Geotechnical lecture presentation · Sheet piles and braced cuts

Braced cut in sand

Lecture: braced (strutted) excavation in sand; the Peck pressure envelope, strut loads, sheet pile and wale design, steel verification and base heave; a worked example on every topic.

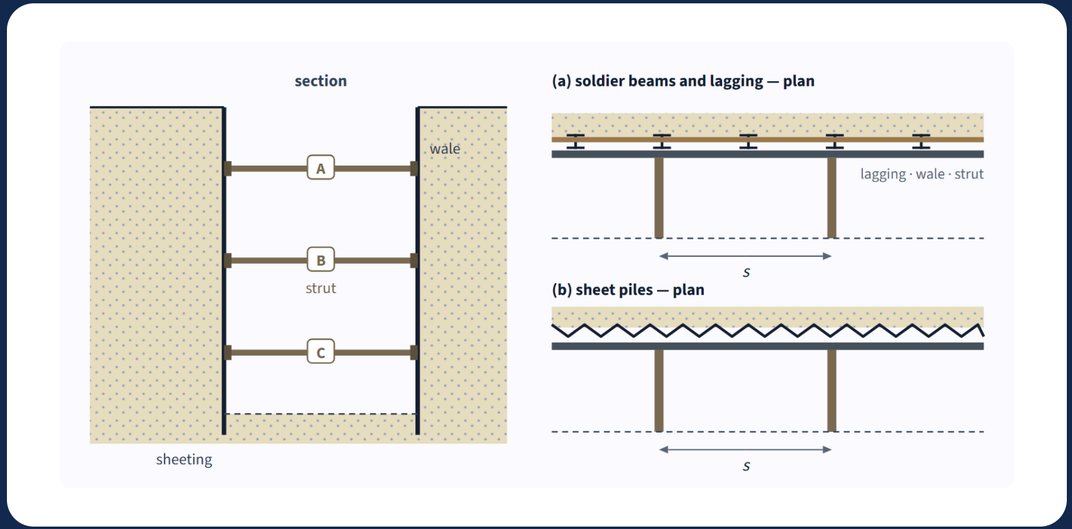
Excavation support

Peck envelope (sand)

Strut · waler · sheet pile

EN 1993 · AISC 360

Piping at the base



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 support loads
- For sand, the **Peck envelope $\sigma_a = 0.65\ \gamma\ H\ K_a$** and three rules for K_a in multiple sand layers
- The addition of **water** and **surcharge** as separate diagrams alongside the envelope
- From the envelope to **support loads** (hinged segments, reactions A, B_1, B_2, C, $P = R \cdot s$)
- The design of the **sheet pile** (M_{max}, S) and the **wale** ($M = R\ s^2/8$); verification of the steel with **EN 1993-1-1 and AISC 360-22**
- Check of **piping** at the base of the sand (exit gradient, $FS = i_{cr}/i_{exit}$)

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: sand ($\gamma = 17.6$ kN/m³, $\varphi' = 32°$), depth H = 8.00 m, width B = 6.00 m; struts at depths 1.80 · 4.60 · 7.30 m, horizontal spacing s = 3.00 m. Forces are per meter of wall (strut loads multiplied by s).

Support system and its elements

THEORY

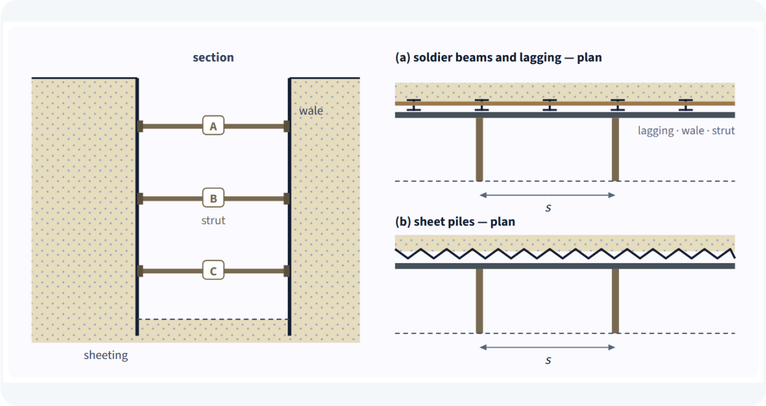
Deep excavations with vertical faces (urban basements, cut-and-cover tunnels) are kept open with a temporary **support system**. Two arrangements are common: **vertical piles** (steel or timber) driven in advance, with horizontal timber lagging placed between them as excavation proceeds; or interlocking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

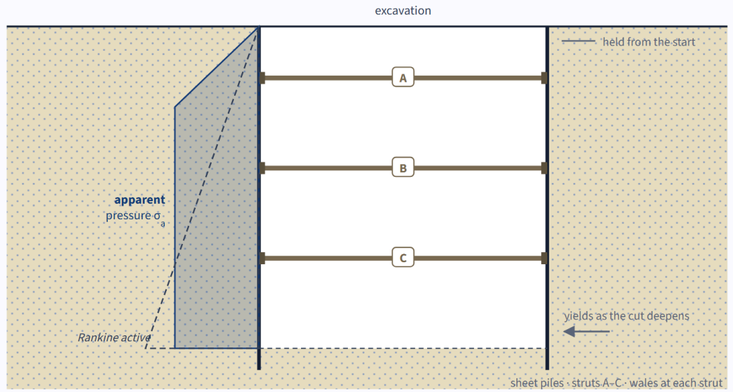
A cantilever retaining wall rotates about its base, and the Rankine active pressure (zero at the top, largest at the bottom) describes it well. **A braced wall moves in the opposite sense**: the first strut is placed while the excavation is shallow, so the upper part barely moves; each subsequent stage 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 ($r = r_o e^{\theta \tan \varphi'}$); 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

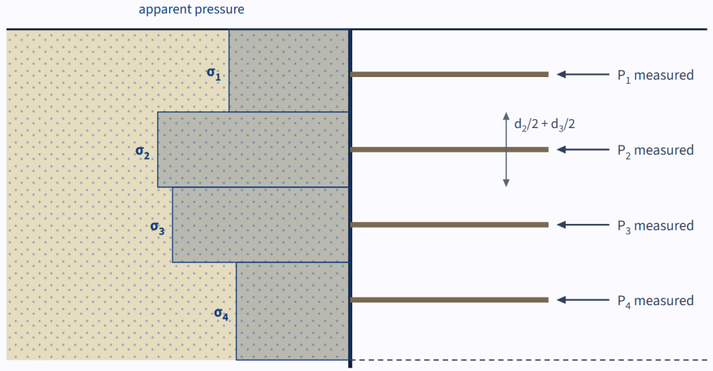
Pressure varies from point to point, and each support must be sized for the **maximum load** it may encounter. Envelopes are constructed for this purpose: in an instrumented excavation, the load on each support is measured, divided by the sheet-pile strip it supports (horizontal spacing s × half the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



each strut load spread over the band of wall it serves: s × (half the distance to each neighbour)

$\sigma_a = 0.65\,\gamma\,H\,K_a$, over the full height

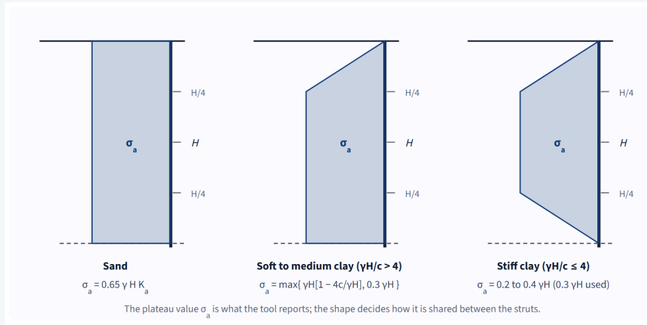
THEORY

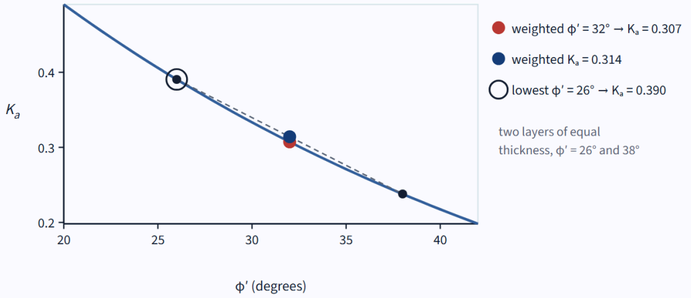
In sand the Peck envelope is a **rectangle**: a single pressure over the full excavation height, 65% of the largest ordinate of the Rankine active triangle:

$$\sigma_a = 0.65\,\gamma H K_a \qquad K_a = \tan^2\left(45^\circ - \frac{\varphi'}{2}\right)$$

The envelopes are provided under the following conditions: excavation depth greater than about 6 m; **groundwater below the excavation base**; drained sand (pore water pressure zero).

The envelope requires a single K_a; the source gives no rule for **multiple sands**. Three rules are used and they disagree, because K_a is a convex function of φ′ (the average taken within the curve lies below the curve): $K_a(\bar{\varphi} = \Sigma\varphi H/H)$, the average $K_a = \Sigma K_{a,i} H/H$, and $K_a(\varphi'_{min})$.





Problem statement

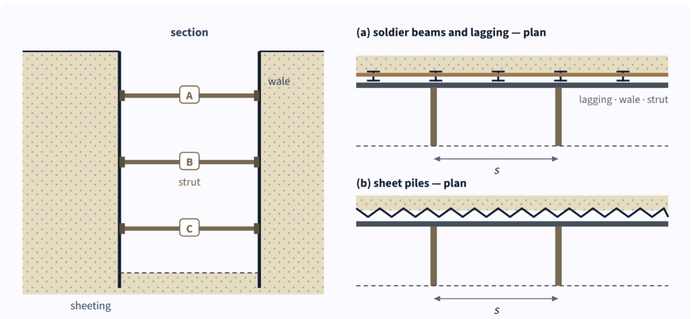
EXAMPLE

Given

Excavation	$H = 8.00$ **m, single sand** $\gamma = 17.6$ **kN/m³,** $\varphi' = 32^\circ$
Three-layer profile	**0–3 m:** $\gamma = 17$, $\varphi' = 28^\circ$**; 3–6 m:** 18, 38°**; below 6 m:** 18.5, 33° **(same excavation)**

REQUIRED

(a) **K_a** and **σ_a**; (b) the K_a and σ_a values of the three rules for the three-layer profile.



Solution

SOLUTION

1 (a) Single sand

$$K_a = \tan^2(45^\circ - 16^\circ) = \mathbf{0.3073}$$

$$\sigma_a = 0.65\,\gamma H K_a = 0.65(17.6)(8.00)(0.3073) = \mathbf{28.12}\ \mathrm{kPa}$$

2 (b) Three layers: weighted $\bar{\gamma}$ = 17.75 kN/m³ according to the excavation height (layer thicknesses 3 + 3 + 2 m)

Rule	K_a	$\sigma_a = 0.65\,\bar{\gamma} H K_a$ (kPa)	P_{max} (kN)
$K_a(\bar{\varphi} = 33.0°)$	0.2948	27.21	308.5
average $K_a = \Sigma K_{a,i} H_i / H$	0.2983	27.53	312.1
$K_a(\varphi'_{min} = 28°)$	0.3610	33.32	377.7

In a single sand the envelope is **28.12 kPa**. The K_a values of the three rules vary between 0.295 – 0.361: the calculation with the lowest φ′ (σ_a = 33.3 kPa) gives a value 22% larger than the averages; the average-taking rules remain inside the curve, and **the conservative one is φ'_{min}.**

Check the solution in GeoTerva ↗

Added beside the envelope, not inside it

THEORY

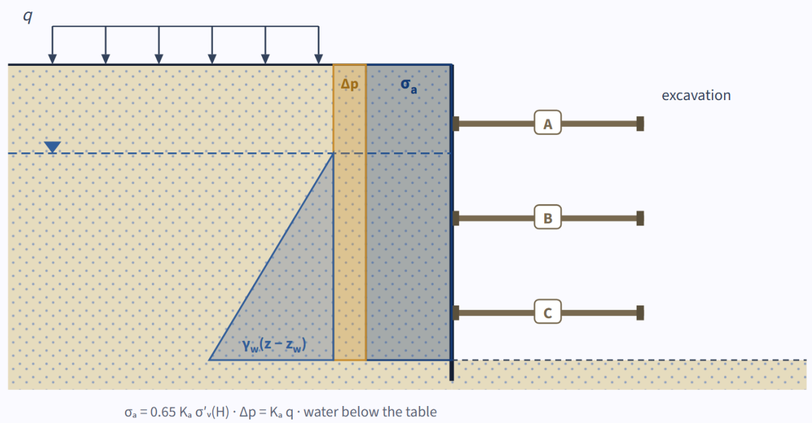
The envelopes are constructed from excavations with no load below the base and beside the cut. The module adds both alongside the envelope as **separate diagrams**; the envelope is not scaled beyond the range in which it was measured.

Water: Peck's sand envelope is an effective stress diagram. If water is inside the excavation, the overburden stress at the base level is taken as effective (natural above water, submerged unit weight below); water also presses on the wall over the full height and the supports carry both together.

Surcharge q is not included in the envelope (the empirical 0.65 would also apply to it); it is carried as its own uniform pressure over the entire height.

$$\sigma'_v(H) = \sum \gamma_i h_i + \sum (\gamma_{sat,i} - \gamma_w) h_i \qquad \sigma_a = 0.65 K_a \sigma'_v(H) \qquad u(z) = \gamma_w (z - z_w)$$

$$\Delta p_q = K_a q \qquad R = \sigma_a H + \Delta p_q H + \tfrac{1}{2} \gamma_w (H - z_w)^2$$



Problem statement

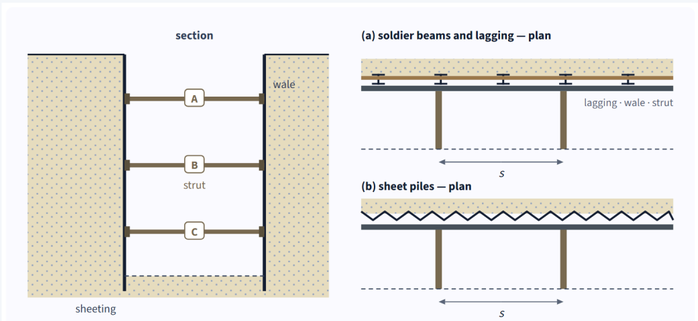
EXAMPLE

Given

Excavation	**The cut in Example 1(a) ($H = 8$ m, $\gamma = 17.6$, $\gamma_{sat} = 20$ kN/m³, $\varphi' = 32°$)**
Cases	**(a) no water, no surcharge; (b) groundwater at $z_w = 3$ m; (c) uniform surcharge $q = 10$ kPa**

REQUIRED

In each case σ_a, the surcharge and water pressure; the **total load R** and the maximum support load.



Solution (1/2)

SOLUTION

1 (b) Water: effective overburden stress and envelope; water also acts over the full height

$$\sigma'_v(H) = 17.6(3) + (20 - 9.81)(5) = 103.75 \text{ kPa}$$

$$\sigma_a = 0.65(0.3073)(103.75) = 20.72 \text{ kPa}$$

$$R = \sigma_a H + \tfrac{1}{2}\gamma_w (H - z_w)^2 = 20.72(8) + \tfrac{1}{2}(9.81)(5)^2 = \mathbf{288.41} \text{ kN/m}$$

Under water the effective overburden decreases (σ_a 28.12 → 20.72 kPa) but the water acts on the sheet pile at full height: R = **288.4 kN/m**.

Solution (2/2)

SOLUTION

2 (c) Surcharge: separate uniform pressure; summary of (a), (b), (c)

$$\Delta p_q = K_a q = 0.3073(10) = 3.073\ \text{kPa}$$

$$R = \sigma_a H + \Delta p_q H = 28.12(8) + 3.073(8) = 249.52\ \text{kN/m}$$

Condition	σ_a (kPa)	Ek (kPa)	R (kN/m)	P_{max} (kN)	M_{max} sheet pile (kN·m/m)
(a) No water, q = 0	28.12	—	224.96	318.8	45.55
(b) Water z_w = 3 m	20.72	u(H) = 49.05	288.38	376.7	37.09
(c) q = 10 kPa	28.12	Δp_q = 3.07	249.52	353.5	50.53

The water envelope **reduces** it but adds water pressure: R 225.0 → **288.4 kN/m**. Surcharge does not touch the envelope; it adds Δp_q = 3.07 kPa (R = 249.5 kN/m).

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:

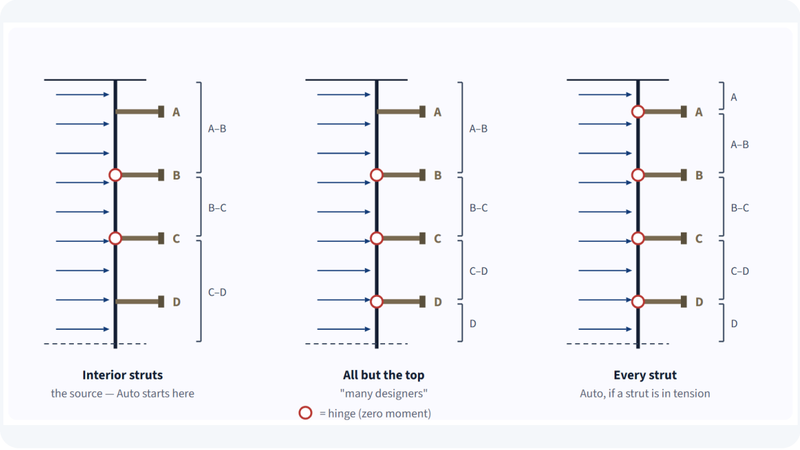
> Draw the pressure envelope and the support levels. Take the sheet pile **hinged at every support level** except the topmost and the lowest (many designers do so at all except the topmost).

> 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

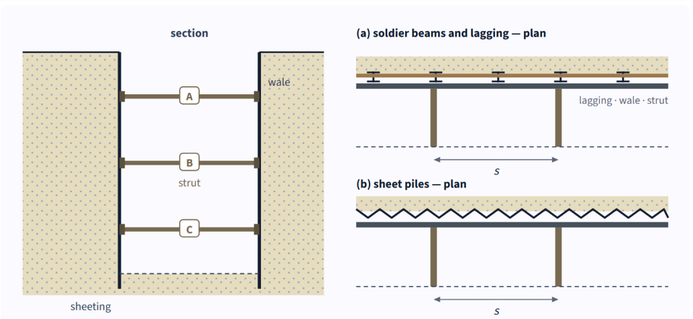
EXAMPLE

Given

Excavation	$H = 8.00$ m; $B = 6$ m; struts $z = 1.80, 4.60, 7.30$ m (A, B, C); horizontal spacing $s = 3.00$ m
Pressure	Example 1(a): continuous rectangular envelope $\sigma_a = 28.12$ kPa; sheet pile hinged at support B (except at the very top and 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 – 4.60	129.35	2.300	A (z = 1.80), B (z = 4.60)
2	4.60 – 8.00	95.61	6.300	B (z = 4.60), C (z = 7.30)

2 First part: moment equilibrium about the first support

$$R_{B_2} = \frac{W(\bar{z} - z_A)}{z_B - z_A} = \frac{129.35(2.300 - 1.80)}{4.60 - 1.80} = \mathbf{23.10} \qquad R_A = W - R_{B_2} = \mathbf{106.25}\ \mathrm{kN/m}$$

The first segment carries the two supports with 106.25 and 23.10 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

$$R_C = \frac{W(\bar{z} - z_B)}{z_C - z_B} = \frac{95.61(6.300 - 4.60)}{7.30 - 4.60} = \mathbf{60.20} \qquad R_{B_1} = W - R_C = \mathbf{35.41}\ \mathrm{kN/m}$$

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	1.80	106.25	106.25	**318.75**
B	4.60	23.10 + 35.41	58.51	**175.53**
C	7.30	60.20	60.20	**180.60**

Maximum support load **P_A = 318.75 kN** (level A): 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

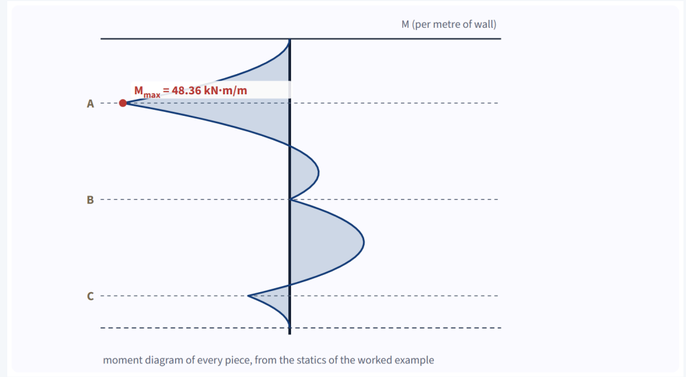
The **sheet pile** is designed from the same parts: the maximum bending moment is found in each part (where the shear changes sign: within the span or at the support where the cantilever meets the strut); the largest is the design moment for unit length of wall. The required section modulus follows from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

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

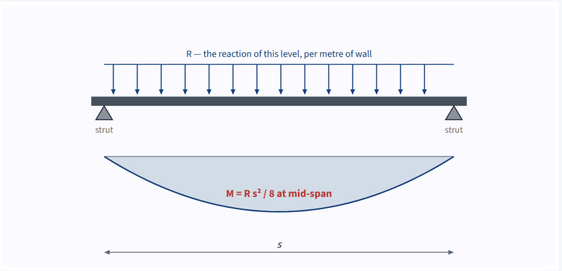
Struts may be considered continuous if properly connected; conservatively, they are taken as **hinged** at the supports: a simply supported beam of span s, with the level reaction R (per unit length of wall) as a distributed load:

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

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

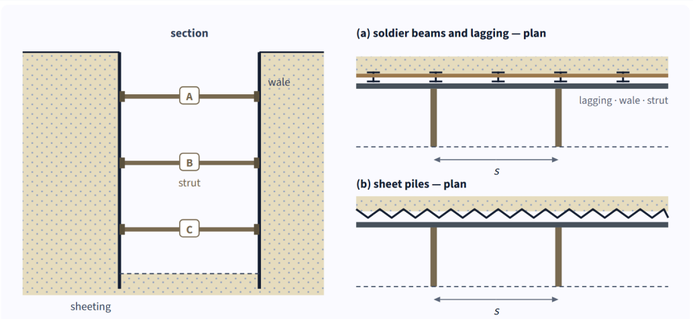
EXAMPLE

Given

Excavation	**depth $H = 27$ ft (8.23 m), sand $\gamma = 17.59$ kN/m³, $\varphi' = 32^\circ$; struts at 6, 15, 24 ft (1.83; 4.57; 7.32 m); horizontal spacing 12 ft (3.66 m); hinge at B**
Steel	**allowable bending stress $\sigma_{allow} = 165\,474$ 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 – 4.57	48.36	1.83
2	4.57 – 8.23	21.49	5.79

2 Required section modulus

$$S = \frac{M_{max}}{\sigma_{allow}} = \frac{48.36}{165{,}474} = 0.0002923\ \mathrm{m^3/m} = \mathbf{292.3}\ \mathrm{cm^3/m}$$

In the sheet pile **M_{max} = 48.36 kN·m/m** (segment 1, z = 1.83 m): **S ≥ 292.3 cm³/m**.

Solution (2/2)

SOLUTION

3 Wale: simple beam at each level (s = 3.658 m)

Level	R (kN/m)	M = R s²/8 (kN·m)	S = M/σ_{allow} (cm³)
A	110.18	184.25	1,113
B	57.30	95.82	579
C	70.52	117.93	713

The largest reaction is **at level A** (R = 110.18 kN/m): the wale is sized with **M_{max} = 184.25 kN·m**, **S ≥ 1,113 cm³**; the wales at the other levels may be smaller.

Check the solution in GeoTerva ↗

Design effects, class, bending and buckling

THEORY

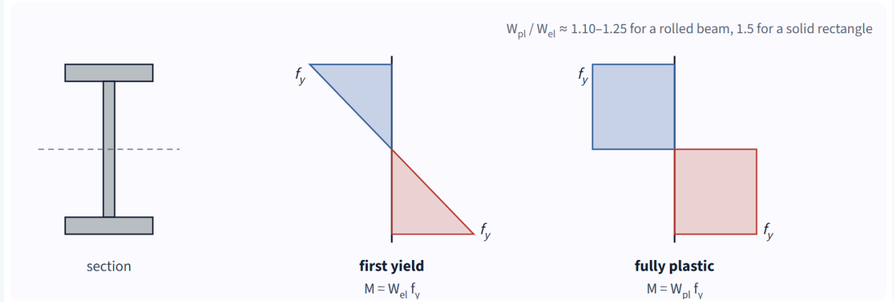
The loads from the envelope are **working values** (the Peck envelope is an upper bound on the measured loads); they are factored to the design value once: $E_d = \gamma_F E_k$. Each element is verified with the yield strength of its own steel grade (f_y); the section is given by the element's **section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

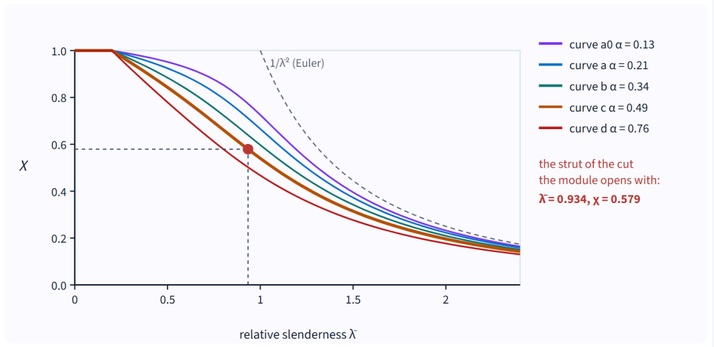
$$M_{c,Rd} = \frac{W f_y}{\gamma_{M0}} \qquad \eta = \frac{M_{Ed}}{M_{c,Rd}} \le 1 \qquad W = W_{pl}\ (\text{class 1–2}),\ W_{el}\ (\text{class 3})$$

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

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

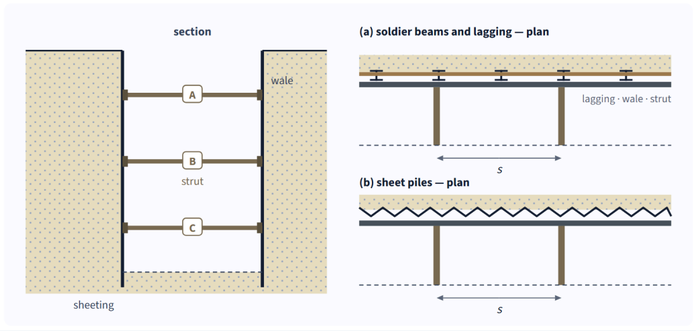
EXAMPLE

Given

Excavation and loads	**The excavation in Example 1(a) and the loads in Example 3: $P_{max} = 318.75$ kN (strut A)**
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): $\eta = \gamma_F M/(W f_y)$

$$\eta_{sp} = \frac{1.35(45.55)}{600 \cdot 10^{-6}(275\,000)} = \frac{61.50}{165.0} = \mathbf{0.373}$$

$$\eta_{belt} = \frac{1.35(119.53)}{1\,000 \cdot 10^{-6}(275\,000)} = \frac{161.37}{275.0} = \mathbf{0.587}$$

Sheet pile η = **0.373**, wale η = **0.587**: both are sufficient ($\eta \leq 1$).

Solution (2/2)

SOLUTION

2 Strut (buckling): $N_{Ed} = 1.35\ P_{max} = 1.35(318.75) = 430.31$ kN

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

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

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

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

η_{strut} = 430.3 / 1,046.4 = **0.411**. 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

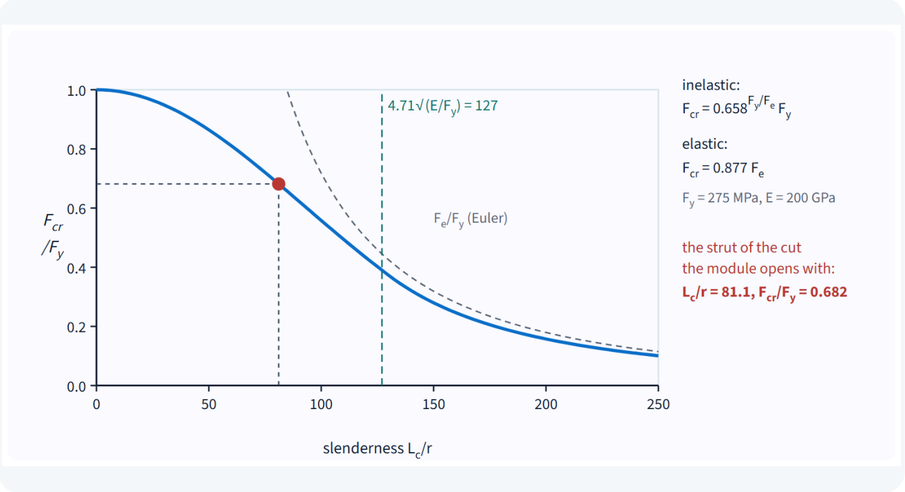
Each member is verified with the **nominal strength R_n** found from the compression, bending and shear provisions. In LRFD the load is factored and the strength reduced by φ; in ASD the working load is compared with the strength divided by the safety factor Ω:

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

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

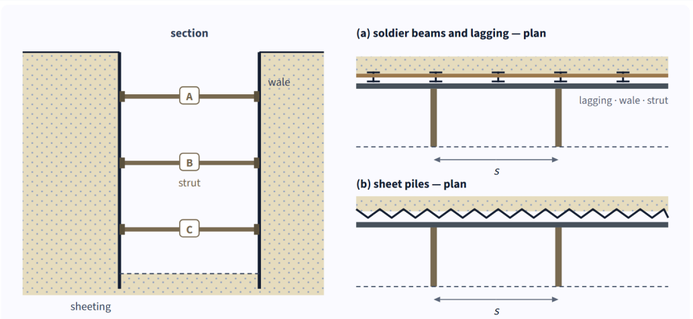
EXAMPLE

Given

Excavation and loads	**The excavation in Example 1(a) and the loads in Example 3: $P_{max} = 318.75$ kN (strut A)**
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)

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

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

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

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

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 = 318.75 kN)

$$\text{LRFD: } P_u = 1.6(318.75) = 510.0\ \text{kN} \leq \phi_c P_n = 0.90(1{,}231.5) = 1{,}108.3\ \text{kN} \quad \eta = \mathbf{0.460}$$

$$\text{ASD: } P_a = 318.75\ \text{kN} \leq \frac{P_n}{\Omega_c} = \frac{1{,}231.5}{1.67} = 737.4\ \text{kN} \quad \eta = \mathbf{0.432}$$

Bracing: LRFD η = **0.460**, ASD η = **0.432**. Sheet pile (LRFD) 0.491, waler 0.773; in EN 1993-1-1 they were 0.373, 0.587 and 0.411.

Check the solution in GeoTerva ↗

Exit gradient and critical gradient

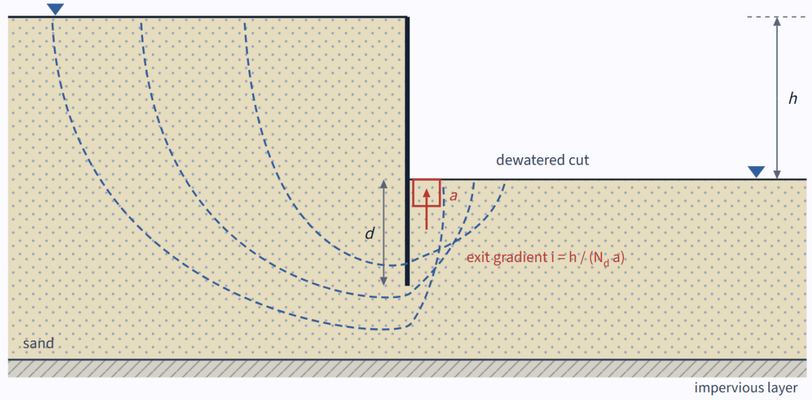
THEORY

A sand base is generally stable, and remains so as long as the water inside the excavation does not stand below the groundwater outside. When the excavation is dewatered, water flows under the wall and up through the base; the high upward gradient at the exit point can lift the grains: **the base boils (piping)**. The largest exit gradient beside the wall is read from the flow net:

$$i_{exit(max)} = \frac{h/N_d}{a} = \frac{h}{N_d\, a} \qquad FS = \frac{i_{cr}}{i_{exit(max)}} \qquad i_{cr} = \frac{G_s - 1}{1 + e}$$

h is the head loss in the flow net, N_d the number of drops, and a the length of the flow element in which the water exits the soil beside the wall. i_{cr} is 0.9–1.1 in most soils (average 1); the required FS ≈ **1.5**.

The exit gradient for a braced excavation in deep sand has been solved theoretically: from two nomographs, the modulus m (with $2L_2/B$ and $2L_1/B$) and $L_2 i_{exit}/h$ are obtained; $i = (L_2 i/h)_{nomograph} \cdot h/L_2$. L_1 is the depth of the external sand to the base level, and L_2 is the embedment of the wall below its base.



Problem statement

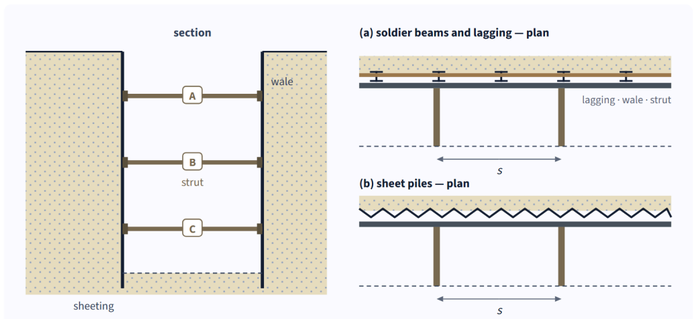
EXAMPLE

Given

Flow net (opening excavation)	**head loss $h = 3.0$ m, number of drops $N_d = 8$, exit element $a = 1.0$ m; $i_{cr} = 1.0$; $FS_{required} = 1.5$**
Chart example (textbook)	$h = 4.5$ m, $L_1 = 5$ m, $L_2 = 4$ m, $B = 5$ m: $2L_1/B = 2$, $B/2L_2 = 0.625$, $m \approx 0.033$, $L_2 i_{exit}/h = 0.54$

REQUIRED

i_{exit} and **FS** by both methods; decide against the required 1.5.



Solution

SOLUTION

1 Flow net: $i_{exit} = h/(N_d\, a)$

$$i_{exit(max)} = \frac{h}{N_d a} = \frac{3.0}{8(1.0)} = \mathbf{0.375}$$

$$FS = \frac{i_{cr}}{i_{exit(max)}} = \frac{1.0}{0.375} = \mathbf{2.67} \;\geq 1.5\;\checkmark$$

2 Chart method (deep sand)

$$i_{exit(max)} = \left(\frac{L_2 i_{exit}}{h}\right)_{nomograph} \frac{h}{L_2} = 0.54 \cdot \frac{4.5}{4} = 0.6075$$

$$FS = \frac{1.0}{0.6075} = \mathbf{1.646} \;\geq 1.5\;\checkmark$$

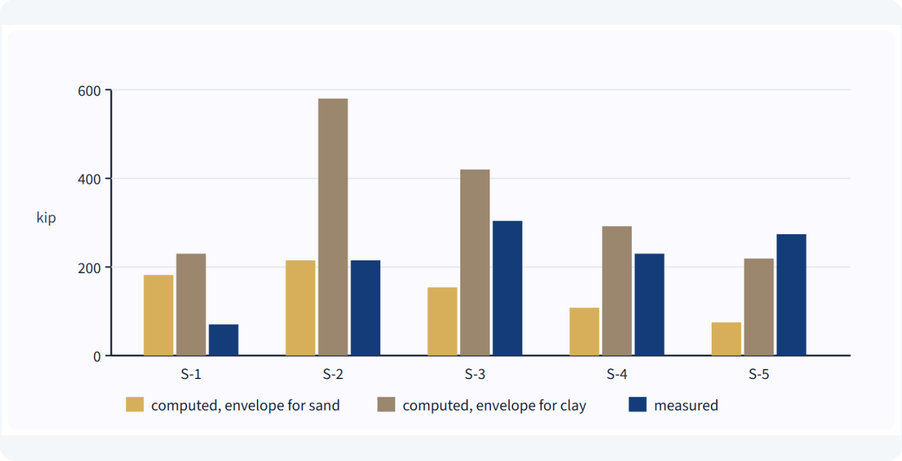
In the flow net the piping safety factor is **2.67**, from the chart **1.65**: both are above the required 1.5.

Check the solution in GeoTerva ↗

Boston and Chicago case studies

THEORY

Boston (test section B): 17.7 m deep, five struts (spaced 3.66 m); the profile was bounded by two envelopes for sand and clay: the sand envelope 53.5 kPa, the soft–medium clay envelope 146.1 kPa; the measured loads turned out very different from both estimates (soil parameters uncertain), and the struts were designed for loads far above the measured ones. **Chicago, National Plaza**: 21.3 m deep, six struts, clays beneath 13 ft of sand fill; the profile was converted to an equivalent clay: c_{av} = 34.91 kPa, γ_{av} = 19.76 kN/m³, $\gamma H/c$ = 12.08 > 4 → σ_a = 282.0 kPa. The Peck envelope gave a pressure about **1.8 times** the observed value: an envelope drawn around many excavations is conservative for a single excavation; field experience in the same soil can substantially reduce overdesign.



What the method assumes

THEORY

The envelopes were compiled from excavations deeper than about 6 m; shallower excavations are outside the data, and the module notes this. **An envelope is not the actual pressure acting on the wall at any given moment**: it gives a safe support load, not a wall deflection.

Groundwater is assumed below the base; the module keeps the sand envelope within the effective overburden and, if water is above, adds the hydrostatic pressure alongside it; seepage into the excavation is checked at the base.

The **sheet pile** is regarded as a chain of simply supported beams at the specified support levels; its own stiffness and the embedment below the base are not used. The **wales** are simple spans between supports (conservative for a continuous wale spanning several supports).

Steel is verified with EN 1993-1-1; class 4 sections are not designed, lateral torsional buckling of the wales is not checked. There is an AISC 360-22 LRFD/ASD option.

Some lateral movement of the wall is expected: the amount depends on the time between excavating a stage and installing the walings and struts. The movement causes **ground loss** in the surrounding soil; from 150 cases, the maximum settlement is ≈ 1.1% of the excavation depth in soft clay, 0.2% in stiff clay and 0.3% in cohesionless soil.

Summary

Key points

Envelope: rotates about the top of the braced wall; the pressure is not the Rankine value but an empirical diagram drawn around the measured loads. **Rectangular in sand**: $\sigma_a = 0.65\ \gamma\ H\ K_a$ (example: 28.12 kPa)

Multiple sands: $K_a(\bar{\varphi})$, the average K_a, and $K_a(\varphi'_{min})$ do not agree (convex); φ'_{min} is conservative

Water and surcharge are separate: water reduces the effective overburden but adds full hydrostatic pressure (R: 225 → 288 kN/m); q adds $\Delta p = K_a q$

Support load: the envelope is divided into parts at the hinges, reactions are found by statics, P = (summed reactions)·s; in the example P_A = 318.75 kN

Sizing: $S = M_{max}/\sigma_{allow}$ (sheet pile), wale $M = R\ s^2/8$; in the textbook excavation the sheet pile is 292 cm³/m, the wale 1 113 cm³

Steel: EN 1993-1-1 η = 0.373 / 0.587 / 0.411 (sheet pile / wale / strut buckling); AISC LRFD strut 0.460, ASD 0.432

Piping: $FS = i_{cr}/i_{exit} \geq 1.5$; in the example 2.67 (flow net) and 1.65 (chart)

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NOTE

All numerical examples have been generated with the module's own calculation engine; the module gives the same values. The worked example on the module page is also these excavations (textbook excavation: 27 ft, γ = 112 pcf, φ' = 32°; the module's opening excavation: 8 m).

Online geotechnical analysis

From theory to practice

Enter your own excavation profile, support levels, groundwater, surcharge and structural elements in GeoTerva; the module gives the envelope, support loads, sheet-pile and waler moments, steel verification and base heave together.

Open the module →

Theory page →

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The example modules need no registration.

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