GEOTECHNICAL ENGINEERING IN THE DESIGN OF STRUCTURES:

Retaining walls

Professor V.N. Georgiannou, MSc, DIC, Ph.D.

NTUA, 9/11/2023

RETAINING WALLS

1. *CONVENTIONAL HEAVY WALLS*

Such walls have an enlarged foundation and they are supported by the large shear stresses developed at the interface between the soil and the foundation due to their large weight.

- mass concrete or stone walls
- reinforced concrete
- hollow walls partly filled with gravels

2. *THIN WALLS* embedded in soil supported by the passive resistance of the soil below excavation level.

- diaphragm wall (e.g. secant pile or contiguous bored pile wall)
- driven sheet pile wall

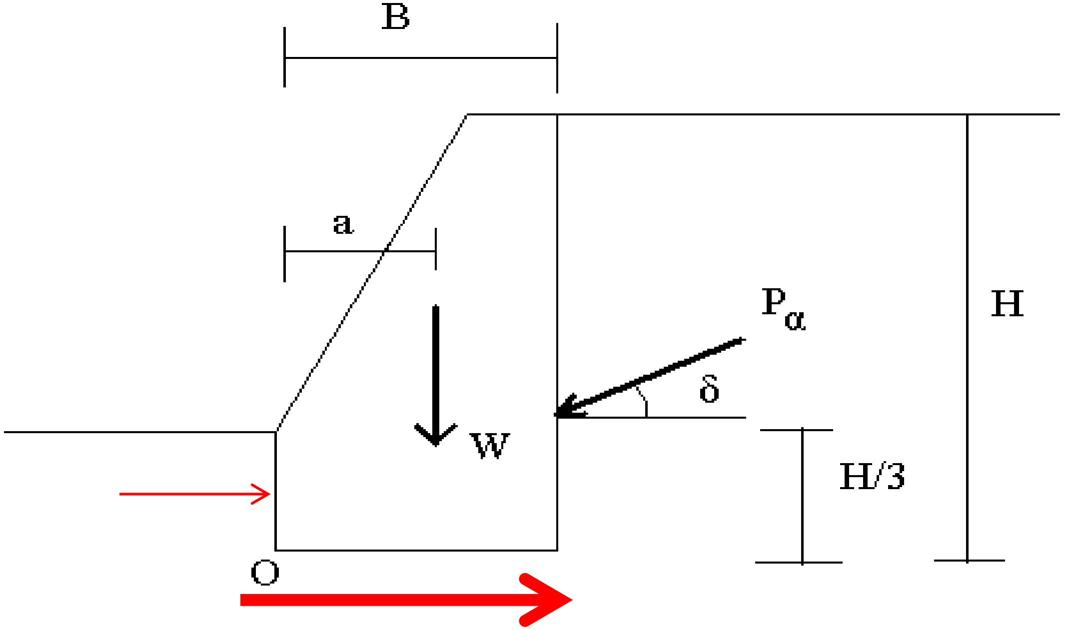
HEAVY WALLS

1. ΕΛΕΓΧΟΣ ΟΛΙΣΘΗΣΗΣ _ 2. ΕΛΕΓΧΟΣ ΑΝΑΤΡΟΠΗΣ

1. SLIDING
2. OVERTURNING

- soil properties:
  φ, c
- angle of shearing resistance at the interface between wall and soil

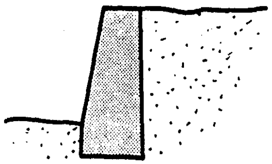
γωνία τριβής τοίχου-εδάφους: δ~0.5-0.75φ



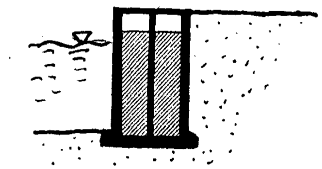
S=Wtanφ

STRUCTURAL FORMS OF RETAINING WALLS

Mass concrete wall



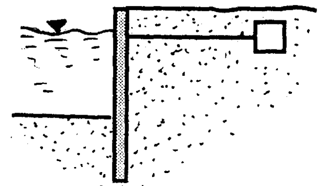
τοίχος βαρύτητας

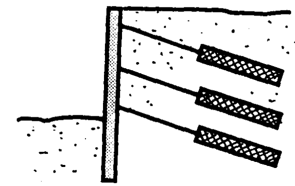


λιμενικός
κρηπιδότοιχος

anchored sheet pile wall

anchored diaphragm wall

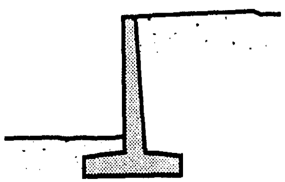




πασσαλοσανίδες ή πασσαλότοιχοι ή διαφράγματα

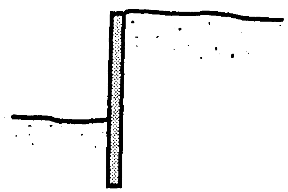
με απλή
αγκύρωση

με προεντεταμένα
αγκύρια



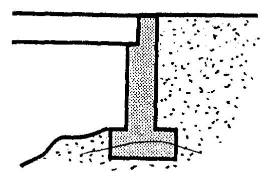
τοίχος πρόβολος
(ωπλισμένου σκυροδέματος)

reinforced concrete cantilever wall



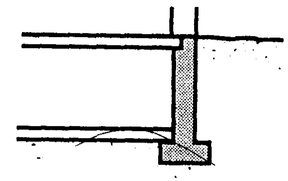
εγκιβωτισμένος
προβολότοιχος

embedded cantilever wall



τοίχος ακροβάθρου
γεφύρας

reinforced concrete bridge abutment

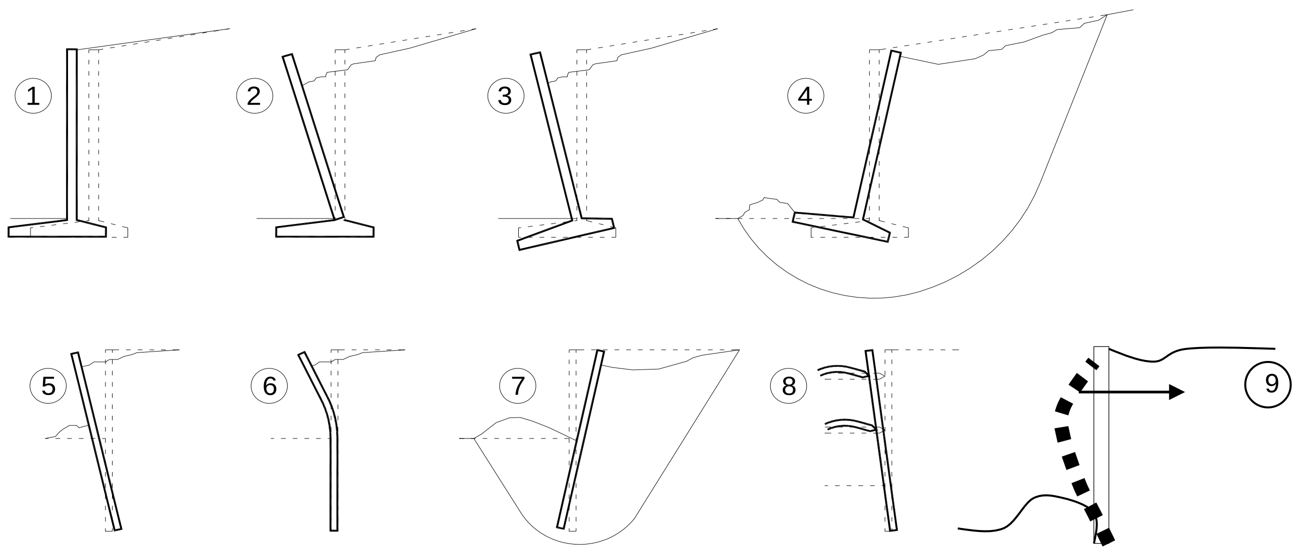


(περιμετρικός)
τοίχος υπογείου

propped wall

The deflections a sheet pile/diaphragm wall may accommodate without overstress are much larger than for a reinforced concrete wall resulting to greater degree of wall soil interaction

POTENTIAL COLLAPSE CONDITIONS of THIN WALLS



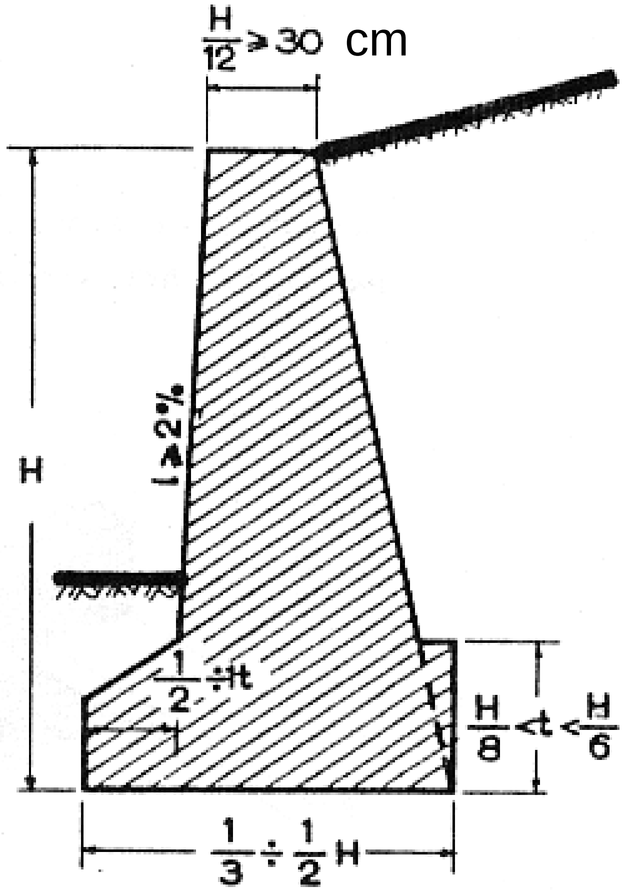
- 1. excessive movement
- 2, 3, 5. forward rotation (cantilever)
- 4, 7. rotation failure of the mass of soil in which the structure is embedded (deep seated slip)
- 8. brittle failure of prop or limited prop yield
- 9. failure of the wall in bending

DESIGN STEPS

- define soil parameters (γ,c,φ), pore water pressure distribution, friction angle at wall-soil interface, δ.
- choose wall type (reinforced concrete, earth, hollow etc.)
- choose foundation level (remove surface material)
- calculate forces and bending moments in the wall
- check factor of safety against sliding, overturning, bearing capacity failure, $q_{ult}=f(N_c, N_\gamma, N_q)$
- check settlements

RETAINING WALLS

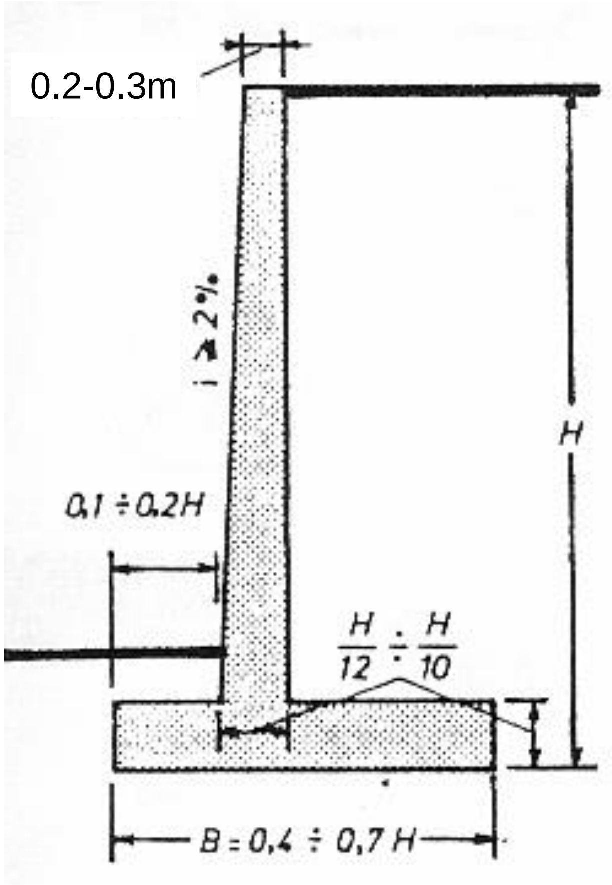
- mass concrete wall



preliminary design – concrete wall

RETAINING WALLS

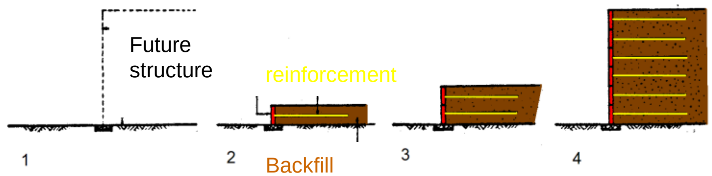
- reinforced concrete wall with footing

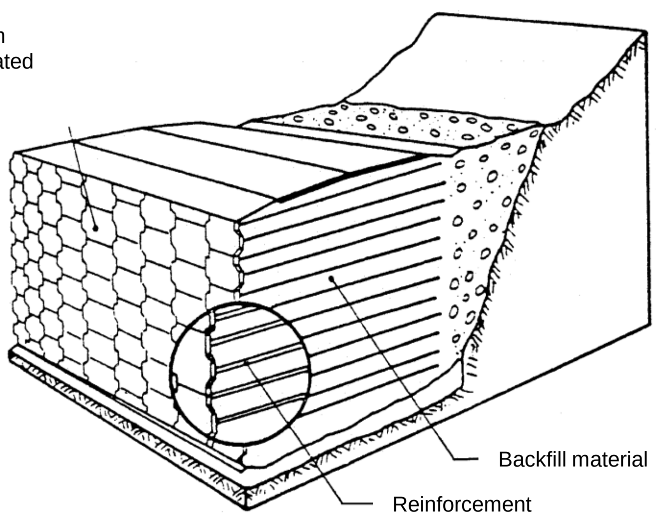


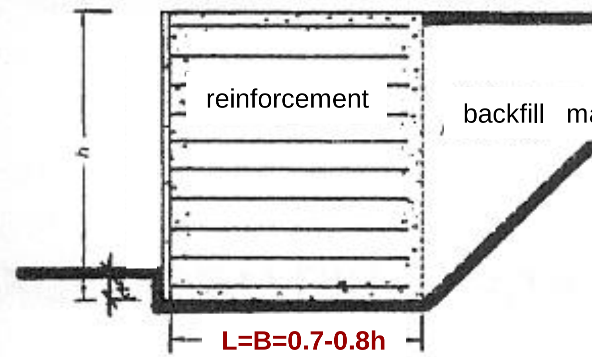
↳ preliminary design

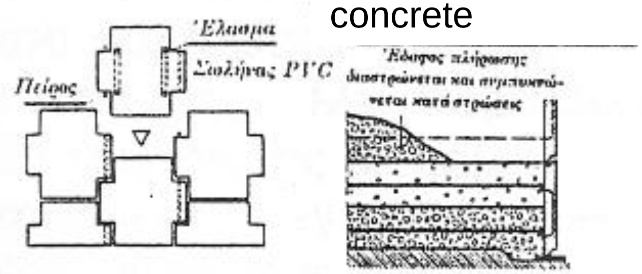
RETAINING WALLS

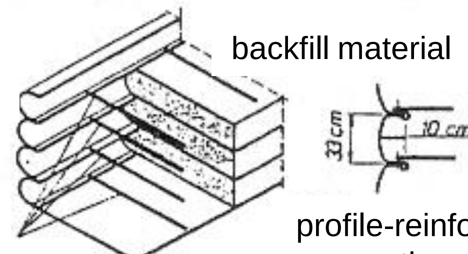
- reinforced earthfill wall





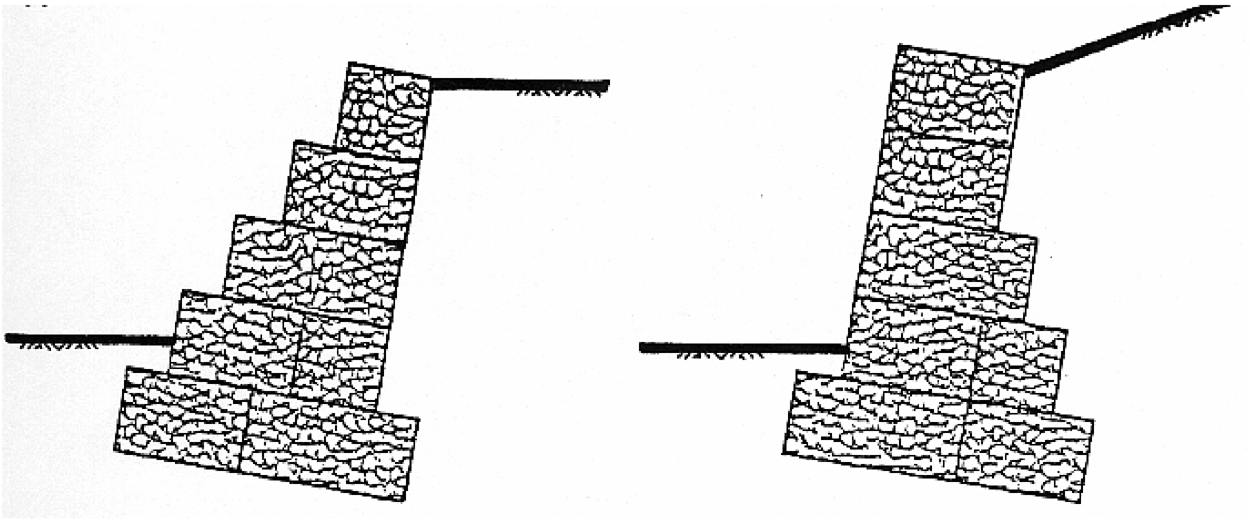






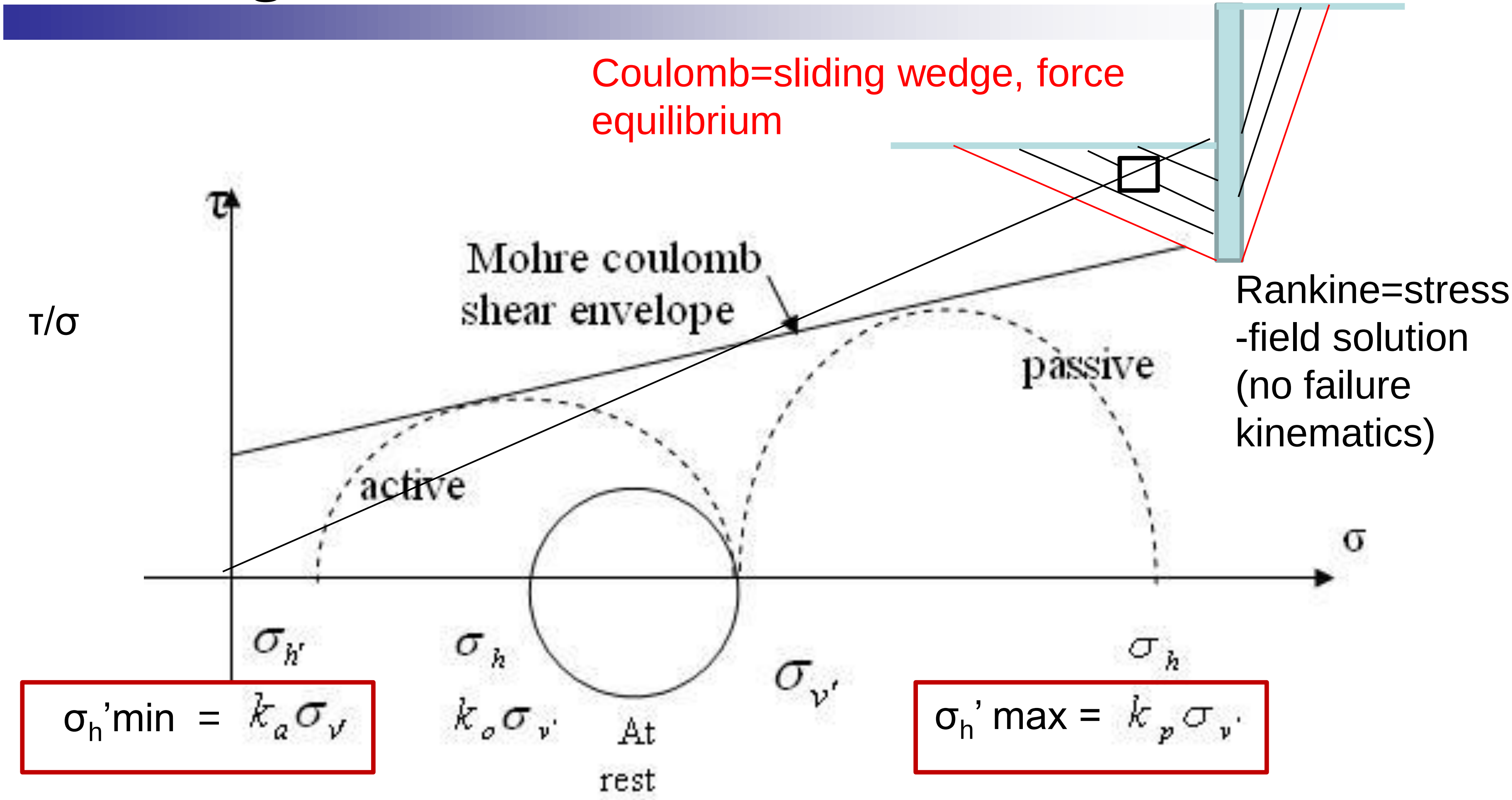
RETAINING WALLS

- encased gravels (συρματοκιβώτιο ή ζαρζανέτι)



wall rotated towards soil mass by 1/10 or 1/6 or 1/4 for wall heights of about 3m

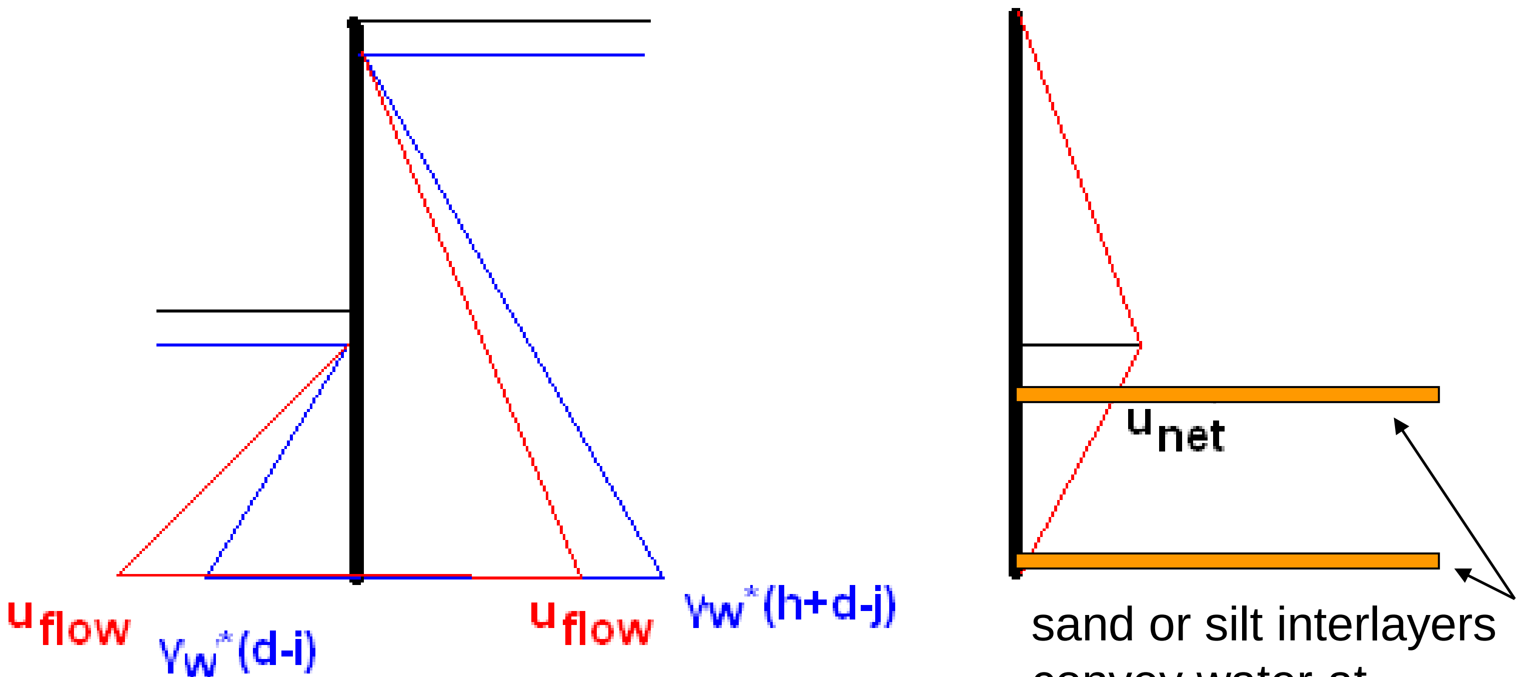
Limiting horizontal stress conditions



DESIGN EARTH PRESSURES

- Wall friction and adhesion: the values of δ/φ' and c_w/c' mobilised are a function of the roughness of the interface and the relevant stress field.
- Effective stress design: the maximum effective wall friction for **active zone** $\delta=2/3\varphi'$, **passive zone** $\delta=1/2\varphi'$, a maximum wall adhesion $c_w=0.5c'$
- horizontal effective earth pressures resulting from soil weight: $\mathbf{p'_a=K_{ac}(\gamma z-u)-2\sqrt{K_{ac}}c'}$ and $\mathbf{p'_p=K_{pc}(\gamma z-u)+2\sqrt{K_{pc}}c'}$, where Kac, Kpc active and passive pressure coefficients [Caquot & Kerisel (logarithmic spiral), etc] for a horizontal surface to the retained material and various δ/φ' values
- $Pa'=-2c'\sqrt{K_{ac}}$ at h=0 negative active pressure; tension cracks develop in a zone of $h_{crit}=2c'/\gamma'\sqrt{K_{ac}}$ The negative pressure is balanced by the same positive pressure over the same depth below. Hence the resultant active pressure is zero at $H_c=2h_{crit}$. For undrained conditions $\mathbf{H_c=4Su/\gamma}$. Use of $c'>5kN/m^2$ in the retained soil results in a significant depth of theoretical negative active effective pressure.
- 1. reduce c' towards the surface to avoid this (realistic due to weathering)
- 2. assume that effective pressure on the wall at any depth should not be less than 5*z kN/m^2

DESIGN WATER PRESSURE

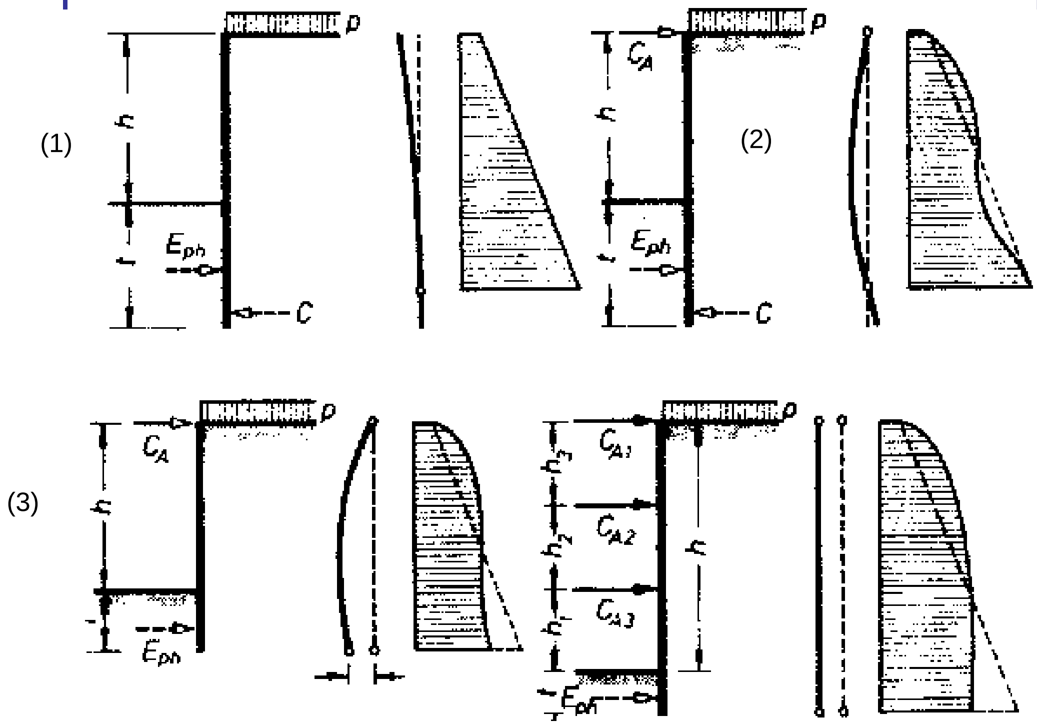


sand or silt interlayers convey water at hydrostatic pressure to base of wall

(a) Gross water pressures

(b) net water pressures

EARTH PRESSURES – WALL FLEXIBILITY AND LOAD REDISTRIBUTION



supports:

- props, struts
- anchors
- pre-stressed anchors

Supports move earth pressure distribution diagram upwards

(1) embedded cantilever wall, triangular earth pressure distribution

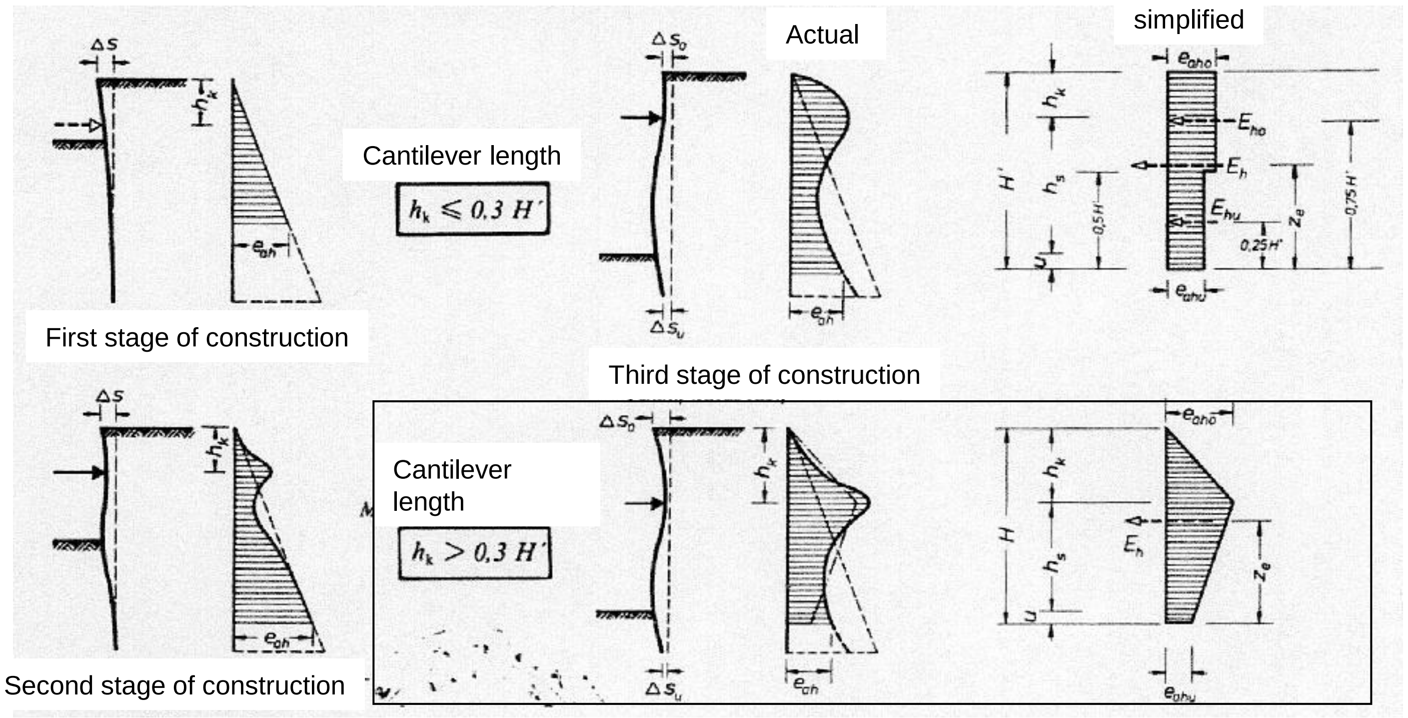
$E_\alpha=0.5*\gamma*k_\alpha*h^2-2c*\sqrt{k_\alpha}*h+p*k_\alpha*h$

(2), (3) single prop, wall deforms in the middle

(4) multi propped wall, horizontal displacement

Influence of wall flexibility on pressure distribution

Simplified active earth pressure distribution for a sheet pile wall

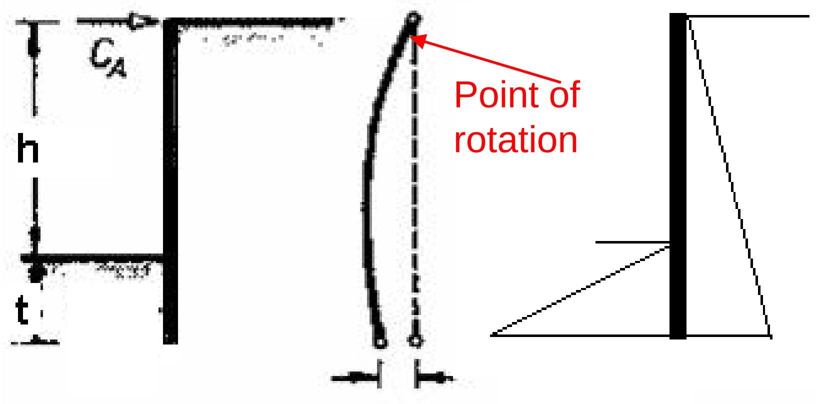


Supports move active earth pressure distribution diagram upwards

Influence of wall flexibility on pressure distribution

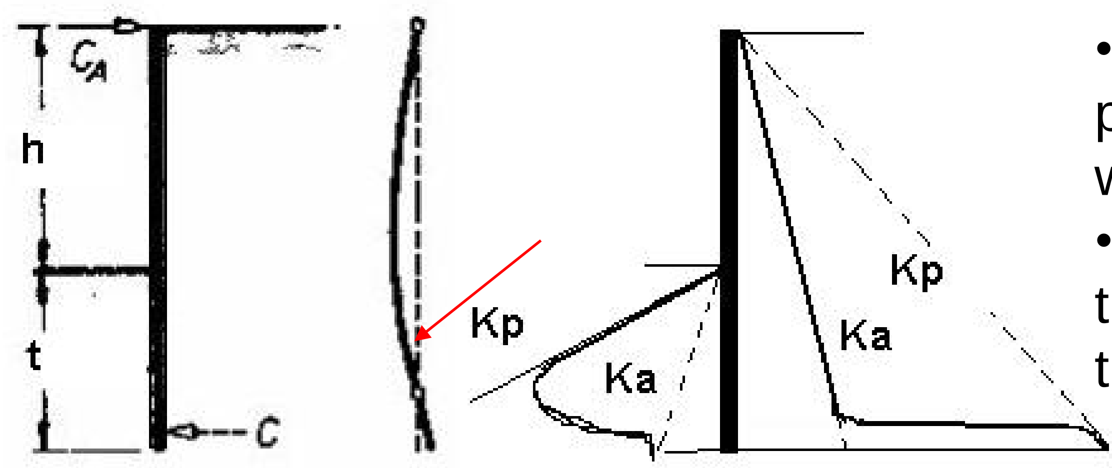
FREE-EARTH AND FIXED-EARTH CONDITIONS

Free earth support conditions- Propped wall



- not sufficient embedment to prevent movement of the toe of the wall
- wall in equilibrium with idealized pressure distribution shown

Fixed earth support conditions- Propped wall



deflected shape earth pressure distribution

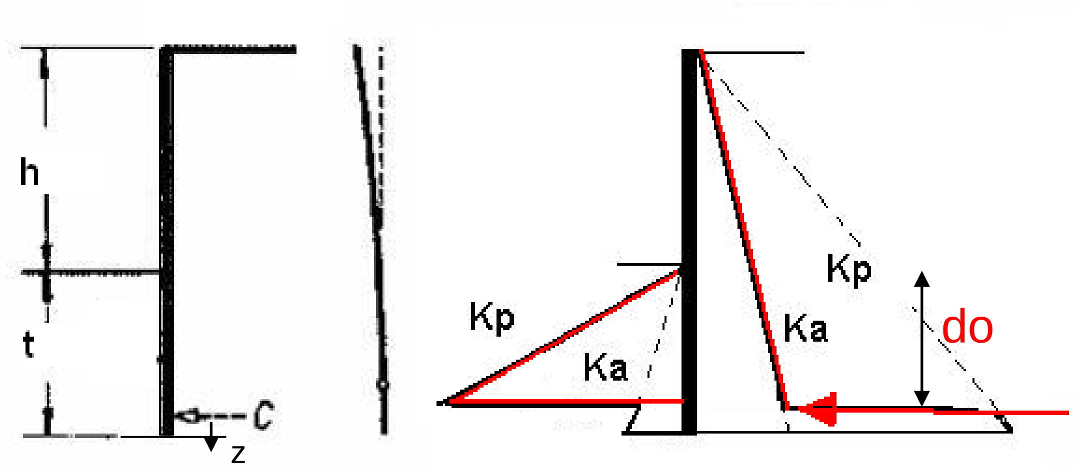
- increased embedment, below point C rotation of the toe of the wall becomes negligible
- a fixing moment is provided by the large reaction at the back of the wall close to its toe

Overall stability checks in terms of rotation about the prop only applicable to free-earth conditions. No failure mechanism relevant to fixed-earth. Reduction of design bending moment, deeper embedment though.

FREE-EARTH AND FIXED-EARTH WALLS -LIMITING CONDITION (on the point of collapse)

• for a **propped** wall, the failure mechanism considered in overall stability calculations is rotation about the position of the prop, assuming free-earth support conditions
• depth of embedment determined by taking moments about the position of the prop

Fixed earth support conditions- cantilever wall

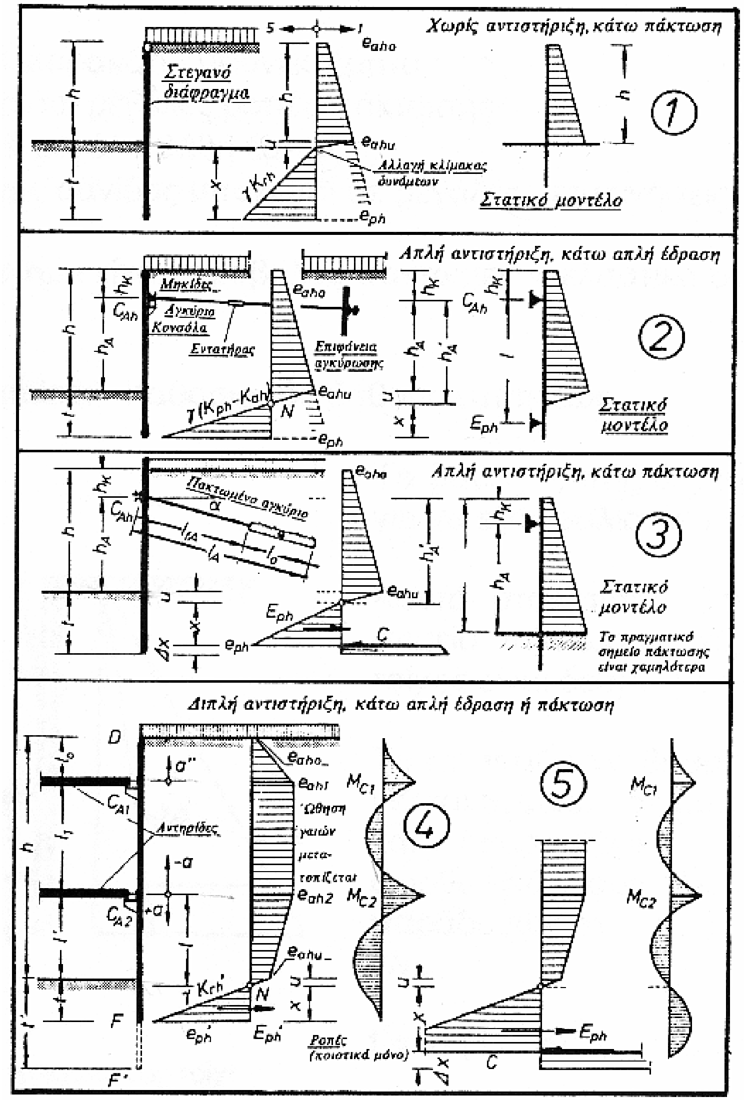


deflected shape earth pressure distribution

• for **cantilever** wall the theoretical pressure distribution is of the fixed-earth form
• at point of rotation full passive pressure at the front of the wall and full passive pressure behind the wall is assumed
• equating horizontal forces and taking moments about C defines t and z (2 equations, 2 unknowns)
• simplification: take moments about C to define do

SIMPLIFIED PRESSURE DISTRIBUTION AT LIMITING CONDITION

(1).
embedded cantilever wall
(2).
propped free-earth support
(3).
propped fixed-earth support
(4).
multi propped free-earth support
(5).
multi propped fixed-earth support



SHEET PILES

Removable thick sheets driven by crane mounted pile drivers

DRIVEN SHEET PILE WALLS

REQUIREMENT: soil strength allows driving process

➡ via hydraulic pressure or blows

- water tight sheet pile walls

➡ metal profiles interfacing to seal against water

➡ profiles can be reused

➡ applications to narrow lanes, close to houses, sewage works

Such walls installed from ground surface are suitable next to existing structures

-the diaphragm wall technique ensures small displacements and settlement

1. Secant piles or contiguous piles are formed in boreholes

   secant pile wall: intersecting piles

   contiguous pile wall: small gaps between adjacent piles

2. The boreholes are cased or filled with liquid (bentonite) before being filled with concrete to avoid soil disturbance

3. Diaphragm walls contrary to driven sheet pile walls have greater stiffness and bending moments

Types of Diaphragm walls

Secant pile wall

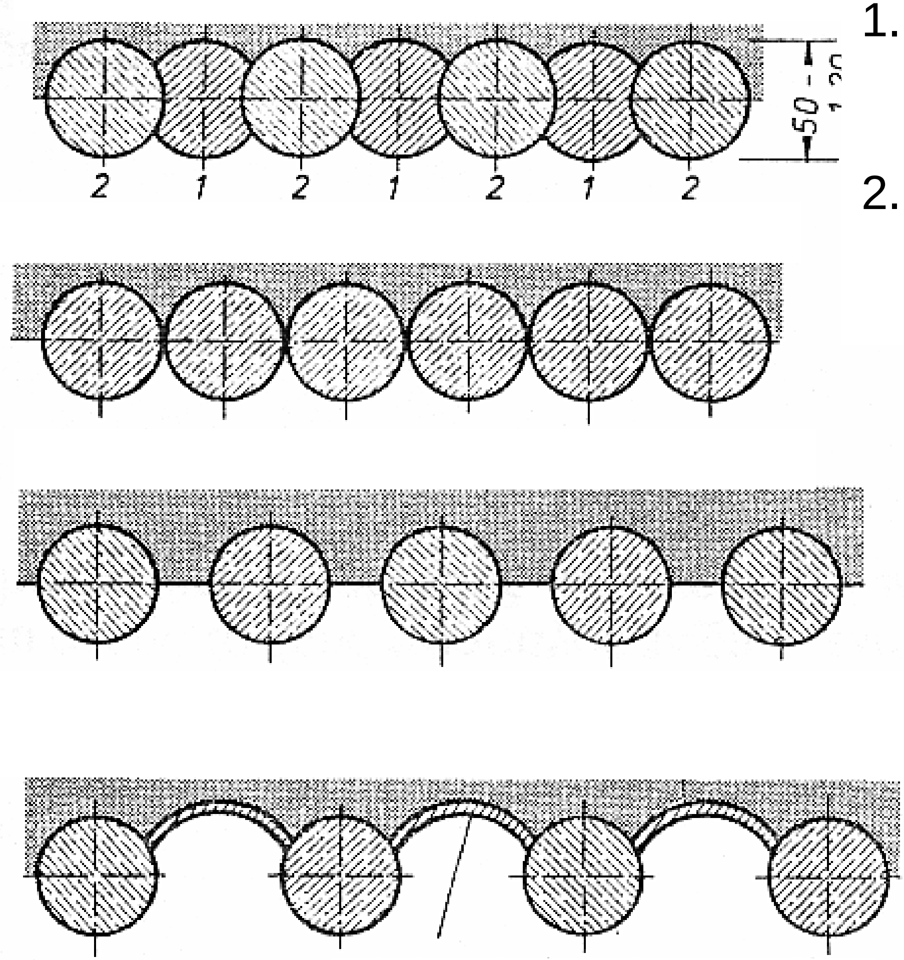
$0<a<1D$,

a=distance between piles, D=pile diameter

Contiguous pile wall

$1D<a<2D$

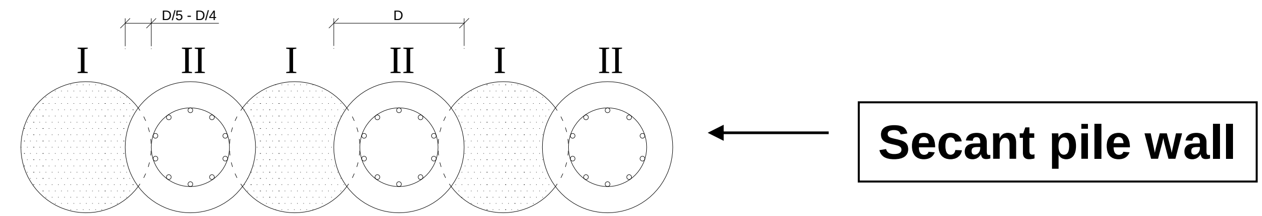
- $a>2D$

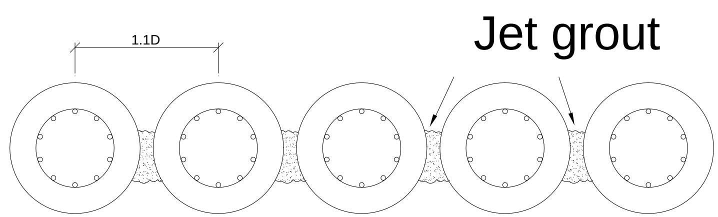


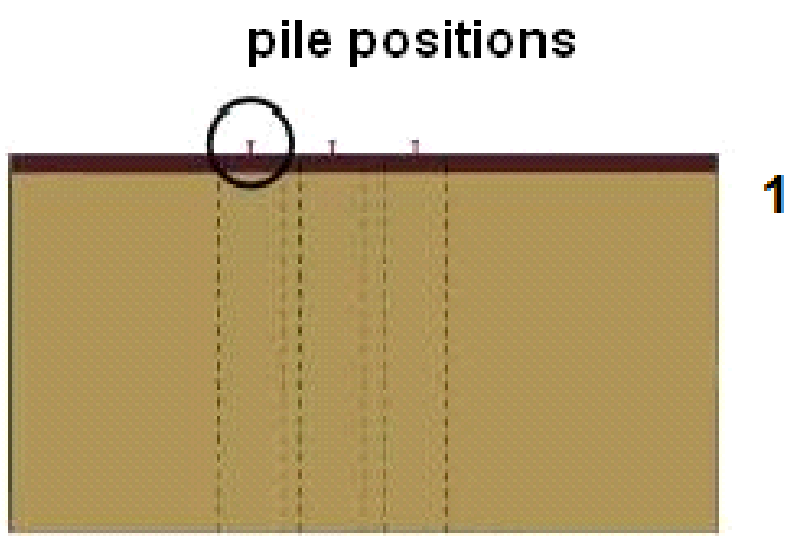
Concrete sprayed lining

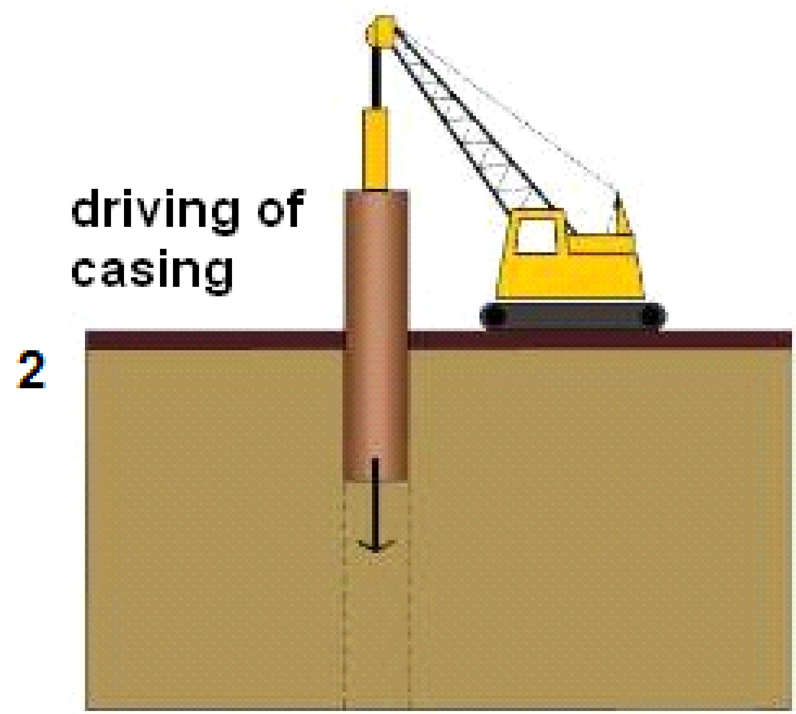
1. Conctere piles formed first
2. Reiforced concrete piles follow

Diaphragm wall construction

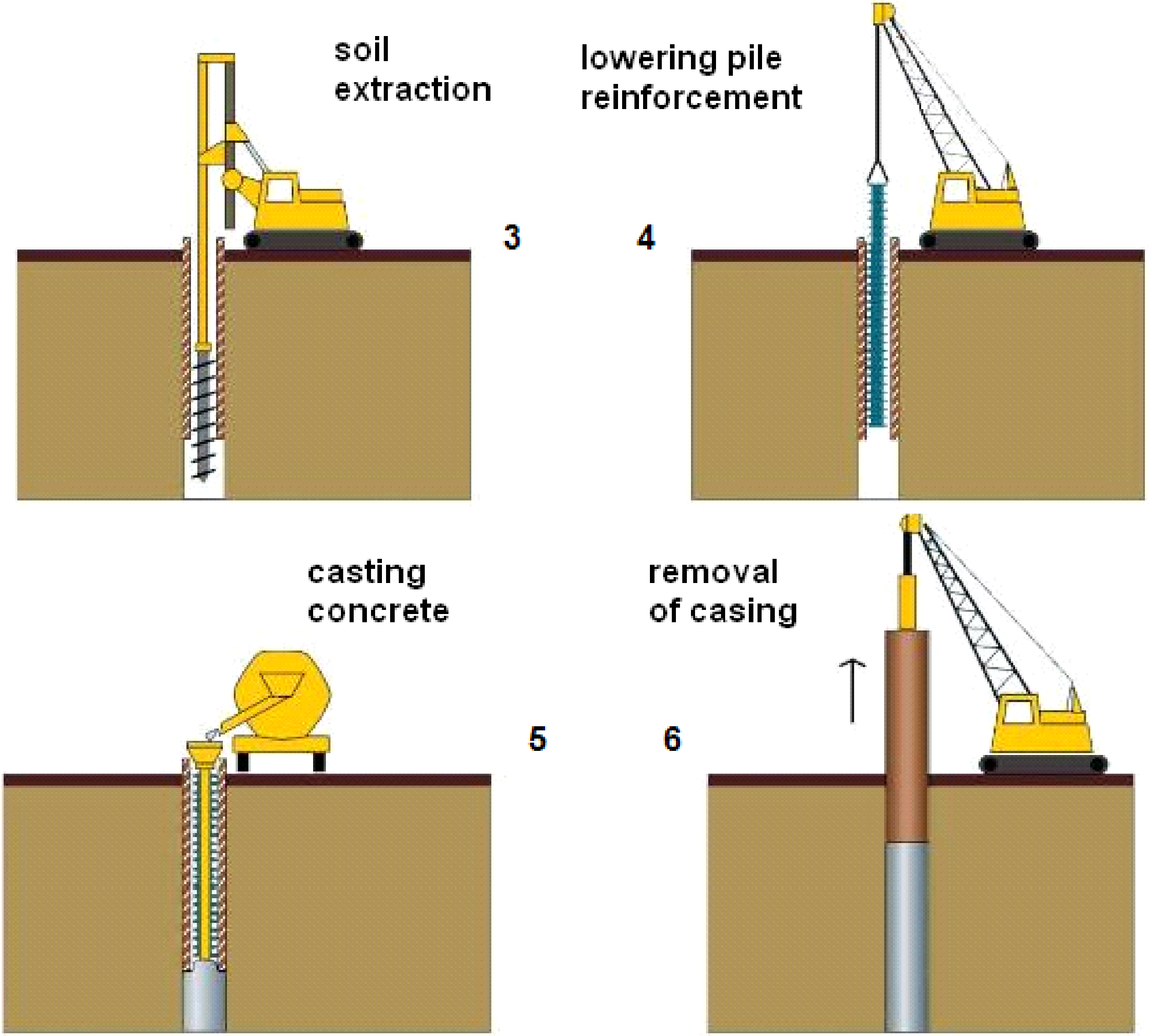




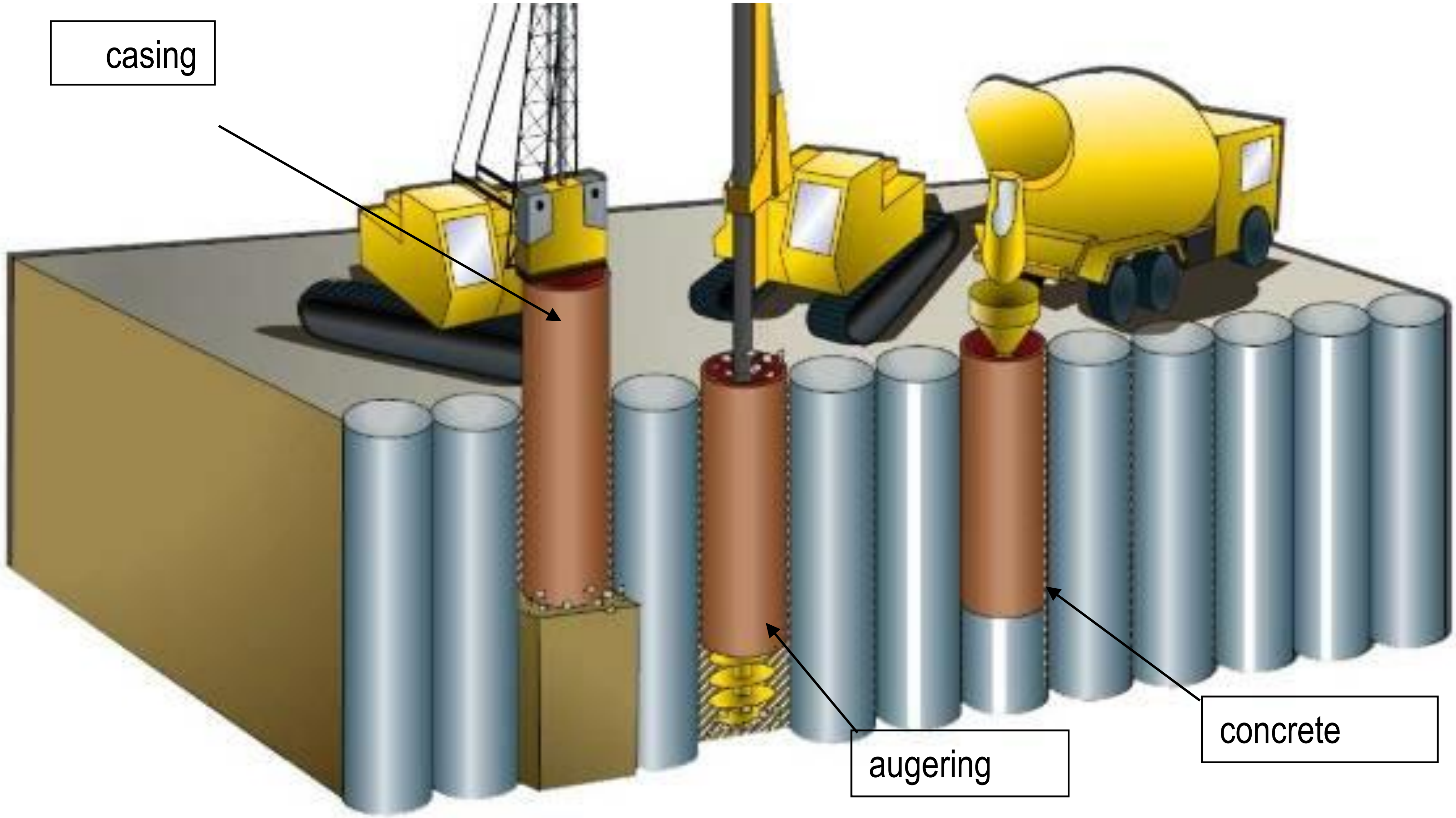




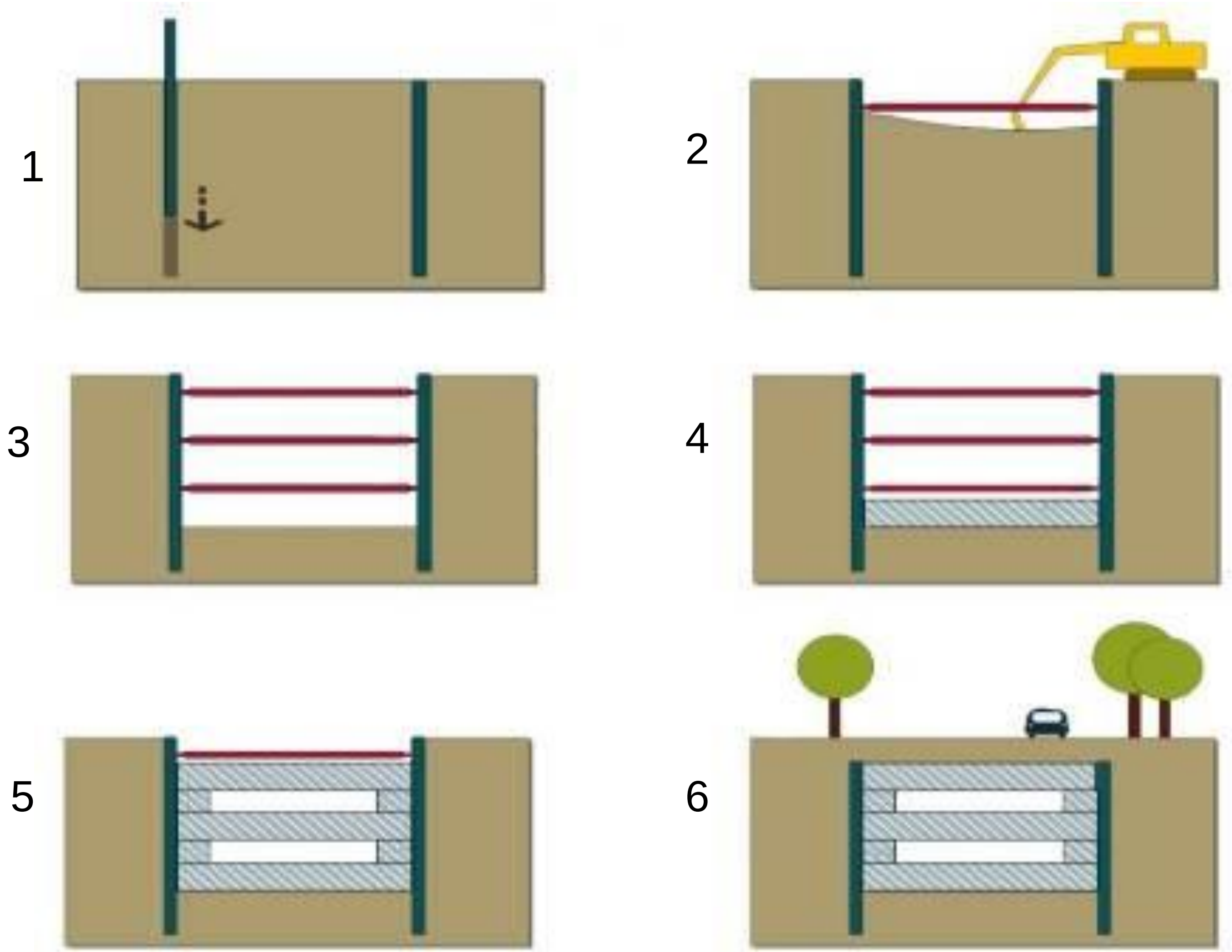
Diaphragm wall construction



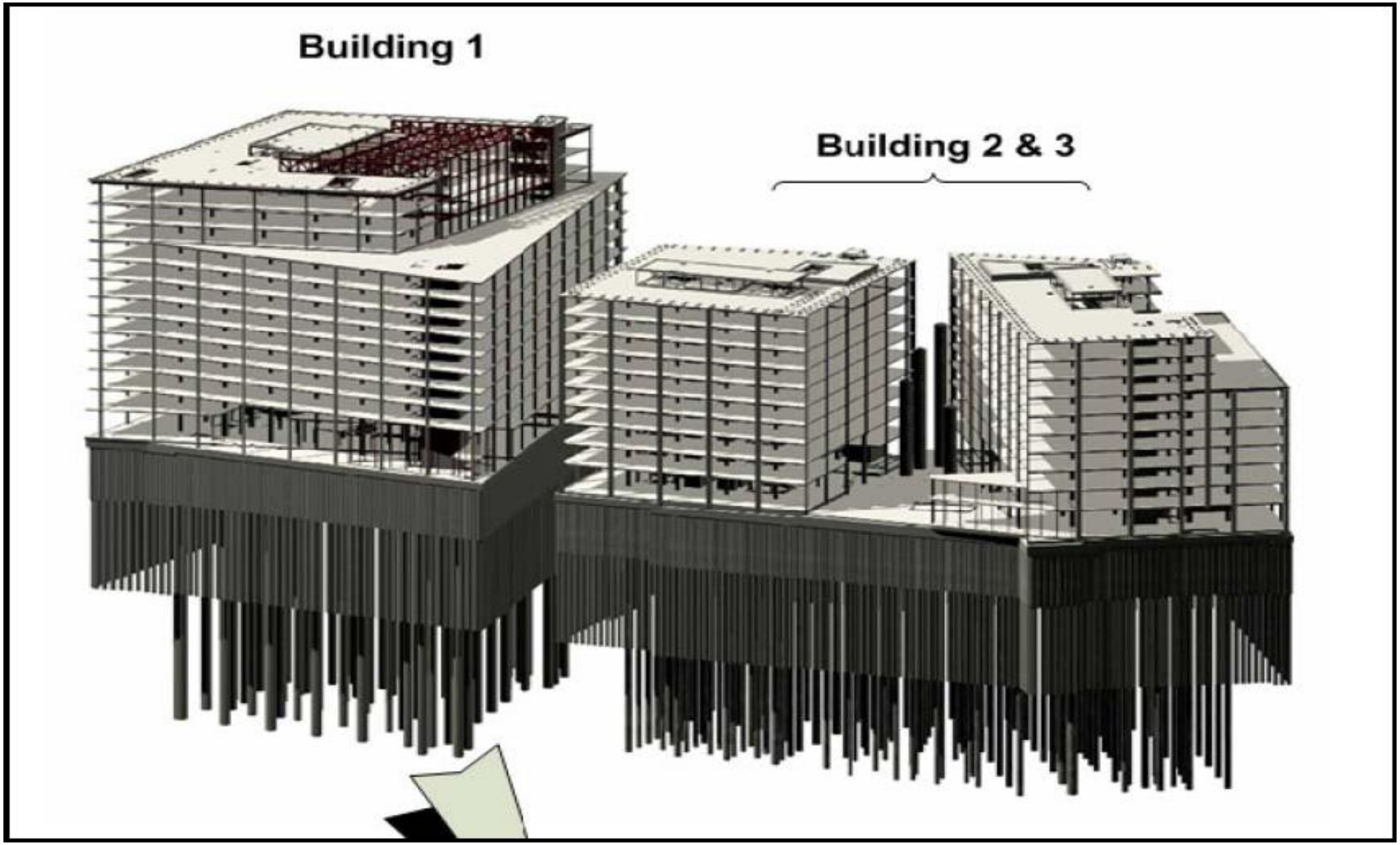
Diaphragm wall line of construction



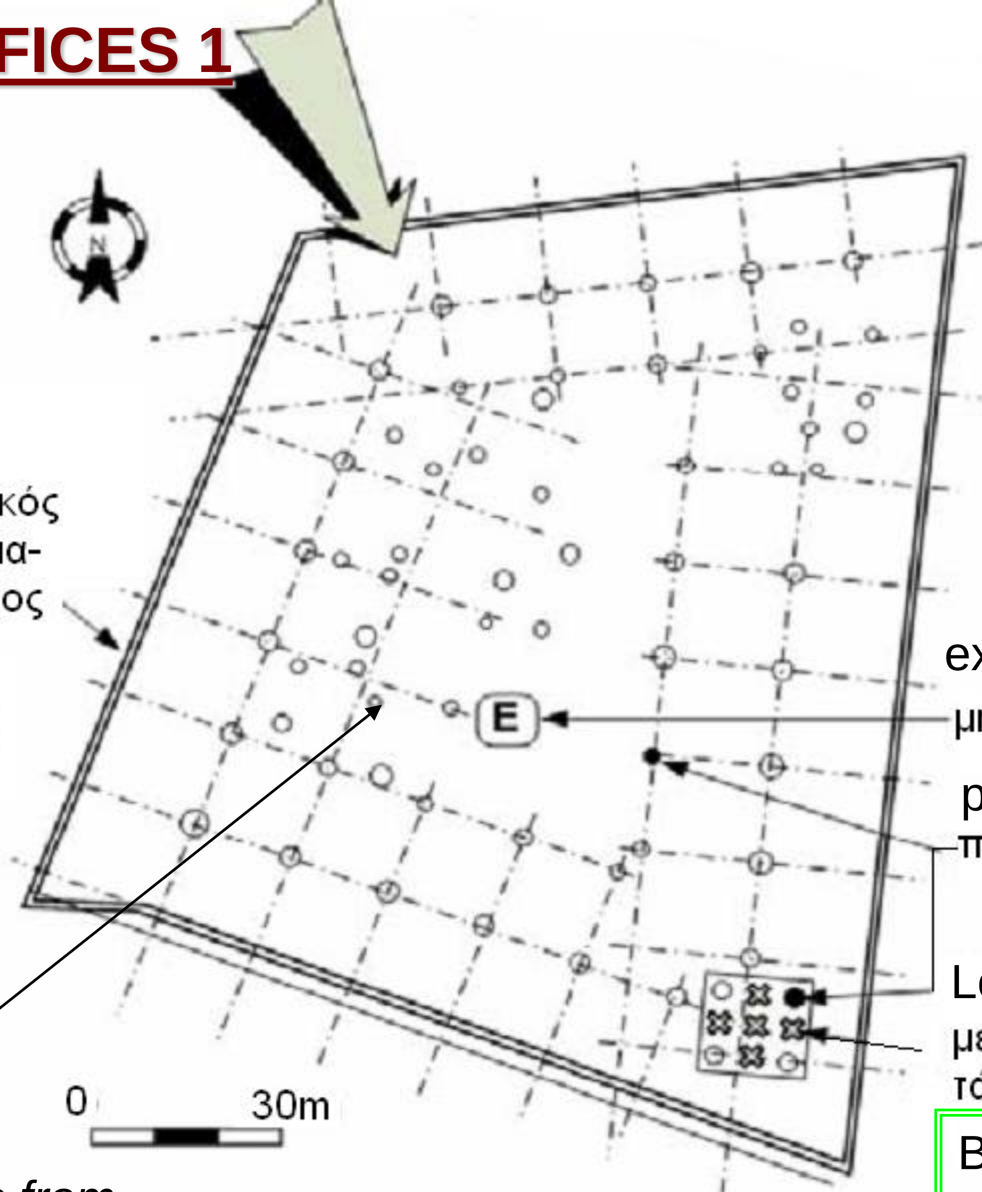
MULTI-PROPPED RETAINING WALLS



PILES



OFFICES 1



περιμετρικός διαφραγμα-τικός τοίχος

(secant pile wall)

extensometer

μηκυνσιόμετρο

piezometer

πιεζόμετρα

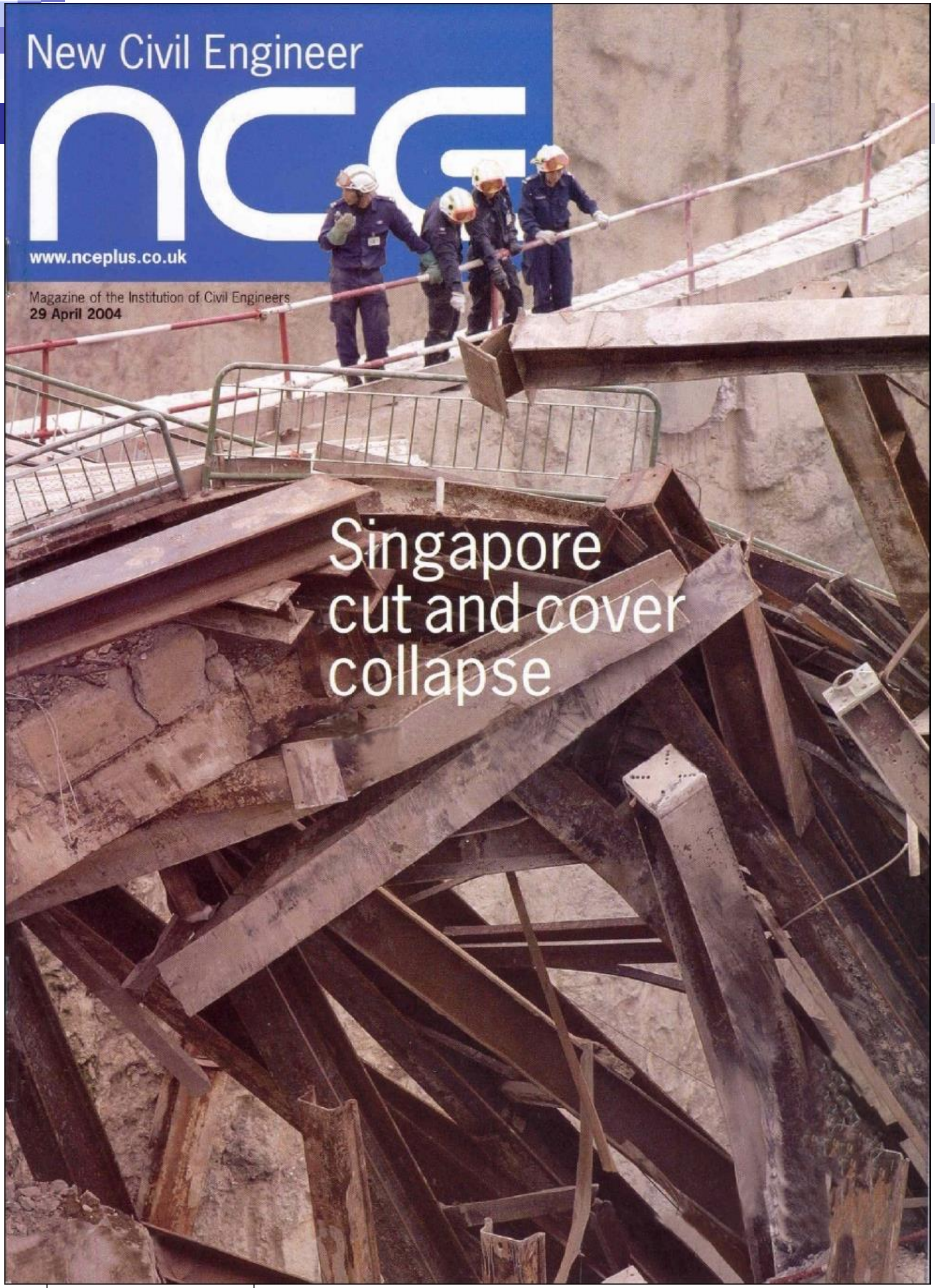
Load cells

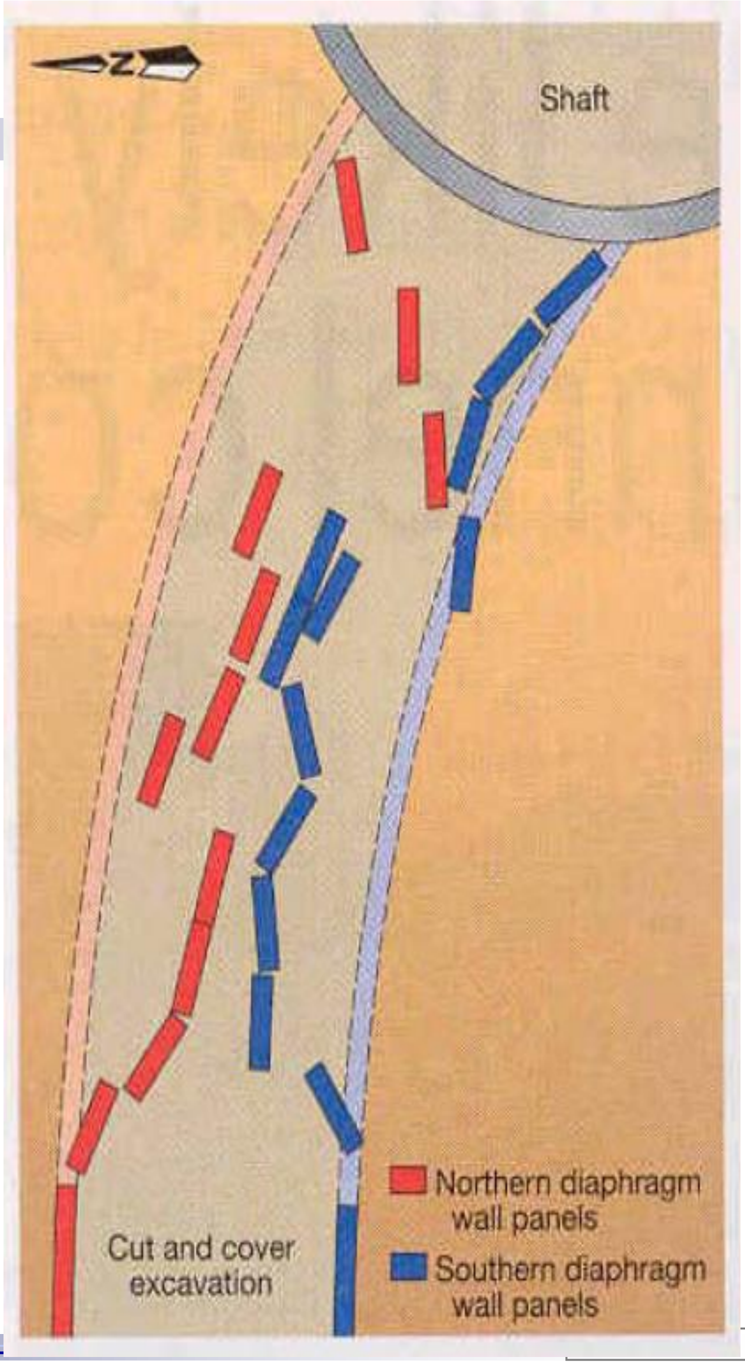
μετρητές τάσεων

Micro-piles from previous structure

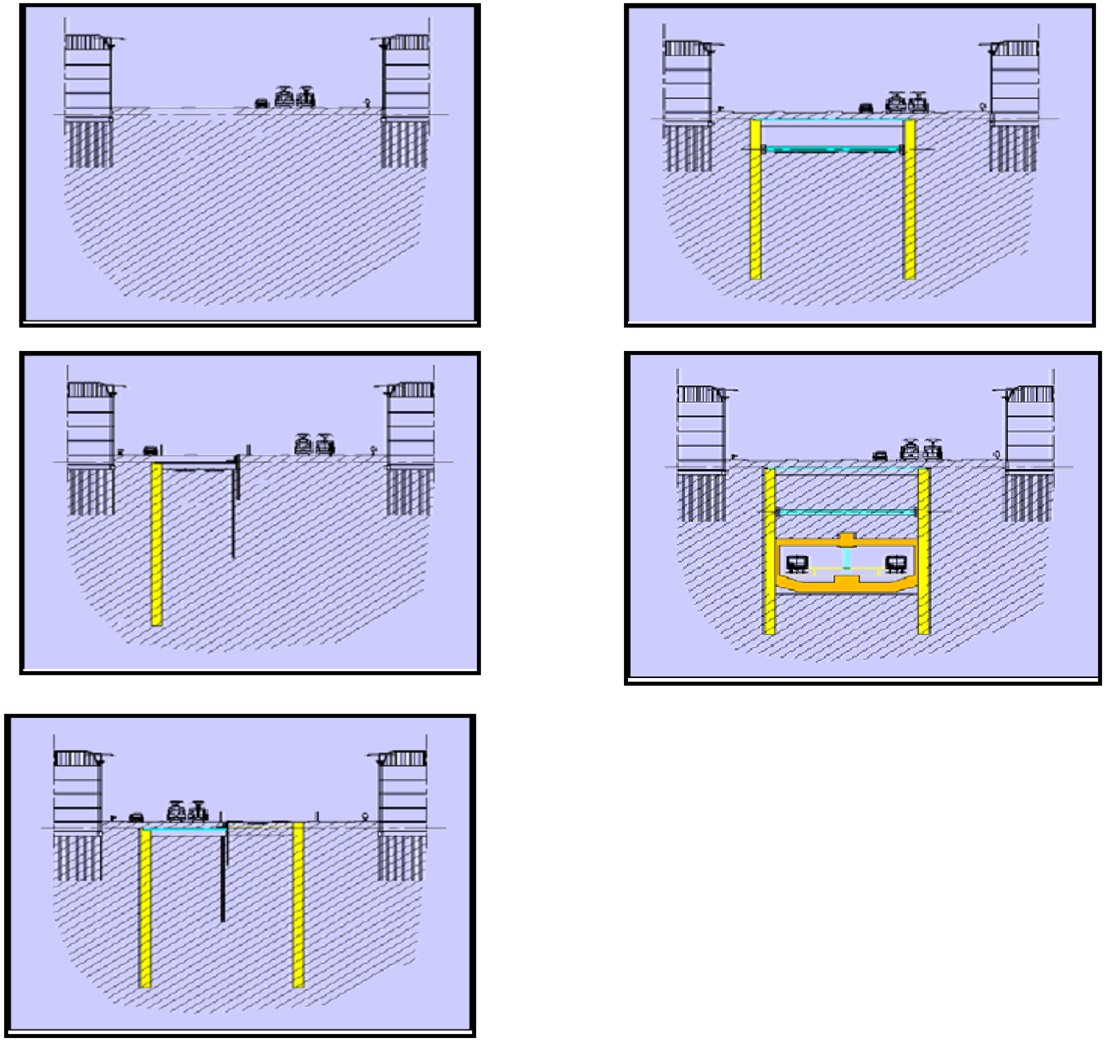
70000 m^2 area
12 storey building
3 levels underground
9m x 9m pile network
200m diaphragm wall

Boreholes filled with bentonite
D = 1.2-2.4m (diameter)
L = 53m (length)
3-stage construction


New Civil Engineer
nce
www.nceplus.co.uk
Magazine of the Institution of Civil Engineers
29 April 2004
Singapore
cut and cover
collapse


N
Shaft
Cut and cover
excavation
Northern diaphragm
wall panels
Southern diaphragm
wall panels

Underground station construction

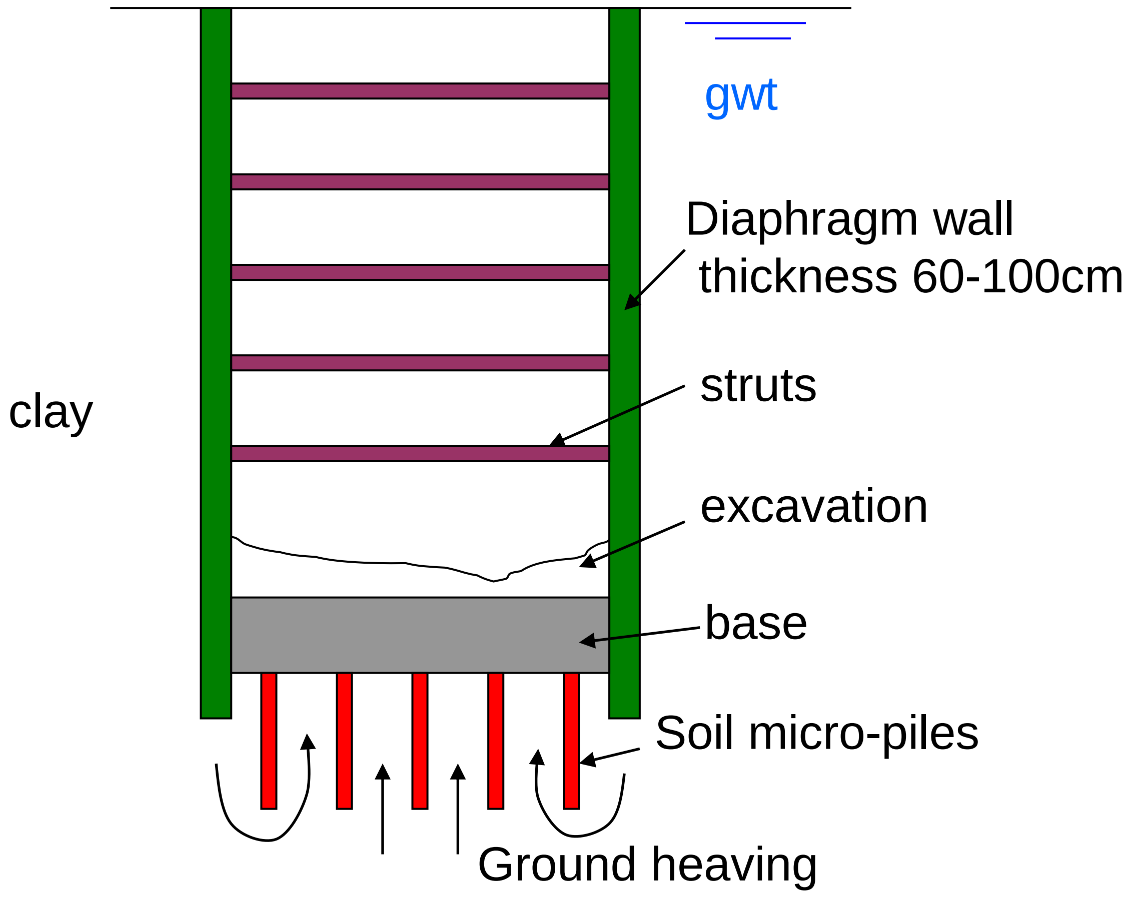


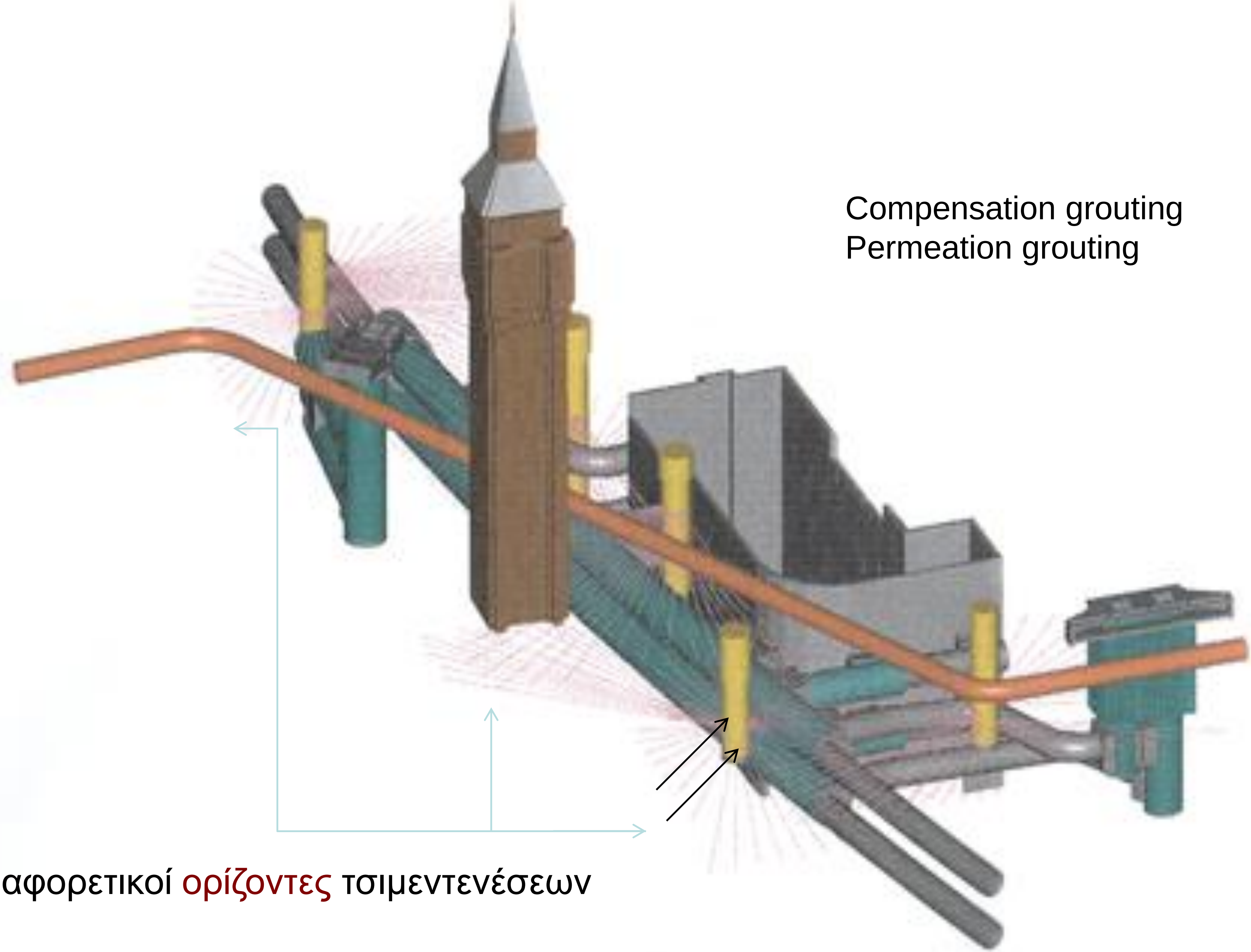


νοικιασε ...
mateco


03 09 2015

Soil heaving





διαφορετικοί ορίζοντες τσιμεντενέσεων

THIN WALLS EMBEDDED IN THE SOIL

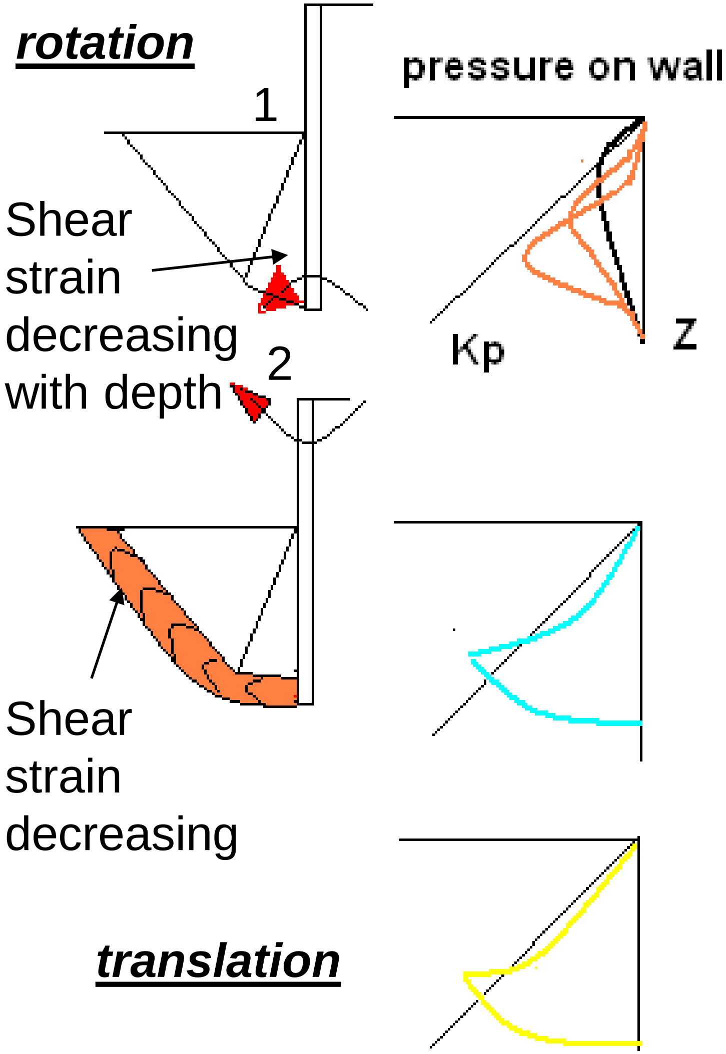
The behaviour of the wall is dependent on the interaction of soil and wall

1. Limit equilibrium methods e.g. Weissenbach, Brom, Blum
2. Elastic methods e.g. Sherif, Wemer

1. Soil yielding along the wall depth does not simulate working conditions (displacements?)
2. Elastic solution under working conditions provides a good simulation of the earth pressure coefficients

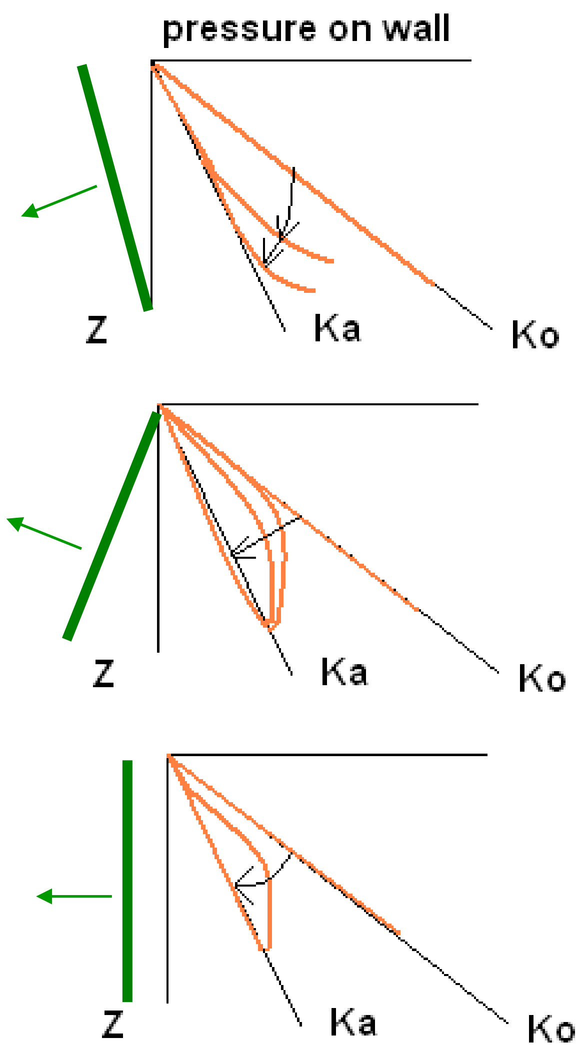
Passive pressure distribution with wall movement

- Rotation about the **toe** of the wall mobilizes the peak stress ratio near the top. With further rotation the soil near the top deforms past peak, while material lower down mobilises peak strength.
- Rotation about the top of the wall causes the shear to be concentrated in a band simulating wedge sliding.
- Wall translation resembles the linear pressure distribution developed by Rankine.



Active pressure distribution with wall movement

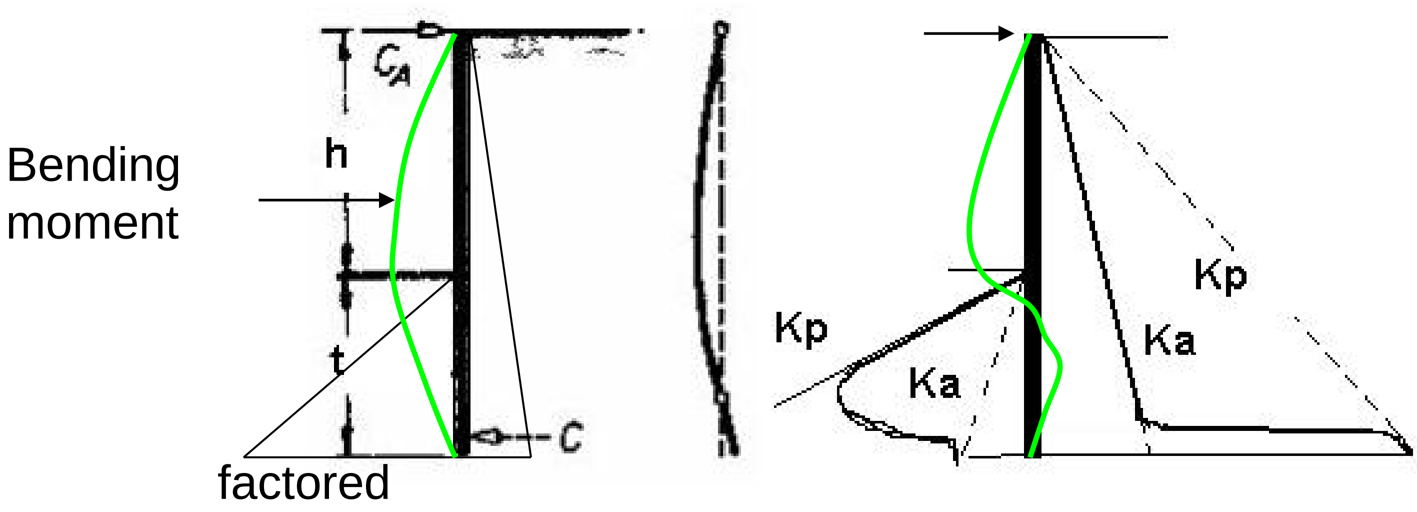
- Rotation about the **toe** of the wall mobilizes the peak stress ratio near the top. At large deformations earth pressure approaches a straight line distribution (Ka or Kp in previous slide)
- Rotation about the top of the wall mobilises active stresses near the bottom of the wall.
- Wall translation resembles the linear pressure distribution developed by Rankine.



LIMIT STATE DESIGN

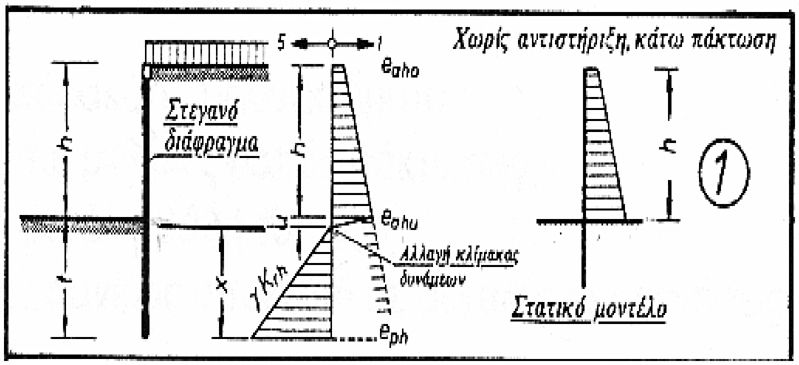
- SERVICABILITY LIMIT STATE DESIGN: ensure a specified threshold deformation is not exceeded and stresses applied to the construction materials will not affect their durability

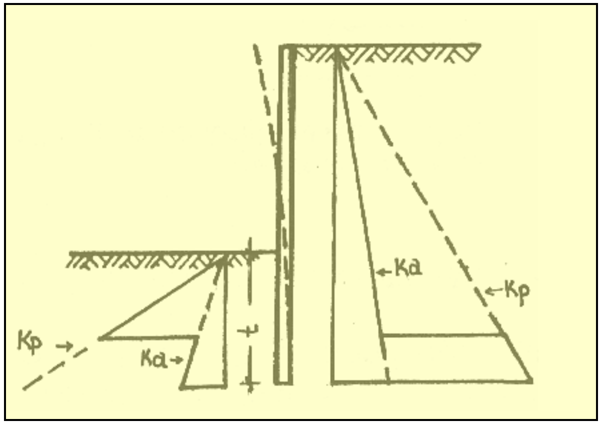
❷ ULTIMATE LIMIT STATE DESIGN: factors of safety so that probability of collapse of the structure acceptably small.

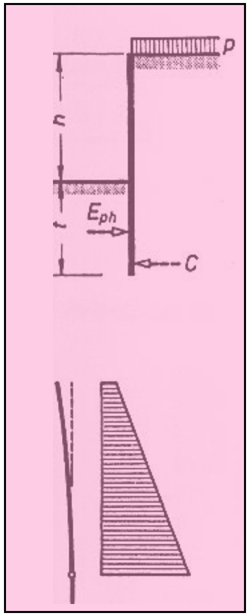


(a) limit equilibrium (b) working condition

DEPTH OF EMBEDMENT (1/3)



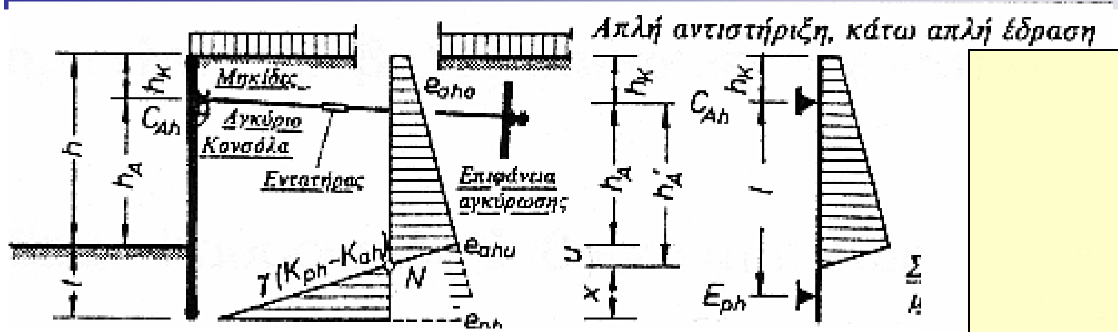


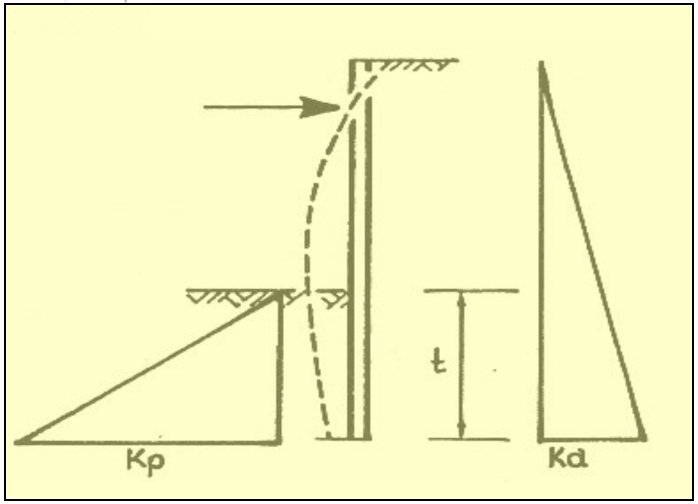


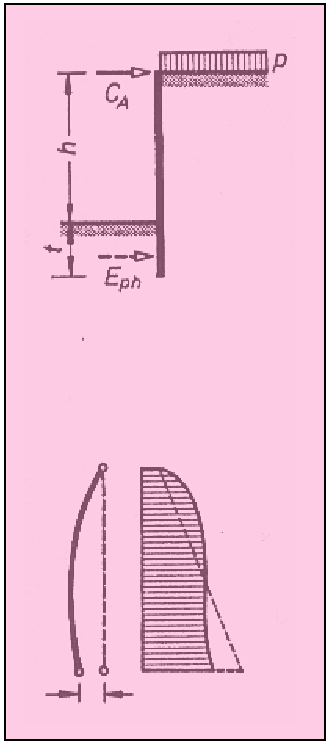
FIXED-EARTH CANTILEVER WALL (H<3M)

- Point of zero rotation around toe
- Moments about toe for depth of embedment, do, calculation, increase do by a small amount (up to 20%)

DEPTH OF EMBEDMENT(2/3)



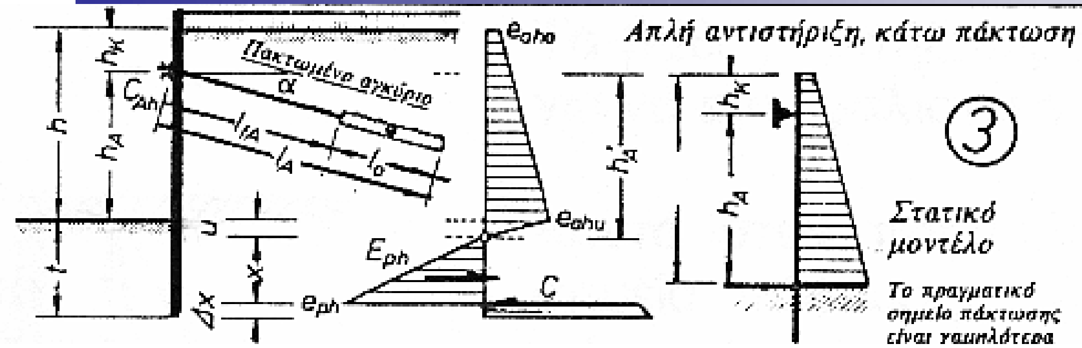


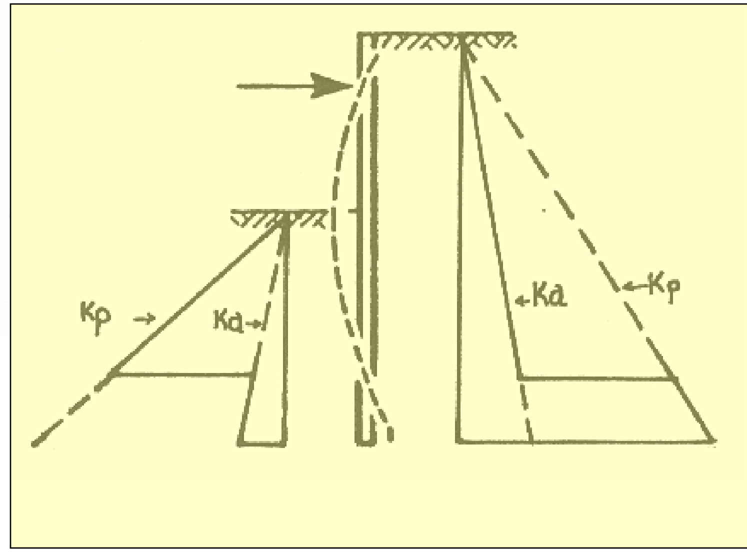


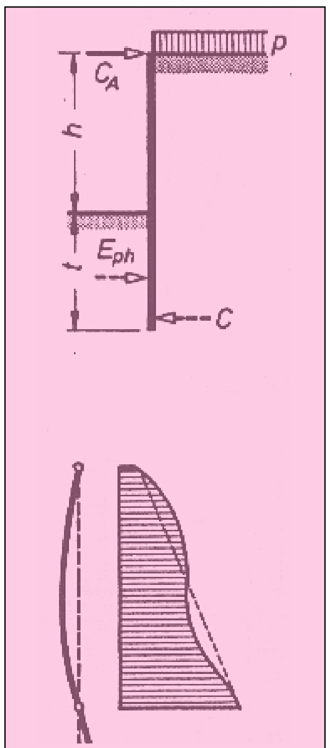
SINGLE PROP FREE-EARTH WALL

- **For depth of embedment (t) calculation take moments about prop. Equating horizontal forces will define prop force**

DEPTH OF EMBEDMENT(3/3)



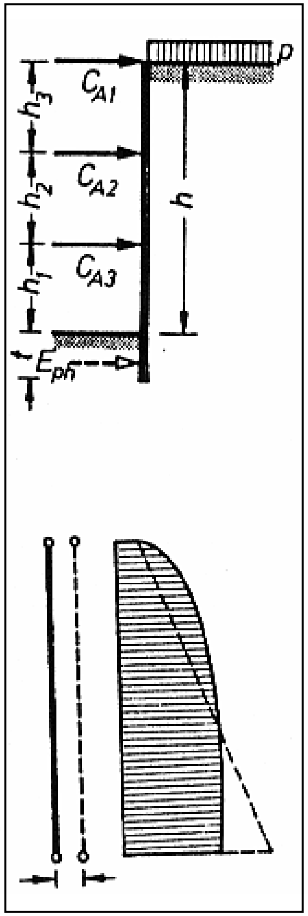


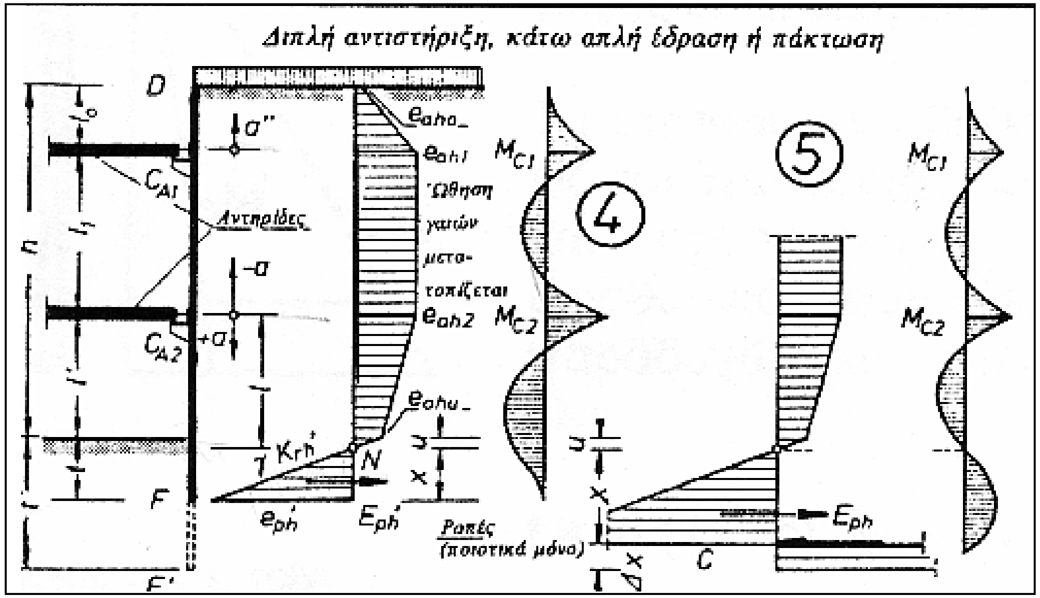


SINGLE PROPPED FIXED-EARTH WALL

- Point of zero rotation around toe
- Above point of zero moment active and passive stresses have been fully developed.
- Under this point passive stresses develop behind the wall and active in front of the wall.
- Depth of embedment is calculated for a beam supported at prop and point of zero rotation and increased by 20% (as for the cantilever).

MULTI PROPPED FREE- OR FIXED-EARTH SUPPORT





Earth pressure distribution (Peck's diagrams)

➔separate beams. From lowest prop downwards previous wall case.

➔continuous beam supported by the props and an extra support below level of excavation at the point where active pressures are zero, solution according to Cross, Kany, etc

FACTORS OF SAFETY

Various methods require:

- ⌧ **passive soil pressure is factored**
- ⌧ **soil strength parameters are factored**

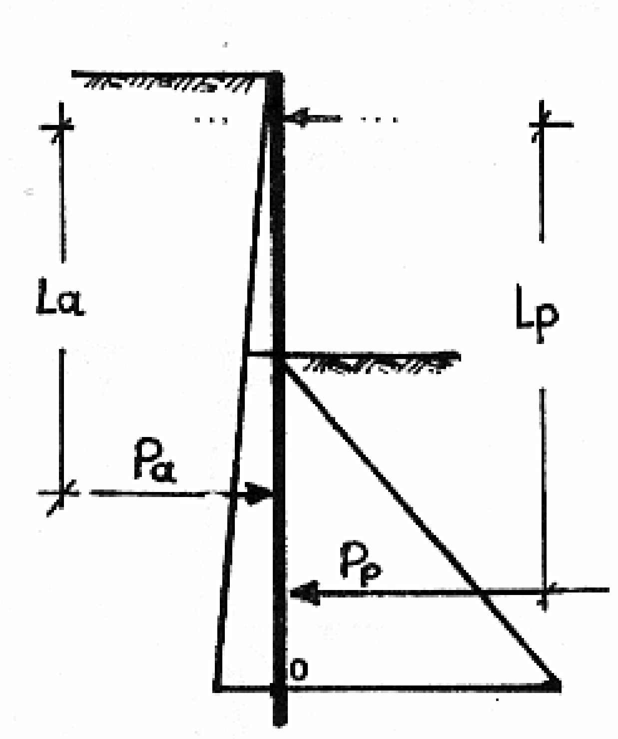
🕮 **providing safety against uncertainties e.g. hydraulic fracture, construction defects, variability of soil strata**

➘ after the application of factors of safety the depth of embedment is calculated to provide safety against failure

FACTORS OF SAFETY (1/4)

⌦ **factor of safety on moments: F_p=1.5-2**

$$F_p = \frac{\text{Moments due to passive pressures}}{\text{Moments due to active +net water pressures}}$$



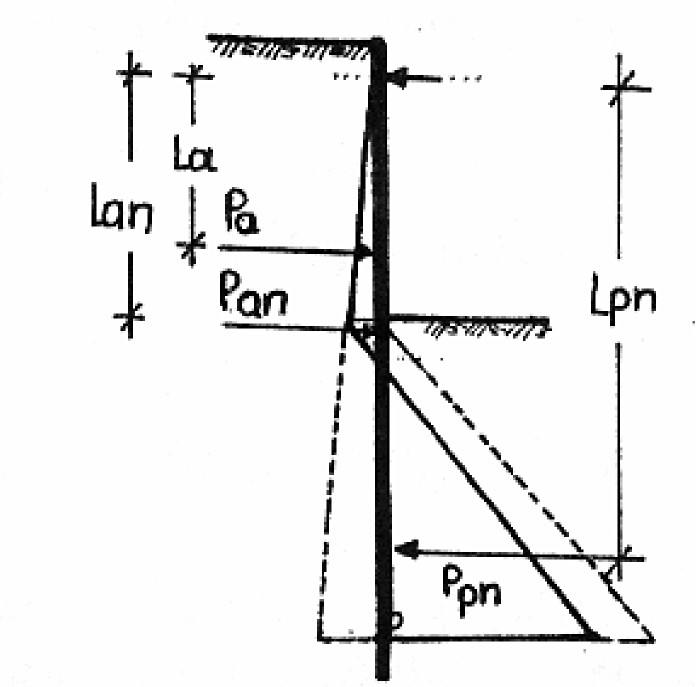
The depth of embedment is calculated, when moments are taken about the prop such that Fp=restoring moments/ overturning moments

$$P_a * L_a \leq \frac{1}{F_p} * (P_p * L_p)$$

FACTORS OF SAFETY (2/4)

- factor of safety on moments: F_{np}=1.5-2

$$F_{np} = \frac{\text{Net passive pressures}}{\text{Active pressures}}$$



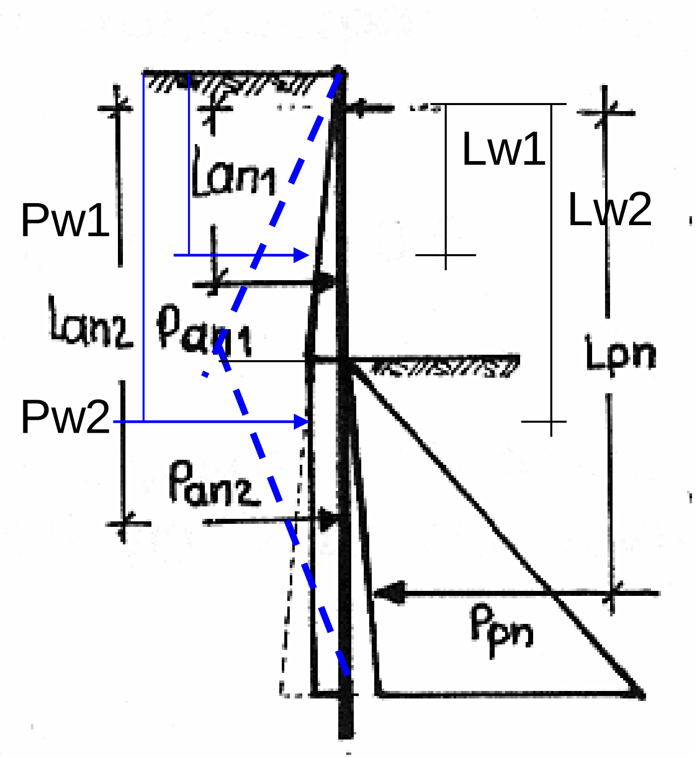
net active pressures

$$(P_a * L_a + P_{an} * L_{an}) \leq \frac{1}{F_{np}} * (P_{pn} * L_{pn})$$

FACTORS OF SAFETY (3/4)

- **factor of safety on moments**: F_r

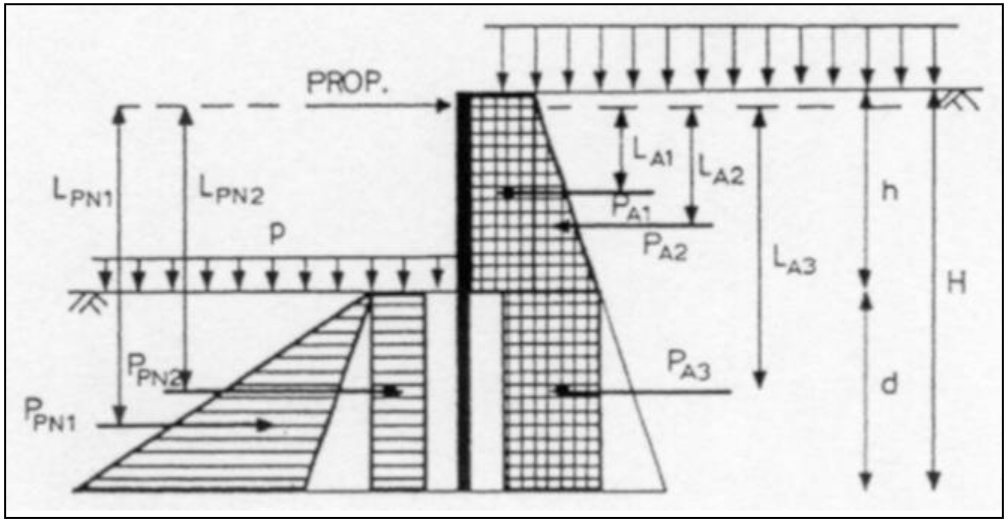
$$F_r = \frac{\text{Net passive pressures}}{\text{Net active pressures+ Net water pressures}}$$



$$F_r = \frac{P_{pn} * L_{pn}}{P_{an1} * L_{an1} + P_{an2} * L_{an2} + P_{w1} * L_{w1} + P_{w2} * L_{w2}} \quad {}^{1}$$

[1] non cohesive soil

FACTOR OF SAFETY (F_r)



$$F_r = \frac{P_{PN1} * L_{PN1} + P_{PN2} * L_{PN2}}{P_{A1} * L_{A1} + P_{A2} * L_{A2} + P_{A3} * L_{A3}}$$

earth pressures for the calculation of F_r for uniformly distributed load

FACTORS OF SAFETY(4/4)

Factor of safety on shear strength: F_s

The depth of embedment is found such that the moment about the prop is zero for the same pressure diagram as in (1/4) $P_p * L_p - P_a * P_a = 0$ except that the soil pressures are derived from factored soil strength parameters. This results in an enhanced active and a reduced passive pressure

soil strength parameters

$$F_s = \frac{\tan\phi}{\tan\phi_{\lim}}$$

[1] non cohesive soil

$\Phi_{\lim}$=angle of shearing resistance at limit equilibrium

$$F_s = \frac{S_u}{S_{u\lim}}$$

undrained loading

$$c_m = \frac{c}{F_c}, \varphi_m = \tan^{-1}(\tan\varphi / F_\varphi), F_c = F_\varphi = 1.2$$

drained loading

DESIGN METHODS

Requirements of various methods on:

- **strength parameters (F_s)**
- **resisting moments (F_r, F_p, F_{pn})**

- **CIRIA,**
- **CP2,**
- **British Steel Handbook,**
- **Rowe method,**
- **Burland-Potts method,**
- **Danish Standards.**

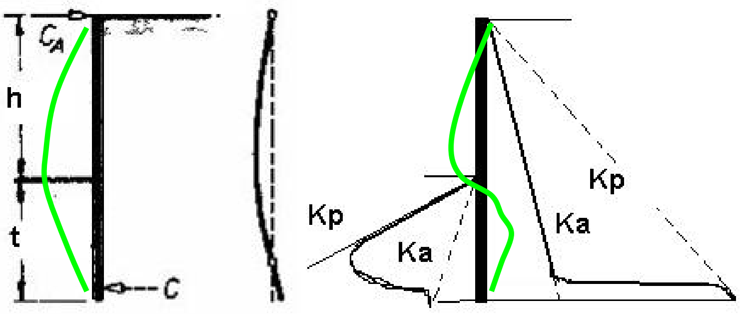
DESIGN METHOD - CIRIA

Propped free-earth wall:

- Depth of embedment
  - factor on moments F_r=2 or F_p=2
  - calculation of bending moment and forces at limiting equilibrium
  - increase prop force by 25%

DESIGN METHOD - CP2

- Propped free- or fixed earth support:
  - Depth of embedment free-earth support



- $F_p=2$
- bending moments and forces at ***working conditions***
- factor of safety on moments F_p
- reduce bending moment by 25%
- increase propped force by 15%

BRITISH STEEL HANDBOOK

- Propped free-earth support wall
  - Depth of embedment
    - $F_{np}=2$
    - bending moments and forces at limiting equilibrium
    - reduce bending moment up to 25% depending on the shape of the wall
    - unfactored propped force

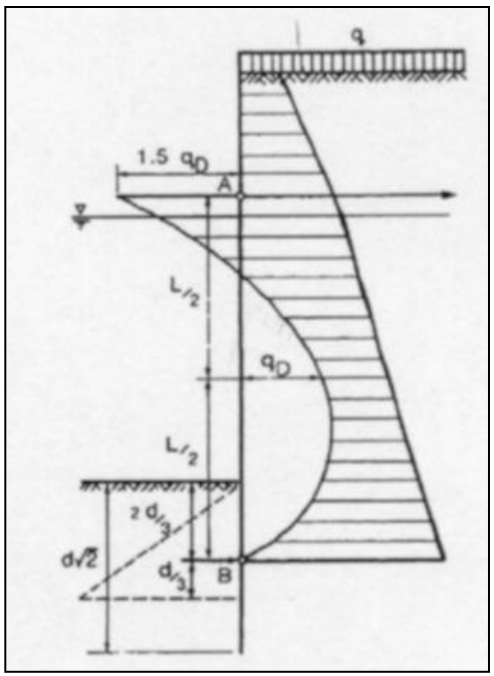
ROWE METHOD

- earth pressure coefficients Coulomb
- active pressures δ=2/3*φ
- passive pressures δ=0
- working conditions, F_p=1.5
- reduce bending moment
- increase prop force

DANISH STANDARDS

Hansen method, empirical

- parabolic active pressures



- assume depth of embedment

Force equilibrium yields reaction at B (active force+prop force)=passive force

- increase embedment by $\sqrt{2}$
- active δ=0, passive pressures δ=1/2*φ
- wall forces and moments: elastic beam supported at B & A and loaded with the active pressure distribution diagram

DESIGN METHODS vs SITE MEASUREMENTS

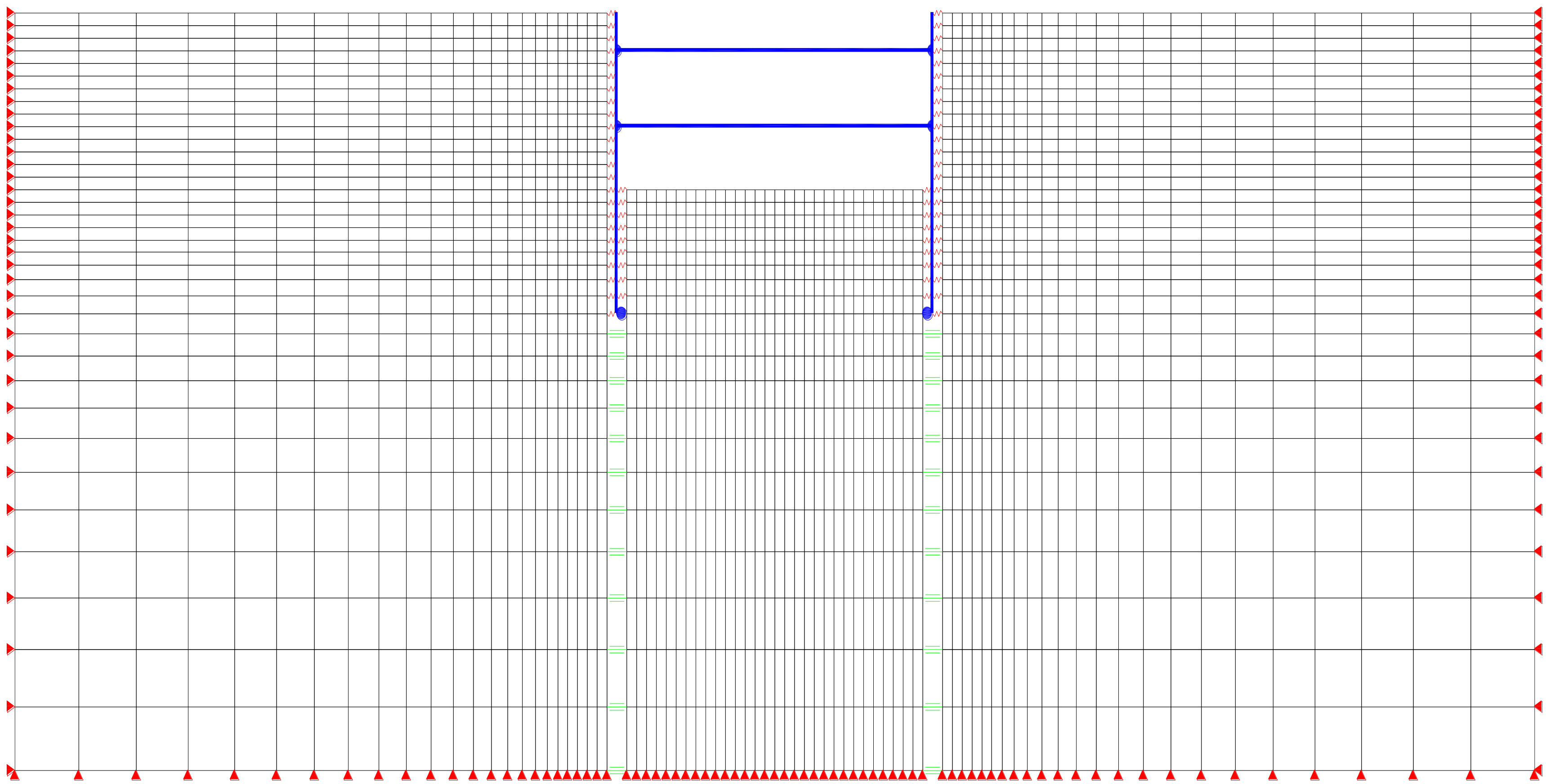
- Usually measured moments smaller, and propped forces larger than values predicted from design methods
- Limit equilibrium methods calculate forces and moments with or without factors of safety
- Under working conditions earth pressures are neither at limit equilibrium nor similar to the predicted values by various methods. None of the methods simulates the construction sequence e.g. pre stress the prop

FINITE ELEMENT ANALYSES

- Simulate soil behaviour (stress-strain relationship),
- soil structure interaction,
- stages of construction,
- initial soil behaviour, before commencement of construction activities.

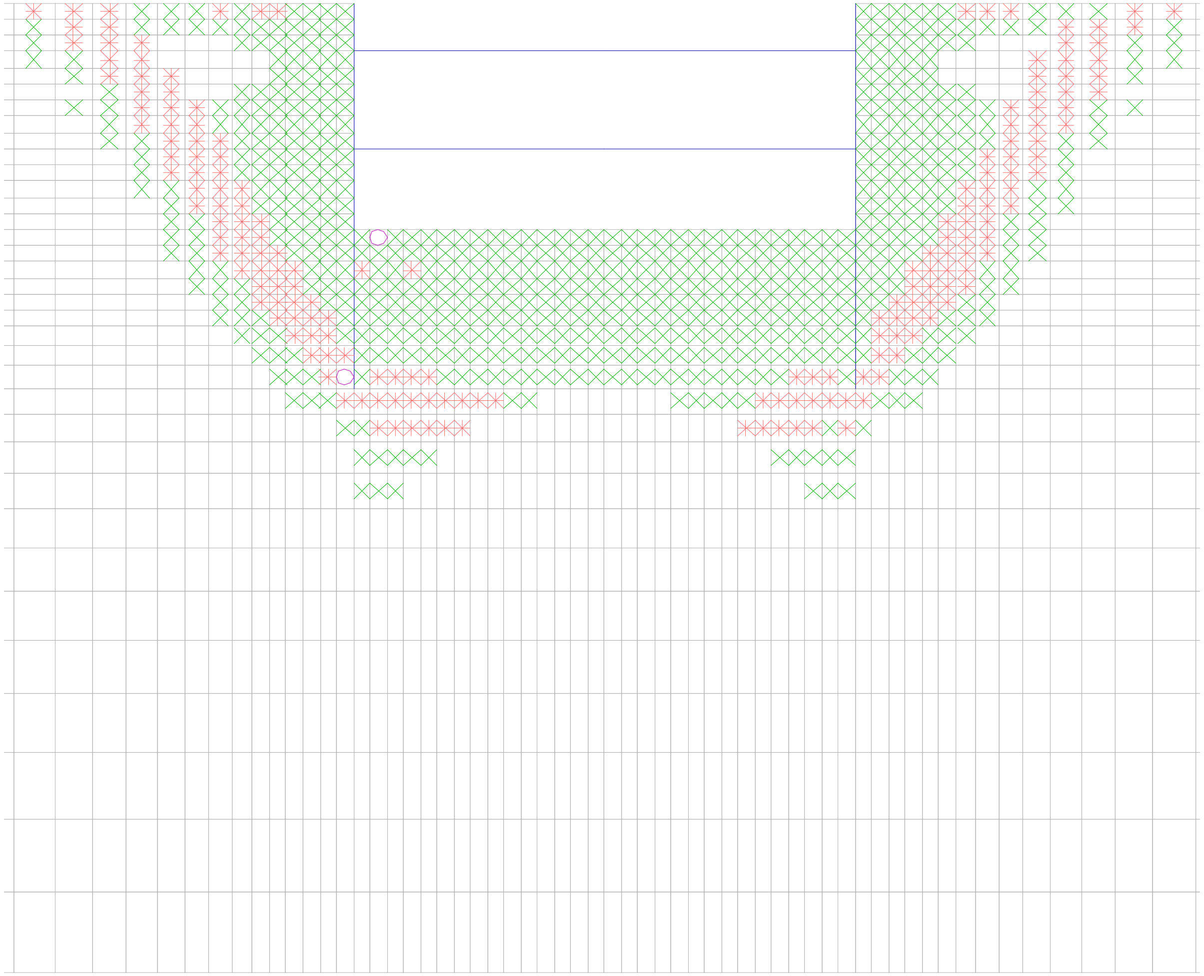
⌦2*D rectangular elements represent the soil and 1*D the thin wall. Excavation is simulated by removing elements

Numerical simulation of soil, wall and struts



- Same displacements
- Interface with friction and cohesion
- Same displacements and rotation
- Only horizontal movement allowed
- Only vertical movement allowed

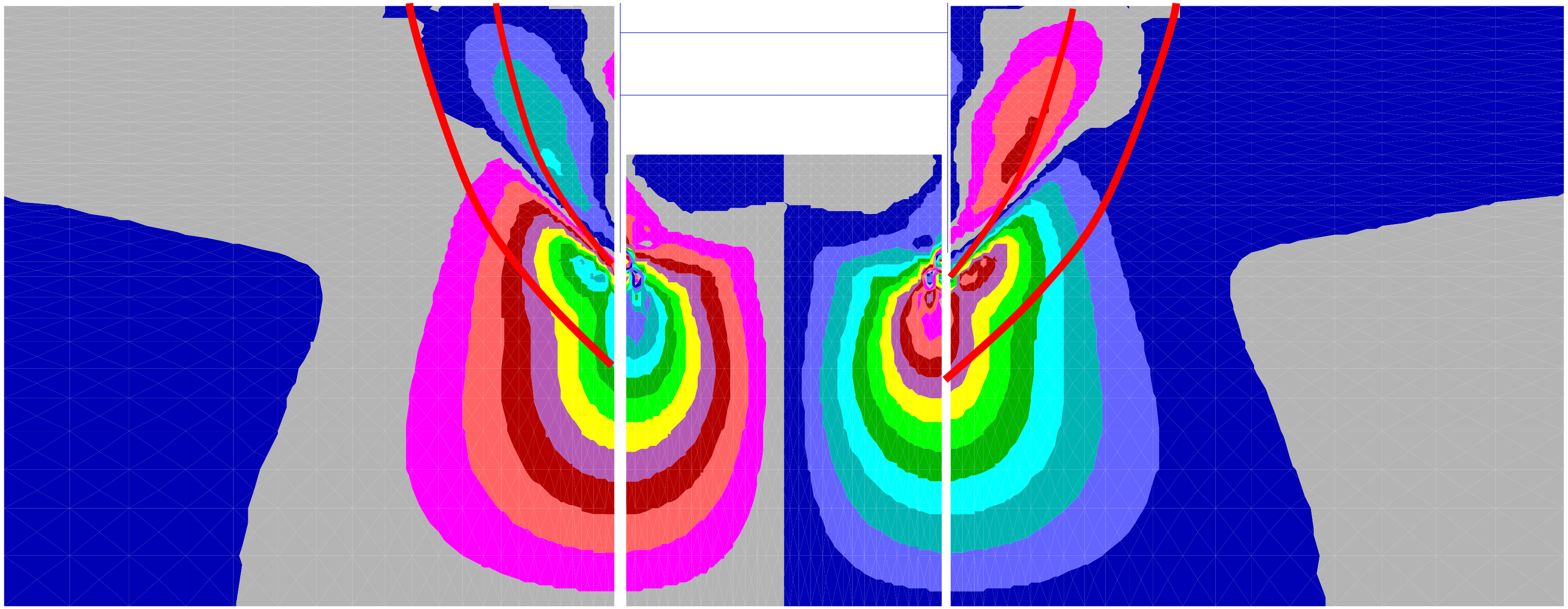
FINITE ELEMENT ANALYSIS



Yielded elements (x) according to the Mohr-Coulomb criterion

FINITE ELEMENT ANALYSIS

Shear stress distribution



Zones of highest shear stresses

British Standards, CIRIA 104, EuroCode 7

- **earth pressures as built**

  Earth pressures with factors of safety, depending on the method. Unsafe design (inadequate embedment) or safe design (adequate depth of embedment)

- **earth pressures at minimum safe embedment**

  reduction of embedment until factored earth pressures (restoring and overturning) at equilibrium

- **earth pressures with maximum safety factors**

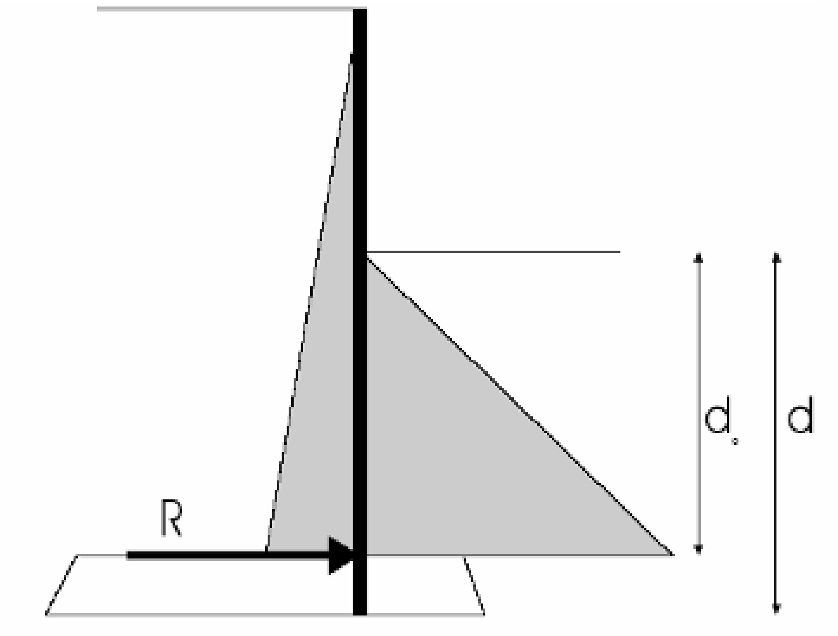
  increase of factors of safety to achieve equilibrium between earth pressures for given embedment

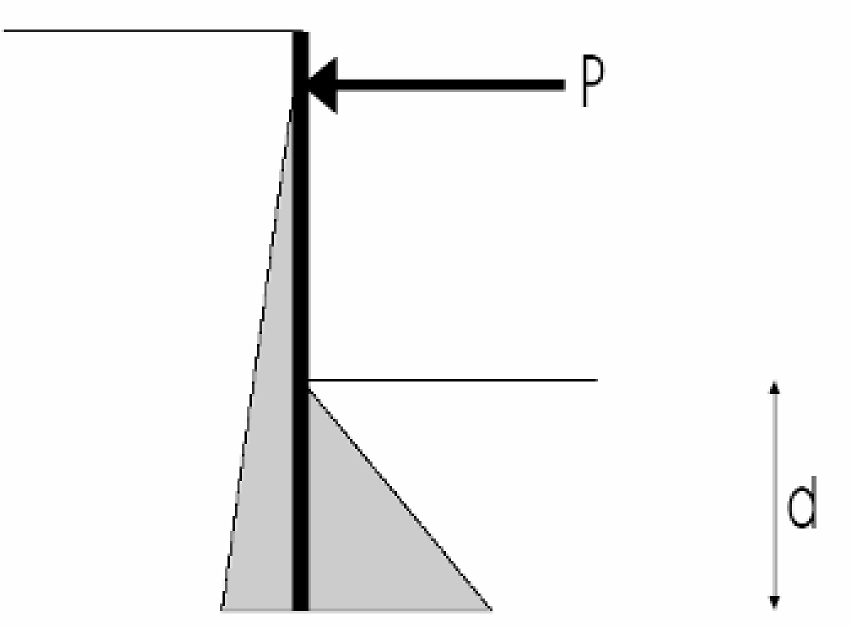
Used for the calculation of bending moments and forces in the wall

- **earth pressures at failure**

  reduction of depth of wall to achieve equilibrium of earth pressures when factors of safety are equal to one

Calculations using ReWaRD

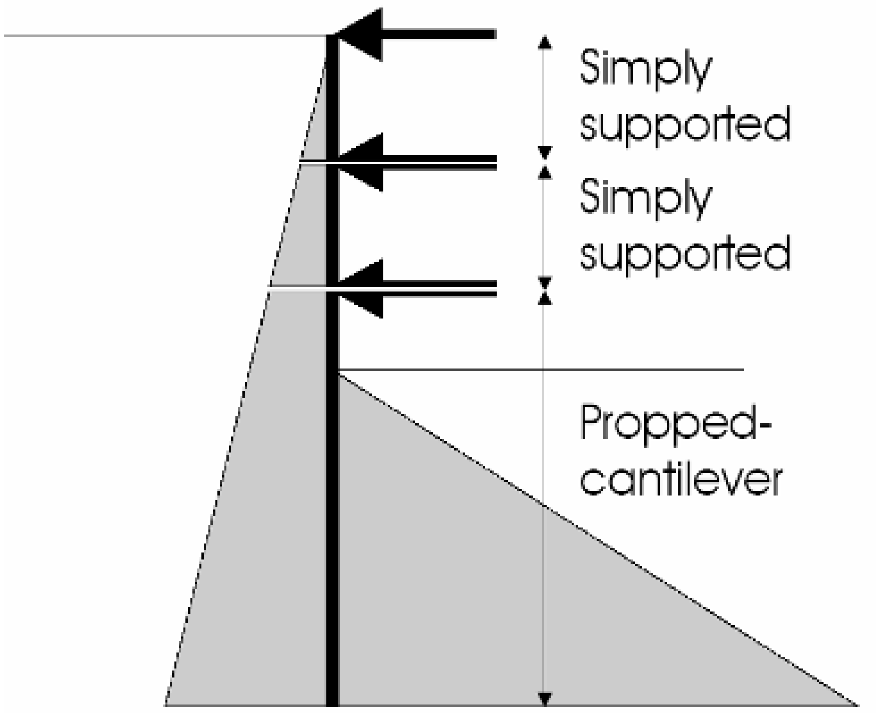




cantilever wall
- fixed earth support

single- propped wall
- free earth support

Calculations using ReWaRD



multi-propped wall

LIMIT STATE CONDITIONS

- ULTIMATE LIMIT STATE DESIGN: factors of safety so that probability of collapse of the structure acceptably small. Stability assessed using deformation (FEM techniques) or limit equilibrium analysis (determination of disrupting and resisting forces about a potential failure surface as in slope stability)
- In ultimate limit state (ULS) design the objective is to check that the probability of collapse of the structure is acceptable

OVERALL STABILITY-MASS CONCRETE WALL

- check bearing capacity of the foundation is adequate to support the weight of the wall, and for stability against sliding due to any applied shear forces on the back and front faces.
- check forces and bending moments in the wall.
- check for rotation failure of the mass of soil including the wall (deep seated slip)
- check for settlements of wall and the soil it supports.
- check for failure due to ***weathering***, ***hydraulic fracture*** at the toe of the wall and check for ***seismic risk***.

LIMIT STATE CONDITIONS

- **ULTIMATE LIMIT STATE DESIGN**: factors of safety so that probability of collapse of the structure acceptably small. Stability assessed using deformation (FEM techniques) or limit equilibrium analysis (determination of disrupting and resisting forces about a potential failure surface as in slope stability)

- **SERVICABILITY LIMIT STATE DESIGN**: ensure
- a specified threshold deformation is not exceeded
- stresses applied to the construction materials will not affect their durability

EURO CODE 7-PARTIAL FACTORS

- 1: ACTIONS: design actions are obtained from characteristic actions by multiplying by the appropriate factor
- 2: MATERIAL PROPERTIES: design material properties obtained by dividing by the appropriate partial factor
- 3: GEOMETRIC PROPERTIES: the design height obtained by adding an appropriate safety margin

In Eurocode 7 pressures arising from weight of soil and water pressures are regarded as unfavourable permanent

In Eurocode 7 pressures from surcharges are regarded as permanent, variable (1.5, 1.5, 1.3) or accidental (1, 1, 1) according to the flags set for each individual surcharge

EURO CODE 7 (ENV1997-1)

Cases A, B, C; limit equilibrium design

- A: Failure of the structure or the soil. Stresses in soil and the structure do not contribute significantly to resistance to failure

- B: Internal failure or excessive deformation of the structure. Structural forces contribute significantly to resistance to failure

- C: Soil failure or excessive deformations in soil. Soil resistance contributes significantly to resistance to failure

FACTORS OF SAFETY

CIRIA 104		A. Moderately Conservative (Permanent works)	
Gross pressure method		F_p=1.2-2.0 for φ'=20-30^o	
Nett pressure method		F_r=1.5-2.0	
Strength factor method		F_s=1.2 φ'>30^o else F_s=1.5	
Action			
DESIGN STANDARD		Unfav. (permanent)	Fav. (permanent)
BS 8002		1.0	
EUROCODE 7	A-B-C	1.0-1.35-1	0.95-1-1
	Serviceability	1.0	1.0

Material Properties				
DESIGN STANDARD		γ_φ	γ_c	γ_{cu}
BS 8002		1.2		1.5
EUROCODE 7	A-B-C	1.1-1-1.25	1.3-1-1.6	1.2-1-1.4
	Serviceability	1.0		
Geometric Properties				
DESIGN STANDARD		Unplanned Excavation (Δ_H)		
BS 8002		10% of the clear height/minimum of 0.5 m		
EUROCODE 7	A-B-C	10% of the clear height/maximum of 0.5 m		
	Serviceability	None		

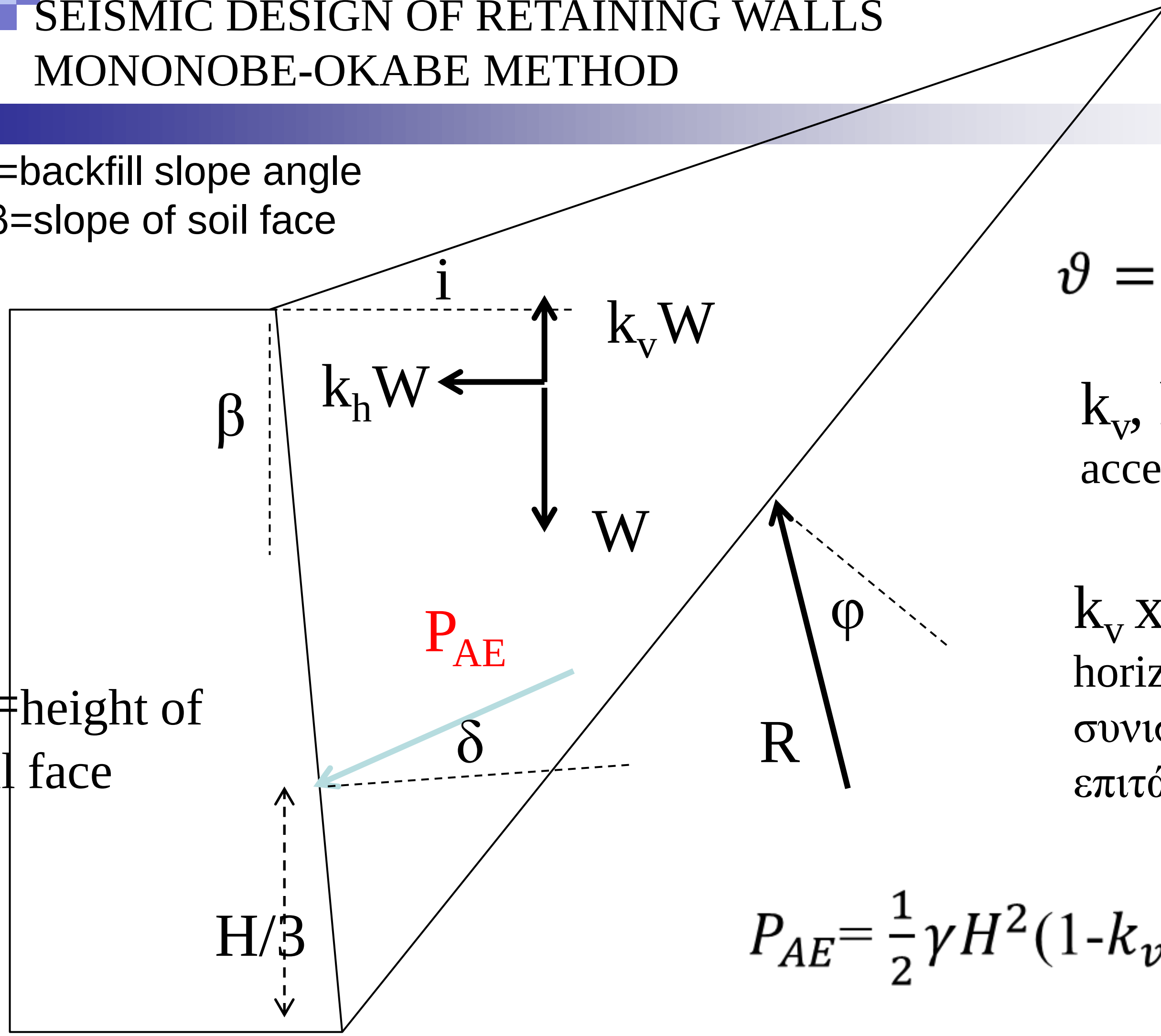
EURO CODE 7 (ENV1997-1)

- UPL: failure due to excess pore water pressures

- HYD: hydraulic fracturing, weathering due to hydraulic gradients

- SERVICEABILITY LIMIT CONDITION: strains and stresses according to cases A, B, C; factors of safety equal to *unity*

i=backfill slope angle
β=slope of soil face



H=height of soil face

$$\vartheta = tan^{-1}\frac{k_h}{1-k_v}$$

k_v, k_h = vertical , horizontal acceleration coefficients

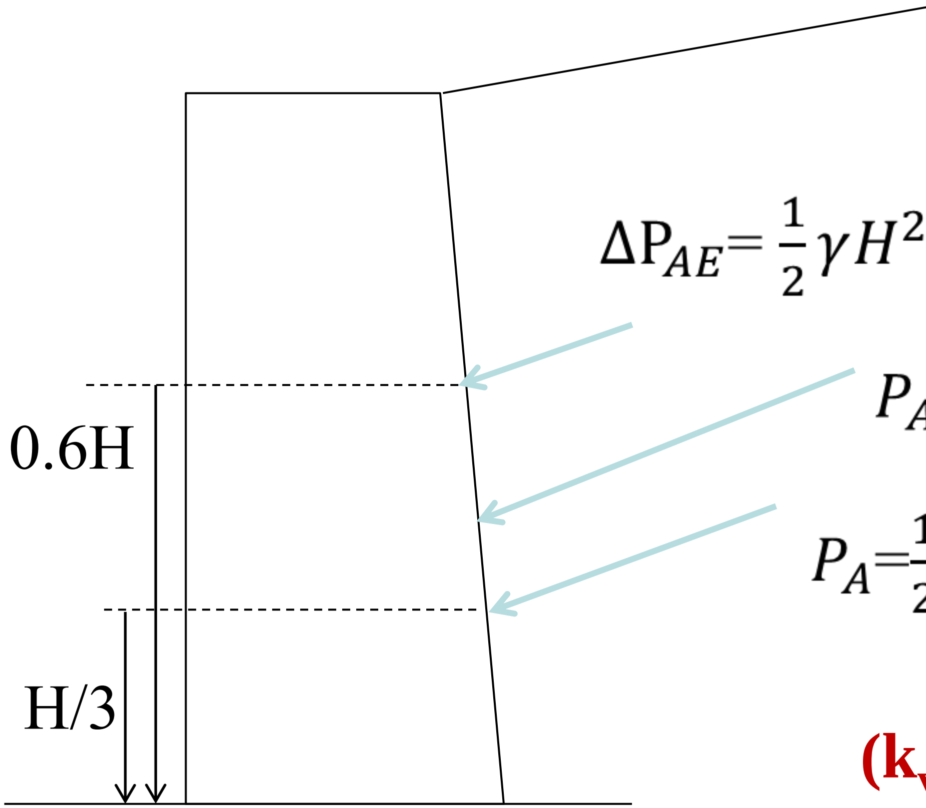
k_v x g, k_h x g = vertical , horizontal seismic acceleration
συνιστώσες σεισμικής επιτάχυνσης

$$P_{AE}=\frac{1}{2}\gamma H^2(1\text{-}k_v)^{*}k_{AE}$$

Seismic active Pressure coefficient

$$k_{AE}=\frac{cos^2(\varphi-\vartheta-\beta)}{cos\vartheta * cos^2\beta * \cos(\vartheta+\beta+\delta) * [1+\sqrt{\frac{\sin(\varphi+\delta)\ \sin(\varphi-\vartheta-i)}{\cos(\delta+\beta+\vartheta)\ \cos(i-\beta)}}]^2}$$

Seed & Whitman



$$\Delta P_{AE} = \frac{1}{2}\gamma H^2\left(\frac{3}{4}k_h\right)$$ Dynamic action

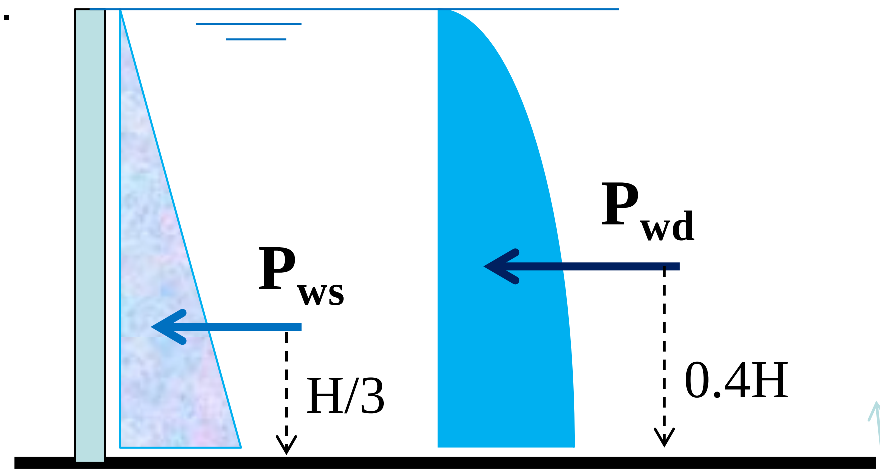
$$P_{AE} = P_A + \Delta P_{AE}$$

$$P_A = \frac{1}{2}\gamma H^2 k_a$$ Static action

(k_v~0)

Ch11_Steven L. Kramer - Geotechnical Earthquake Engineering (1996, Prentice Hall)

1.



Hyrdostatic pressure

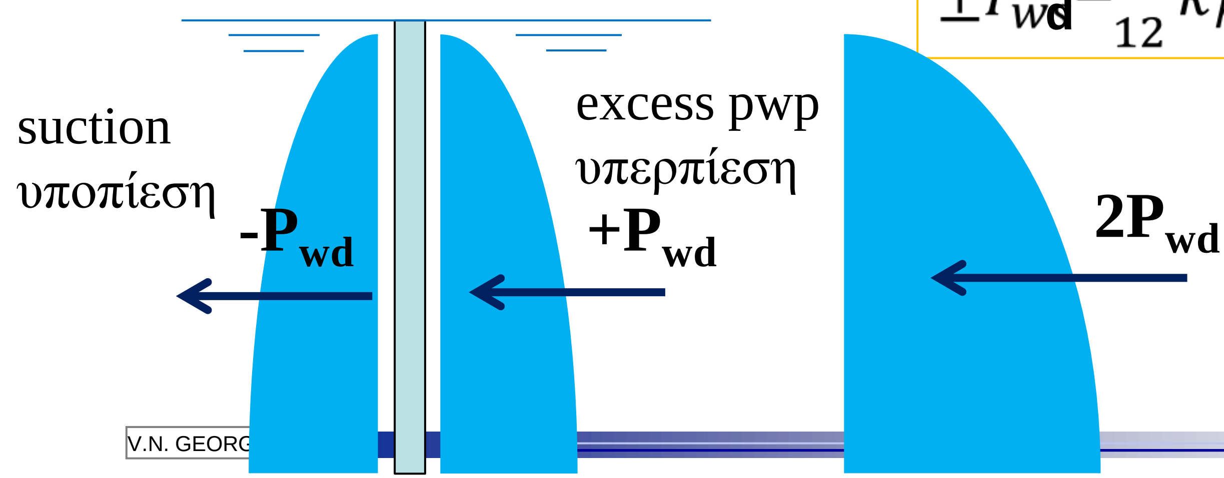
$$P_{ws}=\frac{1}{2}\gamma_w H^2$$

Hydrodynamic water pressure

$$\pm P_{wd}(x)=\frac{7}{8}k_h\gamma_w H\sqrt{x/H}$$

2.

$$\pm P_{wd}=\frac{7}{12}k_h\gamma_w H^2=(1.17k_hP_{ws})$$



HYDRODYNAMIC WATER PRESSURE: *wall retaining saturated soil*

$$\pm P_w = \frac{7}{8} C_e k_h \gamma_w H^2$$

$$C_e = 0.5 - 0.5 tanh * log \frac{2\pi n \gamma_w H^2}{7 E_w k T}$$

n=porocity
E_w=2x10^6 kPa (water compressibility)
K=soil permeability
T=fundamental period of vibration
Ce>0.8 for coarse sand and gravel

the point of application of the *hydrodynamic* water *pressure* lies at a depth below the top of the saturated layer equal to 60% of the height of such layer.