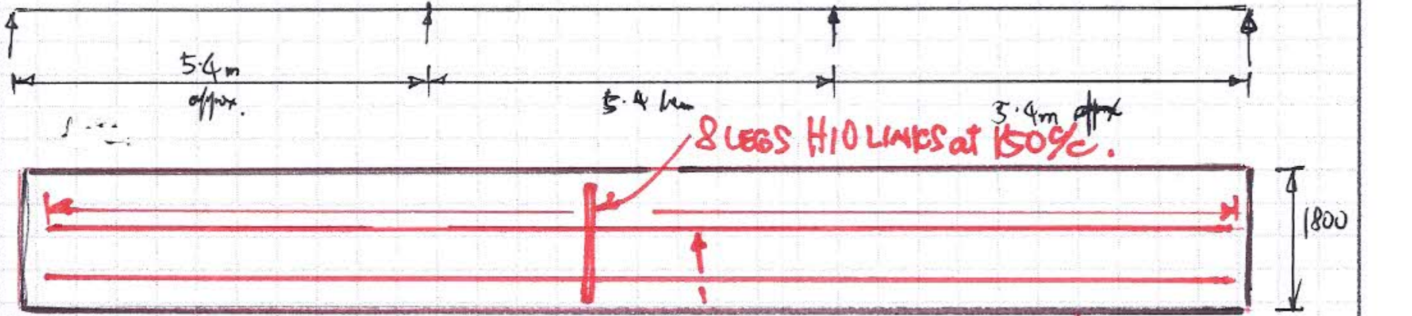
CHARTERED ENGINEERS
BUILDING
DESIGN
CONSULTANTS

INDEX OF CONTENTS, SUMMARY



SCHEMATIC REINFORCEMENT SKETCH

Calculations

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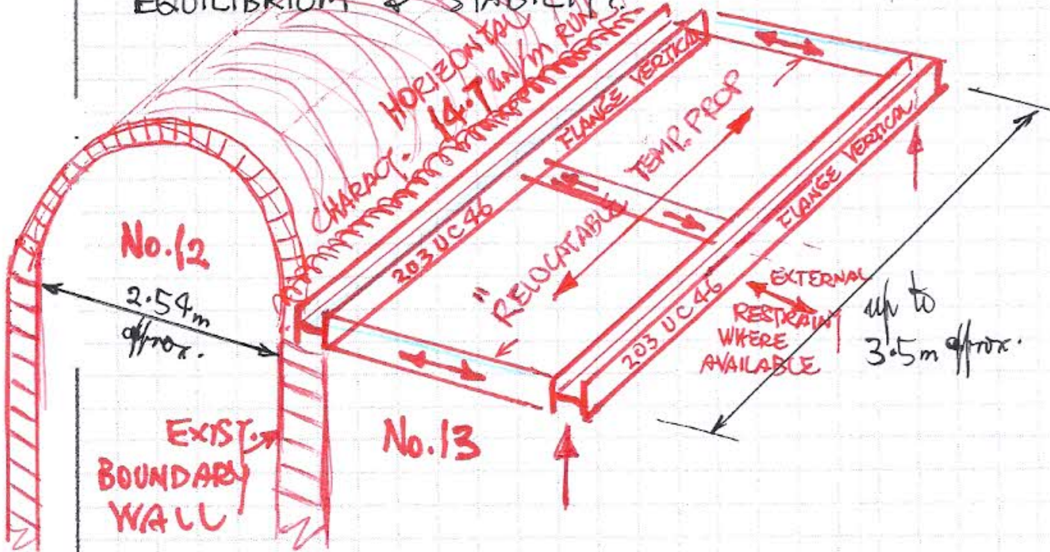

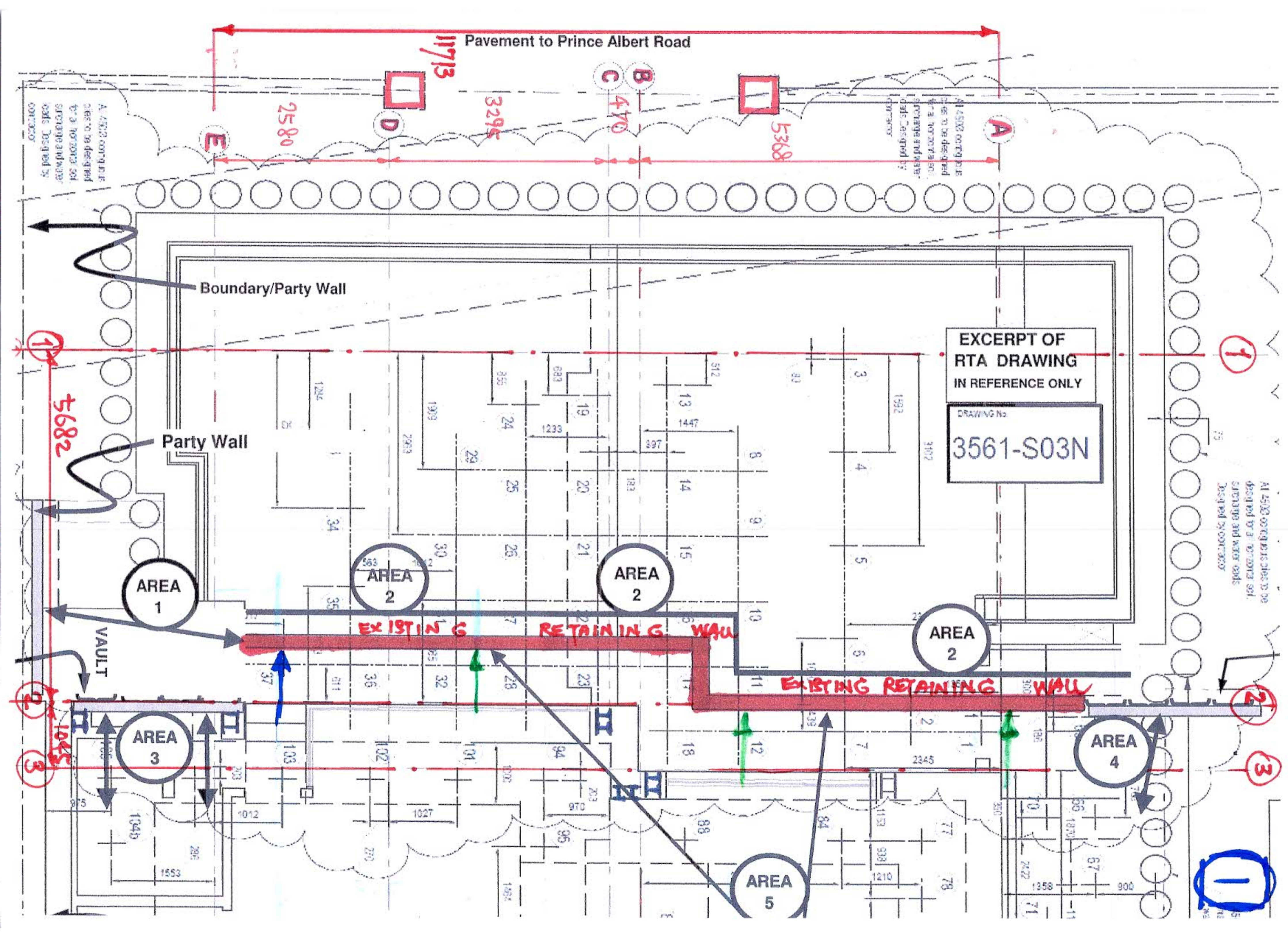
Pavement to Prince Albert Road
11713
5368
470
3295
2580
5682
1045
Boundary/Party Wall
Party Wall
VAULT
EXCERPT OF
RTA DRAWING
IN REFERENCE ONLY
DRAWING No.
3561-S03N
AREA 1
AREA 2
AREA 3
AREA 4
AREA 5
EXISTING RETAINING WALL
EXISTING RETAINING WALL
All 450Ø contiguous piles to be designed for all horizontal soil, surcharge and water loads. Designed by contractor
All 450Ø contiguous piles to be designed for all horizontal soil, surcharge and water loads. Designed by contractor
All 450Ø contiguous piles to be designed for all horizontal soil, surcharge and water loads. Designed by contractor

$18 \times 4 \times 0.35 = 25.2$ kN/m

1:20

10 ϕ at 200 c/c. gives 392 mm² per metre width

$\frac{392}{1000 \times 210} = 0.0019$

2

WT & L CHARTERED BUILDING DESIGN CONSULTANTS

Calculations

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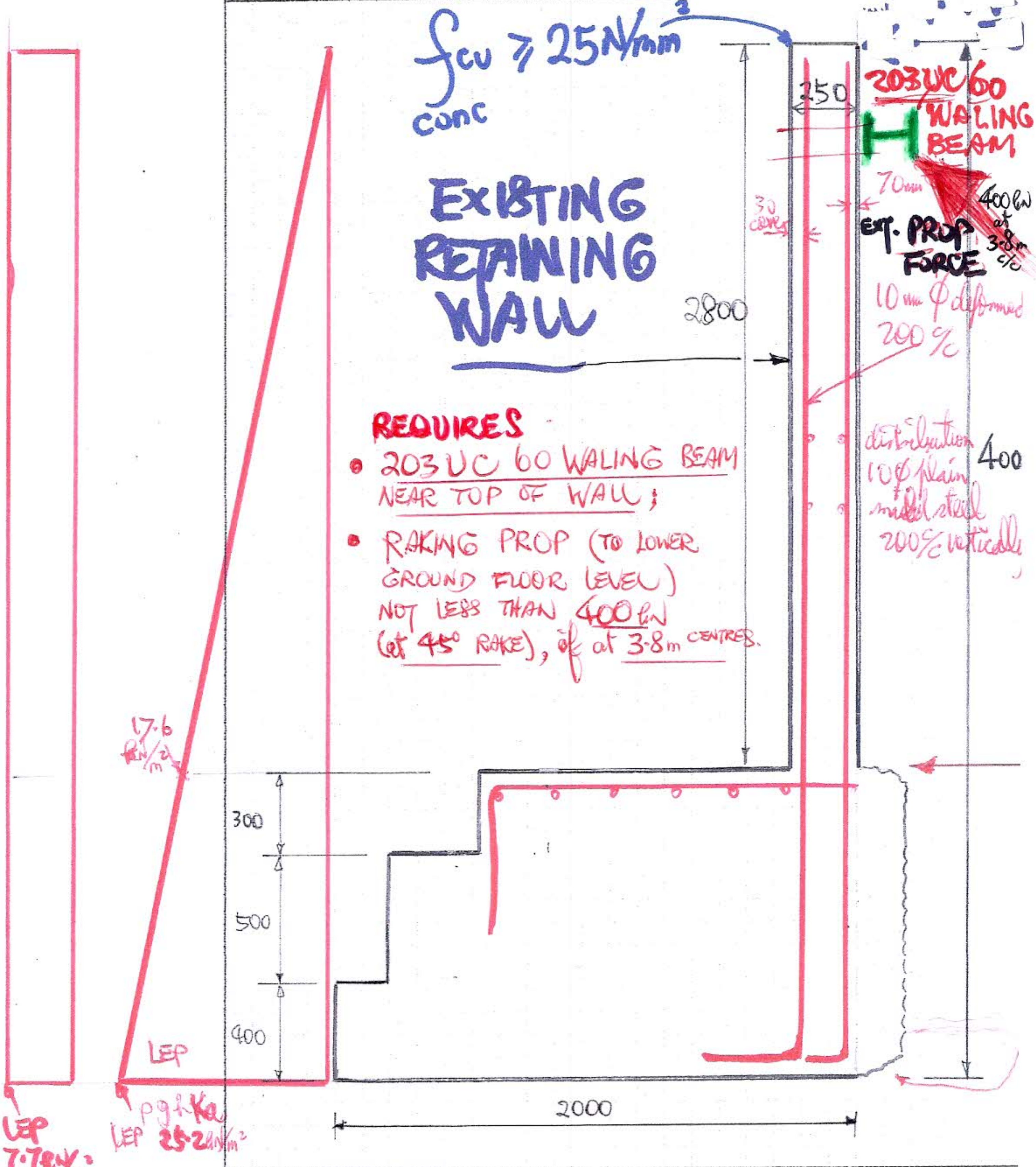


VERTICAL SURCHARGE PRESS under pad = 22 kN/m² $\frac{100 kN}{2.7m \times 1.7m}$

$22 \times 0.35 = 7.7$ kN/m²
25.2

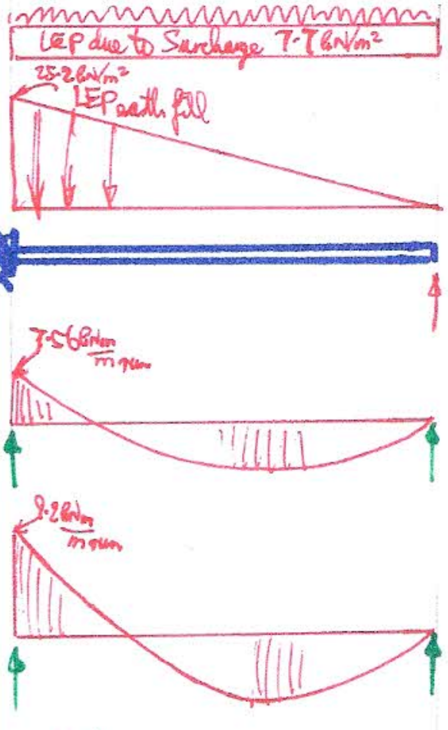
take lateral earth pressure coefficient = 0.35

$22 \times 0.35 = 7.7$ kN/m²

ALL LOAD & LOAD EFFECT. CHARACTERISTIC or WORKING CONDITION

P CE	HORIZONTAL FORCE PER METRE RUN OF WALL	OVERTURNING MOMENT PER METRE RUN OF WALL
(SU)RCHAR GE	$2.8_m \times 1_m \times 7.7\,kN/m^2$ (ht. width) $= 21.6\,kN$	$21.6\,kN \times 1.4_m = 30\,kNm/m$ run.
(LA)TERAL EP. FILL.	$\frac{1}{2} \times 17.6 \times 2.8_m \times 1m = 24.64\,kN$	$24.64\,kN \times \frac{2.8}{3}m = 23\,kNm/m$ run.
Sum	$46.2\,kN \rightarrow$ per m. run of wall.	$53\,kNm$ /m run of wall.
(a)t 2.8m (b)elow top	HORIZONTAL Shearing $\frac{46.2 \times 10^3}{1000 \times 210} = 0.22\,N/mm^2$ permissible shearing stress is $0.8\,N/mm^2$ ∴ OK.	BENDING M/bd^2 $\frac{53 \times 10^6}{1000 \times 210^2} = 1.2$ | $\frac{23 \times 10^6}{1000 \times 210^2}$ $\cdot 0.52$ requires $0.0020 \times 1000 \times 210 = 420\,mm^2/m$ wall. compressive stress in concrete $= 7.5\,N/mm^2$ (less than $10\,N/mm^2$ OK) High yield main reinf. has been provided in fill face of wall. If working tensile stress is at $220\,N/mm^2$, required area of reinf. $0.006 \times 1000 \times 210 = 1260\,N/mm^2$ or even H16 at 200 gives $1005\,mm^2/m$
SURCHARGE	$4_m \times 1_m \times 7.7\,kN/m^2$ (ht. width) $= 30.8\,kN/m$ run	$30.8\,kN \times 2_m = 61.6\,kNm/m$ O/T moment.
LATERAL EARTH PRESS.	$\frac{1}{2} \times 25.2 \times 4_m^2 = 50.4\,kN/m$ run	$50.4\,\frac{kN}{m} \times \frac{4}{3}m = 67.2\,kNm$ /m run.
SUM	$81.2\,kN/m$ run HORIZONTAL PUSH	overturning moment $61.6 + 67.2 = 128.8\,kNm/m$ run.
	RESISTANCE TO SLIDING. $4_m \times 2_m \times 1_m \times 19\frac{kN}{m^3} \times 0.5\,\mu$ (ht. WIDTH LENGTH) $= 76\,kN$ per m. run. JUST OK particularly when IMPOSED LOAD adds to RESIS	RESTORING MOMENT $4_m \times 2_m \times 1_m \times 19\frac{kN}{m^3} \times 1m$ + FRICTION at FILL INTERFACE. $152\,kNm/m$ run + INTERFACE FRICTION.

IF EXTERNAL PROPPING IS APPLIED avoid Overall stability & large F.E.M. @ root of wall



$\frac{M}{bd^2} = \frac{16.7 \times 10^6}{1000 \times 210^2} = 0.379$

$\sigma_{st} = 208\,N/mm^2$

$f_c = 3.8\,N/mm^2$

OK.

4

LATER. PRESS. due to SURCHARGE

Wed. 26th November 2014

Surcharge

REP 7.7 kN/m² x 2.8 m x 1 m = 21.6 kN

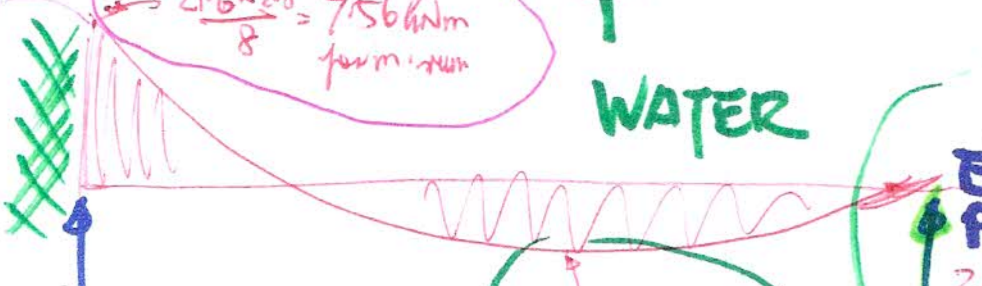
2.8 metre

take external Prop force to be 11 kN/m run

$10 \times 2.8 = 28$ kN/m² total H force

$\frac{28}{2} \times 2.8 = 39.2$ kN

$\frac{21.6 \times 2.8}{8} = 7.56$ kNm per m. run



+ WATER

$\frac{5 \times 21.6}{8} = 13.5$ kN per m. run

$\frac{21.6 \times 2.8}{14.2} = 4.26$ kNm per m. width

EXTERN. PROP FORCE at top of Wall

$\frac{3 \times 21.6}{8} = 8.1$ kN per metre run.

take ext. prop force $= \frac{39.2}{3} = 13.1$ kN/m run

cumulative shear stress

$\frac{(13.5 + 14.7) \times 10^3}{1000 \times 210} = 0.16$ N/mm² in wall below 0.9 N/mm² shear OK.

+ EARTH

EARTH

Lateral pressure. $\frac{1}{2} \times 17.6 \text{ kN/m}^3 \times 2.8_m^2 = 24.64$ kN

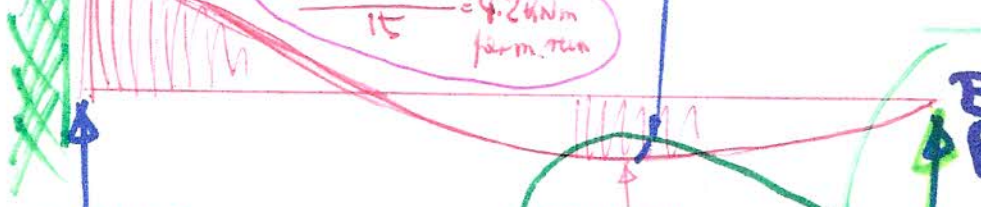
Sum B.M. in span or EXPROP FORCE 8.4 kNm/m

$\frac{M}{bd^2} = \frac{8.4 \times 10^6}{1000 \times 210^2} = 0.19$

tens. stress in reinf. over exprop force 110 N/mm²

EP 2.8×0.35 9.6 kN/m² LATER. FORCE Per metre run of wall. $\frac{9.6}{2} \times 2.8 \times 1$ kN $= 27.4$ kN

$\frac{2 \times 24.64 \times 2.8}{15} = 9.2$ kNm per m. run



EXTERNAL PROP FORCE at top of Wall

0.8×24.64 kN $= 19.7$ kN

$\frac{24.64 \times 2.8}{16.7785} = 4.1$ kNm per m. run

0.2×24.64 kN $= 4.93$ kN per metre run

take external prop force $\frac{27.4}{3} = 9.2$ kN/m.

Sum B.M. moment at lower end of Retaining Wall: 16.7 kNm/m run

$\frac{M}{bd^2} = \frac{16.7 \times 10^6}{1000 \times 210^2} = 0.379$ FOR $\rho = 0.0019$ $\sigma_{steel} = 208$ N/mm² OK.

HORIZONTAL PROPPING FORCE at top of existing Ret. Wall 13 kN per m. run

Calculations — WT&L Chartered Engineers, Building Design Consultants — 5

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EXTERNAL PROP FORCE AT TOP OF EXISTING RETAINING WALL

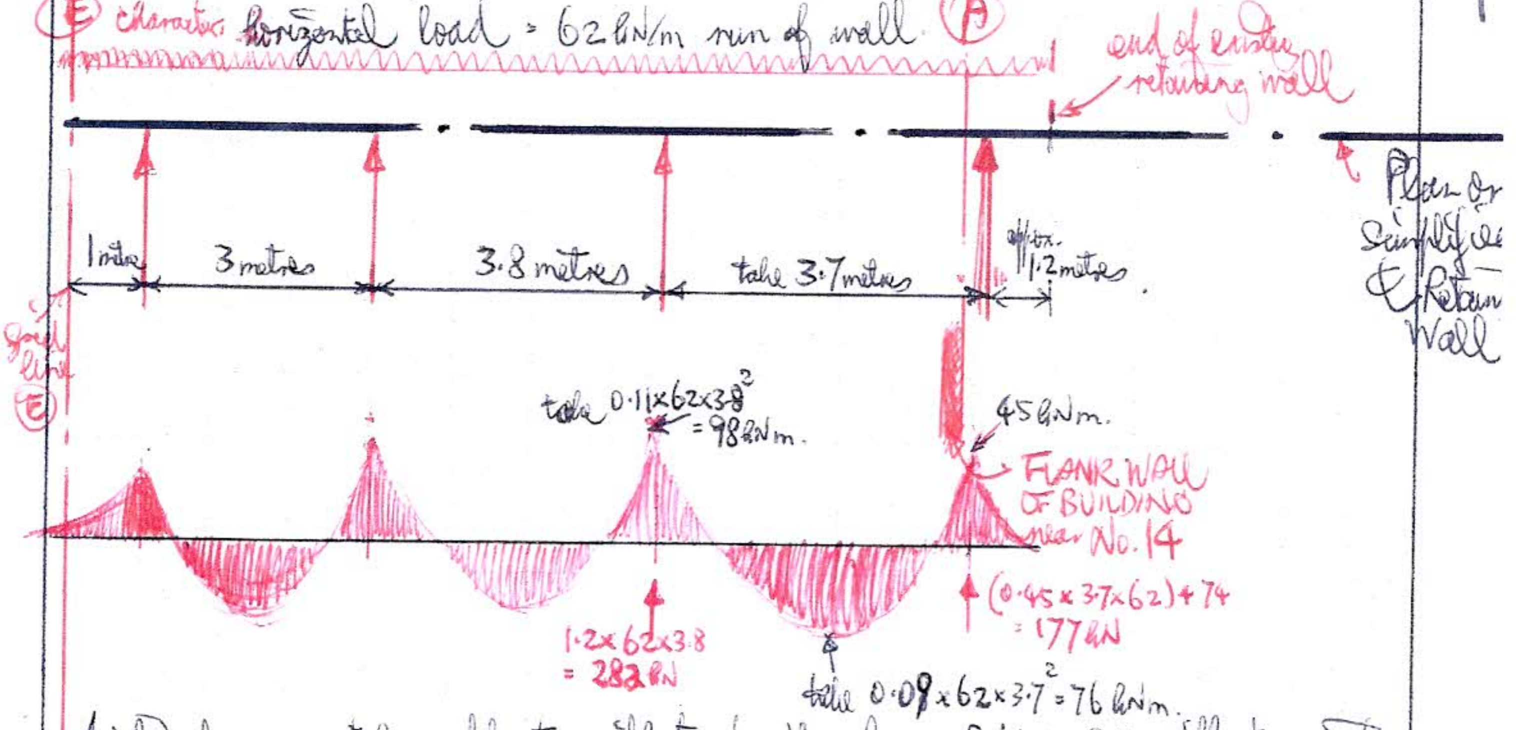
from previous page(s) on assessing HORIZONTAL FORCES TO BE RESISTED BY THE EXISTING Ret. Wall

High level propping force required

$$\text{take}\left(\frac{4\text{ metres}}{2.8\text{ metres}}\right)^2 \times \left(\underset{\text{LEP due to SURCHARGE}}{8\,\tfrac{kN}{m}} + \underset{\text{Horizontal Water Pressure}}{13.1\,\tfrac{kN}{m}} + \underset{\text{LATERAL EARTH PRESS.}}{9.2\,\tfrac{kN}{m}}\right) = 2.04 \times 30.3$$

$= 62$ kN per metre run

LOCATION OF THE RAKING PROPS TO THE WALING BEAM IS RESTRICTED BY WINDOW POSITION IN THE LOWER GROUND STOREY.



Waling beam must be able to withstand the above B.M. & S.F. without assistance (composite action resistance) from existing R.C. wall). Take span length = 3.8 metres.

$\boxed{\text{USE 203UC60}}$: $l/r_{yy} = (3800 \div 52) = 73$; $\frac{D}{T} = \frac{209}{14.2} = 14.7$; $P_{bc} = 167\ N/mm^2$

$\frac{M}{Z_x} = \frac{98 \times 10^6}{584 \times 10^3} = 167\ N/mm^2$ OK

RAKING PROP THROUGH EACH OPENING IN LOWER GROUND STOREY TO HAVE SAFETY COMPRESSION CAPACITY $\geq$ 400 kN (working)

1:50

MAIN RETAINING WALL PARALLEL TO & UNDERNEATH PAVEMENT ALONG PRINCE ALBERT ROAD.

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Calculations

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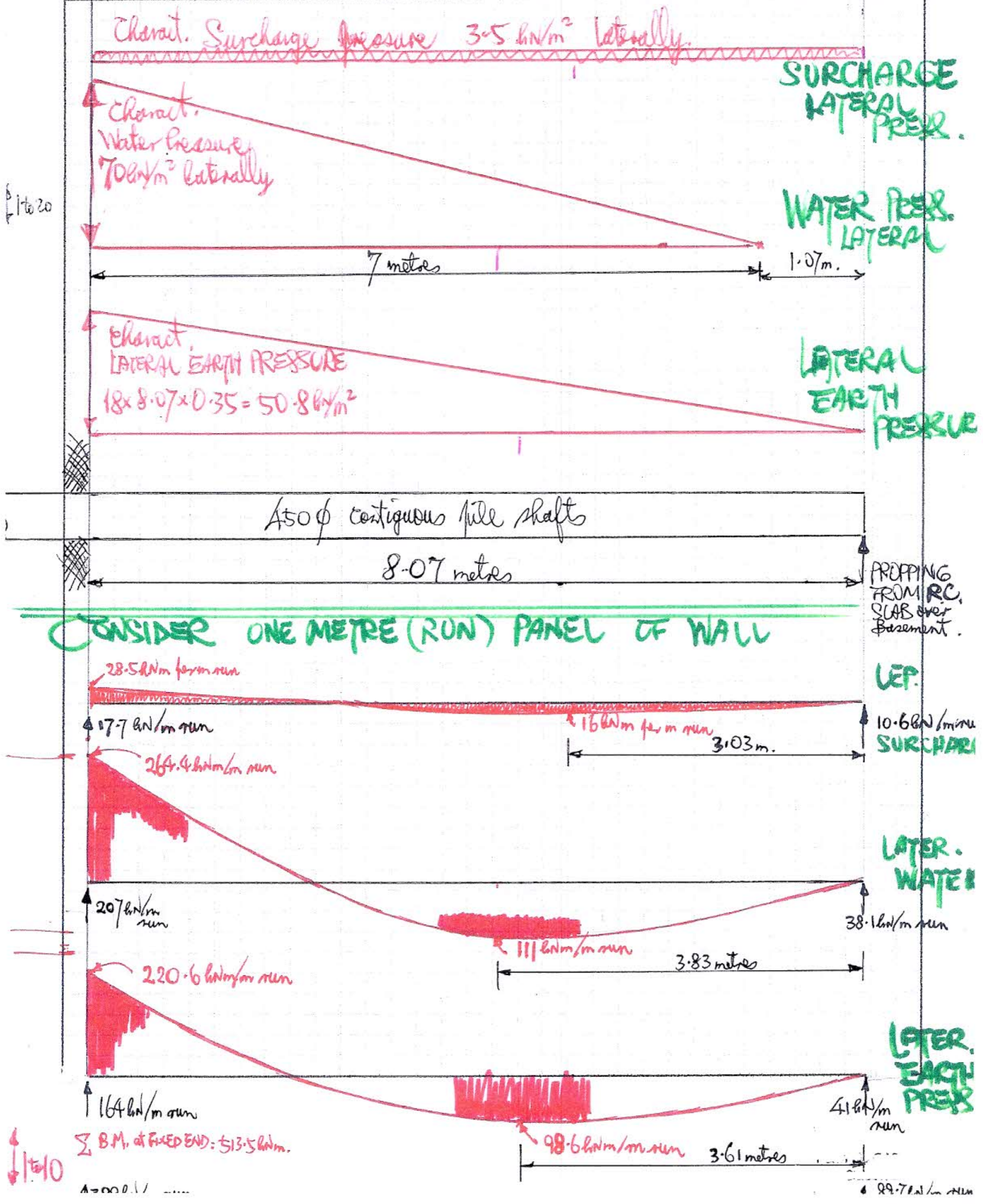
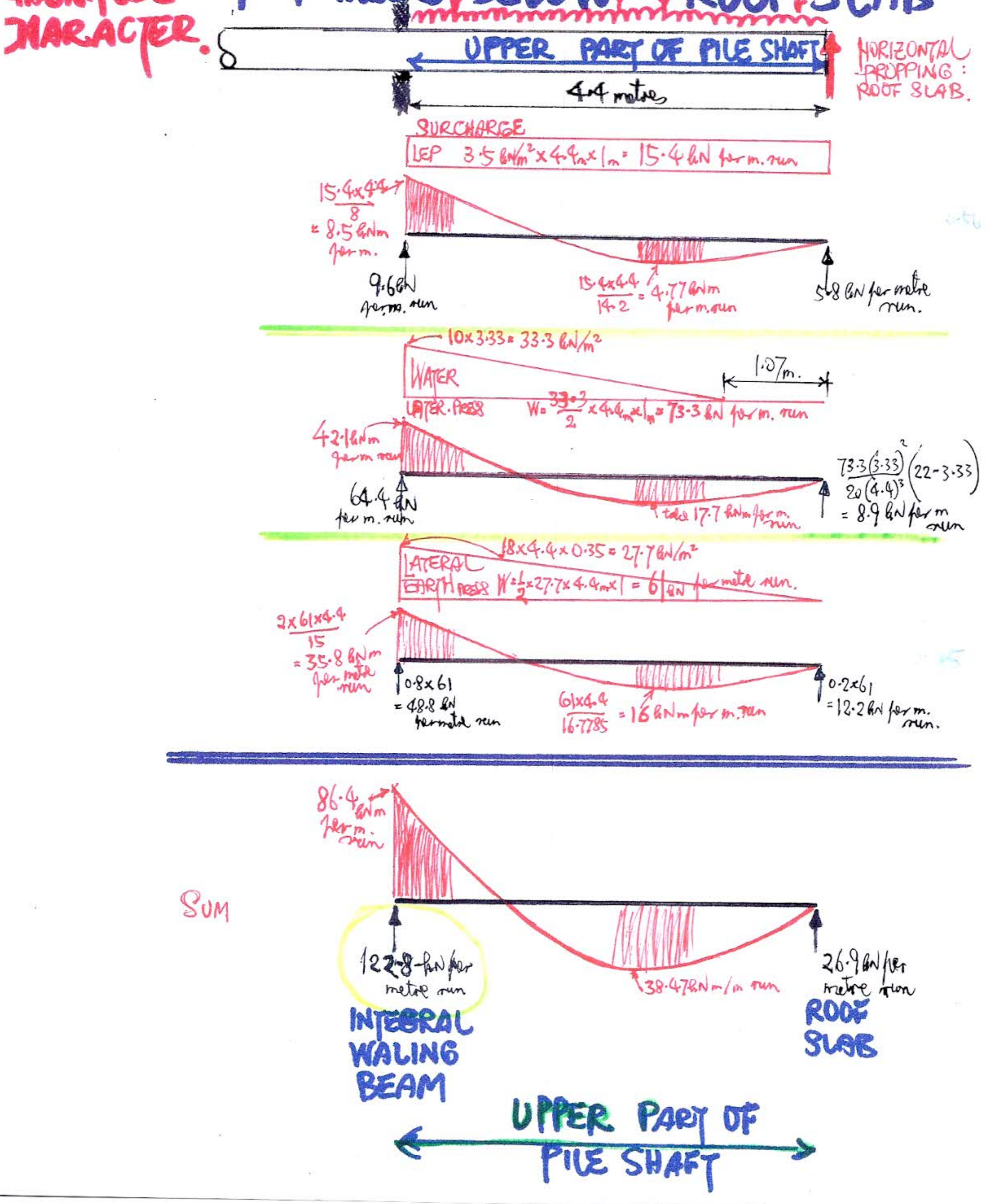


STAGE 1 "TOP-DOWN"

INTEGRAL WALING BEAM AT 4·4 metres BELOW "ROOF SLAB"

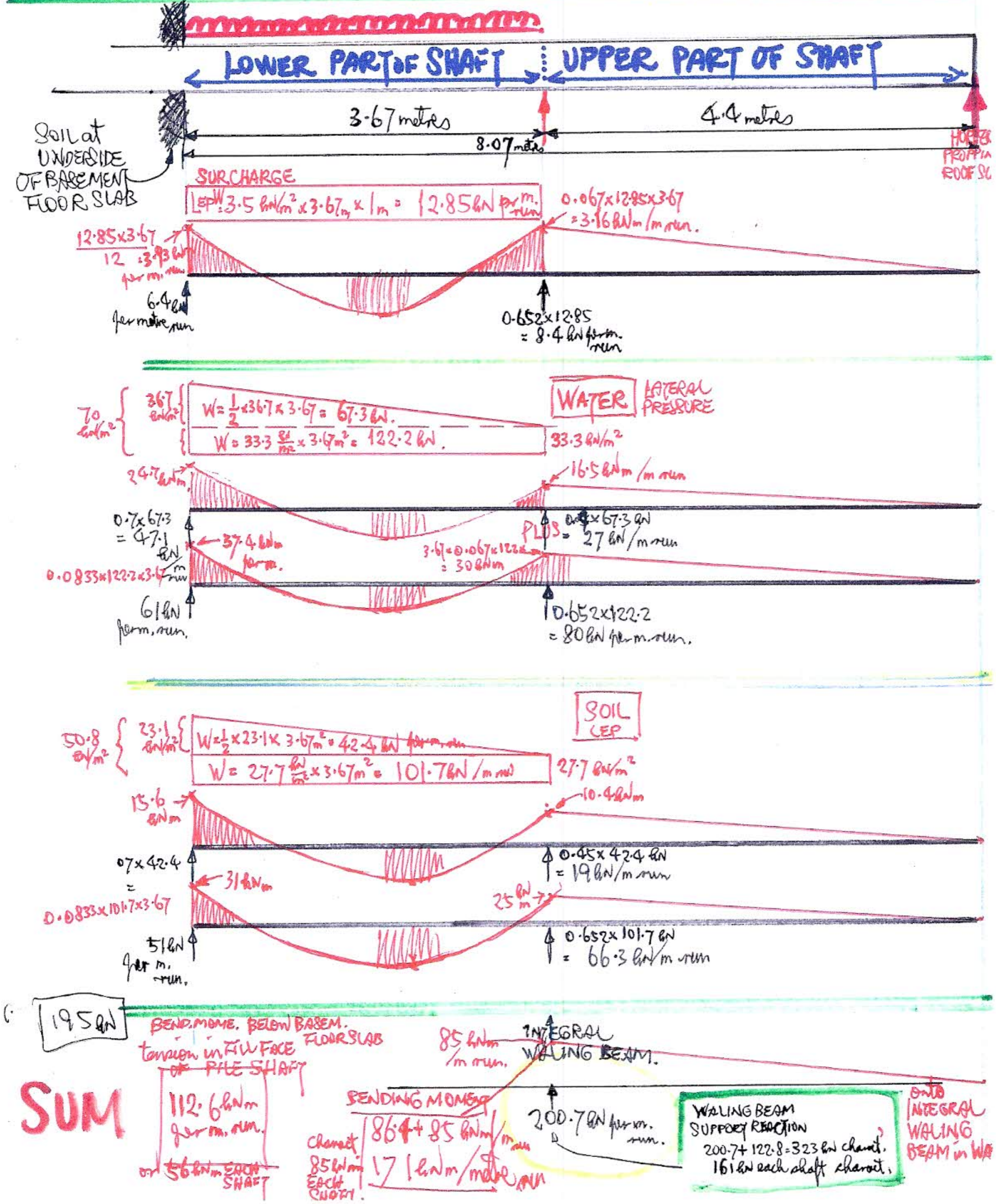
ALL LOAD MAGNITUDE CHARACTER.



STAGE 2 "TOP-DOWN" 8

EXCAVATION BELOW INTEGRAL WALING BEAM

REGION / ZONE 4·4m to 8·07m BELOW "ROOF SLAB



1.4

WT&L CHARTERED ENGINEERS BUILDING DESIGN CONSULTANTS	Project 13 PRINCE ALBERT ROAD			Job Ref. 2014-138	9
	Section MAIN BASEMENT LATERAL LOAD ON PILE SHAFT FRONT WALL			Sheet no./rev.	
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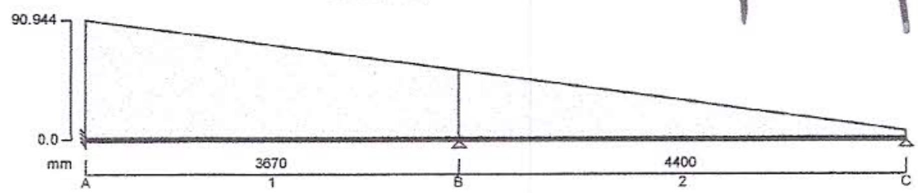
Characteristic
Water 35 kN/m span, EACH SHAFT
LEP 25.4 kN/m span
60.4×1.4 = 84 kN/m span at ULS.

RC BEAM ANALYSIS & DESIGN (BS8110) ULTIMATE LIMIT STATE

RC BEAM ANALYSIS & DESIGN BS8110 OUTPUT FOR ONE PILE SHAFT ONLY.

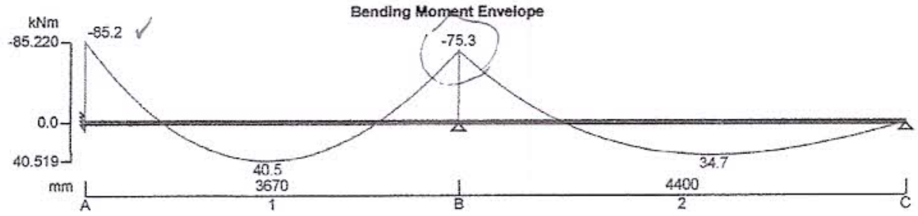
TEDDS calculation version 2.1.11

Load Envelope - Combination 1

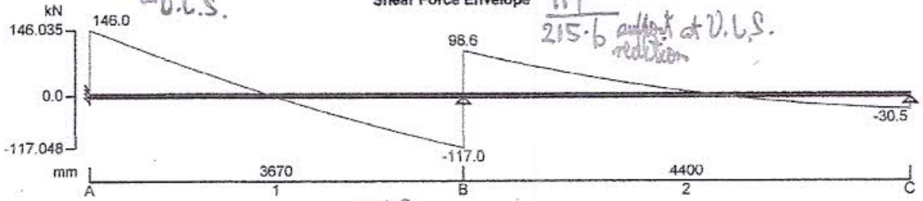


LOAD MAGNITUDE
B.M.
S.F.
all at
U.L.S.

Bending Moment Envelope



Shear Force Envelope



216 kN reaction at U.L.S.

152 kN reaction Characteristic.

35
84
119 kN/m

Support conditions

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

Applied loading

Dead self weight of beam × 1
Dead full VDL 60.5 kN/m to 0 kN/m characteristic magnitude on ONE pile shaft
Imposed full UDL 1.75 kN/m characteristic surcharge LEP component

56
23
89 kN/m each shaft

Load combinations

Load combination 1	Support A	Dead × 1.40
		Imposed × 1.60
	Span 1	Dead × 1.40

216

		Imposed × 1.60
	Support B	Dead × 1.40
		Imposed × 1.60
	Span 2	Dead × 1.40
		Imposed × 1.60
	Support C	Dead × 1.40
		Imposed × 1.60

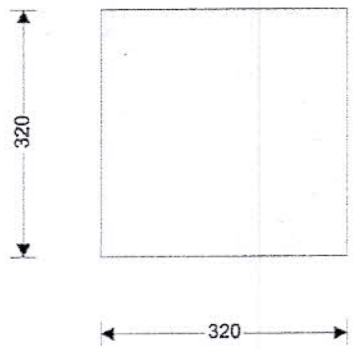
Analysis results

Maximum moment support A;	M_{A_max} = **-85** kNm;	M_{A_red} = **-85** kNm;	ULTIMATE at LIMIT STATE
Maximum moment span 1 at 1791 mm;	M_{s1_max} = **41** kNm;	M_{s1_red} = **41** kNm;	
Maximum moment support B;	M_{B_max} = **-75** kNm;	M_{B_red} = **-75** kNm;	at U.L.S.
Maximum moment span 2 at 2512 mm;	M_{s2_max} = **35** kNm;	M_{s2_red} = **35** kNm;	
Maximum moment support C;	M_{C_max} = **0** kNm;	M_{C_red} = **0** kNm;	
Maximum shear support A;	V_{A_max} = **146** kN;	V_{A_red} = **146** kN	
Maximum shear support A span 1 at 270 mm;	$V_{A\_s1\_max}$ = **122** kN;	$V_{A\_s1\_red}$ = **122** kN	
Maximum shear support B;	V_{B_max} = **-117** kN;	V_{B_red} = **-117** kN	
Maximum shear support B span 1 at 3407 mm;	$V_{B\_s1\_max}$ = **-103** kN;	$V_{B\_s1\_red}$ = **-103** kN	
Maximum shear support B span 2 at 263 mm;	$V_{B\_s2\_max}$ = **85** kN;	$V_{B\_s2\_red}$ = **85** kN	
Maximum shear support C;	V_{C_max} = **-30** kN;	V_{C_red} = **-30** kN	
Maximum shear support C span 2 at 4130 mm;	$V_{C\_s2\_max}$ = **-28** kN;	$V_{C\_s2\_red}$ = **-28** kN	
Maximum reaction at support A;	R_A = **146** kN		
Unfactored dead load reaction at support A;	R_{A_Dead} = **101** kN	} 104 kN characteristic at below basement floor level.	
Unfactored imposed load reaction at support A;	$R_{A\_Imposed}$ = **3** kN		
Maximum reaction at support B;	R_B = **216** kN		
Unfactored dead load reaction at support B;	R_{B_Dead} = **144** kN	} 152 kN characteristic at waling beam level	
Unfactored imposed load reaction at support B;	$R_{B\_Imposed}$ = **8** kN		
Maximum reaction at support C;	R_C = **30** kN		
Unfactored dead load reaction at support C;	R_{C_Dead} = **18** kN		
Unfactored imposed load reaction at support C;	$R_{C\_Imposed}$ = **3** kN		

Rectangular section details

Section width; b = **320** mm

Section depth; h = **320** mm



Concrete details

Concrete strength class; **C40/50**

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

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Modulus of elasticity of concrete; $E_c = 20\text{kN/mm}^2 + 200 \times f_{cu} = \mathbf{30000}\ \text{N/mm}^2$

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

Reinforcement details

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

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

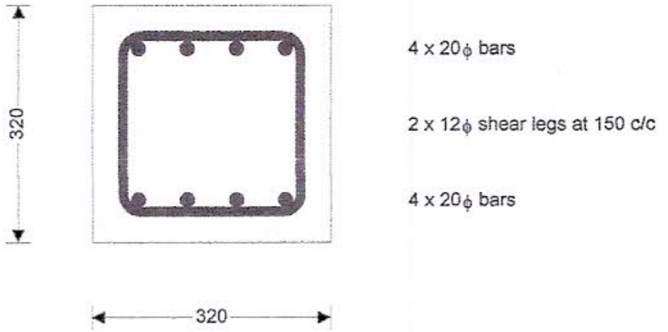
Nominal cover to reinforcement

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

Nominal cover to bottom reinforcement; $c_{nom\_b} = \mathbf{35}$ mm

Nominal cover to side reinforcement; $c_{nom\_s} = \mathbf{35}$ mm

Support B



Design moment resistance of rectangular section (cl. 3.4.4)

Design bending moment; $M = \text{abs}(M_{B\_red}) = \mathbf{75}$ kNm

Depth to tension reinforcement; $d = h - c_{nom\_t} - \phi_v - \phi_{top} / 2 = \mathbf{263}$ mm

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

$K = M / (b \times d^2 \times f_{cu}) = \mathbf{0.068}$

$K' = 0.156$

K' > K - No compression reinforcement is required

Lever arm; $z = \min(d \times (0.5 + (0.25 - K / 0.9)^{0.5}), 0.95 \times d) = \mathbf{241}$ mm

Depth of neutral axis; $x = (d - z) / 0.45 = \mathbf{48}$ mm

Area of tension reinforcement required; $A_{s,req} = M / (0.87 \times f_y \times z) = \mathbf{717}\ \text{mm}^2$

Tension reinforcement provided; $4 \times 20\phi$ bars

Area of tension reinforcement provided; $A_{s,prov} = \mathbf{1257}\ \text{mm}^2$

Minimum area of reinforcement; $A_{s,min} = 0.0013 \times b \times h = \mathbf{133}\ \text{mm}^2$

Maximum area of reinforcement; $A_{s,max} = 0.04 \times b \times h = \mathbf{4096}\ \text{mm}^2$

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

Rectangular section in shear

Design shear force span 1 at 3407 mm; $V = \text{abs}(\min(V_{B\_s1\_max}, V_{B\_s1\_red})) = \mathbf{103}$ kN

Design shear stress; $v = V / (b \times d) = \mathbf{1.223}\ \text{N/mm}^2$

Design concrete shear stress; $v_c = 0.79 \times \min(3,[100 \times A_{s,prov} / (b \times d)]^{1/3}) \times \max(1, (400 /d)^{1/4}) \times (\min(f_{cu}, 40) / 25)^{1/3} / \gamma_m$

$v_c = \mathbf{0.938}\ \text{N/mm}^2$

Allowable design shear stress; $v_{max} = \min(0.8\ \text{N/mm}^2 \times (f_{cu}/1\ \text{N/mm}^2)^{0.5}, 5\ \text{N/mm}^2) = \mathbf{5.000}\ \text{N/mm}^2$

PASS - Design shear stress is less than maximum allowable

Value of v from Table 3.7; $0.5 \times v_c < v < (v_c + 0.4\ \text{N/mm}^2)$

Design shear resistance required; $v_s = \max(v - v_c, 0.4\ \text{N/mm}^2) = \mathbf{0.400}\ \text{N/mm}^2$

Area of shear reinforcement required; $A_{sv,req} = v_s \times b / (0.87 \times f_{yv})$ = **294** mm^2/m

Shear reinforcement provided; 2 × 12ϕ legs at 150 c/c

Area of shear reinforcement provided; $A_{sv,prov}$ = **1508** mm^2/m

PASS - Area of shear reinforcement provided exceeds minimum required

Maximum longitudinal spacing; $s_{vl,max} = 0.75 \times d$ = **197** mm

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

Design shear force span 2 at 263 mm; $V = \max(V_{B\_s2\_max}, V_{B\_s2\_red})$ = **85** kN

Design shear stress; $v = V / (b \times d)$ = **1.012** N/mm^2

Design concrete shear stress; $v_c = 0.79 \times \min(3,[100 \times A_{s,prov} / (b \times d)]^{1/3}) \times \max(1, (400 /d)^{1/4}) \times (\min(f_{cu}, 40) / 25)^{1/3} / \gamma_m$

v_c = **0.938** N/mm^2

Allowable design shear stress; $v_{max} = \min(0.8\ N/mm^2 \times (f_{cu}/1\ N/mm^2)^{0.5}, 5\ N/mm^2)$ = **5.000** N/mm^2

PASS - Design shear stress is less than maximum allowable

Value of v from Table 3.7; $0.5 \times v_c < v < (v_c + 0.4\ N/mm^2)$

Design shear resistance required; $v_s = \max(v - v_c, 0.4\ N/mm^2)$ = **0.400** N/mm^2

Area of shear reinforcement required; $A_{sv,req} = v_s \times b / (0.87 \times f_{yv})$ = **294** mm^2/m

Shear reinforcement provided; 2 × 12ϕ legs at 150 c/c

Area of shear reinforcement provided; $A_{sv,prov}$ = **1508** mm^2/m

PASS - Area of shear reinforcement provided exceeds minimum required

Maximum longitudinal spacing; $s_{vl,max} = 0.75 \times d$ = **197** mm

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

Spacing of reinforcement (cl 3.12.11)

Actual distance between bars in tension; $s = (b - 2 \times (c_{nom\_s} + \phi_v + \phi_{top}/2)) /(N_{top} - 1) - \phi_{top}$ = **49** mm

Minimum distance between bars in tension (cl 3.12.11.1)

Minimum distance between bars in tension; $s_{min} = h_{agg} + 5$ mm = **25** mm

PASS - Satisfies the minimum spacing criteria

Maximum distance between bars in tension (cl 3.12.11.2)

Design service stress; $f_s = (2 \times f_y \times A_{s,req}) / (3 \times A_{s,prov} \times \beta_b)$ = **190.2** N/mm^2

Maximum distance between bars in tension; $s_{max} = \min(47000\ N/mm / f_s, 300\ mm)$ = **247** mm

PASS - Satisfies the maximum spacing criteria

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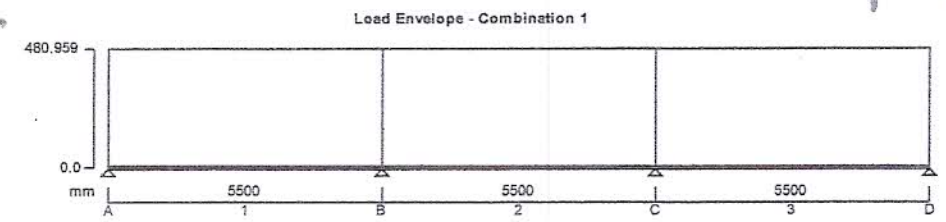
RC BEAM ANALYSIS & DESIGN (BS8110)

INTEGRAL WALING BEAM AT 4.4m BELOW FRONT BASEMENT ROOF SLAB; NEAR PAVEMENT ALONG PRINCE ALBERT ROAD

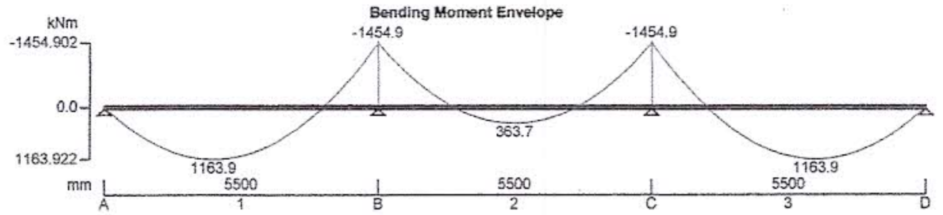
RC BEAM ANALYSIS & DESIGN BS8110

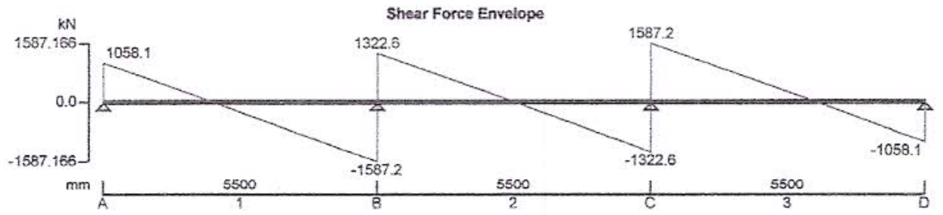
TEDDS calculation version 2.1.11

322kN/m x 1.4



LOAD MAGNIT & RESULTS at ULTIMATE LIMIT STATE.





2078 kN Charact

2078 kN Charact.

Support conditions

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

Applied loading

Dead self weight of beam × 1

Dead full UDL 323 kN/m characteristic $1.4 \times 323 = 453$ kN/m run at ULTIM LIM. STATE

Load combinations

Load combination 1	Support A	Dead × 1.40
		Imposed × 1.60

	Span 1	Dead × 1.40
		Imposed × 1.60
	Support B	Dead × 1.40
		Imposed × 1.60
	Span 2	Dead × 1.40
		Imposed × 1.60
	Support C	Dead × 1.40
		Imposed × 1.60
	Span 3	Dead × 1.40
		Imposed × 1.60
	Support D	Dead × 1.40
		Imposed × 1.60

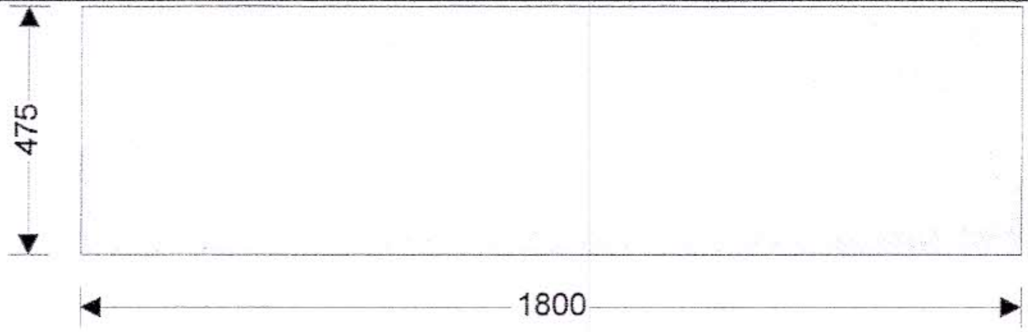
Analysis results

Maximum moment support A;	M_{A_max} = **0** kNm;	M_{A_red} = **0** kNm;
Maximum moment span 1 at 2200 mm;	M_{s1_max} = **1164** kNm;	M_{s1_red} = **1164** kNm;
Maximum moment support B;	M_{B_max} = **-1455** kNm;	M_{B_red} = **-1455** kNm;
Maximum moment span 2 at 2750 mm;	M_{s2_max} = **364** kNm;	M_{s2_red} = **364** kNm;
Maximum moment support C;	M_{C_max} = **-1455** kNm;	M_{C_red} = **-1455** kNm;
Maximum moment span 3 at 3300 mm;	M_{s3_max} = **1164** kNm;	M_{s3_red} = **1164** kNm;
Maximum moment support D;	M_{D_max} = **0** kNm;	M_{D_red} = **0** kNm;
Maximum shear support A;	V_{A_max} = **1058** kN;	V_{A_red} = **1058** kN
Maximum shear support A span 1 at 425 mm;	$V_{A\_s1\_max}$ = **854** kN;	$V_{A\_s1\_red}$ = **854** kN
Maximum shear support B;	V_{B_max} = **-1587** kN;	V_{B_red} = **-1587** kN
Maximum shear support B span 1 at 5086 mm;	$V_{B\_s1\_max}$ = **-1383** kN;	$V_{B\_s1\_red}$ = **-1383** kN
Maximum shear support B span 2 at 414 mm;	$V_{B\_s2\_max}$ = **1118** kN;	$V_{B\_s2\_red}$ = **1118** kN
Maximum shear support C;	V_{C_max} = **1587** kN;	V_{C_red} = **1587** kN
Maximum shear support C span 2 at 5075 mm;	$V_{C\_s2\_max}$ = **-1118** kN;	$V_{C\_s2\_red}$ = **-1118** kN
Maximum shear support C span 3 at 425 mm;	$V_{C\_s3\_max}$ = **1383** kN;	$V_{C\_s3\_red}$ = **1383** kN
Maximum shear support D;	V_{D_max} = **-1058** kN;	V_{D_red} = **-1058** kN
Maximum shear support D span 3 at 5075 mm;	$V_{D\_s3\_max}$ = **-854** kN;	$V_{D\_s3\_red}$ = **-854** kN
Maximum reaction at support A;	R_A = **1058** kN	
Unfactored dead load reaction at support A;	R_{A_Dead} = **756** kN	
Maximum reaction at support B;	R_B = **2910** kN	
Unfactored dead load reaction at support B;	R_{B_Dead} = **2078** kN	
Maximum reaction at support C;	R_C = **2910** kN	
Unfactored dead load reaction at support C;	R_{C_Dead} = **2078** kN	
Maximum reaction at support D;	R_D = **1058** kN	
Unfactored dead load reaction at support D;	R_{D_Dead} = **756** kN	

Rectangular section details

Section width;	b = **1800** mm
Section depth;	h = **475** mm

WT&L Chartered Engineers Building Design Consultants	Project 13 PRINCE ALBERT ROAD			Job Ref. 2014-138	15
	Section HIDDEN WALING BEAM DESIGN ALONG PAVEMENT			Sheet no./rev.	
	Calc. by WT	Date NOV14	Chk'd by GC	Date NOV14	App'd by	Date



Concrete details

Concrete strength class; **C40/50**

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

Modulus of elasticity of concrete; $E_c = 20kN/mm^2 + 200 \times f_{cu}$ = **30000** N/mm^2

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

Reinforcement details

Characteristic yield strength of reinforcement; f_y = **500** N/mm^2

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

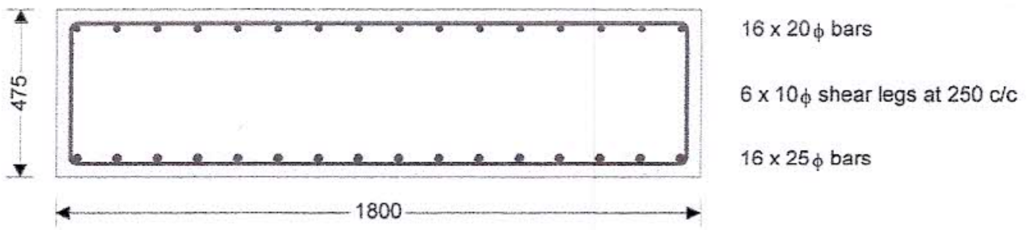
Nominal cover to reinforcement

Nominal cover to top reinforcement; c_{nom_t} = **35** mm

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

Nominal cover to side reinforcement; c_{nom_s} = **35** mm

<u>Mid span 1</u>



Design moment resistance of rectangular section (cl. 3.4.4) - Positive midspan moment

Design bending moment; $M = abs(M_{s1\_red})$ = **1164** kNm

Depth to tension reinforcement; $d = h - c_{nom\_b} - \phi_v - \phi_{bot} / 2$ = **418** mm

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

$K = M / (b \times d^2 \times f_{cu})$ = **0.074**

$K' = 0.156$

K' > K - No compression reinforcement is required

Lever arm; $z = \min(d \times (0.5 + (0.25 - K / 0.9)^{0.5}), 0.95 \times d)$ = **380** mm

Depth of neutral axis; $x = (d - z) / 0.45$ = **84** mm

Area of tension reinforcement required; $A_{s,req} = M / (0.87 \times f_y \times z)$ = **7048** mm^2

Tension reinforcement provided; 16 × 25ϕ bars

Area of tension reinforcement provided; $A_{s,prov}$ = **7854** mm^2

Minimum area of reinforcement; $A_{s,min} = 0.0013 \times b \times h$ = **1112** mm^2

Maximum area of reinforcement; $A_{s,max} = 0.04 \times b \times h$ = **34200** mm^2

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

Rectangular section in shear

Shear reinforcement provided; $6 \times 10\phi$ legs at 250 c/c

Area of shear reinforcement provided; $A_{sv,prov}$ = **1885** mm^2/m

Minimum area of shear reinforcement (Table 3.7); $A_{sv,min} = 0.4N/mm^2 \times b / (0.87 \times f_{yv})$ = **1655** mm^2/m

PASS - Area of shear reinforcement provided exceeds minimum required

Maximum longitudinal spacing (cl. 3.4.5.5); $s_{vl,max} = 0.75 \times d$ = **313** mm

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

Design concrete shear stress; $v_c = 0.79N/mm^2 \times \min(3,[100 \times A_{s,prov} / (b \times d)]^{1/3}) \times \max(1, (400mm / d)^{1/4}) \times (\min(f_{cu}, 40N/mm^2) / 25N/mm^2)^{1/3} / \gamma_m$ = **0.750** N/mm^2

Design shear resistance provided; $v_{s,prov} = A_{sv,prov} \times 0.87 \times f_{yv} / b$ = **0.456** N/mm^2

Design shear stress provided; $v_{prov} = v_{s,prov} + v_c$ = **1.206** N/mm^2

Design shear resistance; $V_{prov} = v_{prov} \times (b \times d)$ = **906.1** kN

Shear links provided valid between 400 mm and 4000 mm with tension reinforcement of 7854 mm^2

Spacing of reinforcement (cl 3.12.11)

Actual distance between bars in tension; $s = (b - 2 \times (c_{nom\_s} + \phi_v + \phi_{bot}/2)) /(N_{bot} - 1) - \phi_{bot}$ = **87** mm

Minimum distance between bars in tension (cl 3.12.11.1)

Minimum distance between bars in tension; $s_{min} = h_{agg} + 5$ mm = **25** mm

PASS - Satisfies the minimum spacing criteria

Maximum distance between bars in tension (cl 3.12.11.2)

Design service stress; $f_s = (2 \times f_y \times A_{s,req}) / (3 \times A_{s,prov} \times \beta_b)$ = **299.1** N/mm^2

Maximum distance between bars in tension; $s_{max} = \min(47000 N/mm / f_s, 300 mm)$ = **157** mm

PASS - Satisfies the maximum spacing criteria

Span to depth ratio (cl. 3.4.6)

Basic span to depth ratio (Table 3.9); $span\_to\_depth_{basic}$ = **26.0**

Design service stress in tension reinforcement; $f_s = (2 \times f_y \times A_{s,req})/ (3 \times A_{s,prov} \times \beta_b)$ = **299.1** N/mm^2

Modification for tension reinforcement

$f_{tens} = \min(2.0, 0.55 + (477N/mm^2 - f_s) / (120 \times (0.9N/mm^2 + (M / (b \times d^2)))))$ = **0.872**

Modification for compression reinforcement

$f_{comp} = \min(1.5, 1 + (100 \times A_{s2,prov} / (b \times d)) / (3 + (100 \times A_{s2,prov} / (b \times d))))$ = **1.182**

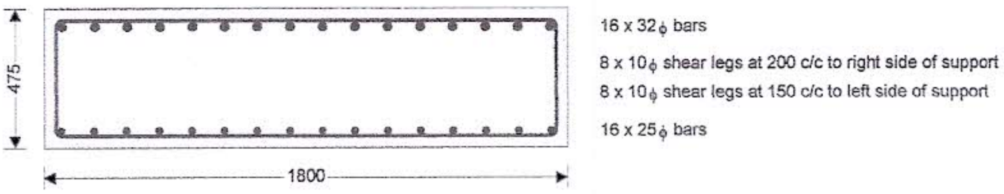
Modification for span length; f_{long} = 1.000

Allowable span to depth ratio; $span\_to\_depth_{allow} = span\_to\_depth_{basic} \times f_{tens} \times f_{comp}$ = **26.8**

Actual span to depth ratio; $span\_to\_depth_{actual} = L_{s1} / d$ = **13.2**

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

<u>**Support B**</u>



Design moment resistance of rectangular section (cl. 3.4.4)

Design bending moment; $M = abs(M_{B\_red})$ = **1455** kNm

Depth to tension reinforcement; $d = h - c_{nom\_t} - \phi_v - \phi_{top} / 2$ = **414** mm

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

$K = M / (b \times d^2 \times f_{cu})$ = **0.094**

$K' = 0.156$

K' > K - No compression reinforcement is required

Lever arm; $z = \min(d \times (0.5 + (0.25 - K / 0.9)^{0.5}), 0.95 \times d) = \mathbf{365}$ mm

Depth of neutral axis; $x = (d - z) / 0.45 = \mathbf{109}$ mm

Area of tension reinforcement required; $A_{s,req} = M / (0.87 \times f_y \times z) = \mathbf{9169}$ mm^2

Tension reinforcement provided; $16 \times 32\phi$ bars

Area of tension reinforcement provided; $A_{s,prov} = \mathbf{12868}$ mm^2

Minimum area of reinforcement; $A_{s,min} = 0.0013 \times b \times h = \mathbf{1112}$ mm^2

Maximum area of reinforcement; $A_{s,max} = 0.04 \times b \times h = \mathbf{34200}$ mm^2

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

Rectangular section in shear

Design shear force span 1 at 5086 mm; $V = \text{abs}(\min(V_{B\_s1\_max}, V_{B\_s1\_red})) = \mathbf{1383}$ kN

Design shear stress; $v = V / (b \times d) = \mathbf{1.856}$ N/mm^2

Design concrete shear stress; $v_c = 0.79 \times \min(3,[100 \times A_{s,prov} / (b \times d)]^{1/3}) \times \max(1, (400 /d)^{1/4}) \times$ $(\min(f_{cu}, 40) / 25)^{1/3} / \gamma_m$

$v_c = \mathbf{0.887}$ N/mm^2

Allowable design shear stress; $v_{max} = \min(0.8 \text{ N/mm}^2 \times (f_{cu}/1 \text{ N/mm}^2)^{0.5}, 5 \text{ N/mm}^2) = \mathbf{5.000}$ N/mm^2

PASS - Design shear stress is less than maximum allowable

Value of v from Table 3.7; $(v_c + 0.4 \text{ N/mm}^2) < v < v_{max}$

Design shear resistance required; $v_s = \max(v - v_c, 0.4 \text{ N/mm}^2) = \mathbf{0.969}$ N/mm^2

Area of shear reinforcement required; $A_{sv,req} = v_s \times b / (0.87 \times f_{yv}) = \mathbf{4009}$ mm^2/m

Shear reinforcement provided; $8 \times 10\phi$ legs at 150 c/c

Area of shear reinforcement provided; $A_{sv,prov} = \mathbf{4189}$ mm^2/m

PASS - Area of shear reinforcement provided exceeds minimum required

Maximum longitudinal spacing; $s_{vl,max} = 0.75 \times d = \mathbf{311}$ mm

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

Design shear force span 2 at 414 mm; $V = \max(V_{B\_s2\_max}, V_{B\_s2\_red}) = \mathbf{1118}$ kN

Design shear stress; $v = V / (b \times d) = \mathbf{1.501}$ N/mm^2

Design concrete shear stress; $v_c = 0.79 \times \min(3,[100 \times A_{s,prov} / (b \times d)]^{1/3}) \times \max(1, (400 /d)^{1/4}) \times$ $(\min(f_{cu}, 40) / 25)^{1/3} / \gamma_m$

$v_c = \mathbf{0.887}$ N/mm^2

Allowable design shear stress; $v_{max} = \min(0.8 \text{ N/mm}^2 \times (f_{cu}/1 \text{ N/mm}^2)^{0.5}, 5 \text{ N/mm}^2) = \mathbf{5.000}$ N/mm^2

PASS - Design shear stress is less than maximum allowable

Value of v from Table 3.7; $(v_c + 0.4 \text{ N/mm}^2) < v < v_{max}$

Design shear resistance required; $v_s = \max(v - v_c, 0.4 \text{ N/mm}^2) = \mathbf{0.614}$ N/mm^2

Area of shear reinforcement required; $A_{sv,req} = v_s \times b / (0.87 \times f_{yv}) = \mathbf{2540}$ mm^2/m

Shear reinforcement provided; $8 \times 10\phi$ legs at 200 c/c

Area of shear reinforcement provided; $A_{sv,prov} = \mathbf{3142}$ mm^2/m

PASS - Area of shear reinforcement provided exceeds minimum required

Maximum longitudinal spacing; $s_{vl,max} = 0.75 \times d = \mathbf{311}$ mm

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

Spacing of reinforcement (cl 3.12.11)

Actual distance between bars in tension; $s = (b - 2 \times (c_{nom\_s} + \phi_v + \phi_{top}/2)) /(N_{top} - 1) - \phi_{top} = \mathbf{80}$ mm

Minimum distance between bars in tension (cl 3.12.11.1)

Minimum distance between bars in tension; $s_{min} = h_{agg} + 5 \text{ mm} = \mathbf{25}$ mm

PASS - Satisfies the minimum spacing criteria

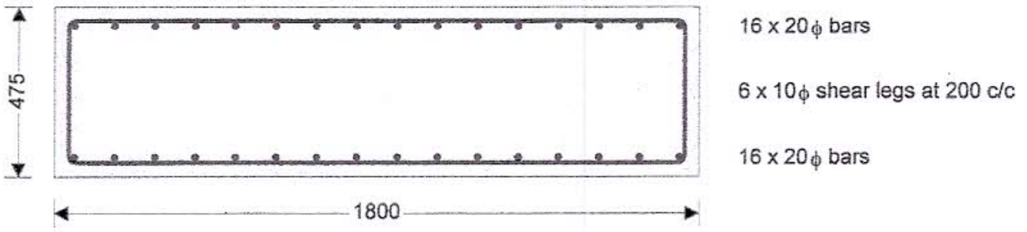
Maximum distance between bars in tension (cl 3.12.11.2)

Design service stress; $f_s = (2 \times f_y \times A_{s,req}) / (3 \times A_{s,prov} \times \beta_b)$ = **237.5** N/mm^2

Maximum distance between bars in tension; s_{max} = min(47000 N/mm / f_s, 300 mm) = **198** mm

PASS - Satisfies the maximum spacing criteria

Mid span 2



Design moment resistance of rectangular section (cl. 3.4.4) - Positive midspan moment

Design bending moment; M = abs(M_{s2_red}) = **364** kNm

Depth to tension reinforcement; $d = h - c_{nom\_b} - \phi_v - \phi_{bot} / 2$ = **420** mm

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

$K = M / (b \times d^2 \times f_{cu})$ = **0.023**

K' = 0.156

K' > K - No compression reinforcement is required

Lever arm; $z = \min(d \times (0.5 + (0.25 - K / 0.9)^{0.5}), 0.95 \times d)$ = **399** mm

Depth of neutral axis; $x = (d - z) / 0.45$ = **47** mm

Area of tension reinforcement required; $A_{s,req} = M / (0.87 \times f_y \times z)$ = **2096** mm^2

Tension reinforcement provided; 16 × 20ϕ bars

Area of tension reinforcement provided; $A_{s,prov}$ = **5027** mm^2

Minimum area of reinforcement; $A_{s,min} = 0.0013 \times b \times h$ = **1112** mm^2

Maximum area of reinforcement; $A_{s,max} = 0.04 \times b \times h$ = **34200** mm^2

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

Rectangular section in shear

Shear reinforcement provided; 6 × 10ϕ legs at 200 c/c

Area of shear reinforcement provided; $A_{sv,prov}$ = **2356** mm^2/m

Minimum area of shear reinforcement (Table 3.7); $A_{sv,min}$ = 0.4N/mm^2 $\times b / (0.87 \times f_{yv})$ = **1655** mm^2/m

PASS - Area of shear reinforcement provided exceeds minimum required

Maximum longitudinal spacing (cl. 3.4.5.5); $s_{vl,max} = 0.75 \times d$ = **315** mm

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

Design concrete shear stress; v_c = 0.79N/mm^2 × min(3,[100 × $A_{s,prov}$ / (b × d)]$^{1/3}$) × max(1, (400mm /d)$^{1/4}$) × (min(f_{cu}, 40N/mm^2) / 25N/mm^2)$^{1/3}$ / γ_m = **0.645** N/mm^2

Design shear resistance provided; $v_{s,prov} = A_{sv,prov} \times 0.87 \times f_{yv} / b$ = **0.569** N/mm^2

Design shear stress provided; $v_{prov} = v_{s,prov} + v_c$ = **1.215** N/mm^2

Design shear resistance; $V_{prov} = v_{prov} \times (b \times d)$ = **918.2** kN

Shear links provided valid between 900 mm and 4600 mm with tension reinforcement of 5027 mm^2

Spacing of reinforcement (cl 3.12.11)

Actual distance between bars in tension; $s = (b - 2 \times (c_{nom\_s} + \phi_v + \phi_{bot}/2)) / (N_{bot} - 1) - \phi_{bot}$ = **93** mm

Minimum distance between bars in tension (cl 3.12.11.1)

Minimum distance between bars in tension; $s_{min} = h_{agg}$ + 5 mm = **25** mm

PASS - Satisfies the minimum spacing criteria

Maximum distance between bars in tension (cl 3.12.11.2)

Design service stress; $f_s = (2 \times f_y \times A_{s,req}) / (3 \times A_{s,prov} \times \beta_b) = \mathbf{139.0}\ N/mm^2$

Maximum distance between bars in tension; $s_{max} = \min(47000\ N/mm / f_s, 300\ mm) = \mathbf{300}\ mm$

PASS - Satisfies the maximum spacing criteria

Span to depth ratio (cl. 3.4.6)

Basic span to depth ratio (Table 3.9); $span\_to\_depth_{basic} = \mathbf{26.0}$

Design service stress in tension reinforcement; $f_s = (2 \times f_y \times A_{s,req}) / (3 \times A_{s,prov} \times \beta_b) = \mathbf{139.0}\ N/mm^2$

Modification for tension reinforcement

$$f_{tens} = \min(2.0, 0.55 + (477N/mm^2 - f_s) / (120 \times (0.9N/mm^2 + (M / (b \times d^2))))) = \mathbf{1.927}$$

Modification for compression reinforcement

$$f_{comp} = \min(1.5, 1 + (100 \times A_{s2,prov} / (b \times d)) / (3 + (100 \times A_{s2,prov} / (b \times d)))) = \mathbf{1.181}$$

Modification for span length; $f_{long} = 1.000$

Allowable span to depth ratio; $span\_to\_depth_{allow} = span\_to\_depth_{basic} \times f_{tens} \times f_{comp} = \mathbf{59.2}$

Actual span to depth ratio; $span\_to\_depth_{actual} = L_{s2} / d = \mathbf{13.1}$

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

;

chord AXIAL COMP. FORCE.

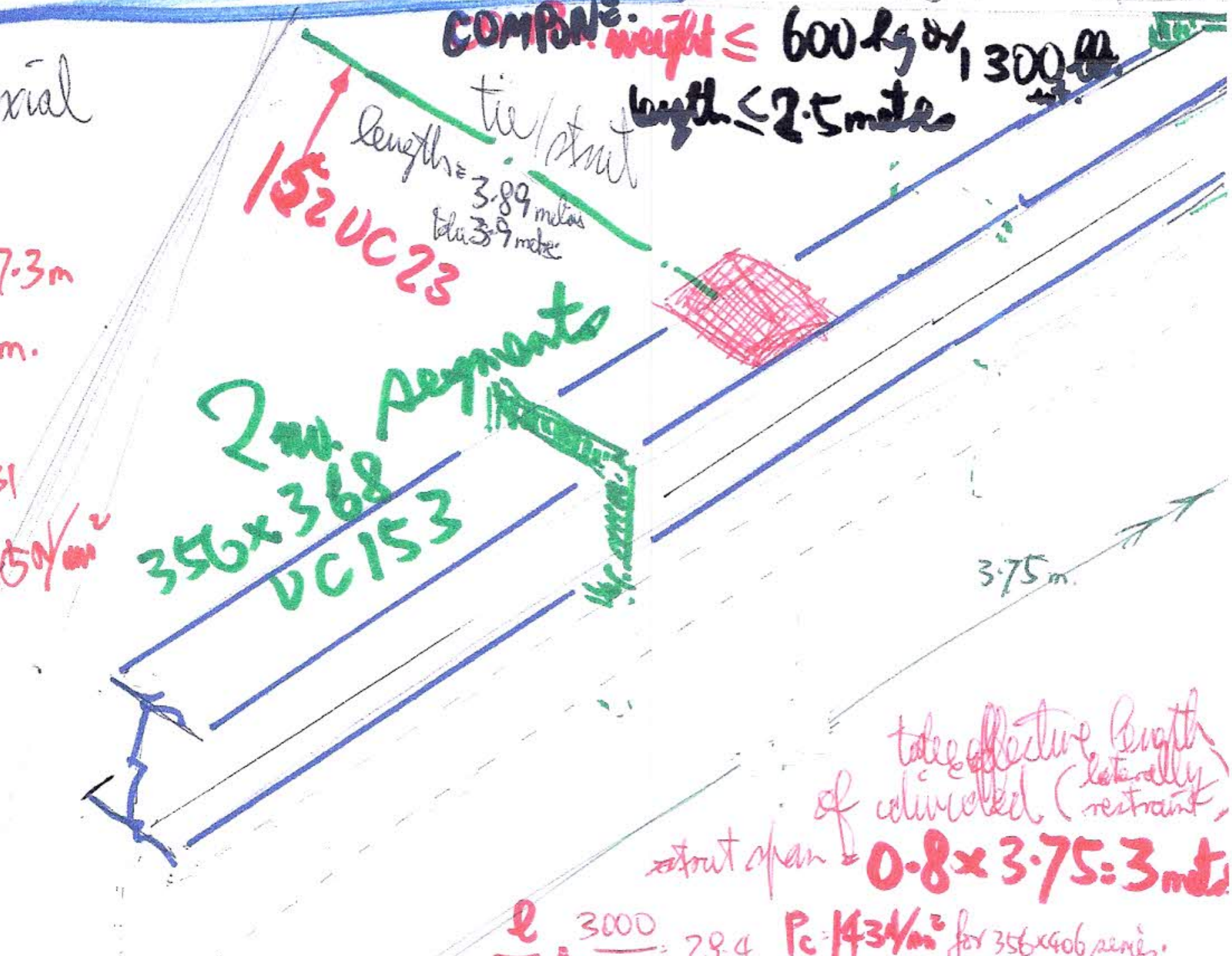
27th Nov & 28th November 2014.

axial

$0{\cdot}7 \times 7{\cdot}3\text{m} = 5{\cdot}1\text{m}.$

$\frac{5000}{158} = 31$

$P_c = 145\ \text{N/mm}^2$



take effective length of column (laterally restraint)
strut span = $0{\cdot}8 \times 3{\cdot}75 = 3$ metre

$\frac{l}{r} = \frac{3000}{102} = 29.4$ $P_c = 143\ \text{N/mm}^2$ for 356×406 series.

3·75m

$3750 \times 0{\cdot}7 = 2625$

take 5% of axial to be secondary / lateral

strut force $= \frac{5}{100} \times 2939 = 147$ kN.

allowable for 356×406 UC 235
axial comp.
$29900 \times 143\ \text{N/mm}^2 = 4276$ kN

* lateral tie/strut.

$\frac{l}{r} = \frac{3800}{37} = 105$ $\left(\frac{D}{T} = \frac{152}{6.8} = 22\right)$ $P_c = 74\ \text{N/mm}^2$

allow. axial comp. $74 \times 2970 = 219.78$ kN

$\frac{219.78}{147} = 1.5$ OK.

try lower weight series.
$\frac{3000}{95} = 31.5$ $P_c = 142\ \text{N/mm}^2$.
$\frac{2939 \times 10^3}{142\ \text{N/mm}^2} = 207\ \text{cm}^2$

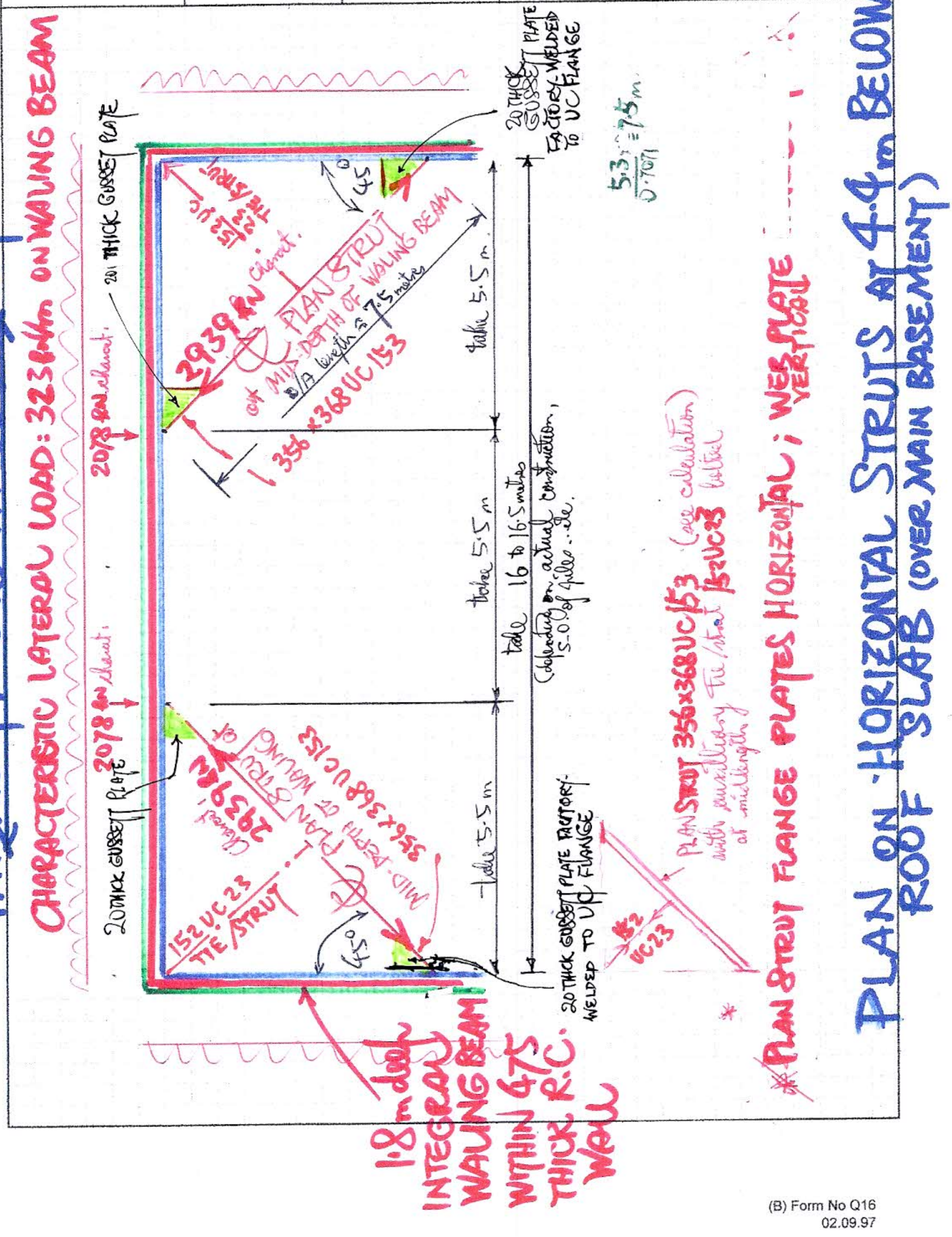
use 356×368 UC 153

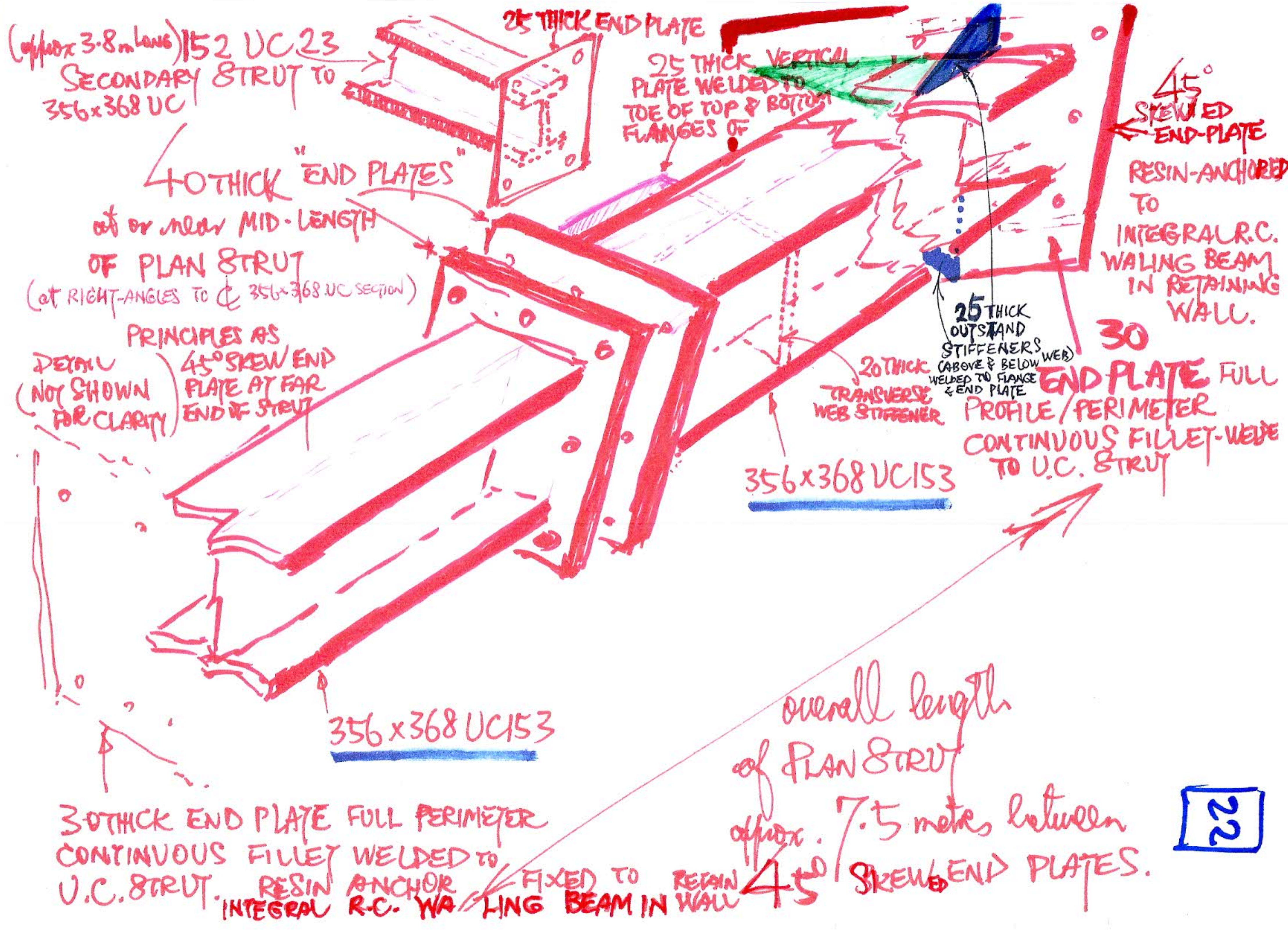
$\frac{l}{r} = \frac{2625}{95} = 27$ $P_c = 145\ \text{N/mm}^2$
$19500 \times 145 = 2842\,000 = 2842$ kN

Calculations

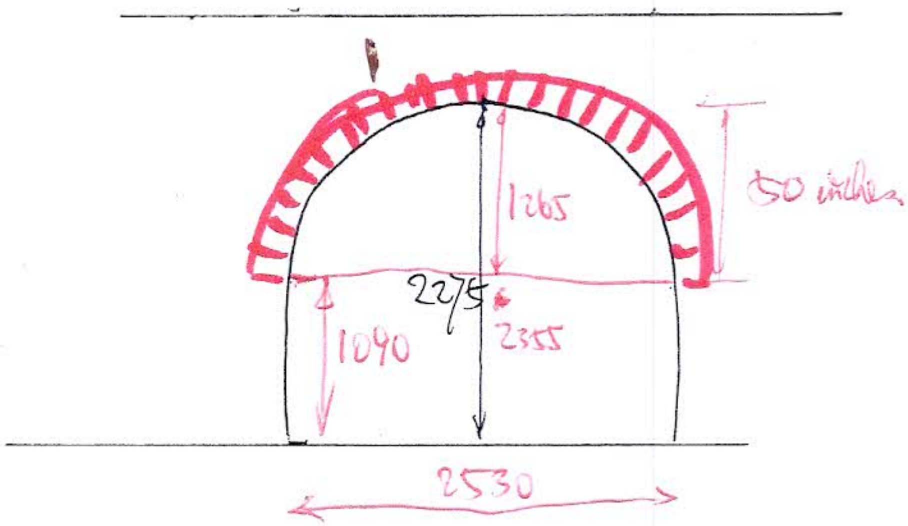
Job Ref: 2014 - 138			
Calc.By	Checked WT	Date NOV, 2014	Project 13 PRINCE ALBERT ROAD

Page No

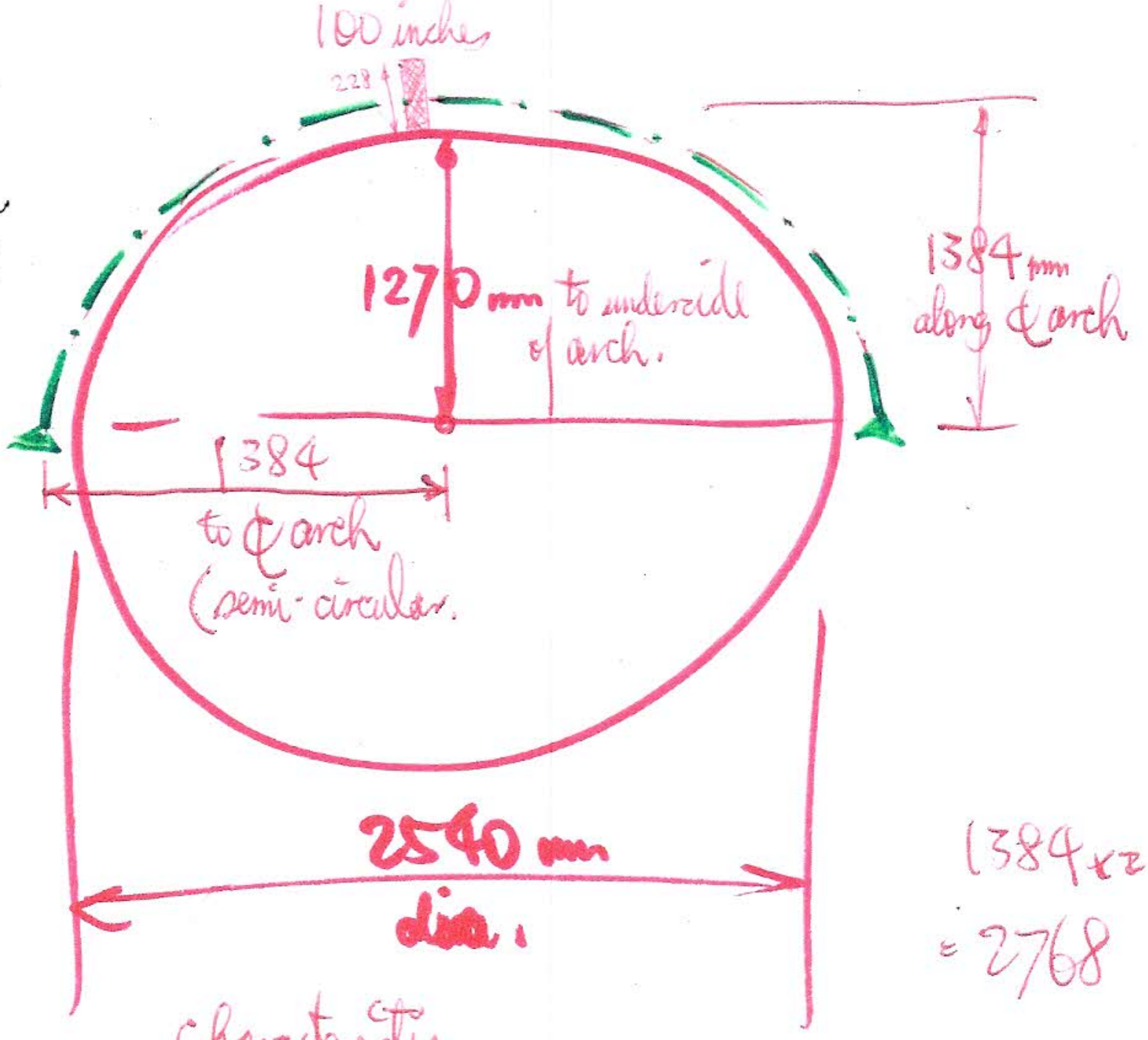




(approx 3.8m long) 152 UC 23
SECONDARY STRUT TO
356x368 UC
25 THICK END PLATE
25 THICK VERTICAL
PLATE WELDED TO
TOE OF TOP & BOTTOM
FLANGES OF
45°
SKEWED
END-PLATE
RESIN-ANCHORED
TO
INTEGRAL R.C.
WALING BEAM
IN RETAINING
WALL.
40 THICK "END PLATES"
at or near MID-LENGTH
OF PLAN STRUT
(at RIGHT-ANGLES TO ℄ 356x368 UC SECTION)
PRINCIPLES AS
DETAIL (NOT SHOWN FOR CLARITY)
45° SKEW END PLATE AT FAR END OF STRUT
25 THICK
OUTSTAND
STIFFENERS
(ABOVE & BELOW WEB)
WELDED TO FLANGE & END PLATE
20 THICK
TRANSVERSE
WEB STIFFENER
30
END PLATE FULL
PROFILE/PERIMETER
CONTINUOUS FILLET-WELDED
TO U.C. STRUT
356x368 UC153
356x368 UC153
overall length
of PLAN STRUT
approx. 7.5 metres between
45° SKEWED END PLATES.
30 THICK END PLATE FULL PERIMETER
CONTINUOUS FILLET WELDED TO
U.C. STRUT. RESIN ANCHOR FIXED TO RETAIN WALL
INTEGRAL R.C. WALING BEAM IN WALL



2355
−1090
1265



1384 × 2
= 2768

4.5 inches.
114 mm.

characteristic
take uniformly distributed load at

take 0.65 × 22 kN/m² = ; 14.3 kN/m² charact. D.L.

take 5 kN/m² charact L.L.

WT&L CHARTERED ENGINEERS BUILDING DESIGN CONSULTANTS

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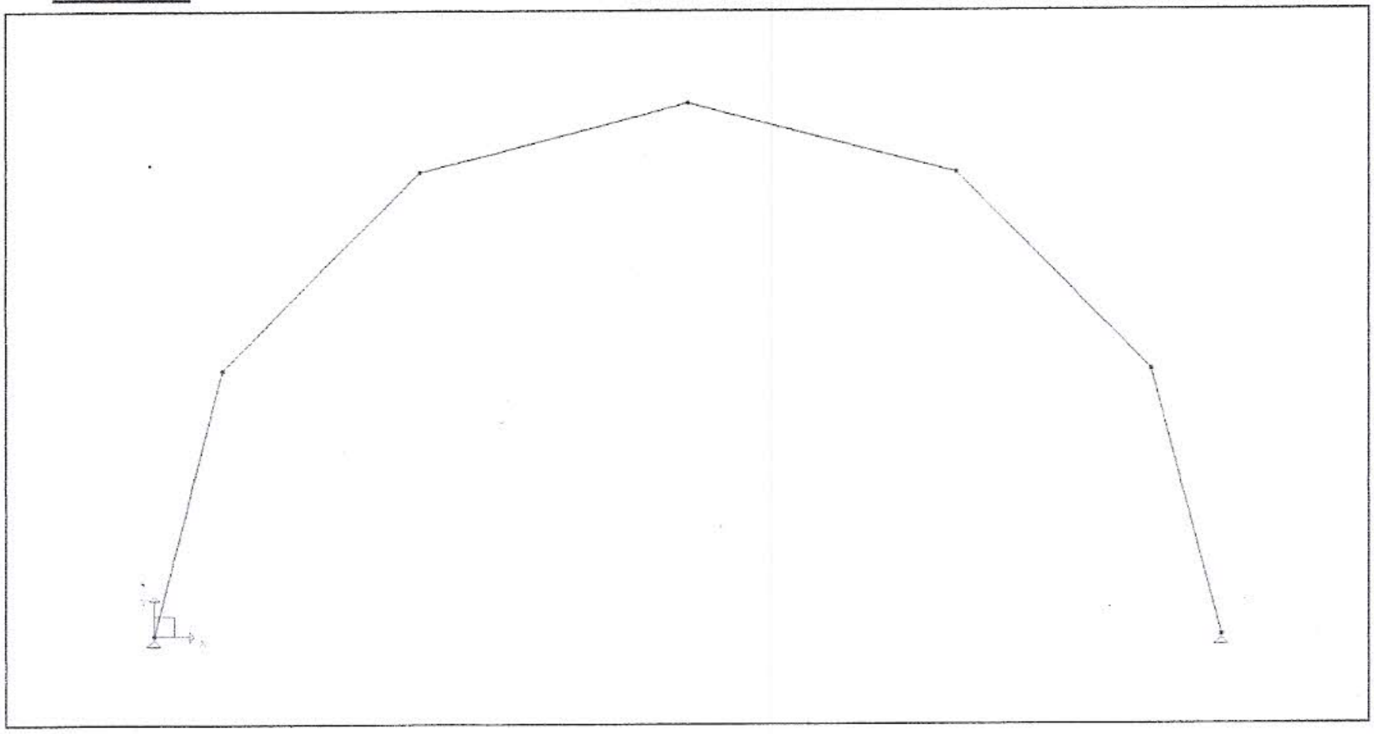
EXISTING MASONRY ARCH UNDER PORTICO					
Calc. by	Date	Chk'd by	Date	App'd by	Date
W	NOV14	GC	NOV14		

GEOMETRY



BASIC MEMBER LOAD DATA

Member No	W1 at X1 kN/m	Distance X1/Length	W2 at X2 kN/m	Distance X2/Length	Load Type
1	-20.0000	0.0000	-20.0000	1.0000	Y Global
2	-20.0000	0.0000	-20.0000	1.0000	Y Global
3	-20.0000	0.0000	-20.0000	1.0000	Y Global
4	-20.0000	0.0000	-20.0000	1.0000	Y Global
5	-20.0000	0.0000	-20.0000	1.0000	Y Global
6	-20.0000	0.0000	-20.0000	1.0000	Y Global

REACTIONS

Joint No	Ld Case No	X-Force kN	Y-Force kN	Z-Moment kN-m
1	1	14.7466	43.1001	0.0000
3	1	-14.7466	43.0139	0.0000
4	1	0.0000	0.0000	0.0000
5	1	0.0000	0.0000	0.0000
7	1	0.0000	0.0000	0.0000
8	1	0.0000	0.0000	0.0000
9	1	0.0000	0.0000	0.0000

$\frac{118}{141} \times 2.77 = 2.32$ metre

25

CHARTERED ENGINEERS
BUILDING
DESIGN
CONSULTANTS

$2.77\,m \times 15\,kN/m^2 = 42\,kN.$

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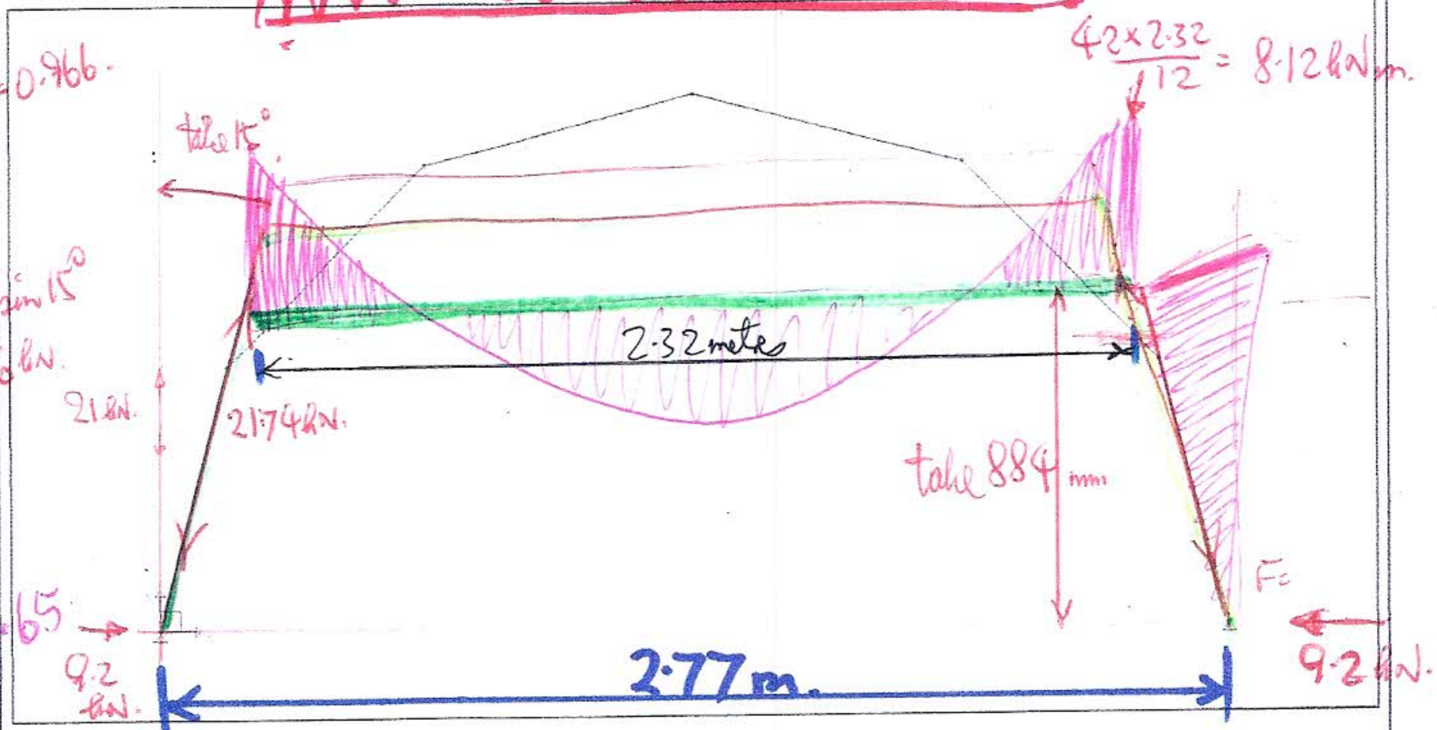
Calc. by	Date	Chk'd by	Date	App'd by	Date
W	NOV14	GC	NOV14		

take Total W = 42 kN.

GEOMETRY



BASIC MEMBER LOAD DATA

Member No	W1 at X1 kN/m	Distance X1/Length	W2 at X2 kN/m	Distance X2/Length	Load Type
1	-20.0000	0.0000	-20.0000	1.0000	Y Global
2	-20.0000	0.0000	-20.0000	1.0000	Y Global
3	-20.0000	0.0000	-20.0000	1.0000	Y Global
4	-20.0000	0.0000	-20.0000	1.0000	Y Global
5	-20.0000	0.0000	-20.0000	1.0000	Y Global
6	-20.0000	0.0000	-20.0000	1.0000	Y Global

REACTIONS

Joint No	Ld Case No	X-Force kN	Y-Force kN	Z-Moment kN-m
1	1	14.7466	43.1001	0.0000
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5	1	0.0000	0.0000	0.0000
7	1	0.0000	0.0000	0.0000
8	1	0.0000	0.0000	0.0000
9	1	0.0000	0.0000	0.0000

9.2 kN from Bending Moment.
5.6 kN from Horizontal comp. of Thrust.

14.8 kN.

$F \times 0.884\,m = 8.12\,kNm.$

BEAM TO RESIST HORIZONTAL THRUST ALONG SPRINGING OF MASONRY ARCH UNDER PORTICO

Calculations

WT & L — Chartered Engineers, Building Design Consultants

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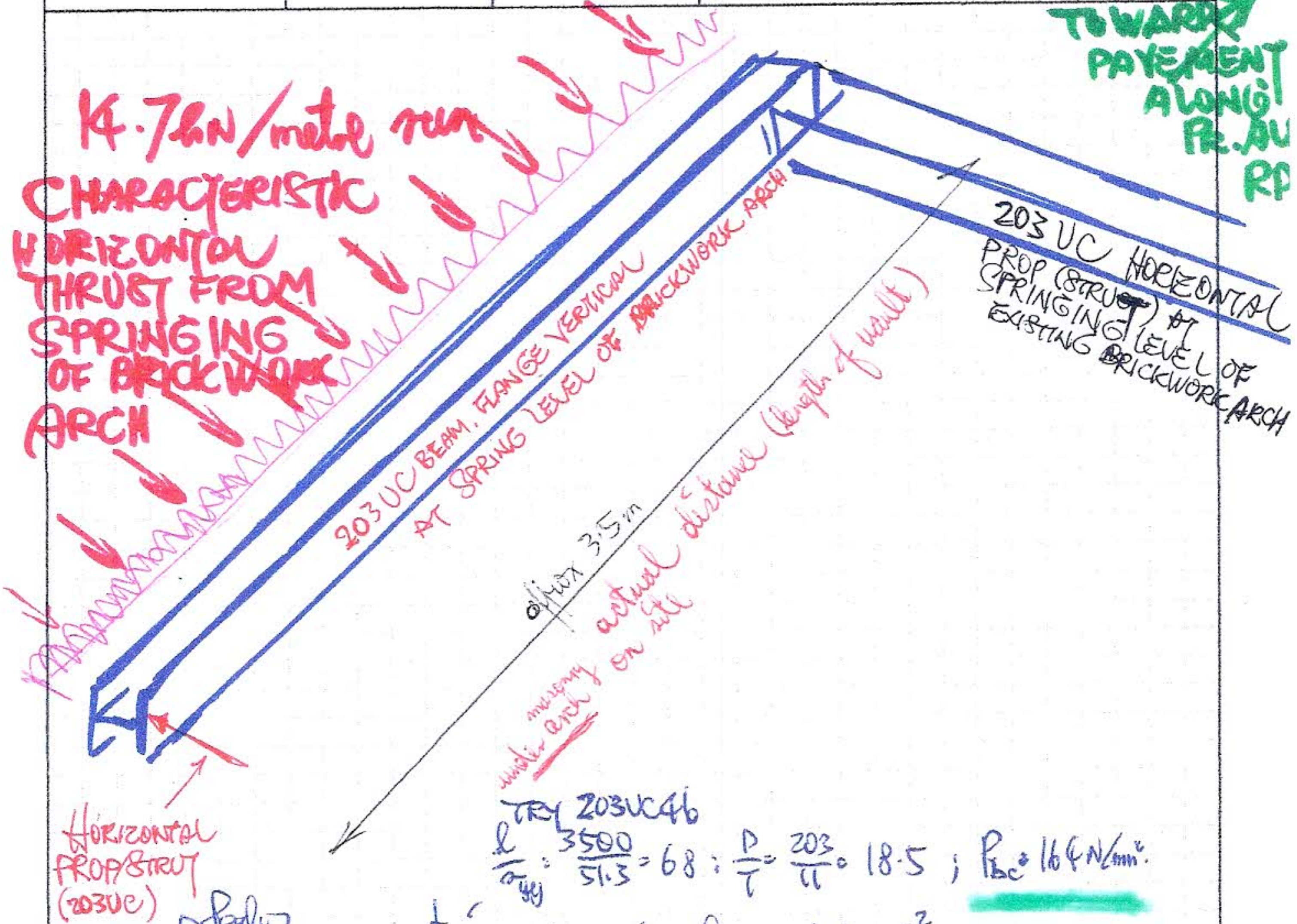

TRY 203UC46

$\frac{l}{r_{yy}} = \frac{3500}{51.3} = 68$; $\frac{D}{T} = \frac{203}{11} = 18.5$; $P_{bc} = 164\ N/mm^2$

* Peak Bending moment in 3.5m span beam $\frac{14.7 \times 3.5^2}{8} = 22.5\ kNm$.

$\frac{M}{Z} = \frac{22.5 \times 10^6}{450 \times 10^3} = \pm 50\ N/mm^2$ OK.

Maximum span of beam ; allowable to resist charact. ↔ load at 14.7kN/m span

ca 5.5 metres Peak B.M. = $(14.7 \times 5.5^2) \div 8 = 55.6\ kNm$; $\frac{M}{Z} = \frac{55.6 \times 10^6}{450 \times 10^3} = \pm 124\ N/mm^2$

$\frac{l_e}{r_{yy}} = \frac{5500}{51.3} = 107$; $\frac{D}{T} = 18.5$; $P_{bc} = 137\ N/mm^2$; OK.

203 UC 46 OK for span up to 5.5m when U.D.L intensity = 14.7 kN/m characteristic.