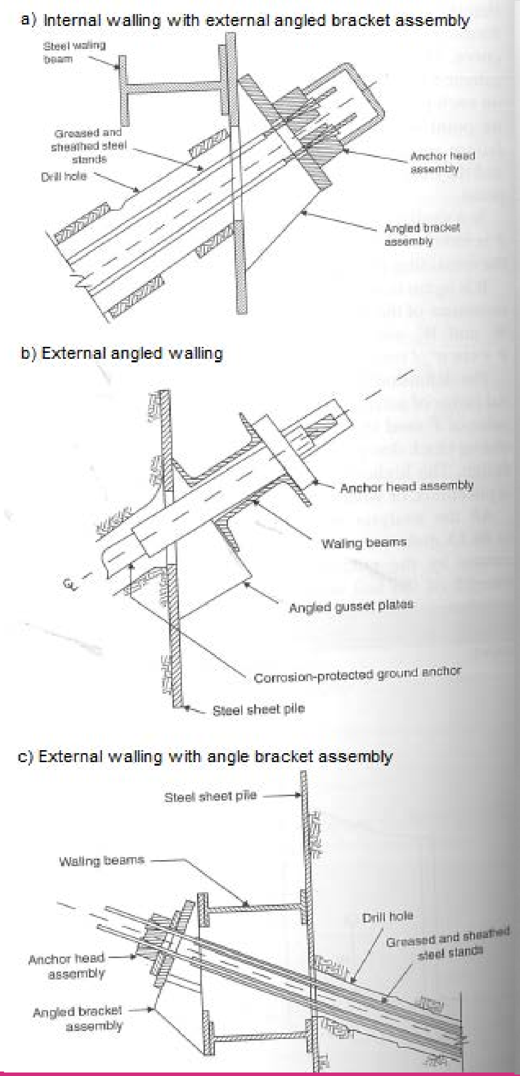


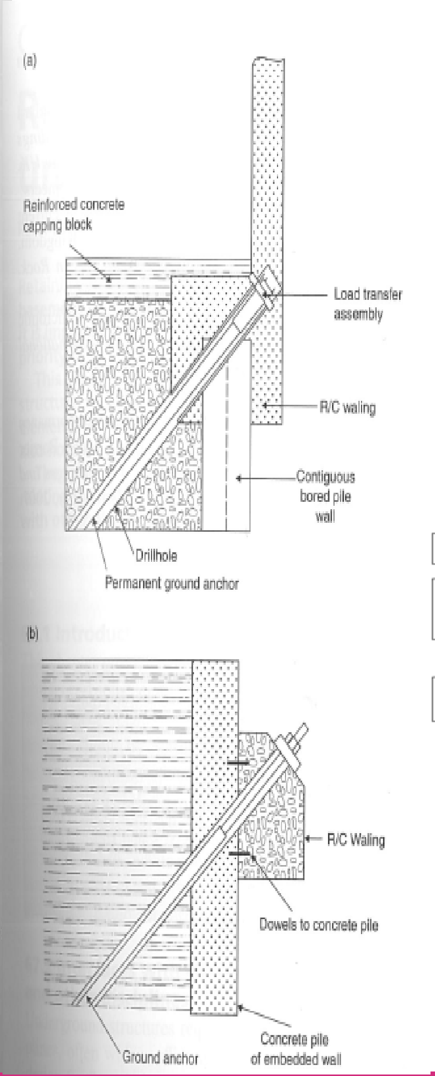
Type of ground anchorage	Typical characteristics	Typical ground conditions and design assumptions
Type A	Straight shafted borehole temporarily lined or unlined depending on borehole stability. Grout placed under gravity by tremie, packer or cartridge	Rock or very stiff to hard consistency fine grained soils. Resistance from side shear at ground/grout interface
Type B	Low pressure grouting (typically < 1 MPa) via lining tube or in situ packer at the top of the fixed anchor length. Effective diameter of the borehole increased as grout permeates/compacts ground locally along fixed anchor length	Weak fissured rocks and fine to coarse grained soils. Resistance primarily from side shear, but an end bearing component may be assumed in design when calculating ultimate capacity
Type C	High pressure grouting (typically > 2 MPa) via tube-a-manchette systems (or similar) allowing multiple phased injections (if required) along fixed anchor length. Ground hydro-fractured resulting in a grout fissure system extending beyond the nominal borehole diameter	Fine grained soils. Design assumes uniform shear along fixed anchor length
Type D	Borehole is enlarged by bells or under-reams along fixed anchor length. Grout placed under gravity by tremie	Can be used in coarse grained soils and cohesive soils in conjunction with local ground improvement (pre-injection of cement or chemical grout in the ground around the fixed anchor length prior to borehole drilling) to improve side wall stability over the enlarged length. Resistance from side shear and end bearing

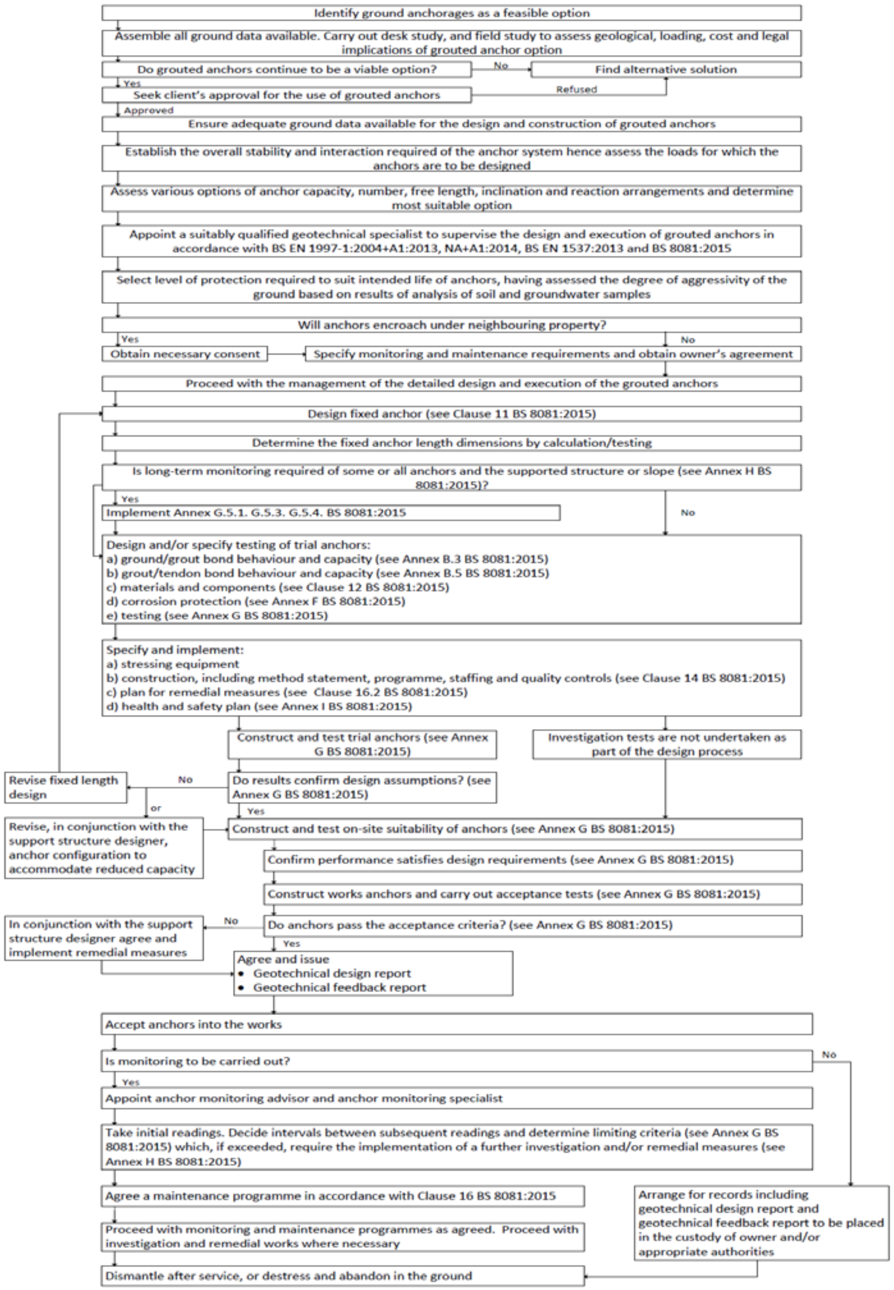
Structural design of wall support system - ground anchorages

Activity	Wall designer's responsibilities: overall design activities	Anchor designer's responsibilities: specialist activities	Key interfaces between the wall and the anchor designer
1	Acquisition of legal authorisation and entitlement to encroach on third party property (if required)		
2	Provision of site investigation data for the design and construction of ground anchorages – borings near fixed anchor locations and outside the site working area, as appropriate	Assessment of site investigation data with regard to design assumptions	Planning, scope and details of additional site investigation established in consultation between wall and anchor designer
3	Decision to use ground anchorages, required trials and testing and provision of a specification (see also 4 below)	Selection of ground anchorage type, components and details, including determination of fixed anchor dimensions	Common agreement on ground model at site and appropriateness of selected ground anchorage type and testing regime
4	Overall design of anchored structure, including calculations of horizontal restraint required and its level(s) of action. Determination of minimum free anchor length for overall wall stability. Specification of anchor spacing and orientation, anchor loads (SLS and ULS) for overall stability of the supported wall structure, including load transfer mechanism from anchors to the wall structure and any sequence of anchor loading required by the supported wall structure	Determination of individual anchor spacing and orientation and anchor loads (if not specified by the wall designer). Supply and install ground anchorage system. Execution and assessment of on-site anchor load testing	Common agreement that specific anchor arrangement satisfies design requirements of supported structure for overall vertical and lateral stability, recognising that anchor force inclinations will exert a vertical component of load on the supported wall structure. The free anchor length may be increased by the anchor designer to ensure that the fixed length is satisfactorily embedded in suitable founding strata
5	Definition of anchor life (permanent or temporary) and requirement for corrosion protection	Detailing of the specific corrosion protection system	
6	Specification for monitoring ground anchorage behaviour and for interpretation of the results	Supply and install ground anchorage monitoring system	Common agreement on monitoring frequency, trigger levels, reporting requirements and contingency measures (as appropriate)
7	Technical supervision of the works on site to ensure compliance with overall design intent of supported wall structure	Quality control of works on site to ensure compliance with specified requirements	Common agreement on key components of the overall design philosophy of the supported wall structure to which special attention should be directed
8	Maintenance specification for ground anchorages	Maintenance of ground anchorages as specified	

Structural design of wall support system ground anchorages






Identify ground anchorages as a feasible option

Assemble all ground data available. Carry out desk study, and field study to assess geological, loading, cost and legal implications of grouted anchor option

Do grouted anchors continue to be a viable option?

No

Find alternative solution

Yes

Seek client's approval for the use of grouted anchors

Refused

Approved

Ensure adequate ground data available for the design and construction of grouted anchors

Establish the overall stability and interaction required of the anchor system hence assess the loads for which the anchors are to be designed

Assess various options of anchor capacity, number, free length, inclination and reaction arrangements and determine most suitable option

Appoint a suitably qualified geotechnical specialist to supervise the design and execution of grouted anchors in accordance with BS EN 1997-1:2004+A1:2013, NA+A1:2014, BS EN 1537:2013 and BS 8081:2015

Select level of protection required to suit intended life of anchors, having assessed the degree of aggressivity of the ground based on results of analysis of soil and groundwater samples

Will anchors encroach under neighbouring property?

Yes

No

Obtain necessary consent

Specify monitoring and maintenance requirements and obtain owner's agreement

Proceed with the management of the detailed design and execution of the grouted anchors

Design fixed anchor (see Clause 11 BS 8081:2015)

Determine the fixed anchor length dimensions by calculation/testing

Is long-term monitoring required of some or all anchors and the supported structure or slope (see Annex H BS 8081:2015)?

Yes

Implement Annex G.5.1. G.5.3. G.5.4. BS 8081:2015

No

Design and/or specify testing of trial anchors:
a) ground/grout bond behaviour and capacity (see Annex B.3 BS 8081:2015)
b) grout/tendon bond behaviour and capacity (see Annex B.5 BS 8081:2015)
c) materials and components (see Clause 12 BS 8081:2015)
d) corrosion protection (see Annex F BS 8081:2015)
e) testing (see Annex G BS 8081:2015)

Specify and implement:
a) stressing equipment
b) construction, including method statement, programme, staffing and quality controls (see Clause 14 BS 8081:2015)
c) plan for remedial measures (see Clause 16.2 BS 8081:2015)
d) health and safety plan (see Annex I BS 8081:2015)

Construct and test trial anchors (see Annex G BS 8081:2015)

Investigation tests are not undertaken as part of the design process

Revise fixed length design

No

Do results confirm design assumptions? (see Annex G BS 8081:2015)

or

Yes

Revise, in conjunction with the support structure designer, anchor configuration to accommodate reduced capacity

Construct and test on-site suitability of anchors (see Annex G BS 8081:2015)

Confirm performance satisfies design requirements (see Annex G BS 8081:2015)

Construct works anchors and carry out acceptance tests (see Annex G BS 8081:2015)

In conjunction with the support structure designer agree and implement remedial measures

No

Do anchors pass the acceptance criteria? (see Annex G BS 8081:2015)

Yes

Agree and issue
- Geotechnical design report
- Geotechnical feedback report

Accept anchors into the works

Is monitoring to be carried out?

No

Yes

Appoint anchor monitoring advisor and anchor monitoring specialist

Take initial readings. Decide intervals between subsequent readings and determine limiting criteria (see Annex G BS 8081:2015) which, if exceeded, require the implementation of a further investigation and/or remedial measures (see Annex H BS 8081:2015)

Agree a maintenance programme in accordance with Clause 16 BS 8081:2015

Arrange for records including geotechnical design report and geotechnical feedback report to be placed in the custody of owner and/or appropriate authorities

Proceed with monitoring and maintenance programmes as agreed. Proceed with investigation and remedial works where necessary

Dismantle after service, or destress and abandon in the ground

Presentation Outline

- **Introduction: EC7 compliant design of embedded retaining walls**

- **Design by calculation**
  - EC7 fundamentals: limit states
  - Design Approach 1: combination 1 and 2 and associated partial factors
  - Parameter selection
  - Choice of water levels/φ'/wall friction and adhesion/unplanned excavation

- **Design by The Observational Method: outline**

- **Other significant revisions to C580 guidance**
  - Derivation of weak rock parameters
  - Update and expansion of ground movements database
  - Choice of wall : construction tolerances, use of nomograms and typical structural details
  - Guidance on the use of finite element analysis for ULS calculations
  - EC2 and EC3 compliant structural design of wall and its support system
  - Durability & plastic design of steel walls and crack width considerations & reinforcement design of reinforced concrete walls

- **Closing comments**

Closing comments

Updating C580 has been a daunting but necessary task

- EC 7 compliant design compatible with EC2 and EC3 requirements
  - by calculation
    - Interpretation of code
    - Choice of characteristic parameters
    - Choice of water levels/φ’/wall friction and adhesion
    - Unplanned excavation
  - by The Observation Method
    - New frame work
    - *Ab initio* and *Ipso Tempore*

Closing comments

- Other significant changes and additions:

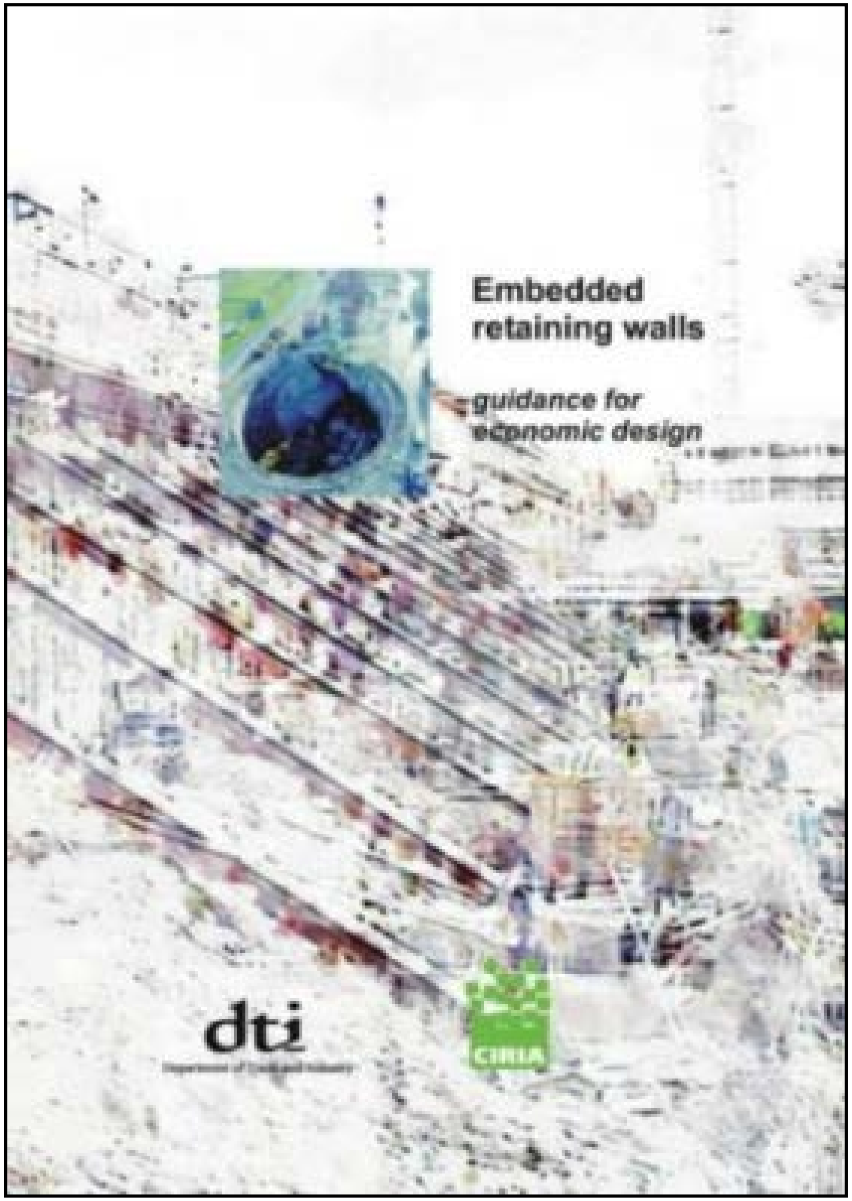
  - Derivation of parameters of weak rocks
  - Update and expansion of ground movement database
  - Choice of wall type: construction tolerances, use of nomograms and typical structural details
  - Guidance on geothermal piles and carbon calculation
  - Guidance on the use of finite element analysis for ULS calculations
  - Structural design of wall and support system to EC2 and EC3 requirements
  - Durability and plastic design of steel walls and crack width considerations and reinforcement design of reinforced concrete walls

C760

Embedded Retaining walls:

guidance for design

THE OBSERVATIONAL METHOD



Embedded retaining walls

guidance for economic design

dti

CIRIA

The Observational Method

Ab Initio = From the start

(where the OM is planned from the start of the project)

Ipso Tempore = In the moment

(where the design of the embedded wall and its support system is reassessed during construction and the OM is implemented after start of construction)

Tempura

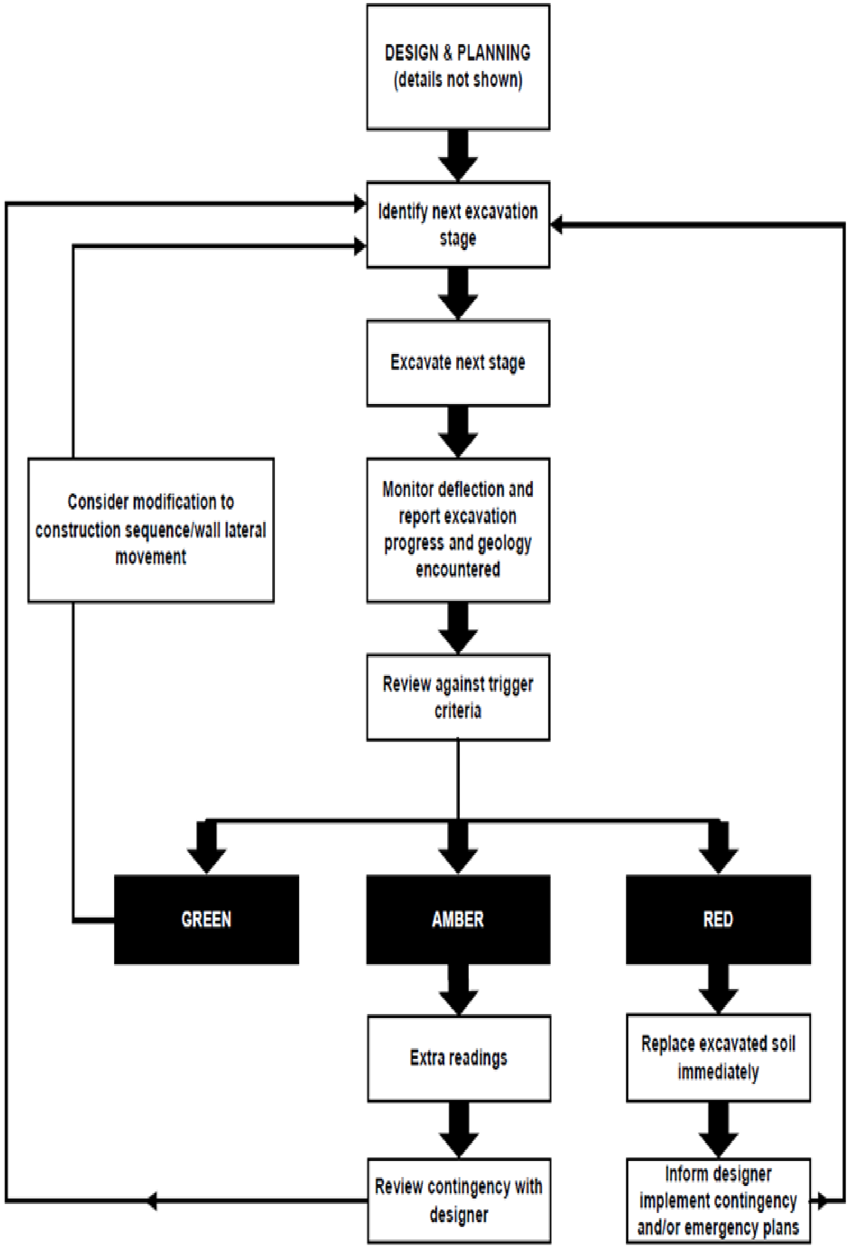
Presentation Outline

- **Background**
  - Peck (1969)
  - CIRIA C185 (Nicholson *et al.*, 1999)
  - EC7 (2004)

- **New C760 Observational Method implementation Framework**
  - ***Ab Initio***
    - **A** – optimistically pro-active
    - **B** – cautiously pro-active
  - ***Ipso tempore***
    - **C** – pro-active to make modifications
    - **D** – reactive to make corrections

- **Observational Method application**
  - Illustrative scenarios

- **Closing comments**



The Observational Method: background

Definition (C185):

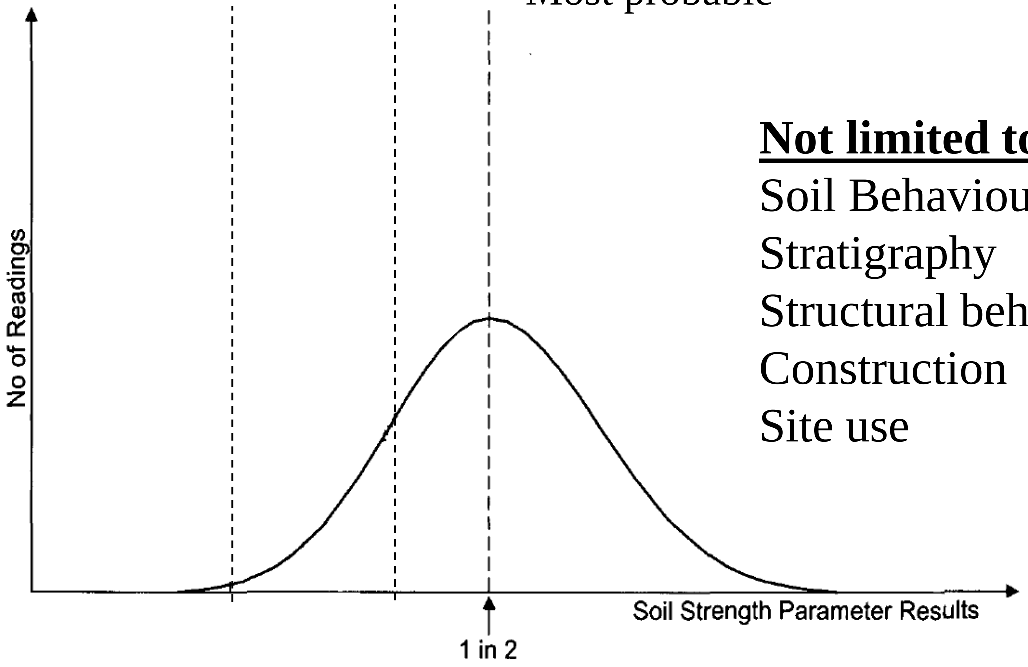
“The Observational Method in ground engineering is a continuous, managed, integrated process of design, construction control, monitoring and review that enables previously defined modifications to be incorporated during or after construction as appropriate. The objective is to achieve greater overall economy without compromising safety. The method can be adopted from the inception of a project, or later if benefits are identified”

The Observational Method: Parameter definition

Worst credible

Characteristic

Most probable



Not limited to soil parameters

Soil Behaviour

Stratigraphy

Structural behaviour

Construction

Site use

The Observational Method: background

Chapter 8 of C580 - “Areas of further work and research”:

> *“Significant cost savings can be achieved by adopting a risk based approach to design and construction through the use of the Observational Method”*

The Observational Method – Peck (1969)

Peck’s Rankine Lecture 1969

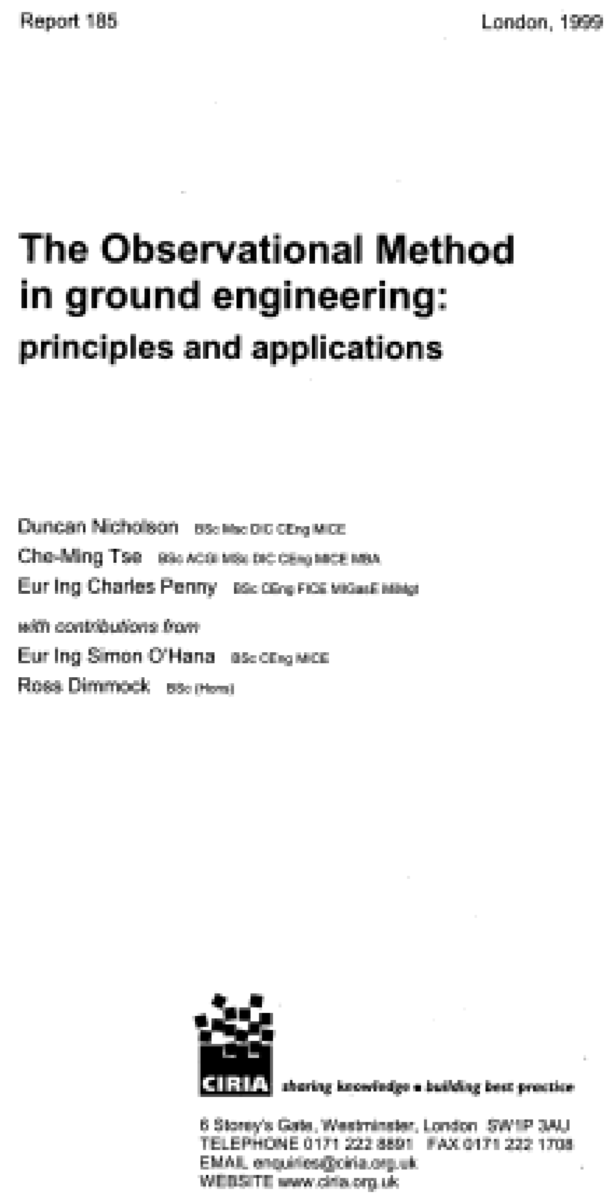
- *Ab Initio*
  - OM designed before construction starts
  - *“the intended use of the Observational Method from the inception of the construction phase”* Peck 1969
  - Maximises material savings by using “most probable” behaviour

- Best Way Out
  - OM introduced during construction
  - *“construction has already started and some unexpected development has occurred, or whenever a failure or accident threatens or has already taken place”* Peck 1969

The Observational Method
– C185 (Nicholson *et al.*, 1999)

CIRIA C185 (Nicholson *et al.*, 1999):

- *Ab Initio*
  - Starts with “characteristic”
  - Progressive modification to “most probable”
  - Limited material savings
  - No guidance on “Best way out”



Report 185 London, 1999

The Observational Method in ground engineering: principles and applications

Duncan Nicholson BSc MSc DIC CEng MICE
Che-Ming Tse BSc ACGI MSc DIC CEng MICE MBA
Eur Ing Charles Penny BSc CEng FICE MIGasE MIHigt

with contributions from
Eur Ing Simon O'Hana BSc CEng MICE
Ross Dimmock BSc (Hons)

CIRIA sharing knowledge ▪ building best practice

6 Storey's Gate, Westminster, London SW1P 3AU
TELEPHONE 0171 222 8891 FAX 0171 222 1708
EMAIL enquiries@ciria.org.uk
WEBSITE www.ciria.org.uk

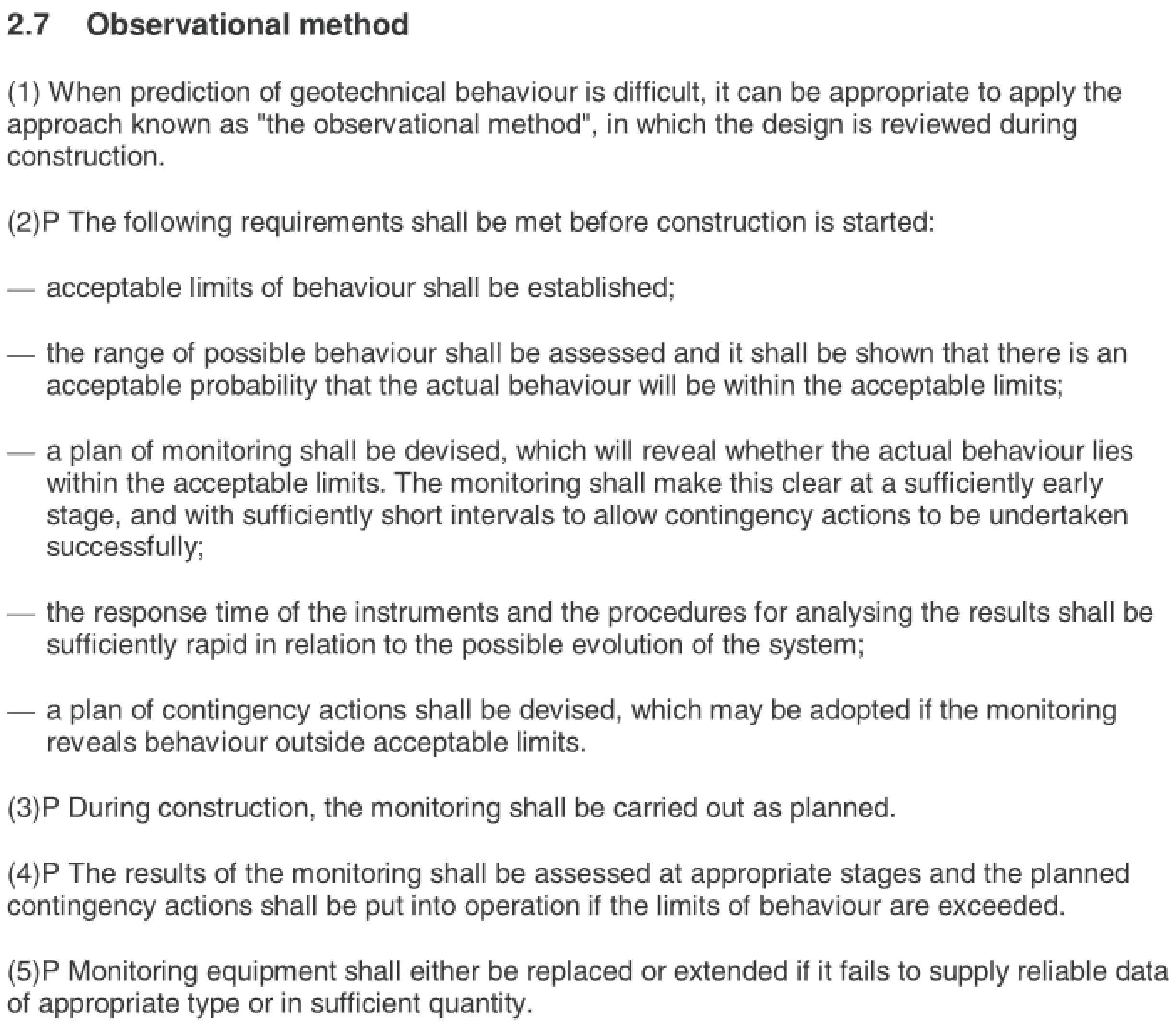
The Observational Method – EC7 (BS EN 1997-1:2004)

Eurocode 7

First code in UK to cover OM

12 pages introducing “Design by Calculation”

221 words on OM



2.7 Observational method

(1) When prediction of geotechnical behaviour is difficult, it can be appropriate to apply the approach known as "the observational method", in which the design is reviewed during construction.

(2)P The following requirements shall be met before construction is started:

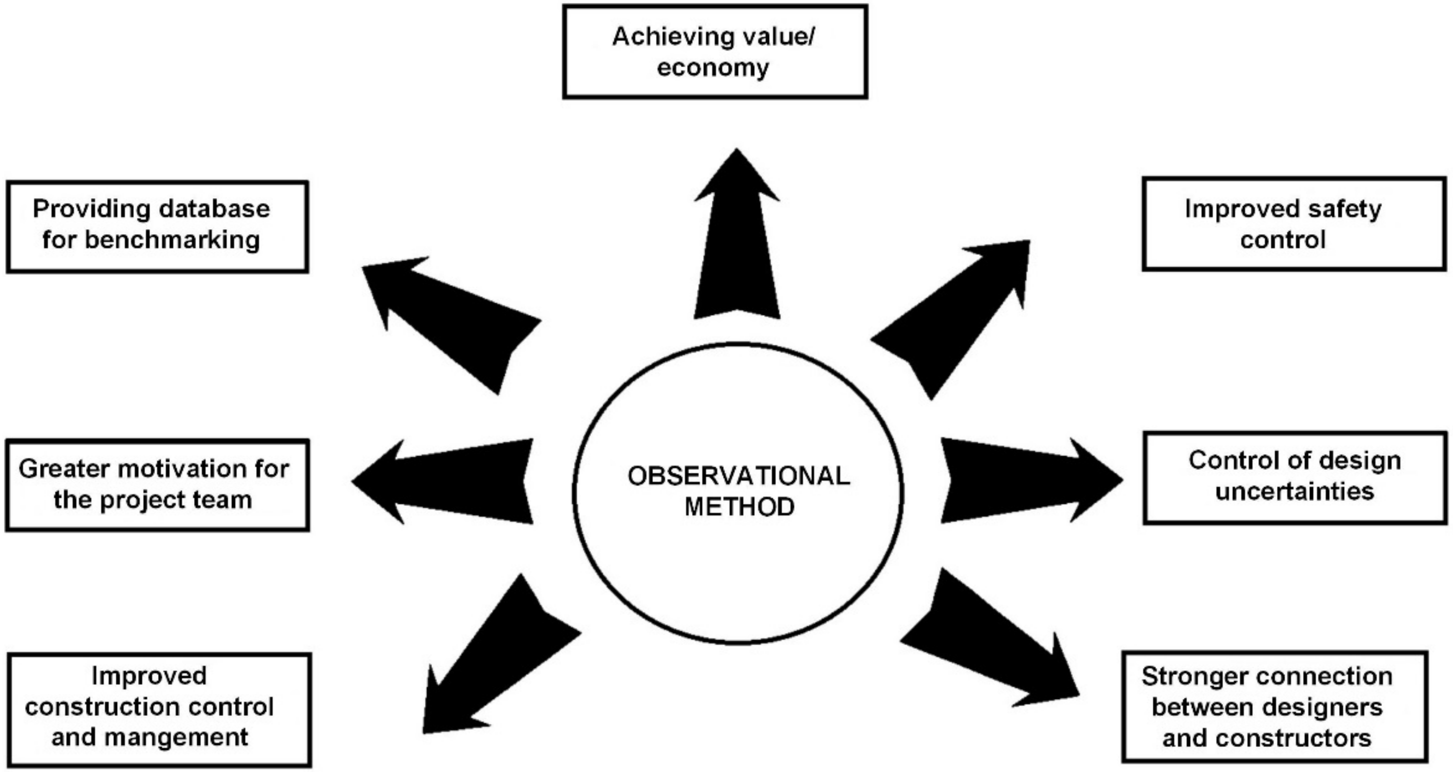
- acceptable limits of behaviour shall be established;
- the range of possible behaviour shall be assessed and it shall be shown that there is an acceptable probability that the actual behaviour will be within the acceptable limits;
- a plan of monitoring shall be devised, which will reveal whether the actual behaviour lies within the acceptable limits. The monitoring shall make this clear at a sufficiently early stage, and with sufficiently short intervals to allow contingency actions to be undertaken successfully;
- the response time of the instruments and the procedures for analysing the results shall be sufficiently rapid in relation to the possible evolution of the system;
- a plan of contingency actions shall be devised, which may be adopted if the monitoring reveals behaviour outside acceptable limits.

(3)P During construction, the monitoring shall be carried out as planned.

(4)P The results of the monitoring shall be assessed at appropriate stages and the planned contingency actions shall be put into operation if the limits of behaviour are exceeded.

(5)P Monitoring equipment shall either be replaced or extended if it fails to supply reliable data of appropriate type or in sufficient quantity.

The Observational Method



- One important consideration when applying OM to embedded retaining walls: once the wall is installed, it cannot be changed
- Therefore, changes can only be made to the construction sequence and wall support

Presentation Outline

- Background
  - Peck (1969)
  - CIRIA C185 (Nicholson *et al*., 1999)
  - EC7 (2004)

- **New C760 Observational Method implementation Framework**
  - ***Ab Initio***
    - **A** – optimistically pro-active
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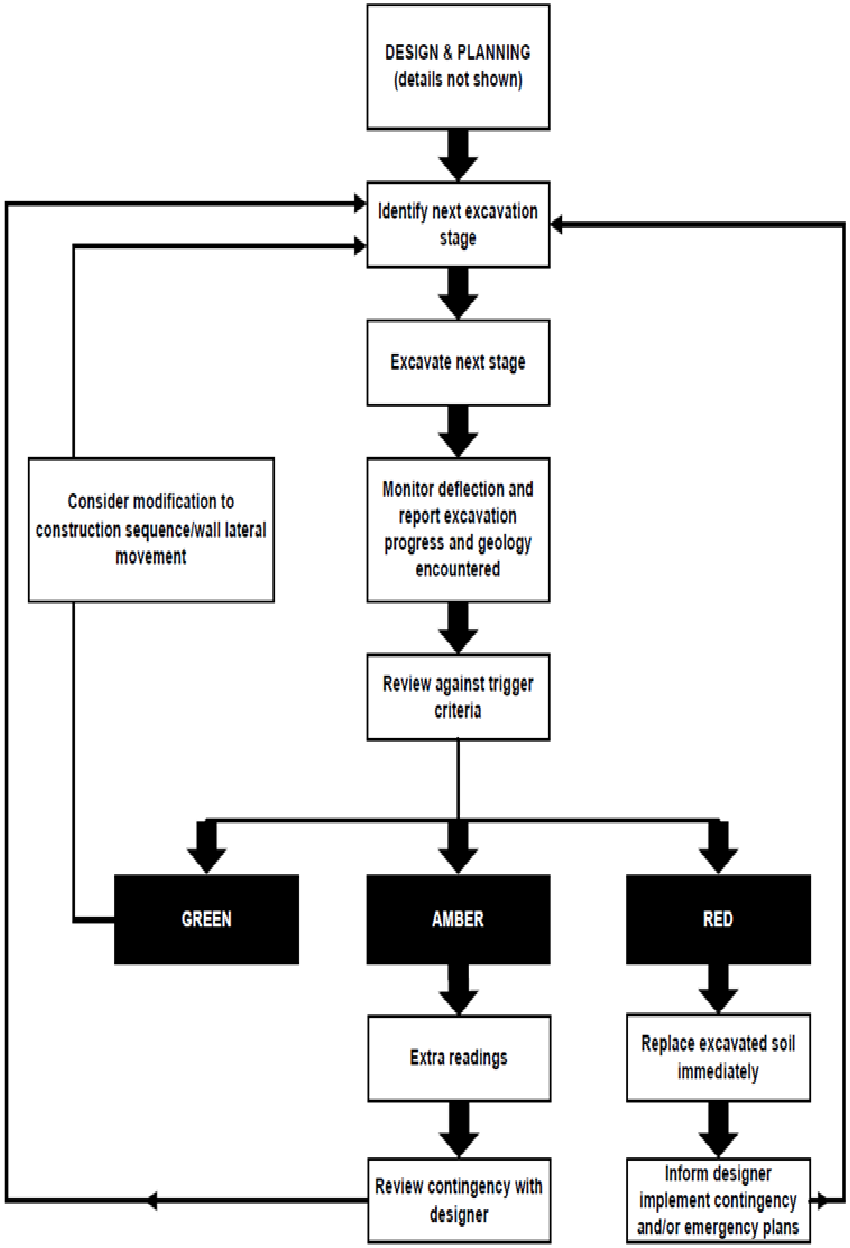
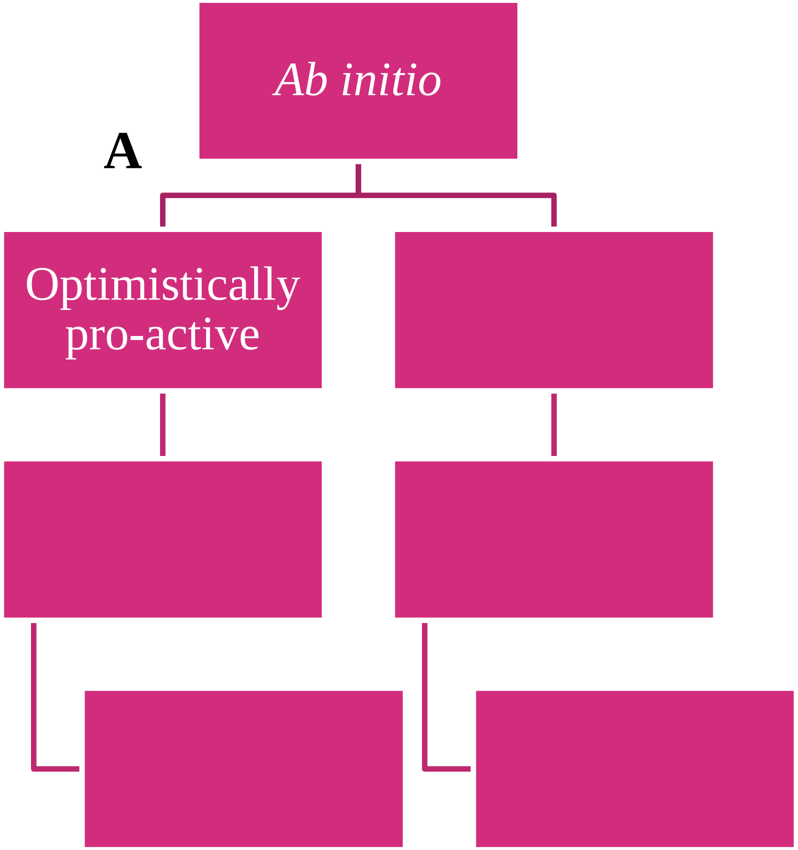
- Observational Method application
  - Illustrative scenarios

- Closing comments



The Observational Method

New proposed framework – C760



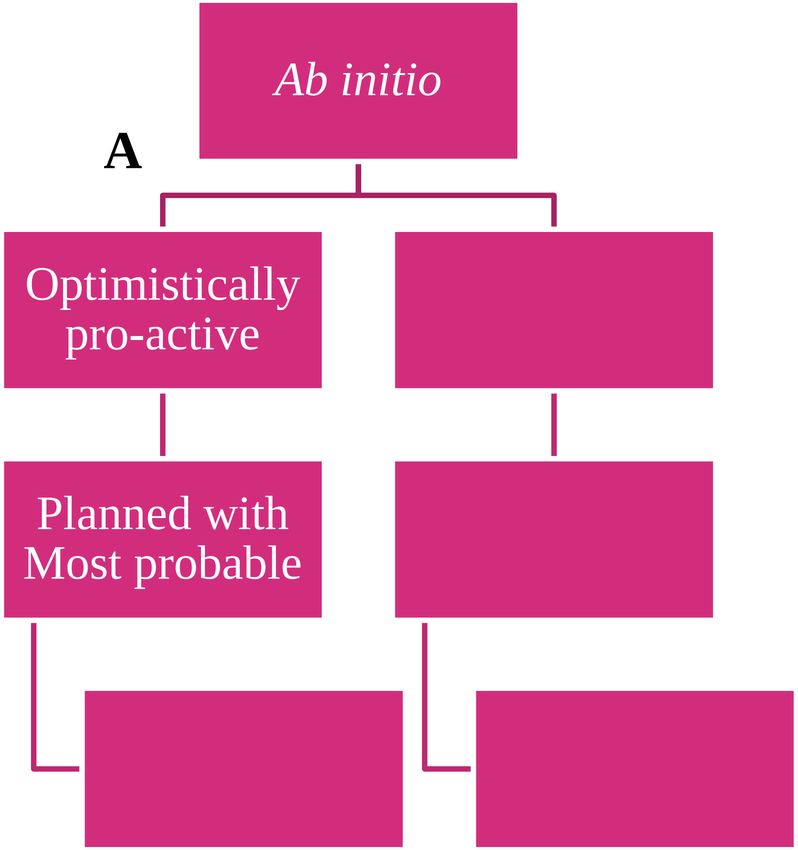
A

Ab initio

Optimistically pro-active

The Observational Method

New proposed framework – C760



Ab initio

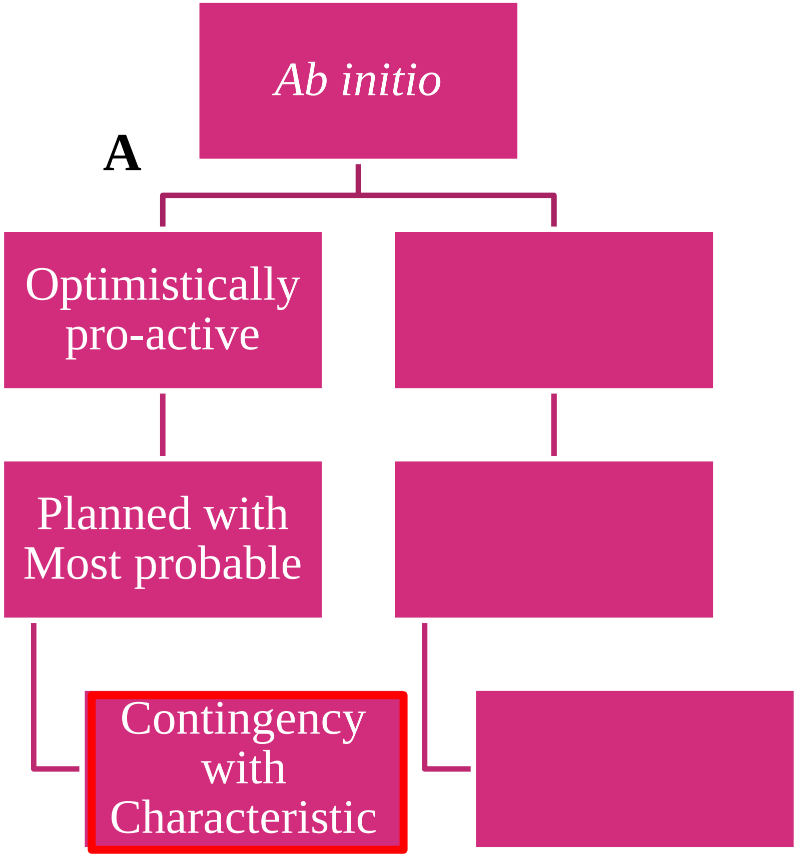
A

Optimistically pro-active

Planned with Most probable

The Observational Method

New proposed framework – C760



Ab initio

A

Optimistically pro-active

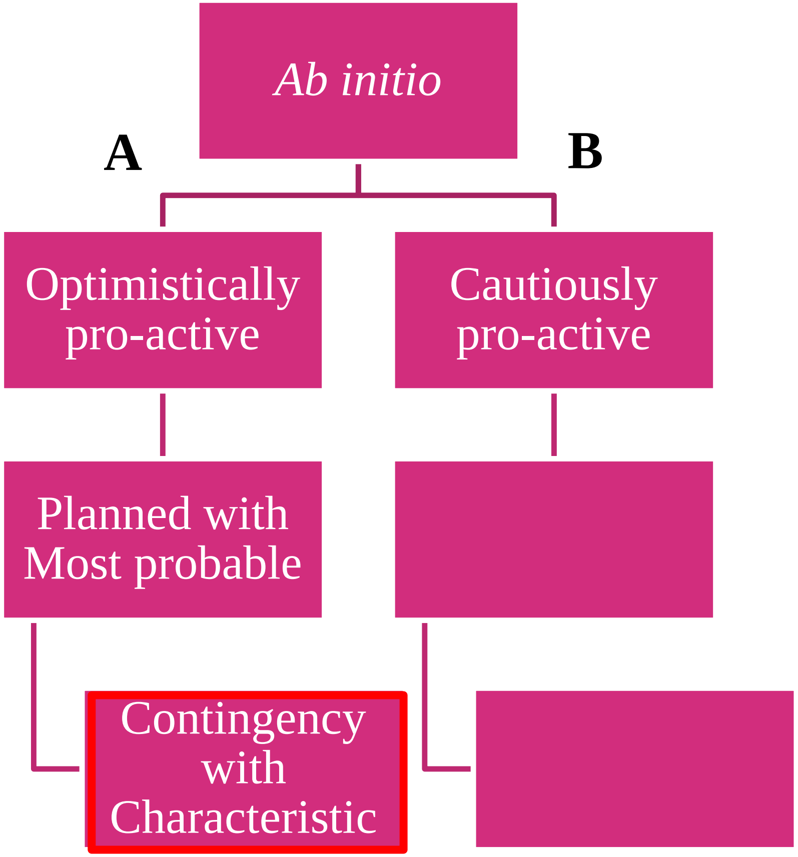
Planned with Most probable

Contingency with Characteristic

Ab initio in Peck 1969

The Observational Method

New proposed framework – C760



Ab initio

A

B

Optimistically pro-active

Cautiously pro-active

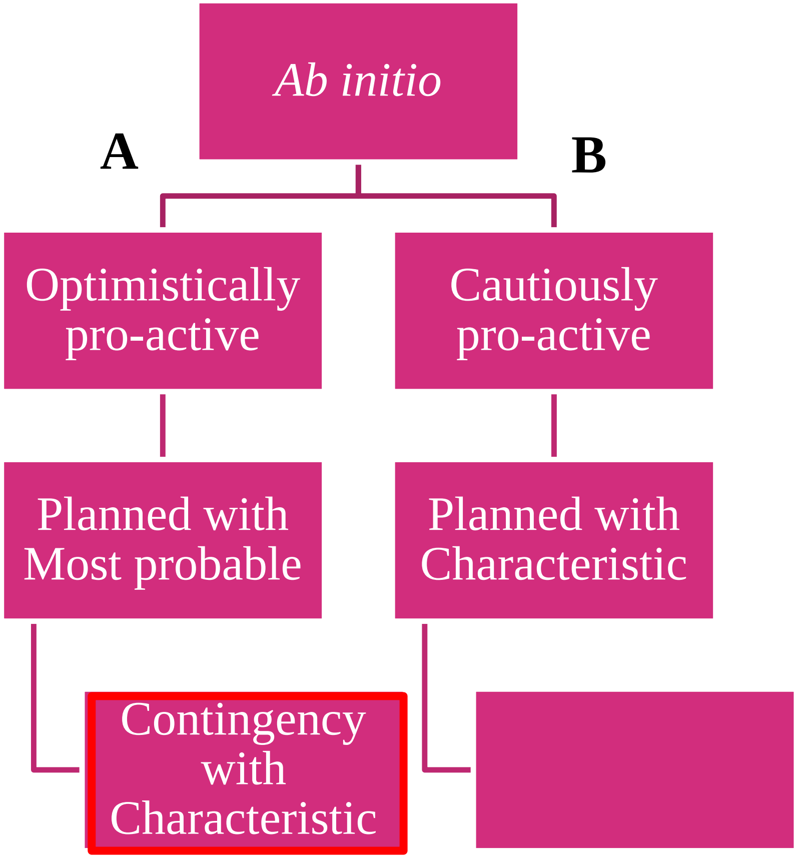
Planned with Most probable

Contingency with Characteristic

Ab initio in Peck 1969

The Observational Method

New proposed framework – C760



Ab initio

A

B

Optimistically pro-active

Cautiously pro-active

Planned with Most probable

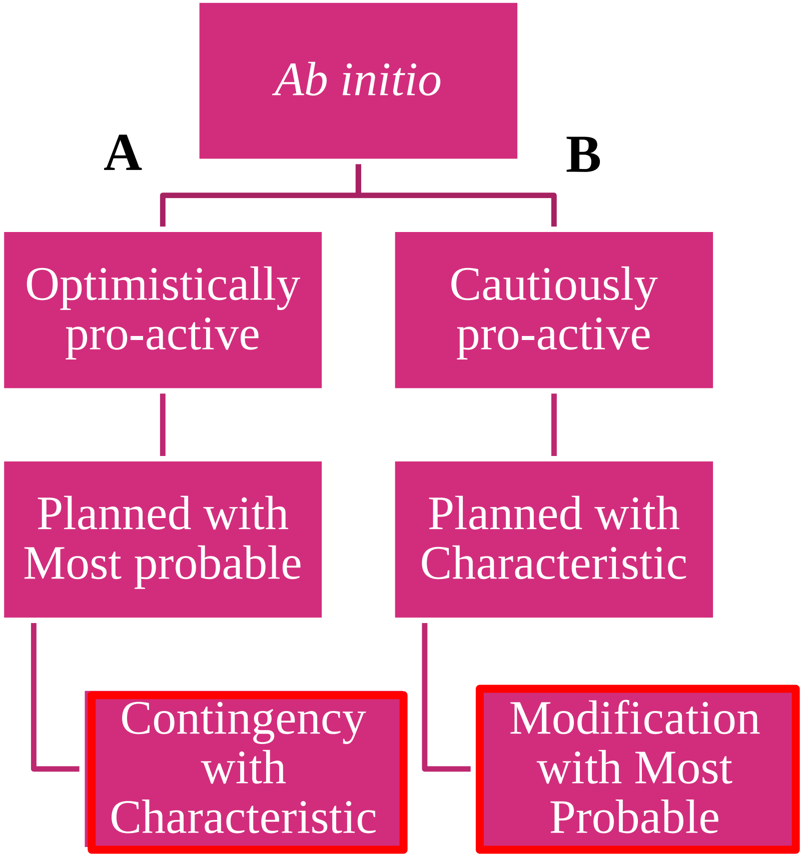
Planned with Characteristic

Contingency with Characteristic

Ab initio in Peck 1969

The Observational Method

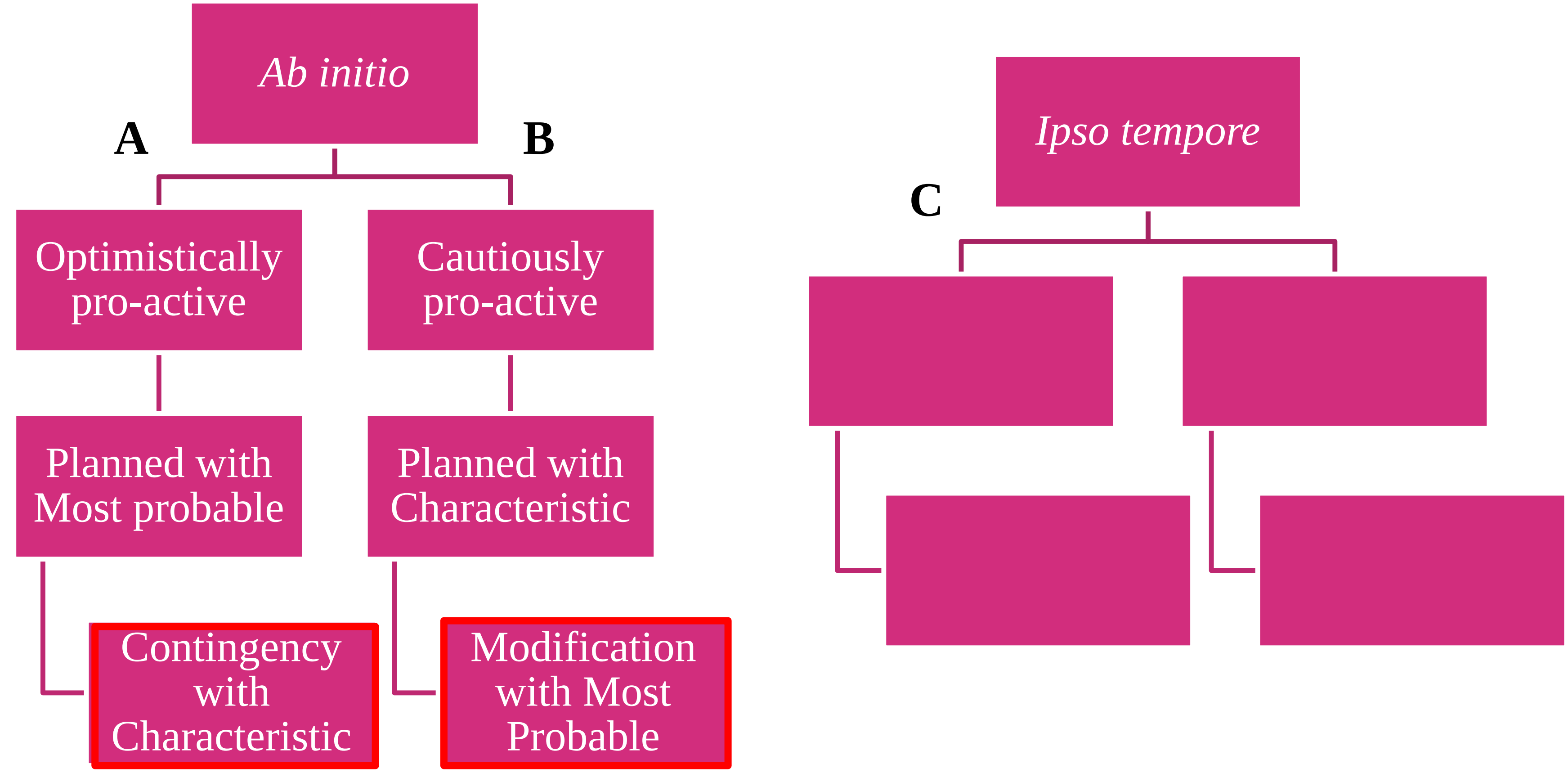
New proposed framework – C760



Ab initio in Peck 1969 *Ab initio* in C185

The Observational Method

New proposed framework – C760



Ab initio

A

B

Optimistically pro-active

Cautiously pro-active

Planned with Most probable

Planned with Characteristic

Contingency with Characteristic

Modification with Most Probable

Ipso tempore

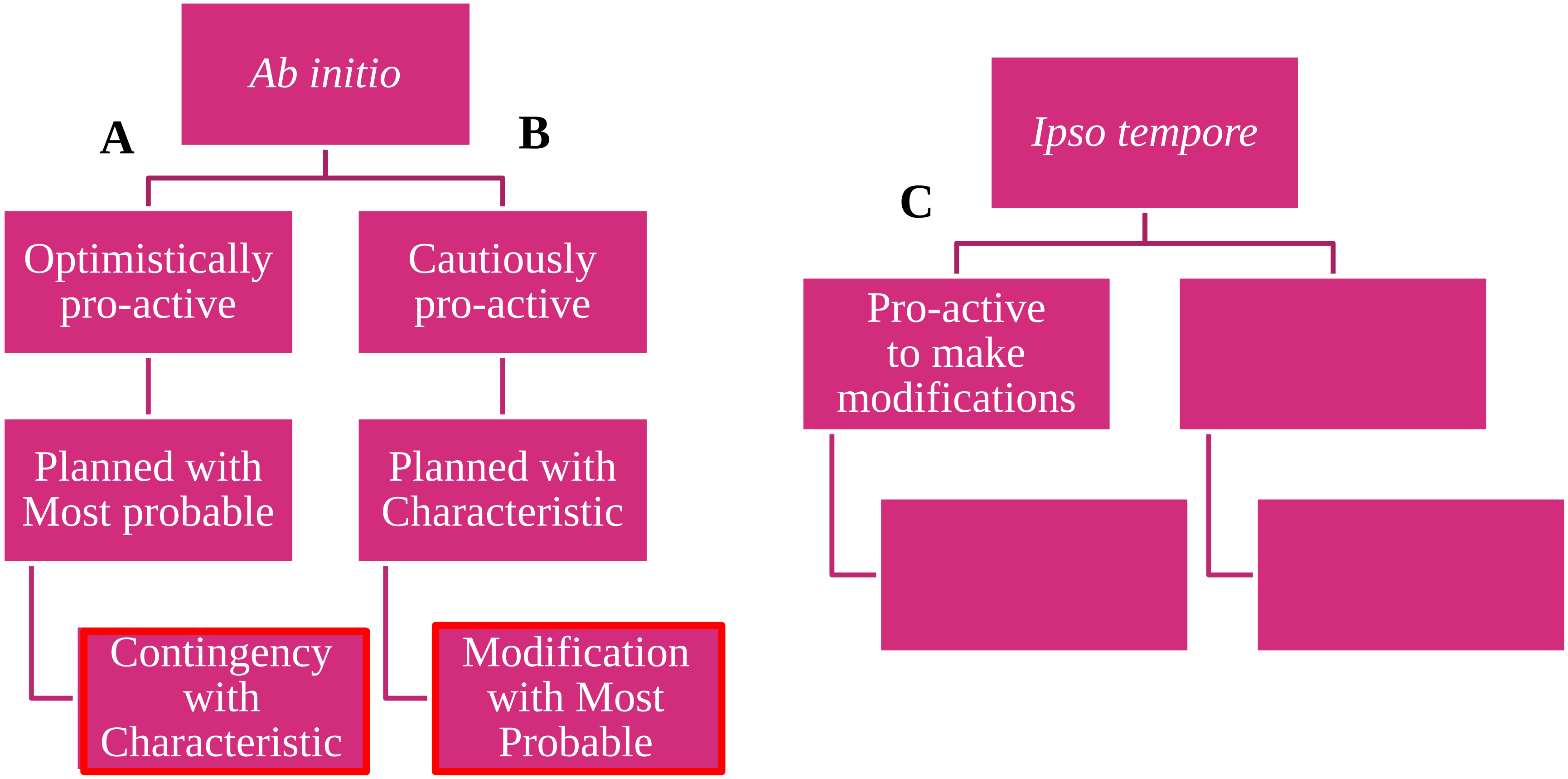
C

Ab initio in Peck 1969

Ab initio in C185

The Observational Method

New proposed framework – C760



Ab initio

A

B

Optimistically pro-active

Cautiously pro-active

Planned with Most probable

Planned with Characteristic

Contingency with Characteristic

Modification with Most Probable

Ipso tempore

C

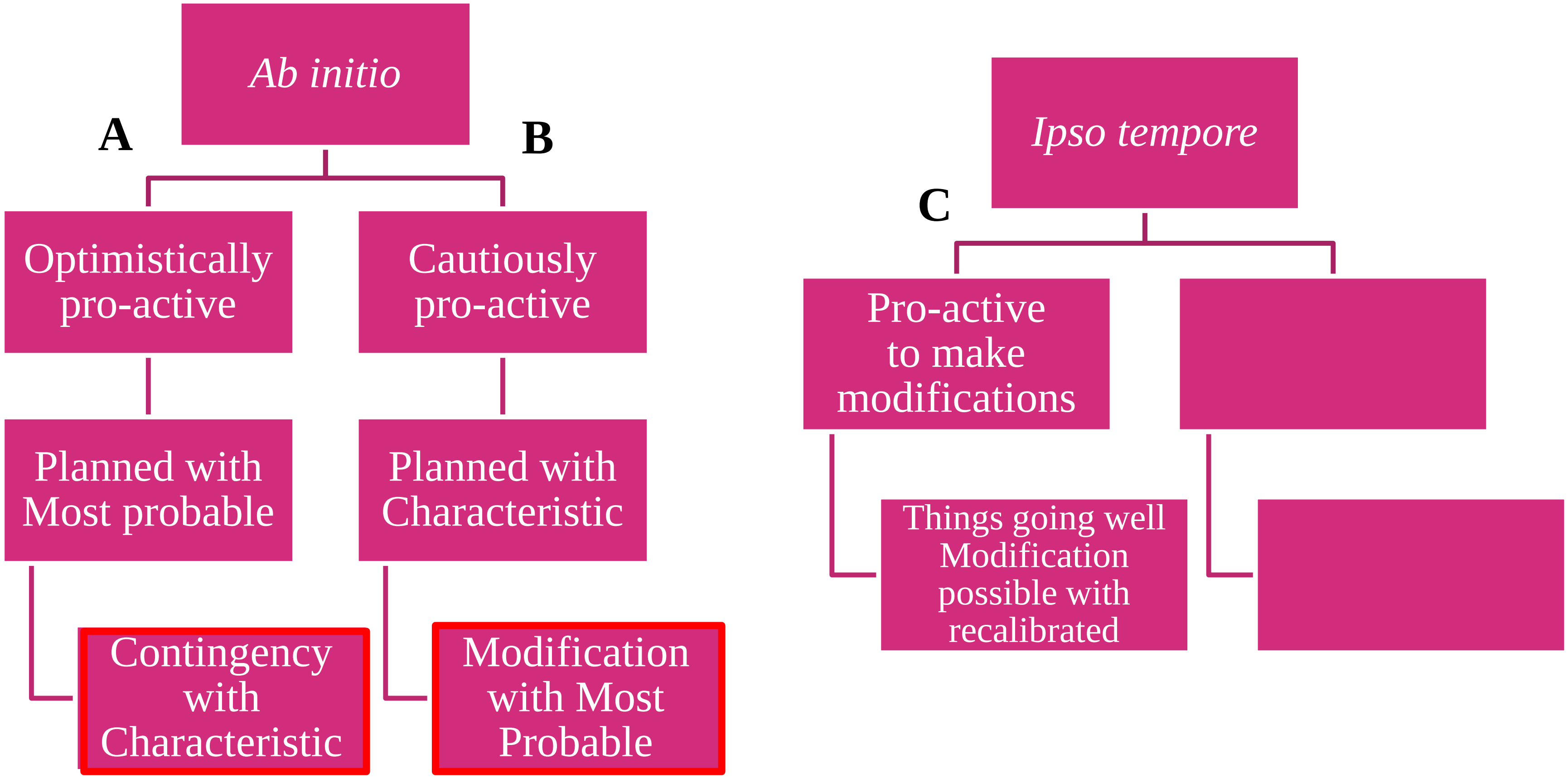
Pro-active to make modifications

Ab initio in Peck 1969

Ab initio in C185

The Observational Method

New proposed framework – C760



Ab initio

A

B

Optimistically pro-active

Cautiously pro-active

Planned with Most probable

Planned with Characteristic

Contingency with Characteristic

Modification with Most Probable

Ipso tempore

C

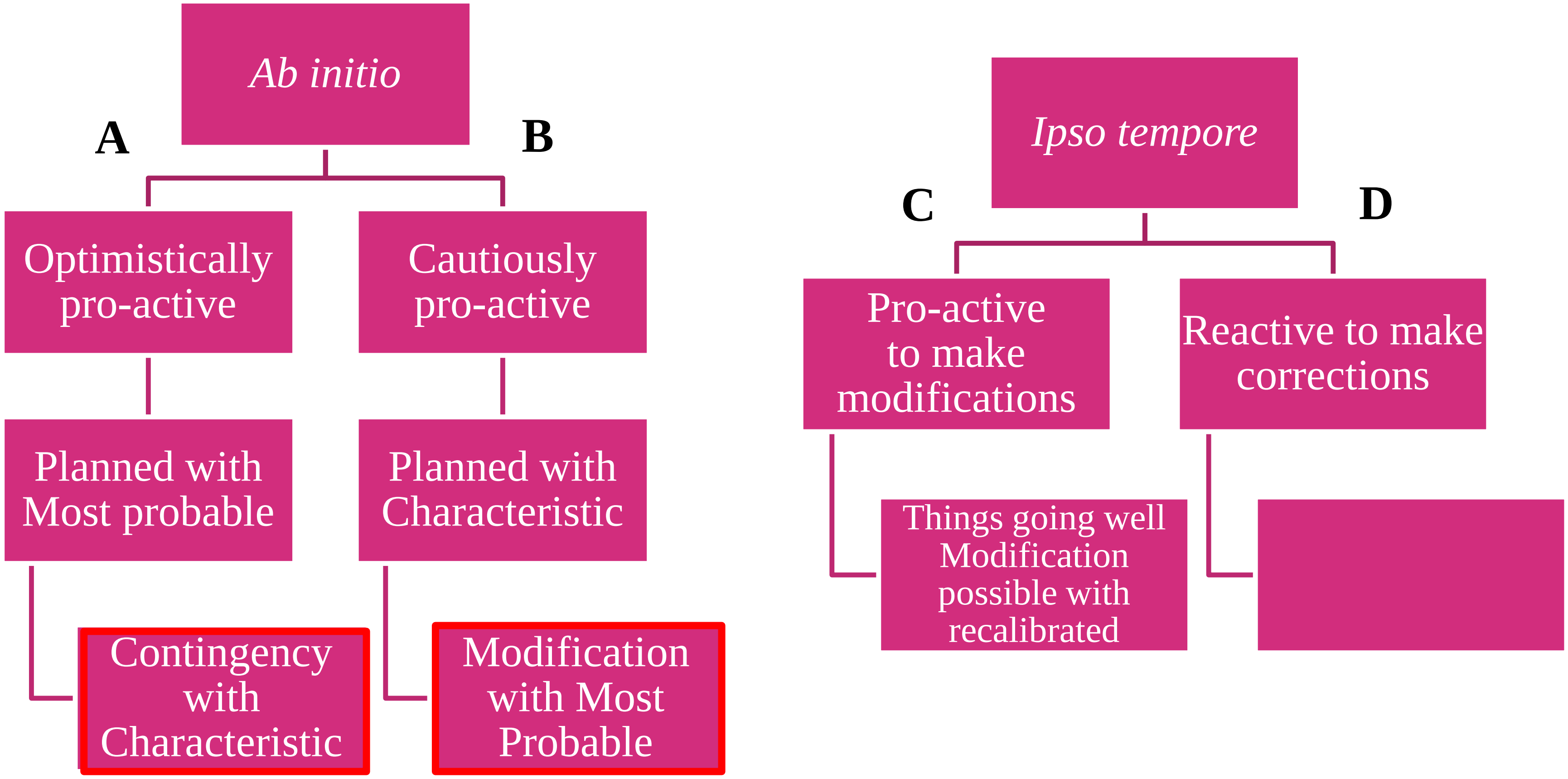
Pro-active to make modifications

Things going well Modification possible with recalibrated

Ab initio in Peck 1969 *Ab initio* in C185

The Observational Method

New proposed framework – C760



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Modification with Most Probable

Ipso tempore

C

D

Pro-active to make modifications

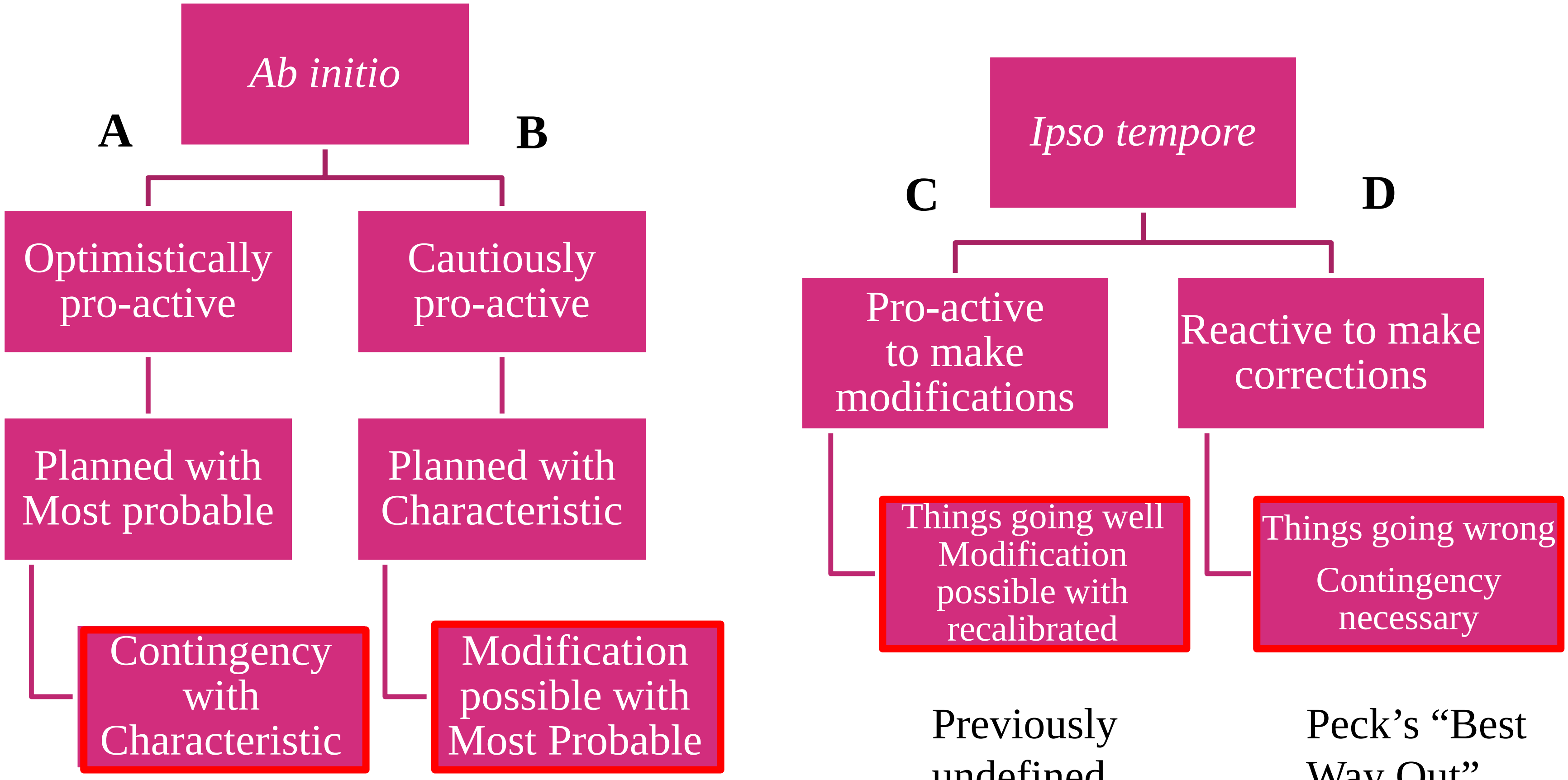
Reactive to make corrections

Things going well Modification possible with recalibrated

Ab initio in Peck 1969 *Ab initio* in C185

The Observational Method

New proposed framework – C760



Ab initio

A

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Planned with Most probable

Contingency with Characteristic

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Planned with Characteristic

Modification possible with Most Probable

Ipso tempore

C

Pro-active to make modifications

Things going well Modification possible with recalibrated

D

Reactive to make corrections

Things going wrong Contingency necessary

Ab initio in Peck 1969

Ab initio in C185

Previously undefined

Peck's "Best Way Out"

New C760 Observational Method Framework

- ***Ab Initio*** – OM design before work starts.

  A) – starts with "most probable" design (as Peck, 1969).
  (maximises material and programme savings)

  B) – starts with "characteristic" design (as C185 – Nicholson *et al.*, 1999)
  (limited material savings; potential programme savings)

- ***Ipso Tempore*** – OM starts after wall construction.

  C) – Modification plan – if design overestimated movements (NEW)
  (limited material savings; potential programme savings)

  D) – Contingency plan – if design underestimated movements. (as Peck's "best way out" - to avert breach of a limit state)

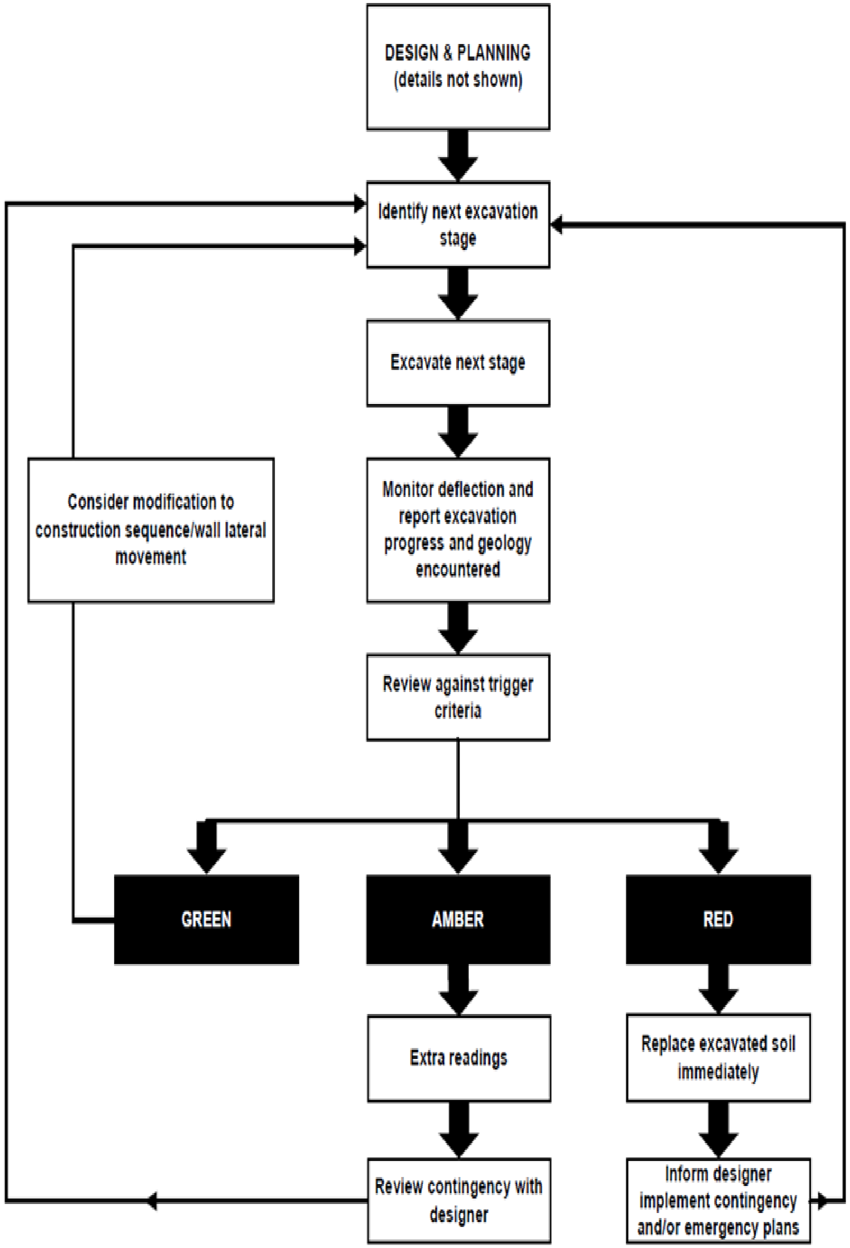
New C760 Observational Method Framework - design

Approach of OM		*Ab initio* (from the start)		*Ipso tempore* (in the moment)	
		A **Optimistically pro-active (Peck 1969)**	**B** **Cautiously pro-active (CIRIA C185)**	**C** **Pro-active modifications (Not previously defined)**	**D** **Reactive corrections (Peck 1969)**
OM design work starts		Before construction		After construction.	
Design	Existing back analysis	Necessary	Preferable, not essential	Necessary - initial construction stages	
	Start OM design using	Most Probable parameters	Characteristic parameters	Characteristic parameters	
	Design objectives	Optimise wall strength & depth & wall support	Reduce wall support	Reduce wall support	Increase wall support
	Alternative designs	Use as Contingency plan	Use as Modification plan	Develop and use as Modification plan	Develop and use as Contingency plan

Approach	Ab initio (from the start)		Ipso tempore (in the moment)	
	A - optimistically pro-active	**B - cautiously pro-active**	**C - pro-active to make modifications**	**D - reactive to make corrections**
When implemented	Observational Method is planned from project inception		Starts with conventional design with no explicit intention of applying the Observational Method	
Back analysis requirements	Necessary before construction starts from available reliable and relevant case history data	Preferable, but not essential	Necessary – from assessment of initial construction stages	
Analysis assumptions for design of the wall and its support system	Wall embedment, design and construction sequence in accordance with "design by calculation" method (see section 7.3) adopting "most probable" parameters	Wall embedment depth, design and construction sequence in accordance with "design by calculation" method (see section 7.3) adopting characteristic parameters	Wall embedment, design and construction sequence in accordance with "design by calculation" method (see section 7.3) adopting characteristic parameters	
Implementation	Most probable wall design and associated construction sequence implemented on site Alternative construction sequence fully developed in accordance with "design by calculation" (section 7.3) adopting characteristic parameters for use as contingency, depending upon the actual performance of the wall and its support system	Characteristic wall design and associated construction sequence implemented on site Alternative construction sequence fully developed in accordance with "design by calculation" method (section 7.3) adopting "most probable" parameters for use on site, depending on actual performance of the wall and its support system	Monitoring, observations and back analysis during construction show wall performing better than anticipated. Ground, material and structural parameters and ground and analytical models recalibrated on this basis. Construction sequence modified and fully developed in accordance with "design by calculation" method (section 7.3) adopting "recalibrated" parameters	Monitoring and observations during construction show wall not performing in accordance with design predictions. Additional measures put in place to prevent breach of a limit state e.g. damage to nearby structures or to prevent catastrophic collapse
Advantages and possible savings	Maximum potential for savings in materials and construction programme duration	Savings in construction programme duration but no wall material savings, although some savings in materials may be possible due to reduced wall support requirements	Possible savings in wall support system during excavation in front of the wall by modifying construction sequence and support system requirements. Only likely to be feasible on large projects with long construction duration	

Presentation Outline

- Background
  - Peck (1969)
  - CIRIA C185 (Nicholson *et al.*, 1999)
  - EC7 (2004)
- New C760 Observational Method implementation Framework
  - *Ab Initio*
    - A – optimistically pro-active
    - B – cautiously pro-active
  - *Ipso tempore*
    - C – pro-active to make modifications
    - D – reactive to make corrections
- **Observational Method application**
  - Illustrative scenarios
- Closing comments


DESIGN & PLANNING (details not shown)

Identify next excavation stage

Excavate next stage

Monitor deflection and report excavation progress and geology encountered

Review against trigger criteria

Consider modification to construction sequence/wall lateral movement

GREEN

AMBER

RED

Extra readings

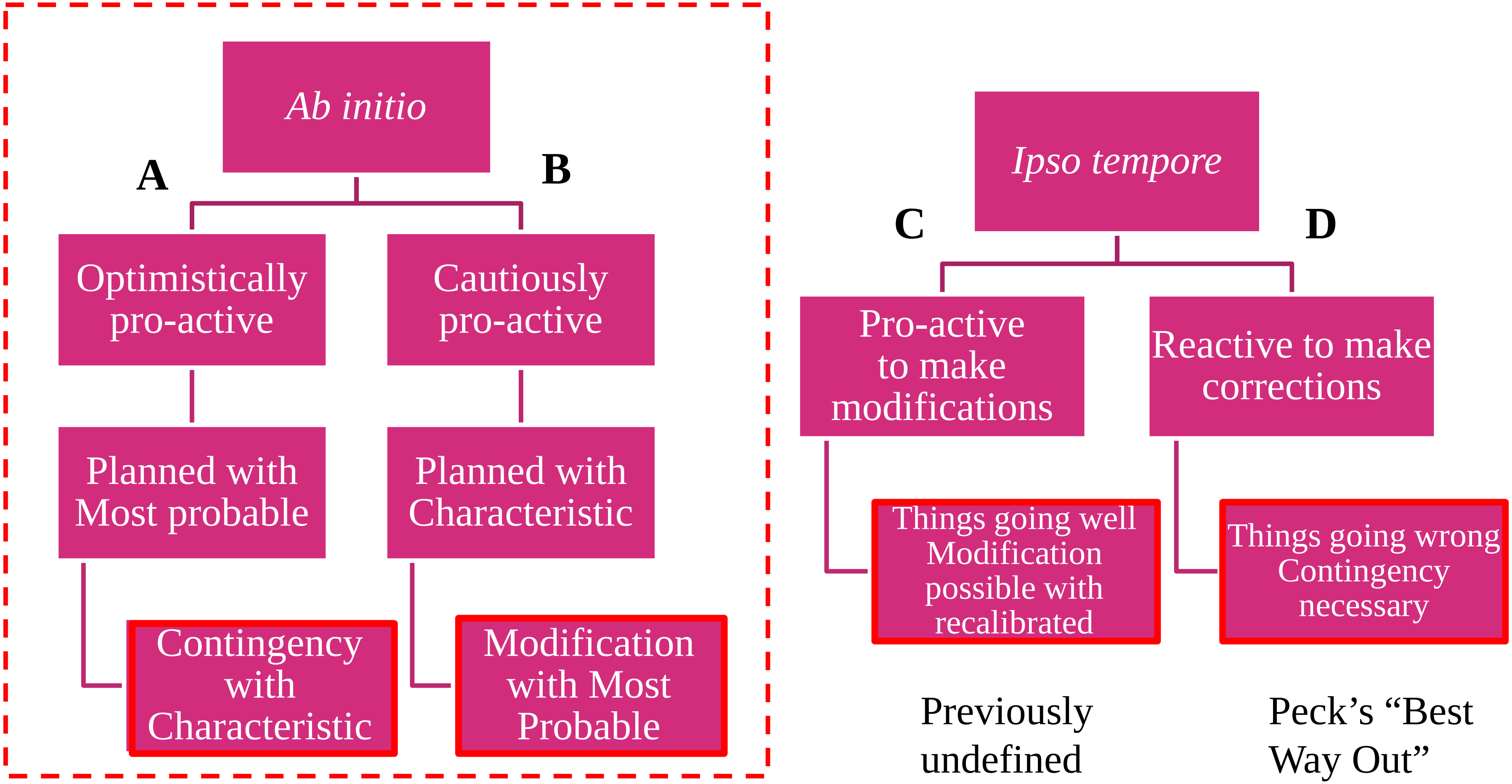
Review contingency with designer

Replace excavated soil immediately

Inform designer implement contingency and/or emergency plans

The Observational Method

New proposed framework – C760



Ab initio

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Optimistically pro-active

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Things going well Modification possible with recalibrated

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Reactive to make corrections

Things going wrong Contingency necessary

Previously undefined

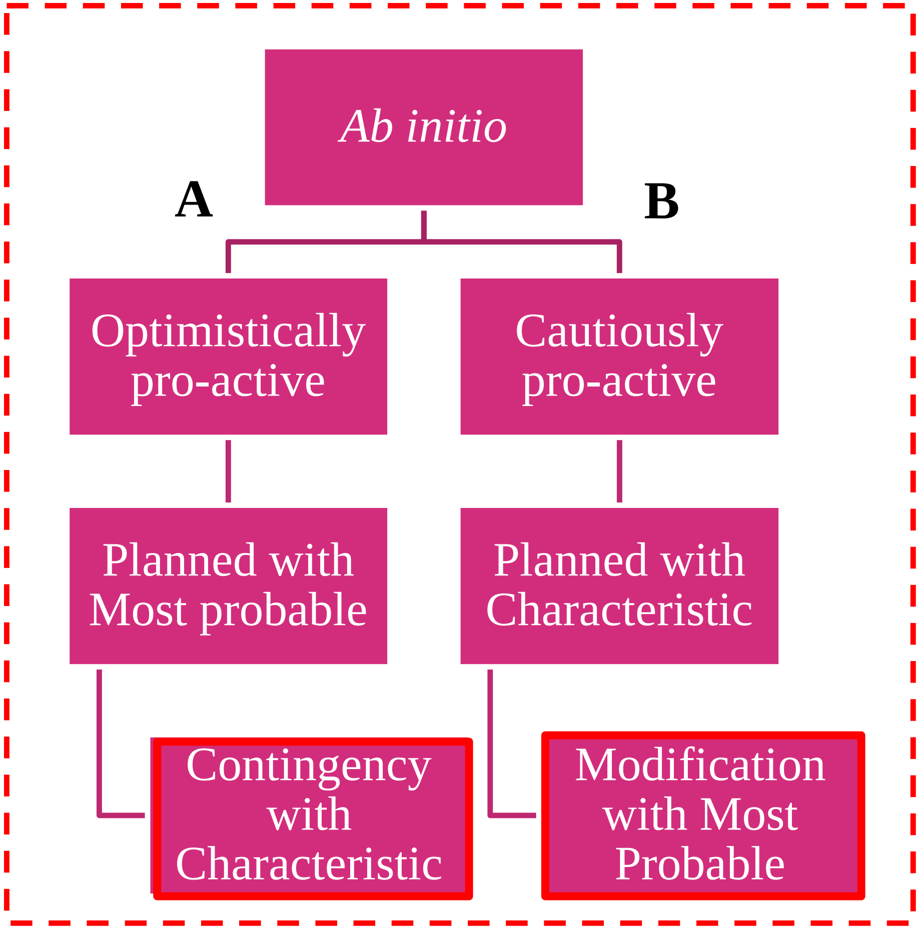
Peck’s “Best Way Out”

Ab initio in Peck 1969

Ab initio in C185

The Observational Method

New proposed framework – C760

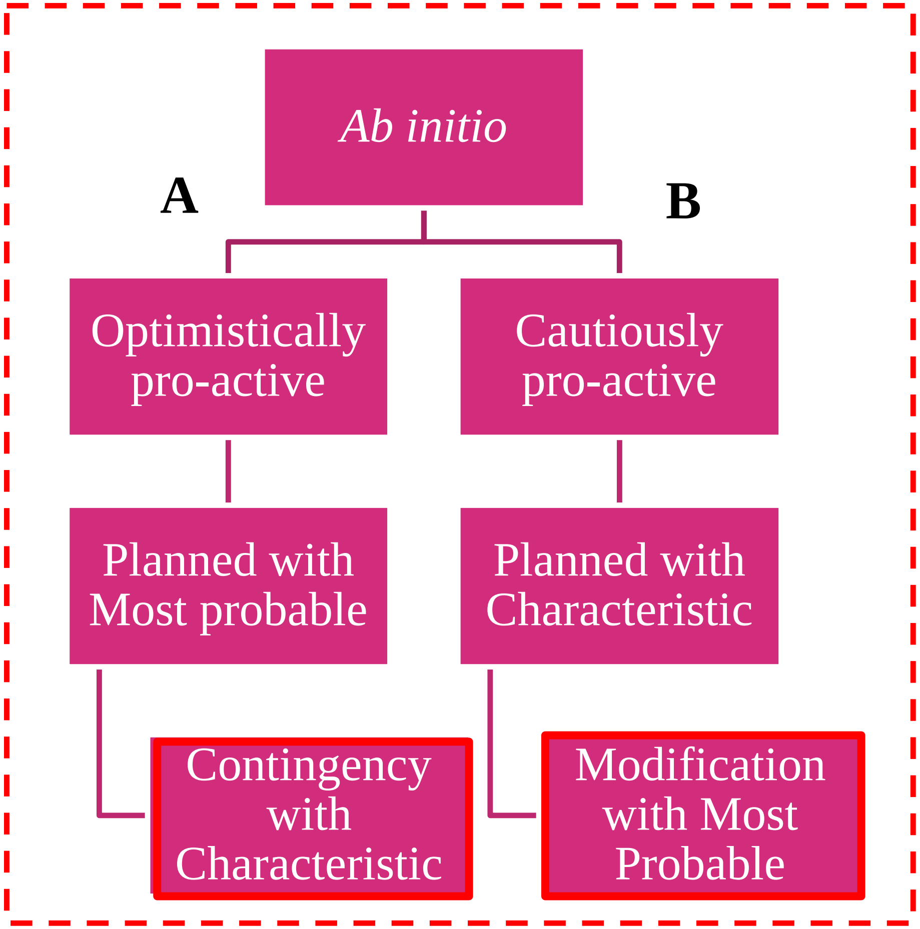


Ab initio in Peck 1969 *Ab initio* in C185

***Ab Initio* Approach**	**Scenario 1**	**Scenario 2**
A Optimistically pro-active		
B Cautiously pro-active		

The Observational Method

New proposed framework – C760



Ab initio

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Modification with Most Probable

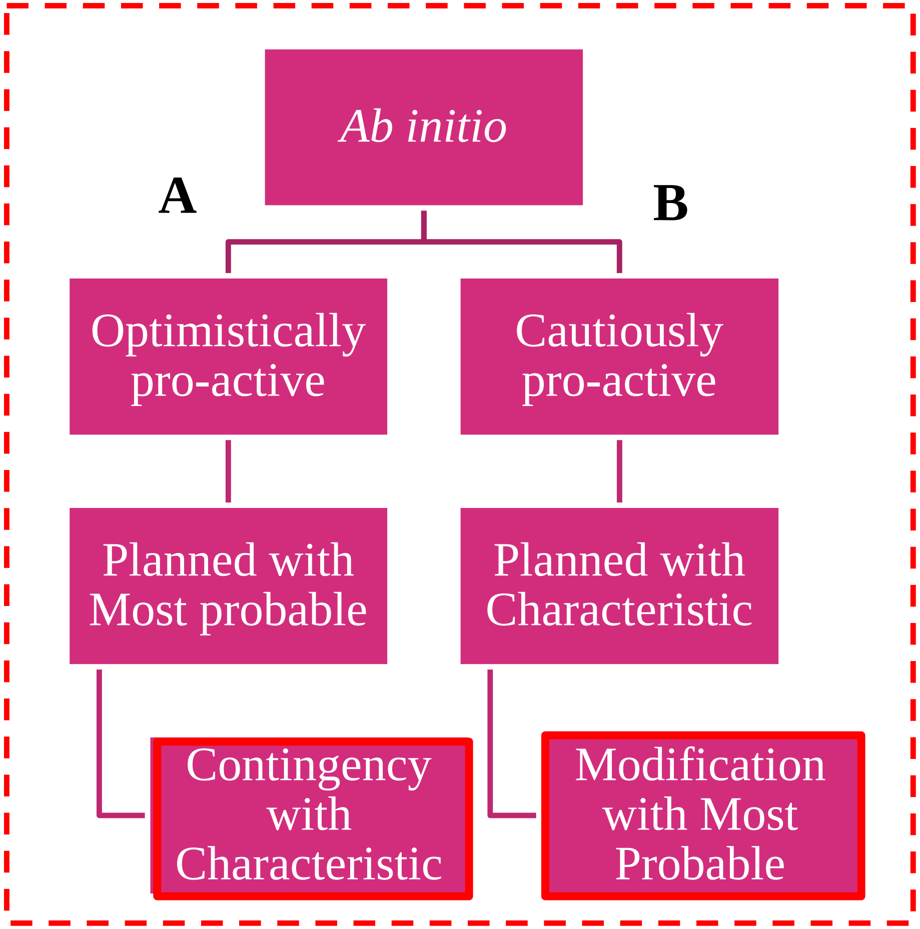
Ab initio in Peck 1969

Ab initio in C185

Ab Initio **Approach**	**Scenario 1**	**Scenario 2**
A Optimistically pro-active	✓ **No Contingency**	✘ **Contingency needed**
B Cautiously pro-active		

The Observational Method

New proposed framework – C760



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Planned with Characteristic

Contingency with Characteristic

Modification with Most Probable

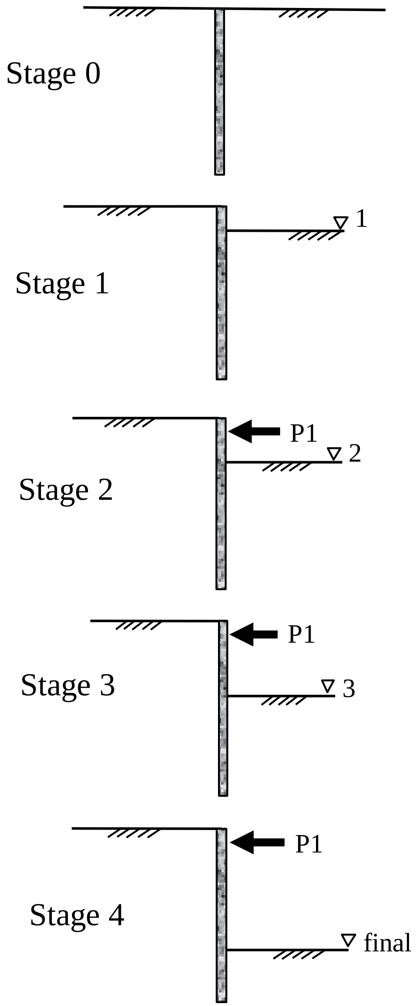
Ab initio in Peck 1969

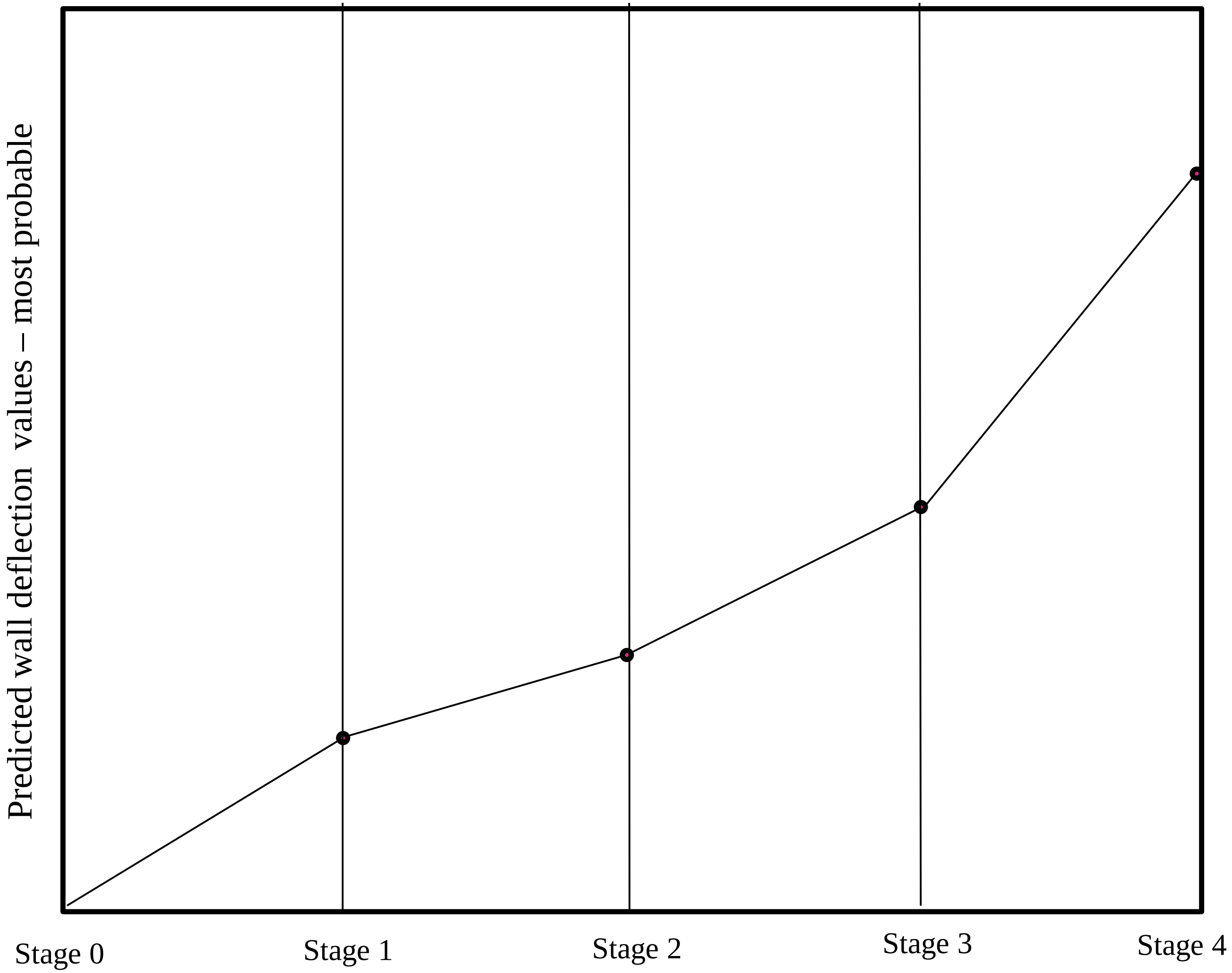
Ab initio in C185

***Ab Initio* Approach**	**Scenario 1**	**Scenario 2**
A Optimistically pro-active	✓ **No Contingency**	✘ **Contingency needed**
B Cautiously pro-active	✓ **Modification Possible**	✘ **Modification not possible**

Ab initio Approach A – design construction sequences

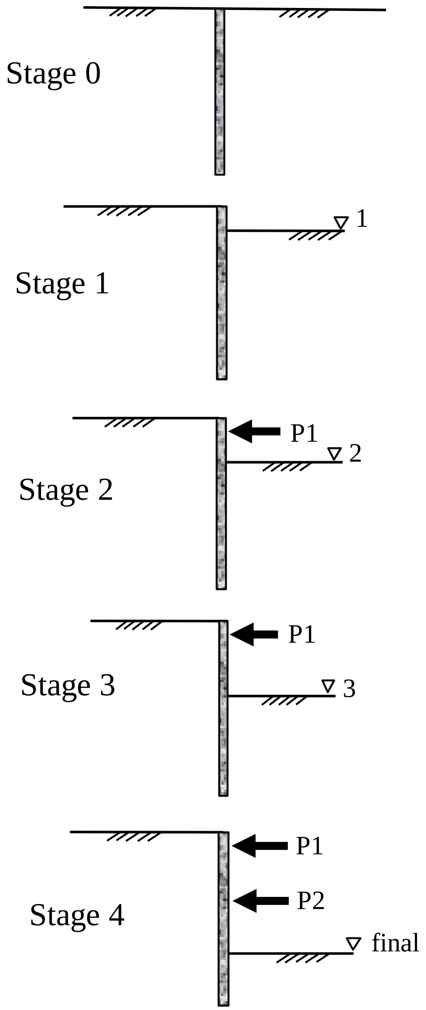
Most probable design construction sequence

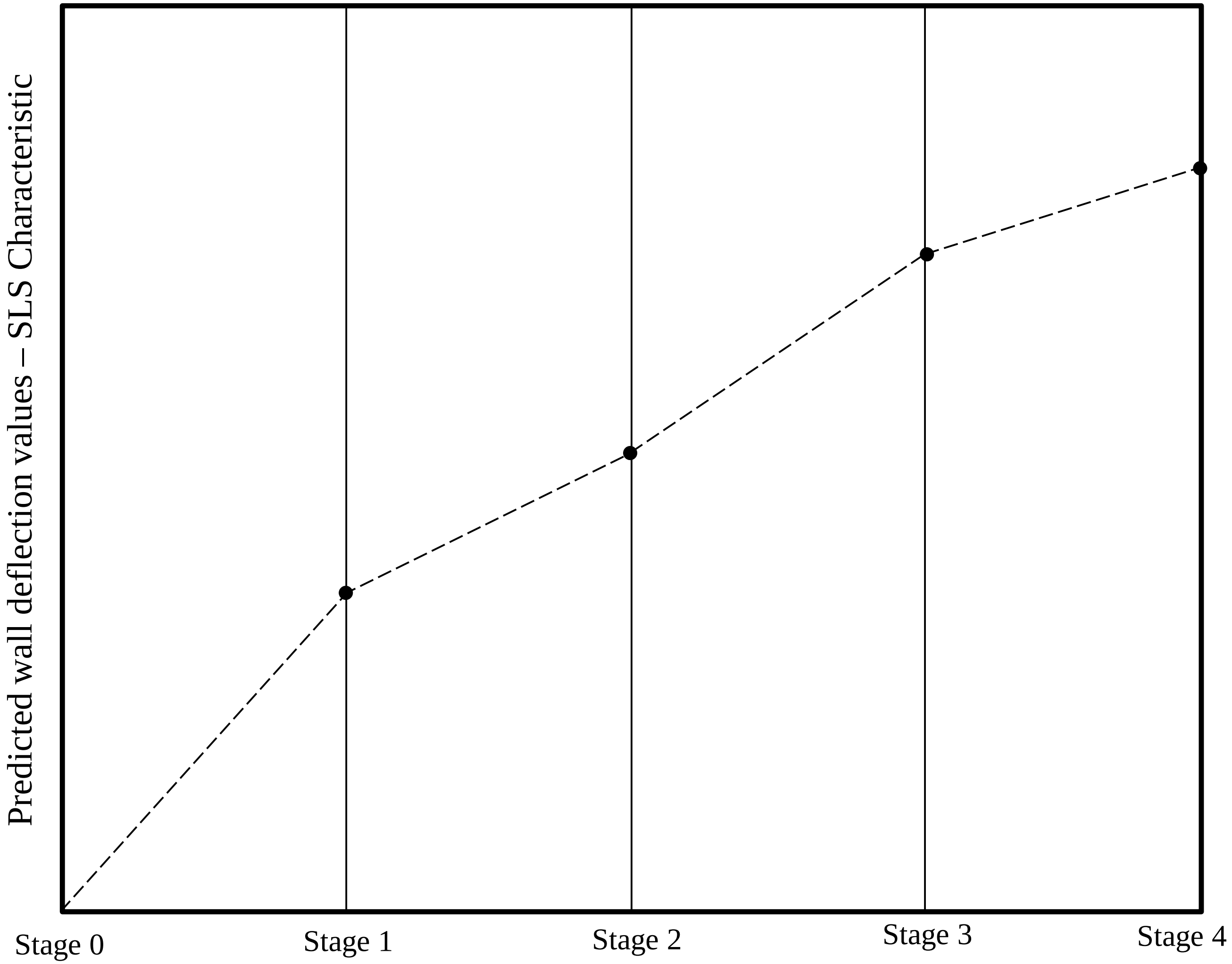




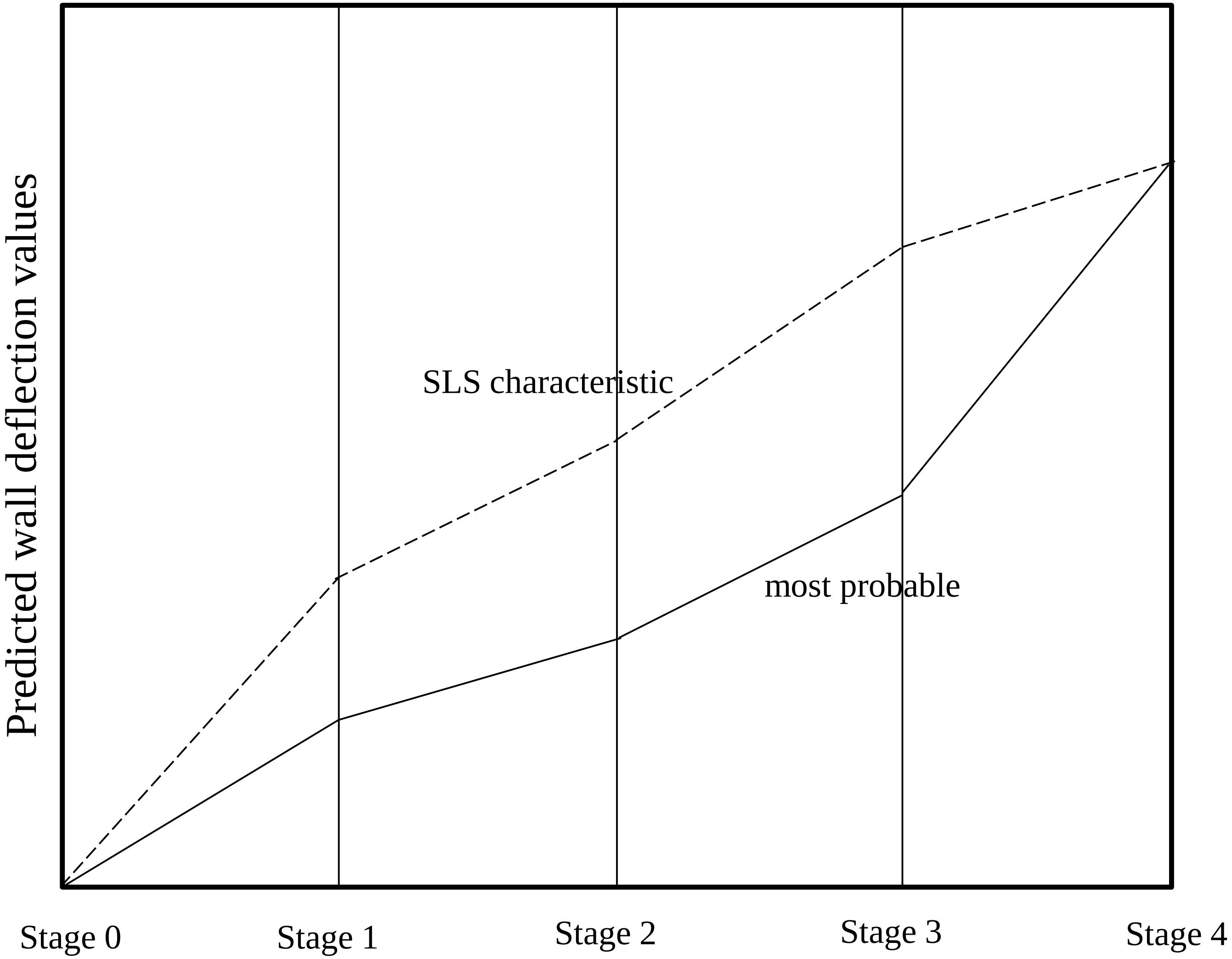
Ab initio Approach A – design construction sequences

Characteristic design construction sequence

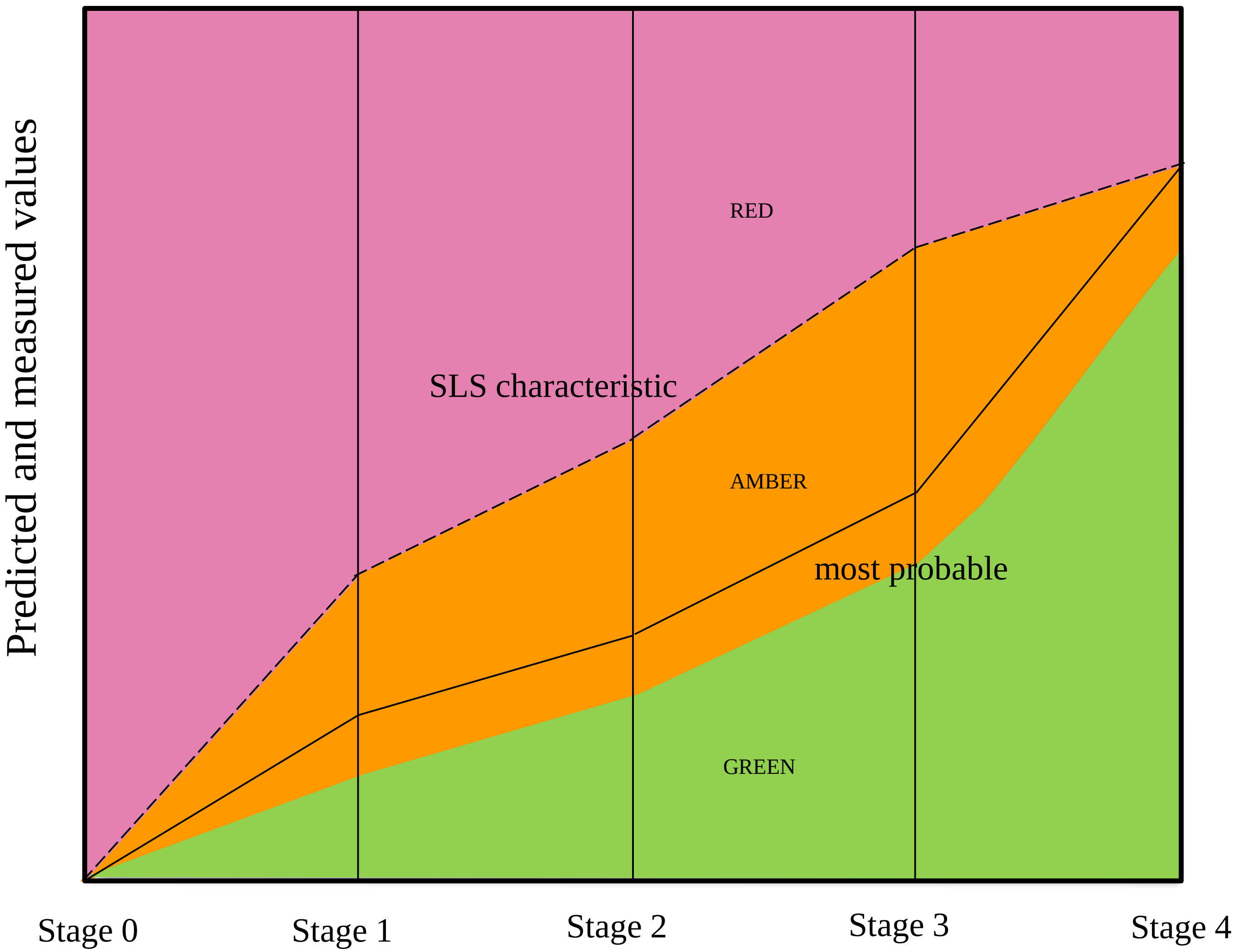




Ab initio Approach A – design construction sequences



Ab initio Observational Method – trigger levels

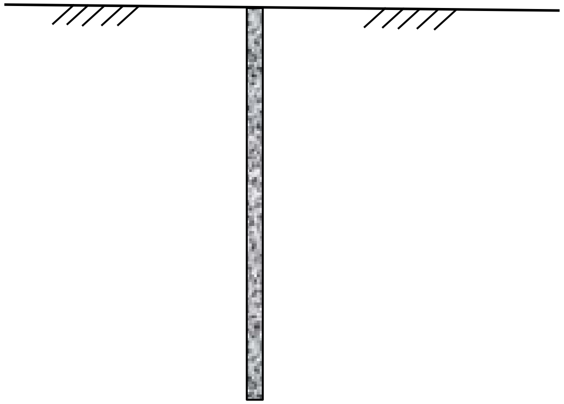


Ab initio – Approach A

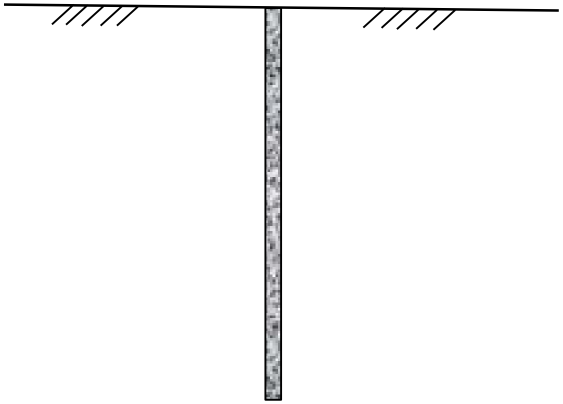
Most probable construction sequence

Characteristic construction sequence

Stage 0: Install wall



Stage 0: Install wall

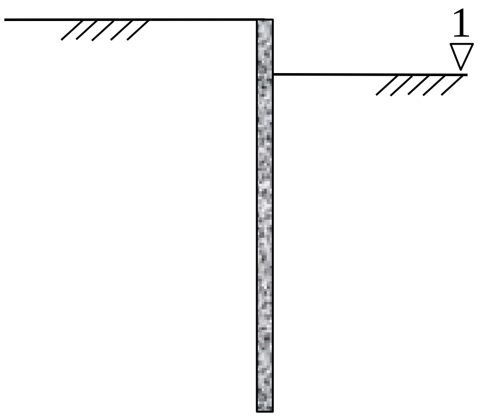


Ab initio – Approach A

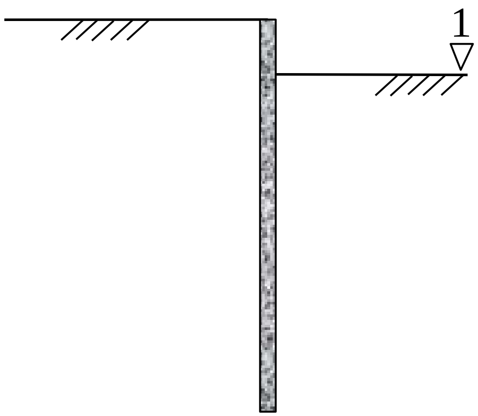
Most probable construction sequence

Characteristic construction sequence

Stage 1: Dig to excavation level 1
(just below 1st prop level)



Stage 1: Dig to excavation level 1
(just below 1st prop level)



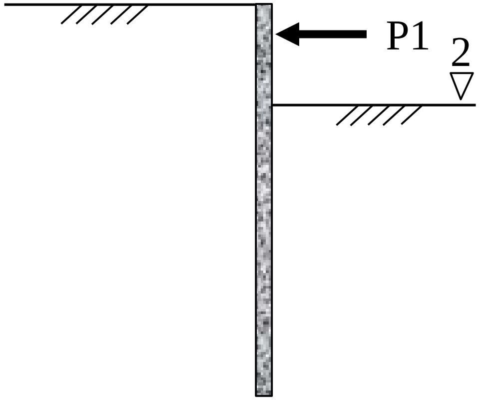
Ab initio – Approach A

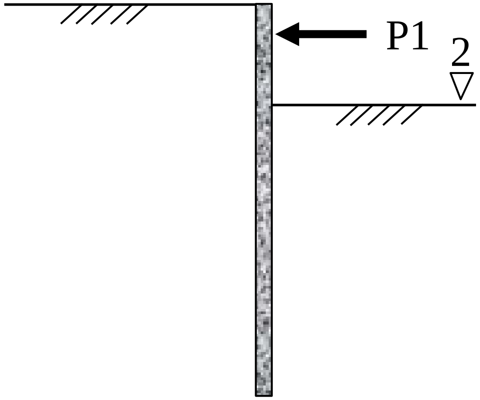
Most probable construction sequence

Characteristic construction sequence

Stage 2: Install 1st pop (P1) and dig to excavation level 2

Stage 2: Install 1st pop (P1) and dig to excavation level 2

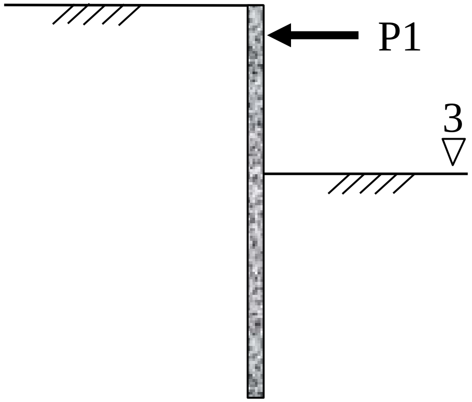




Ab initio – Approach A

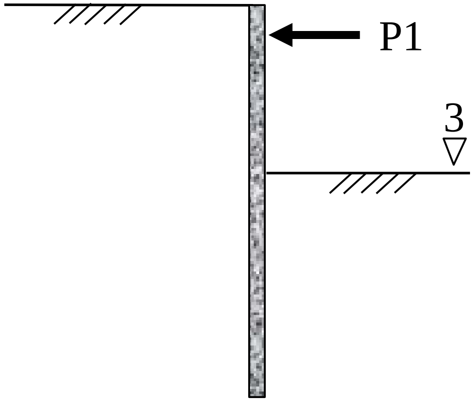
Most probable construction sequence

Stage 3: Dig to excavation level 3



Characteristic construction sequence

Stage 3: Dig to excavation level 3
(just below 2nd prop level)

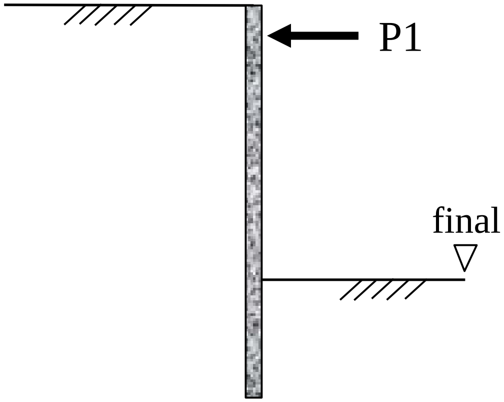


Ab initio – Approach A

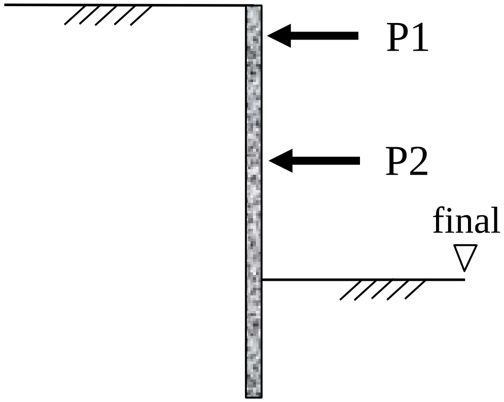
Most probable construction sequence

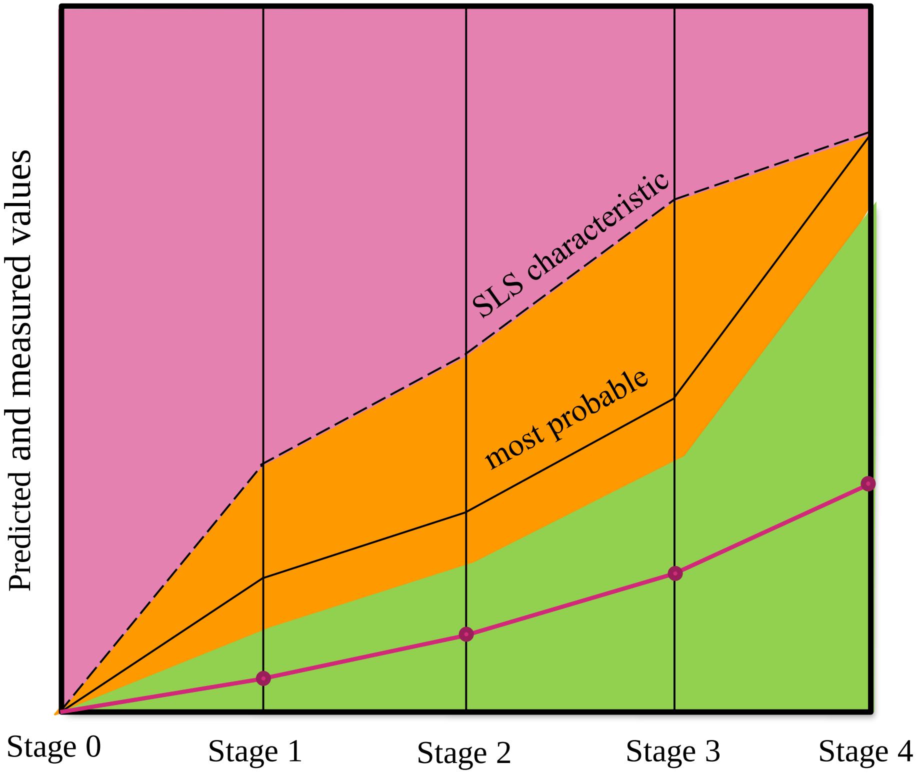
Characteristic construction sequence

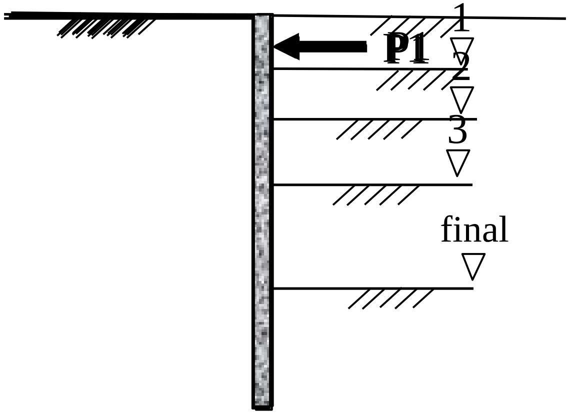
Stage 4: Dig to final excavation level

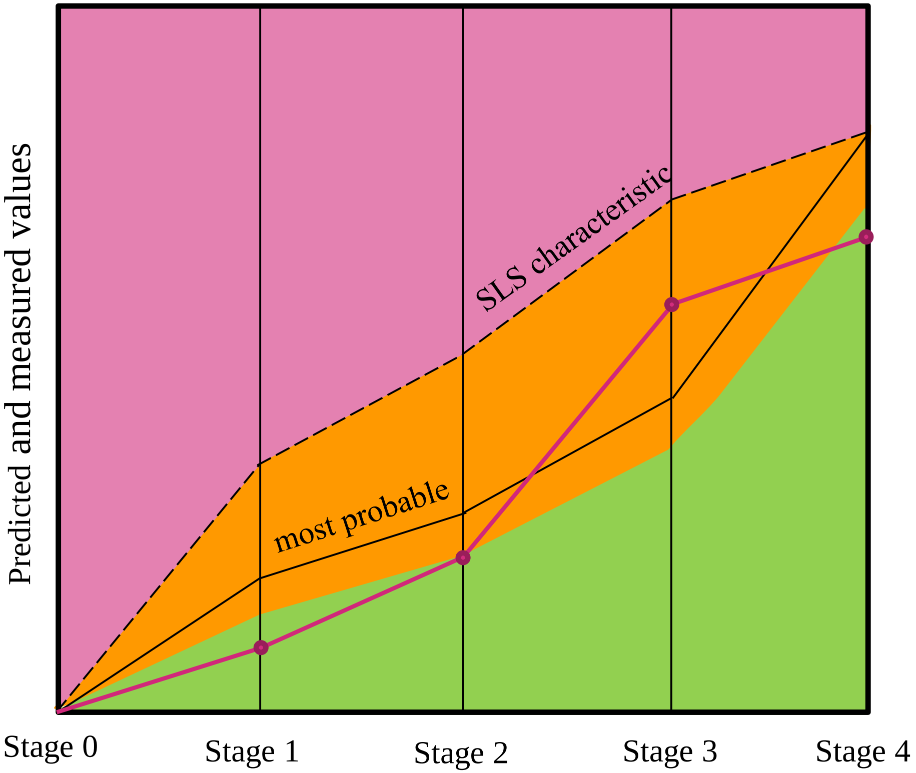


Stage 4: Install 2nd prop (P2) and dig to final excavation level




Predicted and measured values
SLS characteristic
most probable
Stage 0
Stage 1
Stage 2
Stage 3
Stage 4


P1
1
2
3
final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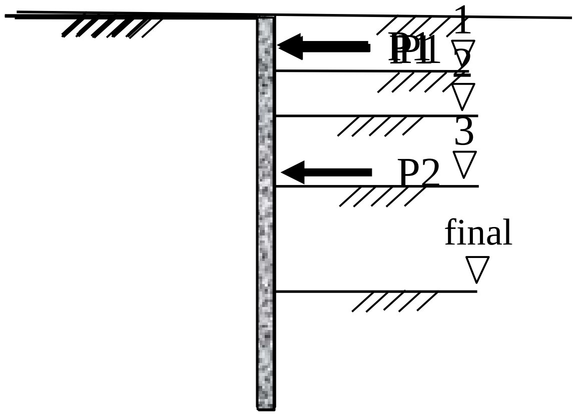
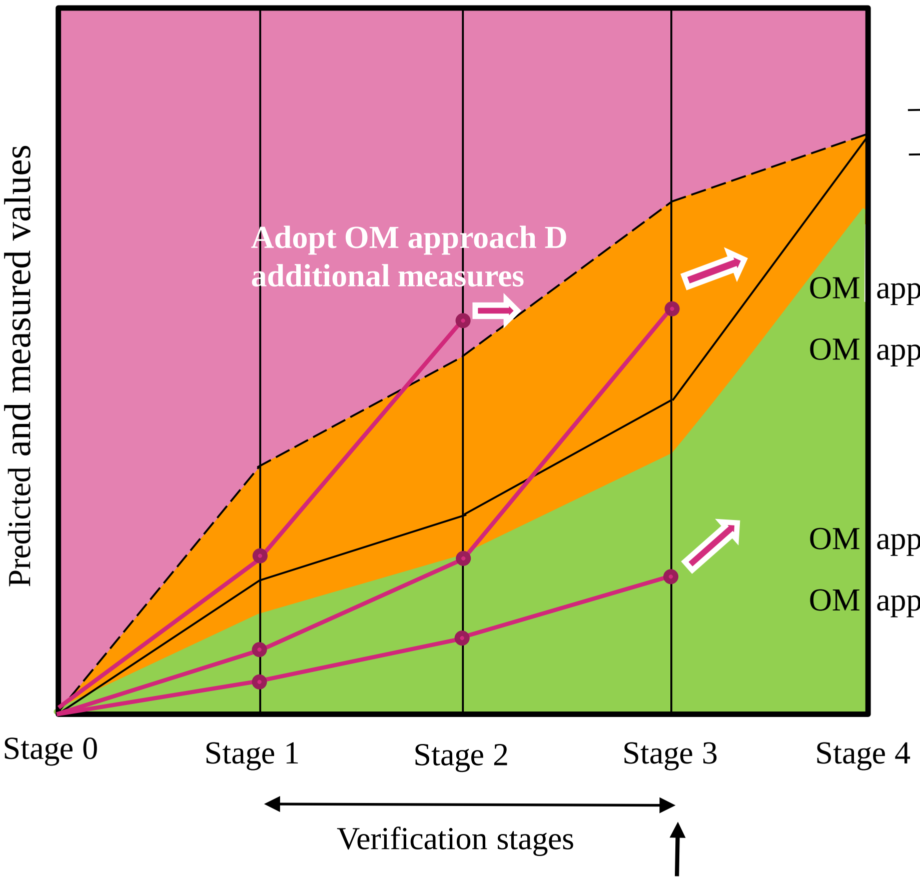
Stage 0
Stage 1
Stage 2
Stage 3
Stage 4


1
P1
2
3
P2
final



--- SLS characteristic

—— Most probable

OM approach A: adopt alternative characteristic approach

OM approach B: continue with characteristic approach

OM approach A: continue with most probable approach

OM approach B: adopt most probable approach

Decision

OM approach A : to continue with most probable approach or to revert to characteristic approach?

OM approach B: to continue with characteristic approach or to apply the most probable approach?

The Observational Method

New proposed framework – C760

OM Approach	***Ab initio* (from the start)**	
	A Optimistically pro-active	**B** Cautiously pro-active
OM design works starts	Before construction	
Starting design	Most probable	Characteristic
Alternative design	Contingency plan	Modification plan
Benefits	Optimised	Program and support only

The Observation Method – *Ab initio*

What will govern the choice between optimistic and cautious?

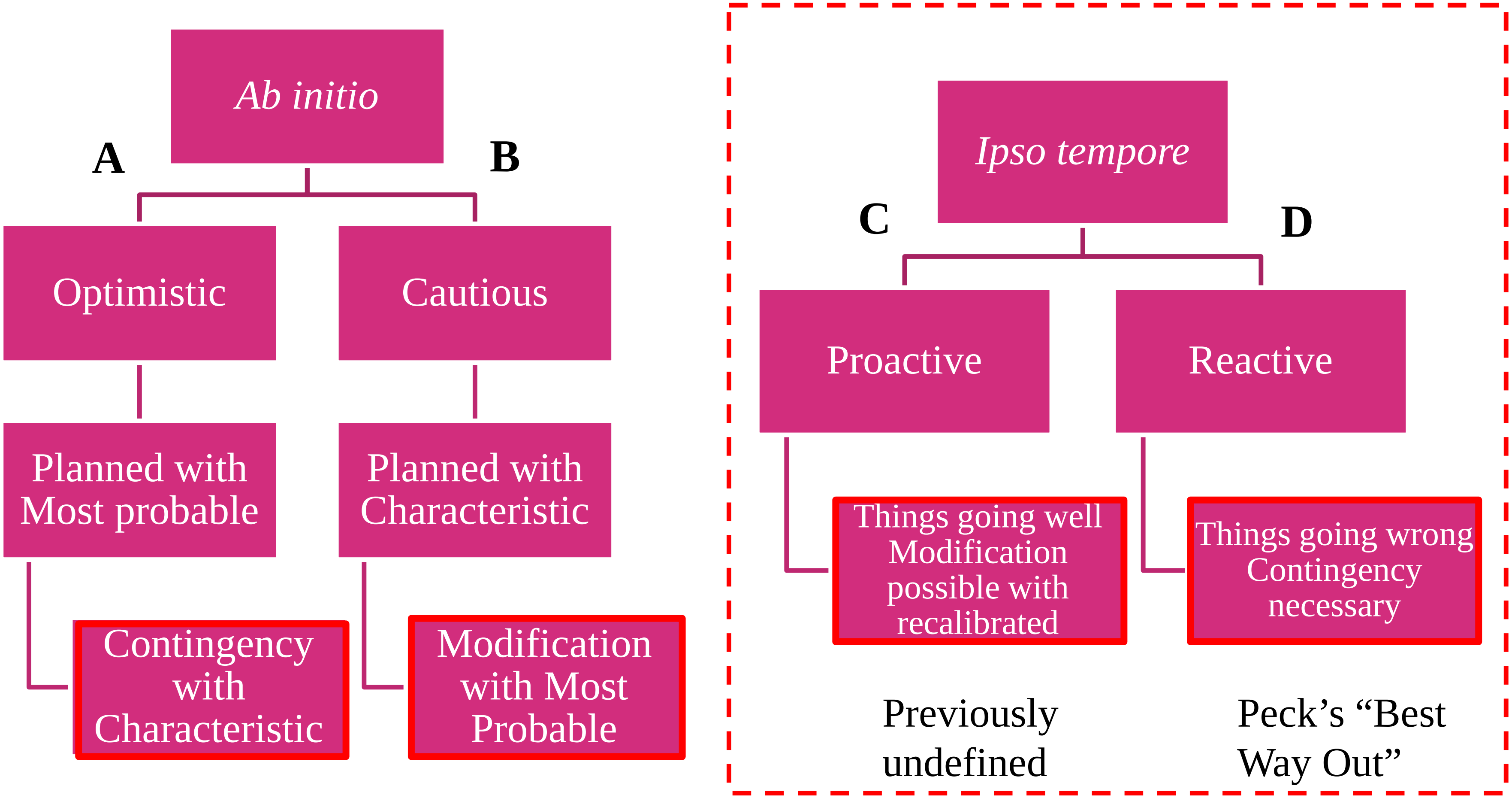
- Familiarity with ground conditions
  - Designer and contractor
  - Availability and reliability of documented case history data

- Contractual environment

- Appetite for risk management and cost savings

The Observational Method

New proposed framework – C760



Ab initio

A

B

Optimistic

Cautious

Planned with Most probable

Planned with Characteristic

Contingency with Characteristic

Modification with Most Probable

Ipso tempore

C

D

Proactive

Reactive

Things going well Modification possible with recalibrated

Things going wrong Contingency necessary

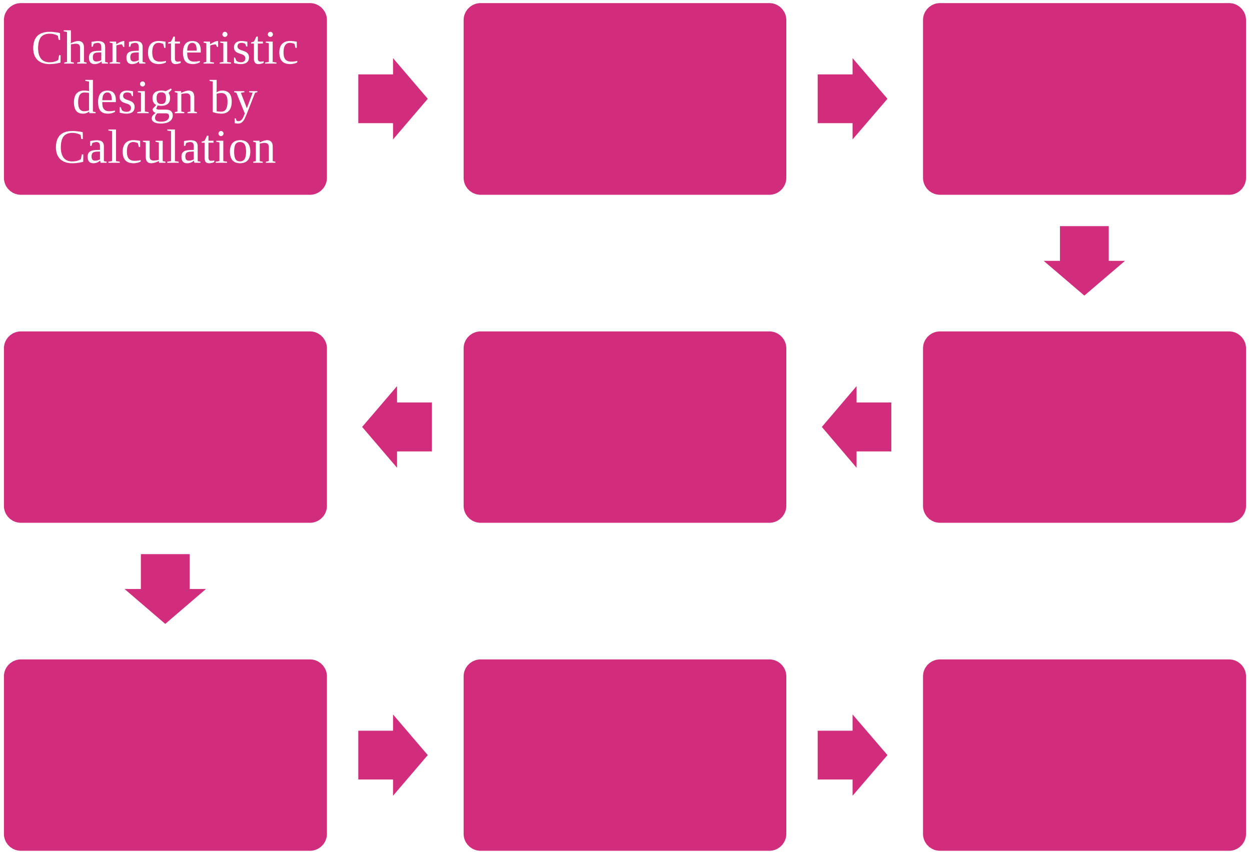
Previously undefined

Peck’s “Best Way Out”

Ab initio in Peck 1969 *Ab initio* in C185

The Observational Method

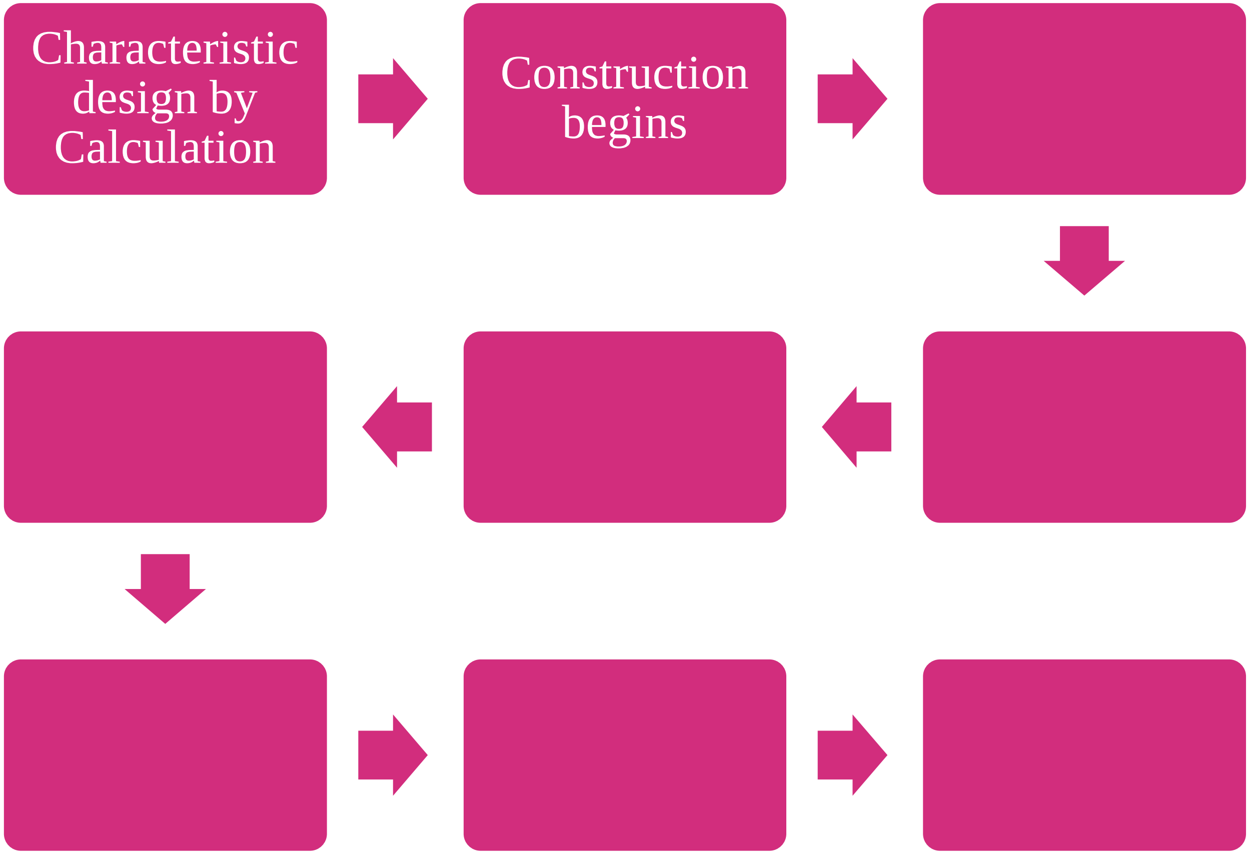
Approach C – *ipso tempore* pro-active



Characteristic design by Calculation

The Observational Method

Approach C – *ipso tempore* pro-active

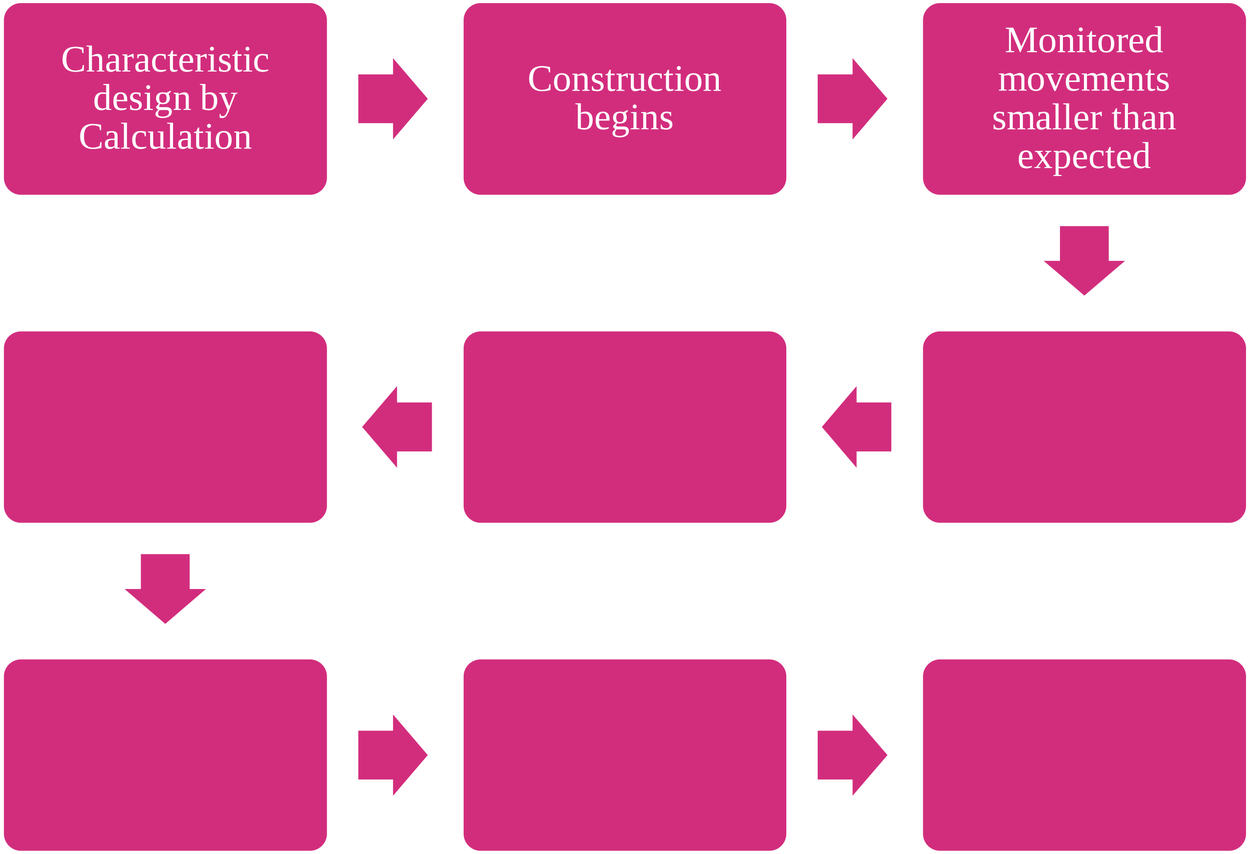


Characteristic design by Calculation

Construction begins

The Observational Method

Approach C – *ipso tempore* pro-active



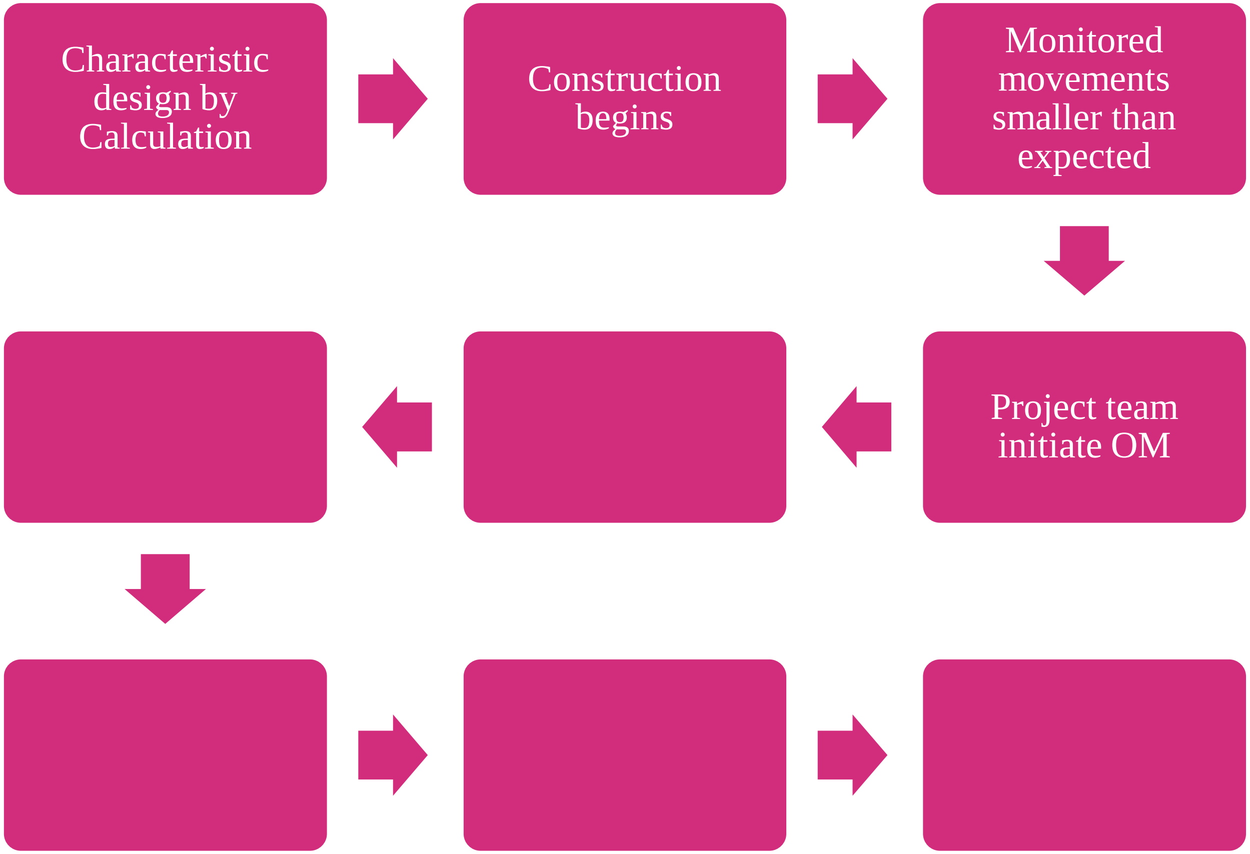
Characteristic design by Calculation

Construction begins

Monitored movements smaller than expected

The Observational Method

Approach C – *ipso tempore* pro-active



Characteristic design by Calculation

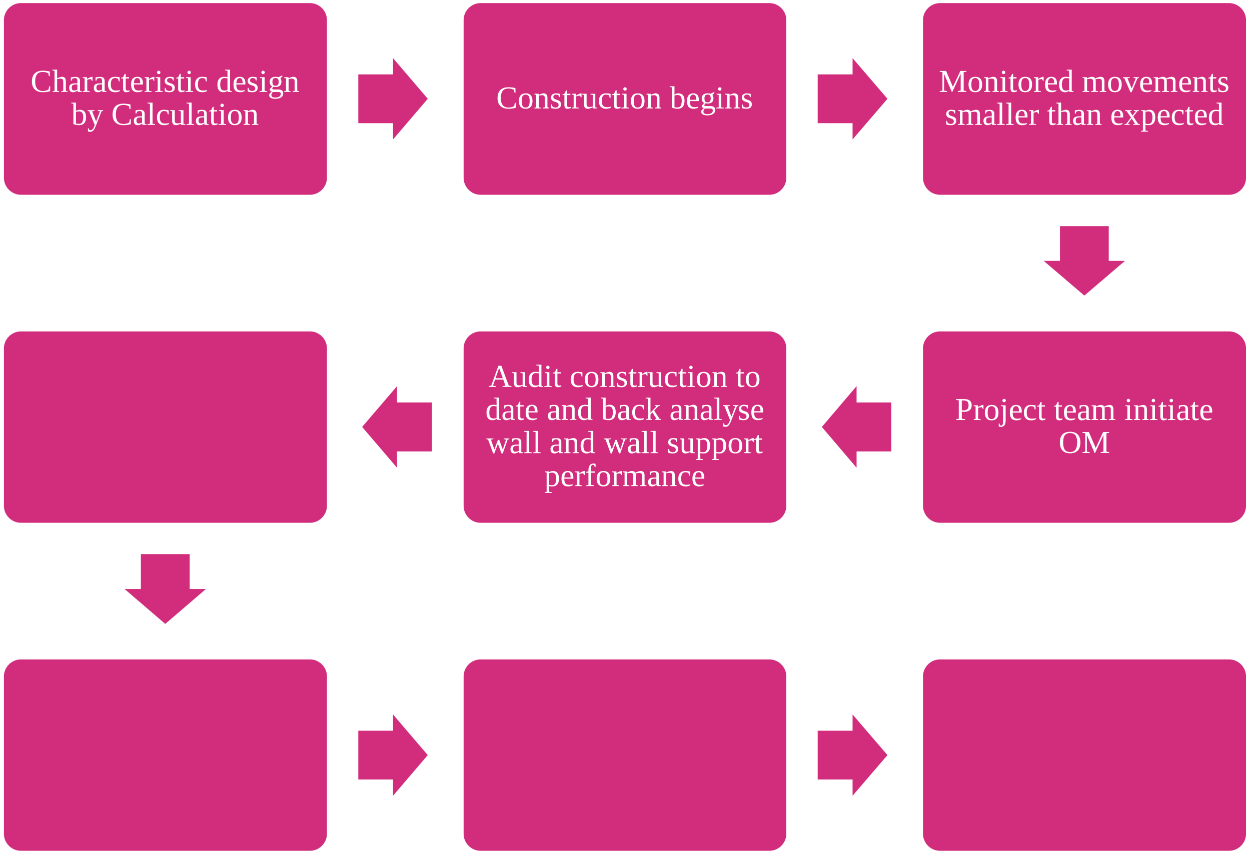
Construction begins

Monitored movements smaller than expected

Project team initiate OM

The Observational Method

Approach C – *ipso tempore* pro-active



Characteristic design by Calculation

Construction begins

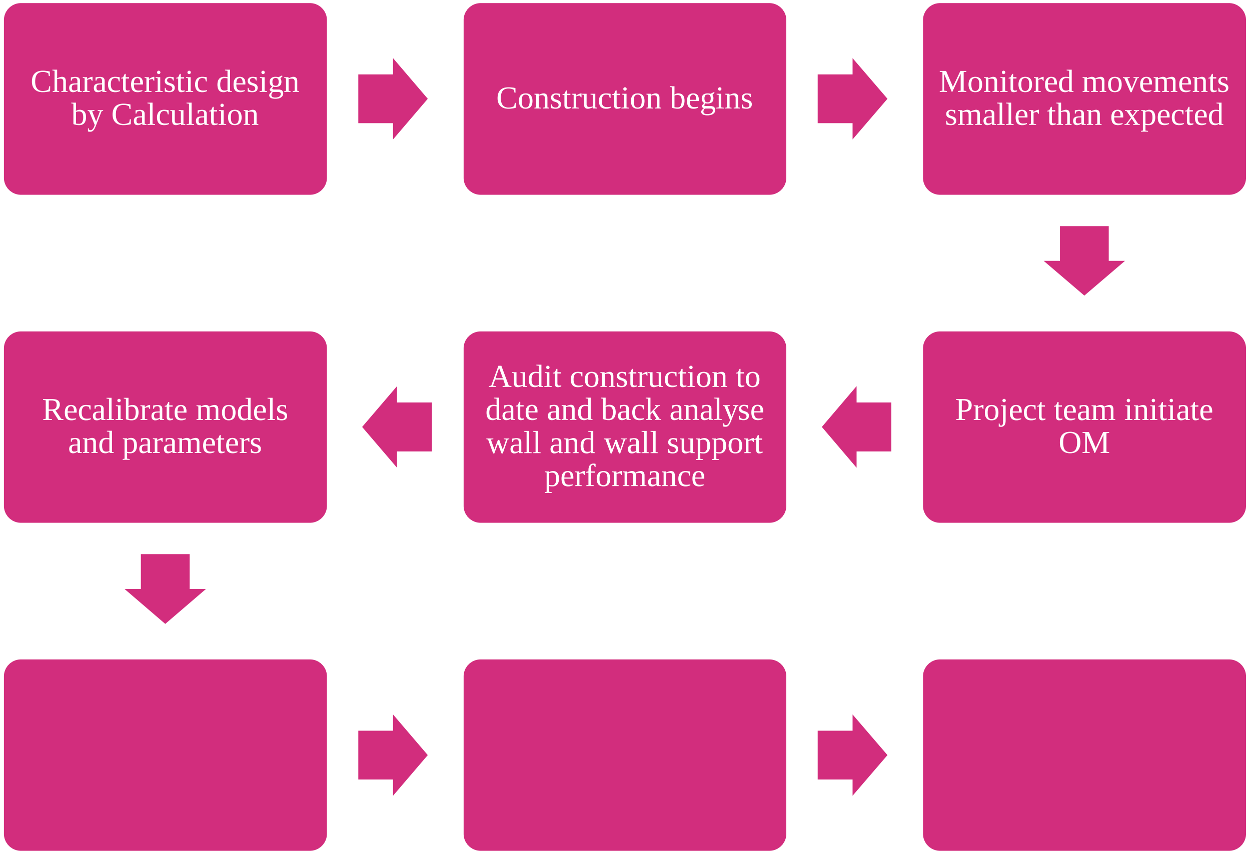
Monitored movements smaller than expected

Project team initiate OM

Audit construction to date and back analyse wall and wall support performance

The Observational Method

Approach C – *ipso tempore* pro-active



Characteristic design by Calculation

Construction begins

Monitored movements smaller than expected

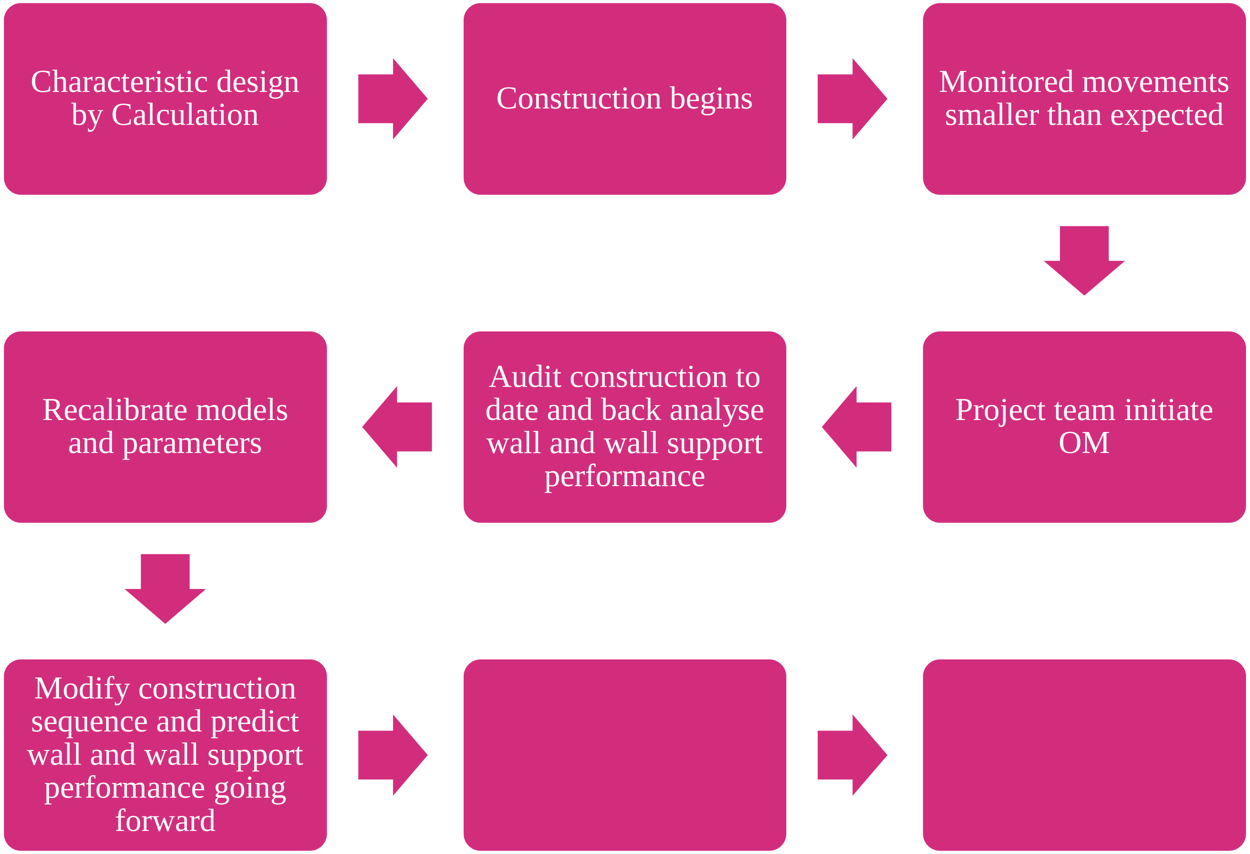
Project team initiate OM

Audit construction to date and back analyse wall and wall support performance

Recalibrate models and parameters

The Observational Method

Approach C – *ipso tempore* pro-active



Characteristic design by Calculation

Construction begins

Monitored movements smaller than expected

Project team initiate OM

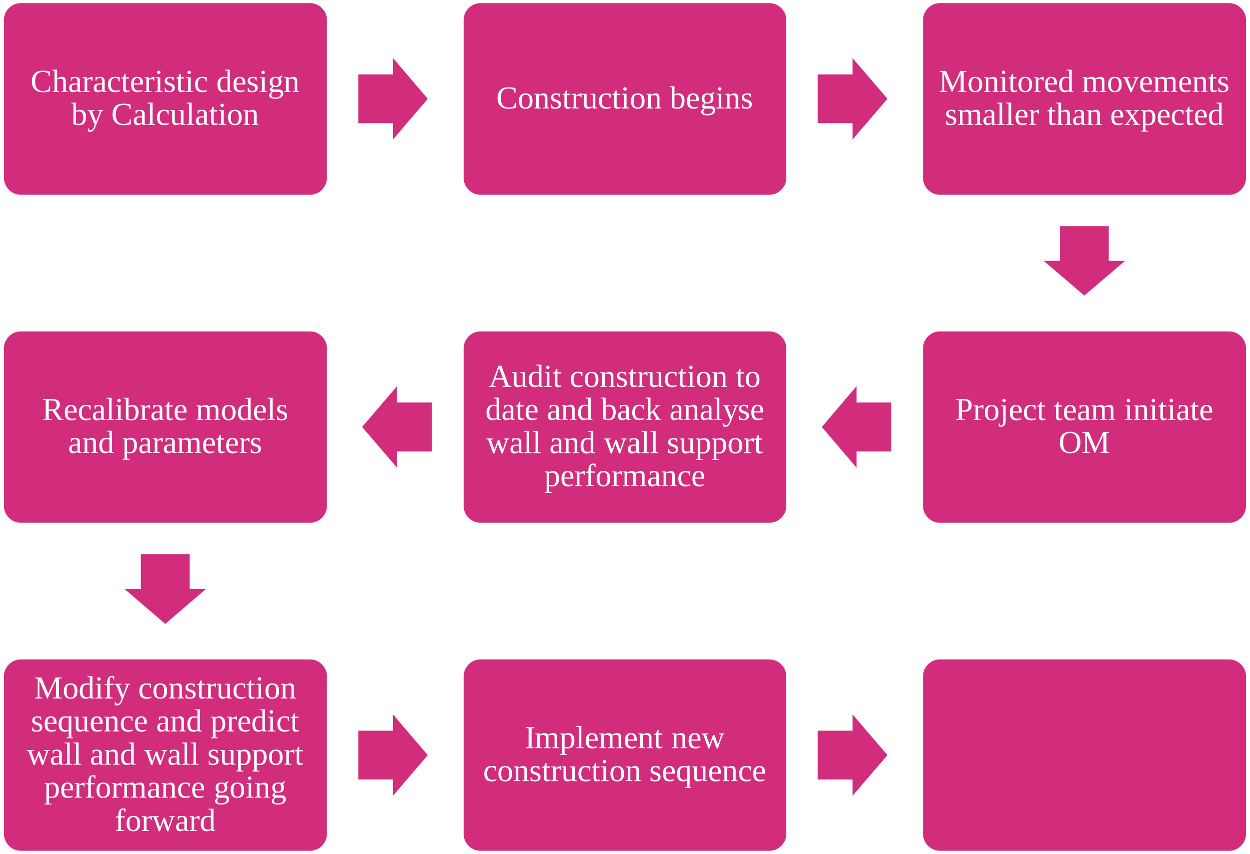
Audit construction to date and back analyse wall and wall support performance

Recalibrate models and parameters

Modify construction sequence and predict wall and wall support performance going forward

The Observational Method

Approach C – *ipso tempore* pro-active



Characteristic design by Calculation

Construction begins

Monitored movements smaller than expected

Project team initiate OM

Audit construction to date and back analyse wall and wall support performance

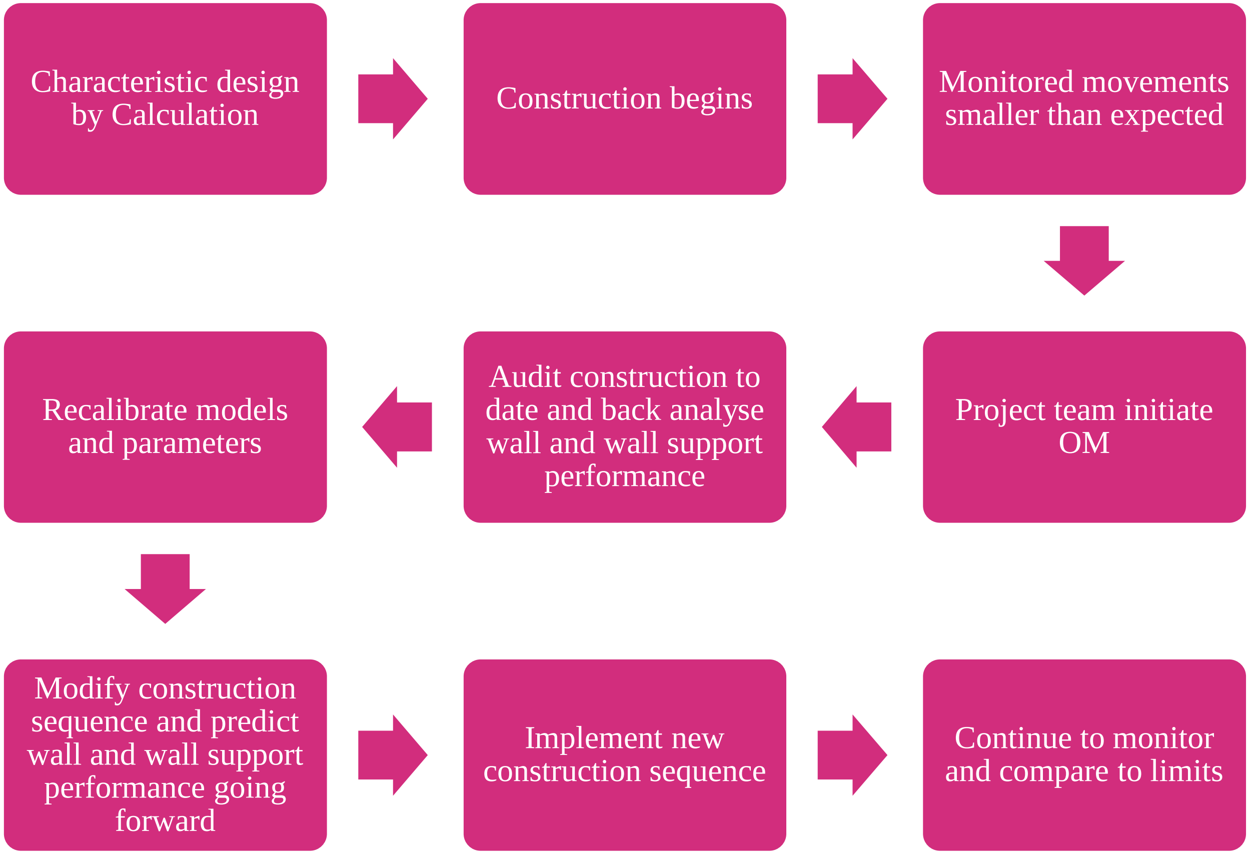
Recalibrate models and parameters

Modify construction sequence and predict wall and wall support performance going forward

Implement new construction sequence

The Observational Method

Approach C – *ipso tempore* pro-active



Characteristic design by Calculation

Construction begins

Monitored movements smaller than expected

Project team initiate OM

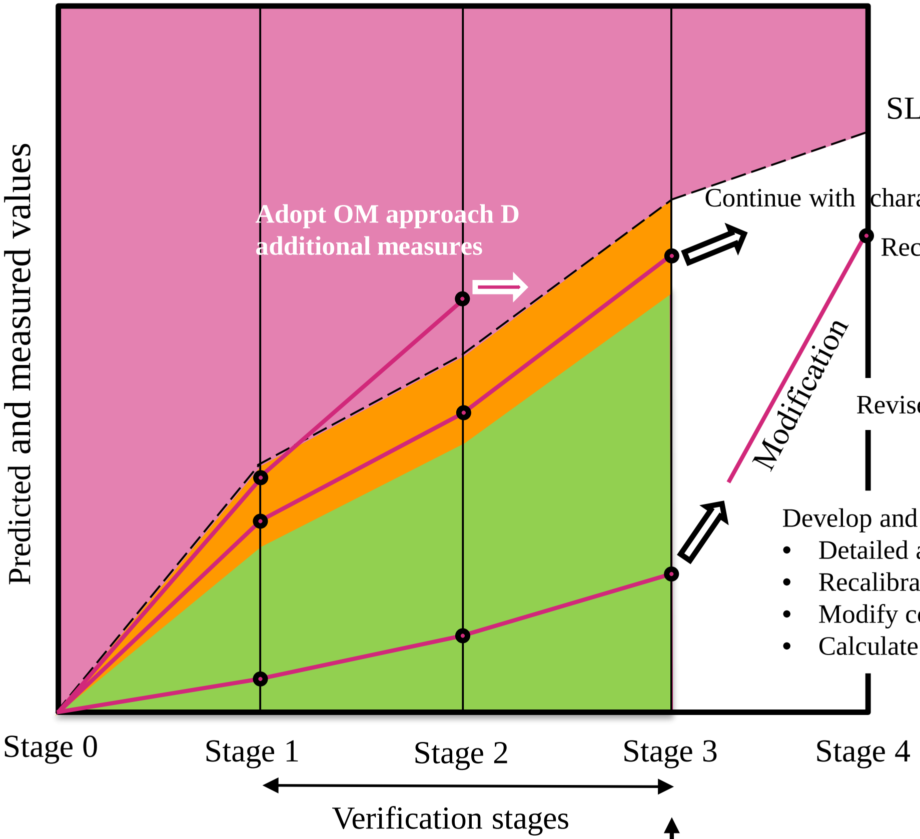
Audit construction to date and back analyse wall and wall support performance

Recalibrate models and parameters

Modify construction sequence and predict wall and wall support performance going forward

Implement new construction sequence

Continue to monitor and compare to limits



SLS characteristic

Continue with characteristic approach

Recalibrated (with modified construction sequence)

Revise trigger levels for modified construction sequence.

Develop and adopt modified construction sequence after undertaking the following:

- Detailed audit of construction sequence to date
- Recalibrate models and parameters on basis of back analyses of actual performance
- Modify construction sequence going forward
- Calculate recalibrated wall and wall support performance for modified construction sequence

Decision

Whether to continue with characteristic approach or to recalibrate and modify future construction sequences?

The Observational Method

Approach D – *ipso tempore* reactive

Reactive is similar to pro-active except trying to prevent SLS or ULS failure rather than make improvements

- Movements are observed to be greater than expected
- Recalibrate parameters and behaviour
- Plan contingency measures (e.g. additional propping, etc.)
- Redefine limits based on recalibrated model
- Implement contingency plan
- Continue to monitor against trigger limits

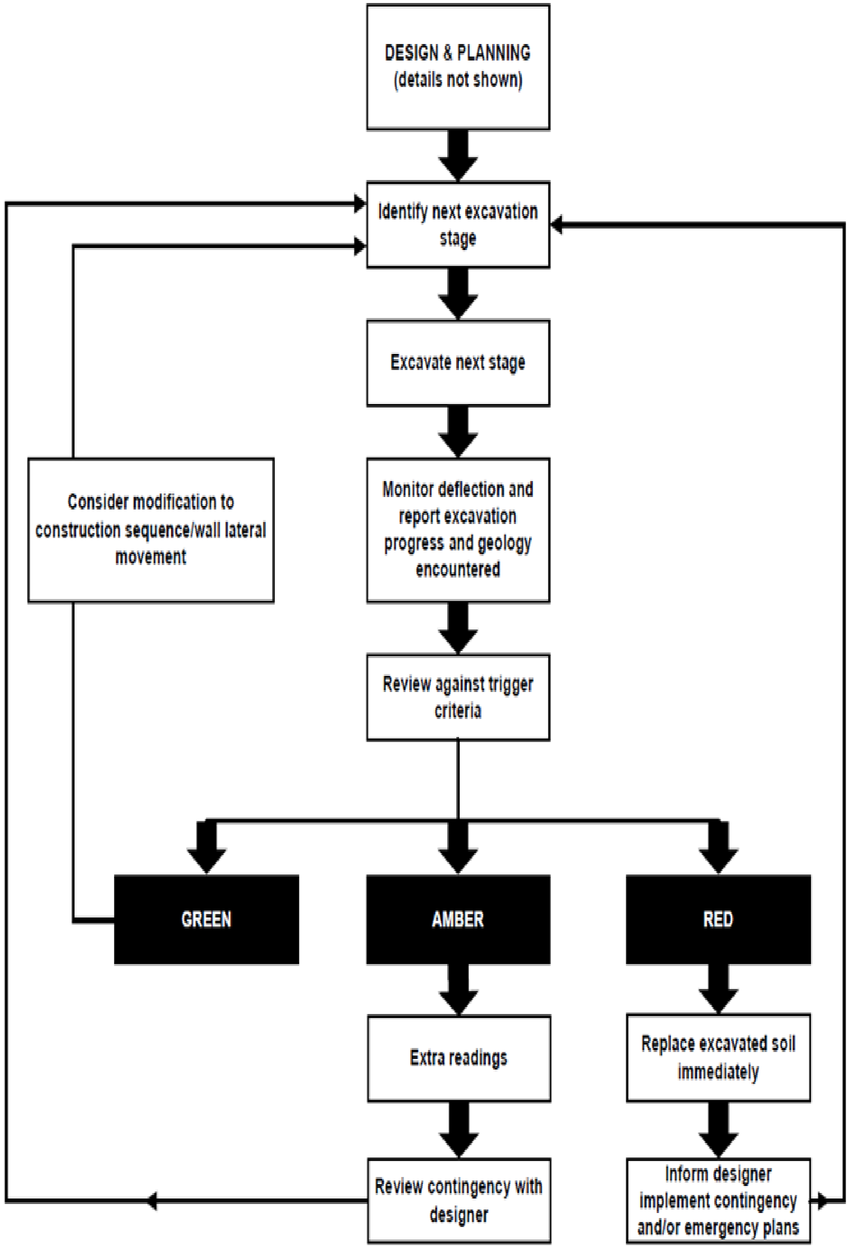
Presentation Outline

- Background
  - Peck (1969)
  - CIRIA C185 (Nicholson *et al.*, 1999)
  - EC7 (2004)

- New C760 Observational Method implementation Framework
  - *Ab Initio*
    - **A** – optimistically pro-active
    - **B** – cautiously pro-active
  - *Ipso tempore*
    - **C** – pro-active to make modifications
    - **D** – reactive to make corrections

- Observational Method application
  - Illustrative scenarios

- **Closing comments**

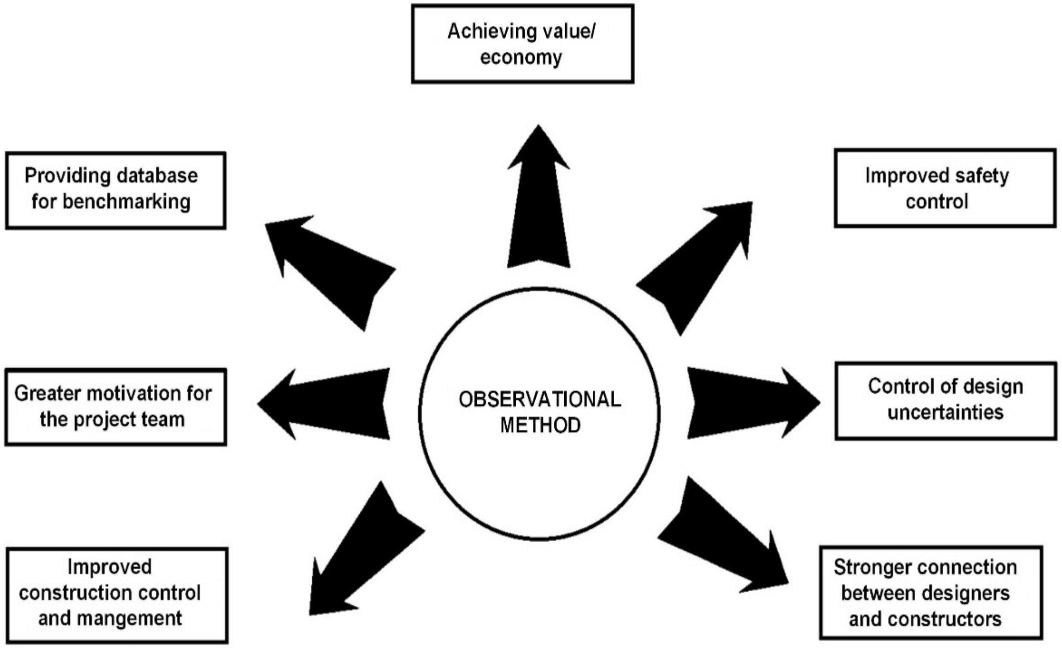


The Observational Method – closing comments

The Observational Method can offer significant savings in cost and programme with improved safety with explicit risk management procedures

C760 provides a robust framework for the application of OM within Eurocode environment

Application of OM requires all members of the design and construction teams to have clear understandings of overall objectives, key criteria and clarity of individual and collective responsibilities – very close collaborative working explicitly managing safety and risk



OBSERVATIONAL METHOD

Achieving value/ economy

Improved safety control

Control of design uncertainties

Stronger connection between designers and constructors

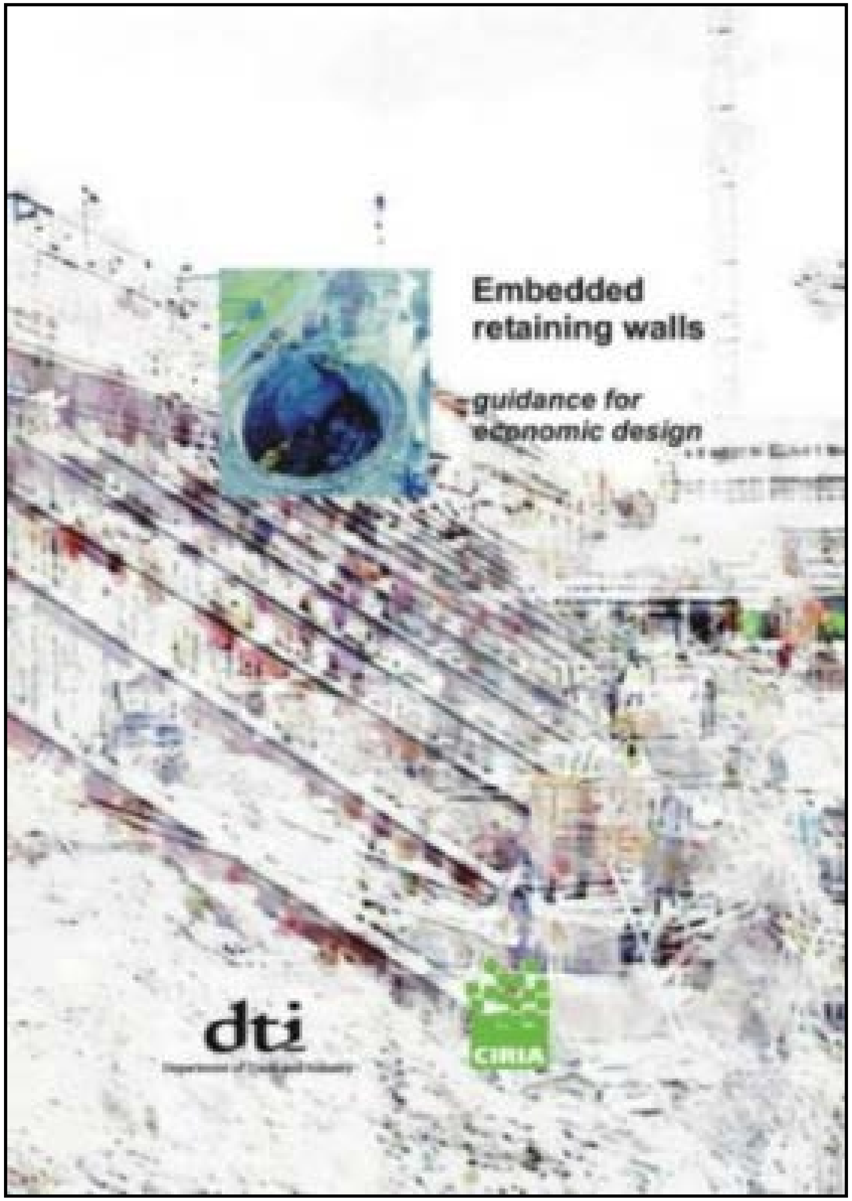
Improved construction control and mangement

Greater motivation for the project team

Providing database for benchmarking

C760
Embedded Retaining walls:
guidance for design

WORKED EXAMPLE



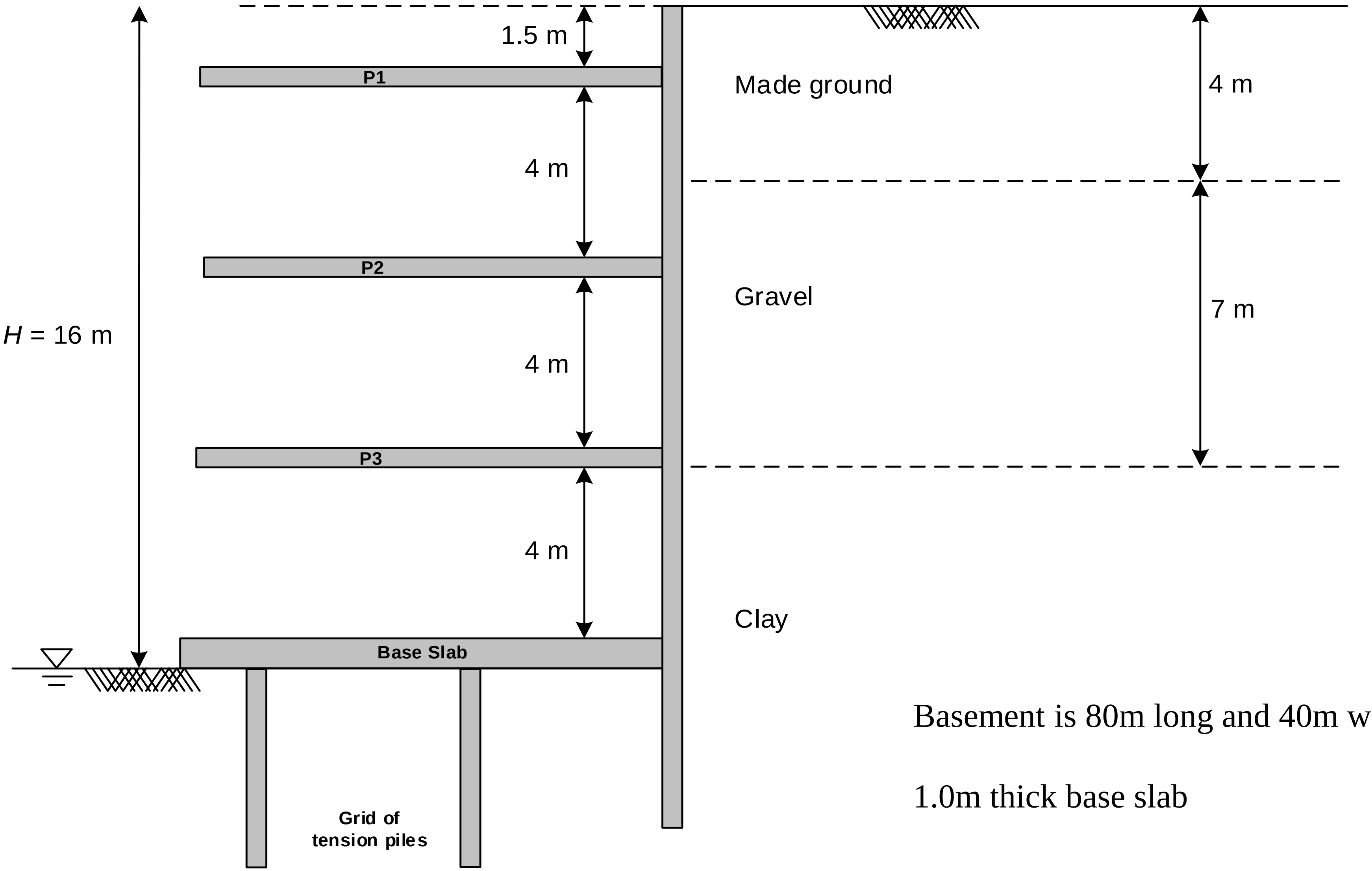
Embedded retaining walls

guidance for economic design

dti

CIRIA

Worked Example – geometry and stratigraphy

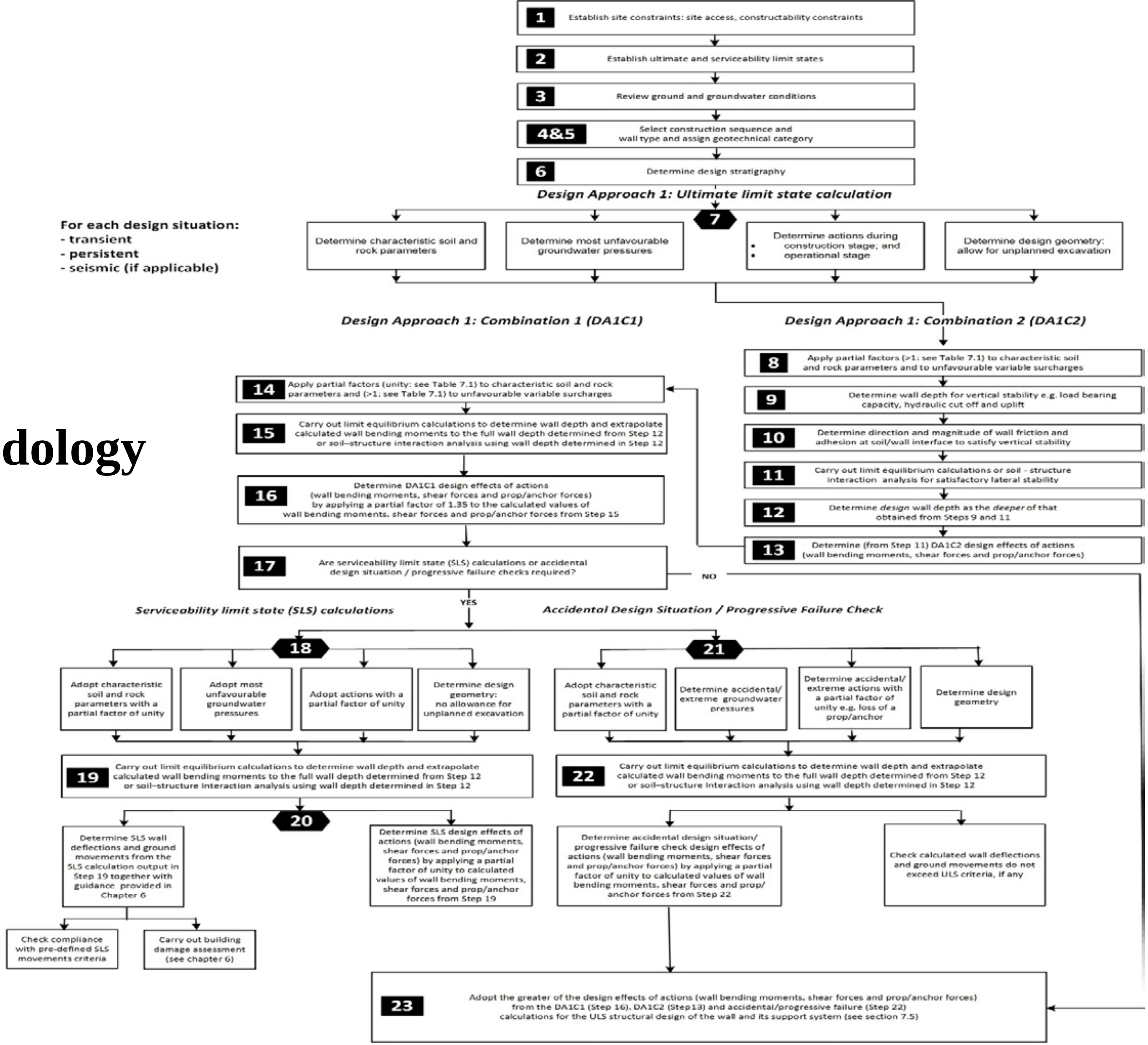


Basement is 80m long and 40m wide

1.0m thick base slab

Worked Example

Design methodology



1 Establish site constraints: site access, constructability constraints

2 Establish ultimate and serviceability limit states

3 Review ground and groundwater conditions

4&5 Select construction sequence and wall type and assign geotechnical category

6 Determine design stratigraphy

Design Approach 1: Ultimate limit state calculation

7

For each design situation:
- transient
- persistent
- seismic (if applicable)

Determine characteristic soil and rock parameters

Determine most unfavourable groundwater pressures

Determine actions during
- construction stage; and
- operational stage

Determine design geometry: allow for unplanned excavation

Design Approach 1: Combination 1 (DA1C1)

Design Approach 1: Combination 2 (DA1C2)

8 Apply partial factors (>1: see Table 7.1) to characteristic soil and rock parameters and to unfavourable variable surcharges

9 Determine wall depth for vertical stability e.g. load bearing capacity, hydraulic cut off and uplift

10 Determine direction and magnitude of wall friction and adhesion at soil/wall interface to satisfy vertical stability

11 Carry out limit equilibrium calculations or soil - structure interaction analysis for satisfactory lateral stability

12 Determine *design* wall depth as the *deeper* of that obtained from Steps 9 and 11

13 Determine (from Step 11) DA1C2 design effects of actions (wall bending moments, shear forces and prop/anchor forces)

14 Apply partial factors (unity: see Table 7.1) to characteristic soil and rock parameters and (>1: see Table 7.1) to unfavourable variable surcharges

15 Carry out limit equilibrium calculations to determine wall depth and extrapolate calculated wall bending moments to the full wall depth determined from Step 12 or soil–structure interaction analysis using wall depth determined in Step 12

16 Determine DA1C1 design effects of actions (wall bending moments, shear forces and prop/anchor forces) by applying a partial factor of 1.35 to the calculated values of wall bending moments, shear forces and prop/anchor forces from Step 15

17 Are serviceability limit state (SLS) calculations or accidental design situation / progressive failure checks required?

NO

YES

Serviceability limit state (SLS) calculations

Accidental Design Situation / Progressive Failure Check

18

Adopt characteristic soil and rock parameters with a partial factor of unity

Adopt most unfavourable groundwater pressures

Adopt actions with a partial factor of unity

Determine design geometry: no allowance for unplanned excavation

19 Carry out limit equilibrium calculations to determine wall depth and extrapolate calculated wall bending moments to the full wall depth determined from Step 12 or soil–structure interaction analysis using wall depth determined in Step 12

20

Determine SLS wall deflections and ground movements from the SLS calculation output in Step 19 together with guidance provided in Chapter 6

Check compliance with pre-defined SLS movements criteria

Carry out building damage assessment (see chapter 6)

Determine SLS design effects of actions (wall bending moments, shear forces and prop/anchor forces) by applying a partial factor of unity to calculated values of wall bending moments, shear forces and prop/anchor forces from Step 19

21

Adopt characteristic soil and rock parameters with a partial factor of unity

Determine accidental/ extreme groundwater pressures

Determine accidental/ extreme actions with a partial factor of unity e.g. loss of a prop/anchor

Determine design geometry

22 Carry out limit equilibrium calculations to determine wall depth and extrapolate calculated wall bending moments to the full wall depth determined from Step 12 or soil–structure interaction analysis using wall depth determined in Step 12

Determine accidental design situation/ progressive failure check design effects of actions (wall bending moments, shear forces and prop/anchor forces) by applying a partial factor of unity to calculated values of wall bending moments, shear forces and prop/ anchor forces from Step 22

Check calculated wall deflections and ground movements do not exceed ULS criteria, if any

23 Adopt the greater of the design effects of actions (wall bending moments, shear forces and prop/anchor forces) from the DA1C1 (Step 16), DA1C2 (Step13) and accidental/progressive failure (Step 22) calculations for the ULS structural design of the wall and its support system (see section 7.5)

Worked Example – design methodology steps: initial steps

Establish site constraints

- Basement in urban environment
- Super structure construction above basement: retaining wall is load bearing in short term and long term with permanent load of 150kN/m and a variable load of 50kN/m at all construction stages
- Access road behind wall at offset > 1m
- Permanent long term sub-slab drainage

Establish limit states

- ULS
- SLS : watertightness, permissible ground surface movements, etc.

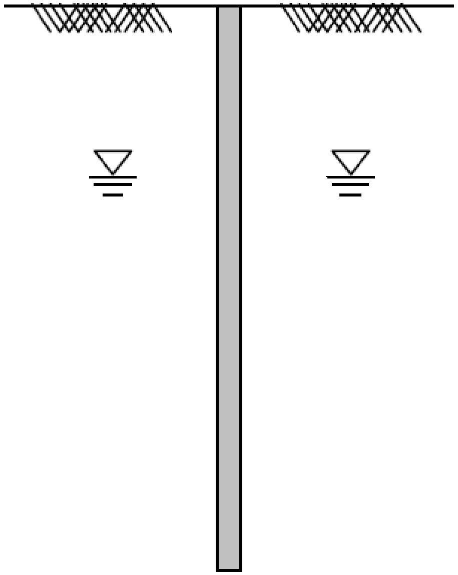
Review ground and groundwater conditions

Select construction sequence, wall type, assign geotechnical category

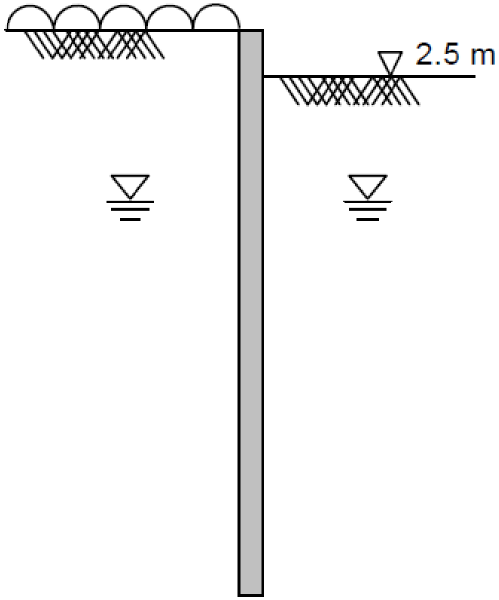
- Geotechnical category 2
- 900mm hard-hard secant pile wall (external casing diameter is 880mm: resulting pile diameter is 900mm due to presence of outside driving teeth; auger diameter is 750mm)

Determine design geometry and stratigraphy

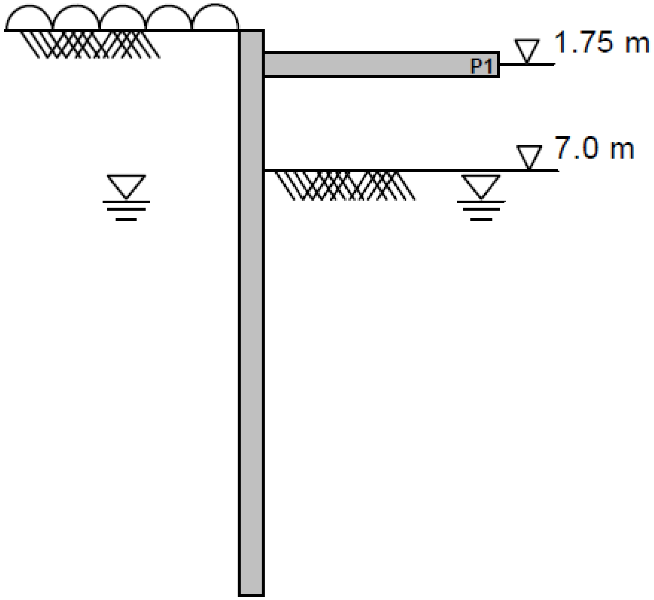
Worked Example – construction sequence stages 1 to 3



Stage 1: Install wall

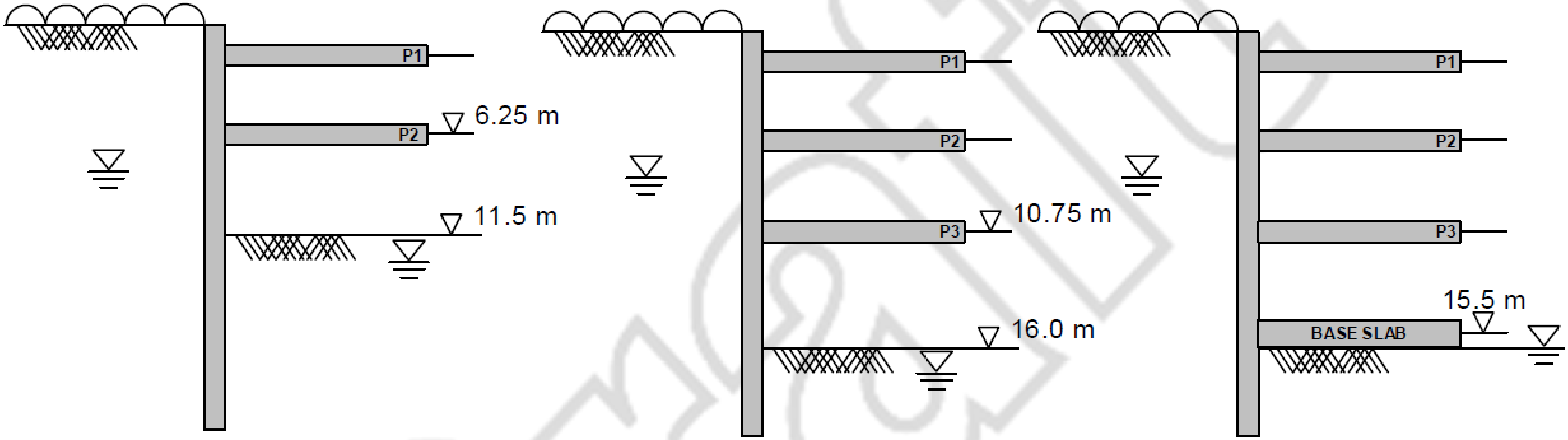


Stage 2: Apply a 10 kPa surcharge outside the wall and excavate to 2.5 m depth



Stage 3: Install (500 mm thick)slab at 1.75 m depth and excavate to 7.0 m depth

Worked Example – construction sequence stages 4 to 6

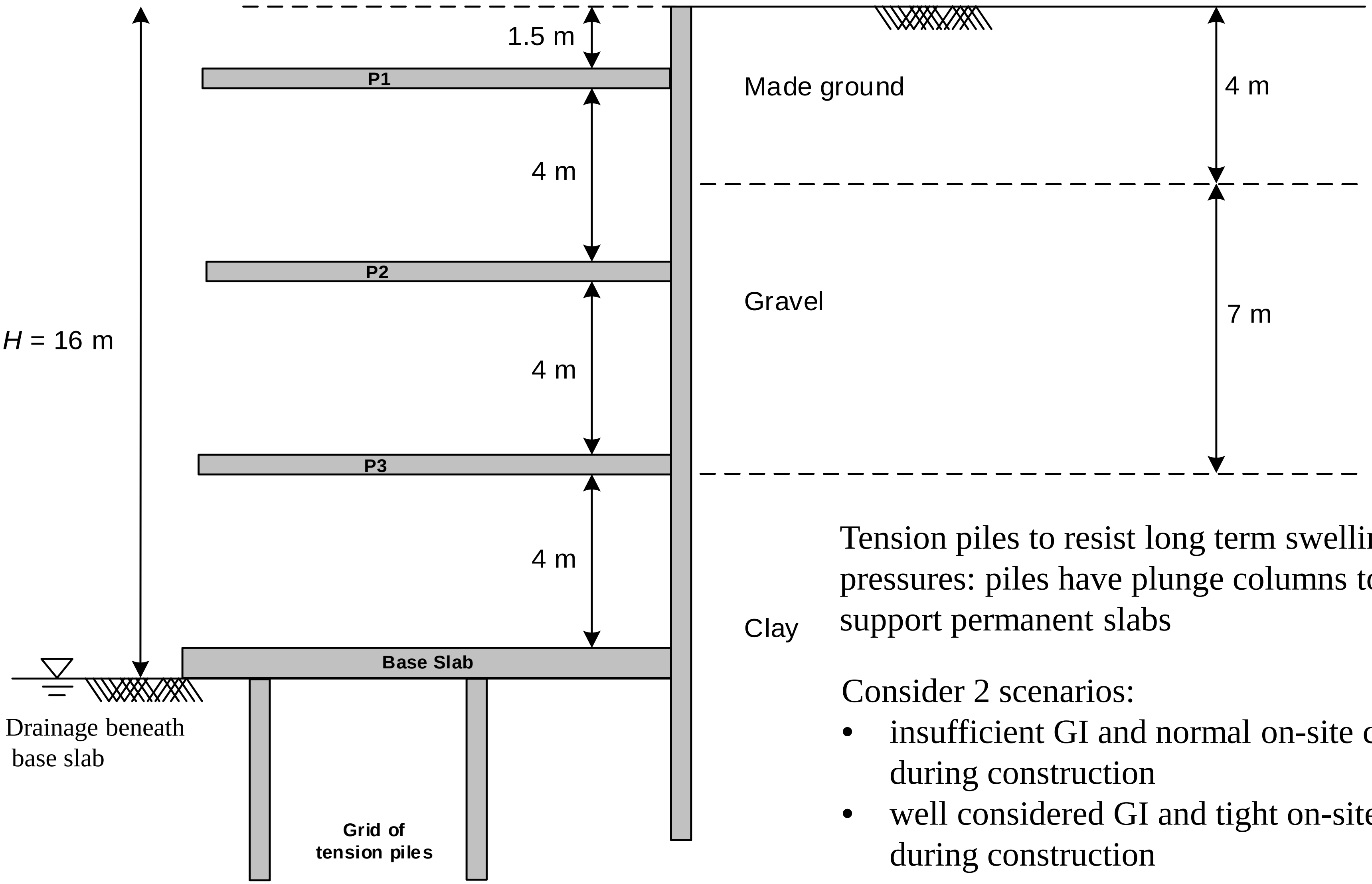


Stage 4: Install (500 mm thick) slab at 6.25 m depth, dewater the gravel and excavate to 11.5 m depth

Stage 5: Install (500 mm thick) slab at 10.75 m depth and excavate to 16.0m depth

Stage 6: Install (1 m thick) base slab at 15.5 m depth with permanent drainage beneath. Clay drains in long term, with water at base slab soffit level.

Worked Example – geometry and stratigraphy



Drainage beneath
base slab

Tension piles to resist long term swelling pressures: piles have plunge columns to support permanent slabs

Consider 2 scenarios:

- insufficient GI and normal on-site control during construction
- well considered GI and tight on-site control during construction

Worked Example

Nomogram for interlock of segmented cased piled walls

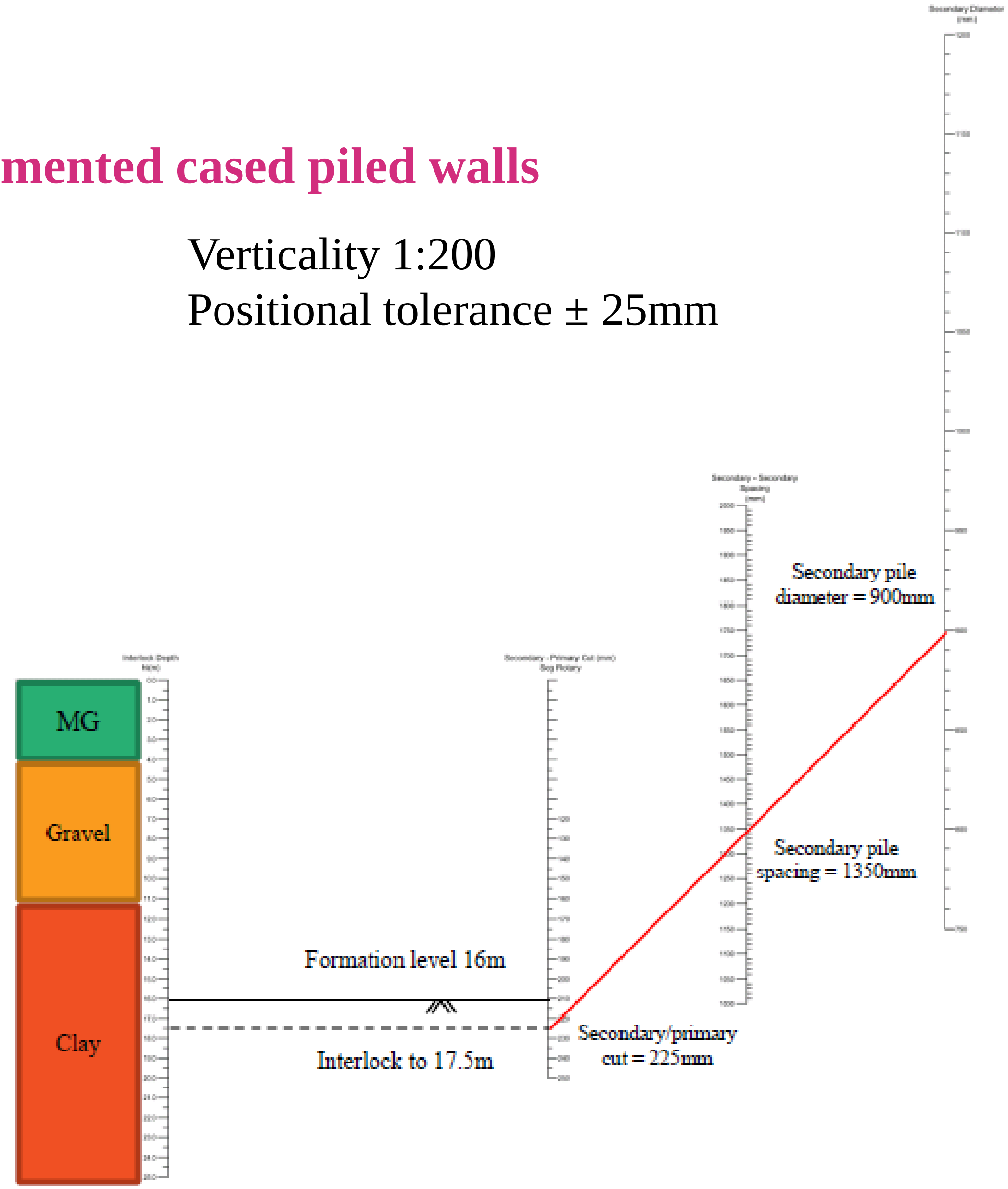
Required interlock between primary and secondary piles is 1m below formation

In this case 17.5m below ground level (including 0.5m for unplanned excavation)

Use nomogram to determine spacing between secondary piles = 1350mm

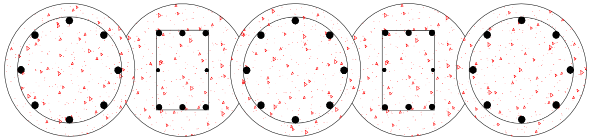
Spacing between primary and secondary piles is 675mm (i.e. 1350/2). Cut into primary pile (225mm) leaves half primary pile intact

Verticality 1:200
Positional tolerance ± 25mm



Worked Example – typical wall sizes

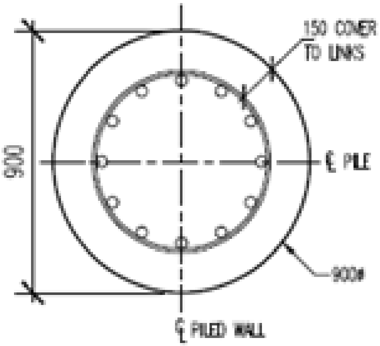
Wall Type	**Pile Diameter/Casing Size**	
	Casing Size (mm)	Drilling Tool Size (mm)
Secant CFA (Hard-Firm)	N/A	600
		750
		900
		1050
		1200
Secant Cased CFA (Hard-Firm)	700	600
	850	750
	880	780
	1000	900
Secant Segmental Cased Rotary (Hard-Firm, Hard-Hard)	508	450
	620	520
	750	650
	880	**750**
	1000	900
	1180	1050
	1300	1200
	1500	1350
	1650	1500
	2000	1830



900mm hard-hard secant pile wall (external casing diameter is 880mm: resulting pile diameter is 900mm due to presence of outside driving teeth; auger diameter is 750mm)

Worked Example – wall structural parameters

$I_{sec} = \pi D_{sec}^4 / 64.s$



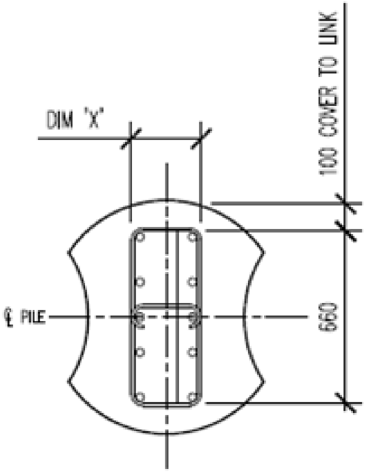
The stiffness of the cased section of the secondary piles (I_{sec}) where D = 900mm and s = 1350mm is:

$$I_{sec} = 0.024\text{m}^4/\text{m}$$

The cutting of the primary piles of diameter D_{prim} by the secondary piles, results in a complex shape that does not lend itself easily to calculation

The wall stiffness (I_{prim}) can be approximated by treating the primary piles as an equivalent rectangle of width w and depth d given by the following equations:

$$w = s - D_{sec}$$



$$d = 2\sqrt{\left(\frac{D_{prim}}{2}\right)^2 - \left(\frac{s - D_{sec}}{2}\right)^2}$$

$$I_{prim} = \frac{1}{12} w d^3 = 0.013\text{m}^4/\text{m}$$

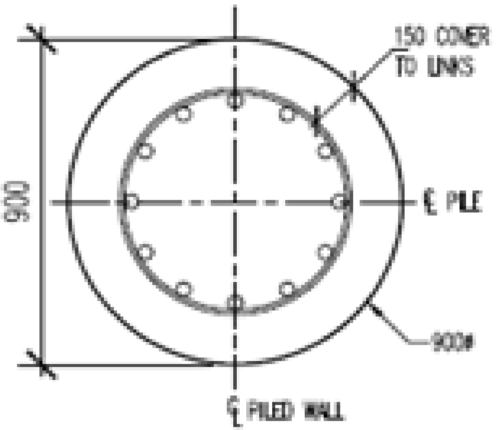
$$I_{wall} = I_{sec} + I_{prim} = 0.037\text{m}^4/\text{m}$$

Worked Example – wall structural parameters

Below the cased section of the wall (> 17.5m below ground level), the wall stiffness is derived from the secondary piles only with a reduced diameter assumed to be equal to the auger diameter (750mm) at 1350mm spacing

Wall $I_{wall} = \pi D^{4} / 64.s$

$I_{wall} = 0.0115\text{m}^4/\text{m}$



Young's modulus of concrete, E_0 = 30 MPa (C32/40 concrete, see Table 3.1 of EC2 Part 1)

Wall EI during construction = $0.7\, E_0\, I$

Wall EI in the long term = $0.5\, E_0\, I$

Worked Example – prop structural parameters

The props (slabs) act axially across the width of the basement and have a stiffness (k) given by:

Prop stiffness, $k = EA / l_{eff}\, s$

Where

l_{eff} is the effective length of the prop (point of zero axial movement).
For a symmetric basement, effective length of the prop is half the basement width (20m)

s is the prop (slab) spacing. The slabs are continuous across the basement: prop spacing is unity

Young's modulus of the concrete E_0 is taken to be 30 MPa (C32/40).
During construction, E is taken to be $0.7E_0$ and in the long term E is taken to be $0.5E_0$

Slab Thickness	Stiffness k during construction	Stiffness k in long term
500mm (P1, P2 and P3)	525,000kN/m/m	375,000kN/m/m

Worked Example – DA1 Combination 2 PLAXIS analysis

Strategy 1

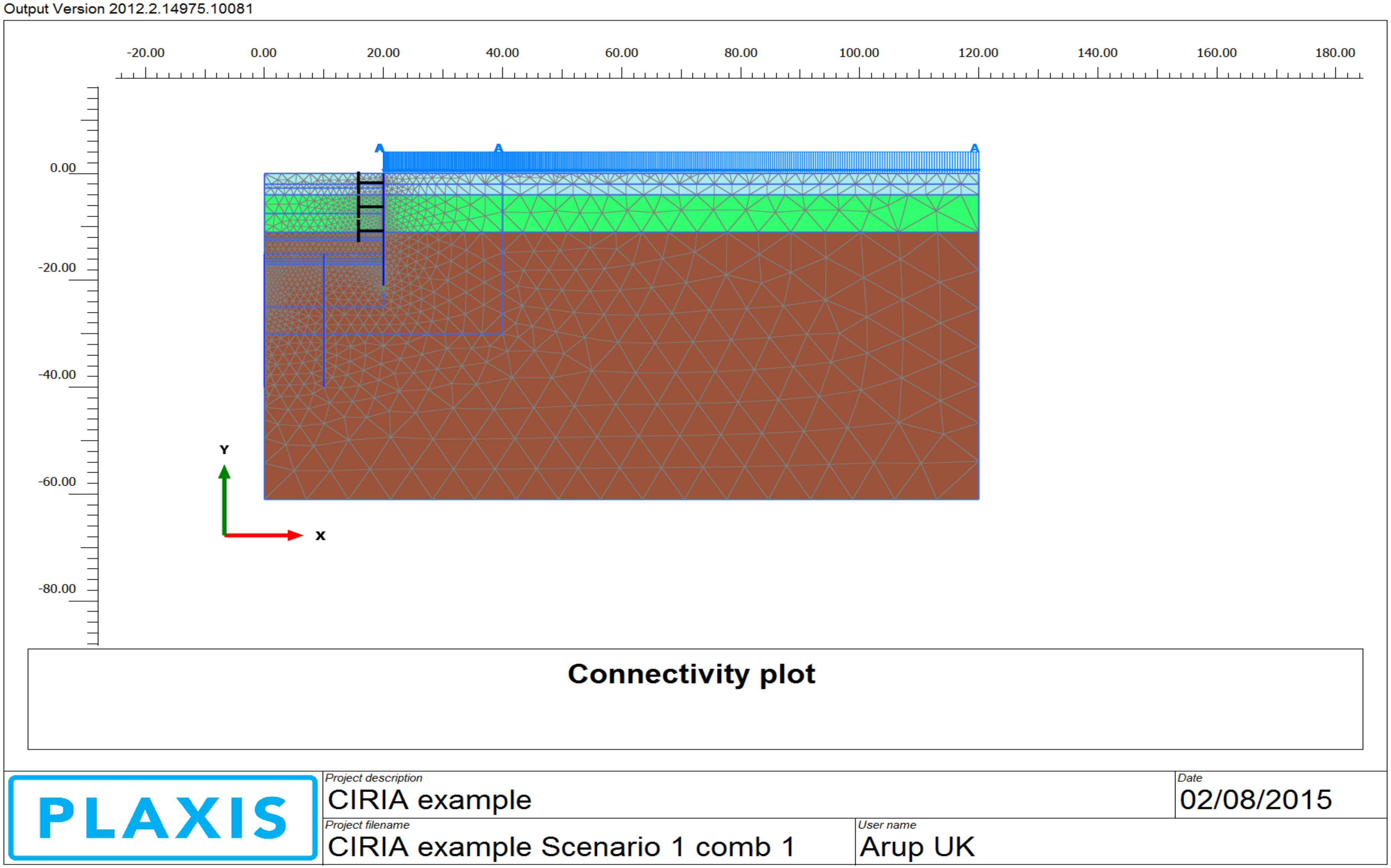
- Carry out whole analysis with the soil strength already reduced by the requisite partial factors

Strategy 2

- Start and continue the analysis to each key stage with the full characteristic soil strength and then reduce soil strength by the required partial factor to investigate the effect on each stage separately

Strategy 2 adopted in design worked example in C760

Worked Example – PLAXIS model



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Connectivity plot

Project description CIRIA example		*Date* 02/08/2015
Project filename CIRIA example Scenario 1 comb 1	*User name* Arup UK	

Worked Example – PLAXIS analysis assumptions

- Linear elastic perfectly plastic soil model. Short term undrained behaviour of clay stratum represented by a Tresca model. Long term behaviour by a Mohr-Coulomb model

- All structural elements modelled as linear elastic

- Excavation disturbance to a depth of 0.5m below formation level

- No softening of the clay assumed during excavation on the restraining side

- Interface friction:

  - Made Ground, gravel and clay stratum (long term only): $\delta = \varphi$
  - Clay stratum (short term only): $c_w = 0.5c_u$

Worked Example – scenario 1: minimal site investigation and no specified requirements for site control

- One cable percussion borehole to base of clay stratum
- SPTs undertaken in all strata
- Classification testing undertaken on disturbed samples recovered from all strata
- No installations in borehole for determination of ground water level
- No on-site supervision of site investigation by the designer
- No special controls specified during construction: requirement for unplanned excavation at each excavation stage is the lesser of 0.5m or 10% of the total height retained for cantilever walls or the height retained below the lowest support level for propped or anchored walls

Worked Example – scenario 2: well planned site investigation and specified tight on-site management to avoid over excavation

- C760 guidance compliant with EC7 requirements followed (i.e. typically, borehole spacings < 20m apart to depths > 3 times retained height for a cantilever wall or > 2 times retained height for a propped wall)

- 4 boreholes (3 of which proved the depth of clay stratum)

- In-situ SPTs undertaken in all boreholes

- Triaxial testing of thin walled clay samples

- Large shear box testing of gravel samples to directly assess angle of friction

- Classification and index testing of all strata

- Installation of standpipe piezometers in the gravel layer in 3 boreholes, including pressure transducers and a data logger to get continuous readings for an adequate period of time

- Full time on-site supervision of site investigation by the designer

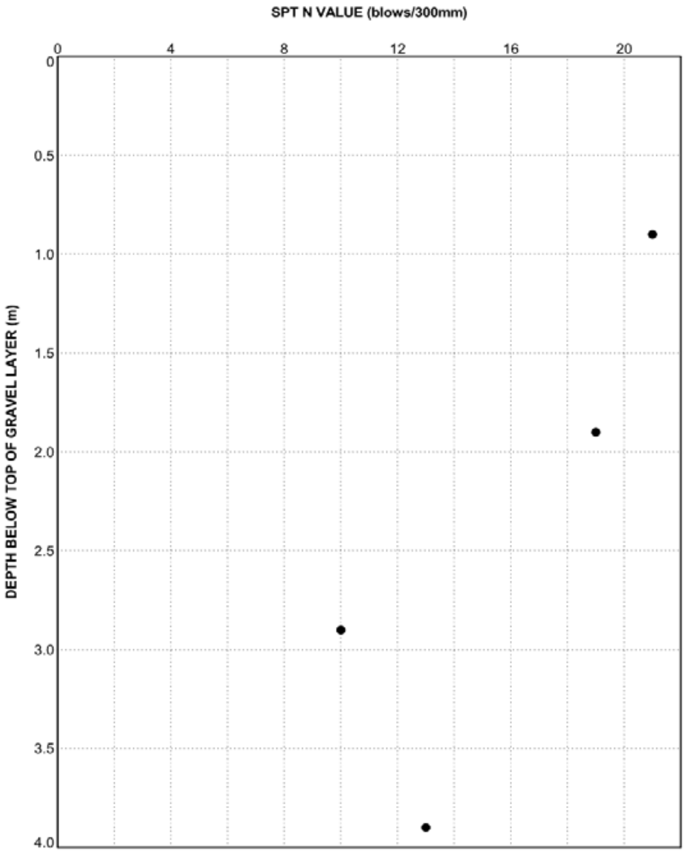
- Special controls specified and tight on-site management in place during construction to avoid unplanned excavation: 100mm practical minimal tolerance for unplanned excavation allowed for at each excavation stage in analysis

Worked Example – scenarios 1 and 2

derivation of characteristic parameters – Made Ground

Stratum	Bulk unit weight γ_b	Young's modulus E'	Angle of shearing resistance $\varphi_{cv}' / \varphi_{pk}'$	Effective cohesion c'	Coefficient of permeability k	Poisson's ratio v'
Made Ground	18	5MPa	25° / 25°	0kPa	1 x 10^{-6} m/s	0.3

Worked Example – scenarios 1 and 2:
derivation of characteristic parameters – gravel deposit



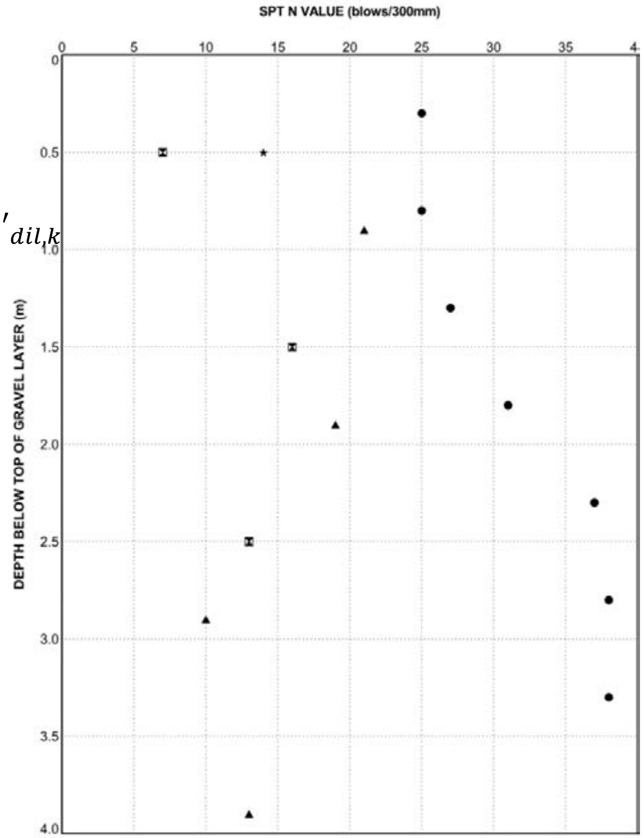
$$\varphi'_{cv,k} = 30 + \varphi'_{ang,k} + \varphi'_{PSD,k}$$

$$\varphi'_{pk,k} = 30 + \varphi'_{ang,k} + \varphi'_{PSD,k} + \varphi'_{dil,k}$$

E’ = 2.0 N_{60} (MPa)

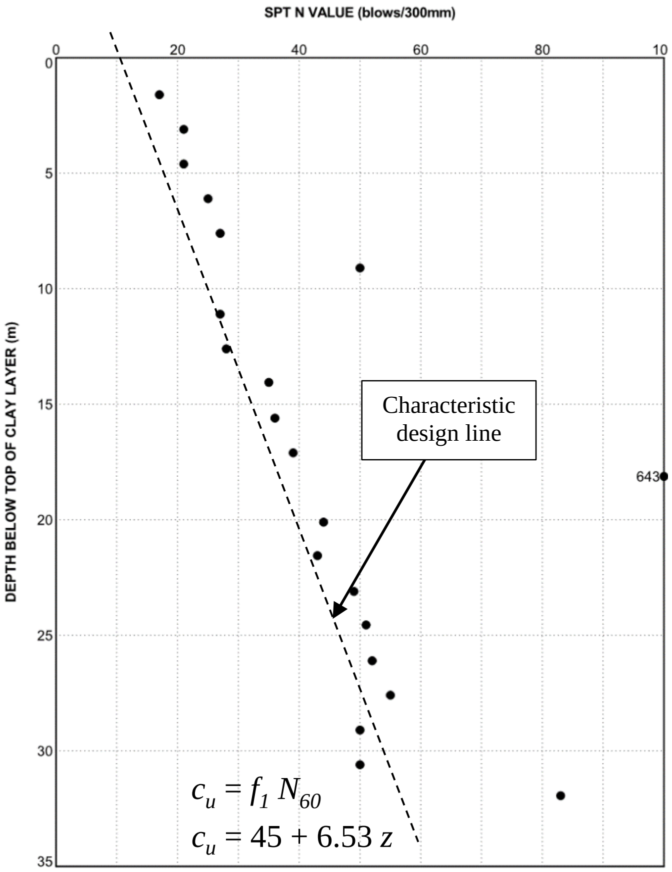
$K_0 = 1 - \sin \varphi$

Shear box test results in scenario 2



Scenario	Bulk unit weight γ_b	Young’s modulus *E’*	Angle of shearing resistance φ_{cv}’ / φ_{pk}’	Effective cohesion *c’*	Coefficient of permeability *k*	Poisson’s ratio *v’*
1	19	20 MPa	34°/36°	0	1 x 10^{-4} m/s	0.25
2	19	40 MPa	36°/39°	0	1 x 10^{-4} m/s	0.25

Worked Example – scenarios 1 and 2: derivation of characteristic parameters – clay stratum



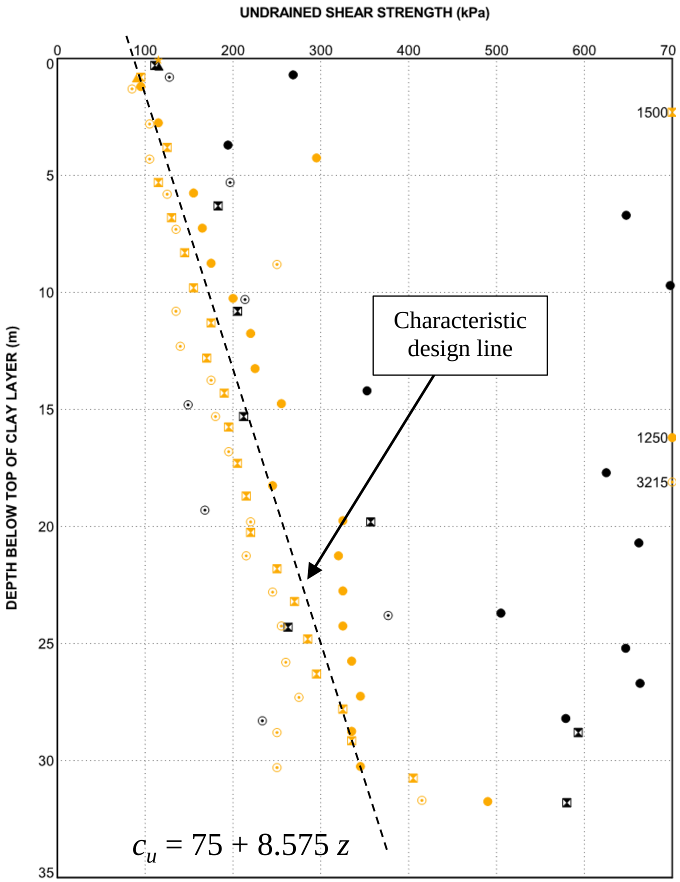
$c_u = f_1 N_{60}$

$c_u = 45 + 6.53\, z$

$E_u = 1000\, c_u$

$E' = 750\, c_u$

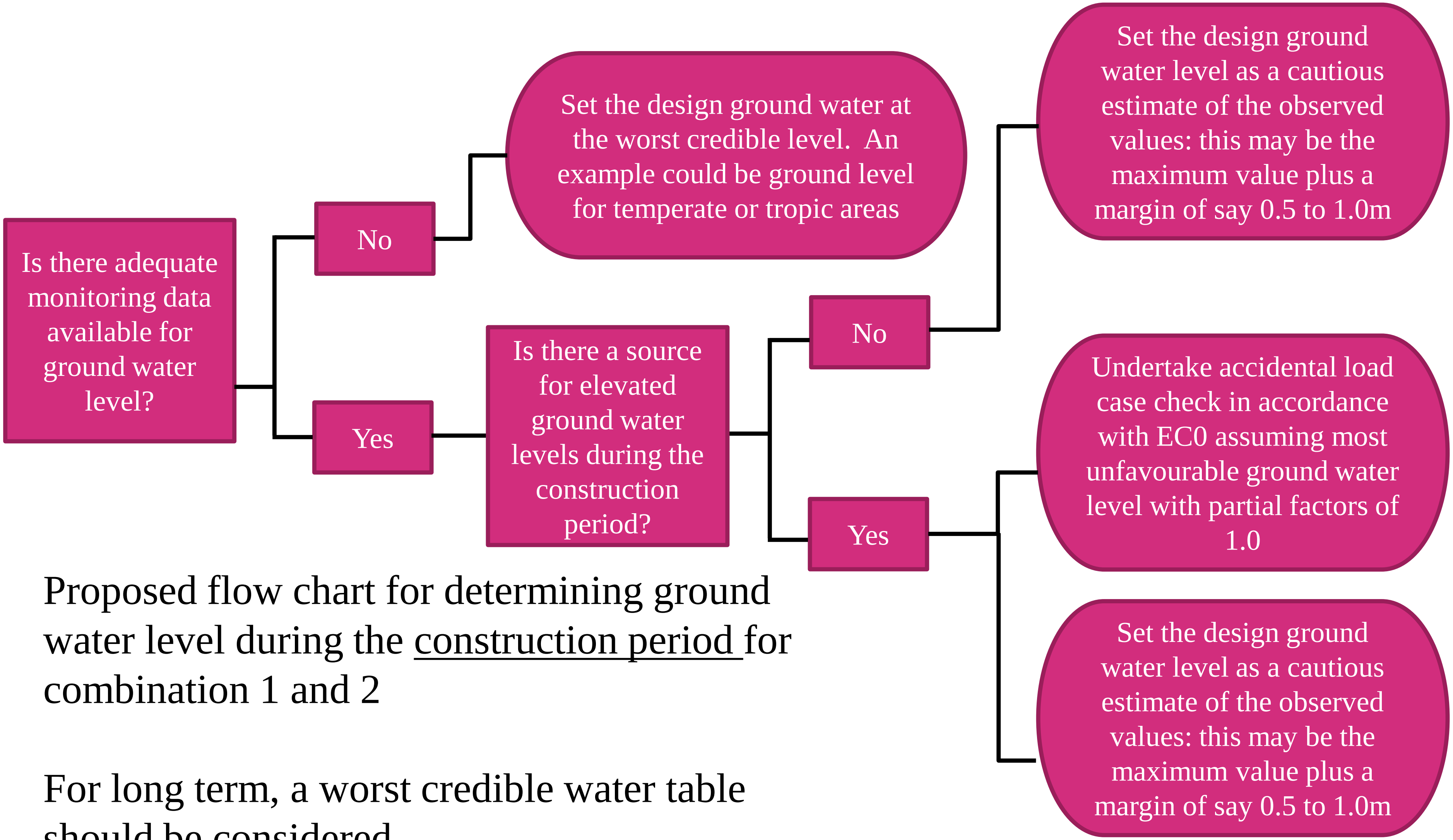
Triaxial test results in scenario 2



$c_u = 75 + 8.575\, z$

Stratum	Bulk unit weight γ_b	Angle of shearing resistance φ_{cv}'	Effective cohesion c'	Coefficient of permeability k	Poisson's ratio v' / v_u
Clay	20	25°	5kPa	1×10^{-9} m/s	0.25 / 0.5

Design considerations – water levels



Is there adequate monitoring data available for ground water level?

No

Set the design ground water at the worst credible level. An example could be ground level for temperate or tropic areas

Yes

Is there a source for elevated ground water levels during the construction period?

No

Set the design ground water level as a cautious estimate of the observed values: this may be the maximum value plus a margin of say 0.5 to 1.0m

Yes

Undertake accidental load case check in accordance with EC0 assuming most unfavourable ground water level with partial factors of 1.0

Set the design ground water level as a cautious estimate of the observed values: this may be the maximum value plus a margin of say 0.5 to 1.0m

Proposed flow chart for determining ground water level during the construction period for combination 1 and 2

For long term, a worst credible water table should be considered

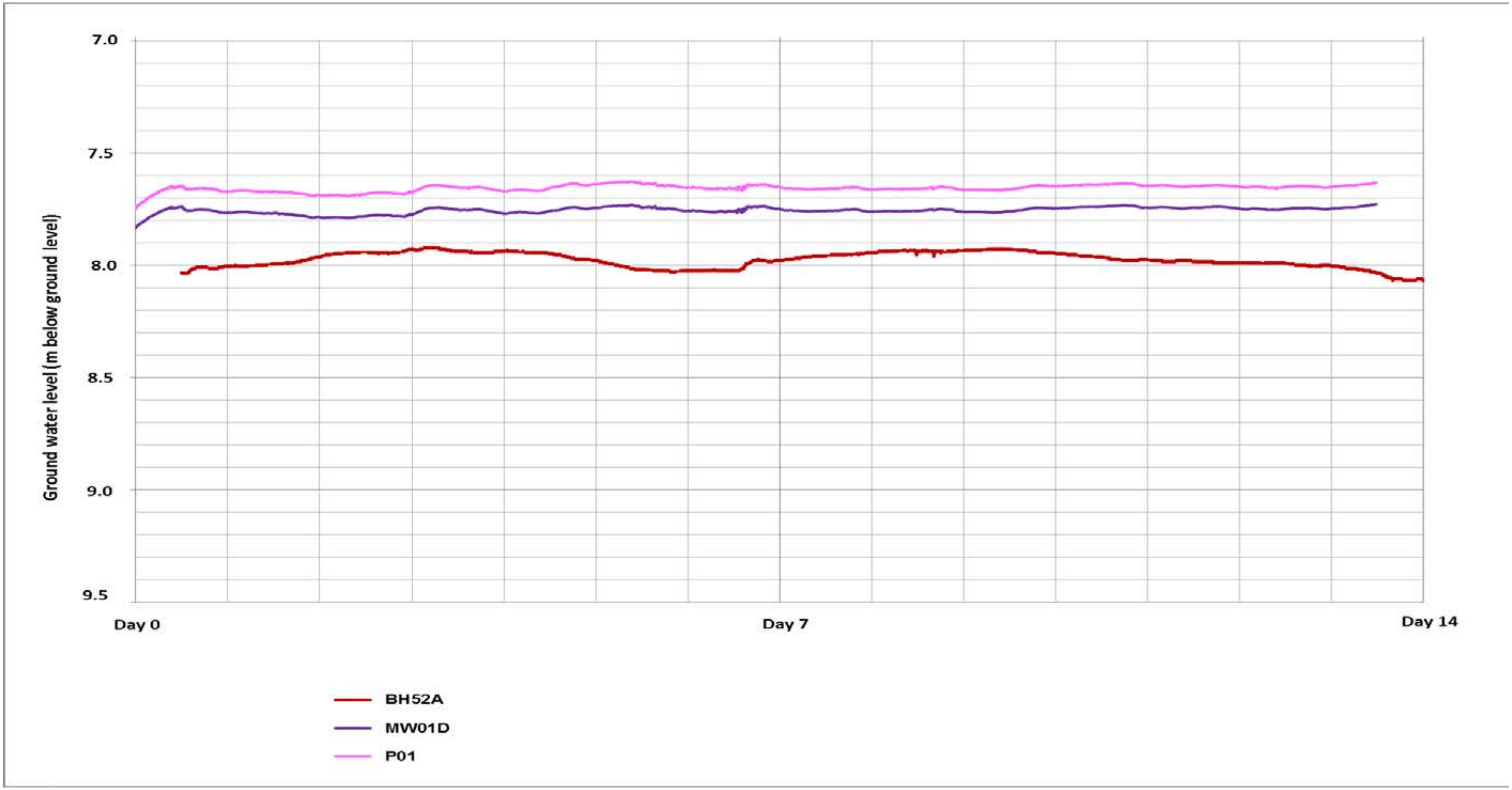
Worked Example – scenario 1: Groundwater

No groundwater monitoring installations: no reliable measurement of groundwater level

Driller’s logs indicate a water strike in the gravel at a depth of 7.5m below ground, rising to 7.1m below ground over 20 minutes

A design level of 2m below ground level judiciously chosen (during construction and in the long term)

Worked Example – scenario 2: Groundwater



Design water level = 7m during construction and 2m in the long term

Worked Example - Partial factors and design parameters

$$X_d = \frac{X_k}{\gamma_M}$$

$c'_d = c_k' / \gamma_c$

$\tan \varphi'_d = \tan \varphi_k' / \gamma_\varphi$

$c_{u,d} = c_{k,u} / \gamma_{cu}$

$q_{u,d} = q_{k,u} / \gamma_{qu}$

Design Approach 1	**Soil and rock parameters**				**Unfavourable actions**	
	γ_φ	γ_c	γ_{cu}	γ_{qu}	Permanent	Variable
Combination 1 DA1C1	1.0	1.0	1.0	1.0	1.35	1.5
Combination 2 DA1C2	1.25	1.25	1.4	1.4	1.0	1.3

DA1C1 design effects of actions $E_d = \gamma_E E_k = 1.35 \text{ x } E_k$

DA1C2 design effects of actions $E_d = \gamma_E E_k = 1.00 \text{ x } E_k$

Worked Example – summary of design parameters

Parameter		Scenario 1		Scenario 2	
		DA1 Combination1	DA1 Combination 2	DA1 Combination1	DA1 Combination 2
Made Ground	γ_b	18 kN/m^3		18 kN/m^3	
	φ'	25°	20.5°	25°	20.5°
	c’	0kPa	0kPa	0kPa	0kPa
	E’	5MPa	5MPa	5MPa	5MPa
	v’	0.3		0.3	
	k	1 x 10^{-6} m/s		1 x 10^{-6} m/s	
	δ	25°	20.5°	25°	20.5°
	K_0	0.58	0.65	0.58	0.65
Gravel	γ_b	19 kN/m^3		19 kN/m^3	
	φ'	36°	30.2°	39°	32.9°
	c’	0kPa	0kPa	0kPa	0kPa
	E’	20MPa	20MPa	40MPa	40MPa
	v’	0.25		0.25	
	k	1 x 10^{-4} m/s		1 x 10^{-4} m/s	
	δ	36°	30.2°	39°	32.9°
	K_0	0.41	0.50	0.37	0.46
Clay	γ_b	20 kN/m^3		20 kN/m^3	
	c_u	45 + 6.53z kPa	32 + 4.66z kPa	75 + 8.575z kPa	53.6 + 6.13z kPa
	E_u	45 + 6.53z MPa	45 + 6.53z MPa	75 + 8.575z MPa	75 + 8.575z MPa
	v_u	0.5	0.5	0.5	0.5
	c_w	22.5 + 3.27z kPa	16 + 2.34z kPa	37.5 + 4.29z kPa	26.8 + 3.06z kPa
	φ'	25°	20.5°	25°	20.5°
	c’	5kPa	4kPa	5kPa	4kPa
	E’	33.75 + 4.9z MPa	33.75 + 4.9z MPa	56.25 + 6.43zMPa	56.25 + 6.43zMPa
	v’	0.25		0.25	
	k	1 x 10^{-9} m/s		1 x 10^{-9} m/s	
	δ	25°	20.5°	25°	20.5°
	K_0	1.0	1.0	1.0	1.0

Worked Example – design actions

10kPa ground surface surcharge on retained side of retaining wall

Design actions (surcharge values) of:
- 11kPa for DA1C1 calculation
- 13kPa for DA1C2 calculation
- 10kPa for SLS calculation

Partial factors applied to unfavourable actions (surcharges)

Combination	Variable	Permanent
DA1C1	1.11*	1.0*
DA1C2	1.3	1.0

*** Note these values are modified to take account of the 1.35 partial factor that is applied to the effect of actions for DA1C1 as discussed in Chapter 7 section 7.3**

The wall is also subject to 150kN/m of permanent load and 50kN/m of variable load applied vertically at the top of the wall throughout the construction sequence

Design actions (vertical loads) applied to the wall at its top:
- 205kN/m for DA1C1 calculation
- 215kN/m for DA1C2 calculation
- 200kN/m for SLS calculation

Worked Example

Stages in PLAXIS DA1C2 analysis

Strategy 2 adopted

Stage	Description	Behaviour of clay layer	Comment
1	Initialise stress with characteristic soil parameters	Drained	
2	Install wall and tension piles	Undrained	
3	Apply 13kPa surcharge behind wall and 215kN/m line load to top of wall	Undrained	
4	Excavate to 2.5m depth plus allowance for over excavation	Undrained	Unplanned excavation: 0.25m for Scenario 1 (2.75m) 0.1m for Scenario 2 (2.6m)
4a	Apply partial factors to soil parameters	Undrained	
5	Install P1 slab at depth of 1.75m and excavate to 7.0m plus allowance for over excavation	Undrained	P1 slab modelled as fixed end anchor Unplanned excavation: 0.5m for Scenario 1 (7.5m) 0.1m for Scenario 2 (7.1m)
5a	Apply partial factors to soil parameters	Undrained	
6	Install P2 slab at depth of 6.25m and excavate to 11.5m plus allowance for over excavation	Undrained	P1 slab modelled as fixed end anchor Unplanned excavation: 0.5m for Scenario 1 (12.0m) 0.1m for Scenario 2 (11.6m) Apply softening in clay to 0.5m depth
6a	Apply partial factors to soil parameters	Undrained	
7	Install P3 slab at depth of 10.75m and excavate to 16.0m plus allowance for over excavation	Undrained	P1 slab modelled as fixed end anchor Unplanned excavation: 0.5m for Scenario 1 (16.5m) 0.1m for Scenario 2 (16.1m) Apply softening in clay to 0.5m depth
7a	Apply partial factors to soil parameters	Undrained	
8	Replace over excavated material and construct base slab	Undrained	Unplanned excavated material replaced with clay. Base slab modelled with solid elements with properties of concrete (linear elastic)
9	Switch clay parameters to drained	Drained	
10	Consolidate for 100 years to dissipate excess pore pressures in clay layer and reach steady state seepage under the wall	Drained	Fix pore water pressure on boundary of mesh to represent source. Set zero flow boundary on base of mesh and along the line of symmetry.
10a	Apply partial factors to soil parameters	Drained	
11	Switch clay parameters to undrained	Undrained	
12	Check for water pressure 2m below ground level	Undrained	Raise ground water level to check for future rise in ground water level
12a	Apply partial factors to soil parameters	Undrained	

Highlighted stages are ultimate limit state checks at key stages

Worked Example

Stages in PLAXIS DA1C1 analysis

Resulting effects of actions (wall bending moments, shear forces and prop loads) are multiplied by 1.35 to obtain design effects of actions

Stage	Description	Behaviour of clay layer	Comment
1	Initialise stress with characteristic soil parameters	Drained	
2	Install wall and tension piles	Undrained	
3	Apply 11kPa surcharge behind wall and 205kN/m line load to top of wall	Undrained	
4	Excavate to 2.5m depth plus allowance for over excavation	Undrained	Unplanned excavation: 0.25m for Scenario 1 (2.75m) 0.1m for Scenario 2 (2.6m)
5	Install P1 slab at depth of 1.75m and excavate to 7.0m plus allowance for over excavation	Undrained	P1 slab modelled as fixed end anchor Unplanned excavation: 0.5m for Scenario 1 (7.5m) 0.1m for Scenario 2 (7.1m)
6	Install P2 slab at depth of 6.25m and excavate to 11.5m plus allowance for over excavation	Undrained	P1 slab modelled as fixed end anchor Unplanned : 0.5m for Scenario 1 (12.0m) 0.1m for Scenario 2 (11.6m) Apply softening in clay to 0.5m depth
7	Install P3 slab at depth of 10.75m and excavate to 16.0m plus allowance for over excavation	Undrained	P1 slab modelled as fixed end anchor Unplanned excavation: 0.5m for Scenario 1 (16.5m) 0.1m for Scenario 2 (16.1m) Apply softening in clay to 0.5m depth
8	Replace over excavated material and construct base slab	Undrained	Unplanned excavated material replaced with clay. Base slab modelled with solid elements with properties of concrete (linear elastic)
9	Switch clay parameters to drained	Drained	
10	Consolidate for 100 years to dissipate excess pore pressures in clay layer and reach steady state seepage under the wall	Drained	Fix pore water pressure on boundary of mesh to represent source. Set zero flow boundary on base of mesh and along the line of symmetry.
11	Switch clay parameters to undrained	Undrained	
12	Check for water pressure 2m below ground level	Undrained	Raise ground water level to check for future rise in ground water level

Worked Example

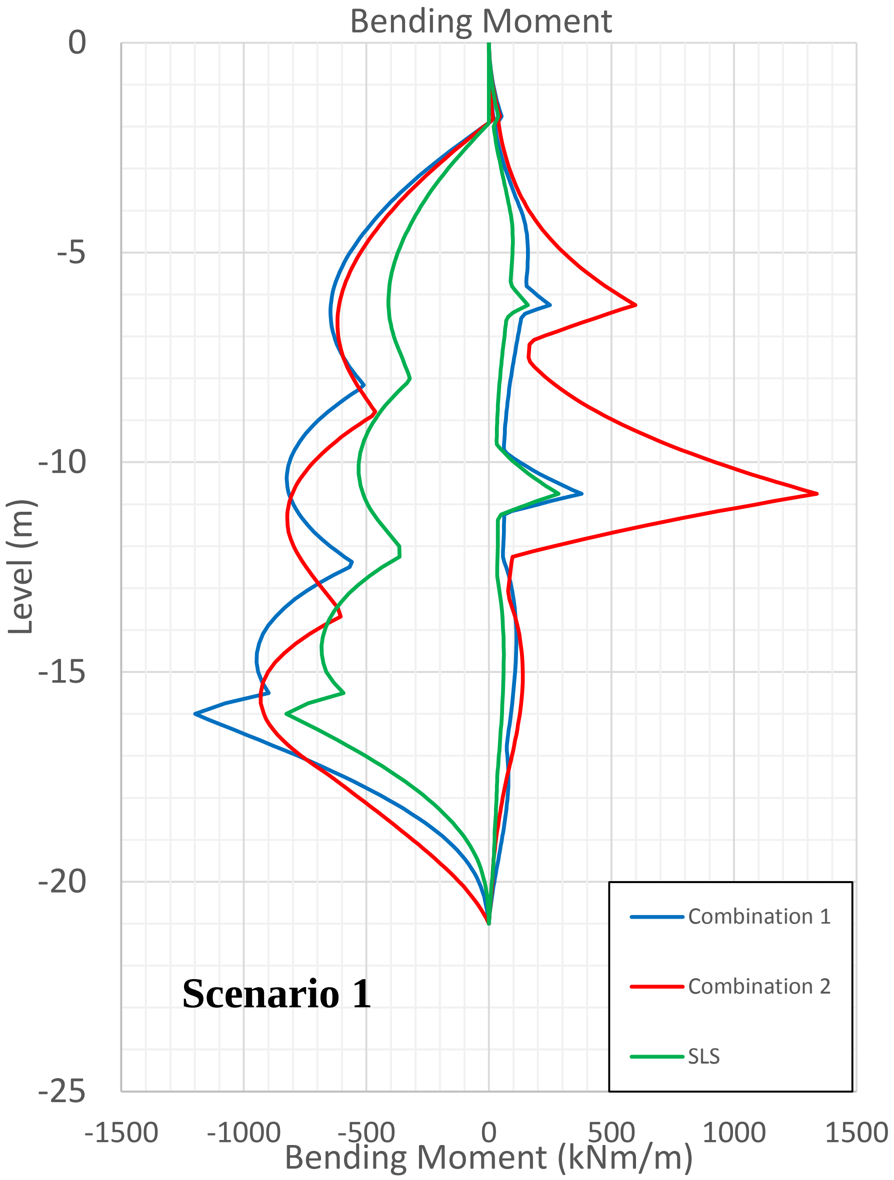
Stages in PLAXIS SLS analysis

Stage	Description	Behaviour of clay layer	Comment
1	Initialise stress with characteristic soil parameters	Drained	
2	Install wall and tension piles	Undrained	
3	Apply 10kPa surcharge behind wall and 200kN/m line load to top of wall	Undrained	
4	Excavate to 2.5m depth	Undrained	No unplanned excavation:
5	Install P1 slab at depth of 1.75m and excavate to 7.0m	Undrained	P1 slab modelled as fixed end anchor No unplanned excavation
6	Install P2 slab at depth of 6.25m and excavate to 11.5m	Undrained	P1 slab modelled as fixed end anchor No unplanned excavation
7	Install P3 slab at depth of 10.75m and excavate to 16.0m	Undrained	P1 slab modelled as fixed end anchor No unplanned excavation or allowance for softening
8	Replace over excavated material and construct base slab	Undrained	Base slab modelled with solid elements with properties of concrete (linear elastic)
9	Switch clay parameters to drained	Drained	
10	Consolidate for 100 years to dissipate excess pore pressures in clay layer and reach steady state seepage under the wall	Drained	Fix pore water pressure on boundary of mesh to represent source. Set zero flow boundary on base of mesh and along the line of symmetry.
11	Switch clay parameters to undrained	Undrained	

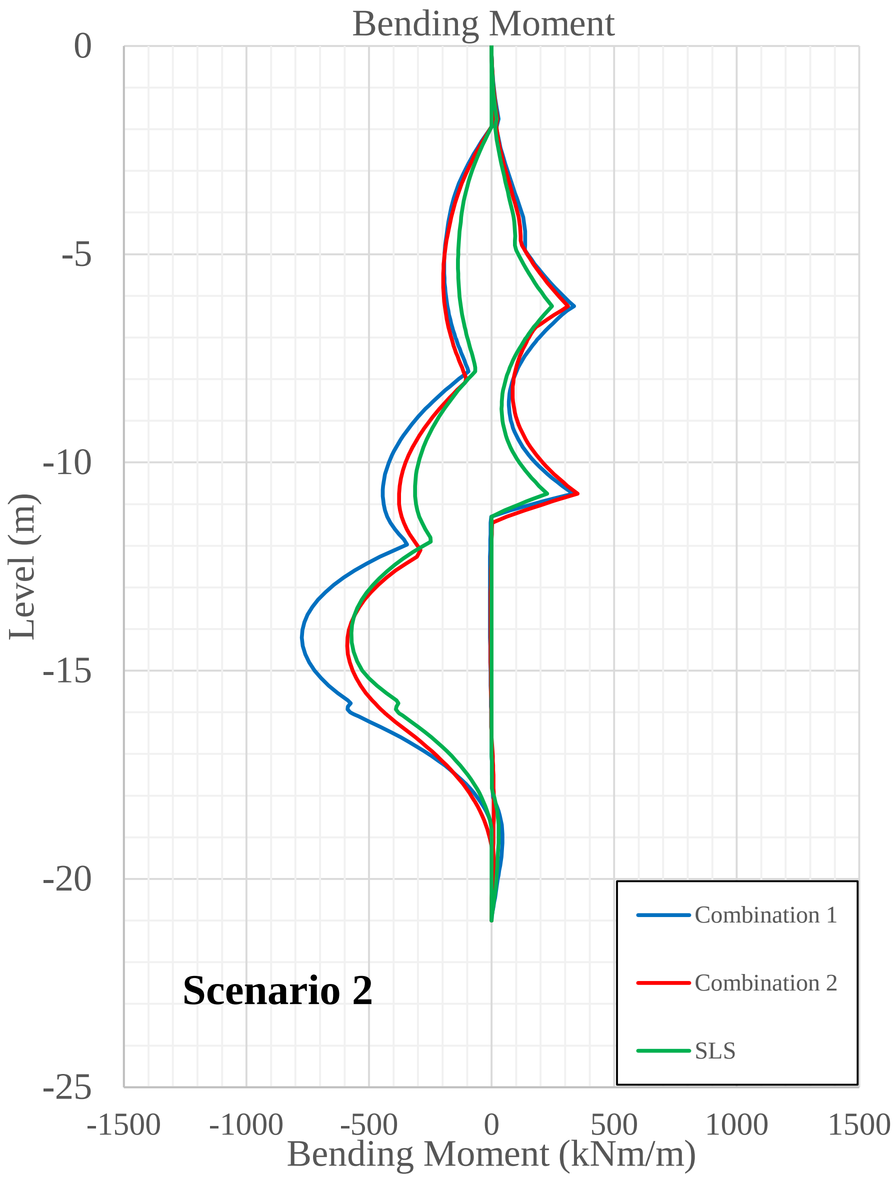
Worked Example – design effects of actions (prop/slab forces)

Prop	**DA1C1**		**DA1C2**		**SLS**	
	Scenario 1 kN/m	**Scenario 2** kN/m	**Scenario 1** kN/m	**Scenario 2** kN/m	**Scenario 1** kN/m	**Scenario 2** kN/m
P1	295	160	263	136	202	115
P2	737	513	832	441	506	376
P3	1150	950	1616	794	831	672

Worked Example – design effects of actions (wall bending moments)

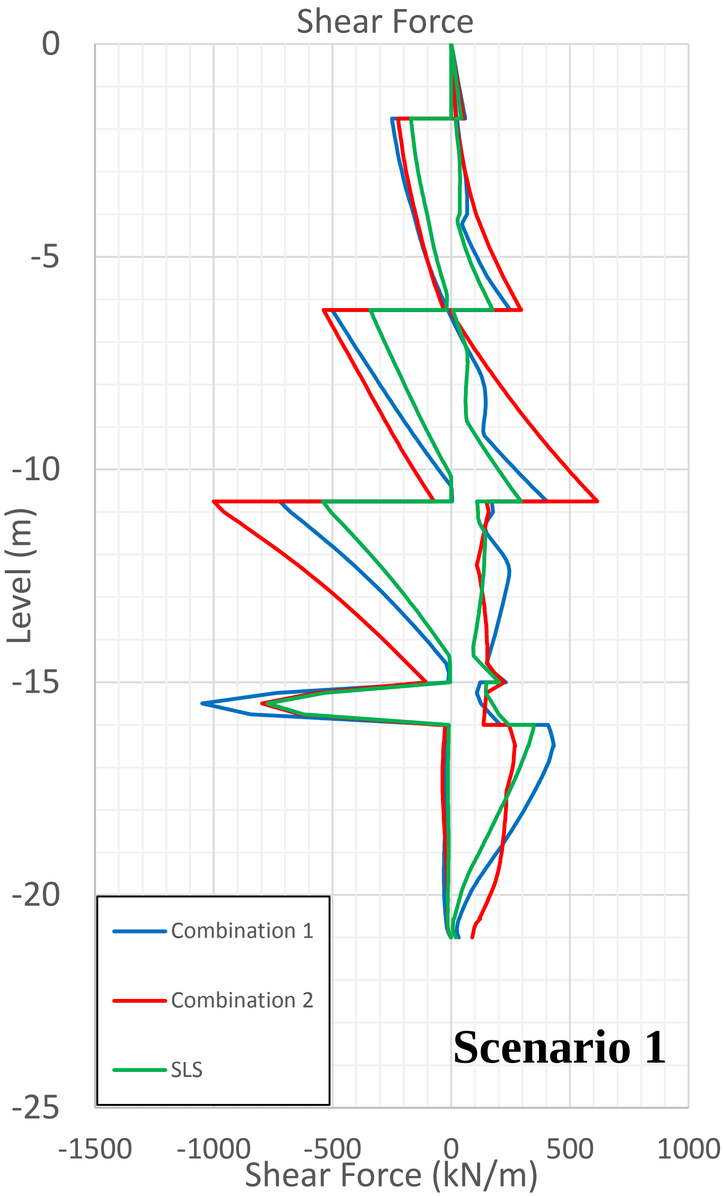


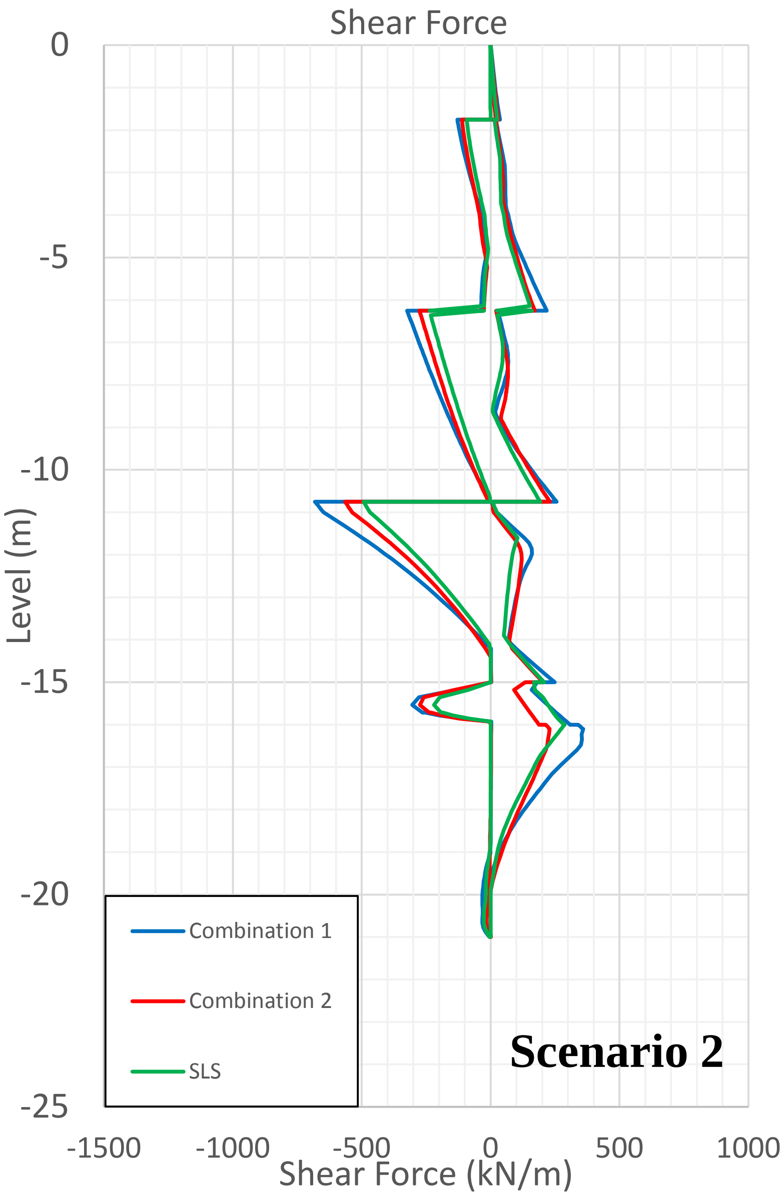
Scenario 1



Scenario 2

Worked Example – design effects of actions (wall shear forces)





Worked Example – comparison of Scenario 1 and 2 analyses

Scenario	Maximum wall bending moment (kNm/m)	Maximum wall shear force (kN/m)	Maximum prop/slab force (kN/m)	Maximum computed SLS wall deflection (mm)	Maximum computed SLS ground surface settlement behind wall (mm)
1	-1200 / 1340	1050	1616	33	10
2	-770 / 350	673	950	20	5
% reduction with Scenario 2	**36% / 74%**	**36%**	**41%**	**39%**	**50%**

Significant benefits in undertaking an appropriate well considered site investigation together with good on-site management and control during construction: huge potential savings

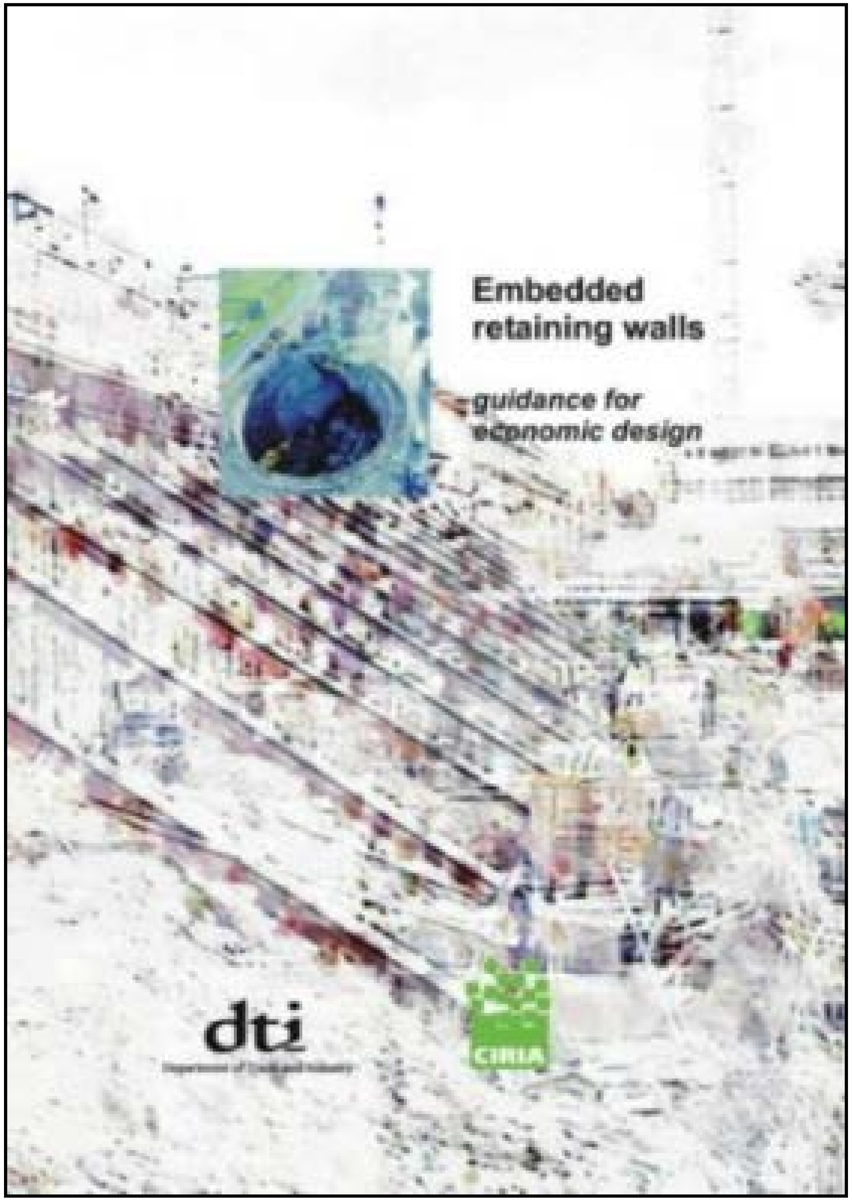
Why then are site investigations typically so poor?

C760

Embedded Retaining walls:

guidance for design

ANALYSIS



Embedded retaining walls

guidance for economic design

dti

CIRIA

OUTLINE

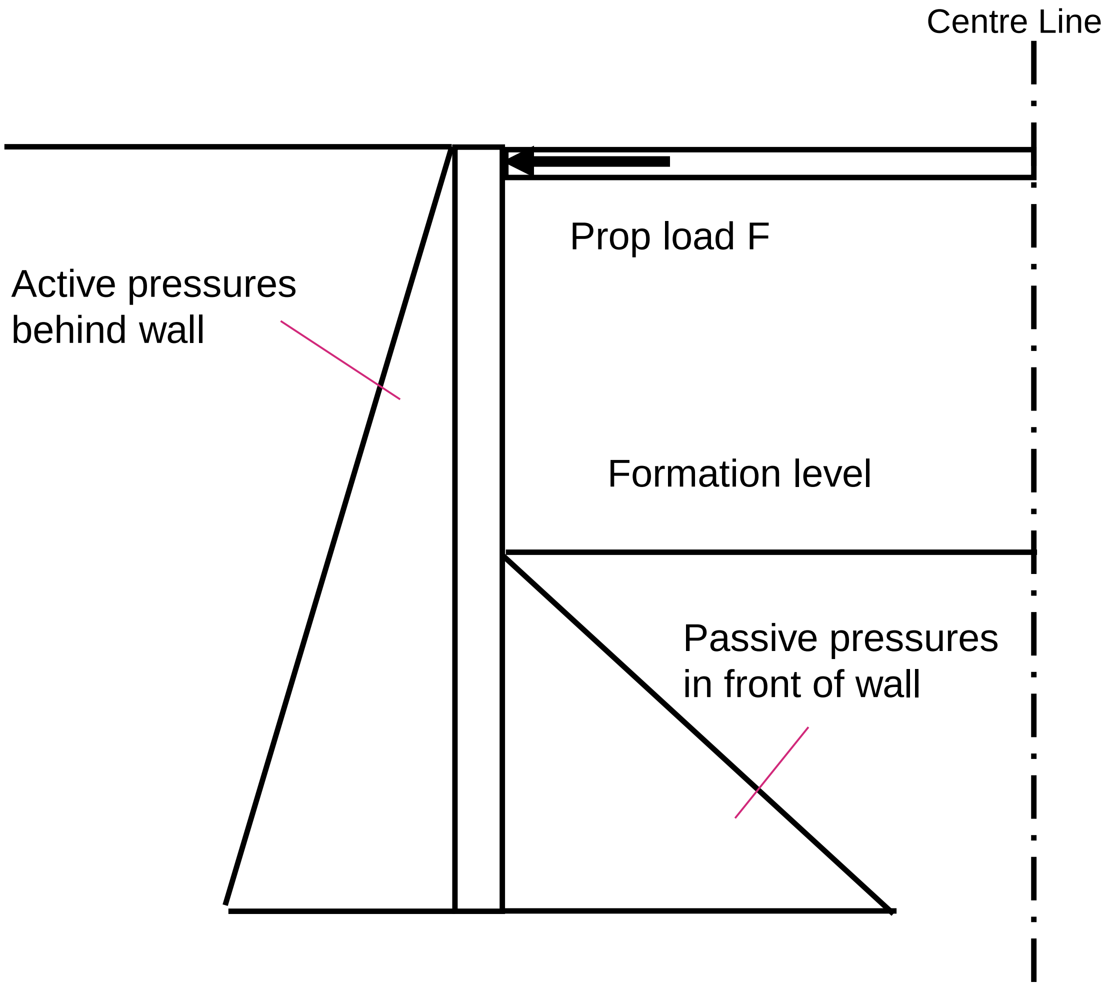
- LIMIT EQUILIBRIUM ANALYSIS
    - Active and passive pressures
    - Pore water pressures (steady state seepage)
    - Unpropped walls
    - Walls propped near the crest
    - Estimating displacements
- MODELLING MORE COMPLEX SITUATIONS
    - Multiple props
    - Wall flexibility
    - Soil / structure interaction analyses
    - Earth berms
    - Modelling installation effects for in situ walls
- WORKED EXAMPLES

LIMIT EQUILIBRIUM ANALYSIS

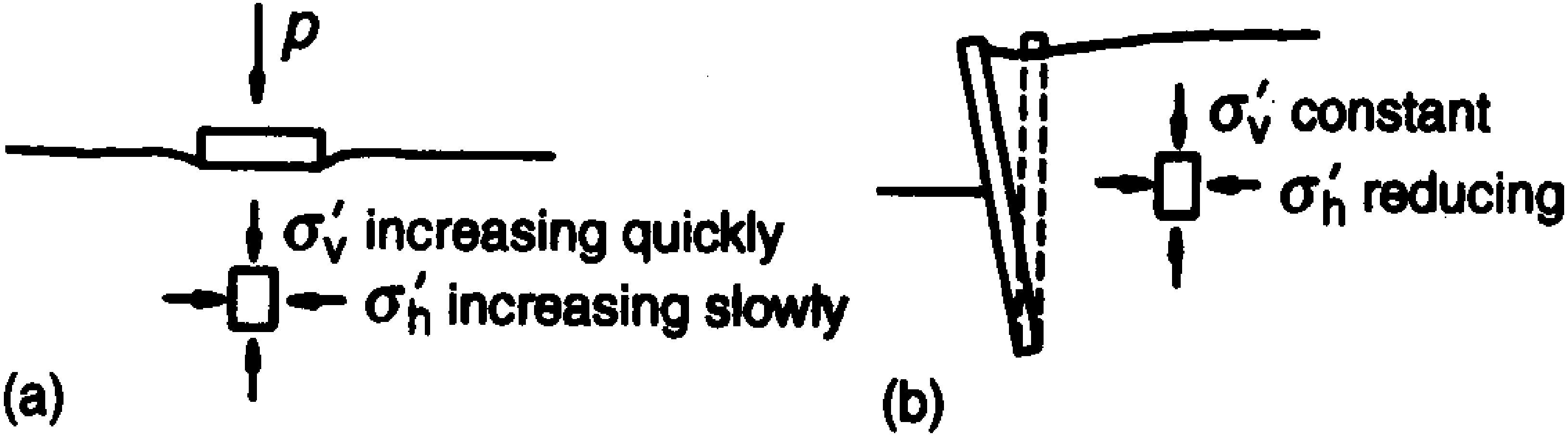
- Active and passive pressures
- Pore water pressures (steady state seepage)
- Unpropped walls
- Walls propped near the crest
- Estimating displacements

Embedded retaining walls: principles

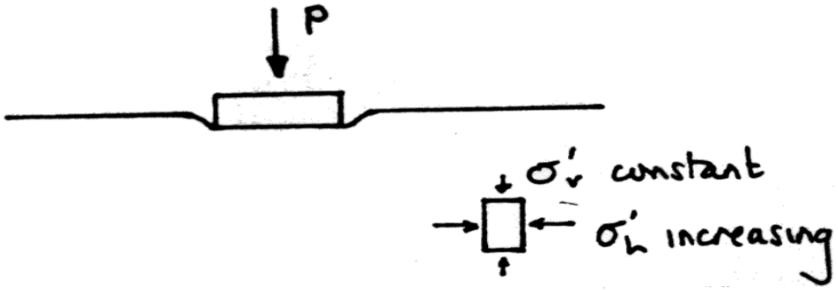
Lateral stresses from the retained ground are resisted by lateral stresses in front of the wall plus any prop or anchor loads



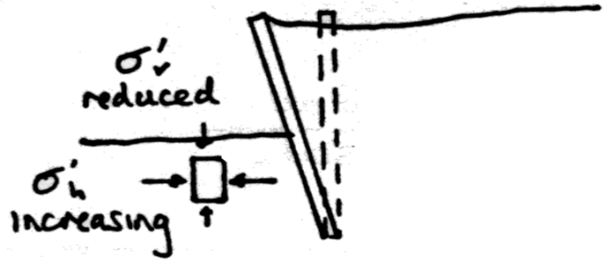
Active failure: major principal stress vertical



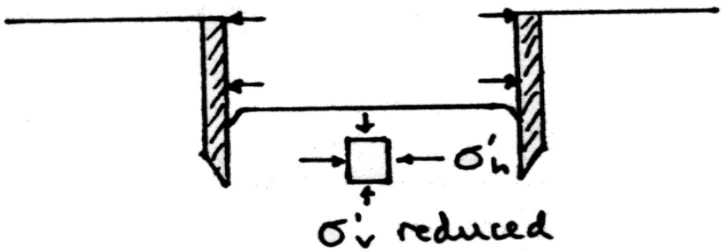
Passive failure: major principal stress horizontal



(a) to either side of a foundation



(b) in front of a retaining wall



(c) below the floor of an excavation

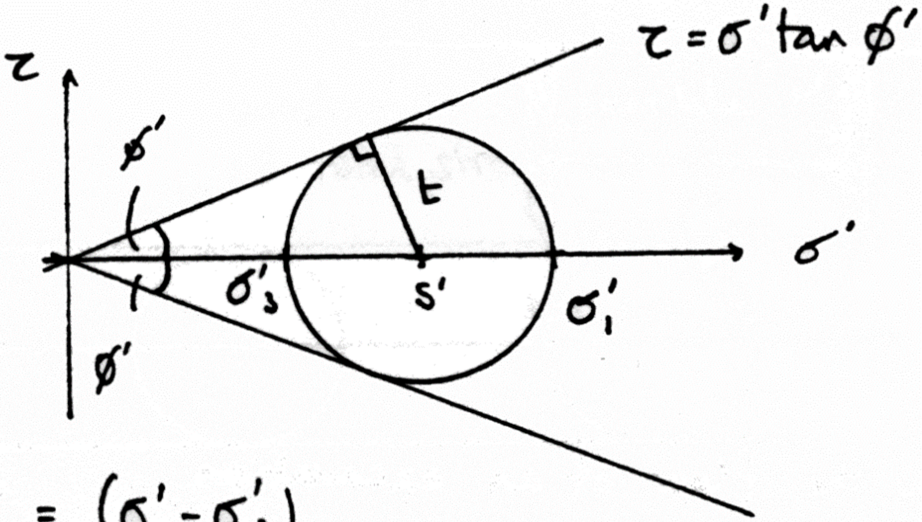
Active and passive pressures: effective stress

- Active: $\sigma'_h = (1-\sin\phi')/(1+\sin\phi').\sigma'_v$
- Passive: $\sigma'_h = (1+\sin\phi')/(1-\sin\phi').\sigma'_v$
- K_a, K_p (= σ'_h/σ'_v) are active and passive earth pressure coefficients

$s' = \frac{1}{2}(\sigma'_1 + \sigma'_3)$

$t = \frac{1}{2}(\sigma'_1 - \sigma'_3)$

$\sin\phi' = t/s'$



$\therefore (\sigma'_1 + \sigma'_3)\sin\phi' = (\sigma'_1 - \sigma'_3)$

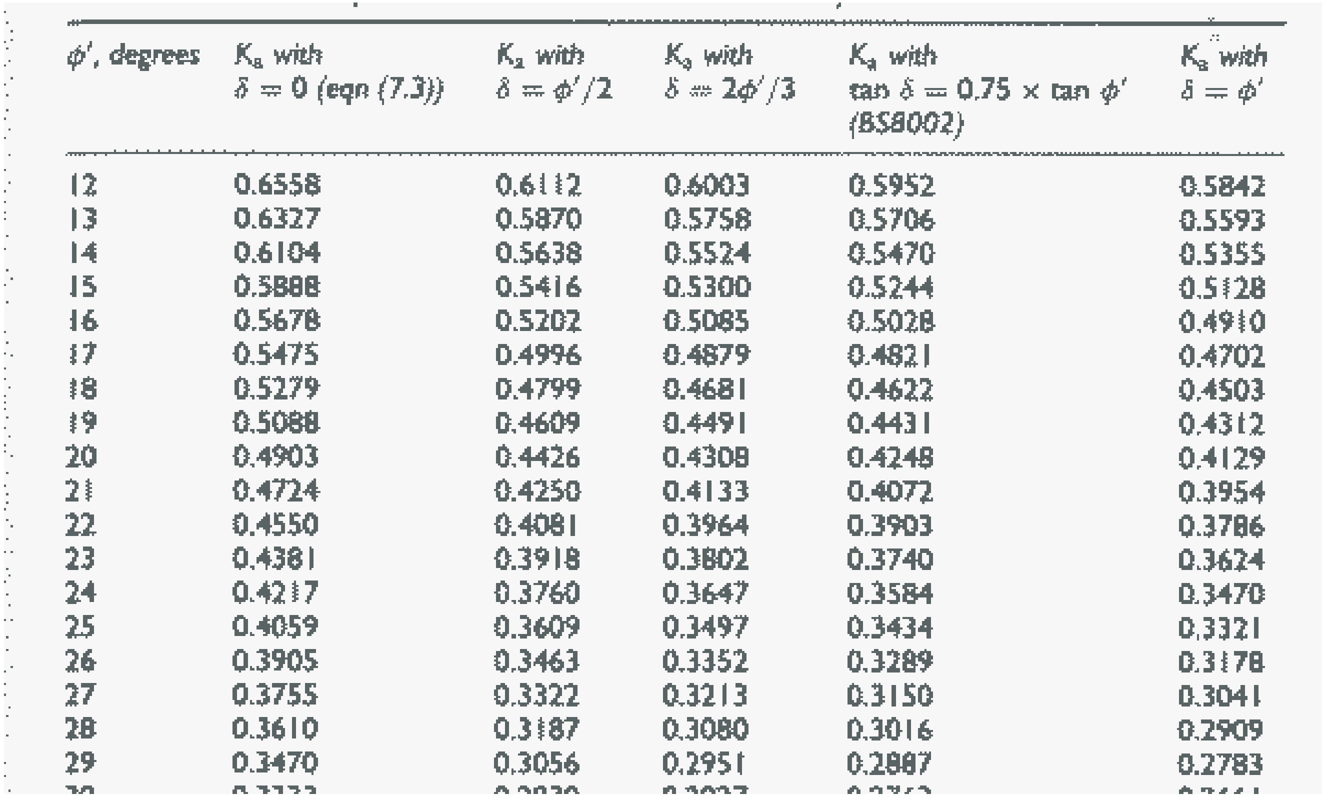
or $\dfrac{\sigma'_1}{\sigma'_3} = \dfrac{1+\sin\phi'}{1-\sin\phi'}$

Active and passive earth pressure coefficients: effects of wall friction

- Wall friction (angle δ) generally increases passive pressures and reduces active pressures, assuming soil behind wall moves down and in soil front of wall moves up relative to the wall
- Opposite directions of relative soil/wall movement have the opposite effect
- Look up earth pressure coefficients in tables for given value of wall friction angle δ/ϕ'
- Can also account for sloping backfill and wall batter (see SMCA 10.7 and 10.8 [3rd Edition])
- Many tables give horizontal component K_a or $K_p = \sigma'_h/(\gamma z - u)$

Tabulated active and passive earth pressure coefficients

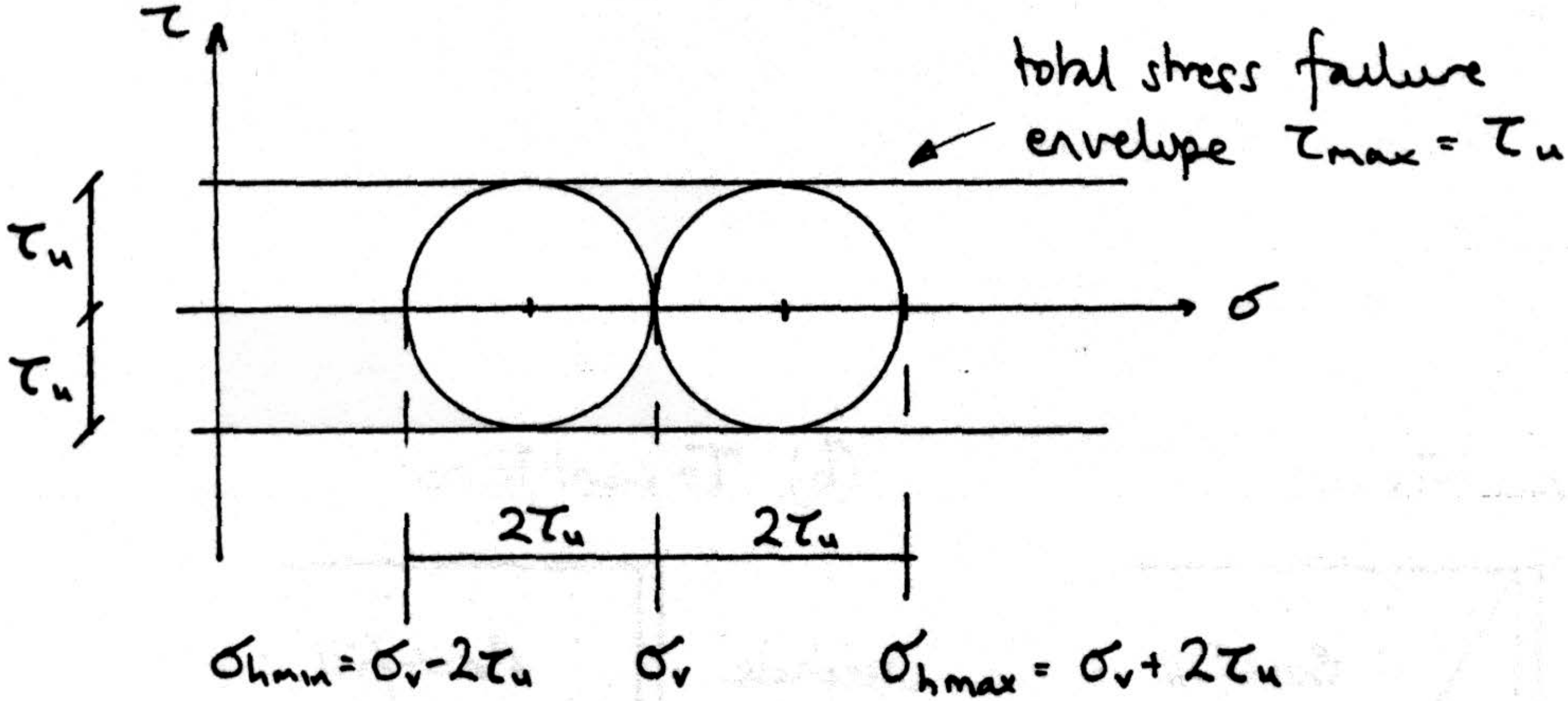
- See eg SMCA pages 423 and 424 [2nd Edition] or pages 476 and 477 [3rd Edition] for full tables (these are derived using a lower bound approach given by Sokolovskii, 1960)



ϕ', degrees	K_a with $\delta = 0$ (eqn (7.3))	K_a with $\delta = \phi'/2$	K_a with $\delta = 2\phi'/3$	K_a with $\tan\delta = 0.75 \times \tan\phi'$ (BS8002)	K_a with $\delta = \phi'$
12	0.6558	0.6112	0.6003	0.5952	0.5842
13	0.6327	0.5870	0.5758	0.5706	0.5593
14	0.6104	0.5638	0.5524	0.5470	0.5355
15	0.5888	0.5416	0.5300	0.5244	0.5128
16	0.5678	0.5202	0.5085	0.5028	0.4910
17	0.5475	0.4996	0.4879	0.4821	0.4702
18	0.5279	0.4799	0.4681	0.4622	0.4503
19	0.5088	0.4609	0.4491	0.4431	0.4312
20	0.4903	0.4426	0.4308	0.4248	0.4129
21	0.4724	0.4250	0.4133	0.4072	0.3954
22	0.4550	0.4081	0.3964	0.3903	0.3786
23	0.4381	0.3918	0.3802	0.3740	0.3624
24	0.4217	0.3760	0.3647	0.3584	0.3470
25	0.4059	0.3609	0.3497	0.3434	0.3321
26	0.3905	0.3463	0.3352	0.3289	0.3178
27	0.3755	0.3322	0.3213	0.3150	0.3041
28	0.3610	0.3187	0.3080	0.3016	0.2909
29	0.3470	0.3056	0.2951	0.2887	0.2783

Active and passive pressures: total stress

- Active: $\sigma_h = \sigma_v - 2.\tau_u$
- applies only to clay soils in the short term
- Passive: $\sigma_h = \sigma_v + 2.\tau_u$
- τ_u is the undrained shear strength



Active and passive total stresses: effects of wall adhesion

- Active:

  $\sigma_h = (\gamma z + q) - [2\sqrt{(1+\tau_w/\tau_u)}].\tau_u$

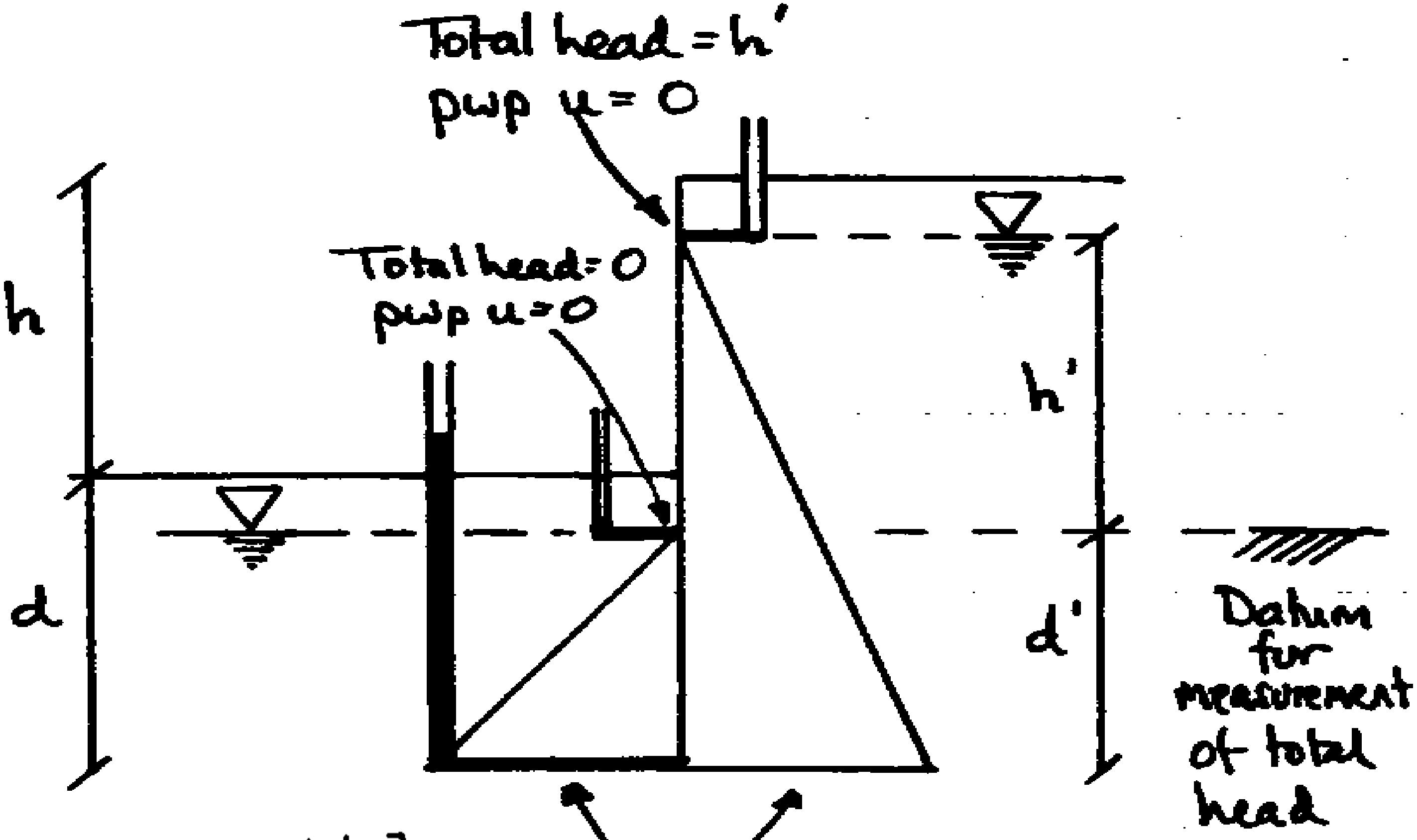
- Passive:

  $\sigma_h = (\gamma z + q) + [2\sqrt{(1+\tau_w/\tau_u)}].\tau_u$

This expression is an approximation: its numerical value should not exceed 2.57 (= 1 + $\pi/2$)

- q is surface surcharge
- τ_u (c_u, s_u) is undrained shear strength
- τ_w (c_w s_w) is wall adhesion

Pore water pressures: ‘linear seepage’ approximation for long term pore water pressures

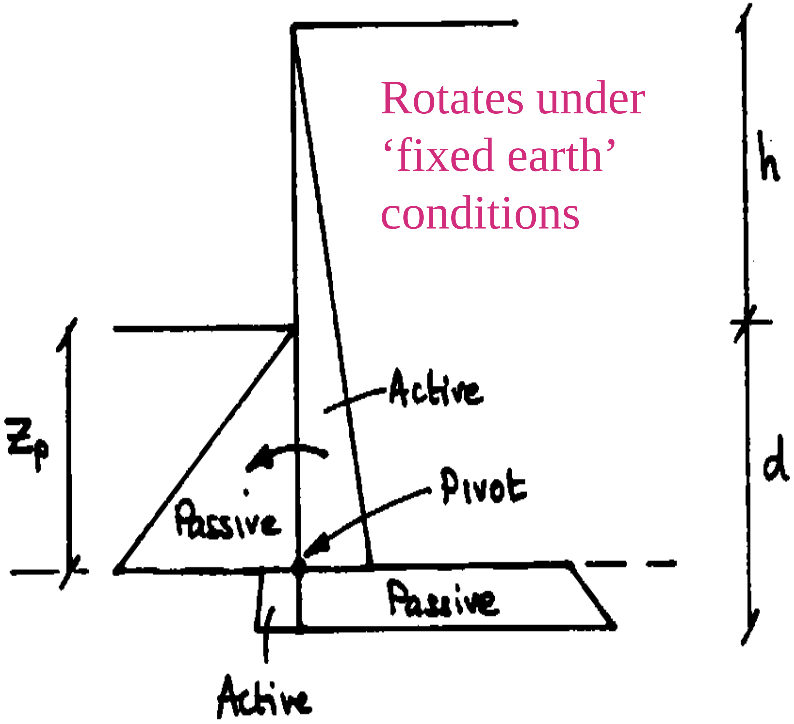


pwp $u = \gamma_w \left[d' + \frac{h'd'}{2d'+h'} \right]$ total head $= h' \times \frac{d'}{2d'+h'}$

Pore water pressures: ‘linear seepage’ model

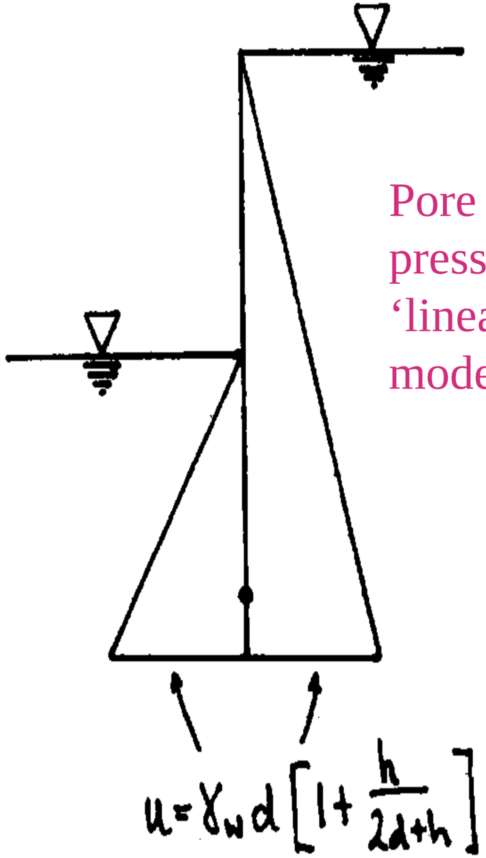
- Only applies in a uniform, isotropic soil
- Does NOT apply when there are several layers of soil of different permeability
- Does NOT apply if the wall is permeable (e.g. unsealed contiguous piles)
- Does NOT apply if soil is anisotropic
- If in doubt, draw a flownet (or use a seepage program)

Unpropped embedded walls (fixed earth support)



Rotates under ‘fixed earth’ conditions

Lateral effective stresses (earth pressures)

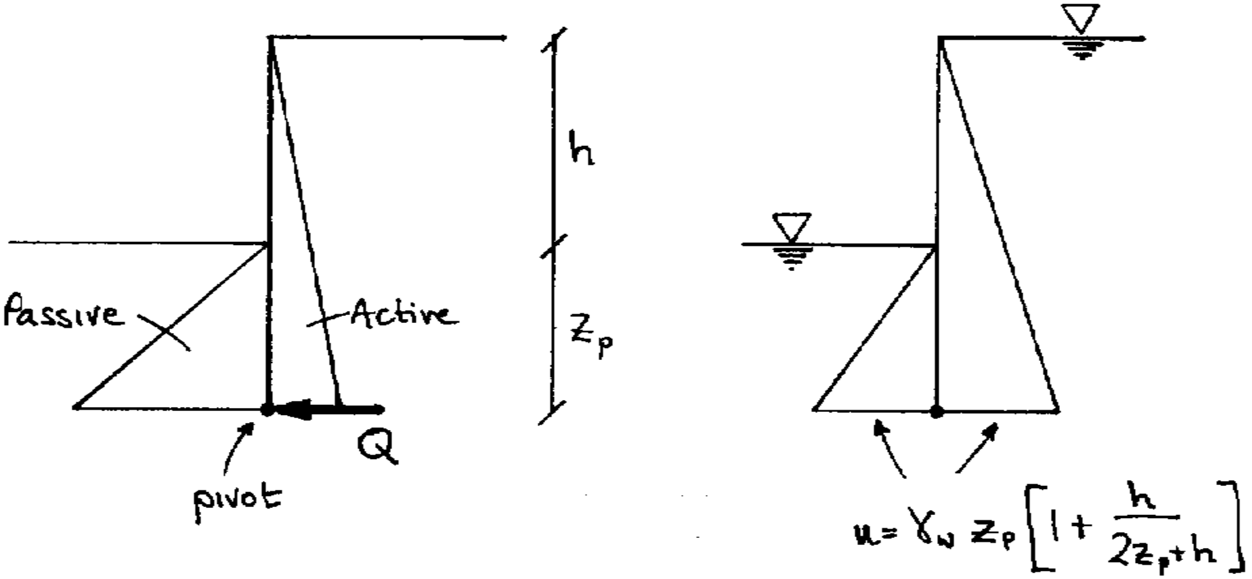


$$u = \gamma_w d\left[1 + \frac{h}{2d+h}\right]$$

Pore water pressures from ‘linear seepage’ model

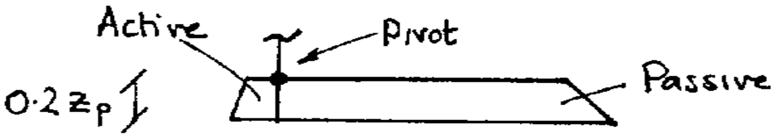
Pore water pressures

Unpropped embedded retaining walls: approximate long-term limiting stress distribution



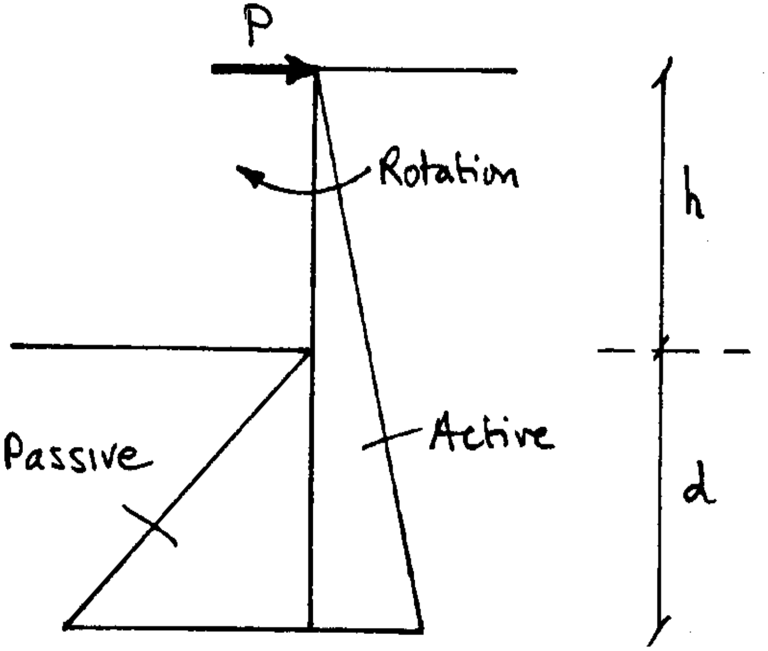
(a) effective stresses

(b) pore water pressures



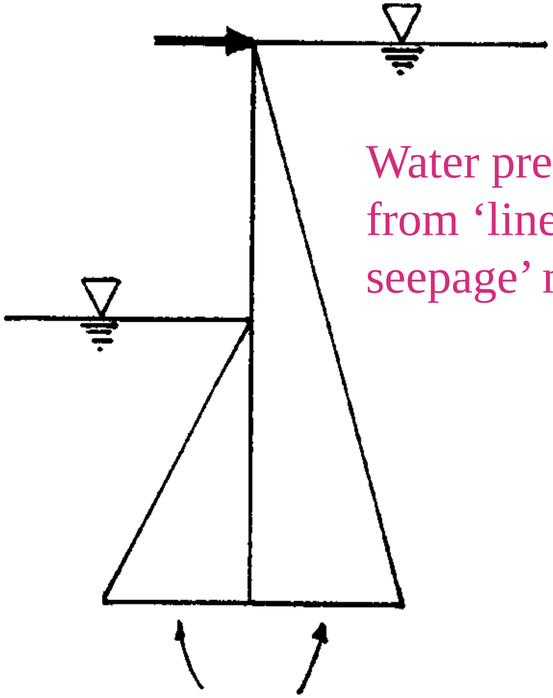
(c) check that resultant of limiting stresses below pivot is ≥ Q

Embedded retaining walls propped at the crest: long term limiting stress distribution (free earth support)



Rotates under ‘free earth’ conditions

Lateral effective stresses (earth pressures)



Water pressures from ‘linear seepage’ model

$$u = \gamma_w d \left[1 + \frac{h}{2d+h} \right]$$

Pore water pressures

Some comments on Limit Equilibrium Analysis (1)

- Traditionally used for geotechnical calculations because “elasticity” is difficult to quantify for soils
- Relies (approximately) on the lower bound theorem of plasticity: if we can find a stress field that is in **equilibrium** and does not violate the **failure criterion**, the solution is inherently **safe**
- Serviceability deemed satisfied by the application of an appropriate (partial) factor (of safety) to either the loads or the material strength

Some comments on Limit Equilibrium Analysis (2)

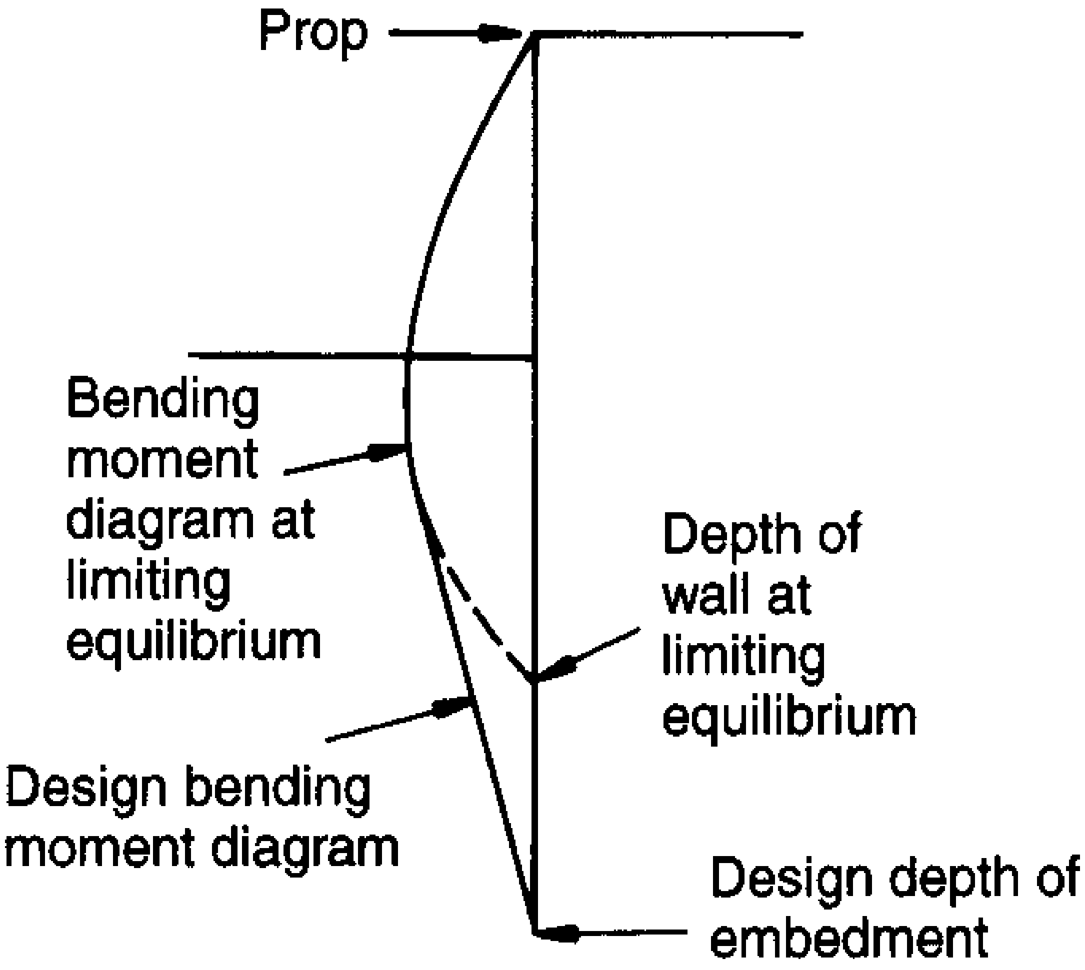
- In structures, it doesn’t usually matter whether we apply the factor to the load or to the material strength; linearity means that the numerical value is the same
- For some geotechnical calculations e.g. foundations, the factor could be applied to either the loads or the soil strength (or resistance), but non-linearity means that the numerical correlation may not be obvious
- Hence the C1 / C2 approach of the Eurocode: we apply partial factors greater than one to the strength and the loads in turn, and take the worse case for design

For embedded retaining walls

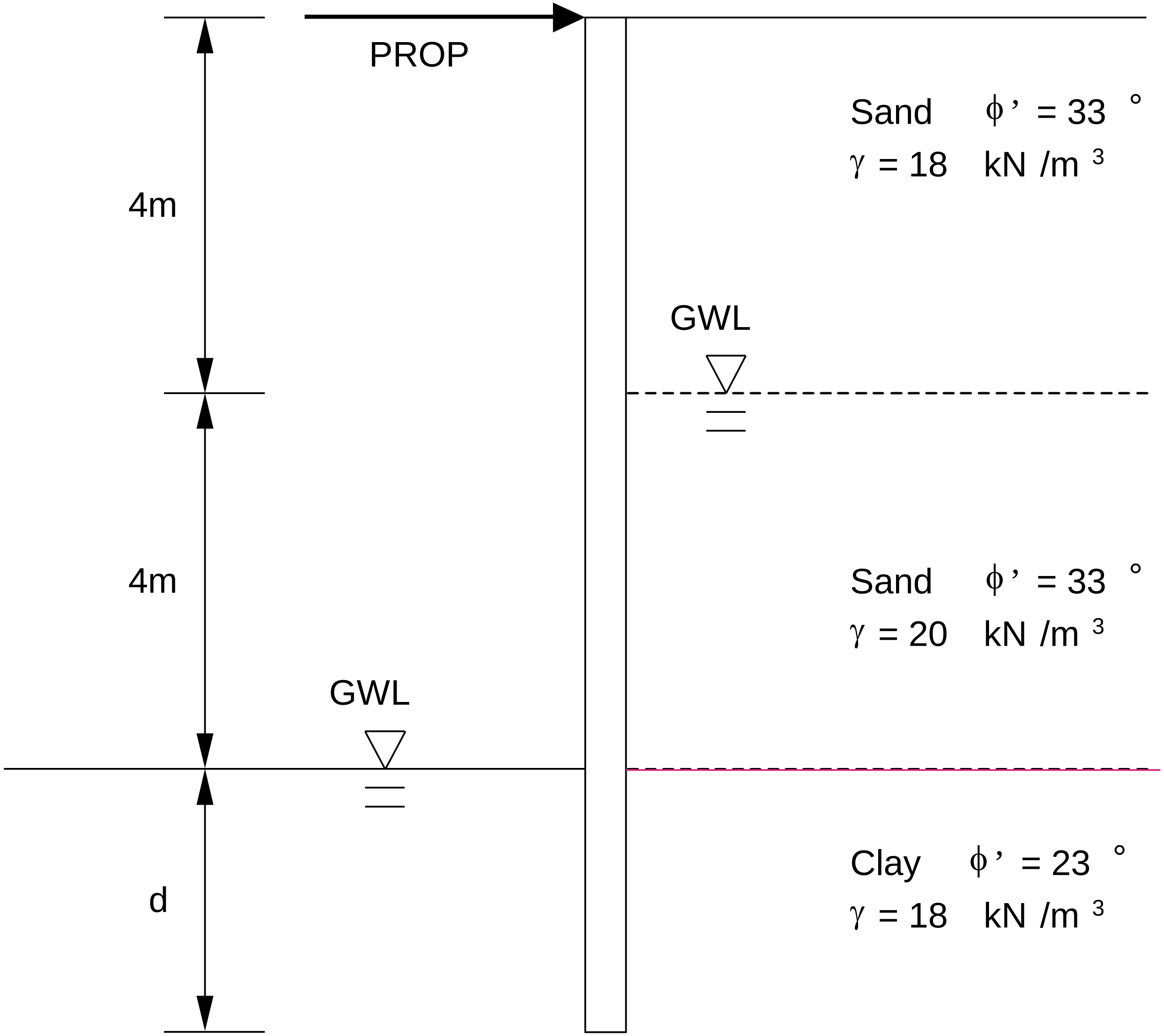
- The loads at “failure in the ground” depend on the soil strength
- Equivalence / duality of factoring loads vs factoring strength (ie partial factors greater than unity) does not quite work
- In C1, partial factors greater than 1 are applied to external variable loads and calculated structural stress resultants (prop loads, bending moments, shear forces) and a partial factor of 1 to the soil strength
- In C2, partial factors greater than one are applied to the soil strength and external variable loads but not to the structural stress resultants

For embedded retaining walls

- C1 analysis uses the C2 embedment depth in a soil structure interaction calculation (e.g. pseudo finite element or finite element), but
- In a limit equilibrium calculation, a new embedment depth that is at true limiting equilibrium is determined



Example

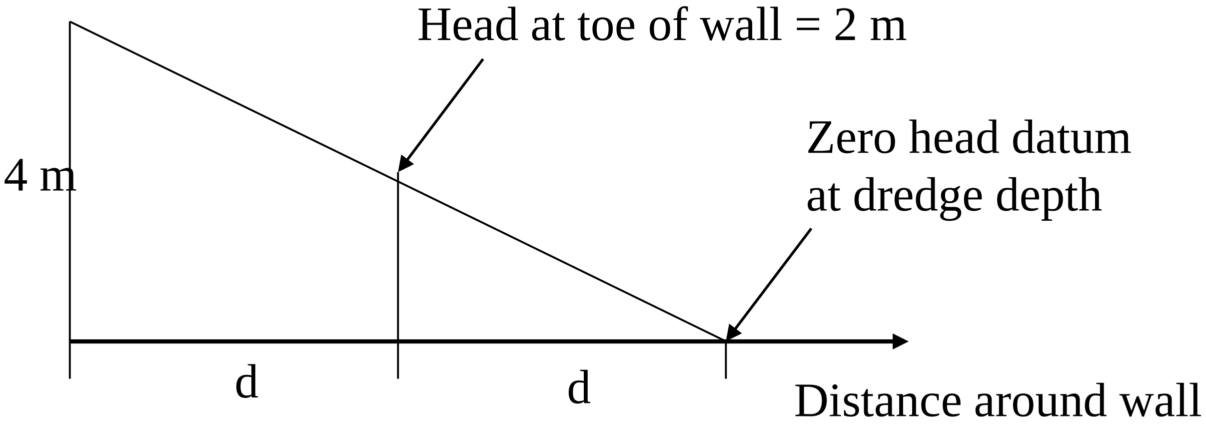

PROP
4m
4m
d
GWL
GWL
Sand ϕ' = 33 °
γ = 18 kN /m ³
Sand ϕ' = 33 °
γ = 20 kN /m ³
Clay ϕ' = 23 °
γ = 18 kN /m ³

Example

- Suggest a suitable assumption concerning the long term pore water pressures around the wall.
- Calculate the effective stresses and pore water pressure at depths behind and in front of the wall between which changes in stress may be treated as linear with depth. Use earth pressure coefficients $\sigma'_h/(\gamma z - u)$ of 0.2983 (active, sand); 0.4413 (active, clay); and 2.586 (passive, clay). (These correspond to a partial factor on $\tan\phi'$ of 1.25, and wall friction $\delta = \phi'_{mob}$ on both sides of the wall).
- Hence show that the design depth of embedment is in the region of 11.1 m, and calculate the corresponding prop load.
- The retained height includes the required 0.1 m overdig

Solution: long-term pore water pressures

- Assume seepage from the water table behind the wall to a water table at formation level in front, with all head being lost as the water flows through the relatively low permeability clay. Assume that the head loss (from 4 m at the clay surface behind the wall to zero at the excavated surface in front) is distributed linearly around the wall.
- The head at the toe of the wall (depth d) is then 2 m, and the pore water pressure at depth d is (d + 2) m × γ_w.



Solution: active pressures

- Calculate values of horizontal total stress at key points behind the wall
- $K_{aS} = 0.2983$; $K_{aC} = 0.4413$

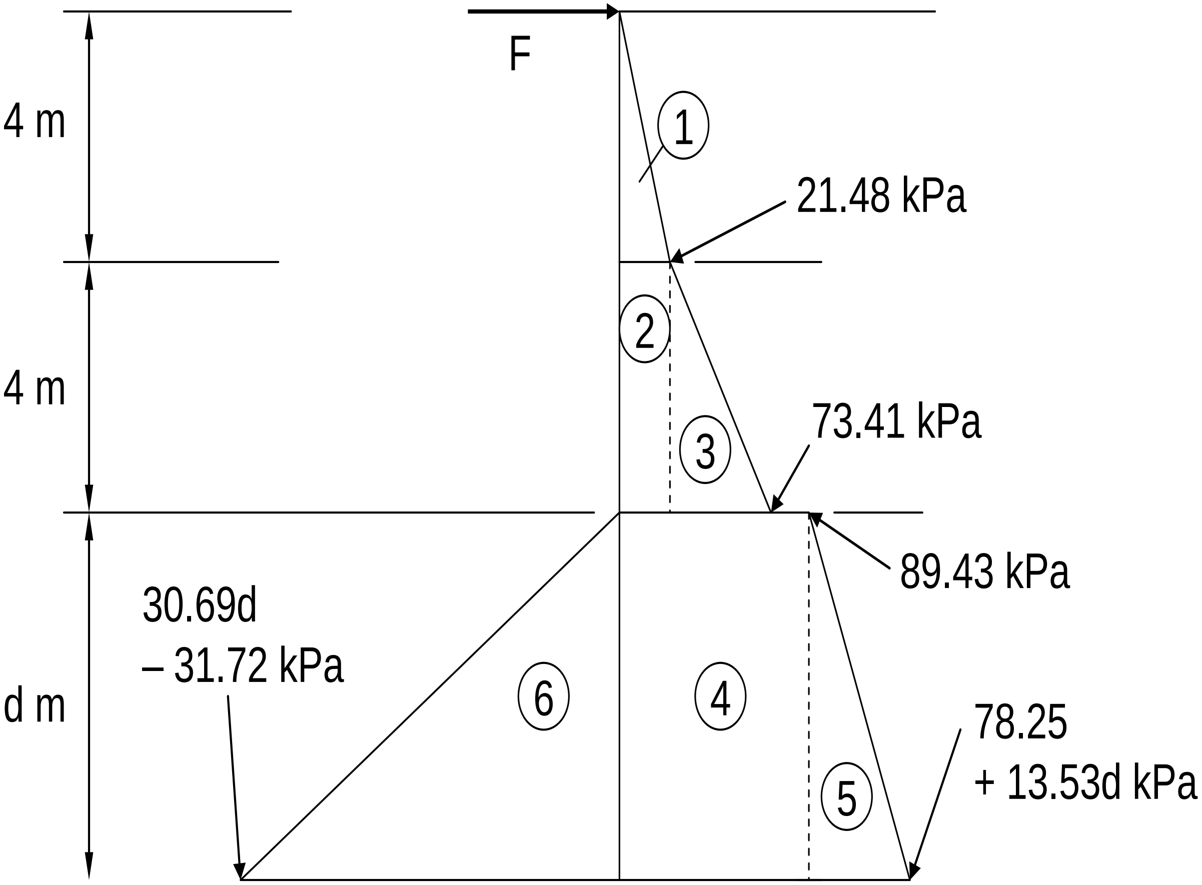
Depth, m behind wall	γz (kPa)	Pore pressure u (kPa)	$\sigma'_h = K_a (\gamma z - u)$, (kPa)	$\sigma_h = \sigma'_h + u$, (kPa)
0, Sand				
4, Sand				
8, Sand				
8, Clay				
8 + d, Clay				

Solution: passive pressures

- Calculate values of horizontal total stress at key points in front of the wall
- $K_{pC} = 2.586$

Depth, m behind wall	γz (kPa)	Pore pressure u (kPa)	$\sigma'_h = K_p(\gamma z - u)$, (kPa)	$\sigma_h = \sigma'_h + u$, (kPa)
0, Clay				
d, Clay				

Total stresses on wall, divided into stress blocks



Take moments about prop to determine wall embedment d

Stress block	Average stress within stress block, kPa	Depth of stress block, m	Resultant force, kN/m	Lever arm about prop, m
1	21.48 ÷ 2 = 10.74	4	42.96	4 × 2/3 = 2.67
2	21.48	4	85.92	4 + (4 ÷ 2) = 6
3	(73.41 – 21.48) ÷ 2 = 25.97	4	103.86	4 + (4 × 2/3) = 6.67
4	89.43	d	89.43d	8 + d/2
5	(78.25 + 13.53d – 89.43) ÷ 2 = (6.765d - 5.59)	d	$6.675d^2$ – 5.59d	8 + 2d/3
6	(30.69d – 31.72) ÷ 2 = (15.345d – 15.86)	d	$15.345d^2$ – 15.86d	8 + 2d/3

Solution: moment and horizontal equilibrium

For moment equilibrium about the prop:

$[114.703] + [515.52] + [692.75] + [715.44d + 44.72d^2] = [5.78d^3 + 62.51d^2 - 82.16d]$

(the term on the right hand side represents the effect of stress block 6 minus stress block 5)

$\Rightarrow 5.78d^3 + 17.79d^2 - 797.6d = 1322.76$

By trial and error, the condition of moment equilibrium is satisfied with

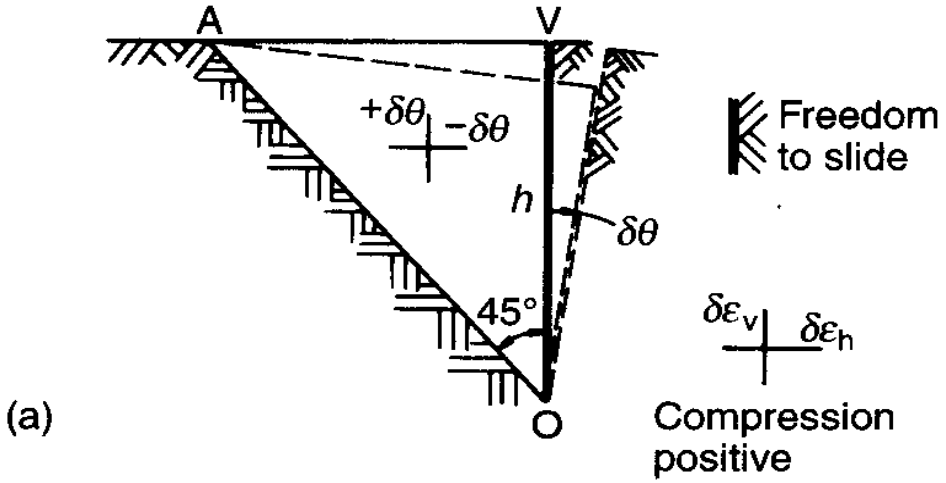
$\underline{d = 11.145\text{m}}$

The corresponding prop load may be calculated from the condition of horizontal force equilibrium for the wall. Substituting d = 11.145 m into the expressions for the forces in the table:

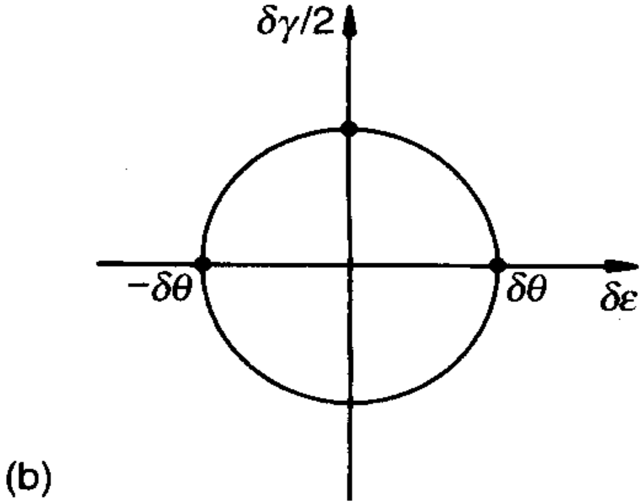
$F = [42.96] + [85.92] + [103.86] + [89.43 \times 11.145] - [(8.67 \times 11.145^2) - (10.27 \times 11.145)] \Rightarrow \underline{F = 207(.25)\text{ kN/m}}$

Idealized deformation pattern and shear strain for a rotating rigid bulkhead

(a) Idealised shear strains

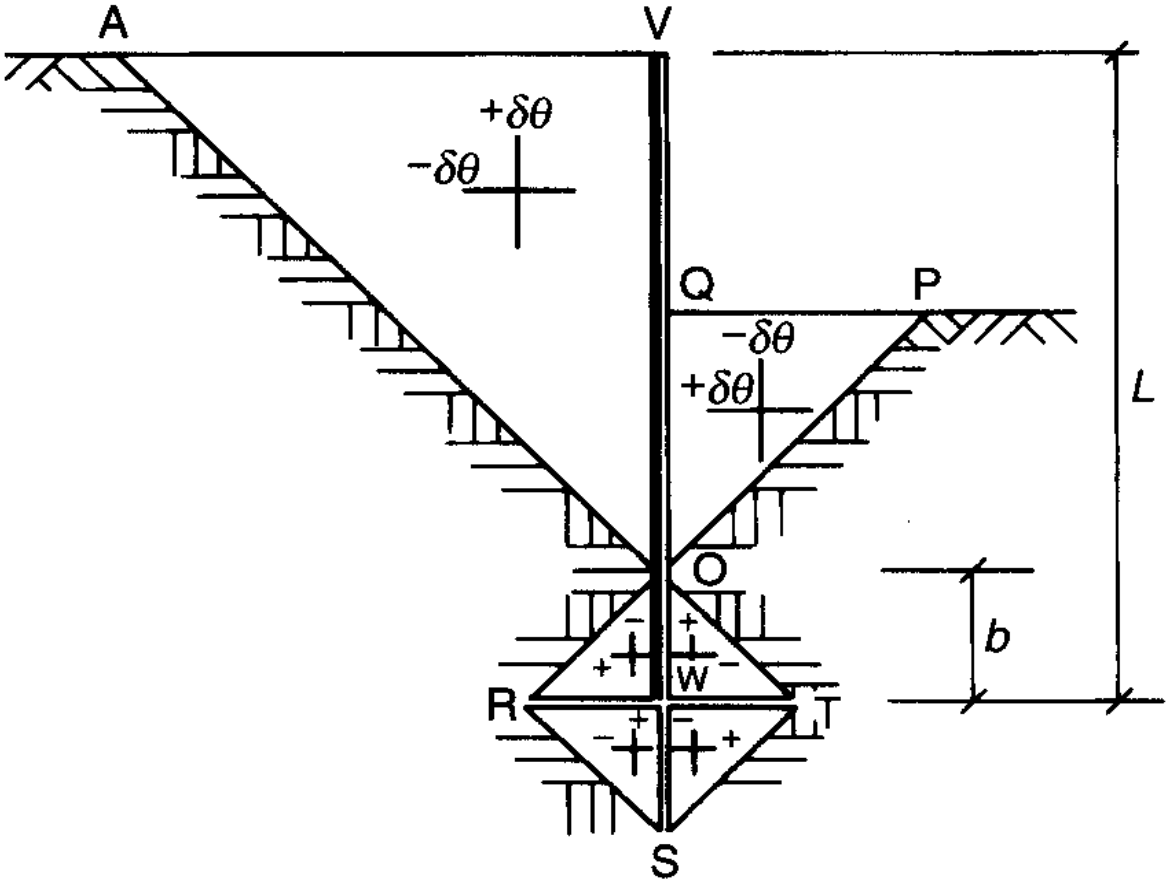


(b) Mohr circle of strain

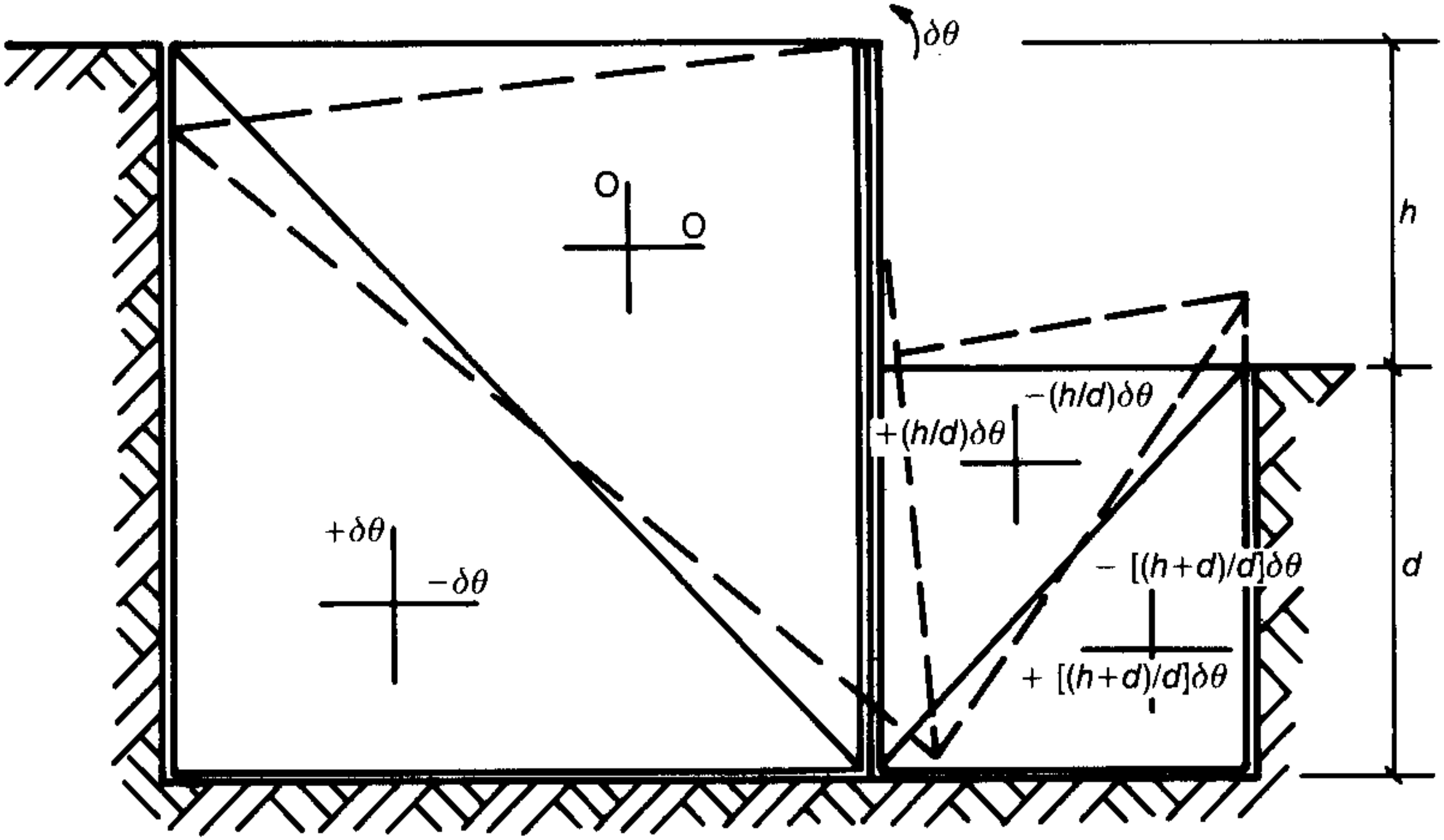


Idealised deformation pattern for a rigid unpropped wall

Shear strain increment $\delta\gamma = 2 \times$ increment of wall rotation $\delta\theta$



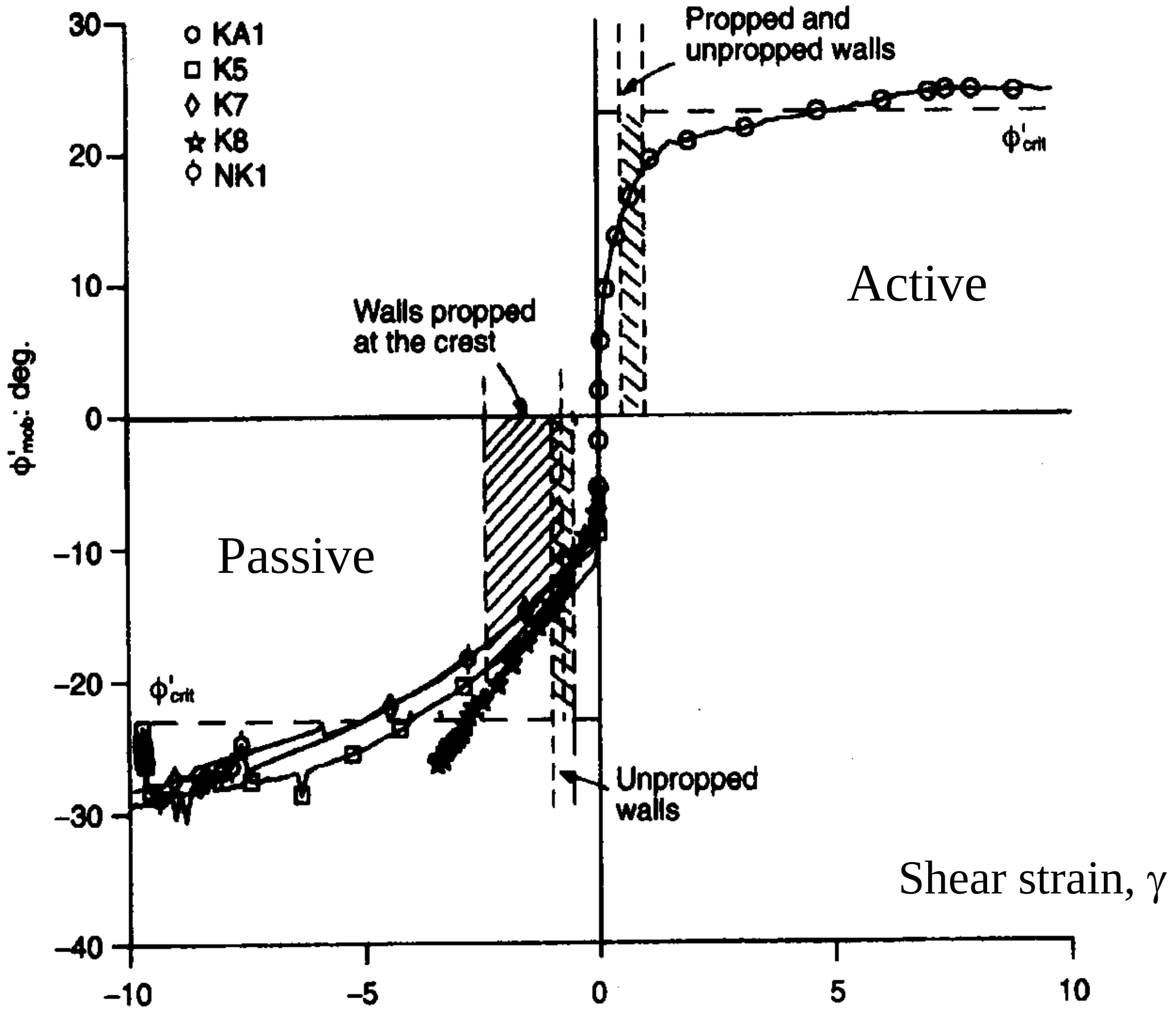
Rigid wall propped at the crest



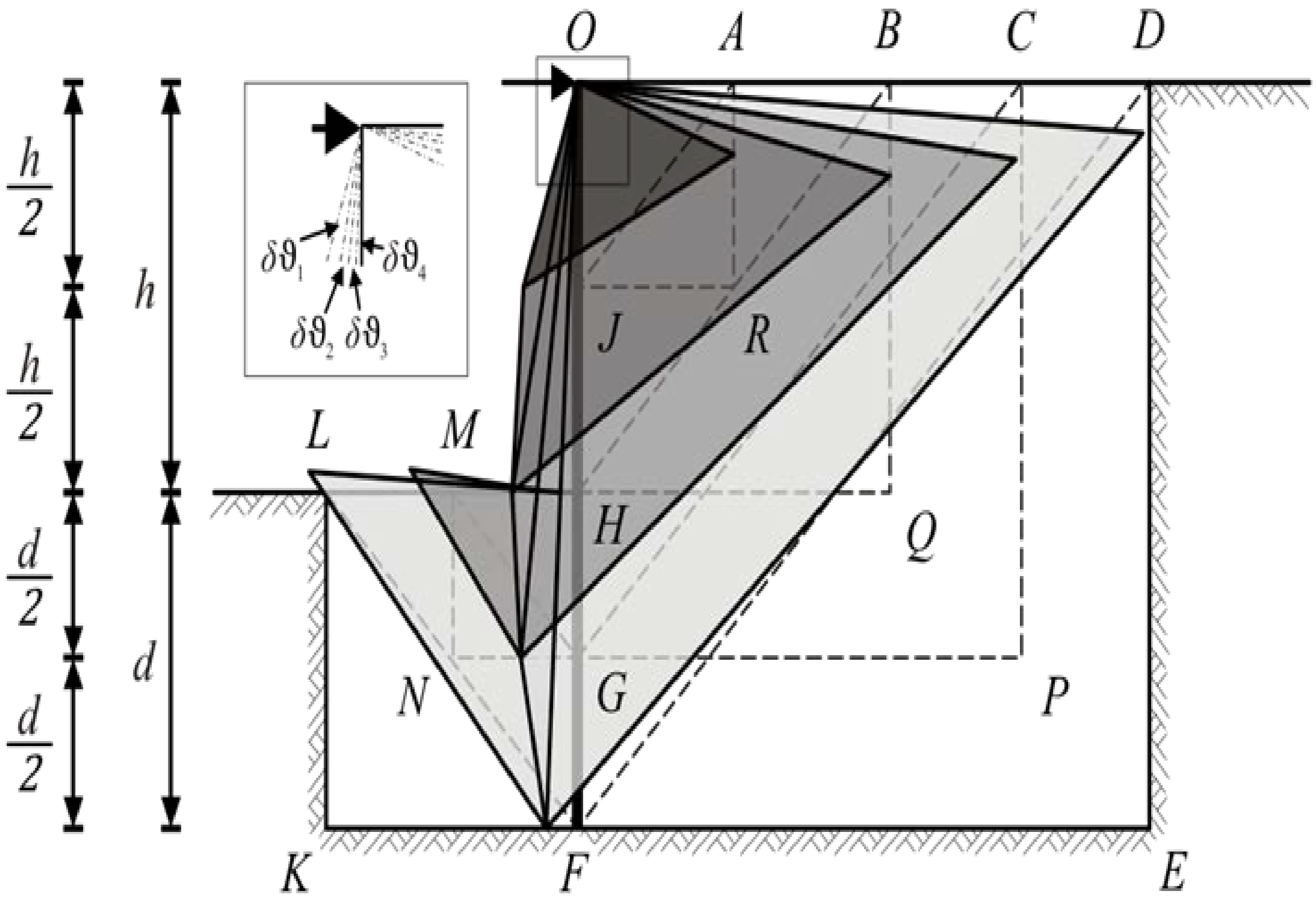
Behind wall as for an unpropped wall, $\delta\gamma = 2\delta\theta$

In front of wall, $\delta\gamma = 2(1+h/d)\delta\theta$

Relative strength mobilisation rates (stiffnesses)



Superposition of displacement mechanisms to account for wall flexibility (rotation concentrated at joints)

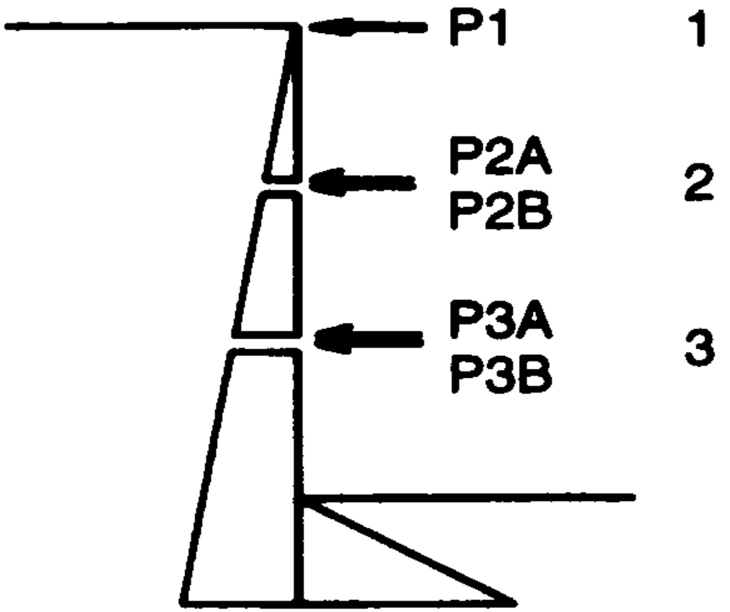


MODELLING MORE COMPLEX SITUATIONS

- Multiple props
- Wall flexibility
- Soil / structure interaction analyses
- Modelling installation effects for in situ walls
- Earth berms

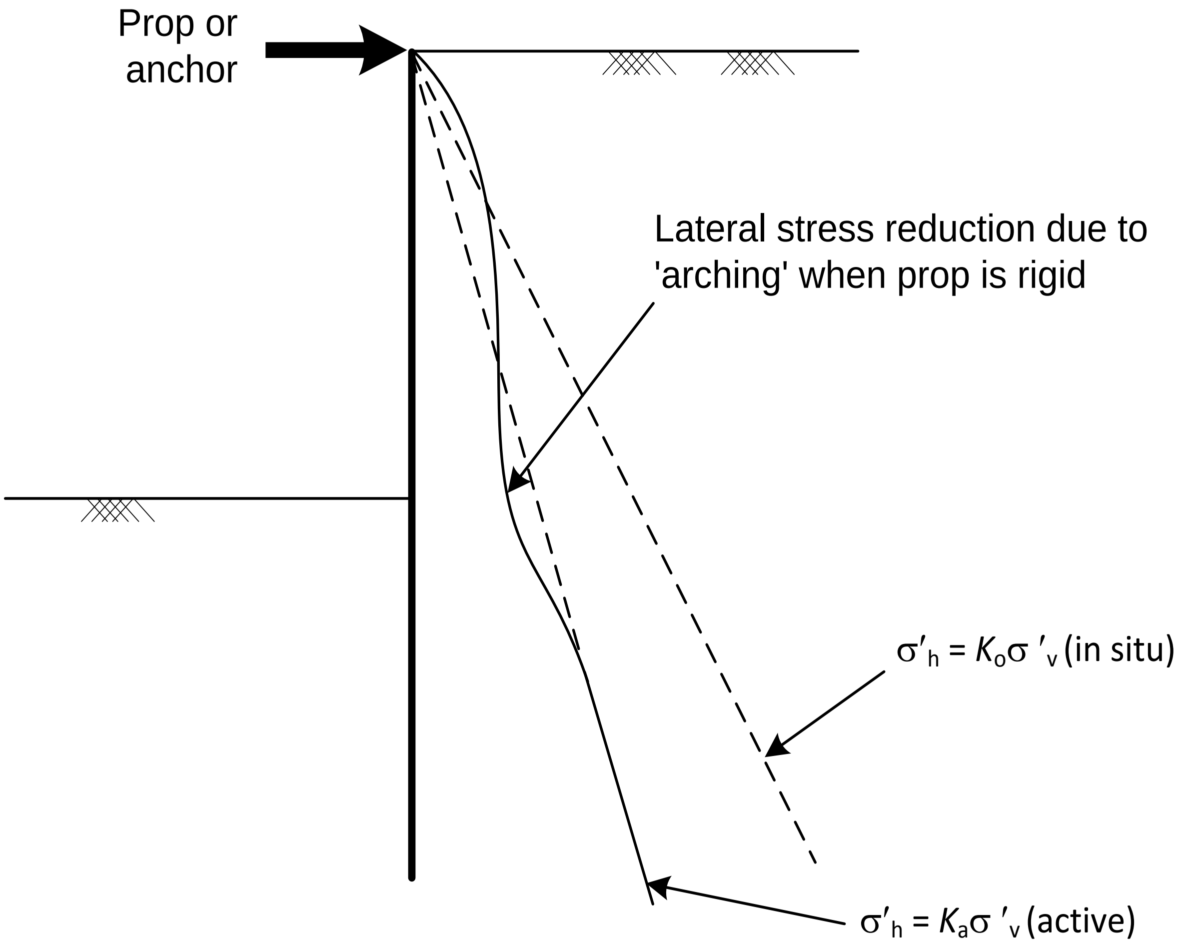
Multi-propped embedded retaining walls: long-term limiting stress distribution

- Are statically indeterminate
- To determine wall depth and prop loads cut wall into sections (see right)
- OR use an elastic soil-structure interaction analysis (see later)

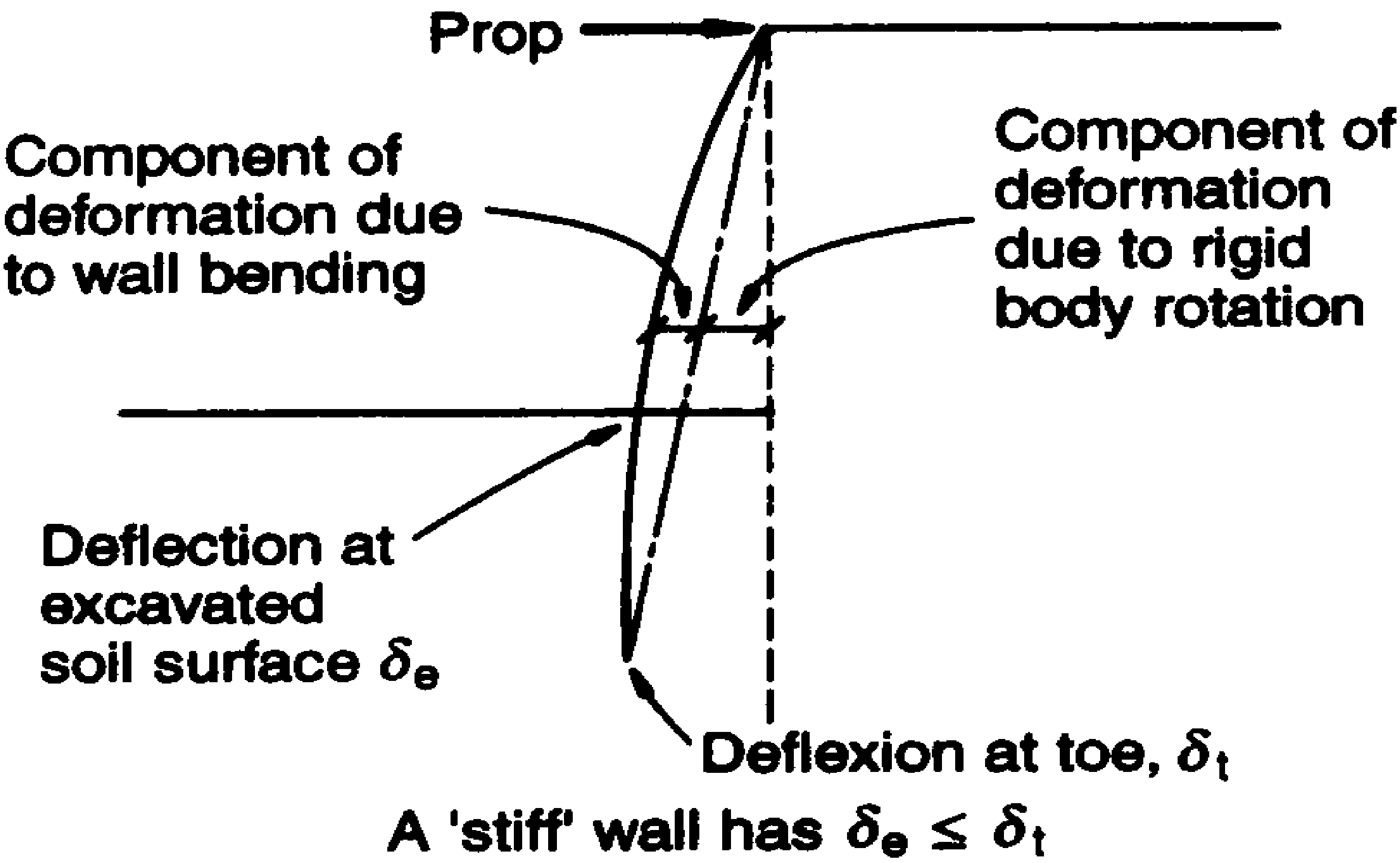


Wall span between adjacent props analysed as simply-supported beam; lowest section analysed as propped cantilever.
Total load in prop 2 = P2A + P2B.
Total load in Prop 3 = P3A + P3B etc.

Effect of a rigid prop on lateral stresses behind the wall

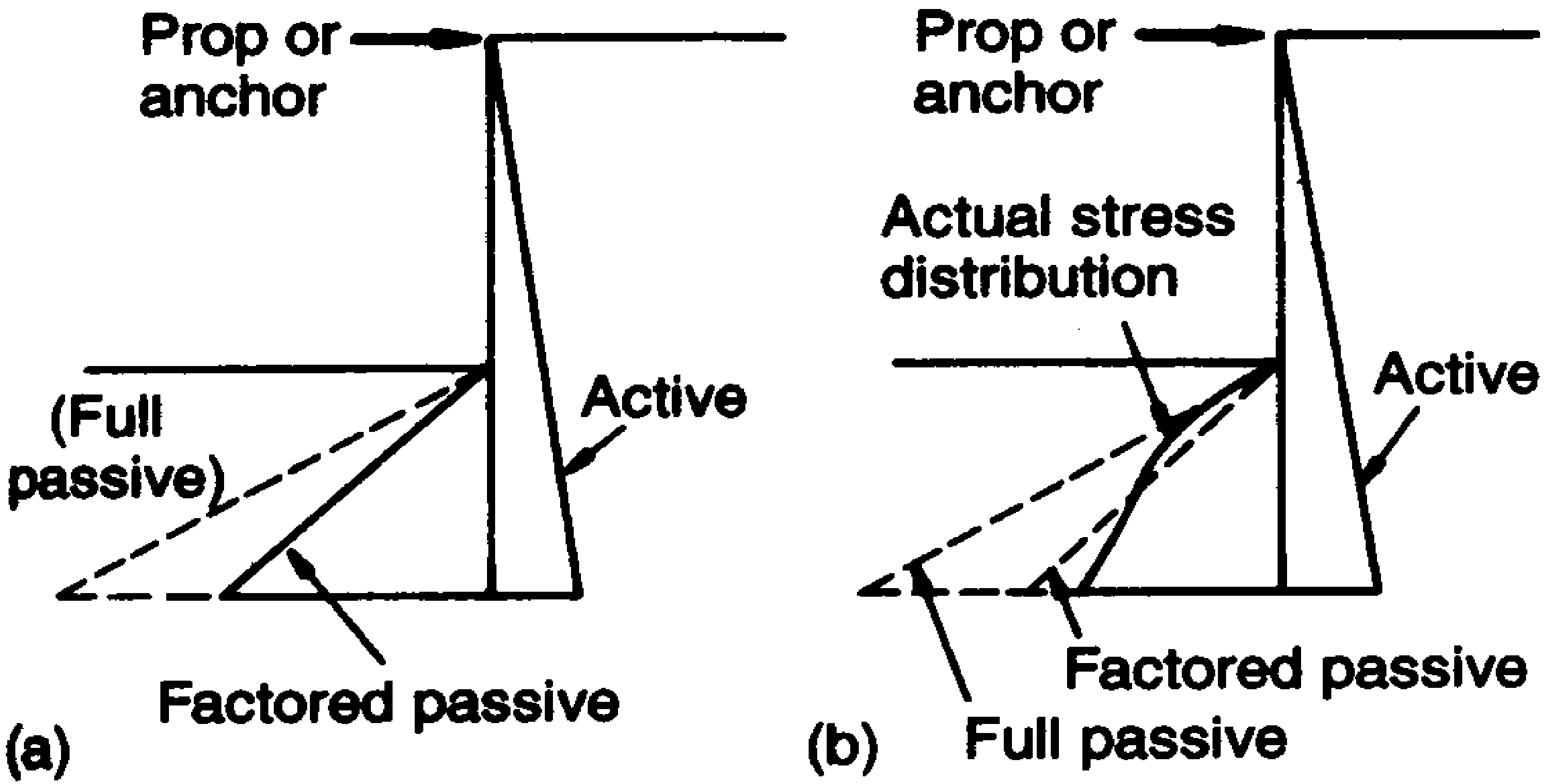


Deformation of a flexible wall

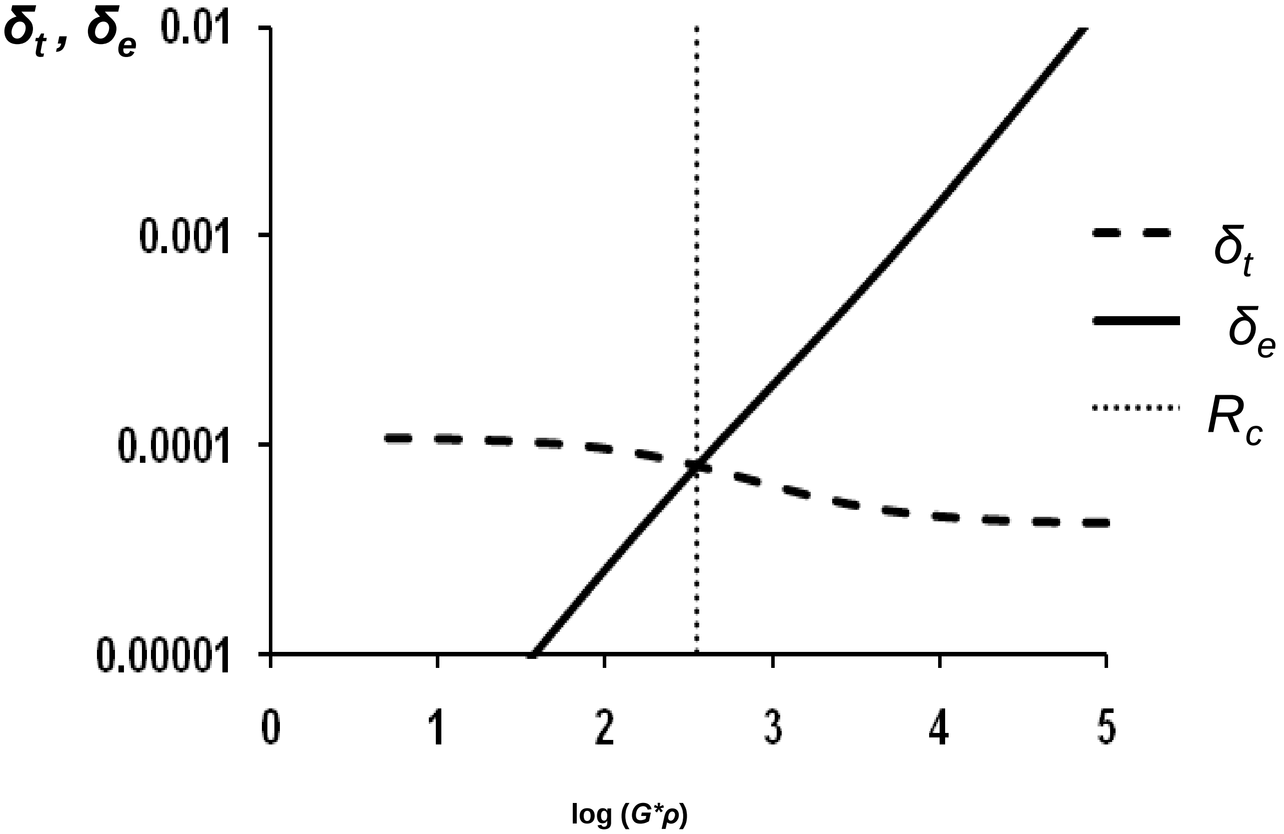


A 'stiff' wall has $\delta_e \leq \delta_t$

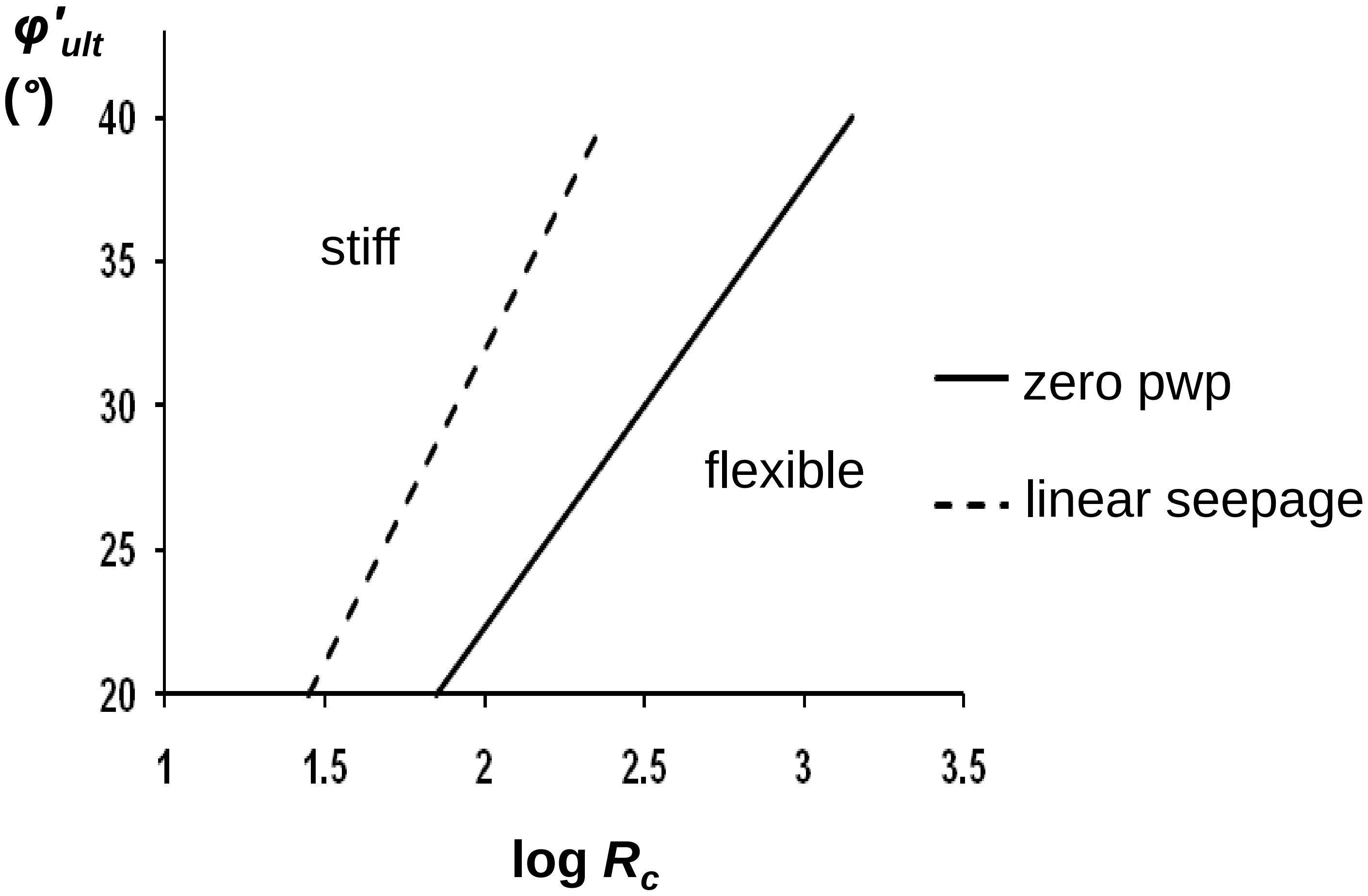
Lateral stress redistribution due to wall flexibility effects



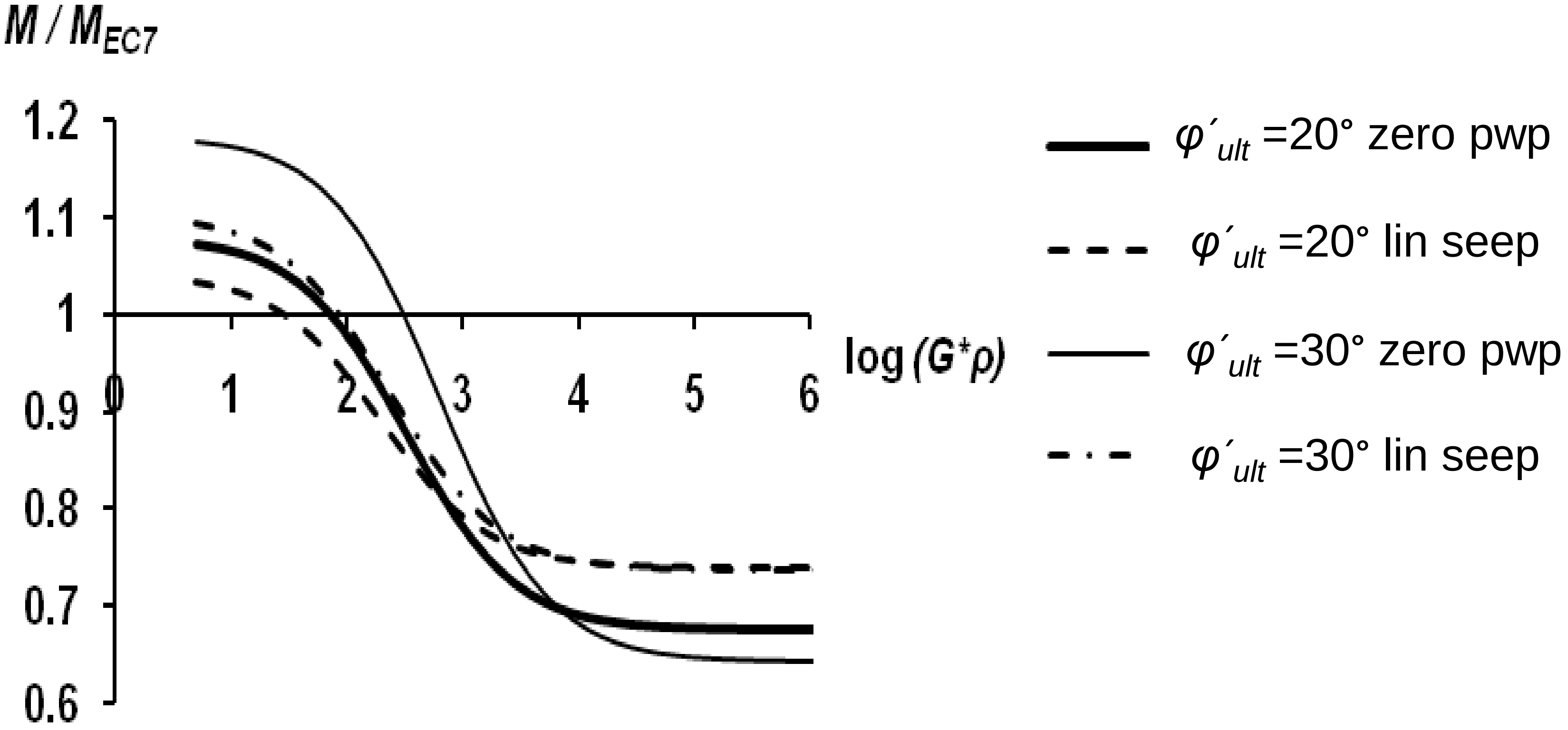
Critical value of $(G^*H^4/EI) = R_c$ at which $\delta_e = \delta_t$ for given soil strength and pore pressure conditions



Critical flexibility number R_c for different soil strengths and pore pressure conditions

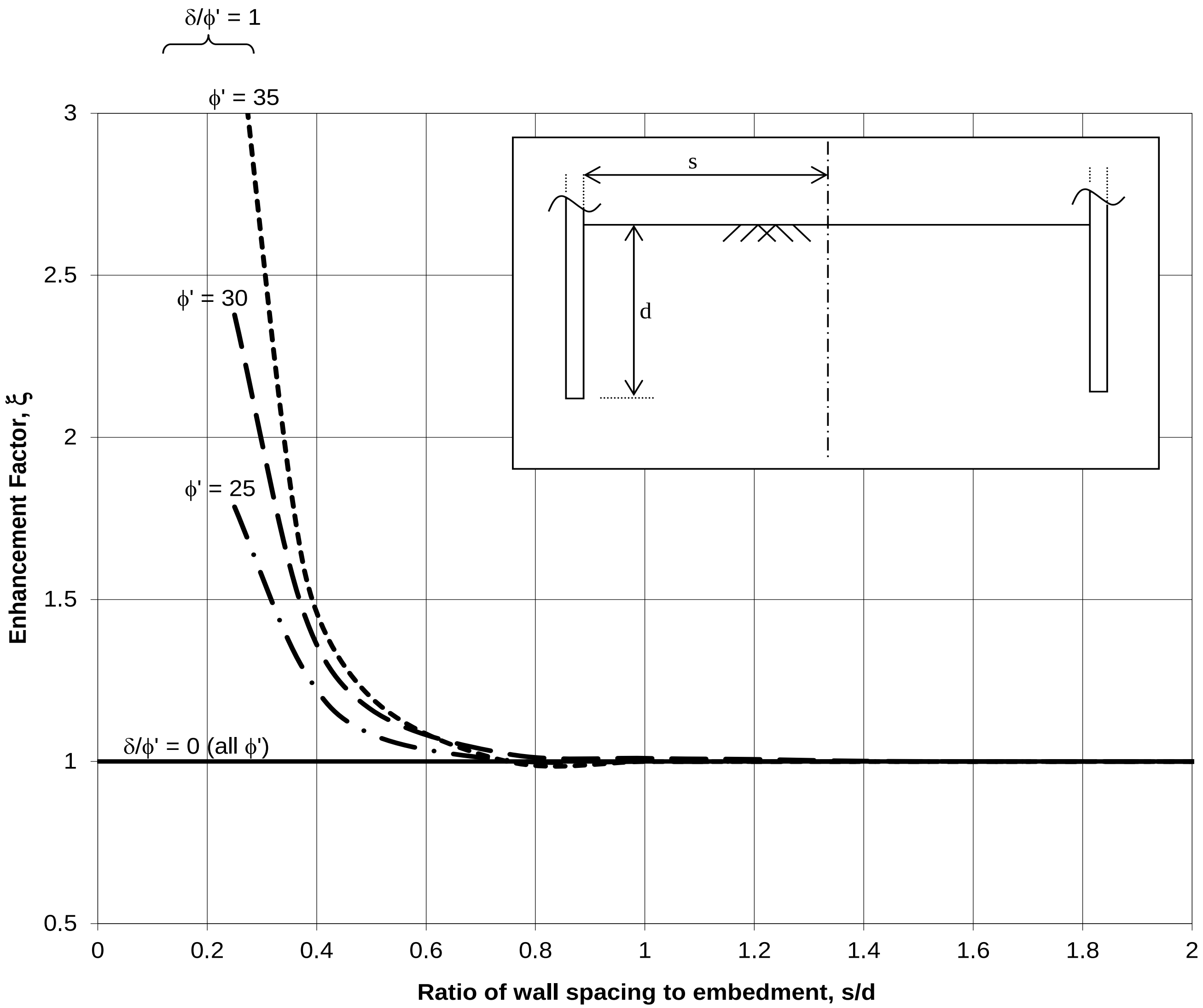


Moment reduction curves accounting for wall flexibility



A unique curve is obtained for given soil strength and pore pressure conditions with G^*H^4/EI as the relative soil/wall stiffness

Passive pressure enhancement factors for interacting walls



Soil / structure interaction analyses

Types

- Subgrade reaction, beam on springs, pseudo FE
- Finite element / finite difference

Solve numerically the equations associated with

- Equilibrium (of stresses and forces)
- Compatibility (of strains and displacements)
- Constitutive behaviour of the soil (elasto-plastic)

Soil model can be more sophisticated

- Effective stress, total stress or consolidation
- Anisotropic
- Stress and strain dependent (non-linear) stiffness

Able to model complex geometry (FE and FD)

Modelling in situ concrete wall installation effects in analyses starting with the wall in place

Set pore pressures to the in situ values

Where the soil constitutive model does not distinguish between the stiffnesses of the soil behind and in front of the wall,

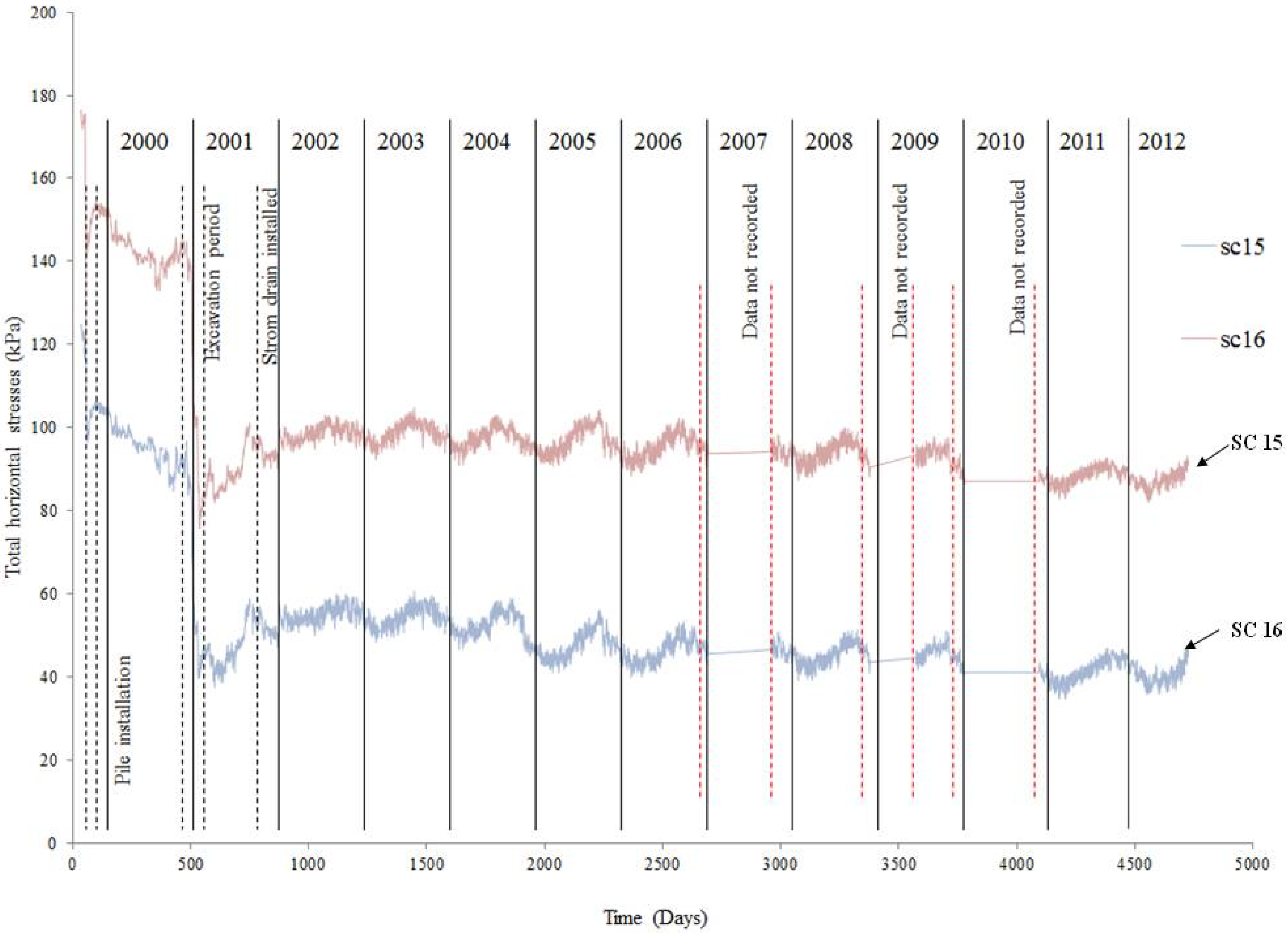
- for heavily overconsolidated deposits having $K_0 > 1$, set $K_{initial} = 1$ (overestimate of actual lateral stress relief is compensated for by the neglect of the increased stiffness of the soil on the retained side of the wall)
- If $K_0 < 1$, set $K_{initial} = K_0$

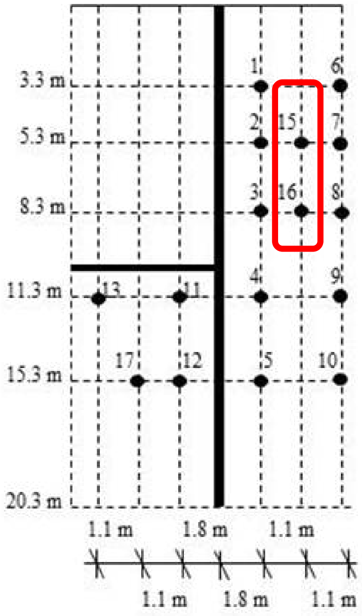
Modelling in situ concrete wall installation effects in analyses starting with the wall in place

If the soil constitutive model does allow for the effect of recent stress history on soil stiffness (ie the soil behind the wall is stiffer than that in front)

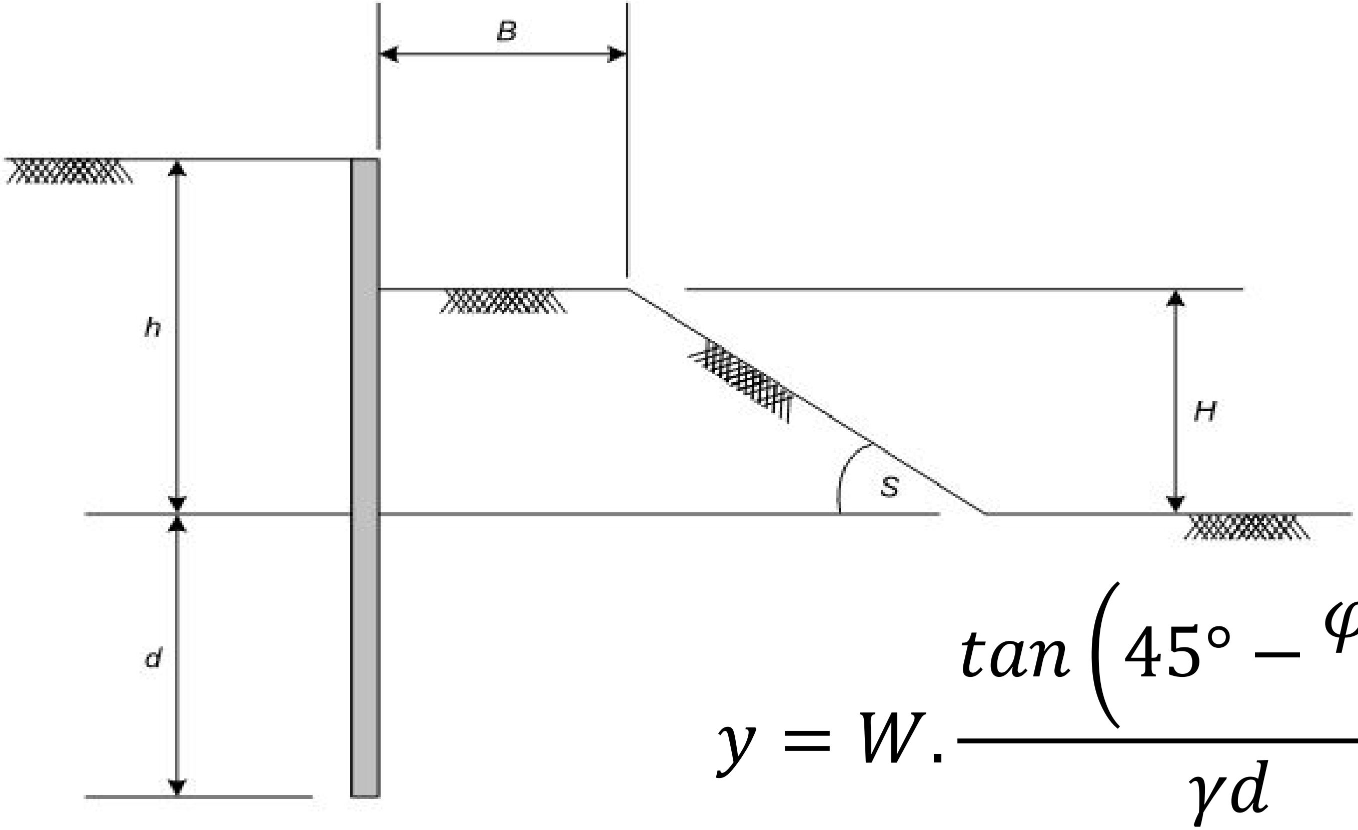
- in an overconsolidated deposit, reduce the in situ lateral earth pressure coefficient K_0 by 10-20%
- If $0.5 < K_0 < 1$, set the pre-excavation lateral earth pressure coefficient $K_{init} = K_0$
- If $K_0 < 0.5$, set $K_{init} = 0.5$ to represent a possible increase during installation to approximately the bentonite support pressure

No tendency for high in situ lateral stresses to re-establish in the long-term (data from HS1, Ashford)





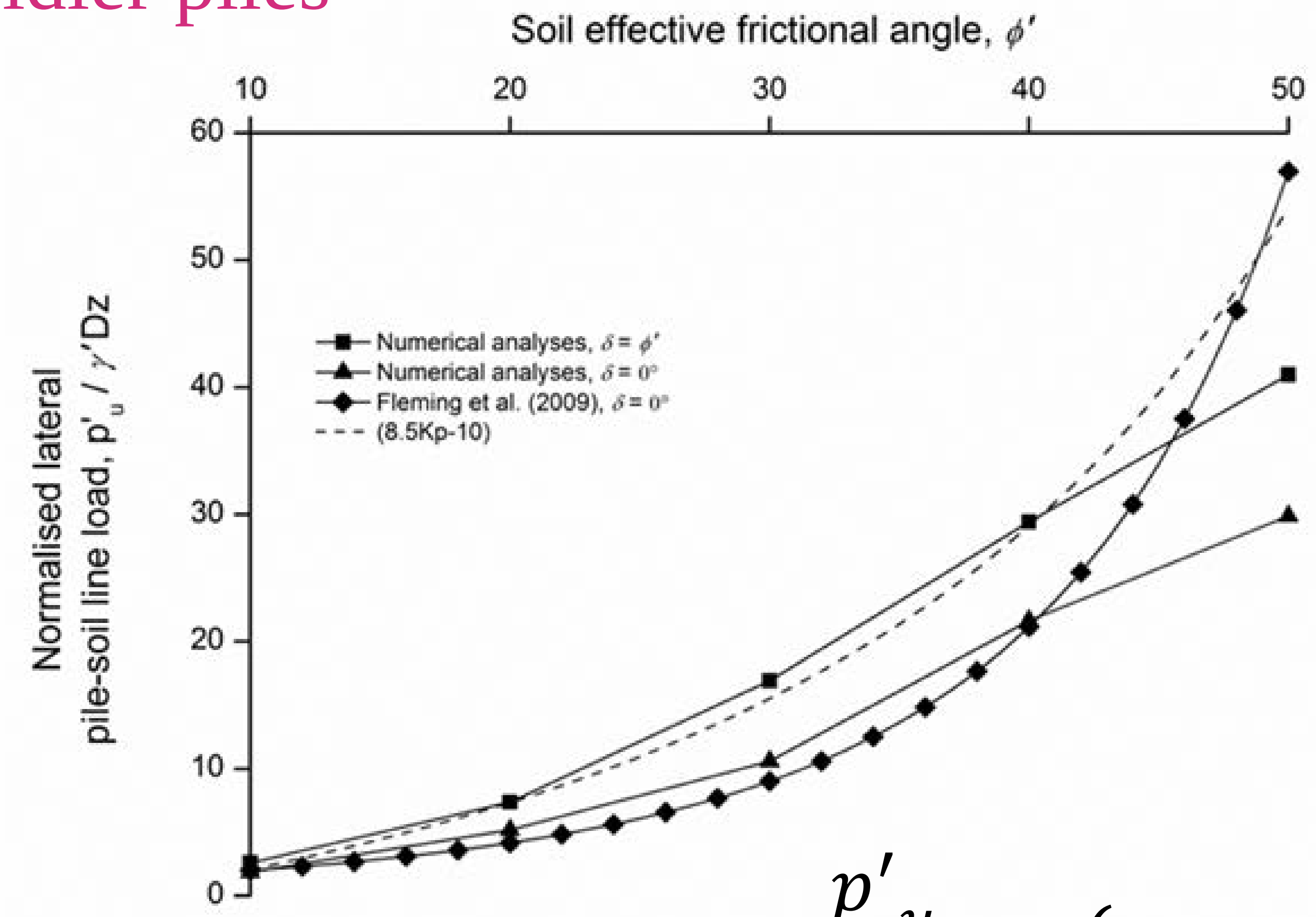
Modified "raised effective formation level" approach for modelling berms in effective stress analysis



$$y = W.\frac{\tan\left(45^\circ - {\varphi'}/_{2}\right)}{\gamma d}$$

y = raised effective formation level
W = berm weight

Limiting lateral load at depth on kingpost wall soldier piles



$$\frac{p'_u}{\sigma'_v . d} = \left(8.5K_p - 10\right)$$

WORKED EXAMPLES

Idealised Worked Examples

- Effective stress analysis:
  - Example 1: cantilever wall
  - Example 2: singly propped wall

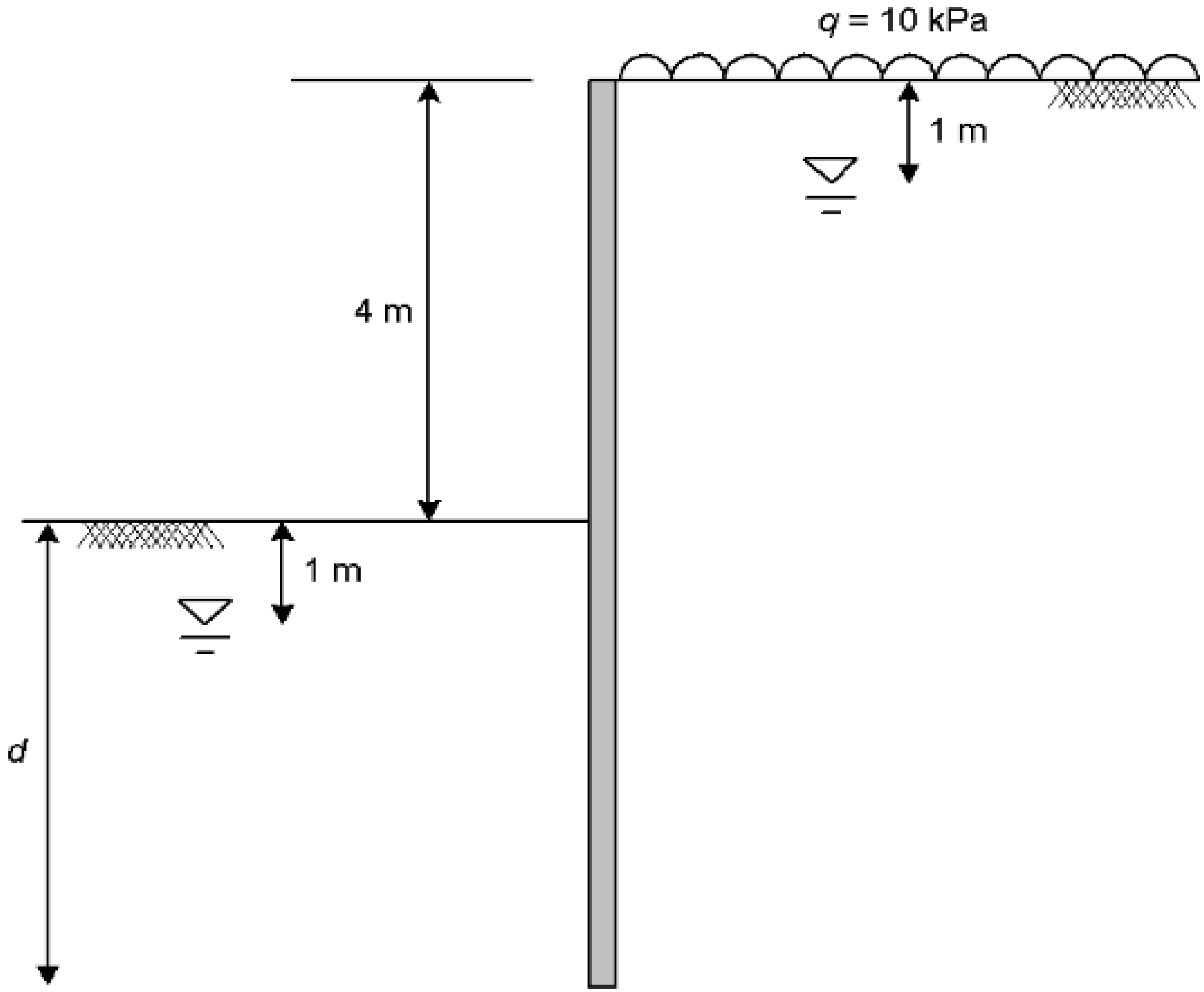
- Total stress analysis:
  - Example 3: cantilever wall
  - Example 4: singly propped wall

Idealised Worked Examples

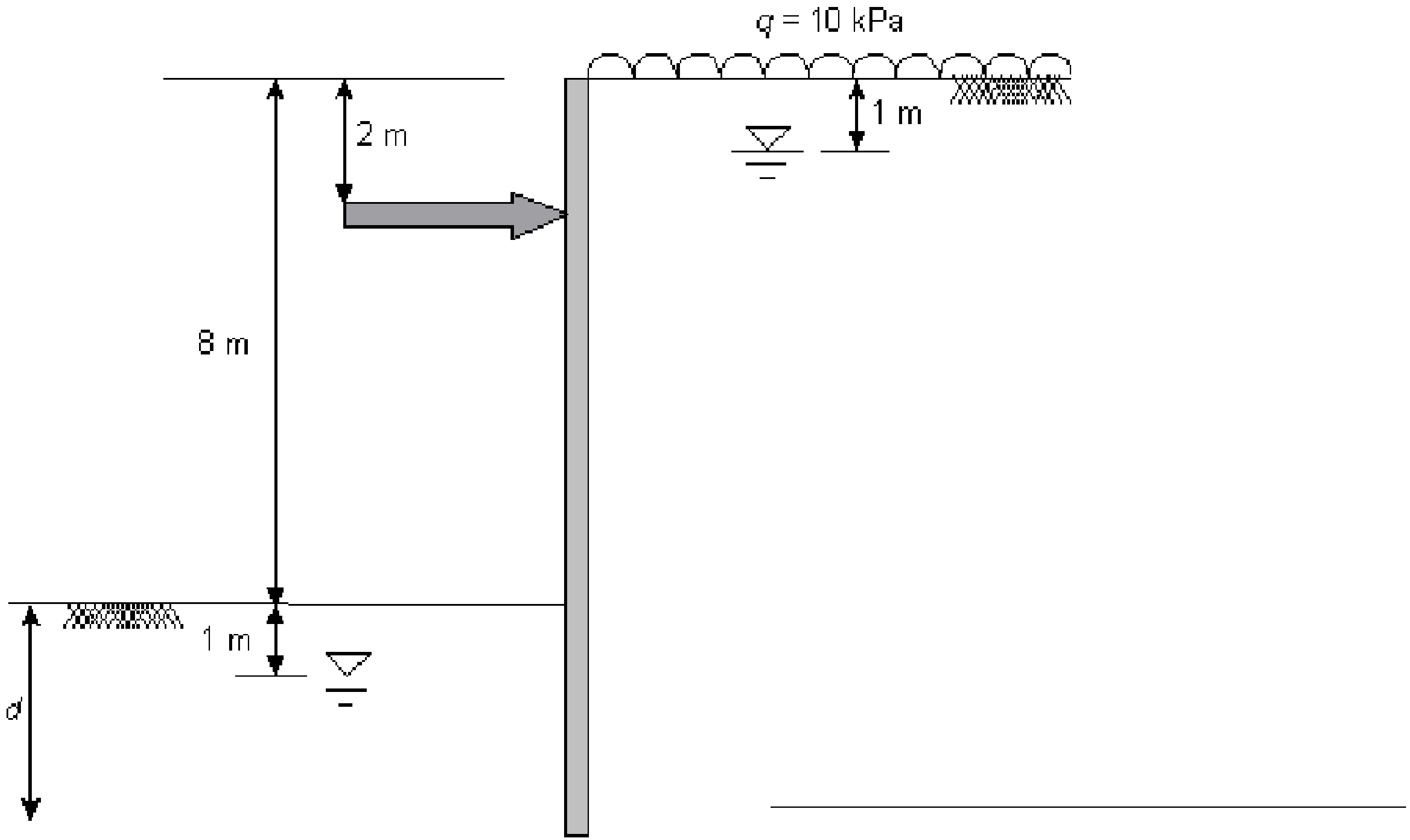
Solved using

• limit equilibrium	Spreadsheet FREW WALLAP
• pseudo-finite element	FREW WALLAP
• finite element	PLAXIS

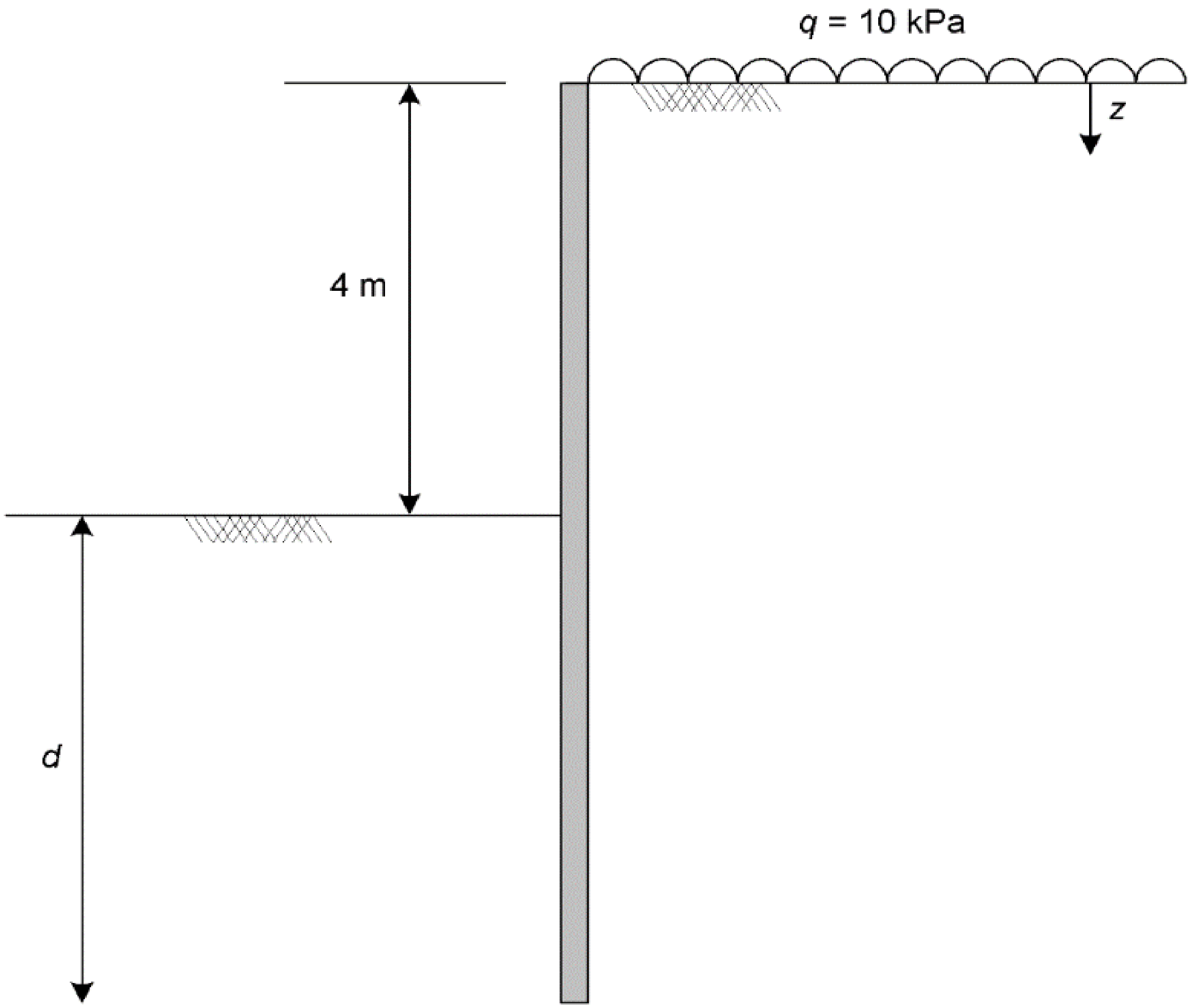
Example 1: effective stress, cantilever



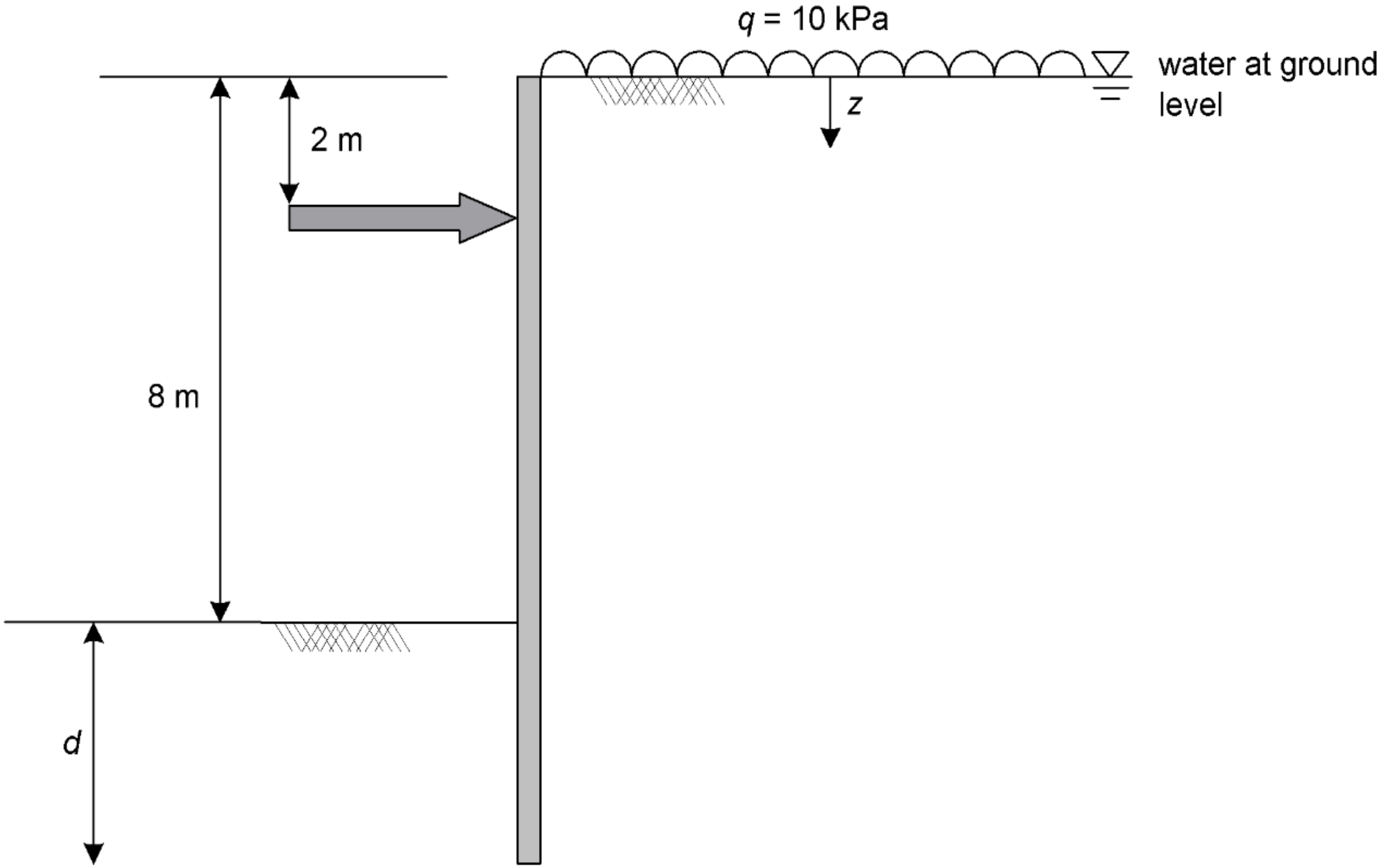
Example 2: effective stress, propped



Example 3: total stress, cantilever



Example 4: total stress, propped



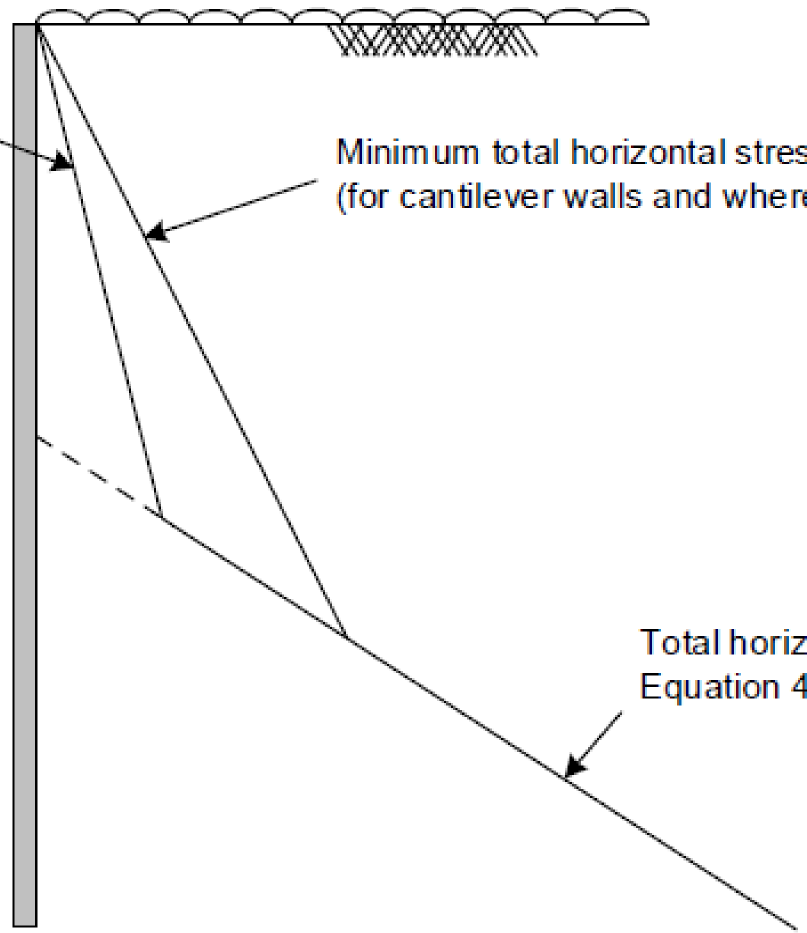
Key assumptions and partial factors (1)

	Calculation approach		
	EC7 DA1 Com.1	EC7 DA1 Com.2	SLS
Factor on effective cohesion c′	1	1.25	1
Factor on angle of shearing resistance ϕ′	1	1.25	1
Effective stress soil/wall adhesion	0	0	0
Maximum angle of soil/wall friction δ	ϕ′	ϕ′	ϕ′
Factor on undrained shear strength c_u	1	1.4	1
Maximum soil wall adhesion c_w	$c_u/2$	$c_u/2$	$c_u/2$
Factor on Imposed Loads	1.1 (V) 1.0 (P)	1.3 (V) 1.0 (P)	1 (all)
Factor on structural effects	1.35	1	1

Key assumptions and partial factors (2)

	Calculation approach		
	EC7 DA1 Com.1	EC7 DA1 Com.2	SLS
Expected surcharge applied to active (retained) side (assumed Variable)	10 kPa		
Allowance for unexpected surcharge	0 kPa		
Unplanned excavation (assuming a well-controlled site)	0.1m	0.1m	None
Maximum depth of soil softening applied to total stress analysis	0.5m below lowered excavation level		
Tension crack in total stress analysis	MEFP of 5z for propped walls Flooded hydrostatic ($\gamma_w z$) for unpropped walls		

Tension cracks (dry and flooded)



MEFP = 5z
(where water is not
expected)

Minimum total horizontal stress = $\gamma_w z$
(for cantilever walls and where water is expected)

Total horizontal stress calculated from
Equation 4.9

Seems straightforward, but……

- C1 wall depth calculated independently in LEA; C1 calculation uses C2 depth in pseudo FE and FE analyses
- How to calculate pore pressures - linear seepage or solving the flow equations? (the latter is probably only realistic in a full finite element analysis)
- Active or passive pressures behind the wall above the prop? (if passive, the values are reduced because wall friction is reversed)

Seems straightforward, but……

- In a full LE calculation for an unpropped wall, do you modify the active and passive pressure coefficients below the pivot to account for the reversal in the relative direction of wall friction? (experience and back-analysis of full scale walls and model tests suggests not; embedment depths will be increased significantly if you do)

Seems straightforward, but……

- In an undrained full LE calculation for an unpropped wall, do you take an MEFP of 5z kPa (z measured below the water table) below the pivot in front of the wall? (it generally makes little difference compared with assuming conventional active pressures in this region)

Seems straightforward, but……

- Commercial programs will not always let you use the parameters you want, eg c'_w/c' different from δ/ϕ'
- Node placement / spacing might limit resolution in some pseudo FE programs
- Modelling (flooded or dry) tension cracks and / or specification of tension crack depth may be difficult
- In FEA, do you reduce the strength at the start ("strategy 1") or at each stage separately ("strategy 2")?

Further input data and assumptions for pseudo FE and FEA: cantilever walls

Stage 0: install wall

- Post-installation lateral earth pressure coefficient $K_0 = 1.0$

Stage 1: excavation to final formation level

- Excavation to 4.1 m depth in ULS analysis and 4.0 m depth in SLS analysis

Soil and wall stiffness (both stages)

- Wall EI = 469 000 kNm^2/m (~ a hard/hard secant bored pile wall comprising 750 mm pile diameters at 650 mm spacing, with EI reduced by 30% to allow for long-term cracking
- Soil Young’s modulus = 48 MPa (effective stress, Ex 1), 60 MPa (undrained, Ex 3): shear modulus G = 20 MPa

Further input data and assumptions for pseudo FE and FEA: propped walls

Stage 0: install wall

- Post-installation lateral earth pressure coefficient $K_0 = 1.0$

Stage 1: excavation to 1 m below prop level

- Excavation to 3.1 m depth in ULS analysis and 3.0 m depth in SLS analysis

Stage 2: install prop and excavate to final formation level

- Install prop. Prop stiffness, k = 100 MN/m/m
- Excavate to 8.1 m depth in ULS analysis and 8.0 m depth in SLS analysis

Soil and wall stiffness (all stages)

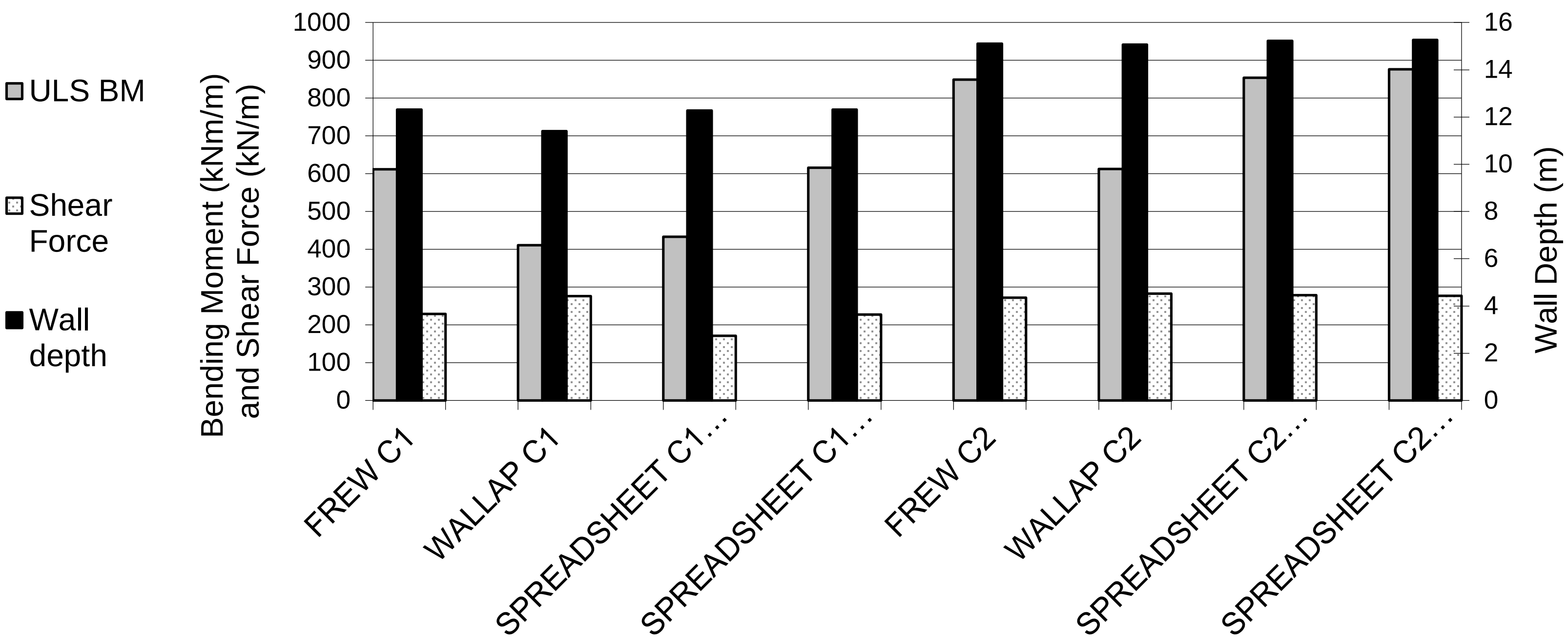
- As for cantilever walls

Finite element analyses

- Modelled ½ a symmetrical excavation of width 32 m
- Mesh 46 m wide × 50 m deep
- Vertical boundaries on rollers; bottom horizontal boundary pinned
- Wall modelled using thin plate elements
- Interface properties "from adjacent soil" with roughness reduction and no tension allowed
- Pore pressures calculated according to steady state seepage in effective stress analyses
- Flooded tension crack and MEFP simulated by the application of pressures to relevant depths

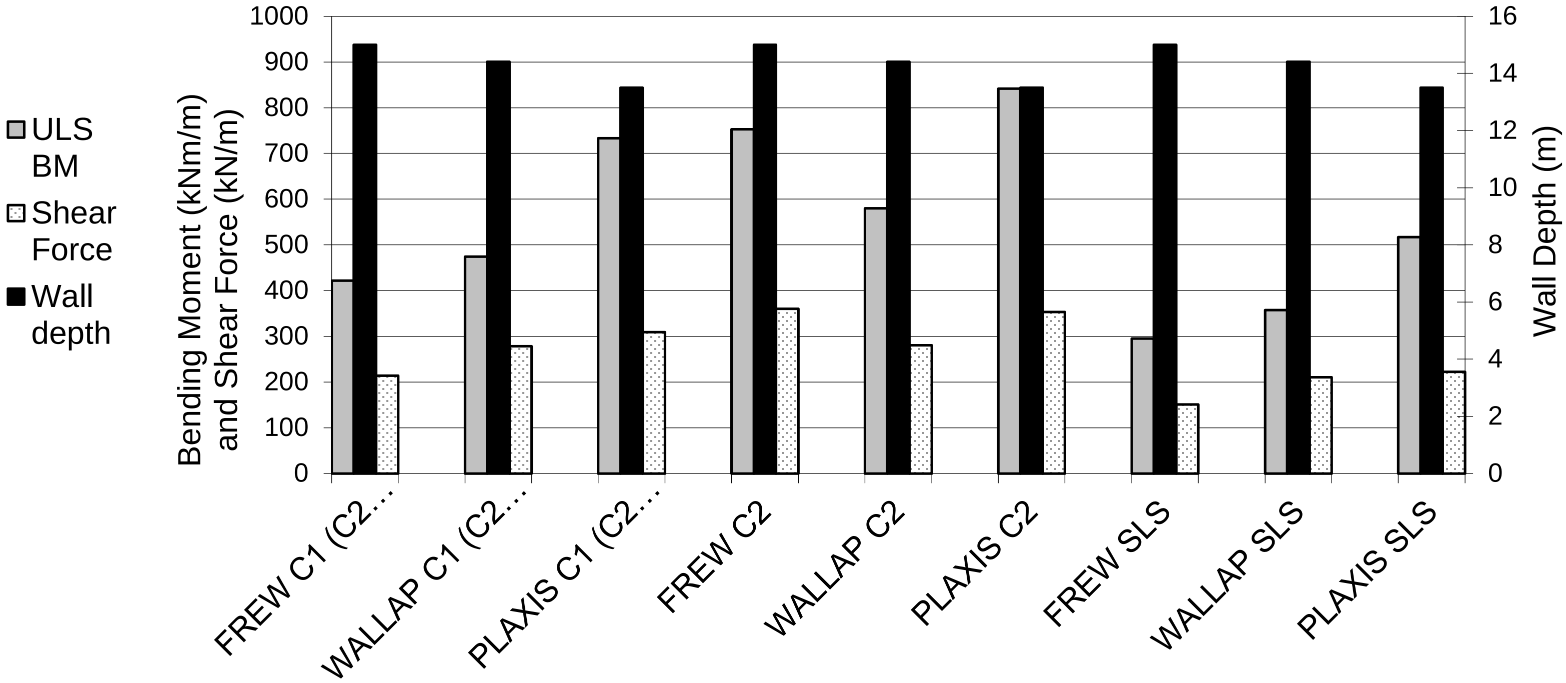
Typical results, Example 2: LEA

Propped wall	**Limit equilibrium**							
Effective Stress	**FREW**	**WALLAP**	**SPREADSHEET**		**FREW**	**WALLAP**	**SPREADSHEET**	**SPREADSHEET**
(see Figure xx)	**DA1 C1**	**DA1 C1**	**DA1 C1 PaP**	**DA1 C1 AaP**	**DA1 C2**	**DA1 C2**	**DA1 C2 PaP**	**DA1 C2 AaP**
Wall depth (m)	12.3	11.4	12.3	12.3	15.1	15.1	15.2	15.3
Max w all BM (kNm/m)	612	411	433	616	849	613	854	876.0
Max w all SF (kN/m)	229	276	171	227	272	283	279	277.0
MaxProp Load (kN)	245	496	300	242	291	478	338	296.0

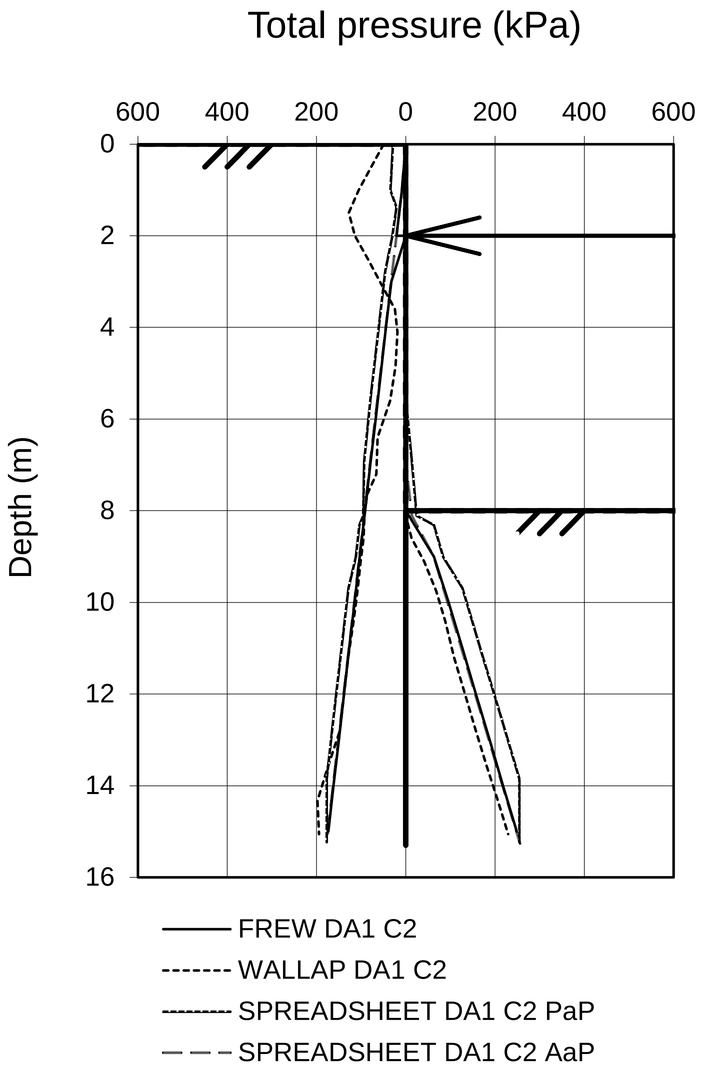


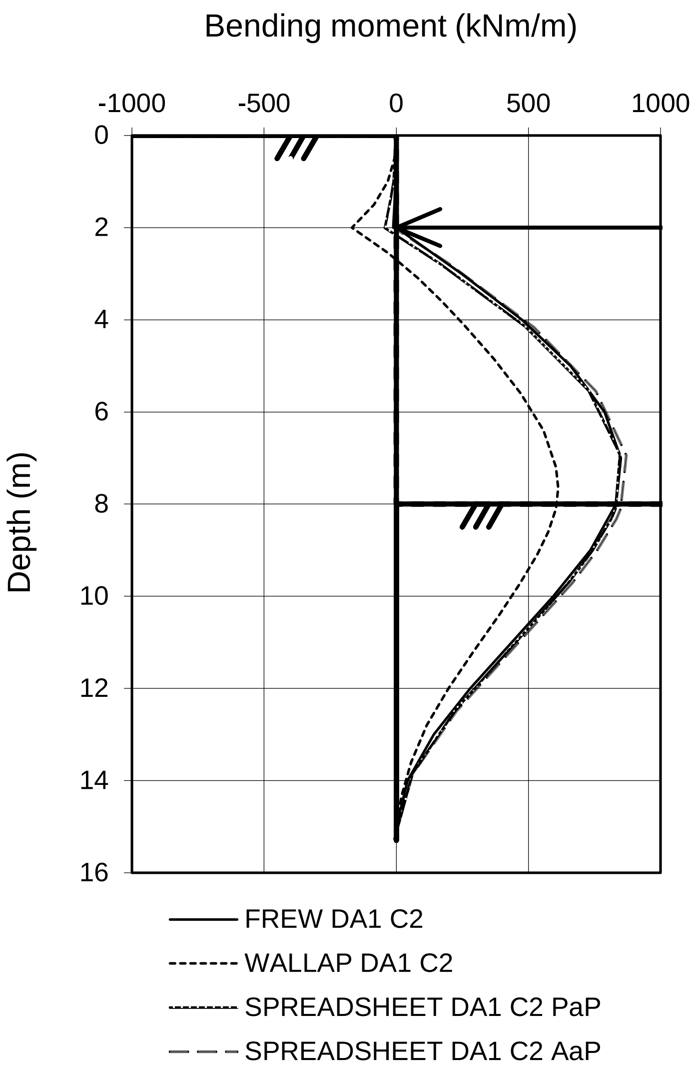
Typical results, Example 2: SSI

Propped wall	**Pseudo-finite element**		**Finite element**	**Pseudo-finite element**		**Finite element**	**Pseudo-finite element**		**Finite element**
Effective Stress	**FREW**	**WALLAP**	**Plaxis**	**FREW**	**WALLAP**	**Plaxis**	**FREW**	**WALLAP**	**Plaxis**
(see Figure xx)	**DA1 C1 (C2 Depth)**	**DA1 C1 (C2 Depth)**	**DA1 C1 (C2 Depth)**	**DA1 C2**	**DA1 C2**	**DA1 C2**	**SLS**	**SLS**	**SLS**
Wall depth (m)	15.0	14.4	13.5	15.0	14.4	13.5	15.0	14.4	13.5
Max wall BM (kNm/m)	422	474	456	753	580	842	295	357	436
Max wall SF (kN/m)	214	278	268	360	280	353	151	210	191
MaxProp Load (kN)	378	493	387	564	476	412	263	367	276

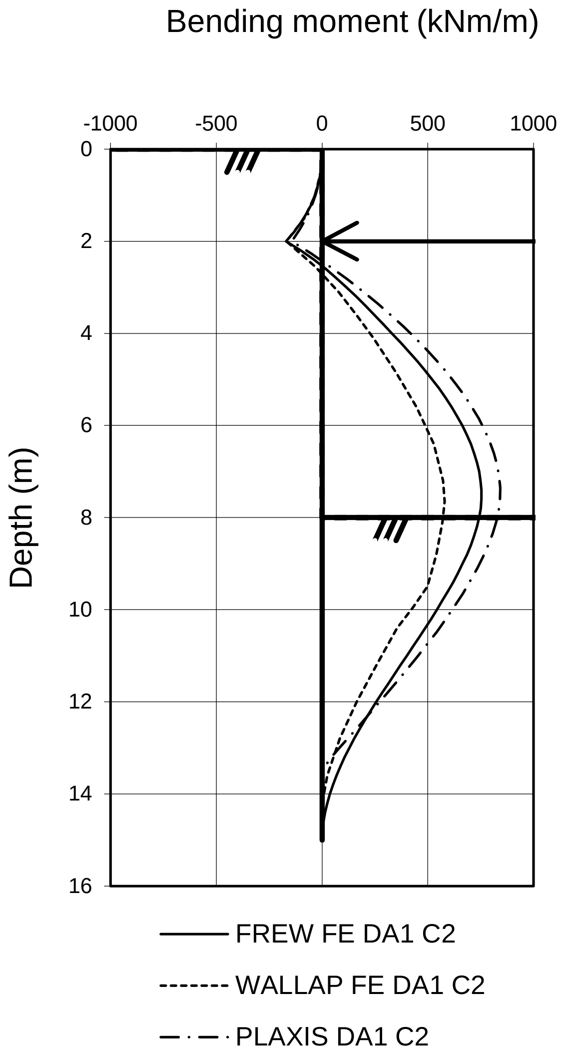


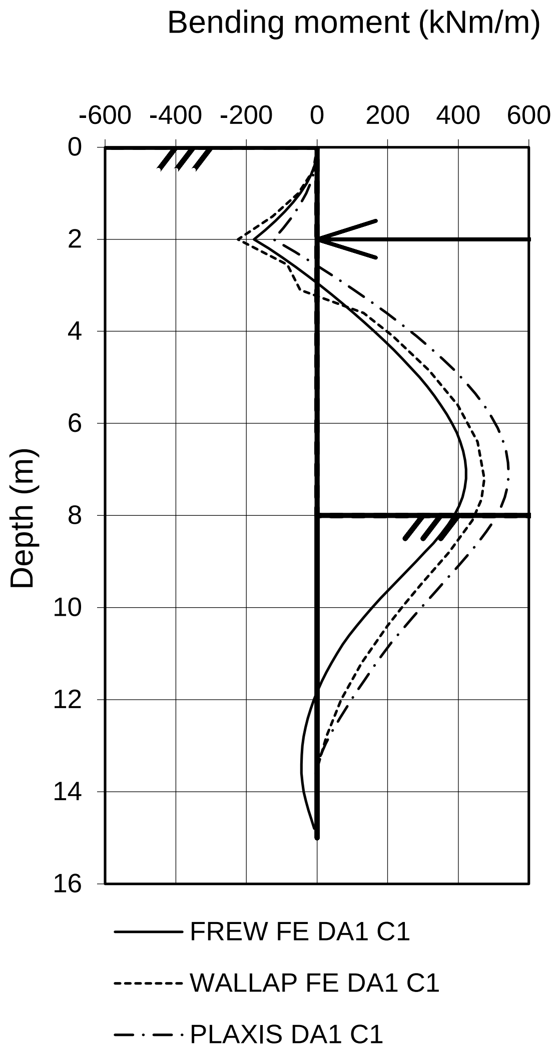
Typical results, Example 2: LEA DA1 C2

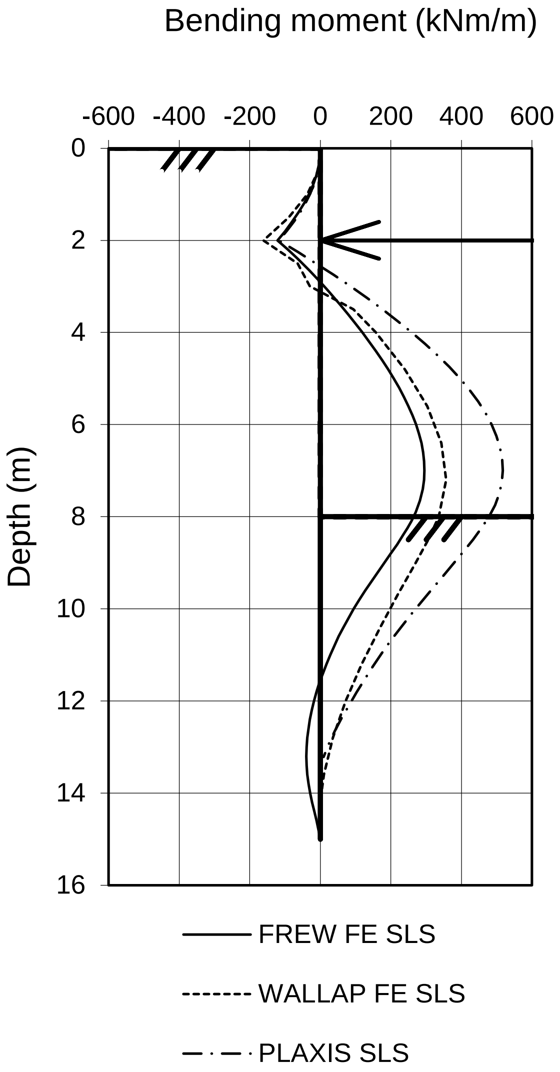




Typical results, Ex 2: SSI Bending Moments







Example calculations: summary

- All methods gave broadly similar embedment depths with differences due to method probably masked by differences in input assumptions
- SSI analyses gave generally lower SLS structural effects (BM & SF) than LEA, especially for propped walls
- Prop loads are sensitive to (assumptions about) the stresses behind the wall above the prop
- Wall embedment by definition governed by C2
- Structural effects greater for C2 than C1 in effective stress analyses, but greater for C1 than C2 in total stress analyses