This document presents the temporary works design and structural verification for the propping of the 4.5 m tall precast concrete wall panels forming part of the 430773 Works Return Tank - Knapp Mill project. The calculations demonstrate the stability of the panels during installation and throughout the execution phase, ensuring that all temporary conditions are safely managed in accordance with the relevant Eurocodes and the UK National Annex.

The temporary support arrangement considered in this design consists of:

1 Titan RSK 6 props installed at high level

| A | 07.01.2026 | Initial issue | LK |

PRECAST PANEL PROPPING CALCULATION

Constants

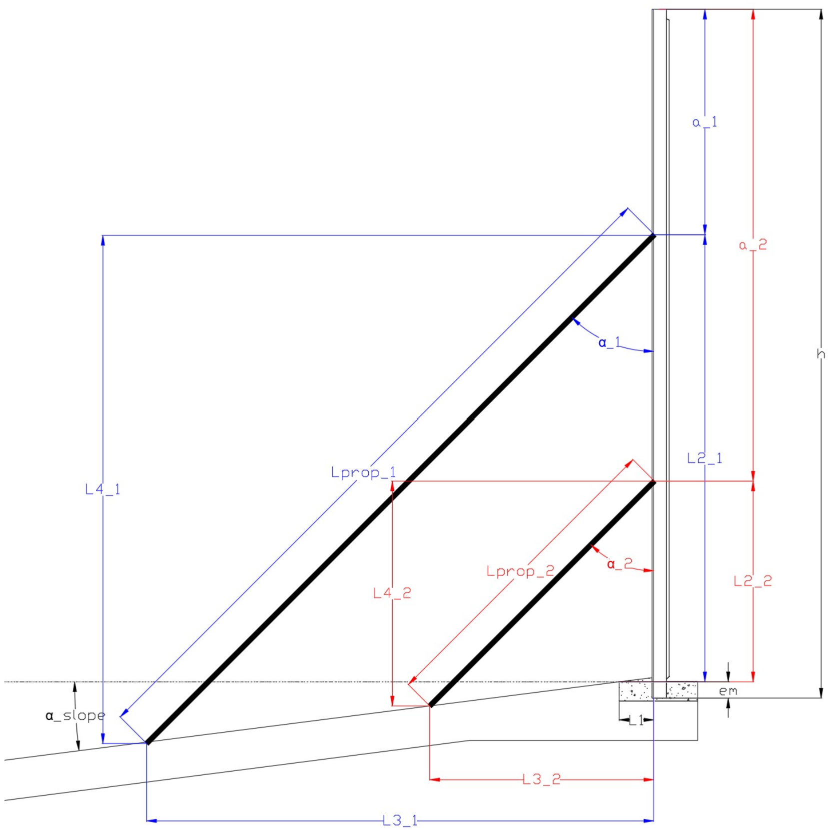
Global Variables:

Gravitational acceleration $g=9.807\ m \cdot s^{-2}$

Air density $\rho_{air} := 1.226\ \dfrac{kg}{m^3}$ (UK parameter)

Wall panel weight in kg : $W := 4280 \cdot kg$

$W_N := W \cdot g$ $W_N = 41.97\ kN$



Geometry:

Height of wall panel: $h := 4.5 \cdot m$

Tank regular wall panel width: $b := 2.11 \cdot m$

Embedment below the top of base: $em := -0.3 \cdot m$

Mounting height of prop(s) (distance from top of wall panel): $a := [1.5]^{T}\ m$

Number of props in wall panel at each level: $n := [1]^{T}$

Angle of prop(s) in relation to the vertical wall panel: $\alpha := [45]^{T}\ deg$

Average width of inside ring beam: $L1 := 0.450\ m$

Slope of base: $\alpha_{SL} := 0\ deg$

$i := 0 .. \text{length}(a) - 1$

Lengths of propping:

$$L2_i := h - a_i - em = [3.3]\ m$$

$$L3_i := -\left(\frac{\left(L1 \cdot \tan\left(\alpha_{SL}\right) - L2_i\right)}{\tan\left(90\ deg - \alpha_i\right) - \tan\left(\alpha_{SL}\right)}\right) = [3.3]\ m$$

$$L4_i := \tan\left(90\ deg - \alpha_i\right) \cdot L3_i = [3.3]\ m$$

$$Lprop_i := \sqrt{L3_i^{\,2} + L4_i^{\,2}} = [4.67]\ m$$

Wind load calculation (In Accordance with Eurocode EN1991-1-4)

Fundamental value of the basic wind speed $v_{b.0} := 23.7 \ \frac{m}{s}$

Directional factor $c_{dir} := 1$

Season factor $c_{season} := 1$

(Can be smaller depending on season - See relevant NA)

Return-period adjustment (EN 1991-1-6 Table 3.1)

Probability factor (EN 1991-1-4 Clause 4.2(2)P — Note 5 + Expression (4.2))

≤ 3 days → 2-year return period

$p := \frac{1}{2} = 0.5$ $\quad K_{note5} := 0.2$ $\quad n_{note5} := 0.5$

$$c_{prop} := \left| \frac{1 - K_{note5} \cdot \ln(-\ln(1-p))}{1 - K_{note5} \cdot \ln(-\ln(0.98))} \right|^{n_{note5}} = 0.776$$

Basic wind speed $v_b := c_{dir} \cdot c_{season} \cdot c_{prop} \cdot v_{b.0} = 18.401 \ \frac{m}{s}$

Reference height $z := h$ (Wall Height above terrain)

Terrain category $\begin{bmatrix} z_0 \\ z_{min} \end{bmatrix} :=$ Terrain category: II

$z_{max} := 200\ m$

$z_{0.II} := 0.05\ m$

Terrain factor $k_r := 0.19 \cdot \left(\frac{z_0}{z_{0.II}}\right)^{0.07} = 0.19$

Roughness factor $c_r(z) := \text{if}\left(z \le z_{min}, k_r \cdot \ln\left(\frac{z_{min}}{z_0}\right), k_r \cdot \ln\left(\frac{z}{z_0}\right)\right)$ $c_r(z) = 0.855$

Orography factor $c_o(z) := 1$

To calculate factor accounting for speed up of wind near slopes hills and ridges see EN1991-1-4, A.3

Mean wind velocity $v_m(z) := c_r(z) \cdot c_o(z) \cdot v_b$ $v_m(z) = 15.733\ \frac{m}{s}$

Turbulence factor $k_I := 1$

Turbulent component standard deviation $\sigma_v := k_r \cdot v_b \cdot k_I = 3.496\ \frac{m}{s}$

Turbulence intensity factor $I_v(z) := \text{if}\left(z < z_{min}, \frac{\sigma_v}{v_m(z_{min})}, \frac{\sigma_v}{v_m(z)}\right)$ $I_v(z) = 0.222$

Peak velocity pressure $q_{p1}(z) := \left(1 + 7 \cdot I_v(z)\right) \cdot \frac{1}{2} \cdot \rho_{air} \cdot v_m(z)^2$ $q_{p1}(z) = 0.388\ \frac{kN}{m^2}$

EN 12812 8.2.4.1 - Safety factor 1.5* $q_p(z)$ *0.7

$q_p(z) := 1.5\ q_{p1}(z) \cdot 0.7$

$v_p(z) := \sqrt{\left(1 + 7 \cdot I_v(z)\right)} \cdot v_m(z)$ $v_p(z) = 25.151\ \frac{m}{s}$

Structural factor $c_s c_d := 1$ (For height less than 15m)

$A_{ref} := b \cdot h = 9.495\ m^2$

$d\_b := \frac{145}{2110} = 0.069$

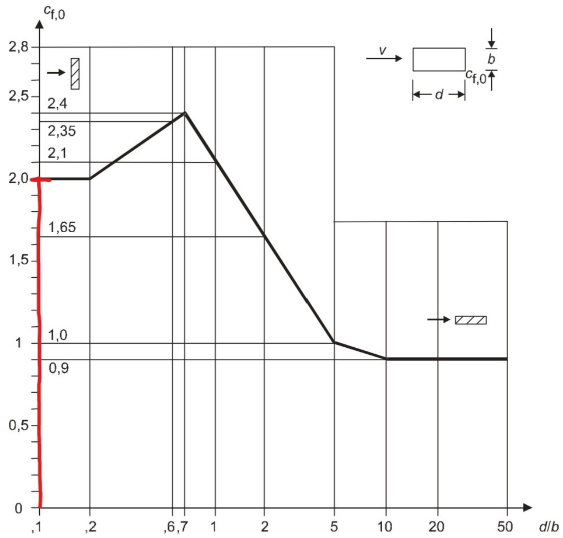


Figure 7.23 — Force coefficients $c_{f,0}$ of rectangular sections with sharp corners and without free end flow

force coefficient $c_f := 2.0$ (EN-1991-1-4 Figure 7.23)

For a fully installed tank the force coefficient according to EN-1991-1-4 is: $c_{fcyl} = 0.524$

$$F_w := c_s c_d \cdot c_f \cdot q_p(z) \cdot A_{ref} = 7.7\ kN$$

$$w := \frac{F_w}{h} + \frac{W_N \cdot 0.025}{h} = 1.95\ \frac{kN}{m}$$

(With additional 2½ % horizontal loading due to self weight of panel - BS5975 - 2019)

Total horisontal load on panel:

$$w \cdot h = 8.8\ kN$$

<u>Horisontal reactions in the prop(s) and at the bottom of the wall panel</u>:

$$R_H := \begin{array}{l} \text{if length}(a) = 1 \\ \quad rh1 \leftarrow \dfrac{h^2 \cdot w}{2 \cdot h - 2 \cdot a_0} \\ \quad rh3 \leftarrow w \cdot h - \dfrac{h^2 \cdot w}{2 \cdot h - 2 \cdot a_0} \\ \quad \begin{bmatrix} rh1 & rh3 \end{bmatrix}^T \\ \text{else if length}(a) = 2 \\ \quad rh2 \leftarrow \dfrac{w \cdot \left(\left(2 \cdot \left(h - a_1 \right) + 2 \cdot \left(h - a_0 \right) \right) \cdot a_0^{\,2} - \left(h - a_0 \right)^3 \right)}{8 \cdot \left(h - a_1 \right) \cdot \left(h - a_1 - \left(h - a_0 \right) \right)} + \dfrac{\left(h - a_0 \right) \cdot w}{8} \\ \quad rh1 \leftarrow \dfrac{w \cdot \left(\left(a_0 - h \right) \cdot a_1^{\,2} + \left(a_0^{\,2} - 3 \cdot h^2 \right) \cdot a_1 + a_0^{\,3} + h^3 \right)}{8 \cdot \left(a_0 - h \right) \cdot \left(a_1 - a_0 \right)} \\ \quad rh3 \leftarrow w \cdot h - rh1 - rh2 \\ \quad \begin{bmatrix} rh1 & rh2 & rh3 \end{bmatrix}^T \end{array}$$

$$R_H = \begin{bmatrix} 6.59 \\ 2.2 \end{bmatrix} \, kN$$

Control of force equilibrium:

$\sum R_H = 8.78 \; kN$ $\qquad w \cdot h = 8.78 \; kN$

Loads in individual prop(s)
(positive in compression)

$$F_i := \frac{R_{H_i}}{\sin\left(\alpha_i\right)} \cdot \frac{1}{n_i}$$

$$F = \begin{bmatrix} 9.31 \end{bmatrix} \; kN$$

Total vertical reactions from the prop(s):

$$R_{V_i} := F_i \cdot n_i \cdot \cos\left(\alpha_i\right)$$

This

$$R_{V_i} = \begin{bmatrix} 6.59 \end{bmatrix} \; kN$$

Uplift from wind load

If vertical reactions in props (RV) are higher than the self-weight (WN), extra fixing to the base is necessary. Simple equilibrium between the weight and the upwards prop reactions.

$$W_N - \sum R_V = 35.387\ kN$$

$$\text{if}\left(W_N > \sum R_V, \text{“OK”}, \text{“!!”}\right) = \text{“OK”}$$

Friction against bottom of wall panel:

The horisontal reaction at the base of the wall should be lower than the friction force, based on the panel self-weight against the sub-base:

Reaction to be carried by friction:

$$R_a := \left(W_N - \sum R_V\right) \cdot fric = 5.308\ kN$$

Coeff. of friction: $fric := 0.15$

$$\text{if}\left(R_a > R_{H_{\text{last}(R_H)}}, \text{“OK”}, \text{“!!”}\right) = \text{“OK”}$$

Prop selection :

Type order ref.	TITAN RS no. 2	TITAN RS no. 3	TITAN RSK no. 1	TITAN RSK no. 3	TITAN RSK no. 4	TITAN RSK no. 6	TITAN RSK no. 8
adjustment m ft	1,70 - 2,90 5'-7" – 9'-6"	2,10 - 3,60 6'-11" – 11'-10"	0,90 - 1,50 3' – 4'-11"	1,80 - 3,20 5'-11" – 10'-6"	2,60 - 4,00 8'-6" – 13'-1"	4,60 - 6,00 15'-1" – 19'-8"	6,20 - 7,60 20'-4" – 24'-11"
perm. axial load on compression* kN lbs	37 - 18 8300 - 4000	24 - 8 5400 - 1800	40 9000	40 - 29,2 - 15,4 9000 - 6600 - 3500	38,8 - 23,3 -12,8 8700 - 5200 - 2900	30,5 - 18,4 - 9,9 6900 - 4100 - 2200	40 - 20,1 - 9,1 9000 - 4500 - 2000
on tension* kN lbs.	25 5600	25 5600	40 9000	40 9000	40 9000	40 9000	40 9000
weight approx. kg (lbs)	14 (31)	17 (37)	11 (24)	19 (42)	23 (51)	38 (84)	72 (159)
outer tube Ø mm (in)	57 (2 1/4")	57 (2 1/4")	70 (2 3/4")	70 (2 3/4")	70 (2 3/4")	83 (3 1/4")	108 (4 1/4")

Prop(s) at level 1:

The length of prop(s) 1: $Lprop_0 = 4.67\ m$

Using:

Prop at level 1: Titan RSK 6

Permissible axial load on compression : $F_{p1} := 29.29\ kN$ (From Linear interpolation)
(In tension: 40 kN capacity)

Load in prop(s) 1: $F_0 = 9.31\ kN$

Check: $\text{if}\left(F_{p1} > F_0, \text{“OK”}, \text{“!!”}\right) = \text{“OK”}$

Prop fixing in the base:

Design resistance

Anchor size		**6**		**8**			**10**			**14**		
Type	**HUS3-**	**H,C,A, I, P**	**H,C,A, I,**	**H,C,HF**			**H,C,HF**			**H,HF**		
Non-cracked concrete												
Tension N_{Rd}	[kN]	3,9	5,0	6,0	8,0	10,7	8,0	13,3	18,0	11,4	17,7	28,8
Shear V_{Rd}	[kN]	*5,4*	*8,3*	8,3	*12,7*	*14,7*	8,8	*20,0*	*22,7*	22,7	*35,4*	*41,3*
Cracked concrete												
Tension N_{Rd}	[kN]	1,4	3,3	4,0	6,0	8,0	6,0	10,0	12,6	7,9	12,4	20,0
Shear V_{Rd}	[kN]	*3,8*	*8,3*	5,8	*12,7*	*14,7*	6,2	*20,0*	*22,7*	15,9	*24,8*	*40,4*

Recommended loads[a)]

Anchor size		**6**		**8**			**10**			**14**		
Type	**HUS3-**	**H,C,A, I, P**	**H,C,A, I,I-flex**	**H,C,HF**			**H,C,HF**			**H,HF**		
Non-cracked concrete												
Tension N_{Rec}	[kN]	2,8	3,6	4,3	5,7	7,6	5,7	9,5	12,9	8,1	12,6	20,6
Shear V_{Rec}	[kN]	3,9	*5,9*	5,9	9,1	10,5	6,3	14,3	16,2	16,2	25,3	29,5
Cracked concrete												
Tension N_{Rec}	[kN]	1,0	2,4	2,9	4,3	5,7	4,3	7,1	9,0	5,6	8,9	14,3
Shear V_{Rec}	[kN]	2,7	*5,9*	4,1	9,1	10,5	4,4	14,3	16,2	11,4	17,7	28,9

a) With overall partial safety factor for action γ = 1,4. The partial safety factors for action depend on the type of loading and shall be taken from national regulations.

Using Hilti Screw anchor HUS3-H14x100

Design values (steel failure)

Tension Capacity: $N_{Rec} := 14.3 \cdot kN$

Shear Capacity: $V_{Rec} := 28.9 \cdot kN$

*Pull out failure not decisive

Pry-out failure design capacity:

$h_{ef} := 66.3$ (Effective anchorage)

$f_{ck.cube.base} := 30$ (Base concrete strength C30/37)

$$N_{Rk.c} := N \cdot 7.2 \cdot \sqrt{f_{ck.cube.base}} \cdot h_{ef}^{\ 1.5} = 21.289\ kN$$

$$V_{Rd.cp} := \frac{2 \cdot N_{Rk.c}}{1.5} = 28.386\ kN$$ (According to ETAG001)

$$F_{sd.min} := min\left(V_{Rec}, V_{Rd.cp}\right) = 28.386\ kN$$

Loads:

$$V_{b_i} := F_i \cdot \cos\left(90\ deg - \alpha_i - \alpha_{SL}\right) = [6.586]\ kN \qquad N_{b_i} := F_i \cdot \sin\left(90\ deg - \alpha_i - \alpha_{SL}\right) = [6.586]\ kN$$

Utilization, base Prop(s):

$$U_i := \sqrt{\left(\frac{V_{b_i}}{F_{sd.min}}\right)^{1.5} + \left(\frac{N_{b_i}}{N_{Rec}}\right)^{1.5}} \quad U = [0.65]$$

$$\textbf{if}\left(U_i < 1, \text{“OK”}, \text{“!!”}\right) = [\text{“OK”}]$$

Fixation of bottom wall if relevant (shear only) - one fixation each panel:

$$U_{bw} := \frac{R_{H_{\text{last}\,(R_H)}}}{F_{sd.min}} = 0.077$$

$$\textbf{if}\left(U_{bw} < 1, \text{“OK”}, \text{“!!”}\right) = \text{“OK”}$$

Connection loading of prop(s) to precast panel:

Normal force (tension):

$$N_{sd_i} := R_{H_i} \cdot \frac{1}{n_i} = [6.59]\ kN$$

Shear force:

$$V_{sd_i} := F_i \cdot \cos\left(\alpha_i\right) = [6.59]\ kN$$

Capacity of embedded anchor:

$N_{rd} := 17\ kN \cdot 1.18 \quad V_{rd} := 11\ kN \cdot 1.18$

Using: SSRCP1675 (CERTEX)

table 34
Solid Rod Fixing Socket with Crossbar

Product Code	Thread Dia. (M)	L (mm)	D (mm)	g (mm)	L1 (mm)	a (mm)	e (mm)	f (mm)	Tension Load (kN)	Shear Load (kN)
SSRCP1060	10	60	16	6	50	10.0	35	40	6.0	4.0
SSRCP1250	12	50	20	10	75	12.0	22	28	9.0	5.6
SSRCP1275	12	75	20	10	75	12.0	40	46	9.0	5.6
SSRCP1675	16	75	22	10	75	14.5	40	45	17.0	11.0
SSRCP16100	16	100	22	10	75	14.5	65	70	17.0	11.0
SSRCP2075	20	75	28	12	90	18.5	35	40	23.0	17.0
SSRCP20100	20	100	28	12	90	18.5	60	65	23.0	17.0
SSRCP24100	24	100	35	16	100	20.0	55	60	30.0	24.0

Panel anchor utilization:

$$\frac{N_{sd_i}}{N_{rd}} + \frac{V_{sd_i}}{V_{rd}} = [0.836]$$

$$\text{if}\left(\frac{N_{sd_i}}{N_{rd}} + \frac{V_{sd_i}}{V_{rd}} \le 1.2, \text{“OK”}, \text{“!!”}\right) = [\text{“OK”}]$$

According to CERTEX Technical documentation