Gamble, King and Noone Limited	Project		Job Ref.			
Consulting Structural Engineers & Building Surveyors	New Dwelling at Gerrards Hollow, Gee Cross. SK14 5DT				C23.07.32	
5 Queen Street, Oldham. OL1 1RD.	Section				Sheet no./rev.	
Tel – 0161 652 1183		Structura	l Calculations		1	
www.gknltd.co.uk	Calc. by	Date	Chk'd by	Date	App'd by	Date
C C	S.King	Nov. 2023				

Ref:

#### Calculations

Output

## <u>NOTES</u>

The structural drawing and calculations will need to be approved by a suitable checking Authority such as Building Control. This approval must be sought by the Client/Contractor prior to any works taking place or any materials being ordered.

## **DESIGN INFORMATION / PHILOSOPHY**

It is proposed to construct a new dwelling at a site off Apethorn Lane, Gerrards Hollow, Gee-Cross, Hyde.

The proposed development will comprise of a 3 storey dwelling. The lower ground floor will include an area for parking which is independent to the new dwelling.

The lower ground floor itself will include a double garage, W.C , Utility, store and a staircase to access the upper storey.

The ground floor consists of a living room, dining/kitchen and a hall which has the staircases which facilitate the 1<sup>st</sup> floor and the lower ground floor. The ground floor has a terrace which is directly above the parking area.

The first floor includes 3 bedrooms , 1 with en-suite, a bathroom and storage area.

The dwelling will be built into a steep embankment and therefore will require an appropriate retaining structure in order for the dwelling to be constructed in a safe manner. This retaining structure will be installed independently from the new dwelling.

We are not responsible for the quality of workmanship. This exercise is to justify the retaining structure only.

A slope stability assessment has been carried out by Ashton Bennett Consultancy (Engineering Geologists & Environmental Scientists). The report was conducted to clarify the ground stability for the proposed development.

These calculations cover a design check of the permanent contiguous bored piled retaining wall structure together with the capacity calculations for the support piles.

Gamble, King and Noone Limited	Project				Job Ref.			
Consulting Structural Engineers & Building Surveyors	New Dwel	ling at Gerrards	Hollow, Gee Cr	oss. SK14 5DT	C23.07.32			
5 Queen Street, Oldham. OL1 1RD.	Section	<b>e</b>			Sheet no./rev.			
Tel – 0161 652 1183		Structura	I Calculations		2			
www.gknltd.co.uk	Calc. by S.King	Date Nov. 2023	Chk'd by	Date	App'd by	Date		
Ref:	С	alculations			Outp	Output		
Contigous Bored Piled Reta	aining wa	II check (W	<u>all A)</u>					
Details of the proposed retaining struct no. 2221, drawing no. 07.	ure have bee	en taken from the	e drawings supp	blied by Northern	Design, proje	ect reference		
One section of wall has been identified	for this analy	/sis, labelled Se	ction A-A on C2	3.07.32 Sketch0 <sup>7</sup>	1			
The section has been chosen to repres retaining wall.	ent the gene	ral soil and struc	ctural criteria wh	iich is applicable	to the require	ements of the		
SECTION A-A :								
These are installed in close proxim with T16 bars and connected into a section of wall has been installed a footpath.	a continuous at this level to	sting retaining w ring beam (capp provide enhance	all to the footpa bing beam) whic bed retaining me	th of Apethorn La th is reinforced w easures to the exi	ane. The pile: ith T16 bars : isting retainir	a are reinforce also. This ng wall to the		
There has been a line of 600mm diame deep and are socketed (1.5m appr	eter reinforce ox.) into bed	d concrete auge rock which retur	r piles installed n the lower sec	lower down the s tion of ground.	lope, these a	are also 7.5m		
A 500mm thick R.C slab has been insta higher line of CFA piles. The slab i line of Stone faced lego blocks. Th	alled which a s reinforced e lego blocks	cts as a capping with 2 layers of <i>i</i> s have been con	beam to the lov A393 mesh top nected back int	wer line of CFA p and bottom. The o the piles via ret	iles and spar slab provides par hoops.	ns across to th s support for a		
The stone faced lego blocks are installe	ed in an inter	connected sequ	ence and are (6	rows high) which	n is 3.6m reta	ained height.		
A 10kN/m <sup>2</sup> surcharge will be considered	d due to vehi	cular traffic (Ape	ethorn Lane)					
Ground conditions:								
The borehole sample test results show strength at 6.0m. (Details can be found	that the clay on page 8 ( <sup>-</sup>	is on medium to Fable 2 of the Sl	o high strength f ope Stability As	rom 1.0m – 5.0m sessment).	depth and v	ery high		
Groundwater was not encountered duri	ing the drilling	g of the borehole	e except as sma	III seepages.				
The wall will have a proprietary drain in	stalled at the	back of the wal	l to relieve any	hydrostatic press	ure.			



Gamble, King and Noone Limited	Project				Job Ref.		
Consulting Structural Engineers & Building Surveyors	New Dwel	ling at Gerrards	Hollow, Gee Cro	oss. SK14 5DT	C23.07.32		
5 Queen Street, Oldham. OL1 1RD.	Section		Sheet no./rev.	Sheet no./rev.			
Tel – 0161 652 1183		Structura	I Calculations			4	
www.gknltd.co.uk	Calc. by	Date	Chk'd by	Date	App'd by	Date	
	S.King	Nov. 2023					
Ref:	C	alculations			Outp	ut	
				Imposed	d × 1.60		
		Span 1		$Dead \times$	1.40		
				Imposed × 1.60			
	Support B		Dead ×	1.40			
				Imposed	d × 1.60		
	Span 2		Dead ×	1.40			
				Imposed	d × 1.60		
		Support C		Dead ×	1.40		
			Imposed	d × 1.60			
	Span 3		Dead ×	1 40			
			Imposed	d × 1.60			
	Support D	Support D Dead -			1.40		
				Imposed	1 × 1.60		
Analysis results		M. – O	kNm:	M	0 kNm		
Maximum moment support A,	nm.	$M_{\star} = 0$	kNm:	IVIA_red —	• • kNm:		
Maximum moment support B	,	$M_{\rm D} = -0$	$M_{B max} = -0 \text{ kNm} \cdot M_{B red} =$				
Maximum moment support D,	nm.	$M_{n2} = 0$	$M_{\text{LO}} = 0 \text{ KNm}; \qquad M_{\text{LO}} = 0$				
Maximum moment support C:	,	$M_{C,max} = 0$	$M_{s2}$ max - 0 KNIII, $M_{s2}$ red -				
Maximum moment span 3 at 270 r	nm:	$M_{s3 max} = 0$	kNm:	Mc_red =	= <b>0</b> kNm:		
Maximum moment support D:	,	$M_{D, max} = 0$	kNm:	MD red =	= <b>0</b> kNm:		
Maximum shear support A;		V <sub>A max</sub> = <b>1</b>	kN;	V <sub>A red</sub> =	1 kN		
Maximum shear support A span 1	at 384 mm;	V <sub>A s1 max</sub> =	<b>-2</b> kN;	VA s1 red	= <b>-2</b> kN		
Maximum shear support B;		V <sub>B_max</sub> = <b>-2</b>	kN;	V <sub>B_red</sub> =	<b>-2</b> kN		
Maximum shear support B span 1	at 66 mm;	V <sub>B_s1_max</sub> =	<b>1</b> kN;	V <sub>B_s1_red</sub>	= <b>1</b> kN		
Maximum shear support B span 2	at 384 mm;	$V_{B_{s2}max} =$	<b>-1</b> kN;	V <sub>B_s2_red</sub>	= <b>-1</b> kN		
Maximum shear support C;		V <sub>C_max</sub> = <b>2</b>	kN;	$V_{C_{red}} =$	<b>2</b> kN		
Maximum shear support C span 2	at 66 mm;	V <sub>C_s2_max</sub> =	<b>1</b> kN;	Vc_s2_red	= <b>1</b> kN		
Maximum shear support C span 3	at 384 mm;	V <sub>C_s3_max</sub> =	<b>-1</b> kN;	V <sub>C_s3_red</sub>	= <b>-1</b> kN		
Maximum shear support D;		V <sub>D_max</sub> = -1	kN;	$V_{D_{red}} =$	<b>-1</b> kN		
Maximum shear support D span 3	at 66 mm;	V <sub>D_s3_max</sub> =	<b>2</b> kN;	VD_s3_red	= <b>2</b> kN		
Maximum reaction at support A;		R <sub>A</sub> = <b>1</b> kN					
Maximum reaction at support B;		R <sub>B</sub> = <b>4</b> kN					
Maximum reaction at support C;		$R_c = 4 kN$	$R_c = 4 kN$				
Maximum reaction at support D;		R₀ <b>= 1</b> kN					
Rectangular section details							
Section width; b	= <b>500</b> mm;		Section depth;		h = <b>450</b> mm		



Gamble, King and Noone Limited	Project				Job Ref.		
Consulting Structural Engineers & Building Surveyors	New Dwel	ling at Gerrards	Hollow, Gee Cro	oss. SK14 5DT	C23	.07.32	
5 Queen Street, Oldham. OL1 1RD.	Section				Sheet no./rev.		
Tel – 0161 652 1183		Structura	l Calculations			6	
www.gknltd.co.uk	Calc. by	Date	Chk'd by	Date	App'd by	Date	
U U	S.King	Nov. 2023					
Ref:	C	alculations			Outpu	t	
<u>Mid span 1</u>							
450		500	3 x 16φ bars 2 x 8φ shear 3 x 16φ bars	legs at 200 c/c			
<b>Design moment resistance of r</b> Design bending moment; M K	ectangular se I = 0 kNm; = 0.000;	-500 →	ا Depth to tensio <b>۲ - ۸ - ۸</b>	n reinf.; <b>Io compressior</b>	d = <b>384</b> mm K' = <b>0.156</b> h reinforceme	nt is required	
Lever arm; z	= <b>365</b> mm;		Depth of neutra	l axis;	x = <b>43</b> mm		
Area of tension reinf req'd; A	<sub>s,req</sub> <b>= 1</b> mm <sup>2</sup> ;		Tension reinf p	rovided;	$3 \times 16\phi$ bars		
Area of tension reinf prov; A	<sub>s,prov</sub> = 603 mr	n²;	Minimum area	of reinf;	A <sub>s,min</sub> = <b>293</b> m	1m <sup>2</sup>	
Maximum area of reinf; A	<sub>s,max</sub> = <b>9000</b> m	ım²					
P	ASS - Area o	f reinforcemen	t provided is gr	reater than area	of reinforcen	nent required	
Rectangular section in shear							
Shear reinforcement provided; 2	× 8     k legs at 2	00 c/c					
Area of shear reinf provided; A	<sub>sv,prov</sub> = <b>503</b> m P/	m²/m; ASS - Area of s	Minimum area <i>hear reinforcen</i>	of shear reinf; <b>nent provided e</b>	A <sub>sv,min</sub> = 460 r exceeds minin	nm²/m n <b>um required</b>	
Max longitudinal spacing; s	ul,max = 288 mn PASS - Long	n itudinal spacin	g of shear reinf	orcement prov	ided is less th	an maximum	
Spacing of reinforcement (cl 3.	12.11)	-		-			
Actual dist between bars; s	= <b>168</b> mm;		Min dist betwee	en bars; <b>S - Satisfies the</b>	s <sub>min</sub> = <b>25</b> mm s minimum sp	acing criteria	
Design service stress: fs	= <b>0.4</b> N/mm <sup>2</sup> :		Max distance b	etween bars:	s <sub>max</sub> = 300 mr	n	
5	<b>,</b>		PASS	S - Satisfies the	maximum sp	acing criteria	
Span to depth ratio (cl. 3.4.6)						-	
Span to depth ratio (T 3 9).	oan to denth	basic = 26.0 <sup>.</sup>	Service stress i	n tension rein	fs = <b>0.4</b> N/mm	2	
Modification for tension reinf: f.	ens = 2.000:	,	Modification for	comp reinf:	$f_{\text{somp}} = 1.095$		
Modification for span > 10m <sup>-</sup>	$n_{\rm ma} = 1.000^{\circ}$		Allowable span	to depth ratio	span to dent	hallow = 56.9	
Actual span to depth ratio	pan to denth	actual = <b>1</b> .2			span_to_dop		
· · · · · · · · · · · · · · · · · · ·		PAS	S - Actual span	to depth ratio i	s within the a	llowable limit	

Gamble, King and Noone Limited	Project				Job Ref.		
Consulting Structural Engineers & Building Surveyor	New Dwel	ling at Gerrards	Hollow, Gee Cro	oss. SK14 5DT	C23.	07.32	
5 Oueen Street, Oldham, OL1 1RD.	Section				Sheet no./rev.		
Tel = 0161 652 1183		Structura	I Calculations			7	
www.sknltd.co.uk	Calc. by	Date	Chk'd by	Date	App'd by	Date	
www.Brinterco.un	S.King	Nov. 2023					
Ref:	C	alculations	I	I	Output		
Support B					· ·		
<u></u>			1				
	r	• •	3 x 16∳ bars				
				lass at 200 ala			
$\frac{1}{4}$							
	le	<u> </u>	3 x 16∳ bars				
<b>•</b>							
	L	500	I				
	•	-500	1				
Design moment resistance of	rectangular se	ection (cl. 3 4 4)					
Design bending moment:	M = 0  kNm		Depth to tensio	n reinf ·	d = <b>384</b> mm		
Boolgh Bollang Monont,	$\leq = 0.000^{\circ}$		Doptil to toholo		K' = 0.156		
	<b>C C C C C C C C C C</b>		reinforcemer	nt is reauired			
Lever arm:	z = <b>365</b> mm:		Depth of neutra	ll axis:	x = <b>43</b> mm		
Area of tension reinf reg'd:	$s_{reg} = 1 \text{ mm}^2$ :		Tension reinf p	rovided:	$3 \times 16\phi$ bars		
Area of tension reinf prov:	A <sub>s.prov</sub> = 603 mr	n²;	Minimum area	of reinf;	A <sub>s,min</sub> = <b>293</b> mm <sup>2</sup>		
Maximum area of reinf;	A <sub>s.max</sub> = <b>9000</b> m	im²		,			
	PASS - Area o	f reinforcemen	t provided is gi	eater than area	of reinforcen	nent required	
Postangular soction in shear			, ,				
Shear - span 1 at 66 mm	l = 1 k N		Shear stress:		y = 0.005  N/m	m <sup>2</sup>	
Allowable design shear stress:	$m_{max} = 4.866 \text{ N/}$	mm <sup>2</sup>	oncar stress,	ss, v − 0.003 N/IIIII-			
		PAS	SS - Desian she	ar stress is les	s than maxim	um allowable	
Value of v from Table 3.7: v <	0.5vc						
Design shear resistance reg'd:	/s = <b>0.400</b> N/mi	m <sup>2</sup> :	Area of shear r	einf rea'd:	A <sub>sv reg</sub> = <b>460</b> mm <sup>2</sup> /m		
Shear reinforcement provided:	$2 \times 8\phi$ leas at 2	00 c/c:	Area of shear r	einf. prov:	$A_{sv,req} = 503 \text{ mm}^2/\text{m}$		
1 ,	P	ASS - Area of s	hear reinforcen	nent provided e	xceeds minin	num reauired	
Max longitudinal spacing;	s <sub>vl,max</sub> = <b>288</b> mn	n					
	PASS - Long	itudinal spacin	g of shear reinf	orcement provi	ded is less th	an maximum	
Shear - span 2 at 384 mm;	√ = <b>1</b> kN;		Shear stress;		v = <b>0.007</b> N/m	m <sup>2</sup>	
Allowable design shear stress;	/ <sub>max</sub> = <b>4.866</b> N/	mm²					
		PAS	SS - Design she	ar stress is les	s than maxim	um allowable	
Value of v from Table 3.7; v <	0.5vc						
Design shear resistance req'd;	/s = <b>0.400</b> N/mi	m²;	Area of shear r	einf req'd;	A <sub>sv,req</sub> = <b>460</b> m	nm²/m	
Shear reinforcement provided;	$2 \times 8\phi$ legs at 2	00 c/c;	Area of shear r	einf. prov;	A <sub>sv,prov</sub> = <b>503</b> r	mm²/m	
	P	ASS - Area of s	hear reinforcen	nent provided e	xceeds minin	num required	
Max longitudinal spacing;	s <sub>vl,max</sub> = <b>288</b> mn	n					
	PASS - Long	itudinal spacing	g of shear reinf	orcement provi	ded is less th	an maximum	
Spacing of reinforcement (cl 3	.12.11)						
Actual dist between bars;	s = <b>168</b> mm;		Min dist betwee	en bars;	s <sub>min</sub> = <b>25</b> mm		
			PAS	S - Satisfies the	minimum spa	acing criteria	
Design service stress;	s = <b>0.5</b> N/mm <sup>2</sup> ;		Max distance b	etween bars;	s <sub>max</sub> = <b>300</b> mn	า	
			PASS	S - Satisfies the	maximum spa	acing criteria	

Gamble, King and Noone Limited	Project			Job Ref.		
Consulting Structural Engineers & Building Surveyors	New Dwelling at Gerrards	s Hollow, Gee Cro	ss. SK14 5DT	C23	3.07.32	
5 Queen Street, Oldham. OL1 1RD.	Section			Sheet no./rev.		
Tel – 0161 652 1183	Structur	al Calculations			8	
www.gknltd.co.uk	Calc. by Date	Chk'd by	Date	App'd by	Date	
	S.King Nov. 2023					
Ref:	Calculations			Outp	ut	
Mid span 2						
450		3 x 16∳ bars 2 x 8∳ shear I 3 x 16∳ bars	egs at 200 c/c			
<b>Design moment resistance of re</b> Design bending moment; M K	<pre></pre>	<ul> <li>▶</li> <li>Depth to tensior</li> <li>K' &gt; K - N</li> </ul>	n reinf.; o compressior	d = 384 mm K' = 0.156 n reinforceme	ent is required	
Lever arm; z	= <b>365</b> mm;	Depth of neutral	ıl axis; x = <b>43</b> mm			
Area of tension reinf req'd; A	<sub>s,req</sub> = <b>0</b> mm²;	Tension reinf pr	ovided;	$3 \times 16\phi$ bars		
Area of tension reinf prov; A	s,prov = <b>603</b> mm <sup>2</sup> ;	Minimum area o	of reinf;	A <sub>s,min</sub> = <b>293</b> mm <sup>2</sup>		
Maximum area of reinf; A	<sub>s,max</sub> = <b>9000</b> mm <sup>2</sup>					
P	ASS - Area of reinforceme	nt provided is gro	eater than area	a of reinforce	ment required	
Rectangular section in shear						
Shear reinforcement provided; 2	$\times$ 8 $\phi$ legs at 200 c/c					
Area of shear reinf provided; A	<sub>sv,prov</sub> = 503 mm²/m; PASS - Area of s	Minimum area c shear reinforcem	of shear reinf; Nent provided e	A <sub>sv,min</sub> = 460 exceeds mini	mm²/m mum required	
Max longitudinal spacing; s <sub>v</sub>	<sub>/l,max</sub> = <b>288</b> mm					
	PASS - Longitudinal spaci	ng of shear reinfo	orcement prov	ided is less t	han maximum	
Spacing of reinforcement (cl 3.1	12.11)					
Actual dist between bars; s	= <b>168</b> mm;	Min dist betwee <b>PASS</b>	n bars; <b>3 - Satisfies the</b>	s <sub>min</sub> = 25 mm e <i>minimum s</i>	າ <b>oacing criteria</b>	
Design service stress; fs	= <b>0.1</b> N/mm <sup>2</sup> ;	Max distance be	etween bars;	s <sub>max</sub> = <b>300</b> m	ım	
		PASS	- Satisfies the	maximum s	pacing criteria	
Span to depth ratio (cl. 3.4.6)						
Span to depth ratio (T.3.9); s	pan_to_depth <sub>basic</sub> = <b>26.0</b> ;	Service stress in	n tension rein;	f <sub>s</sub> = <b>0.1</b> N/mr	m²	
Modification for tension reinf; $f_{te}$	ens = <b>2.000</b> ;	Modification for	comp reinf;	f <sub>comp</sub> = <b>1.095</b>		
Modification for span > 10m; f <sub>lo</sub>	ong = <b>1.000</b> ;	Allowable span	to depth ratio;	span_to_depth <sub>allow</sub> = <b>56.9</b>		
Actual span to depth ratio; sp	pan_to_depth <sub>actual</sub> = <b>1.2</b>					
	PAS	S - Actual span	to depth ratio i	is within the	allowable limit	

Gamble, King and Noone Limited	Project				Job Ref.		
Consulting Structural Engineers & Building Surveyors	New Dwel	ling at Gerrards	Hollow, Gee Cro	oss. SK14 5DT	C23.	07.32	
5 Queen Street, Oldham. OL1 1RD.	Section				Sheet no./rev.		
Tel – 0161 652 1183		Structura	I Calculations			9	
www.gknltd.co.uk	Calc. by	Date	Chk'd by	Date	App'd by	Date	
	S.King	Nov. 2023					
Ref:	C	alculations			Outpu	t	
Support C							
•			]				
	ſ	• •	3 x 16 <sub>∲</sub> bars				
450-			$2 \times 8_{\varphi}$ shear	legs at 200 c/c			
			3 x 164 bars				
•		y	5 x τοφ bars				
	-	-500	•				
Design moment resistance of r	octangular co	ction (cl. 3.4.4)					
Design hending moment:	l = <b>0</b> kNm <sup>.</sup>	cuon (ci. 3.4.4)	Denth to tensio	n reinf ·	d = <b>384</b> mm		
Design bending moment, K	i = 0.000		Deptil to tellsio	n renn.,	K' = 0.156		
	0.000,		reinforceme	nt is reauired			
Lever arm; z	= <b>365</b> mm;		Depth of neutra	ıl axis;	x = <b>43</b> mm		
Area of tension reinf req'd; A	$s_{s,req} = 1 \text{ mm}^2;$		Tension reinf p	rovided;	$3 \times 16\phi$ bars		
Area of tension reinf prov; A	s,prov = 603 mr	n²;	Minimum area	of reinf;	A <sub>s,min</sub> = <b>293</b> mm <sup>2</sup>		
Maximum area of reinf; A	<sub>s,max</sub> = <b>9000</b> m	1m <sup>2</sup>					
F	PASS - Area o	f reinforcemen	t provided is gr	reater than area	of reinforcen	nent required	
Rectangular section in shear							
Shear - span 2 at 66 mm; V	′ = <b>1</b> kN;		Shear stress;		v <b>= 0.007</b> N/mm <sup>2</sup>		
Allowable design shear stress; v	max = <b>4.866 N</b> /	mm²					
		PAS	SS - Design she	ar stress is less	s than maxim	um allowable	
Value of v from Table 3.7; $v < 0$	).5vc	0				0.	
Design shear resistance req'd; v	s = 0.400 N/mr	m²;	Area of shear r	einf req'd;	A <sub>sv,req</sub> = <b>460</b> m	1m²/m	
Shear reinforcement provided; 2	× 8¢ legs at 2	00 c/c;	Area of shear r	eint. prov;	$A_{sv,prov} = 503$ r	mm²/m 	
Max longitudinal spacing:	- 288 mn	455 - Area of s	near reinforcen	nent provided e	xceeas minin	ium requirea	
Max longitudinal spacing, 3	PASS - 1 ong	'' itudinal spacin	a of shear reinf	orcement provi	ded is less th	an maximum	
Shear - span 3 at 384 mm: V	′ = 1 kN:	icaamar opaom	Shear stress:		v = 0.005 N/m	lm²	
Allowable design shear stress; v	<sub>max</sub> = <b>4.866 N</b> /i	mm²	,				
-		PAS	S - Design she	ar stress is less	s than maxim	um allowable	
Value of v from Table 3.7; $v < 0$	).5vc						
Design shear resistance req'd; v	s = <b>0.400</b> N/mr	m²;	Area of shear r	einf req'd;	A <sub>sv,req</sub> = <b>460</b> m	nm²/m	
Shear reinforcement provided; 2	$\times$ 8 $\phi$ legs at 2	00 c/c;	Area of shear r	einf. prov;	A <sub>sv,prov</sub> = 503 r	mm²/m	
	P	ASS - Area of s	hear reinforcen	nent provided e	xceeds minin	num required	
Max longitudinal spacing; s	<sub>vl,max</sub> = <b>288</b> mn	n 				_	
	PASS - Long	itudinal spacing	g of shear reinf	orcement provi	ded is less th	an maximum	
Spacing of reinforcement (cl 3.	12.11)						
Actual dist between bars; s	= <b>168</b> mm;		Min dist betwee	en bars;	s <sub>min</sub> = <b>25</b> mm		
<b>5</b> · · · · · ·	<b></b>		PAS	S - Satisfies the	minimum sp	acing criteria	
Design service stress; $f_s$	= <b>0.5</b> N/mm <sup>2</sup> ;		Max distance b	etween bars;	s <sub>max</sub> = 300 mn	n naine aritari	
			PASS	- Satisties the	maximum sp	acing criteria	

Gamble, King and Noone Limited	Project				Job Ref.		
Consulting Structural Engineers & Building Surveyors	New Dwel	ling at Gerrards	Hollow, Gee Cro	oss. SK14 5DT	C23.	07.32	
5 Queen Street, Oldham. OL1 1RD.	Section				Sheet no./rev.		
Tel – 0161 652 1183		Structura	l Calculations			10	
www.gknltd.co.uk	Calc. by	Date	Chk'd by	Date	App'd by	Date	
U U	S.King	Nov. 2023					
Ref:	C	alculations			Outpu	t	
<u>Mid span 3</u>							
$3 \times 16\phi \text{ bars}$ $2 \times 8\phi \text{ shear legs at 200 c/c}$ $3 \times 16\phi \text{ bars}$							
Design moment resistance of re	ectangular se I = 0 kNm; - 0 000;	-500 → ►	Depth to tensio	n reinf.;	d = <b>384</b> mm		
ĸ	– <b>0.000</b> ,		K' > K - N	lo compression	n reinforcement is required		
l ever arm: z	= 365 mm <sup>.</sup>		Depth of neutra	al axis:	x = <b>43</b> mm		
Area of tension reinf reg'd: A	$s_{reg} = 1 \text{ mm}^2$ :		Tension reinf p	rovided:	$3 \times 16\phi$ bars		
Area of tension reinf prov: A	s.prov = 603 mr	n <sup>2</sup> :	Minimum area	of reinf:	$A_{s.min} = 293 \text{ mm}^2$		
Maximum area of reinf; A	<sub>s,max</sub> = <b>9000</b> m	m <sup>2</sup>					
P	ASS - Area o	f reinforcemen	t provided is gr	reater than area	of reinforcen	nent required	
Rectangular section in shear							
Shear reinforcement provided; 2	× 8¢ legs at 2	00 c/c					
Area of shear reinf provided; A	<sub>sv,prov</sub> = <b>503</b> m	m²/m;	Minimum area	of shear reinf;	A <sub>sv,min</sub> = <b>460</b> n	nm²/m	
May longitudinal anaging	P/	ASS - Area of s	hear reinforcen	nent provided e	exceeds minin	num required	
Max longitudinal spacing, s	PASS = I ong	l itudinal snacin	a of shear reint	orcement provi	idad is lass th	an maximum	
Specing of winforcement (al.2)	1 A00 - Long	tuumui spuem	g or shear renn	orecinent provi			
Actual dist between bars:	- 168 mm <sup>.</sup>		Min dist betwee	n hare:	s . − <b>25</b> mm		
	- <b>100</b> mm,		PAS	S - Satisfies the	minimum sn	acina criteria	
Design service stress:	= 0.4 N/mm <sup>2</sup> :		Max distance b	etween bars:	s <sub>max</sub> = <b>300</b> mn	n	
,	····,		PASS	S - Satisfies the	maximum sp	acing criteria	
Span to denth ratio (cl. $3.4.6$ )				-		•	
Span to depth ratio $(T 3 9)$	han to denth	nonia = 26 0 <sup>.</sup>	Service stress i	n tension rein:	$f_{0} = 0.4 \text{ N/mm}$	2	
Modification for tension reinf: f	ans = 2.000:	asic <b>24:4</b> ,	Modification for	comp reinf:	Is - 0.4 IN/IIIII <sup>-</sup> f <sub>comp</sub> = 1.095		
Modification for span > 10m: fir	ong = <b>1.000</b> :		Allowable span	to depth ratio:	span to dept	h <sub>allow</sub> = <b>56.9</b>	
Actual span to depth ratio:	pan to depth	actual = <b>1.2</b>			<u>-</u> br		
,		PASS	S - Actual span	to depth ratio i	s within the al	llowable limit	





Gamble, King and Noone Limited	Project				Job Ref.		
Consulting Structural Engineers & Building Surveyors	New Dwell	ing at Gerrards	Hollow, Gee C	ross. SK14 5DT	C23.07.32		
5 Queen Street, Oldham. OL1 1RD.	Section				Sheet no./rev.		
Tel – 0161 652 1183		Structura	l Calculations			13	
www.gknltd.co.uk	Calc. by	Date	Chk'd by	Date	App'd by	Date	
	S.King	Nov. 2023					
Ref:	Ca	alculations			Outp	out	
Support B		Vertically r	estrained				
		Rotationall	y free				
Applied loading							
Beam loads		Dead self v	veight of beam	× 1			
		PL from Su	ırcharge - Impo	sed point load 12	.38 kN at 37	50 mm	
		PL from So	oil - Imposed po	oint load 169 kN a	t 2500 mm		
Load combinations							
Load combination 1		Support A		Dead ×	1.40		
				Impose	d x 1 60		
				Dead x	1 40		
				Impose	d x 1 60		
		Support B		Dead x	1 40		
		ouppoir B		Impose	1. <del>4</del> 0		
				impose	4 ^ 1.00		
Analysis results							
Maximum moment;		M <sub>max</sub> = 484	<b>.9</b> kNm;	$M_{min} = C$	$M_{min} = 0 \text{ KNM}$		
Maximum snear;		V <sub>max</sub> = 195	.8 KN;	$V_{min} = -105.7 \text{ KN}$			
		omax = 19.9	mm;	δmin <b>= U</b>			
Infactored dood load reaction at s	upport A:	$RA_{max} - 1$	<b>1</b> KN	RA_min -	195.0 KIN		
Linfactored imposed load reaction at s	at support $A$ ,		$R_{A \text{ Imposed}} = 118.9 \text{ kN}$				
Maximum reaction at support B:	aroupporr,	$R_{B max} = 10$	R <sub>B max</sub> = 105.7 kN: R <sub>B min</sub> =				
Unfactored dead load reaction at s	upport B:	$R_B Dead = 4$	.1 kN				
Unfactored imposed load reaction	at support B;	R <sub>B Imposed</sub> =	<b>62.5</b> kN				
Section details							
Section type:		CHS 457 0	x10.0 (Tata St	eel Celsius (Gr3!	55 Gr420 Gi	-460))	
Steel grade:		S275	x10.0 (100 01		0 01420 01	400))	
From table 9: Design strength p	v						
Thickness of element;		t = <b>10.0</b> mr	n				
Design strength;		p <sub>y</sub> = <b>275</b> N	/mm²				
Modulus of elasticity;		E = 20500	0 N/mm²				
				Ar o			

Gamble, King and Noone Limited	Project				Job Ref.			
Consulting Structural Engineers & Building Surveyors	New Dwellin	g at Gerrards	Hollow, Gee	e Cross. SK14 5DT	C2	3.07.32		
5 Queen Street, Oldham. OL1 1RD.	Section				Sheet no./rev.			
Tel – 0161 652 1183		Structura		14				
www.gknltd.co.uk	Calc. by	Date	Chk'd by	Date	App'd by	Date		
	S.King	Nov. 2023						
Ref:	Cal	culations	Outp	out				
Lateral restraint								
		Span 1 ha	s lateral rest	traint at supports only	/			
Effective length factors								
Effective length factor in major axis	\$;	K <sub>x</sub> = <b>1.00</b>						
Effective length factor in minor axis	s;	K <sub>y</sub> = <b>1.00</b>						
Effective length factor for lateral-to	Effective length factor for lateral-torsional buckling; KLT.A = 1.00;							
		K <sub>LT.B</sub> = <b>1.0</b>	0;					
Classification of cross sections	- Section 3.5							
		ε = √[275 ľ	V/mm² / p <sub>y</sub> ] =	= 1.00				
Tubular sections - Table 12								
		D / t = 45.7	7×ε<=50×	< ε <sup>2</sup> ; Class 2	compact			
					Section is c	lass 2 compact		
Shear capacity - Section 4.2.3						-		
Design shear force;		Fv = max(a	abs(V <sub>max</sub> ), at	os(V <sub>min</sub> )) = <b>195.8</b> kN				
Shear area;		A <sub>v</sub> = 0.6 × A = <b>8426</b> mm <sup>2</sup>						
Design shear resistance;		P <sub>v</sub> = 0.6 ×	p <sub>y</sub> × A <sub>v</sub> = <b>13</b>	<b>90.2</b> kN				
		PA	SS - Design	shear resistance e	xceeds des	ign shear force		
Moment capacity - Section 4.2.5								
Design bending moment;		M = max(a	ıbs(M <sub>s1_max</sub> ),	abs(M <sub>s1_min</sub> )) = <b>484</b> .	9 kNm			
Moment capacity low shear - cl.4.2	2.5.2;	M₀ = min(p	$M_c = min(p_v \times S, 1.2 \times p_v \times Z) = 506.8 \text{ kNm}$					
		P	ASS - Mome	ent capacity exceed	ls design be	ending moment		
Check vertical deflection - Section	on 2.5.2							
Consider deflection due to dead ar	nd imposed loa	ds						
Limiting deflection;		$\delta_{\text{lim}} = L_{s1} / $	360 = <b>20.83</b>	<b>3</b> mm				
Maximum deflection span 1;		$\delta$ = max(al	os(δ <sub>max</sub> ), abs	s(δ <sub>min</sub> )) = <b>19.877</b> mm				
		PAS	SS - Maximu	um deflection does	not exceed	deflection limit		

450mm diameter steel pile is suitable for lateral loads worst case.

Gamble, King and Noone Limited	Project				Job Ref.		
Consulting Structural Engineers & Building Surveyo	s New Dwel	ling at Gerrards	Hollow, Gee Cr	oss. SK14 5D	т	C23.0	7.32
5 Oueen Street. Oldham. OL1 1RD.	Section				Sheet no./r	ev.	
Tel – 0161 652 1183		Structura	l Calculations			1	5
www.gknltd.co.uk	Calc. by	Date	Chk'd by	Date	App'd by		Date
	S.King	Nov. 2023					
Ref:	C	alculations	<u>.</u>		Ou	Itput	
Check 450mm dia. Pile fo	<sup>,</sup> compress	sive and ten	sile resista	nce.			
PILE ANALYSIS							
In accordance with EN 1997-1	2004 incorpor	ating Corrigen	dum dated Feb	ruary 2009 a	nd the UK na	ationa	l annex
					Tedds ca	lculatio	n version 1.0.08
Design summary							
Description Uni	t Actua	l Allo	wable Ut	ilisation R	esult	]	
Axial, compression	kN	4	479 C	.008	PASS		
Axial, tension	kN	185.1	378.7 0	.489	PASS		
		<b>⊲</b> ——450 mm-	<b></b>				
			and a second sec				
Pile details							
Installation method;	Drilled;		Shape;		450 mm o	diame	ter
Length;	_ = <b>7500</b> mm						
Material details							
Material:	Concrete:		Concrete stren	ath class:	C30/37		
Partial safety factor concrete:	$v_{\rm C} = 1.50$				a = 0 85	5	
Characteristic compression cylin	der strength:			,		•	
	$a = 30 \text{ N/mm}^{2}$		Design compre	esive strenat	n' f <sub>od</sub> = <b>17 0</b>	N/mm	n <sup>2</sup>
Moon value, evlinder strength:	ck = 30 N/mm	o <sup>2</sup> ·	Secont modulu	solve strengt		9 kNI/r	nm <sup>2</sup>
Modulus of electicity:		$kN/mm^2$	Secant modulu	is of elasticity		O KIN/I	
modulus of elasticity,	$- \Box_{cm} - 32.0$						
Geometric properties							
Bearing area;	A <sub>bearing</sub> = <b>0.159</b>	m²;	Pile perimeter;		Perim <sub>pile</sub> =	= 1.41	<b>4</b> m
Moment of inertia;	= <b>201289</b> cm <sup>4</sup>						



Gamble, King and Noone Limited	Project			Job Ref.				
Consulting Structural Engineers & Building Surveyors	New Dwel	ling at Gerrards	C23.07.32					
5 Queen Street, Oldham. OL1 1RD.	Section		Sheet no./rev.					
Tel – 0161 652 1183		Structura	al Calculations	3		17		
www.gknltd.co.uk	Calc. by	Date	Chk'd by	Date	App'd by	Date		
C C	S.King	Nov. 2023						
Ref:	С	alculations	Outpu	ıt				
Design compr. resistance; R	<sub>c,d,C2</sub> = <b>479</b> kN	d,C2 = <b>479</b> kN; F <sub>c,d,C2</sub> / R <sub>c,d,C2</sub> = <b>0.008</b>						
	PASS - Design compressive resistance exceeds design load							
Axial tensile resistance								
Load combination 1: A1 + M1 + R	1							
Design tension load; Ft	<sub>,d,C1</sub> = <b>246.8</b> k	N;	Part. resist.	factor, shaft tens.;	γ <sub>s,t,R1</sub> = <b>1</b>			
Design tensile resistance; R	t,d,C1 = <b>757.3</b> k	kN;	F <sub>t,d,C1</sub> / R <sub>t,d,C1</sub> = <b>0.326</b>					
			PASS - De	sign tensile resis	tance exceed	s design load		
Load combination 2: A2 + M1 + R	4							
Design tension load; Ft	<sub>,d,C2</sub> = <b>185.1</b> k	N;	Part. resist. factor, shaft tens.; $\gamma_{s,t,R4} = 2$					
Design tensile resistance; R	t,d,C2 = <b>378.7</b>	d,C2 = <b>378.7</b> kN; F <sub>t,d,C2</sub> / R <sub>t,d,C2</sub> = <b>0.489</b>						
			PASS - De	sign tensile resis	tance exceed	s design load		
. ,								

450mm diameter CFA pile is suitable for purpose.

Gamble, King and Noone Limited	Project				Job Ref.	
Consulting Structural Engineers & Building Surveyors	New Dwell	ling at Gerrards	C23.07.32			
5 Queen Street, Oldham. OL1 1RD.	Section				Sheet no./rev.	
Tel – 0161 652 1183		Structura	l Calculations		18	
www.gknltd.co.uk	Calc. by S.King	Date Nov. 2023	Chk'd by	Date	App'd by	Date

Ref:

#### Calculations

Output

# Check 3.6m High Lego Block Retaining Wall

Lego block dimensions = 1500mm long x 600mm wide x 600mm deep

Lego blocks have been installed onto a 500mm R.C concrete slab.

The lego blocks are not required to provide any retaining resistance due to the fact the CFA system has been installed directly behind. This has then been filled with no fines concrete with A393 fabric mesh. The lego block is acting as a shutter and has been installed upto a height of 3600mm. (6 rows of 600mm blocks) the sequence of installation should be verifyed by the Contractor who has installed them. By inspection the lego blocks are suitable for purpose. The 500mm thick slab will be checked for suitablity to support the self weight of the lego blocks.

# Check 500mm R.C slab – A393 mesh top and bottom







;

Gamble, King and Noone Limited	Project				Job Ref.				
Consulting Structural Engineers & Building Surveyors	New Dwell	ing at Gerrards	Hollow, Gee Cro	oss. SK14 5DT	C23.07.32				
5 Queen Street, Oldham. OL1 1RD.	Section			Sheet no./rev.					
Tel – 0161 652 1183		Structura	l Calculations	1		20			
www.gknltd.co.uk	Calc. by	Date	Chk'd by	Date	App'd by	Date			
	S.King	NOV. 2023							
Ref:	Ca	alculations			Output	t			
Check as simply supported slab									
RC SLAB DESIGN (BS8110:PAR	<u>T1:1997)</u>				TEDDS calculati	on version 1.0.04			
CONCRETE SLAB DESIGN (CL 3	<u>3.5.3 &amp; 4)</u>								
SIMPLE ONE WAY SPANNING S	LAB DEFINI	ΓΙΟΝ							
; Overall depth of slab; h =	500 mm								
; Cover to tension reinforcer	; Cover to tension reinforcement resisting sagging; $c_b = 40$ mm								
; Trial bar diameter; D <sub>tryx</sub> = <b>10</b> mm									
Depth to tension steel (resisting sagging)									
$d_x = h - c_b - D_{tryx}/2 = 455 \text{ mm}$									
; Characteristic strength of reinforcement; f <sub>y</sub> = <b>500</b> N/mm <sup>2</sup>									
; Characteristic strength of o	concrete; f <sub>cu</sub> =	<b>35</b> N/mm <sup>2</sup>							
ONE WAY SPANNING SLAB (CL	<u>. 3.5.4)</u>								
MAXIMUM DESIGN MOMENTS I	N SPAN								
; Design sagging moment (p	per m width of	slab); m <sub>sx</sub> = <b>57</b>	<b>.0</b> kNm/m						
<u>CONCRETE SLAB DESIGN – SA</u>	<u>GGING – OU</u>	TER LAYER O	F STEEL (CL 3.	<u>5.4)</u>					
; Design sagging moment (p	per m width of	slab); m <sub>sx</sub> = <b>57</b>	<b>.0</b> kNm/m						
; Moment Redistribution Fac	ctor; β <sub>bx</sub> = <b>1.0</b>								
Area of reinforcement required									
;; $K_x = abs(m_{sx}) / (d_x^2 \times f_{cu})$	= 0.008								
K' <sub>x</sub> = min (0.156 , (0.402 ×	(β <sub>bx</sub> - 0.4)) - (	0.18 × (β <sub>bx</sub> - 0.4	) <sup>2</sup> )) = <b>0.156</b>						
· · ·			Outer compre	ession steel not	required to re	sist sagging			
One-way Spanning Slab requirin	ng tension st	eel only (saggi	<u>ng) - mesh</u>		-				
;; $z_x = \min ((0.95 \times d_x), (d_x \times (0.95 \times d_x)))$	0.5+√(0.25-K <sub>×</sub>	/0.9)))) = <b>432</b> m	m						
Neutral axis depth; $x_x = (d_x)$	<sub>x</sub> - z <sub>x</sub> ) / 0.45 =	<b>51</b> mm							
Area of tension steel required									
;;; $A_{sx\_req} = abs(m_{sx}) / (1/\gamma_{ms} \times$	$f_y \times z_x$ ) = 303	mm²/m							
Tension steel									
;;Use A393 Mesh;									
A <sub>sx_prov</sub> = A <sub>sl</sub> = <b>393</b> mm <sup>2</sup> /m	; A <sub>sy_prov</sub> = A <sub>st</sub>	<b>= 393</b> mm²/m							
$D_x = d_{sl} = 10 \text{ mm}; D_y = d_{st}$	= <b>10</b> mm								
		A	rea of tension	steel provided s	ufficient to re	esist sagging			
Check min and max areas of ste	<u>el resisting s</u>	agging							
; I otal area of concrete; $A_c = h = 50$	JUUUU mm²/m								
; Minimum % reinforcement	; к <b>= 0.13</b> %								

amble, King and Noone Limited	Project				Job Ref.	
nsulting Structural Engineers & Building Surveyors	New Dwel	ling at Gerrards	Hollow, Gee C	cross. SK14 5DT	C23.07.32	
5 Queen Street, Oldham. OL1 1RD.	L 1RD. Section				Sheet no./rev.	04
Tel – 0161 652 1183	Colo by	Structura		Data	App'd by	21 Dete
www.gknltd.co.uk	S.King	Nov. 2023	Спка ру	Date	Арра бу	Date
f:	 C	alculations	<u> </u>		Outpu	ıt.
$A_{st min} = k \times A_c = 650 mm^2$	/m				•	
- A <sub>st max</sub> = 4 % × A <sub>c</sub> = 20000	mm²/m					
Steel defined:						
: Outer steel resisting saggi	na: A <sub>sx prov</sub> =	786 mm²/m				
, ealer electrocientig eagg	<b></b>		Less	than min area of	outer steel (	saaaina) O
: Inner steel resisting saggi	na: A <sub>sv. prov</sub> = 7	786 mm²/m	2000			
, initial account actioning accegan	19, 7 sy_plov		Less	than min area of	inner steel (	sagging) O
			Slab mesh ha	s been nested top	and bottom	therefore
SHEAR RESISTANCE OF CONC	KETE SLAB	<u>5 (CL 3.5.5)</u>				
Outer tension steel resisting sa	gging mome	nts				
; Depth to tension steel fror	n compressio	n tace; d <sub>x</sub> = <b>455</b>	mm			
; Area of tension reinforcem	ent provided	(per m width of	slab); A <sub>sx_prov</sub> =	= <b>393</b> mm²/m		
; Design ultimate shear force	e (per m widt	h of slab); V <sub>x</sub> =	<b>119</b> kN/m			
; Characteristic strength of	concrete; f <sub>cu</sub> =	= <b>35</b> N/mm <sup>2</sup>				
Applied shear stress						
v <sub>x</sub> = V <sub>x</sub> / d <sub>x</sub> = <b>0.26</b> N/mm <sup>2</sup>						
Check shear stress to clause 3.	5.5.2					
$v_{allowable}$ = min ((0.8 N <sup>1/2</sup> /mm) × $\sqrt{f_{c}}$	u ), 5 N/mm² )	) = <b>4.73</b> N/mm <sup>2</sup>				
					She	ar stress - (
Shear stresses to clause 3.5.5.3						
Design shear stress						
$f_{cu_ratio} = if (f_{cu} > 40 \text{ N/mm}^2)$	, 40/25 , f <sub>cu</sub> /(2	25 N/mm²)) = <b>1</b> .	400			
$v_{cx}$ = 0.79 N/mm <sup>2</sup> × min(3)	$100 \times A_{sx\_pro}$	$_{v}$ / d <sub>x</sub> ) <sup>1/3</sup> × max(0	).67,(400 mm /	$(d_x)^{1/4}) \ / \ 1.25 \times f_{cu_{-}}$	ratio <sup>1/3</sup>	
v <sub>cx</sub> = <b>0.30</b> N/mm <sup>2</sup>						
Applied shear stress						
v <sub>x</sub> = <b>0.26</b> N/mm <sup>2</sup>						
				No she	ear reinforcei	ment requii
SHEAR PERIMETERS FOR A RE	CTANGULA		ATED LOAD ((	CL 3.7.7)		
; Length of loaded rectangle	e; l = <b>600</b> mm		<b>`</b>			
; Width of loaded rectangle	w = <b>1000</b> mr	n				
: Depth to tension steel d	= <b>455</b> mm					
: Dimension from edge of lo	ad to shear r	erimeter: I <sub>n</sub> = kr	. × d <sub>x</sub> = <b>683</b> mn	n: where: k <sub>n</sub> = 1.50	)	
For punching shear cases not affe	cted by free e	edges or holes.		,,,	-	
Total length of inner porim	eter at edge	of loaded area	llo == 2 ∨ /l ±	(w) = 3200  mm		
			$u_{0}gen - 2 \times (1 + $	$w_j = 3200 \text{ mm}$		
i otal length of outer perim	ieter at Ip fron	i loaded area; u	$g_{gen} = 2 \times (I + W)$	) + 8 × Ip = 8660 n	nm	
PUNCHING SHEAR AT CONCEN	ITRATED LO	ADS (CL 3.7.7)	<u>!</u>			
Tension steel resisting sagging						
; Total length of inner perim	eter at edge	of loaded area;	u₀ = <b>3200</b> mm			

Gamble, King and Noone Limited Project					Job Ref.			
Consulting Structural Engineers & Building Surveyors	New Dwel	ling at Gerrards	Hollow, Gee Cro	oss. SK14 5DT	C23.07.32			
5 Queen Street, Oldham. OL1 1RD.	Section	<b>e</b> , , , , , , , , , , , , , , , , , , ,			Sheet no./rev.			
Tel – 0161 652 1183	Cala hu	Structura	Calculations	Data	Appld by	22 Data		
www.gknltd.co.uk	S.King	Nov. 2023		Dale	Арра бу	Dale		
Ref:	C	alculations	<u> </u>		Outpu	it it		
; Total length of outer perim	eter at dimen	ision l <sub>p</sub> from load	led area; u = <b>86</b>	<b>60</b> mm				
; Depth to outer steel; $d_x = 4$	<b>455</b> mm							
; Depth to inner steel; $d_y = 4$	<b>15</b> mm							
Average depth to "tension"	" steel; d <sub>av</sub> = (	(d <sub>x</sub> + d <sub>y</sub> )/2 = <b>250</b>	. <b>0</b> mm					
; Area of outer steel per m effective through the perimeter; A <sub>sx_prov</sub> = <b>393</b> mm <sup>2</sup> /m								
; Area of inner steel per m effective through the perimeter; A <sub>sy_prov</sub> = <b>393</b> mm <sup>2</sup> /m								
; Max shear effective across	; Max shear effective across either perimeter under consideration; V <sub>p</sub> = <b>119</b> kN							
; Characteristic strength of a	; Characteristic strength of concrete; f <sub>cu</sub> = <b>35</b> N/mm <sup>2</sup>							
Applied shear stress								
Stress around loaded area; $v_{max} = V_p / (u_0 \times d_{av}) = 0.149 \text{ N/mm}^2$								
Stress around perimeter; $v = V_p / (u \times d_{av}) = 0.055 \text{ N/mm}^2$								
Check shear stress to clause 3.7	7.7.2							
v <sub>allowable</sub> = min ((0.8 N <sup>1/2</sup> /mi	m) × $\sqrt{(f_{cu})}$ , 5	N/mm <sup>2</sup> ) = <b>4.73</b>	<b>3</b> N/mm <sup>2</sup>					
					Shea	ar stress - OK		
Shear stresses to clause 3.7.7.4								
Design shear stress								
f <sub>cu_ratio</sub> = if (f <sub>cu</sub> > 40 N/mm <sup>2</sup>	, 40/25 , f <sub>cu</sub> /(2	25 N/mm²)) = <b>1.</b> 4	400					
; Effective steel area for shear	strength de	etermination:;	A <sub>s_eff</sub> = <b>393</b> ı	mm²/m;				
$v_c = 0.79 \text{ N/mm}^2 \times \text{min}(3,$	100×( A <sub>s_eff</sub> /	d <sub>av</sub> ) ) <sup>1/3</sup> × max((	).67, (400 mm /	d <sub>av</sub> ) <sup>1/4</sup> ) / 1.25 × fo	cu_ratio <sup>1/3</sup>			
v <sub>c</sub> = <b>0.429</b> N/mm <sup>2</sup>								
				No she	ear reinforce	ment required		
CONCRETE SLAB DEFLECTION	CHECK (CL	<u> </u>						
; Slab span length; l <sub>x</sub> = <b>2.50</b>	<b>0</b> m							
; Design ultimate moment ir	n shorter spar	n per m width; m	<sub>sx</sub> = <b>57</b> kNm/m					
; Depth to outer tension ste	el; d <sub>x</sub> = <b>455</b> m	ım						
Tension steel								
; Area of outer tension reinf	orcement pro	vided; A <sub>sx_prov</sub> =	<b>393</b> mm²/m					
; Area of tension reinforcem	nent required;	A <sub>sx_req</sub> = <b>303</b> mi	m²/m					
; Moment Redistribution Fa	ctor; β <sub>bx</sub> = <b>1.0</b>	0						
Modification Factors								
;Basic span / effective depth ratio	(Table 3.9); ra	atio <sub>span_depth</sub> = <b>20</b>	I					
The modification factor for spans in	n excess of 1	0m (ref. cl 3.4.6	4) has not been	included.				
; f_s = 2 × f_y × A_{sx_req} / (3 × A_{sx_prov} × )	β <sub>bx</sub> ) = <b>257.2</b> Ι	N/mm <sup>2</sup>						
factor <sub>tens</sub> = min ( 2 , 0.55 + ( 477 N	/mm² - f <sub>s</sub> ) / (	120 × ( 0.9 N/m	m² + m <sub>sx</sub> / d <sub>x</sub> ²)))	= 2.000				
Calculate Maximum Span								
This is a simplified approach and f 3.4.6.4 and 3.4.6.7.	urther attentio	on should be giv	en where specia	al circumstances	exist. Refer to	o clauses		
Maximum span; I <sub>max</sub> = rati	$o_{span_depth}  imes factors$	$actor_{tens} \times d_x = 13$	<b>3.20</b> m					

Gamble, King and Noone Limited	Project				Job Ref.		
Consulting Structural Engineers & Building Surveyors	Consulting Structural Engineers & Building Surveyors New Dwelling at Gerrards Hollow, Gee Cross. SK14 5DT					23.07.32	
5 Queen Street, Oldham. OL1 1RD.	Section				Sheet no./rev		
Tel – 0161 652 1183	Cala hu	Structura		Data	Analdhu	23	
www.gknltd.co.uk	S.King	Nov. 2023	Clik d by	Date	Арра бу	Dale	
Ref	Calculations						
Check the actual beam span					out		
Actual span/depth ratio: Ix	/ d <sub>x</sub> = <b>5.49</b>						
Span depth limit: ratiospan	depth × factorte	ms = <b>40.00</b>					
- F - · · · · F · · · · · · · · · · · ·				Span/l	Depth ratio	check satisfied	
CHECK OF NOMINAL COVER (S	SAGGING) -	(BS8110:PT 1, )	<u> </u>				
; Slab thickness; h = <b>500</b> m	m						
; Effective depth to bottom	outer tension	reinforcement;	d <sub>x</sub> = <b>455.0</b> mm	I			
; Diameter of tension reinfo	rcement; D <sub>x</sub> =	= <b>10</b> mm					
; Diameter of links; L <sub>diax</sub> = <b>0</b>	mm						
Cover to outer tension reinforcem	ent						
$c_{tenx} = h - d_x - D_x / 2 = 40.0$	<b>)</b> mm						
Nominal cover to links steel							
c <sub>nomx</sub> = c <sub>tenx</sub> - L <sub>diax</sub> = <b>40.0</b> r	nm						
Permissable minimum nominal co	ver to all reinf	forcement (Table	e 3.4)				
; c <sub>min</sub> = <b>40</b> mm							
				Cover over	steel resisti	ng sagging OK	
3							
500mm Thick R.C slab wi	th 2xA393	3 mesh top	and botto	m (40mm cov	/er) is ad	equate to	
sup	port stone	e faced lego	blocks (3	3.6m high)			



Gamble, King and Noone Limited	Project				Job Ref.				
Consulting Structural Engineers & Building Surveyors	New Dwell	ling at Gerrards	Hollow, Gee C	ross. SK14 5DT	C	23.07.32			
5 Queen Street Oldham OL1 1RD	Section				Sheet no./rev	·.			
		Structura	l Calculations			25			
	Calc. by	Date	Chk'd by	Date	App'd by	Date			
www.gkmtd.co.uk	S.Kina	Nov. 2023	,						
			<u> </u>						
Ref:	Ca	alculations			Outp	out			
Support B		Vertically r	Vertically restrained						
		Rotationall	y free						
Applied loading									
Beam loads		Dead self	weight of beam	× 1					
		PL from Su	PL from Surcharge - Imposed point load 12.38 kN at 3750 mm						
		PL from So	oil - Imposed po	oint load 169 kN a	t 2500 mm				
Load combinations									
Load combination 1		Support A		Dead ×	1.40				
		Capport		Impose	d v 1 60				
					1 10				
					1. <del>4</del> 0				
		Current D		Inpose	u × 1.00				
		Support B		Deau ×	1.40				
				Impose	d × 1.60				
Analysis results									
Maximum moment;		M <sub>max</sub> = <b>486</b>	i kNm;	M <sub>min</sub> = 0	<b>)</b> kNm				
Maximum shear;		V <sub>max</sub> = <b>196</b>	V <sub>max</sub> = <b>196.5</b> kN;		V <sub>min</sub> = <b>-106.4</b> kN				
Deflection;		δ <sub>max</sub> = <b>14.4</b>	δ <sub>max</sub> = <b>14.4</b> mm;		$\delta_{\min} = 0 \text{ mm}$				
Maximum reaction at support A;		R <sub>A_max</sub> = 19	96.5 kN;	R <sub>A_min</sub> =	<b>196.5</b> kN				
Unfactored dead load reaction at s	upport A;	RA_Dead = 4	.5 kN						
Unfactored imposed load reaction	at support A;	R <sub>A_Imposed</sub> =	= 118.9 kN	_					
Maximum reaction at support B;		$R_{B_{max}} = 10$	$R_{B_{max}} = 106.4 \text{ kN};$ $R_{B_{min}}$			= 106.4 KN			
Unfactored dead load reaction at s	support B;	$R_{B_{Dead}} = 4$	$R_{B_{Dead}} = 4.5 \text{ kN}$						
Unfactored imposed load reaction	at support B;	RB_Imposed =	62.5 KIN						
Section details									
Section type;		CHS 508.0	CHS 508.0x10.0 (Tata Steel Celsius (Gr355 Gr420 Gr460))						
Steel grade;		S275							
From table 9: Design strength p	y .								
Thickness of element;		t = <b>10.0</b> mr	n						
Design strength;		$p_y = 275 N$	/mm²						
		E = 20500	U N/mm²						
				10					
				×					
	•	508							
				//					
				//					
			/	/					

Gamble, King and Noone Limited	Project				Job Ref.			
Consulting Structural Engineers & Building Surveyors	New Dwell	ing at Gerrards	Hollow, Gee Cr	oss. SK14 5DT	C2	23.07.32		
5 Queen Street, Oldham. OL1 1RD.	Section				Sheet no./rev			
Tel – 0161 652 1183		Structura	l Calculations			26		
www.gknltd.co.uk	Calc. by	Date	Chk'd by	Date	App'd by	Date		
-	S.King	Nov. 2023						
Ref:	Calculations			Output				
Lateral restraint								
		Span 1 has	s lateral restrain	it at supports only	1			
Effective length factors								
Effective length factor in major axis	;	K <sub>x</sub> = 1.00						
Effective length factor in minor axis	s;	K <sub>y</sub> = <b>1.00</b>						
Effective length factor for lateral-to	rsional buckli	ng; K <sub>LT.A</sub> = <b>1.0</b>	0;					
		K <sub>LT.B</sub> = <b>1.0</b>	D;					
Classification of cross sections	- Section 3.5	ł						
		ε = √[275 Ν	J/mm <sup>2</sup> / p <sub>y</sub> ] = <b>1.0</b>	00				
Tubular sections - Table 12								
		D/t = 50.8	× د <= 140 × د <sup>2</sup>	2. Class 3	semi-comp	act		
		D / ( = 00.0		, Class e Sectio	n is class 3	semi-compact		
				00010		oeini ooinpuot		
Snear capacity - Section 4.2.3		Г. — тару/а						
Design shear force;		$F_v = \max(a)$	$DS(V_{max}), aDS(V)$	min)) = 196.5 KIN				
Snear area;		$A_v = 0.6 \times 10^{-10}$	A = 9387 mm²	N 1-N 1				
Design snear resistance;		$P_v = 0.6 \times$	$p_y \times A_v = 1548.5$	) KIN	vaaada daa	ian abaar faraa		
		PAS	ss - Design she	ear resistance ex	ceeus des	ign shear lorce		
Moment capacity - Section 4.2.5								
Design bending moment;		M = max(a	bs(M <sub>s1_max</sub> ), abs	s(M <sub>s1_min</sub> )) = <b>486</b> k	۸m			
Effective plastic modulus - Section	on 3.5.6							
Limiting value for class 2 compact	flange;	$\beta_{2f} = 10 \times \epsilon$	: = 10					
Limiting value for class 3 semi-con	npact flange;	β <sub>3f</sub> = 15 × ε	: = 15					
Limiting value for class 2 compact	web;	β <sub>2w</sub> = 100 >	$\beta_{2w} = 100 \times \epsilon = 100$					
Limiting value for class 3 semi-con	npact web;	β <sub>3w</sub> = 120 >	< ε <b>= 120</b>					
Effective plastic modulus - cl.3.5.6	4							
S	<sub>eff</sub> = min(Z +	1.485 × (S - Z) :	< [√[(140 / (D / t	)) × (275 N/mm² /	py)] - 1], S)	= <b>2469106</b> mm <sup>3</sup>		
Moment capacity low shear - cl.4.2	5.2;	M <sub>c</sub> = min(p	$_y  imes S_{eff},  1.2  imes p_y$	× Z) = 630.4 kNr	n			
		PA	ASS - Moment	capacity exceed	s design be	ending moment		
Check vertical deflection - Section	on 2.5.2							
Consider deflection due to dead ar	nd imposed lo	ads						
Limiting deflection;		$\delta_{\text{lim}} = L_{s1} / 3$	360 = <b>20.833</b> m	m				
Maximum deflection span 1;		$\delta = \max(ab)$	$\mathbf{bs}(\delta_{max})$ , abs $(\delta_{mi})$	n)) = <b>14.427</b> mm				
· · ·		PAS	S - Maximum d	deflection does	not exceed	deflection limit		

600mm diameter steel pile is suitable for lateral loads worst case.

sulting Structural Engineers & Building Surveyo	•				JOD IVEI.		
	s New Dwell	New Dwelling at Gerrards Hollow, Gee Cross. SK14 5DT C23.07.32					
5 Queen Street, Oldham. OL1 1RD.	Section				Sheet no./rev.		
Tel – 0161 652 1183		Structura	al Calculations			27	
www.gknltd.co.uk	Calc. by	Date	Chk'd by	Date	App'd by	Date	
0	S.King	Nov. 2023					
f	C	alculations			Outou	ı <del>t</del>	
	0				Outpu		
<u>PILE ANALYSIS</u>	<u>compress</u>	ive and ter	n <mark>sile resista</mark> dum dated Feb	<u>NCE.</u> ruary 2009 and	the UK nation	nal annex	
Design summary	p	gg		,	Tedds calcula	tion version 1.0.08	
Description Uni	t Actual	Allo	wable Ut	ilisation Res	sult		
Axial, compression	kN	119	641.2 0	.186 P.	ASS		
Axial, tension	kN	185.1	504.9 0	.367 P.	ASS		
<b>Pile details</b> Installation method;	Drilled;		Shape;		600 mm diam	neter	
Pile details Installation method; Length; Material details	Drilled; L = <b>7500</b> mm		Shape;		600 mm diam	neter	
<b>Pile details</b> Installation method; Length; <b>Material details</b> Material;	Drilled; L = <b>7500</b> mm Concrete;		Shape; Concrete stren	gth class;	600 mm diam C30/37	neter	
<b>Pile details</b> Installation method; Length; <b>Material details</b> Material; Partial safety factor, concrete;	Drilled; L = <b>7500</b> mm Concrete; <sub>/c</sub> = <b>1.50</b> ;		Shape; Concrete stren Coefficient α <sub>cc</sub>	gth class;	600 mm diam C30/37 α <sub>cc</sub> = <b>0.85</b>	neter	
<b>Pile details</b> Installation method; Length; <b>Material details</b> Material; Partial safety factor, concrete; Characteristic compression cylin	Drilled; _ = <b>7500</b> mm Concrete; <sub>/C</sub> = <b>1.50</b> ; der strength;		Shape; Concrete streng Coefficient $\alpha_{cc}$	gth class;	600 mm diam C30/37 α <sub>cc</sub> = <b>0.85</b>	neter	
<b>Pile details</b> Installation method; Length; <b>Material details</b> Material; Partial safety factor, concrete; Characteristic compression cylin Mean value, cylinder strength; Modulus of elasticity;	Drilled; L = <b>7500</b> mm Concrete; <sub>/C</sub> = <b>1.50</b> ; der strength; f <sub>ck</sub> = <b>30</b> N/mm <sup>2;</sup> f <sub>cm</sub> = <b>38.0</b> N/mn E = E <sub>cm</sub> = <b>32.8</b> I	n²; kN/mm²	Shape; Concrete streng Coefficient α <sub>cc</sub> Design compre Secant modulu	gth class; ; ssive strength; s of elasticity;	600 mm diam C30/37 α <sub>cc</sub> = <b>0.85</b> f <sub>cd</sub> = <b>17.0</b> N/m E <sub>cm</sub> = <b>32.8</b> kN	neter nm² N/mm²	
<ul> <li>Pile details</li> <li>Installation method;</li> <li>Length;</li> <li>Material details</li> <li>Material;</li> <li>Partial safety factor, concrete;</li> <li>Characteristic compression cylin</li> <li>Mean value, cylinder strength;</li> <li>Modulus of elasticity;</li> <li>Geometric properties</li> <li>Bearing area;</li> <li>Moment of inertia;</li> </ul>	Drilled; L = <b>7500</b> mm Concrete; $\gamma_{C} = 1.50;$ der strength; $f_{cm} = 38.0 N/mm$ E = E <sub>cm</sub> = <b>32.8</b> I Abearing = <b>0.283</b> H = <b>636173</b> cm <sup>4</sup>	n²; kN/mm² m²;	Shape; Concrete streng Coefficient α <sub>cc</sub> Design compre Secant modulu Pile perimeter;	gth class; ; ssive strength; s of elasticity;	600 mm diam C30/37 $\alpha_{cc} = 0.85$ $f_{cd} = 17.0 \text{ N/m}$ $E_{cm} = 32.8 \text{ kN}$ Perim <sub>pile</sub> = 1.8	neter nm² V/mm² 885 m	



Gamble, King and Noone Limited	Project			Job Ref.				
Consulting Structural Engineers & Building Surveyors	New Dwel	ling at Gerrards	Hollow, Gee	Cross. SK14 5DT	C23.07.32			
5 Queen Street, Oldham. OL1 1RD.	Section		Sheet no./rev.					
Tel – 0161 652 1183		Structural Calculations				29		
www.gknltd.co.uk	Calc. by	Date	Chk'd by	Date	App'd by	Date		
	S.King	Nov. 2023						
Ref:	С	Calculations			Output			
Design compr. resistance; R	c,d,C2 = <b>641.2</b>	<n;< th=""><th colspan="5">F<sub>c,d,C2</sub> / R<sub>c,d,C2</sub> = 0.186</th></n;<>	F <sub>c,d,C2</sub> / R <sub>c,d,C2</sub> = 0.186					
		PA	SS - Design d	compressive resist	ance exceeds	s design load		
Axial tensile resistance								
Load combination 1: A1 + M1 + R	1							
Design tension load; Ft	<sub>.d,C1</sub> = <b>246.8</b> k	N;	Part. resist. factor, shaft tens.; $\gamma_{s,t,R1} = 1$					
Design tensile resistance; R <sub>i</sub>	<sub>,d,C1</sub> = <b>1009.8</b>	kN;	F <sub>t,d,C1</sub> / R <sub>t,d,C</sub>	c1 = <b>0.244</b>				
			PASS - De	esign tensile resist	ance exceeds	s design load		
Load combination 2: A2 + M1 + R	4							
Design tension load; Ft	<sub>.d,C2</sub> = <b>185.1</b> k	N;	Part. resist.	factor, shaft tens.;	γs,t,R4 <b>= 2</b>			
Design tensile resistance; Ri	,d,C2 = <b>504.9</b> k	κN;	F <sub>t,d,C2</sub> / R <sub>t,d,C</sub>	c2 = 0.367				
		PASS - Design tensile resistance exceeds						

600mm diameter CFA pile is suitable for purpose.

;

Gamble King and Noone Limited	Project				Job Ref.				
Counciliant Structured Environme & Building Structure	New Dwell	ing at Gerrards	Hollow. Gee Cr	oss. SK14 5DT	C23	.07.32			
Consulting Structural Engineers & Building Surveyors	Section				Sheet no./rev.				
		Structura	I Calculations			30			
	Calc. by	Date	Chk'd by	Date	App'd by	Date			
www.gknita.co.uk	S.King	Nov. 2023							
			<u> </u>						
Ref:	Ca	alculations			Output				
<u>Design of new reinforced c</u> <u>vertical load scenario)</u>	oncrete re	etaining wa	II (WALL A)	<u>) (No</u>					
Max. height of retaining wall = 3.5m Ma	ax. retained h	eight.							
2.5kN/m² Surcharge (Garden) worst ca	se								
The wall will have a proprietary drain ir	istalled at the	back of the wal	l to relieve any h	ydrostatic press	ure.				
The base has been designed as propp	The base has been designed as propped at the bottom due to the construction a new concrete floor slab.								
RETAINING WALL ANALYSIS (BS 8002:1994)									
		1700	<b> ⊲</b> 35 <b>0⊳ </b> 300 <b> ∢</b> −						
				3 kN/m²					
	Prop -								
	4	2350	▶						
Wall details									
Retaining wall type:		Cantilever	propped at ba	se					
Height of retaining wall stem;		h <sub>stem</sub> = <b>350</b>	<b>0</b> mm						
Thickness of wall stem;		t <sub>wall</sub> = 350 r	nm						
Length of toe;		I <sub>toe</sub> = <b>1700</b>	mm						
Length of heel;		I <sub>heel</sub> = 300 i	mm						
Overall length of base;		I <sub>base</sub> = I <sub>toe</sub> +	I <sub>heel</sub> + t <sub>wall</sub> = 235	<b>60</b> mm					
Thickness of base;		t <sub>base</sub> = <b>350</b>	mm						
Depth of downstand;		d <sub>ds</sub> = <b>0</b> mm	1						
Position of downstand;		l <sub>ds</sub> = <b>1600</b> I	mm						

Gamble, King and Noone Limited	Project				Job Ref.			
Consulting Structural Engineers & Building Surveyors	New Dwell	ling at Gerrards	Hollow, Gee	Cross. SK14 5DT	C2	3.07.32		
5 Queen Street, Oldham. OL1 1RD.	Section				Sheet no./rev.			
Tel – 0161 652 1183		Structura	I Calculations			31		
www.gknltd.co.uk	Calc. by	Date	Chk'd by	Date	App'd by	Date		
	S.King	Nov. 2023						
Ref:	C	alculations		·	Outp	ut		
Thickness of downstand;		t <sub>ds</sub> = <b>350</b> m	ım					
Height of retaining wall;		h <sub>wall</sub> = h <sub>stem</sub>	+ t <sub>base</sub> + d <sub>ds</sub> =	<b>- 3850</b> mm				
Depth of cover in front of wall;	Depth of cover in front of wall; d <sub>cover</sub> = <b>100</b> mm							
Depth of unplanned excavation;	Depth of unplanned excavation; d <sub>exc</sub> = <b>100</b> mm							
Height of ground water behind wal	l;	h <sub>water</sub> = <b>0</b> m	ım					
Height of saturated fill above base	,	h <sub>sat</sub> = max(	h <sub>water</sub> - t <sub>base</sub> - o	d <sub>ds</sub> , 0 mm) = <b>0</b> mm				
Density of wall construction;		γ <sub>wall</sub> = <b>23.6</b>	kN/m³					
Density of base construction;		γ <sub>base</sub> = <b>23.6</b>	3 kN/m³					
Angle of rear face of wall;		α <b>= 90.0</b> de	eg					
Angle of soil surface behind wall;		β <b>= 0.0</b> deg	3					
Effective height at virtual back of w	/all;	h <sub>eff</sub> = h <sub>wall</sub> +	$-I_{heel} \times tan(\beta)$	= <b>3850</b> mm				
Retained material details								
Mobilisation factor:		M = 1.5						
Moist density of retained material:		w = 1.0	N/m <sup>3</sup>					
Seturated density of retained material,	ym = 10.0 K	$\gamma_{\rm m} = -210  {\rm kN/m^3}$						
Design shoer strength:	urated density of retained material; $\gamma_s = 21.0 \text{ kN/m}^3$							
Design snear strengtn;		φ <sup>*</sup> = 25.0 de	eg					
Angle of wall friction;		δ = <b>19.3</b> de	èg					
Base material details								
Firm clay								
Moist density;		γ <sub>mb</sub> = <b>18.0</b>	kN/m³					
Design shear strength;		φ' <sub>b</sub> = <b>24.2</b> c	y'₅ = <b>24.2</b> deg					
Design base friction;		δ <sub>b</sub> = <b>18.6</b> d	eg					
Allowable bearing pressure;		P <sub>bearing</sub> = 1	00 kN/m²					
Using Coulomb theory								
Active pressure coefficient for reta	ined material							
$K_a = sin(\alpha + \alpha)$	$(sin(lpha)^2$ / $(sin(lpha)^2)$	$\times \sin(\alpha - \delta) \times [1 - \delta]$	+ √(sin(φ' + δ)	$\times \sin(\phi' - \beta) / (\sin(\alpha))$	$(\alpha + \delta) \times \sin(\alpha + \delta)$	⊦ β)))]²) = <b>0.358</b>		
Passive pressure coefficient for ba	se material							
	K <sub>p</sub> = sin(9	90 - φ' <sub>b</sub> )² / (sin(90	Ο - δ <sub>b</sub> ) × [1 - √(	$(\sin(\phi_{b} + \delta_{b}) \times \sin(\phi_{b}))$	' <sub>b</sub> ) / (sin(90 +	$\delta_b)))]^2) = 4.187$		
At-rest pressure								
At-rest pressure for retained mater	ial:	K₀ = 1 – sir	η(φ') = <b>0.577</b>					
	,		(,,)					
Surpharge lead on plan:		Suraharaa	- 2 E kN1/m2					
Applied vertical dead lead on well:			- 2.3 KN/III-					
Applied vertical live load on wall:			kNI/m					
Position of applied vertical load on	wall		n n/111					
Applied horizontal dead load on w	wan, all:		kNI/m					
Applied horizontal live load on wall	an,  •		N/m					
Height of applied horizontal load o	n wall:	$h_{load} = 0$ m	m					



Camble Ving and Naana Limited	Project				Job Ref		
Gamble, King and Noone Limited	New Dwel	ling at Gerrards	Hollow Gee	Cross SK14 5DT	C23.07.32		
Consulting Structural Engineers & Building Surveyors	Section			01033. 01(14 0D1	Shoot no /rov		
5 Queen Street, Oldham. OL1 1RD.	Section	Structure	Sheet no./rev.	22			
Tel – 0161 652 1183		Structura				33	
www.gknltd.co.uk	Calc. by	Date	Chk'd by	Date	App'd by	Date	
	S.King	Nov. 2023					
Ref:	С	alculations			Output		
Total restoring moment;		M <sub>rest</sub> = M <sub>wa</sub>	<sub>all</sub> + M <sub>base</sub> + N	1 <sub>m_r</sub> = <b>118.6</b> kNm/m			
Check bearing pressure							
Surcharge;		M <sub>sur_r</sub> = w <sub>s</sub>	$ur  imes (I_{base} - I_{he})$	<sub>el</sub> / 2) = <b>1.7</b> kNm/m			
Soil in front of wall;		$M_{p_r} = w_p$	< I <sub>toe</sub> / 2 = <b>2.6</b>	kNm/m			
Total moment for bearing;		M <sub>total</sub> = M <sub>re</sub>	est - Mot + Msu	<sub>r_r</sub> + M <sub>p_r</sub> = <b>58.7</b> kNm	ı/m		
Total vertical reaction;		R = W <sub>total</sub> =	= <b>71.0</b> kN/m				
Distance to reaction;		$x_{bar} = M_{tota}$	ı / R = <b>827</b> m	m			
Eccentricity of reaction;		e = abs((l <sub>b</sub>	<sub>ase</sub> / 2) - x <sub>bar</sub> )	= <b>348</b> mm			
				Reaction acts v	within middl	e third of base	
Bearing pressure at toe;		p <sub>toe</sub> = (R /	$I_{base}$ ) + (6 × F	$R \times e / I_{base}^2$ ) = 57.1 k	N/m <sup>2</sup>		
Bearing pressure at heel;		$p_{heel} = (R /$	$^{\prime}$ I <sub>base</sub> ) - (6 $ imes$ F	$R \times e / I_{base}^2$ ) = 3.4 kN	l/m²		
1						_	

PASS - Maximum bearing pressure is less than allowable bearing pressure

Gamble, King and Noone Limited	Project				Job Ref.			
Consulting Structural Engineers & Building Surveyors	New Dwel	ling at Gerrards	Hollow, Gee Cro	oss. SK14 5DT	C23.	07.32		
5 Queen Street, Oldham. OL1 1RD.	Section		Sheet no./rev.					
Tel – 0161 652 1183		Structura		34				
www.gknltd.co.uk	Calc. by	Date	Chk'd by	Date	App'd by	Date		
	S.King	Nov. 2023						
Ref:	С	alculations			Outpu	t		
RETAINING WALL DESIGN (BS	8002:1994 <u>)</u>							
				Т	EDDS calculation	version 1.2.01.08		
Ultimate limit state load factors								
Dead load factor;		$\gamma_{f_d} = 1.4$						
Live load factor;		γ <sub>f_l</sub> = <b>1.6</b>						
Earth and water pressure factor;		γ <sub>f_e</sub> = <b>1.4</b>						
Factored vertical forces on wall								
Wall stem;		$W_{wall_f} = \gamma_{f_c}$	$1 \times h_{stem} \times t_{wall} \times \gamma$	<sub>/wall</sub> = <b>40.5</b> kN/m				
Wall base;		$W_{base_f} = \gamma_{f}$	$_{d}  imes I_{base}  imes t_{base}  imes$	<sub>γbase</sub> = <b>27.2</b> kN/r	n			
Surcharge;		$W_{sur_f} = \gamma_{f_l}$	$\times$ Surcharge $\times$ I <sub>h</sub>	<sub>eel</sub> = <b>1.2</b> kN/m				
Moist backfill to top of wall;		$W_{m_w_f} = \gamma_{f_w}$	$_{\rm d} \times {\sf I}_{\sf heel} \times ({\sf h}_{\sf stem}$ -	$h_{sat}$ ) $\times \gamma_m = 26.5$	kN/m			
Soil in front of wall;		$W_{p_f} = \gamma_{f_d} \times$	$1_{toe}  imes d_{cover}  imes \gamma_{min}$	₀ = <b>4.3</b> kN/m				
Total vertical load;		$W_{total_f} = W_{v}$	wall_f + Wbase_f + W	/ <sub>sur_f</sub> + W <sub>m_w_f</sub> + W	<sub>p_f</sub> = <b>99.6</b> kN/n	n		
Factored horizontal at-rest force	s on wall							
Surcharge;		$F_{sur_f} = \gamma_{f_i}$	$\times$ K <sub>0</sub> $\times$ Surcharge	e × h <sub>eff</sub> = <b>8.9</b> kN/r	n			
Moist backfill above water table;	$F_{m_a_f} = \gamma_{f_e}$	$_{e}  imes 0.5  imes K_0  imes \gamma_m$	× (h <sub>eff</sub> - h <sub>water</sub> ) <sup>2</sup> =	<b>107.8</b> kN/m				
Total horizontal load;	F <sub>total_f</sub> = F <sub>su</sub>	ur_f + Fm_a_f = <b>116</b>	<b>6.7</b> kN/m					
Calculate propping force								
Passive resistance of soil in front of wall; $F_{p_f} = \gamma_{f_e} \times 0.5 \times K_p \times \cos(\delta_b) \times (d_{cover} + t_{base} + d_{ds} - d_{exc})^2 \times \gamma_{mb} = 6.1$								
kN/m								
Propping force;		F <sub>prop_f</sub> = ma	ax(F <sub>total_f</sub> - F <sub>p_f</sub> - (	W <sub>total_f</sub> - W <sub>sur_f</sub> - W	$v_{p_f}) \times tan(\delta_b), 0$	0 kN/m)		
		F <sub>prop_f</sub> = <b>78</b>	<b>.9</b> kN/m					
Factored overturning moments								
Surcharge;		M <sub>sur_f</sub> = F <sub>su</sub>	$r_f \times (h_{eff} - 2 \times d_{off})$	<sub>is</sub> ) / 2 = <b>17.1</b> kNm	ı/m			
Moist backfill above water table;		$M_{m_a_f} = F_n$	$M_{m_a_f} = F_{m_a_f} \times (h_{eff} + 2 \times h_{water} - 3 \times d_{ds}) / 3 = 138.4 \text{ kNm/m}$					
Total overturning moment;		M <sub>ot_f</sub> = M <sub>sur</sub>	M <sub>ot_f</sub> = M <sub>sur_f</sub> + M <sub>m_a_f</sub> = <b>155.5</b> kNm/m					
Restoring moments								
Wall stem;		$M_{wall_f} = w_w$	$_{\text{vall}_{f}} \times (I_{\text{toe}} + t_{\text{wall}})$	2) = <b>75.9</b> kNm/m	ı			
Wall base;		M <sub>base_f</sub> = w	$base_f \times I_{base} / 2 =$	<b>31.9</b> kNm/m				
Surcharge;		$M_{sur_f} = w$	$s_{ur_f} \times (I_{base} - I_{heel})$	/ 2) = <b>2.6</b> kNm/m	l			
Moist backfill;		M <sub>m_r_f</sub> = (w	m_w_f × (Ibase - Ihee	el / 2) + w <sub>m_s_f</sub> × (I	<sub>base</sub> - I <sub>heel</sub> / 3))	= 58.2		
kNm/m								
Soil in front of wall;		$M_{p_r_f} = w_{p_f}$	_f × I <sub>toe</sub> / 2 = <b>3.6</b> I	(Nm/m				
Total restoring moment;		$M_{rest_f} = M_v$	$_{vall_f} + M_{base_f} + N$	1 <sub>sur_r_f</sub> + M <sub>m_r_f</sub> + 1	M <sub>p_r_f</sub> = <b>172.3</b>	kNm/m		
Factored bearing pressure								
Total moment for bearing;		$M_{total_f} = M_f$	rest_f - Mot_f = <b>16.</b>	<b>3</b> kNm/m				
Total vertical reaction;		R <sub>f</sub> = W <sub>total_f</sub>	<b>= 99.6</b> kN/m					
Distance to reaction; $x_{bar_f} = M_{total_f} / R_f = 169 \text{ mm}$								
Eccentricity of reaction;	e <sub>f</sub> = abs((l <sub>base</sub> / 2) - x <sub>bar_f</sub> ) = <b>1006</b> mm							
		_ :	F	eaction acts ou	tside middle	third of base		
Bearing pressure at toe; $p_{toe_f} = R_f / (1.5 \times x_{bar_f}) = 393.4 \text{ kN/m}^2$								
$p_{heel_f} = 0 \text{ kiv/m}^2 = 0 \text{ kiv/m}^2$								
Rearing process of stem / test		rate = p <sub>toe</sub>	$max(n = \sqrt{r-1}) = 1$	$i 0.00 \text{ KIN/III}^/\text{III}$	$-0 k N l/m^2$			
bearing pressure at stem / toe;		Pstem_toe_f =	max(ptoe_f - (rate	$5 \times \text{Itoe}$ , U KIN/II <sup>2</sup> )				

Gamble, King and Noone Limited	Project				Job Ref.		
Consulting Structural Engineers & Building Surveyors	New Dwell	ling at Gerrards	Hollow, Gee Cro	oss. SK14 5DT	C23	3.07.32	
5 Queen Street, Oldham. OL1 1RD.	Section				Sheet no./rev.		
Tel – 0161 652 1183		Structura	I Calculations			35	
www.gknltd.co.uk	Calc. by	Date	Chk'd by	Date	App'd by	Date	
	S.King	Nov. 2023					
Ref:	Ca	alculations			Outpu	ut	
Bearing pressure at mid stem;		p <sub>stem_mid_f</sub> =	max(p <sub>toe_f</sub> - (rate	$e \times (I_{toe} + t_{wall} / 2)$	), 0 kN/m²) =	<b>0</b> kN/m <sup>2</sup>	
Bearing pressure at stem / heel;		Pstem_heel_f =	= max(p <sub>toe_f</sub> - (rat	$e \times (I_{toe} + t_{wall})), C$	) kN/m²) = <b>0</b> l	kN/m²	
Design of reinforced concrete re	taining wall	too (BS 8002-1	994)				
	tannig wan	<u>10e (B3 8002.1</u>	<u>334)</u>				
Material properties		£ 00 N//					
Characteristic strength of concrete	; mont:	$T_{cu} = 30 \text{ N/r}$	nm²				
	ment,	ly – 500 N/					
Base details							
Minimum area of reinforcement;		K = 0.13 %	m				
Cover to remore ment in toe,		Ctoe – <b>30</b> m	m				
Calculate shear for toe design			. ,				
Shear from bearing pressure;	$V_{toe\_bear} = 3$	$3 \times p_{toe_f} \times x_{bar_f} / $	2 = 99.6 kN/m				
Shear from weight of base;	Vtoe_wt_base	= $\gamma f_d \times \gamma base \times I_{too}$	$e \times t_{base} = 19.7 \text{ kN}$	N/m			
l otal shear for toe design;		$V_{toe} = V_{toe}$	bear - Vtoe_wt_base =	= <b>79.9</b> KIN/M			
Calculate moment for toe desigr	ı						
Moment from bearing pressure;	$3 \times p_{\text{toe}_f} \times X_{\text{bar}_f}$	< (Itoe - Xbar_f + t <sub>wal</sub>	/2)/2= <b>16</b>	<b>9.9</b> kNm/m			
Moment from weight of base; $M_{toe_wt_base} = (\gamma_{f_d} \times \gamma_{base} \times t_{base} \times (I_{toe} + t_{wall}))$ Total memory for too desires:						<b>3</b> kNm/m	
I otal moment for toe design;		$M_{toe} = M_{toe}$	_bear - M <sub>toe_wt_base</sub>	= <b>149.6</b> kNm/m			
350	•	•	•	•	•		
	<b>⊲</b> —150— <b>→</b>						
Check toe in bending							
Width of toe;		b = <b>1000</b> m	nm/m				
Depth of reinforcement;		$d_{toe} = t_{base}$	- c <sub>toe</sub> - (φ <sub>toe</sub> / 2) =	= <b>312.0</b> mm			
Constant;		$K_{toe} = M_{toe}$	/ (b × $d_{toe}^2$ × $f_{cu}$ )	= 0.051			
			Co	ompression rein	nforcement i	s not required	
Lever arm;		z <sub>toe</sub> = min(0 z <sub>toe</sub> = <b>293</b> r	).5 + √(0.25 - (m nm	in(K <sub>toe</sub> , 0.225) / 0	).9)),0.95) × c	toe	
Area of tension reinforcement requ	iired;	As_toe_des =	$M_{toe}$ / (0.87 $\times$ fy :	× z <sub>toe</sub> ) = <b>1173</b> mn	n²/m		
Minimum area of tension reinforce	ment;	$A_{s\_toe\_min} =$	$\mathbf{k} \times \mathbf{b} \times \mathbf{t}_{\text{base}} = 45$	5 <b>5</b> mm²/m			
Area of tension reinforcement requ	iired;	As_toe_req =	Max(As_toe_des, A	.s_toe_min) = <b>1173</b> r	mm²/m		
Reinforcement provided;		16 mm dia	.bars @ 150 mr	n centres			
Area of reinforcement provided;		As_toe_prov =	1340 mm²/m	vidad at the met-	ining well t	o io odor+-	
		rajj - Keir	norcement prov	vided at the reta	uuung wall to	is adequate	
Check shear resistance at toe				••••			
Design shear stress;		$v_{toe} = V_{toe} /$	(b × d <sub>toe</sub> ) = <b>0.25</b>	<b>6</b> N/mm <sup>2</sup>			

	Draigat				Job Dof				
Gamble, King and Noone Limited		lling at Gerrards	C23 07 32						
Consulting Structural Engineers & Building Surveyors	Section			0033. 01(14 001	Sheet no /rev	10.07.02			
5 Queen Street, Oldnam. OLT TRD.		Structura	I Calculations			36			
101 - 01010521105	Calc. by	Date	Chk'd by	Date	App'd by	Date			
www.gkintu.co.uk	S.King	S.King Nov. 2023							
Ref:	C	alculations	I		Outp	out			
Allowable shear stress:		V <sub>adm</sub> = min	/0.8 × √(f <sub>cu</sub> / 1	$N/mm^{2}$ , 5) × 1 N/r	mm <sup>2</sup> = <b>4.382</b> N/mm <sup>2</sup>				
,		PASS	Design shea	r stress is less th	nan maximu	m shear stress			
From BS8110:Part 1:1997 – Tab	e 3.8		U						
Design concrete shear stress;		v <sub>c_toe</sub> = <b>0.5</b>	<b>39</b> N/mm²						
<i>v</i> <sub>toe</sub> < <i>v</i> <sub>c_toe</sub> - <i>No shear reinforcement required</i>									
Design of reinforced concrete re	taining wall	l heel (BS 8002:	<u>1994)</u>						
Material properties									
Characteristic strength of concrete	;	f <sub>cu</sub> = <b>30</b> N/ı	mm²						
Characteristic strength of reinforce	ment;	f <sub>y</sub> = <b>500</b> N/	mm <sup>2</sup>						
Base details									
Minimum area of reinforcement;		k = 0.13 %							
Cover to reinforcement in heel;		c <sub>heel</sub> = <b>30</b> n	nm						
Calculate shear for heel design									
Shear from weight of base;		Vheel wt base	= $\gamma_{f d} \times \gamma_{base} \times$	I <sub>heel</sub> × t <sub>base</sub> = <b>3.5</b> k	N/m				
Shear from weight of moist backfil	Shear from weight of moist backfill;			kN/m					
Shear from surcharge;		V <sub>heel_sur</sub> = v	v <sub>sur_f</sub> = <b>1.2</b> kN/r	n					
Total shear for heel design;		$V_{heel} = V_{heel}$	el_wt_base + Vheel_	_wt_m + V <sub>heel_sur</sub> = 3	<b>1.1</b> kN/m				
Calculate moment for heel desig	ın								
Moment from weight of base;		Mheel_wt_base	e = (γ <sub>f_d</sub> × γ <sub>base</sub> )	imes t <sub>base</sub> $ imes$ (I <sub>heel</sub> + t <sub>wall</sub>	/ 2) <sup>2</sup> / 2) = <b>1</b>	<b>.3</b> kNm/m			
Moment from weight of moist back	fill;	$M_{heel\_wt\_m} = w_{m\_w\_f} \times (I_{heel} + t_{wall}) / 2 = 8.6 \text{ kNm/m}$							
Moment from surcharge;		$M_{heel\_sur} = w_{sur_f} \times (I_{heel} + t_{wall}) / 2 = 0.4 \text{ kNm/m}$							
Total moment for heel design;		M <sub>heel</sub> = M <sub>he</sub>	el_wt_base + Mhee	el_wt_m + M <sub>heel_sur</sub> =	<b>10.3</b> kNm/m				
	<b>↓</b> 150 <b>→</b>								
<b>▲</b>	•	•	•	• •	•				
350-									
Check neel in bending		h <b>- 1000</b> n	m/m						
Depth of reinforcement		$d_{\text{hool}} = t_{\text{hool}}$		(2) = <b>312 0</b> mm					
Constant:	Kheel = Mhe	uheel – Ubase – Cheel – ( $\phi$ heel / 2) = 312.0 MM							
		Theer - IVine		Compression rei	nforcement	is not reauired			
Lever arm;		Z <sub>heel</sub> = min Zhaol = <b>296</b>	0.5 + √(0.25 - mm	(min(K <sub>heel</sub> , 0.225)	/ 0.9)),0.95)	× d <sub>heel</sub>			
Area of tension reinforcement requ	iired;	∠neer - 230 As heel des =	: M <sub>heel</sub> / (0.87 ×	∶ f <sub>y</sub> × z <sub>heel</sub> ) = <b>80</b> mr	ım²/m				

Area of tension reinforcement required; Minimum area of tension reinforcement; Area of tension reinforcement required; Reinforcement provided;

## $A_{s\_heel\_req} = Max(A_{s\_heel\_des}, A_{s\_heel\_min}) = 455 \text{ mm}^2/\text{m}$

16 mm dia.bars @ 150 mm centres

 $A_{s\_heel\_min}$  = k × b × t<sub>base</sub> = **455** mm<sup>2</sup>/m

Gamble, King and Noone Limited	Project				Job Ref.				
Consulting Structural Engineers & Building Surveyors	New Dwel	ling at Gerrards	Hollow, Gee	Cross. SK14 5DT	C23	.07.32			
5 Queen Street, Oldham. OL1 1RD.	Section				Sheet no./rev.				
Tel – 0161 652 1183		Structura	I Calculations	5		37			
www.gkpltd.co.uk	Calc. by	Date	Chk'd by	Date	App'd by	Date			
www.gkintu.co.uk	S.King	Nov. 2023							
Ref:	C	alculations			Outpu	it			
Area of reinforcement provided;		$A_{s\_heel\_prov}$	= <b>1340</b> mm²/r	n					
		PASS - Rein	forcement pr	ovided at the retai	ining wall hee	el is adequate			
Check shear resistance at heel									
Design shear stress:		Vheel = Vhee	$(h \times d_{had}) =$	0 100 N/mm <sup>2</sup>					
Allowable about stress,				$N/mm^2$ E) $\therefore 1 N/m$	$am^2 = 4.393$ N	1/100 102			
Allowable shear stress,		Vadm - IIIII	1111 <sup>-</sup> - 4.302 N	1/11111-					
		PASS	- Design sne	ar stress is less th	an maximum	i snear stress			
From BS8110:Part 1:1997 – Tab	e 3.8								
Design concrete shear stress;		Vc_heel = <b>0.</b>	<b>539</b> N/mm²						
	v <sub>heel</sub> < v <sub>c_heel</sub> - No shear reinforcement required								
Design of reinforced concrete r	taining wall	ctom (BS 8002	-1004)						
Design of remorced concrete re	tanning wan	Stelli (BS 6002	.1554)						
Material properties									
Characteristic strength of concrete	;	f <sub>cu</sub> = <b>30</b> N/	mm²						
Characteristic strength of reinforce	ement;	fy = <b>500</b> N/	mm²						
Wall details									
Minimum area of reinforcement:		k = 0.13 %							
Cover to reinforcement in stem:	Cetem = 30	c <sub>stem</sub> = <b>30</b> mm							
Cover to reinforcement in wall:		$C_{\text{unell}} = 30 \text{ n}$	h						
Factored horizontal at-rest force	es on stem								
Surcharge;		Fs_sur_f = γ <sub>f</sub>	$_{\rm I} \times {\rm K}_0 \times {\rm Surch}$	$harge \times (h_{eff} - t_{base} - t_{base})$	d <sub>ds</sub> ) = <b>8.1</b> kN/i	m			
Moist backfill above water table;		$F_{s_m_a_f} = 0$	$F_{s_m_a_f} = 0.5 \times \gamma_{f_e} \times K_0 \times \gamma_m \times (h_{eff} - t_{base} - d_{ds} - h_{sat})^2 = 89.1 \text{ kN/m}$						
Calculate shear for stem design									
Shear at base of stem;		V <sub>stem</sub> = F <sub>s</sub>	<sub>sur f</sub> + F <sub>smaf</sub>	- F <sub>prop f</sub> = <b>18.3</b> kN/m	า				
Calculate moment for stem desi	an	-							
	gn	M 5	(1-		(				
Surcharge,		IVIs_sur – Fs	$VIs_{sur} = F_{sur_f} \times (Nstem + t_{base}) / 2 = 15.6 \text{ kNm/m}$						
Moist backfill above water table;		$M_{s_m_a} = F_a$	$M_{s_m_a} = F_{s_m_a_f} \times (2 \times h_{sat} + h_{eff} - d_{ds} + t_{base} / 2) / 3 = 119.6 \text{ kNm/m}$						
Total moment for stem design;		M <sub>stem</sub> = M <sub>s</sub>	_ <sub>sur</sub> + M <sub>s_m_a</sub> =	<b>135.1</b> kNm/m					
$\bullet$									
3100									
n n n n n n n n n n n n n n n n n n n					$\leq$				
	• •	•	•	• •	•				
▼									
	150 N								
	◄								
Check wall stem in bending									
Width of wall stem:		h - 1000 n	nm/m						
Dopth of roinforcements		0 – 1000 II		( ) - 240 0					
Depth of reinforcement;		Ustem = Twall	- Cstem - (Φster	m/2) = 310.0  mm					
Constant;		K <sub>stem</sub> = M <sub>st</sub>	$K_{stem} = M_{stem} / (b \times d_{stem}^2 \times f_{cu}) = 0.047$						
				Compression rein	nforcement is	not required			
Lever arm;		z <sub>stem</sub> = min	(0.5 + √(0.25	- (min(K <sub>stem</sub> , 0.225)	/ 0.9)),0.95) ×	dstem			

Gamble, King and Noone Limited	Project				Job Ref.			
Consulting Structural Engineers & Building Surveyors	New Dwel	ling at Gerrards	Hollow, Gee	e Cross. SK14 5DT	C2	C23.07.32		
5 Queen Street, Oldham. OL1 1RD.	Section				Sheet no./rev.			
Tel – 0161 652 1183		Structura	al Calculation	าร		38		
www.gknltd.co.uk	Calc. by	Date	App'd by	Date				
	S.King	Nov. 2023						
Ref:	С	alculations	Output					
		Zstem = <b>293</b>	mm					
Area of tension reinforcement requ	Area of tension reinforcement required;			$87 \times f_y \times z_{stem}$ ) = 106	<b>1</b> mm²/m			
Minimum area of tension reinforce	ement;	As_stem_min	= $\mathbf{k} \times \mathbf{b} \times \mathbf{t}_{wal}$	ı = <b>455</b> mm²/m				
Area of tension reinforcement requ	uired;	As_stem_req =	m_des, As_stem_min) = <b>1</b>	1061 mm²/m				
Reinforcement provided;		20 mm dia						
Area of reinforcement provided;		As_stem_prov						
		PASS - Reinf	orcement p	rovided at the retai	ining wall st	em is adequate		
Check shear resistance at wall s	stem							
Design shear stress;		v <sub>stem</sub> = V <sub>stem</sub> / (b × d <sub>stem</sub> ) = <b>0.059</b> N/mm <sup>2</sup>						
Allowable shear stress;		v <sub>adm</sub> = min	v <sub>adm</sub> = min(0.8 × √(f <sub>cu</sub> / 1 N/mm²), 5) × 1 N/mm² = <b>4.382</b> N/mm²					
		PASS	- Design sh	ear stress is less t	han maximu	m shear stress		
From BS8110:Part 1:1997 – Tab	le 3.8							
Design concrete shear stress;		v <sub>c_stem</sub> = <b>0</b> .	628 N/mm <sup>2</sup>					
			Vs	<sub>tem</sub> < v <sub>c_stem</sub> - No sh	ear reinforc	ement required		
Check retaining wall deflection								
Basic span/effective depth ratio;		ratio <sub>bas</sub> = 7	,					
Design service stress;		$f_s = 2 \times f_y$	< As_stem_req /	$(3 \times A_{s\_stem\_prov}) = 1$	<b>68.8</b> N/mm <sup>2</sup>			
Modification factor; fa	ctor <sub>tens</sub> = min	(0.55 + (477 N/r	nm <sup>2</sup> - f <sub>s</sub> )/(120	$0 \times (0.9 \text{ N/mm}^2 + (\text{M}))$	<sub>stem</sub> /(b × d <sub>stem</sub>	<sup>2</sup> )))),2) = <b>1.66</b>		
Maximum span/effective depth rat	Maximum span/effective depth ratio; ratio <sub>max</sub> = ratio <sub>bas</sub> $\times$ factor <sub>tens</sub> = <b>11.65</b>							
Actual span/effective depth ratio;		ratio <sub>act</sub> = h	11.29					

PASS - Span to depth ratio is acceptable



Gamble, King and Noone Limited	Project				Job Ref.			
Consulting Structural Engineers & Building Surveyors	New Dwel	ling at Gerrards	Hollow, Gee C	Cross. SK14 5DT	C23.07.32			
5 Queen Street, Oldham. OL1 1RD.	Section		Sheet no./rev.					
Tel – 0161 652 1183		Structura	al Calculations			40		
www.gknltd.co.uk	Calc. by	Date	Chk'd by	Date	App'd by	Date		
	S.King	Nov. 2023						
Ref:	C	alculations			Outp	Output		
Design of new reinforced c	oncrete r	etaining wa	II (WALL E	<u> 3) (vertical</u>				
load scenario)								
<b>_</b>								
Mary haight of actoining well - 0.5m M		- : 4						
Max. height of retaining wall = 3.5m Ma	ax. retained h	eight.						
No surcharge but will have vertical load	d from the pro	posed dwelling						
5	1	1 5						
Dead load								
Main roof = 9.0m / 2 x 1.00kN/m²	= 4.50kN/m							
1 <sup>st</sup> floor = 5.5m / 2 x 0.55kN/m <sup>2</sup>	= 1.51kN/m							
Ground floor = 5.5m / 2 x 0.55kN/m <sup>2</sup>	= 1.51kN/m							
Ext. Rear Wall = 4.0m high x 4.0kN/m <sup>2</sup>	= 16.0kN/m							
Total dead load	d = 23.52kN/	m						
Superload								
$\frac{64000}{1000}$ Main roof = 0.0m / 2 x 1.00kN/m <sup>2</sup>	- 1 50kN/m							
$1^{st}$ floor = 5.5m / 2 x 1.00kN/m <sup>2</sup>	- 4.30KN/III	I						
$-5.51172 \times 1.5000000$	- 4.13KN/III							
	- 4. ISKN/III							
	u = 12.0 KIN/II							
The wall will have a proprietary drain in	stalled at the	back of the wa	I to relieve any	/ hydrostatic press	ure.			
The base has been designed as propp	ed at the bott	om due to the c	onstruction a r	new concrete floor	slab.			

RETAINING WALL ANALYSIS (BS 8002:1994)

TEDDS calculation version 1.2.01.08



Gamble, King and Noone Limited	Project				Job Ref.				
Consulting Structural Engineers & Building Surveyors	New Dwel	ling at Gerrards	oss. SK14 5DT	C23.07.32					
5 Queen Street, Oldham. OL1 1RD.	Section				Sheet no./rev.				
Tel – 0161 652 1183		Structura	I Calculations	1		42			
www.gknltd.co.uk	Calc. by	Date	Chk'd by	Date	App'd by	Date			
	5.King	NOV. 2023							
Ref:	C	alculations			Outpu	t			
Design shear strength;		φ' = <b>25.0</b> d	eg						
Angle of wall friction;		δ <b>= 19.3</b> de	èg						
Base material details									
Firm clay									
Moist density;		γ <sub>mb</sub> = <b>18.0</b>	kN/m <sup>3</sup>						
Design shear strength;		φ' <sub>b</sub> = <b>24.2</b> α	deg						
Design base friction;		δ <sub>b</sub> = <b>18.6</b> d	eg						
Allowable bearing pressure;		Pbearing = 1	00 kN/m²						
Using Coulomb theory									
Active pressure coefficient for reta	ined material								
$K_a = \sin(\alpha + \alpha)$	φ')² / (sin(α)² :	$\times \sin(\alpha - \delta) \times [1 - \delta]$	+ √(sin(φ' + δ) ×	sin(φ' - β) / (sin(α	$-\delta$ ) × sin( $\alpha$ +	β)))] <sup>2</sup> ) <b>= 0.358</b>			
Passive pressure coefficient for ba	ise material	$(1)^2/(-i\pi/0)$				S ()(12) - 4 407			
	$\kappa_p = \sin(s)$	90 - φ <sub>b</sub> )- / (sin(90	J - ₀₀) × [1 - ∿(Sii	$n(\phi_{\rm b} + o_{\rm b}) \times \sin(\phi_{\rm b})$	b) / (SIN(90 + a	ob)))] <sup>2</sup> ) = <b>4.187</b>			
At-rest pressure			(						
At-rest pressure for retained mater	rial;	$K_0 = 1 - si$	n(φ') = <b>0.577</b>						
Loading details									
Surcharge load on plan;									
Applied vertical dead load on wall;		VVdead = 23	W <sub>live</sub> = <b>12.8</b> kN/m						
Position of applied vertical load on	wall.	$v_{\text{live}} = 12.0$							
Applied horizontal dead load on w	all:	$F_{dead} = 0.0$	kN/m						
Applied horizontal live load on wal	l;	F <sub>live</sub> = <b>0.0</b> k	(N/m						
Height of applied horizontal load o	n wall;	h <sub>load</sub> = <b>0</b> m	m						
			36 1	0					
			<b>†</b>	0.CT 1					
			××	*>					
				A					
				A					
				B					
	~~~~								
40.5									
				Loads shown	in KN/m, pressure	es snown in kN/m <sup>2</sup>			

Gamble, King and Noone Limited	Project				Job Ref.		
Consulting Structural Engineers & Building Surveyors	New Dwel	ling at Gerrards	oss. SK14 5DT	C	23.07.32		
5 Queen Street, Oldham. OL1 1RD.	Section				Sheet no./rev	<i>.</i>	
Tel – 0161 652 1183	Structural Calculations					43	
www.gknltd.co.uk	Calc. by	Date	Chk'd by	Date	App'd by	Date	
	S.King	Nov. 2023					
Ref:	C	alculations	Outj	Output			
Vertical forces on wall							
Wall stem;		$w_{wall} = h_{ster}$	$_{n} \times t_{wall} \times \gamma_{wall} =$	<b>28.9</b> kN/m			
Wall base;		$W_{base} = I_{base}$	$_{e}  imes t_{base}  imes \gamma_{base}  imes$	= <b>18.6</b> kN/m			
Moist backfill to top of wall;		$w_{m_w} = I_{heel}$	$\times$ (h <sub>stem</sub> - h <sub>sat</sub> ) >	<pre>γ<sub>m</sub> = 12.6 kN/m</pre>			
Soil in front of wall;		$w_p = I_{toe} \times c$	$d_{cover} \times \gamma_{mb} = 3.$	<b>1</b> kN/m			
Applied vertical load;		$W_v = W_{dead}$	+ W <sub>live</sub> = <b>36.3</b> I	κN/m			
Total vertical load;		$W_{total} = W_{wa}$	all + Wbase + Wm_v	$w + w_p + W_v = 99.$	<b>5</b> kN/m		
Horizontal forces on wall							
Moist backfill above water table;		F <sub>m_a</sub> = 0.5	$\times$ K <sub>a</sub> $\times$ cos(90 -	$\alpha$ + $\delta$ ) × $\gamma_m$ × (h <sub>eff</sub>	- $h_{water})^2 = 4$	<b>15.1</b> kN/m	
Total horizontal load; $F_{total} = F_{m_a} = 45.1 \text{ kN/m}$							
Calculate propping force							
Passive resistance of soil in front of	of wall;	$F_p = 0.5 \times$	$K_p  imes cos(\delta_b)  imes (\mathbf{c})$	d <sub>cover</sub> + t <sub>base</sub> + d <sub>ds</sub>	- $d_{exc})^2 \times \gamma_{mt}$	<sub>o</sub> = <b>4.4</b> kN/m	
Propping force;	$F_{prop} = max$	$F_{prop} = max(F_{total} - F_{p} - (W_{total} - w_{p} - W_{live}) \times tan(\delta_{b}), 0 \; kN/m)$					
		F <sub>prop</sub> = <b>12.6</b>	s kN/m				
Overturning moments							
Moist backfill above water table;		$M_{m_a} = F_{m_a}$	$_{a} \times (h_{eff} + 2 \times h_{v})$	<sub>vater</sub> - 3 × d <sub>ds</sub> ) / 3 =	= <b>57.8</b> kNm/	m	
Total overturning moment;		$M_{ot} = M_{m_a}$	= <b>57.8</b> kNm/m				
Restoring moments							
Wall stem;		M <sub>wall</sub> = w <sub>wa</sub>	$II \times (I_{toe} + t_{wall} / 2)$	) = <b>54.2</b> kNm/m			
Wall base;		M <sub>base</sub> = w <sub>ba</sub>	ase × I <sub>base</sub> / 2 <b>= 2</b>	<b>0.9</b> kNm/m			
Moist backfill;		M <sub>m_r</sub> = (w <sub>m</sub>	$_w \times (I_{base} - I_{heel})$	′2) + w <sub>m_s</sub> × (I <sub>base</sub>	- I <sub>heel</sub> / 3)) =	<b>27.1</b> kNm/m	
Design vertical dead load;		M <sub>dead</sub> = W <sub>d</sub>	$_{ead} \times I_{load} = 44.1$	kNm/m			
Total restoring moment;		M <sub>rest</sub> = M <sub>wa</sub>	II + M <sub>base</sub> + M <sub>m_</sub>	+ M <sub>dead</sub> = <b>146.3</b>	kNm/m		
Check bearing pressure							
Soil in front of wall;		$M_{p_r} = w_p \times$	I <sub>toe</sub> / 2 = <b>2.6</b> kN	lm/m			
Design vertical live load;		M <sub>live</sub> = W <sub>live</sub>	e × I <sub>load</sub> = <b>24</b> kNr	n/m			
Total moment for bearing;		M <sub>total</sub> = M <sub>re</sub>	st - Mot + Mp_r +	M <sub>live</sub> = <b>115.1</b> kNm	n/m		
Total vertical reaction;		R = W <sub>total</sub> =	<b>99.5</b> kN/m				
Distance to reaction;		$x_{bar} = M_{total}$	/ R = <b>1157</b> mm				
Eccentricity of reaction;		e = abs((l <sub>ba</sub>	ase / 2) - x <sub>bar</sub> ) = 3	<b>32</b> mm			
				Reaction acts	within midd	lle third of base	
Bearing pressure at toe;		p <sub>toe</sub> = (R / I	$_{base})$ - (6 × R × $\epsilon$	e / I <sub>base</sub> <sup>2</sup> ) = <b>40.5</b> kl	N/m <sup>2</sup>		
Bearing pressure at heel;		p <sub>heel</sub> = (R /	$I_{base}$ ) + (6 × R ×	e / I <sub>base</sub> <sup>2</sup> ) = <b>48</b> kN	l/m <sup>2</sup>		
	PAS	SS - Maximum I	bearing pressu	ıre is less than a	llowable be	earing pressure	

Gamble, King and Noone Limited	Project				Job Ref.				
Consulting Structural Engineers & Building Surveyors	New Dwel	ling at Gerrards	oss. SK14 5DT	C23.07.32					
5 Queen Street, Oldham. OL1 1RD.	Section		Sheet no./rev.						
Tel – 0161 652 1183		Structura	l Calculations			44			
www.gknltd.co.uk	Calc. by	Date	Chk'd by	Date	App'd by	Date			
	S.King	Nov. 2023							
Ref:	C	alculations			Outpu	t			
RETAINING WALL DESIGN (BS	3002:1994 <u>)</u>								
				Т	EDDS calculation	version 1.2.01.08			
Ultimate limit state load factors									
Dead load factor;		γ <sub>f_d</sub> = <b>1.4</b>							
Live load factor;		γ <sub>f_l</sub> = <b>1.6</b>							
Earth and water pressure factor;		γ <sub>f_e</sub> = 1.4							
Factored vertical forces on wall									
Wall stem;		$W_{wall_f} = \gamma_{f_c}$	$1 \times h_{stem} \times t_{wall} \times \gamma$	wall = <b>40.5</b> kN/m					
Wall base;		$W_{base_f} = \gamma_{f_i}$	$d \times I_{base} \times t_{base} \times d$	γ <sub>base</sub> = <b>26</b> kN/m					
Moist backfill to top of wall;		$W_{m_w_f} = \gamma_f$	$d \times I_{heel} \times (h_{stem} -$	h <sub>sat</sub> ) × γ <sub>m</sub> = <b>17.6</b>	kN/m				
Soil in front of wall;		$W_{p_f} = \gamma_{f_d} \times$	$I_{toe} \times d_{cover} \times \gamma_{mt}$	₀ = <b>4.3</b> kN/m					
Applied vertical load;		$VV_{v_f} = \gamma_{f_d}$	< W <sub>dead</sub> + γ <sub>f_l</sub> × W	/ <sub>live</sub> = <b>53.4</b> kN/m	444.01.01/				
l otal vertical load;		$VV$ total_f = $W_V$	vall_f + Wbase_f + W	$m_w_f + W_{p_f} + VV_{v_f}$	_f = 141.8 KIN/f	n			
Factored horizontal at-rest force	s on wall	_	a = 1/	<i>и</i>					
Moist backfill above water table;	$F_{m_a_f} = \gamma_{f_e}$	$\times 0.5 \times K_0 \times \gamma_m$	$\times (h_{eff} - h_{water})^2 =$	107.8 kN/m					
I OTAI NORIZONTAI IOAD; $F_{total_f} = F_{m_a_f} = 107.8 \text{ kN/m}$									
Calculate propping force	<b>.</b>	_							
Passive resistance of soil in front c	of wall;	$F_{p_f} = \gamma_{f_e} \times$	$0.5 \times K_p \times \cos(\delta)$	$\delta_{b}) \times (d_{cover} + t_{base})$	, + d <sub>ds</sub> - d <sub>exc</sub> )²⇒	< γ <sub>mb</sub> = <b>6.1</b>			
Propping lorce,		$F_{\text{prop}_f} = \Pi c$	IX(Ftotal_f - Fp_f - ( 3 kN/m	VVtotal_f - Wp_f - γf_l	× vv <sub>live</sub> ) × tari(	$o_b$ , $U KIN/III)$			
<b>-</b>		1 prop_1 – <b>02</b>	<b>3</b> KIN/III						
Factored overturning moments		M - F		b 2d )//	) - <b>430 4</b> kMm	100			
Total overturning moment:		$M_{ab} f = M_{m}$	$\text{IVI}_{m_a,f} - \Gamma_{m_a,f} \times (\text{IIeff} + 2 \times \text{IIwater} - 3 \times \text{Uds}) / 3 = 138.4 \text{ KNM/M}$ $M_{\text{eff}} = M_{m_a,f} = 138.4 \text{ KNm/m}$						
Restoring moments				2) - 75.0 k Mm/m					
Wall base:		$M_{\text{wall}_{f}} = W_{W}$	$\frac{1}{100} = \frac{1}{100} = \frac{1}$						
Moist backfill		$M_{m,r,f} = (w)$	$\frac{1}{1000} = \frac{1}{1000} + \frac{1}{10000} + \frac{1}{10000} + \frac{1}{100000} + \frac{1}{10000000000000000000000000000000000$						
kNm/m		wiii_i_i = (wi		(i	Dase - Meer / O))	- 07.5			
Soil in front of wall;		$M_{p,r,f} = W_{p}$	<sub>f</sub> × I <sub>toe</sub> / 2 = <b>3.6</b> k	Nm/m					
Design vertical load;		$M_{v f} = W_{v f}$	× I <sub>load</sub> = 100.1 kl	Nm/m					
Total restoring moment;		M <sub>rest_f</sub> = M <sub>w</sub>	<sub>/all_f</sub> + M <sub>base_f</sub> + N	$I_{m_r_f} + M_{p_r_f} + M$	<sub>v_f</sub> = <b>246.9</b> kN	m/m			
Factored bearing pressure									
Total moment for bearing;		$M_{total_f} = M_r$	<sub>est_f</sub> - M <sub>ot_f</sub> = <b>108</b>	<b>.5</b> kNm/m					
Total vertical reaction;		R <sub>f</sub> = W <sub>total_f</sub>	= <b>141.8</b> kN/m						
Distance to reaction;		$\mathbf{x}_{bar_f} = \mathbf{M}_{tot}$	<sub>al_f</sub> / R <sub>f</sub> = <b>765</b> mn	n					
Eccentricity of reaction;		e <sub>f</sub> = abs((I <sub>b</sub>	<sub>ase</sub> / 2) - x <sub>bar_f</sub> ) =	360 mm					
				Reaction acts v	vithin middle	third of base			
Bearing pressure at toe;		$p_{toe_f} = (R_f)$	$(I_{base}) + (6 \times R_f >$	$(e_f / I_{base^2}) = 123$	.6 KN/m²				
Bearing pressure at heel; $p_{heel_f} = (R_f / l_{base}) - (6 \times R_f \times e_f / l_{base}^2) = 2.5 \text{ kN/m}^2$									
Rearing pressure at stem / too:		nate = (ptoe	_t = Pheel_f) / Ibase =	- $33.00$ KIV/III <sup>+</sup> /M	= 32 1 kN/m <sup>2</sup>				
Bearing pressure at stem? loe,	Searing pressure at stem / toe; p <sub>stem_tc</sub>				$= 32.1 \text{ kN/m}^2$	2.7 kN/m <sup>2</sup>			
Bearing pressure at stem / heel		Pstem_haal_f =	= max(ptoe_r - (rat	$e \times (I_{\text{top}} + t_{\text{wall}}))  ($	) kN/m <sup>2</sup> ) = <b>13</b> :	<b>3</b> kN/m <sup>2</sup>			
		►2rem_ueei_t -			,, in j = 10.				

Gamble, King and Noone Limited	Project				Job Ref.		
Consulting Structural Engineers & Building Surveyors	New Dwel	ling at Gerrards	Hollow, Gee Cro	oss. SK14 5DT	C23.07.32		
5 Queen Street, Oldham. OL1 1RD.	Section				Sheet no./rev.		
Tel – 0161 652 1183		Structura	l Calculations			45	
www.gknltd.co.uk	Calc. by	Date	Chk'd by	Date	App'd by	Date	
-	S.King	Nov. 2023					
Ref:	С	alculations			Outpu	t	
Design of reinforced concrete re	taining wall	toe (BS 8002:1	<u>994)</u>				
Material properties							
Characteristic strength of concrete	;	f <sub>cu</sub> = <b>30</b> N/r	nm²				
Characteristic strength of reinforce	ment;	fy = <b>500</b> N/	mm²				
Base details							
Minimum area of reinforcement:		k = 0.13 %					
Cover to reinforcement in toe:		<sub>Ctoe</sub> = <b>40</b> m	m				
Calculate aboar for too design							
Shoer from boaring procedure:		<u> </u>	n (+ n · · · ·	) v l (2 - <b>132 3</b>	kNI/m		
Shear from weight of base:		V toe_bear - (	ptoe_r · pstem_toe_r	-10.7 k			
Total shear for too design:		V toe_wt_base	<ul> <li>γf_d × γbase × Ito</li> </ul>	$e \times l_{base} = 13.7 \text{ Km}$	N/111		
		v toe - v toe_	bear - V toe_wt_base -	- 112.7 KIN/III			
Calculate moment for toe design	1	N.4	(0. <b></b> )			<b>D d</b> 1/N 1 /	
Moment from bearing pressure;	IVI <sub>toe_bear</sub> = (	$2 \times p_{\text{toe}_f} + p_{\text{stem}_f}$	_mid_f) × (Itoe + twall	$(1/2)^2/0 = 150$	<b>5.1</b> KINM/M		
Moment from weight of base;	Moment from weight of base; M <sub>tc</sub>				2)²/2) = 20.3	KNM/M	
		IVItoe = IVItoe	_bear - IVItoe_wt_base	= 137.8 KINM/M			
T T I							
-350					$\leq$		
	•	•	•	•	•		
•							
	<b>⊲</b> —150— <b>▶</b>						
	1 1						
Check toe in bending							
Width of toe;		b = <b>1000</b> m	ım/m				
Depth of reinforcement;		d <sub>toe</sub> = t <sub>base</sub> -	$- C_{\text{toe}} - (\phi_{\text{toe}} / 2) =$	= 302.0 mm			
Constant;		$K_{toe} = M_{toe}$	/ (b × d <sub>toe<sup>2</sup></sub> × f <sub>cu</sub> )	= 0.050			
				ompression rein	norcement is	not required	
Lever arm;		$z_{\text{toe}} = \min(0$	).5 + ∿(0.25 - (m mm	IIN(K <sub>toe</sub> , 0.225) / 0	J.9)),U.95) × di	toe	
	uirodu	∠ <sub>toe</sub> = <b>284</b> r			m <sup>2</sup> /m		
Area or tension reinforcement requ	ment:	As_toe_des =	$\frac{1}{1000} / (U.87 \times 1)^{-1}$	$\times 2$ toe) = 1115 mn	11 7111		
	ured:	$A_{s_{toe_{min}}} =$	$\mathbf{x} \times \mathbf{D} \times \mathbf{l}_{base} = 4$	) = 11111 <sup>4</sup> /111	mm²/m		
Reinforcement provided:	meu,	As_toe_req =	hars @ 150 m	ns_toe_min) — 11113 [ m centres	11(11 /(11		
Area of reinforcement provided.			1340 mm <sup>2</sup> /m				
, and of the northern provided,	PASS - Reinforcement provided at the retaining wall toe is adequate						
Chack shoar resistance at too							
		$V_{\rm terr} = M_{\rm e}/I$	(h × d) - 0 27	<b>3</b> N/mm <sup>2</sup>			
Allowable shear stress:		vtoe – vtoe /	$V_{\text{toe}} = V_{\text{toe}} / (D \times d_{\text{toe}}) = 0.373 \text{ N/mm}^2$				
Allowable Silear Siless,		vadm – IIIIII( DACC	Design shear $r$	strass is lose th	an mavimum	shoar stross	
From BS8110 Part 1 1997 – Tabl	e 3.8	FAJJ -	Design siledi	311 C33 13 1833 [[]	an maxiillulli	SIITAI SUESS	

Gamble, King and Noone Limited	Project				Job Ref.			
Consulting Structural Engineers & Building Surveyors	New Dwel	ling at Gerrards	Hollow, Gee Cr	oss. SK14 5DT	C23.	.07.32		
5 Queen Street, Oldham. OL1 1RD.	Section				Sheet no./rev.			
Tel – 0161 652 1183		Structura	Structural Calculations			46		
www.gknltd.co.uk	Calc. by	Date	Chk'd by	Date	App'd by	Date		
	S.King	Nov. 2023						
Ref:	С	alculations			Outpu	t		
Design concrete shear stress;		vc_toe = <b>0.5</b>	50 N/mm²					
			Vtoe	< v <sub>c_toe</sub> - No she	ar reinforcen	nent required		
Design of reinforced concrete re	etaining wall	heel (BS 8002:	<u>1994)</u>					
Material properties								
Characteristic strength of concrete	;	f <sub>cu</sub> = <b>30</b> N/r	mm²					
Characteristic strength of reinforce	ement;	f <sub>y</sub> = <b>500</b> N/	mm²					
Base details								
Minimum area of reinforcement;		k = <b>0.13</b> %	,					
Cover to reinforcement in heel;		c <sub>heel</sub> = <b>40</b> n	nm					
Calculate shear for heel design								
Shear from bearing pressure:		V <sub>beel bear</sub> =	(Dheel f + Dstem he	$(1.1) \times  _{\text{heel}} / 2 = 1.0$	6 kN/m			
Shear from weight of base:		Vheel wt base	$= vf d \times vbase \times b$	$\frac{1}{2} \times \frac{1}{2} = \frac{2}{3} k$	V/m			
Shear from weight of moist backfil	l:	Vheel wt m =	$W_{m w f} = 17.6 \text{ k}$	N/m				
Total shear for heel design:	.,	$V_{\text{heel}} = -V$	heel bear + Vheel wt	base + Vheel wt m =	<b>18.4</b> kN/m			
Calculate moment for heel desir	nn							
Moment from bearing pressure:	<b>J</b> 11	Mhaal haar =	$(2 \times D_{bask}) \in \pm D_{add}$		$(12)^2 / 6 = 0$	6 kNm/m		
Moment from weight of base:		M.		t. v(l t	$(2)^2 (2) - 0 = 0$	kNm/m		
Moment from weight of base,	zfill•	M	$e = (\gamma I_d \land \gamma base \land$	t $(1 - 4 - 4 - 4 - 4 - 4 - 4 - 4 - 4 - 4 - $	/ 2) / 2) <b>- 0.0</b> m/m			
Total moment for heel design:	XIIII,	M – – N	- wm_w_t×(Ineel + 4 + M	$+ M_{1} + M_{2}$	- <b>5</b> kNm/m			
rotal moment for heer design,	<b>↓</b> 150▶	Ivineel – – Iv	Ineel_bear I IVineel_v	vt_base ' lvineel_wt_m	- <b>J</b> KINII/III			
	• •	•	•	• •	•			
350					$\langle \rangle$			
l l l l l l l l l l l l l l l l l l l								
Check heel in bending								
Width of heel;		b = <b>1000</b> n	nm/m					
Depth of reinforcement;		d <sub>heel</sub> = t <sub>base</sub>	– Cheel – (¢heel / 2	2) = <b>302.0</b> mm				
Constant;		K <sub>heel</sub> = M <sub>he</sub>	$_{\rm el}$ / (b × d <sub>heel</sub> <sup>2</sup> × fo	cu) = <b>0.002</b>				
			С	ompression reir	forcement is	not required		
Lever arm;		Z <sub>heel</sub> = min	(0.5 + √(0.25 - (r	nin(K <sub>heel</sub> , 0.225) /	(0.9)),0.95) ×	dheel		
		Z <sub>heel</sub> = <b>287</b>	mm		21			
Area of tension reinforcement requ	Area of tension reinforcement required;			y × Z <sub>heel</sub> ) = <b>40</b> mn	n²/m			
Minimum area of tension reinforce	As_heel_min =	$= \mathbf{K} \times \mathbf{b} \times \mathbf{t}_{\text{base}} = 4$	155 mm²/m	2.				
Area of tension reinforcement requ	As_heel_req =	Max(As_heel_des,	As_heel_min) = <b>455</b>	mm²/m				
Reinforcement provided;		16 mm dia	a.bars @ 150 m	m centres				
Area of reinforcement provided;		As_heel_prov	$A_{s_{heel}prov} = 1340 \text{ mm}^2/\text{m}$					
		PASS - Rein	forcement prov	idea at the retai	ning wall hee	i is adequate		

Gamble, King and Noone Limited	Project				Job Ref.						
Consulting Structural Engineers & Building Surveyors	New Dwel	ling at Gerrards	Hollow, Gee (	Cross. SK14 5DT	C23.07.32						
5 Queen Street, Oldham. OL1 1RD.	Section				Sheet no./rev.						
Tel – 0161 652 1183		Structura	l Calculations			47					
www.gknltd.co.uk	Calc. by	Date	Chk'd by	Date	App'd by	Date					
	S.King	Nov. 2023									
Ref:	C	alculations			Outpu	ıt					
Check shear resistance at heel											
Design shear stress;		$v_{heel} = V_{hee}$	$_{\rm I}$ / (b × d <sub>heel</sub> ) =	<b>0.061</b> N/mm <sup>2</sup>							
Allowable shear stress;		v <sub>adm</sub> = min	$(0.8 \times \sqrt{f_{cu}} / 1)$	N/mm <sup>2</sup> ), 5) × 1 N/n	nm² = <b>4.382</b> N	l/mm²					
		PASS	Design shea	ar stress is less th	an maximun	n shear stress					
From BS8110:Part 1:1997 – Tabl	e 3.8										
Design concrete shear stress;		Vc_heel = <b>0.</b>	550 N/mm²								
			Vhe	el < Vc_heel - No she	ear reinforce	ment required					
Design of reinforced concrete re	taining wall	stem (BS 8002	:1994 <u>)</u>								
Material properties											
Characteristic strength of concrete	;	f <sub>cu</sub> = <b>30</b> N/r	mm²								
Characteristic strength of reinforce	ment;	f <sub>y</sub> = <b>500</b> N/	mm²								
Wall details											
Minimum area of reinforcement;	Minimum area of reinforcement;										
Cover to reinforcement in stem;	Cover to reinforcement in stem;										
Cover to reinforcement in wall;		c <sub>wall</sub> = <b>40</b> m	c <sub>wall</sub> = <b>40</b> mm								
Factored horizontal at-rest forces on stem											
Moist backfill above water table; $F_{s_ma_f} = 0.5 \times \gamma_{f_e} \times K_0 \times \gamma_m \times (h_{eff} - t_{base} - d_{ds} - h_{sat})^2 = 89.1 \text{ kN/m}$											
Calculate shear for stem design											
Shear at base of stem;		V <sub>stem</sub> = F <sub>s_r</sub>	m_a_f - F <sub>prop_f</sub> =	<b>26.8</b> kN/m							
Calculate moment for stem desi	gn										
Moist backfill above water table;	-	Ms_m_a = Fs	s_m_a_f × (2 × h	<sub>sat</sub> + h <sub>eff</sub> - d <sub>ds</sub> + t <sub>base</sub>	/ 2) / 3 = 119	<b>.6</b> kNm/m					
Total moment for stem design;		M <sub>stem</sub> = M <sub>s</sub>	_m_a = <b>119.6</b> kl	Nm/m							
					$\langle \rangle$						
	• •	•	•	• •	•						
<b>↓</b>											
	<b>∢</b> —150— <b>▶</b>										
Check wall stem in bending		h - 1000 m	m/m								
Depth of reinforcement:		deters = ture		(2) - 300.0  mm							
Constant:		Kotom = Mat	$v_{\text{stem}} = (\psi_{\text{stem}})^2$	$x f_{ev}$ = 0.044							
				Compression rei	nforcement i	s not required					
Lever arm:	Z <sub>stem</sub> = min	(0.5 + √(0.25 -	(min(K <sub>stem</sub> , 0.225)	(0,9)).0.951 >	< d <sub>stem</sub>						
	Z <sub>stem</sub> = 284	$z_{\text{sterm}} = -1111(0.3 + 3(0.23 - (1111)(1\sterm, 0.223) / 0.3)), 0.33) \times 0.330$									
Area of tension reinforcement reau	Area of tension reinforcement required:				As stem des = $M_{stem} / (0.87 \times f_v \times z_{stem}) = 966 \text{ mm}^2/\text{m}$						
Minimum area of tension reinforce	Minimum area of tension reinforcement			As stem min = $k \times b \times t$ wall = 455 mm <sup>2</sup> /m							
Area of tension reinforcement requ	, iired;	$A_{s\_stem\_req} = Max(A_{s\_stem\_des}, A_{s\_stem\_min}) = 966 \text{ mm}^2/\text{m}$									
•		1	, <u> </u>								

Gamble, King and Noone Limited	Project				Job Ref.		
Consulting Structural Engineers & Building Surveyors	New Dwelling at Gerrards Hollow, Gee Cross. SK14 5DT				C23.07.32		
5 Queen Street, Oldham. OL1 1RD.	Section Structural Calculations				Sheet no./rev. 48		
Tel – 0161 652 1183							
www.gknltd.co.uk	Calc. by	Date	Chk'd by	Date	App'd by	Date	
	S.King	Nov. 2023					
Ref:	Calculations				Output		
Reinforcement provided;	20 mm dia.bars @ 150 mm centres						
Area of reinforcement provided;	A <sub>s_stem_prov</sub> = <b>2094</b> mm <sup>2</sup> /m						
PASS - Reinforcement provided at the retaining wall stem is adequate							
Check shear resistance at wall s	tem						
Design shear stress;	v <sub>stem</sub> = V <sub>stem</sub> / (b × d <sub>stem</sub> ) = <b>0.089</b> N/mm <sup>2</sup>						
Allowable shear stress;	$v_{adm}$ = min(0.8 × $\sqrt{(f_{cu} / 1 N/mm^2)}$ , 5) × 1 N/mm <sup>2</sup> = <b>4.382</b> N/mm <sup>2</sup>						
		PASS	- Design sh	ear stress is less th	nan maximu	m shear stress	
From BS8110:Part 1:1997 – Table	ə 3.8						
Design concrete shear stress;	= <b>0.640</b> N/mm <sup>2</sup>						
			Vst	<sub>tem</sub> < v <sub>c_stem</sub> - No she	ear reinforc	ement required	
Check retaining wall deflection							
Basic span/effective depth ratio;		ratio <sub>bas</sub> = 7	7				
Design service stress;		$f_s = 2 \times f_y$	$f_s = 2 \times f_y \times A_{s\_stem\_req} / (3 \times A_{s\_stem\_prov}) = 153.8 \text{ N/mm}^2$				
Modification factor; fac	$ctor_{tens} = min(0.55 + (477 N/mm^2 - f_s)/(120 \times (0.9 N/mm^2 + (M_{stem}/(b \times d_{stem}^2)))),2) = 1.76$						
Maximum span/effective depth ratio	o;	ratio <sub>max</sub> =	ratio <sub>max</sub> = ratio <sub>bas</sub> × factor <sub>tens</sub> = <b>12.31</b>				
Actual span/effective depth ratio;	ratio <sub>act</sub> = h	ratio <sub>act</sub> = h <sub>stem</sub> / d <sub>stem</sub> = <b>11.67</b>					
				PASS - Span t	o depth rati	o is acceptable	

