KGM | ENGINEERING

TRILLIUM RAILWAY SHORING – Hwy 406 WIDENING MTO 2011-2005

ANCHORED SHORING DESIGN DOCUMENTATION

Soldier [illegible]e with ti[illegible]ing

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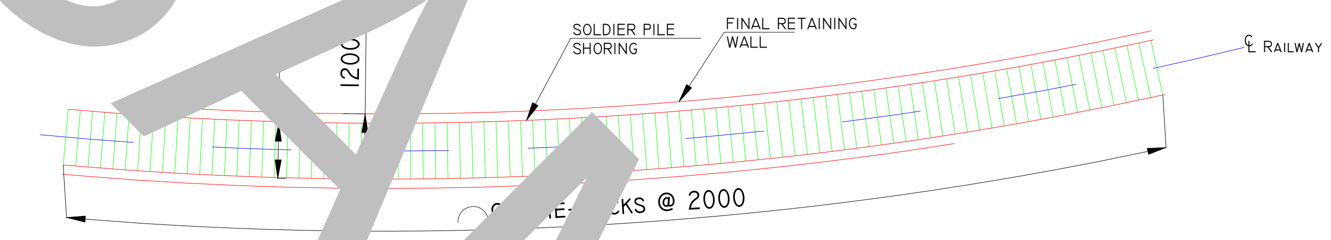
ANCHORED SOLDIER PILE
(Anchor depth: 1.5 m)

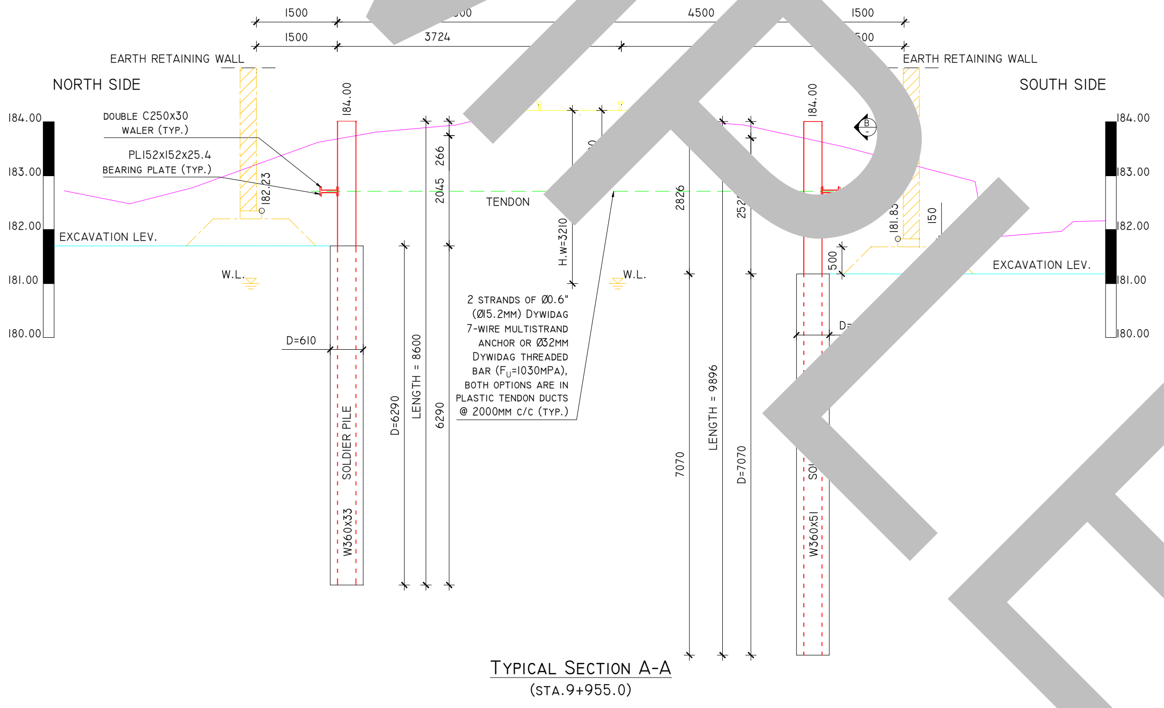
1. Input parameters

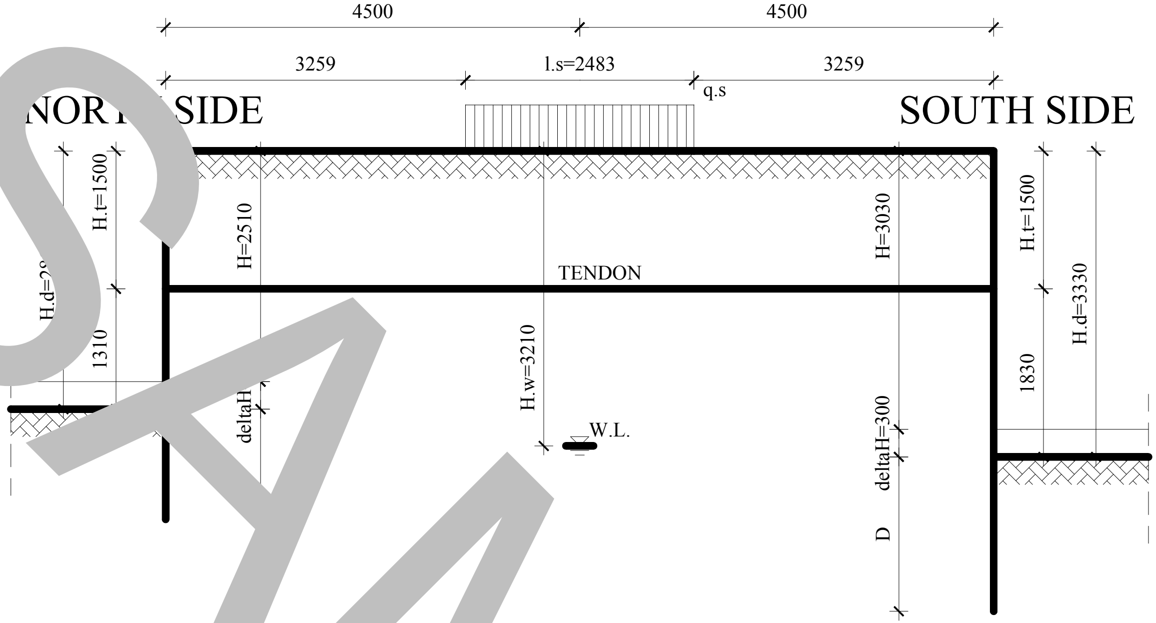
1.1. Geometry

1.1.1. Plan view



1.1.2. Cross section @ 9+955





The first element of the vectors refe[illegible] the nort[illegible], the seco[illegible] the [illegible]hern side of the railway.

Height from top of the soldier pile to botto[illegible] excavation: $:= \begin{pmatrix} 2.51 \\ 3.0\text{[illegible]} \end{pmatrix}$

Uncertainty in the depth of excavation: ΔH [illegible] m

Design height of excavation from top of the soldier pile: $_d := H + \Delta$[illegible] $\begin{pmatrix} 2.81 \\ \text{[illegible]} \end{pmatrix}$

Depth of the tendon: $H_t := 1.5\text{m}$

[illegible]sign Criteria"
11. 12. 13. pts.

Based on the documents provided by Terraprobe Inc. the water level is at the elevation of [illegible]n, this is a temporary state, therefore an assumed surplus of 1 m will be used.

Depth of water level from top of the soldier pile: $H_w := 3.21\text{m}$

Width of the track (tie length): $l_s := 8\text{ft} = 2.438\text{ m}$

"Design C[illegible]"
9

At passive earth pressure side, tributary width of caissons is taken as double of diameter of caisson (2Φ), according to AREMA 28.5.3.2.

1.2. Standards, regulations

AREMA Part 20 - Flexible Sheet Pile Bulkheads
AREMA Pa Temporary Structures for Construction

For sold piles with ber lagging:
mporary Sh g Design Load Diagrams
porary Sho g Construction Procedure
D n Criteri 11. Jan. 06.

.3. Ma ial properties

1. S

Cinder fill mater rganics o ying silty clay.

Passive earth pressure coeffi t: $K_p := 3$

The given data apply to the sold pile type of on. The Temporary Shoring Design Load Diagrams and the "Design Criteria 10.pt." indicat at on the r ed **instead of the active earth pressure the at rest earth pressure** (which is greater in itude) s be ied, with a minimum coefficient of 0.5. Further on this assumption will be used, how r in ot pects regulations of AREMA Part 20 will be adapted.

At rest earth pressure coefficient: 0 :

"Design Criteria" 10.pt.

Unit weight of the moist soil: oist := 20. m^3

Unit weight of submerged soil (**assumed**): γ_{su} 21 $\frac{kN}{m^3}$

1.3.2. Water

Unit weight of water: $\gamma_w := 10 \frac{kN}{m^3}$

1.3.3. Tendon

In **this case** (notably when the anchor is 1.5 m below the rail) the **tendon** type gi the Temporary Shoring Construction Procedure will be utilized.

Dywidag Multistrand

- 0.6" dia. (15.24 mm),
- 7-wire,
- 270 ksi (1862 MPa),
- Low relaxation,
- Greased and coated,
- Conforming to CSA G279-87 (ASTM A 416)

"D Criteria" pt.

Data provided by the manufacturer:

Minimum ultimate strength: $f_{pu} := 270\mathrm{ksi} = 1862 \cdot \mathrm{MPa}$ (based on provided construction procedure)

Nomina[illegible]oss section[illegible]a per strand: $A_s := 0.217\ \mathrm{in}^2 = 140 \cdot \mathrm{mm}^2$

[illegible]ltimate st[illegible]th of a [illegible]d: $F_{pu} := f_{pu} \cdot A_s = 261 \cdot \mathrm{kN}$

[illegible]aximum jac[illegible] force: $F_{pj} := 0.8 F_{pu} = 208 \cdot \mathrm{kN}$

M[illegible]off force: $F_{pl} := 0.7 F_{pu} = 182 \cdot \mathrm{kN}$

Note: the tendons [illegible]unded, an[illegible]ed at the two sides.

"Design Criteria" 14. 22.6. 22.8.pt

1.3.4. Steel soldier [illegible]

Yield stress: $F_y := 350\mathrm{MPa}$

Young modulus: $E_s := 200\mathrm{GPa}$

1.4. Safety factors

Soldier pile tensile yield strength: (AREMA-[illegible].3.[illegible] F[illegible]p $:= \frac{2}{3} = 0$[illegible]

Anchor allowable stress: (AREMA 8-20.5.7.a(2).) FS_{anch}[illegible]

"Design Criteria" 16.pt.

Passive resistance: (AREMA 8-28.5.1.2.) F[illegible] := [illegible]

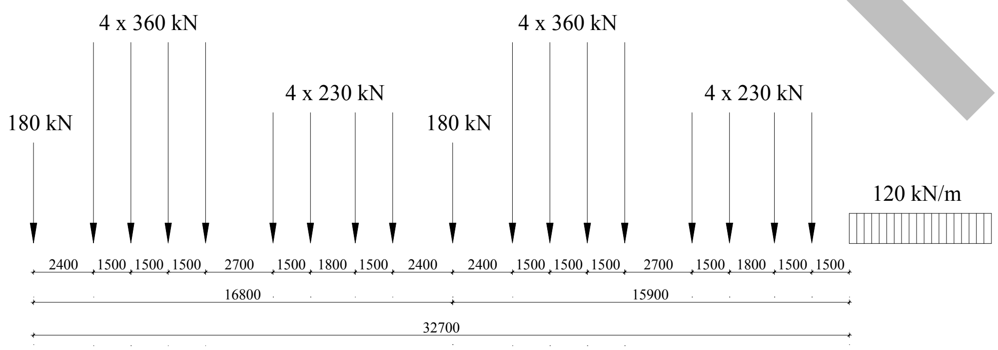
"Design Criteria" 21.pt.

1.5. Loads

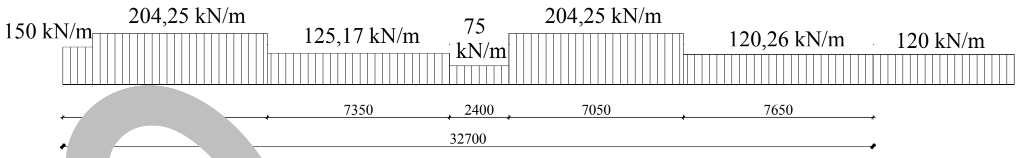
1.5.1. Live loads

1.5.1.1. Vertical

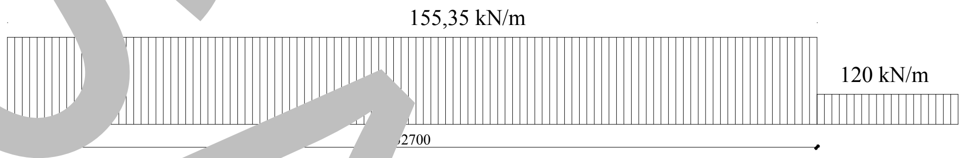
The vertical live load is the Cooper E 90, which is equal to the Cooper E 80 (EM 36[illegible]ultiplied by 1.125.
The Cooper E 80 can be found in AREMA Part 28, fig. 8-2-1. as follows:



These axial loads can be considered as distributed loads due to the effect of the substructure of the railway.
Let's take first the approximation of coupling the similar axial loads:



[illegible] we dist[illegible]e all the a[illegible] loads along the length of the carriage, we get:

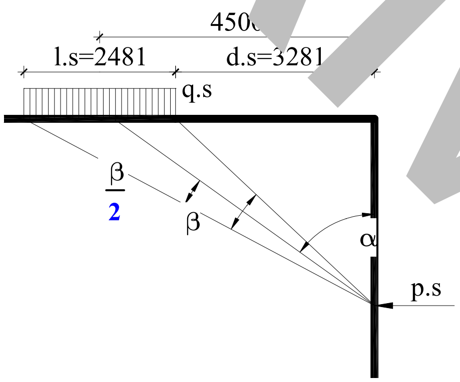


According to t[illegible]g[illegible]riteria" 9[illegible]he applied strip load magnitude is the following:

The strip load (Cooper E90, A[illegible]010): $\frac{90\text{kip}}{5\text{ft}\cdot l_s} = 107.731\cdot\text{kPa}$

"Design Criteria" 9.pt.

1.5.1.2. Horizontal



[illegible]om the Boussinesque solution:

$$p_s = \frac{2\cdot q_s}{\pi}\cdot\left[\beta + \sin(\beta)\cdot(\sin(\alpha))^2 - \sin(\beta)\cdot(\cos(\alpha))^2\right]$$

Chan[illegible] [illegible]artes coordinates:

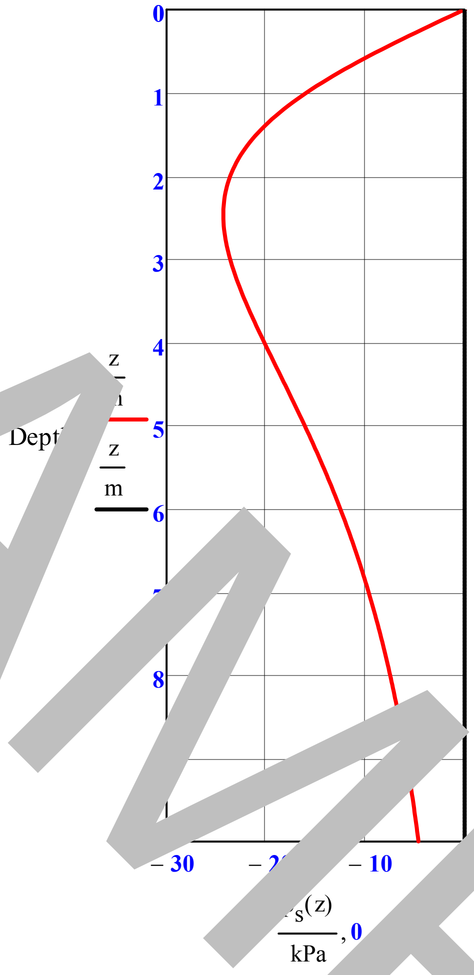
$$\alpha = \frac{1}{2}\cdot\left([illegible] \left(\frac{d_s + l_s}{z}\right) [illegible] \left(\frac{d_s}{z}\right)\right)$$

$$[illegible]\ \text{atan}\left(\frac{d_s + l_s}{z}\right) - \text{atan}\left(\frac{d_s}{z}\right)$$

$l_s =$ [illegible] m $\quad d_s$ [illegible] m [illegible] 3.281 m

The horizontal strip load as the function of the depth:

$$p_s(z) := \begin{cases} 0 & \text{if } z = 0 \\ \left[-\frac{2\cdot q_s}{\pi}\cdot\left[\begin{array}{l}\text{atan}\left(\frac{d_s + l_s}{z}\right) - \text{atan}\left(\frac{d_s}{z}\right) \ldots \\ + \sin\left(\text{atan}\left(\frac{d_s + l_s}{z}\right) - \text{atan}\left(\frac{d_s}{z}\right)\right)\cdot\left[\sin\left[\frac{1}{2}\cdot\left(\text{atan}\left(\frac{d_s + l_s}{z}\right) + \text{atan}\left(\frac{d_s}{z}\right)\right)\right]\right]^2 \ldots \\ + -1\cdot\left[\sin\left(\text{atan}\left(\frac{d_s + l_s}{z}\right) - \text{atan}\left(\frac{d_s}{z}\right)\right)\cdot\left[\cos\left[\frac{1}{2}\cdot\left(\text{atan}\left(\frac{d_s + l_s}{z}\right) + \text{atan}\left(\frac{d_s}{z}\right)\right)\right]\right]^2\right]\end{array}\right]\right] & \text{otherwis[illegible]} \end{cases}$$



The horizontal [illegible] sure fron
Cooper E90 live loa[illegible] ting on the sh

2. NORTHERN SIDE

2.1. Horizontal dead loads - northern side

2.1.1. Functions

Retained side Tributary area of a pile: $X := 2\mathrm{m}$

At rest earth pressure

$$p_{z.0}(z) := \begin{cases} -K_0\cdot\gamma_{moist}\cdot z & \text{if } z < H_w \\ -K_0\cdot\left[\gamma_{moist}\cdot H_w + \left(\gamma_{subm} - \gamma_w\right)\cdot\left(z - H_w\right)\right] & \text{otherwise} \end{cases}$$

Water pressure

$$u_0(z) := \begin{cases} 0 & \text{if } z < H_w \\ -\gamma_w\cdot\left(z - H_w\right) & \text{otherwise} \end{cases}$$

Resisting side

Passive earth pressure
According to the "Design Criteria" (20.pt.) the passive resistance of the soil should be ignored within a width at least 1.5 times the pile width.

Pile width: $\Phi := 0.61\mathrm{m}$

$\Delta_p := 1.5\cdot\Phi = 0.915\ \mathrm{m}$

"Design Criteria"
20.pt.

$$p_{z.p}(z) := \frac{1}{FS_{rp}}\cdot\begin{cases} 0 & \text{if } z < H_{d_1} + \Delta_p \\ K_p\cdot\left(\gamma_{subm} - \gamma_w\right)\cdot\left[z - \left(H_{d_1} + \Delta_p\right)\right] & \text{otherwise} \end{cases}$$

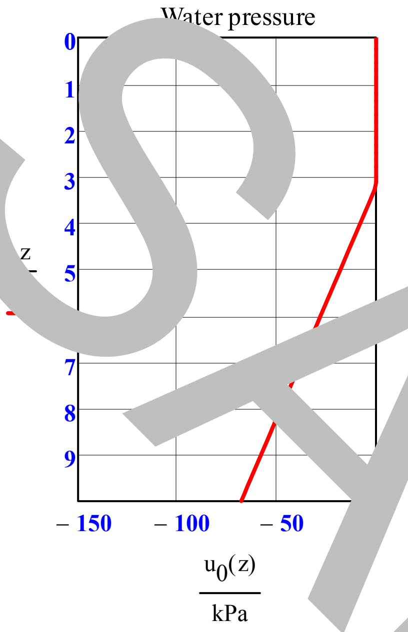
"Design Criteria"
21.pt.

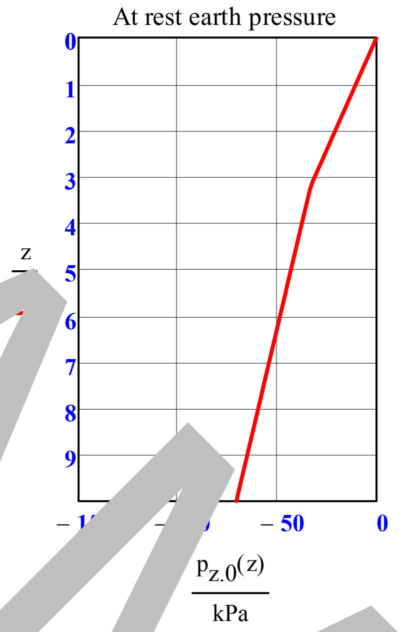
Water pressure

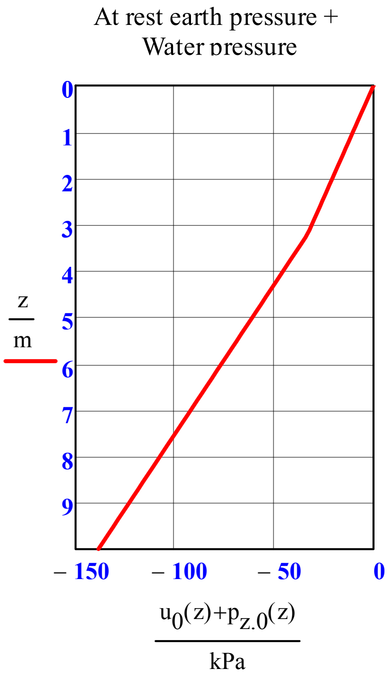
$$u_p(z) := \begin{cases} 0 & \text{if } z < H_{d_1} + \Delta_p \\ \gamma_w\cdot\left[z - \left(H_{d_1} + \Delta_p\right)\right] & \text{otherwise} \end{cases}$$

2.1.2. Graphs

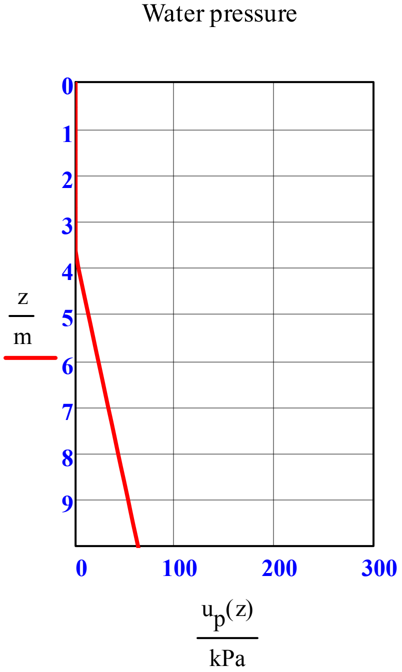
Retained side

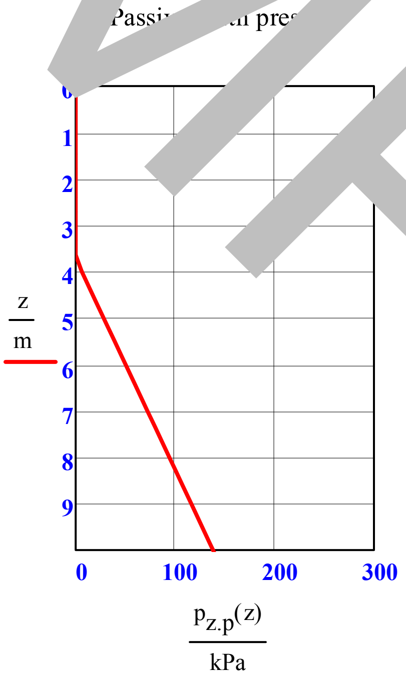

Water pressure

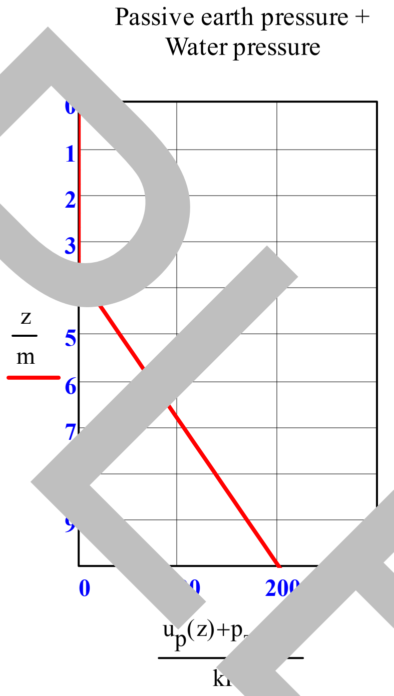

At rest earth pressure


At rest earth pressure +
Water pressure

Resisting side

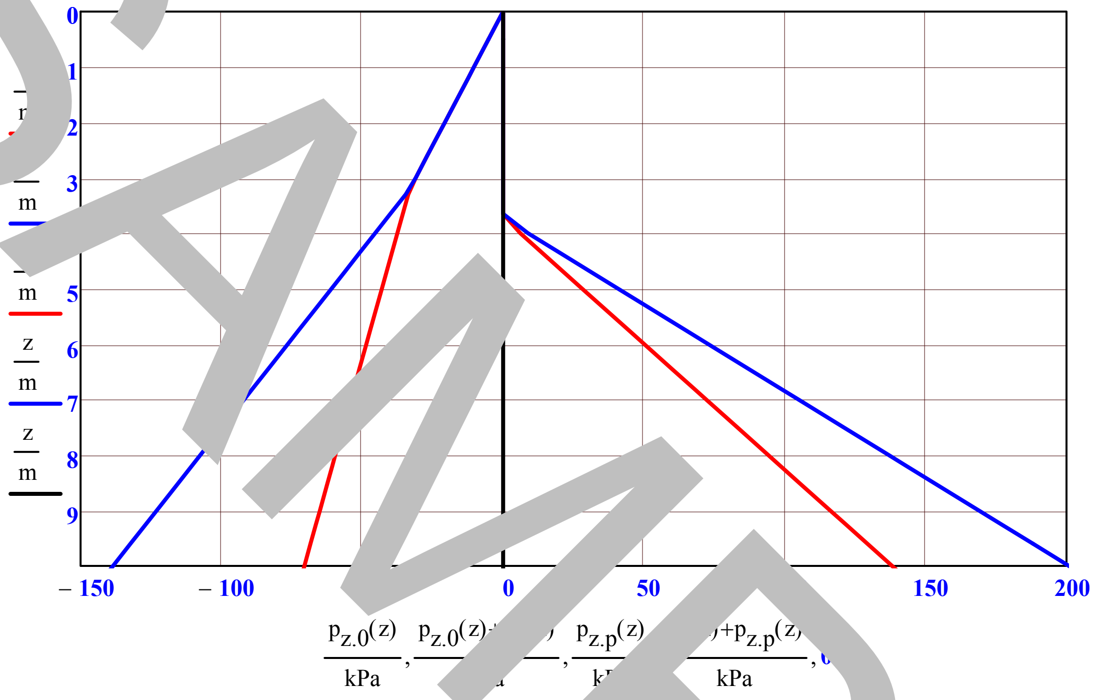

Water pressure


Passive earth pressure


Passive earth pressure +
Water pressure

In a joint diagram

	Retained side		Resisting side
Blue:	At rest earth pressure + Water pressure	Blue:	Passive earth pressure + Water pressure
Red:	At rest earth pressure	Red:	Passive earth pressure



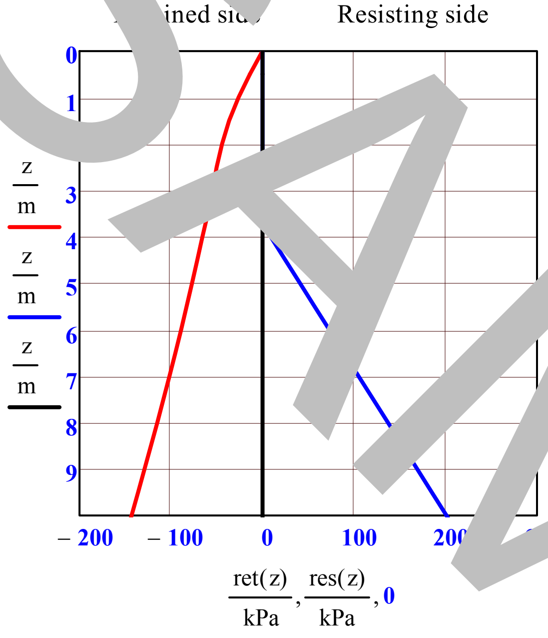
2.2. Design calculation - northern side

2.2.1. Figure of horizontal loads

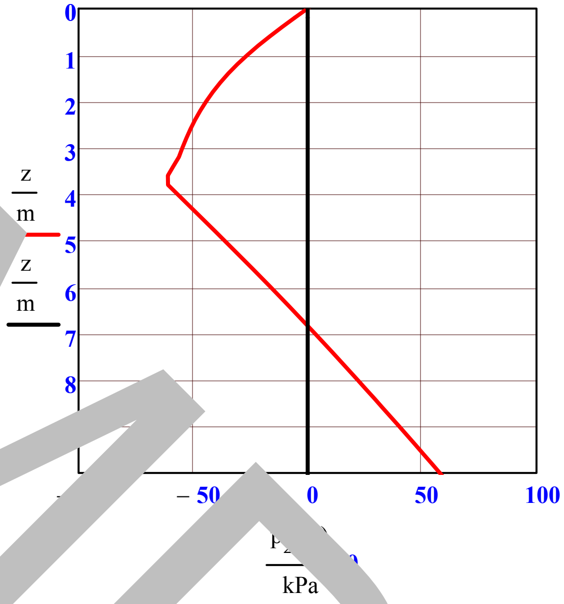
Pressure on the retained side: $ret(z) := p_{z.0}(z) + u_0(z) + p_s(z)$

Pressure on the resisting side: $res(z) := p_{z.p}(z) + u_p(z)$

Resultant of the two actions: $p_{\Sigma}(z) := res(z) + ret(z)$



The sum of the pressures acting on the wall.



The depth at which the pressure becomes zero: $z_1 := \mathrm{root}(p_{\Sigma}$ [illegible] $) = 6.834\ m$

2.2.2. Depth of embedment

2.2.2.1. Anchor force (horizontal equilibrium)

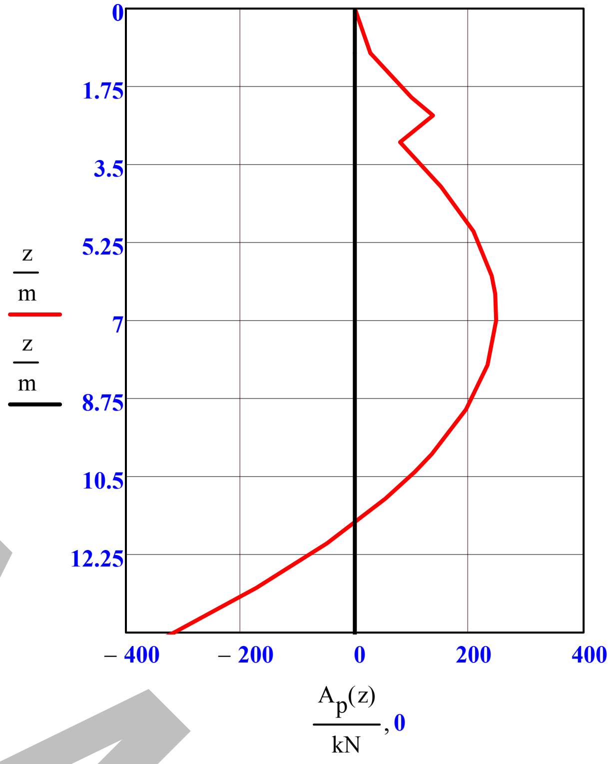
The force in the anchor equals to the negative resultant of the distributed loads:

$$A_p(z) := \begin{array}{|l} \left[\left(-\int_0^z p_{\Sigma}(t)\,dt\right)\cdot X\right] \quad \text{if } z \le H_1 \\ \left[\left(-\int_0^{H_1} p_{\Sigma}(t)\,dt\right)\cdot X - \left(\int_{H_1}^{z_1} p_{\Sigma}(t)\,dt\right)\Phi - \left(\int_{z_1}^{z} p_{\Sigma}(t)\,dt\right)\cdot 2\Phi\right] \quad \text{if } z > H_1 \end{array}$$

In the equations "X" denotes tributary width above excavation level, "Φ" denotes width of concrete caisson where at-rest soil pressure is present, '2Φ" denotes equivalent width of concrete caisson where passive soil pressure is present according to AREMA 28.5.3.2.

The depth at which the resultant of the pressures becomes zero:

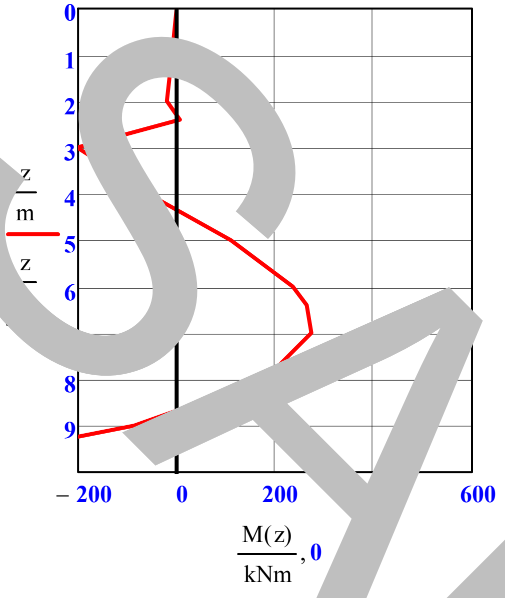
$z_2 := \mathrm{root}\left(A_p(var), var\right) = 11.55\ \mathrm{m}$



2.2.2.2. Depth of embedment (bending moment at the bottom)

The bending moment for a cross section at depth "z":

$$M(z) := \begin{cases} \left[\left[\int_0^z p_\Sigma(t)(z-t)\,dt\right]\cdot X + \begin{cases} 0 & \text{if } z < H_t \\ A_p(z)\cdot\left(z - H_t\right) & \text{otherwise} \end{cases}\right] & \text{if } z \le H_1 \\ \left[\left[\int_0^{H_1} p_\Sigma(t)\cdot(z-t)\,dt\right]\cdot X + \left[\int_{H_1}^{z_1} p_\Sigma(t)\cdot(z-t)\,dt\right]\cdot\Phi + \left[\int_{z_1}^{z} p_\Sigma(t)\cdot(z-t)\,dt\right]\cdot\Psi\right] + A_p(z)\cdot\left(z - H_t\right) & \text{if } z > H_1 \end{cases}$$



Length of soldier pile equals to the coordinate where M(z) becomes zero:

$$L_{w_1} := \mathrm{root}(M(var), var) = 8.745\ \mathrm{m}$$

Applied wall length:

$$L_{app.w_1} := \mathrm{ceil}\left(\frac{L_{w_1}}{0.1\mathrm{m}}\right)0.1\mathrm{m} \qquad \boxed{L_{app.w_1} = 8.8\ \mathrm{m}}$$

Depth of the embedment shall be at least 3m according to the "Design Criteria" (23.pt.):

$$D_1 := \max\left(L_{app.w_1} - H_{d_1}, 3\mathrm{m}\right) \qquad \boxed{D_1 = 5.99\ \mathrm{m}} \qquad \boxed{\text{"Design Criteria" 23.pt.}}$$

2.2.3. Maximum bending moment

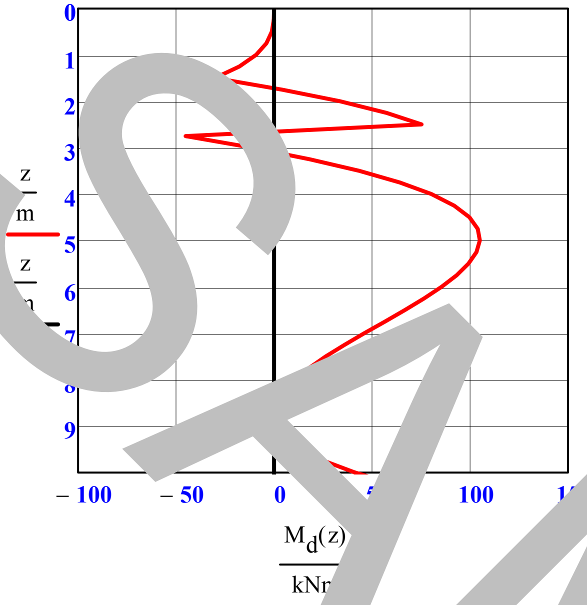
Bending moment:

$$M_d(z) := \begin{cases} \left[\left[\int_0^z p_\Sigma(t)(z-t)\,dt\right]\cdot X + \begin{cases} 0 & \text{if } z < H_t \\ A_p\left(L_{w_1}\right)\cdot\left(z - H_t\right) & \text{otherwise} \end{cases}\right] & \text{if } z \le H_1 \\ \left[\left[\int_0^{H_1} p_\Sigma(t)\cdot(z-t)\,dt\right]\cdot X + \left[\int_{H_1}^{z_1} p_\Sigma(t)\cdot(z-t)\,dt\right]\cdot\Phi + \left[\int_{z_1}^{z} p_\Sigma(t)\cdot(z-t)\,dt\right]\cdot 2\Phi\right] + A_p\left(L_{w_1}\right)\cdot\left(z - H_t\right) & \text{if } z > H_1 \end{cases}$$

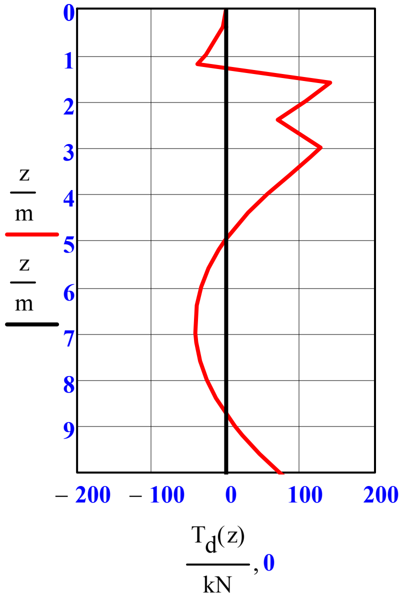
Shear force:

$$T_d(z) := \begin{cases} \left[\left(\int_0^z p_\Sigma(t)\,dt\right)\cdot X + \begin{cases} 0 & \text{if } z < H_t \\ A_p\left(L_{w_1}\right) & \text{otherwise} \end{cases}\right] & \text{if } z \le H_1 \\ \left[\left(\int_0^{H_1} p_\Sigma(t)\,dt\right)\cdot X + \left(\int_{H_1}^{z_1} p_\Sigma(t)\,dt\right)\Phi + \left(\int_{z_1}^{z} p_\Sigma(t)\,dt\right)\cdot 2\Phi\right] + A_p\left(L_{w_1}\right) & \text{if } z > H_1 \end{cases}$$

Moment diagram



Shear force diagram



The position of the maximum bending moment:

$z_3 := \text{root}(T_d(\text{var}), \text{var}) = 4.966\ \text{m}$

The maximum bending moment:

$M_{max_1} := M_d(z_3)$

M_{max_1} = [illegible]05·kNm

Required section modulus:

$$W_{nec_1} := \frac{M_{max_1}}{FS_{sp} F_y}$$

W_{nec_1} = [illegible]42.086[illegible]

Applied section: W360x33

Section modulus: $S_{x.360x33} := 474 \cdot 10^3 mm^3$

Inertia: $I_{x.360x33} := 82.7 \cdot 10^6 mm^4$

Area: $A_{360x33} := 4170 mm^2$

2.2.4. Anchor pull

The calculated magnitude:

$A_1 := A_p(L_{w_1})$

$A_1 = 205.953 \cdot kN$

The maximum allowable load of a strand:

$F_{allow} := \frac{F_{pu}}{FS}$ $F_{allow} = 130 \cdot kN$ "Design Criria" 16.pt.

Require umber o ds per tendon:

$a_1 := ce\left(\frac{A_1}{llow}\right) =$ $n_{a_1} = 2$

.2.5. Tota esign load of one tie back

$F_{p} := \; 205.953 \cdot kN$

Lock-off load of a te ef.: "Cons tion Procedure")

$F_{pl} := 1.1 \cdot F_{p\;t.n}$ $F_{pl} = 227 \cdot kN$

Proof load of a tendon: (ref.: "C ction Proced

$F_{pp} := 1.33 \cdot F_{pd.tot.n}$ $F_{pp} = 274 \cdot kN$

2.2.6. Total amount o oad e sh ing wall for one-meter length (excavated height)

2.2.6.1. Due to soil

At rest earth pressure:

$f_{p.0_1} := \int_0^{H} p_{z.0}(t) \; -40.862 \cdot kN$ "Design Criteria" 22.1.pt.

2.2.6.2. Due to train load

$f_{q_1} := \int_0^{H_{d_1}} p_s(t)\,dt = -48.482 \cdot \frac{kN}{m}$ "n Criteria" 22.2.pt.

2.2.6.3. Due to hydrostatic pressure

$f_{u_1} := \int_0^{H_{d_1}} u_0(t)\,dt = 0 \cdot \frac{kN}{m}$ "Design Criteria" 22.3.pt.

3. SOUTHERN SIDE

3.1. Horizontal dead loads - southern side

3.1.1. Functions

Retained s[illegible]

Tributary area of a pile: $X := 2\text{m}$

At rest earth pressure

$$p_{z}\text{[illegible]} := \begin{cases} \text{[illegible]}\cdot\gamma_{moist}\cdot z & \text{if } z < \text{[illegible]} \\ -K_0\cdot\left[\gamma\text{[illegible]}_w + \left(\gamma_{sub}\text{[illegible]}\gamma_w\right)\cdot\left(z - H_w\right)\right] & \text{otherwise} \end{cases}$$

Water pressure

$$u_0(z) := \begin{cases} 0 & \text{if } z < H_w \\ -\gamma_w\cdot\left(z - H_w\right) & \text{otherwise} \end{cases}$$

Resisting side

Passive earth pressure
According to the "Design Criteria" (20.pt.) t[illegible]ve resistan[illegible] the soil s[illegible] ignored within a width at least 1.5 times the pile width.

Pile width:

Φ [illegible]1m

$\Delta_p := 1.5\cdot\Phi =$ [illegible]15 m

"Design Criteria"
20.pt.

$$p_{z.p}(z) := \frac{1}{FS_{rp}}\cdot\begin{cases} 0 & \text{if } z < H_{d_2} + \Delta_p \\ K_p\cdot\left(\gamma_{subm} - \gamma_w\right)\cdot\left[z - \left(H_{d_2} + \Delta_p\right)\right] & \text{otherwise} \end{cases}$$

"Design Criteria"
21.pt.

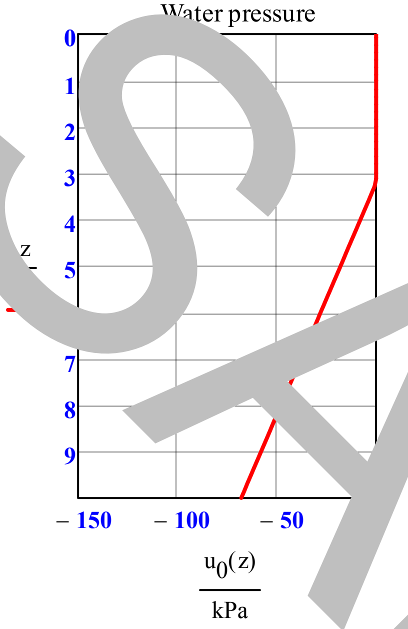
Water pressure

$$u_p(z) := \begin{cases} 0 & \text{if } z < H_{d_2} + \Delta_p \\ \gamma_w\cdot\left[z - \left(H_{d_2} + \Delta_p\right)\right] & \text{otherwise} \end{cases}$$

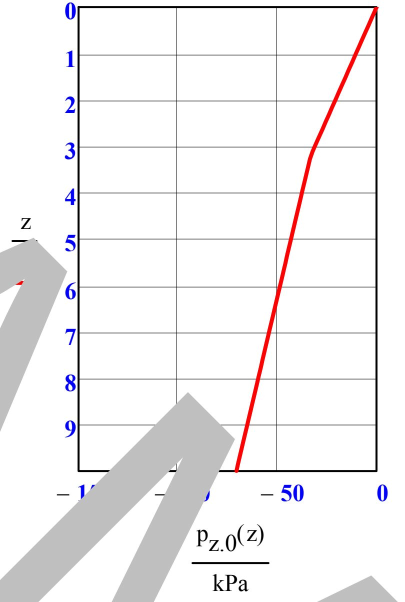
3.1.2. Graphs

Retained side

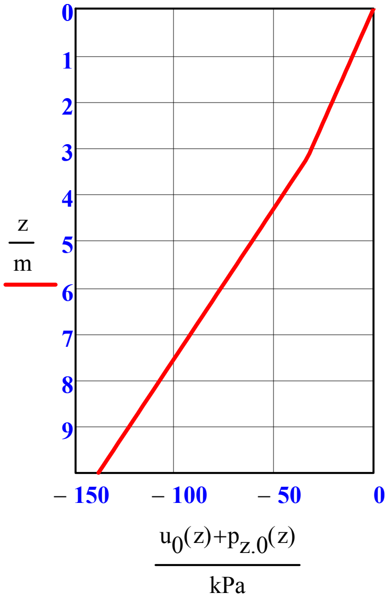
Water pressure



At rest earth pressure

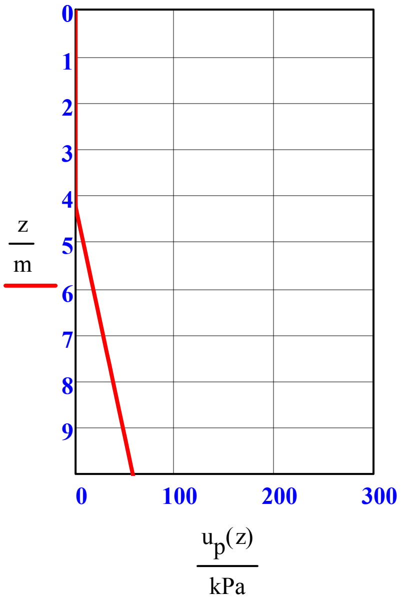


At rest earth pressure +
Water pressure

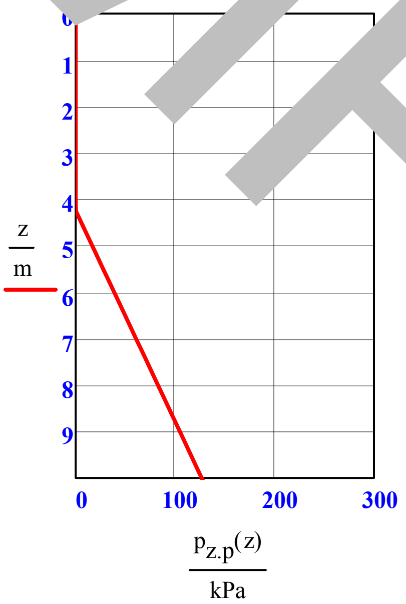


Resisting side

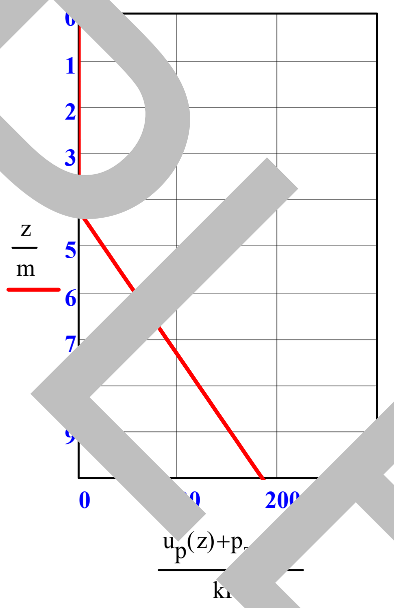
Water pressure



Passive earth pressure



Passive earth pressure +
Water pressure



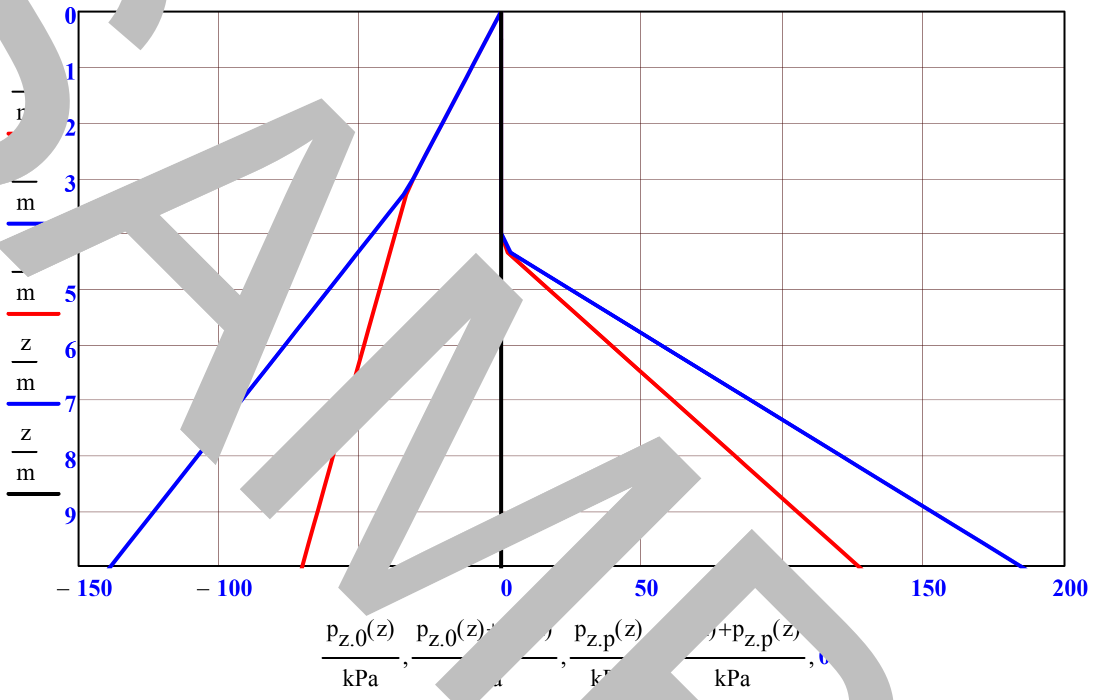
In a joint diagram

Retained side

Blue: At rest earth pressure +
Water pressure
Red: At rest earth pressure

Resisting side

Blue: Passive earth pressure +
Water pressure
Red: Passive earth pressure



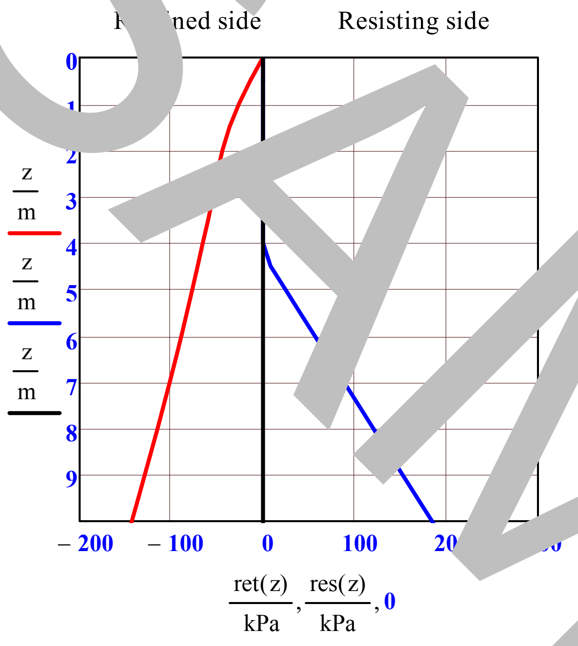
3.2. Design calculation - southern side

2.2.1. Fig[illegible]e of horizontal loads

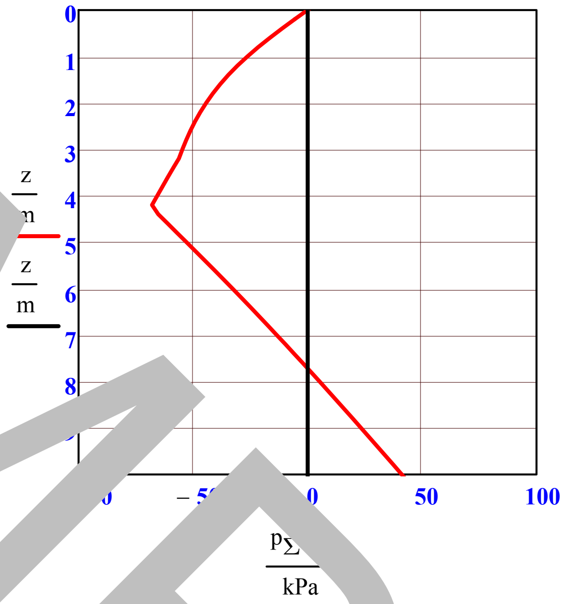
Pressure [illegible] the retai[illegible]side: $ret(z) := p_{z.0}(z) + u_0(z) + p_s(z)$

Pressure [illegible]the resisting [illegible]de: $res(z) := p_{z.p}(z) + u_p(z)$

[illegible]esultant [illegible]e two ac[illegible]s: $p_\Sigma(z) := res(z) + ret(z)$



The sum of the pressures acting on the wall.



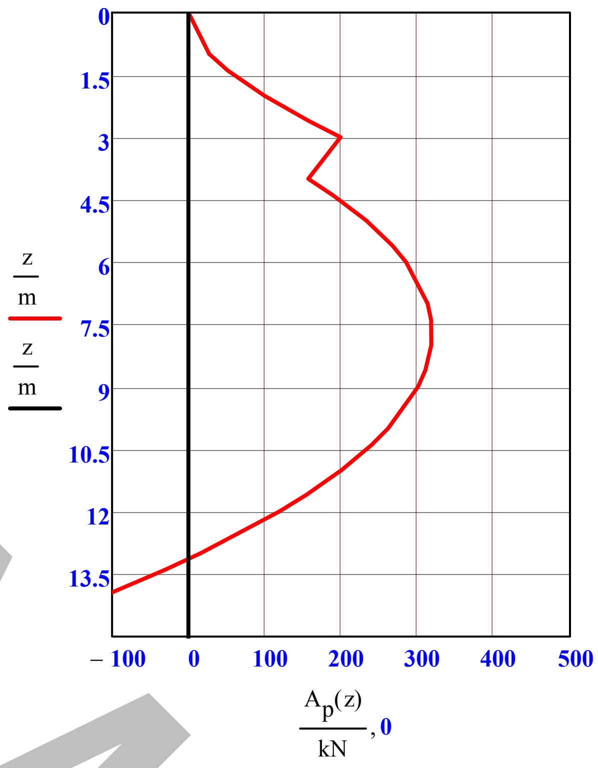
The depth at which the pressure becomes zero: $z_1 := \text{root}($[illegible]$\text{ar}), \text{va}$[illegible] $7.719\ \text{m}$

3.2.2. Depth of embedment

3.2.2.1. Anchor force (horizontal equilibrium)

The force in the anchor equals to the negative resultant of the distributed loads:

$$A_p(z) := \left| \begin{array}{ll} \left[\left(-\int_0^z p_\Sigma(t)\,dt\right)\cdot X\right] & \text{if } z \le H_2 \\ \left[\left(-\int_0^{H_2} p_\Sigma(t)\,dt\right)\cdot X - \left(\int_{H_2}^{z_1} p_\Sigma(t)\,dt\right)\Phi - \left(\int_{z_1}^{z} p_\Sigma(t)\,dt\right)\cdot 2\Phi\right] & \text{if } z > H_2 \end{array} \right.$$



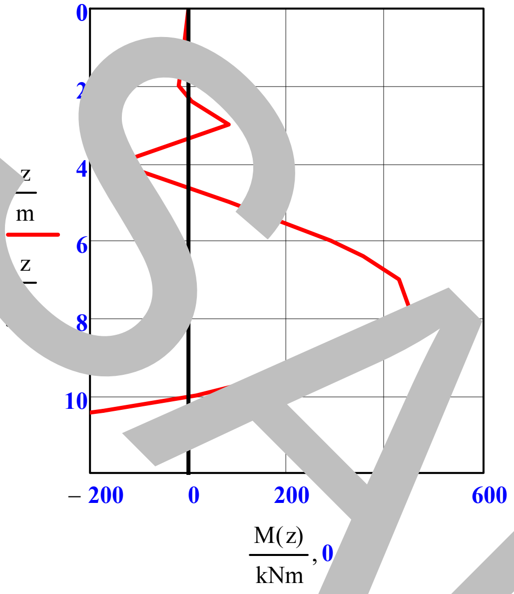
The dep[illegible]t which the [illegible]ultant of the press[illegible] becomes z[illegible]

$z_2 := \text{root}(\text{[illegible]}var), va\text{[illegible]}) \text{[illegible]} \mathbf{13.141}\ m$

3.2.2.2. Depth of embedment (be[illegible]g moment[illegible] the bott[illegible])

The bending moment for a cross section at depth "z":

$$M(z) := \left\| \begin{array}{l} \left[\left[\int_0^z p_\Sigma(t)(z-t)\,dt \right] \cdot X + \left| \begin{array}{ll} 0 & \text{if } z < H_t \\ A_p(z)\cdot(z-H_t) & \text{otherwise} \end{array} \right. \right] \text{ if } z \le \text{[illegible]} \\ \left[\left[\int_0^{H_2} p_\Sigma(t)\cdot(z-t)\,dt \right] \cdot X + \left[\int_{H_2}^{z_1} p_\Sigma(t)\cdot(z-t)\,dt \right] \cdot \Phi + \left[\int_{z_1}^{z} p_\Sigma(t)\cdot(z-t) \text{[illegible]} \right] \text{[illegible]} \right] + A_p(z)\cdot(z-H_t) \text{ if } z > H_2 \end{array} \right.$$



Length of soldier pile equals to the coordinate where M(z) becomes zero:

$$L_{w_2} := \mathrm{root}(M(var), var) = 10.037\,m$$

Applied wall length:

$$L_{app.w_2} := \mathrm{ceil}\left(\frac{L_{w_2}}{0.1m}\right) 0.1m \qquad \boxed{L_{app.w_2} = 10.1\,m}$$

Depth of the embedment shall be at least 3m according to the "Design Criteria" (23.pt.):

$$D_2 := \max\left(L_{app.w_2} - H_{d_2}, 3m\right) \qquad \boxed{D_2 = 6.77\,m} \qquad \boxed{\text{"Design Criteria" 23.pt.}}$$

3.2.3. Maximum bending moment

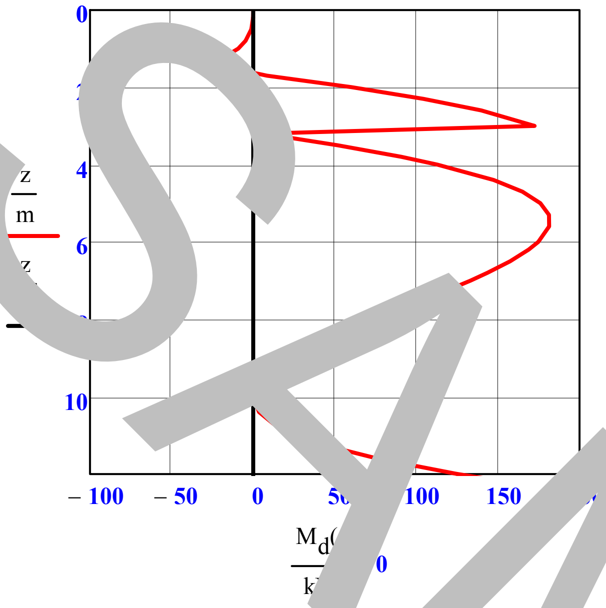
Bending moment:

$$M_d(z) := \left|\begin{array}{ll} \left[\left[\int_0^z p_\Sigma(t)(z-t)\,dt\right]\cdot X + \left|\begin{array}{ll} 0 & \text{if } z < H_t \\ A_p(L_{w_2})\cdot(z - H_t) & \text{otherwise} \end{array}\right.\right] & \text{if } z \le H_2 \\ \left[\left[\int_0^{H_2} p_\Sigma(t)\cdot(z-t)\,dt\right]\cdot X + \left[\int_{H_2}^{z_1} p_\Sigma(t)\cdot(z-t)\,dt\right]\cdot\Phi + \left[\int_{z_1}^{z} p_\Sigma(t)\cdot(z-t)\,dt\right]\cdot 2\Phi\right] + A_p(L_{w_2})\cdot(z - H_t) & \text{if } z > H_2 \end{array}\right.$$

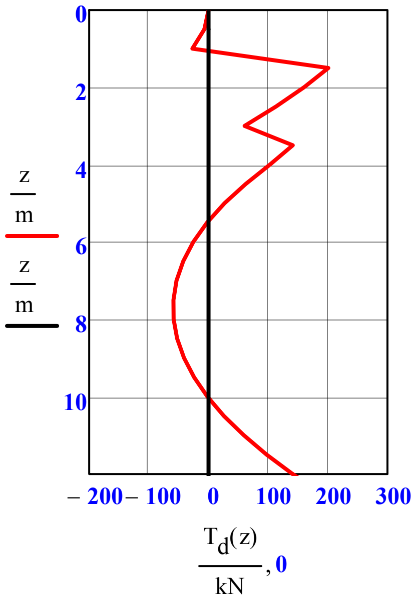
Shear force:

$$T_d(z) := \left|\begin{array}{ll} \left[\left(\int_0^z p_\Sigma(t)\,dt\right)\cdot X + \left|\begin{array}{ll} 0 & \text{if } z < H_t \\ A_p(L_{w_2}) & \text{otherwise} \end{array}\right.\right] & \text{if } z \le H_2 \\ \left[\left(\int_0^{H_2} p_\Sigma(t)\,dt\right)\cdot X + \left(\int_{H_2}^{z_1} p_\Sigma(t)\,dt\right)\cdot\Phi + \left(\int_{z_1}^{z} p_\Sigma(t)\,dt\right)\cdot 2\Phi\right] + A_p(L_{w_2}) & \text{if } z > H_2 \end{array}\right.$$

Moment diagram



Shear force diagram



The position of the maximum bendi[illegible]oment:

$z_3 := \mathrm{root}\left(T_d(\mathrm{var}), \mathrm{var}\right) = 5.455\,\mathrm{m}$

The maximum bending moment:

$M_{max_2} := M_d\left(z_3\right)$ $M_{max_2} = 1$[illegible]kNm

Required section modulus:

$$W_{nec_2} := \frac{M_{max_2}}{FS_{sp}\,F_y}$$ $W_{nec_2} = 779356.9$[illegible]m^3

Applied section: W360x51

Section modulus: $S_{x.360x51} := 796 \cdot 10^3\,\mathrm{mm}^3$

Inertia: $I_{x.360x51} := 141 \cdot 10^6\,\mathrm{mm}^4$

Area: $A_{360x51} := 6450\,\mathrm{mm}^2$

3.2.4. Anchor pull

The calculated magnitude:

$A_2 := A_r(\quad _2)$ $\qquad$ $A_2 = 259.576\cdot kN$

The maximum allowable load of a strand:

$F_{allow} := \frac{F_{pu}}{\quad ch}$ $\qquad$ $\boxed{F_{allow} = 130\cdot kN}$ $\qquad$ "Design Criteria" 16.pt.

Required number of strands per tendon:

$n_{a_2} := \left(\frac{\quad _2}{F_{allow}}\right) = 2$ $\qquad$ $\boxed{n_{a_2} = 2}$

3.2.5. Total design load of the tie back

$F_{pd.tot.s} := A_2 = 259.576\cdot kN$

Lock-off load of a tendon: (ref.: "Construction Procedure")

$F_{pl} := 1.1\cdot F_{pd.tot.s}$ $\qquad$ $\boxed{F_{pl} = 286\cdot kN}$

Proof load of a tendon: (ref.: "Construction Procedure")

$F_{pp} := 1.33\cdot F_{pd.tot.s}$ $\qquad$ $\boxed{F_{pp} = 345\cdot kN}$

3.2.6. Total amount of load on the shoring wall for one meter length (excavated height)

3.2.6.1. Due to soil

At rest earth pressure:

$$f_{p.0_1} := \int_0^{H_{d_2}} p_{z.0}(t)\, dt = -57.35\cdot\frac{kN}{m}$$

"Design Criteria" 22.1.pt.

3.2.6.2. Due to train load

$$f_{q_1} := \int_0^{H_{d_2}} p_s(t)\, dt = -60.653\cdot\frac{kN}{m}$$

"Design Criteria" 22.2.pt.

3.2.6.3. Due to hydrostatic pressure

$$f_{u_1} := \int_0^{H_{d_2}} u_0(t)\, dt = -0.072\cdot\frac{kN}{m}$$

"Design Criteria" 22.3.pt.

4. FE MODEL

The Finite Element Model is calculated using the Plaxis 8.2. software.
The name of the file is 2012-01-26_soldier_pile.plx.

"Design Criteria"
14.pt.

The Plaxis geotechnical finite element software is capable of modeling soil-structure interaction, and handles tie-backs, anchors properly, as points which have nonzero deflection.

The software is generally used for plane strain problems. Taking into consideration that the shoring wall has a tributary width of [illegible] above excavation level and Φ=0.61m below excavation level, the elastic modulus of the soil which is resisting to the moveme[illegible] the soldier pile has to be decreased. The ratio of the decrease is the ratio of the two tributary width.

4.1. Geometry

Height of th[illegible]

L[illegible]$_{pp.w} = \begin{pmatrix} 8.8 \\ 10.1 \end{pmatrix}$ m

Depth of the embedment:

[illegible] $\begin{pmatrix} 5.99 \\ 6.77 \end{pmatrix}$ m

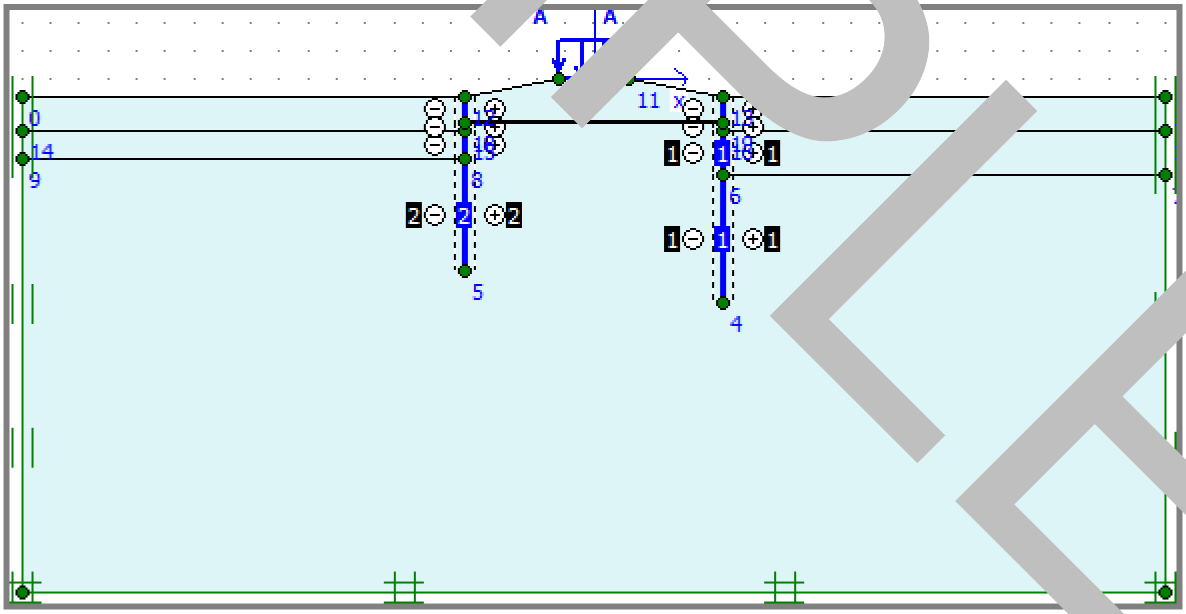
Excavation depths (design v[illegible]s):

$D_{ex.1} := H_t + 30cm$

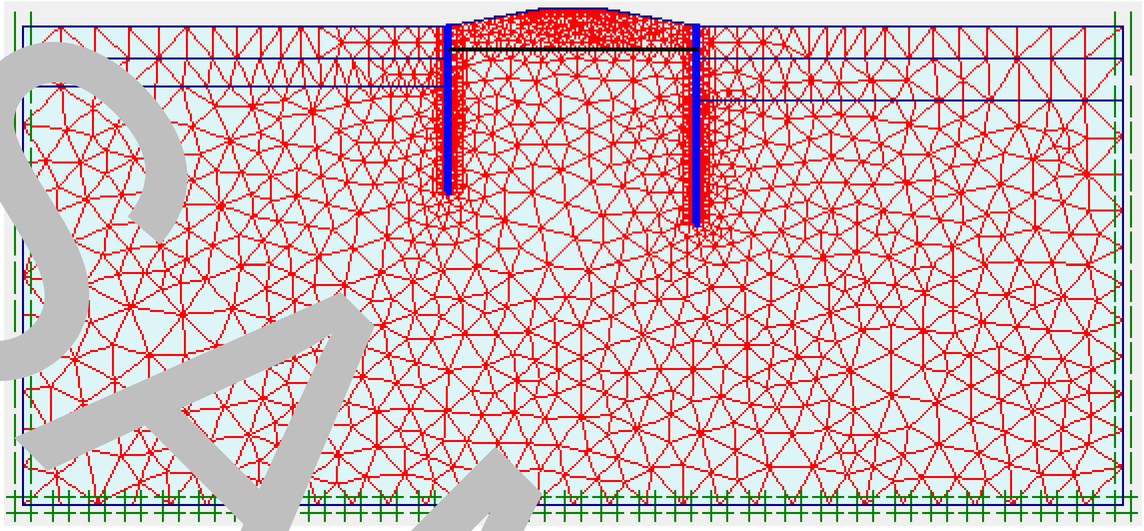
$D_{ex.1} = 1$[illegible]

$D_{ex.2} := H_d$

$D_{ex.2} =$ [illegible] $.33$ m



4.2. The mesh



4.3. Loads

4.3.1. Live load

Magnitude: $q_s = 1[illegible]1 \cdot \frac{kN}{m^2}$

Length: $l_s = 2.438$ [illegible]

4.3.2. Anchorage

Calculated design load of a tendon:

$A_p := \max(F_{pd.tot.n}, F_{pd.tot.s})$ $A_p = 259.576 \cdot kN$

The more sophisticated FE model indicates that the soldier pile does not behave like a rigid body, therefore in order to reduce the top displacement the tieback prestressing force should be decreased.

Reduced prestressing force of one tendon: $F_{app} := 20\% \cdot A_p = 51.915 \cdot kN$

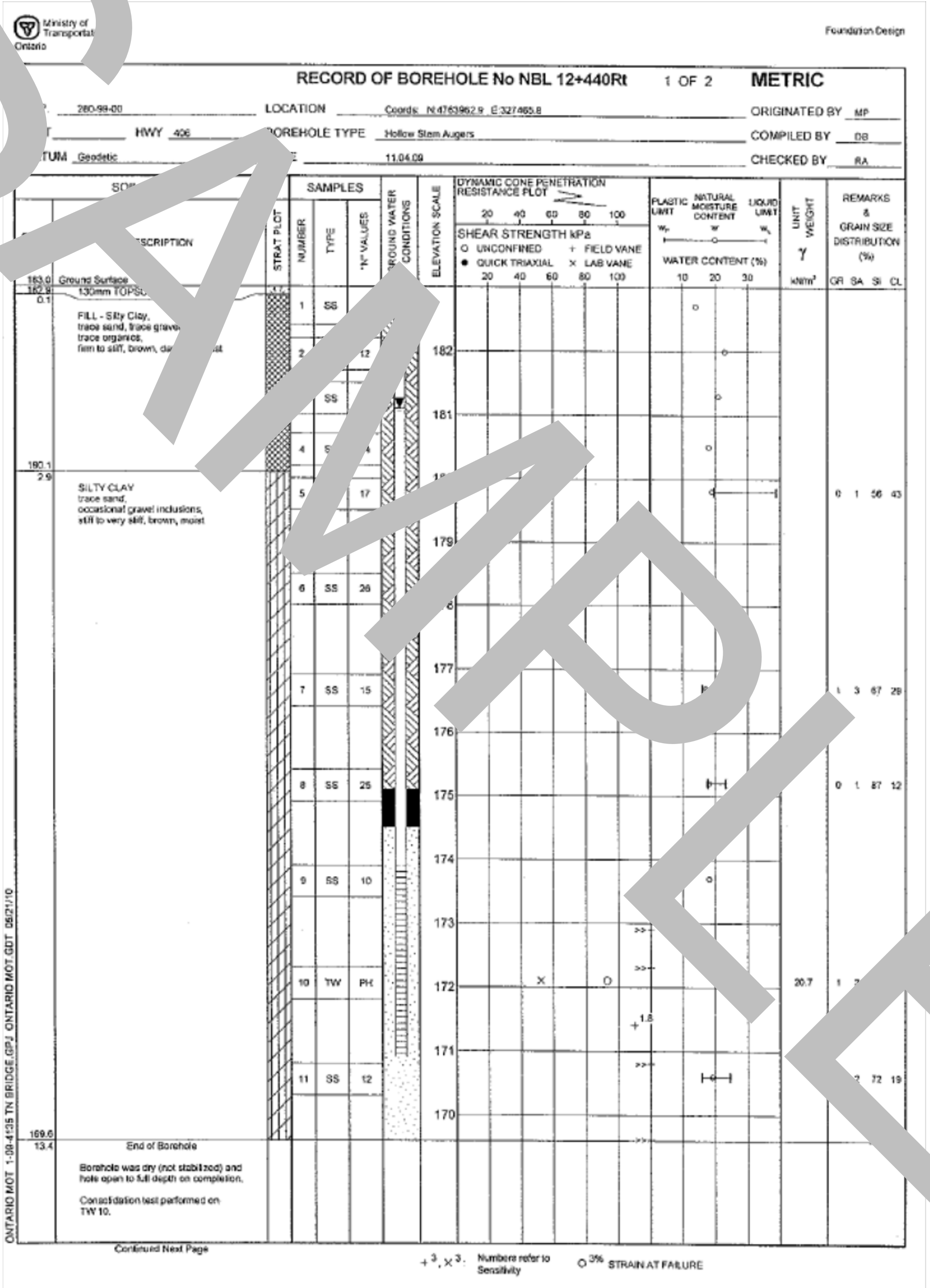
Spacing of tendons (equals to spacing of soldier piles): $s_{app.a} := X = 2\,m$

Unit prestress force to be used in FE model: $f_d := \frac{F_{app}}{s_{app.a}} = 25.958 \cdot \frac{kN}{m}$

4.4. Material properties

4.4.1. Soil

We take [illegible]L 12+4[illegible] borehole (**Terraprobe soil report *(File No. 1-09-4135)***) as the basis of soil properties.



Ministry of Transportat[illegible]
Ontario

Foundation Design

RECORD OF BOREHOLE No NBL 12+440Rt 1 OF 2 **METRIC**

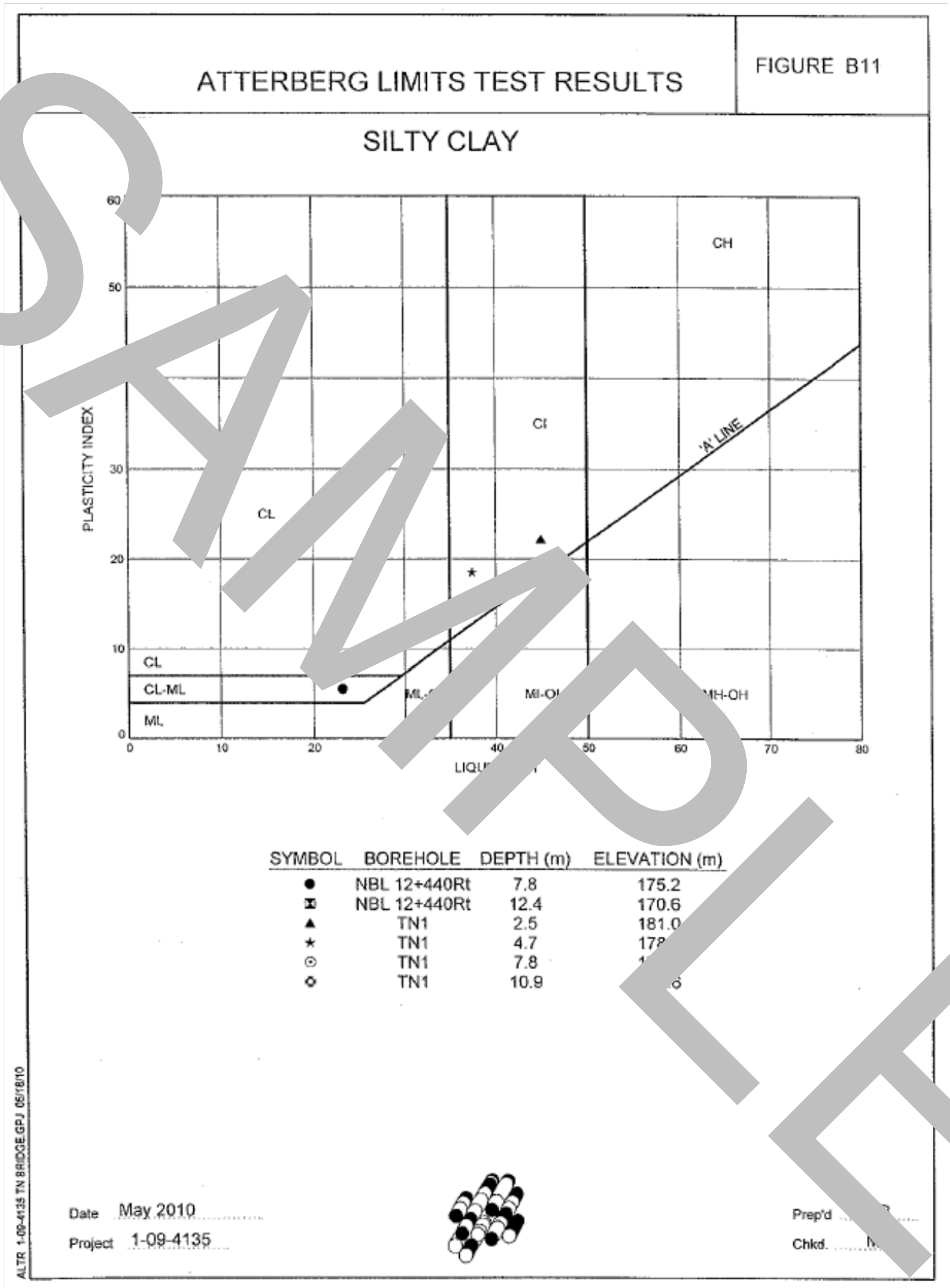
[illegible]. 280-98-00 LOCATION Coords: N:4763962.9 E:327465.8 ORIGINATED BY MP
[illegible]T HWY 406 [illegible]OREHOLE TYPE Hollow Stem Augers COMPILED BY DB
[illegible]TUM Geodetic [illegible] 11.04.09 CHECKED BY RA

Elev. / Depth	Soil description	Sample number	Type	"N" values	Elevation scale
183.0	Ground Surface				
182.9 0.1	130mm TOPS[illegible]	1	SS		
	FILL - Silty Clay, trace sand, trace grave[illegible] trace organics, firm to stiff, brown, da[illegible] [illegible]st	2	[illegible]	12	182
		[illegible]	SS	[illegible]	181
		4	S[illegible]	4	
180.1 2.9	SILTY CLAY trace sand, occasional gravel inclusions, stiff to very stiff, brown, moist	5	[illegible]	17	
					179
		6	SS	26	
					177
		7	SS	15	176
		8	SS	25	175
					174
		9	SS	10	173
		10	TW	PH	172
					171
		11	SS	12	170
169.6 13.4	End of Borehole Borehole was dry (not stabilized) and hole open to full depth on completion. Consolidation test performed on TW 10.				

Sample	Unit weight γ kN/m³	GR	SA	SI	CL
5		0	1	56	43
7		1	3	67	29
8		0	1	87	12
10	20.7	1	[illegible]	[illegible]	[illegible]
11		[illegible]	2	72	19

Continued Next Page

ONTARIO MOT 1-09-4135 TN BRIDGE.GPJ ONTARIO MOT.GDT 05/21/10



ATTERBERG LIMITS TEST RESULTS

FIGURE B11

SILTY CLAY

SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	NBL 12+440Rt	7.8	175.2
⧗	NBL 12+440Rt	12.4	170.6
▲	TN1	2.5	181.0
★	TN1	4.7	178[illegible]
⊙	TN1	7.8	1[illegible]
○	TN1	10.9	[illegible]6

Date May 2010

Project 1-09-4135

Prep'd [illegible]

Chkd. [illegible]

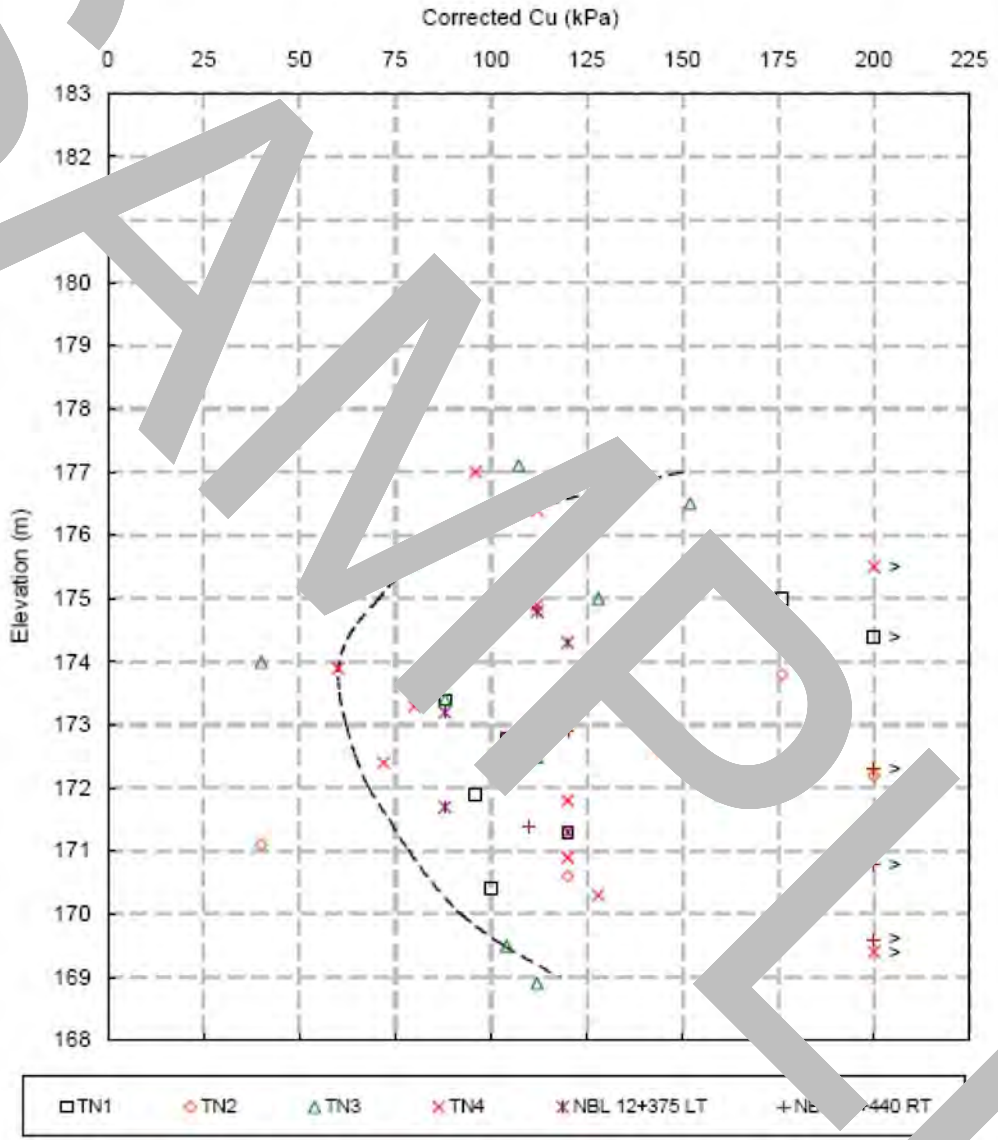
ALTR 1-09-4135 TN BRIDGE.GPJ 05/18/10

CORRECTED UNDRAINED SHEAR STRENGTH

FIGURE B18

HWY 406 TWINNING - TRILLIUM OVERHEAD (NBL)

Silty Clay



Field Shear Vane Correction

Applied Correction Factors

Morris & Williams (1994) 0.89 (Elev.>177m) 1.00 (Elev.<177m)

(μ = 1.18 EXP(-0.08 Ip) + 0.57)

Project No. : 1-09-4135

Date : September, 2010

Terraprobe Inc.

Prepared By : HW

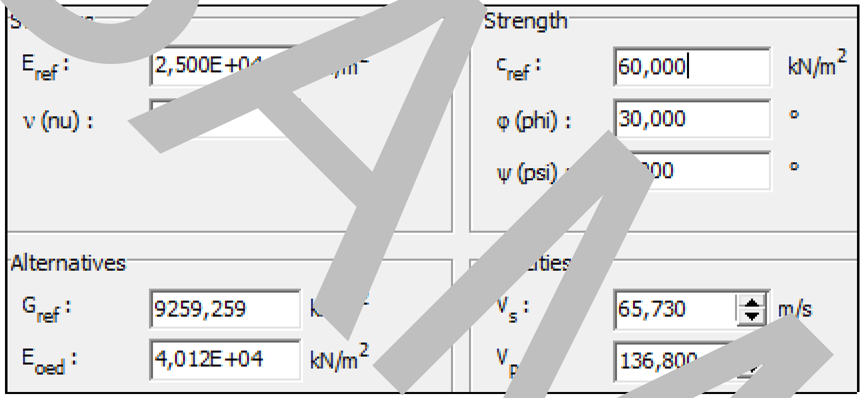
Checked By : RA

Cinder fill material with organics overlying silty clay.

Unit weight of moist soil: $\gamma_{moist} := 20.7 \frac{kN}{m^3}$

Unit weight of submerged soil: $\gamma_{subm} := 21 \frac{kN}{m^3}$

The Mohr-Coulomb plasticity criteria is used.



E_{ref} : 2,500E+0 ... /m

ν (nu) :

c_{ref} : 60,000 kN/m²

φ (phi) : 30,000 °

ψ (psi) : 00 °

G_{ref} : 9259,259 k

E_{oed} : 4,012E+04 kN/m²

V_s : 65,730 m/s

V_p : 136,800

Material properties of soil

Young's modulus: (based on plasticity index, liquid limit, plastic limit) $E_{ref} := 25000 \frac{kN}{m^2}$

We assumed no friction between soil and shoring wall: "Design Criteria" 27.pt.

Decreased Young's modulus: $E_{de} \dots ref \cdot \frac{2}{X} \dots 15250 \cdot \frac{kN}{m^2}$

Poisson coefficient (Typical average value for clay): $\nu := 0.35$

Cohesion (min. of statistical evaluation): $c_{ref} := 60 \frac{kN}{m^2}$

Angle of inner friction (based on the given passive earth pressure coefficient): $\varphi := 30°$

4.4.2. Anchor

Axial stiffness of the tendons:

$n_{app} := \max\left(n_{a_1}, n_{a_2}\right) = 2$

$EA_t := E_s \cdot n_{app} \cdot A_s$ $EA_t = 55999.888 \cdot kN$

4.4.3. Wall

Stiffness of W360x33:

Axial: $EA_s := E_s \cdot A_{360x33} = \mathbf{834000} \cdot kN$

$$\boxed{\frac{EA_s}{X} = \mathbf{417000} \cdot \frac{kN}{m}}$$

Bending: $EI_s := E_s \cdot I_{x.360x33} = \mathbf{16540} \cdot kN \cdot m^2$

$$\boxed{\frac{EI_s}{X} = \mathbf{8270} \cdot \frac{kN \cdot m^2}{m}}$$

W360x51

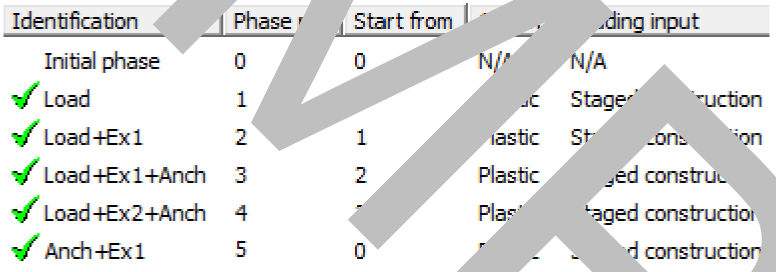
Axial: $EA_s := E_s \cdot A_{360x51} = \mathbf{12[illegible]00} \cdot kN$

$$\boxed{\frac{EA_s}{X} = \mathbf{645000} \cdot \frac{kN}{m}}$$

Bending: $EI_s := E_s \cdot I_{x.360x51} = \mathbf{[illegible]200} \cdot kN \cdot m^2$

$$\boxed{\frac{EI_s}{X} = \mathbf{14100} \cdot \frac{kN \cdot m^2}{m}}$$

4.5. Construction stages



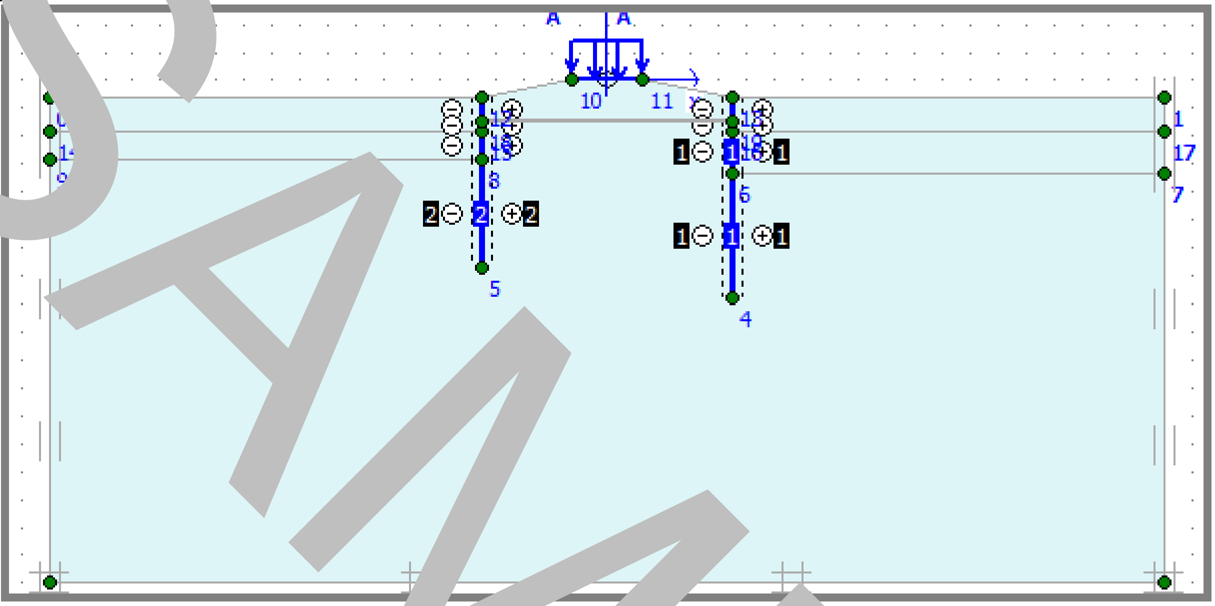
Identification	Phase no	Start from	[illegible]	Loading input
Initial phase	0	0	N/A	N/A
✓ Load	1	[illegible]	Plastic	Staged construction
✓ Load+Ex1	2	1	Plastic	Staged construction
✓ Load+Ex1+Anch	3	2	Plastic	Staged construction
✓ Load+Ex2+Anch	4	[illegible]	Plastic	Staged construction
✓ Anch+Ex1	5	0	[illegible]	Staged construction

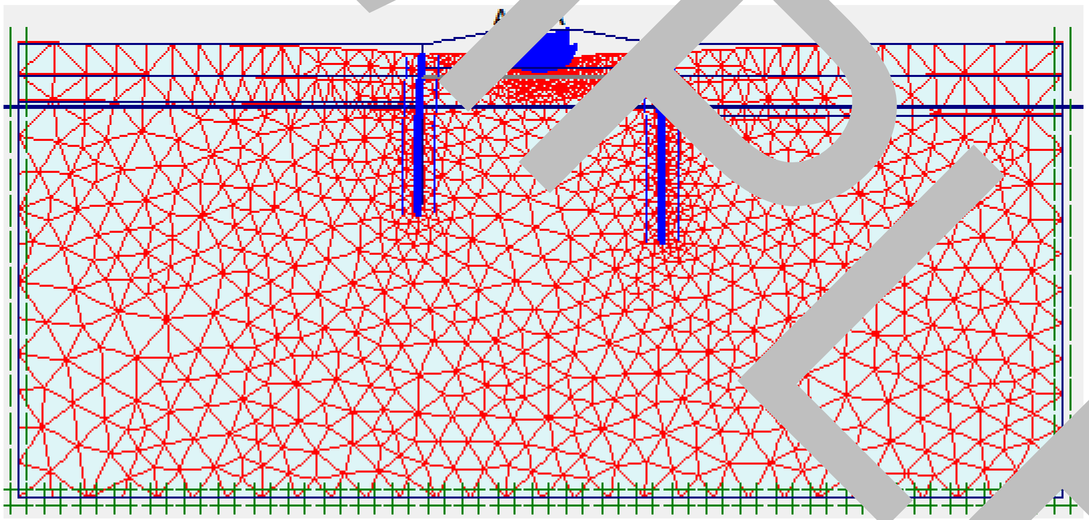
"Design Criteria"
3. 5. 14. pt.

4.5.1. Phase 1

The wall is driven in,
Live load is applied,
No excav

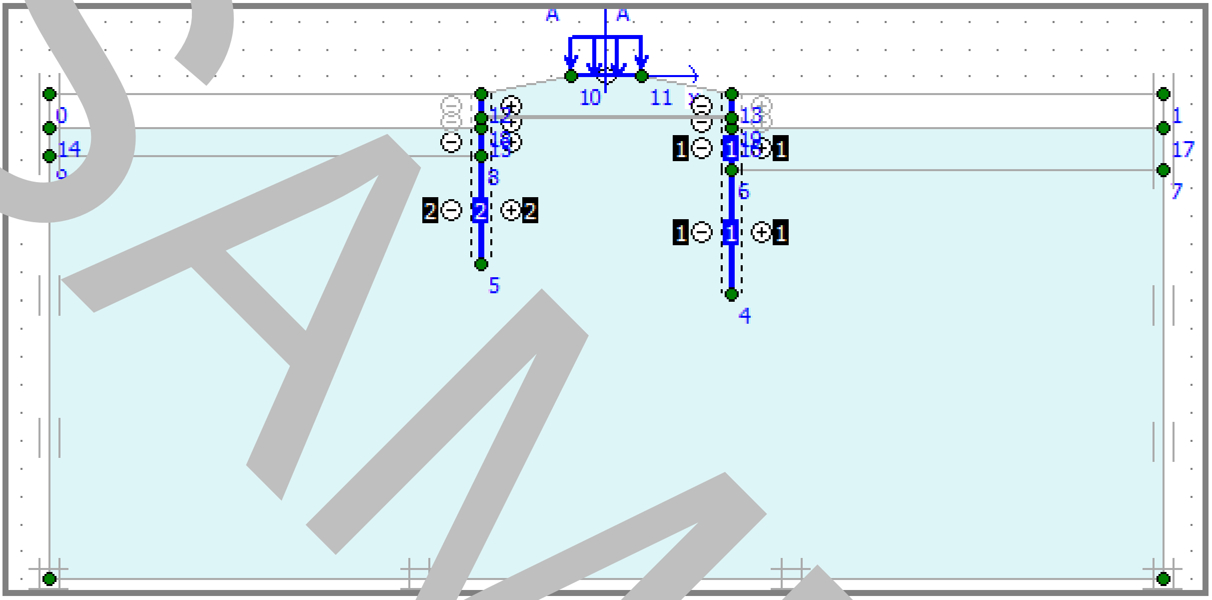
Starting m the initi ndition.

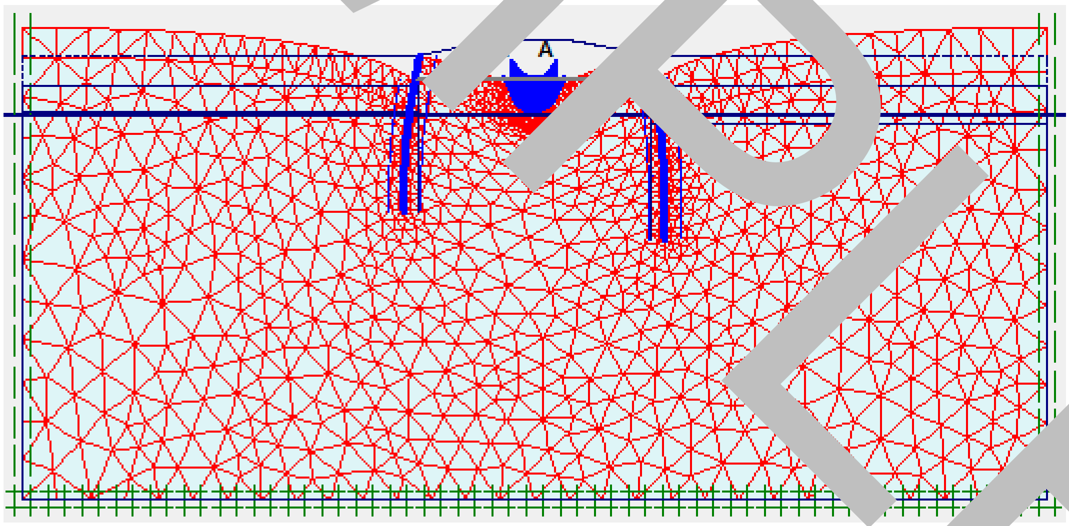




4.5.2. Phase 2

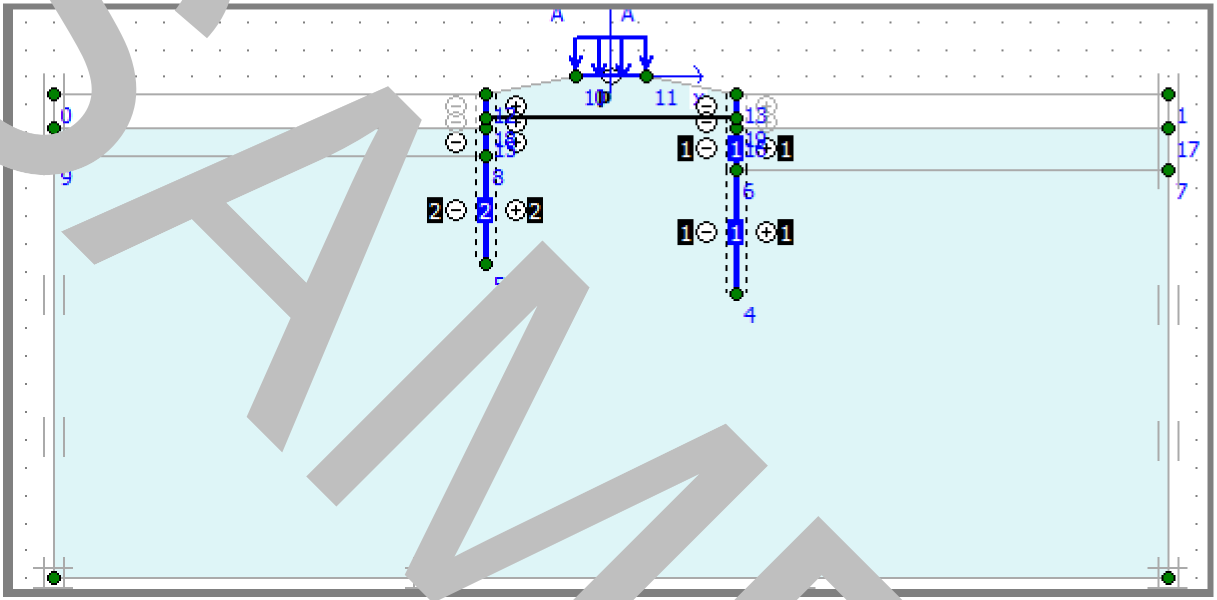
The wall is driven in,
Live load is applied,
Excavati[illegible]low anchorage level.
Starting [illegible]n Phase [illegible]

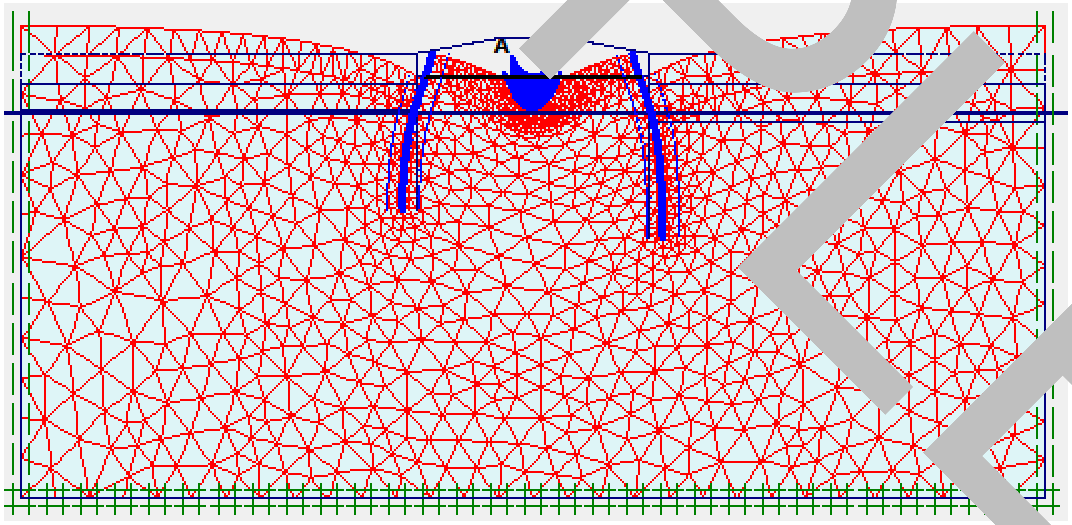




4.5.3. Phase 3

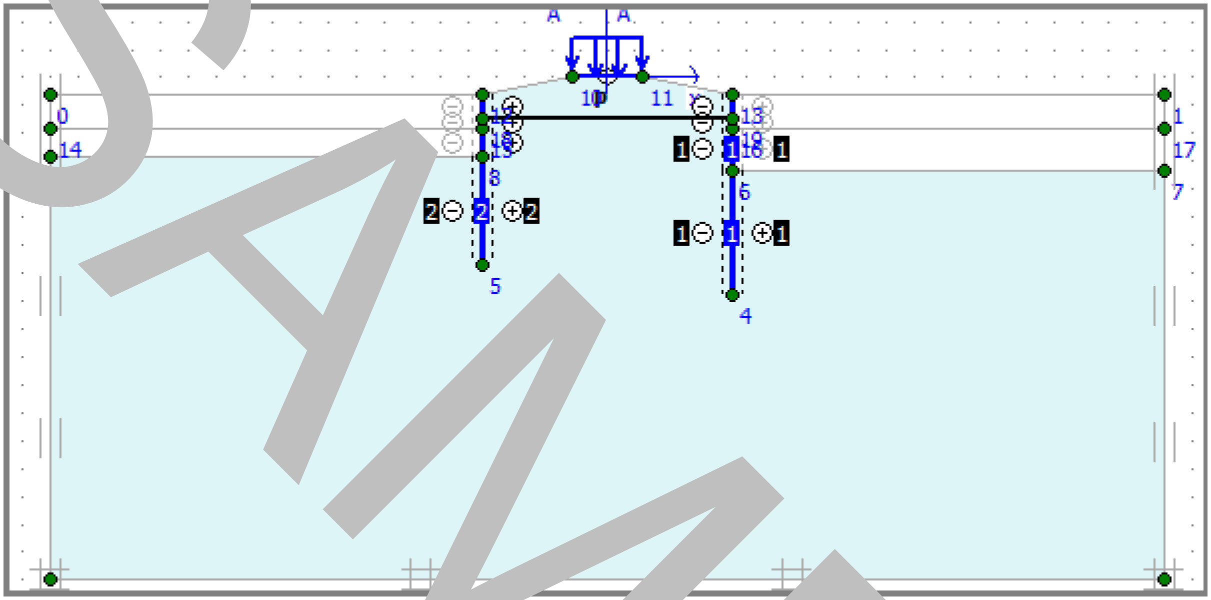
The wall is driven in,
Live load is applied,
Excavati [illegible] low anchorage level.
Anchor [illegible] prestre [illegible]
Starting [illegible] m Phase 2.

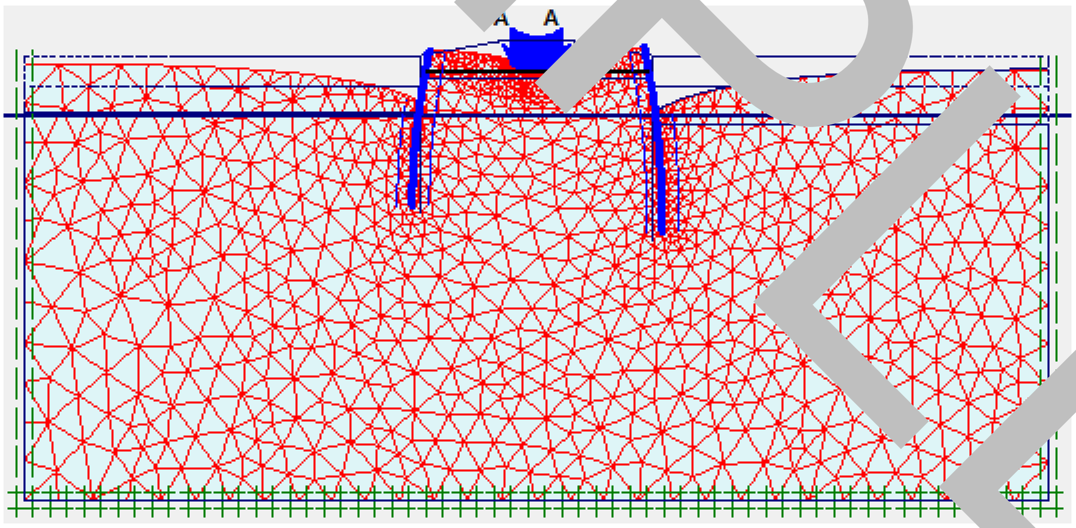




4.5.4. Phase 4

The wall is driven in,
Live load is applied.
Excavation to the final level.
Anchors are applied.
Starting from Phase 3.

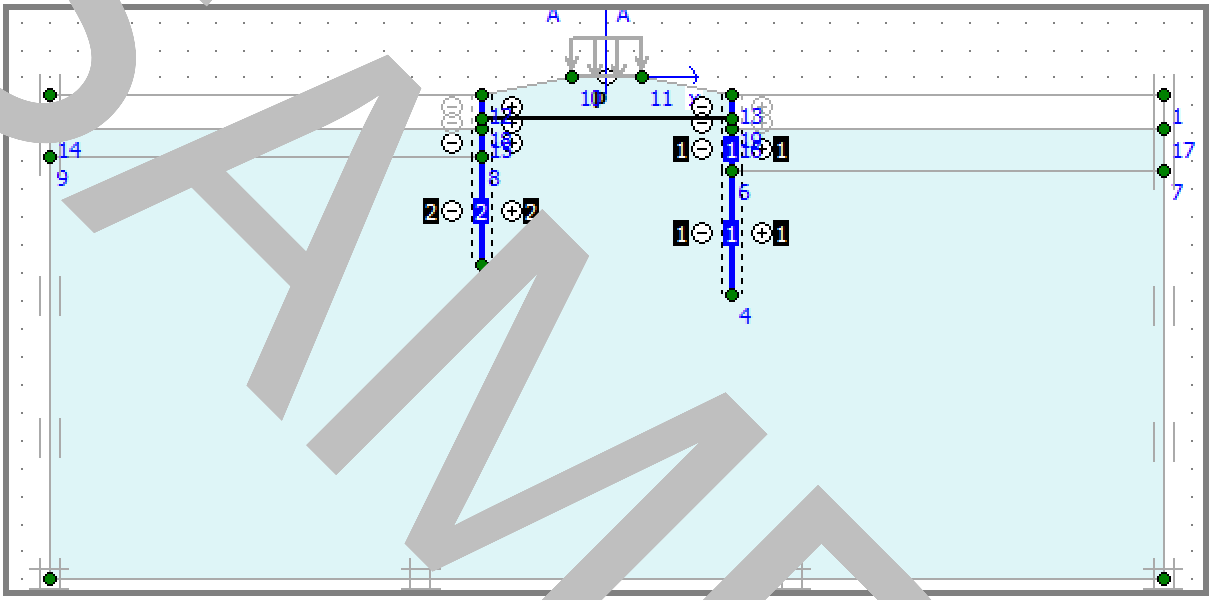


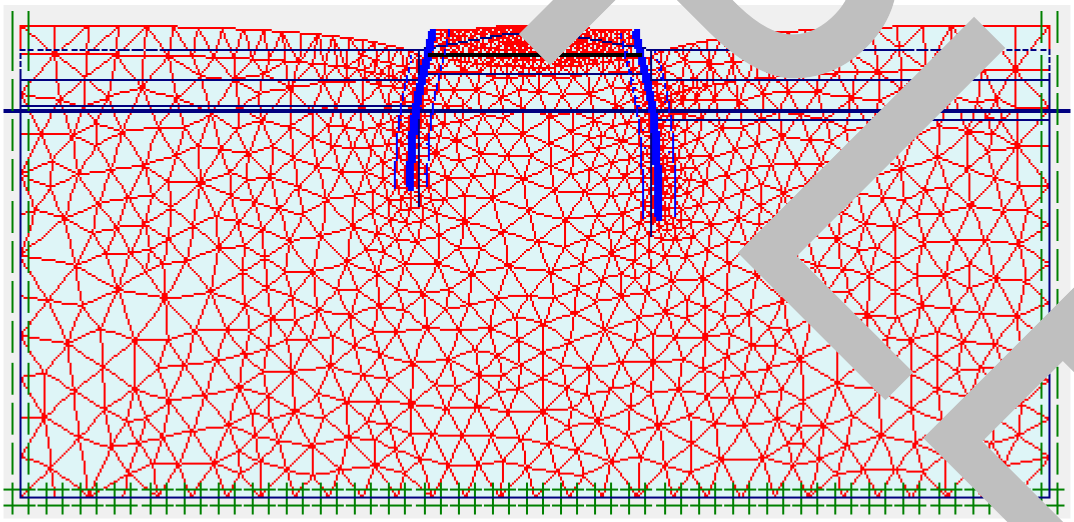


4.5.5. Phase 5

Control phase to check the soil without the load

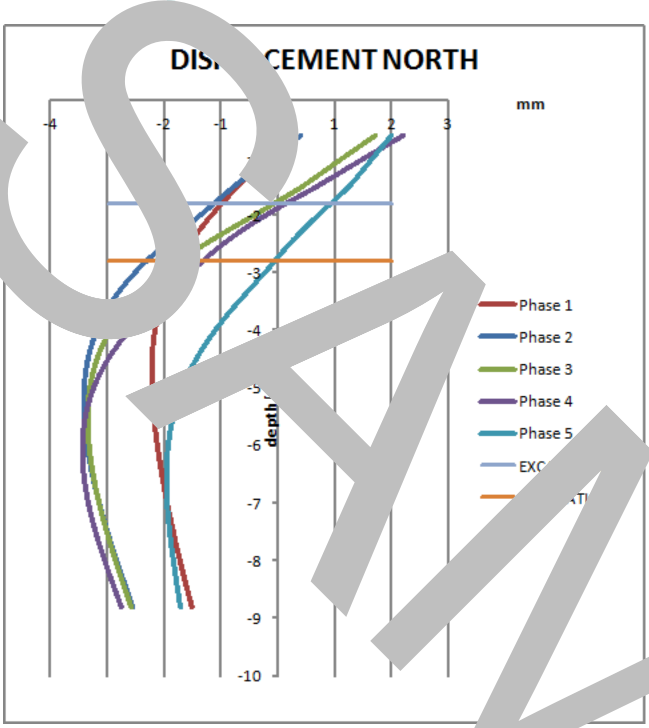
The wall is driven.
Live load is excluded.
Excavation to the final level.
Anchors are applied.
Starting from the initial phase.

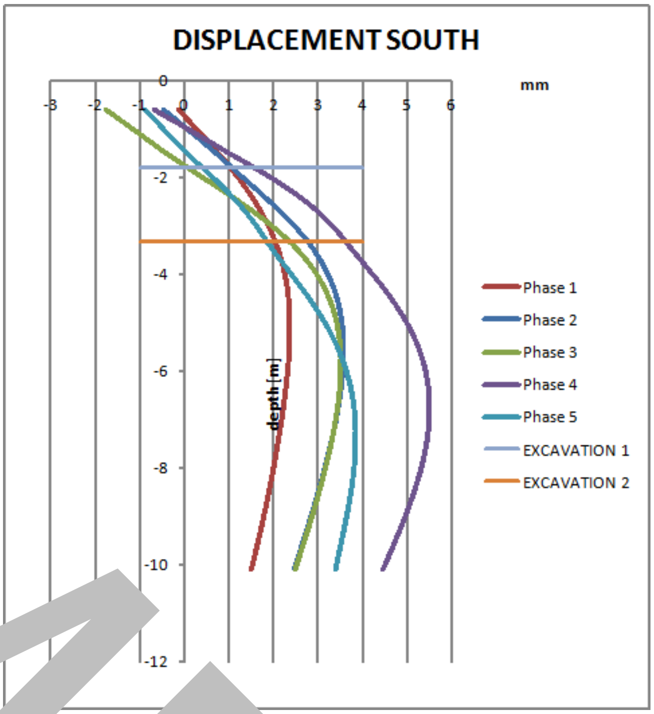




4.6. Results

4.6.1. Displacements





The lateral displacements are limited to the 0.1% of the e[illegible]ation heig[illegible] Phases [illegible]

"Design Criteria"
15. pt.

$\Delta s_{max} := 0.001 \cdot H_{d_2} = 3.33 \cdot mm$

The maximum displacement satisfies this criteria, and we ca[illegible]il the requi[illegible] of displacement (5mm) while the tiebacks are being stressed ([illegible]ase 5).

"Design Criteria"
30. pt.

4.6.2. Check of soil's plastic resistance in prestressing phase

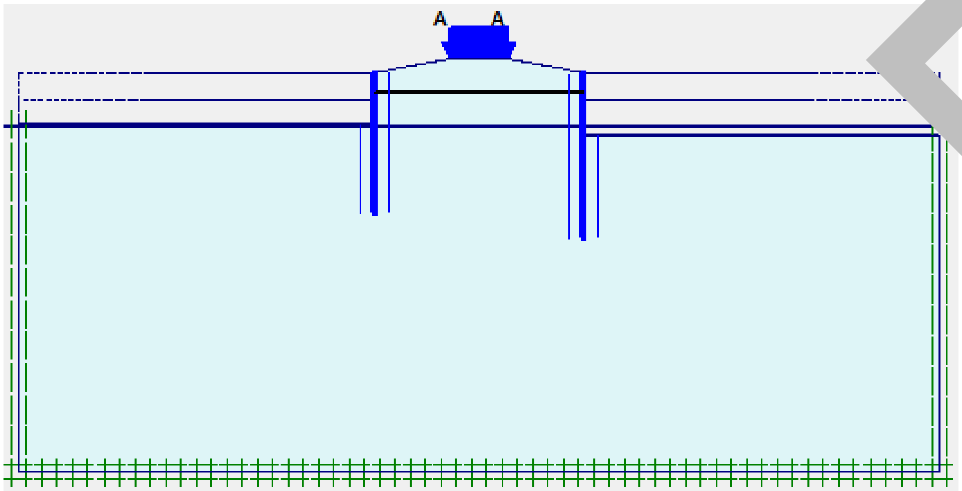


Figure shows prestressing [illegible]se.
Red squares would repre[illegible]
plastic points on the f[illegible].
No plasticity is pre[illegible]ecause o[illegible]
[illegible]estressing.

"[illegible]gn Criteri[illegible]
6. pt.

5. Summary

5.1. Soldier pile

Height [illegible] the soldier [illegible]: $L_{app.w} = \begin{pmatrix} 8.8 \\ 10.1 \end{pmatrix} m$

Depth of the embedment: $D = \begin{pmatrix} 5.99 \\ 6.77 \end{pmatrix} m$

Section: South side: W360x51

North side: W360x33

5.2. Tie-back[illegible]

Number of strands in a tendon: $n_{app} = 2$

Spacing: $s_{app.a} = 2\ m$

Design load of a tendon: $A_p = 260 \cdot kN$

Prestressing force of the tendons: $F_{app} = 51.915 \cdot kN$

Lock-off load of a tendon:
(ref.: "Construction Procedure") $F_{pl} :=$ [illegible] $\cdot F_{app}$ $F_{pl} =$ [illegible] $\cdot kN$

Proof load of a tendon:
(ref.: "Construction Procedure") $F_{pp} := 1.33 \cdot F_{app}$ $F_{pp} = 69 \cdot kN$

Note: The lock-off load and the proof load are calculated based [illegible] the applied prestressing force rather than the design load. This is the result of the restriction for the displacement criter[illegible]