Geotechnical lecture presentation · Sheet piles and braced cuts

Braced cut in clay

Lecture: braced (strutted) excavation in clay; clay envelopes, tension crack, strut loads, sheet pile and wale sizing, steel verification and base heave; a worked example for each topic.

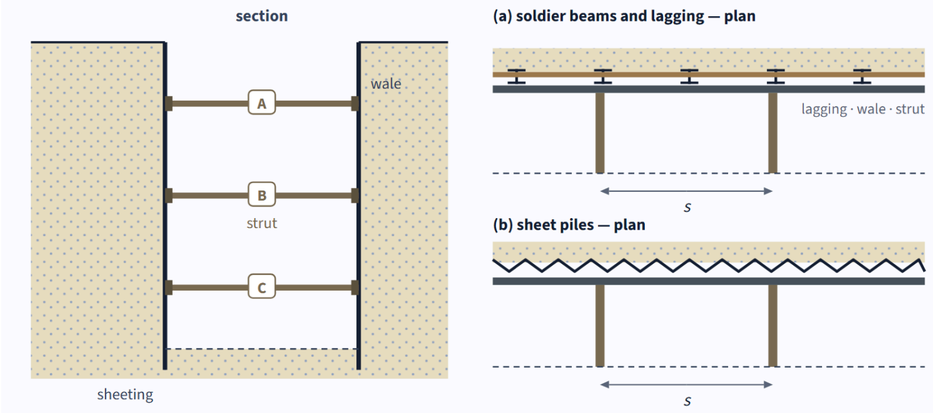
Excavation support

Peck envelope (clay)

Tension crack · first strut

Strut · wale · sheet pile

Base heave



Introduction

What will we learn in this lecture?

- **why the pressure in a braced cut is not Rankine** and **how the envelopes** are constructed from measured loads
- The two envelopes of the clay by the **γH/c stability number**: soft–medium clay (γH/c > 4) and stiff clay (≤ 4)
- Adding the **surcharge** as a separate pressure, that water in clay changes only the unit weight; the **tension crack $z_c = 2c/\gamma$** and the first support
- From the envelope, the **support loads** (hinged segments), and the sizing of the **sheet pile** and **wale**
- To verify the steel with **EN 1993-1-1 and AISC 360-22**
- The **base heave** check of the excavation base in clay (Terzaghi column, Chang's correction)

LECTURE FLOW

Each topic is covered in **three steps**:

1 · **Theory**: what is the rule, why is it so?

2 · **Problem statement**: given and required

3 · **Solution**: step by step, with numbers

Example excavation: clay (γ = 18 kN/m³, c = 35 kPa, φ = 0), depth H = 7.00 m, width B = 6.00 m; struts at depths of 1.00, 3.50, 6.00 m, horizontal spacing s = 3.00 m. Forces are per metre of wall (strut loads multiplied by s).

Support system and its elements

THEORY

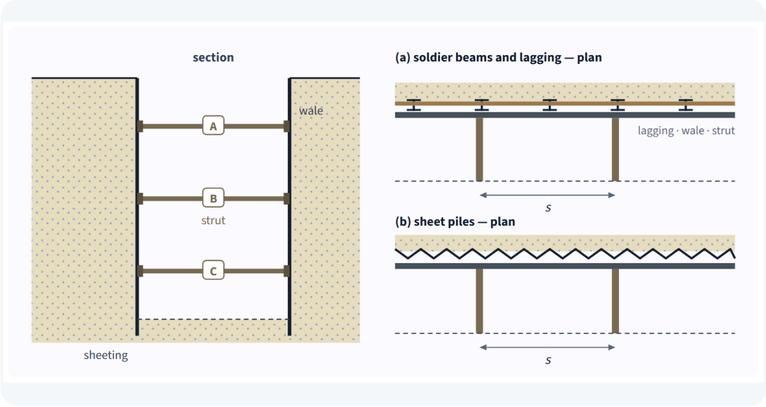
Deep excavations with vertical faces (urban basements, cut-and-cover tunnels) are kept open with a temporary **support system**. Two arrangements are common: **vertical piles** (steel or timber) driven in advance, with horizontal timber lagging placed between them as excavation proceeds; or interlocking **sheet piles** forming a continuous wall.

At each support level, horizontal **wales** are installed as soon as the excavation reaches that level; **struts** are horizontal steel compression members that cross the excavation from wale to wale.

Selecting the members requires the lateral pressure that the support system will take. The theory gives the total thrust, not its **distribution**: the distribution depends on the soil, the sequence and rate of the work, the crew and the equipment.

The design pressures are therefore **empirical envelopes**: drawn around the pressures back-calculated from support loads measured in actual excavations.

During construction, the strut spacing should be at least about 2.75 m vertically.



Rotates about the top of the wall

THEORY

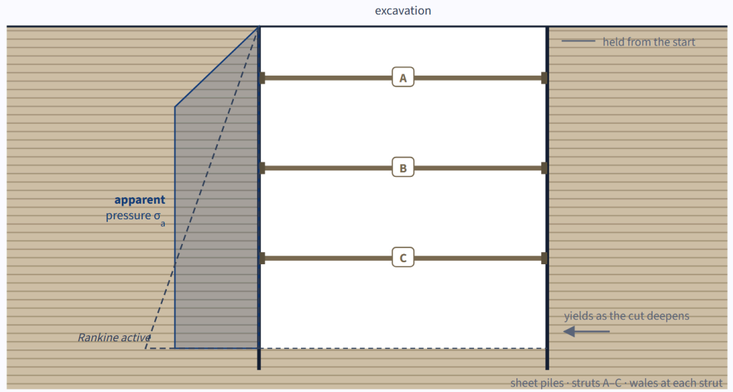
A cantilever retaining wall rotates about its base, and the Rankine active pressure (zero at the top, largest at the bottom) describes it well. **A braced wall moves in the opposite sense**: the first strut is placed while the excavation is shallow, so the upper part barely moves; each subsequent stage is excavated before the strut is installed, and the interval lengthens with depth, so the wall deflects progressively lower: when excavation is complete it has **rotated about its top**.

Near the top the pressure stays close to the **at-rest value** (the soil has moved very little).

At the bottom the pressure may drop below the Rankine active value (the soil has long exceeded plastic equilibrium).

The resultant thrust P_a acts **above** the one-third point of the triangle: at height n_aH from the base, $n_a > 1/3$.

The total thrust is found with the logarithmic spiral wedge ($r = r_0 e^{\theta \tan \varphi'}$); in granular soil it is 15–20% above Coulomb's value. But since its distribution and n_a depend on the soil and the site, excavations are designed with **envelopes**.



How were the envelopes constructed?

THEORY

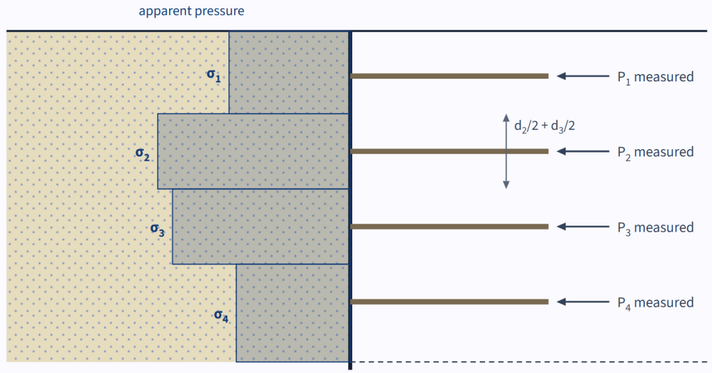
Pressure varies from point to point, and each support must be sized for the **maximum load** it may encounter. Envelopes are constructed for this purpose: in an instrumented excavation, the load on each support is measured, divided by the sheet-pile strip it supports (horizontal spacing s × half the distances to the supports above and below), and converted to an **apparent pressure** over this band:

$$\sigma_1 = \frac{P_1}{s\left(d_1 + \frac{d_2}{2}\right)} \qquad \sigma_i = \frac{P_i}{s\left(\frac{d_i}{2} + \frac{d_{i+1}}{2}\right)}$$

d_1 is the depth of the first strut, d_i the spacings between subsequent struts. The envelope of all diagrams drawn one over the other is an **apparent pressure envelope**.

Peck's envelopes are based on sand loads measured in Berlin, Munich and New York subway excavations; he constructed them by the same method for soft–medium and stiff clays.

The envelope is **not** the pressure acting on the wall at any given instant: the load taken from it for a given support has been drawn so as to remain on the safe side of the measured loads.



Soft–medium or stiff: γH/c

THEORY

In clay the shape of the envelope is determined by the **stability number γH/c** (c undrained cohesion, φ = 0). A **soft-to-medium clay** with γH/c > 4 is worked far beyond its strength by the excavation:

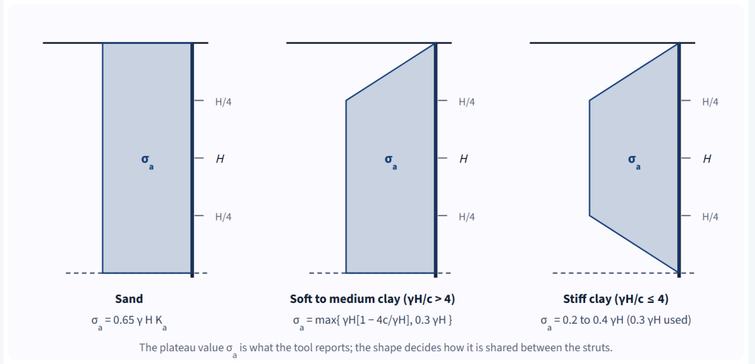
$$\frac{\gamma H}{c} > 4: \quad \sigma_a = \max\left(\gamma H\left[1 - \frac{4c}{\gamma H}\right],\ 0.3\gamma H\right)$$

The envelope is zero at the surface, increases linearly in the upper quarter, and remains constant down to the base; the larger of the two expressions governs. **Stiff clay** is closer to its elastic range:

$$\frac{\gamma H}{c} \le 4: \quad \sigma_a = 0.2\gamma H \text{ with } 0.4\gamma H \ \ (\text{average } 0.3\gamma H)$$

In stiff clay the envelope increases in the upper quarter, is constant in the middle half, and returns to zero in the lower quarter. The conditions are the same as for the sand envelope (≳ 6 m depth, water below the base, clay undrained).

If the cut passes through several clays, the source averages them by thickness: $c_{av} = \Sigma c_i H_i/H$, $\gamma_{av} = \Sigma \gamma_i H_i/H$, then the clay envelopes are used with the averages.



Problem statement

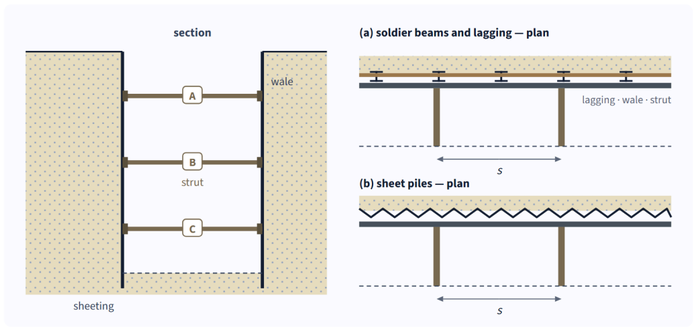
EXAMPLE

Given

Open excavation	$H = 7.0$ **m,** $\gamma = 18$ **kN/m³,** $c = 35$ **kPa**
Softer clay	**same excavation,** $c = 20$ **kPa**

REQUIRED

For both clays, **γH/c**, the type of envelope and σ_a.



Solution

SOLUTION

1 Opening clay (c = 35 kPa)

$$\frac{\gamma H}{c} = \frac{18(7)}{35} = \mathbf{3.60} \le 4 \Rightarrow \text{stiff clay}$$

$$\sigma_a = 0.3\gamma H = 0.3(18)(7) = \mathbf{37.80}\ \text{kPa}$$

2 Softer clay (c = 20 kPa)

$$\frac{\gamma H}{c} = \frac{126}{20} = \mathbf{6.30} > 4 \Rightarrow \text{soft–medium clay}$$

$$\sigma_a = \gamma H\left[1 - \frac{4c}{\gamma H}\right] = 126(1 - 0.635) = \mathbf{46.00}\ \text{kPa} \ge 0.3\gamma H = 37.8$$

The opening clay is taken as stiff: σ_a = **37.80 kPa** (largest support load P = 271.3 kN). When cohesion drops to 20 kPa the excavation behaves as soft–medium clay: σ_a = **46.0 kPa** and the base of the envelope becomes a constant plateau (P = 338.1 kN).

Check the solution in GeoTerva ↗

Next to the envelope; with unit weight; location of the first strut

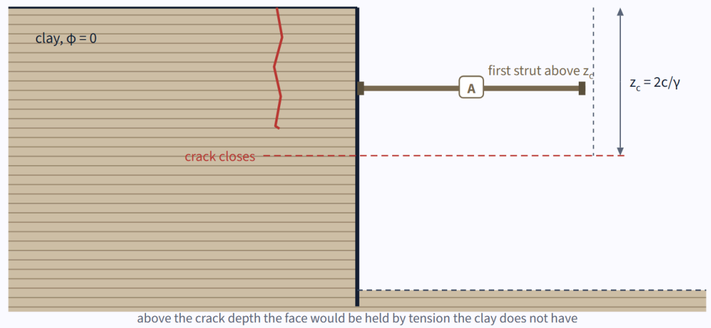
THEORY

Surcharge q is not included in the envelope (the empirical coefficients would then also apply to it); it is carried as its own uniform pressure over the full height, and in undrained clay $K_a = 1$: $\Delta p_q = q$.

Water: clay envelopes are undrained, total stress formulations; water in the clay is already contained in the unit weight and the undrained strength. Water above the base is drawn and noted, not added as a separate load; the only thing it changes is the soil weight: below water the saturated unit weight, and in the layer cut by water the thickness-weighted average of the two values, $\gamma = (\gamma z_w + \gamma_{sat}(H - z_w))/H$.

Tension crack: the clay face can stand unsupported down to the depth at which the tension crack closes (the depth at which the cohesive Rankine active pressure is zero). For $\varphi = 0$, $K_a = 1$; the first support must be placed **above** this depth, so that the wall does not bear on an open face that the clay cannot hold.

$$\Delta p_q = q \qquad z_c = \frac{2c}{\gamma}\ (q = 0), \qquad z_c = \frac{2c - q}{\gamma}\ (\text{surcharge q})$$



Problem statement

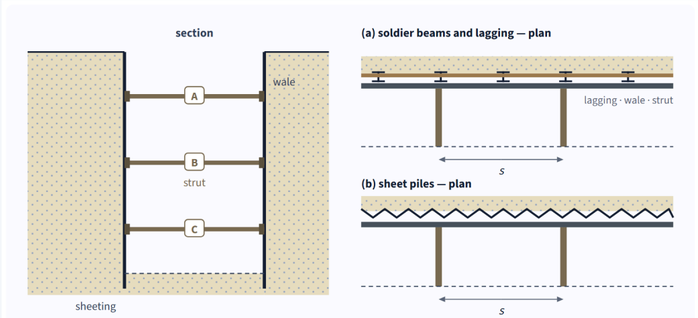
EXAMPLE

Given

Excavation	**The open excavation in Example 1 ($H = 7$ m, $\gamma = 18$, $\gamma_{sat} = 19$ kN/m³, $c = 35$ kPa); first support at $z = 1.0$ m**
Cases	**(a) q = 0, no water; (b) surcharge $q = 10$ kPa; (c) groundwater $z_w = 3$ m**

REQUIRED

z_c, whether the position of the first support is suitable; in (c) $\bar{\gamma}$ and $\bar{\gamma}H/c$; in each case the total load R and the largest support load.



Solution (1/2)

SOLUTION

1 Tension crack and initial support

$$z_c = \frac{2c}{\gamma} = \frac{2(35)}{18} = \mathbf{3.889}\ \mathrm{m}\ > 1.00\ \mathrm{m}\ \checkmark$$

$$z_c = \frac{2c - q}{\gamma} = \frac{70 - 10}{18} = 3.333\ \mathrm{m}\ \ (q = 10\ \mathrm{kPa})$$

The first support (z = 1.0 m) is well above the tension crack depth (3.89 m): suitable. With surcharge the crack drops to 3.33 m.

Solution (2/2)

SOLUTION

2 Water (c): $\bar{\gamma} = (\gamma z_w + \gamma_{sat}(H - z_w))/H$; summary of the three cases

$$\bar{\gamma} = \frac{18(3) + 19(4)}{7} = 18.571\ \text{kN/m}^3 \qquad \frac{\bar{\gamma}H}{c} = \frac{18.571(7)}{35} = 3.714$$

$$\sigma_a = 0.3\bar{\gamma}H = 39.00\ \text{kPa}$$

Condition	γH/c	σ_a (kPa)	z_c (m)	R (kN/m)	P_{max} (kN)
(a) q = 0, no water	3.60	37.80	3.89	198.4	271.3
(b) q = 10 kPa	3.60	37.80	3.33	268.4	334.3
(c) water z_w = 3 m	3.71	39.00	—	204.8	279.8

Surcharge adds only $\Delta p_q = q = 10$ kPa (R 198.4 → 268.4 kN/m); water only raises $\bar{\gamma}$ to 18.57 kN/m³ (σ_a 37.8 → 39.0 kPa).

Check the solution in GeoTerva ↗

From envelope to support loads

THEORY

Struts are horizontal members: they carry the pressure along their axes and bend under their own weight; capacity depends on slenderness, and wide excavations may require additional support. The source gives a simple and conservative method for the load:

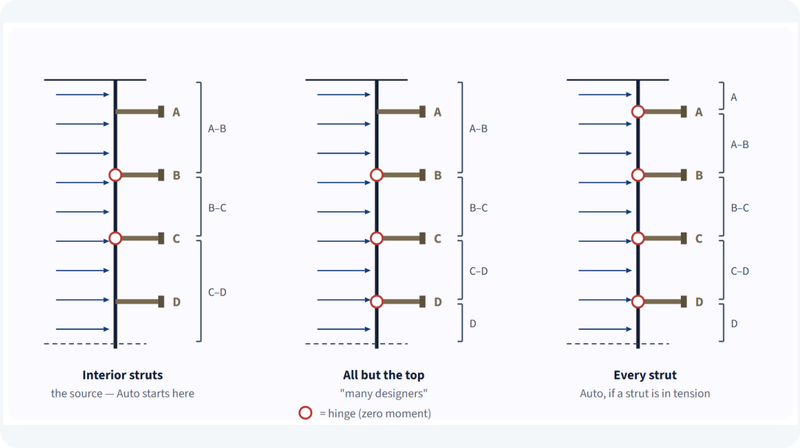
> Draw the pressure envelope and the support levels. Take the sheet pile **hinged at every support level** except the topmost and the lowest (many designers do so at all except the topmost).

> Divide the wall at the hinges into **simple cantilevers at the ends and simple beams in between**; find the reactions of each part by statics (for a four-support cut, A, B_1, B_2, C_1, C_2, D).

> The load of a support is the sum of the reactions it receives × the horizontal spacing s.

$$P_A = A\,s \quad P_B = (B_1 + B_2)\,s \quad P_C = (C_1 + C_2)\,s \quad P_D = D\,s$$

Each segment is statically determinate: it carries two supports; moment and horizontal force equilibrium about one of the supports gives both of them. Since the hinge support belongs to both sides, it collects **two reactions**. The reactions are per unit length of the wall (kN/m); s converts them to a support load.



Problem statement

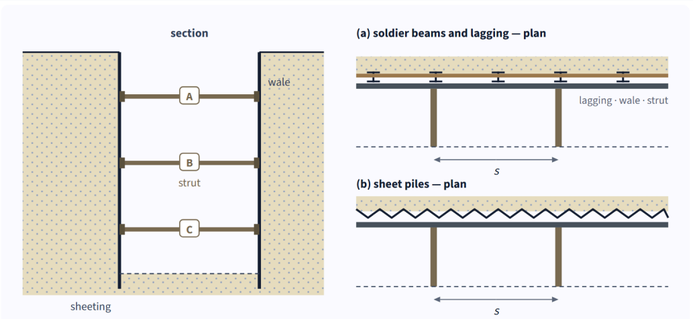
EXAMPLE

Given

Excavation	$H = 7.00$ m; $B = 6$ m; struts $z = 1.00, 3.50, 6.00$ m (A, B, C); horizontal spacing $s = 3.00$ m
Pressure	Example 1: stiff clay (γH/c = 3.6): trapezoidal envelope $\sigma_a = 0.3\gamma H = 37.8$ kPa (increases in the upper quarter, drops to zero in the lower quarter); sheet pile hinged at support B (except at the very top and very bottom)

REQUIRED

The **resultant W** of the parts and its arm **z̄**; the reactions **A, B_1, B_2, C**; the support loads **P_A, P_B, P_C**.



Solution (1/2)

SOLUTION

1 Divide the envelope into parts at the hinges; the resultant and moment arm of each part (depth from the top surface)

Piece	Range (m)	W (kN/m)	z̄ (m)	Supports
1	0.00 – 3.50	99.23	2.139	A (z = 1.00), B (z = 3.50)
2	3.50 – 7.00	99.23	4.861	B (z = 3.50), C (z = 6.00)

2 First part: moment equilibrium about the first support

$$R_{B_2} = \frac{W(\bar{z} - z_A)}{z_B - z_A} = \frac{99.23(2.139 - 1.00)}{3.50 - 1.00} = \mathbf{45.20} \qquad R_A = W - R_{B_2} = \mathbf{54.02}\ \mathrm{kN/m}$$

The first span carries the two struts with 54.02 and 45.21 kN/m; the reaction of the uppermost support (A) is larger because it also includes the load of the cantilever portion.

Solution (2/2)

SOLUTION

3 Second part

$$R_C = \frac{W(\bar{z} - z_B)}{z_C - z_B} = \frac{99.23(4.861 - 3.50)}{6.00 - 3.50} = \mathbf{54.02} \qquad R_{B_1} = W - R_C = \mathbf{45.20}\ \mathrm{kN/m}$$

4 Each support sums the reactions it receives; load = total × s (s = 3.00 m)

Level	z (m)	Reactions received (kN/m)	Total (kN/m)	P = R·s (kN)
A	1.00	54.02	54.02	**162.06**
B	3.50	45.21 + 45.21	90.42	**271.26**
C	6.00	54.02	54.02	**162.06**

Maximum support load **P_B = 271.26 kN** (level B): this load governs the strut sizing. Since support B belongs to two parts, it sums two reactions.

Check the solution in GeoTerva ↗

Maximum moment and required section modulus

THEORY

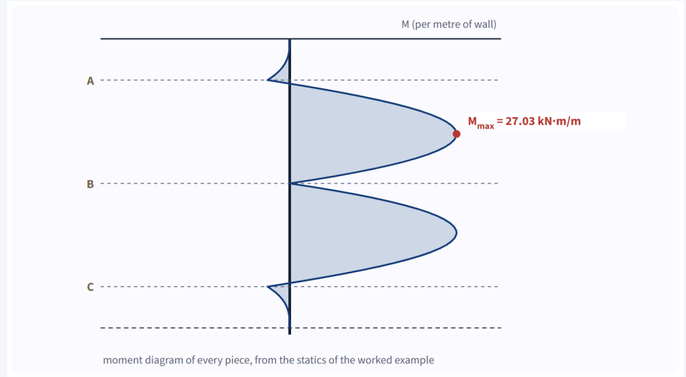
The **sheet pile** is designed from the same parts: the maximum bending moment is found in each part (where the shear changes sign: within the span or at the support where the cantilever meets the strut); the largest is the design moment for unit length of wall. The required section modulus follows from the allowable bending stress, and a section with at least that value is selected from the table:

$$M_{max} = \max_{parts} |M| \qquad S = \frac{M_{max}}{\sigma_{allow}}$$

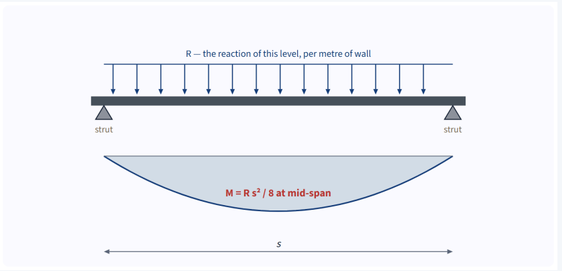
Struts may be considered continuous if properly connected; conservatively, they are taken as **hinged** at the supports: a simply supported beam of span s, with the level reaction R (per unit length of wall) as a distributed load:

$$M_{max} = \frac{R\,s^2}{8} \qquad S = \frac{M_{max}}{\sigma_{allow}}$$

The level with the largest reaction is governing. Struts are sometimes connected to the sheet pile at points that will provide the lateral support they require.



moment diagram of every piece, from the statics of the worked example



Problem statement

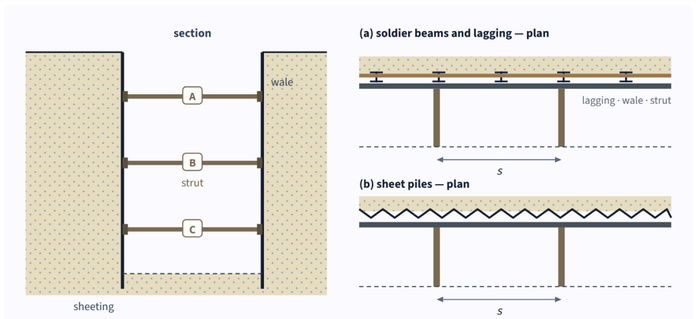
EXAMPLE

Given

Excavation	**The excavation in Example 1 ($H = 7$ m, stiff clay, $\sigma_a = 37.8$ kPa); struts at 1.0 · 3.5 · 6.0 m; horizontal spacing $s = 3.0$ m; hinge at B**
Steel	**allowable bending stress $\sigma_{allow} = 170\,000$ kPa**

REQUIRED

The sheet pile's **M_{max}** and required **S** (cm³/m); at the level of the largest reaction, the wale's **M_{max}** and **S** (cm³).



Solution (1/2)

SOLUTION

1 Sheet pile: maximum moment in each part

Piece	Range (m)	M_{max} (kN·m/m)	Location z (m)
1	0.00 – 3.50	27.03	2.30
2	3.50 – 7.00	27.03	4.70

2 Required section modulus

$$S = \frac{M_{max}}{\sigma_{allow}} = \frac{27.03}{170{,}000} = 0.0001590\ \mathrm{m^3/m} = \mathbf{159.0}\ \mathrm{cm^3/m}$$

In the sheet pile **M_{max} = 27.03 kN·m/m** (segment 1, z = 2.30 m): **S ≥ 159.0 cm³/m**.

Solution (2/2)

SOLUTION

3 Wale: simple beam at each level (s = 3.000 m)

Level	R (kN/m)	M = R s²/8 (kN·m)	S = M/σ_{allow} (cm³)
A	54.02	60.77	357
B	90.42	101.72	598
C	54.02	60.77	357

The largest reaction is **at level B** (R = 90.42 kN/m): the wale is sized with **M_{max} = 101.72 kN·m, S ≥ 598 cm³**; the wales at the other levels may be smaller.

Check the solution in GeoTerva ↗

Design effects, class, bending and buckling

THEORY

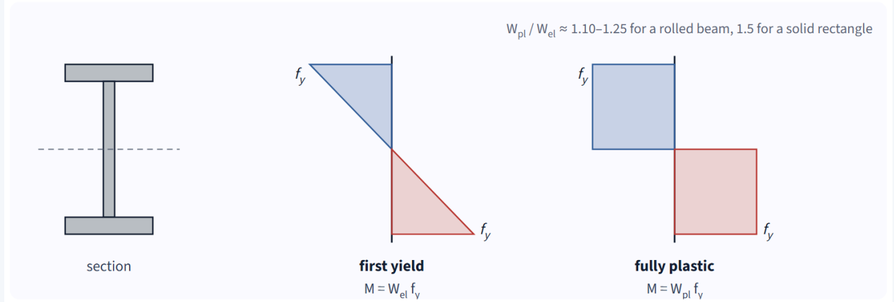
The loads from the envelope are **working values** (the Peck envelope is an upper bound on the measured loads); they are factored to the design value once: $E_d = \gamma_F E_k$. Each element is verified with the yield strength of its own steel grade (f_y); the section is given by the element's **section modulus** (sheet pile: W_{el} per metre; wale: W; strut: A and I about the weak axis).

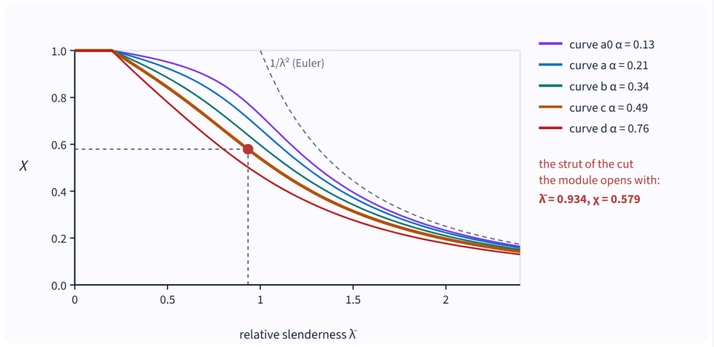
$$M_{c,Rd} = \frac{W f_y}{\gamma_{M0}} \qquad \eta = \frac{M_{Ed}}{M_{c,Rd}} \le 1 \qquad W = W_{pl}\ (\text{class 1–2}),\ W_{el}\ (\text{class 3})$$

$$N_{cr} = \frac{\pi^2 EI}{L_{cr}^2} \qquad \bar{\lambda} = \sqrt{\frac{A f_y}{N_{cr}}} \qquad \Phi = 0.5\left[1 + \alpha(\bar{\lambda} - 0.2) + \bar{\lambda}^2\right] \qquad \chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}} \le 1 \qquad N_{b,Rd} = \frac{\chi A f_y}{\gamma_{M1}}$$

The strut buckles about its weak axis: I is the small moment of inertia, $L_{cr} = K L_b$ is the effective length (excavation width, if no king pile or cross bracing passes in between). α comes from the buckling curve (curve c: 0.49).

For sheet piles the same resistance applies per unit width (EN 1993-5; interlock slip β_B for U-profiles, 1.0 for Z-profiles). Since the compression flange of the wale bears against the wall, lateral torsional buckling is not governing.





Problem statement

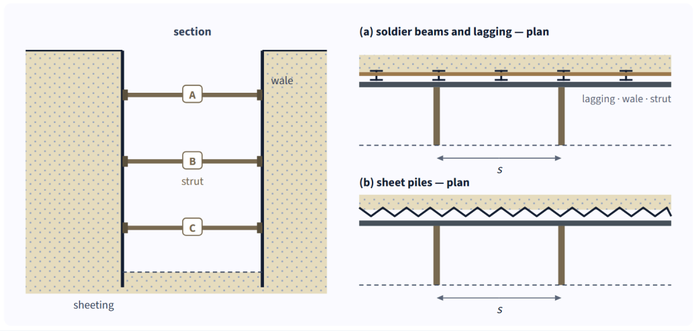
EXAMPLE

Given

Excavation and loads	**The cut in Example 1 and the loads in Example 3: $P_{max} = 271.26$ kN (support B)**
Elements	**S275 ($f_y = 275$ MPa), $\gamma_F = 1.35$, $\gamma_{M0} = \gamma_{M1} = 1.00$: sheet pile $W_{el} = 600$ cm³/m; wale $W_{el,y} = 1\,000$ cm³; strut $A = 65.7$ cm², $I = 3\,598$ cm⁴, curve c, $L_{cr} = 6.00$ m**

REQUIRED

η for the sheet pile, the most loaded wale and the most loaded strut (≤ 1).



Solution (1/2)

SOLUTION

1 Sheet pile and wale (bending): $\eta = \gamma_F M/(W f_y)$

$$\eta_{sp} = \frac{1.35(27.03)}{600 \cdot 10^{-6}(275\,000)} = \frac{36.49}{165.0} = \mathbf{0.221}$$

$$\eta_{belt} = \frac{1.35(101.72)}{1\,000 \cdot 10^{-6}(275\,000)} = \frac{137.33}{275.0} = \mathbf{0.499}$$

Sheet pile η = **0.221**, wale η = **0.499**: both are sufficient ($\eta \leq 1$).

Solution (2/2)

SOLUTION

2 Strut (buckling): $N_{Ed} = 1.35\,P_{max} = 1.35(271.26) = 366.20$ kN

$$N_{cr} = \frac{\pi^2(210\,000\,000)(3\,598 \cdot 10^{-8})}{6.00^2} = 2{,}071.5\ \text{kN}$$

$$\bar{\lambda} = \sqrt{\frac{A f_y}{N_{cr}}} = \sqrt{\frac{1\,806.8}{2{,}071.5}} = 0.9339$$

$$\Phi = 0.5\left[1 + 0.49(0.9339 - 0.2) + 0.9339^2\right] = 1.1159$$

$$\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}} = 0.5791 \qquad N_{b,Rd} = \chi A f_y = 1{,}046.4\ \text{kN}$$

η_{strut} = 366.2 / 1,046.4 = **0.350**. All three members are adequate ($\eta \le 1$); **buckling** governs the strut ($N_{c,Rd}$ = 1,806.8 kN, $N_{b,Rd}$ = 1,046.4 kN).

Check the solution in GeoTerva ↗

LRFD or ASD; classes, bending, shear, column curve

THEORY

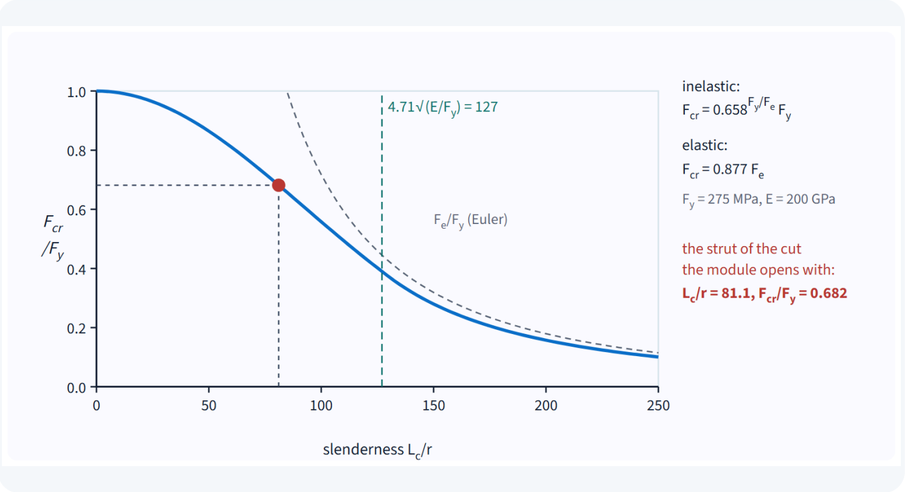
Each member is verified with the **nominal strength R_n** found from the compression, bending and shear provisions. In LRFD the load is factored and the strength reduced by φ; in ASD the working load is compared with the strength divided by the safety factor Ω:

$$R_u \le \phi R_n \ (\text{LRFD}) \qquad R_a \le \frac{R_n}{\Omega} \ (\text{ASD}) \qquad P_u = \gamma_H P,\ M_u = \gamma_H M,\ \gamma_H = 1.6$$

The loads coming out of the envelope are the **lateral earth pressure H** of ASCE 7-22: in LRFD combinations, it enters with 1.6 wherever it is added to the load (always in excavation bracing), and with 1.0 in ASD. The sheet pile and the wale given with the module are taken with the strength claimed by the module: F_yZ at compact, F_yS at first yield. Since the wale is laterally restrained by the wall, $L_b = 0$ and lateral-torsional buckling is not a limit state.

$$F_e = \frac{\pi^2 E}{(L_c/r)^2} \qquad F_{cr} = 0.658^{F_y/F_e} F_y \ \left(\frac{L_c}{r} \le 4.71\sqrt{E/F_y}\right) \ \text{otherwise } 0.877F_e \qquad P_n = F_{cr} A_g$$

A single column curve serves every section (no curve to be selected as in EN 1993-1-1); r is the small radius of gyration, $L_c = K\,L_b$ the effective length; $L_c/r \le 200$ is recommended. AISC, unlike EN 1993-1-1, does not reduce the flexural strength due to shear force.



Problem statement

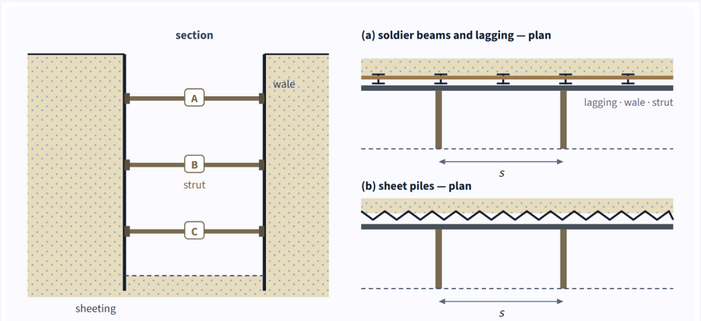
EXAMPLE

Given

Excavation and loads	**The cut in Example 1 and the loads in Example 3: $P_{max} = 271.26$ kN (support B)**
Elements	**Elements from Example 5 (S275, $F_y = 275$ MPa); $E = 200$ GPa; LRFD $\gamma_H = 1.6$, $\phi_c = \phi_b = 0.90$; ASD $\Omega_c = 1.67$, $\Omega_b = 1.67$ ($M_n = F_y S$)**

REQUIRED

For the most heavily loaded strut: **L_c/r, F_e, F_{cr}, P_n**; LRFD and ASD ratios; LRFD ratio for the sheet pile and the wale.



Solution (1/2)

SOLUTION

1 Support: column curve (A_g = 65.7 cm², r = 7.40 cm, L_c = 6.00 m)

$$\frac{L_c}{r} = \frac{600}{7.40} = 81.1 \;\leq 4.71\sqrt{E/F_y} = 127$$

$$F_e = \frac{\pi^2 E}{(L_c/r)^2} = \frac{\pi^2(200\,000)}{(81.1)^2} = 300.3\ \text{MPa}$$

$$F_{cr} = 0.658^{275/300.3}(275) = 187.4\ \text{MPa}$$

$$P_n = F_{cr}A_g = 187.4 \times 6\,570\ \text{mm}^2 = \mathbf{1{,}231.5}\ \text{kN}$$

At the support **nominal strength P_n = 1,231.5 kN**; slenderness L_c/r = 81.1, in the inelastic buckling range (limit 127).

Solution (2/2)

SOLUTION

2 LRFD and ASD ratios (P = 271.26 kN)

$$\text{LRFD: } P_u = 1.6(271.26) = 434.0\ \text{kN} \leq \phi_c P_n = 0.90(1{,}231.5) = 1{,}108.3\ \text{kN} \quad \eta = \mathbf{0.392}$$

$$\text{ASD: } P_a = 271.26\ \text{kN} \leq \frac{P_n}{\Omega_c} = \frac{1{,}231.5}{1.67} = 737.4\ \text{kN} \quad \eta = \mathbf{0.368}$$

Bracing: LRFD η = **0.392**, ASD η = **0.368**. Sheet pile (LRFD) 0.291, waler 0.658; in EN 1993-1-1 they were 0.221, 0.499 and 0.350.

Check the solution in GeoTerva ↗

Terzaghi column, Chang's correction

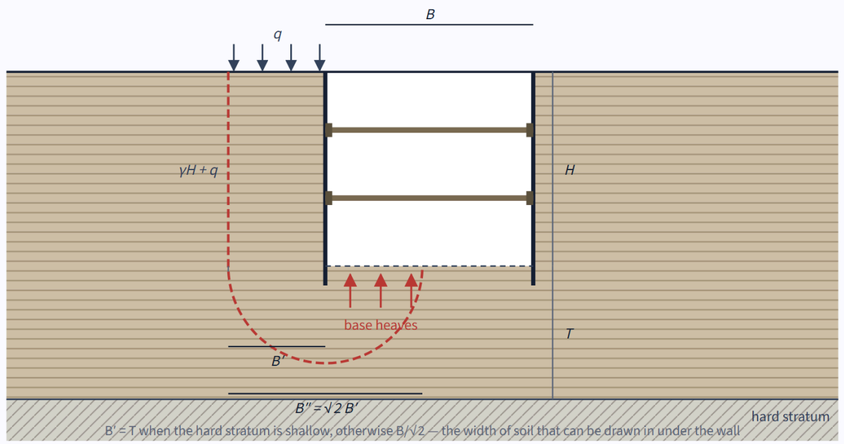
THEORY

A braced cut can fail even without the struts collapsing: the weight of the soil beside the excavation and of the surcharge squeezes the clay below the base inward and upward. The soil column of width B′ bears on the clay at base level with a bearing capacity cN_c (N_c = 5.7); the shearing resistance cH on the side faces relieves it:

$$FS = \frac{cN_c}{\gamma H + q - cH/B'}$$

The equation has been **corrected in three respects**: the side shear load does not reduce, it **adds resistance**; the base strip of width B″ behaves like an upward-loaded foundation; since the excavation base is unbounded, N_c = 5.14 (smooth foundation):

$$FS = \frac{5.14\,c\left(1 + 0.2\frac{B''}{L}\right) + \frac{cH}{B'}}{\gamma H + q} \qquad B' = \min\left(T, \frac{B}{\sqrt{2}}\right),\ B'' = \sqrt{2}\,B'$$



B excavation width, L plan length, T clay thickness below the base; q surcharge beside the excavation. In fully collapsed excavations FS ≈ 1, while in partially/minimally failed excavations it comes out somewhat above; required FS ≈ **1.5**.

In homogeneous clay, if FS < 1.5 the sheet piles are driven deeper (generally not more than B/2 below the base); the embedded length carries a force taken from published design charts.

Problem statement

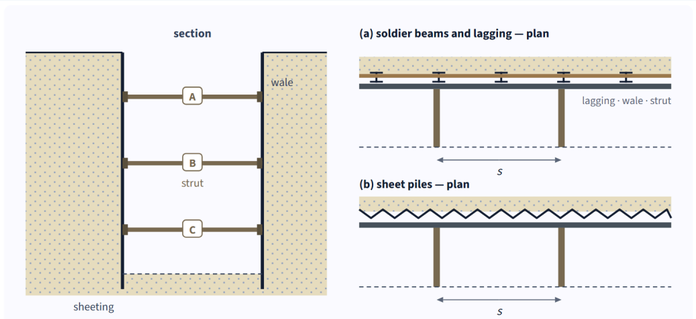
EXAMPLE

Given

Open excavation	$H = 7$ m, $B = 6$ m, $L = 20$ m; clay $\gamma = 18$, $c = 35$ kPa, clay thickness below the base $T = 4$ m; no surcharge
Book excavation	4 m wide, 15 m long, 6 m deep; $\gamma = 17$, $c = 40$ kPa; stiff layer 2 m below the base

REQUIRED

For both excavations: **B′**, **B″** and **FS** (≥ 1.5?); in the open excavation, if there is no hard layer and the width is 10 m?



Solution

SOLUTION

1 Excavation at an angle: B/√2 = 4.24 m > T = 4 m, therefore B′ = T

$$B' = T = 4.00\ \text{m} \qquad B'' = \sqrt{2}B' = 5.66\ \text{m}$$

$$FS = \frac{5.14(35)\left(1 + 0.2\frac{5.66}{20}\right) + \frac{35(7)}{4.00}}{18(7)} = \frac{190.1 + 61.2}{126} = \mathbf{1.99} \geq 1.5\ \checkmark$$

2 Basement excavation (T = 2 m ≤ B/√2 = 2.83 m: B′ = T = 2.00 m, B″ = 2.83 m)

$$FS = \frac{5.14(40)\left(1 + 0.2\frac{2.83}{15}\right) + \frac{40(6)}{2.00}}{17(6)} = \mathbf{3.27}$$

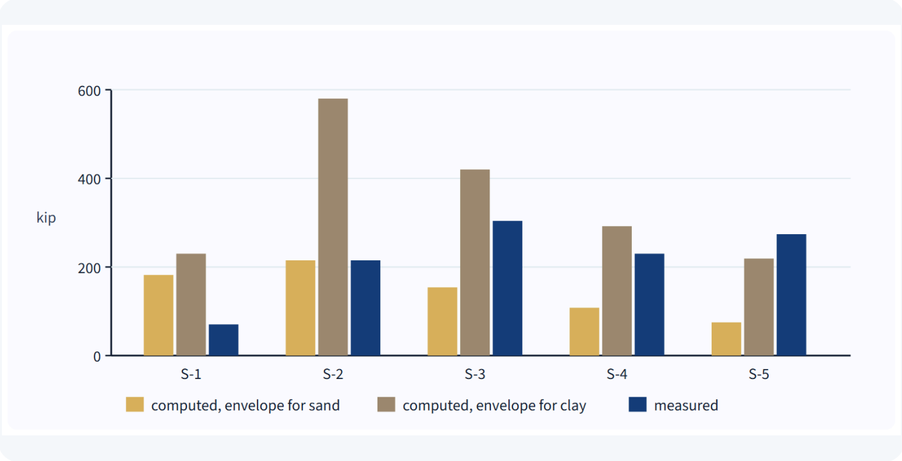
The open-cut excavation gives **FS = 1.99**, the book excavation **3.27**: both ≥ 1.5. If the stiff layer is not encountered (B′ = B/√2), FS for the book excavation drops to 2.96; the open-cut excavation drops to 1.80 for a 10 m width and without the stiff layer (it decreases in the conservative direction): base heave worsens with width and with the absence of the stiff layer.

Check the solution in GeoTerva ↗

Boston and Chicago case studies

THEORY

Boston (test section B): 17.7 m deep, five supports; the profile was bounded by two envelopes for sand and clay (sand envelope 53.5 kPa, soft–medium clay envelope 146.1 kPa); the measured loads turned out very different from both estimates (soil parameters are uncertain), and the supports had been designed for loads far above the measured ones. **Chicago, National Plaza**: 21.3 m deep, six supports; the profile was converted to an equivalent clay (c_{av} = 34.91 kPa, γ_{av} = 19.76 kN/m³, γH/c = 12.08 > 4, σ_a = 282.0 kPa). The Peck envelope gave a pressure about **1.8 times** the observed one: an envelope drawn around many excavations is conservative for a single excavation; on the same soil, field experience can substantially reduce overdesign.



What the method assumes

THEORY

The envelopes were collected from excavations deeper than about 6 m; **the envelope is not the actual pressure on the wall at any instant**: it gives a safe support load, not a wall deflection. The clay is **undrained**; pore water pressure is not additionally accounted for.

Multiple clays are averaged by thickness (c_{av}, γ_{av}); γH/c is selected with these averages. Surcharge is added beside the envelope (Δp = q); water only changes the unit weight.

A **sheet pile** is counted as a chain of discrete hinges; its own stiffness and the embedment below the base are not used. The **wales** are simple spans between supports (retainers). Steel is verified with EN 1993-1-1 (class 4 is not designed) or with AISC 360-22.

Base heave is checked with Chang's corrected equation; the embedment of the sheet pile below the base is not counted (conservative). For multiple layers, the formula is used with GeoTerva's layered extension.

Lateral movement and ground loss around the excavation are expected during construction; if a stiff layer is present, the sheet piles are embedded into it. In soft clay, at γH/c ≈ 8 large displacements and failure of the struts may occur.

Summary

Key points

Envelope: $\gamma H/c > 4$ soft–medium ($\sigma_a = \max(\gamma H[1 - 4c/\gamma H], 0.3\gamma H)$); ≤ 4 stiff (≈ $0.3\gamma H$). Example: $\gamma H/c = 3.6$, $\sigma_a = 37.8$ kPa

Surcharge and water: $\Delta p = q$ is added separately; water changes only $\bar{\gamma}$. **Tension crack** $z_c = 2c/\gamma = 3.89$ m; the first support (1.0 m) is above it

Support load: the envelope is divided into parts at the hinges, reactions are found by statics; in the example $P_B = 271.26$ kN (sums the two reactions)

Sizing: sheet pile $M_{max} = 27.03$ kN·m/m ($S = 159$ cm³/m), wale $M = R\,s^2/8 = 101.72$ kN·m ($S = 598$ cm³; $\sigma_{allow} = 170$ MPa)

Steel: EN 1993-1-1 $\eta = 0.221 / 0.499 / 0.350$ (sheet pile / wale / strut); AISC LRFD strut 0.392, ASD 0.368

Base heave: $FS = (5.14c(1 + 0.2B''/L) + cH/B')/(\gamma H + q) \geq 1.5$; 2.00 for the open excavation, 3.27 for the textbook excavation

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NOTE

All numerical examples were generated with the module's own calculation engine; the module yields the same values. The worked example on the module page is also this excavation (H = 7 m, γ = 18 kN/m³, c = 35 kPa, struts at 1 · 3.5 · 6 m).

GeoTerva

Online geotechnical analysis

From theory to practice

Enter your excavation's clay profile, support levels, surcharge, water and members in GeoTerva; the module reports the envelope, tension crack, support loads, sheet-pile and wale moments, steel verification and base heave together.

Open the module →

Theory page →

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If you would like to use them in your lectures: info@geoterva.com