Example – Example Information

podk · wpodk · Block odpwk · Parcel dpow

SHORING DESIGN REPORT

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SYMBOLS AND ABBREVIATIONS

Geometry

Table 1: Symbol · Description · Unit

Symbol	Description	Unit
H	Excavation depth	m
L	Total length of the excavation support element	m
d	Embedment depth, d = L − H	m
D	Bored pile diameter	m
s	Pile or soldier spacing	m
t	Diaphragm wall thickness	m
z	Depth (positive downward from ground surface)	m
L_{free}	Anchor free length	m
Lroot	Anchor bond length	m
D_k	Anchor bond body (grout body) diameter	m
S	Anchor or strut plan spacing	m
α	Anchor inclination (angle to the horizontal)	°

Soil parameters

Table 2: Symbol · Description · Unit

Symbol	Description	Unit
γ	Natural unit weight	kN/m³
γ_{sat}	Saturated unit weight	kN/m³
γ'	Effective (submerged) unit weight, $\gamma' = \gamma_{sat} - \gamma_w$	kN/m³
γ_w	Unit weight of water (9.807)	kN/m³
phi'	Effective angle of internal friction	°
c'	Effective cohesion	kPa
c_u	Undrained shear strength	kPa
δ	Soil–wall interface friction angle	°
v	Poisson's ratio	—
OCR	Overconsolidation ratio	—
E_{oed}	Oedometer (constrained) modulus of elasticity	kPa
E_s	Modulus of elasticity (triaxial)	kPa
EM	Pressuremeter modulus	kPa

Stress and pressure

Table 3: Symbol · Description · Unit

Symbol	Description	Unit
σ_v	Total vertical stress	kPa
σ'_v	Effective vertical stress, $\sigma'_v = \sigma_v - u$	kPa
u	Pore water pressure	kPa
K_a	Active soil pressure coefficient	—
K_0	At-rest earth pressure coefficient	—
K_p	Passive earth pressure coefficient	—
K_{AE}	Seismic active pressure coefficient (Mononobe–Okabe)	—
pa	Active horizontal pressure	kPa
pp	Passive horizontal pressure	kPa
Pa	Active thrust (resultant of the pressure)	kN/m
Pp	Passive resistance (resultant of the pressure)	kN/m
q	Surcharge (surface load)	kPa
z0	Tension crack depth	m

Structural quantities

Table 4: Symbol · Description · Unit

Symbol	Description	Unit
δ_h	Horizontal displacement	mm
$\delta_{h,lim}$	Allowable horizontal displacement	mm
δ_v	Vertical settlement	mm
θ	Angular distortion, $\theta = \Delta\delta v / L$	—
M	Bending moment	kNm/m
V	Shear force	kN/m
N	Axial (normal) force	kN/m
EI	Bending stiffness	kN·m²/m
EA	Axial stiffness	kN/m
I	Moment of inertia	m⁴/m
A	Cross-sectional area	m²/m
Wel	Elastic section modulus	m³/m
Wpl	Plastic section modulus	m³/m
k_h	Horizontal subgrade reaction modulus	kN/m³
Mp	Plastic moment capacity	kNm
Vr	Shear resistance	kN

Material

Table 5: Symbol · Description · Unit

Symbol	Description	Unit
f_{ck}	Characteristic compressive strength of concrete	kPa
f_{cd}	Design compressive strength of concrete, $f_{cd} = f_{ck}/1.50$	kPa
f_{ctk}	Characteristic tensile strength of concrete	kPa
f_{ctd}	Design tensile strength of concrete	kPa
f_{yk}	Characteristic yield strength of the reinforcement	kPa
f_{yd}	Design yield strength of the reinforcement, $f_{yd} = f_{yk}/1.15$	kPa
fy	Yield strength of the structural steel	kPa
E_s	Modulus of elasticity of steel	kPa
Asw	Stirrup cross-sectional area	mm²
γ_c	Concrete material factor (1.50)	—
γ_s	Steel material factor (1.15)	—

Safety and checks

Table 6: Symbol · Description · Unit

Symbol	Description	Unit
GS	Factor of safety	—
γ_{Rk}	Characteristic factor of safety (overall stability)	—
$\gamma_{G,dst}$	Partial factor for permanent action (1.35)	—
τ_f	Anchor bond skin friction	kPa
τ_c	Strand–grout bond stress	MPa
ξ	Correlation factor	—
α_a	Adhesion factor	—
N_c, N_q	Bearing capacity factors	—
i	Hydraulic gradient	—
k_h, k_v	Horizontal and vertical seismic coefficients	—
Sds	Short-period design spectral acceleration coefficient	—
r	Behaviour factor (KDYY Table 2.8)	—

Abbreviations

Table 7: Abbreviation · Description

Abbreviation	Description
KDY	Excavation support structure
KDYY	Excavation Support Structures Regulation

Abbreviation	Description
TBDY	Turkish Building Earthquake Code
TS500	Requirements for Design and Construction of Reinforced Concrete Structures
AGB	Monitoring zone
ULS	Ultimate limit state (ULS)
SLS	Serviceability limit state (SLS)
GLE	Generalised Limit Equilibrium (GLE)
P–M	Axial force – moment interaction diagram

> **UNIT SYSTEM —** The kN–m unit system is used in the report. Lengths are in metres, forces in kilonewtons, stresses and pressures in kPa (kN/m²) and displacements in millimetres. Quantities given per metre (kNm/m, kN/m) refer to a 1 metre strip of the wall.

1. PROJECT DESCRIPTION

1.1 Project record

Table 8: Information · Value

Information	Value
Client / owner	o
Project name	pkdwpsokd
Province	podk
District	wpodk
District	wopdwk
Block	odpwk
Sheet	dpowk
Parcel	dpow
Design engineer	kdpow
Title	kdpw
Chamber registration no	okdpw
Firm	okdw
Report no	pokdw
Report date	podkw

1.2 Structure to be built

Table 9: Information · Value

Information	Value
Structural system	Reinforced concrete
Foundation type	Raft
Number of storeys	Z+2
Foundation base depth	2 m

Information	Value
Foundation base level	0 m
Foundation base area	0 m²

1.3 General description of the excavation support system

A total of 1 excavation faces have been designed within the scope of the project. The wall type, excavation depth and support arrangement of the faces are summarised below. The detailed analysis of each face is presented in its own section.

Table 10: Face · Wall type · Support system

Face	Wall type	Support system	Excavation H (m)	Wall L (m)	Embedment (m)	Face length (m)
Street Face	Bored pile wall	Anchored system	10	14	4	39

The total excavation perimeter is 39 m and the deepest excavation is 10 m.

2. INVENTORY OF ADJACENT STRUCTURES, ROADS AND UTILITIES

Adjacent structures, roads and utility lines liable to be affected by the excavation are documented below. The inventory is an input to the settlement and angular distortion assessment; it is not merely a record list.

2.1 Adjacent structures

Table 11: No. · Block/Parcel · Year

No.	Block/Parcel	Year	Storey	Structural system	Foundation	Foundation depth (m)	Face	Distance (m)
1	2783/1	1988	B+Z+4	Reinforced concrete	Strip	2.5	Street Face	6
2	2783/3	2018	B+Z+4	Reinforced concrete	Raft	2	—	6

2.1.1 Cafer Apartmanı

Table 12: Information · Value

Information	Value
Block / Parcel	2783 / 1
Year of construction	1988
Structural system	Reinforced concrete
Number of storeys	B+Z+4
Foundation type	Strip
Foundation base depth	2.5 m
Foundation base level	-3 m
Foundation base area	485 m²
Nearest face	Street Face

Information	Value
Distance to the excavation line	6 m

Yapı temeli kazı tabanının üzerindedir; ancak kazı hattına mesafesi (6 m) is greater than the horizontal extent of the active wedge developing from this depth (3.99 m).

(Figure — Cafer Apartmanı — presented in Appendix A.)

2.1.2 Ayşe Apartmanı

Table 13: Information · Value

Information	Value
Block / Parcel	2783 / 3
Year of construction	2018
Structural system	Reinforced concrete
Number of storeys	B+Z+4
Foundation type	Raft
Foundation base depth	2 m
Foundation base level	-2.5 m
Foundation base area	444 m²
Nearest face	—
Distance to the excavation line	6 m

(Figure — Ayşe Apartmanı — presented in Appendix A.)

2.2 Surrounding roads

Table 14: No. · Name · Lagging

No.	Name	Lagging	Width (m)	Traffic	Slope (%)	Lighting	Underpass	Overpass	Infrastructure	Face	Distance (m)
1	Mustafa Kemal Paşa Bulvarı	Asphalt	15	Heavy	3	Yes	None	None	None	Street Face	3
2	2784 Sokak	Interlocking paver	6	Light	4	Yes	None	None	None	—	3

(Figure — Mustafa Kemal Paşa Bulvarı — presented in Appendix A.)

(Figure — 2784 Sokak — presented in Appendix A.)

> **TRAFFIC LOAD —** Traffic has been specified as heavy on road(s) Mustafa Kemal Paşa Bulvarı. The traffic load must be modeled as a surcharge, and whether heavy vehicle traffic will be permitted at the roadside during excavation must be stated explicitly in the site instructions.

2.3 Utility lines

Table 15: No. · Name · Type

No.	Name	Type	Depth (m)	Face	Distance (m)
1	İZSU	Water	1.5	Street Face	3

UTILITY LINE CLOSE TO EXCAVATION — İZSU (Su) kazı hattına, kazı derinliğinin yarısından daha yakındır. Bu hatların kazı öncesinde yerinde tespiti (aç-kapa çukur veya dedektörle), gerekiyorsa deplasesi ve kazı süresince ayrıca izlenmesi gerekir. Doğalgaz ve su hatlarında sızıntı, zemin dayanımını yerel olarak düşürdüğü için ilave risk oluşturur.

3. GEOTECHNICAL SITE CHARACTERISATION

3.1 Soil investigation record

Table 16: Information · Value

Information	Value
Investigation firm	podkw
Report no	podkw
Report date	podkw
Number of boreholes	—
Deepest borehole	—

3.2 Soil profile

The soil profile used in the design is given below. All faces use the SAME profile.

Table 17: No. · Layer · Thickness (m)

No.	Layer	Thickness (m)	Status	γ (kN/m³)	γ_{sat} (kN/m³)	ϕ' (°)	c' (kPa)	c_u (kPa)	δ/ϕ' (—)	δ (°)	K_0	E_{oed} (kPa)
1	Well Graded Sand	10	Drained	20	20.4	34	1	—	0.667	22.7	0.441	95040
2	Compacted Silty Sand	10	Drained	18.9	19.2	33	3	—	0.646	21.3	0.455	36111

The total defined profile thickness is 20 m.

The wall–soil friction angle has been entered as a ratio of the soil's ϕ' angle (δ/ϕ', 0…1); the analysis uses δ = (δ/ϕ')·ϕ', so δ ≤ ϕ' holds in every layer.

3.3 Groundwater

No groundwater is defined at any face. Water pressure has not been included in the analysis.

ABSENCE OF WATER IS AN ASSUMPTION — The absence of groundwater shall be supported by the soil investigation and by seasonal observations. If the excavation coincides with the rainy season, drainage measures shall be taken to remove the surface water accumulating in the excavation and to prevent the backfill from becoming saturated.

4. DESIGN ASSUMPTIONS AND BASIS OF DESIGN

Design code: KDYY (+ TBDY 2018) — Excavation Support Structures Regulation

4.1 Materials

Table 18: Material · Class · Characteristic resistance

Material	Class	Characteristic resistance	Modulus of elasticity
Concrete	C30	f_{ck} = 30000 kN/m²	32000000 kN/m²
Reinforcing steel	B420C	f_{yk} = 420000 kN/m²	200000000 kN/m²
Structural steel	S355H	f_y = 355000 kN/m²	200000000 kN/m²
Prestressing strand	—	anchor table (breaking load)	195000000 kN/m² (195 GPa, Settings)
Cement grout	—	w = 25 kN/m³	10000000 kN/m² (10 GPa, Settings)

The design strengths have been derived in accordance with TS500:

$$f_{cd} = f_{ck} / 1.50 \qquad f_{ctd} = f_{ctk} / 1.50 \qquad f_{yd} = f_{yk} / 1.15$$

4.2 Soil pressure method

Table 19: Subject · Selection

Subject	Selection
Active pressure method (drained)	Mazindrani
Passive pressure method (drained)	Caquot–Kérisel
Undrained condition method	Total Stress (ϕ = 0)
Subgrade reaction model	Schmitt model
Minimum loading condition	Applied (0.15·σ'$_v$)
Mesh spacing	0.05 m

Rankine limit values:

$$K_a = \tan^2(45° - \phi'/2) \qquad K_p = \tan^2(45° + \phi'/2)$$

The cohesion effect has been applied so as to reduce the active pressure and to increase the passive pressure.

> **MINIMUM LOADING CONDITION — In zones where the active pressure is zero or negative, a floor value of 0.15·σ'$_v$ has been applied. In cohesive soil, complete loss of the active pressure in the upper zone is an unreliable assumption because of the risk of long-term loss of cohesion.**

4.3 Earthquake

Seismic action has not been taken into account. This choice must be justified with regard to the service life of the excavation support structure and the requirements of the code.

4.4 Partial factors and factors of safety

Table 20: Check · Required value · Source

Check	Required value	Source
Embedment depth (cantilever)	GS ≥ 1.50	Common practice
Embedment depth (supported)	GS ≥ 1.30	Common practice
Overall stability (static)	γ_{Rk} ≥ 1.50	KDYY
Overall stability (seismic)	γ_{Rk} ≥ 1.10	KDYY
Anchor strand rupture	GS ≥ 1.60	Design assumption
Anchor pull-out	GS ≥ 1.50	Design assumption
Pile axial bearing	GS ≥ 2.50	Common practice
Basal heave	GS ≥ 1.50	Common practice
Sand boiling	GS ≥ 1.50	Common practice

DA2 (EC7 DESIGN APPROACH 2) — The anchor verdict is given with partial factors: $\gamma_{G,dst}$ = 1.50, pull-out γ_{Re} = 1.40, strand γ_s = 1.55 (Settings). The anchor FoS rows above are for information.

4.5 Serviceability limits

The horizontal displacement limits have been determined in accordance with KDYY according to the support system:

Table 21: System · Limit · Source

System	Limit	Source
Internally supported (strutted)	$\delta_{h,lim}$ = 2.5‰ · H	KDYY 2.12.10
Anchored	$\delta_{h,lim}$ = 3.0‰ · H	KDYY 2.12.7
Cantilever	$\delta_{h,lim}$ = 1.0% · H	KDYY 2.12.9

COLLAPSE CRITERION — Separately from the serviceability limits, a displacement exceeding 3% of the largest FREE (unsupported) excavation height seen up to that stage is taken as collapse. Displacement of this order is outside the small-strain assumption on which the solution rests; the spring model and the pressure distribution lose their validity there.

4.6 Calculation method

The excavation support structure is modelled as a beam system connected to the soil by horizontal springs. Each node has horizontal displacement and rotation degrees of freedom.

$$[K] \cdot \{d\} = \{F\}$$

The global stiffness matrix consists of three contributions: the bending stiffness of the beam elements (EI), the horizontal stiffness of the soil springs (k_h × nodal tributary area) and the axial stiffness of the support members. The load vector comes from the active pressure, water pressure, surcharge and anchor prestress.

The soil springs are not linear: they behave elastically between the active and passive limits and yield on reaching the limit. This behaviour has been solved by initial stress iteration: after the elastic solution, the excess force at the nodes exceeding the limit is reapplied as a balancing load and the process is repeated until convergence.

4.7 Staged construction

The construction stages have been modelled and the displacement of each stage is carried over to the next. The free excavation depth below the support is taken as 0.5 m.

> **WHY STAGED —** A real excavation is not carried out in a single step. Before a support is installed the soil is excavated some distance below it, and at that moment the system acts as a cantilever. A single-step analysis does not see this temporary condition and shows the displacement smaller than it is.

5. STREET FACE

5.1 Description and geometry of the face

Table 22: Information · Value

Information	Value
Wall type	Bored pile wall
Support system	Anchored system
Excavation depth (H)	10 m
Wall length (L)	14 m
Embedment depth (d)	4 m
Embedment / excavation ratio (d/H)	0.40
Face length	39 m
Excavation base level	-9.00 m
Wall top level (derived)	+1.00 m
Pile diameter	0.8 m
Pile spacing	1 m
Capping beam	0.8 × 0.8 m
Cap beam position	Inside the soil (top at ground level); the entered total height includes the beam, net pile length 13.2 m

5.1.1 Section and stiffness per unit length

Table 23: Quantity · Value · Unit

Quantity	Value	Unit
Cross-sectional area	0.502655	m^2/m
Moment of inertia	0.020106	m^4/m
Bending stiffness (EI)	643398	$kN \cdot m^2/m$
Axial stiffness (EA)	16084954	kN/m

Bored pile: circular section values divided by the pile spacing. EI effectiveness factor=1.

5.1.2 Excavation and site geometry

Table 24: Information · Value

Information	Value
Excavation side geometry	Horizontal
Retained ground surface	Horizontal
Groundwater	None
Tension crack	Taken into account
Filling of the crack with water	Taken into account

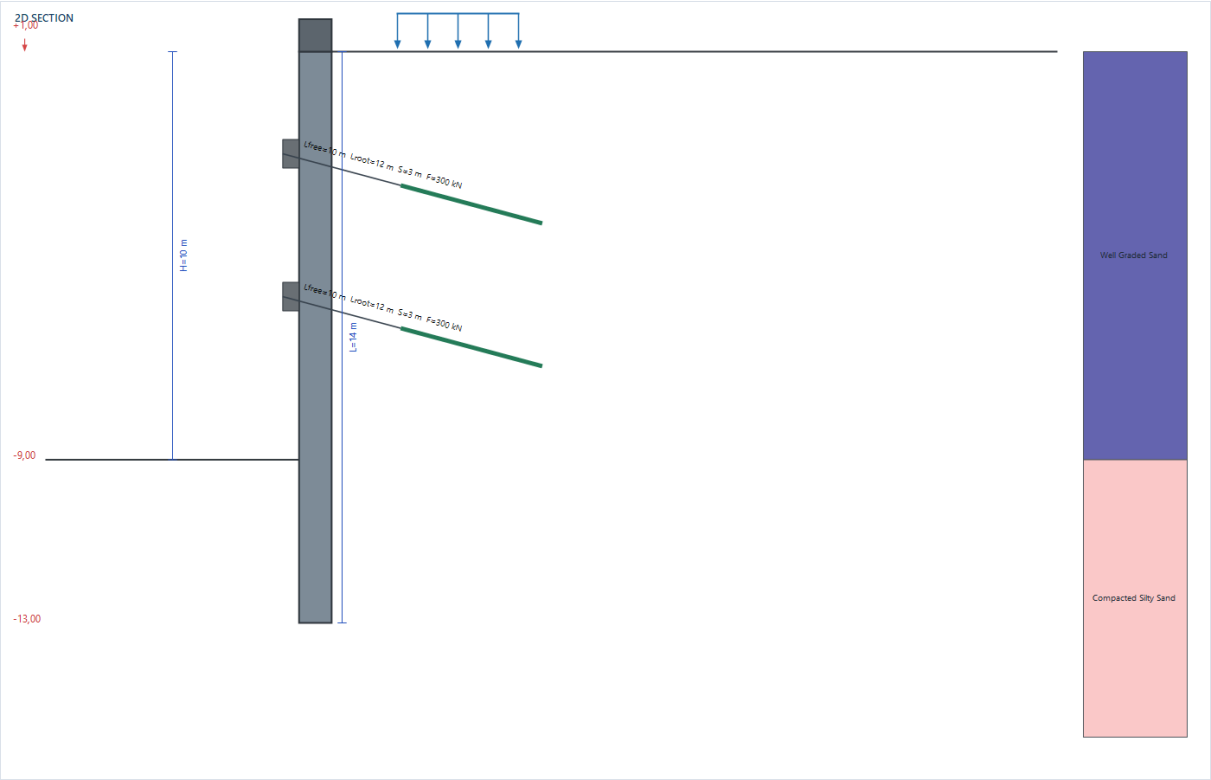


Figure 1: Street Face — section

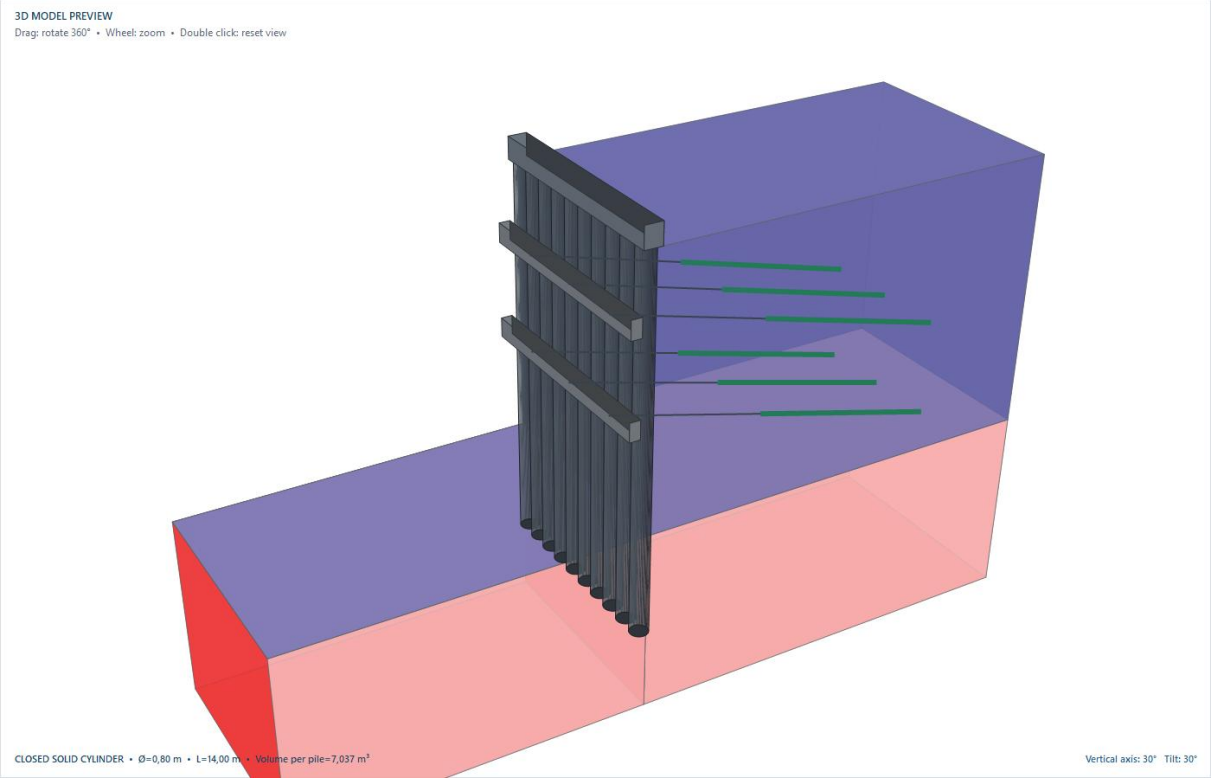


Figure 2: Street Face — three-dimensional model

5.1.3 System type and expected behaviour

Table 25: Information · Value

Information	Value
System type	2-ROW SUPPORTED
Number of effective support rows	2

Information	Value
Anchor / strut	2 anchors · 0 struts

In multi-row support the upper zone remains almost fixed; the bulge moves downward, between the lowest support and the excavation base. As the maximum displacement occurs at depth, the settlement trough behind the excavation is wider and flatter; the angular distortion imposed on the adjacent structure is therefore smaller even at the same displacement.

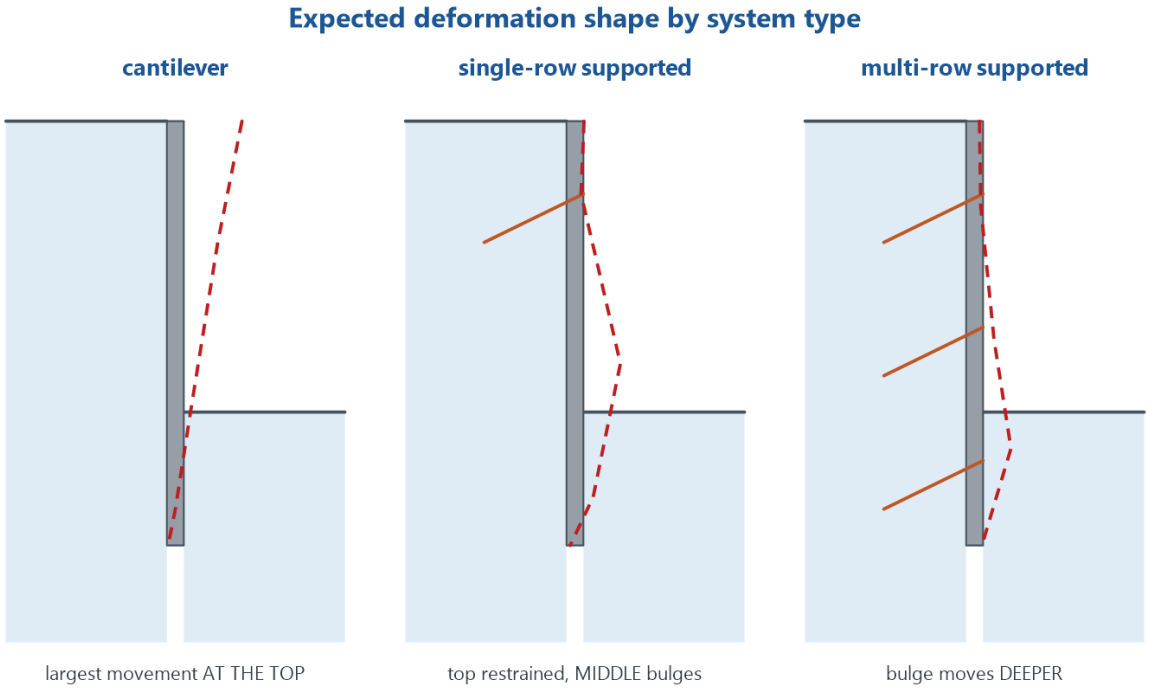


Figure 3: Deformation shape by system type

How to use this section — The shape given here is not a CALCULATION RESULT but the behaviour expected from the system type. If the displacement curve from the analysis does not resemble this shape, the input shall first be checked and the result then interpreted. The same comparison also applies to inclinometer readings during excavation.

5.2 Support arrangement

5.2.1 Anchors

Table 26: No. · Depth (m) · Free length (m)

No.	Depth (m)	Free length (m)	Bond length (m)	Inclination (°)	Plan spacing (m)	Bond body diameter (m)	Strand	Prestress (kN)	Status
1	2.5	10	12	15	3	0.15	3×0.60 in	300	Prestressed
2	6	10	12	15	3	0.15	3×0.60 in	300	Prestressed

The anchor stiffness uses a strand modulus Et = 195 GPa (prestressing strand, Settings), a grout modulus Eg = 10 GPa and a grout unit weight w = 25 kN/m³. The rebar modulus (200000000 kN/m²) is not used for the anchor strand.

$$kt = Et \cdot At / Lf$$

Table 27: E values used for the anchor stiffness and the PLAXIS export

No.	Et (GPa)	At (cm²)	Et·At (kN)	Lf (m)	Eroot (GPa)	(EA)root (kN)	L_b,eff (m)	Et·At/Lf (kN/m)	kt (kN/m)
1	195	4.20	81900	10	10	176715	—	8190	8190
2	195	4.20	81900	10	10	176715	—	8190	8190

PLAXIS export: the root embedded beam uses E = Eroot and w from the table above; the free-length node-to-node anchor is written with EA = Et·At (not divided by the spacing; Lspacing is given separately).

Table 28: Consistency of the support location with the waler beam decision

Member	No.	Location	Waler beam	Result
Anchor	1	Between two elements	Yes	CONSISTENT
Anchor	2	Between two elements	Yes	CONSISTENT

> **WALER BEAM DECISION —** At a support standing on the axis of a vertical element a waler beam is NOT MANDATORY; the load bears directly on that element. A waler may still be used — it spreads the load, strengthens the area around the hole and takes up the verticality tolerance. This is a design choice; the program does not interfere.

5.2.2 Anchor free zone check

The bond zone of the anchor shall lie outside the active wedge that may form behind the wall. If the bond body remains inside the wedge, the anchor is tied to the very mass it is trying to retain and contributes nothing to the system.

$$\text{failure plane angle} = 45° + \phi'/2 = 62° \quad \text{(from the excavation base)}$$

$$\text{offset } X = \max(\ 2.00\ \text{m}\ ;\ 0.20 \cdot H\ ) = 2\ \text{m}$$

Table 29: Anchor · Depth (m) · Free length (m)

Anchor	Depth (m)	Free length (m)	Required (m)	Result
1	2.5	10	5.67	SATISFACTORY
2	6	10	3.98	SATISFACTORY

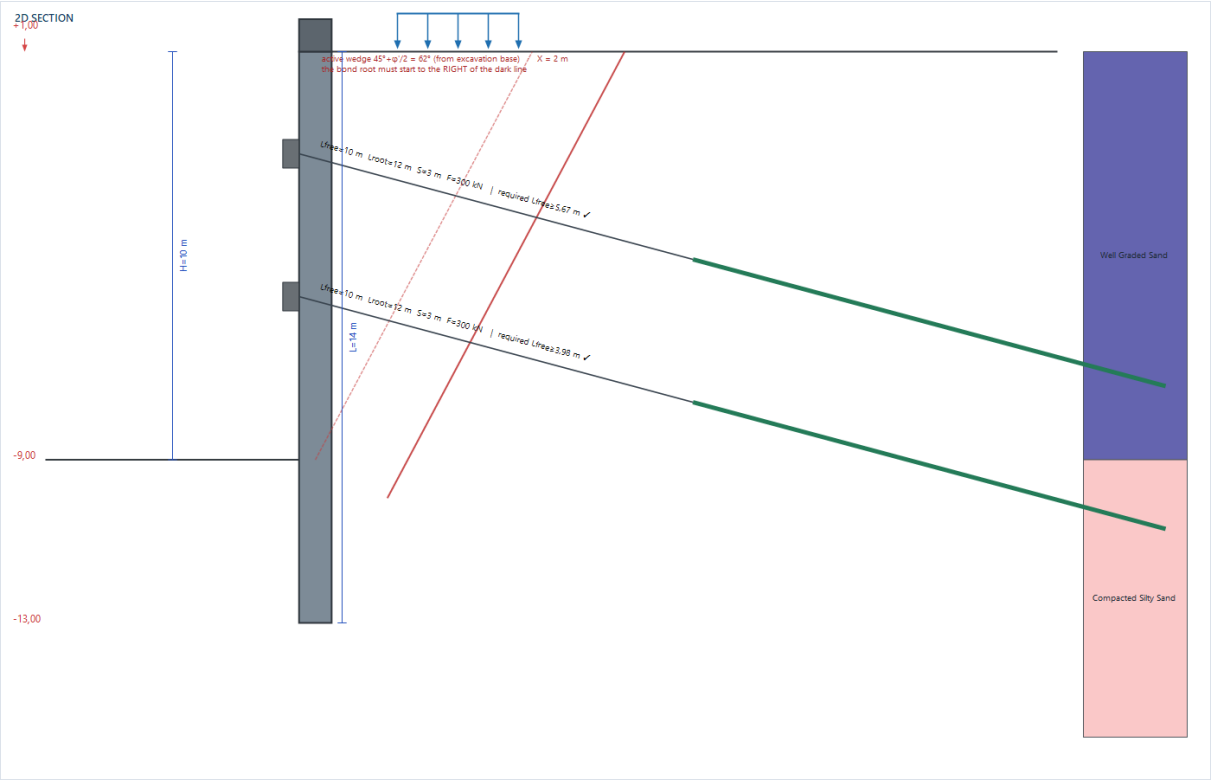


Figure 4: Street Face — section

5.3 Construction stages

The excavation is carried out in stages, not in a single step. The displacement of each stage is carried over to the next; support elements are included in the system from the stage at which they are installed.

Table 30: No. · Stage · Excavation level (m)

No.	Stage	Excavation level (m)	Active anchor	Active strut	Description
1	Initial state	0	—	—	Wall construction and initial K_o state
2	Free excavation: 3.00 m	3	—	—	Free excavation for the installation of Anchor 1 (support NOT YET active)
3	Anchor 1 installation	3	1	—	Anchor 1 installed; the prestress (if any) was applied to the soil at this stage
4	Free excavation: 6.50 m	6.5	1	—	Free excavation for the installation of Anchor 2 (support NOT YET active)
5	Anchor 2 installation	6.5	1, 2	—	Anchor 2 installed; the prestress (if any) was applied to the soil at this

No.	Stage	Excavation level (m)	Active anchor	Active strut	Description
					stage
6	Final excavation	10	1, 2	—	Net excavation level

5.4 Initial analysis parameters

The quantities below follow from the soil profile and the water conditions; they do not change with the excavation stages. They are therefore given once per face.

5.4.1 Vertical stress profile

The total vertical stress, the pore water pressure and the effective vertical stress are shown below along the depth. The pressure coefficients are applied to the EFFECTIVE stress; the water pressure is added separately and in full.

$$\sigma_v = \Sigma\ \gamma \cdot h \qquad u = \gamma_w \cdot (z - zw) \qquad \sigma'_v = \sigma_v - u$$

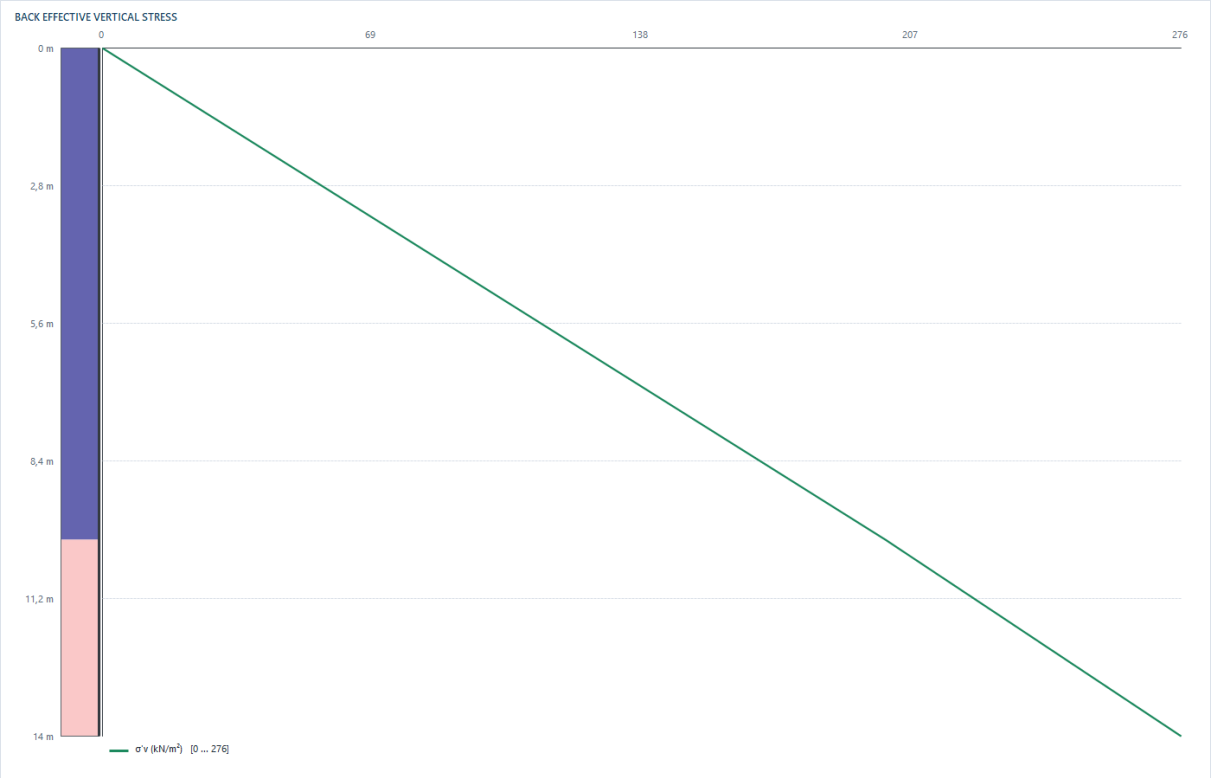


Figure 5: Street Face — Initial state — Back effective vertical stress

Table 31: Street Face — vertical stress profile on the retained soil side

Depth (m)	σ_v (kPa)	u (kPa)	σ'_v (kPa)
3.5	70	0	70
7	140	0	140
10.5	209.5	0	209.5
14	275.6	0	275.6

5.4.2 Lateral pressure limits ($K_a \cdot K_0 \cdot K_p$)

The soil springs behave elastically between the active and passive limits; on reaching the limit they yield.

In the solution the active/passive coefficients are computed node by node with the selected method (Mazindrani / Caquot–Kérisel; Total Stress (ϕ = 0) in undrained layers), depending on the wall friction δ and the vertical effective stress σ'_v. The K_a and K_p values below are Rankine comparison values for δ = 0; the K_0 column is the value used in the solution (Jaky·OCR^sinϕ', or the manually entered value).

$$pa = K_a \cdot \sigma'_v - 2c' \cdot \sqrt{K_a} \qquad p0 = K_0 \cdot \sigma'_v \qquad pp = K_p \cdot \sigma'_v + 2c' \cdot \sqrt{K_p}$$

Table 32: Street Face — lateral pressure coefficients by layer

Layer	ϕ' (°)	c' (kPa)	K_a (Rankine, δ=0)	K_0 (used)	K_p (Rankine, δ=0)
Well Graded Sand	34	1	0.283	0.441	3.537
Compacted Silty Sand	33	3	0.295	0.455	3.392

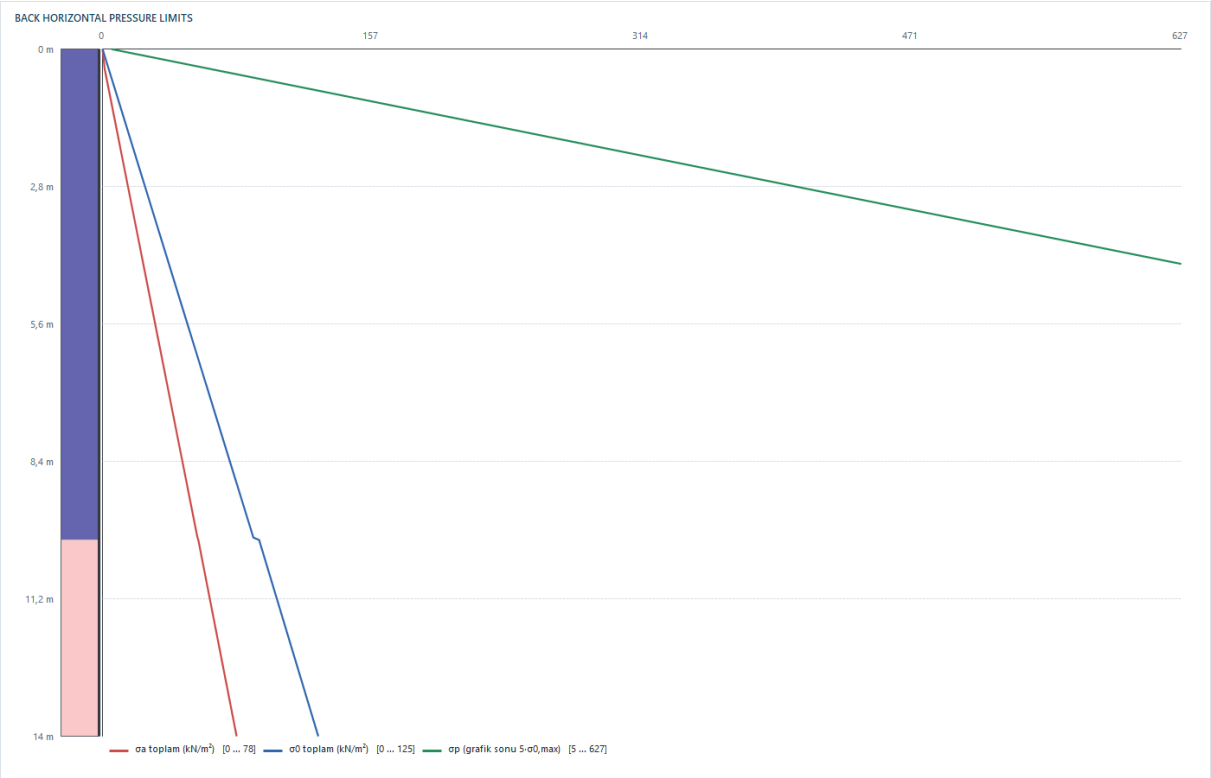


Figure 6: Street Face — Initial state — Back horizontal pressure limits

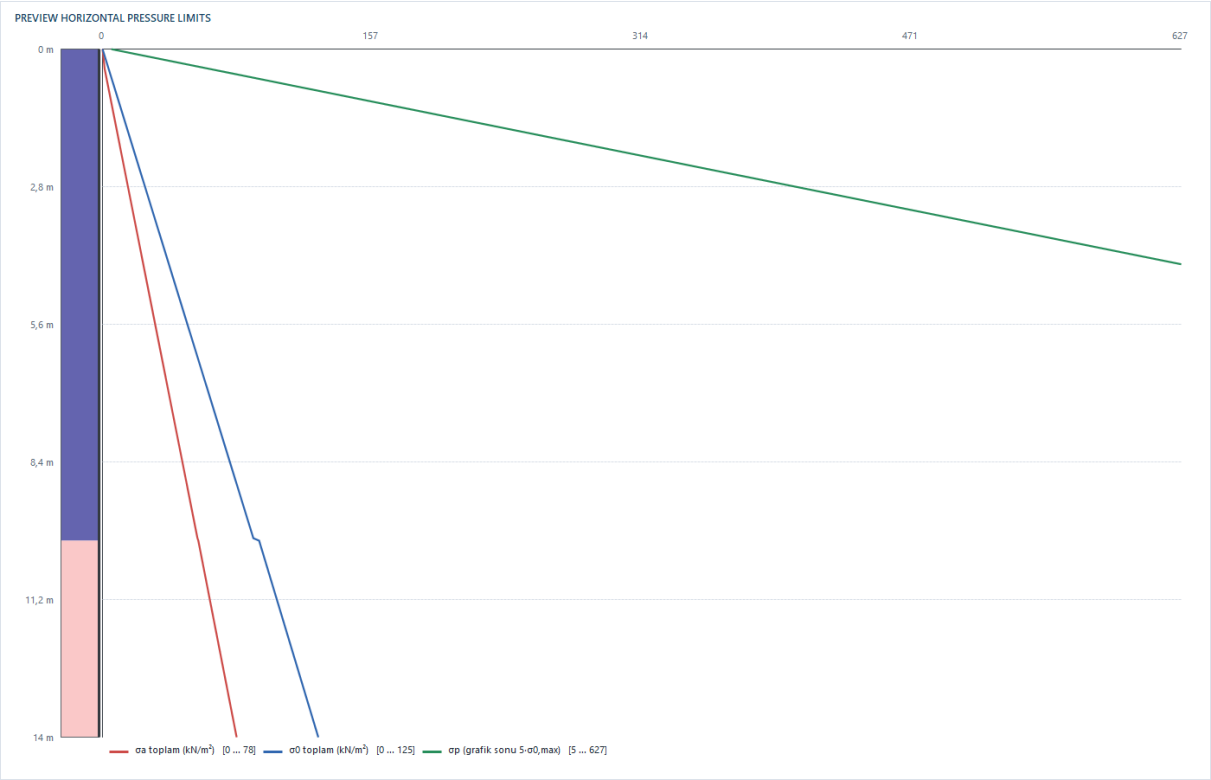


Figure 7: Street Face — Initial state — Preview horizontal pressure limits

5.5 Stage-by-stage analysis results

Displacement, internal forces and soil pressures are presented separately for each construction stage. The graphs are taken directly from the program's drawing engine and are identical to the drawings seen on screen.

5.5.1 Free excavation: 3.00 m

The excavation has been taken down to level 3 m; at this stage 3 m of excavation has been carried out. Since no support has yet been installed, at this stage the system works as a CANTILEVER and the free excavation height is 3 m.

Table 33: Quantity · Value · Unit

Quantity	Value	Unit
Excavation level	3	m
Maximum horizontal displacement	1.1	mm
Maximum bending moment	37.6	kNm/m
Maximum shear force	23.1	kN/m
Convergence	Satisfied	—
Iteration	8	—

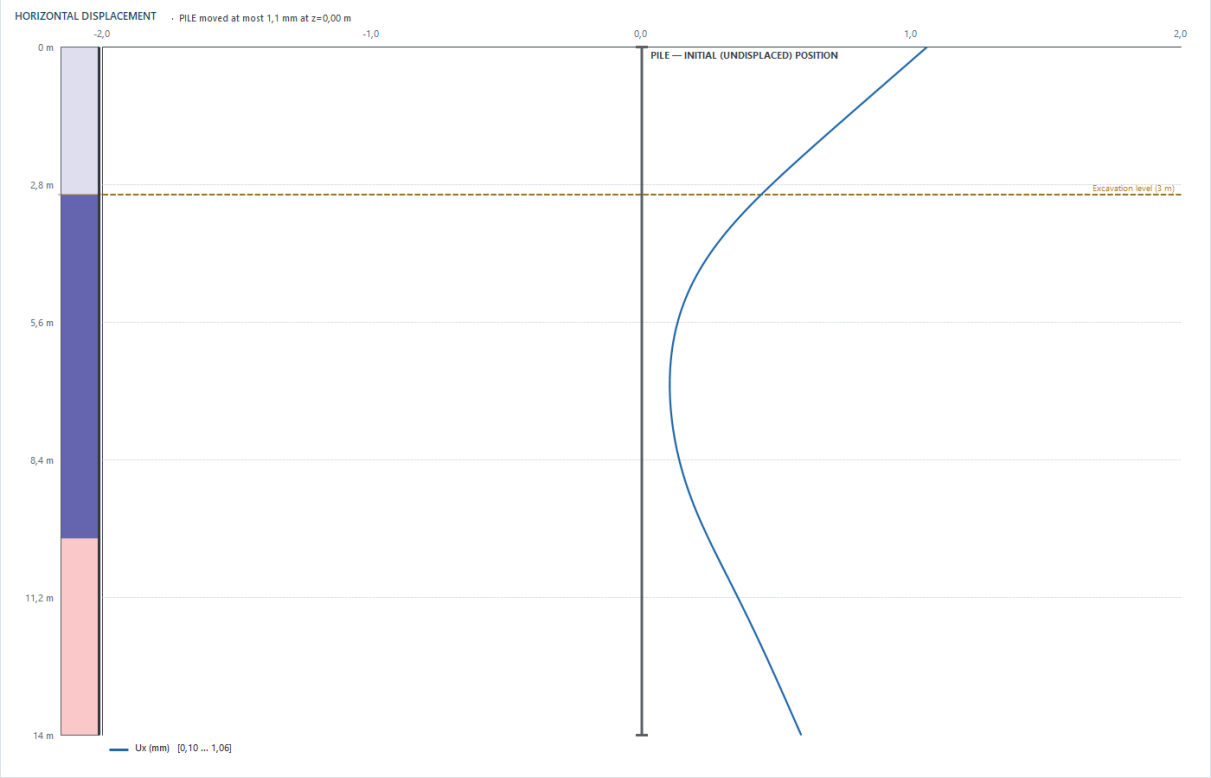


Figure 8: Street Face — Free excavation: 3.00 m — Horizontal displacement

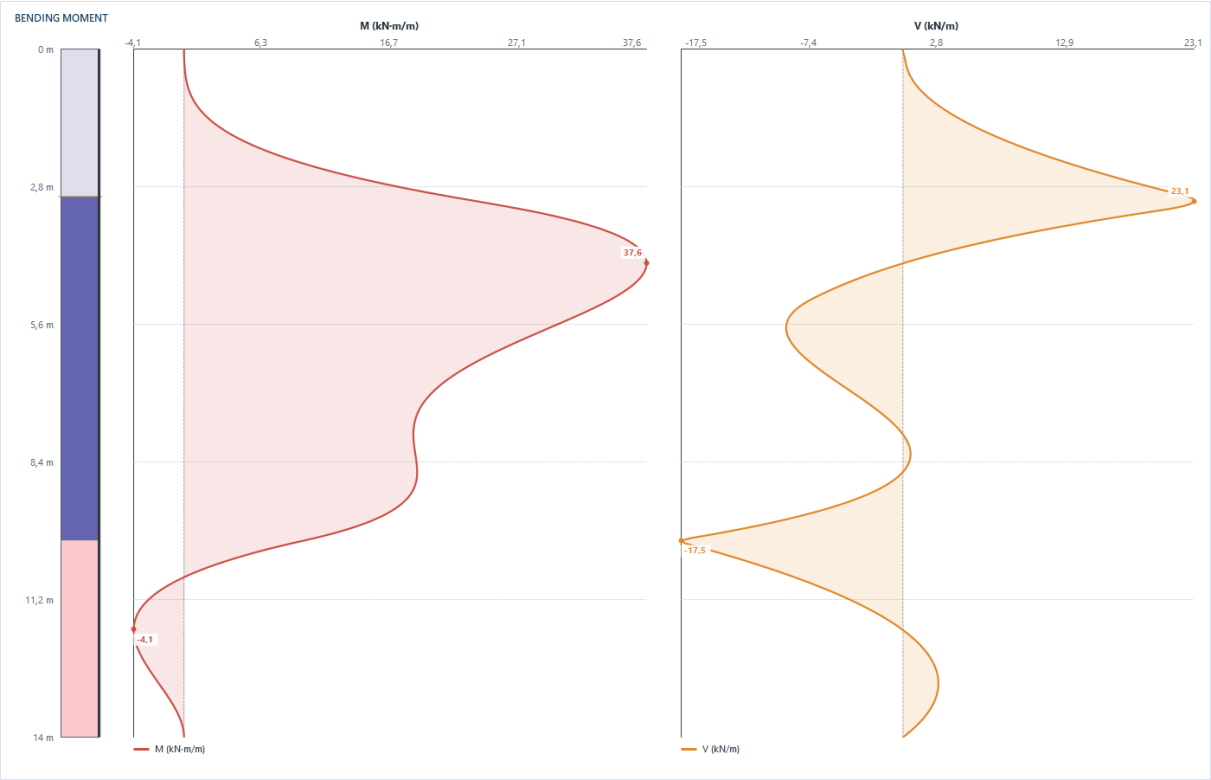


Figure 9: Street Face — Free excavation: 3.00 m — Bending moment and shear force

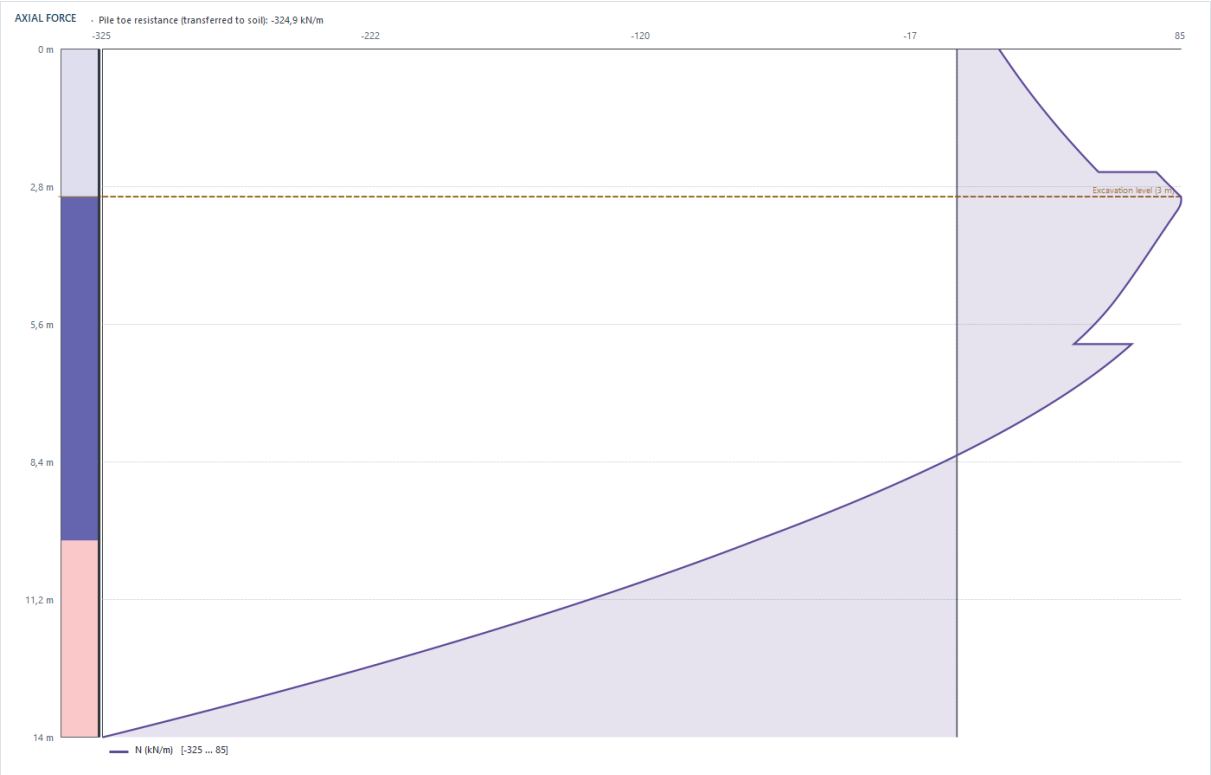


Figure 10: Street Face — Free excavation: 3.00 m — Axial force

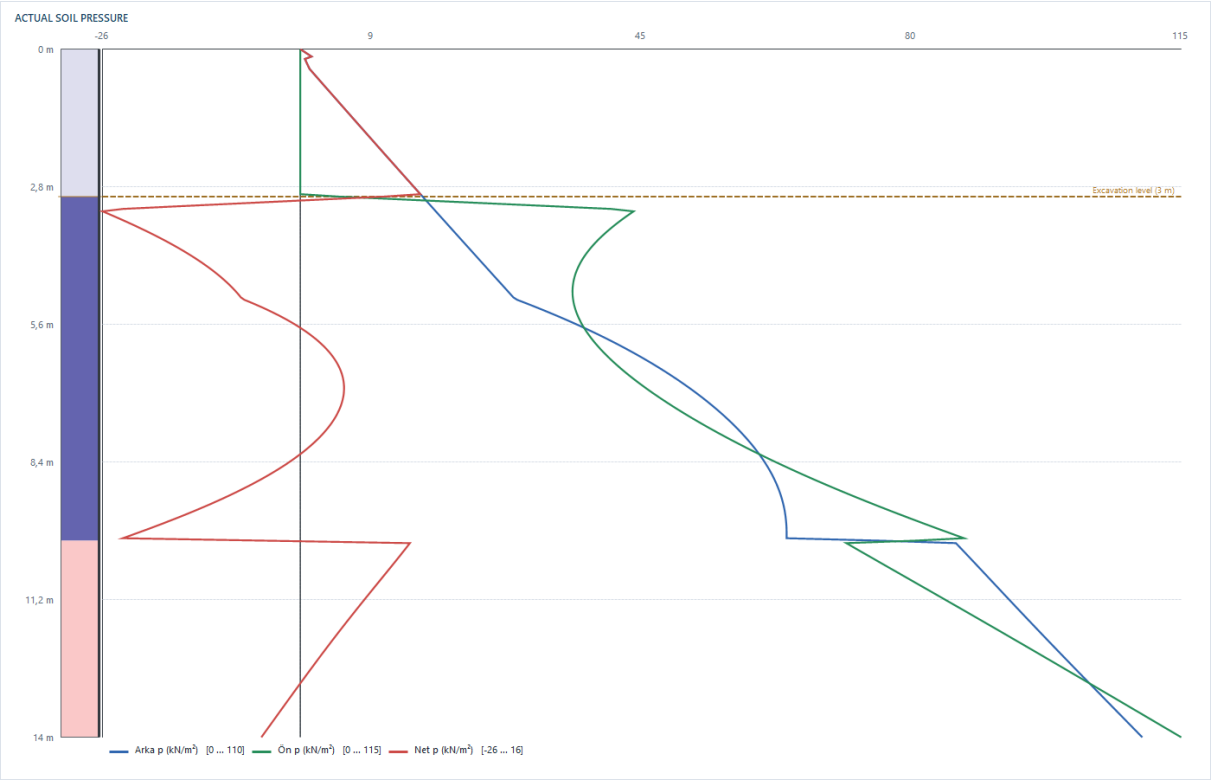


Figure 11: Street Face — Free excavation: 3.00 m — Actual soil pressure

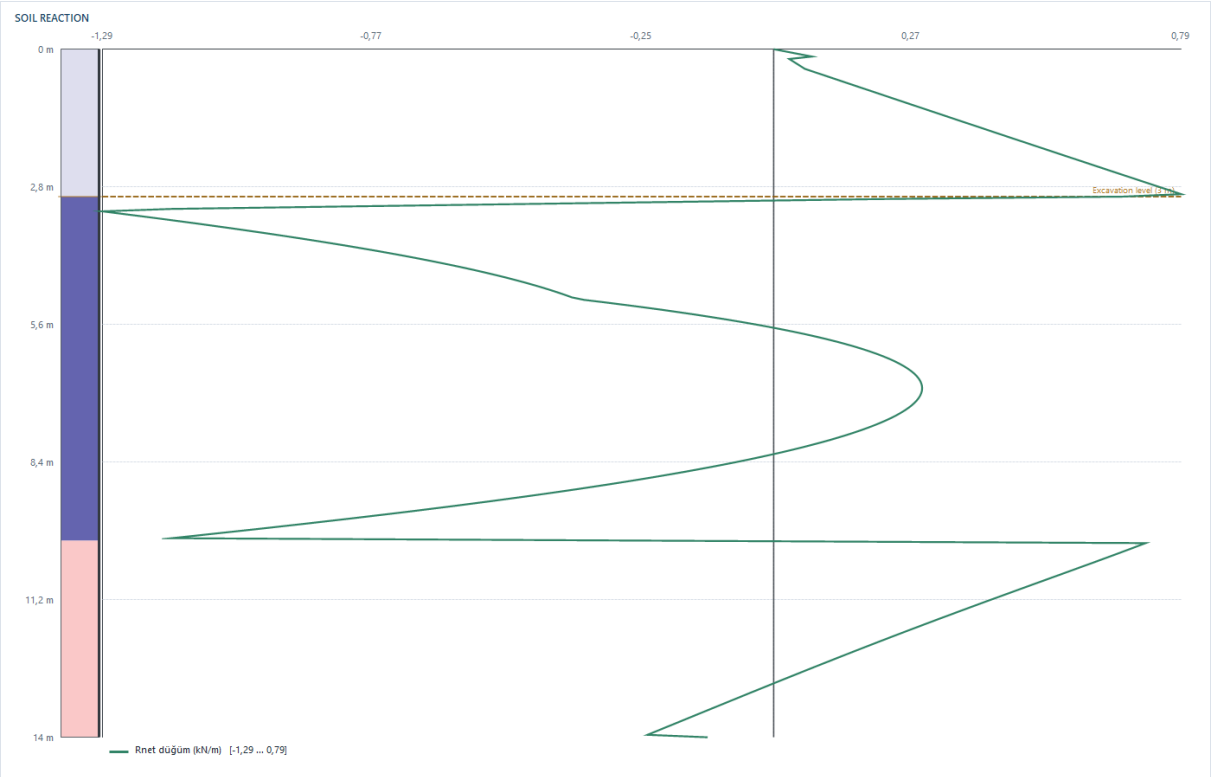


Figure 12: Street Face — Free excavation: 3.00 m — Soil reaction

5.5.2 Anchor 1 installation

Anchor row no. 1 has been installed at level 2.5 m and a prestress force of 300 kN applied; free length 10 m, bond length 12 m, inclined at 15° to the horizontal, plan spacing 3 m, and 3 strands of 0.60 in have been used.

Table 34: Quantity · Value · Unit

Quantity	Value	Unit
Excavation level	3	m
Maximum horizontal displacement	0.9	mm
Maximum bending moment	61.3	kNm/m
Maximum shear force	56.3	kN/m
Convergence	Satisfied	—
Iteration	22	—

Table 35: Support · No. · Depth (m)

Support	No.	Depth (m)	Total horizontal force (kN/m)
Anchor	1	2.5	-96.1

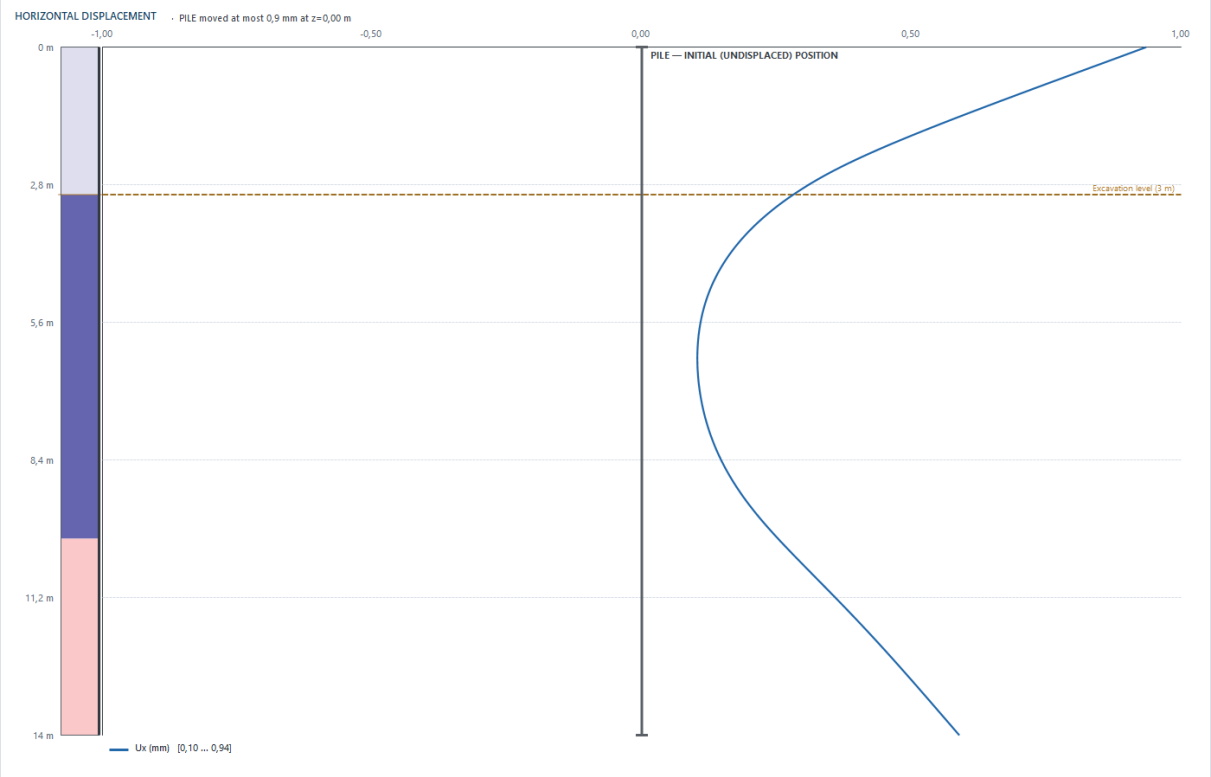


Figure 13: Street Face — Anchor 1 installation — Horizontal displacement

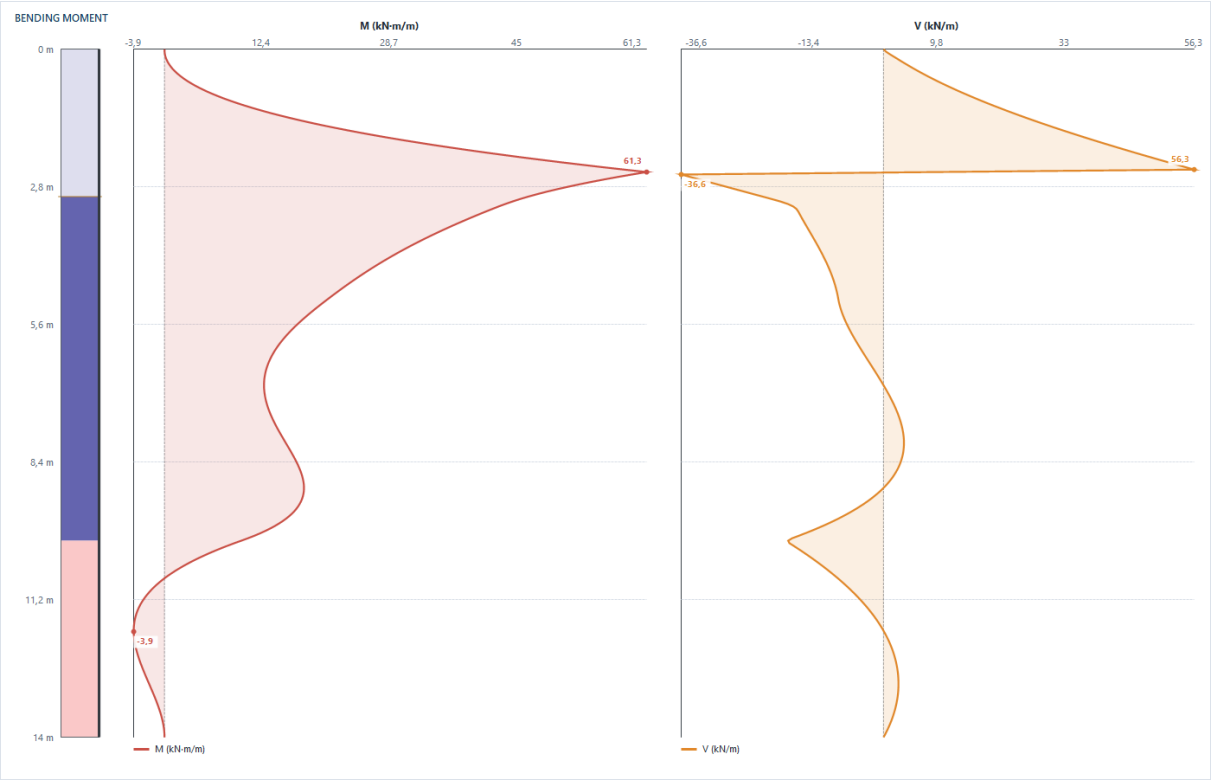


Figure 14: Street Face — Anchor 1 installation — Bending moment and shear force



Figure 15: Street Face — Anchor 1 installation — Axial force

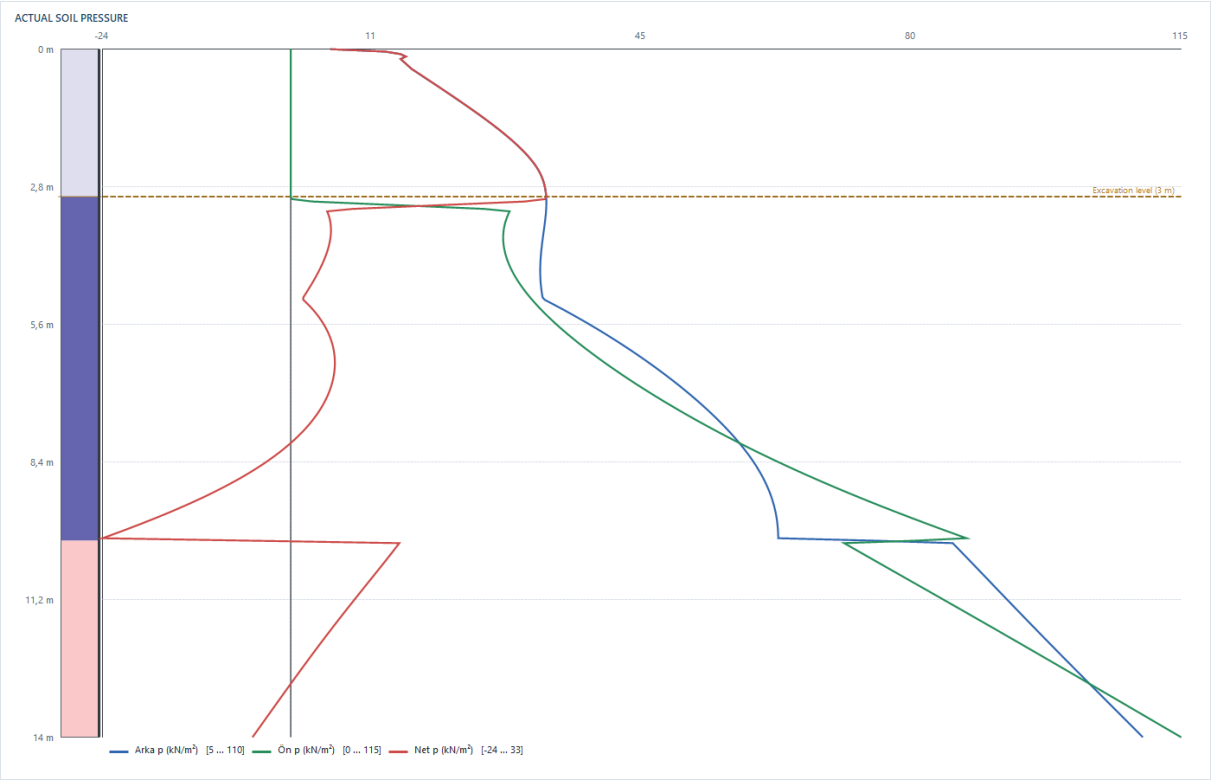


Figure 16: Street Face — Anchor 1 installation — Actual soil pressure

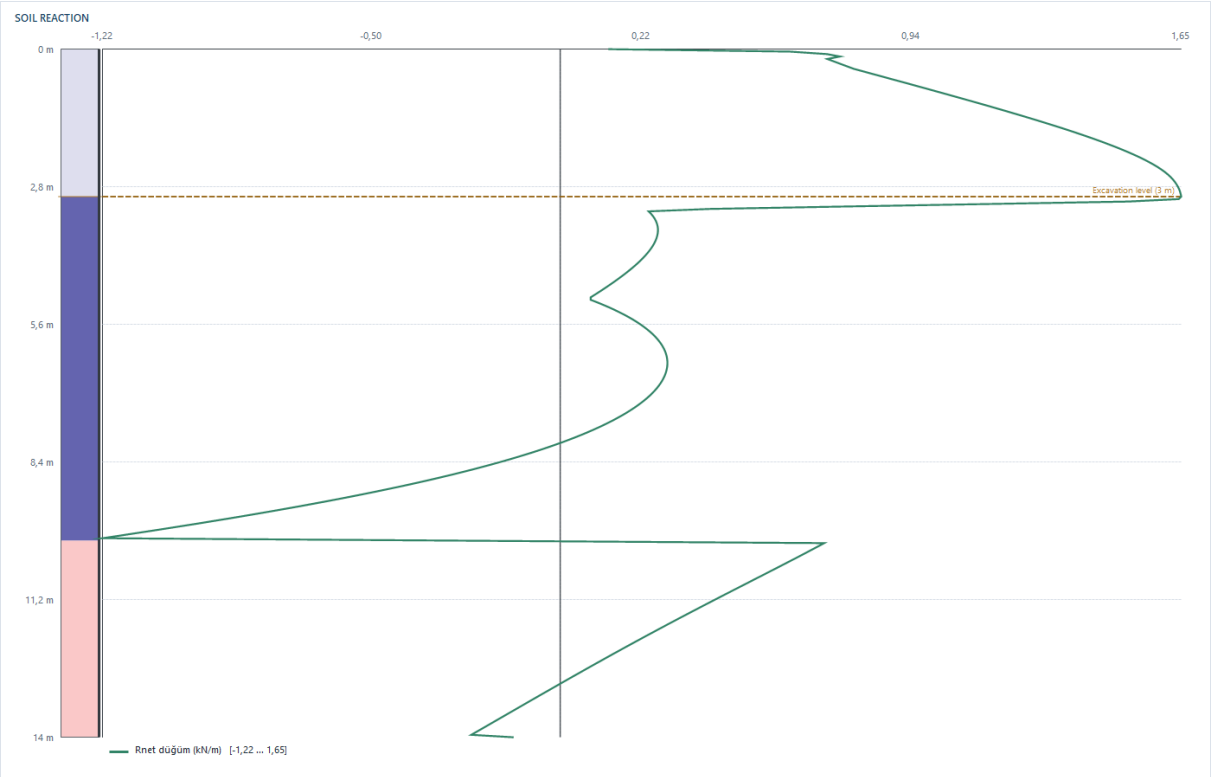


Figure 17: Street Face — Anchor 1 installation — Soil reaction

5.5.3 Free excavation: 6.50 m

The excavation has been taken down from level 3 m to level 6.5 m; at this stage 3.5 m of excavation has been carried out. Since the lowest support is at level 2.5 m, the unsupported (free) excavation height is 4 m.

Table 36: Quantity · Value · Unit

Quantity	Value	Unit
Excavation level	6.5	m
Maximum horizontal displacement	1.2	mm
Maximum bending moment	46.4	kNm/m
Maximum shear force	58.1	kN/m
Convergence	Satisfied	—
Iteration	26	—

Table 37: Support · No. · Depth (m)

Support	No.	Depth (m)	Total horizontal force (kN/m)
Anchor	1	2.5	-95.4

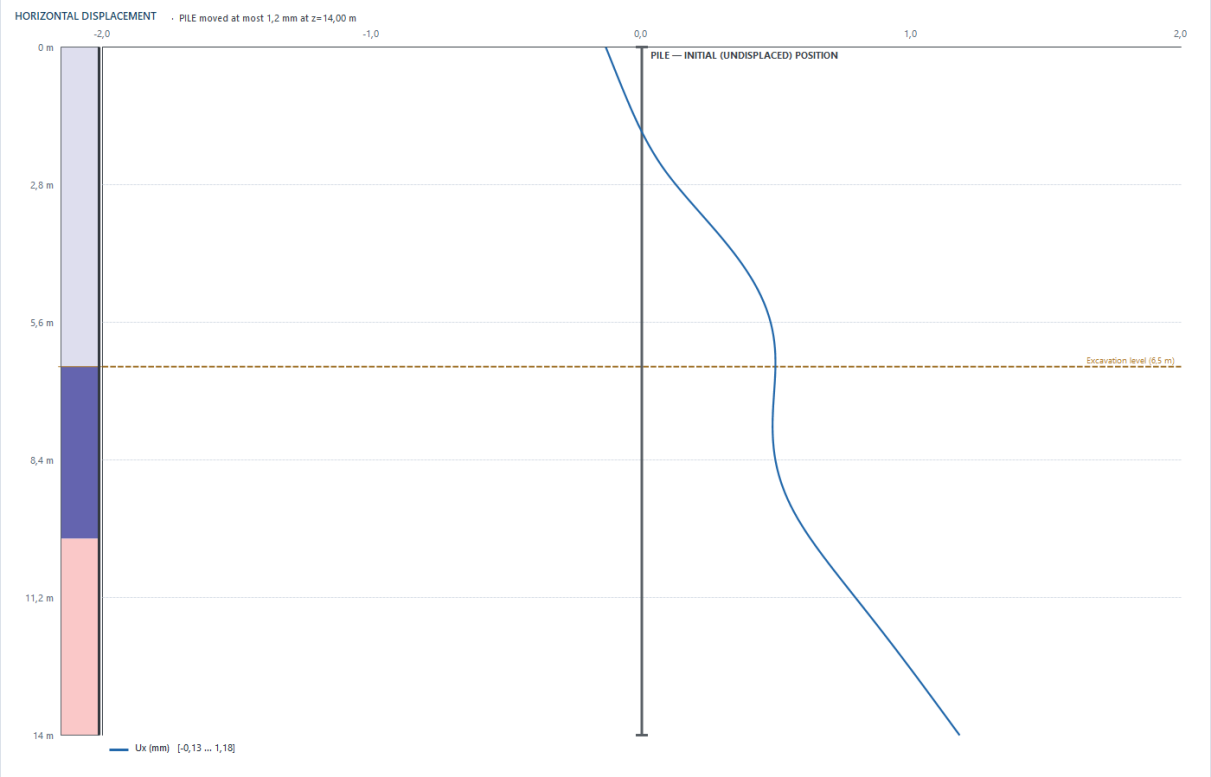


Figure 18: Street Face — Free excavation: 6.50 m — Horizontal displacement

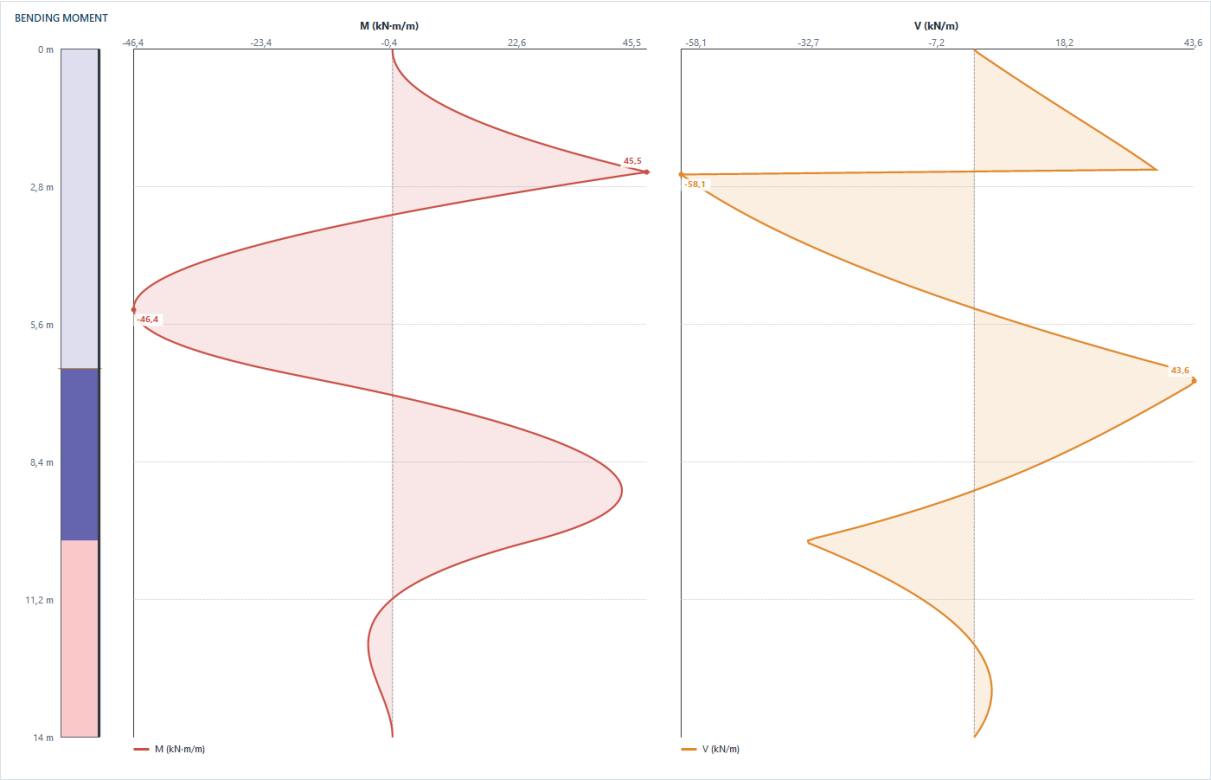


Figure 19: Street Face — Free excavation: 6.50 m — Bending moment and shear force



Figure 20: Street Face — Free excavation: 6.50 m — Axial force

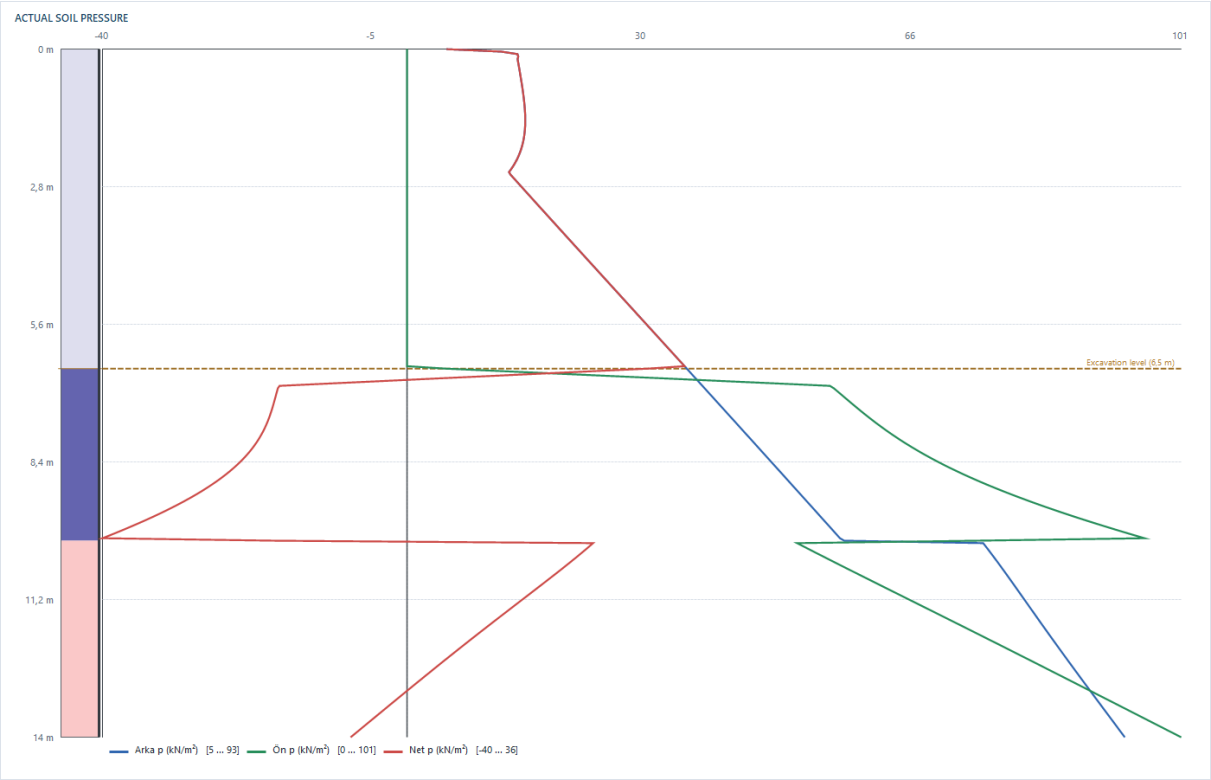


Figure 21: Street Face — Free excavation: 6.50 m — Actual soil pressure

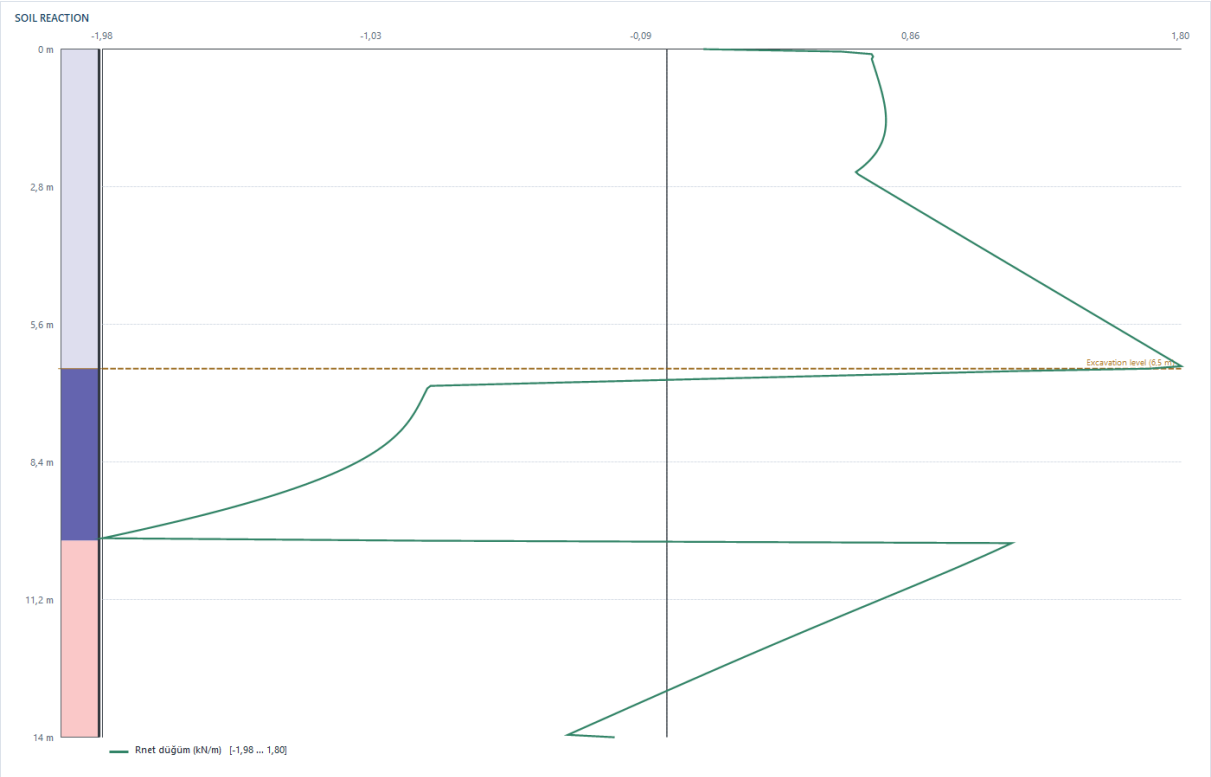


Figure 22: Street Face — Free excavation: 6.50 m — Soil reaction

5.5.4 Anchor 2 installation

Anchor row no. 2 has been installed at level 6 m and a prestress force of 300 kN applied; free length 10 m, bond length 12 m, inclined at 15° to the horizontal, plan spacing 3 m, and 3 strands of 0.60 in have been used.

Table 38: Quantity · Value · Unit

Quantity	Value	Unit
Excavation level	6.5	m
Maximum horizontal displacement	1.2	mm
Maximum bending moment	39.8	kNm/m
Maximum shear force	62.9	kN/m
Convergence	Satisfied	—
Iteration	4	—

Table 39: Support · No. · Depth (m)

Support	No.	Depth (m)	Total horizontal force (kN/m)
Anchor	1	2.5	-95.3
Anchor	2	6	-96.2

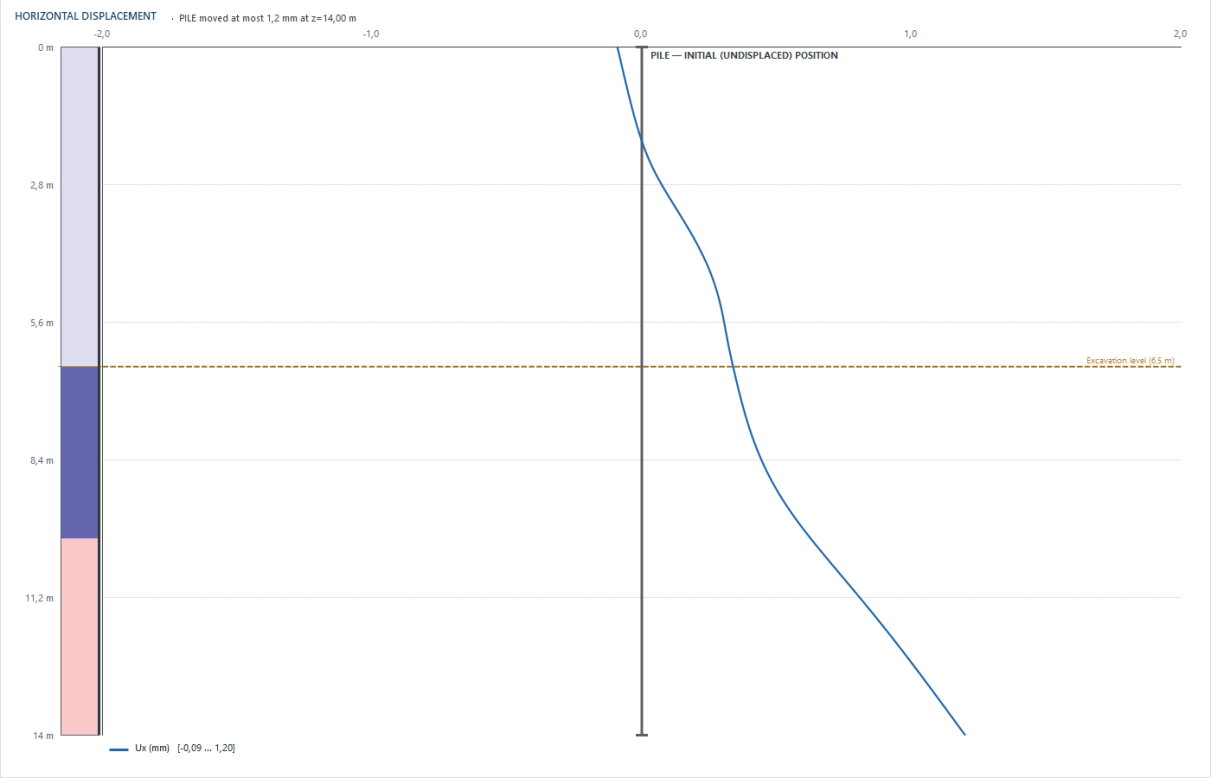


Figure 23: Street Face — Anchor 2 installation — Horizontal displacement

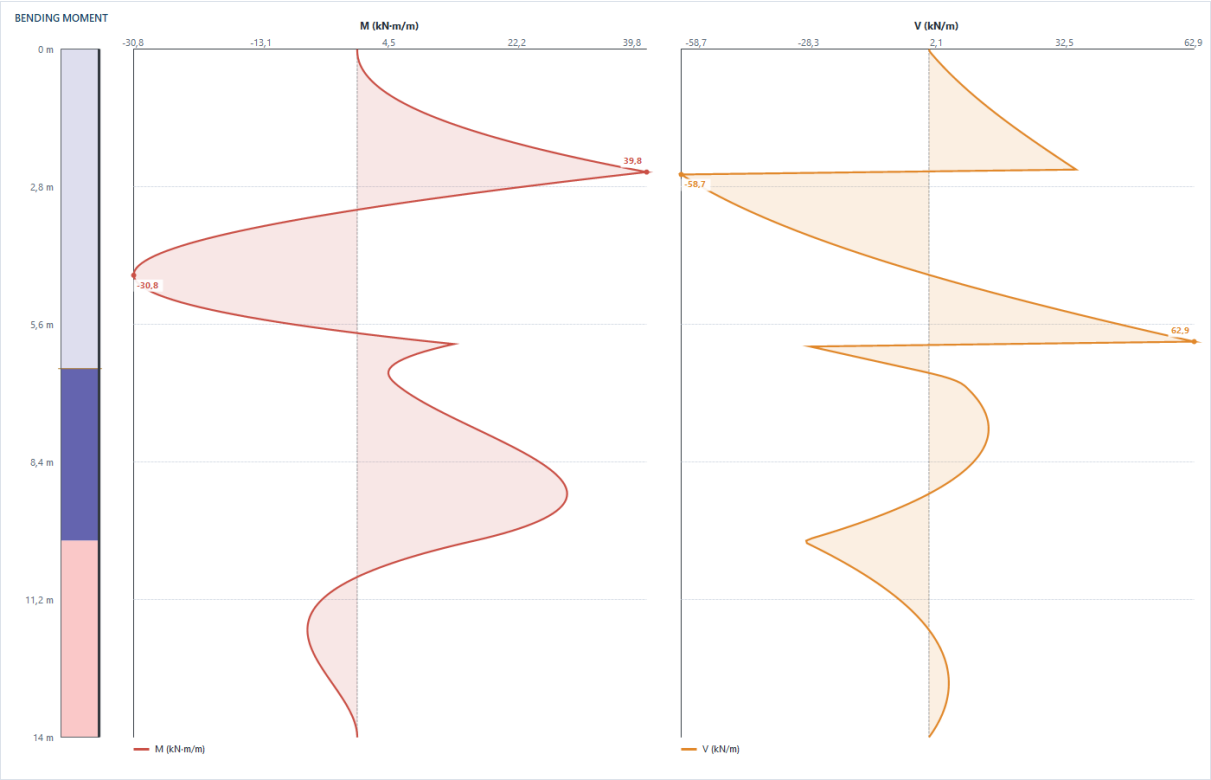


Figure 24: Street Face — Anchor 2 installation — Bending moment and shear force



Figure 25: Street Face — Anchor 2 installation — Axial force

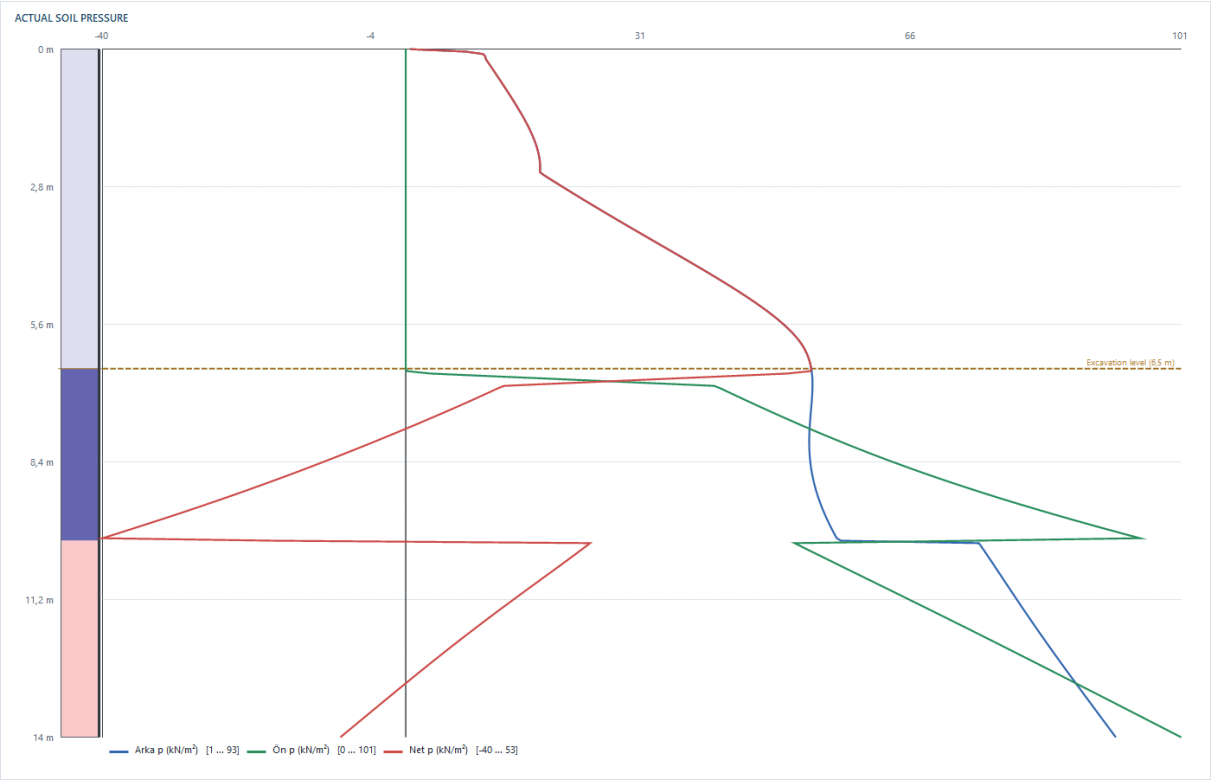


Figure 26: Street Face — Anchor 2 installation — Actual soil pressure

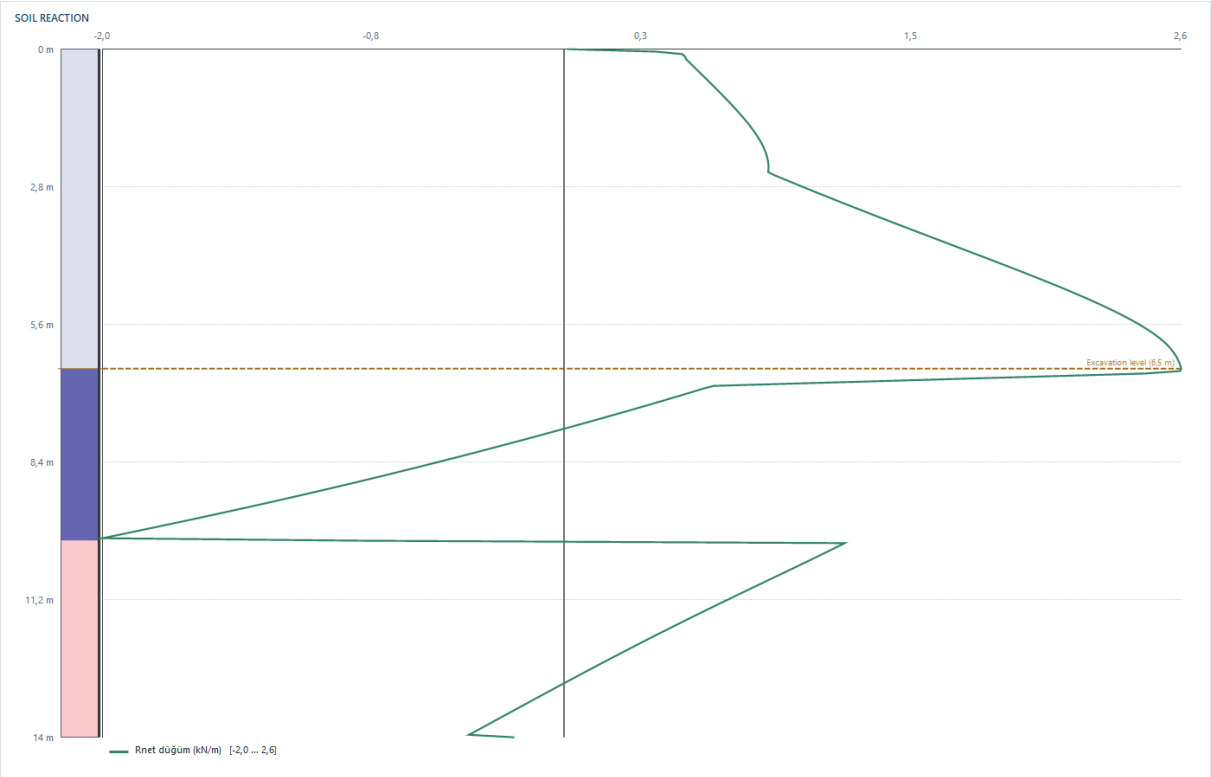


Figure 27: Street Face — Anchor 2 installation — Soil reaction

5.5.5 Final excavation

The excavation has been taken down from level 6.5 m to level 10 m; at this stage 3.5 m of excavation has been carried out. Since the lowest support is at level 6 m, the unsupported (free) excavation height is 4 m.

Table 40: Quantity · Value · Unit

Quantity	Value	Unit
Excavation level	10	m
Maximum horizontal displacement	4.8	mm
Maximum bending moment	210.6	kNm/m
Maximum shear force	94.3	kN/m
Convergence	Satisfied	—
Iteration	10	—

Table 41: Support · No. · Depth (m)

Support	No.	Depth (m)	Total horizontal force (kN/m)
Anchor	1	2.5	-99
Anchor	2	6	-105.8

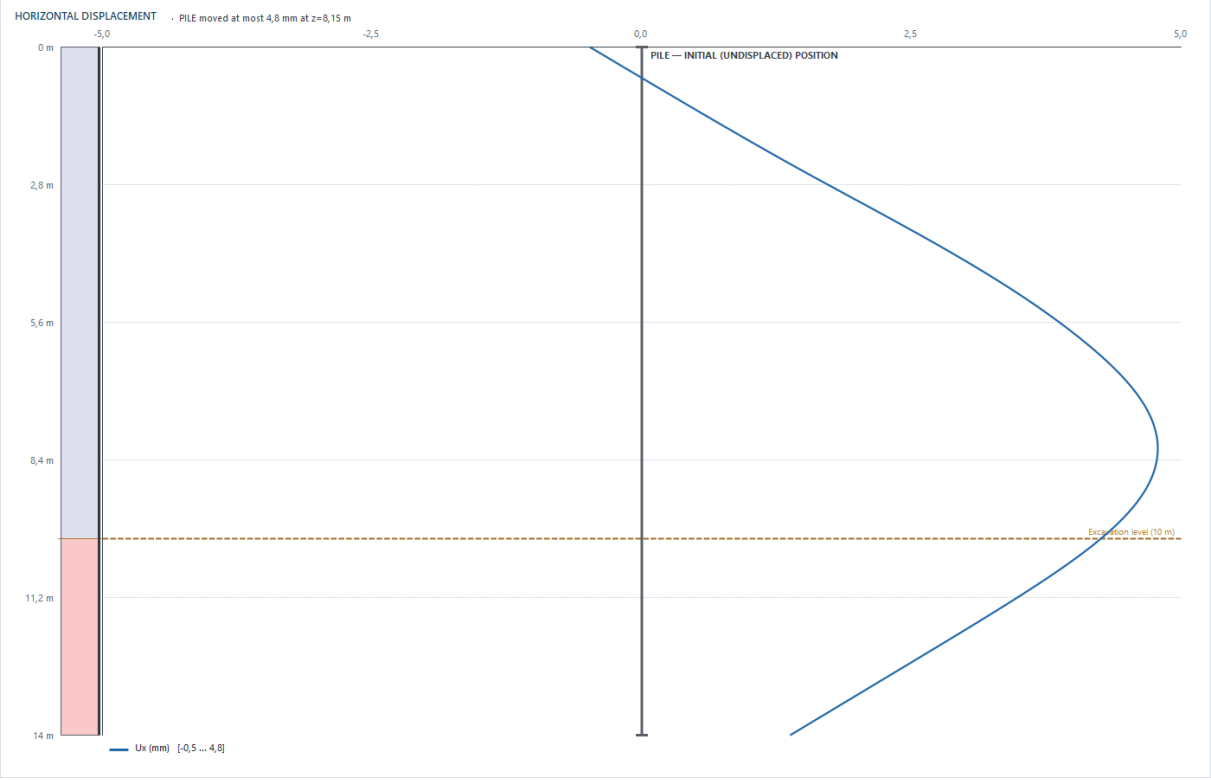


Figure 28: Street Face — Final excavation — Horizontal displacement

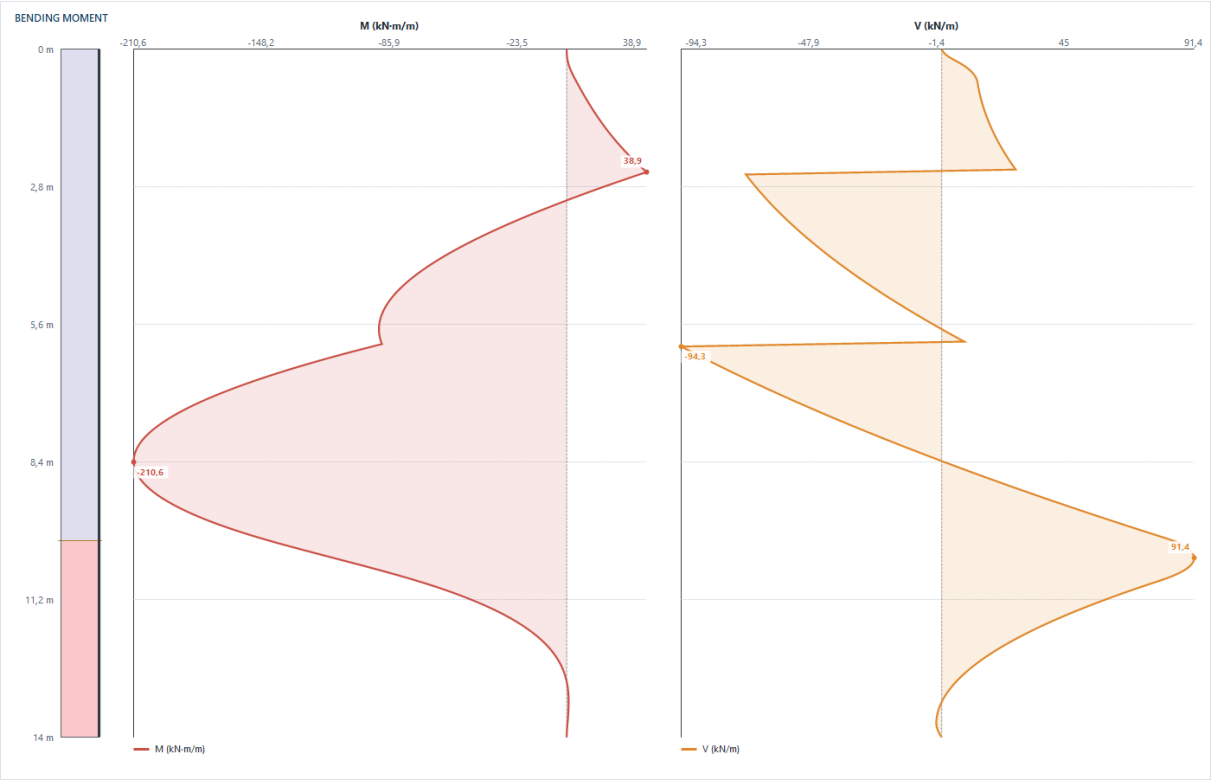


Figure 29: Street Face — Final excavation — Bending moment and shear force



Figure 30: Street Face — Final excavation — Axial force

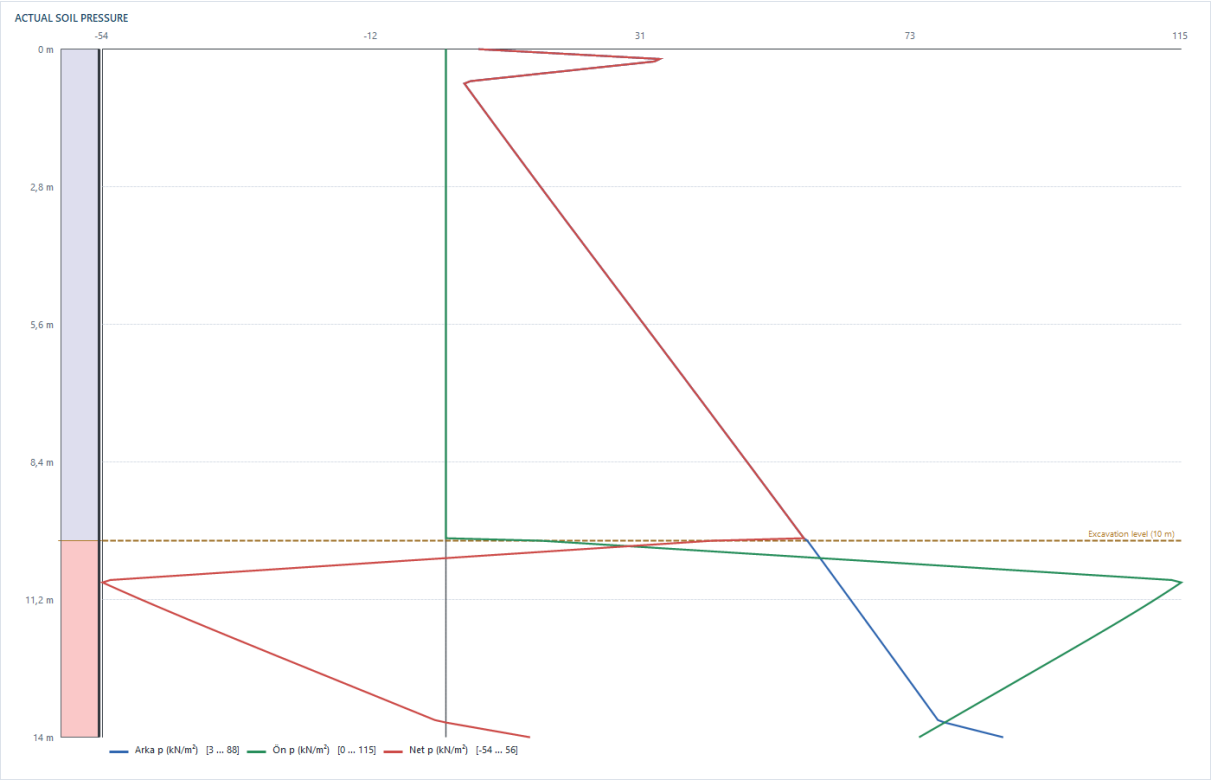


Figure 31: Street Face — Final excavation — Actual soil pressure

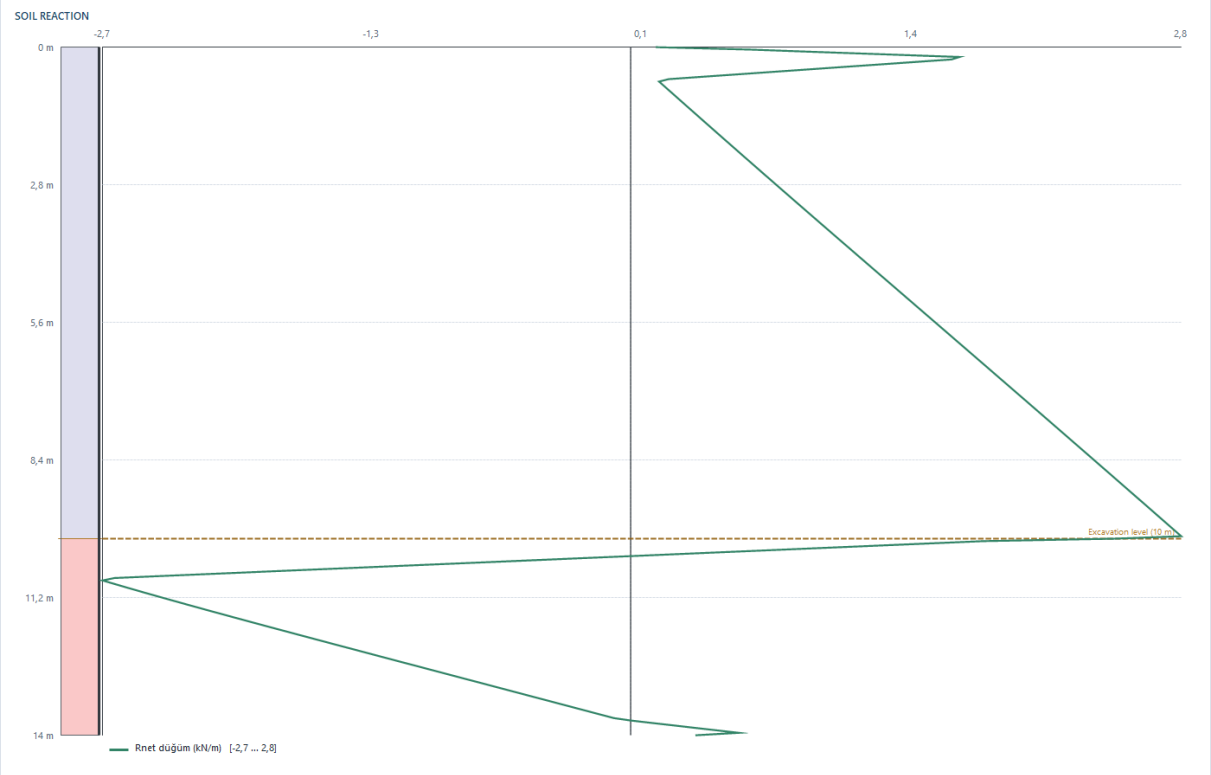


Figure 32: Street Face — Final excavation — Soil reaction

5.6 Envelope values and design actions

The design actions have been obtained by combining the most unfavourable values of all stages. The reinforced concrete and steel design has been carried out on these envelope values.

Table 42: Quantity · Envelope value · Unit

Quantity	Envelope value	Unit	Governing stage
Maximum horizontal displacement	4.8	mm	Final excavation
Maximum bending moment	210.6	kNm/m	Final excavation
Maximum shear force	94.3	kN/m	Final excavation

Table 43: Check · Computed · Limit

Check	Computed	Limit	Result
Horizontal displacement	4.8 mm	30 mm (Anchored, 2.12.7: $\delta_{h,lim}$ = ‰3.0×H.)	SATISFACTORY

5.7 Geotechnical checks

5.7.1 Embedment depth and passive resistance

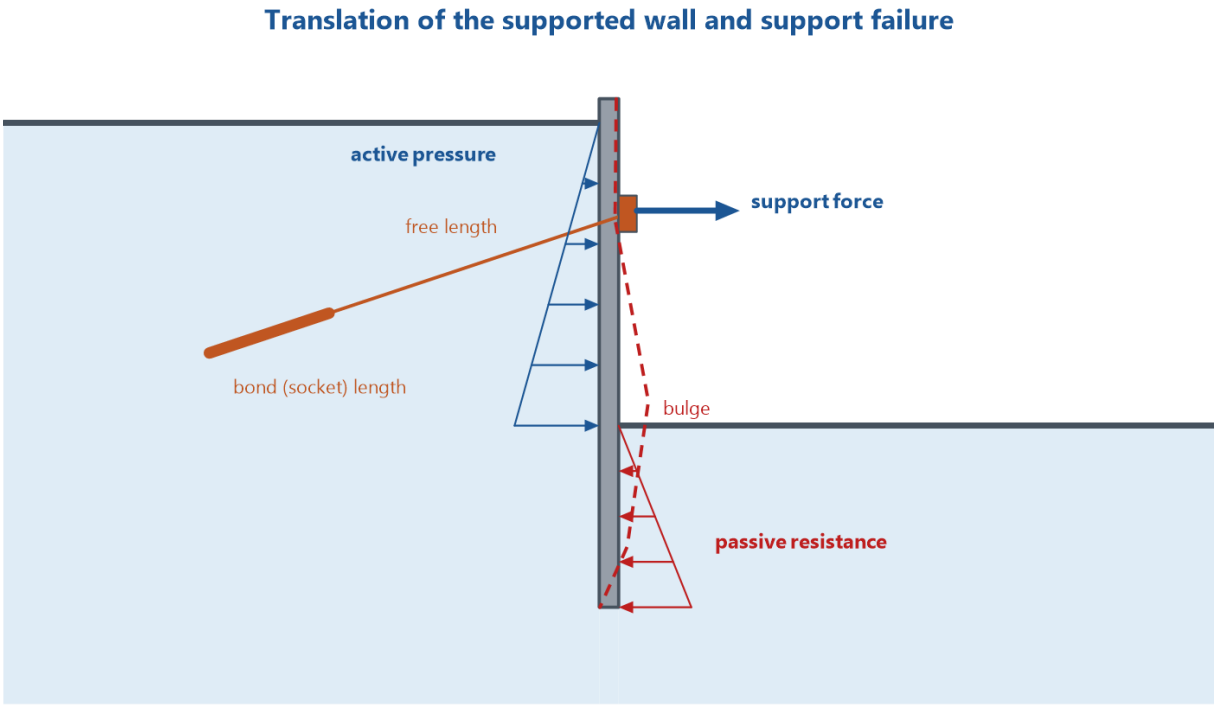


Figure 33: Translation of the supported wall

What does this check prevent? — In a wall supported by anchors or struts, failure takes the form of a system held at the support level bulging and translating between the support and the excavation base. Three separate limit states are asked at the same time: the bending/shear resistance of the wall section, the passive resistance over the embedment depth and the capacity of the support element itself. If the support fails, the system turns into a cantilever at once; the safety of the anchor capacity is therefore more critical than that of the wall section. [KDYY — limit state: translation, failure of a support element]

The part of the wall below the excavation base (the embedment) resists the overturning effect of the active thrust by its passive resistance. Moment equilibrium has been established about a point near the wall toe in a cantilever system and about the uppermost support point in a supported system.

$$Pa = \int [\sigma_a'(z) + u_{back}(z)]\, dz \qquad Pp = \int [\sigma_p'(z) + u_{front}(z)]\, dz \quad \text{(layer by layer, including water and surcharge)}$$

$$FoS = M_{resisting} / M_{overturning}$$

Supported · excavation 10 m · bond 4 m (d/H=0.40) · thrusts integrated layer by layer from the node stresses (active: Mazindrani, passive: Caquot–Kérisel (Total Stress in undrained layers))

Table 44: Quantity · Value · Unit

Quantity	Value	Unit
Excavation depth (H)	10	m
Embedment depth (d)	4	m
Active thrust (Pa)	548.7	kN/m
Passive resistance (Pp)	1050.6	kN/m
Overturning moment	2052.7	kNm/m
Resisting moment	6964.8	kNm/m
Factor of safety	3.393	—
Required	1.30	—
Result	SATISFACTORY	—

5.7.2 Mobilisation of the passive resistance

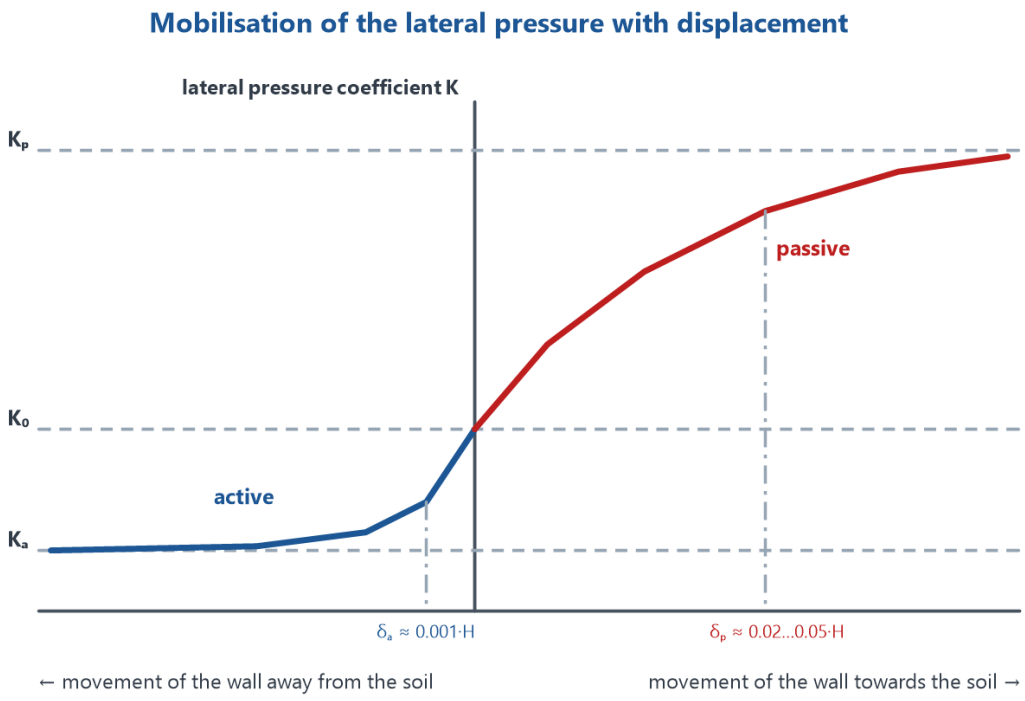


Figure 34: Mobilisation of the passive resistance

What does this check prevent? — The active pressure is fully developed by a displacement of the order of one per mille; the passive resistance, however, reaches its full value only after a movement of two to five per cent of the excavation depth. Using the full K_p in a check therefore means accepting that the system will undergo that displacement. In works with a displacement limit only part of the passive resistance is mobilised; this means an embedment depth that passes the check may fail the displacement criterion. The two criteria shall be asked separately. [KDYY — mobilisation ratio of passive resistance]

The embedment factor of safety measures the total capacity; the mobilisation ratio shows how much of the available passive reserve is actually used.

Table 45: Quantity · Value · Unit

Quantity	Value	Unit
Mobilised passive resistance	357.4	kN/m
Available passive resistance	1050.6	kN/m
Mobilisation ratio	%34	—

ASSESSMENT — The surcharge contribution is included in the active thrust: the surface load via $K_a \cdot q$, strip/line/point loads via Boussinesq $\Delta\sigma v \cdot K_a$ (same as the solver).

ASSESSMENT — This check is STATIC: no earthquake (Mononobe–Okabe) increase has been applied; for the embedment adequacy under the earthquake case the solution of the 'Earthquake check' stage shall govern.

ASSESSMENT — Caquot active: $|\delta|/\phi' = 0.67 > 2/3$ (Well Graded Sand). In this range the approximate ρ relation may lie 3-4% above the Kerisel-Absi table; the result is on the safe side but somewhat conservative.

ASSESSMENT — Retained side, Well Graded Sand: there are nodes where water pressure is applied within the tension crack zone (otherwise the pressure would have been 0).

ASSESSMENT — Retained Well Graded Sand: there are nodes where the computed at-rest pressure falls BELOW the active limit (with sloping retained soil K_a may exceed K_0). At these nodes σ_0 has been raised to σ_a — in the spring model the at-rest pressure cannot be below the active limit. In this zone the soil is no longer at rest but in the ACTIVE state.

5.7.3 Displacement and collapse criterion

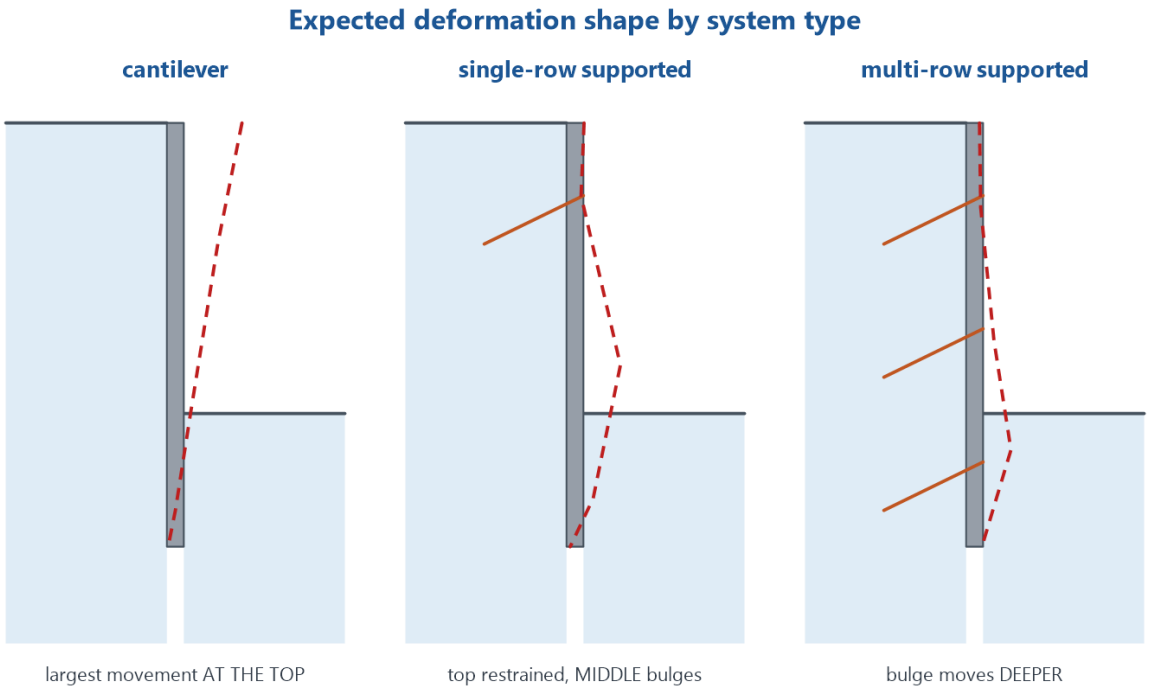


Figure 35: Deformation shape by system type

What does this check prevent? — Even if the computed maximum displacement is similar for the same excavation depth, the SHAPE of the deformation varies with the system type, and the effect on the structure behind is governed by that shape. In a cantilever system the movement concentrates at the top; the settlement trough is adjacent to the wall and steeply inclined. As supports are added the top is restrained, the bulge moves downward and the trough moves away from the wall and flattens. Therefore, at the position closest to the adjacent structure, shifting the maximum displacement deeper is as useful as reducing it. Deviation of the measured shape from the expected shape is the first thing to look at in instrumentation: a profile inconsistent with the calculation usually indicates that a support is not working. [KDYY — expected behaviour by system type]

Separately from the serviceability limits, it has been checked at every stage whether the displacement exceeds 3% of the largest FREE excavation height seen up to that point.

Table 46: Stage · Excavation (m) · Free excavation (m)

Stage	Excavation (m)	Free excavation (m)	Displacement (mm)	Threshold (mm)	Ratio	Result
Free excavation: 3.00 m	3	3	1.1	90	%0.04	SATISFACTORY
Anchor 1 installation	3	3	0.9	90	%0.03	SATISFACTORY
Free excavation: 6.50 m	6.5	4	1.2	120	%0.03	SATISFACTORY
Anchor 2 installation	6.5	4	1.2	120	%0.03	SATISFACTORY
Final excavation	10	4	4.8	120	%0.12	SATISFACTORY

The Broms (1965) method is not applicable: There is at least one DRAINED layer in the embedment zone — Broms' pu = 9·su·B relation has no counterpart in drained soil. This method is NOT APPLICABLE (this does not mean that the embedment is inadequate).

5.7.4 Hydraulic checks and basal heave

Since there is no water level difference, the sand boiling check is not applicable.

The uplift check is not applicable: The uplift check requires the PLAN AREA and the PERIMETER of the excavation; no closed excavation perimeter is defined in the layout plan. Enter the faces in the Plan section or load an AutoCAD file.

As the soil below the excavation base is not undrained clay, the basal heave check is not applicable.

5.7.5 Anchor checks

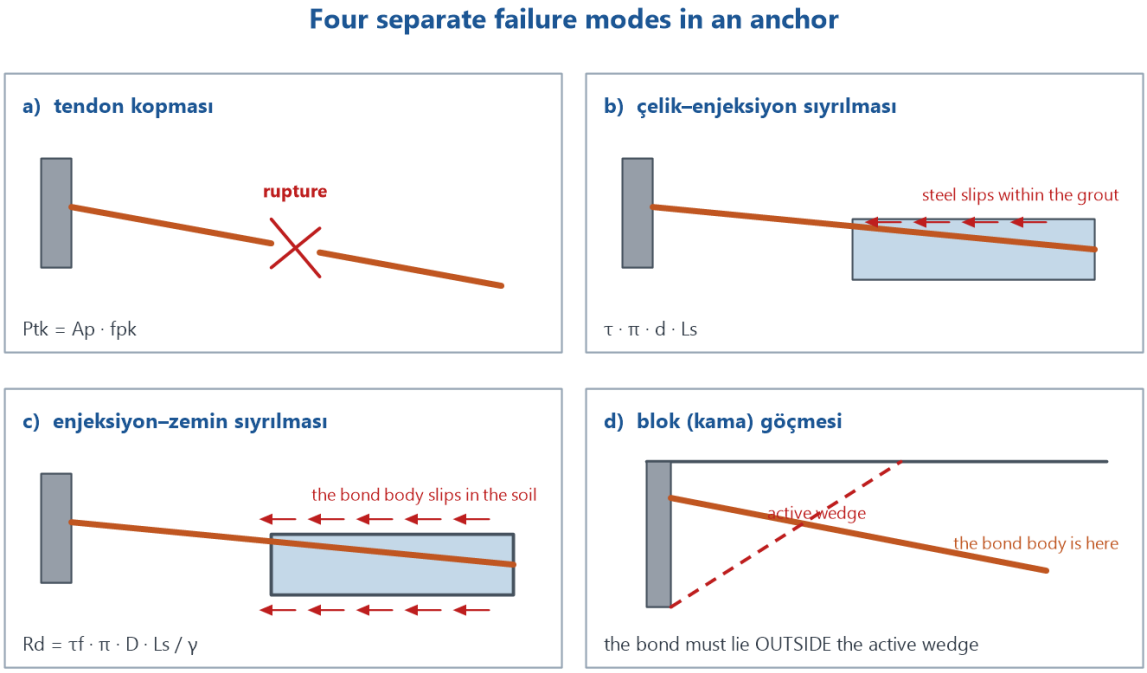


Figure 36: Failure modes in an anchor

> **What does this check prevent? —** The capacity of an anchor is not a single number; it is the smallest of four independent limit states. The tensile strength of the tendon and the steel–grout bond are under the control of fabrication and are covered safely by calculation. The third one carries the high uncertainty: the unit skin friction between the grout body and the soil varies several-fold with soil class and grouting method; for this reason the pull-out test is not a verification but a part of the design. The fourth is not a capacity but a geometric condition: the bond length must lie outside the active wedge, otherwise the anchor is tied to the very mass it restrains. [KDYY / EN 1537 — anchor limit states]

For each anchor the three failure modes have been checked separately and the smallest factor of safety has been taken as governing.

$$\text{Strand rupture:} \quad R_{steel} = n \cdot P_{rupture}$$

$$\text{Bond pull-out:} \quad R_a = \tau_f \cdot L_{root} \quad (\text{if } \tau_f \text{ is derived, } \pi \cdot D_k \cdot L_{root} \cdot \tau_f / \xi)$$

$$\text{Grout bond: } \tau_c, \text{ from user input or from the TS500/ACI relation}$$

Table 47: Safety criteria used in the anchor checks

Criterion	Value
Required FoS — strand rupture (information)	1.60
Required FoS — pull-out (information)	1.50

Criterion	Value
Shoring system	Temporary
DA2 partial factors (governing) $\gamma_{G,dst} \cdot \gamma_{Re} / \gamma_s$	1.50 · 1.40 / 1.55

Eurocode 7 Design Approach 2 (DA2) is active: the anchor verdict is given with partial factors; Ed = $\gamma_{G,dst}$·Td ≤ R_d = R_k/γ_{Re}. Classical safety factors (FoS) remain below for information only and do not take part in the overall result.

Table 48: Anchor check — DA2 (governing)

Anchor	Check	Ed (kN)	R_d (kN)	$\gamma_{G,dst}$	γ_{Re}	Utilisation	Result
1	Strand rupture	461.8	504	1.50	1.55	0.92	SATISFACTORY
1	Soil-grout pull-out	461.8	666.5	1.50	1.40	0.69	SATISFACTORY
2	Strand rupture	494	504	1.50	1.55	0.98	SATISFACTORY
2	Soil-grout pull-out	494	868.4	1.50	1.40	0.57	SATISFACTORY

Table 49: Anchor · Depth (m) · Design load (kN)

Anchor	Depth (m)	Design load (kN)	Strand capacity (kN)	GS	Pull-out capacity (kN)	GS	Result (DA2)
1	2.5	307.8	781.2	2.54	933.1	3.03	SATISFACTORY
2	6	329.3	781.2	2.37	1215.8	3.69	SATISFACTORY

Table 50: Anchor · Strand-grout capacity (kN) · GS

Anchor	Strand-grout capacity (kN)	GS	Result
1	1740.9	5.66	SATISFACTORY
2	1740.9	5.29	SATISFACTORY

Table 51: Anchor · Governing failure mode

Anchor	Governing failure mode
1	DA2 / EC7 partial factors
2	DA2 / EC7 partial factors

ASSESSMENT — Anchor 1: Design load = solved axial force 307.8 kN (> prestress 300 kN; governing stage: Final excavation)

ASSESSMENT — Anchor 2: Design load = solved axial force 329.3 kN (> prestress 300 kN; governing stage: Final excavation)

ASSESSMENT — The soil-grout pull-out resistance (RootSoilResistance) is a value entered by the user on the Anchors tab — in this setting the program DOES NOT PRODUCE the value from the soil type. You may request its derivation with the 'Total stress (α_a·Su)' or 'Effective stress (K1·σ'$_v$·tanϕ')' option.

ASSESSMENT — The factors of safety (RequiredSafetyFactorSteel/Pullout) reflect the TYPICAL ranges in DIN 4125/EN 1537 practice — the project's own specification/code may prescribe a different value; it must be set by the user.

ASSESSMENT — The tendon/grout bond τ_c has been derived via TS500: C1=1/(4·C0), $f_{ctd}=0.35\cdot\sqrt{f_c}$, $\tau_c=C1\cdot f_{ctd}$ (f_c=30 MPa, C0=0.24).

ASSESSMENT — In strand-grout pull-out, ds is the NOMINAL COMBINED diameter equivalent to the total area of the strands (the bundle is assumed contiguous within the grout) — counting each strand as a separate circle would overstate the pull-out surface.

ASSESSMENT — In actual practice, INDEPENDENTLY of the computed capacity, verification of every anchor by an ON-SITE stressing/proof load test is essential — this calculation is intended for PRELIMINARY DESIGN only.

Whether the bond body of the anchor lies outside the active wedge is checked in section has been checked in 5.2.2. The Kranz internal stability check below assesses the sliding safety of the block formed jointly by the wall and the anchors: even if the bond body lies outside the wedge, the block may slide as a whole.

```
Fi = allowable anchor force (from Kranz block equilibrium)          FoS = Fi / F
```

Table 52: Anchor · Depth (m) · Block base angle θ (°)

Anchor	Depth (m)	Block base angle θ (°)	Block weight W (kN)	Allowable Fi (kN)	Actual F (kN)	GS	Result
1	2.5	25.5	3339.9	735.4	102.6	7.17	SATISFACTORY
2	6	14	3872.3	1387.9	109.8	12.64	SATISFACTORY

ASSESSMENT — The internal stability has been solved by the FULL FORCE EQUILIBRIUM of the ABCD block set up on the slip surface between the theoretical wall base point (A) and the MIDDLE point (B) of the anchor bond (Kranz type). The forces of the ACTIVE anchors other than the anchor being verified are included as forces acting on the block.

ASSESSMENT — The slip surface has been assumed to be a SINGLE PLANE joining A to B. The actual critical surface may come out different (circular or polygonal), particularly in multi-layered soil or where a distinct weak layer is present — in that case the slope/overall stability analysis shall be taken as governing.

ASSESSMENT — The block weight Wi is calculated from the area between the ground surface and the slip plane, using the thickness-weighted unit weights of the layers; below the water table the SUBMERGED unit weight (γ') is used and, consistently, the hydrostatic water forces on surfaces AD, BC and AB are added separately to the equilibrium (effective stress approach, 13.09.2026). Surface and strip/line surcharges ON the block (the portion within the horizontal extent of the block) are INCLUDED in Wi; point loads are not included under the plane strain assumption.

ASSESSMENT — The active thrusts Ea (AD) and Eai (BC) are cohesive: $Ea = \tfrac{1}{2}\cdot K_a\cdot\gamma\cdot H^2 - 2c\cdot\sqrt{K_a}\cdot H + q\cdot K_a\cdot H$ (≥0). In an UNDRAINED layer $\phi=0$, $K_a=1$ and $c=c_u$ are taken (also on the slip surface $\phi_i=0$, $Ci=c_u\cdot L$); in a drained layer φ', c' (13.09.2026).

5.7.6 Pile axial bearing capacity and settlement

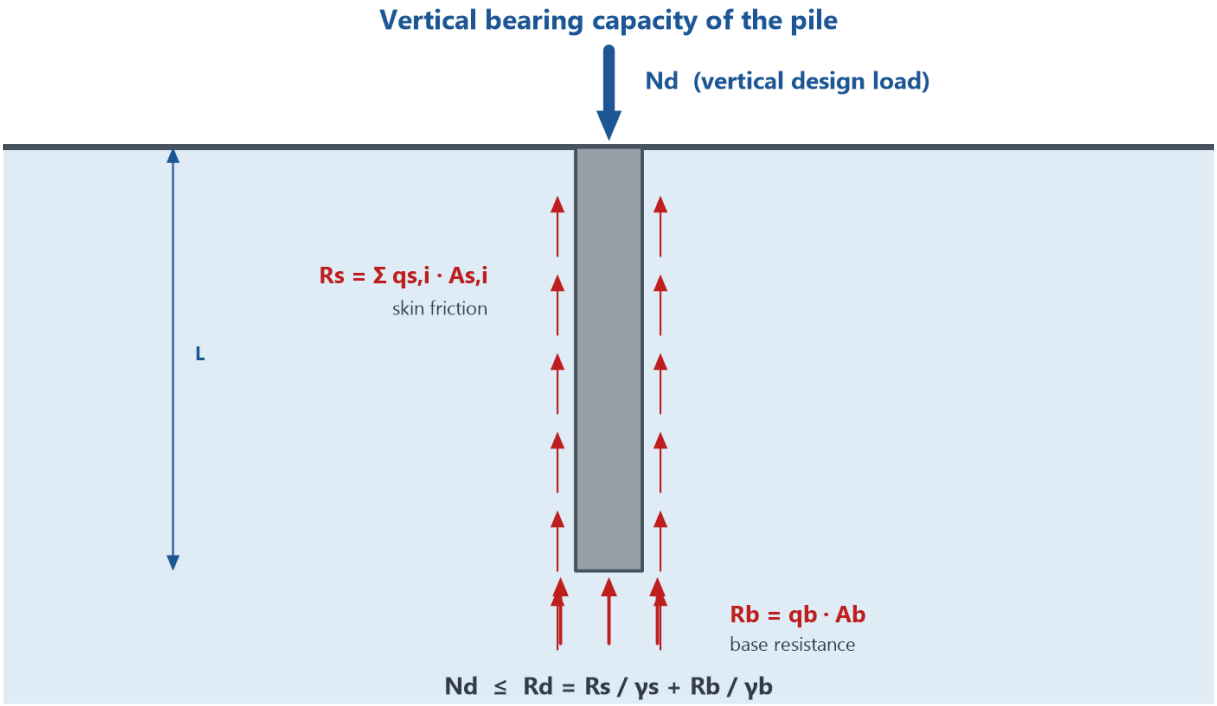


Figure 37: Vertical bearing capacity

What does this check prevent? — The shoring pile is designed as a member working under horizontal load, but since the anchor is inclined, each anchor row imposes a vertical component on the pile. Together with the self-weight of the pile, this load is carried by skin friction and base resistance. Skin friction is mobilised at once along the pile with a small settlement; base resistance, however, requires a settlement of the order of five to ten per cent of the diameter. Therefore at service load the greater part of the bearing comes from the shaft, and a design leaning on base resistance alone fails on the settlement criterion even if it passes the check. [KDYY / TS EN 1997 — axial bearing capacity]

The vertical component arising from the anchor inclination and the self-weight of the wall impose an axial load on the pile. Skin friction and end bearing are computed separately.

$$\text{Undrained skin friction: } q_s = \alpha \cdot c_u \qquad \text{Drained skin friction: } q_s = K \cdot \sigma'_v \cdot \tan\delta$$

$$\text{End resistance (Vesic): } q_b = c \cdot N_c + \sigma'_v \cdot N_q \qquad \text{FoS} = (Qs + Qb) / N$$

Table 53: Quantity · Value · Unit

Quantity	Value	Unit
Pile diameter	0.8	m
Pile length	14	m
Applied axial load N	361.2	kN
Skin friction capacity Qs	930.1	kN
End resistance capacity Qb	3672.8	kN
Unit end resistance q_b	7306.9	kPa
Total capacity Qt = Qs + Qb	4603	kN
Factor of safety FoS = Qt / N	12.74	—
Required	2.50	—
Result	SATISFACTORY	—

$$GS = (930.1 + 3672.8) / 361.2 = 12.74$$

The settlement has been summed from three components by the Vesic (1977) semi-empirical method:

$$s = s_{elastic} + s_{base} + s_{skin}$$

Table 54: Settlement component · Value · Unit

Settlement component	Value	Unit
Elastic shortening $s_{elastic}$	0.28	mm
End mobilisation s_{end}	1.48	mm
Skin mobilisation s_{skin}	0.03	mm
Total settlement s	1.8	mm
Limit min(25 mm ; 0.10·D)	25	mm
Result	SATISFACTORY	—

$$s = 0.28 + 1.48 + 0.03 = 1.8 \text{ mm}$$

Skin 930.1 kN + toe 3672.8 kN = 4603 kN; applied 361.2 kN (361.2 kN/m per metre strip × pile spacing 1 m).

ASSESSMENT — The settlement was computed by the Vesic (1977) semi-empirical method: elastic shortening + base mobilisation + shaft mobilisation. THE GROUP EFFECT IS NOT CONSIDERED — since the piles are close to one another in a spaced bored pile wall, the actual settlement may be somewhat LARGER; in this respect the result is not on the safe side.

ASSESSMENT — Negative skin friction (fill settlement) and time-dependent consolidation settlement are NOT INCLUDED; the method gives the immediate settlement.

LIMITATION — Group effect has not been taken into account; in spaced piles the actual settlement is somewhat larger and the result is in this respect not on the safe side. Negative skin friction and consolidation settlement are likewise not included in the calculation.

5.8 Overall stability (slope stability)

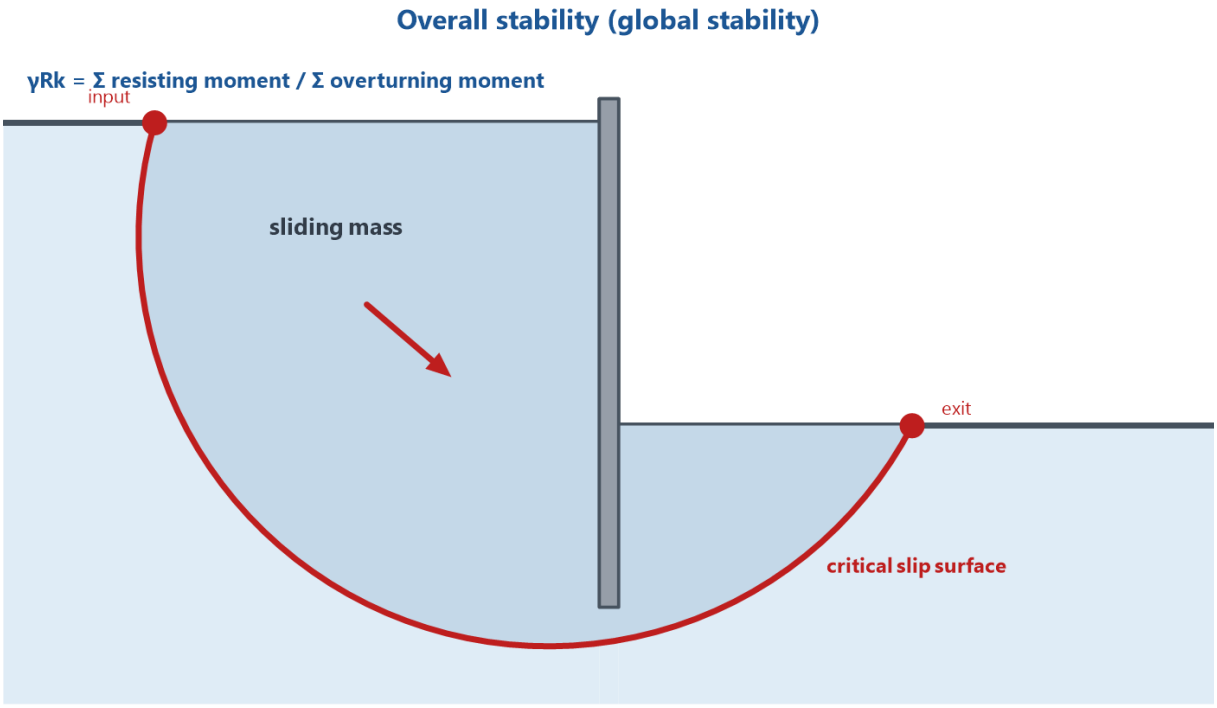


Figure 38: Overall stability

> **What does this check prevent? —** Even if the wall and the support members are adequate on their own, a soil mass containing the wall may slide as a whole. In this check the wall is not a structural member but a source of resistance cutting the slip surface; the anchors likewise add resistance where they cross the surface. The critical surface either passes below the wall toe or cuts the vertical element directly. In the second case the factor of safety is governed no longer by the soil but by the shear capacity of the pile. [TBDY 2018 Eq. 16.30 / KDYY — overall stability]

Even if the section check passes, whether the system will slide together with the soil has been investigated by limit equilibrium methods. The wall is modelled not as a rigid block but as an element having SHEAR AND FLEXURAL capacity; the rigid block assumption attributes infinite strength to the wall and inflates the factor of safety on the unsafe side.

$$\text{transferred force} = \min(V_r \ , \ 2 \cdot Mp \ / \ L_{hinge})$$

Table 55: Quantity · Value

Quantity	Value
Characteristic factor of safety (γ_{Rk})	1.878
Required	1.50
Governing case	static
Critical surface generator	ThreePoint
FS method	Spencer
TBDY 2018 Eq. 16.30 (separate circular search, information)	1.856
Result	SATISFACTORY

5.8.1 KDYY partial factor verification

$$\text{static case: design FoS} = \gamma_{Rk} \ / \ \gamma_{G,dst} = 1.878 \ / \ 1.35 = 1.391 \quad (\geq 1.00)$$

SCOPE — The ratio above applies the permanent action factor only. A surcharge is a VARIABLE action and is amplified by 1.50 in KDYY; in sections where the surcharge dominates this ratio is somewhat optimistic.

5.8.2 Method comparison

The critical surface has been found by one method and the same surface has been re-solved with all limit equilibrium methods. Convergence of the methods is a measure of the robustness of the result.

MAIN γ_{Rk} AND TBDY 16.30 — The main value comes from the critical surface searched with the Spencer method (moment and force equilibrium together); the table row of the same name equals the main value. The other methods (Janbu, Bishop, …) on the same surface are for information. The TBDY 2018 Equation 16.30 value is given for information from a separate circular surface search: in the code's slice resistance the cohesion term is c'·b (b = slice width), which is cosα times the classical Bishop term; hence the 16.30 value is usually lower.

Table 56: Method (classical definition) · GK · Equilibrium type

Method (classical definition)	GK	Equilibrium type	Note
TBDY 2018 (16.30)	1.858	moment	—
Fellenius (Ordinary)	1.416	moment	—
Bishop Simplified	1.866	moment	—
Janbu Simplified	1.892	force	—
Janbu Corrected	1.965	force (corrected)	—
Spencer	1.878	moment + force	—
Morgenstern–Price	—	moment + force	Did not converge
Corps of Engineers	—	force	Did not converge
Lowe–Karafiath	3.379	force	—

Scatter: 1.416 … 3.379 (138.6%)

5.8.3 The critical mechanism CUTS the vertical element

The critical slip surface does NOT pass around the toe of the vertical element; it CUTS its body above the toe (14.00 m). The shear capacity of the element is mobilised on this surface and has contributed 361.5 kN/m to the factor of safety.

THIS IS NOT A SEARCH DEFECT — The search also generates surfaces passing BELOW the toe of the vertical element. The critical one turns out to be the surface cutting the wall because the deep mechanism mobilises a LARGER mass and a longer slip arc, and therefore gives a higher factor of safety. When the shear capacity of the vertical element is increased, the critical mechanism shifts to the deep surface; after that shift, however, the factor of safety is limited by the SOIL and further increase in capacity does not change the result. The benefit of increasing capacity is bounded by this ceiling.

5.8.4 Critical slip surface and search details

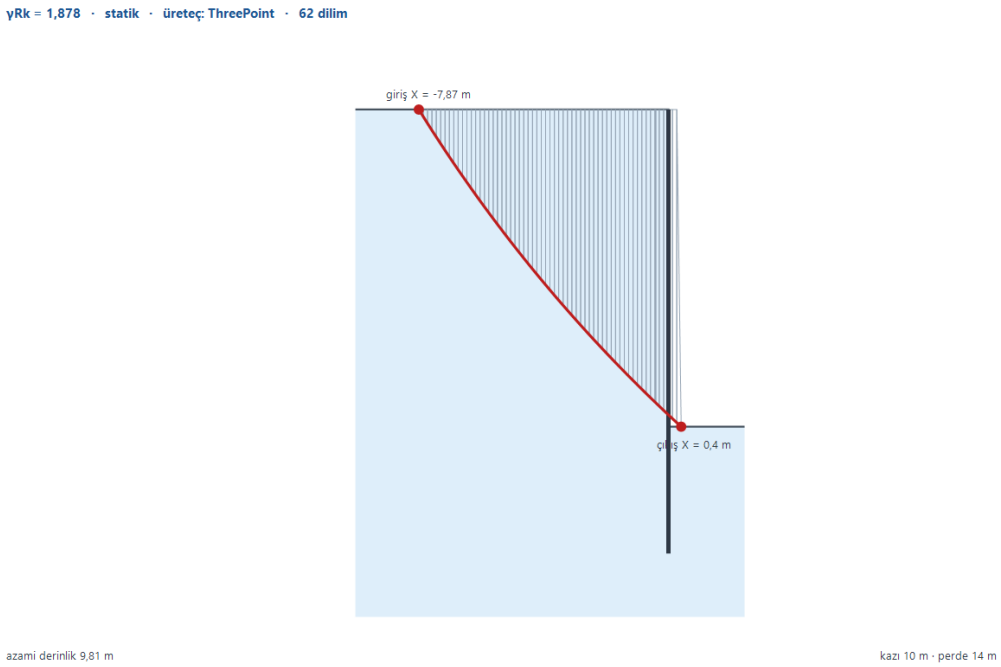


Figure 39: Street Face — critical slip surface and slices (γ_{Rk} = 1.878, ThreePoint)

Table 57: Quantity · Value · Unit

Quantity	Value	Unit
Entry point (X)	-7.87	m
Exit point (X)	0.4	m
Surface length (horizontal)	8.27	m
Maximum depth	9.81	m
Number of slices	62	pcs
Total surfaces generated	20680	pcs
Valid (kinematically admissible) surface	463	pcs
Surface generator	ThreePoint	—
Contributing support element	3	pcs
Resisting contribution of the supports ($\Sigma M/R_d$ — moment equilibrium)	496.7	kN/m
Infinite slope factor	6250	—

Table 58: Mechanism family · Number of surfaces · Lowest γ_{Rk}

Mechanism family	Number of surfaces	Lowest γ_{Rk}
CUTTING the pile (governing)	303	1.878
NOT cutting the pile — passing below the toe (level -14 m)	40	2.324

The second row is a CEILING: by increasing the shear capacity of the pile, γ_{Rk} can be raised at most to 2.324, because a mechanism that does not cut the pile is unaffected by that capacity. Since this value exceeds the required factor of safety, increasing the reinforcement or the diameter of the section is a route that reaches the target.

In a comparison against a finite element solution (ϕ-c reduction), the comparable quantity is also the second row: there the vertical member is modelled as a plate that does not fail in shear, so the mechanism cutting the member does not develop in the same way.

During the search 20680 surfaces were generated, and the 463 surfaces passing the kinematic and geometric filters were solved. Surfaces rejected as invalid are mechanisms in which the mass cannot move (the driving force is negative), which pass through the structure, or which do not reach the ground surface.

5.8.5 Loading cases

The search was performed separately for each load case and the most unfavourable result taken as governing. In the seismic case the vertical acceleration is applied both upward and downward ($\pm k_v$); the two may render different mechanisms critical.

Table 59: Load case · γ_{Rk} · Required

Load case	γ_{Rk}	Required	Result
static	1.878	1.50	SATISFACTORY

5.8.6 Slice breakdown of the critical surface

The sliding mass is divided into vertical slices; the driving and resisting forces are computed from the weight of each slice and the inclination of its base. The factor of safety is obtained over the sum of all slices:

$$FoS = \Sigma\ (resisting\ force)\ /\ \Sigma\ (driving\ force)$$

The driving force is the component of the slice weight tangential to the slip surface, while the resisting force is the shear strength mobilised along the base (cohesion and friction contribution). The contribution of the support members is added separately to this sum.

Table 60: Slice · X1 (m) · X2 (m)

Slice	X1 (m)	X2 (m)	Width (m)	Base level (m)	Top level (m)	Height (m)	Weight (kN/m)	Driving (kN/m)	Resisting (kN/m)
1	-7.87	-7.73	0.138	-0.11	0	0.11	3.1	2.6	2.6
2	-7.73	-7.6	0.138	-0.33	0	0.33	3.7	3.1	3
3	-7.6	-7.46	0.138	-0.55	0	0.55	4.3	3.6	3.5
4	-7.46	-7.32	0.138	-0.76	0	0.76	4.9	4.1	4
5	-7.32	-7.18	0.138	-0.98	0	0.98	5.4	4.6	4.4
6	-7.18	-7.04	0.138	-1.19	0	1.19	6	5	4.8
7	-7.04	-6.91	0.138	-1.39	0	1.39	6.6	5.5	5.3
8	-6.91	-6.77	0.138	-1.6	0	1.6	7.2	5.9	5.7
9	-6.77	-6.63	0.138	-1.8	0	1.8	7.7	6.4	6.1
10	-6.63	-6.49	0.138	-2	0	2	8.3	6.8	6.5
11	-6.49	-6.35	0.138	-2.2	0	2.2	8.8	7.2	6.9
12	-6.35	-6.22	0.138	-2.4	0	2.4	9.4	7.7	7.3
13	-6.22	-6.08	0.138	-2.59	0	2.59	9.9	8.1	7.7
14	-6.08	-5.94	0.138	-2.78	0	2.78	10.4	8.5	8.1
15	-5.94	-5.8	0.138	-2.97	0	2.97	11	8.9	8.5
16	-5.8	-5.67	0.138	-3.16	0	3.16	11.5	9.2	8.8
17	-5.67	-5.53	0.138	-3.35	0	3.35	12	9.6	9.2
18	-5.53	-5.4	0.127	-3.53	0	3.53	11.5	9.2	8.8

Slice	X1 (m)	X2 (m)	Width (m)	Base level (m)	Top level (m)	Height (m)	Weight (kN/m)	Driving (kN/m)	Resisting (kN/m)
19	-5.4	-5.39	0.01	-3.62	0	3.62	0.8	0.6	0.6
20	-5.39	-5.25	0.138	-3.71	0	3.71	10.2	8.2	7.8
21	-5.25	-5.11	0.138	-3.9	0	3.9	10.7	8.5	8.2
22	-5.11	-4.98	0.138	-4.07	0	4.07	11.2	8.9	8.5
23	-4.98	-4.84	0.138	-4.25	0	4.25	11.7	9.2	8.9
24	-4.84	-4.7	0.138	-4.43	0	4.43	12.2	9.6	9.2
25	-4.7	-4.56	0.138	-4.6	0	4.6	12.7	9.9	9.5
26	-4.56	-4.42	0.138	-4.77	0	4.77	13.2	10.3	9.9
27	-4.42	-4.29	0.138	-4.94	0	4.94	13.6	10.6	10.2
28	-4.29	-4.15	0.138	-5.11	0	5.11	14.1	10.9	10.5
29	-4.15	-4.01	0.138	-5.28	0	5.28	14.6	11.2	10.8
30	-4.01	-3.87	0.138	-5.45	0	5.45	15	11.5	11.1
31	-3.87	-3.74	0.138	-5.61	0	5.61	15.5	11.8	11.4
32	-3.74	-3.6	0.138	-5.77	0	5.77	15.9	12.1	11.7
33	-3.6	-3.46	0.138	-5.94	0	5.94	16.4	12.4	12
34	-3.46	-3.32	0.138	-6.1	0	6.1	16.8	12.7	12.3
35	-3.32	-3.18	0.138	-6.25	0	6.25	17.2	13	12.6
36	-3.18	-3.05	0.138	-6.41	0	6.41	17.7	13.3	12.9
37	-3.05	-2.91	0.138	-6.57	0	6.57	18.1	13.5	13.2
38	-2.91	-2.77	0.138	-6.72	0	6.72	18.5	13.8	13.4
39	-2.77	-2.63	0.138	-6.87	0	6.87	19	14.1	13.7
40	-2.63	-2.49	0.138	-7.03	0	7.03	19.4	14.3	14
41	-2.49	-2.36	0.138	-7.18	0	7.18	19.8	14.6	14.2
42	-2.36	-2.22	0.138	-7.33	0	7.33	20.2	14.8	14.5
43	-2.22	-2.08	0.138	-7.47	0	7.47	20.6	15	14.8
44	-2.08	-1.94	0.138	-7.62	0	7.62	21	15.3	15
45	-1.94	-1.81	0.138	-7.77	0	7.77	21.4	15.5	15.3
46	-1.81	-1.67	0.138	-7.91	0	7.91	21.8	15.7	15.5
47	-1.67	-1.53	0.138	-8.05	0	8.05	22.2	16	15.8
48	-1.53	-1.39	0.138	-8.19	0	8.19	22.6	16.2	16
49	-1.39	-1.25	0.138	-8.34	0	8.34	23	16.4	16.2
50	-1.25	-1.12	0.138	-8.47	0	8.47	23.4	16.6	16.5
51	-1.12	-0.98	0.138	-8.61	0	8.61	23.7	16.8	16.7
52	-0.98	-0.84	0.138	-8.75	0	8.75	24.1	17	17
53	-0.84	-0.7	0.138	-8.89	0	8.89	24.5	17.2	17.2
54	-0.7	-0.56	0.138	-9.02	0	9.02	24.9	17.4	17.4
55	-0.56	-0.43	0.138	-9.16	0	9.16	25.2	17.5	17.6
56	-0.43	-0.4	0.027	-9.24	0	9.24	5	3.5	3.5
57	-0.4	-0.29	0.111	-9.3	0	9.3	20.6	14.3	14.4
58	-0.29	-0.15	0.138	-9.42	0	9.42	26	17.9	18.1
59	-0.15	-0.01	0.138	-9.55	0	9.55	26.3	18.1	18.3
60	-0.01	0.12	0.138	-9.68	0	9.68	26.7	18.2	18.5

Slice	X1 (m)	X2 (m)	Width (m)	Base level (m)	Top level (m)	Height (m)	Weight (kN/m)	Driving (kN/m)	Resisting (kN/m)
61	0.12	0.26	0.138	-9.81	0	9.81	27	18.4	18.7
62	0.26	0.4	0.138	-9.94	-5	4.94	13.6	9.2	9.5

Table 61: Total · Value · Unit

Total	Value	Unit
Total weight of the sliding mass	939.7	kN/m
Total driving force Σ D	697.8	kN/m
Total resisting force Σ R (soil)	684.2	kN/m
Contribution of the support members ($\Sigma M/R_d$ — moment equilibrium)	496.7	kN/m
Factor of safety γ_{Rk}	1.878	—

```
with soil contribution FoS = 684.2 / 697.8 = 0.980
```

> **ITERATION —** In methods that satisfy force and moment equilibrium (Bishop, Janbu, Spencer, Morgenstern–Price) the factor of safety appears on BOTH SIDES of the equation at once: the normal force at the slice base depends on the FoS, and the FoS on the normal force. The solution is therefore obtained by successive approximation: the normal forces are computed with an initial FoS, a new FoS is obtained, and the process is repeated until the difference between two successive values falls to a negligible level. The value above is the converged solution.

5.8.7 Engine diagnostic logs

The lines below show how the model was built and which elements were included in the calculation; they are given for checking purposes.

- Governing FS method: Spencer. The other methods (Janbu, Bishop, M-P, …) on the same critical surface are for INFORMATION.
- TBDY 2018 Eq. 16.30 (circular surfaces, separate search, statik): γ_{Rk} = 1.856 (information; not governing).
- 11 segments -> 15 split segments -> 14 edges; 0 dangling edges discarded; 4 closed regions found.
- 1 yuzey sursarji dilim agirliklarina added.
- Method selection: all surfaces solved with 'Spencer'.
- Impenetrable (bedrock) layer not marked; no composite surface generated.
- 3049 two-block wedge surfaces generated.
- 0 “undercutting” surfaces generated (0 from structure/block lower bounds).
- 1 material interfaces scanned; 503 weak-layer-following surfaces generated.
- Local refinement: 8 seeds, 386 additional evaluations; minimum factor of safety 1.8998 -> 1.8780.
- Excavation support mode: 2712 candidate surfaces were eliminated because they lie entirely BEHIND the wall (x_{exit} < 0 m) — these are not a global failure of the excavation support but the surcharge's own bearing capacity mechanism, which is checked separately.
- 42 candidate surfaces were rejected because they run along a side boundary of the model (the box, not the soil, would govern).

• Pre-screening (fast mode): the best 15 candidates per generator and the first 40 of the overall ranking by the reference method (Bishop / Janbu) were solved with the selected method; 8476 candidates were not solved (threshold: 1.5 × the best reference).

• 65 candidate surfaces were discarded because 'Spencer' did not converge or its root was not admissible; no fallback to another method.

• [DESTEK] Support criterion: MOMENT equilibrium — sign of the moment (circular/composite surface, centre of rotation exists)

• [DESTEK] Support 'Bored pile wall (shear resistance)': CONTRIBUTES — head (0; 0), bond length beyond the surface 4.34 m / 14 m · fore kazık, TS500 Vc (yalnız beton; etriye katkısı YOK) · Mp=604 kNm (information — in reinforced concrete the flexural limit is NOT APPLICABLE, shear is GOVERNING)

• [DESTEK] Support 'Anchor 1': CONTRIBUTES — head (-0.4; -2.5), bond length beyond the surface 12 m / 12 m · anchor, ACTIVE — allowable 488.3 kN (ultimate 781.2 / 1.60, tendon rupture); not divided

• [DESTEK] Support 'Anchor 2': CONTRIBUTES — head (-0.4; -6), bond length beyond the surface 12 m / 12 m · anchor, ACTIVE — allowable 488.3 kN (ultimate 781.2 / 1.60, tendon rupture); not divided

STRUCTURAL DESIGN BASIS AND RESULTS

The actions are taken from the envelope values of all construction stages. Reinforced concrete sections have been designed to TS500 with N–M interaction, and steel sections with elastic bending and plastic shear resistance.

1. Design effects and load factor

The design values have been obtained by increasing the actions with an action factor of 1.15.

$$M_d = 1.15 \cdot Mk \qquad V_d = 1.15 \cdot Vk$$

Table 62: Face · Mk (kNm/m) · Vk (kN/m)

Face	Mk (kNm/m)	Vk (kN/m)	M_d (kNm/m)	V_d (kN/m)
Street Face	210.6	94.3	242.2	108.4

> **EACH FACE IS DESIGNED SEPARATELY —** Reinforcement is face-specific. A different excavation depth and support arrangement produce different internal forces; solving all faces with the same reinforcement means unnecessary cost on shallow faces and an inadequate section on deep faces. Each face is given separately below.

2. Street Face — section design

Wall type: Bored pile wall. Reinforcement has been set manually BY THE USER.

2.1 Bored pile — circular section

The analysis results are for a 1 metre wall strip. As the reinforced concrete check is made on A SINGLE PILE, the effects are converted to per-pile values by multiplying by the plan spacing and the design factor. This conversion is applied to ALL THREE EFFECTS (moment, shear, axial); if one is omitted, capacity and demand are compared in different reference frames.

$$\text{per pile} = (1\ \text{m strip value}) \times s \times \gamma = (\ \dots\ ) \times 1 \times 1.15$$

Table 63: Quantity · Value · Unit

Quantity	Value	Unit
Pile spacing s	1	m
Action design factor γ	1.15	—
Envelope moment (1 m strip)	210.6	kNm/m
Envelope shear (1 m strip)	94.3	kN/m

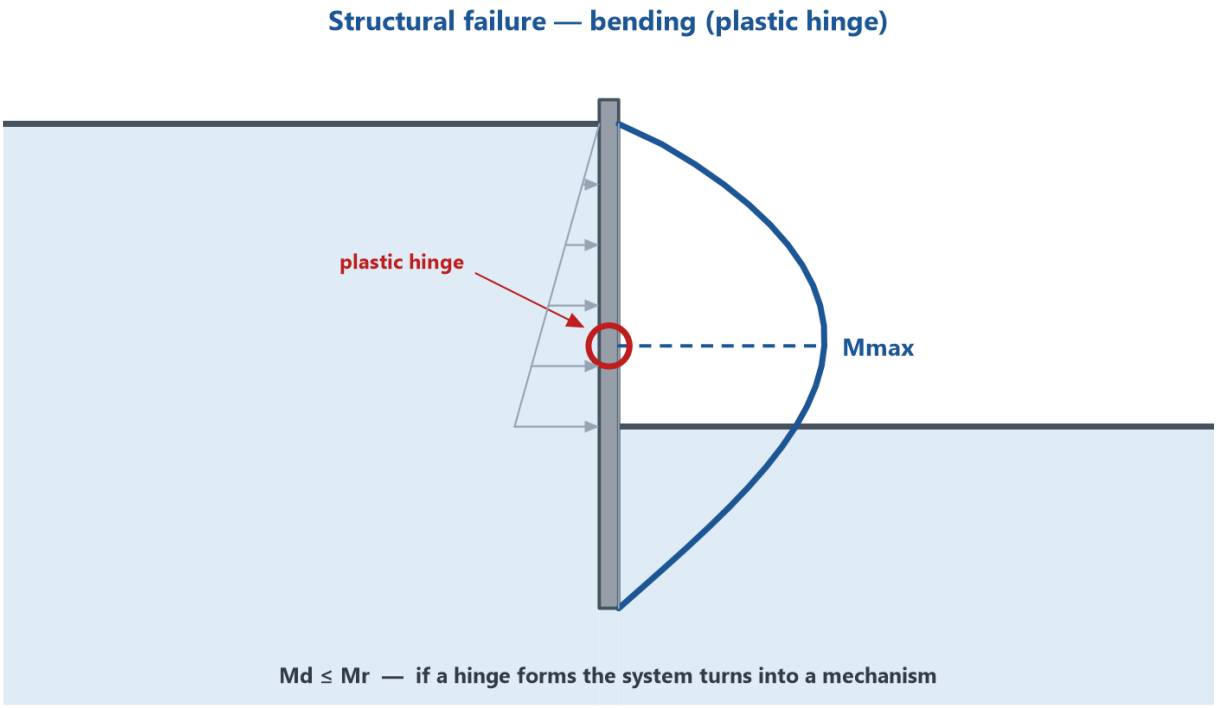


Figure 40: Bending failure

What does this check prevent? — When the bending moment in a wall section exceeds the moment capacity, a plastic hinge forms at that section. A single hinge alone does not mean collapse; if the system is sufficiently redundant the load redistributes to other sections. In a cantilever wall, however, the first hinge is directly a mechanism. The bending check must therefore be read together with the number of supports in the system: the same M_d/M_r ratio is collapse in a cantilever and redistribution in a multi-row anchored system. [TS 500 / TBDY 2018 — flexural strength]

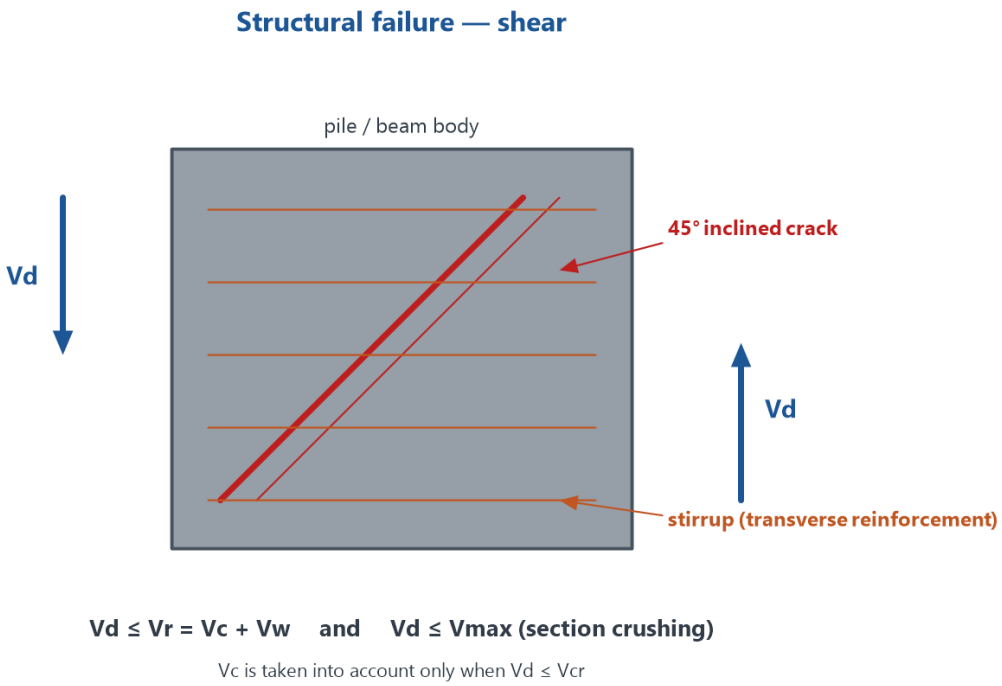


Figure 41: Shear failure

What does this check prevent? — Unlike bending, shear failure is not ductile: once the inclined crack has opened, the section separates without warning. The shear resistance is therefore provided with a higher margin of safety than the flexural resistance, and the aim in design is that failure always initiates in bending. There are two separate limits: the capacity limit, which can be raised by transverse reinforcement, and the crushing limit, which is set by the section dimensions and cannot be exceeded by adding reinforcement. In a section governed by the latter the solution is not to reduce the stirrup spacing but to enlarge the section. [TS 500 Section 8 — shear strength]

2.1.1 Section and longitudinal reinforcement

REINFORCEMENT SET MANUALLY — The reinforcement of this face has been changed manually by the user; the automatic design does not override this face. The checks below have been carried out on the manually entered reinforcement.

Table 64: Quantity · Value · Unit

Quantity	Value	Unit
Pile diameter D	0.8	m
Pile spacing s	1	m
Gross section area Ac = πD²/4	0.5027	m²
Concrete cover	3.5	cm
Longitudinal reinforcement	17 × Ø20	—
Area of one bar Ab	3.142	cm²
Total reinforcement area As	53.41	cm²
Reinforcement ratio ρ = A_s/Ac	1.063	%
TS500 minimum ratio	1.000	%
Ratio check	SATISFIED	—

$$A_s = 17 \times 3.142 = 53.41 \text{ cm}^2$$

$$\rho = 53.41 / 5026.55 = 1.063\%$$

The pile is checked separately for the CONFINEMENT zone, where shear and moment concentrate, and for the GENERAL zone. The confinement zone extends ±1 m about each support and, in a cantilever system, about the final excavation level; the stirrup spacing is reduced there. Each zone is checked with ITS OWN envelope actions — checking one with the envelope of the other is asking the wrong question.

2.1.2 Bending and shear — CONFINEMENT zone

$$M_{pile} = M_d \cdot s \cdot \gamma = 168.2 \times 1 \times 1.15 = 193.4 \text{ kNm}$$

$$V_{pile} = 108.4 \text{ kN} \qquad N_{pile} = 302.9 \text{ kN}$$

Table 65: Quantity · Value · Unit

Quantity	Value	Unit
Design axial force Nd	302.9	kN
Design moment Md	193.4	kNm
Neutral axis x	18.85	cm
Solution iteration	30	pcs
Axial equilibrium residual	-1.097	kN
Moment capacity Mr	676.5	kNm
Utilisation ratio M_d/M_r	0.286	—
Result	SATISFACTORY	—

$$M_d / M_r = 193.4 / 676.5 = 0.286$$

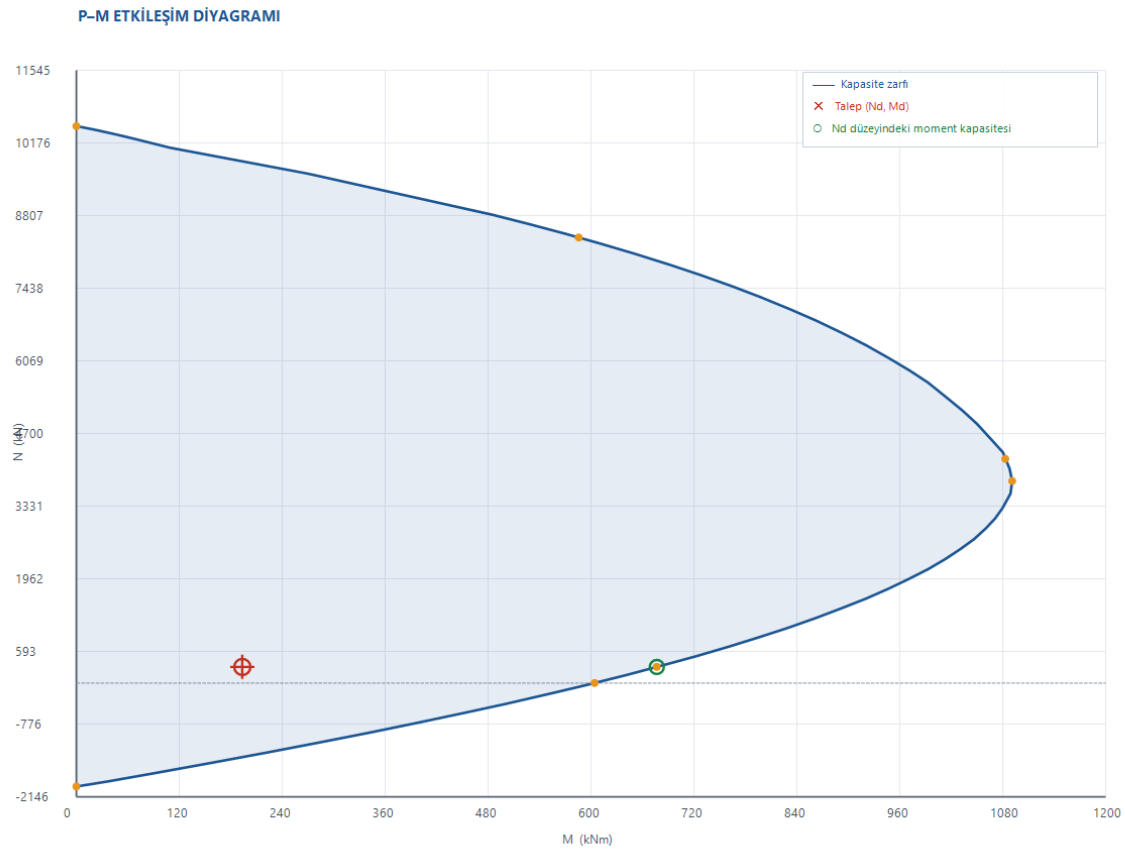


Figure 42: Street Face — P–M interaction diagram of the pile section — sarılma bölgesi (Ø80 / 17Ø20)

The shear resistance is the sum of the concrete and stirrup contributions to TS500 Section 8.1.4 (pile — confinement zone).

$$V_c = 0.8 \cdot 0.65 \cdot f_{ctd} \cdot b_w \cdot d \qquad V_w = (A_{sw}/s) \cdot f_{yd} \cdot d$$

$$V_r = V_c + V_w \le V_{max} = 0.22 \cdot f_{cd} \cdot b_w \cdot d$$

Table 66: Quantity · Value · Unit

Quantity	Value	Unit
Characteristic concrete strength f_{ck}	30000	kN/m²
Design concrete strength f_{cd}	20000	kN/m²
Concrete tensile strength f_{ctd}	1278	kN/m²
Section width bw	0.8	m
Effective depth d	0.68	m
Stirrup	Ø10/7.5 cm, 2 legs	—
Stirrup area Asw	1.571	cm²
Concrete contribution Vc	361.5	kN
Stirrup contribution Vw	520.1	kN
Total resistance Vr	881.7	kN
Upper limit Vmax	2393.6	kN
Design shear Vd	108.4	kN
Utilisation ratio V_d/V_r	0.123	—
Result	SATISFACTORY	—

$$Vc = 0.8 \times 0.65 \times 1278 \times 0.8 \times 0.68 = 361.5 \text{ kN}$$

$$Vw = (1.571 / 7.5) \times 365217 \times 0.68 = 520.1 \text{ kN}$$

$$V_r = 361.5 + 520.1 = 881.7 \text{ kN} \quad \geq \quad V_d = 108.4 \text{ kN}$$

2.1.3 Bending and shear — GENERAL zone

$$M_{pile} = M_d \cdot s \cdot \gamma = 210.6 \times 1 \times 1.15 = 242.2 \text{ kNm}$$

$$V_{pile} = 105.1 \text{ kN} \qquad N_{pile} = 352.2 \text{ kN}$$

Table 67: Quantity · Value · Unit

Quantity	Value	Unit
Design axial force Nd	352.2	kN
Design moment Md	242.2	kNm
Neutral axis x	19.21	cm
Solution iteration	46	pcs
Axial equilibrium residual	-0.153	kN
Moment capacity Mr	687.7	kNm
Utilisation ratio M_d/M_r	0.352	—
Result	SATISFACTORY	—

$$M_d / M_r = 242.2 / 687.7 = 0.352$$

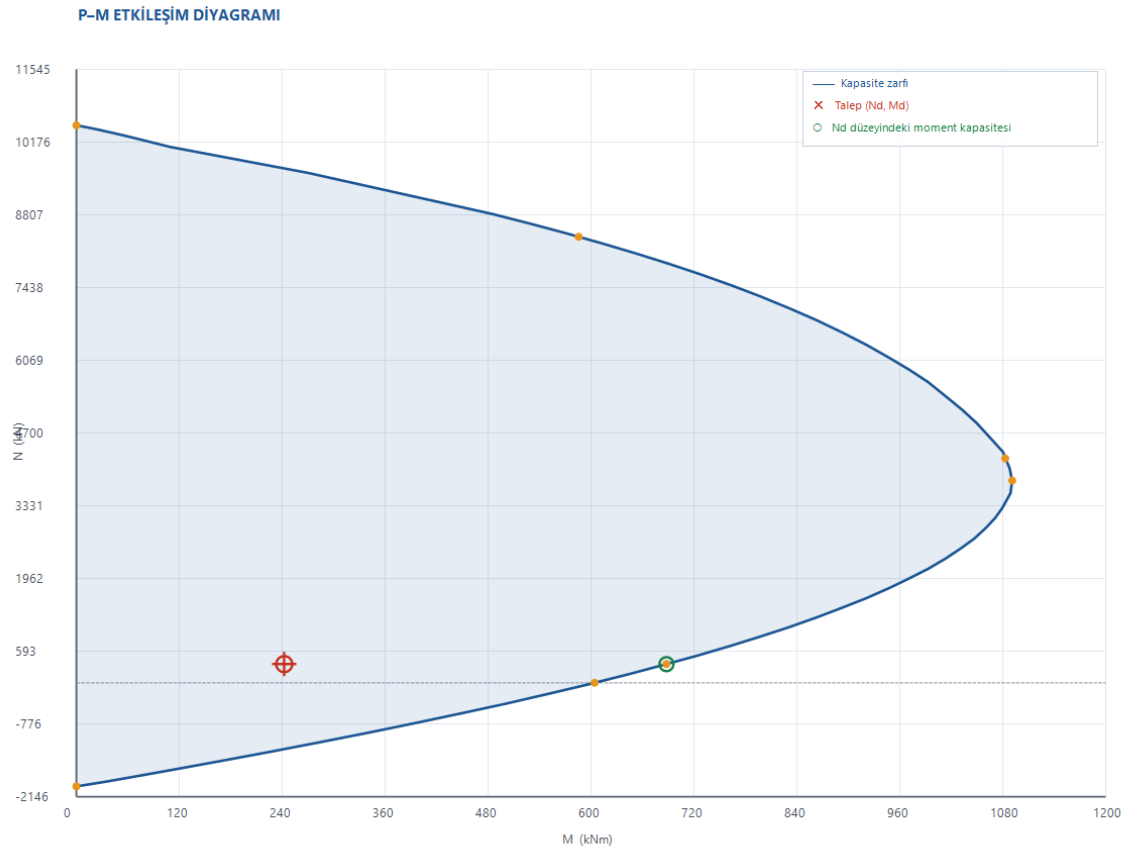


Figure 43: Street Face — P–M interaction diagram of the pile section — genel bölge (Ø80 / 17Ø20)

The shear resistance is the sum of the concrete and stirrup contributions to TS500 Section 8.1.4 (pile).

$$Vc = 0.8 \cdot 0.65 \cdot f_{ctd} \cdot bw \cdot d \qquad Vw = (Asw/s) \cdot f_{yd} \cdot d$$

$$V_r = Vc + Vw \le Vmax = 0.22 \cdot f_{cd} \cdot bw \cdot d$$

Table 68: Quantity · Value · Unit

Quantity	Value	Unit
Characteristic concrete strength f_{ck}	30000	kN/m²
Design concrete strength f_{cd}	20000	kN/m²
Concrete tensile strength f_{ctd}	1278	kN/m²
Section width bw	0.8	m
Effective depth d	0.68	m
Stirrup	Ø10/15 cm, 2 legs	—
Stirrup area Asw	1.571	cm²
Concrete contribution Vc	361.5	kN
Stirrup contribution Vw	260.1	kN
Total resistance Vr	621.6	kN
Upper limit Vmax	2393.6	kN
Design shear Vd	105.1	kN
Utilisation ratio V_d/V_r	0.169	—
Result	SATISFACTORY	—

$$Vc = 0.8 \times 0.65 \times 1278 \times 0.8 \times 0.68 = 361.5 \text{ kN}$$

$$Vw = (1.571 / 15) \times 365217 \times 0.68 = 260.1 \text{ kN}$$

$$V_r = 361.5 + 260.1 = 621.6 \text{ kN} \quad \ge \quad V_d = 105.1 \text{ kN}$$

2.2 Capping beam

The capping beam ties the pile heads together, distributes the horizontal load among the piles and prevents the pile heads from moving differentially. It is assumed to act as a continuous beam over the pile spacing.

$$M \approx w \cdot s^2 / 10 \qquad As \approx M / (0.9 \cdot d \cdot f_{yd})$$

Table 69: Quantity · Value · Unit

Quantity	Value	Unit
Section width b	0.8	m
Section depth h	0.8	m
Concrete cover	3	cm
Effective depth d	0.75	m
Top reinforcement	11 × Ø14	—
Bottom reinforcement	11 × Ø14	—
Web reinforcement (each face)	4 × Ø16	—
Tension reinforcement area As	16.93	cm²
Moment capacity $Mr \approx 0.9 \cdot d \cdot f_{yd} \cdot A_s$	417.4	kNm
Stirrup	Ø8/20 cm	—
Design moment $M_d = M_{d,wall} \cdot \gamma \cdot s^2/10$	24.2	kNm
Minimum tension reinforcement $A_{s,min} = 0.8 \cdot f_{ctd}/f_{yd} \cdot b \cdot d$ (TS 500 Cl. 7.3)	16.8	cm²
TS 500 beam minimum $A_{s,min} = 0.8 \cdot f_{ctd}/f_{yd} \cdot b \cdot d$	16.8	cm²
Required tension reinforcement A_s,req = max(A_s,calc;	16.8	cm²

Quantity	Value	Unit
$A_{s,min}$; $A_{s,min}$,TS 500)		
Minimum stirrups Asw/s ≥ 0.3·f_{ctd}/fywd·bw (TS 500)	8.4	cm²/m
Provided stirrups Asw/s (4 legs)	10.05	cm²/m
Stirrup spacing limit s ≤ min(d/2; 20 cm)	20	cm
Moment check M_r ≥ Md	SATISFIED	—
Minimum reinforcement check A_s(top) ≥ $A_{s,min}$	SATISFIED	—
Minimum stirrup and spacing check (TS 500)	SATISFIED	—

$$A_s = 11 \times 1.539 = 16.93 \text{ cm}^2$$

$$M_r = 0.9 \times 0.75 \times 365217 \times 0.001693 = 417.4 \text{ kNm}$$

$$M_d = 210.6 \times 1.15 \times 1^2 / 10 = 24.2 \text{ kNm} \quad \rightarrow \quad M_r = 417.4 \text{ kNm} \geq Md$$

2.3 Waler beam

The waler beam distributes the anchor forces to the wall. It has been assumed to act as a continuous beam between the anchors and the moment has been taken approximately as P·S/10.

$$M = P \cdot S / 10 = 329.3 \times 3 / 10 = 98.8 \text{ kNm}$$

Anchor 2: Design load = solved axial force 329.3 kN (> prestress 300 kN; governing stage: Final excavation)

Table 70: Quantity · Value · Unit

Quantity	Value	Unit
Anchor force P	329.3	kN
Plan spacing S	3	m
Section width b	0.4	m
Section depth h	0.7	m
Effective depth d	0.65	m
Top reinforcement	3 × Ø18	—
Bottom reinforcement	3 × Ø18	—
Side reinforcement	4 × Ø14	—
Tension reinforcement area As	15.27	cm²
Design moment Md	98.8	kNm
Moment capacity Mr	326.2	kNm
Utilisation ratio	0.303	—
Stirrup	Ø8/5 cm	—
Result	SATISFACTORY	—

$$M_r = 0.9 \times 0.65 \times 365217 \times 0.001527 = 326.2 \text{ kNm}$$

2.4 Anchor head zone

The anchor force is transferred to the waler beam through the bearing plate over a very small area. Even if the bending and shear checks of the beam pass, this local zone may fail. The three modes are checked separately.

2.4.1 Local bearing in concrete

The load spreads from the plate area (A_1) within the beam over a larger area (A_2); this spread increases the resistance. The increase factor is computed as $\sqrt{(A_2/A_1)}$ and is limited to 2.0.

$$\sigma = P / A_1 \qquad f_{bd} = f_{cd} \cdot \sqrt{(A_2/A_1)} \leq 2 \cdot f_{cd}$$

Table 71: Anchor · P (kN) · Plate area A_1 (m^2)

Anchor	P (kN)	Plate area A_1 (m^2)	σ (kPa)	Spread $\sqrt{(A_2/A_1)}$	f_{bd} (kPa)	Result
1	307.8	0.09	3420.5	1.76	35276.7	SATISFACTORY
2	329.3	0.09	3659.2	1.76	35276.7	SATISFACTORY

2.4.2 Punching

The plate may punch through the beam. The critical perimeter is taken at a distance d/2 from the plate edge.

$$u_1 = 2(a + b) + \pi \cdot d \qquad \tau = P / (u_1 \cdot d) \leq f_{ctd}$$

Table 72: Anchor · d (m) · u_1 (m)

Anchor	d (m)	u_1 (m)	τ (kPa)	f_{ctd} (kPa)	Result
1	0.64	3.211	149.8	1278	SATISFACTORY
2	0.64	3.211	160.3	1278	SATISFACTORY

2.4.3 Steel plate bending

The plate acts as a cantilever outside the bearing (trumpet) circle. A thin plate bends and cannot distribute the pressure uniformly to the beam.

$$c = (\min(a,b) - D_{bearing}) / 2 \qquad M = \sigma \cdot c^2 / 2 \qquad t_{req} = \sqrt{(6M / f_y)}$$

Table 73: Anchor · Cantilever c (mm) · M (kNm/m)

Anchor	Cantilever c (mm)	M (kNm/m)	Required t (mm)	Provided t (mm)	Result
1	90	13.85	15.3	30	SATISFACTORY
2	90	14.82	15.8	30	SATISFACTORY

ASSESSMENT — The local crushing resistance has been increased by the spread of the load within the beam (√(A2/A1), at most 2.0). This increase depends on the presence of ANTI-CRACKING reinforcement in the spread zone; additional stirrups/mesh shall be placed around the plate in the waler beam.

ASSESSMENT — Plate bending was calculated assuming a cantilever outside the bearing (trumpet) circle. If the plate is thinner than required, the pressure concentrates at the centre of the plate and the actual bearing stress becomes LARGER than calculated; the plate thickness is therefore a precondition for the validity of the concrete checks.

QUANTITY TAKE-OFF SUMMARY

The quantities below are derived from the face lengths and the section definitions. The detailed item breakdown and the proposed tender pay items are given in the Quantity take-off section.

1. Face lengths and pile counts

Table 74: Face · Type · Length (m)

Face	Type	Length (m)	Spacing (m)	Count	Length (m)	Total
Street Face	Bored pile wall	39	1	40	14	560

The total excavation perimeter is 39 m. Pile/soldier count 40, total length 560 m.

2. Concrete quantity

Table 75: Face · Number of piles · Diameter (m)

Face	Number of piles	Diameter (m)	Pile concrete (m³)	Capping beam (m³)	Total (m³)
Street Face	40	0.8	281.49	24.96	306.45

The total net concrete quantity is 306.45 m³. The order quantity shall be determined by adding the construction waste ratio.

3. Anchor quantity

Table 76: Face · Row · Type

Face	Row	Type	Depth (m)	Spacing (m)	Count	Length (m)	Drilling (m)
Street Face	1	Prestressed	2.5	3	13	22.5	286
Street Face	2	Prestressed	6	3	13	22.5	286

A total of 26 anchors and 572 m of drilling are foreseen.

The count is the number of anchors drawn in the face elevation. Pile axes are read from the plan (a pile at both ends, spacing fitted to the length). Depending on the selected position, the first anchor is placed midway between the first two piles or on the axis of the first pile where the head plate fits on the face; the following ones at the horizontal anchor spacing; the head plate fits at the end of the face. Length (tendon) = free length + bond length + 0.50 m head allowance; drilling = free length + bond length (the head allowance is not drilled).

> **THE QUANTITY TAKE-OFF IS A PRELIMINARY VALUE —** The quantities above have been generated from the project geometry. Construction waste, splice lengths, reinforcement laps and differences arising from site conditions shall be assessed separately at the tender stage.

MONITORING AND INSTRUMENTATION PLAN DURING EXCAVATION

Whether an excavation support structure behaves as designed can only be established by measurement. The calculation is a prediction based on soil parameters; the actual behaviour on site may deviate from it. Instrumentation is the only way of seeing this deviation before damage occurs.

1. Quantities to be measured

- Deformations in the support structure and in the surrounding soil/structures/utilities
- Stresses in the structural members (anchor, nail, strut)
- Groundwater pressure and water ingress

2. Measurement systems

Table 77: System · Measured · Utilisation

System	Measured	Utilisation
Inclinometer	Horizontal displacement along depth	Gives the deformation profile of the wall and the retained soil; it is the most common system
Optical survey (triangulation)	x, y, z displacement	At the wall head and on the surrounding structures; verifies the inclinometer, scans many points
Surface levelling	Vertical settlement	Tracks the settlement trough over adjacent structures, roads and utilities
Anchor load cell	Anchor force	Monitors prestress loss and force increase
Strut strain gauge	Axial force	Monitors the temperature and load variation
Piezometer	Pore water pressure	Monitors the drawdown and the seepage effect
Crack gauge	Crack opening	Tracks the development of existing cracks in the adjacent structures

3. Layout rules

- At least one inclinometer is installed on each face.
- At least three inclinometers are installed on faces longer than 60 m.
- Inclinometers are placed at least 20 m away from the corners where possible; at points close to a corner the measured displacement does not represent the actual behaviour of the face because of the three-dimensional corner effect.
- In systems having an embedment zone the inclinometer is installed DEEPER than the vertical structural member; only in this way can overall stability be monitored.
- On critical faces (adjacent structure close, excavation deep, soil weak) the number of measurement points is increased.

3.1 Minimum number of inclinometers for this project

Table 78: Face · Length (m) · Excavation (m)

Face	Length (m)	Excavation (m)	Minimum inclinometers	Rationale
Street Face	39	10	1	At least one per face

At least 1 inclinometers in total shall be provided in the project. This number is a MINIMUM; it shall be increased according to the position of the adjacent structures and the soil conditions.

3.2 Inclinometer depth and LOCATION

Table 79: Face · Wall length (m) · Borehole depth (m)

Face	Wall length (m)	Borehole depth (m)	Borehole positions (from the start of the face, m)
Street Face	14	17.5	INK-1: 5.0 · INK-2: 27.0

The borehole positions are also shown on the FACE ELEVATION sheet (INSTRUMENTATION layer); this table in the report and the marks on the sheet are produced from the same source.

The inclinometer must extend deeper than the wall toe so that a slip surface passing beneath the wall can be captured. An inclinometer terminating at the wall toe cannot see an overall stability failure in which the whole mass moves together.

3.3 Monitoring zone (AGB) and surface measurement points

The inclinometer measures the HORIZONTAL movement of the wall over depth. The criterion for damage to the adjacent structure, however, is the angular distortion, and that follows from the surface SETTLEMENT behind the excavation — the inclinometer cannot see it. For this reason surface measurement points (prisms/targets) are additionally installed behind the wall.

No separate empirical rule HAS BEEN ESTABLISHED for the width of the monitoring zone: the program already computes the settlement envelope behind the excavation, and the AGB is the width of influence of that envelope. Writing a second definition would mean defining the same quantity in two different ways.

Table 80: Face · Excavation (m) · AGB width (m)

Face	Excavation (m)	AGB width (m)	Source of the width	Surface point
Street Face	10	20	influence width of the settlement envelope (Kum)	12

At each monitoring section there are 4 rows of points running back from the wall: wall face · AGB×0.25 · AGB×0.60 · AGB×1.00. The sections are repeated along the face at approximately 20 m intervals.

> **WHY A SINGLE ROW IS NOT ENOUGH —** A single point adjacent to the wall gives the MAGNITUDE of the settlement but NOT ITS PROFILE. The angular distortion $\beta_{max} = \Delta\delta v/\Delta x$, which is the damage criterion, is the difference in slope between two points; it cannot be computed from a single point. For this reason several arrays along the AGB have been provided at each section. The end of the trough (the AGB limit) must also be measured, otherwise the profile does not close.

The AGB limit, the surface measurement points and the inclinometer boreholes are shown on the SECTION and FACE ELEVATION sheets in the INSTRUMENTATION layer; these tables in the report and the marks on the sheet are produced from the same source.

4. Reading frequency

Table 81: Period · Frequency · Rationale

Period	Frequency	Rationale
Before excavation (zero reading)	At least twice	Reference value; if a single reading is erroneous the whole series shifts
During active excavation	At least once a week	Deformation develops fastest in this period
At the critical stage (before support)	Daily	System in the most unfavourable condition; free excavation is the largest
After anchor stressing	Stressing day and the following day	Prestress loss and system response
After completion of the excavation	Monthly	Creep and consolidation effects
When the alarm level is exceeded	Daily or continuous	The action plan is activated

5. Alarm levels and action plan

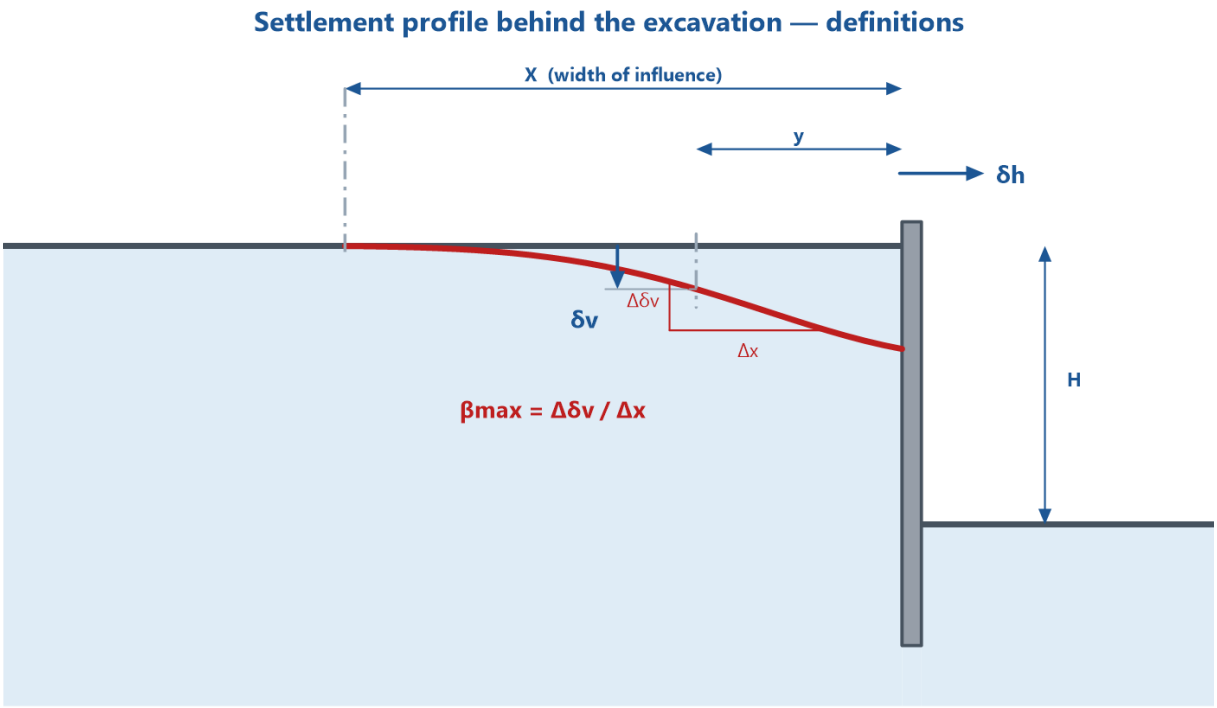


Figure 44: Settlement profile definitions

What does this check prevent? — Damage behind the excavation depends less on the magnitude of the settlement than on the shape of its profile. A structure carries a uniform 20 mm settlement without distress, yet cracks if the same 20 mm occurs over a short distance. The criterion is therefore not δ_v but the angular distortion β_{maks}, the difference in slope between two points. The influence width X is a function of the excavation depth and the soil class; where the adjacent structure stands within this width determines the δ_v and β_{maks} it experiences. The horizontal component δ_h is often neglected; yet in the structures closest to the excavation it is what initiates the tension crack. [KDYY — definitions of deformation behind the excavation]

The alarm levels are defined in terms of horizontal displacement. The red alarm limit is the serviceability limit set in the design; the yellow alarm is taken as 70–80% of that value.

Table 82: Street Face — alarm level · Displacement (mm) · Action

Street Face — alarm level	Displacement (mm)	Action
GREEN (normal)	< 15	Continue readings at the planned frequency
YELLOW (caution)	15 – 22.5	Reading frequency is increased, the excavation rate is reviewed, the design engineer is informed
ORANGE (warning)	22.5 – 30	Excavation is stopped, additional support is assessed, daily monitoring
RED (limit)	> 30	Excavation is stopped immediately, counterweight/backfill and additional support are applied, adjacent structures are assessed for evacuation

The limit value has been determined as 30 mm for the Anchored system on the basis of 2.12.7: $\delta_{h,lim}$ = ‰3.0×H..

5.2 Anchor head load cells (KDYY Handbook Table 8-6)

Table 83: Alarm levels for the anchor head load cells

Face	Load cell	Lock-off load Pg (kN)	Outside YELLOW (±25%)	Outside RED (±40%)
Street Face	YKH-A1	300	225 – 375	180 – 420
Street Face	YKH-A2	300	225 – 375	180 – 420

The alarm is TWO-WAY: both an increase and a loss of load is an alarm. An increase indicates that the soil behind or the water pressure is loading the system more than expected; a loss indicates creep in the bond zone or loss of lock-off. YELLOW: The Geotechnical Officer is informed, readings are taken more frequently and measurements on the adjacent structures are started. RED: If the load increases, an additional dewatering well is opened; if displacements are also increasing, additional anchors are installed.

> **ALARM LEVELS SHALL BE CONSISTENT WITH THE DESIGN —** An alarm limit loses its meaning if it is set independently of the displacement predicted by the analysis. If the computed maximum displacement already falls in the yellow alarm range, the system has no reserve from the outset and the design shall be reviewed.

6. Points to be observed during excavation

6.1 Construction sequence

- The excavation shall not be taken below the specified stage depth before the support element is installed. In the design the free excavation below support 0.5 m has been adopted; exceeding this value on site creates a loading condition never encountered in the calculation.
- The stage below shall not be started before the stressing of the anchors is complete.
- The excavation shall be advanced in sections along the face, simultaneously with support installation, and not in a single pass.

• Heavy plant and material stockpiles shall not be kept behind the wall in a way that exceeds the surcharge value defined in the design.

6.2 Water management

• Surface water accumulating within the excavation shall be drained without delay; standing water at the excavation base reduces the passive resistance.

• Water accumulating behind the wall creates hydrostatic pressure not allowed for in the analysis. Drainage holes and the diversion of surface water are part of the works.

• If dewatering is carried out, the risk of consolidation settlement in the adjacent structures shall be assessed separately; this settlement is independent of the excavation deformation.

6.3 Adjacent structures and utilities

• Before excavation, existing cracks and damage in adjacent structures shall be surveyed, photographed and formally recorded. Without this survey, a crack observed during excavation cannot be distinguished as pre-existing or excavation-induced.

• Levelling points on the adjacent structures shall be established before excavation starts and at least two zero readings shall be taken.

• The position of the utility lines shall be established before excavation by trial pit or detector; the positions on the sketch often do not reflect reality.

• Leakage in natural gas and water lines creates additional risk because it locally reduces the soil strength; monitoring must be carried out throughout the excavation.

6.4 Unforeseen conditions

• If a soil layer, fill, cavity or water ingress not foreseen in the design is encountered at the excavation face, construction shall be stopped and the design engineer notified. Exposure of the excavation face is the only opportunity to verify the soil profile.

• Cracking on the wall face, deformation at the anchor head or bending of the waler beam requires immediate intervention.

• If a sudden jump is observed in the measurement system, the reading shall first be verified (it may be a measurement error); if confirmed, the action plan shall be implemented.

ANCHOR TEST PROGRAMME

The test (proof) load has been taken by the KDYY definition as Pp = 1.25 · Pd; Pd is the anchor design load (the value in the anchor check). The test-load upper limit selected in Settings is 20% above the lock-off load (Pg · 1.20). The number of anchors to be tested is the LARGER of 3% of the total anchors on the face and at least 3 per face. The ratio and counts are the designer's decision and can be changed in Settings.

Table 84: Anchor test program

Face	Row	Level	Number of anchors	Lock-off load Pg (kN)	Design load Pd (kN)	Test load Pp (kN)	Test load upper limit (kN)
Street Face	1	-1.50	13	300	307.8	384.8	360
Street Face	2	-5.00	13	300	329.3	411.7	360
Street Face	—	—	26	face total	—	3 tests	—

A total of 26 anchors are provided in the project; acceptance tests shall be carried out on at least 3 of them. The anchors to be tested shall be selected so that every face and every row is represented, and from the areas where the soil conditions are the most unfavourable.

ACCEPTANCE TEST — KDYY 3.2.4.3

Proof load Pp = 1.25 · Pd (anchor design load). Preload (seating) load Pa = 0.10 · Pp. Soil of the bond zone: fine-grained soil.

Table 85: Load steps and hold times (fine-grained soil)

Cycle	Load step	Minimum hold time [min]
0	Pa	1
1	0.40 Pp	1
2	0.55 Pp	1
3	0.70 Pp	1
4	0.85 Pp	1
5	1.00 Pp	15

At intermediate load steps (during loading and unloading) the load is held constant for 1 minutes. At the end of the test, unloading down to the load Pa is carried out using the same increments.

In the final increment, under the proof load, the strand head elongation is read at the following times [min]: 1 → 2 → 3 → 4 → 5 → 7 → 10 → 15.

Table 86: Observation times and limit values at load Pp

Criterion	Minimum monitoring	Extended monitoring
ta [min]	5	5
tb [min]	15	≥ 30
Displacement Δs = sb − sa [mm]	≤ 0.25	—
Creep rate α_{krip} [mm]	—	≤ 1.0

$$\alpha_{krip} = (sb - sa) / \log_{10}(tb / ta)$$

A test not satisfying the Δs condition IS NOT STOPPED; the observation period is extended and it is checked that the creep rate satisfies the condition $\alpha_{krip} \leq 1.0$ mm.

APPARENT STRAND FREE LENGTH (Lapp)

$$Lapp = At \cdot Et \cdot \Delta sel / (Pp - Pa)$$

$$0.80 \cdot Ltf + Lte \leq Lapp \leq Ltf + Lte + 0.50 \cdot Ltb$$

Here At is the total cross-sectional area of the strands, Et the elastic modulus of the strand, Δsel the elastic shortening at the end of the test, Ltf the free length of the strand, Ltb the bond length of the strand and Lte the strand length inside the jack. This check shows that the test result is consistent with the AS-DESIGNED geometry of the anchor; it does not replace the creep criterion, it is additional to it.

ANCHOR CORROSION PROTECTION

The environmental conditions have been taken as agresif olmayan (Settings → KDYY factors). This choice is determined by the chemistry of the soil and the groundwater — sulphate, chloride, pH, stray current — and the program cannot derive it from the soil record.

Table 87: Required corrosion protection systems (KDYY Table 3.7 and 3.8)

Face	Anchor	Type	System	Required protection	Suitable systems
Street Face	1	Prestressed strand/bar	Temporary	Not mandatory	—
Street Face	2	Prestressed strand/bar	Temporary	Not mandatory	—

> **CORROSION PROTECTION —** For temporary anchors and in non-aggressive conditions, KDYY Table 3.7 DOES NOT REQUIRE corrosion protection. The aggressiveness of the environment is determined from the soil and groundwater chemistry (sulphate, chloride, pH, stray current) and is selected in Settings; the program CANNOT DERIVE it from the soil record.

In place of an anchor whose test result is not accepted, a new anchor is installed in the same row adjacent to the tested anchor and the test is repeated.

APPENDICES

APPENDIX A — Surroundings inventory photographs



Figure 45: Figure — Cafer Apartmanı



Figure 46: Figure — Ayşe Apartmanı

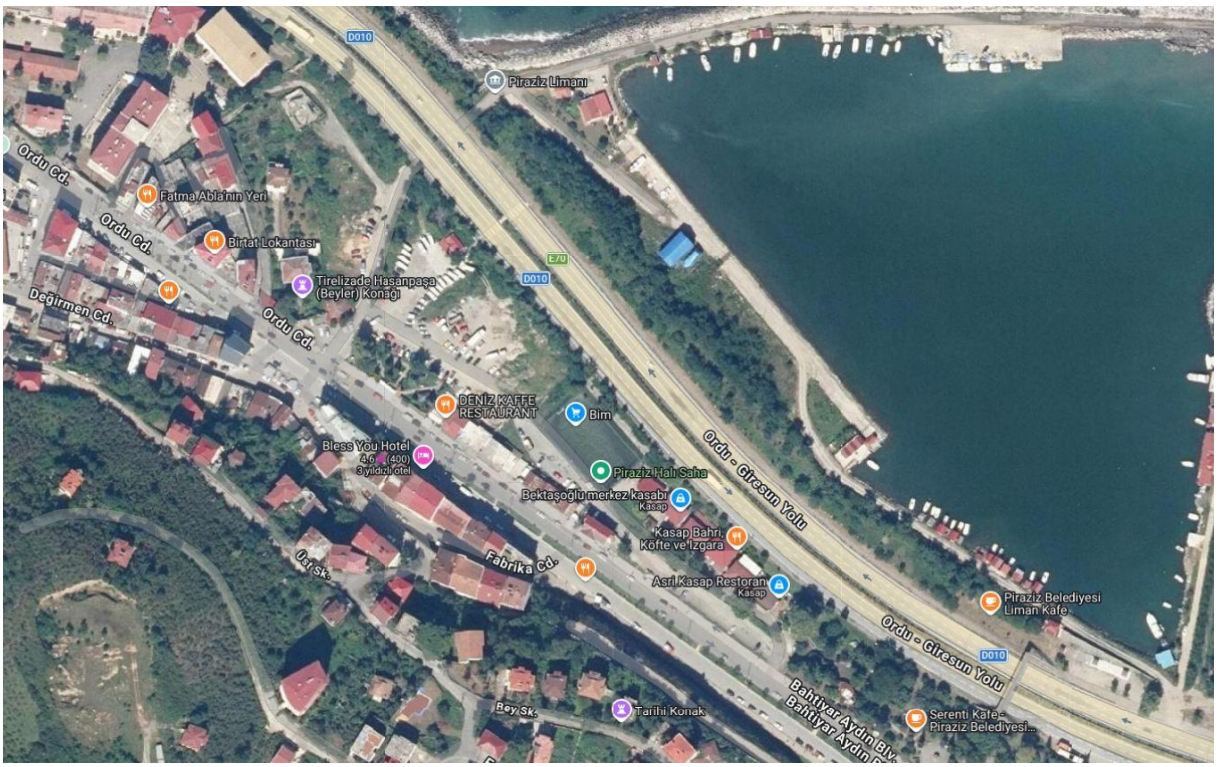


Figure 47: Figure — Mustafa Kemal Paşa Bulvarı

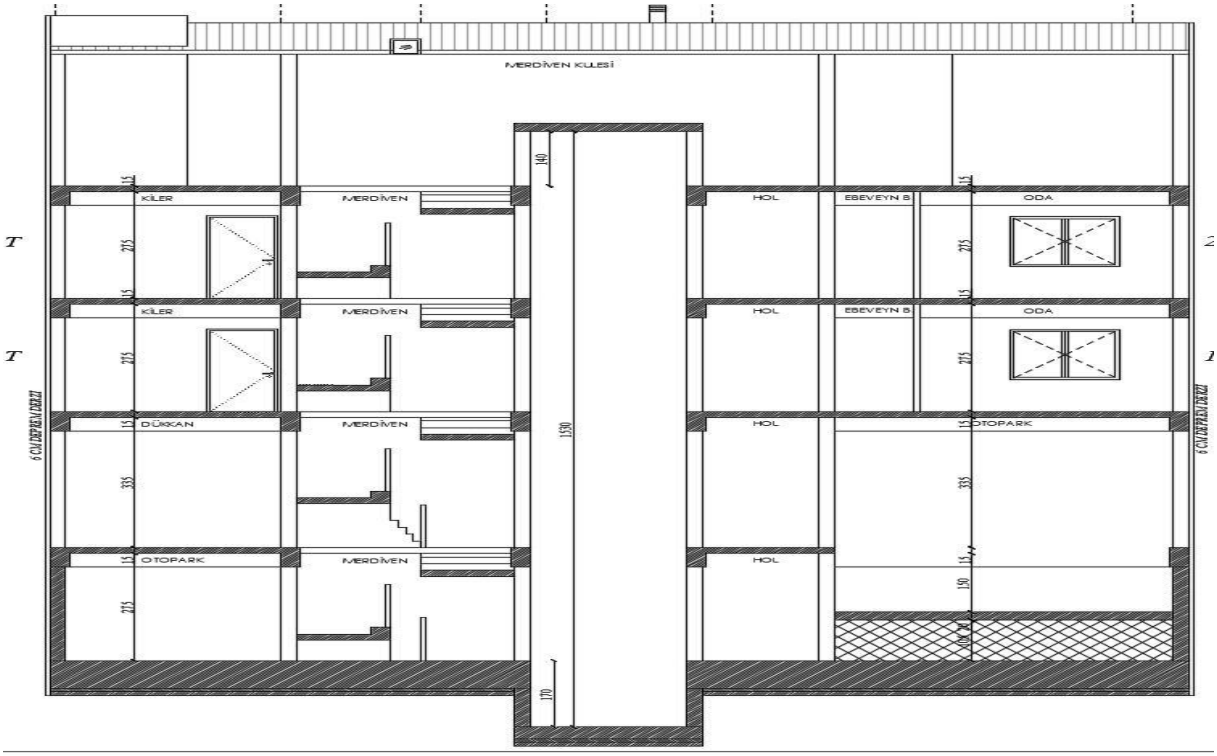


Figure 48: Figure — 2784 Sokak

APPENDIX B — Methods used and references

Table 88: Subject · Method / source

Subject	Method / source
Active and passive pressure	Rankine, Coulomb, Mazindrani, Caquot–Kérisel, TBDY 2018
Subgrade reaction modulus	Schmitt, Chadeisson, Terzaghi–Bowles, Ménard,

Subject	Method / source
	Chinese Code (m), Vesic
Seismic	Mononobe–Okabe
Reinforced concrete section	TS500 — N–M interaction, equivalent rectangular stress block
Reinforced concrete shear	TS500 Section 8.1.4
Steel section	Plastic shear strength ($f_y/\sqrt{3}$), elastic bending
Overall stability	Fellenius, Bishop, Janbu, Spencer, Morgenstern–Price, Corps of Engineers, Lowe–Karafiath, TBDY 16.30
Anchor internal stability	Kranz
Pile axial capacity and settlement	Vesic (1977)
Surface settlement (independent check)	Clough & O'Rourke (1990)
Instrumentation	KDYY Design Manual Chapter 8

APPENDIX C — Assumptions and limitations

The items below are outside the scope of the design or have been assessed with the stated assumptions. They must be known when reading the report.

- The water pressure is assumed hydrostatic; flow around the wall toe (seepage) has not been modelled.
- The beam–spring solution assumes SMALL DEFORMATION; at large displacements the change of geometry and the redistribution of the pressure distribution are not taken into account.
- The overall stability analysis searches for mass sliding mechanisms; it does not model the ROTATION of the wall itself. The two mechanisms have been checked separately.
- Group effect, negative skin friction and consolidation settlement are not taken into account in the pile settlement.
- The axial force arising from thermal expansion along the strut axis has not been taken into account.
- The soil parameters have been taken from the soil investigation report; the values in the program library are for preliminary definition only and do not replace the investigation.

APPENDIX D — Responsibility

This report has been prepared by the design engineer identified in the record above, on the basis of the stated soil investigation data and project geometry. If the soil conditions on site differ from those assumed in the report, or if the excavation geometry or the loading condition changes, the design shall be reassessed.

The excavation support structure shall be monitored during and after construction in accordance with the instrumentation plan defined in this report. Without monitoring, the validity of the design cannot be verified.

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APPENDIX 2A (IMPLEMENTATION APPENDIX): STRUCTURAL CONDITION ASSESSMENT TECHNICAL REPORT

Table 89: Appendix 2A identification

Information	Value
Report Subject	Excavation area inspection for pkdwpsokd located at wopdwk Neighbourhood block odpwk parcel dpow
Inspection Date	10.11.2026
Report Date	podkw
Prepared By	Gökhan Demirbaş (Ground Inspection)

The points identified as a result of the inspections carried out on 10.11.2026 by the technical staff of okdw regarding the surrounding and nearby structures that may be affected by the excavation to be carried out within the scope of pkdwpsokd, and regarding the slope condition, together with the opinions on these points, are presented below.

1. GENERAL INFORMATION

1.1 Purpose and Scope of the Inspection

It covers the inspections aimed at identifying the effects of the planned excavation at the construction site on the existing buildings and determining the measures to be taken, for the pkdwpsokd planned to be built within Block odpwk, Parcel dpow. The maximum excavation depth is 10 m, and the inspection covers a zone of influence 20 m wide from the excavation line.

1.2 Information on the Structure to Be Built

Table 90: Information on the structure to be built

Information	Value
Total Number of Floors	Z+2
Number of Basement Floors	None
Foundation Base Level (relative to survey grid)	0 m
Structural System Type	Reinforced concrete
Foundation System	Raft
Building Importance Factor (I)	1
Foundation Footprint Area	—
Design Year	—
Planned Construction Year	—
Nearest Distance to Excavation Support Line	5 m
Farthest Distance to Excavation Support Line	5 m

1.3 Planned Excavation Information

Table 91: Planned excavation information

Information	Line 1

Information	Line 1
Line name	Street Face
Line length (from program)	39 m
Line height (from program)	10 m
Line slope	—
Excavation support system type (from the program)	Bored pile wall — anchored
Zone of influence distance L (from program)	20 m

The total excavation support line length is 39 m, and the maximum excavation depth is 10 m. The zone of influence is taken as 20 m wide, twice the maximum excavation depth.

The excavation area is located on Block odpwk, Parcel dpow, wopdwk Neighborhood.

2. SITE OBSERVATIONS

2.1 Plan Showing the Excavation Area and Its Surroundings

(Figure — Plan showing the excavation area and its surroundings — presented in the Appendix 2A photographs section.)

The excavation area, the excavation support structure lines and the zone of influence with a width of at least twice the maximum excavation depth are shown on the plan.

2.2 On-Site Inspections and Findings

2.2.1 Existing Building Information

2.2.1.1 Cafer Apartmanı

Table 92: Cafer Apartmanı — structural condition assessment

Information	Value
Total Number of Floors	B+Z+4
Number of Basement Floors	Yes
Foundation Base Level (relative to survey grid)	-3 m
Structural Description of the Building	—
Structural System Type	Reinforced concrete
Foundation System	Strip
Building Importance Factor (I)	1
Foundation Footprint Area	485 m²
Damage Condition	—
Design Year	—
Year Built	1988
Nearest Excavation Support Line	Street Face
Nearest Distance to Excavation Support Line	6 m (WITHIN the excavation zone of influence — L = 20 m)
Farthest Distance to Excavation Support Line	5 m (WITHIN the excavation zone of influence — L = 20 m)
Reinforced Concrete Project	None
Architectural Project	None

Information	Value
Notes	—

> **DAMAGE STATUS NOT ENTERED —** The existing damage status for Cafer Apartmanı has not been specified. Bu alan EK-2A'nın geri alınamaz alanıdır: kazı sırasında ya da sonrasında ortaya çıkan bir hasar tartışmasında, kazı ÖNCESİ yapılmış tespit tek dayanaktır ve sonradan doldurulamaz. Yapı hasarsız ise bu da açıkça yazılmalıdır.

(Figure — Cafer Apartmanı — presented in the Appendix 2A photographs section.)

2.2.1.2 Ayşe Apartmanı

Table 93: Ayşe Apartmanı — structural condition assessment

Information	Value
Total Number of Floors	B+Z+4
Number of Basement Floors	Yes
Foundation Base Level (relative to survey grid)	-2.5 m
Structural Description of the Building	—
Structural System Type	Reinforced concrete
Foundation System	Raft
Building Importance Factor (I)	1
Foundation Footprint Area	444 m²
Damage Condition	—
Design Year	—
Year Built	2018
Nearest Excavation Support Line	—
Nearest Distance to Excavation Support Line	6 m
Farthest Distance to Excavation Support Line	5 m
Reinforced Concrete Project	None
Architectural Project	None
Notes	—

> **DAMAGE STATUS NOT ENTERED —** The existing damage status for Ayşe Apartmanı has not been specified. Bu alan EK-2A'nın geri alınamaz alanıdır: kazı sırasında ya da sonrasında ortaya çıkan bir hasar tartışmasında, kazı ÖNCESİ yapılmış tespit tek dayanaktır ve sonradan doldurulamaz. Yapı hasarsız ise bu da açıkça yazılmalıdır.

(Figure — Ayşe Apartmanı — presented in the Appendix 2A photographs section.)

2.2.2 Infrastructure – Superstructure Information

2.2.2.1 İZSU

Table 94: İZSU — infrastructure assessment

Information	Value
Infrastructure Type	Water

Information	Value
Responsible Authority	—
Nearest Excavation Support Line	Street Face
Infrastructure Base Level (relative to survey grid)	0 m
Estimated Depth	1.5 m
Infrastructure Slope	0 %
Culvert / Underpass	None
Nearest Distance to Excavation Support Line	3 m (WITHIN the excavation zone of influence — L = 20 m)
Farthest Distance to Excavation Support Line	2 m (WITHIN the excavation zone of influence — L = 20 m)
Notes	—

> **RESPONSIBLE AUTHORITY NOT ENTERED —** The responsible authority for line İZSU has not been specified. Kazı öncesinde deşarj planı bu kurumdan alınır ve hat kesilirse ilk aranacak muhatap odur; hattın TÜRÜNÜ bilmek kurumu bilmek değildir.

2.2.3 Road Information

2.2.3.1 Mustafa Kemal Paşa Bulvarı

Table 95: Mustafa Kemal Paşa Bulvarı — road survey

Information	Value
Road Surface Material	Asphalt
Road Width	15 m
Traffic Condition	Heavy
Road Slope	3 %
Road Levels	—
Road Lighting	Yes
Overpass – Underpass	Overpass: none · Underpass: none
Infrastructure Within the Road Body	None
Nearest Excavation Support Line	Street Face
Nearest Distance to Excavation Support Line	3 m (WITHIN the excavation zone of influence — L = 20 m)
Farthest Distance to Excavation Support Line	3 m (WITHIN the excavation zone of influence — L = 20 m)
Notes	—

(Figure — Mustafa Kemal Paşa Bulvarı — presented in the Appendix 2A photographs section.)

2.2.3.2 2784 Sokak

Table 96: 2784 Sokak — road survey

Information	Value
Road Surface Material	Interlocking paver
Road Width	6 m

Information	Value
Traffic Condition	Light
Road Slope	4 %
Road Levels	—
Road Lighting	Yes
Overpass – Underpass	Overpass: none · Underpass: none
Infrastructure Within the Road Body	None
Nearest Excavation Support Line	—
Nearest Distance to Excavation Support Line	3 m
Farthest Distance to Excavation Support Line	3 m
Notes	—

(Figure — 2784 Sokak — presented in the Appendix 2A photographs section.)

2.2.4 Adjacent Parcel Information

There are no adjacent parcels within the excavation area and zone of influence.

3. FINDINGS AND ASSESSMENTS

There is too little risk for enviroment and the buildings around.

This is our opinion and conclusion. 10.11.2026

Table 97: EK-2A signature

Prepared By	Signature
Gökhan Demirbaş Ground Inspection	

APPENDIX 2A PHOTOGRAPHS



Figure 49: Figure — Plan showing the excavation area and its surroundings



Figure 50: Figure — Cafer Apartmanı



Figure 51: Figure — Ayşe Apartmanı

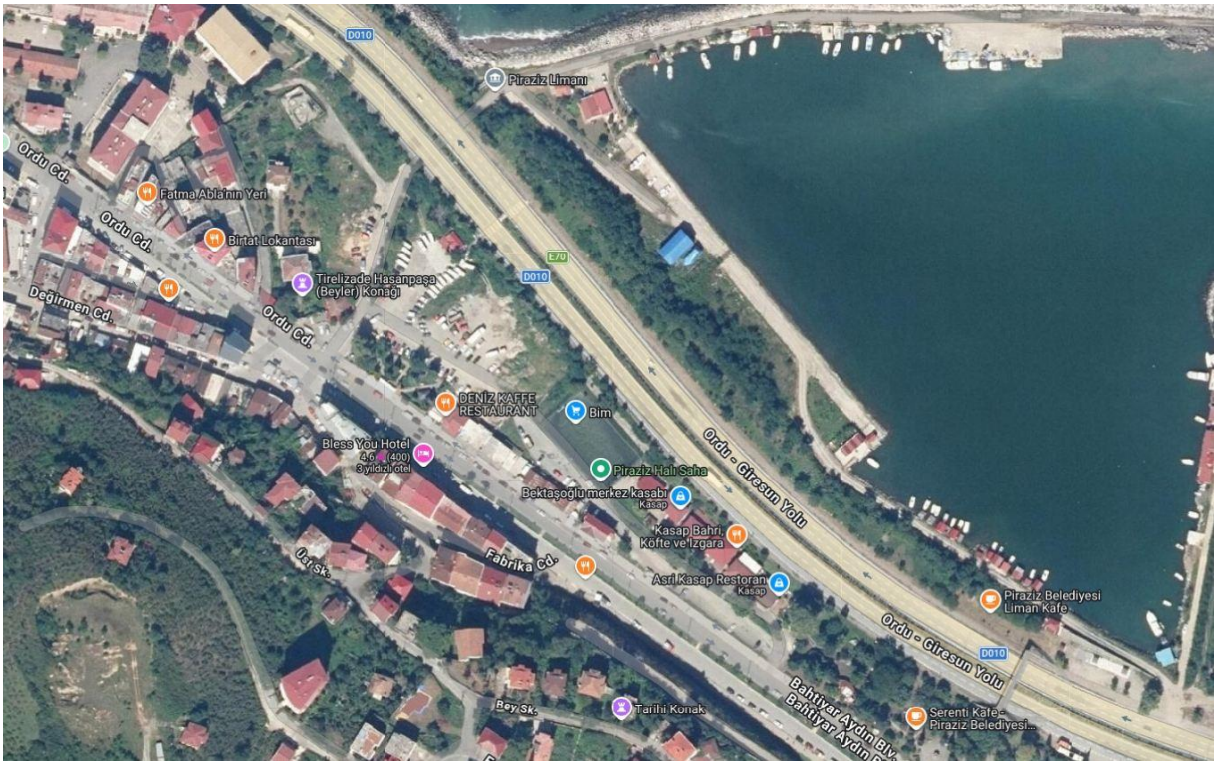


Figure 52: Figure — Mustafa Kemal Paşa Bulvarı

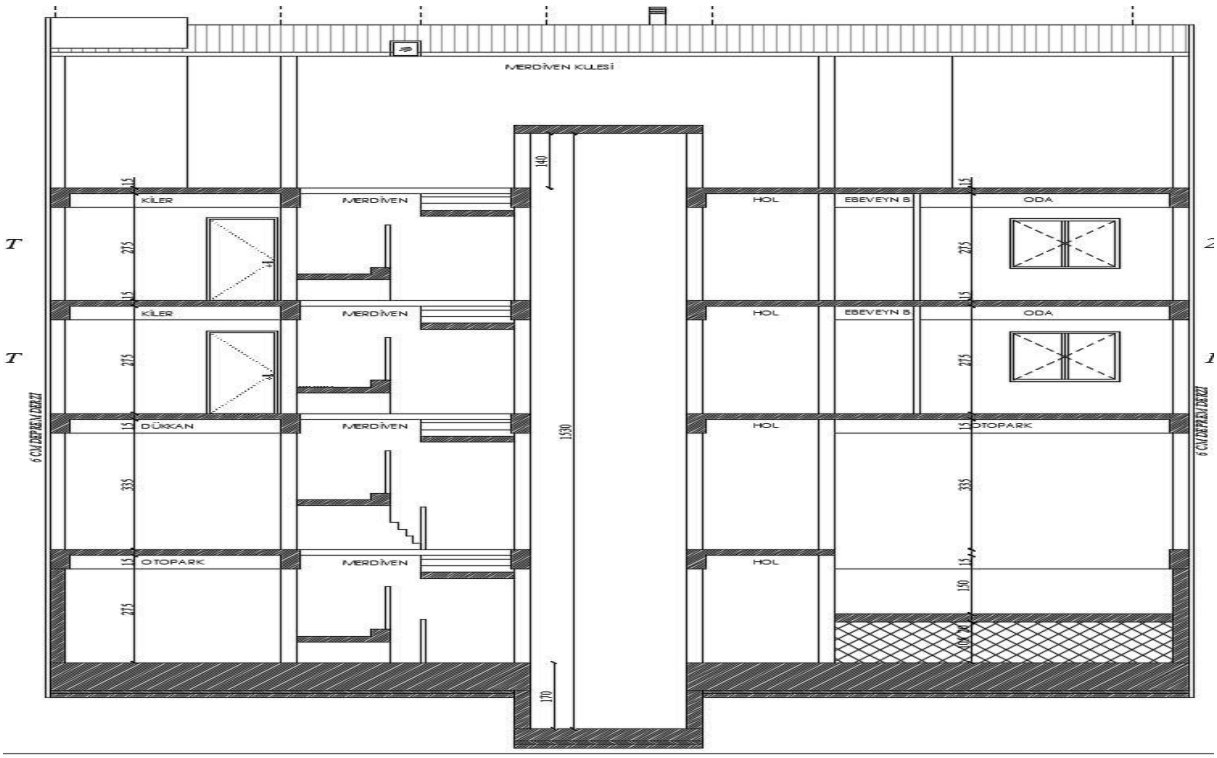


Figure 53: Figure — 2784 Sokak