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Temporary works Design
For
Construction of 2 level Basement
at
UCL
For
FOI
MACE

APRIL 2016

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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.

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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.



RC BEAM ANALYSIS & DESIGN (EN1992-1)

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 \times 0				
	Permanent full UDL 122	Permanent full UDL 122.2 kN/m			
Load combinations					
Load combination 1	Support A	Permanent \times 1.35			
		Variable \times 1.50			
	Span 1	Permanent \times 1.35			
		Variable \times 1.50			
	Support B	Permanent \times 1.35			
		Variable \times 1.50			

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		Span 2		Permane	ent $ imes$ 1.35			
				Variable	× 1.50			
		Support C		Permane	ent $ imes$ 1.35			
				Variable	× 1.50			
		Span 3		Perman	ent $ imes$ 1.35			
				Variable	× 1.50			
		Support D		Perman	ent $ imes$ 1.35			
				Variable	× 1.50			
Analysis results								
Maximum moment support A;		$M_{A_{max}} = 4$	72 kNm;	$M_{A_{red}} =$	472 kNm;			
Maximum moment span 1 at 5	23 mm;	$M_{s1_max} = 4$	95 kNm;	Ms1_red =	495 kNm;			
Maximum moment support B;		$M_{B_{max}} = -2$	2965 kNm;	$M_{B_{red}} =$	-2965 kNm;			
Maximum moment span 2 at 7	943 mm;	Ms2_max = 2	M _{s2_max} = 2239 kNm;		M _{s2_red} = 2239 kNm;			
Maximum moment support C;	nent support C;		M _{C_max} = -2983 kNm;		M _{C_red} = -2983 kNm;			
Maximum moment span 3 at 6	561 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;		VA_max = 86	5 kN;	$V_{A_{red}} = $	86 kN			
Maximum shear support A spa	an 1 at 1443 mm;	VA_s1_max =	-153 kN;	VA_s1_red	= -153 kN			
Maximum shear support B;		V _{B_max} = 13	310 kN;	$V_{B_red} =$	1310 kN			
Maximum shear support B spa	an 1 at 5573 mm;	$V_{B_{s1}max} =$	-829 kN;	V _{B_s1_red}	= -829 kN			
Maximum shear support B spa	an 2 at 1428 mm;	$V_{B_s2_max} =$	1071 kN;	V _{B_s2_red}	= 1071 kN			
Maximum shear support C;		Vc_max = -1	313 kN;	$V_{C_{red}} =$	-1313 kN			
Maximum shear support C spa	an 2 at 14473 mm;	$V_{C_{s2}max} =$	-1073 kN;	V _{C_s2_red}	= -1073 kN			
Maximum shear support C spa	an 3 at 1428 mm;	VC_s3_max =	843 kN;	VC_s3_red	= 843 kN			
Maximum shear support D;	0 1 5070	$V_{D_{max}} = -2$	3 kN;	$V_{D_red} =$	-23 kN			
Maximum shear support D spa	an 3 at 5273 mm;	$VD_s3_max =$	216 KN;	VD_s3_red	= 216 KN			
Maximum reaction at support /	4; estima et eveneut Ar	RA = 80 KN	CALN					
Maximum reaction at support	action at support A;	RA_Permanent	: = 04 KIN					
Maximum reaction at support	D, action at augment P	$\mathbf{n}_{\mathrm{B}} = \mathbf{23/9}$	1769 LN					
Maximum reaction at support (\sim .	R ₀ – 2305						
Linfactored permanent load re-	o, action at support C	Ro Port	– 177/ kN					
Maximum reaction at support f		$\mathbf{R}_{D} = 23 \mathbf{k} \mathbf{N}$						
Unfactored permanent load rea	action at support D:	RD Permanent	. = 17 kN					
Rectangular section details	Sopport D	, <u>.</u> , ormanen						
nectangular section details								



Section depth;



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Concrete details (Table 3.1 - Strength and defor	mation characteristics for concrete)
Concrete strength class;	C40/50
Characteristic compressive cylinder strength;	f _{ck} = 40 N/mm ²
Characteristic compressive cube strength;	$f_{ck,cube} = 50 \text{ N/mm}^2$
Mean value of compressive cylinder strength;	$f_{cm} = f_{ck} + 8 \text{ N/mm}^2 = 48 \text{ N/mm}^2$
Mean value of axial tensile strength;	$f_{ctm} = 0.3 \text{ N/mm}^2 \times (f_{ck}/ 1 \text{ N/mm}^2)^{2/3} = 3.5 \text{ N/mm}^2$
Secant modulus of elasticity of concrete;	$E_{cm} = 22 \text{ kN/mm}^2 \times [f_{cm}/10 \text{ N/mm}^2]^{0.3} = 35220 \text{ N/mm}^2$
Partial factor for concrete (Table 2.1N);	γc = 1.50
Compressive strength coefficient (cl.3.1.6(1));	$\alpha_{cc} = 0.85$
Design compressive concrete strength (exp.3.15);	$f_{cd} = \alpha_{cc} \times f_{ck} / \gamma_C = 22.7 \text{ N/mm}^2$
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);	γs = 1.15
Design yield strength of reinforcement;	$f_{yd} = f_{yk} / \gamma_S = 435 \text{ N/mm}^2$
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	
	10 x 25 ₀ bars



Rectangular section in flexure (Section 6.1)

Minimum moment factor (cl.9.2.1.2(1));	$\beta_1 = 0.25$
Design bending moment;	$M = max(abs(M_{A_red}), \beta_1 \times abs(M_{s1_red})) = \textbf{472} \text{ kNm}$
Depth to tension reinforcement;	$d = h - c_{nom_b} - \phi_v - \phi_{bot} / 2 = 1443 \text{ mm}$
Percentage redistribution;	m _r A = 0 %
Redistribution ratio;	$\delta = min(1 - m_{rA}, 1) = 1.000$
	$K = M / (b \times d^2 \times f_{ck}) = 0.004$
	$\text{K'} = 0.598 \times \delta \text{ - } 0.181 \times \delta^2 \text{ - } 0.21 = \textbf{0.207}$
	K' > K - No compression reinforcement is required
Lever arm;	z = min((d / 2) × [1 + (1 - $3.53 \times K$) ^{0.5}], 0.95 × d) = 1370 mm
Depth of neutral axis;	x = 2.5 × (d - z) = 180 mm
Area of tension reinforcement required;	$A_{s,req} = M / (f_{yd} \times z) = 792 \text{ mm}^2$
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);	$A_{s,min} = max(0.26 \times f_{ctm} \ / \ f_{yk}, \ 0.0013) \times b \times d = \textbf{3948} \ mm^2$
Maximum area of reinforcement (cl.9.2.1.1(3));	$A_{s,max} = 0.04 \times b \times h = 90000 \text{ mm}^2$

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	DAGO Ama							
	PASS - Area	of reinforce	ment provide	a is greater than	area of reinfor	cement requirea		
Rectangular section in shear (Section 6.2)							
Design shear force at support A;	;	$V_{Ed,max} = a$	abs(max(V _{A_ma}	$A_{A_{red}}) = 86 \text{ kN}$				
Angle of comp. shear strut for m	aximum shear;	$\theta_{max} = 45$	deg					
Maximum design shear force (e)	(p.6.9);	$V_{Rd,max} = k$	$\mathbf{D} \times \mathbf{Z} \times \mathbf{V}_1 \times \mathbf{f}_{cd}$	$/ (\cot(\theta_{max}) + \tan(\theta_{max}))$	max)) = 11/41 Kl	N 		
Design shear force span 1 at 14	PA55	- Design sne	min(V)	$\frac{1}{10000000000000000000000000000000000$	n maximum de N	sign snear force		
Design shear stress:	45 mm,		$(11111)(VA_{s1}_{max})$	$V_{A_{s1}_{red}} = 133 \text{ K}$	IN .			
Strength reduction factor (cl 6.2	3(3)).	$v_{Ea} = v_{Ea}$	$(0 \land 2) = 0.07$ [1 - fat / 250 N	$1/mm^2$ – 0 504				
Compression chord coefficient (2(3)	α _{em} = 1 00		(iiiii] – 0.304				
Angle of concrete compression s	strut (cl $6 2.3$):							
	$\theta = m^2$	$in(max(0.5 \times A))$	$Asin[min(2 \times v_F)]$	$Ed / (\alpha_{cw} \times f_{cd} \times V_1)$	1)], 21,8 dea), 4	5dea) = 21.8 dea		
Area of shear reinforcement requ	uired (exp.6.13):	Asy reg = VE	-u × p \ (tvu × c	$ot(\theta)) = 103 \text{ mm}^2/\text{n}$	י, ביוס מכשי, י	546g)e		
Shear reinforcement provided:	a	6 × 10¢ le	as at 300 c/c					
Area of shear reinforcement prov	vided:	$A_{sy proy} = 1$	571 mm ² /m					
Minimum area of shear reinforce	ement (exp.9.5N)	: Asy.min = 0	.08 N/mm ² × b	$0 \times (f_{ck} / 1 \text{ N/mm}^2)^{0.1}$	⁵ / f _{vk} = 1518 mi	m²/m		
	, , , , , , , , , , , , , , , , , , ,	PASS - Area	of shear rein	forcement provid	ed exceeds mi	inimum required		
Maximum longitudinal spacing (e	exp.9.6N);	$S_{vl,max} = 0.$	75 × d = 1082	? mm				
	PASS - Loi	ngitudinal sp	acing of shea	nr reinforcement p	provided is less	s than maximum		
Crack control (Section 7.3)								
Maximum crack width;		w _k = 0.3 n	าฑ					
Design value modulus of elastici	ty reinf (3.2.7(4))); E _s = 2000	00 N/mm ²					
Mean value of concrete tensile s	strength;	$f_{\text{ct,eff}} = f_{\text{ctm}}$	= 3.5 N/mm ²					
Stress distribution coefficient;		$k_{c} = 0.4$						
Non-uniform self-equilibrating st	ress coefficient;	k = min(m	ax(1 + (300 m	$m - min(h, b)) \times 0.3$	35 / 500 mm, 0.	.65), 1) = 0.65		
Actual tension bar spacing;		$S_{bar} = (b -$	$2 \times (C_{nom_s} + \phi)$	ν) - φ _{bot}) / (N _{bot} - 1)	= 151 mm			
Maximum stress permitted (Tabl	e 7.3N);	σ _s = 280 №	N/mm²					
Concrete to steel modulus of ela	ist. ratio;	$\alpha_{cr} = E_s / I$	∃cm = 5.68					
Distance of the Elastic NA from I	bottom of beam;	y = (b × h ² mm	2 / 2 + A _{s,prov} ×	$(\alpha_{cr} - 1) \times (h - d)) /$	(b × h + A _{s,prov} :	$\times (\alpha_{cr} - 1)) = 743$		
Area of concrete in the tensile zo	one;	$A_{ct} = b \times y$	/ = 1114505 m	1m²				
Minimum area of reinforcement	required (exp.7.1); A _{sc,min} = k	$k \times k \times f_{ct,eff} \times A$	$A_{ct} / \sigma_s = 3637 \text{ mm}^2$	2			
PAS	SS - Area of ten	sion reinforc	ement provid	led exceeds minir	num required i	for crack control		
Quasi-permanent value of variat	ole action;	ψ2 = 0.30						
Quasi-permanent limit state mor	nent;	MQP = abs	s(Ma_c21) + ψ2	$\times abs(M_{A_{c22}}) = 35$	0 kNm			
Permanent load ratio;		$R_{PL} = M_{QF}$	• / M = 0.74					
Service stress in reinforcement;		$\sigma_{sr} = f_{yd} \times$	A _{s,req} / A _{s,prov} ×	$< R_{PL} = 52 \text{ N/mm}^2$				
Maximum bar spacing (Tables 7	.3N);	Sbar,max = 3	300 mm					
	P	ASS - Maxim	um bar spaci	ng exceeds actua	n bar spacing f	or crack control		
Minimum bar spacing								
Minimum bottom bar spacing;		$S_{bot,min} = (I$	$-2 \times c_{nom_s}$ -	$2 imes \phi_v$ - ϕ_{bot}) / (N _{bot}	- 1) = 151 mm			
Minimum allowable bottom bar s	pacing;	Sbar_bot,min	= max(ø _{bot} , h _{ag}	_{gg} + 5 mm, 20 mm)	+ φ _{bot} = 50 mm			
Minimum top bar spacing;		$S_{top,min} = (I$	$rac{-2 \times c_{nom_s}}{-}$	$2 \times \phi_v$ - $\phi_{top}) / (N_{top})$	- 1) = 151 mm			
Minimum allowable top bar space								
within allowable top bar opae	ing;	Sbar_top,min	= max(\u00f6 _{top} , h _{ag}	_{gg} + 5 mm, 20 mm)	+ φ _{top} = 50 mm			

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Area of concrete in the tensile zone;	A _{ct} = b × y = 1114505 mm ²
Minimum area of reinforcement required (exp.7.1);	$A_{sc,min} = k_c \times k \times f_{ct,eff} \times A_{ct} / \sigma_s = 3637 \text{ mm}^2$
PASS - Area of tension	on reinforcement provided exceeds minimum required for crack control
Quasi-permanent value of variable action;	ψ ₂ = 0.30
Quasi-permanent limit state moment;	$M_{QP} = abs(M_{s1_c21}) + \psi_2 \times abs(M_{s1_c22}) = 366 \text{ kNm}$
Permanent load ratio;	$R_{PL} = M_{QP} / M = 0.74$
Service stress in reinforcement;	$\sigma_{sr} = f_{yd} \times A_{s,req} \ / \ A_{s,prov} \times R_{PL} = \textbf{54} \ N/mm^2$
Maximum bar spacing (Tables 7.3N);	s _{bar,max} = 300 mm
PAS	S - Maximum bar spacing exceeds actual bar spacing for crack control
Minimum bar spacing	
Minimum bottom bar spacing;	$s_{bot,min} = (b - 2 \times c_{nom_s} - 2 \times \phi_v - \phi_{bot}) / (N_{bot} - 1) = 151 \text{ mm}$
Minimum allowable bottom bar spacing;	$s_{bar_{bot,min}} = max(\phi_{bot}, h_{agg} + 5 \text{ mm}, 20 \text{ mm}) + \phi_{bot} = 50 \text{ mm}$
Minimum top bar spacing;	$s_{top,min} = (b - 2 \times c_{nom_s} - 2 \times \phi_v - \phi_{top}) / (N_{top} - 1) = 150 \text{ mm}$
Minimum allowable top bar spacing;	$S_{bar_{top,min}} = max(\phi_{top}, h_{agg} + 5 mm, 20 mm) + \phi_{top} = 64 mm$
	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 × [11 + 1.5 × (f _{ck} / 1 N/mm ²) ^{0.5} × ρ_{m0} / ρ_{m} + 3.2 ×
	$(f_{ck} / 1 N/mm^2)^{0.5} \times (\rho_{m0} / \rho_m - 1)^{1.5}] = 1820.132$
Reinforcement factor (exp.7.17);	K _s = min(A _{s,prov} / A _{s,req} × 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 _{s1} / d = 4.853
	PASS - Actual span to depth ratio is within the allowable limit
Support B	



Rectangular section in flexure (Section 6.1)

Design bending moment; Depth to tension reinforcement; Percentage redistribution;

Redistribution ratio;

$$\begin{split} M &= abs(M_{B_red}) = \textbf{2965} \text{ kNm} \\ d &= h - c_{nom_t} - \varphi_v - \varphi_{top} \ / \ 2 = \textbf{1428} \ mm \\ m_{rB} &= \textbf{0} \ \% \\ \delta &= min(1 - m_{rB}, \ 1) = \textbf{1.000} \\ K &= M \ / \ (b \times d^2 \times f_{ck}) = \textbf{0.024} \end{split}$$

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					0			
		K' = 0.59	8×δ-0.181× אי	$\delta^2 - 0.21 = 0.207$, occion roinfor	omont is requires		
l ever arm:		z = min($(d/2) \times [1 + (1)]$	- 3.53 × K) ^{0.5}]. 0.5	95 × d) = 1356	mm		
Depth of neutral axis:		$x = 2.5 \times$	(d - z) = 178 n	nm	00 / u / = 1000			
Area of tension reinforcement requ	ired;	$A_{s,reg} = N$	$(1 / (f_{vd} \times z) = 50)$	29 mm ²				
Tension reinforcement provided;		12 × 25¢	bars					
Area of tension reinforcement prov	ided;	A _{s,prov} =	5890 mm²					
Minimum area of reinforcement (e>	(p.9.1N);	A _{s,min} = r	$max(0.26 \times f_{ctm})$	/ f _{yk} , 0.0013) × b :	× d = 3907 mm ²	2		
Maximum area of reinforcement (c	l.9.2.1.1(3));	A _{s,max} = 0	$0.04 \times b \times h = 9$	90000 mm ²				
· · · · · · · · · · · · · · · · · · ·	PASS - Area	of reinforc	ement provide	ed is greater tha	n area of reinfo	orcement required		
Rectangular section in shear (Se	ection 6.2)							
Design shear force at support B;		V _{Ed,max} =	abs(max(V _{B_m}	ax, V _{B_red})) = 1310) kN			
Angle of comp. shear strut for max	imum shear;	$\theta_{max} = 45$	5 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 sh	ear force at s	upport is less th	an maximum d	design shear force		
Design shear force span 1 at 5573	mm;	$V_{Ed} = ab$	s(min(V _{B_s1_max}	, V _{B_s1_red})) = 829) kN			
Design shear stress;		$v_{Ed} = V_{Ed}$	$(b \times z) = 0.40$	08 N/mm²				
Strength reduction factor (cl.6.2.3($V_1 = 0.6$	v ₁ = 0.6 × [1 - f _{ck} / 250 N/mm ²] = 0.504						
Compression chord coefficient (cl.6	5.2.3(3));	$\alpha_{cw} = 1.0$	00					
Angle of concrete compression stru	ut (cl.6.2.3);	((a =	A · F · /0	,, ,				
	$\theta = m$	$n(max(0.5 \times$	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 requir	ed (exp.6.13);	$A_{sv,req} = V$	$v_{Ed} \times b / (t_{yd} \times c)$	$ot(\theta)) = 563 \text{ mm}^2$	/m			
Shear reinforcement provided;	l-	6 × 10φ I	egs at 300 c/c					
Area of shear reinforcement provid	ea; ant (ave 0 ENI)	Asv,prov =	15/1 mm²/m					
winninum area of snear reinforcem	ent (exp.9.5N)	, $A_{sv,min} =$	u.uo IN/IIIM ² × I	o × (I _{ck} / I IN/MM ²	j / i _{yk} = 1518 vided exceeds /	minimum requirer		
Maximum longitudinal spacing (ex	o.9.6N);	Svl,max = (0.75 × d = 107 1	mm	ACA GAUGEUS			
	PASS - Lor	gitudinal s	pacing of shea	ar reinforcement	t provided is le	ess than maximum		
Design shear force span 2 at 1428	mm;	V _{Ed} = ma	ax(VB_s2_max, VB	_s2_red) = 1071 kN	I			
Design shear stress;		$v_{\text{Ed}} = V_{\text{Ed}}$	u / (b × z) = 0.5 2	27 N/mm²				
Strength reduction factor (cl.6.2.3(3));	$v_1 = 0.6$	$v_1 = 0.6 \times [1 - f_{ck} / 250 \text{ N/mm}^2] = 0.504$					
Compression chord coefficient (cl.	6.2.3(3));	α _{cw} = 1.0	$\alpha_{cw} = 1.00$					
Angle of concrete compression stru	ut (cl.6.2.3);							
	$\theta = mi$	n(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 requir	ed (exp.6.13);	$A_{sv,req} = V$	$v_{Ed} \times b / (f_{yd} \times c)$	$ot(\theta)) = 727 \text{ mm}^2$	²/m			
Shear reinforcement provided;		6 × 10φ I	egs at 300 c/c					
Area of shear reinforcement provid	ed;	Asv,prov =	1571 mm²/m			0 .		
Minimum area of shear reinforcem	ent (exp.9.5N)	; Asv,min =	0.08 N/mm ² × I	o × (f _{ck} / 1 N/mm ²) ^{0.5} / f _{yk} = 1518	mm²/m minimum roquiroq		
Maximum longitudinal enacing (ov	9 6 11.	г АЗЗ - АГС С	a UI SHEAF FEIR	Inorcement prov	ideu exceeas l	minininini required		
Maximum longituumai spacing (exp	PASS - Lor	svi,max = 0	pacina of she	ar reinforcemen	t provided is le	ess than maximum		
Track control (Section 7.3)								
Maximum crack width		$W_k = 0.3$	mm					
Design value modulus of elasticity	reinf (3.2.7(4))	; E _s = 200	000 N/mm ²					
Mean value of concrete tensile stre	enath:	fct.eff = fct	m = 3.5 N/mm ²					

 $k_c = \boldsymbol{0.4}$

 $k = min(max(1 + (300 \text{ mm - } min(h, b)) \times 0.35 \text{ / } 500 \text{ mm}, 0.65), 1) = \textbf{0.65}$

 $s_{bar} = (b - 2 \times (c_{nom_s} + \phi_v) - \phi_{top}) / (N_{top} - 1) = 123 \text{ mm}$

Stress distribution coefficient;

Actual tension bar spacing;

Non-uniform self-equilibrating stress coefficient;

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	By	31/03/16		Max 10	Approved	A == 10			
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Maximum stress permitted (Tab	le 7.3N);	σs = 301 N	l/mm ²						
Concrete to steel modulus of el	ast. ratio;	$\alpha_{cr} = E_s / E_s$	_{cm} = 5.68						
Distance of the Elastic NA from	bottom of beam;	$y = (b \times h^2)$	/ 2 + A _{s,prov} ×	$(\alpha_{cr} - 1) \times (h - d)$) / (b \times h + A _{s,pro}	_v × (α _{cr} - 1)) = 742			
		mm							
Area of concrete in the tensile z	one;	$A_{ct} = b \times y$	= 1112703 m	1m ²					
Minimum area of reinforcement	required (exp.7.	1); A _{sc,min} = k _c	$\timesk\times f_{\text{ct,eff}}\times A$	$A_{ct} / \sigma_s = 3367 \text{ mm}$	n²				
PA	SS - Area of ten	sion reinforce	ement provid	led exceeds min	nimum required	l for crack control			
Quasi-permanent value of variable action;		$\psi_2 = 0.30$	$\psi_2 = 0.30$						
Quasi-permanent limit state mo	ment;	$M_{QP} = abs$	$M_{QP} = abs(M_{B_c21}) + \psi_2 \times abs(M_{B_c22}) = 2196 \text{ kNm}$						
Permanent load ratio;		$R_{PL} = M_{QP}$	$R_{PL} = M_{QP} / M = 0.74$						
Service stress in reinforcement;		$\sigma_{\text{sr}} = f_{\text{yd}} \times I$	$\sigma_{sr} = f_{yd} \times A_{s,req} / A_{s,prov} \times R_{PL} = \textbf{275} \ N/mm^2$						
Maximum bar spacing (Tables 7	7.3N);	Sbar,max = 1	s _{bar,max} = 150 mm						
	F	PASS - Maximu	ım bar spaci	ng exceeds acti	ual bar spacing	for crack control			
Minimum bar spacing									
Minimum bottom bar spacing;		Sbot,min = (b	- 2 \times Cnom_s -	$2\times\varphi_v$ - $\varphi_{bot})$ / (N	bot - 1) = 123 mm	n			
Minimum allowable bottom bar	spacing;	Sbar_bot,min =	$S_{bar_bot,min} = max(\phi_{bot}, h_{agg} + 5 mm, 20 mm) + \phi_{bot} = 50 mm$						
Minimum top bar spacing;		$s_{top,min} = (b)$	$s_{top,min} = (b - 2 \times c_{nom_s} - 2 \times \phi_v - \phi_{top}) / (N_{top} - 1) = 123 \text{ mm}$						
Minimum allowable top bar space	cing;	Sbar_top,min =	$S_{bar_top,min} = max(\phi_{top}, h_{agg} + 5 mm, 20 mm) + \phi_{top} = 50 mm$						
			PASS -	Actual bar spac	ing exceeds m	inimum allowable			
Mid span 2									
		• • • • •	10 x 20¢ b	ars					
		6 x 10φ shear legs at 300 c/c							

12 x 25∳ bars

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

4

*

Design bending moment;	M = abs(M _{s2_red}) = 2239 kNm
Depth to tension reinforcement;	d = h - c _{nom_b} - φ _v - φ _{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 × (d - z) = 180 mm
Area of tension reinforcement required;	A _{s,req} = M / (f _{yd} × z) = 3758 mm ²
Tension reinforcement provided;	$12 \times 25\phi$ bars
Area of tension reinforcement provided;	A _{s,prov} = 5890 mm ²
Minimum area of reinforcement (exp.9.1N);	$A_{s,min} = max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times b \times d = 3948 \text{ mm}^2$
Maximum area of reinforcement (cl.9.2.1.1(3));	$A_{s,max} = 0.04 \times b \times h = 90000 \text{ mm}^2$
PASS - Area o	of reinforcement provided is greater than area of reinforcement required

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 Rectangular section in shear (Section 6.2)									
Shear reinforcement provided;	, ,	6 ×	10¢ leg	s at 300 c/c						
Area of shear reinforcement prov	vided;	A _{sv} ,	prov = 1 5	5 71 mm²/m						
Minimum area of shear reinforcement (exp.9.5N); $A_{sv,min} = 0.08 \text{ N/mm}^2 \times b \times (f_{ck} / 1 \text{ N/mm}^2)^{0.5} / f_{yk} = 1518 \text{ mm}^2/\text{m}^2$										
		PASS	- Area d	of shear reinfo	orcement provide	d exceeds mi	nimum required			
Maximum longitudinal spacing (e	exp.9.6N);	S _{vI,n}	nax = 0.7	′5 × d = 1082 r	nm					
	PASS - L	ongitudi	nal spa	cing of shear	reinforcement pr	ovided is less	s than maximum			
Design shear resistance (assum	ing $\cot(\theta)$ is 2.	5); V _{pro}	ov = 2.5	$\times A_{sv,prov} \times z \times z$	f _{yd} = 2339.8 kN					
Shear lii	nks provided	valid bet	ween 0	mm and 1590	0 mm with tensio	n reinforcem	ent of 5890 mm ²			
Crack control (Section 7.3)										
Maximum crack width;		Wk :	= 0.3 m	m						
Design value modulus of elastici	ty reinf (3.2.7(4	4)); Es :	= 20000	0 N/mm ²						
Mean value of concrete tensile s	trength;	f _{ct,et}	$f = f_{ctm} =$	- 3.5 N/mm²						
Stress distribution coefficient;		kc =	- 0.4							
Non-uniform self-equilibrating st	ress coefficien	t; k =	min(ma	ix(1 + (300 mm	n - min(h, b)) × 0.3	5 / 500 mm, 0.	65), 1) = 0.65			
Actual tension bar spacing;		Sbar	$s_{bar} = (b - 2 \times (c_{nom_s} + \phi_v) - \phi_{bot}) / (N_{bot} - 1) = 123 \text{ mm}$							
Maximum stress permitted (Table 7.3N);			σ _s = 301 N/mm ²							
Concrete to steel modulus of elast. ratio;			$\alpha_{\rm cr} = E_{\rm s} / E_{\rm cm} = 5.68$							
Distance of the Elastic NA from bottom of beam;			$(b \times h^2)$	$/2 + A_{s,prov} \times (e)$	$\alpha_{cr} - 1) \times (h - d)) / ($	$b \times h + A_{s,prov}$	$(\alpha_{cr} - 1)) = 742$			
	mm	۱		0						
Area of concrete in the tensile zone;			= b × y	= 1112431 mm	1 ²					
Minimum area of reinforcement i	required (exp. /	(.1); A _{sc}	$min = K_c$	\times K \times f _{ct,eff} \times A _c	$t / \sigma_s = 3367 \text{ mm}^2$					
	55 - Area of te	ension re		ment provide	a exceeas minim	um requirea t	or crack control			
Quasi-permanent value of variat		Ψ2 :	= 0.30							
Quasi-permanent limit state mon	nent;		= abs(_IVI _{s2_c21}) + Ψ2 × / M – Ο 7 4	$abs(IVI_{s2_{c22}}) = 103$	9 KINITI				
Service stress in reinforcement:		Ger	$= f_{vd} \times A$	$A_{\rm a rog} / A_{\rm a prov} \times I$	B _{PI} = 205 N/mm ²					
Maximum bar spacing (Tables 7	.3N):	Shar	s _{bar,max} = 200 mm							
······································	,,	PASS - I	Maximu	m bar spacing	g exceeds actual	bar spacing f	or crack control			
Minimum bar spacing					-					
Minimum bottom bar spacing:		Sbot	.min = (b	- 2 × Cnom s - 2	$\times \phi_{V} - \phi_{bot}) / (N_{bot} -$	1) = 123 mm				
Minimum allowable bottom bar s	pacing;	Sbar	bot.min =	max(ϕ_{bot} , h _{agg}	+ 5 mm, 20 mm) +	- φ _{bot} = 50 mm				
Minimum top bar spacing;		Stop	Stop.min = $(\mathbf{b} - 2 \times C_{\text{nom}} + 2 \times \phi_{\text{V}} - \phi_{\text{top}}) / (N_{\text{top}} - 1) = 151 \text{ mm}$							
Minimum allowable top bar spac	ing;	Sbar	_top,min =	max(\u00f6top, hagg	+ 5 mm, 20 mm) +	- φ _{top} = 45 mm				
				PASS - A	ctual bar spacing	exceeds min	imum allowable			
Deflection control (Section 7.4)									
Reference reinforcement ratio;		ρ _{m0}	$= (f_{ck} / $	1 N/mm²) ^{0.5} / 1	000 = 0.006					
Required tension reinforcement	ratio;	ρ_{m}	= A _{s,req} /	′ (b × d) = 0.00	2					
Required compression reinforce	ment ratio;	ρ' m	= A _{s2,rec}	, / (b × d) = 0.0	00					
Structural system factor (Table 7	′.4N);	K _b :	= 1.5							
Basic allowable span to depth ra	tio (7.16a);	spa (f _{ck}	in_to_d / 1 N/m	${ m epth_{basic}=K_b imes}$ ${ m m^2)^{0.5} imes}(ho_{ m m0}/ ho_{ m m0}$	$[11 + 1.5 \times (f_{ck} / 1)]$ om - 1) ^{1.5}] = 198.63	N/mm ²) ^{0.5} × ρ_r 2	_no / $ ho_m$ + 3.2 $ imes$			
Reinforcement factor (exp.7.17);		Ks	= min(A	$_{\rm s,prov}$ / A $_{\rm s,req}$ $ imes$ 5	00 N/mm² / f _{yk} , 1.5) = 1.500				
Flange width factor;		F1	F1 = 1.000							
Long span supporting brittle part	ition factor;	F2	F2 = 1.000							
Allowable span to depth ratio;		spa 60.	ın_to_d 000	epth _{allow} = min(span_to_depth _{basic}	$\times K_s \times F1 \times F1$	2, 40 × K _b) =			
Actual span to depth ratio;		spa	in_to_d	$epth_{actual} = L_{s2}$	/ d = 11.023					

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

Support C 12 x 256 bars 500 $6 \; x \; 10_{\varphi}$ shear legs at 300 c/c $12 \text{ x} 25_{\varphi} \text{ bars}$ 1500 Rectangular section in flexure (Section 6.1) Design bending moment; $M = abs(M_{C red}) = 2983 kNm$ Depth to tension reinforcement; $d = h - c_{nom_t} - \phi_v - \phi_{top} / 2 = 1428 \text{ mm}$ 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 $z = min((d / 2) \times [1 + (1 - 3.53 \times K)^{0.5}], 0.95 \times d) = 1356 mm$ Lever arm: Depth of neutral axis; $x = 2.5 \times (d - z) = 178 \text{ mm}$ Area of tension reinforcement required; $A_{s,req} = M / (f_{yd} \times z) = 5059 \text{ mm}^2$ Tension reinforcement provided; $12 \times 25\phi$ bars Area of tension reinforcement provided; A_{s,prov} = 5890 mm² Minimum area of reinforcement (exp.9.1N); $A_{s,min} = max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times b \times d = 3907 \text{ mm}^2$ Maximum area of reinforcement (cl.9.2.1.1(3)); $A_{s.max} = 0.04 \times b \times h = 90000 \text{ mm}^2$ 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; $V_{Ed,max} = abs(max(V_{C_max}, V_{C_red})) = 1313 \text{ kN}$ Angle of comp. shear strut for maximum shear; $\theta_{max} = 45 \text{ deg}$ Maximum design shear force (exp.6.9); $V_{Rd,max} = b \times z \times v_1 \times f_{cd} / (cot(\theta_{max}) + tan(\theta_{max})) = 11619 \text{ kN}$ PASS - Design shear force at support is less than maximum design shear force Design shear force span 2 at 14473 mm; $V_{Ed} = abs(min(V_{C_s2_max}, V_{C_s2_red})) = 1073 \text{ kN}$ Design shear stress; $v_{Ed} = V_{Ed} / (b \times z) = 0.528 \text{ N/mm}^2$ $v_1 = 0.6 \times [1 - f_{ck} / 250 \text{ N/mm}^2] = 0.504$ Strength reduction factor (cl.6.2.3(3)); Compression chord coefficient (cl.6.2.3(3)); $\alpha_{cw} = 1.00$ Angle of concrete compression strut (cl.6.2.3); $\theta = \min(\max(0.5 \times Asin[\min(2 \times v_{Ed} / (\alpha_{cw} \times f_{cd} \times v_1), 1)], 21.8 \text{ deg}), 45 \text{ deg}) = 21.8 \text{ deg})$ Area of shear reinforcement required (exp.6.13); $A_{sv,req} = v_{Ed} \times b / (f_{yd} \times cot(\theta)) = 728 \text{ mm}^2/\text{m}$ Shear reinforcement provided; $6 \times 10\phi$ legs at 300 c/c Area of shear reinforcement provided; Asv,prov = 1571 mm²/m Minimum area of shear reinforcement (exp.9.5N); $A_{sv,min} = 0.08 \text{ N/mm}^2 \times b \times (f_{ck} / 1 \text{ N/mm}^2)^{0.5} / f_{yk} = 1518 \text{ mm}^2/\text{m}$ PASS - Area of shear reinforcement provided exceeds minimum required Maximum longitudinal spacing (exp.9.6N); $S_{vl,max} = 0.75 \times d = 1071 \text{ mm}$

PASS - Longitudinal spacing of shear reinforcement provided is less than maximumDesign shear force span 3 at 1428 mm;VEd = max(VC s3 max, VC s3 red) = 843 kN

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Decian chear stress:			$(b \times z) = 0.414$ N	l/mm ²						
Strongth reduction factor (al 6.2)	2(2)).	$v_{Ed} = v_{Ed}$	$(0 \times 2) = 0.414$ N	$m^{21} = 0.504$						
	S(3)),	$\mathbf{v}_1 = \mathbf{0.0 \times [}$	1 - 1 _{ck} / 230 N/III	11] = 0.504						
	(1.0.2.3(3)),	$\alpha_{cw} = 1.00$								
Angle of concrete compressions	θ – m	$\sin(\max(0.5 \times \Delta))$	sin[min(2 × v _{Ed} /	$(\alpha_{ov} \times f_{od} \times v_1)$ 1)]	21.8 dea) 4	5dea) – 21 8 dea				
Area of shear reinforcement requ	uired (exp 6 13)		$x = b / (f_{val} \times cot)$	$(0.00 \times 100 \times 01), 1)$, 21.0 dog), 4	ucg) = 21.0 ucg				
Shear reinforcement provided:		6 × 10φ lea	1×0 / (i) 2×00 (i))) = 312 mm /m						
Area of shear reinforcement provided,	vided.	6×1000 leg	571 mm ² /m							
Minimum area of shear reinforce	ment (exp 9 5N). Asy min = 0.0	$0.8 \text{ N/mm}^2 \times \text{b} \times ($	f _{ok} / 1 N/mm ²) ^{0.5} /	f _{vk} = 1518 mn	n²/m				
		PASS - Area c	of shear reinfor	cement provided	d exceeds mi	nimum reauired				
Maximum longitudinal spacing (e	exp.9.6N):	Sylmax = 0.7	′5 × d = 1071 mm	n						
	PASS - Lo	naitudinal spa	cing of shear re	inforcement pro	ovided is less	than maximum				
Creak control (Section 7.2)		3	9	· · · · · · · · · · · · · · · · · · ·						
Maximum crack width:		w ₄ – 0 3 m	m							
Design value modulus of elastici	ity reinf (3 2 7(4)	$W_{R} = 0.0 \text{ mm}$	$F_{r} = 200000 \text{ N/mm}^2$							
Mean value of concrete tensile s	strength:	$f_{ct eff} = f_{ctm} =$	$f_{ct eff} = f_{ctm} = 3.5 \text{ N/mm}^2$							
Stress distribution coefficient:	, i oligui,	$k_{c} = 0.4$	k _c = 0.4							
Non-uniform self-equilibrating st	ress coefficient:	k = min(ma	k = min(max(1 + (300 mm - min(h, b)) × 0.35 / 500 mm, 0.65), 1) = 0.65							
Actual tension bar spacing:	,	s _{bar} = (b - 2	$S_{bar} = (b - 2 \times (c_{nom s} + \phi_v) - \phi_{top}) / (N_{top} - 1) = 123 \text{ mm}$							
Maximum stress permitted (Tabl	le 7.3N):	$\sigma_{\rm s} = 301 {\rm N}$	$\sigma_{\rm s} = 301 \text{ N/mm}^2$							
Concrete to steel modulus of ela	ist. ratio:	$\alpha_{cr} = E_s / E_s$	$\alpha_{\rm cr} = E_{\rm s} / E_{\rm cm} = 5.68$							
Distance of the Elastic NA from	bottom of beam	$v = (b \times h^2)$	$y = (b \times h^2 / 2 + A_{s,prov} \times (\alpha_{cr} - 1) \times (h - d)) / (b \times h + A_{s,prov} \times (\alpha_{cr} - 1)) = 742$							
		mm	mm							
Area of concrete in the tensile zo	one;	$A_{ct} = b \times y$	= 1112703 mm ²							
Minimum area of reinforcement	required (exp.7.	1); $A_{sc,min} = k_c$	imes k $ imes$ f _{ct,eff} $ imes$ A _{ct} /	σ _s = 3367 mm²						
PAS	SS - Area of ter	nsion reinforce	ment provided	exceeds minimu	ım required f	or crack control				
Quasi-permanent value of variat	ole action;	$\psi_2 = 0.30$								
Quasi-permanent limit state mor	ment;	M _{QP} = abs($M_{C_{c21}} + \psi_2 \times al$	os(M _{C_c22}) = 2210	kNm					
Permanent load ratio;		$R_{PL} = M_{QP}$	/ M = 0.74							
Service stress in reinforcement;		$\sigma_{sr} = f_{yd} \times A$	$A_{s,req}$ / $A_{s,prov}$ $ imes$ R_{P}	L = 277 N/mm²						
Maximum bar spacing (Tables 7	.3N);	Sbar,max = 15	Sbar,max = 150 mm							
	F	PASS - Maximu	m bar spacing	exceeds actual	bar spacing f	or crack control				
Minimum bar spacing										
Minimum bottom bar spacing;		Sbot,min = (b	- 2 × c _{nom_s} - 2 ×	φ _v - φ _{bot}) / (N _{bot} -	1) = 123 mm					
Minimum allowable bottom bar s	spacing;	Sbar_bot,min =	max(\$bot, hagg +	5 mm, 20 mm) +	φ _{bot} = 50 mm					
Minimum top bar spacing;		$s_{top,min} = (b$	$s_{top,min} = (b - 2 \times c_{nom_s} - 2 \times \phi_v - \phi_{top}) / (N_{top} - 1) = 123 \text{ mm}$							
Minimum allowable top bar spac	ing;	Sbar_top,min =	max(\u00f6top, hagg +	5 mm, 20 mm) +	φ _{top} = 50 mm					
			PASS - Act	ual bar spacing	exceeds min	imum allowable				

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Area of concrete in the tensile zone;	A _{ct} = b × y = 1114505 mm ²
Minimum area of reinforcement required (exp.7.1);	$A_{sc,min} = k_c \times k \times f_{ct,eff} \times A_{ct} / \sigma_s = \textbf{3637} mm^2$
PASS - Area of tension	on reinforcement provided exceeds minimum required for crack control
Quasi-permanent value of variable action;	ψ ₂ = 0.30
Quasi-permanent limit state moment;	$M_{\text{QP}} = abs(M_{s3_c21}) + \psi_2 \times abs(M_{s3_c22}) = 420 \text{ kNm}$
Permanent load ratio;	$R_{PL} = M_{QP} / M = 0.74$
Service stress in reinforcement;	$\sigma_{sr} = f_{yd} \times A_{s,req} \ / \ A_{s,prov} \times R_{PL} = \textbf{62} \ N/mm^2$
Maximum bar spacing (Tables 7.3N);	s _{bar,max} = 300 mm
PAS	S - Maximum bar spacing exceeds actual bar spacing for crack control
Minimum bar spacing	
Minimum bottom bar spacing;	$s_{bot,min} = (b - 2 \times c_{nom_s} - 2 \times \phi_v - \phi_{bot}) / (N_{bot} - 1) = 151 \text{ mm}$
Minimum allowable bottom bar spacing;	$s_{bar_{bot,min}} = max(\phi_{bot}, h_{agg} + 5 \text{ mm}, 20 \text{ mm}) + \phi_{bot} = 50 \text{ mm}$
Minimum top bar spacing;	$S_{top,min} = (b - 2 \times c_{nom_s} - 2 \times \phi_v - \phi_{top}) / (N_{top} - 1) = 151 \text{ mm}$
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 × [11 + 1.5 × (f _{ck} / 1 N/mm ²) ^{0.5} × ρ_{m0} / ρ_{m} + 3.2 ×
	$(f_{ck} / 1 N/mm^2)^{0.5} \times (\rho_{m0} / \rho_m - 1)^{1.5}] = 1477.745$
Reinforcement factor (exp.7.17);	K _s = min(A _{s,prov} / A _{s,req} × 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
Support D	



Rectangular section in flexure (Section 6.1)

Minimum moment factor (cl.9.2.1.2(1)); Design bending moment; Depth to tension reinforcement; Percentage redistribution; Redistribution ratio;
$$\begin{split} \beta_1 &= \textbf{0.25} \\ M &= max(abs(M_{D_red}), \, \beta_1 \times abs(M_{s3_red})) = \textbf{142} \ kNm \\ d &= h \ \text{-} \ c_{nom_t} \ \text{-} \ \varphi_v \ \text{-} \ \varphi_{top} \ \text{/} \ 2 = \textbf{1428} \ mm \\ m_{rD} &= \textbf{0} \ \% \\ \delta &= min(1 \ \text{-} \ m_{rD}, \ \textbf{1}) = \textbf{1.000} \end{split}$$

BRIDGES POUND	Location				Job No					
CONSULTING ENGINEERS		UCL	for Mace	M1993						
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	Base	ment Temp	orary Work	s Design	1	16				
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		K – M / /k	$a \times d^2 \times f_{\rm el}$ - 0	001						
		K' = 0.59	$S \times U \times I_{CK} = U$ $R \times S = 0.181 \times U$	$\delta^2 = 0.21 = 0.207$						
		N = 0.030	× 101.01 - 0.101	$\sim K - No compress$	ion reinforce	ment is required				
l ever arm.		z = min(l)	ע 1/2) × [1 + (1	- 3 53 × K) ^{0.5} 1 0 95 ×	v d) – 1356 mi	m				
Depth of neutral axis:		2 – mm(() x – 2 5 ×	(d _{- 7}) – 178 m	0.00 × 1()], 0.00 /	(u) = 1000 mi					
Area of tension reinforcement	required:	$\Lambda = 2.0 \times$	$(0^{-2}) = 170$ fm $(1 \times 7) = 24$	1 mm ²						
Tension reinforcement provide	required,	10 ∨ 25a	/ (1yu ~ 2) - 2 -	•••••••						
Area of tension reinforcement	provided:	Δ	900 mm ²							
Minimum area of reinforcement	p(0)(aea, 0)	$\Delta_{s, min} = m$	303 mm $28 \times f_{stm}$	/ f 0 0013) × b × d	– 3007 mm ²					
Maximum area of reinforcement	(e,p,g,h),	$A_{s,min} = 0$	0.20×1000	10000 mm^2	- 3907 11111					
Maximum area of reinforceme	PASS - Area	ns,max = 0	.04 ^ 0 ^ 11 = 3	od is areater than a	rea of reinfor	coment required				
••••			ment provide	a is greater than a		cement required				
Minimum bottom reinforcem	ent at supports									
Minimum reinforcement factor	(cl.9.2.1.4(1));	$\beta_2 = 0.25$	2							
Area of reinforcement to adjac	ent span;	$A_{s,span} = 4$	1909 mm²	2						
Minimum bottom reinforcemer	it to support;	$A_{s2,min} = F$	52 × As,span = 12	227 mm²						
Bottom reinforcement provided	Bottom reinforcement provided;			$10 \times 20\phi$ bars						
Area of bottom reinforcement	providea;	$A_{s2,prov} = 3$	3142 mm²		una of volutor					
PAS	5 - Area of reinfo	rcement prov	lided is great	er than minimum a	rea of reinford	cement required				
Rectangular section in shea	r (Section 6.2)									
Design shear force at support	D;	$V_{Ed,max} =$	abs(max(V _{D_ma}	ax, V _{D_red})) = 23 kN						
Angle of comp. shear strut for	Angle of comp. shear strut for maximum shear;			$\theta_{max} = 45 \text{ deg}$						
Maximum design shear force	(exp.6.9);	V _{Rd,max} =	$b \times z \times v_1 \times f_{cd}$	$/ (\cot(\theta_{max}) + \tan(\theta_{max}))$	ax)) = 11619 kl	N				
	PASS	- Design she	ear force at su	ipport is less than	maximum de	sign shear force				
Design shear force span 3 at s	52/3 mm;	$V_{Ed} = mat$	$X(V_{D_{s3}_{max}}, V_{D})$	$_{s3_{red}} = 216 \text{ kN}$						
Design snear stress;	2.2(2))	VEd = VEd	$V_{Ed} = V_{Ed} / (D \times Z) = 0.106 \text{ N/mm}^2$							
Strength reduction factor (cl.6.	.2.3(3));	$V_1 = 0.6 \times$	$v_1 = 0.6 \times [1 - t_{ck} / 250 \text{ N/mm}^2] = 0.504$							
Compression chord coefficient	t (cl.6.2.3(3));	$\alpha_{cw} = 1.0$	$\alpha_{\rm CW} = 1.00$							
Angle of concrete compression	n strut (cl.6.2.3);	· / /o =	A : F : (O		1.04.0 1 4					
Area of abase using the second	$\theta = m$	$m(max(0.5 \times 10^{\circ}))$	ASIN[$min(2 \times V)$	$Ed / (\alpha_{cw} \times T_{cd} \times V_1), 1)$	j, ∠1.8 aeg), 4	oueg) = 21.8 deg				
Area or snear reinforcement re	equirea (exp.6.13);	$A_{sv,req} = V$	$A_{sv,req} = V_{Ed} \times D / (I_{vd} \times COI(\theta)) = 147 \text{ mm}^2/\text{m}$							
Snear reinforcement provided	; rovidod:	6 × 10φ le	$b \times 10\phi$ legs at 300 C/C							
Minimum area of chear relation	iovided,	$A_{sv,prov} = 0$	$A_{\text{sv,prov}} = 1071 \text{ IIIIII7/III}$ $A_{\text{sv}} = 0.09 \text{ N/mm}^2 \text{ y b } \text{ y } (f_{\text{sv}}/1) \text{ N/mm}^2 \text{ N} 5 / f_{\text{sv}} = 1519 \text{ mm}^2 \text{ /mm}^2 \text{ /m}^2$							
winimum area of shear reinfor	cement (exp.a.oN)	PASS = Arco		$D = (1ck / 1 N/IIIII)^{0.0}$	$r_{\rm IVK} = 1318$ m ²	inimum roquirod				
Maximum longitudinal enacing		- AICA	75 × d = 1071	mm	u exceeds mi	ininiani required				
Maximum longitudinal spacing	DACC - 1 -	ovi,max = 0 naitudinal co	a c i n a of ebor	nini ar reinforcement ar	ovided is los	s than mavimum				
	FA33 - LU	ingituuniai sp		n rennorcement pr	Structu 13 1853	s than maximum				
Urack control (Section 7.3)										
IVIAXIMUM CRACK WIDTN;	inity raise (2.0.7/4)	$W_k = 0.3 f$	11/11 100 N/mm ²							
Mean value of concrete topold	icity rell11 (J.2.7(4) a strangth:), ⊏s = ∠∪∪ (f. "_ f	$= 3.5 \text{N/mm}^2$							
Stress distribution coefficient:	ะ รถษายุแม	$l_{ct,eff} = l_{ctm}$	i = 3.3 in/iiiii1²							
Non-uniform self-oquilibration	stress coefficient:	$r_c = 0.4$		$m = \min(h = h) \setminus (0, 2)$	5 / 500 mm 0	65) 1) - 0 65				
	Suess coenicient;		$K = \min(\max(1 + (300 \text{ mm} - \min(h, b)) \times 0.35 / 500 \text{ mm}, 0.65), 1) = 0.65$							
Maximum stress parmitted /Tr		$s_{\text{bar}} = (D - C_{\text{bar}})$		w) - ψtop) / (Iνtop - I) =						
Concrete to stool modulus of a	aure 7.01N),	$\sigma = \mathbf{Z} \mathbf{O} \mathbf{U}$	E E 69							
Distance of the Electic NA free	riasi. Idliu, n bottom of boom	$u_{cr} = E_s /$	∟cm = 3.08 2 / 3 . Λ	$(\alpha + 1) \times (b + d)) / ($	hyh A	x (a 1)) 740				
	in bollom of beam;	y = (b × h	$\mathbf{y} = (\mathbf{D} \times \mathbf{n}^{2} / 2 + \mathbf{A}_{s, prov} \times (\alpha_{cr} - 1) \times (\mathbf{n} - \mathbf{d})) / (\mathbf{D} \times \mathbf{h} + \mathbf{A}_{s, prov} \times (\alpha_{cr} - 1)) = 743$							
		111111								

 $A_{ct} = b \times y = 1114732 \text{ mm}^2$

Area of concrete in the tensile zone;

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Minimum area of reinforcement required (exp.7.1);	$A_{sc,min} = k_c \times k \times f_{ct,eff} \times A_{ct} / \sigma_s = 3638 \text{ mm}^2$
PASS - Area of tension	on reinforcement provided exceeds minimum required for crack control
Quasi-permanent value of variable action;	ψ2 = 0.30
Quasi-permanent limit state moment;	$M_{QP} = abs(M_{D_c21}) + \psi_2 \times abs(M_{D_c22}) = 419 \text{ kNm}$
Permanent load ratio;	$R_{PL} = M_{QP} / M = 2.95$
Service stress in reinforcement;	$\sigma_{sr} = f_{yd} \times A_{s,req} \ / \ A_{s,prov} \times R_{PL} = \textbf{63} \ N/mm^2$
Maximum bar spacing (Tables 7.3N);	Sbar,max = 300 mm
PAS	S - Maximum bar spacing exceeds actual bar spacing for crack control
Minimum bar spacing	
Minimum bottom bar spacing;	$S_{bot,min} = (b - 2 \times c_{nom_s} - 2 \times \phi_v - \phi_{bot}) / (N_{bot} - 1) = 151 \text{ mm}$
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;	$s_{top,min} = (b - 2 \times c_{nom_s} - 2 \times \phi_v - \phi_{top}) / (N_{top} - 1) = 151 \text{ mm}$
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.



Sheet No 18 By 31/03/16 Checked Approved Approved Approved Materials Name Density Youngs Modulus Shear Modulus Thermal Coefficient Materials Name Density Youngs Modulus Shear Modulus Thermal Coefficient Sections 34 14.2 0.00001 0.00001 Sections Major Minor A, Ac. (cm?) (cm?) (cm?) (cm?) (cm?) R 1500x1500 22:500 4.x10? 18750 18750 Element Length Nodes Section Material Releases Rotate 1 7 1 2 R 1500x1500 Concrete (EC2 Fixed Fixed 2 15.9 2 3 R 1500x1500 Concrete (EC2 Fixed Fixed 3 6.7 3 4 R 1500x1500 Concrete (EC2 Fixed Fixed Name <th< th=""><th>BRID CONSU</th><th>JLTING ENG</th><th>UND NEERS</th><th colspan="6">Location Job No UCL for Mace M1993</th><th>993</th></th<>	BRID CONSU	JLTING ENG	UND NEERS	Location Job No UCL for Mace M1993						993			
$\begin{array}{ c c c c c c c c c c c c c c c c c c c$					Basement Temporary Works Design						et No	3	
Shift 31/03/16 PLN Mar 16 Shift Apr 16 Materials Name Density Youngs Modulus Shear Modulus Thermal Coefficient Name Density Youngs Modulus Shear Modulus Thermal Coefficient (kg/m²) kN/mm² kN/mm² "eC4" Concrete (EC2 2500 34 14.2 0.00001 Sections Mare Area Moment of inertia Shear area Major Minor Ay Ay (cm²) (cm²) (cm²) (cm²) (cm²) R 1500x1500 22500 4.×10² 18750 18750 Elements Element End Axial Start End (m) Start End Start End Axial 1 7 1 2 R 1500x1500 Concrete (EC2 Fixed Fixed 1 7 1 2 R 1500x1500 Concrete (EC2 Fixed Fixed 1 7 1 3 130x1500 Concrete (EC2 Fixed Fixed Name Start End End Member Member 1 3 Loading SSS				Bv	Dust		Check	ked		Solgh	Appro	ved	•
Materials Name Density Youngs Modulus Shear Modulus Thermal Coefficient Concrete (EC2 normal) 2500 34 14.2 0.00001 Sections Name Area Moment of inertia Shear area Major Minor Ay Ay Ay (cm?) (cm?) (cm?) (cm?) (cm?) R 1500x1500 22500 4.x10? 18750 18750 Elements Length Nodes Section Material Releases Rotates (m) Start End Start End Axial moment moment moment Axial 1 7 1 2 R 1500x1500 Concrete (EC2 Fixed fix				SMH		31/03/16	RLN			Mar 16	SMH		Apr 16
Name Density (kg/m ²) Youngs Modulus kN/mm ² Shear Modulus Nmm ² Thermal Coefficient Concrete (EC2 normal) Sections	Materia	als											
Image: Normal basis KN/mm ² KN/mm ² N/mm^2 <td>Na</td> <td>me</td> <td>Density</td> <td>y</td> <td>Young</td> <td>s Modulus</td> <td>Shear</td> <td>Modı</td> <td>ılus</td> <td>Thermal</td> <td>Coefficient</td> <td></td> <td></td>	Na	me	Density	y	Young	s Modulus	Shear	Modı	ılus	Thermal	Coefficient		
Concrete (EC2 normal) 2500 34 14.2 0.00001 Sections Maior Minor Ay Az Name Area Moment of inertia Shear area Major Minor Ay Az (cm ²) (cm ²) (cm ²) (cm ²) R 1500x1500 22500 4.×10 ² 18750 18750 Elements Start End Start End Axial (m) Start End moment moment moment 1 7 1 2 R 1500x1500 Concrete (EC2) Fixed Fixed 1 7 1 2 R 1500x1500 Concrete (EC2) Fixed Fixed 2 15.9 2 3 R 1500x1500 Concrete (EC2) Fixed Fixed Name Elements Start End mommal) Member Elements Start End Member 1 3 . . Start End Member Start End .			(kg/m^3))	kN	I/mm ²	kN	/mm ²		0	C-1	_	
Sections Name Area Moment of inertia Shear area Major Minor A, A. (cm ²) (cm ³) (cm ²) (cm ²) Istoox1500 22500 4.×10 ² 18750 18750 Elements Element Length Nodes Section Material Releases Rotates (m) Start End Start End Axial moment moment 1 7 1 2 R1500x1500 Concrete (EC2) Fixed Fixed 2 15.9 2 3 R1500x1500 Concrete (EC2) Fixed Fixed Start End Members Elements Elements Elements Elements Name Elements Elements Elements Elements Elements Elements Start End Members Elements Elements Elements Elements Start End Elements Start End Member Elements Elements	Concret norr	te (EC2 nal)	2500			34	1	4.2		0.0	0001		
Name Area Moment of inertia Shear area Major Minor Ay Az (cm ²) (cm ²) (cm ²) (cm ²) R 1500x 1500 22500 4×10^2 18750 18750 Elements Section Material Releases Rotate (m) Start End Axial moment moment 1 7 1 2 R 1500x 1500 Concrete (EC2) Fixed Fixed Fixed 2 15.9 2 3 R 1500x 1500 Concrete (EC2) Fixed Fixed Fixed 3 6.7 3 4 R 1500x 1500 Concrete (EC2) Fixed Fixed Fixed Members Start Elements	Sectio	ns											
$\begin{tabular}{ c c c c c c } \hline Minor & A, & A$	Na	me	Area	Мо	ment of	inertia	:	Shear	area				
(cm ²)(cm ²)(cm ²)(cm ²)(cm ²)R 1500x150022500 $4,x10^2$ $4,x10^2$ 1875018750ElementsElementLengthNodesSectionMaterialReleasesRotate(m)StartEndStartEndAxialmomentmoment1712R 1500x1500Concrete (EC2)FixedFixedFixed215.923R 1500x1500Concrete (EC2)FixedFixedFixed36.734R 1500x1500Concrete (EC2)FixedFixedFixedMembersElementsElementsElementsStartEndKeelaseKeelaseNameElementsStartEndMember13LoadingPermanent-LoadingQ S S S S S S S S S S S S S S S S S S S				Majo	or	Minor	A_y			Az			
R 1500x1500 22500 4×10^7 4×10^7 18750 18750 Elements (m) Start End Nodes Section Material Releases Rotate 1 7 1 2 R 1500x1500 Concrete (EC2) Fixed Fixed Fixed 2 15.9 2 3 R 1500x1500 Concrete (EC2) Fixed Fixed Fixed 3 6.7 3 4 R 1500x1500 Concrete (EC2) Fixed Fixed Fixed Members Name Elements Start End Member I 3 Image: Start End Image: Start End Image: Start End Image: Start Image: Sta			(cm^2)	(cm ²	4)	(cm ⁴)	(cm ²))	(c	2m ²)			
Belements Element Length Nodes Section Material Releases Rotates (m) Start End moment mome	R 1500	0x1500	22500	4.×1	07	$4. \times 10^{7}$	1875	0	18	3750			
Element Length Nodes Section Material Releases Kotae (m) Start End moment moment moment 1 7 1 2 R 1500x1500 Concrete (EC2 Fixed Fixed Fixed 2 15.9 2 3 R 1500x1500 Concrete (EC2 Fixed Fixed Fixed 3 6.7 3 4 R 1500x1500 Concrete (EC2 Fixed Fixed Fixed Members Name Elements normal) Fixed Fixed Fixed Member 1 3 3 Member S S S Loading Permanent - Loading Member Member Deflection Axial deflection Member results Load case: Permanent Member Deflection Axial deflection Member Offection Axial deflection Member Image: Pois Max Pos Min Pos Min </td <td>Eleme</td> <td>nts</td> <td>NT - 1-</td> <td></td> <td>0.</td> <td></td> <td>М</td> <td> 1</td> <td></td> <td></td> <td>D 1</td> <td></td> <td>Datata</td>	Eleme	nts	NT - 1-		0.		М	1			D 1		Datata
$I(m) Surt End \\ moment moment \\ moment moment \\ moment \\$	Element	Length	Node	S End	Se	ction	Ма	terial		Start	Keleases End	Avial	Kotateo
$1 7 1 2 R \ 1500 \times 1500 Concrete (EC2 normal) \\ 1 7 1 2 R \ 1500 \times 1500 Concrete (EC2 normal) \\ 2 15.9 2 3 R \ 1500 \times 1500 Concrete (EC2 normal) \\ 3 6.7 3 4 R \ 1500 \times 1500 Concrete (EC2 normal) \\ \hline Members \\ \hline Name \\ \hline Start \\ \hline Elements \\ \hline Start \\ \hline End \\ \hline Member \\ \hline 1 \\ \hline 3 \\ \hline Loading \\ \hline \hline \\ \hline $		(111)	Start	EHQ						moment	moment	Axiai	
$\begin{array}{c ccccccccccccccccccccccccccccccccccc$	1	7	1	2	R 150	00x1500	Concre	ete (E	C2	Fixed	Fixed	Fixed	
3 6.7 3 4 R 1500x1500 Concrete (EC2 Fixed Fixed Fixed normal) Members Name Elements Start End Member 1 3 Loading Permanent-Loading Permanent-Loading Permanent-Loading Permanent-Loading Permanent-Loading Results Total deflection Member results Load case: Permanent Member Deflection Axial deflection Member 0 0 0 0 0	2	15.9	2	3	3 R 1500x1500		noi Concre	rmal) ete (E	C2	Fixed	Fixed	Fixed	
Members Name Elements <u>Start End</u> Member 1 3 Loading Permanent-Loading Permanent-Loading Permanent-Loading Permanent-Loading Permanent-Loading Permanent-Loading Permanent-Loading Permanent-Loading Nember SS SS	3	6.7	3	4	R 150	00x1500	Concre	rmai) ete (E rmal)	C2	Fixed	Fixed	Fixed	
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B Constant		.				Perman	ent - Loading	I					
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Member Results Total deflection Member results Load case: Permanent Member Axial deflection Member Min Pos Min Member Min Min Min Member Deflection Axial deflection Member Deflection Axial deflection Member Deflection Axial deflection Mamber Ideflection Min Min Mamber Deflection Axial deflection Mamber Ideflection Min Min Mamber Deflection Axial deflection Min Mamber Ideflection Min Min Min			122		122				¢ 5 7	122	122		
Member Results Total deflection Member results Load case: Permanent Member Deflection Axial deflection Pos Max Pos Min (m) (m) (m) (m) Member 0.2			× ×		Y					Y	y		
Results Total deflection Member results Load case: Permanent Member Deflection Member Deflection Member Deflection Member Deflection Member Deflection Max Pos Min Momber 14.047 2.4 5.044 0.2 0 0 0 0			Ž			М	ember						
Results Total deflection Member results Load case: Permanent Member Deflection Axial deflection Pos Max Pos Min Pos Min (m) (mm) (m) (mm) (m) (mm) (m) (mm) Mamber 14.047 2.4 5.044 0.2 0 0 0 0 0													
Total deflection Member results Load case: Permanent Member Deflection Pos Max Pos Min Pos Max Pos Min (m) (mm) (m) (mm) (m) (mm) (m) (mm) Mambar Mambar	Result	s											
Member results Load case: Permanent Member Deflection Pos Max Pos Min (m) (m) (m) (m) (m) Mamber 14.047 2.4 5.044 0.2 0 0	Total d	<u>-</u> Iofloation											
Member results Load case: Permanent Member Deflection Axial deflection Pos Max Pos Min Pos Min (m) (mm) (m) (mm) (m) (mm) (m) Mamber 14.047 2.4 5.044 0.2 0 0 0 0													
Load case: Permanent Member Deflection Axial deflection Pos Max Pos Min Pos Max Pos Min (m) (mm) (m) (mm) (m) (mm) (m) (mm) Mamber 14.047 2.4 5.044 0.2 0 0 0 0	Membe	er results											
Member Deflection Axial deflection Pos Max Pos Min Pos Max Pos Min (m) (mm) (m) (mm) (m) (mm) (mm) (mm) Mamber 14.047 2.4 5.044 0.2 0 0 0	Load c	ase: Perm	anent		<i>a</i>								
Pos Max Pos Min Pos Max Pos Min (m) (mm) (m) (mm) (m) (mm) (mm) (mm) Mambar 14.047 2.4 5.044 0.2 0 0 0		nber	P	De	eflection		N.C.	F		Axia	I deflection		M
(m) (mm) (mm) (m) (mm) (mm) Mambar 14.047 2.4 5.044 0.2 0 0 0 0	Men		Pos	Max		POS	Min	P	os	Max	Pos		Min
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BRIDGES POUND	Location				Job No	
CONSULTING ENGINEERS		UCL fo	M1993			
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	Base	ement Tempo	orary Works D	Design	2	20
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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;	$\beta = 0 \deg$
Depth of cover;	d _{cover} = 0 mm
Retained soil properties	
Soil type;	Loose brick hardcore
Moist density;	$\gamma_{mr} = 14.1 \text{ kN/m}^3$
Saturated density;	γsr = 17.1 kN/m ³
Characteristic effective shear resistance angle;	φ'r.k = 28 deg
Characteristic wall friction angle;	$\delta_{r.k} = 14 \text{ deg}$
Loading details	
Variable surcharge load;	Surcharge _Q = 10 kN/m ²



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.

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CONSULTING ENGINEERS		UCL for Mace				1993
					Sheet No	
	Base	ement Tempo	orary Works D	Design	2	22
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	SMH	31/03/16	RLN	Mar 16	SMH	Apr 16

ANALYSIS

Loading

Tedds calculation version 1.0.10



Results

Total base reactions

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

Element end forces

Load combination: a (Strength)

Element	Length	Nodes	Axial force	Shear force	Moment
	(m)	Start/End	(kN)	(kN)	(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	Location				Job No	
CONSULTING ENGINEERS		UCL fo	М	1993		
					Sheet No	
	Base	ement Tempo	2	:3		
	Ву		Checked		Approved	
	SMH	31/03/16	RLN	Mar 16	SMH	Apr 16





Member results

Envelope - Strength combinations

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

BRIDGES POUND	Location				Job No	
CONSULTING ENGINEERS		UCL for Mace				1993
					Sheet No	
	Base	ement Tempo	orary Works D	Design	2	4
	Ву		Checked		Approved	
	SMH	31/03/16	RLN	Mar 16	SMH	Apr 16

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 = $(2 \times 19.5) / 0.044 = 891$ mm 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.

BRIDGES POUND	Location				Job No	
CONSULTING ENGINEERS	UCL for Mace				М	1993
					Sheet No	
	Base	ment Tempo	orary Works D	Design	2	25
	Ву		Checked		Approved	
	SMH	31/03/16	RLN	Mar 16	SMH	Apr 16

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

Mean value of compressive cylinder strength;

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.

RC BEAM DESIGN (EN1992-1)		
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 defo	ormation characteristics for concrete)	
Concrete strength class;	C32/40	
Characteristic compressive cylinder strength;	f _{ck} = 32 N/mm ²	
Characteristic compressive cube strength;	$f_{ck,cube} = 40 \text{ N/mm}^2$	

 $f_{cm} = f_{ck} + 8 \text{ N/mm}^2 = 40 \text{ N/mm}^2$

	Location				Job No	
C O N S U L T I N G E N G I N E E R S		UCL	for Mace		1	M1993
	Base	ment Temp	orary Work	s Design	Sheet No	26
	Ву	•	Checked		Approved	
	SMH	31/03/16	RLN	Mar 16	SMH	Apr 16
		<i>t</i> 0.0	N1/2 /f. /	A N1/2)2/3 0 0 1	N1/2	
Mean value of axial tensile stren	igtn;	$T_{ctm} = 0.3$	N/MM ² × (Tck/	$(10 \text{ N/mm}^2)^{2/3} = 3.0$	N/MM^2	
Secant modulus of elasticity of c	concrete;	Ecm = 22	KIN/IIIII ⁻ × [Icm/	$10 \text{ N/mm}^2]^{0.0} = 33$	340 N/mm ²	
Partial factor for concrete (Table	2.1N);	$\gamma_{\rm C} = 1.50$				
Compressive strength coefficien	t (cl.3.1.6(1));	$\alpha_{\rm cc} = 0.85$)	N/ 2		
Design compressive concrete st	rength (exp.3.15); $f_{cd} = \alpha_{cc} \times$	t _{ck} / γ _C = 18.1	N/mm²		
Maximum aggregate size;		h _{agg} = 20	mm			
Reinforcement details						
Characteristic yield strength of re	einforcement;	f _{yk} = 500 l	N/mm²			
Partial factor for reinforcing steel	l (Table 2.1N);	γs = 1.15				
Design yield strength of reinforce	ement;	$f_{yd} = f_{yk} / \gamma$	_{/S} = 435 N/mm	2		
Nominal cover to reinforcement	nt					
Nominal cover to top reinforcem	ent;	Cnom t = 3	5 mm			
Nominal cover to bottom reinford	cement;	Cnom b = 5	0 mm			
Nominal cover to side reinforcen	nent;	Cnom_s = 5	0 mm			
	↓ .	_750	6 x 25 _∲ b	ars		
Rectangular section in flexure	(Section 6.1) -	-750 Positive mide	6 x 25∳ b →	ars t		
Rectangular section in flexure Design bending moment;	(Section 6.1) -	-750 Positive mid: M = 809 k	6 x 25∳ b → span moment kNm	ars t		
Rectangular section in flexure Design bending moment; Depth to tension reinforcement;	(Section 6.1) -	-750 Positive mid M = 809 H d = h - cnd	6 x 25 _φ b span moment «Nm bm_b - φv - φbot /	ars t 2 = 678 mm		
Rectangular section in flexure Design bending moment; Depth to tension reinforcement; Percentage redistribution;	(Section 6.1) -	750 Positive mids M = 809 k d = h - cno mr = 0 %	6 x 25 _φ b span moment «Nm bm_b - φv - φbot /	ars t 2 = 678 mm		
Rectangular section in flexure Design bending moment; Depth to tension reinforcement; Percentage redistribution; Redistribution ratio;	(Section 6.1) -	-750 Positive mid M = 809 H d = h - c _{no} m _r = 0 % δ = min(1	6 x 25φ b span moment span moment span moment	ars t 2 = 678 mm 00		
Rectangular section in flexure Design bending moment; Depth to tension reinforcement; Percentage redistribution; Redistribution ratio;	(Section 6.1) -	-750 Positive mid M = 809 k d = h - Cno m _r = 0 % δ = min(1 K = M / (b	$6 \times 25_{\phi} b$ $\Rightarrow $ $span moment kNm bm_b - \phi_v - \phi_{bot} / - m_r, 1) = 1.00 b \times d^2 \times f_{ck}) = 0$	ars 2 = 678 mm 00 0 73		
Rectangular section in flexure Design bending moment; Depth to tension reinforcement; Percentage redistribution; Redistribution ratio;	(Section 6.1) -	-750 -	$6 \times 25\phi b$ \Rightarrow $span moments$ spa	ars 2 = 678 mm 00 0.073 δ^2 - 0.21 = 0.207		
Rectangular section in flexure Design bending moment; Depth to tension reinforcement; Percentage redistribution; Redistribution ratio;	(Section 6.1) -	-750 Positive mide $M = 809 H$ $d = h - c_{nd}$ $m_r = 0 \%$ $\delta = min(1)$ $K = M / (b)$ $K' = 0.598$	$6 \times 25\phi b$ \Rightarrow $span moment span moment sm_b - \phi_v - \phi_{bot} / f_{ck} = 0 \Rightarrow x d^2 \times f_{ck} = 0 \Rightarrow x \delta - 0.181 \times K'$	ars 2 = 678 mm 00 .073 δ^2 - 0.21 = 0.207 > K - No compre	ssion reinforc	ement is required
Rectangular section in flexure Design bending moment; Depth to tension reinforcement; Percentage redistribution; Redistribution ratio; Lever arm;	(Section 6.1) -	-750 Positive mids $M = 809 k$ $d = h - c_{nd}$ $m_r = 0 \%$ $\delta = min(1)$ $K = M / (k)$ $K' = 0.598$ $z = min((d))$	$6 \times 25_{\phi} b$ $\Rightarrow 6 \times 25_{\phi} b$	ars 2 = 678 mm 00 0 .073 δ^2 - 0.21 = 0.207 <i>> K - No compre</i> - 3.53 × K) ^{0.5}], 0.9	<i>ssion reinforc</i> 5 × d) = 630 m	rement is required m
Rectangular section in flexure Design bending moment; Depth to tension reinforcement; Percentage redistribution; Redistribution ratio; Lever arm; Depth of neutral axis;	(Section 6.1) -	-750 Positive mide $M = 809 H$ $d = h - c_{nd}$ $m_r = 0 \%$ $\delta = min(1)$ $K = M / (b)$ $K' = 0.598$ $z = min((d)$ $x = 2.5 \times b)$	$6 \times 25\phi b$ span moment (Nm $bm_b - \phi v - \phi_{bot} / (1 - mr, 1) = 1.00)$ $b \times d^2 \times f_{ck} = 0$ $b \times d^2 \times f_$	ars 2 = 678 mm 00 0.073 δ^2 - 0.21 = 0.207 <i>> K</i> - <i>No compre</i> - 3.53 × K) ^{0.5}], 0.9	<i>ssion reinforc</i> 5 × d) = 630 m	r ement is required m
Rectangular section in flexure Design bending moment; Depth to tension reinforcement; Percentage redistribution; Redistribution ratio; Lever arm; Depth of neutral axis; Area of tension reinforcement re	(Section 6.1) -	-750 Positive mids $M = 809 \text{ k}$ $d = h - c_{nc}$ $m_r = 0 \%$ $\delta = min(1)$ $K = M / (b)$ $K' = 0.598$ $z = min((c)$ $x = 2.5 \times A_{s,req} = M$	$6 \times 25_{\phi} b$ $\Rightarrow $ $span moment kNm pm_{b} - \phi_{v} - \phi_{bot} / (1 - m_{r}, 1) = 1.00 p \times d^{2} \times f_{ck} = 0 3 \times \delta - 0.181 \times K' d/2) \times [1 + (1 - (1 - z)) = 118 m / (1 + (1 - z)) = 29$	ars 2 = 678 mm 20 0.073 $\delta^2 - 0.21 = 0.207$ > <i>K - No compre</i> - 3.53 × K) ^{0.5}], 0.9 nm 152 mm ²	<i>ssion reinforc</i> 5 × d) = 630 m	r ement is required m
Rectangular section in flexure Design bending moment; Depth to tension reinforcement; Percentage redistribution; Redistribution ratio; Lever arm; Depth of neutral axis; Area of tension reinforcement re Tension reinforcement provided;	(Section 6.1) -	-750 Positive mids $M = 809 H$ $d = h - c_{nd}$ $m_r = 0 \%$ $\delta = min(1)$ $K = M / (b)$ $K' = 0.598$ $z = min((c)$ $x = 2.5 \times A_{s,req} = M$ $6 \times 25\phi b$	$6 \times 25_{\phi} b$ $\Rightarrow b$ $\Rightarrow b$ $\Rightarrow b$ $\Rightarrow c = 0$	ars t 2 = 678 mm 00 0.073 $\delta^2 - 0.21 = 0.207$ > <i>K - No compre</i> - 3.53 × K) ^{0.5}], 0.9 1m 152 mm ²	ssion reinforc 5 × d) = 630 m	r ement is required m
Rectangular section in flexure Design bending moment; Depth to tension reinforcement; Percentage redistribution; Redistribution ratio; Lever arm; Depth of neutral axis; Area of tension reinforcement re Tension reinforcement provided; Area of tension reinforcement pr	(Section 6.1) -	Positive mide $M = 809 H$ $d = h - Cnc$ $m_r = 0 \%$ $\delta = min(1)$ $K = M / (b)$ $K' = 0.598$ $z = min((c)$ $x = 2.5 \times A_{s,req} = M$ $6 \times 25\phi b$ $A_{s,prov} = 2$	$6 \times 25_{\phi} b$ span moment (Nm $bm_b - \phi_v - \phi_{bot} / (1 - m_r, 1) = 1.00$ $b \times d^2 \times f_{ck} = 0$ $3 \times \delta - 0.181 \times K'$ $d/2) \times [1 + (1 + (1 - 2)) = 118 m / (1 + (1 - 2)) = 29$ ars 945 mm^2	ars 2 = 678 mm 00 0.073 $\delta^2 - 0.21 = 0.207$ > <i>K - No compre</i> - 3.53 × K) ^{0.5}], 0.9 1m 152 mm ²	<i>ssion reinforc</i> 15 × d) = 630 m	r ement is required m
Rectangular section in flexure Design bending moment; Depth to tension reinforcement; Percentage redistribution; Redistribution ratio; Lever arm; Depth of neutral axis; Area of tension reinforcement re Tension reinforcement provided; Area of tension reinforcement pr Minimum area of reinforcement of	equired; rovided; (exp.9.1N);	-750 Positive mids $M = 809 H$ $d = h - Cno$ $mr = 0 \%$ $\delta = min(1)$ $K = M / (b)$ $K' = 0.598$ $z = min((c)$ $x = 2.5 \times$ $A_{s,req} = M$ $6 \times 25\phi b$ $A_{s,prov} = 2$ $A_{s,min} = m$	$6 \times 25_{\phi} b$ span moment (Nm $pm_b - \phi_v - \phi_{bot} / (1 - m_r, 1) = 1.00$ $p \times d^2 \times f_{ck} = 0$ $a \times \delta - 0.181 \times K'$ $d/2) \times [1 + (1)$ (d - z) = 118 m $/ (f_{yd} \times z) = 29$ ars $945 mm^2$ $ax(0.26 \times f_{ctm})$	ars t 2 = 678 mm 00 0.073 $\delta^2 - 0.21 = 0.207$ > K - No compre - 3.53 × K) ^{0.5}], 0.9 1m 152 mm ² (fyk, 0.0013) × b ×	ssion reinforc 5 × d) = 630 m ∶ d = 799 mm²	r ement is required m
Rectangular section in flexure Design bending moment; Depth to tension reinforcement; Percentage redistribution; Redistribution ratio; Lever arm; Depth of neutral axis; Area of tension reinforcement re Tension reinforcement provided; Area of tension reinforcement provided; Area of tension reinforcement provided; Minimum area of reinforcement Maximum area of reinforcement	equired; ; rovided; (exp.9.1N); (cl.9.2.1.1(3)); ess than 0.5% I	Positive mids M = 809 H $d = h - Cno m_r = 0 \%\delta = min(1)K = M / (b)K' = 0.598z = min((c)x = 2.5 \timesA_{s,req} = M6 \times 25\phi bA_{s,prov} = 2A_{s,min} = mA_{s,max} = 0person that desires that the set of the set $	$6 \times 25_{\phi} \text{ b}$ span moment (Nm $pm_b - \phi_v - \phi_{bot} / (1 - m_r, 1) = 1.00$ $p \times d^2 \times f_{ck} = 0$ $3 \times \delta - 0.181 \times K'$ $d / 2) \times [1 + (1 + (1 - 2)) = 118 \text{ m} / (1 + (2 - 2)) = 118 \text{ m} / $	ars 2 = 678 mm 00 0.073 $\delta^2 - 0.21 = 0.207$ > <i>K - No compre</i> - 3.53 × K) ^{0.5}], 0.9 10 152 mm ² / fyk, 0.0013) × b × 22500 mm ² ent so deemed at	<i>ssion reinforc</i> 15 × d) = 630 m : d = 799 mm² cceptable as 7	rement is required m Femporary Works.
Rectangular section in flexure Design bending moment; Depth to tension reinforcement; Percentage redistribution; Redistribution ratio; Lever arm; Depth of neutral axis; Area of tension reinforcement re Tension reinforcement provided; Area of tension reinforcement provided; Maximum area of reinforcement provided is to Rectangular section in shear (equired; ; rovided; (exp.9.1N); (cl.9.2.1.1(3)); ess than 0.5% I (Section 6.2)	-750 Positive mids $M = 809 H$ $d = h - Cnd$ $m_r = 0 \%$ $\delta = min(1)$ $K = M / (b)$ $K' = 0.598$ $z = min((c)$ $x = 2.5 \times$ $A_{s,req} = M$ $6 \times 25\phi b$ $A_{s,prov} = 2$ $A_{s,min} = m$ $A_{s,max} = 0$ Pess than des	$6 \times 25_{\phi} b$ span moment (Nm om_b - $\phi_V - \phi_{bot} / (1 - m_r, 1) = 1.00$ $0 \times d^2 \times f_{ck} = 0$ $3 \times \delta - 0.181 \times K'$ $d/2) \times [1 + (1 + (1 - 4))]$ $(d - z) = 118 m^2$ ars 945 mm ² $ax(0.26 \times f_{ctm})$ $.04 \times b \times h = 2$ <i>ign requirement</i>	ars 2 = 678 mm 2 = 678 mm 300 300 $\delta^2 - 0.21 = 0.207$ > K - No compre $- 3.53 \times \text{K})^{0.5}$], 0.9 300 52 mm^2 $\sqrt{f_{yk}}$, 0.0013) \times b \times 22500 mm^2 ent so deemed at	<i>ssion reinforc</i> 5 × d) = 630 m ∶ d = 799 mm² <i>cceptable as</i> 7	rement is required m Femporary Works.
Rectangular section in flexure Design bending moment; Depth to tension reinforcement; Percentage redistribution; Redistribution ratio; Lever arm; Depth of neutral axis; Area of tension reinforcement re Tension reinforcement provided; Area of tension reinforcement pr Minimum area of reinforcement of Maximum area of reinforcement Note the Area Provided is In Rectangular section in shear (Design shear force at span s1;	equired; ; ; rovided; (exp.9.1N); (cl.9.2.1.1(3)); ess than 0.5% l a (Section 6.2)	Positive mide $M = 809 H$ $d = h - Cno mr = 0 %$ $\delta = min(1)$ $K = M / (b)$ $K' = 0.598$ $z = min((c)$ $x = 2.5 \times$ $A_{s,req} = M$ $6 \times 25\phi b$ $A_{s,prov} = 2$ $A_{s,min} = m$ $A_{s,max} = 0$ Poses than des $V_{Ed,max} = 3$	$6 \times 25_{\phi} \text{ b}$ span moment (Nm $pm_b - \phi_v - \phi_{bot} / (1 - mr, 1) = 1.00$ $p \times d^2 \times f_{ck} = 0$ $8 \times \delta - 0.181 \times K'$ $d / 2) \times [1 + (1 + (1 - 2)) = 118 \text{ m} / (1 + 2) = 29$ ars 945 mm ² ax(0.26 \times f_{ctm}) = .04 \times b \times h = 2 ign requirement abs(max(Vs1_m	ars 2 = 678 mm 2 = 678 mm 00 0.073 $\delta^2 - 0.21 = 0.207$ > K - No compre $- 3.53 × K)^{0.5}$], 0.9 1000 1000 1000 1000 1000 1000 1000 1000 1000	<i>ssion reinforc</i> 5 × d) = 630 m ∶ d = 799 mm ² <i>cceptable as 1</i> kN	ement is required m
Rectangular section in flexure Design bending moment; Depth to tension reinforcement; Percentage redistribution; Redistribution ratio; Lever arm; Depth of neutral axis; Area of tension reinforcement re Tension reinforcement provided; Area of tension reinforcement pr Minimum area of reinforcement of Maximum area of reinforcement Note the Area Provided is In Rectangular section in shear (Design shear force at span s1; Angle of comp. shear strut for m	equired; (Section 6.1) - (Section 6.1) - (c(section 6.1)); (exp.9.1N); (cl.9.2.1.1(3)); (cl.9.2.1); (cl.9.2.1.1(3)); (cl.9.2.1); (cl.9.	Positive mids M = 809 k d = h - Cnc $m_r = 0 \%$ $\delta = min(1)$ K = M / (b) K' = 0.598 z = min((c) $x = 2.5 \times$ $A_{s,req} = M$ $6 \times 25\phi$ b. $A_{s,prov} = 2$ $A_{s,min} = m$ $A_{s,max} = 0$ pess than des $V_{Ed,max} = 3$ $\theta_{max} = 45$	$6 \times 25_{\phi} b$ span moment (Nm $pm_b - \phi_v - \phi_{bot} / (1 - m_r, 1) = 1.00$ $p \times d^2 \times f_{ck} = 0$ $p \times d^2 \times f_{ck} = 0$	ars 2 = 678 mm 2 = 678 mm 00 0.073 $\delta^2 - 0.21 = 0.207$ > K - No compre $- 3.53 × K)^{0.5}$], 0.9 52 mm^2 152 mm^2 1	<i>ssion reinforc</i> 5 × d) = 630 m ≎ d = 799 mm² <i>cceptable as</i> 7 kN	rement is required m

Design shear force ;

 $V_{Rd,max} = b \times z \times v_1 \times f_{cd} / (cot(\theta_{max}) + tan(\theta_{max})) = 2242 \text{ kN}$ **PASS - Design shear force at support is less than maximum design shear force** $V_{Ed} = \textbf{310 kN}$

BRIDGES POUND	Location				Job No			
C O N S U L T I N G E N G I N E E R S		UCL f	or Mace		N	/1993		
					Sheet No			
	Base	Basement Temporary Works Design				27		
	Ву		Checked		Approved			
	SMH	31/03/16	RLN	Mar 16	SMH	Apr 16		
Design shear stress;		$v_{Ed} = V_{Ed} /$	$(b \times z) = 0.65$	6 N/mm²				
Strength reduction factor (cl.6.2.	3(3));	$v_1 = 0.6 \times [$	1 - f _{ck} / 250 N/	/mm²] = 0.523				
Compression chord coefficient (cl.6.2.3(3));	$\alpha_{cw} = 1.00$						
Angle of concrete compression	strut (cl.6.2.3);							
	$\theta = m$	in(max($0.5 \times A$	$sin[min(2 \times v_{E})]$	$_{\rm d}$ / ($lpha_{\rm cw} imes f_{\rm cd} imes v_1$),1)], 21.8 deg), 4	45deg) = 21.8 deg		
Area of shear reinforcement req	uired (exp.6.13);	$A_{sv,req} = v_{Ec}$	$h \times b / (f_{yd} \times co)$	$t(\theta)) = 452 \text{ mm}^2/\text{m}^2$				
Shear reinforcement provided;		$4 imes 10\phi$ leg	s at 450 c/c					
Area of shear reinforcement pro	vided;	$A_{sv,prov} = 69$	98 mm²/m					
Minimum area of shear reinforce	ement (exp.9.5N)); A _{sv,min} = 0.0	08 N/mm ² × b	× (f _{ck} / 1 N/mm ²) ^{0.5}	/ f _{yk} = 679 mn	n²/m		
		PASS - Area d	of shear reinf	orcement provide	ed exceeds m	inimum required		
Maximum longitudinal spacing (exp.9.6N);	$S_{vl,max} = 0.7$	′5 × d = 508 m	ım				
	PASS - Lo	ngitudinal spa	cing of shear	r reinforcement pi	rovided is les	s than maximum		
Minimum bar spacing								
Minimum bottom bar spacing;		Sbot.min = (b	- 2 × Cnom s - 2	$2 \times \phi_{V} - \phi_{bot}$ / (N _{bot} ·	- 1) = 121 mm			
Minimum allowable bottom bar s	pacing:	Sbar bot.min =	Shar bot min = $max(\phi_{bot}, h_{ang} + 5 mm, 20 mm) + \phi_{bot} = 50 mm$					
Minimum top bar spacing:	,	Stop min = (b)	$S_{120} = \frac{(b - 2)}{(b - 2)} (b - 2) (b - 2$					
Minimum allowable top bar space	ina.	Shar ton min =	: max(@top_h_a	+5 mm 20 mm	$+ \phi_{top} = 41 mm$	1		
			PASS - A	Ctual bar spacing	n exceeds mi	nimum allowable		
			1 400 7	iotaan bar opdonig	,			

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

BRIDGESPC	UND	Location				Job No		
CONSULTING ENC	SINEERS		UCL for Mace				M1993	
						Sheet No		
		Base	Basement Temporary Works Design			2	28	
		Ву		Checked		Approved		
		SMH	31/03/16	RLN	Mar 16	SMH	Apr 16	

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.

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

Appendix A

Capping Beam Revised Analysis

BRIDGES POUND	Location				Job No		
CONSULTING ENGINEERS		UCL for Mace				M1993	
					Sheet No		
	Base	Basement Temporary Works Design			ŀ	A 1	
	Ву		Checked		Approved		
	SMH	31/03/16	RLN	Mar 16	SMH	Apr 16	

ANALYSIS

Geometry



Loading



Tedds calculation version 1.0.10

BRIDGES POUND	Location				Job No	
CONSULTING ENGINEERS	UCL for Mace				M1993	
					Sheet No	
	Basement Temporary Works Design			A	42	
	Ву		Checked		Approved	
	SMH	31/03/16	RLN	Mar 16	SMH	Apr 16

Results

Element end forces

Load combination: LoadCombination1 (Strength)

Element	Length	Nodes	Axial force	Shear force	Moment
	(m)	Start/End	(kN)	(kN)	(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

BRIDGES POUND	Location				Job No	
CONSULTING ENGINEERS	UCL for Mace				M1993	
					Sheet No	
	Base	Basement Temporary Works Design			ŀ	43
	Ву		Checked		Approved	
	SMH	31/03/16	RLN	Mar 16	SMH	Apr 16

Element	Length	Nodes	Axial force	Shear force	Moment
	(m)	Start/End	(kN)	(kN)	(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	Nodes	Axial force	Shear force	Moment
	(m)	Start/End	(kN)	(kN)	(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

BRIDGES POUND	Location				Job No	
CONSULTING ENGINEERS	UCL for Mace				M1993	
					Sheet No	
	Base	Basement Temporary Works Design			A	A 4
	Ву		Checked		Approved	
	SMH	31/03/16	RLN	Mar 16	SMH	Apr 16

Element	Length	Nodes	Axial force	Shear force	Moment	
	(m)	Start/End	(kN)	(kN)	(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	Location				Job No	
CONSULTING ENGINEERS	UCL for Mace				M1993	
					Sheet No	
	Basement Temporary Works Design		A 5			
	Ву		Checked		Approved	
	SMH	31/03/16	RLN	Mar 16	SMH	Apr 16

Forces





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



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BRIDGES POUND	Location				Job No	
CONSULTING ENGINEERS		UCL fo	or Mace		М	1993
					Sheet No	
	Base	ement Tempo	orary Works D	Design	A	Appendix B
	Ву		Checked		Approved	
	SMH	31/03/16	RLN	Mar 16	SMH	Apr 16

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.
- 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.



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MGF 406 UC Series -See Section 3 **MGF 600 Series** MGF Davitsafe -See Section 6 MGF Laddersafe -See Section 6 **MGF Trench Sheets -MGF Pole Ladder**

See Section 5



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.



Handling Points WLL = 12.0t

Tel. 01942 402 704



Strut Flange Connection Detail 600 Series Struts and Extensions are connected to each other via a flange plate (Ø850x30mm) 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).



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.



Face to Face Dimension

Safe Working Load for MGF 600 Series (kN)

MGF TECHNICAL FILI MGF 600 SERIES STRU



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

4.4.4 Issue 2



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



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

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



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	-50°C to +50°C	-50°C to +50°C
Approx. Pre-Load	300kN	500kN
Locating Pins	Ф 30	Ф 30



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	Туре А	Туре В
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 Νο. Φ32 holes	16 Νο. Φ32 holes
Pin Detail	Ф 90 (817М40/EN24T)	Ф 90 (817М40/EN24T)

2500kN Type B Swivel Assembly



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



MGF TECHNICAL FILE MGF 600 SERIES STRUT



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).

Face to Face	:	2500kN Hydraulic	
Dimension (m)	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.



Typical Basement Wall Application





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

Appendix C

Trench Sheeting Calculations

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CADS Piled Wall Suite Version 5.31	Project	M1993
Design of embedded retaining walls and cofferdams	File Name	910 - sheet check.pws
UCL for Mace	Engineer	SMH
Temp Works to SE Corner	Date	11/04/2016

Pile geometry

Pile top Level23.15mPile Length5mPile toe level18.15mActive ground slope0Degrees (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	Ка Кр	Кас Крс
23.15	Hardcore	17.00	18.00	48000	0			40 40	.50 .50	.19 8.38	
22.75	Clay Fill	19.00	20.00	13867	0			26 26	.50 .50	.35 3.40	
21.80	GRAVEL	21.00	23.00	46800	0			38 38	.50 .50	.21 7.23	
21.25	Sandy CLAY	20.50	20.50	48000	0			29 29	.50 .50	.31 4.04	
17.85	Sandy CLAY	19.50	19.50	30000	0			28 28	.50 .50	.32 3.81	

Construction sequence

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

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Code of practice

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Code of practice or reference document Application of pressures for stability FOS on moments (stability check) ULS factor on Tan(Phi) values ULS fFactor on drained cohesion values ULS factor on undrained cohesion values ULS factor on active soil pressures ULS factor on passive soil pressures ULS factor on passive soil pressures ULS factor on passive water pressures ULS factor on passive water pressures ULS factor on loads applied to the soil ULS factor on loads applied to the wall FOS on embedment (stability check) Correction factor on cantilever embedment	Eurocode 7 ULS Design Approach 1 Combination 1 Not applicable for FOS=1 on moments 1.00 1.00 1.00 1.35 1.00 1.35 1.35 1.11 1.50 1.00 1.20
Wall analysis detail options	
Nominal Phi for load distribution Depth of water filled tension cracks Density of water Minimum equivalent fluid density Depth of passive softened soil Continuity model for wall analysis	30.0 Degrees .0 m 10.0 kN/m3 5.0 kN/m3 1.0 m Pins at second and lower props
Deflection parameters	
Wall moment of inertia Wall Youngs modulus	801 cm4/m 21000000 kN/m2
Properties for prop at 22.15 Prop/Tie cross sectional area Prop/Tie Youngs modulus Prop/Tie length Prop/Tie spacing Waling moment of inertia Waling Youngs modulus Prop/Tie preload	200 cm2 each 21000000 kN/m2 10.0 m 6.0 m Waling deflection not included Waling deflection not included 0 kN

0.0 mm

Prop/Tie spacing Waling moment of inertia Waling Youngs modulus Prop/Tie preload Initial lack of fit

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Stage ref.4Stage typePassive side excavation



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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 13.6	2.8	.0	.0	.0	.0	2.8	0	1	-2.7		.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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Deflection diagram (mm)

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Graphical results from analysis of stage ref 4 continued



Bending Moment Diagram (kNm/m)



Shear Force Diagram (kN/m)

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Table of envelope for wall forces

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Calc	Bending	Bending	Shear	Shear	Prop
Level	Minimum	Maximum	Minimum	Maximum	Force
m	kNm/m	kNm/m	kN/m	kN/m	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	

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Design of embedded retaining walls and cofferdams	File Name	e10 - sheet check.pws
UCL for Mace	Engineer	SMH
Temp Works to SE Corner	Date	11/04/2016

Structural design of wall

Wall section properties

Sheet pile section ref Mabey M12

Wall material properties

Yield stress of steel Bending Stress Ratio Allowable bending stress Allowable shear stress	355 1.05 338 202	N/mm2 N/mm2 N/mm2	
Wall structural design checks			
Check description	or Limit	or Actual	Units
Max. bending moment Design stress check	18	61	kNm/m
Min. section modulus Design stress check	54	182	cm3/m
Maximum shear force	33	387	kN/m

Design stress check

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UCL for Mace	Engineer	SMH
Temp Works to ACBE Building	Date	11/04/2016

Pile geometry

Pile top Level21.15mPile Length2mPile toe level19.15mActive ground slope0Degrees (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	Кас Крс
23.15	Hardcore	17.00	18.00	48000	0			40 40	.50 .50	.19 8.38	
22.75	Clay Fill	19.00	20.00	13867	0			26 26	.50 .50	.35 3.40	
21.80	GRAVEL	21.00	23.00	46800	0			38 38	.50 .50	.21 7.23	
21.25	Sandy CLAY	20.50	20.50	48000	0			29 29	.50 .50	.31 4.04	
17.85	Sandy CLAY	19.50	19.50	30000	0			28 28	.50 .50	.32 3.81	

Construction sequence

Stage Ref	tage Ref Stage Type	Level or Angle m/deg. k	Width Length m m	ength m		
1 2 A	Active surcharge Passive side excavation	19.90 20.23	90.0	.0		

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30.0 Degrees

.0 m

1.0 m

10.0 kN/m3

5.0 kN/m3

Pins at second and lower props

Code of practice

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

Wall analysis detail options

Nominal Phi for load distribution Depth of water filled tension cracks Density of water Minimum equivalent fluid density Depth of passive softened soil Continuity model for wall analysis

Deflection parameters

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

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Stage ref.2Stage typePassive side excavation



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UCL for Mace	Engineer	SMH
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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

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Deflection diagram (mm)

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Graphical results from analysis of stage ref 2 continued



Bending Moment Diagram (kNm/m)



Shear Force Diagram (kN/m)

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Design of embedded retaining walls and cofferdams	File Name	e m1993 - abce block.pws
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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 Design stress check	1	12	kNm/m
Min. section modulus Design stress check	5	47	cm3/m
Maximum shear force Design stress check	11	109	kN/m

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CADS Piled Wall Suite Version 5.31	Project	M1993
Design of embedded retaining walls and cofferdams	File Name	e m1993 - tunnel.pws
UCL for Mace	Engineer	SMH
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Pile geometry

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Pile top Level20.75mPile Length2mPile toe level18.75mActive ground slope0Degrees (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	Ка Кр	Кас Крс
23.15	Hardcore	17.00	18.00	48000	0			40 40	.50 .50	.19 8.38	
22.75	Clay Fill	19.00	20.00	13867	0			26 26	.50 .50	.35 3.40	
21.80	GRAVEL	21.00	23.00	46800	0			38 38	.50 .50	.21 7.23	
21.25	Sandy CLAY	20.50	20.50	48000	0			29 29	.50 .50	.31 4.04	
17.85	Sandy CLAY	19.50	19.50	30000	0			28 28	.50 .50	.32 3.81	

Construction sequence

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

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CADS Piled Wall Suite Version 5.31	Project	M1993
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UCL for Mace	Engineer	SMH
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Code of practice

Code of practice or reference document Application of pressures for stability FOS on moments (stability check) ULS factor on Tan(Phi) values ULS fFactor on drained cohesion values ULS factor on undrained cohesion values ULS factor on active soil pressures ULS factor on passive soil pressures ULS factor on passive soil pressures ULS factor on passive water pressures ULS factor on passive water pressures ULS factor on loads applied to the soil ULS factor on loads applied to the wall FOS on embedment (stability check) Correction factor on cantilever embedment	Eurocode 7 ULS Design Approach 1 Combination 1 Not applicable for FOS=1 on moments 1.00 1.00 1.35 1.00 1.35 1.35 1.11 1.50 1.00 t 1.20
Wall analysis detail options	
Nominal Phi for load distribution Depth of water filled tension cracks Density of water Minimum equivalent fluid density Depth of passive softened soil Continuity model for wall analysis	30.0 Degrees .0 m 10.0 kN/m3 5.0 kN/m3 1.0 m Pins at second and lower props
Deflection parameters	
Wall moment of inertia Wall Youngs modulus	89 cm4/m 21000000 kN/m2
Properties for prop at 20.75 Prop/Tie cross sectional area Prop/Tie Youngs modulus Prop/Tie length	200 cm2 each 21000000 kN/m2 10.0 m

Prop/Tie Youngs modulus Prop/Tie length Prop/Tie spacing Waling moment of inertia Waling Youngs modulus Prop/Tie preload Initial lack of fit 200 cm2 each 200000 kN/m2 10.0 m 6.0 m Waling deflection not included Waling deflection not included 0 kN

0.0 mm

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Stage ref.4Stage typePassive side excavation



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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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CADS Piled Wall Suite Version 5.31	Project	M1993
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Deflection diagram (mm)
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Graphical results from analysis of stage ref 4 continued



Bending Moment Diagram (kNm/m)



Shear Force Diagram (kN/m)

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Table of envelope for wall forces

Calc	Bending	Bending	Shear	Shear	Prop
Level	Minimum	Maximum	Minimum	Maximum	Force
m	kNm/m	kNm/m	kN/m	kN/m	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	

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Design of embedded retaining walls and cofferdams	File Name	e m1993 - tunnel.pws
UCL for Mace	Engineer	SMH
Temp Works to SE Corner	Date	11/04/2016

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 Design stress check	1	12	kNm/m
Min. section modulus Design stress check	6	47	cm3/m
Maximum shear force Design stress check	14	109	kN/m