BRIDGES POUND CONSULTING ENGINEERS	Location				Job No	
	UCL for Mace				M1993	
	Basement Temporary Works Design				Sheet No 0	
	By		Checked		Approved	
	SMH	31/03/16	RLN	Mar 16	SMH	Apr 16

Temporary Works Design

For

Construction of 2 level Basement

at

UCL

For

MACE

APRIL 2016

Introduction

Mace have engaged Bridges Pound Ltd to undertake the design of Temporary works for their project at UCL.

The Scheme comprises the construction of a new Student Centre of reinforced concrete construction. The new building has two storey basement contained within a secant piled retaining wall.

The extent of the Temporary works has been identified by mace and comprises;

The temporary propping of the Capping beam to the Secant Piled wall during excavation and construction of the basement.

Works to stabilise and confirm the adequacy of the existing pavement vaults.

The stability of the existing buildings to the East, South and West of the site during the excavation works associated with the development and whether any additional propping is required.

The construction of the “link tunnel” to the rear of the Bloomsbury Theatre.

Stability of the bottom of the access ramp during construction.

Design Information

Access has been provided to the Conject site where all the project drawings are maintained so a full set of Architects and Structural Engineers drawings has been made available.

Basement Propping

The Secant Piled wall has been designed by Keltbray Piling using the design loads and forces provided by the Structural Engineer, Curtins.

At the time of undertaking the initial assessment the final design had not been made available and a preliminary design had been provided. This identified a force in the prop at Capping beam level of 122 kN/m, which was been used in the subsequent design.

Subsequent to this the Final Pile Design has been provided and reference should be made to Keltbray Piling document P230-Des-01 Rev C00. This has identified reduced prop loads in all elevations making the original design safe. From these calculations the prop Loads are as follows;

DS1 – 26 Gordon Street Elevation (South) – 106 kN/m
DS2 – ACBE Elevation (West) – 34 kN/m
DS3 – Bloomsbury Theatre (North) – 80 kN/m
DS4 – Gordon Street (East) – 110 kN/m

All these loads are un-factored loads. A revised analysis has been undertaken showing that the Moments, shears and prop forces have been reduced which can be seen in Appendix A.

The final Prop loads are as noted below. The props will be pre-loaded to eliminate axial shortening to a value of 80% of the predicted load as shown.

Prop Ref	Design Axial Load	Pre-Load
P1	1689 kN	1351 kN
P2	1414kN	1130 kN
P3	1420 kN	1150 kN
P4	1208 kN	970 kN

The piled wall is designed to span from the basement 2 slab level to capping beam level with only one level of propping, The propping is to be provided at Capping beam level and has been designed so as to minimise impact on the construction of the basement and ground floor structures and to minimise the impact on the vertical elements.

The above requirement has meant that corner braces only are to be provided with the capping beam acting as a waler between props.

The capping beam is designed as a continuous beam with rotational restraint at corners, which will be provided by the continuity of reinforcement in these locations.

The locations of the props is not ideal but has been chosen to minimise disruption to the basement formation.

The basement is almost square in shape and the temporary propping is shown on drawings 031-51-1000-DR-Y-00001 – B1 to 00005 B1. The capping beam design is as follows.

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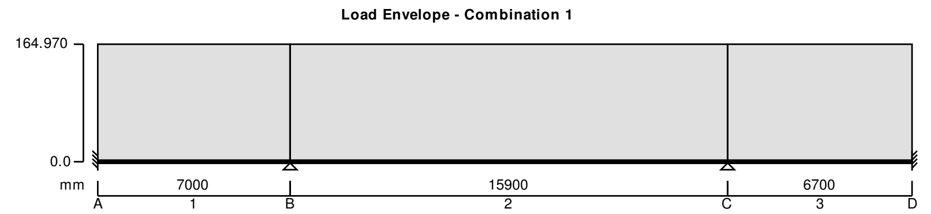
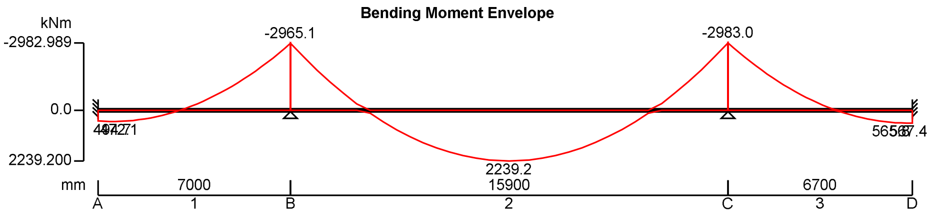
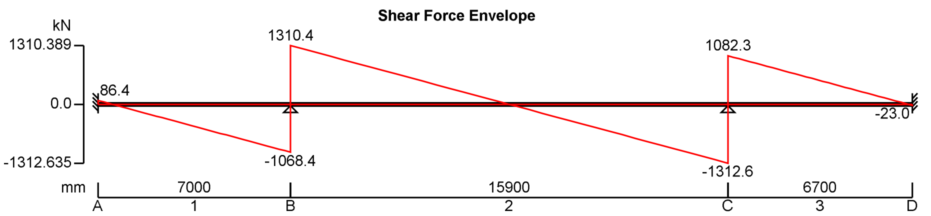
<u>RC BEAM ANALYSIS & DESIGN (EN1992-1)</u>

In accordance with UK national annex

TEDDS calculation version 2.1.15







Support conditions

Support A	Vertically restrained
	Rotationally restrained
Support B	Vertically restrained
	Rotationally free
Support C	Vertically restrained
	Rotationally free
Support D	Vertically restrained
	Rotationally restrained

Applied loading

Permanent self weight of beam × 0

Permanent full UDL 122.2 kN/m

Load combinations

Load combination 1	Support A	Permanent × 1.35
		Variable × 1.50
	Span 1	Permanent × 1.35
		Variable × 1.50
	Support B	Permanent × 1.35
		Variable × 1.50

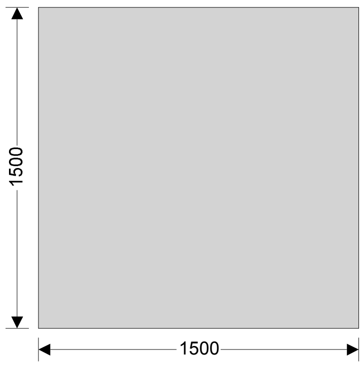
	Span 2	Permanent × 1.35
		Variable × 1.50
	Support C	Permanent × 1.35
		Variable × 1.50
	Span 3	Permanent × 1.35
		Variable × 1.50
	Support D	Permanent × 1.35
		Variable × 1.50

Analysis results

Maximum moment support A;	M_{A_max} = **472** kNm;	M_{A_red} = **472** kNm;
Maximum moment span 1 at 523 mm;	M_{s1_max} = **495** kNm;	M_{s1_red} = **495** kNm;
Maximum moment support B;	M_{B_max} = **-2965** kNm;	M_{B_red} = **-2965** kNm;
Maximum moment span 2 at 7943 mm;	M_{s2_max} = **2239** kNm;	M_{s2_red} = **2239** kNm;
Maximum moment support C;	M_{C_max} = **-2983** kNm;	M_{C_red} = **-2983** kNm;
Maximum moment span 3 at 6561 mm;	M_{s3_max} = **567** kNm;	M_{s3_red} = **567** kNm;
Maximum moment support D;	M_{D_max} = **0** kNm;	M_{D_red} = **0** kNm;
Maximum shear support A;	V_{A_max} = **86** kN;	V_{A_red} = **86** kN
Maximum shear support A span 1 at 1443 mm;	$V_{A\_s1\_max}$ = **-153** kN;	$V_{A\_s1\_red}$ = **-153** kN
Maximum shear support B;	V_{B_max} = **1310** kN;	V_{B_red} = **1310** kN
Maximum shear support B span 1 at 5573 mm;	$V_{B\_s1\_max}$ = **-829** kN;	$V_{B\_s1\_red}$ = **-829** kN
Maximum shear support B span 2 at 1428 mm;	$V_{B\_s2\_max}$ = **1071** kN;	$V_{B\_s2\_red}$ = **1071** kN
Maximum shear support C;	V_{C_max} = **-1313** kN;	V_{C_red} = **-1313** kN
Maximum shear support C span 2 at 14473 mm;	$V_{C\_s2\_max}$ = **-1073** kN;	$V_{C\_s2\_red}$ = **-1073** kN
Maximum shear support C span 3 at 1428 mm;	$V_{C\_s3\_max}$ = **843** kN;	$V_{C\_s3\_red}$ = **843** kN
Maximum shear support D;	V_{D_max} = **-23** kN;	V_{D_red} = **-23** kN
Maximum shear support D span 3 at 5273 mm;	$V_{D\_s3\_max}$ = **216** kN;	$V_{D\_s3\_red}$ = **216** kN
Maximum reaction at support A;	R_A = **86** kN	
Unfactored permanent load reaction at support A;	$R_{A\_Permanent}$ = **64** kN	
Maximum reaction at support B;	R_B = **2379** kN	
Unfactored permanent load reaction at support B;	$R_{B\_Permanent}$ = **1762** kN	
Maximum reaction at support C;	R_C = **2395** kN	
Unfactored permanent load reaction at support C;	$R_{C\_Permanent}$ = **1774** kN	
Maximum reaction at support D;	R_D = **23** kN	
Unfactored permanent load reaction at support D;	$R_{D\_Permanent}$ = **17** kN	

Rectangular section details

Section width;	b = **1500** mm
Section depth;	h = **1500** mm



Concrete details (Table 3.1 - Strength and deformation characteristics for concrete)

Concrete strength class; **C40/50**

Characteristic compressive cylinder strength; f_{ck} = **40** N/mm^2

Characteristic compressive cube strength; $f_{ck,cube}$ = **50** N/mm^2

Mean value of compressive cylinder strength; $f_{cm} = f_{ck} + 8$ N/mm^2 = **48** N/mm^2

Mean value of axial tensile strength; $f_{ctm} = 0.3$ N/mm$^2 \times (f_{ck}/$ 1 N/mm$^2)^{2/3}$ = **3.5** N/mm^2

Secant modulus of elasticity of concrete; $E_{cm} = 22$ kN/mm$^2 \times [f_{cm}/10$ N/mm$^2]^{0.3}$ = **35220** N/mm^2

Partial factor for concrete (Table 2.1N); γ_C = **1.50**

Compressive strength coefficient (cl.3.1.6(1)); α_{cc} = **0.85**

Design compressive concrete strength (exp.3.15); $f_{cd} = \alpha_{cc} \times f_{ck} / \gamma_C$ = **22.7** N/mm^2

Maximum aggregate size; h_{agg} = **20** mm

Reinforcement details

Characteristic yield strength of reinforcement; f_{yk} = **500** N/mm^2

Partial factor for reinforcing steel (Table 2.1N); γ_S = **1.15**

Design yield strength of reinforcement; $f_{yd} = f_{yk} / \gamma_S$ = **435** N/mm^2

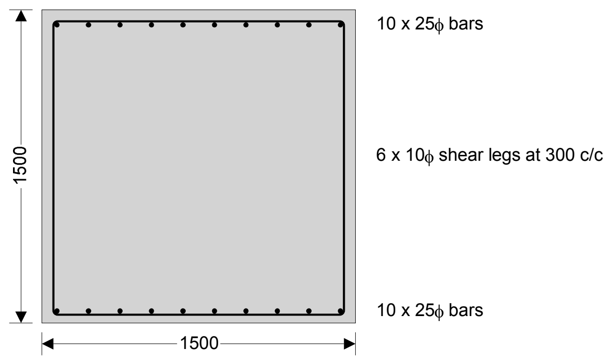
Nominal cover to reinforcement

Nominal cover to top reinforcement; c_{nom_t} = **50** mm

Nominal cover to bottom reinforcement; c_{nom_b} = **35** mm

Nominal cover to side reinforcement; c_{nom_s} = **50** mm

Support A



Rectangular section in flexure (Section 6.1)

Minimum moment factor (cl.9.2.1.2(1)); β_1 = **0.25**

Design bending moment; $M = \max(\mathrm{abs}(M_{A\_red}), \beta_1 \times \mathrm{abs}(M_{s1\_red}))$ = **472** kNm

Depth to tension reinforcement; $d = h - c_{nom\_b} - \phi_v - \phi_{bot} / 2$ = **1443** mm

Percentage redistribution; m_{rA} = **0** %

Redistribution ratio; $\delta = \min(1 - m_{rA}, 1)$ = **1.000**

$K = M / (b \times d^2 \times f_{ck})$ = **0.004**

$K' = 0.598 \times \delta - 0.181 \times \delta^2 - 0.21$ = **0.207**

K' > K - No compression reinforcement is required

Lever arm; $z = \min((d / 2) \times [1 + (1 - 3.53 \times K)^{0.5}], 0.95 \times d)$ = **1370** mm

Depth of neutral axis; $x = 2.5 \times (d - z)$ = **180** mm

Area of tension reinforcement required; $A_{s,req} = M / (f_{yd} \times z)$ = **792** mm^2

Tension reinforcement provided; 10 × 25ϕ bars

Area of tension reinforcement provided; $A_{s,prov}$ = **4909** mm^2

Minimum area of reinforcement (exp.9.1N); $A_{s,min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times b \times d$ = **3948** mm^2

Maximum area of reinforcement (cl.9.2.1.1(3)); $A_{s,max} = 0.04 \times b \times h$ = **90000** mm^2

PASS - Area of reinforcement provided is greater than area of reinforcement required

Rectangular section in shear (Section 6.2)

Design shear force at support A; $V_{Ed,max} = abs(max(V_{A\_max}, V_{A\_red})) =$ **86** kN

Angle of comp. shear strut for maximum shear; $\theta_{max} = 45$ deg

Maximum design shear force (exp.6.9); $V_{Rd,max} = b \times z \times v_1 \times f_{cd} / (cot(\theta_{max}) + tan(\theta_{max})) =$ **11741** kN

PASS - Design shear force at support is less than maximum design shear force

Design shear force span 1 at 1443 mm; $V_{Ed} = abs(min(V_{A\_s1\_max}, V_{A\_s1\_red})) =$ **153** kN

Design shear stress; $v_{Ed} = V_{Ed} / (b \times z) =$ **0.074** N/mm^2

Strength reduction factor (cl.6.2.3(3)); $v_1 = 0.6 \times [1 - f_{ck} / 250\ N/mm^2] =$ **0.504**

Compression chord coefficient (cl.6.2.3(3)); $\alpha_{cw} =$ **1.00**

Angle of concrete compression strut (cl.6.2.3);

$\theta = min(max(0.5 \times Asin[min(2 \times v_{Ed} / (\alpha_{cw} \times f_{cd} \times v_1),1)], 21.8\ deg), 45deg) =$ **21.8** deg

Area of shear reinforcement required (exp.6.13); $A_{sv,req} = v_{Ed} \times b / (f_{yd} \times cot(\theta)) =$ **103** mm^2/m

Shear reinforcement provided; $6 \times 10\phi$ legs at 300 c/c

Area of shear reinforcement provided; $A_{sv,prov} =$ **1571** mm^2/m

Minimum area of shear reinforcement (exp.9.5N); $A_{sv,min} = 0.08\ N/mm^2 \times b \times (f_{ck} / 1\ N/mm^2)^{0.5} / f_{yk} =$ **1518** mm^2/m

PASS - Area of shear reinforcement provided exceeds minimum required

Maximum longitudinal spacing (exp.9.6N); $s_{vl,max} = 0.75 \times d =$ **1082** mm

PASS - Longitudinal spacing of shear reinforcement provided is less than maximum

Crack control (Section 7.3)

Maximum crack width; $w_k =$ **0.3** mm

Design value modulus of elasticity reinf (3.2.7(4)); $E_s =$ **200000** N/mm^2

Mean value of concrete tensile strength; $f_{ct,eff} = f_{ctm} =$ **3.5** N/mm^2

Stress distribution coefficient; $k_c =$ **0.4**

Non-uniform self-equilibrating stress coefficient; $k = min(max(1 + (300\ mm - min(h, b)) \times 0.35 / 500\ mm, 0.65), 1) =$ **0.65**

Actual tension bar spacing; $s_{bar} = (b - 2 \times (c_{nom\_s} + \phi_v) - \phi_{bot}) / (N_{bot} - 1) =$ **151** mm

Maximum stress permitted (Table 7.3N); $\sigma_s =$ **280** N/mm^2

Concrete to steel modulus of elast. ratio; $\alpha_{cr} = E_s / E_{cm} =$ **5.68**

Distance of the Elastic NA from bottom of beam; $y = (b \times h^2 / 2 + A_{s,prov} \times (\alpha_{cr} - 1) \times (h - d)) / (b \times h + A_{s,prov} \times (\alpha_{cr} - 1)) =$ **743** mm

Area of concrete in the tensile zone; $A_{ct} = b \times y =$ **1114505** mm^2

Minimum area of reinforcement required (exp.7.1); $A_{sc,min} = k_c \times k \times f_{ct,eff} \times A_{ct} / \sigma_s =$ **3637** mm^2

PASS - Area of tension reinforcement provided exceeds minimum required for crack control

Quasi-permanent value of variable action; $\psi_2 =$ **0.30**

Quasi-permanent limit state moment; $M_{QP} = abs(M_{A\_c21}) + \psi_2 \times abs(M_{A\_c22}) =$ **350** kNm

Permanent load ratio; $R_{PL} = M_{QP} / M =$ **0.74**

Service stress in reinforcement; $\sigma_{sr} = f_{yd} \times A_{s,req} / A_{s,prov} \times R_{PL} =$ **52** N/mm^2

Maximum bar spacing (Tables 7.3N); $s_{bar,max} =$ **300** mm

PASS - Maximum bar spacing exceeds actual bar spacing for crack control

Minimum bar spacing

Minimum bottom bar spacing; $s_{bot,min} = (b - 2 \times c_{nom\_s} - 2 \times \phi_v - \phi_{bot}) / (N_{bot} - 1) =$ **151** mm

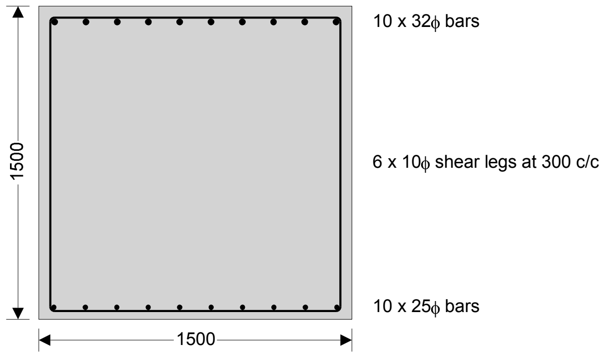
Minimum allowable bottom bar spacing; $s_{bar\_bot,min} = max(\phi_{bot}, h_{agg} + 5\ mm, 20\ mm) + \phi_{bot} =$ **50** mm

Minimum top bar spacing; $s_{top,min} = (b - 2 \times c_{nom\_s} - 2 \times \phi_v - \phi_{top}) / (N_{top} - 1) =$ **151** mm

Minimum allowable top bar spacing; $s_{bar\_top,min} = max(\phi_{top}, h_{agg} + 5\ mm, 20\ mm) + \phi_{top} =$ **50** mm

PASS - Actual bar spacing exceeds minimum allowable

Mid span 1



Rectangular section in flexure (Section 6.1) - Positive midspan moment

Design bending moment; $M = abs(M_{s1\_red})$ = **495** kNm

Depth to tension reinforcement; $d = h - c_{nom\_b} - \phi_v - \phi_{bot} / 2$ = **1443** mm

Percentage redistribution; $m_{rs1} = M_{s1\_red} / M_{s1\_max} - 1$ = **0** %

Redistribution ratio; $\delta = \min(1 - m_{rs1}, 1)$ = **1.000**

$K = M / (b \times d^2 \times f_{ck})$ = **0.004**

$K' = 0.598 \times \delta - 0.181 \times \delta^2 - 0.21$ = **0.207**

K' > K - No compression reinforcement is required

Lever arm; $z = \min((d / 2) \times [1 + (1 - 3.53 \times K)^{0.5}], 0.95 \times d)$ = **1370** mm

Depth of neutral axis; $x = 2.5 \times (d - z)$ = **180** mm

Area of tension reinforcement required; $A_{s,req} = M / (f_{yd} \times z)$ = **830** mm^2

Tension reinforcement provided; 10 × 25ϕ bars

Area of tension reinforcement provided; $A_{s,prov}$ = **4909** mm^2

Minimum area of reinforcement (exp.9.1N); $A_{s,min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times b \times d$ = **3948** mm^2

Maximum area of reinforcement (cl.9.2.1.1(3)); $A_{s,max} = 0.04 \times b \times h$ = **90000** mm^2

PASS - Area of reinforcement provided is greater than area of reinforcement required

Rectangular section in shear (Section 6.2)

Shear reinforcement provided; 6 × 10ϕ legs at 300 c/c

Area of shear reinforcement provided; $A_{sv,prov}$ = **1571** mm^2/m

Minimum area of shear reinforcement (exp.9.5N); $A_{sv,min} = 0.08\ N/mm^2 \times b \times (f_{ck} / 1\ N/mm^2)^{0.5} / f_{yk}$ = **1518** mm^2/m

PASS - Area of shear reinforcement provided exceeds minimum required

Maximum longitudinal spacing (exp.9.6N); $s_{vl,max} = 0.75 \times d$ = **1082** mm

PASS - Longitudinal spacing of shear reinforcement provided is less than maximum

Design shear resistance (assuming cot(θ) is 2.5); $V_{prov} = 2.5 \times A_{sv,prov} \times z \times f_{yd}$ = **2339.8** kN

Shear links provided valid between 0 mm and 7000 mm with tension reinforcement of 4909 mm^2

Crack control (Section 7.3)

Maximum crack width; w_k = **0.3** mm

Design value modulus of elasticity reinf (3.2.7(4)); E_s = **200000** N/mm^2

Mean value of concrete tensile strength; $f_{ct,eff} = f_{ctm}$ = **3.5** N/mm^2

Stress distribution coefficient; k_c = **0.4**

Non-uniform self-equilibrating stress coefficient; $k = \min(\max(1 + (300\ mm - \min(h, b)) \times 0.35 / 500\ mm, 0.65), 1)$ = **0.65**

Actual tension bar spacing; $s_{bar} = (b - 2 \times (c_{nom\_s} + \phi_v) - \phi_{bot}) / (N_{bot} - 1)$ = **151** mm

Maximum stress permitted (Table 7.3N); σ_s = **280** N/mm^2

Concrete to steel modulus of elast. ratio; $\alpha_{cr} = E_s / E_{cm}$ = **5.68**

Distance of the Elastic NA from bottom of beam; $y = (b \times h^2 / 2 + A_{s,prov} \times (\alpha_{cr} - 1) \times (h - d)) / (b \times h + A_{s,prov} \times (\alpha_{cr} - 1))$ = **743** mm

Area of concrete in the tensile zone; $A_{ct} = b \times y =$ **1114505** mm^2

Minimum area of reinforcement required (exp.7.1); $A_{sc,min} = k_c \times k \times f_{ct,eff} \times A_{ct} / \sigma_s =$ **3637** mm^2

PASS - Area of tension reinforcement provided exceeds minimum required for crack control

Quasi-permanent value of variable action; $\psi_2 =$ **0.30**

Quasi-permanent limit state moment; $M_{QP} = abs(M_{s1\_c21}) + \psi_2 \times abs(M_{s1\_c22}) =$ **366** kNm

Permanent load ratio; $R_{PL} = M_{QP} / M =$ **0.74**

Service stress in reinforcement; $\sigma_{sr} = f_{yd} \times A_{s,req} / A_{s,prov} \times R_{PL} =$ **54** N/mm^2

Maximum bar spacing (Tables 7.3N); $s_{bar,max} =$ **300** mm

PASS - Maximum bar spacing exceeds actual bar spacing for crack control

Minimum bar spacing

Minimum bottom bar spacing; $s_{bot,min} = (b - 2 \times c_{nom\_s} - 2 \times \phi_v - \phi_{bot}) / (N_{bot} - 1) =$ **151** mm

Minimum allowable bottom bar spacing; $s_{bar\_bot,min} = \max(\phi_{bot}, h_{agg} + 5\text{ mm}, 20\text{ mm}) + \phi_{bot} =$ **50** mm

Minimum top bar spacing; $s_{top,min} = (b - 2 \times c_{nom\_s} - 2 \times \phi_v - \phi_{top}) / (N_{top} - 1) =$ **150** mm

Minimum allowable top bar spacing; $s_{bar\_top,min} = \max(\phi_{top}, h_{agg} + 5\text{ mm}, 20\text{ mm}) + \phi_{top} =$ **64** mm

PASS - Actual bar spacing exceeds minimum allowable

Deflection control (Section 7.4)

Reference reinforcement ratio; $\rho_{m0} = (f_{ck} / 1\text{ N/mm}^2)^{0.5} / 1000 =$ **0.006**

Required tension reinforcement ratio; $\rho_m = A_{s,req} / (b \times d) =$ **0.000**

Required compression reinforcement ratio; $\rho'_m = A_{s2,req} / (b \times d) =$ **0.000**

Structural system factor (Table 7.4N); $K_b =$ **1.3**

Basic allowable span to depth ratio (7.16a); $span\_to\_depth_{basic} = K_b \times [11 + 1.5 \times (f_{ck} / 1\text{ N/mm}^2)^{0.5} \times \rho_{m0} / \rho_m + 3.2 \times (f_{ck} / 1\text{ N/mm}^2)^{0.5} \times (\rho_{m0} / \rho_m - 1)^{1.5}] =$ **1820.132**

Reinforcement factor (exp.7.17); $K_s = \min(A_{s,prov} / A_{s,req} \times 500\text{ N/mm}^2 / f_{yk}, 1.5) =$ **1.500**

Flange width factor; $F1 =$ **1.000**

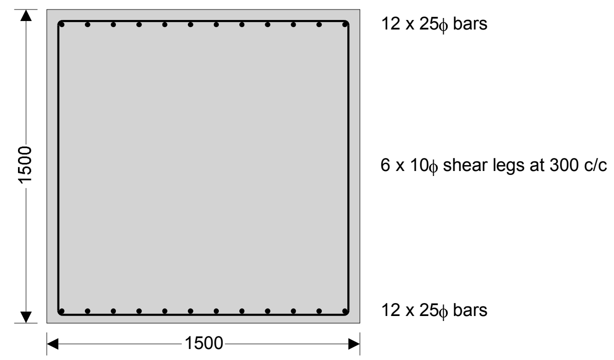
Long span supporting brittle partition factor; $F2 =$ **1.000**

Allowable span to depth ratio; $span\_to\_depth_{allow} = \min(span\_to\_depth_{basic} \times K_s \times F1 \times F2, 40 \times K_b) =$ **52.000**

Actual span to depth ratio; $span\_to\_depth_{actual} = L_{s1} / d =$ **4.853**

PASS - Actual span to depth ratio is within the allowable limit

<u>Support B</u>



Rectangular section in flexure (Section 6.1)

Design bending moment; $M = abs(M_{B\_red}) =$ **2965** kNm

Depth to tension reinforcement; $d = h - c_{nom\_t} - \phi_v - \phi_{top} / 2 =$ **1428** mm

Percentage redistribution; $m_{rB} =$ **0** %

Redistribution ratio; $\delta = \min(1 - m_{rB}, 1) =$ **1.000**

$K = M / (b \times d^2 \times f_{ck}) =$ **0.024**

$K' = 0.598 \times \delta - 0.181 \times \delta^2 - 0.21 = \mathbf{0.207}$

K' > K - No compression reinforcement is required

Lever arm; $z = \min((d / 2) \times [1 + (1 - 3.53 \times K)^{0.5}], 0.95 \times d) = \mathbf{1356}$ mm

Depth of neutral axis; $x = 2.5 \times (d - z) = \mathbf{178}$ mm

Area of tension reinforcement required; $A_{s,req} = M / (f_{yd} \times z) = \mathbf{5029}$ mm^2

Tension reinforcement provided; $12 \times 25\phi$ bars

Area of tension reinforcement provided; $A_{s,prov} = \mathbf{5890}$ mm^2

Minimum area of reinforcement (exp.9.1N); $A_{s,min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times b \times d = \mathbf{3907}$ mm^2

Maximum area of reinforcement (cl.9.2.1.1(3)); $A_{s,max} = 0.04 \times b \times h = \mathbf{90000}$ mm^2

PASS - Area of reinforcement provided is greater than area of reinforcement required

Rectangular section in shear (Section 6.2)

Design shear force at support B; $V_{Ed,max} = abs(\max(V_{B\_max}, V_{B\_red})) = \mathbf{1310}$ kN

Angle of comp. shear strut for maximum shear; $\theta_{max} = 45$ deg

Maximum design shear force (exp.6.9); $V_{Rd,max} = b \times z \times v_1 \times f_{cd} / (\cot(\theta_{max}) + \tan(\theta_{max})) = \mathbf{11619}$ kN

PASS - Design shear force at support is less than maximum design shear force

Design shear force span 1 at 5573 mm; $V_{Ed} = abs(\min(V_{B\_s1\_max}, V_{B\_s1\_red})) = \mathbf{829}$ kN

Design shear stress; $v_{Ed} = V_{Ed} / (b \times z) = \mathbf{0.408}$ N/mm^2

Strength reduction factor (cl.6.2.3(3)); $v_1 = 0.6 \times [1 - f_{ck} / 250 \text{ N/mm}^2] = \mathbf{0.504}$

Compression chord coefficient (cl.6.2.3(3)); $\alpha_{cw} = \mathbf{1.00}$

Angle of concrete compression strut (cl.6.2.3);

$\theta = \min(\max(0.5 \times \text{Asin}[\min(2 \times v_{Ed} / (\alpha_{cw} \times f_{cd} \times v_1),1)], 21.8 \text{ deg}), 45\text{deg}) = \mathbf{21.8}$ deg

Area of shear reinforcement required (exp.6.13); $A_{sv,req} = v_{Ed} \times b / (f_{yd} \times \cot(\theta)) = \mathbf{563}$ mm^2/m

Shear reinforcement provided; $6 \times 10\phi$ legs at 300 c/c

Area of shear reinforcement provided; $A_{sv,prov} = \mathbf{1571}$ mm^2/m

Minimum area of shear reinforcement (exp.9.5N); $A_{sv,min} = 0.08 \text{ N/mm}^2 \times b \times (f_{ck} / 1 \text{ N/mm}^2)^{0.5} / f_{yk} = \mathbf{1518}$ mm^2/m

PASS - Area of shear reinforcement provided exceeds minimum required

Maximum longitudinal spacing (exp.9.6N); $s_{vl,max} = 0.75 \times d = \mathbf{1071}$ mm

PASS - Longitudinal spacing of shear reinforcement provided is less than maximum

Design shear force span 2 at 1428 mm; $V_{Ed} = \max(V_{B\_s2\_max}, V_{B\_s2\_red}) = \mathbf{1071}$ kN

Design shear stress; $v_{Ed} = V_{Ed} / (b \times z) = \mathbf{0.527}$ N/mm^2

Strength reduction factor (cl.6.2.3(3)); $v_1 = 0.6 \times [1 - f_{ck} / 250 \text{ N/mm}^2] = \mathbf{0.504}$

Compression chord coefficient (cl.6.2.3(3)); $\alpha_{cw} = \mathbf{1.00}$

Angle of concrete compression strut (cl.6.2.3);

$\theta = \min(\max(0.5 \times \text{Asin}[\min(2 \times v_{Ed} / (\alpha_{cw} \times f_{cd} \times v_1),1)], 21.8 \text{ deg}), 45\text{deg}) = \mathbf{21.8}$ deg

Area of shear reinforcement required (exp.6.13); $A_{sv,req} = v_{Ed} \times b / (f_{yd} \times \cot(\theta)) = \mathbf{727}$ mm^2/m

Shear reinforcement provided; $6 \times 10\phi$ legs at 300 c/c

Area of shear reinforcement provided; $A_{sv,prov} = \mathbf{1571}$ mm^2/m

Minimum area of shear reinforcement (exp.9.5N); $A_{sv,min} = 0.08 \text{ N/mm}^2 \times b \times (f_{ck} / 1 \text{ N/mm}^2)^{0.5} / f_{yk} = \mathbf{1518}$ mm^2/m

PASS - Area of shear reinforcement provided exceeds minimum required

Maximum longitudinal spacing (exp.9.6N); $s_{vl,max} = 0.75 \times d = \mathbf{1071}$ mm

PASS - Longitudinal spacing of shear reinforcement provided is less than maximum

Crack control (Section 7.3)

Maximum crack width; $w_k = \mathbf{0.3}$ mm

Design value modulus of elasticity reinf (3.2.7(4)); $E_s = \mathbf{200000}$ N/mm^2

Mean value of concrete tensile strength; $f_{ct,eff} = f_{ctm} = \mathbf{3.5}$ N/mm^2

Stress distribution coefficient; $k_c = \mathbf{0.4}$

Non-uniform self-equilibrating stress coefficient; $k = \min(\max(1 + (300 \text{ mm} - \min(h, b)) \times 0.35 / 500 \text{ mm}, 0.65), 1) = \mathbf{0.65}$

Actual tension bar spacing; $s_{bar} = (b - 2 \times (c_{nom\_s} + \phi_v) - \phi_{top}) / (N_{top} - 1) = \mathbf{123}$ mm

Maximum stress permitted (Table 7.3N); σ_s = **301** N/mm^2

Concrete to steel modulus of elast. ratio; $\alpha_{cr} = E_s / E_{cm} =$ **5.68**

Distance of the Elastic NA from bottom of beam; $y = (b \times h^2 / 2 + A_{s,prov} \times (\alpha_{cr} - 1) \times (h - d)) / (b \times h + A_{s,prov} \times (\alpha_{cr} - 1)) =$ **742** mm

Area of concrete in the tensile zone; $A_{ct} = b \times y =$ **1112703** mm^2

Minimum area of reinforcement required (exp.7.1); $A_{sc,min} = k_c \times k \times f_{ct,eff} \times A_{ct} / \sigma_s =$ **3367** mm^2

PASS - Area of tension reinforcement provided exceeds minimum required for crack control

Quasi-permanent value of variable action; ψ_2 = **0.30**

Quasi-permanent limit state moment; $M_{QP} = abs(M_{B\_c21}) + \psi_2 \times abs(M_{B\_c22}) =$ **2196** kNm

Permanent load ratio; $R_{PL} = M_{QP} / M =$ **0.74**

Service stress in reinforcement; $\sigma_{sr} = f_{yd} \times A_{s,req} / A_{s,prov} \times R_{PL} =$ **275** N/mm^2

Maximum bar spacing (Tables 7.3N); $s_{bar,max}$ = **150** mm

PASS - Maximum bar spacing exceeds actual bar spacing for crack control

Minimum bar spacing

Minimum bottom bar spacing; $s_{bot,min} = (b - 2 \times c_{nom\_s} - 2 \times \phi_v - \phi_{bot}) / (N_{bot} - 1) =$ **123** mm

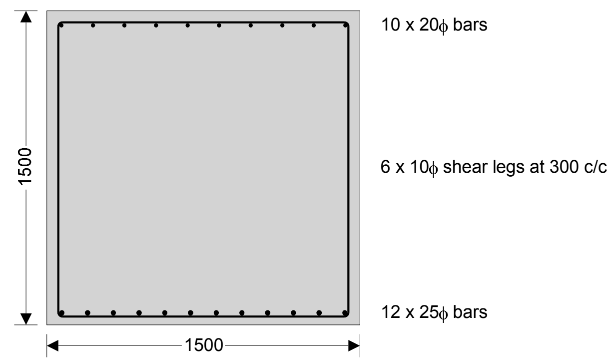
Minimum allowable bottom bar spacing; $s_{bar\_bot,min} = max(\phi_{bot}, h_{agg} + 5\text{ mm}, 20\text{ mm}) + \phi_{bot} =$ **50** mm

Minimum top bar spacing; $s_{top,min} = (b - 2 \times c_{nom\_s} - 2 \times \phi_v - \phi_{top}) / (N_{top} - 1) =$ **123** mm

Minimum allowable top bar spacing; $s_{bar\_top,min} = max(\phi_{top}, h_{agg} + 5\text{ mm}, 20\text{ mm}) + \phi_{top} =$ **50** mm

PASS - Actual bar spacing exceeds minimum allowable

Mid span 2



Rectangular section in flexure (Section 6.1) - Positive midspan moment

Design bending moment; $M = abs(M_{s2\_red}) =$ **2239** kNm

Depth to tension reinforcement; $d = h - c_{nom\_b} - \phi_v - \phi_{bot} / 2 =$ **1443** mm

Percentage redistribution; $m_{rs2} = M_{s2\_red} / M_{s2\_max} - 1 =$ **0** %

Redistribution ratio; $\delta = min(1 - m_{rs2}, 1) =$ **1.000**

$K = M / (b \times d^2 \times f_{ck}) =$ **0.018**

$K' = 0.598 \times \delta - 0.181 \times \delta^2 - 0.21 =$ **0.207**

K' > K - No compression reinforcement is required

Lever arm; $z = min((d / 2) \times [1 + (1 - 3.53 \times K)^{0.5}], 0.95 \times d) =$ **1370** mm

Depth of neutral axis; $x = 2.5 \times (d - z) =$ **180** mm

Area of tension reinforcement required; $A_{s,req} = M / (f_{yd} \times z) =$ **3758** mm^2

Tension reinforcement provided; 12 × 25ϕ bars

Area of tension reinforcement provided; $A_{s,prov}$ = **5890** mm^2

Minimum area of reinforcement (exp.9.1N); $A_{s,min} = max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times b \times d =$ **3948** mm^2

Maximum area of reinforcement (cl.9.2.1.1(3)); $A_{s,max} = 0.04 \times b \times h =$ **90000** mm^2

PASS - Area of reinforcement provided is greater than area of reinforcement required

Rectangular section in shear (Section 6.2)

Shear reinforcement provided; $6 \times 10\phi$ legs at 300 c/c

Area of shear reinforcement provided; $A_{sv,prov}$ = **1571** mm^2/m

Minimum area of shear reinforcement (exp.9.5N); $A_{sv,min} = 0.08\ N/mm^2 \times b \times (f_{ck} / 1\ N/mm^2)^{0.5} / f_{yk}$ = **1518** mm^2/m

PASS - Area of shear reinforcement provided exceeds minimum required

Maximum longitudinal spacing (exp.9.6N); $s_{vl,max} = 0.75 \times d$ = **1082** mm

PASS - Longitudinal spacing of shear reinforcement provided is less than maximum

Design shear resistance (assuming cot(θ) is 2.5); $V_{prov} = 2.5 \times A_{sv,prov} \times z \times f_{yd}$ = **2339.8** kN

Shear links provided valid between 0 mm and 15900 mm with tension reinforcement of 5890 mm^2

Crack control (Section 7.3)

Maximum crack width; w_k = **0.3** mm

Design value modulus of elasticity reinf (3.2.7(4)); E_s = **200000** N/mm^2

Mean value of concrete tensile strength; $f_{ct,eff} = f_{ctm}$ = **3.5** N/mm^2

Stress distribution coefficient; k_c = **0.4**

Non-uniform self-equilibrating stress coefficient; $k = \min(\max(1 + (300\ mm - \min(h, b)) \times 0.35 / 500\ mm, 0.65), 1)$ = **0.65**

Actual tension bar spacing; $s_{bar} = (b - 2 \times (c_{nom\_s} + \phi_v) - \phi_{bot}) / (N_{bot} - 1)$ = **123** mm

Maximum stress permitted (Table 7.3N); σ_s = **301** N/mm^2

Concrete to steel modulus of elast. ratio; $\alpha_{cr} = E_s / E_{cm}$ = **5.68**

Distance of the Elastic NA from bottom of beam; $y = (b \times h^2 / 2 + A_{s,prov} \times (\alpha_{cr} - 1) \times (h - d)) / (b \times h + A_{s,prov} \times (\alpha_{cr} - 1))$ = **742** mm

Area of concrete in the tensile zone; $A_{ct} = b \times y$ = **1112431** mm^2

Minimum area of reinforcement required (exp.7.1); $A_{sc,min} = k_c \times k \times f_{ct,eff} \times A_{ct} / \sigma_s$ = **3367** mm^2

PASS - Area of tension reinforcement provided exceeds minimum required for crack control

Quasi-permanent value of variable action; ψ_2 = **0.30**

Quasi-permanent limit state moment; $M_{QP} = abs(M_{s2\_c21}) + \psi_2 \times abs(M_{s2\_c22})$ = **1659** kNm

Permanent load ratio; $R_{PL} = M_{QP} / M$ = **0.74**

Service stress in reinforcement; $\sigma_{sr} = f_{yd} \times A_{s,req} / A_{s,prov} \times R_{PL}$ = **205** N/mm^2

Maximum bar spacing (Tables 7.3N); $s_{bar,max}$ = **200** mm

PASS - Maximum bar spacing exceeds actual bar spacing for crack control

Minimum bar spacing

Minimum bottom bar spacing; $s_{bot,min} = (b - 2 \times c_{nom\_s} - 2 \times \phi_v - \phi_{bot}) / (N_{bot} - 1)$ = **123** mm

Minimum allowable bottom bar spacing; $s_{bar\_bot,min} = \max(\phi_{bot}, h_{agg} + 5\ mm, 20\ mm) + \phi_{bot}$ = **50** mm

Minimum top bar spacing; $s_{top,min} = (b - 2 \times c_{nom\_s} - 2 \times \phi_v - \phi_{top}) / (N_{top} - 1)$ = **151** mm

Minimum allowable top bar spacing; $s_{bar\_top,min} = \max(\phi_{top}, h_{agg} + 5\ mm, 20\ mm) + \phi_{top}$ = **45** mm

PASS - Actual bar spacing exceeds minimum allowable

Deflection control (Section 7.4)

Reference reinforcement ratio; $\rho_{m0} = (f_{ck} / 1\ N/mm^2)^{0.5} / 1000$ = **0.006**

Required tension reinforcement ratio; $\rho_m = A_{s,req} / (b \times d)$ = **0.002**

Required compression reinforcement ratio; $\rho'_m = A_{s2,req} / (b \times d)$ = **0.000**

Structural system factor (Table 7.4N); K_b = **1.5**

Basic allowable span to depth ratio (7.16a); $span\_to\_depth_{basic} = K_b \times [11 + 1.5 \times (f_{ck} / 1\ N/mm^2)^{0.5} \times \rho_{m0} / \rho_m + 3.2 \times (f_{ck} / 1\ N/mm^2)^{0.5} \times (\rho_{m0} / \rho_m - 1)^{1.5}]$ = **198.632**

Reinforcement factor (exp.7.17); $K_s = \min(A_{s,prov} / A_{s,req} \times 500\ N/mm^2 / f_{yk}, 1.5)$ = **1.500**

Flange width factor; F1 = **1.000**

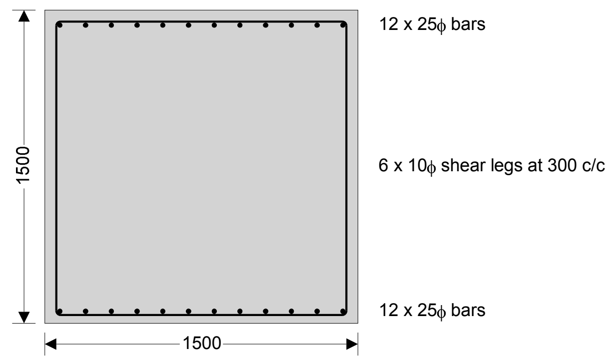
Long span supporting brittle partition factor; F2 = **1.000**

Allowable span to depth ratio; $span\_to\_depth_{allow} = \min(span\_to\_depth_{basic} \times K_s \times F1 \times F2, 40 \times K_b)$ = **60.000**

Actual span to depth ratio; $span\_to\_depth_{actual} = L_{s2} / d$ = **11.023**

PASS - Actual span to depth ratio is within the allowable limit

Support C



Rectangular section in flexure (Section 6.1)

Design bending moment; $M = abs(M_{C\_red})$ = **2983** kNm

Depth to tension reinforcement; $d = h - c_{nom\_t} - \phi_v - \phi_{top} / 2$ = **1428** mm

Percentage redistribution; m_{rC} = **0** %

Redistribution ratio; $\delta = \min(1 - m_{rC}, 1)$ = **1.000**

$K = M / (b \times d^2 \times f_{ck})$ = **0.024**

$K' = 0.598 \times \delta - 0.181 \times \delta^2 - 0.21$ = **0.207**

K' > K - No compression reinforcement is required

Lever arm; $z = \min((d / 2) \times [1 + (1 - 3.53 \times K)^{0.5}], 0.95 \times d)$ = **1356** mm

Depth of neutral axis; $x = 2.5 \times (d - z)$ = **178** mm

Area of tension reinforcement required; $A_{s,req} = M / (f_{yd} \times z)$ = **5059** mm^2

Tension reinforcement provided; 12 × 25φ bars

Area of tension reinforcement provided; $A_{s,prov}$ = **5890** mm^2

Minimum area of reinforcement (exp.9.1N); $A_{s,min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times b \times d$ = **3907** mm^2

Maximum area of reinforcement (cl.9.2.1.1(3)); $A_{s,max} = 0.04 \times b \times h$ = **90000** mm^2

PASS - Area of reinforcement provided is greater than area of reinforcement required

Rectangular section in shear (Section 6.2)

Design shear force at support C; $V_{Ed,max} = abs(\max(V_{C\_max}, V_{C\_red}))$ = **1313** kN

Angle of comp. shear strut for maximum shear; θ_{max} = 45 deg

Maximum design shear force (exp.6.9); $V_{Rd,max} = b \times z \times v_1 \times f_{cd} / (\cot(\theta_{max}) + \tan(\theta_{max}))$ = **11619** kN

PASS - Design shear force at support is less than maximum design shear force

Design shear force span 2 at 14473 mm; $V_{Ed} = abs(\min(V_{C\_s2\_max}, V_{C\_s2\_red}))$ = **1073** kN

Design shear stress; $v_{Ed} = V_{Ed} / (b \times z)$ = **0.528** N/mm^2

Strength reduction factor (cl.6.2.3(3)); $v_1 = 0.6 \times [1 - f_{ck} / 250 \text{ N/mm}^2]$ = **0.504**

Compression chord coefficient (cl.6.2.3(3)); α_{cw} = **1.00**

Angle of concrete compression strut (cl.6.2.3);

$\theta = \min(\max(0.5 \times Asin[\min(2 \times v_{Ed} / (\alpha_{cw} \times f_{cd} \times v_1),1)], 21.8 \text{ deg}), 45\text{deg})$ = **21.8** deg

Area of shear reinforcement required (exp.6.13); $A_{sv,req} = v_{Ed} \times b / (f_{yd} \times \cot(\theta))$ = **728** mm^2/m

Shear reinforcement provided; 6 × 10φ legs at 300 c/c

Area of shear reinforcement provided; $A_{sv,prov}$ = **1571** mm^2/m

Minimum area of shear reinforcement (exp.9.5N); $A_{sv,min} = 0.08 \text{ N/mm}^2 \times b \times (f_{ck} / 1 \text{ N/mm}^2)^{0.5} / f_{yk}$ = **1518** mm^2/m

PASS - Area of shear reinforcement provided exceeds minimum required

Maximum longitudinal spacing (exp.9.6N); $s_{vl,max} = 0.75 \times d$ = **1071** mm

PASS - Longitudinal spacing of shear reinforcement provided is less than maximum

Design shear force span 3 at 1428 mm; $V_{Ed} = \max(V_{C\_s3\_max}, V_{C\_s3\_red})$ = **843** kN

Design shear stress; $v_{Ed} = V_{Ed} / (b \times z) = \mathbf{0.414}$ N/mm^2

Strength reduction factor (cl.6.2.3(3)); $v_1 = 0.6 \times [1 - f_{ck} / 250 \text{ N/mm}^2] = \mathbf{0.504}$

Compression chord coefficient (cl.6.2.3(3)); $\alpha_{cw} = \mathbf{1.00}$

Angle of concrete compression strut (cl.6.2.3);

$\theta = \min(\max(0.5 \times \text{Asin}[\min(2 \times v_{Ed} / (\alpha_{cw} \times f_{cd} \times v_1),1)], 21.8 \text{ deg}), 45\text{deg}) = \mathbf{21.8}$ deg

Area of shear reinforcement required (exp.6.13); $A_{sv,req} = v_{Ed} \times b / (f_{yd} \times \cot(\theta)) = \mathbf{572}$ mm^2/m

Shear reinforcement provided; $6 \times 10\phi$ legs at 300 c/c

Area of shear reinforcement provided; $A_{sv,prov} = \mathbf{1571}$ mm^2/m

Minimum area of shear reinforcement (exp.9.5N); $A_{sv,min} = 0.08 \text{ N/mm}^2 \times b \times (f_{ck} / 1 \text{ N/mm}^2)^{0.5} / f_{yk} = \mathbf{1518}$ mm^2/m

PASS - Area of shear reinforcement provided exceeds minimum required

Maximum longitudinal spacing (exp.9.6N); $s_{vl,max} = 0.75 \times d = \mathbf{1071}$ mm

PASS - Longitudinal spacing of shear reinforcement provided is less than maximum

Crack control (Section 7.3)

Maximum crack width; $w_k = \mathbf{0.3}$ mm

Design value modulus of elasticity reinf (3.2.7(4)); $E_s = \mathbf{200000}$ N/mm^2

Mean value of concrete tensile strength; $f_{ct,eff} = f_{ctm} = \mathbf{3.5}$ N/mm^2

Stress distribution coefficient; $k_c = \mathbf{0.4}$

Non-uniform self-equilibrating stress coefficient; $k = \min(\max(1 + (300 \text{ mm} - \min(h, b)) \times 0.35 / 500 \text{ mm}, 0.65), 1) = \mathbf{0.65}$

Actual tension bar spacing; $s_{bar} = (b - 2 \times (c_{nom\_s} + \phi_v) - \phi_{top}) / (N_{top} - 1) = \mathbf{123}$ mm

Maximum stress permitted (Table 7.3N); $\sigma_s = \mathbf{301}$ N/mm^2

Concrete to steel modulus of elast. ratio; $\alpha_{cr} = E_s / E_{cm} = \mathbf{5.68}$

Distance of the Elastic NA from bottom of beam; $y = (b \times h^2 / 2 + A_{s,prov} \times (\alpha_{cr} - 1) \times (h - d)) / (b \times h + A_{s,prov} \times (\alpha_{cr} - 1)) = \mathbf{742}$ mm

Area of concrete in the tensile zone; $A_{ct} = b \times y = \mathbf{1112703}$ mm^2

Minimum area of reinforcement required (exp.7.1); $A_{sc,min} = k_c \times k \times f_{ct,eff} \times A_{ct} / \sigma_s = \mathbf{3367}$ mm^2

PASS - Area of tension reinforcement provided exceeds minimum required for crack control

Quasi-permanent value of variable action; $\psi_2 = \mathbf{0.30}$

Quasi-permanent limit state moment; $M_{QP} = \text{abs}(M_{C\_c21}) + \psi_2 \times \text{abs}(M_{C\_c22}) = \mathbf{2210}$ kNm

Permanent load ratio; $R_{PL} = M_{QP} / M = \mathbf{0.74}$

Service stress in reinforcement; $\sigma_{sr} = f_{yd} \times A_{s,req} / A_{s,prov} \times R_{PL} = \mathbf{277}$ N/mm^2

Maximum bar spacing (Tables 7.3N); $s_{bar,max} = \mathbf{150}$ mm

PASS - Maximum bar spacing exceeds actual bar spacing for crack control

Minimum bar spacing

Minimum bottom bar spacing; $s_{bot,min} = (b - 2 \times c_{nom\_s} - 2 \times \phi_v - \phi_{bot}) / (N_{bot} - 1) = \mathbf{123}$ mm

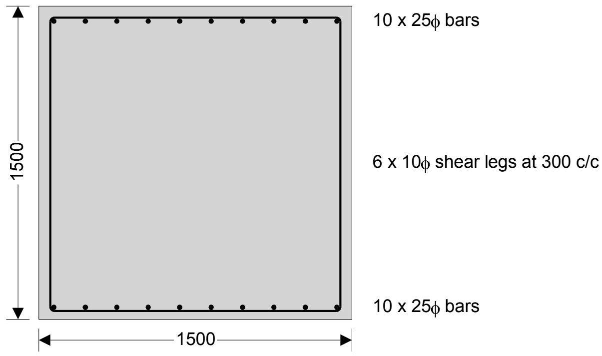
Minimum allowable bottom bar spacing; $s_{bar\_bot,min} = \max(\phi_{bot}, h_{agg} + 5 \text{ mm}, 20 \text{ mm}) + \phi_{bot} = \mathbf{50}$ mm

Minimum top bar spacing; $s_{top,min} = (b - 2 \times c_{nom\_s} - 2 \times \phi_v - \phi_{top}) / (N_{top} - 1) = \mathbf{123}$ mm

Minimum allowable top bar spacing; $s_{bar\_top,min} = \max(\phi_{top}, h_{agg} + 5 \text{ mm}, 20 \text{ mm}) + \phi_{top} = \mathbf{50}$ mm

PASS - Actual bar spacing exceeds minimum allowable

Mid span 3



Rectangular section in flexure (Section 6.1) - Positive midspan moment

Design bending moment;	$M = abs(M_{s3\_red})$ = **567** kNm
Depth to tension reinforcement;	$d = h - c_{nom\_b} - \phi_v - \phi_{bot} / 2$ = **1443** mm
Percentage redistribution;	$m_{rs3} = M_{s3\_red} / M_{s3\_max} - 1$ = **0** %
Redistribution ratio;	$\delta = \min(1 - m_{rs3}, 1)$ = **1.000**
	$K = M / (b \times d^2 \times f_{ck})$ = **0.005**
	$K' = 0.598 \times \delta - 0.181 \times \delta^2 - 0.21$ = **0.207**

K' > K - No compression reinforcement is required

Lever arm;	$z = \min((d / 2) \times [1 + (1 - 3.53 \times K)^{0.5}], 0.95 \times d)$ = **1370** mm
Depth of neutral axis;	$x = 2.5 \times (d - z)$ = **180** mm
Area of tension reinforcement required;	$A_{s,req} = M / (f_{yd} \times z)$ = **952** mm^2
Tension reinforcement provided;	$10 \times 25\phi$ bars
Area of tension reinforcement provided;	$A_{s,prov}$ = **4909** mm^2
Minimum area of reinforcement (exp.9.1N);	$A_{s,min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times b \times d$ = **3948** mm^2
Maximum area of reinforcement (cl.9.2.1.1(3));	$A_{s,max} = 0.04 \times b \times h$ = **90000** mm^2

PASS - Area of reinforcement provided is greater than area of reinforcement required

Rectangular section in shear (Section 6.2)

Shear reinforcement provided;	$6 \times 10\phi$ legs at 300 c/c
Area of shear reinforcement provided;	$A_{sv,prov}$ = **1571** mm^2/m
Minimum area of shear reinforcement (exp.9.5N);	$A_{sv,min} = 0.08\ N/mm^2 \times b \times (f_{ck} / 1\ N/mm^2)^{0.5} / f_{yk}$ = **1518** mm^2/m

PASS - Area of shear reinforcement provided exceeds minimum required

Maximum longitudinal spacing (exp.9.6N);	$s_{vl,max} = 0.75 \times d$ = **1082** mm

PASS - Longitudinal spacing of shear reinforcement provided is less than maximum

Design shear resistance (assuming cot(θ) is 2.5);	$V_{prov} = 2.5 \times A_{sv,prov} \times z \times f_{yd}$ = **2339.8** kN

Shear links provided valid between 0 mm and 6700 mm with tension reinforcement of 4909 mm^2

Crack control (Section 7.3)

Maximum crack width;	w_k = **0.3** mm
Design value modulus of elasticity reinf (3.2.7(4));	E_s = **200000** N/mm^2
Mean value of concrete tensile strength;	$f_{ct,eff} = f_{ctm}$ = **3.5** N/mm^2
Stress distribution coefficient;	k_c = **0.4**
Non-uniform self-equilibrating stress coefficient;	$k = \min(\max(1 + (300\ mm - \min(h, b)) \times 0.35 / 500\ mm, 0.65), 1)$ = **0.65**
Actual tension bar spacing;	$s_{bar} = (b - 2 \times (c_{nom\_s} + \phi_v) - \phi_{bot}) / (N_{bot} - 1)$ = **151** mm
Maximum stress permitted (Table 7.3N);	σ_s = **280** N/mm^2
Concrete to steel modulus of elast. ratio;	$\alpha_{cr} = E_s / E_{cm}$ = **5.68**
Distance of the Elastic NA from bottom of beam;	$y = (b \times h^2 / 2 + A_{s,prov} \times (\alpha_{cr} - 1) \times (h - d)) / (b \times h + A_{s,prov} \times (\alpha_{cr} - 1))$ = **743** mm

Area of concrete in the tensile zone; $A_{ct} = b \times y =$ **1114505** mm^2

Minimum area of reinforcement required (exp.7.1); $A_{sc,min} = k_c \times k \times f_{ct,eff} \times A_{ct} / \sigma_s =$ **3637** mm^2

PASS - Area of tension reinforcement provided exceeds minimum required for crack control

Quasi-permanent value of variable action; $\psi_2 =$ **0.30**

Quasi-permanent limit state moment; $M_{QP} = abs(M_{s3\_c21}) + \psi_2 \times abs(M_{s3\_c22}) =$ **420** kNm

Permanent load ratio; $R_{PL} = M_{QP} / M =$ **0.74**

Service stress in reinforcement; $\sigma_{sr} = f_{yd} \times A_{s,req} / A_{s,prov} \times R_{PL} =$ **62** N/mm^2

Maximum bar spacing (Tables 7.3N); $s_{bar,max} =$ **300** mm

PASS - Maximum bar spacing exceeds actual bar spacing for crack control

Minimum bar spacing

Minimum bottom bar spacing; $s_{bot,min} = (b - 2 \times c_{nom\_s} - 2 \times \phi_v - \phi_{bot}) / (N_{bot} - 1) =$ **151** mm

Minimum allowable bottom bar spacing; $s_{bar\_bot,min} = max(\phi_{bot}, h_{agg} + 5\ mm, 20\ mm) + \phi_{bot} =$ **50** mm

Minimum top bar spacing; $s_{top,min} = (b - 2 \times c_{nom\_s} - 2 \times \phi_v - \phi_{top}) / (N_{top} - 1) =$ **151** mm

Minimum allowable top bar spacing; $s_{bar\_top,min} = max(\phi_{top}, h_{agg} + 5\ mm, 20\ mm) + \phi_{top} =$ **50** mm

PASS - Actual bar spacing exceeds minimum allowable

Deflection control (Section 7.4)

Reference reinforcement ratio; $\rho_{m0} = (f_{ck} / 1\ N/mm^2)^{0.5} / 1000 =$ **0.006**

Required tension reinforcement ratio; $\rho_m = A_{s,req} / (b \times d) =$ **0.000**

Required compression reinforcement ratio; $\rho'_m = A_{s2,req} / (b \times d) =$ **0.000**

Structural system factor (Table 7.4N); $K_b =$ **1.3**

Basic allowable span to depth ratio (7.16a); $span\_to\_depth_{basic} = K_b \times [11 + 1.5 \times (f_{ck} / 1\ N/mm^2)^{0.5} \times \rho_{m0} / \rho_m + 3.2 \times (f_{ck} / 1\ N/mm^2)^{0.5} \times (\rho_{m0} / \rho_m - 1)^{1.5}] =$ **1477.745**

Reinforcement factor (exp.7.17); $K_s = min(A_{s,prov} / A_{s,req} \times 500\ N/mm^2 / f_{yk}, 1.5) =$ **1.500**

Flange width factor; $F1 =$ **1.000**

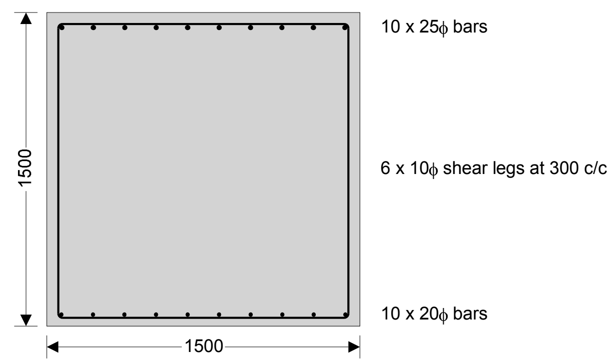
Long span supporting brittle partition factor; $F2 =$ **1.000**

Allowable span to depth ratio; $span\_to\_depth_{allow} = min(span\_to\_depth_{basic} \times K_s \times F1 \times F2, 40 \times K_b) =$ **52.000**

Actual span to depth ratio; $span\_to\_depth_{actual} = L_{s3} / d =$ **4.645**

PASS - Actual span to depth ratio is within the allowable limit

<u>Support D</u>



Rectangular section in flexure (Section 6.1)

Minimum moment factor (cl.9.2.1.2(1)); $\beta_1 =$ **0.25**

Design bending moment; $M = max(abs(M_{D\_red}), \beta_1 \times abs(M_{s3\_red})) =$ **142** kNm

Depth to tension reinforcement; $d = h - c_{nom\_t} - \phi_v - \phi_{top} / 2 =$ **1428** mm

Percentage redistribution; $m_{rD} =$ **0** %

Redistribution ratio; $\delta = min(1 - m_{rD}, 1) =$ **1.000**

$K = M / (b \times d^2 \times f_{ck}) = \mathbf{0.001}$

$K' = 0.598 \times \delta - 0.181 \times \delta^2 - 0.21 = \mathbf{0.207}$

K' > K - No compression reinforcement is required

Lever arm; $z = \min((d / 2) \times [1 + (1 - 3.53 \times K)^{0.5}], 0.95 \times d) = \mathbf{1356}$ mm

Depth of neutral axis; $x = 2.5 \times (d - z) = \mathbf{178}$ mm

Area of tension reinforcement required; $A_{s,req} = M / (f_{yd} \times z) = \mathbf{241}$ mm^2

Tension reinforcement provided; $10 \times 25\phi$ bars

Area of tension reinforcement provided; $A_{s,prov} = \mathbf{4909}$ mm^2

Minimum area of reinforcement (exp.9.1N); $A_{s,min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times b \times d = \mathbf{3907}$ mm^2

Maximum area of reinforcement (cl.9.2.1.1(3)); $A_{s,max} = 0.04 \times b \times h = \mathbf{90000}$ mm^2

PASS - Area of reinforcement provided is greater than area of reinforcement required

Minimum bottom reinforcement at supports

Minimum reinforcement factor (cl.9.2.1.4(1)); $\beta_2 = \mathbf{0.25}$

Area of reinforcement to adjacent span; $A_{s,span} = \mathbf{4909}$ mm^2

Minimum bottom reinforcement to support; $A_{s2,min} = \beta_2 \times A_{s,span} = \mathbf{1227}$ mm^2

Bottom reinforcement provided; $10 \times 20\phi$ bars

Area of bottom reinforcement provided; $A_{s2,prov} = \mathbf{3142}$ mm^2

PASS - Area of reinforcement provided is greater than minimum area of reinforcement required

Rectangular section in shear (Section 6.2)

Design shear force at support D; $V_{Ed,max} = \text{abs}(\max(V_{D\_max}, V_{D\_red})) = \mathbf{23}$ kN

Angle of comp. shear strut for maximum shear; $\theta_{max} = 45$ deg

Maximum design shear force (exp.6.9); $V_{Rd,max} = b \times z \times v_1 \times f_{cd} / (\cot(\theta_{max}) + \tan(\theta_{max})) = \mathbf{11619}$ kN

PASS - Design shear force at support is less than maximum design shear force

Design shear force span 3 at 5273 mm; $V_{Ed} = \max(V_{D\_s3\_max}, V_{D\_s3\_red}) = \mathbf{216}$ kN

Design shear stress; $v_{Ed} = V_{Ed} / (b \times z) = \mathbf{0.106}$ N/mm^2

Strength reduction factor (cl.6.2.3(3)); $v_1 = 0.6 \times [1 - f_{ck} / 250 \text{ N/mm}^2] = \mathbf{0.504}$

Compression chord coefficient (cl.6.2.3(3)); $\alpha_{cw} = \mathbf{1.00}$

Angle of concrete compression strut (cl.6.2.3);

$\theta = \min(\max(0.5 \times \text{Asin}[\min(2 \times v_{Ed} / (\alpha_{cw} \times f_{cd} \times v_1),1)], 21.8 \text{ deg}), 45\text{deg}) = \mathbf{21.8}$ deg

Area of shear reinforcement required (exp.6.13); $A_{sv,req} = v_{Ed} \times b / (f_{yd} \times \cot(\theta)) = \mathbf{147}$ mm^2/m

Shear reinforcement provided; $6 \times 10\phi$ legs at 300 c/c

Area of shear reinforcement provided; $A_{sv,prov} = \mathbf{1571}$ mm^2/m

Minimum area of shear reinforcement (exp.9.5N); $A_{sv,min} = 0.08 \text{ N/mm}^2 \times b \times (f_{ck} / 1 \text{ N/mm}^2)^{0.5} / f_{yk} = \mathbf{1518}$ mm^2/m

PASS - Area of shear reinforcement provided exceeds minimum required

Maximum longitudinal spacing (exp.9.6N); $s_{vl,max} = 0.75 \times d = \mathbf{1071}$ mm

PASS - Longitudinal spacing of shear reinforcement provided is less than maximum

Crack control (Section 7.3)

Maximum crack width; $w_k = \mathbf{0.3}$ mm

Design value modulus of elasticity reinf (3.2.7(4)); $E_s = \mathbf{200000}$ N/mm^2

Mean value of concrete tensile strength; $f_{ct,eff} = f_{ctm} = \mathbf{3.5}$ N/mm^2

Stress distribution coefficient; $k_c = \mathbf{0.4}$

Non-uniform self-equilibrating stress coefficient; $k = \min(\max(1 + (300 \text{ mm} - \min(h, b)) \times 0.35 / 500 \text{ mm}, 0.65), 1) = \mathbf{0.65}$

Actual tension bar spacing; $s_{bar} = (b - 2 \times (c_{nom\_s} + \phi_v) - \phi_{top}) / (N_{top} - 1) = \mathbf{151}$ mm

Maximum stress permitted (Table 7.3N); $\sigma_s = \mathbf{280}$ N/mm^2

Concrete to steel modulus of elast. ratio; $\alpha_{cr} = E_s / E_{cm} = \mathbf{5.68}$

Distance of the Elastic NA from bottom of beam; $y = (b \times h^2 / 2 + A_{s,prov} \times (\alpha_{cr} - 1) \times (h - d)) / (b \times h + A_{s,prov} \times (\alpha_{cr} - 1)) = \mathbf{743}$ mm

Area of concrete in the tensile zone; $A_{ct} = b \times y = \mathbf{1114732}$ mm^2

Minimum area of reinforcement required (exp.7.1); $A_{sc,min} = k_c \times k \times f_{ct,eff} \times A_{ct} / \sigma_s =$ **3638** mm^2

PASS - Area of tension reinforcement provided exceeds minimum required for crack control

Quasi-permanent value of variable action; $\psi_2 =$ **0.30**

Quasi-permanent limit state moment; $M_{QP} = abs(M_{D\_c21}) + \psi_2 \times abs(M_{D\_c22}) =$ **419** kNm

Permanent load ratio; $R_{PL} = M_{QP} / M =$ **2.95**

Service stress in reinforcement; $\sigma_{sr} = f_{yd} \times A_{s,req} / A_{s,prov} \times R_{PL} =$ **63** N/mm^2

Maximum bar spacing (Tables 7.3N); $s_{bar,max} =$ **300** mm

PASS - Maximum bar spacing exceeds actual bar spacing for crack control

Minimum bar spacing

Minimum bottom bar spacing; $s_{bot,min} = (b - 2 \times c_{nom\_s} - 2 \times \phi_v - \phi_{bot}) / (N_{bot} - 1) =$ **151** mm

Minimum allowable bottom bar spacing; $s_{bar\_bot,min} = max(\phi_{bot}, h_{agg} + 5\ mm, 20\ mm) + \phi_{bot} =$ **45** mm

Minimum top bar spacing; $s_{top,min} = (b - 2 \times c_{nom\_s} - 2 \times \phi_v - \phi_{top}) / (N_{top} - 1) =$ **151** mm

Minimum allowable top bar spacing; $s_{bar\_top,min} = max(\phi_{top}, h_{agg} + 5\ mm, 20\ mm) + \phi_{top} =$ **50** mm

PASS - Actual bar spacing exceeds minimum allowable

Deflection

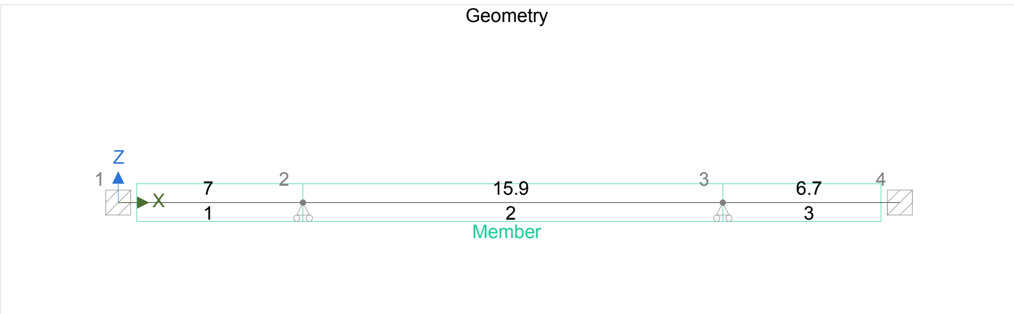
The capping beam is part of the overall structural system and strict limits have been placed on the movement of the wall within the Basement Impact Assessment. The potential deflection of the capping beam is therefore of importance in understanding the overall movement of the wall during basement construction.

A separate analysis of the beam has been undertaken to determine the deflection of the beam under the imposed loads.

ANALYSIS

Tedds calculation version 1.0.10

Geometry



Nodes

Node	Co-ordinates X (m)	Co-ordinates Z (m)	Freedom X	Freedom Z	Freedom Rot.	Coordinate system Name	Coordinate system Angle (°)	Spring X (kN/m)	Spring Z (kN/m)	Spring Rot. kNm/°
1	0	0	Fixed	Fixed	Fixed		0	0	0	0
2	7	0	Free	Fixed	Free		0	0	0	0
3	22.9	0	Free	Fixed	Free		0	0	0	0
4	29.6	0	Fixed	Fixed	Fixed		0	0	0	0

Materials

Name	Density (kg/m³)	Youngs Modulus kN/mm²	Shear Modulus kN/mm²	Thermal Coefficient °C⁻¹
Concrete (EC2 normal)	2500	34	14.2	0.00001

Sections

Name	Area	Moment of inertia		Shear area	
		Major	Minor	A_y	A_z
	(cm²)	(cm⁴)	(cm⁴)	(cm²)	(cm²)
R 1500x1500	22500	$4.\times10^7$	$4.\times10^7$	18750	18750

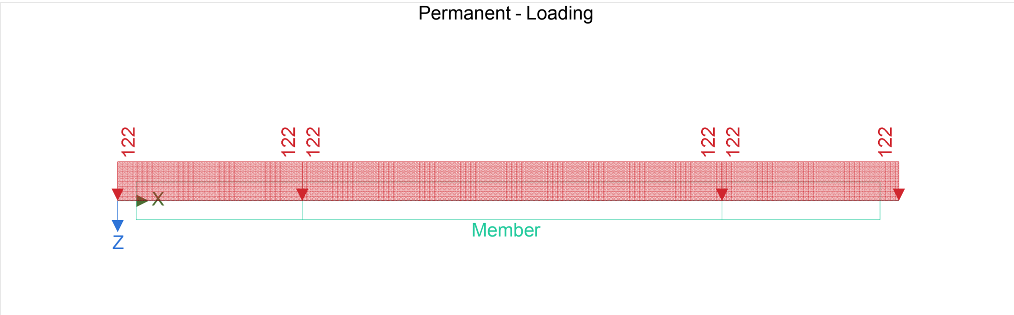
Elements

Element	Length	Nodes		Section	Material	Releases			Rotated
	(m)	Start	End			Start moment	End moment	Axial	
1	7	1	2	R 1500x1500	Concrete (EC2 normal)	Fixed	Fixed	Fixed	
2	15.9	2	3	R 1500x1500	Concrete (EC2 normal)	Fixed	Fixed	Fixed	
3	6.7	3	4	R 1500x1500	Concrete (EC2 normal)	Fixed	Fixed	Fixed	

Members

Name	Elements	
	Start	End
Member	1	3

Loading



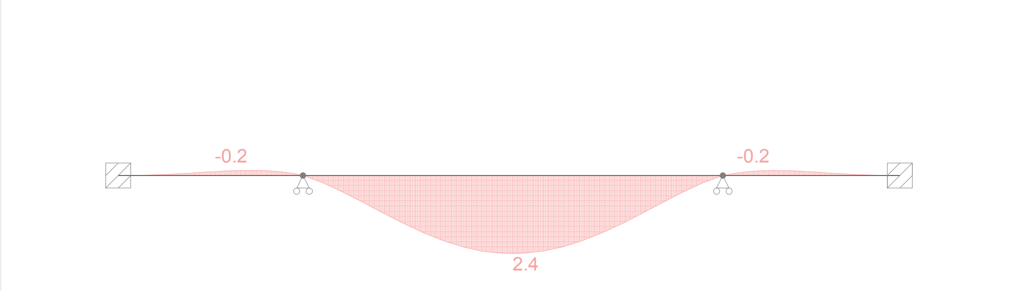
Results

Total deflection

Member results

Load case: Permanent

Member	Deflection				Axial deflection			
	Pos (m)	Max (mm)	Pos (m)	Min (mm)	Pos (m)	Max (mm)	Pos (m)	Min (mm)
Member	14.947	2.4	5.044	-0.2	0	0	0	0



Load case: Permanent

Member	Deflection			
	Pos (m)	Max (mm)	Pos (m)	Min (mm)
Member	14.947	2.4	5.044	-0.2

;

So the maximum deflection is 2.4mm into the site and 0.2mm out of the site.

In addition to deflection of the beams the props will be subject to axial shortening. The props proposed are hydraulic props and so can be pre-loaded to remove this axial shortening.

Props

Use MGF 600 Series Prop. For 10m length axial capacity is 2500kN, see Appendix B.

The Props are to be supported on Corbels onto the Capping beam.

Assessment of Pavement Vaults

Pavement Vaults exist in the pavement to the front of the site.

These are to be investigated to confirm their actual size, whether they have been filled and condition.

It is understood that at present these have been back filled with rubble, not necessarily fully compacted. The internal walls to the vaults are therefore subject to lateral loads for which they were never designed.

The vaults will therefore be strengthened by the provision of a waler at a distance down the wall to provide lateral resistance to the forces exerted by the fill.

Refer to drawing 031-51-1000-DR-Y-00006 – B1

Forces on wall assessed using Coloumb Theory.

RETAINING WALL ANALYSIS

In accordance with EN1997-1:2004 incorporating Corrigendum dated February 2009 and the UK National Annex incorporating Corrigendum No.1

Tedds calculation version 2.6.04

Retaining wall details

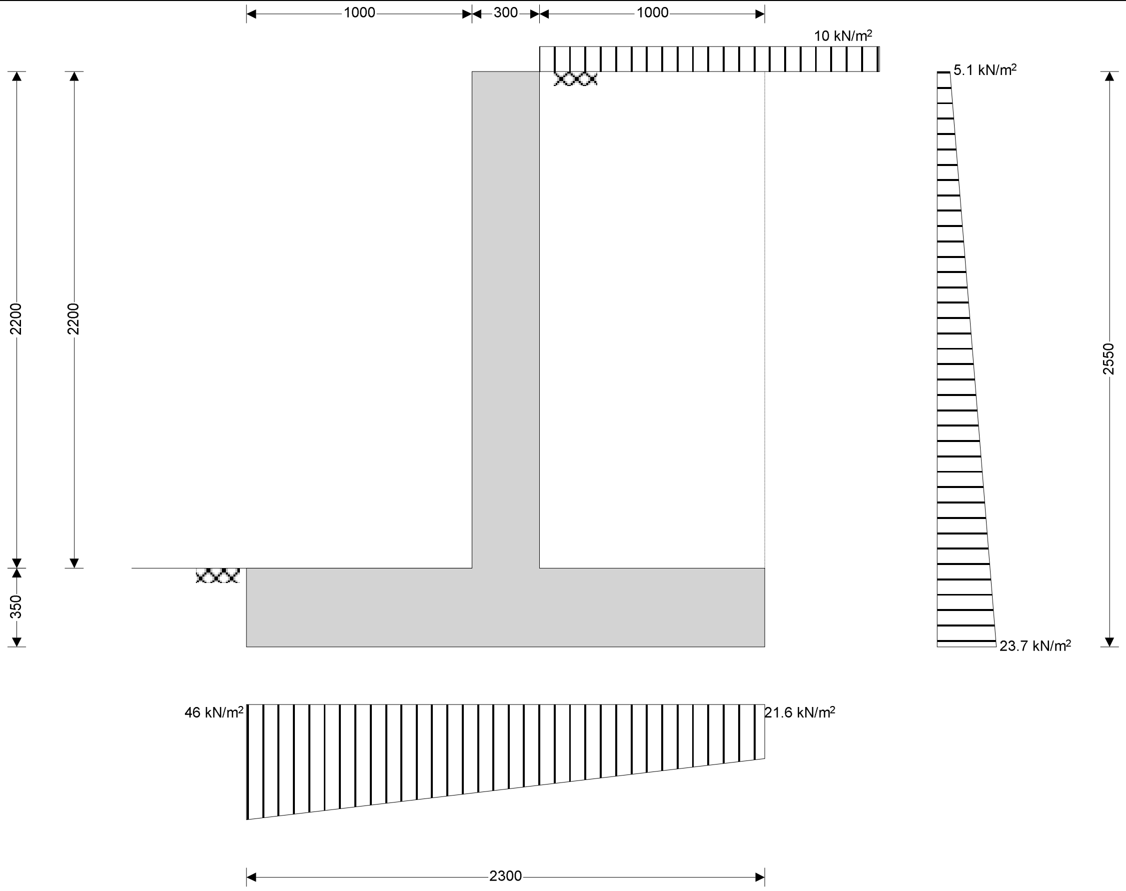
Stem height;	h_{stem} = **2200** mm
Angle to rear face of stem;	α = **90** deg
Height of retained soil;	h_{ret} = **2200** mm
Angle of soil surface;	β = **0** deg
Depth of cover;	d_{cover} = **0** mm

Retained soil properties

Soil type;	Loose brick hardcore
Moist density;	γ_{mr} = **14.1** kN/m^3
Saturated density;	γ_{sr} = **17.1** kN/m^3
Characteristic effective shear resistance angle;	$\phi'_{r.k}$ = **28** deg
Characteristic wall friction angle;	$\delta_{r.k}$ = **14** deg

Loading details

Variable surcharge load;	$Surcharge_Q$ = **10** kN/m^2



Using Coulomb theory

At rest pressure coefficient; $K_0 = 1 - \sin(\phi'_{r.k}) = \mathbf{0.531}$

Retained soil properties

Design effective shear resistance angle; $\phi'_{r.d} = \text{atan}(\tan(\phi'_{r.k}) / \gamma_{\phi'}) = \mathbf{28}$ deg

Design wall friction angle; $\delta_{r.d} = \text{atan}(\tan(\delta_{r.k}) / \gamma_{\phi'}) = \mathbf{14}$ deg

Using Coulomb theory

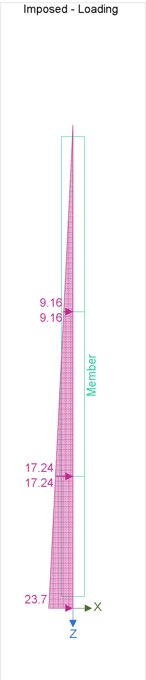
At rest pressure coefficient; $K_0 = 1 - \sin(\phi'_{r.d}) = \mathbf{0.531}$

The wall is tied at its head through connection with the top arch and is supported at the base by the retained ground. The waler therefore provides an intermediate support.

ANALYSIS

Tedds calculation version 1.0.10

Loading



Results

Total base reactions

Load case/combination	Force	
	FX	FZ
	(kN)	(kN)
a (Strength)	-39.1	0

Element end forces

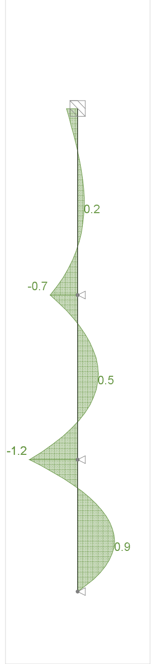
Load combination: a (Strength)

Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	0.6	1	0	-7.8	0
		2	0	-10.6	-1.2
2	0.75	2	0	-8.8	1.2
		3	0	-6	-0.7
3	0.85	3	0	-4.3	0.7
		4	0	-1.5	-0.3

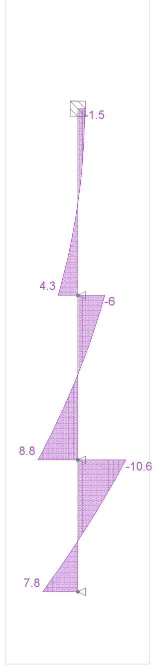
BRIDGES POUND CONSULTING ENGINEERS	Location UCL for Mace	Job No M1993
	Basement Temporary Works Design	Sheet No 23
	By SMH | 31/03/16 | Checked RLN | Mar 16	Approved SMH | Apr 16

Forces

Strength combinations - Moment envelope (kNm)



Strength combinations - Shear envelope (kN)



Member results

Envelope - Strength combinations

Member	Shear force		Moment			
	Pos (m)	Max abs (kN)	Pos (m)	Max (kNm)	Pos (m)	Min (kNm)
Member	0.6	-10.6	0.231	0.9	0.6	-1.2

;

The maximum shear on the wall is 10.6 kN/m and moment is 1.2 kNm/m.

For 215mm thick brickwork the shear stress is therefore .05 N/mm2 and the bending capacity is 1.43 kNm/m so the brickwork capacity is satisfactory.

The UDL on the waler is 19.4 / 1.5 = 13 kN/m. The waler spans a maximum of 3m so the applied moment and shears are 14.6 kNm and 19.5 kN.

Using a RMD Slimshore The maximum moment of resistance is 25.3 kNm and shear capacity exceeds 50kN.

The loads are taken into the masonry using resin anchored ties. The performance of resins in masonry is variable but data is available on suitable systems such as Hilti HIT HY 50 resin injection mortar.

Load capacity of M12 anchor = 3.5 / 80 = 0.044 kN/mm depth

Required length of anchor = (2 x 19.5) / 0.044 = 891mm Use 2 No M12 anchors per connection min 450mm deep = 900mm overall.

At the upper waler the load is 10.3 / 1.5 = 6.86 kN/m so one half of the lower waler load. Therefore a single fixing will be adequate.

Use a secondary vertical tie connected back to the wall to provide support for the waler fixed using 3 No M12 resin anchors 450mm embedment.

Assessment alongside 26 Gordon Street

To the East of the site is 26 Gordon Street. This is a period property with a large masonry gable wall sitting on what have been identified as traditional spread strip footings at relatively high level just below the building basement.

There are 5 No 1500x750 ground beams being cast perpendicular to the gable end of 26 Gordon Street.

The levels are such that the bottom of these beams is just below the identified footing of the gable wall.

The beams support a 1500x1200mm capping beam running parallel to the wall which has been detailed with an offset so that the existing wall footings will not be affected.

Trail pits will need to be excavated at each beam location to identify the footing depth and type but at present it appears no temporary works are required to carry out these permanent works.

Refer to drawing 031-51-1000-DR-Y-00007 – B1

Assessment along the ACBE building.

To the south of the site is the ACBE building. This building has been noted as sitting on a raft foundation

Refer to drawing 031-51-1000-DR-Y-00008 – B1

The existing footings are to the South at or just below the level of the new capping beam and therefore there are no temporary works required in this area.

Between Grids 3 and 5 the Capping beam is at a reduced level. This means that the excavation for this capping beam will be below the formation level of the ACBE building.

In order to maintain a continuous capping beam a temporary capping beam will be installed at the same level as the main capping beam. From Appendix A it can be seen that the loads in this beam are in fact quite small, maximum moment = 810 kNm and Shear = 310 kN. A 750 x 750 beam is more than adequate to take these loads.

<u>RC BEAM DESIGN (EN1992-1)</u>

In accordance with UK national annex

TEDDS calculation version 2.1.15

Rectangular section details

Section width;	b = **750** mm
Section depth;	h = **750** mm

Concrete details (Table 3.1 - Strength and deformation characteristics for concrete)

Concrete strength class;	**C32/40**
Characteristic compressive cylinder strength;	f_{ck} = **32** N/mm^2
Characteristic compressive cube strength;	$f_{ck,cube}$ = **40** N/mm^2
Mean value of compressive cylinder strength;	$f_{cm} = f_{ck}$ + 8 N/mm^2 = **40** N/mm^2

Mean value of axial tensile strength; $f_{ctm} = 0.3 \text{ N/mm}^2 \times (f_{ck}/ 1 \text{ N/mm}^2)^{2/3} = \mathbf{3.0} \text{ N/mm}^2$

Secant modulus of elasticity of concrete; $E_{cm} = 22 \text{ kN/mm}^2 \times [f_{cm}/10 \text{ N/mm}^2]^{0.3} = \mathbf{33346} \text{ N/mm}^2$

Partial factor for concrete (Table 2.1N); $\gamma_C = \mathbf{1.50}$

Compressive strength coefficient (cl.3.1.6(1)); $\alpha_{cc} = \mathbf{0.85}$

Design compressive concrete strength (exp.3.15); $f_{cd} = \alpha_{cc} \times f_{ck} / \gamma_C = \mathbf{18.1} \text{ N/mm}^2$

Maximum aggregate size; $h_{agg} = \mathbf{20}$ mm

Reinforcement details

Characteristic yield strength of reinforcement; $f_{yk} = \mathbf{500} \text{ N/mm}^2$

Partial factor for reinforcing steel (Table 2.1N); $\gamma_S = \mathbf{1.15}$

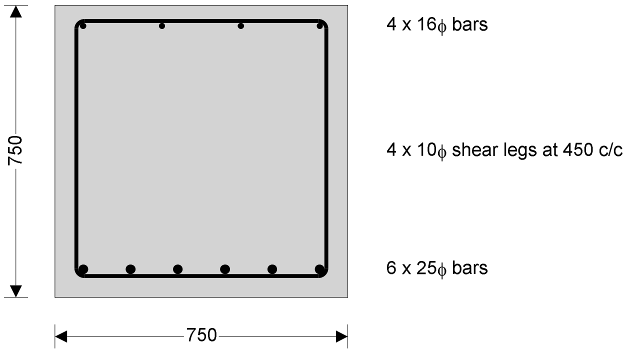
Design yield strength of reinforcement; $f_{yd} = f_{yk} / \gamma_S = \mathbf{435} \text{ N/mm}^2$

Nominal cover to reinforcement

Nominal cover to top reinforcement; $c_{nom\_t} = \mathbf{35}$ mm

Nominal cover to bottom reinforcement; $c_{nom\_b} = \mathbf{50}$ mm

Nominal cover to side reinforcement; $c_{nom\_s} = \mathbf{50}$ mm



Rectangular section in flexure (Section 6.1) - Positive midspan moment

Design bending moment; $M = \mathbf{809}$ kNm

Depth to tension reinforcement; $d = h - c_{nom\_b} - \phi_v - \phi_{bot} / 2 = \mathbf{678}$ mm

Percentage redistribution; $m_r = \mathbf{0}$ %

Redistribution ratio; $\delta = \min(1 - m_r, 1) = \mathbf{1.000}$

$K = M / (b \times d^2 \times f_{ck}) = \mathbf{0.073}$

$K' = 0.598 \times \delta - 0.181 \times \delta^2 - 0.21 = \mathbf{0.207}$

K' > K - No compression reinforcement is required

Lever arm; $z = \min((d / 2) \times [1 + (1 - 3.53 \times K)^{0.5}], 0.95 \times d) = \mathbf{630}$ mm

Depth of neutral axis; $x = 2.5 \times (d - z) = \mathbf{118}$ mm

Area of tension reinforcement required; $A_{s,req} = M / (f_{yd} \times z) = \mathbf{2952} \text{ mm}^2$

Tension reinforcement provided; $6 \times 25\phi$ bars

Area of tension reinforcement provided; $A_{s,prov} = \mathbf{2945} \text{ mm}^2$

Minimum area of reinforcement (exp.9.1N); $A_{s,min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times b \times d = \mathbf{799} \text{ mm}^2$

Maximum area of reinforcement (cl.9.2.1.1(3)); $A_{s,max} = 0.04 \times b \times h = \mathbf{22500} \text{ mm}^2$

Note the Area Provided is less than 0.5% less than design requirement so deemed acceptable as Temporary Works.

Rectangular section in shear (Section 6.2)

Design shear force at span s1; $V_{Ed,max} = \text{abs}(\max(V_{s1\_max}, V_{s1\_red})) = \mathbf{310}$ kN

Angle of comp. shear strut for maximum shear; $\theta_{max} = 45$ deg

Maximum design shear force (exp.6.9); $V_{Rd,max} = b \times z \times v_1 \times f_{cd} / (\cot(\theta_{max}) + \tan(\theta_{max})) = \mathbf{2242}$ kN

PASS - Design shear force at support is less than maximum design shear force

Design shear force ; $V_{Ed} = \mathbf{310}$ kN

Design shear stress; $v_{Ed} = V_{Ed} / (b \times z) =$ **0.656** N/mm^2

Strength reduction factor (cl.6.2.3(3)); $v_1 = 0.6 \times [1 - f_{ck} / 250 \text{ N/mm}^2] =$ **0.523**

Compression chord coefficient (cl.6.2.3(3)); $\alpha_{cw} =$ **1.00**

Angle of concrete compression strut (cl.6.2.3);

$\theta = \min(\max(0.5 \times \text{Asin}[\min(2 \times v_{Ed} / (\alpha_{cw} \times f_{cd} \times v_1),1)], 21.8 \text{ deg}), 45\text{deg}) =$ **21.8** deg

Area of shear reinforcement required (exp.6.13); $A_{sv,req} = v_{Ed} \times b / (f_{yd} \times \cot(\theta)) =$ **452** mm^2/m

Shear reinforcement provided; $4 \times 10\phi$ legs at 450 c/c

Area of shear reinforcement provided; $A_{sv,prov} =$ **698** mm^2/m

Minimum area of shear reinforcement (exp.9.5N); $A_{sv,min} = 0.08 \text{ N/mm}^2 \times b \times (f_{ck} / 1 \text{ N/mm}^2)^{0.5} / f_{yk} =$ **679** mm^2/m

PASS - Area of shear reinforcement provided exceeds minimum required

Maximum longitudinal spacing (exp.9.6N); $s_{vl,max} = 0.75 \times d =$ **508** mm

PASS - Longitudinal spacing of shear reinforcement provided is less than maximum

Minimum bar spacing

Minimum bottom bar spacing; $s_{bot,min} = (b - 2 \times c_{nom\_s} - 2 \times \phi_v - \phi_{bot}) / (N_{bot} - 1) =$ **121** mm

Minimum allowable bottom bar spacing; $s_{bar\_bot,min} = \max(\phi_{bot}, h_{agg} + 5 \text{ mm}, 20 \text{ mm}) + \phi_{bot} =$ **50** mm

Minimum top bar spacing; $s_{top,min} = (b - 2 \times c_{nom\_s} - 2 \times \phi_v - \phi_{top}) / (N_{top} - 1) =$ **205** mm

Minimum allowable top bar spacing; $s_{bar\_top,min} = \max(\phi_{top}, h_{agg} + 5 \text{ mm}, 20 \text{ mm}) + \phi_{top} =$ **41** mm

PASS - Actual bar spacing exceeds minimum allowable

Once the basement has been excavated and the B2 and B1 slabs cast the area behind the temporary capping beam will be excavated and trench sheets installed. Refer to Appendix C for design of sheeting.

These will be back propped from the B1 slab as detailed on drawing 0008.

Assessment along the Bloomsbury Theatre.

The Bloomsbury Theatre is a substantial building which it has been established is founded on a substantial strip footing.

This footing is located at a depth below the proposed capping beam and the service duct and hence no temporary works are required in this area.

Refer to drawing 031-51-1000-DR-Y-00009 – B1

Step in Capping Beam to South West Corner Grid Line 01 / F

Refer to drawing 031-51-1000-DR-Y-00010 – B1

The capping beam steps down by 1500mm to allow services to pass into the basement.

The piles are initially to be brought up to the main capping beam level so that the capping beam is continuous for propping of the excavation. The upper section of the adjacent male piles will be unreinforced so that during excavation a degree of temporary propping will be required.

The excavation depth is 3m, spoil properties are taken from the SI Report by Curtins, Borehole 3.

Use Mabey 5m long M12 sheets propped at a depth of 1m from the surface.

Prop force = 33.8 kN/m

Span 2.4m, Moment = 24.3 kNm shear = 40.6 kN

Using a RMD Slimshore the maximum moment of resistance is 25.3 kNm and shear capacity exceeds 50kN.

Prop ends of RMD waler onto capping beam.

Tunnel Connection to South of Bloomsbury Theatre.

Refer to drawing 031-51-1000-DR-Y-00011 – B1

In this area the new construction runs along and between two existing footings, one to the rear of the Bloomsbury Theatre and one to the "Node".

The drawings illustrate these foundations to be piled so there will be no load imposed on the ground beneath, other than a nominal load.

Initially the pile cap needs to be excavated and formed. This will be within a trench sheeted excavation which will be similar to the excavation in the South West corner so similar ground retention will be provided.

The base of the tunnel above the level of the formation of the adjacent foundations and therefore it is only the ground between the foundations that needs to be supported.

Therefore there will be a maximum face of 1m unsupported.

Use Mabey M6 trench sheets propped to the sides of the existing footing and footed a minimum of 750mm into the ground.

Fix waler anchored back to existing footing, Line Load = 3kN/m

Where there is no footing to the South use a standard RMD waler and prop at max 3m centres.

Appendix A

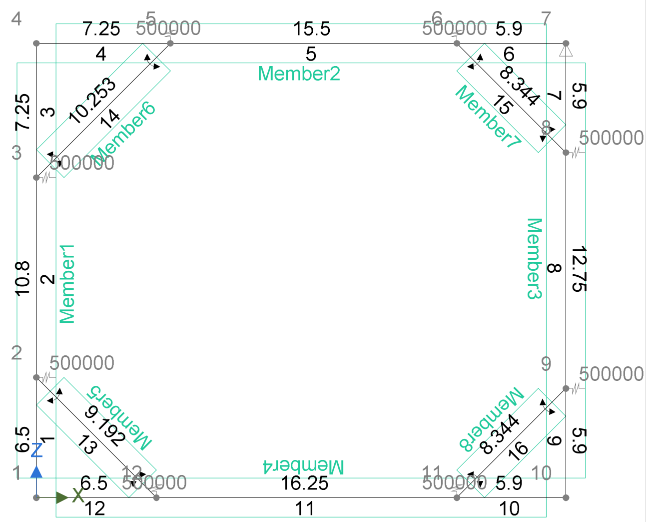
Capping Beam Revised Analysis

ANALYSIS

Tedds calculation version 1.0.10

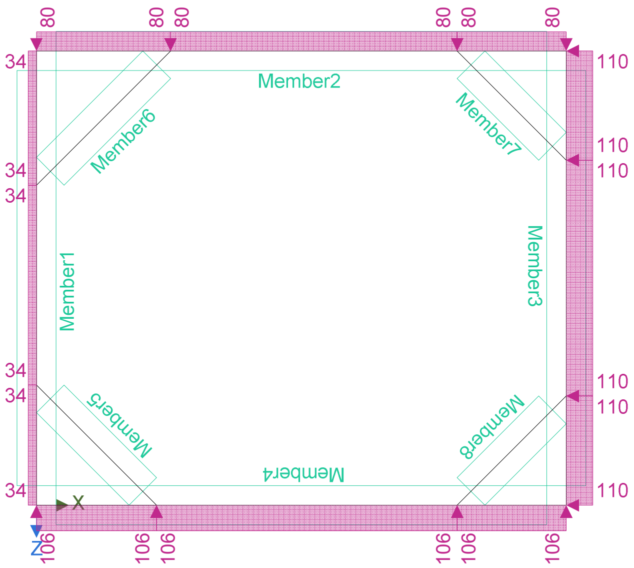
Geometry

Geometry



Loading

Imposed - Loading



Results

Element end forces

Load combination: LoadCombination1 (Strength)

Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	6.5	1	-22	-50.7	-222.6
		2	22	-280.8	-525.4
2	10.8	2	-1528.6	-249	525.4
		3	1528.6	-301.8	-810.6
3	7.25	3	-246.6	-290	810.6
		4	246.6	-79.7	-48.3
4	7.25	4	-79.7	-246.6	48.3
		5	79.7	-623.4	-1414.4
5	15.5	5	-1361.8	-872.2	1414.4
		6	1361.8	-987.8	-2309.9
6	5.9	6	429.7	-786.4	2309.9
		7	-429.7	78.4	241.1
7	5.9	7	313	-129.7	-241.1
		8	-313	-843.8	-1865.5
8	12.75	8	-1478.5	-1025.3	1865.5
		9	1478.5	-1078.5	-2204.7
9	5.9	9	21.7	-916.4	2204.7
		10	-21.7	-57.1	330.4
10	5.9	10	-57.1	-21.7	330.4
		11	57.1	959.8	2565.3
11	16.25	11	-1557.3	1265.5	-2565.3
		12	1557.3	1318.2	2993.1
12	6.5	12	-50.7	1011.5	-2993.1

Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
		1	50.7	22	-222.6
13	9.192	12	-2130.7	0	0
		2	2130.7	0	0
14	10.253	3	-1813.1	0	0
		5	1813.1	0	0
15	8.344	6	-2533.5	0	0
		8	2533.5	0	0
16	8.344	9	-2121.6	0	0
		11	2121.6	0	0

Load combination: (Service)

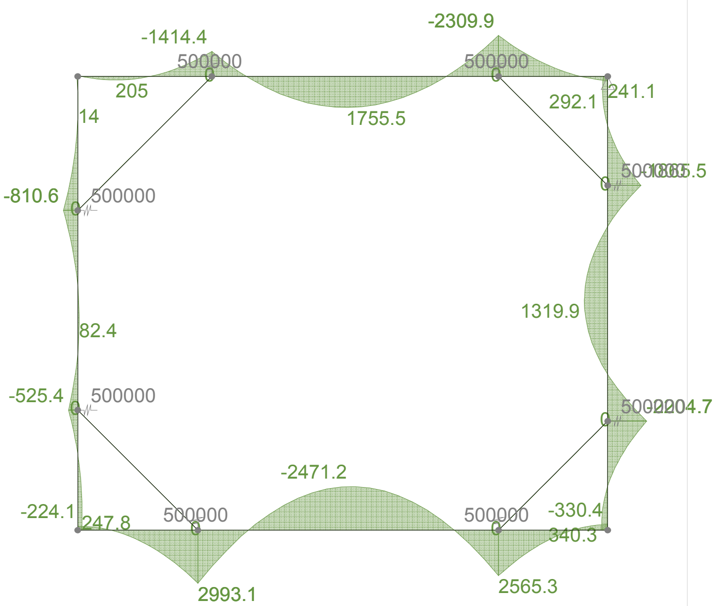
Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	6.5	1	-14.7	-33.8	-148.4
		2	14.7	-187.2	-350.3
2	10.8	2	-1019.1	-166	350.3
		3	1019.1	-201.2	-540.4
3	7.25	3	-164.4	-193.3	540.4
		4	164.4	-53.2	-32.2
4	7.25	4	-53.2	-164.4	32.2
		5	53.2	-415.6	-942.9
5	15.5	5	-907.9	-581.5	942.9
		6	907.9	-658.5	-1539.9
6	5.9	6	286.4	-524.2	1539.9
		7	-286.4	52.2	160.8
7	5.9	7	208.7	-86.5	-160.8
		8	-208.7	-562.5	-1243.6

Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
8	12.75	8	-985.6	-683.5	1243.6
		9	985.6	-719	-1469.8
9	5.9	9	14.5	-611	1469.8
		10	-14.5	-38	220.3
10	5.9	10	-38	-14.5	220.3
		11	38	639.9	1710.2
11	16.25	11	-1038.2	843.7	-1710.2
		12	1038.2	878.8	1995.4
12	6.5	12	-33.8	674.3	-1995.4
		1	33.8	14.7	-148.4
13	9.192	12	-1420.5	0	0
		2	1420.5	0	0
14	10.253	3	-1208.7	0	0
		5	1208.7	0	0
15	8.344	6	-1689	0	0
		8	1689	0	0
16	8.344	9	-1414.4	0	0
		11	1414.4	0	0

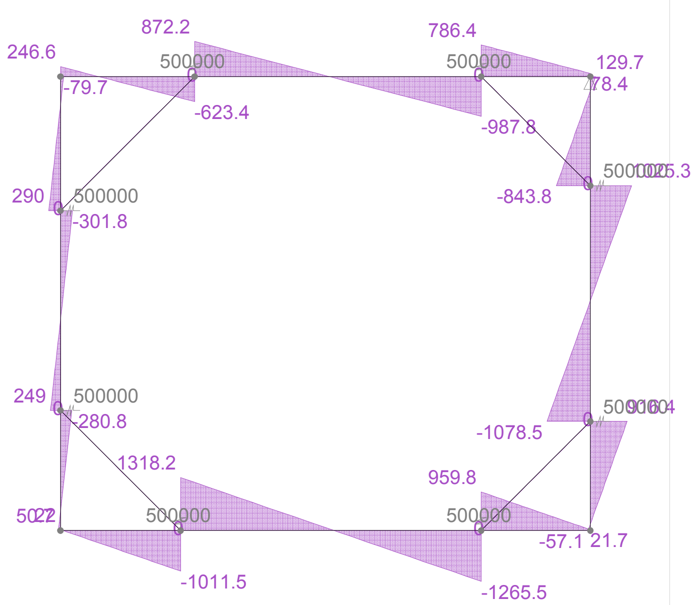
Bridges Pound Consulting Engineers	Location				Job No	
	UCL for Mace				M1993	
	Basement Temporary Works Design				Sheet No A 5	
	By SMH	31/03/16	Checked RLN	Mar 16	Approved SMH	Apr 16

Forces

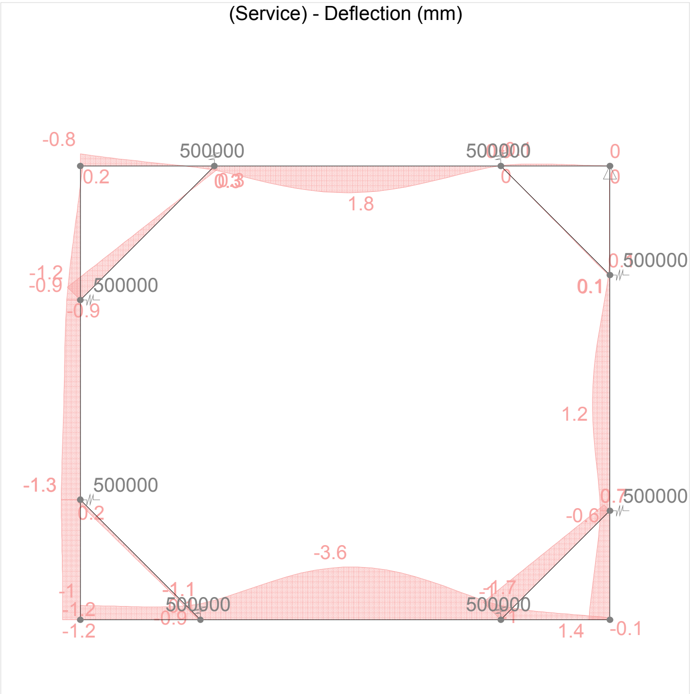
Strength combinations - Moment envelope (kNm)



Strength combinations - Shear envelope (kN)



BRIDGES POUND CONSULTING ENGINEERS	Location	UCL for Mace			Job No	M1993
	Basement Temporary Works Design				Sheet No	A 6
	By SMH	31/03/16	Checked RLN	Mar 16	Approved SMH	Apr 16



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Appendix B

MGF Prop Details

Description

Simple to assemble, heavy duty, modular bracing strut systems designed primarily to be used as cross struts with the MGF 305/406 UC hydraulic bracing system on major excavations. The system can also be used to prop reinforced concrete piles and capping beams forming the walls of major basements structures. Each strut comprises hydraulic ram assemblies together with various length strut extension bars. The system can support loads of up to 2500kN and span up to 30.0m unsupported. Components are extremely heavy and are normally assembled on site prior to being lifted into place and installed within the excavation using large cranes. A variety of end bearings are available allowing the struts to be used at a range of angles.

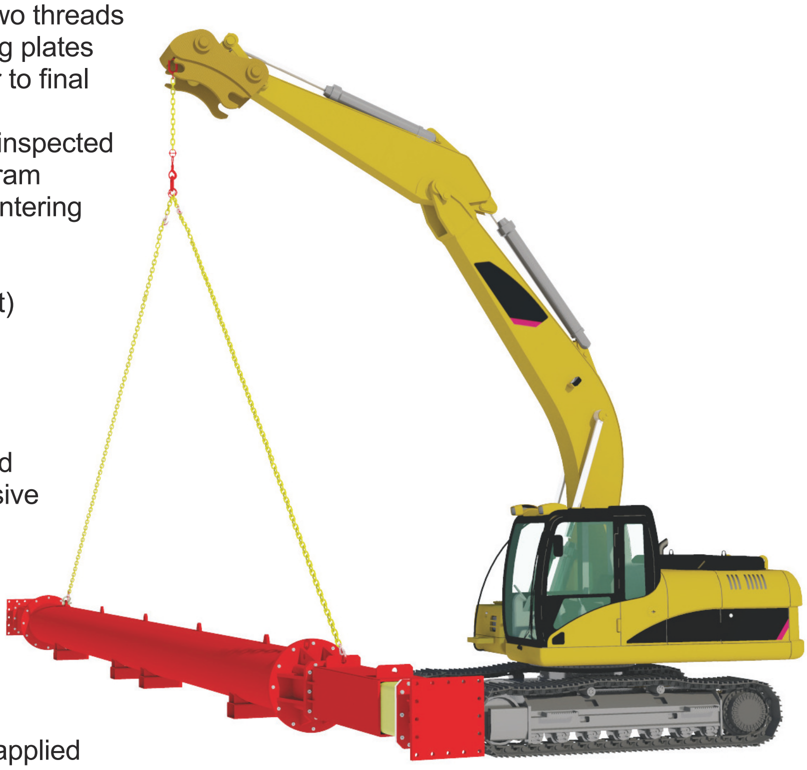
Fabricated from API grade X70 610x12.5 hollow circular steel section, and S355 grade 660x20/25 CHS, the extensions are quickly assembled into the required strut lengths using circular flanged plates c/w bolt, nut and washer assemblies. Final length adjustment is provided by a double acting hydraulic ram providing up to 800mm of stroke. Once located at the correct line and level the struts are pre-loaded (or tightened) against the faces to be supported using a hydraulic pump on the ram. Pre-loading of the legs ensures the strut cannot slip, takes up any slack or hogging in the system and minimises the extent of potential ground movements. Handling points are provided at regular intervals on each leg to assist assembly / removal.

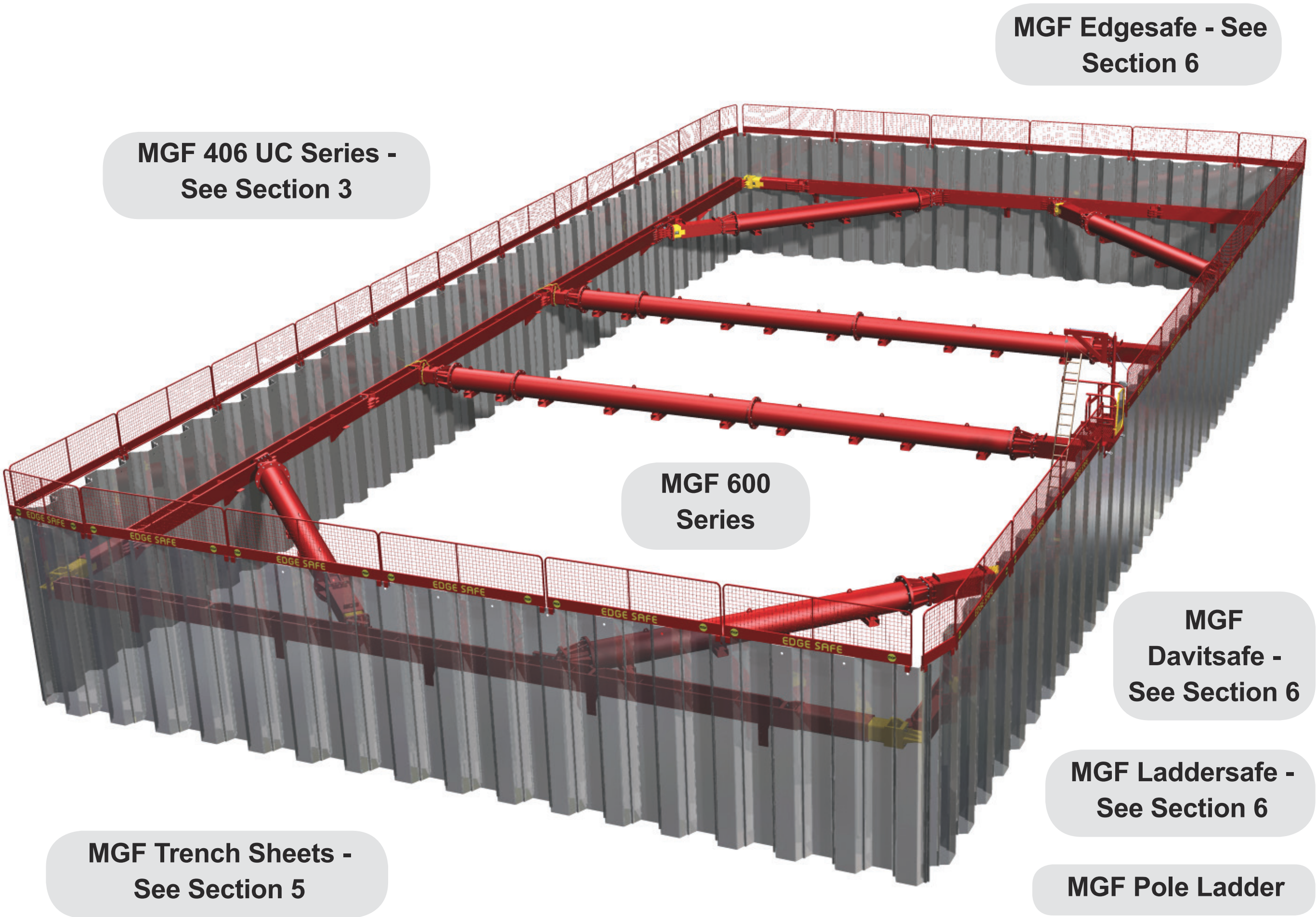
MGF can supply the systems with a full range of suitable handling chains, hydraulic pump installation kits (including bio-degradable shoring fluid and hydraulic hoses) and confined spaces regime equipment.

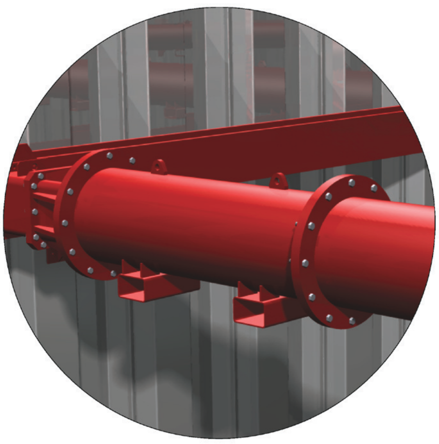
Manufactured and designed in accordance with BS EN 14653:2005 PARTS 1 and 2 Manually operated Shoring Systems for Groundwork Support and BS 5975 (2008) Code of Practice for Temporary Works Procedures and the Permissible Stress Design of Falsework.

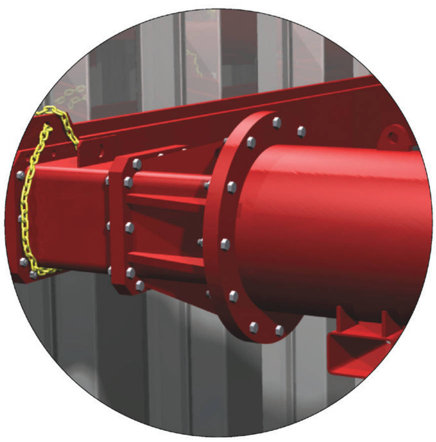
Product Notes

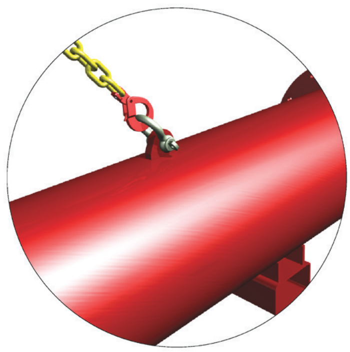
1. Strut systems are heavy and should only be assembled, installed and removed by competent persons in accordance with a site specific detailed design & installation sequence and MGF installation guidelines.
2. Installation is normally carried out by assembling the complete strut and then lowering into place (subject to crane / excavator capacity). Struts are normally long and unbalanced (due to the weight of ram/jack unit) and great care must be taken in preparing the lift / maintaining lift angle (tag lines strongly recommended). On the ram assembly max pre-load pressure of 100Bar (1500psi) must not be exceeded unless design states otherwise.
3. Additional restraining chains or support brackets are normally provided to the brace at intermediate strut locations to carry the additional strut weight.
4. Ensure struts are fully pre-loaded or tightened, end fixings fully packed, all hydraulic ram isolation valves are closed prior to releasing strut from lifting chains and commencing works. When assembling on site ensure that all pins and retaining clips are in place and secured and all flange plate bolts are installed and fully tightened / torqued with a minimum two threads visible beyond the nut. Any gaps in bearing plates must be securely packed using grout prior to final pre-loading of the hydraulic rams.
5. Individual components should be visually inspected for damage, excessive deflection, loss of ram pressure or loose locking collars prior to entering the excavation.
6. Safe access/egress, edge protection (for personnel) and barrier protection (for plant) should always be considered.
7. Prior to removal of systems the complete weight of the strut must be independently supported. Once this is accomplished the hydraulic rams (or struts) must be released and retracted to avoid the need for excessive extraction forces.
8. When installing struts at angles great care must be taken to ensure that the angles match the design, all shear stops are in place and all elements are supported/packed and capable of transmitting loads effectively. On large unsupported spans the pre-load must be applied prior to removing vertical support to minimise sagging.







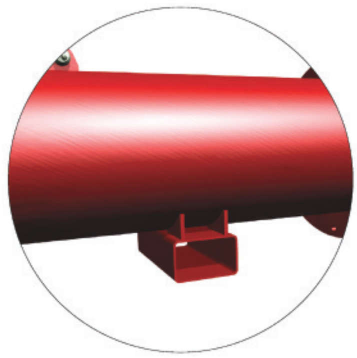




Strut Flange Connection Detail
600 Series Struts and Extensions are connected to each other via a flange plate (Φ850x30mm) using 12 No. grade 8.8 M24 bolts c/w nuts and washers (recommended min. torque 400Nm).

Transition Flange Connection Detail
The transition adaptor is connected to the hydraulic strut or 400 series extension via a square flange plate (520x520x30mm) and connects to 600 Series via a circular end plate(Φ850x30mm) both connections using 12 No. grade 8.8 M24 bolts c/w nuts and washers (recommended min. torque 400Nm).

Handling Points
WLL = 12.0t
Strut assemblies are lifted and handled by attaching MGF lifting chains to the handling/restraining points as shown.
Assemblies can also be handled using a fork lift on the pockets on the underside of the extensions.



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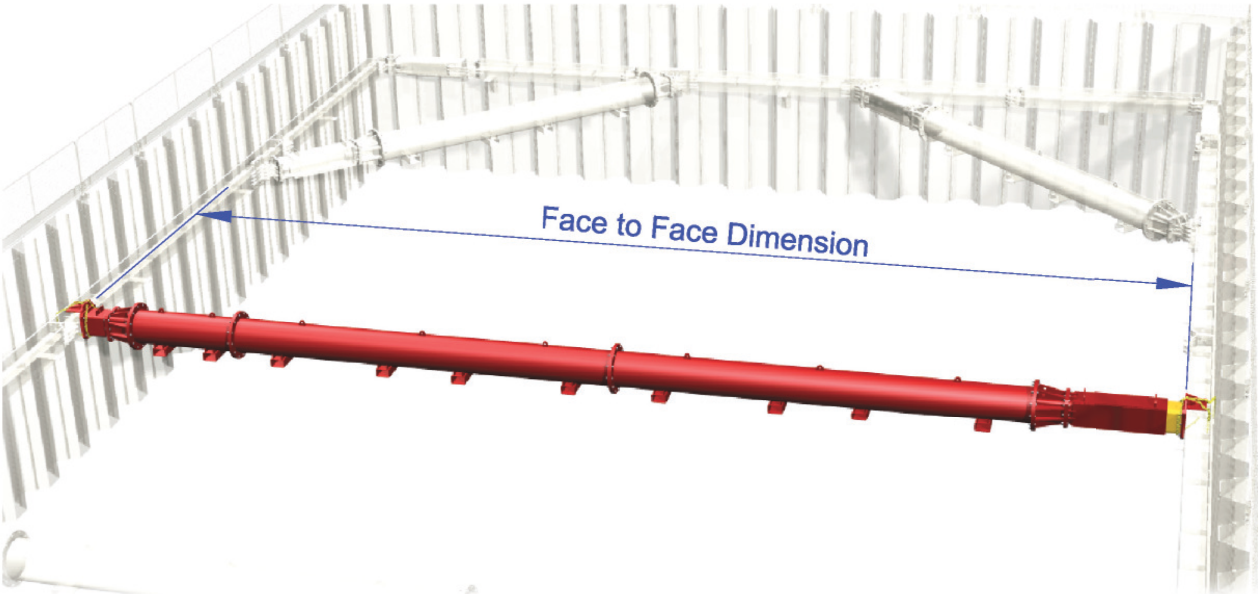
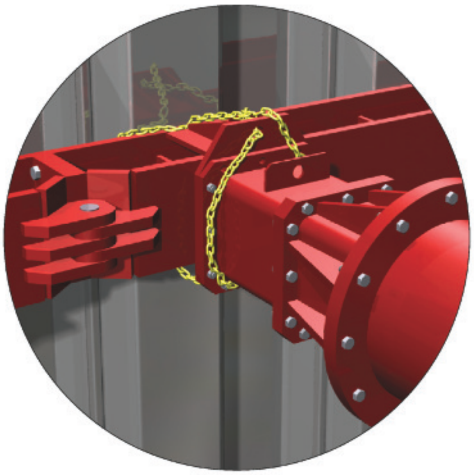
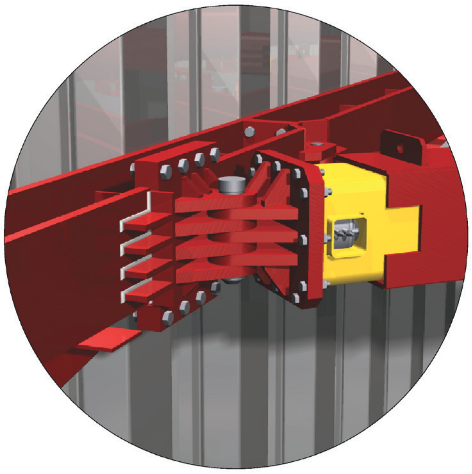
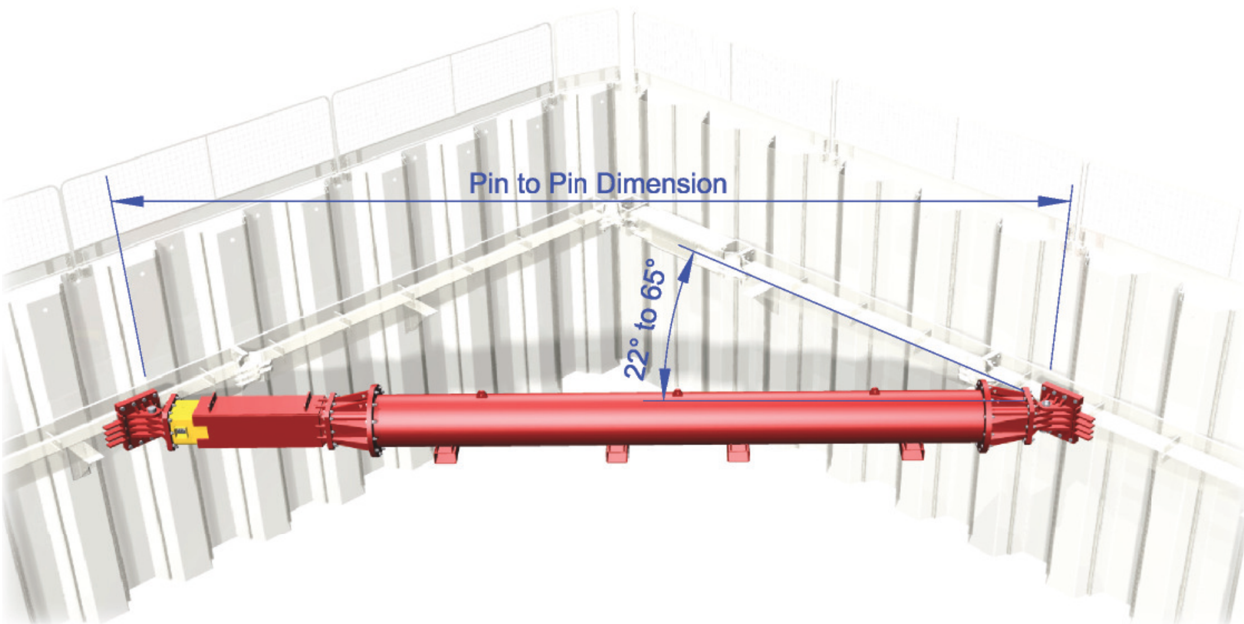




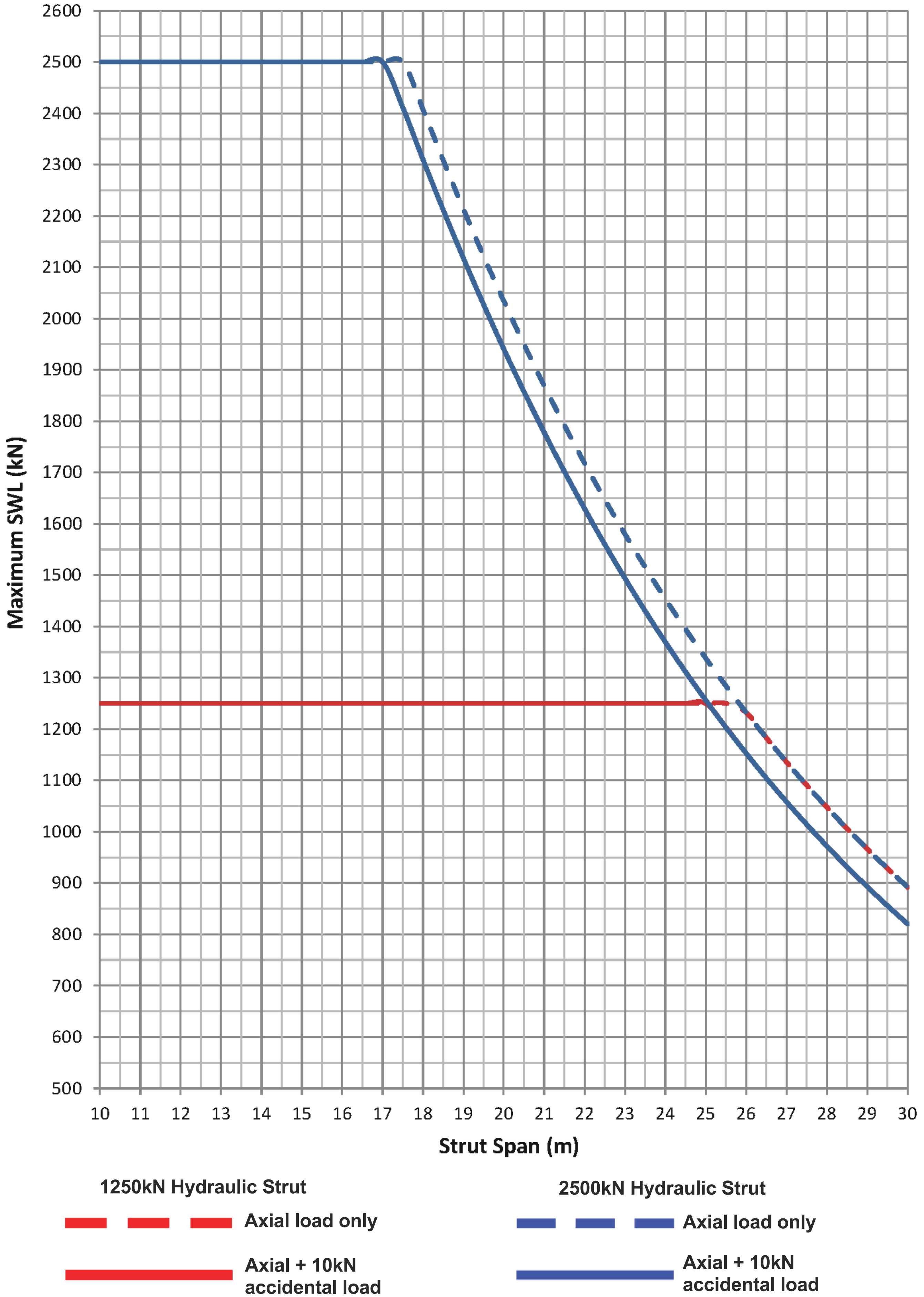


Cleat End Bearing Detail
The end cleat is bolted to the strut or extension/transition using 9 No. grade 8.8 M24 countersunk bolts c/w nuts and washers. The cleat then sits on the UC section. When using this end detail MGF recommend that restraining chains are used to prevent the strut being dislodged if struck accidently.

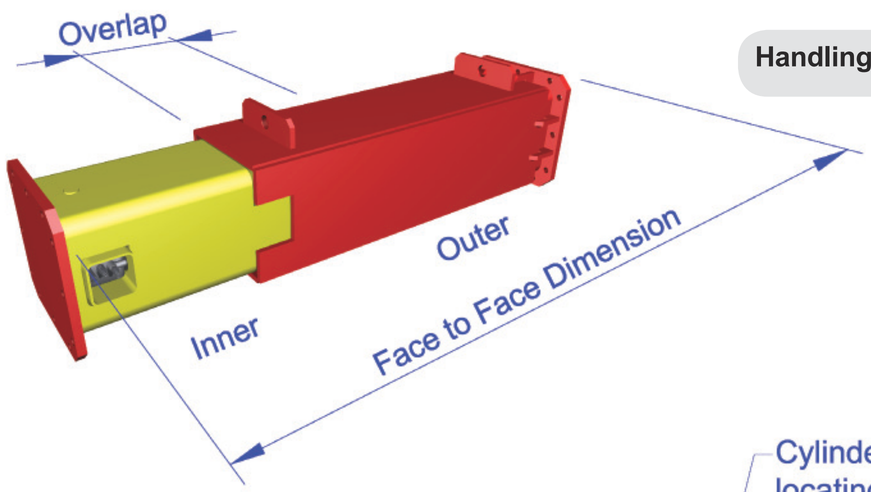
Swivel End Bearing Detail
Swivels can be anchored directly to concrete or clamped to the UC Brace System using 2 No. swivel clamps as detailed on page 4.5.12.



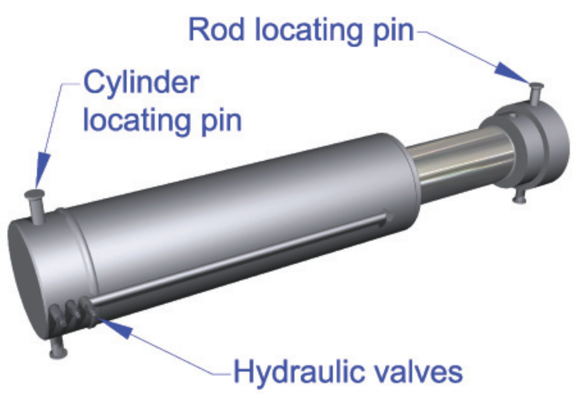
Safe Working Load for MGF 600 Series (kN)



Curves include allowance for self weight deflection, eccentricity and fabrication tolerances.

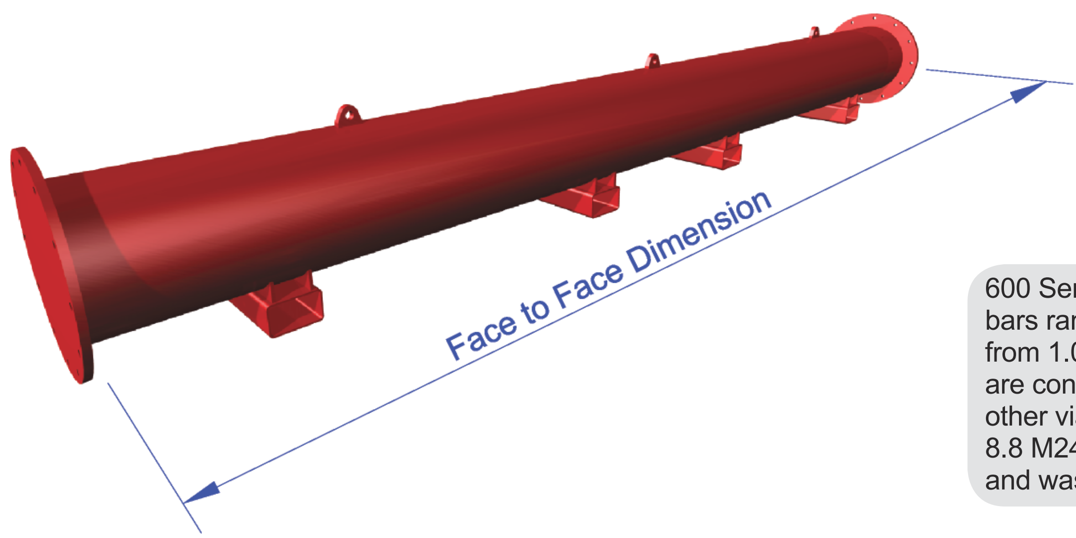


Handling Point WLL = 8t



1250kN and 2500kN hydraulic strut assembly comprises inner and outer sleeved steel box sections housing a double acting (DA) hydraulic ram to provide up to 800mm of leg adjustment.

Product Description	Product ID	Face to Face Dimension (mm)		Weight (kg)
		Min.	Max.	
1250kN Hydraulic Strut	8.400	1476	2276	1047
2500kN Hydraulic Strut	8.500	1685	2485	1716



600 Series extension bars range in length from 1.0m to 11.5m and are connected to each other via 12 No. grade 8.8 M24 bolts c/w nuts and washers.

Product Description	Product ID	Weight (kg)
600 Series 1.0m Extension	9.601	480
600 Series 3.0m Extension	9.603	880
600 Series 4.0m Extension	9.604	1125
600 Series 7.0m Extension	9.607	1680
600 Series 11.5m Extension	9.608	2520

Hydraulic Ram	Inner Section	Outer Section
Specification	350x350x16 SHS (+ 8 No. 100x6 thk. stiffening plates)	400x400x16 SHS
Material Grade	S355	S355
Unit Mass	166kg/m	191kg/m
Axial SWL	1250kN	1250kN
Moment SWL	277kNm	277kNm

Hydraulic Ram	Inner Section	Outer Section
Specification	400x400x16 SHS	450x450x20 SHS
Material Grade	S355	S355
Unit Mass	191kg/m	275kg/m
Axial SWL	2500kN	2500kN
Moment SWL	277kNm	277kNm

Extension Bar	600 Series
Specification	610x12.5 CHS
Material Grade	X70
Unit Mass	184kg/m
Axial SWL	2500kN
Moment SWL	1418kNm

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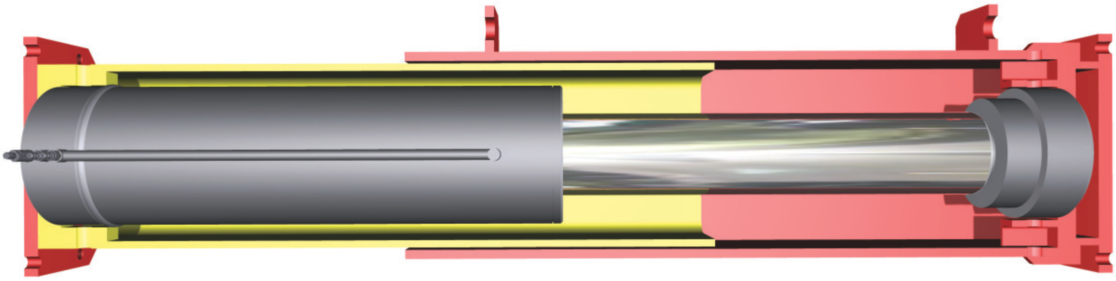
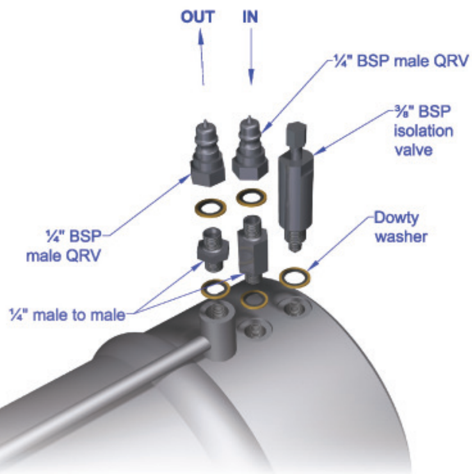
1250kN and 2500kN Double Acting Hydraulic Ram Assembly



Hydraulic Cylinder	1250kN Double Acting	2500kN Double Acting
Material	Steel	Steel
Bore	200mm	250mm
Max .Working Pressure	400 Bar (6000 psi)	500 Bar (7250 psi)
Test Pressure	400 Bar (6000 psi)	500 Bar (7250 psi)
Approx. Working Stroke	800mm	800mm
Axial SWL	1250kN	2500kN
Min. FOS (by test)	2	2
Working Temp Range	-50ºC to +50ºC	-50ºC to +50ºC
Approx. Pre-Load	300kN	500kN
Locating Pins	Φ30	Φ30



Shoring fluid is pumped into the full bore side of the piston through the central male quick release valve (QRV) to extend the ram.
At the same time fluid from the return side of the piston is returned to the pump via the outer male QRV. Retraction is a reverse of extension.
Ensure isolation valve is closed to maintain pre-load pressure and before release/connection of QRV's.

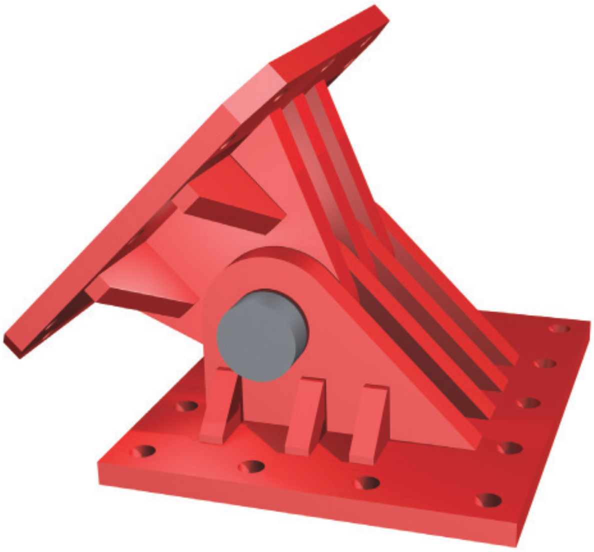
Motorised Pump Units

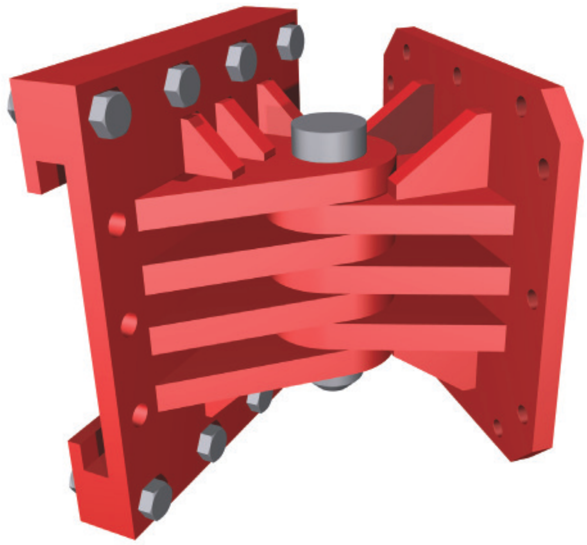
The motorised pumps are used to extend and retract the 600 Series double acting hydraulic rams.
The pumps contain neat bio-degradable Houghto Safe SF25 shoring fluid.
Maximum recommended installation pressure 1500 psi (100 Bar).
MGF supply 2 different types of motorised pump, electric and diesel.

	Electric Pump	Diesel Pump
Rating	110V, 6.5kVA	8kW
Product ID	8.4001 / 8.4003	8.4006
Capacity	120 / 190 litres	100 litres
Shoring Fluid	Houghto Safe SF25B	Houghto Safe SF25B
Installation Pressure	0-1500 psi	0-1500 psi

2500kN Type A Swivel Assembly

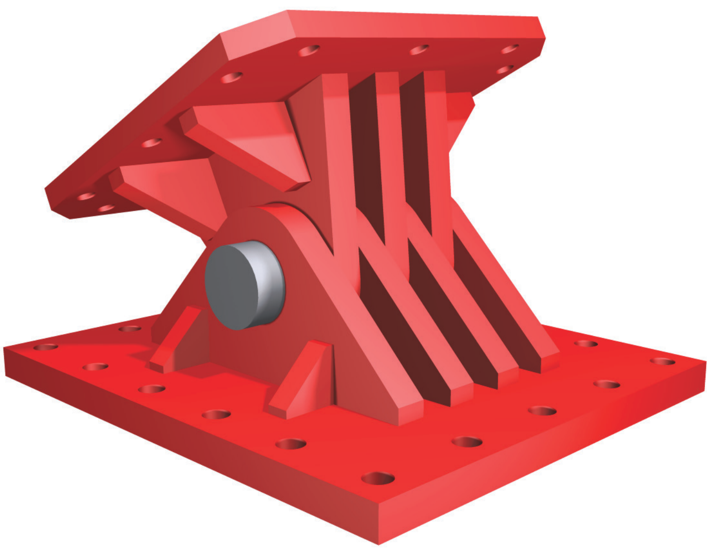
These swivels can be connected directly to concrete structures or the 406 UC Brace Systems by bolting on the associated clamp assemblies detailed on page 4.4.12.

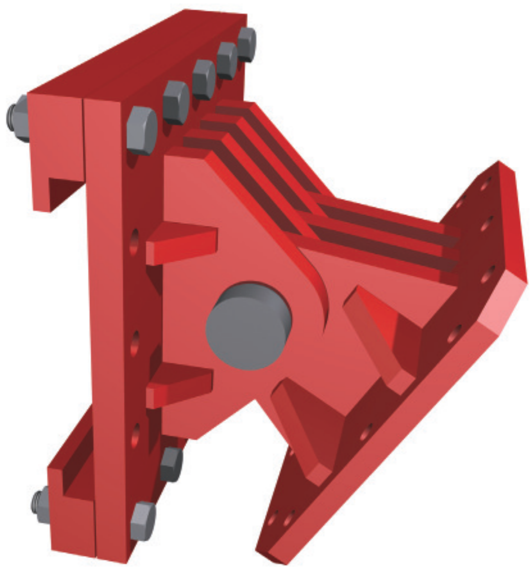




400 Series Swivel	Type A	Type B
Product ID	9.704	9.310
Weight	264kg	320kg
Knee Brace/Cross Strut Operating Range	22°- 65°	65°- 90°
Axial SWL	2500kN	2500kN
Swivel Base Plate	500 x 600 x 30mm thk. (S355)	600 x 600 x 30mm thk. (S355)
Base Plate Hole Details	14 No. Φ32 holes	16 No. Φ32 holes
Pin Detail	Φ90 (817M40/EN24T)	Φ90 (817M40/EN24T)

2500kN Type B Swivel Assembly





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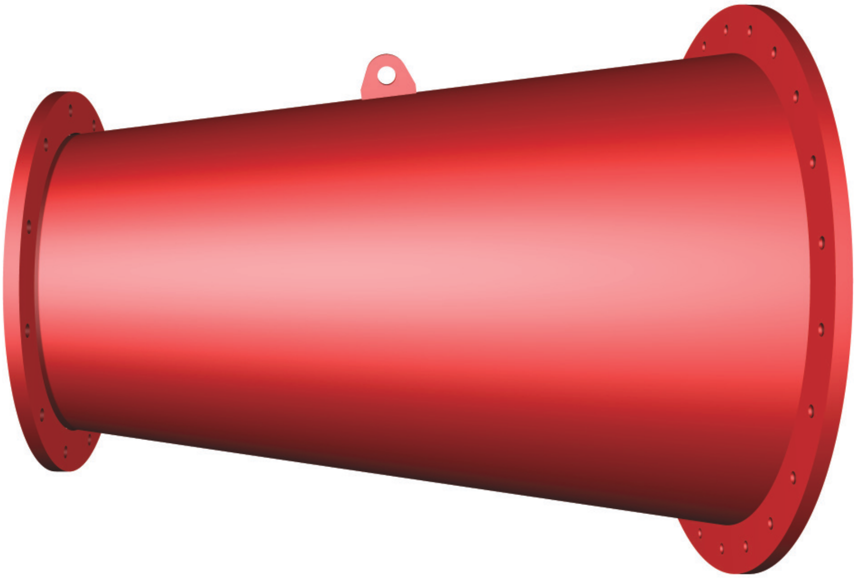
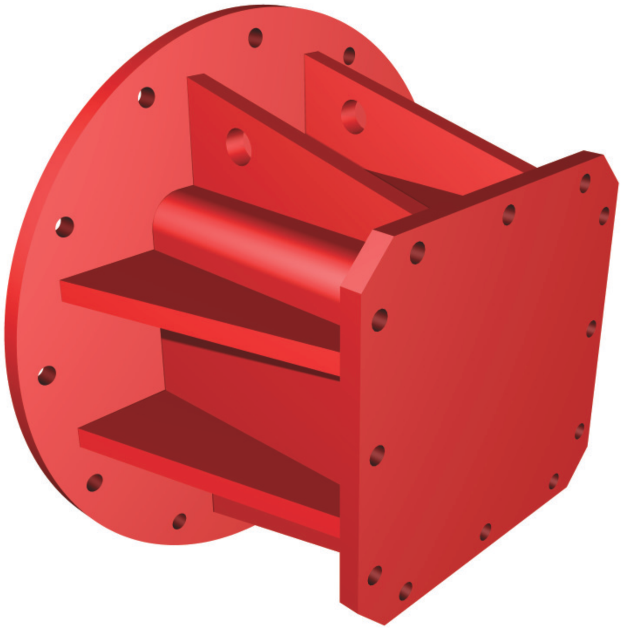
600 Series Adaptors

1000/600 Series Transition

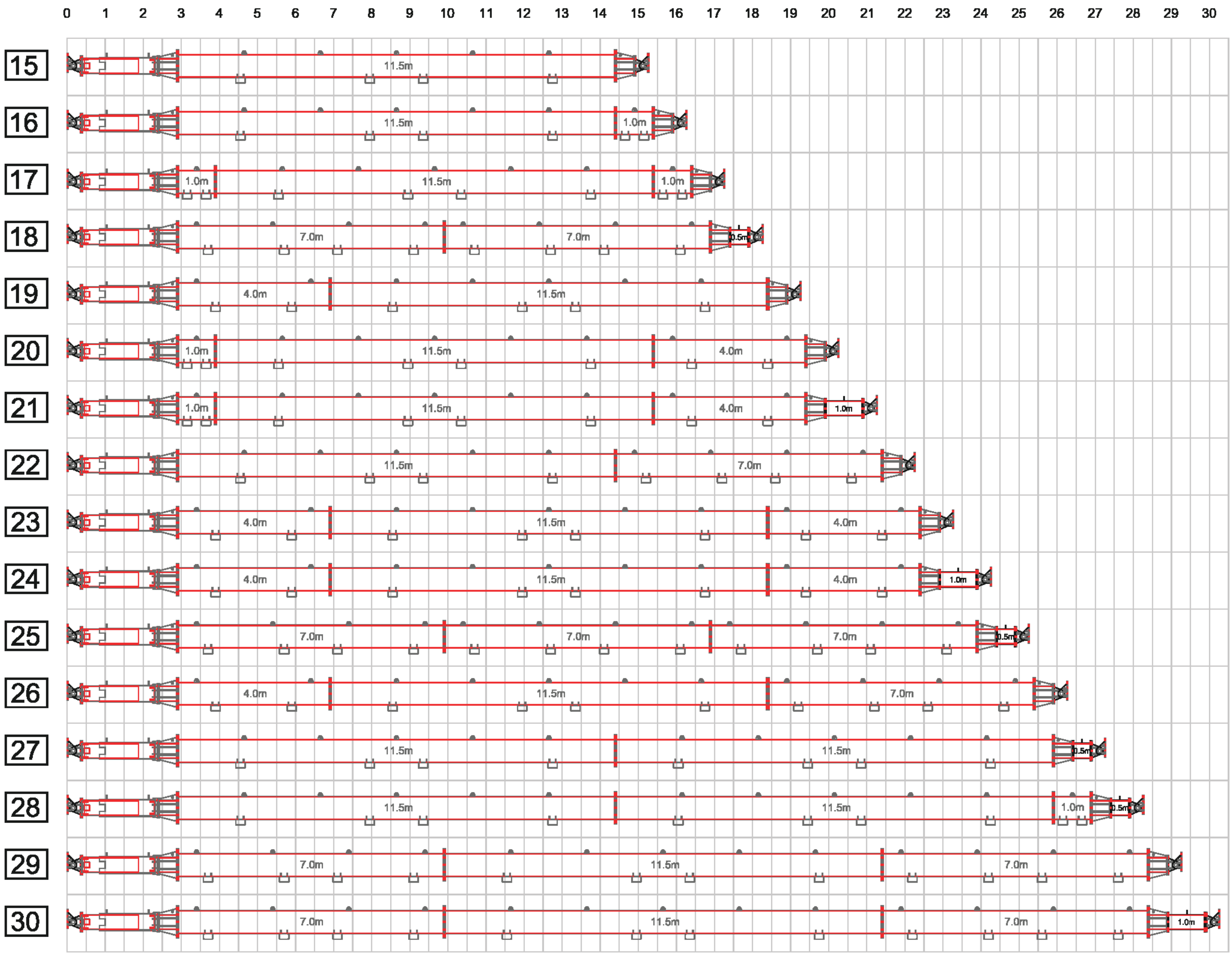


Transition	1000/600	600/400
Product ID	9.800	9.604
Weight	475kg	352kg
Material	14.6 thk. tube, X65	400x400x16 SHS, S355
Bolting Details	12/24 No. grade 8.8 M24 bolts c/w nuts and washers	12 No. grade 8.8 M24 bolts c/w nuts and washers
Strut Adaptor SWL	2500kN	2500kN
Axial SWL	2500kN	2500kN
Moment SWL	1125kNm	396kNm
Joint Moment SWL	396/1125 kNm	277/396 kNm

600/400 Series Transition



600 Series Recommended Extension Combinations



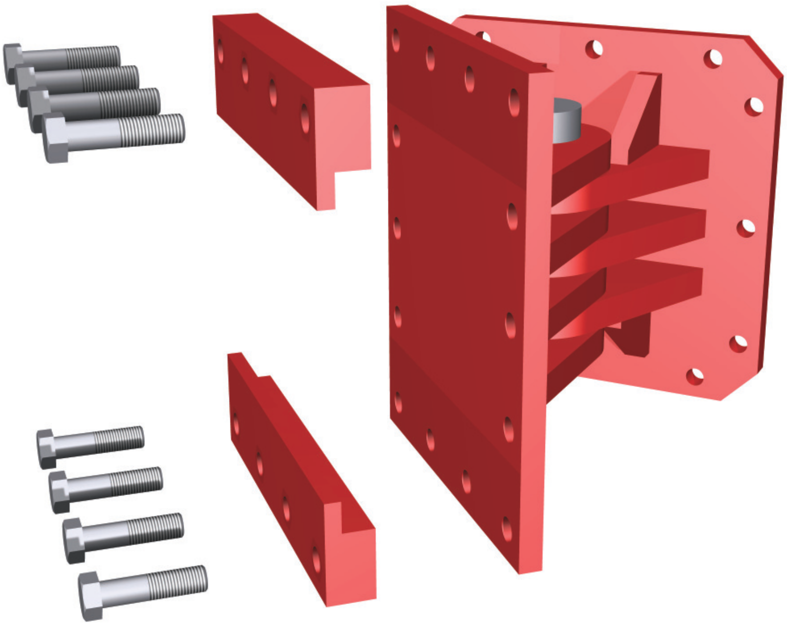
N.B	*Single 0.5m 400 Series extensions should be added to these combinations for intermediate dimensions.* *The strut assemblies are shown at mid-stroke, so each length can vary by up to 400mm in either direction.* *Individual 7m pieces can be exchanged for a 3m and 4m.* *Additional compatible extensions are available (660 diameter / 1000 Series). Contact MGF Design department for details.*

The above strut combinations use the 600 Series Extensions (610 tube).

Tel. 01942 402 704

Face to Face Dimension (m)	2500kN Hydraulic		
	Min. Length (mm)	Max. Length (mm)	Leg Weight (kg)
15	14875	15675	5580
16	15875	16675	6060
17	16875	17675	6540
18	17875	18675	6634
19	18875	19675	6705
20	19875	20675	7185
21	20875	21675	7495
22	21875	22675	7260
23	22875	23675	7830
24	23875	24675	8140
25	24875	25675	8314
26	25875	26675	8385
27	26875	27675	8314
28	27875	28675	8794
29	28875	29675	8940
30	29875	30675	9250

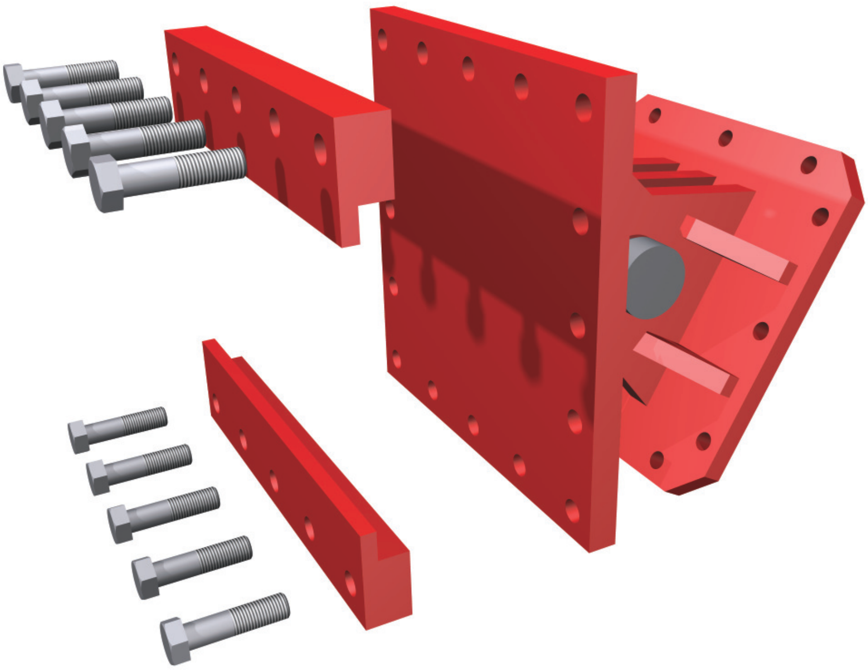
2500kN Swivel Clamping Plates Type A



Swivel Clamp Type A is to be used on 2500kN Swivel Type A, when used on a knee brace connected to 305 UC/406 UC.

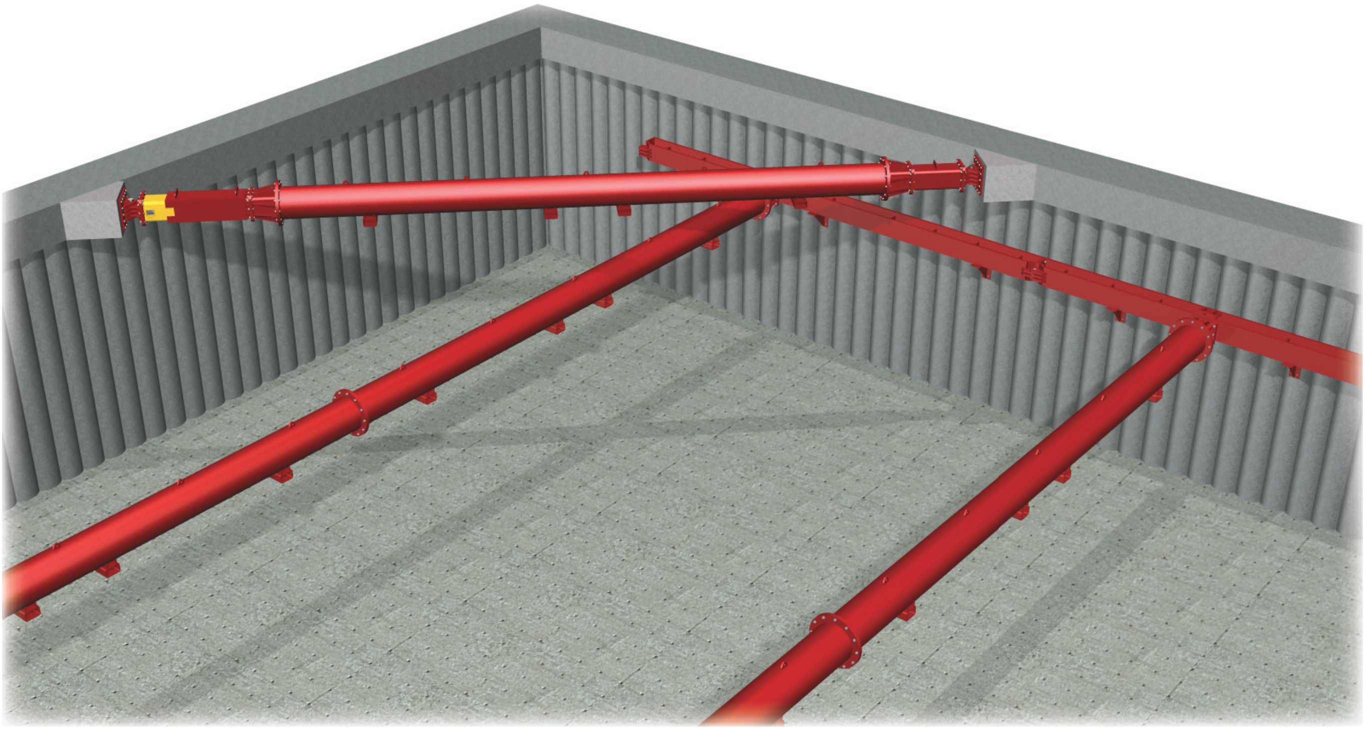
Product ID	9.317A	9.317B
Weight	46kg	54kg
Material	30mm & 40mm thk. flat, 500 long, S275	30mm & 40mm flat, 600 long, S275
Bolting Details	8 No. grade 8.8 M30 bolts c/w nuts and washers	10 No. grade 8.8 M30 bolts c/w nuts and washers
Bearing SWL	2500kN	2500kN

2500kN Swivel Clamping Plates Type B

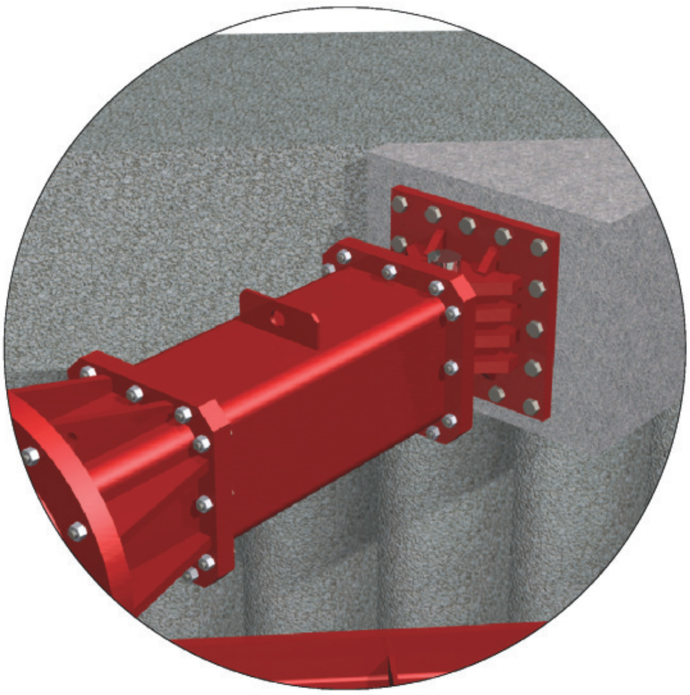


Swivel Clamp Type B is to be used on 2500kN Swivel Type B, when used as a cross strut connected to 305 UC/406 UC.

Tel. 01942 402 704



Typical Basement Wall Application



Typical bearing detail on RC corbel.

Appendix C

Trench Sheeting Calculations

Pile geometry

Pile top Level	23.15	m
Pile Length	5	m
Pile toe level	18.15	m
Active ground slope	0	Degrees (To horizontal)

Soils and ground water initial data (Soils data given for active and passive sides)

Initial Ground Water level 15

Top Level m	Description	Bulk Dens kN/m3	Sat' Dens kN/m3	Young Mod kN/m2	Young Inc. kN/m3	Cu C' kN/m2	C Inc. kN/m3	Phi Deg	Wall Shear Ratio	Ka Kp	Kac Kpc
23.15	Hardcore	17.00	18.00	48000	0			40	.50	.19	
								40	.50	8.38	
22.75	Clay Fill	19.00	20.00	13867	0			26	.50	.35	
								26	.50	3.40	
21.80	GRAVEL	21.00	23.00	46800	0			38	.50	.21	
								38	.50	7.23	
21.25	Sandy CLAY	20.50	20.50	48000	0			29	.50	.31	
								29	.50	4.04	
17.85	Sandy CLAY	19.50	19.50	30000	0			28	.50	.32	
								28	.50	3.81	

Construction sequence

Stage Ref	Stage Type	Level or Angle m/deg.	Load kN(/m)	Offset m	Width m	Length m
1	Active surcharge	23.15	10.0	.0		
2 A	Passive side excavation	22.00				
3	Insert prop	22.15				
4 A	Passive side excavation	20.15				

Code of practice

Code of practice or reference document	Eurocode 7 ULS Design Approach 1 Combination 1
Application of pressures for stability	Not applicable for FOS=1 on moments
FOS on moments (stability check)	1.00
ULS factor on Tan(Phi) values	1.00
ULS fFactor on drained cohesion values	1.00
ULS factor on undrained cohesion values	1.00
ULS factor on active soil pressures	1.35
ULS factor on passive soil pressures	1.00
ULS factor on active water pressures	1.35
ULS factor on passive water pressures	1.35
ULS factor on loads applied to the soil	1.11
ULS factor on loads applied to the wall	1.50
FOS on embedment (stability check)	1.00
Correction factor on cantilever embedment	1.20

Wall analysis detail options

Nominal Phi for load distribution	30.0	Degrees
Depth of water filled tension cracks	.0	m
Density of water	10.0	kN/m3
Minimum equivalent fluid density	5.0	kN/m3
Depth of passive softened soil	1.0	m
Continuity model for wall analysis	Pins at second and lower props	

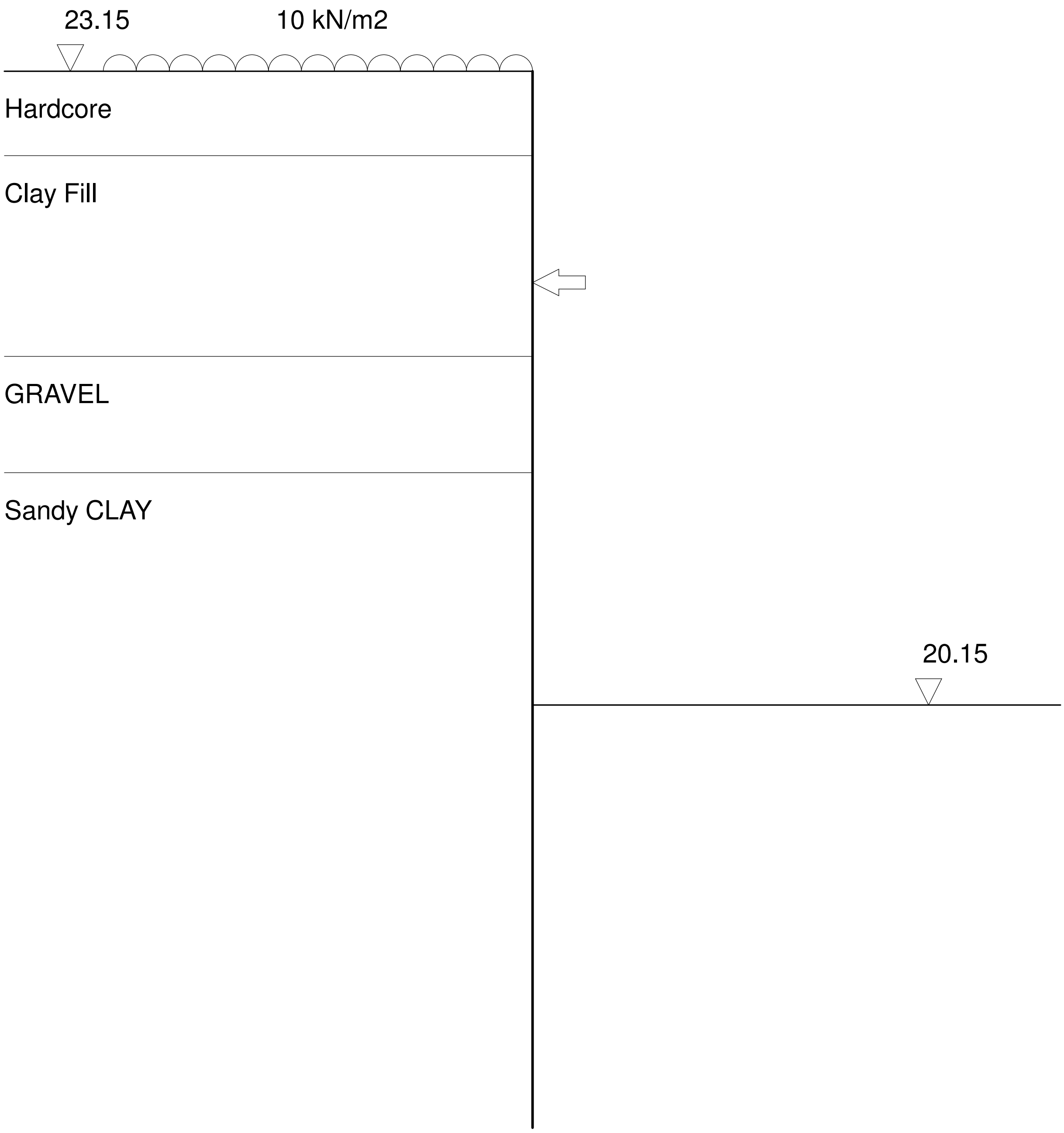
Deflection parameters

Wall moment of inertia	801	cm4/m
Wall Youngs modulus	210000000	kN/m2

Properties for prop at 22.15

Prop/Tie cross sectional area	200	cm2 each
Prop/Tie Youngs modulus	210000000	kN/m2
Prop/Tie length	10.0	m
Prop/Tie spacing	6.0	m
Waling moment of inertia	Waling deflection not included	
Waling Youngs modulus	Waling deflection not included	
Prop/Tie preload	0	kN
Initial lack of fit	0.0	mm

Stage ref. **4**
Stage type **Passive side excavation**



Tabular results from analysis of stage ref 4

Calc Level m	Active Vert kN/m2	Active Earth kN/m2	Active Water kN/m2	Pas' Vert kN/m2	Pas' Earth kN/m2	Pas' Water kN/m2	Total Nett kN/m2	Bend. Moment kNm/m	Shear Force kN/m	Defl't mm	Prop Force kN/m	FOS
23.15	11.1	2.8	.0	.0	.0	.0	2.8	0	-.1	-2.7		.00
23.00	13.6	3.5	.0	.0	.0	.0	3.5	.1	-.6	-2.2		.00
22.75	17.9	4.6	.0	.0	.0	.0	4.6	.3	-1.6	-1.9		.00
22.75	17.9	8.4	.0	.0	.0	.0	8.4	.3	-1.6	-1.8		.00
22.15	29.3	13.8	.0	.0	.0	.0	13.8	3.1	-8.3	3.5	33.8	.00
22.15	29.3	13.8	.0	.0	.0	.0	13.8	3.1	25.5	3.5		.00
22.00	32.1	15.1	.0	.0	.0	.0	15.1	-.5	23.3	4.8		.00
22.00	32.1	15.1	.0	.0	.0	.0	15.1	-.5	23.3	4.8		.00
21.80	36.0	16.9	.0	.0	.0	.0	16.9	-4.9	20.1	6.7		.00
21.80	36.0	10.1	.0	.0	.0	.0	10.1	-4.9	20.1	6.7		.00
21.25	47.5	13.3	.0	.0	.0	.0	13.3	-14.2	13.7	10.8		.00
21.25	47.5	19.7	.0	.0	.0	.0	19.7	-14.3	13.7	10.8		.00
21.00	52.6	21.8	.0	.0	.0	.0	21.8	-17.0	8.5	12.0		.00
20.98	53.0	22.0	.0	.0	.0	.0	22.0	-17.2	8.1	12.0		.00
20.15	70.0	29.0	.0	.0	.0	.0	29.0	-15.5	-13.1	10.6		.00
20.15	70.1	29.0	.0	.0	.0	.0	29.0	-15.5	-13.1	10.6		.00
20.00	73.1	30.3	.0	3.1	12.4	.0	17.9	-13.3	-16.6	9.8		.04
19.05	92.6	38.4	.0	22.6	91.3	.0	-52.9	0	0	5.1		1.00
19.00	93.6	38.8	.0	23.6	95.3	.0	-56.4	0	0	4.8		1.06
18.15	111.1	46.0	.0	41.0	165.7	.0	-119.6	0	0	.6		2.03

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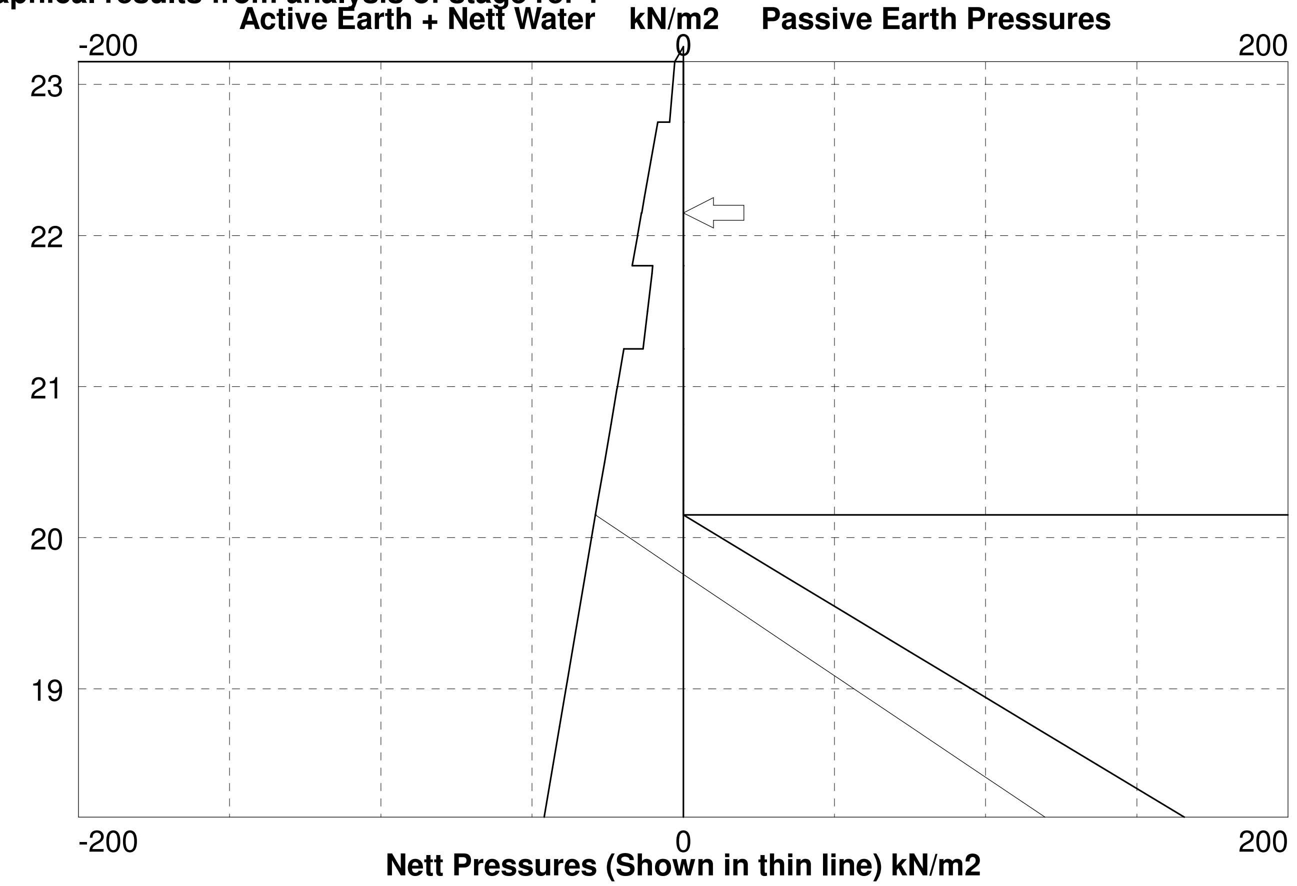
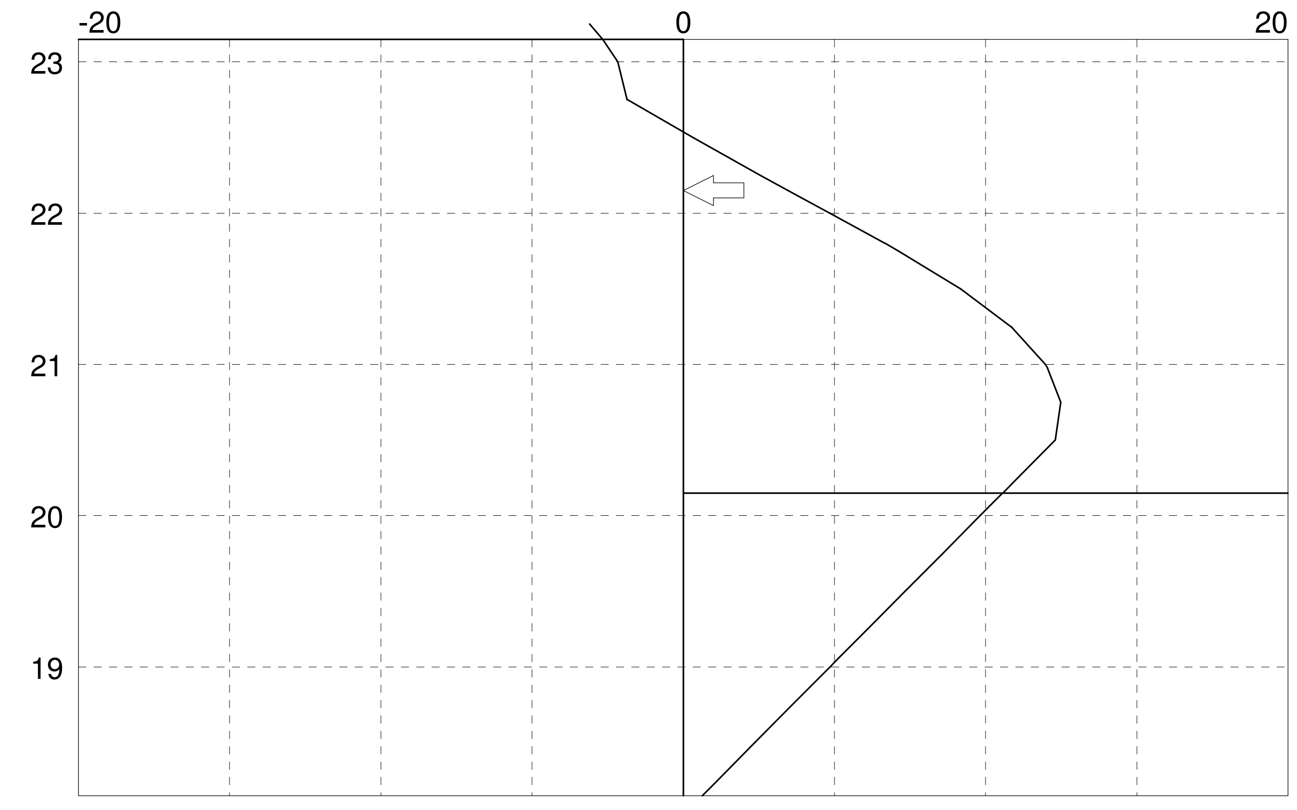
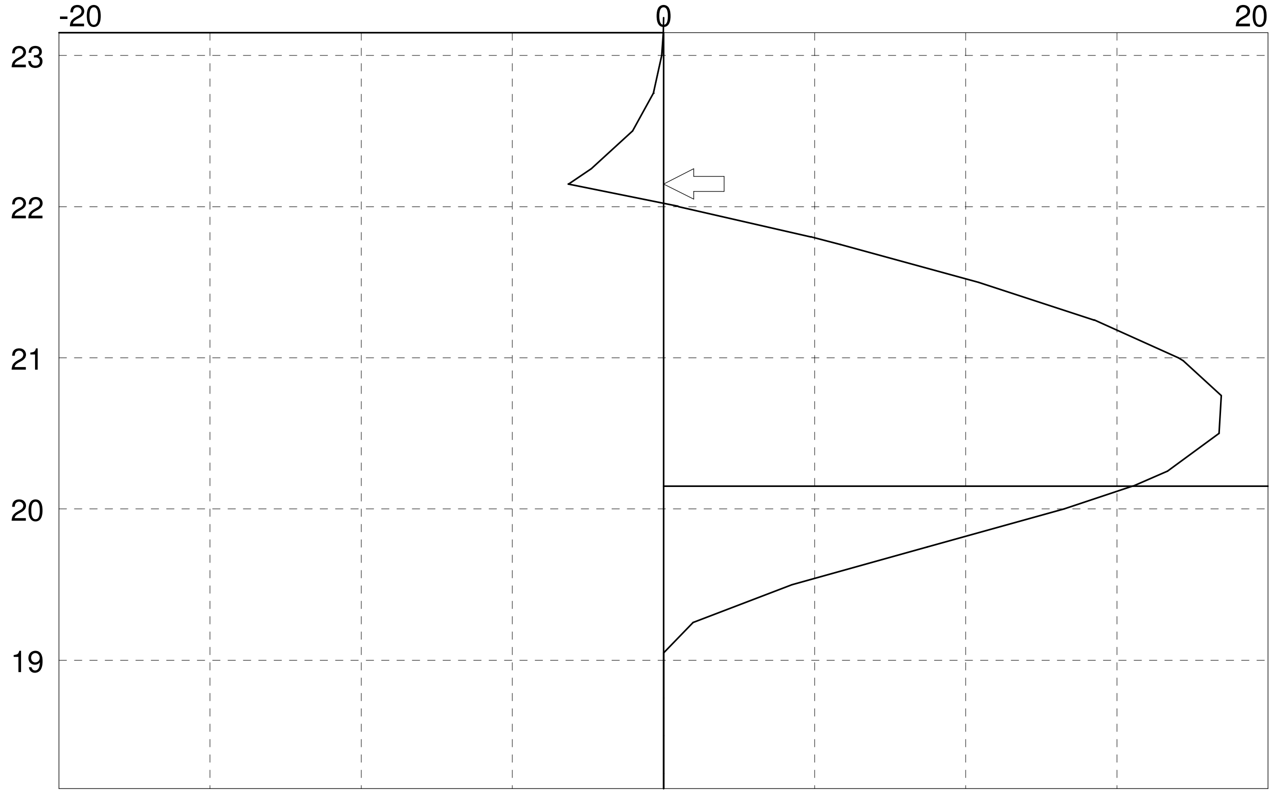
Graphical results from analysis of stage ref 4



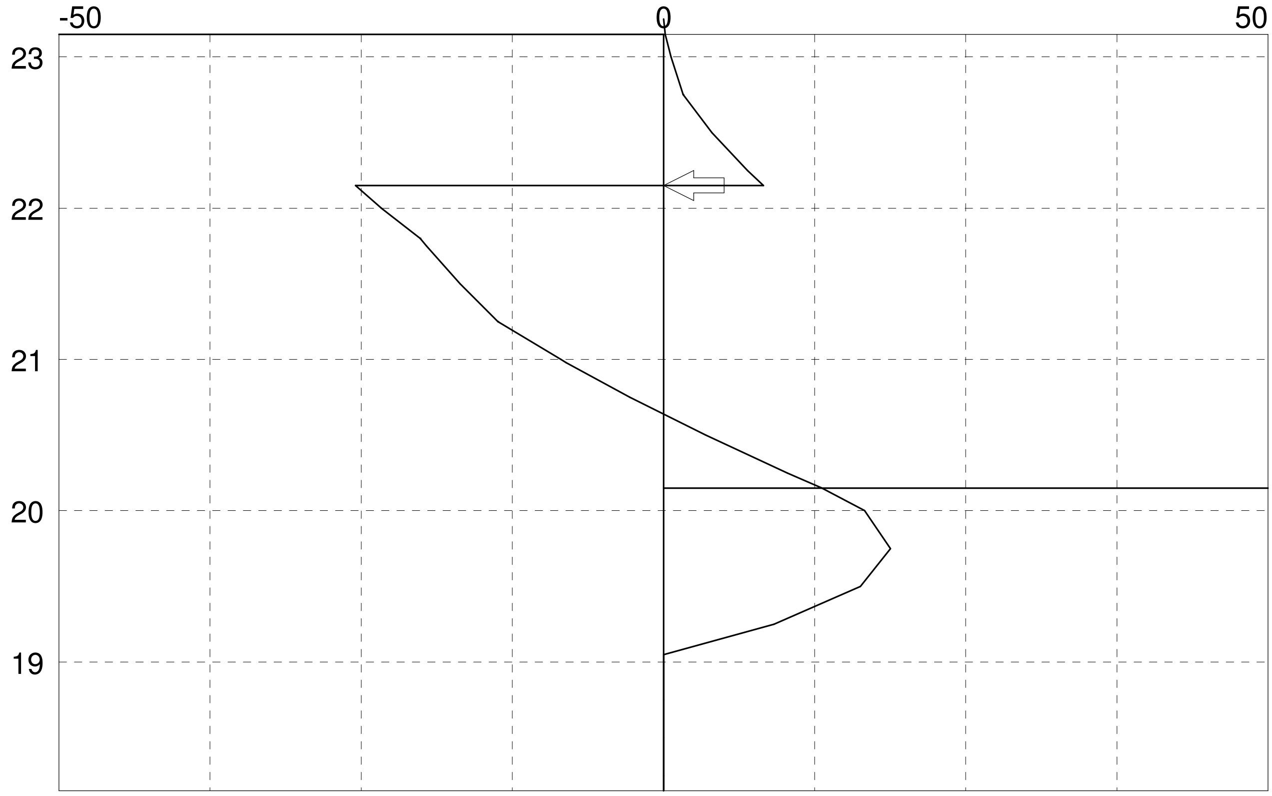


Deflection diagram (mm)

Graphical results from analysis of stage ref 4 continued

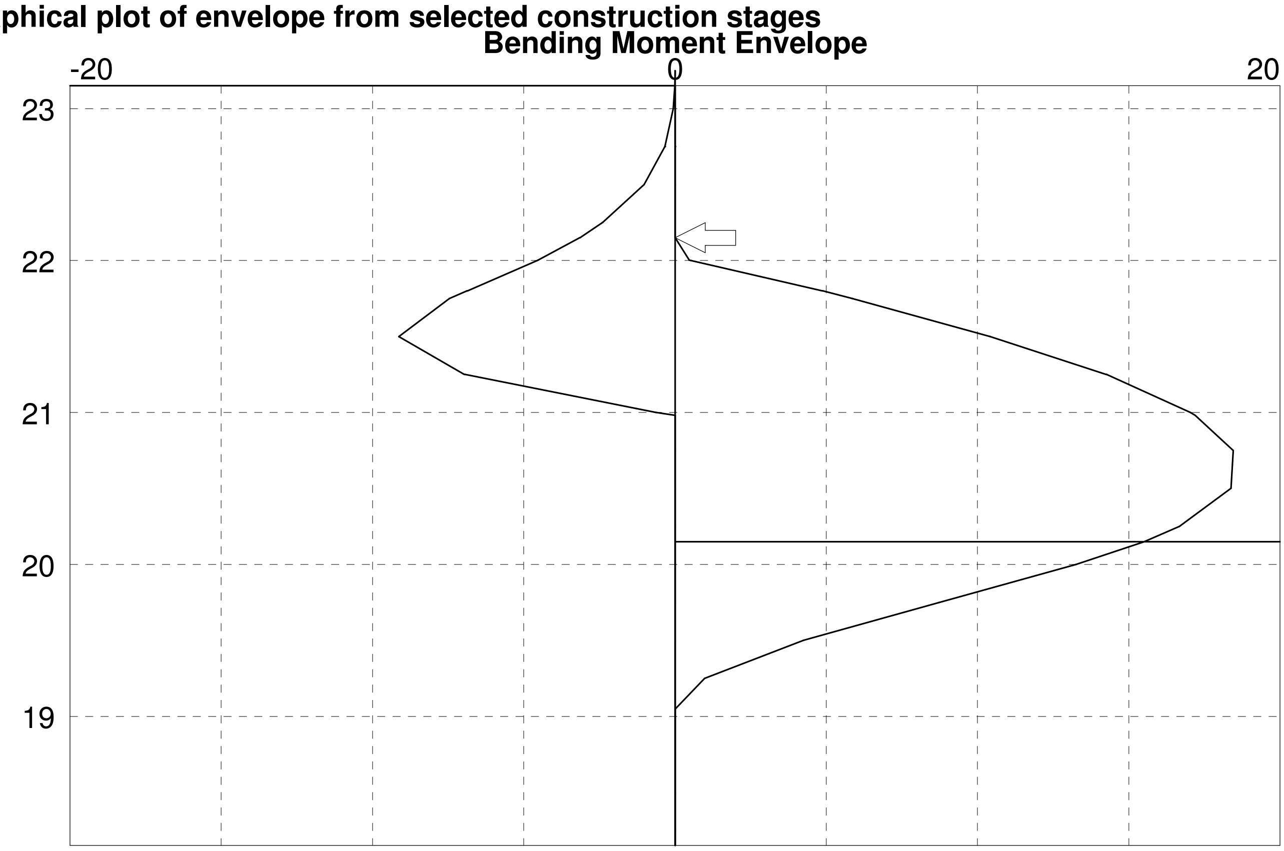


Bending Moment Diagram (kNm/m)



Shear Force Diagram (kN/m)

Graphical plot of envelope from selected construction stages



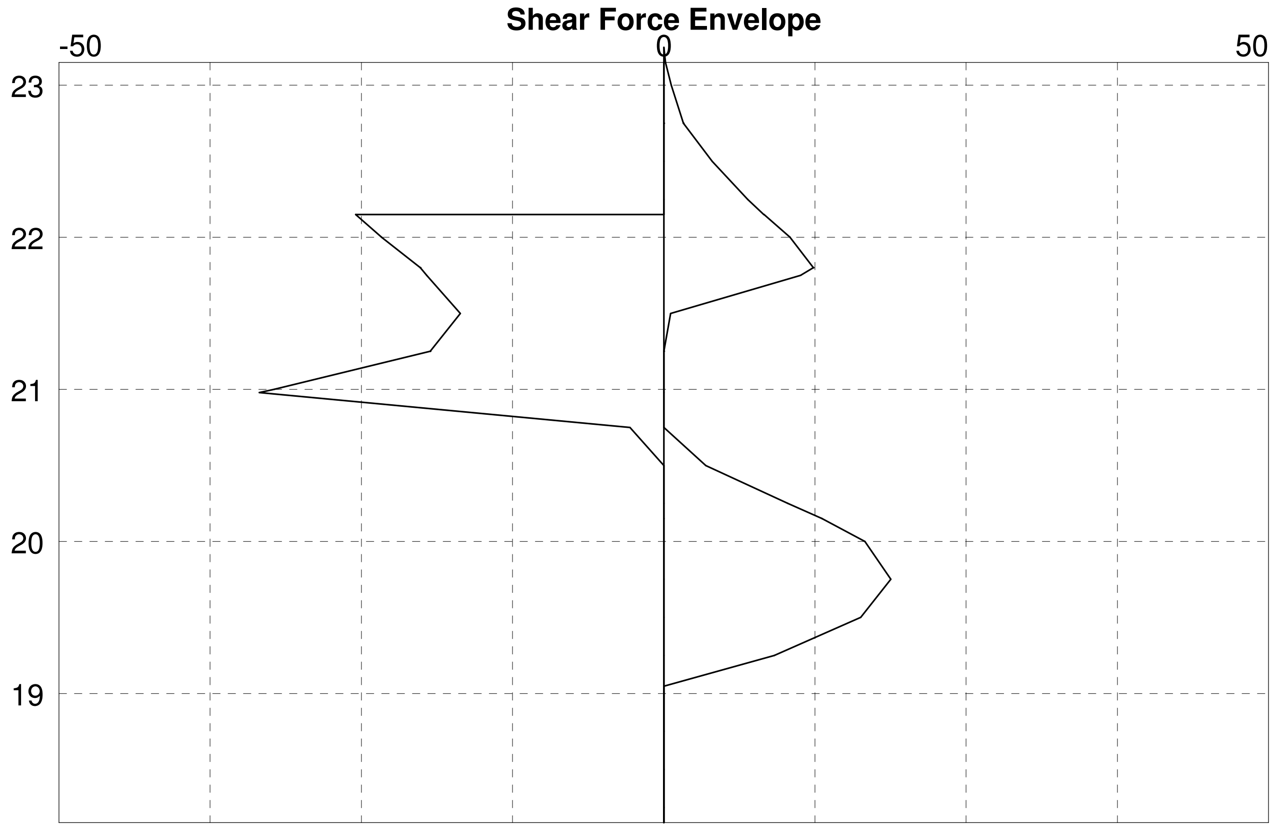


Table of envelope for wall forces

Calc Level m	Bending Minimum kNm/m	Bending Maximum kNm/m	Shear Minimum kN/m	Shear Maximum kN/m	Prop Force kN/m
23.15	.0	.0	-.1	.0	
23.00	.0	.1	-.6	.0	
22.75	.0	.3	-1.6	.0	
22.75	.0	.3	-1.6	.0	
22.15	.0	3.1	-8.3	.0	33.8
22.15	.0	3.1	-8.3	25.5	
22.00	-.5	4.5	-10.4	23.3	
22.00	-.5	4.5	-10.4	23.3	
21.80	-4.9	6.9	-12.3	20.1	
21.80	-4.9	6.9	-12.3	20.1	
21.25	-14.2	7.0	.0	19.3	
21.25	-14.3	7.0	.0	19.3	
21.00	-17.0	.6	.0	32.2	
20.98	-17.2	.0	.0	33.4	
20.15	-15.5	.0	-13.1	.0	
20.15	-15.5	.0	-13.1	.0	
20.00	-13.3	.0	-16.6	.0	
19.05	.0	.0	.0	.0	
19.00	.0	.0	.0	.0	
18.15	.0	.0	.0	.0	

Structural design of wall

Wall section properties

Sheet pile section ref Mabey M12

Wall material properties

Yield stress of steel	355	N/mm2
Bending Stress Ratio	1.05	
Allowable bending stress	338	N/mm2
Allowable shear stress	202	N/mm2

Wall structural design checks

Check description	Required or Limit	Provided or Actual	Units
Max. bending moment	18	61	kNm/m
Design stress check			
Min. section modulus	54	182	cm3/m
Design stress check			
Maximum shear force	33	387	kN/m
Design stress check			

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Pile geometry

Pile top Level	21.15	m
Pile Length	2	m
Pile toe level	19.15	m
Active ground slope	0	Degrees (To horizontal)

Soils and ground water initial data (Soils data given for active and passive sides)

Initial Ground Water level 15

Top Level m	Description	Bulk Dens kN/m3	Sat' Dens kN/m3	Young Mod kN/m2	Young Inc. kN/m3	Cu C' kN/m2	C Inc. kN/m3	Phi Deg	Wall Shear Ratio	Ka Kp	Kac Kpc
23.15	Hardcore	17.00	18.00	48000	0			40	.50	.19	
								40	.50	8.38	
22.75	Clay Fill	19.00	20.00	13867	0			26	.50	.35	
								26	.50	3.40	
21.80	GRAVEL	21.00	23.00	46800	0			38	.50	.21	
								38	.50	7.23	
21.25	Sandy CLAY	20.50	20.50	48000	0			29	.50	.31	
								29	.50	4.04	
17.85	Sandy CLAY	19.50	19.50	30000	0			28	.50	.32	
								28	.50	3.81	

Construction sequence

Stage Ref	Stage Type	Level or Angle m/deg.	Load kN(/m)	Offset m	Width m	Length m
1	Active surcharge	19.90	90.0	.0		
2 A	Passive side excavation	20.23				

Code of practice

Code of practice or reference document	Eurocode 7 ULS Design Approach 1 Combination 1
Application of pressures for stability	Not applicable for FOS=1 on moments
FOS on moments (stability check)	1.00
ULS factor on Tan(Phi) values	1.00
ULS fFactor on drained cohesion values	1.00
ULS factor on undrained cohesion values	1.00
ULS factor on active soil pressures	1.35
ULS factor on passive soil pressures	1.00
ULS factor on active water pressures	1.35
ULS factor on passive water pressures	1.35
ULS factor on loads applied to the soil	1.11
ULS factor on loads applied to the wall	1.50
FOS on embedment (stability check)	1.00
Correction factor on cantilever embedment	1.20

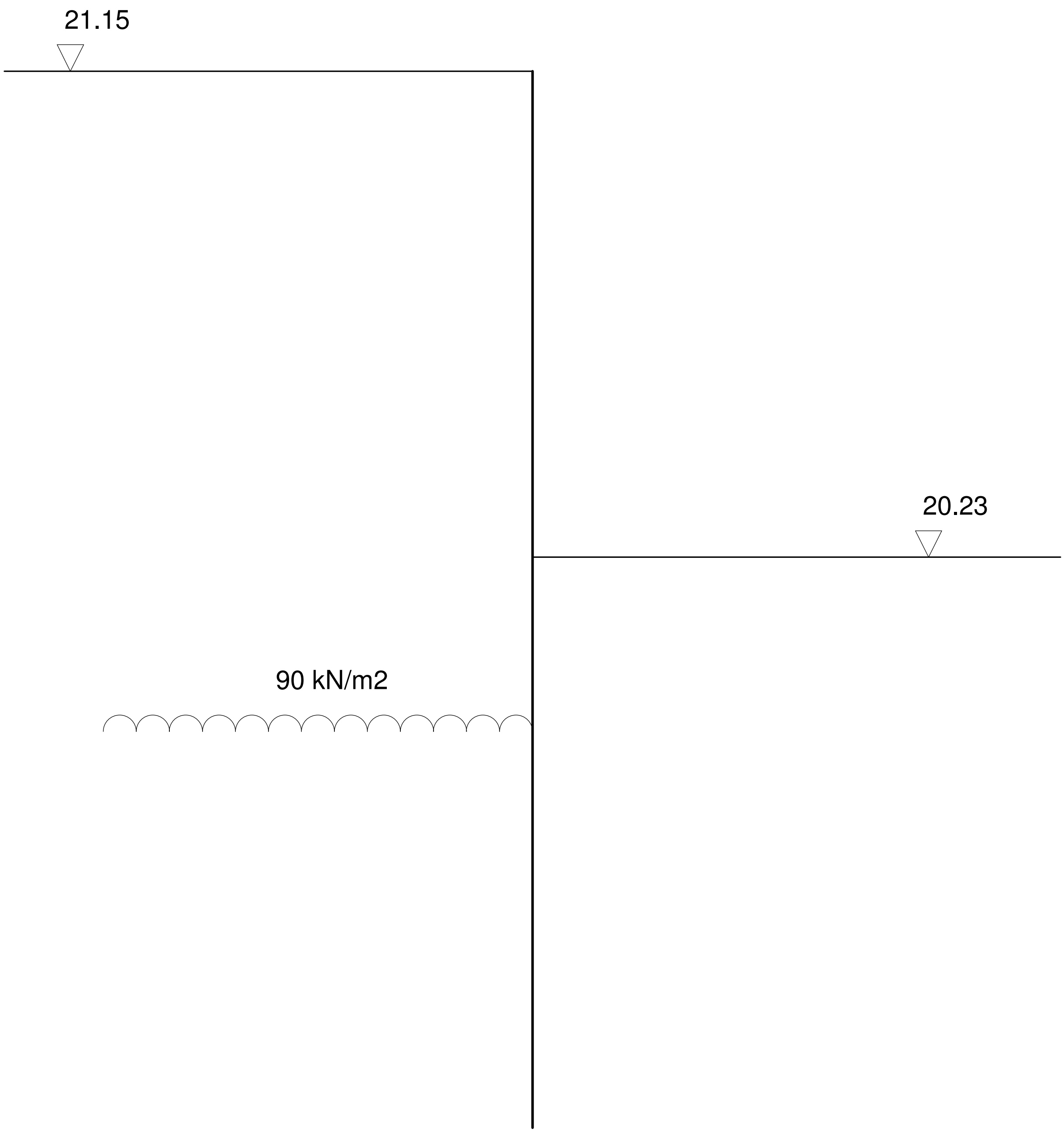
Wall analysis detail options

Nominal Phi for load distribution	30.0	Degrees
Depth of water filled tension cracks	.0	m
Density of water	10.0	kN/m3
Minimum equivalent fluid density	5.0	kN/m3
Depth of passive softened soil	1.0	m
Continuity model for wall analysis	Pins at second and lower props	

Deflection parameters

Wall moment of inertia	89	cm4/m
Wall Youngs modulus	210000000	kN/m2

Stage ref. **2**
Stage type **Passive side excavation**



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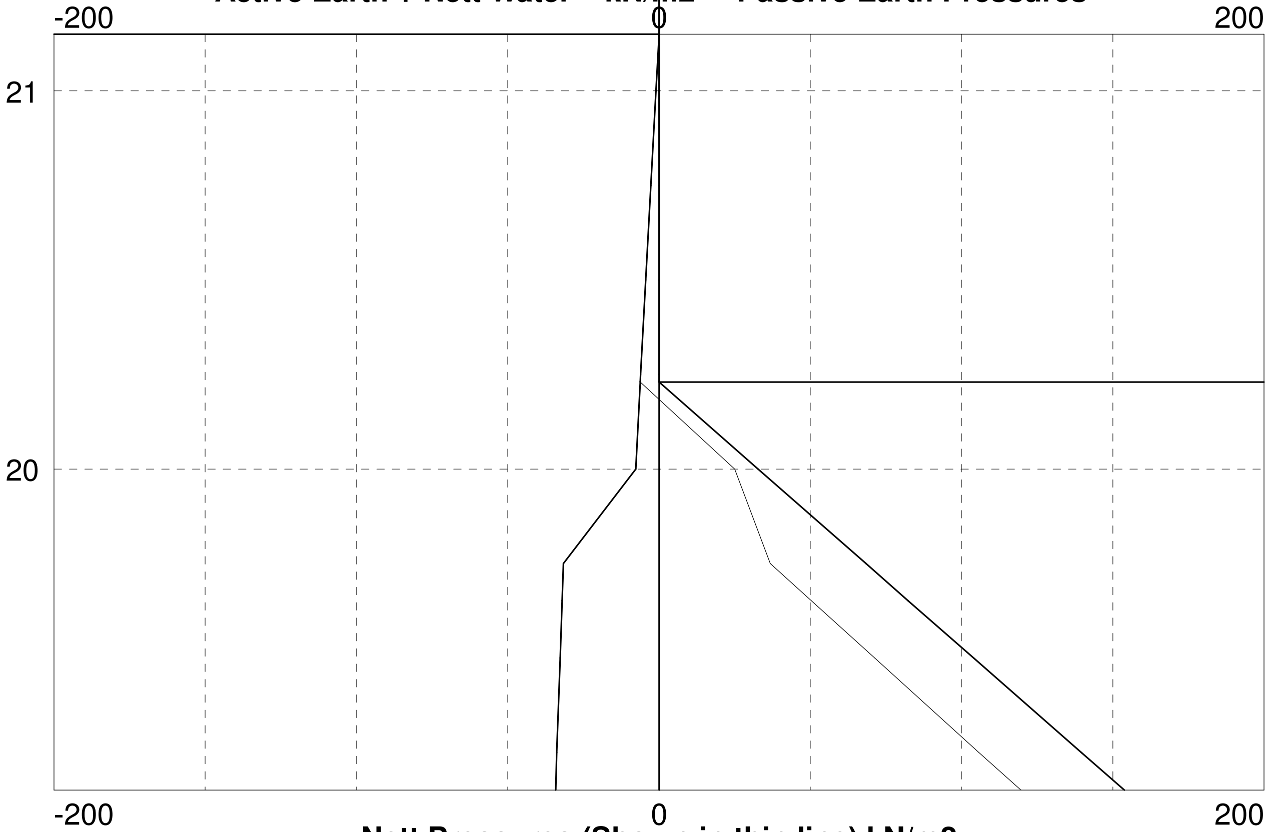
Tabular results from analysis of stage ref 2

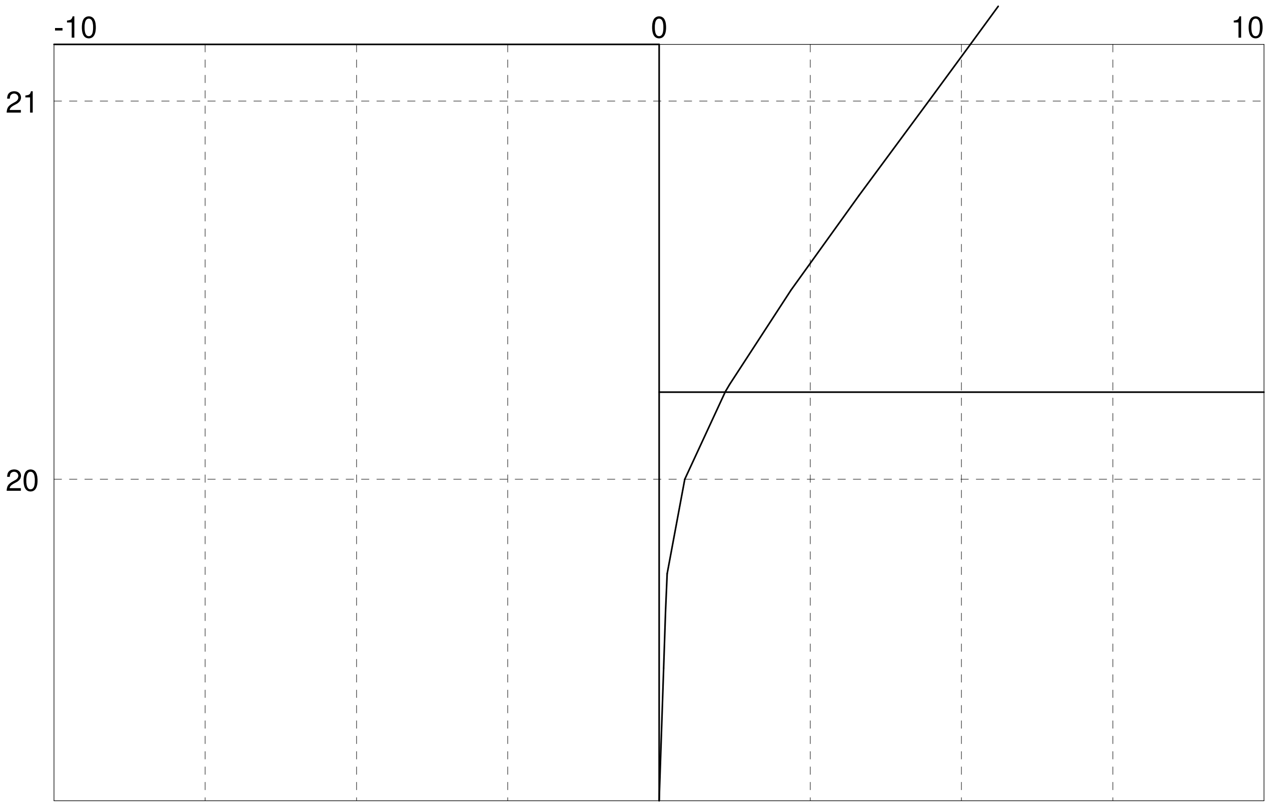
	Calc Level m	Active Vert kN/m2	Active Earth kN/m2	Active Water kN/m2	Pas' Vert kN/m2	Pas' Earth kN/m2	Pas' Water kN/m2	Total Nett kN/m2	Bend. Moment kNm/m	Shear Force kN/m	Defl't mm	Prop Force kN/m	FOS
	21.15	.0	.0	.0	.0	.0	.0	0	0	0	5.1		.00
m	21.00	2.5	1.0	.0	.0	.0	.0	1.0	0	-.1	4.5		.00
m	20.23	15.6	6.2	.0	.0	.0	.0	6.2	.9	-2.8	1.1		.00
m	20.23	15.6	6.2	.0	.0	.0	.0	6.2	.9	-2.9	1.1		.00
m	20.00	19.5	7.8	.0	3.9	32.8	.0	-25.0	1.4	-.7	.4		.11
	19.65	125.4	32.0	.0	9.8	82.3	.0	-50.3	0	11.3	.1		.76
	19.15	133.9	34.2	.0	18.4	153.8	.0	-119.6	0	0	0		1.63

Graphical results from analysis of stage ref 2

Active Earth + Nett Water kN/m2 Passive Earth Pressures

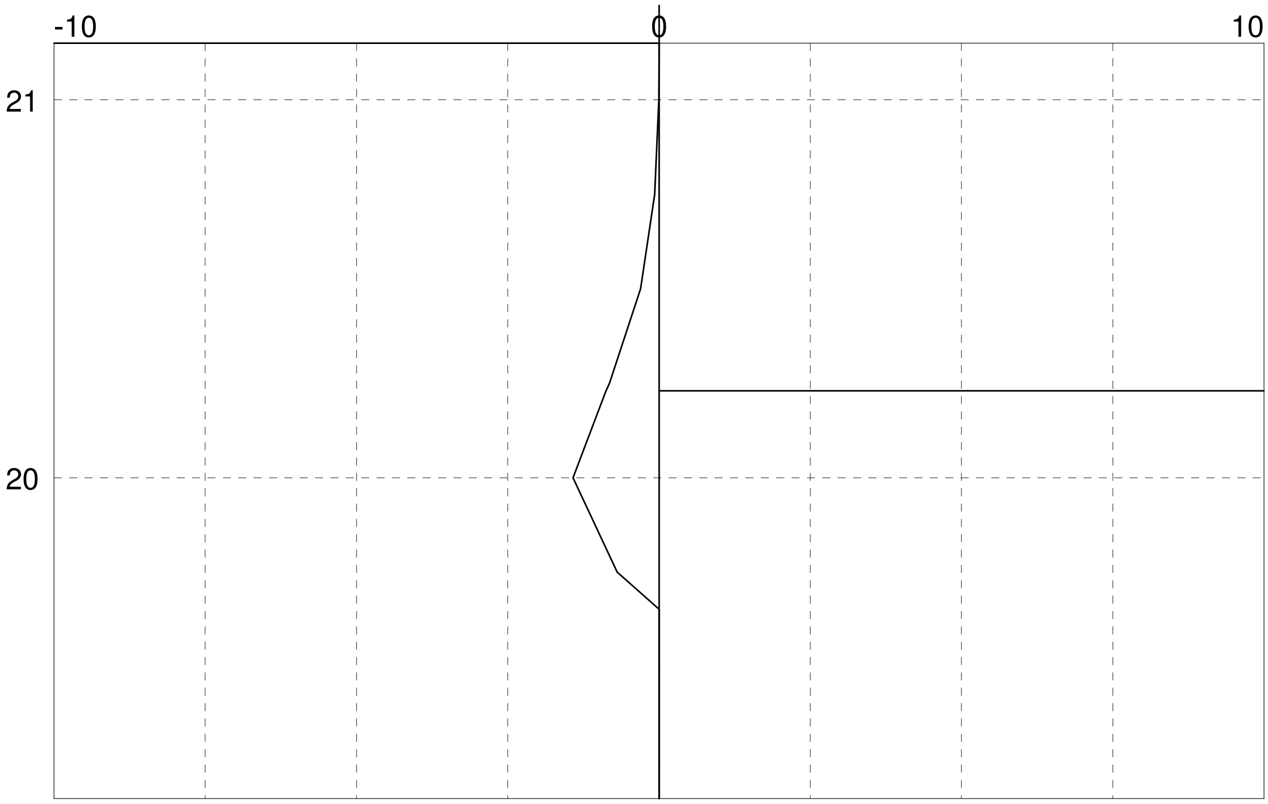


Nett Pressures (Shown in thin line) kN/m2

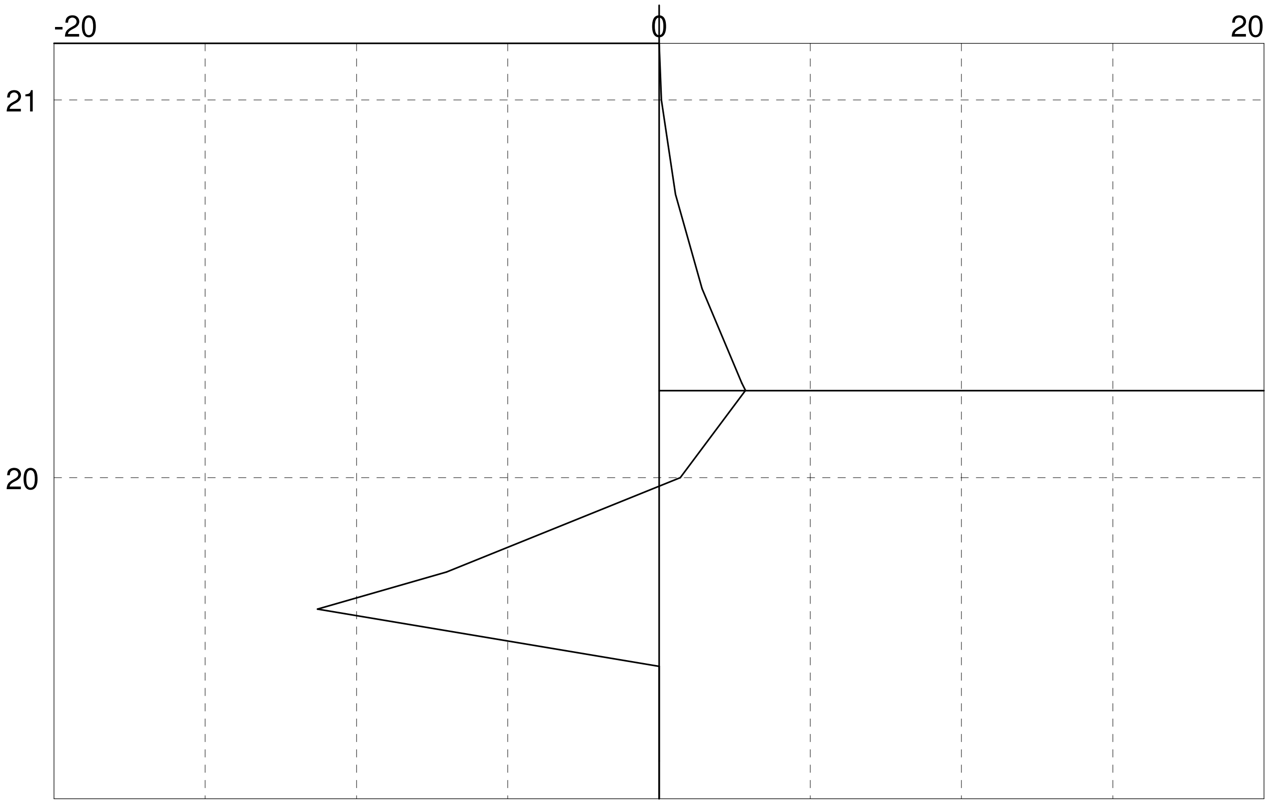


Deflection diagram (mm)

Graphical results from analysis of stage ref 2 continued



Bending Moment Diagram (kNm/m)



Shear Force Diagram (kN/m)

Structural design of wall

Wall section properties

Sheet pile section ref Mabey M6

Wall material properties

Yield stress of steel	275	N/mm2
Bending Stress Ratio	1.05	
Allowable bending stress	261	N/mm2
Allowable shear stress	157	N/mm2

Wall structural design checks

Check description	Required or Limit	Provided or Actual	Units
Max. bending moment	1	12	kNm/m
Design stress check			
Min. section modulus	5	47	cm3/m
Design stress check			
Maximum shear force	11	109	kN/m
Design stress check			

Pile geometry

Pile top Level	20.75	m
Pile Length	2	m
Pile toe level	18.75	m
Active ground slope	0	Degrees (To horizontal)

Soils and ground water initial data (Soils data given for active and passive sides)

Initial Ground Water level 15

Top Level m	Description	Bulk Dens kN/m3	Sat' Dens kN/m3	Young Mod kN/m2	Young Inc. kN/m3	Cu C' kN/m2	C Inc. kN/m3	Phi Deg	Wall Shear Ratio	Ka Kp	Kac Kpc
23.15	Hardcore	17.00	18.00	48000	0			40	.50	.19	
								40	.50	8.38	
22.75	Clay Fill	19.00	20.00	13867	0			26	.50	.35	
								26	.50	3.40	
21.80	GRAVEL	21.00	23.00	46800	0			38	.50	.21	
								38	.50	7.23	
21.25	Sandy CLAY	20.50	20.50	48000	0			29	.50	.31	
								29	.50	4.04	
17.85	Sandy CLAY	19.50	19.50	30000	0			28	.50	.32	
								28	.50	3.81	

Construction sequence

Stage Ref	Stage Type	Level or Angle m/deg.	Load kN(/m)	Offset m	Width m	Length m
1	Active surcharge	20.75	10.0	.0		
2 A	Passive side excavation	20.00				
3	Insert prop	20.75				
4 A	Passive side excavation	19.65				

Code of practice

Code of practice or reference document	Eurocode 7 ULS Design Approach 1 Combination 1
Application of pressures for stability	Not applicable for FOS=1 on moments
FOS on moments (stability check)	1.00
ULS factor on Tan(Phi) values	1.00
ULS fFactor on drained cohesion values	1.00
ULS factor on undrained cohesion values	1.00
ULS factor on active soil pressures	1.35
ULS factor on passive soil pressures	1.00
ULS factor on active water pressures	1.35
ULS factor on passive water pressures	1.35
ULS factor on loads applied to the soil	1.11
ULS factor on loads applied to the wall	1.50
FOS on embedment (stability check)	1.00
Correction factor on cantilever embedment	1.20

Wall analysis detail options

Nominal Phi for load distribution	30.0	Degrees
Depth of water filled tension cracks	.0	m
Density of water	10.0	kN/m3
Minimum equivalent fluid density	5.0	kN/m3
Depth of passive softened soil	1.0	m
Continuity model for wall analysis	Pins at second and lower props	

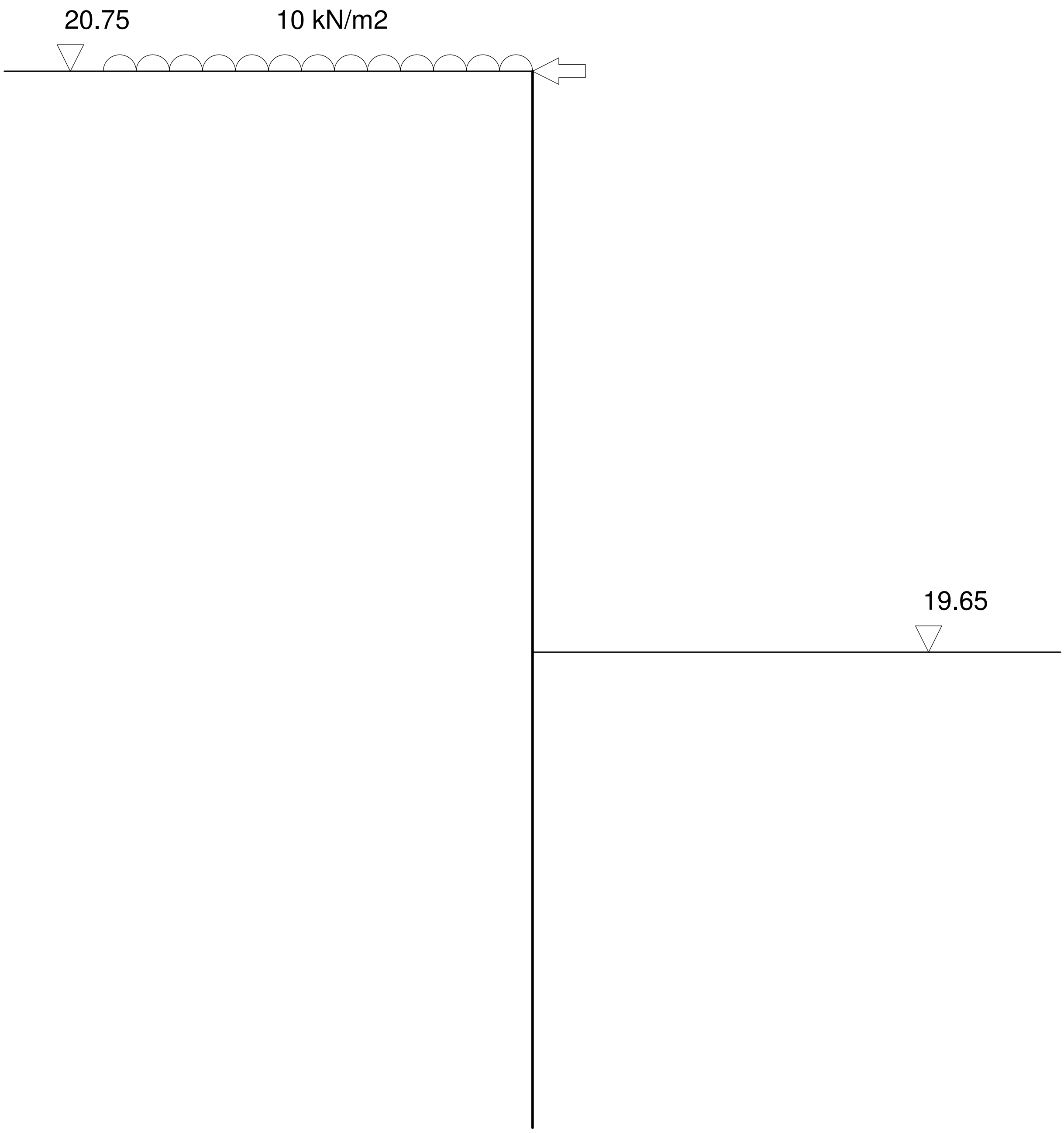
Deflection parameters

Wall moment of inertia	89	cm4/m
Wall Youngs modulus	210000000	kN/m2

Properties for prop at 20.75

Prop/Tie cross sectional area	200	cm2 each
Prop/Tie Youngs modulus	210000000	kN/m2
Prop/Tie length	10.0	m
Prop/Tie spacing	6.0	m
Waling moment of inertia	Waling deflection not included	
Waling Youngs modulus	Waling deflection not included	
Prop/Tie preload	0	kN
Initial lack of fit	0.0	mm

Stage ref. **4**
Stage type **Passive side excavation**



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UCL for Mace Temp Works to SE Corner	Engineer SMH Date 11/04/2016

Tabular results from analysis of stage ref 4

Calc Level m	Active Vert kN/m2	Active Earth kN/m2	Active Water kN/m2	Pas' Vert kN/m2	Pas' Earth kN/m2	Pas' Water kN/m2	Total Nett kN/m2	Bend. Moment kNm/m	Shear Force kN/m	Defl't mm	Prop Force kN/m	FOS
20.75	11.1	2.8	.0	.0	.0	.0	2.8	0	0	5.1	3.0	.00
20.75	11.1	2.8	.0	.0	.0	.0	2.8	0	3.0	5.1		.00
20.00	23.8	6.1	.0	.0	.0	.0	6.1	-1.1	-.4	4.3		.00
20.00	23.9	6.1	.0	.0	.0	.0	6.1	-1.1	-.4	4.3		.00
19.65	29.8	7.6	.0	.0	.0	.0	7.6	-.6	-2.8	3.1		.00
19.65	29.8	7.6	.0	.0	.0	.0	7.6	-.6	-2.8	3.1		.00
m 19.45	33.3	8.8	.0	3.5	29.1	.0	-20.3	0	-1.5	2.4		.65
m 19.38	34.3	9.2	.0	4.5	37.9	.0	-28.7	0	0	2.2		1.04
m 19.00	40.9	11.8	.0	11.0	92.6	.0	-80.8	0	0	.9		3.63
m 18.75	45.1	13.5	.0	15.3	128.2	.0	-114.7	0	0	0		5.25

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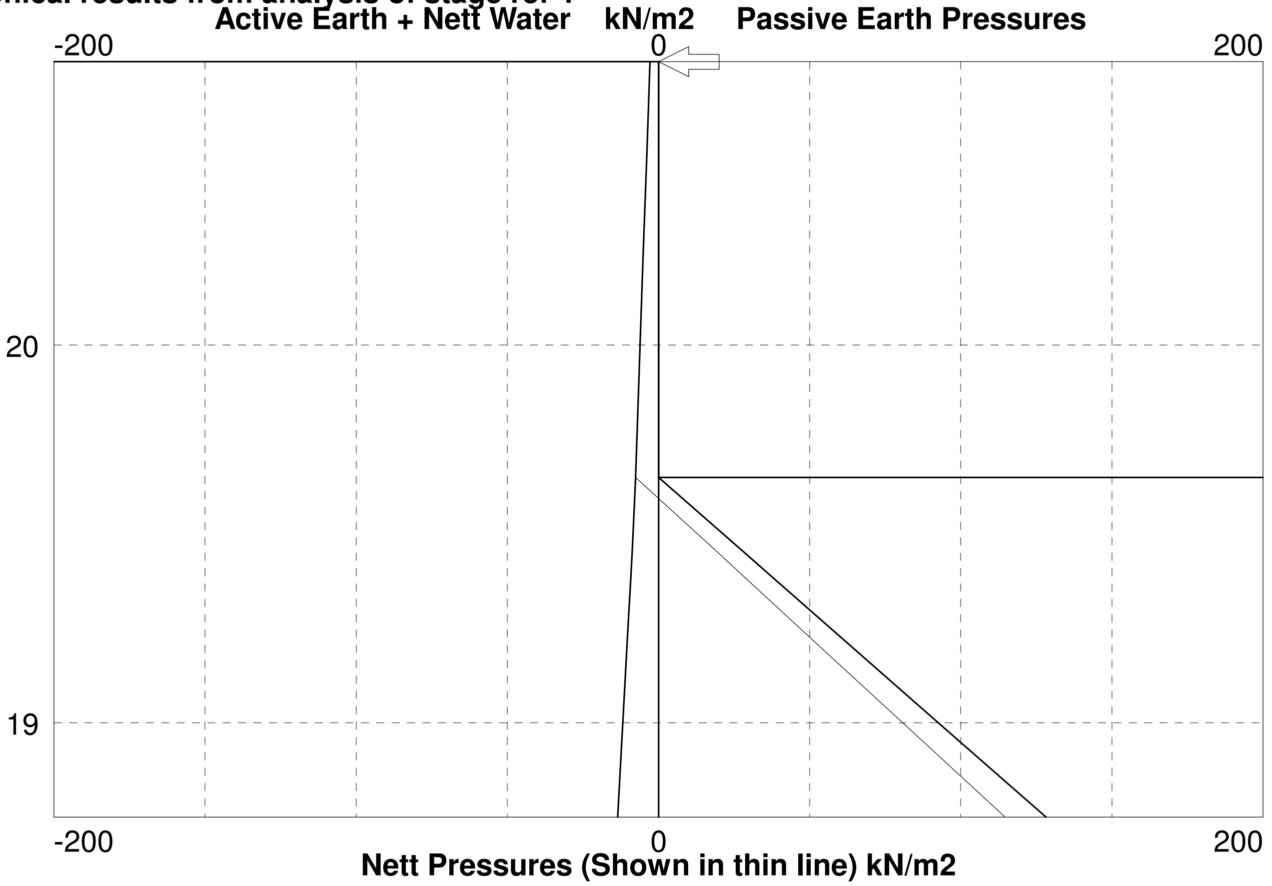
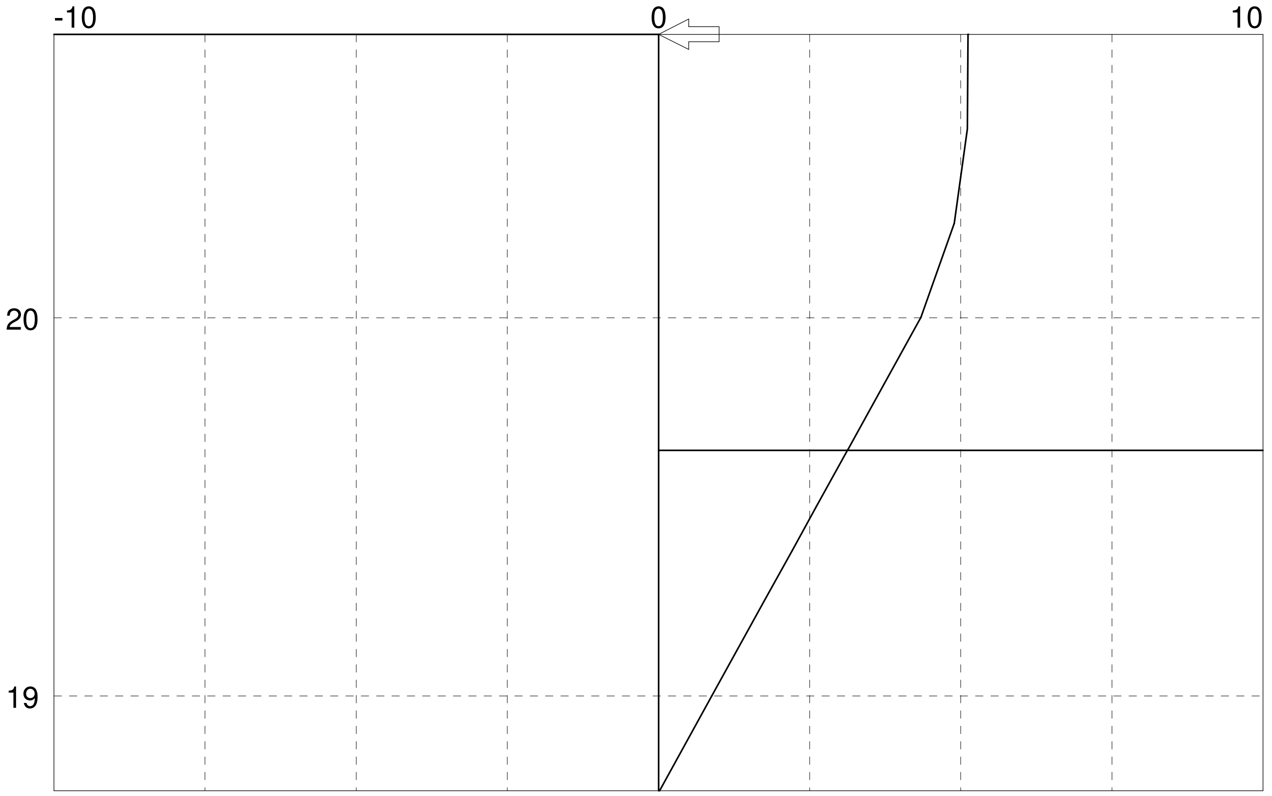
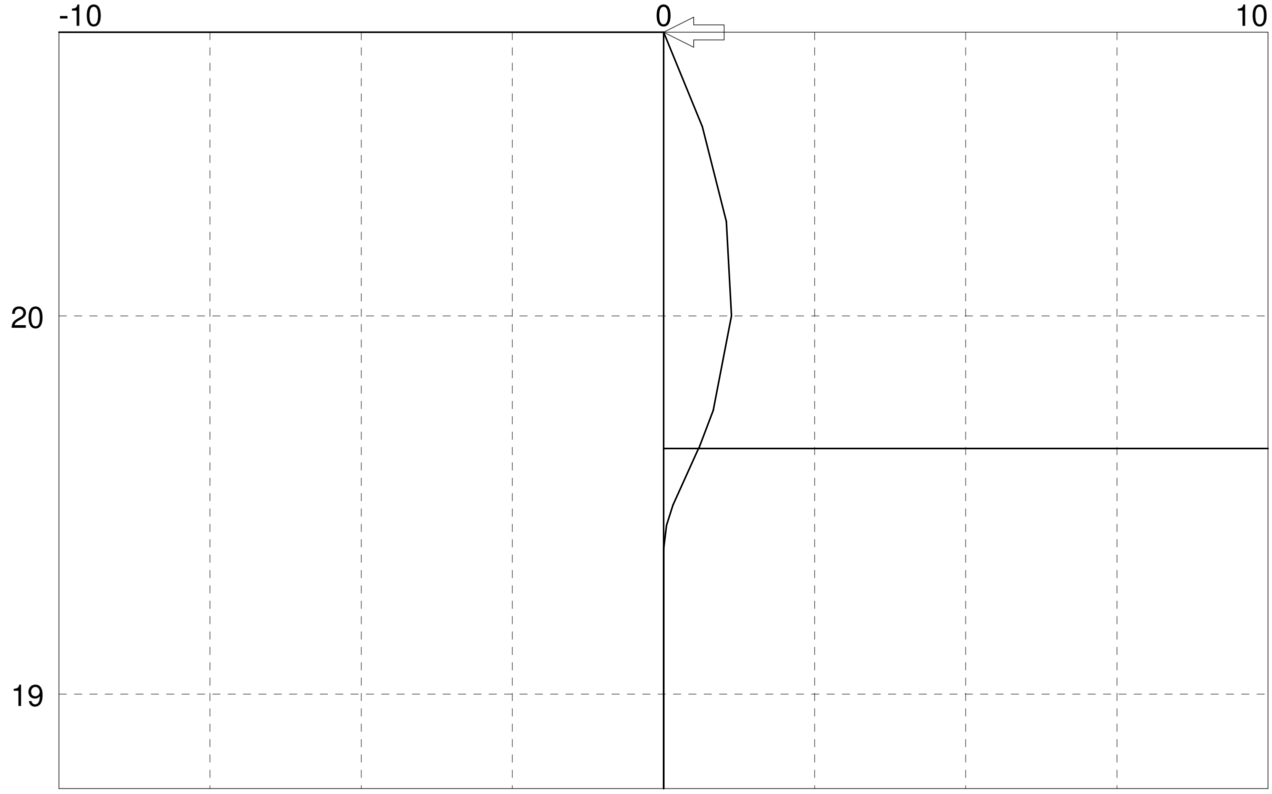
Graphical results from analysis of stage ref 4



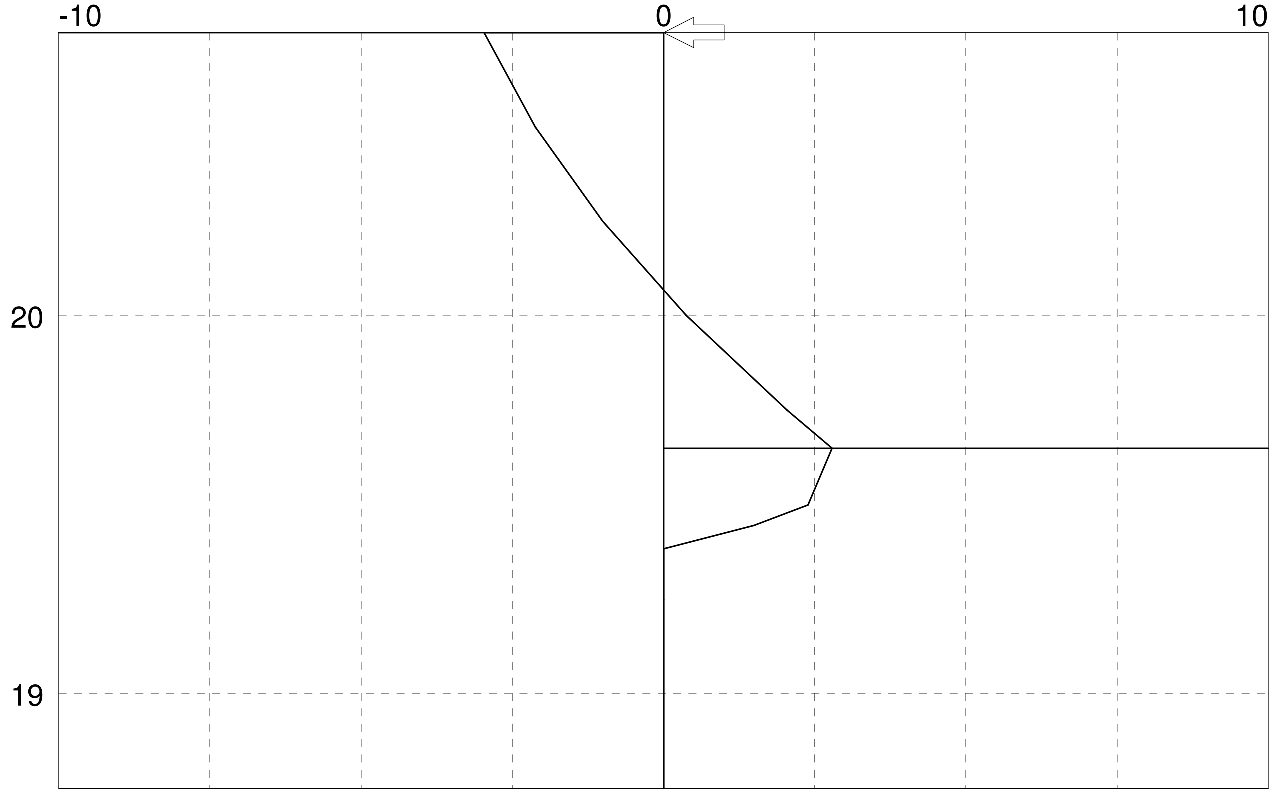


Deflection diagram (mm)

Graphical results from analysis of stage ref 4 continued



Bending Moment Diagram (kNm/m)



Shear Force Diagram (kN/m)

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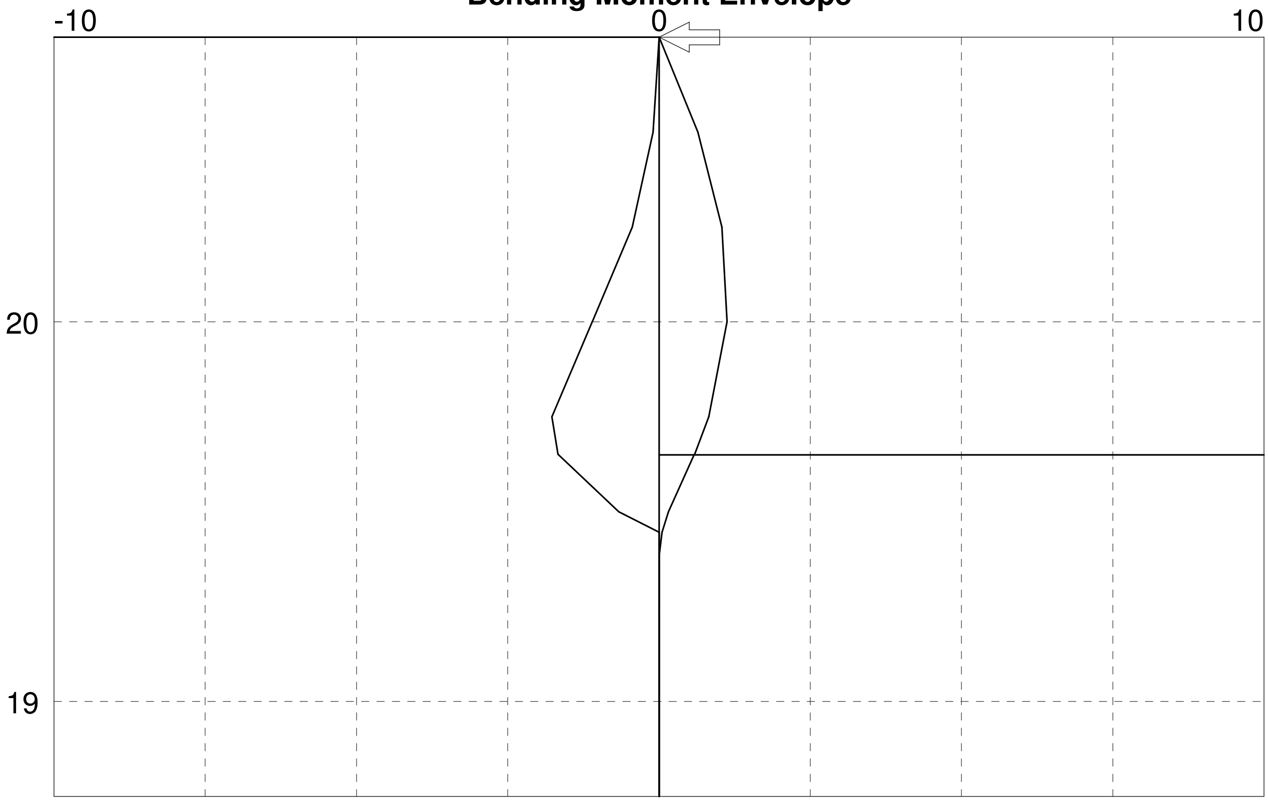
Graphical plot of envelope from selected construction stages

Bending Moment Envelope



Shear Force Envelope

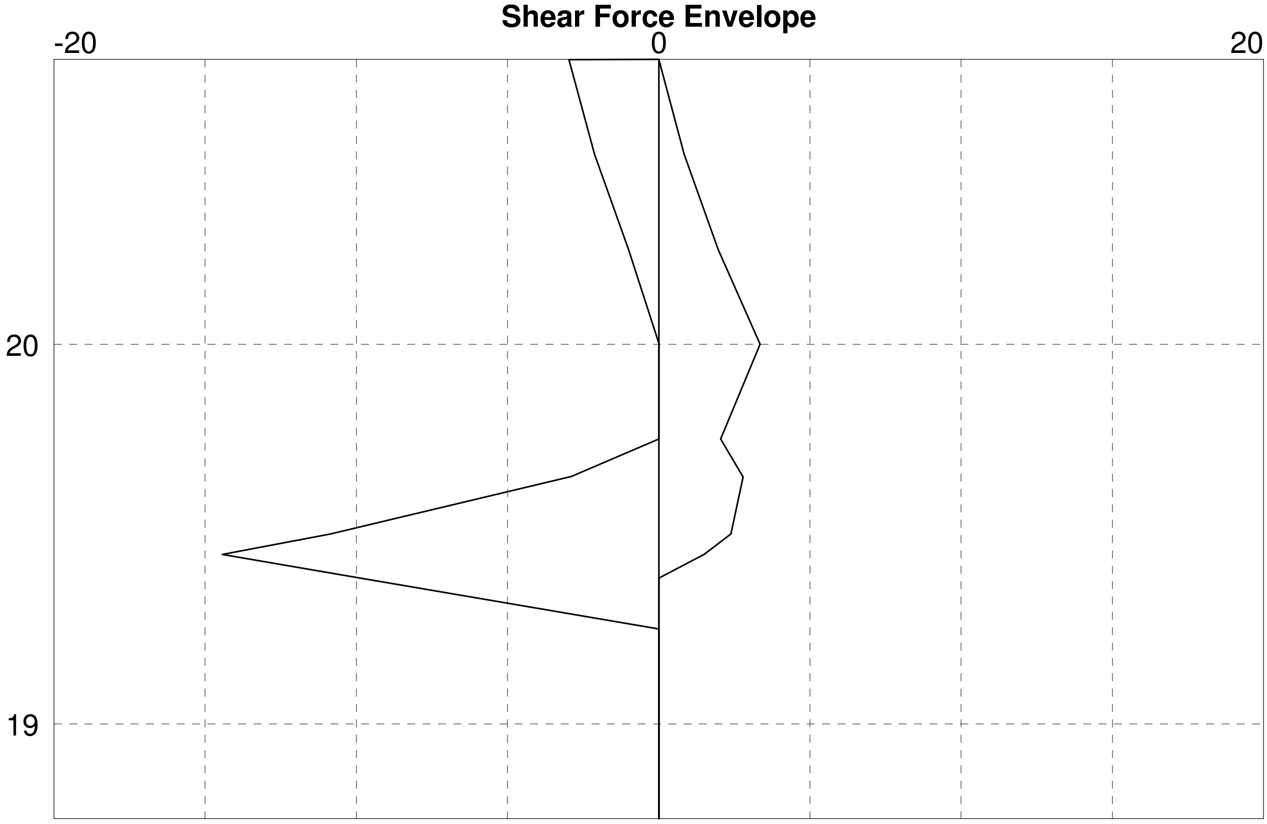


Table of envelope for wall forces

Calc Level m	Bending Minimum kNm/m	Bending Maximum kNm/m	Shear Minimum kN/m	Shear Maximum kN/m	Prop Force kN/m
20.75	.0	.0	.0	.0	3.0
20.75	.0	.0	.0	3.0	
20.00	-1.1	1.1	-3.3	.0	
20.00	-1.1	1.1	-3.3	.0	
19.65	-.6	1.7	-2.8	2.9	
19.65	-.6	1.7	-2.8	3.0	
19.45	.0	.0	-1.5	14.4	
19.38	.0	.0	.0	9.9	
19.00	.0	.0	.0	.0	
18.75	.0	.0	.0	.0	

Structural design of wall

Wall section properties

Sheet pile section ref Mabey M6

Wall material properties

Yield stress of steel	275	N/mm2
Bending Stress Ratio	1.05	
Allowable bending stress	261	N/mm2
Allowable shear stress	157	N/mm2

Wall structural design checks

Check description	Required or Limit	Provided or Actual	Units
Max. bending moment	1	12	kNm/m
Design stress check			
Min. section modulus	6	47	cm3/m
Design stress check			
Maximum shear force	14	109	kN/m
Design stress check			