Issue / Revision Description:			*Date:*	
First Issue			**November-2010**	
Development:	***Title***	***Name [Print]***	***Signature***	***Date***
Author:	**Engineer**	**WP Lam**		***Nov 2010***

Document Status:

Approved

Client:

A GREENER PLACE

Originator:

WT & L
CHARTERED ENGINEERS
BUILDING
DESIGN
CONSULTANTS

Location:

UCL, ARCHAEOLOGY BUILDING

Title:

Structural Design of Support and Holding Down System For Solar Panels On Existing Roof Slab

Format	***Document Number***	***Revision***
A4	**WTL 2010-13.AB**	**00**

Document Reference No. WTL 2010-13.AB

Location(s): UCL, ARCHAEOLOGY BUILDING

00	17 - Nov - 2010	WP Lam	First Issue	For Comment
Rev	Date	By	Section Amended	Details of Amendments

CONTENTS

1.0 OBJECTIVE

To carry out a reconnaissance survey on part of the roof at the Archaeology Building; to view the structural elements which have been identified, by the client ‘A GREENER PLACE, to support solar panels.

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2.0 EXECUTIVE SUMMARY

The north-eastern part of the roof, which comprises ‘reinforced concrete beam ribs with hollow clay pot’ roof slab, & upstand reinforced concrete beams on reinforced concrete columns, walls or loadbearing masonry walls; are considered suitable to support the proposed solar panels.

3.0 OBSERVATION

The roof slab is located approximately 22 metres above adjacent road level. Adjacent buildings are not taller than the Archaeology Building, nor sufficiently close to it; to experience funnelling effect due to wind.

The building was constructed around 1957. The flat roof slab was constructed from insitu reinforced concrete beam ribs spaced apart by hollow clay pots. At the north-eastern area, the slab is supported by a small grid of intersecting upstand reinforced concrete beams . Those upstand beams are supported on loadbearing masonry wall, reinforced concrete columns and walls.

4.0 DISCUSSION

A visual ‘structural condition’ assessment was carried out on site; there was no significant sign of distress or cracking in the area of interest (area designated to support the solar panels.

A theoretical load-carrying / strength assessment has been evaluated, the slab and reinforced concrete beams are considered capable of supporting the solar panels in the proposed arrangement (attached drawing and calculations refer).

5.0 CONCLUSION

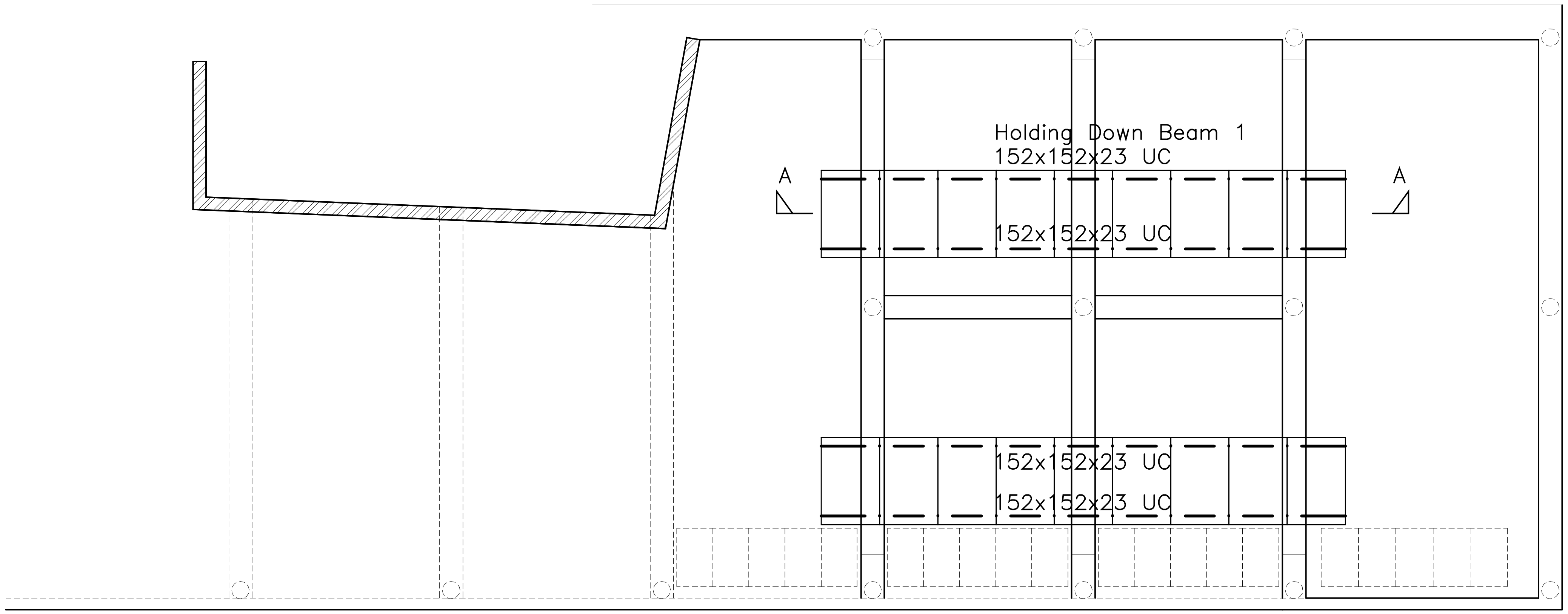
Visual and theoretical structural assessment would indicate that the designated roof area is capable of supporting the proposed solar panels.

The ‘proposed works’ drawing(s) indicate the required support and holding down details.

6.0 APPENDICES

Appendix A

Proposed Drawings By WT&L



STEELWORK NOTES:

1. ALL MATERIALS AND WORKMANSHIP TO BE IN ACCORDANCE WITH THE NATIONAL STRUCTURAL STEELWORK SPECIFICATION (N.S.S.S)

2. STRUCTURAL STEELWORK SECTIONS TO BE GRADE S275 MILD STEEL IN ACCORDANCE WITH B.S. EN 10025.

3. BOLTS TO BE GRADE 8.8 UNLESS NOTED OTHERWISE.

4. ALL WELDS TO BE 6MM CONTINUOUS FILLET, UNLESS NOTED OTHERWISE.

5. CONTRACTOR MUST VERIFY ALL DIMENSIONS ON SITE BEFORE COMMENCING ANY WORK OR PREPARING ANY SHOP DRAWINGS. DUPLICATE COPIES OF SHOP DRAWINGS TO BE FORWARDED TO STRUCTURAL ENGINEER FOR COMMENT 7 DAYS PRIOR TO FABRICATION OF STEEL. NO DIMENSIONS TO BE SCALED FROM DRAWINGS. DISCREPANCIES MUST BE REPORTED TO THE ENGINEER PRIOR TO PROCEEDING.

6. ALL STEELWORK IS TO BE BLAST CLEANED TO A MIN. OF SA2 $\frac{1}{2}$ PRIOR TO GALVANIZING.

7. SUITABLE/APPROVED REMEDIAL CORROSION PROTECTION SHOULD BE APPLIED TO ALL DAMAGED OR IMPAIRED GALVANIZED COATING, IMMEDIATELY ON COMPLETION OF SITE CONNECTION.

8. FOR WELDING OF GALVANIZED COMPONENTS, REFER TO RELEVANT SPECIFICATION.

9. ALL EXTERNAL STEELWORKS TO BE GALVANIZED.

ALL EXTERNAL STEELWORKS SHOULD BE GALVANIZED

CONNECTIONS SHALL BE PREPARED AND COMPLETED AFTER ACCURATE SETTING OUT OF RELEVANT MEMBERS. WELDING SHALL BE CARRIED OUT BY SUITABLY QUALIFIED WELDER(S).

UCL ARCHAEOLOGY BUILDING

ALL CONNECTION WELDS TO BE 6mm FULL CONTINUOUS F.W UNLESS NOTED OTHERWISE

THE CONTRACTOR SHOULD CONSULT AND OBTAIN PERMISSION FROM THE BUILDING OWNER (OWNER'S REPRESENTATIVE) FOR ATTACHMENT OF FIXINGS TO THE BUILDING. WT&L ENGINEERS SHOULD BE NOTIFIED, IF ALTERNATIVE FIXING DETAILS ARE REQUIRED.

Client: A GREENER PLACE

Job Title: UCL ARCHAEOLOGY BUILDING

Drawing Title: HOLDING DOWN SYSTEM GENERAL ARRANGEMENT

Rev	Date	Amendments	Drawn	Approved
A	DEC10	DRAWING UPDATED	WT	G

Notes

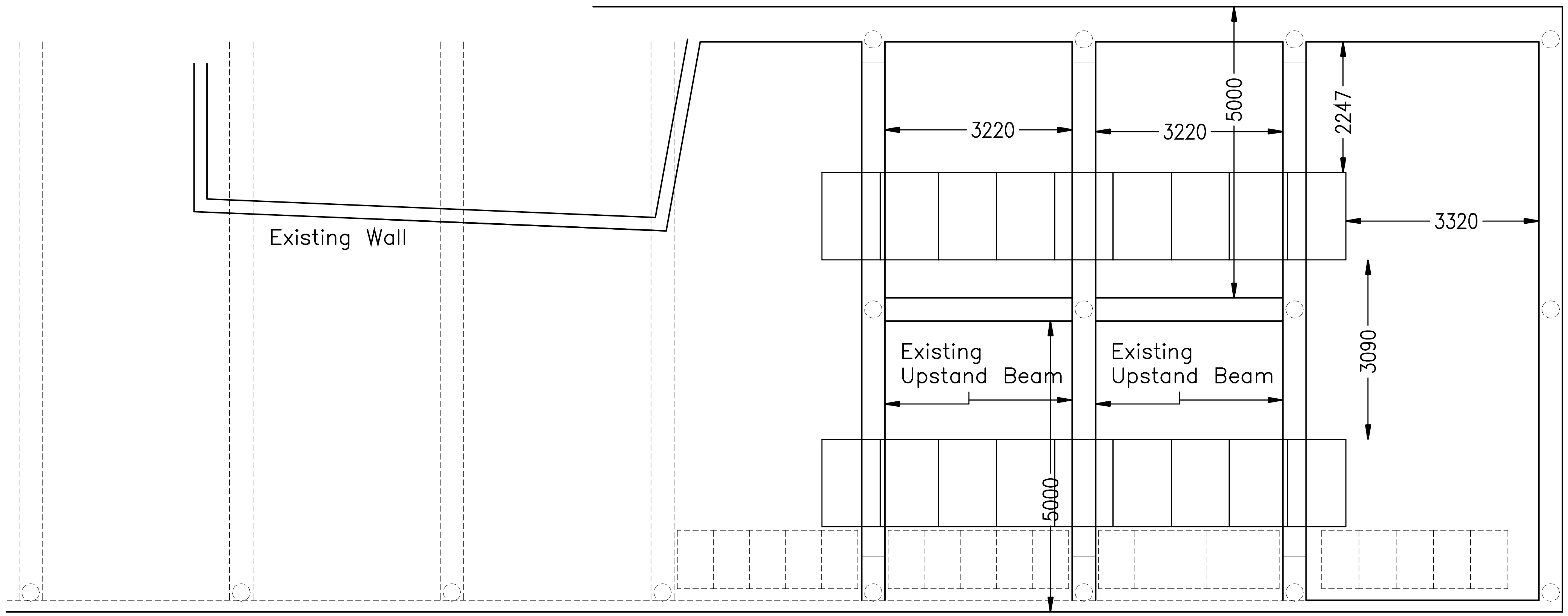
Drawing Size : A3

Date: NOV10
Drawn by: WT
Checked by: L
Approved by: A
Scale: 1:100

WT &L CHARTERED ENGINEERS BUILDING DESIGN CONSULTANTS
ADDRESS: 7 PALACE WAY, WOKING, SURREY, GU22 8JA
TEL: 01483761585

Drawing No: 2010-13.06

Rev: A



UCL ARCHAEOLOGY BUILDING
PROPOSED SOLAR PANELS POSITION

Existing Circular RC Column Below

ANTICIPATED SETTING OUT DIMENSION BASED
ON BEST AVAILABLE INFORMATION ONLY.

Client	A GREENER PLACE
Job Title	UCL ARCHAEOLOGY BUILDING
Drawing Title	ANTICIPATED SETTING OUT ARRANGEMENT

Rev	Date	Amendments	Drawn	Approved
A	DEC10	DRAWING UPDATED	WT	G

Notes

Date	NOV10
Drawn by	WT
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Approved by	A
Drawing Size :	A3
Scale	1:100
Drawing No	2010-13.07
Rev	A

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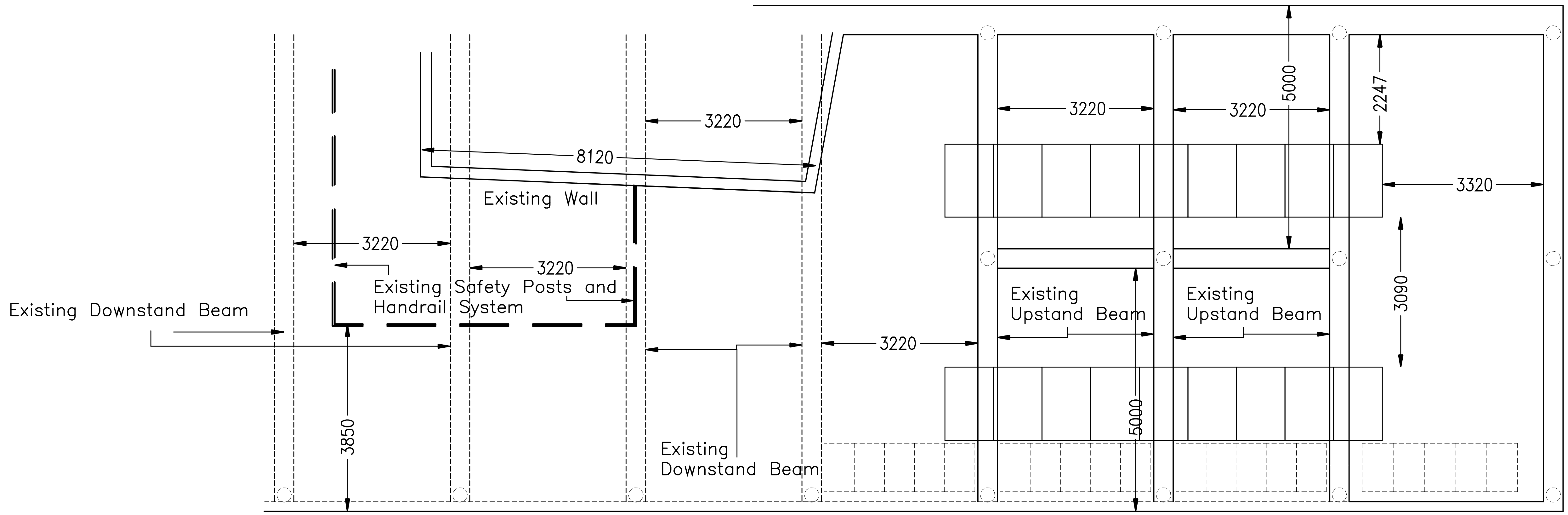

UCL ARCHAEOLOGY BUILDING

ANTICIPATED SETTING OUT DIMENSION BASED ON BEST AVAILABLE INFORMATION ONLY.

○ Existing Circular RC Column Below

Client	A GREENER PLACE
Job Title	UCL ARCHAEOLOGY BUILDING
Drawing Title	EXISTING ROOF DIMENSION

Rev	Date	Amendments	Drawn	Approved
A	DEC10	DRAWING UPDATED	WT	G

Notes

Date	NOV10
Drawn by	WT
Checked by	L
Approved by	A
Drawing Size :	A3
Scale	1:20
Drawing No	2010-13.10
Rev	A

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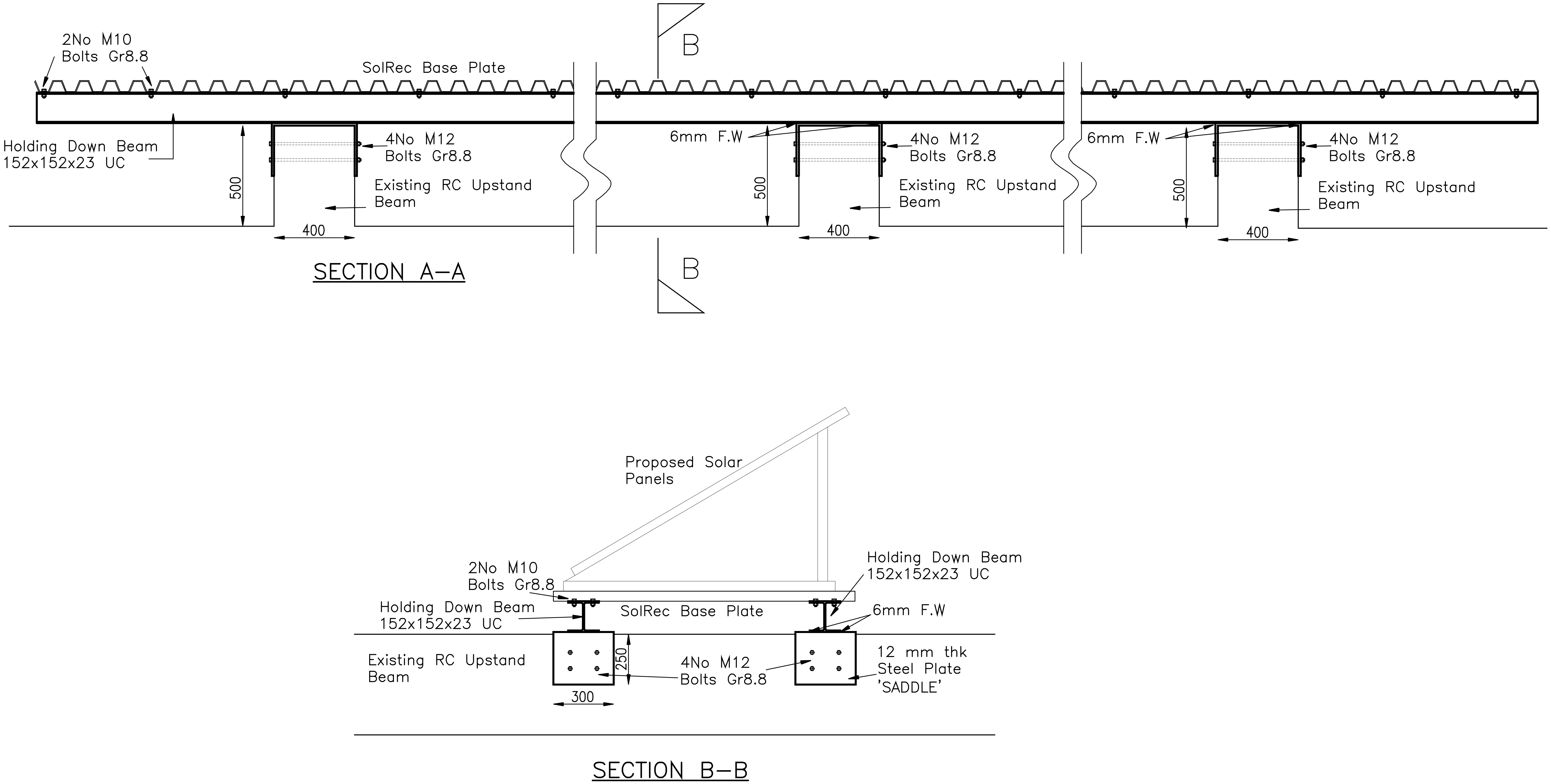

Client	A GREENER PLACE	
Job Title	UCL ARCHAEOLOGY BUILDING	
Drawing Title	DETAILS	

Rev	Date	Amendments	Drawn	Approved
A	DEC10	DRAWING UPDATED	WT	G

Notes

Date	NOV10
Drawn by	WT
Checked by	L
Approved by	A
Drawing Size :	A3
Scale	1:20
Drawing No	2010-13.11
Rev	A

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Appendix B

Assessment And Design Calculations

ARCHAEOLOGY BUILDING

The building was designed by architects, structural engineers, engineers of other disciplines. It was constructed by professional contractors around 1957.

Vertical load-carrying elements of the Archaeology Building comprises load-bearing masonry walls, reinforced concrete columns; depending on the location.

Overall visual inspection / survey of the building was carried out on 26th October, 2010. Observation, discussion …etc. are covered in a separate report.

The current proposal to install a designated number and configuration of solar panels on the flat roof slab has been assessed:

- To ensure the safe load-carrying capacity, integrity of the existing flat roof components (slabs, beams …etc.) can support the small additional distributed downward vertical load associated with the solar panels and their plinths.

- To ensure the existing waterproofing membrane, future roof insulation layer, and other apparatus / equipment are not adversely affected by the installation and continued presence of the solar panels.

- To ensure that storm force wind, shall not lift any panel from its designated position, configuration on the roof.

The following pages of 'strength assessment' calculation show that the existing 'Insitu reinforced concrete beam with voided clay block slab' can safely carry an imposed vertical load at a characteristic distributed intensity of 5 kilonewtons per square metre (based on a span of up to 3.3 metres, as noted in the site survey). The voided reinforced concrete slabs are supported by reinforced concrete upstand beams (downstand beams at internal locations), on reinforced concrete columns. The distributed vertical load intensity of proposed solar panels with the associated supports; amount to less than 0.5 kilonewtons per square metre. That equates to a safe load-carrying capacity at 4.5 kilonewtons per square metre, after installation of the solar panels. Current 'British standard' / 'European Standard' require a 'safe capacity' at just 0.75 kilonewton per square metre.

Underside of the plinths to the solar panels rest on lightweight steel sheets to disperse any concentrated load. The proposed arrangement is such that future waterproofing membrane will not be damaged (punctured), and future insulation layer will not be under any significant compressive strain or imposed creep deflection under sustained load (of very low pressure).

Wind pressure associated with 'Storm force wind' (speed up to 90 miles per hour); has been used to evaluate potential uplifting force on the solar panels. Appropriate light-weight, holding-down beam system; has been designed to ensure no solar panel will be lifted off under strong wind condition.

The current proposed holding down and restraining system of 152UC23 beams, allows future installation of insulation layer, by lifting of their fixed end.

Asset Nr / Project	*Solar panel installation*	Sheet	1
Location	*UCL Archaeology Building*	Rev	
Part of Structure	**Index**	Date	11 November 2010

Reference | Calculations | Output

Assessment of RC Beam (to BS8110)

Page Contents

	Print name (once only)	Signature (once only)	Date (every page)
Prepared by	WL		11 Nov 10
Checked by	AL		

Reference | Calculations | Output

Introduction

Purpose

This calculation is to assess the load carrying capacity of reinforced concrete beam sections subject to crowd loading of 0.75 kN/m^2.

The calculation should not be used for the assessment of reinforced concrete slabs.

The assessment is based on BS8110. However rather than calculating the ultimate moment capacity of the section by determining the lever arm z using a K factor based on the applied bending moment, z is derived by equating the force in the tension steel at yield $(f_y/\gamma_m)A_s$ to the force in the concrete based on a concrete strength of $0.67(f_{cu}/\gamma_m)$ and a depth to the compression zone of 0.9x so that the force in concrete = $0.67(f_{cu}/\gamma_m) \times b \times 0.9x$ and $z = d - 0.45x$.

STRUCTURES

CALCULATION SHEET

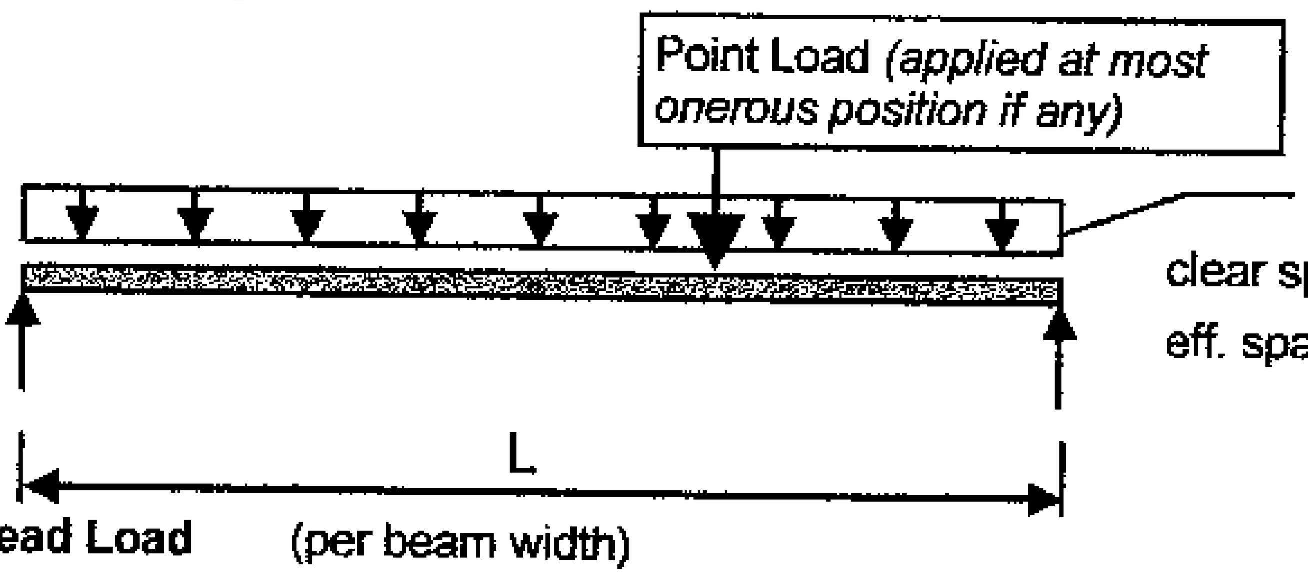
WT &L — CHARTERED ENGINEERS BUILDING DESIGN CONSULTANTS

Asset Nr / Project	**Solar panel installation**	Sheet	3
Location	**UCL Archaeology Building**	Rev	
Part of Structure	***Simply Supported Reinforced Concrete Beam***	Date	)

Reference | Calculations | Output

BS8110-1:1997

Beam Arrangement



cl. 3.4.1.2

clear span = *5000* mm

eff. span L = clear span + eff. depth

= 5000 + *505*

= 5505.0 mm

Dead Load (per beam width)

	depth (m)		breadth (m)		density		γf_1	γf_2	γf_3		
RC Beam	*0.550*	x	*0.400*	x	*24*	x	*1.10* ×	*1.00* ×	*1.05*	=	6.10 kN/m
Waterproofing	*3.610*	x	0.1		kN/m^2	x	*1.10* ×	*1.00* ×	*1.05*	=	0.42 kN/m
Hollow clay tiles			0.5		kN/m	x	*1.10* ×	*8.00* ×	*1.05*	=	4.62 kN/m
Roof slab	*0.040*	x	*3.610*	x	*24*	x	*1.10* ×	*1.00* ×	*1.05*	=	4.00 kN/m
Roof beam	*0.120*	x	*0.100*	x	*24*	x	*1.10* ×	*8.00* ×	*1.05*	=	2.66 kN/m
								Total dead load		=	17.80 kN/m

Output: no. of beams 3610/450 spacing = 8 beams

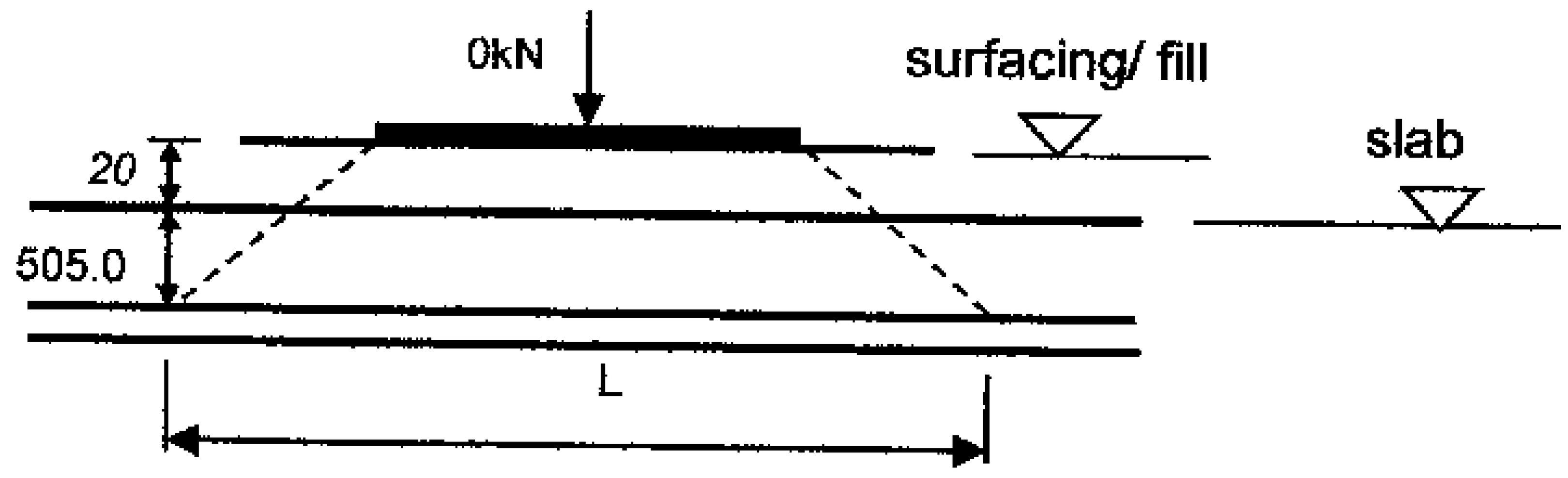
BS6399:part 1

Live Load

0.75 kN/m^2 Crowd Load or 0 kN Point Load on 300mm Square or 1.5kN/m^2 Crowd Load

	breadth (m)		w		γf_1	γf_2	γf_3		
Uniformly distributed load	*3.610*	x	*0.75*	x	*1.00* ×	*1.00* ×	*1.10*	=	2.98 kN/m
	3.610	x	*1.50*	x	*1.00* ×	*1.00* ×	*1.10*	=	5.96 kN/m

Distribute 300x300mm Patch Load at 45° in both directions through *surfacing / fill* to depth of Tension Reinforcement:



Length L = 200 + {(83+115.9)x2} = 1350 mm

∴ Load = 0 / (305 x 1,350) = 0.0 kN/m^2

	P		γf_1	γf_2	γf_3		
Patch load on 300×300 square	0.0	x	*1.00* ×	*1.00* ×	*1.10*	=	0.00 kN/m^2

Applied Bending Moment

eff. span L = *5.51m*

Bending Moment

	w		L			
UD dead load	17.80	×	5.51² /	8 =	67.43 kNm	
UD 0.75kN/m^2 live load	2.98	×	5.51² /	8 =	11.28 kNm	
or 1.5kN/m^2 live load	5.96	×	5.51² /	8 =	22.56 kNm	
				Max BM, SA =	89.99 kNm	

	Print name *(once only)*	Signature *(once only)*	Date *(every page)*
Prepared by	*WL*		*11 Nov 10*
Checked by	*AL*		

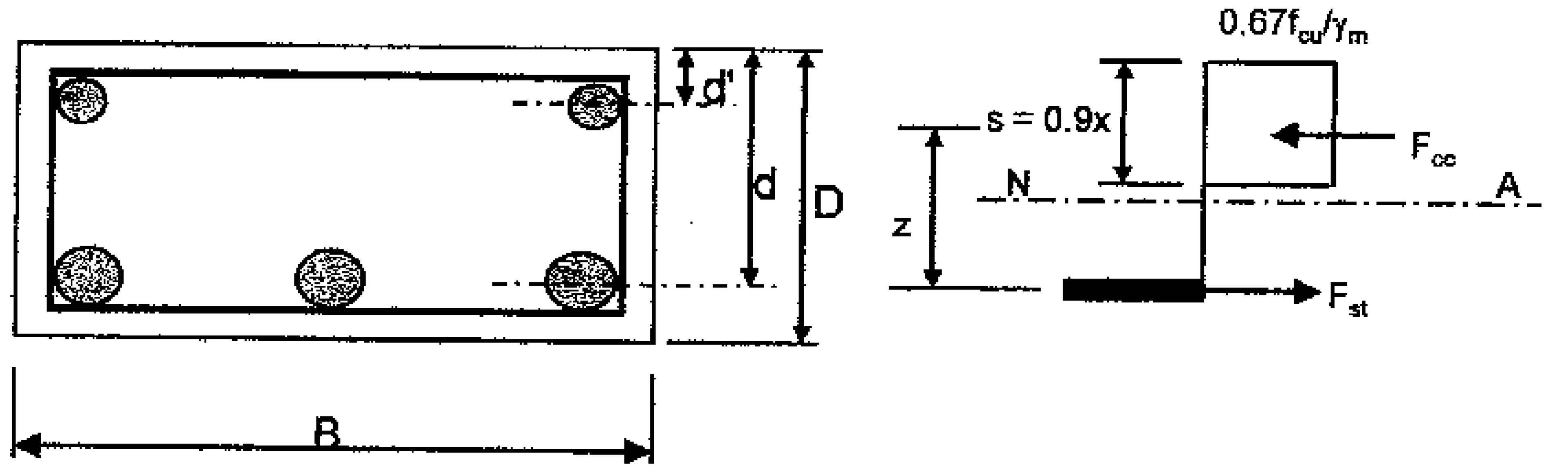
STRUCTURES
CALCULATION SHEET

CHARTERED ENGINEERS BUILDING DESIGN CONSULTANTS

Asset Nr / Project	**Solar panel installation**	Sheet	4
Location	**UCL Archaeology Building**	Rev	
Part of Structure	**Simply Supported Reinforced Concrete Beam**	Date	

Reference | Calculations | Output

BS8110-1:1997



Section Details (input values in highlighted boxes)

Width of section, B =	400	mm	Concrete strength (f_{cu}) =	30	N/mm^2
Depth of section, D =	550	mm	Reinf't strength (f_y) =	250	N/mm^2
Dia. of tensile steel =	20	mm	Cover to bottom steel =	25	mm
Dia. of compr. Steel =	16.0	mm	Cover to top steel =	25	mm
Dia. of links =	10.0	mm	Mat. Factor (conc), γ_{mc} =	1.5	
Mat Fact. (shear), γ_{mv} =	1.25		Mat. Factor (steel), γ_{ms} =	1.15	
No of legs =	0		No of bars (tension) = *(per metre width for slabs)*	3	
Effective Depth, d =	505.0	mm	No of bars (compr) =	2	
Effective Depth, d' =	43.0	mm			
$A_{s\,(tensile)}$ in width b =	942.5	mm^2	$A'_{s\,(compr.)}$ =	402.1	mm^2
Condition Factor (CF) =	1.0		Applied Bending Moment, M =	89.99	kNm

cl. 3.4 on

Moment Capacity

For equilibrium of the compressive and tensile forces:

$$F_{cc} = F_{st} \Rightarrow 0.67f_{cu}/\gamma_m bs = f_y/\gamma_m A_s$$

γ_{mc} = 1.5
γ_{ms} = 1.15

$$0.67 \;/\; 1.5 \times 30 \times 400.0 \times s = 250 \;/\; 1.15 \times 942$$

$$\therefore\; s = 38.2 \text{ mm}$$

$$\Rightarrow x = s \;/\; 0.9 = 42.5 \text{ mm}$$

$$M_u = F_{st} \times z = 0.87f_yA_s(d - s/2)$$

$$M_u = 0.87 \times 250 \times 942 \;(505.0 - 38.2 \;/\; 2) \times 10^{-6}$$

$$M_u = \mathbf{99.60\ kNm}$$

Applied Bending Moment, M = 89.99 kNm

OK

	Print name *(once only)*	Signature *(once only)*	Date *(every page)*
Prepared by	WL		11 Nov 10
Checked by	AL		

STRUCTURES

CALCULATION SHEET

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Asset Nr / Project	**Solar panel installation**	Sheet	5
Location	**UCL Archaeology Building**	Rev	
Part of Structure	**Simply Supported Reinforced Concrete Beam**	Date	

Reference | Calculations | Output

BS8110-1:1997

Shear Capacity check for a Reinforced Concrete Section

Applied Shear Force

cl. 3.4.5.8

Calculate Shear at 2d from support, [Note; no shear enhancement effects] V:

Shear Force	w	L		2d			
UD dead load	17.80 ×	5.51 /	2 -	(1.010 x	17.80)	=	31.02 kN
UD 0.75kN/m² live load	2.98 ×	5.51 /	2 -	(1.010 x	2.98)	=	5.19 kN
or 1.5kN/m² live load	5.96 ×	5.51 /	2 -	(1.010 x	5.96)	=	10.38 kN

$V = V_d + V_L$ = 41.39 kN

Shear Stress

$$v = \frac{V}{b_v d} \ \text{N/mm}^2$$

$$= \frac{41.4 \times 10E3}{400 \times 505}$$

$$v = 0.20 \ \text{N/mm}^2$$

cl. 3.4.5.2

In no case should v exceed $0.8\sqrt{f_{cu}}$ or 5N/mm² = 4.38 or 5.00 N/mm²

∴ **Shear stress within limit**

Output: 0.20

Shear Capacity (ignoring links)

Ultimate Shear Resistance, $V_u = v_c\, b\, d$

cl. 3.4.5.4

Table 3.8

$$\text{shear stress in concrete, } v_c = \frac{0.79}{\gamma_m} \cdot \left[\frac{100A_s}{b_v d}\right]^{1/3} \cdot \left[\frac{400}{d}\right]^{1/4} \cdot (f_{cu}/25)^{1/3}$$

Where the term $\frac{100A_s}{bd}$ should not be taken as > 3.0 = 0.47 ∴ **OK**

& the term $\left[\frac{400}{d}\right]^{1/4}$ should not be taken as < 0.67 = 0.94 ∴ **OK**

$$v_c = \frac{0.79}{1.5}\left[\frac{100 \times 942.48}{400 \times 505}\right]^{1/3}\left[\frac{400}{505}\right]^{1/4}\left[\frac{30}{25}\right]^{1/3}$$

$$= 0.39 \ \text{N/mm}^2$$

V_u = 77.84 kN (In absence of eff. shear reinf)

V = 41.39 kN

∴ **Shear check is ok**

Output: **OK**

SUMMARY

2.1 Table of Resistance and Applied Loads

MEMBER	LOAD EFFECT	TOTAL MEMBER CAPACITY $R_{AD}+R_{AL}$	DEAD LOAD EFFECTS S_{AD}	SPARE CAPACITY FOR LIVE R_{AL}	STANDARD LIVE LOAD EFFECT S_{AL}	RATIO OF CAPY TO STANDARD R_{AL}/S_{AL}	PASSED OR FAILED
RC BEAM	Moment 0.75kN/m² Crowd Load	99.60	67.43	32.18	11.28	2.9	**PASS**
RC BEAM	Moment 1.5kN/m² Crowd Load	99.60	67.43	32.18	22.56	1.4	**PASS**
RC BEAM	Shear 0.75kN/m² Crowd Load	77.84	31.02	46.83	5.19	9.0	**PASS**
RC BEAM	Shear 1.5kN/m² Crowd Load	77.84	31.02	46.83	10.38	4.5	**PASS**

	Print name (once only)	Signature (once only)	Date (every page)
Prepared by	WL		11 Nov 10
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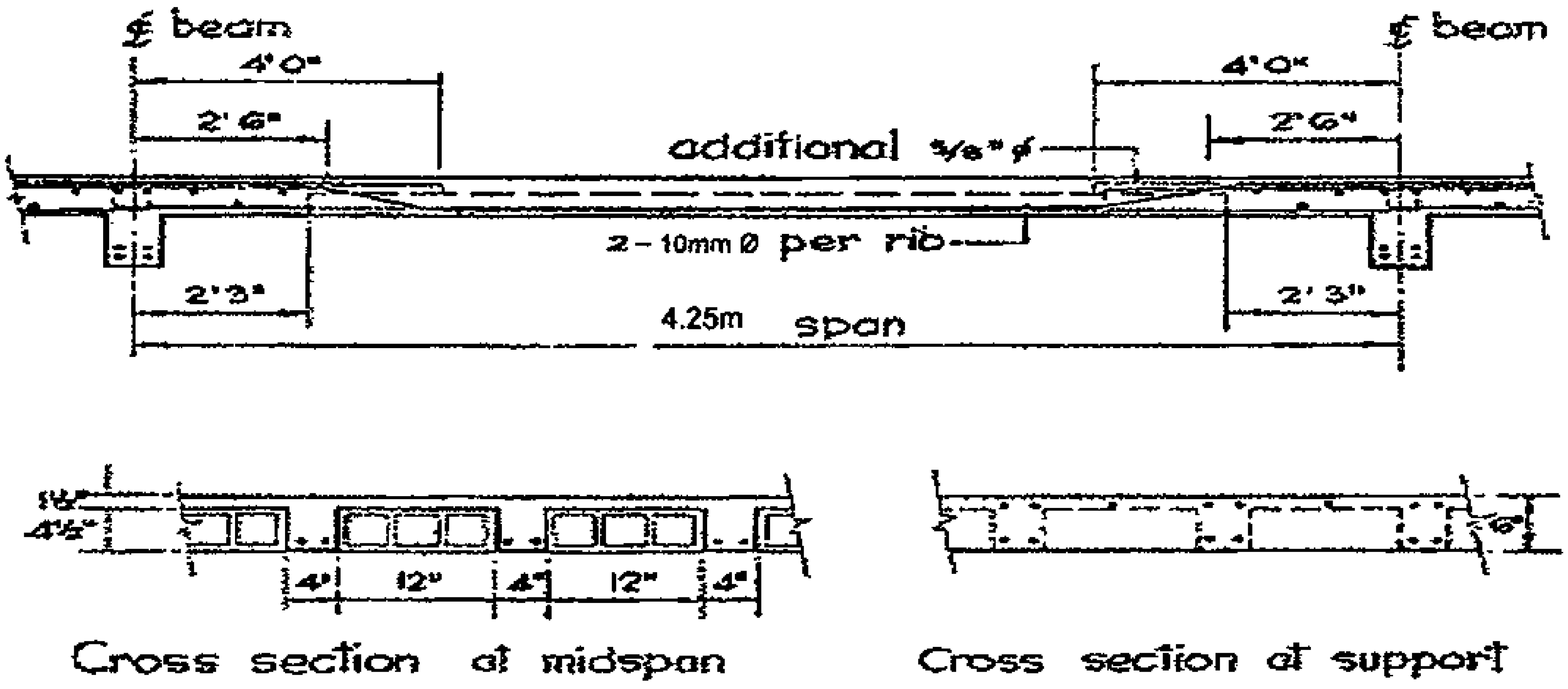
BRIDGES & STRUCTURES

CALCULATION SHEET

Project	Solar panel Installation	Sheet	6
Location	UCL - Archaeology Building	Rev	
Part of Structure	Roof slab - Hollow Clay Floor Construction	Date	

Reference	Calculations	Output



Typical details of hollow clay floor

$1\frac{1}{2}'' = 38.1\,mm$
$4\frac{1}{2}'' = 114.3\,mm$
$4'' = 101.6\,mm$
$12'' = 304.8\,mm$
$16'' = 406.4\,mm$
$6'' = 152.4\,mm$

<u>Nominal Dead load</u>

BS648:1964 Hollow clay blocks $4\frac{1}{2}''$ with Concrete ribs = $132\ kg/m^2$ or $1.32\ kN/m^2$

Concrete topping $= 24\ kN/m^3 \times 0.038\ m$ thick $= 0.91\ kN/m^2$

$2.23\ kN/m^2$

Waterproofing membrane say $= 0.1\ kN/m^2$

$2.33\ kN/m^2$

Live Load $= 0.75\ kN/m^2$ or $1.5\ kN/m^2$

<u>ULS Moment and Shear</u>

$W = (2.33 + 0.75) \times \overset{\gamma_f}{1.1} \times 0.406\,m = 1.38\ kN/m$

Conservative approach & assuming it's single span simply supported.

<u>$0.75\ kN/m^2$ Live Load</u>

Moment $= 1.38 \times 4.25^2 / 8 = 3.12\ kNm$

$V = 1.38 \times 4.25 / 2 = 2.93\ kN$

	Print name (once only)	Signature (once only)	Initials (every page)	Date (every page)
Prepared by	WL			11/11/10
Checked by	AC			11/11/10

BRIDGES & STRUCTURES

CALCULATION SHEET

Project		Sheet	7
Location		Rev	
Part of Structure		Date	

Reference	Calculations	Output

1.5 kN/m² Live Load

$W = (2.33 + 1.5) \times \overset{\gamma_f}{1.1} \times 0.406m = 1.71\,kN/m$

$Moment = 1.71 \times 4.25^2/8 = 3.86\,kNm$

$V = 1.71 \times 4.25/2 = 3.63\,kN$.

Conclusion

The Hollow clay floor is adequate for both 0.75 kN/m² or 1.5 kN/m² Live Load.

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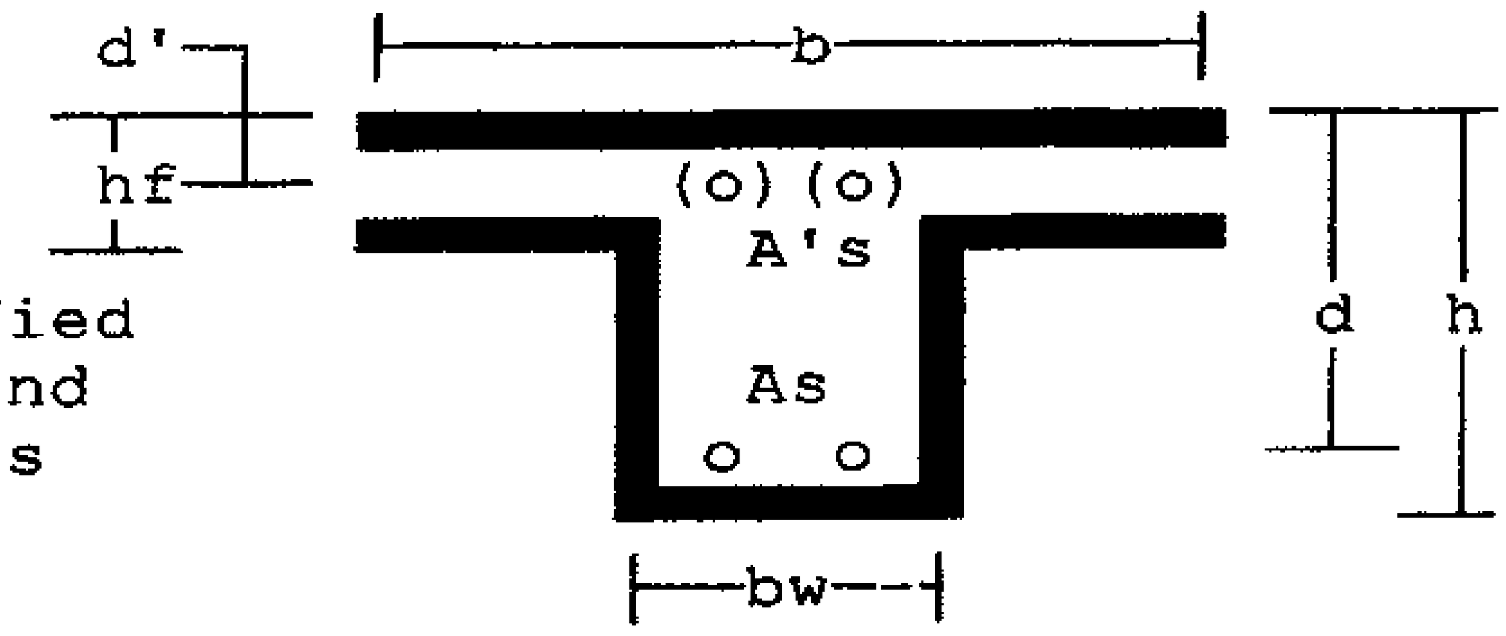
Prepared by	WL			11/11/10
Checked by	AL			11/11/10

11/11/2010

Location: UCL Archaeology Building Hollow Clay Floor Roof

Design of flanged beams to resist bending



Calculations are based on formulae in Clause 3.4.4.4 of BS8110: Part 1 and thus assume the use of a simplified rectangular concrete stress-block, and that the depth to the neutral axis is restricted to 0.5*d.

Design to BS8110(1997) with partial safety factor for steel gammaS=1.05
Moment before redistribution Mbef=3.86 kNm
Beam being analysed is considered as non-continuous.
Characteristic concrete strength fcu=30 N/mm^2
Max.aggregate size (for bar spc.) hagg=20 mm
Char.strength of long'l bars fy=250 N/mm^2
Longitudinal reinforcement is mild steel.
Diameter of tension bars dia=10 mm
Diameter of link legs dial=6 mm
Minimum nominal cover to all steel is 30 mm
Nominal concrete cover cover=20 mm
Overall depth of section h=152.4 mm
Effective depth of section d=121.4 mm
Breadth of flange b=406.4 mm
Breadth of rib bw=100 mm

Thickness of flange hf=38.1 mm

Longitudinal reinforcement

As Mbef=M, no redistribution has occurred.
Thus redistribution ratio betab=1
xmax=0.5*d=60.7 mm and K'=0.156 [Condition (b) in Clause 3.2.2.1].
Max.depth of conc.stress-block dpmax=0.9*xmax=0.9*60.7=54.63 mm
As max.depth of stress-block of 54.63 mm exceeds flange thickness of 38.1 mm , stress-block may extend below flange: check resistance moment.

Moment capacity of flange alone Mfl=b*hf*0.45*fcu*(d-hf/2)/1000000
=406.4*38.1*0.45*30*(121.4-38.1/2)/1000000
=21.394 kNm

As this exceeds M, lower edge of stress-block does not fall below underside of flange and section may be designed as a rectangular beam.

Office: 5658

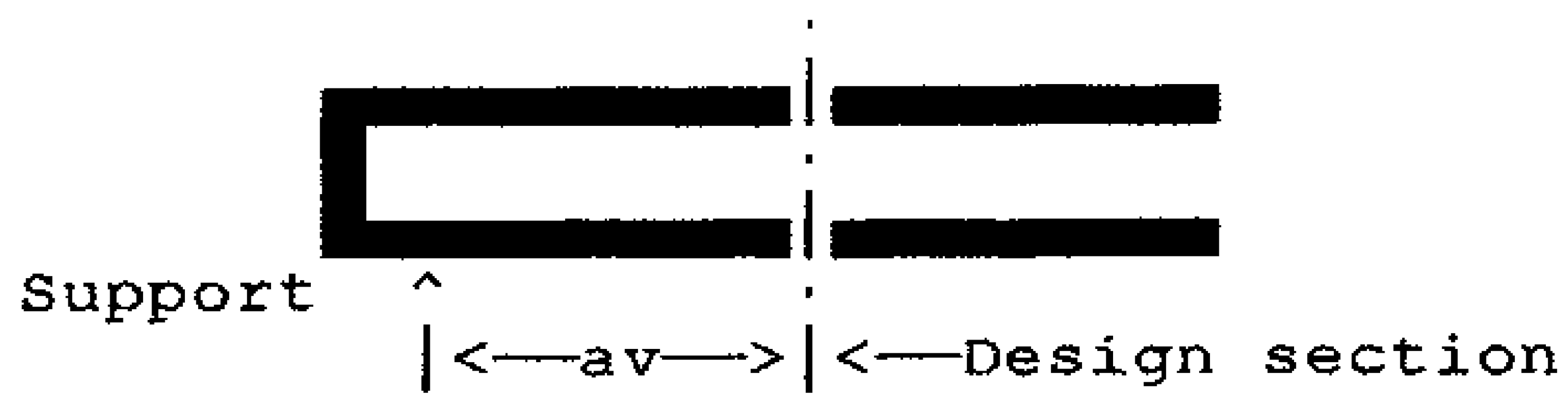
```
Applied-moment factor              K=M*1000*1000/(b*d^2*fcu)
                                    =3.86*1000*1000/(406.4*121.4^2*30)
                                    =0.021482
As this does not exceed the resistance-moment factor K' ( 0.156 ),
no compression steel is required.
and lever arm                      z=d*(0.5+SQR(0.25-K/0.9))
                                    =121.4*(0.5+SQR(0.25-0.021482/0.9))
                                    =118.43 mm
but this must not exceed 0.95*d = 0.95*121.4 =115.33 mm
so make                            z=115.33 mm
Area of tension steel required     As=M*1000*1000/(z*fy/gammaS)
                                     =3.86*1000*1000/(115.33*250/1.05)
                                       =140.57 mm2
Min.number of tension bars reqd.   nbrtmn=2
Number of tension bars provided    nbart=2
Area of tension steel provided      Aspr=157.08 mm2 ( 2 No. 10 mm bars)
Tension steel provided  per0=100*Aspr/(b*hf+bw*(h-hf))
                           =100*157.08/(406.4*38.1+100*(152.4-38.1))
                           =0.58364 % of gross section.
As this does not exceed 4% (see Clause 3.12.6.1), the design is O.K.
Percentage of tension steel provided per=100*Aspr/(bw*h)
                                        =100*157.08/(100*152.4)
                                        =1.0307 % of web area.
As this is not less than minimum of 0.32 %, the design is O.K.
Maximum clear distance allowed between tension bars according to
Clause 3.12.11.2.4 is given by
                           cdist=47000*3*Aspr*betab/(2*fy*As)
                                =47000*3*157.08*1/(2*250*140.57)
                                =315.12 mm
but as this exceeds 300 mm , take cdist=300 mm

    TENSION                            Characteristic strength  250 N/mm2
    REINFORCEMENT                     Diameter of bars          10 mm
    SUMMARY                           Number of bars            2
                                      arranged in a single layer
                                      Cover to all steel        20 mm
                                        Area of steel required    140.57 mm2
                                        Area of steel provided    157.08 mm2
                                      Percentage provided:
                                        of gross section        0.58364 %
                                        of web area             1.0307 %
                                      Weight of steel provided 1.2331 kg/m
                                      Max.permissible spacing  300 mm
                                      Link size assumed        6 mm
```

Shear in rectangular and flanged beams, with av enhancement

Location: UCL Archaeology Building Hollow Clay Floor Roof.

The shear calculations are in accordance with the requirements of Clause 3.4.5



Design to BS8110(1997) with partial safety factor for steel gammaS=1.05
Shear force due to ultimate loads V=3.63 kN
Overall depth of section h=152.4 mm
Effective depth of section d=121.4 mm
Effective breadth for shear bv=100 mm
Characteristic concrete strength fcu=30 N/mm^2
Distance to support av=100 mm
Diameter of compression bars diac=0 mm
Diameter of tension bars dia=10 mm
Diameter of link legs dial=6 mm
Number of tension bars at section nbars=2

Char.strength of link steel fyv=250 N/mm^2
Area of longitudinal steel As=nbars*PI*dia^2/4
=2*3.1416*10^2/4
=157.08 mm^2

Percentages of longitudinal steel:
In terms of (bv*d): per=100*As/(bv*d)
=100*157.08/(100*121.4)
=1.2939 %

In terms of (bv*h): perg=100*As/(bv*h)
=100*157.08/(100*152.4)
=1.0307 %

From Table 3.8 with fcu=25 N/mm2, effective depth=121.4 mm and steel percentage=1.2939 %, design concrete shear stress vc=0.91641 N/mm2. Since characteristic concrete strength is between 25 and 40 N/mm2, increase vc according to formula in footnote below Table 3.8. Then
vc=vc25*(fcu/25)^(1/3)
=0.91641*(30/25)^(1/3)
=0.97384 N/mm^2

Since av is less than 2*d (see Clause 3.4.5.8),
enhanced value of vc=vc*2*d/av
=0.97384*2*121.4/100
=2.3645 N/mm^2

Design shear stress v=V*1000/(bv*d)
=3.63*1000/(100*121.4)
=0.29901 N/mm^2

Number of legs to be provided nlegs=2
Total leg area Asv=nlegs*(PI*dial^2/4)
=2*(3.1416*6^2/4)
=56.549 mm^2

Since v (i.e. 0.29901 N/mm2) does not exceed (vc+0.4) (i.e. 2.7645 N/mm2),
minimum spacing of links sv=fyv/gammaS*Asv/(0.4*bv)
=250/1.05*56.549/(0.4*100)
=336.6 mm
=320 mm (rounded)

Proposed spacing sv greater than 0.75*d= 0.75*121.4 =91.05 mm.
Reset link spacing to limiting value of 90 mm (rounded).

Use 6 mm links (two legs), spaced at 90 mm centres along beam.
Approx.total weight of links 0.8 kg per metre

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BRIDGES & STRUCTURES

CALCULATION SHEET

Project	Solar Panel Installation.	Sheet	12
Location	UCL Archaeology Building	Rev	
Part of Structure	Solar Panel up-lift due to Wind	Date	

Reference	Calculations	Output
	Unit Weight of Solar Panel = 21kg per 1.5m × 1.0m	
	Total weight = 21kg × 10m long = 210 kg or	2.1 kN
	Total uplift force due to wind = 0.597 kN/m² × 1.5 m × 10m long = 9 kN × 1.2 (γf) = 10.8 kN	
	Counter Weight needed = 10.8 kN − 2.1kN = 8.7 kN or 870 kg	
	Try 152 × 152 × 37 Kg/m UC	
	Total length required = 870 / 37 = 24 m length	
	Use 3 × 4m = 12m; 2 × 5m = 10m; 22m.	
	Bracing use 50×50×8 EA (5.9kg/m).	
	Total weight = 5.9kg/m × 12m = 70.8 kg	
	Use 2 × 6m = 12m.	
	Total Counter Weight = 37kg/m × 22m	= 814 kg
	5.9kg/m × 12 m	= 70.8 kg
		884.8 kg > 870 kg ∴ O.K.

	Print name (once only)	Signature (once only)	Initials (every page)	Date (every page)
Prepared by	WL			11/11/10
Checked by	AL			11/11/10

Calculations

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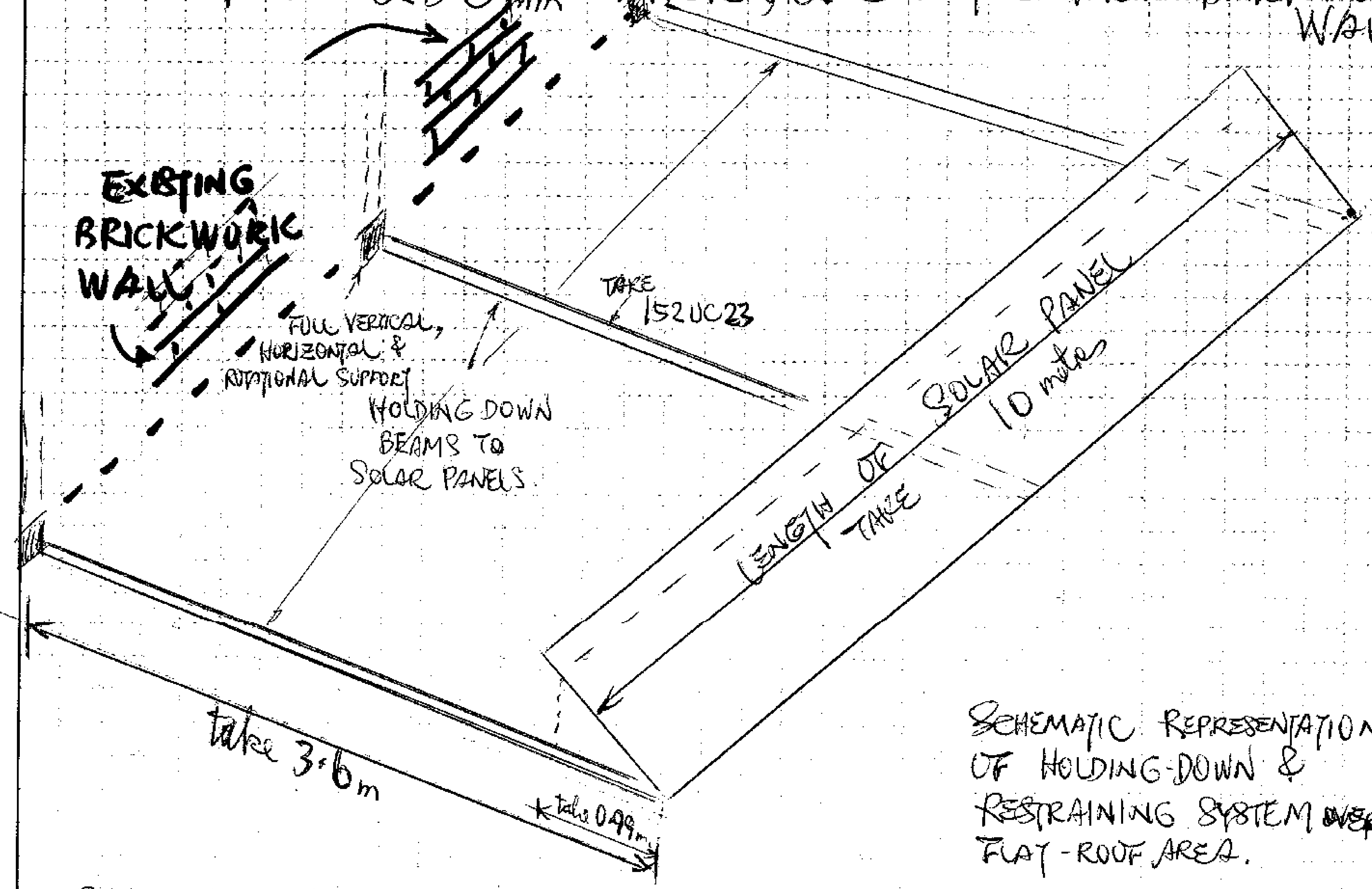
CHARTERED ENGINEERS
BUILDING
DESIGN
CONSULTANTS

at THE ARCHAEOLOGY BUILDING.

ROW OF SOLAR PANELS AWAY FROM THE UPSTAND REINFORCED CONCRETE BEAMS.

- MAIN ANCHORAGE ELEMENT = EXISTING 9" or 230mm THICK, 2·5 METRES HIGH BRICKWORK WALL.



SCHEMATIC REPRESENTATION OF HOLDING-DOWN & RESTRAINING SYSTEM OVER FLAT-ROOF AREA.

- PEAK UPLIFTING WIND LOAD (STORM FORCE CONDITION)

take UPWARD WIND PRESSURE ON SOLAR PANEL AT 0·45 kN/m² (BS 6399: PART 2)

↑ WIND FORCE ON ROW OF PANELS take $0{\cdot}99_m$ (WIDTH) $\times$ 10_m (LENGTH) $\times$ $0{\cdot}45\,\frac{kN}{m^2}$ = 4·46 kN.

CONSIDER INTERNAL 152UC BEAM, TO RESIST VERTICAL LIFTING FORCE OF 2·23 kN

CORRESPONDING BENDING MOMENT AT WALL SUPPORT $2{\cdot}23\,kN \times (3{\cdot}6_m - 0{\cdot}5_m)$
= 6·9 kNm

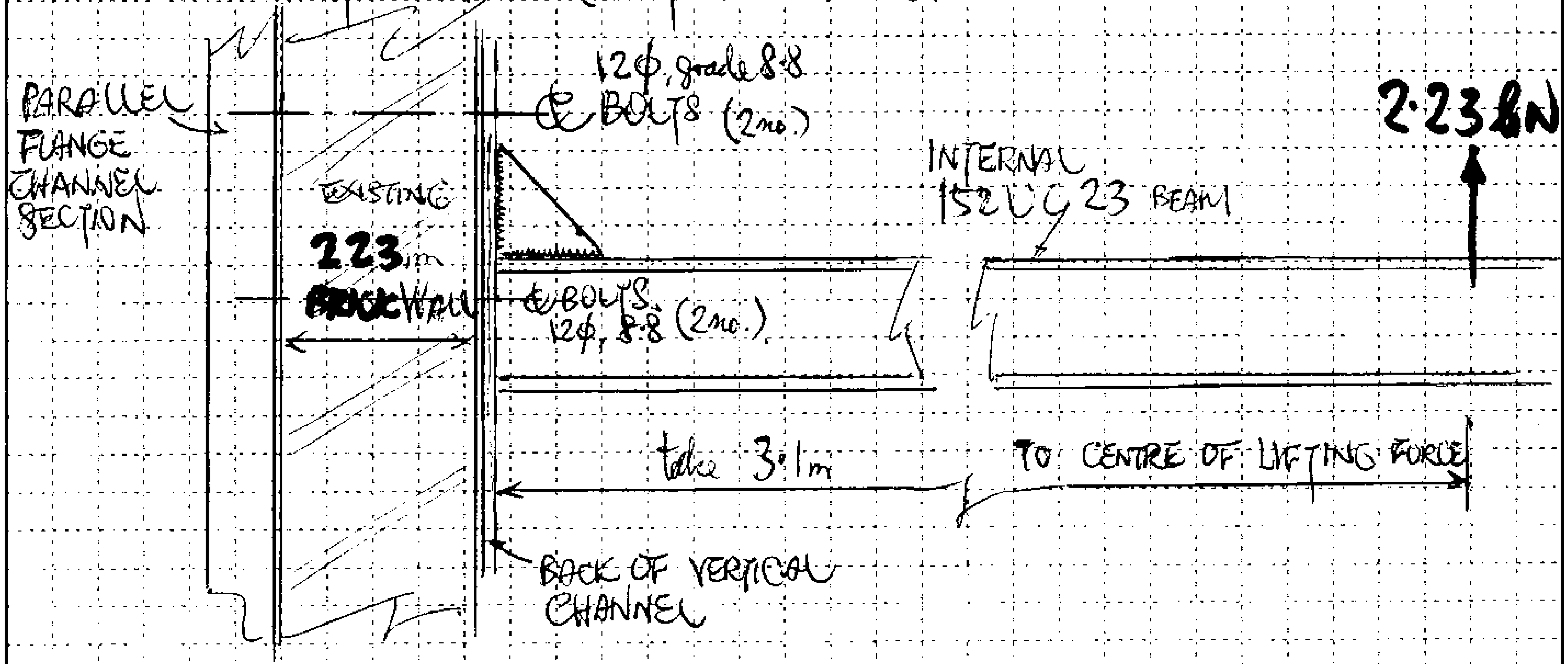
Calculations

WT &L — CHARTERED ENGINEERS BUILDING DESIGN CONSULTANTS

Job Ref:				
Calc.By	Checked	Date	Project	Page No 4

at THE ARCHAEOLOGY BUILDING.

HOLDING-DOWN, RESTRAINING SYSTEM TO LONG ROW OF SOLAR PANELS ON THE FLAT ROOF (AWAY FROM THE UPSTAND BEAMS)



* CONSIDER VERTICAL RESISTANCE AT MASONRY SUPPORT

VERTICAL SHEAR CAPACITY OF 4no. 12φ grade 8.8 bolts:

$4 \times 15.8\ kN = 63.2\ kN$. (greater than 2.23 kN)

VERTICAL SHEAR CAPACITY OF 152UC23 WEB PLATE

$152_{mm} \times 5.8_{mm} \times 140\ N/mm^2 = 123.4\ kN$ (greater than 2.23 kN)

* CONSIDER RESISTANCE OF FIXED END OF BEAM AGAINST ROTATION & BENDING.

APPLIED BENDING MOMENT DUE TO WIND LIFT at ULTIMATE LIMIT STATE.

$\therefore\ 2.23\ kN \times 3.1m = 6.9\ kNm$.

MOMENT OF RESISTANCE DUE TO 2no. 12φ ANCHOR BOLTS (SAFE) $2 \times 18.9 \times 0.18 = 6.9\ kNm$ O.K.

MOMENT OF RESISTANCE OF 152UC23 SECTION $275\ N/mm^2 \times 182 \times 10^3\ mm^3 = 50.1\ kNm$ (greater than 6.9 kNm)

$\therefore$ THE FIXING AT SUPPORT IS SAFE.

Calculations

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Calc.By WT	Checked L	Date Nov10	Project UCL AB	Page No 15

WT & L — Chartered Engineers, Building Design Consultants

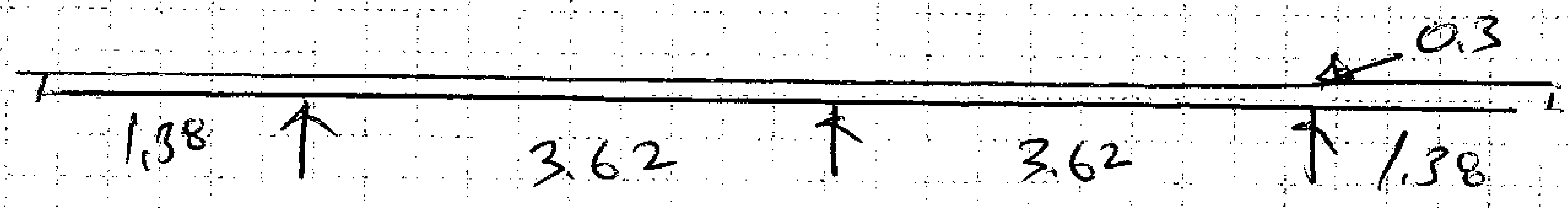
Holding Down Beam Design: (1)

Span = 10m

Loading:

Solar panels = 21kg ~ 0.21kN / 1.5 × 1 = 0.14 kN/m²

= 0.14 × 1.5 = 0.21kN/m ~ 0.3kN/m



UB 152×152×23UC

Design Analysis See computer output:

Check Uplift :

Uplift Pressure = 0.448 kN/m²

= 0.448 × 10 × 1.5 = 6.72 kN

Total Downward Force = 23kg × 10 = 230kg = 2.3kN × 2 = 4.6 kN + 0.21×10

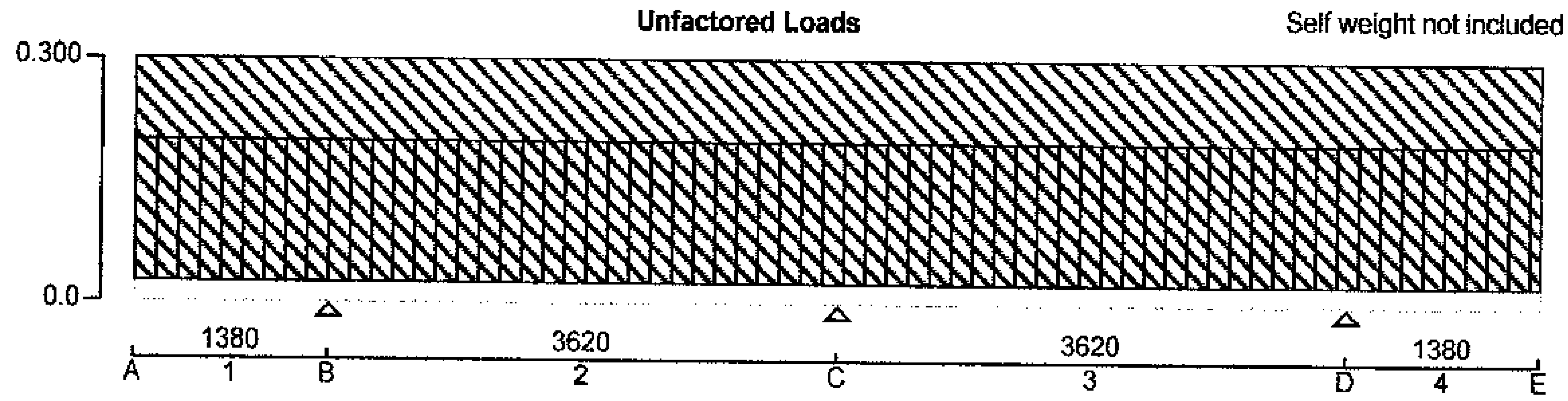
= 6.7kN < 6.72

∴ Use steel saddle for holding down Beam to bolt to Upstand Beams

M12 shear capacity = 31.6kN > 6.72kN

∴ ok

(B) Form No Q16
02.09.97



CONTINUOUS BEAM ANALYSIS - INPUT

BEAM DETAILS

Number of spans = **4**

Material Properties:

Modulus of elasticity = **205** kN/mm^2 Material density = **7860** kg/m^3

Support Conditions:

Support A	Vertically **"Free"**	Rotationally **"Free"**
Support B	Vertically **"Restrained"**	Rotationally **"Free"**
Support C	Vertically **"Restrained"**	Rotationally **"Free"**
Support D	Vertically **"Restrained"**	Rotationally **"Free"**
Support E	Vertically **"Free"**	Rotationally **"Free"**

Span Definitions:

Span 1	Length = **1380** mm	Cross-sectional area = **2925** mm^2	Moment of inertia = **12.5×10^6** mm^4
Span 2	Length = **3620** mm	Cross-sectional area = **2925** mm^2	Moment of inertia = **12.5×10^6** mm^4
Span 3	Length = **3620** mm	Cross-sectional area = **2925** mm^2	Moment of inertia = **12.5×10^6** mm^4
Span 4	Length = **1380** mm	Cross-sectional area = **2925** mm^2	Moment of inertia = **12.5×10^6** mm^4

LOADING DETAILS

Beam Loads:

Load 1 UDL Dead load **0.2** kN/m

Load 2 UDL Live load **0.3** kN/m

LOAD COMBINATIONS

Load combination 1

Span 1	1.4*Dead + 1.6*Live
Span 2	1.4*Dead + 1.6*Live
Span 3	1.4*Dead + 1.6*Live
Span 4	1.4*Dead + 1.6*Live

CONTINUOUS BEAM ANALYSIS - RESULTS

Unfactored support reactions

Support A	Dead load = **0.0** kN	Live load = **0.0** kN	Wind load = **0.0** kN	Other load = **0.0** kN
Support B	Dead load = **-0.6** kN	Live load = **-0.9** kN	Wind load = **0.0** kN	Other load = **0.0** kN
Support C	Dead load = **-0.7** kN	Live load = **-1.1** kN	Wind load = **0.0** kN	Other load = **0.0** kN
Support D	Dead load = **-0.6** kN	Live load = **-0.9** kN	Wind load = **0.0** kN	Other load = **0.0** kN
Support E	Dead load = **0.0** kN	Live load = **0.0** kN	Wind load = **0.0** kN	Other load = **0.0** kN

Support Reactions - Combination Summary

Support A	Max react = **0.0** kN	Min react = **0.0** kN	Max mom = **0.0** kNm	Min mom = **0.0** kNm
Support B	Max react = **-2.4** kN	Min react = **-2.4** kN	Max mom = **0.0** kNm	Min mom = **0.0** kNm
Support C	Max react = **-2.8** kN	Min react = **-2.8** kN	Max mom = **0.0** kNm	Min mom = **0.0** kNm
Support D	Max react = **-2.4** kN	Min react = **-2.4** kN	Max mom = **0.0** kNm	Min mom = **0.0** kNm

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Support E Max react = **0.0** kN Min react = **0.0** kN Max mom = **0.0** kNm Min mom = **0.0** kNm

Support Deflections - Combination Summary

Support A	Max def = **0.1** mm	Min def = **0.1** mm	Max rot = **0.0** deg	Min rot = **0.0** deg
Support B	Max def = **0.0** mm	Min def = **0.0** mm	Max rot = **0.0** deg	Min rot = **0.0** deg
Support C	Max def = **0.0** mm	Min def = **0.0** mm	Max rot = **0.0** deg	Min rot = **0.0** deg
Support D	Max def = **0.0** mm	Min def = **0.0** mm	Max rot = **0.0** deg	Min rot = **0.0** deg
Support E	Max def = **0.1** mm	Min def = **0.1** mm	Max rot = **0.0** deg	Min rot = **0.0** deg

Beam Max/Min results - Combination Summary

Maximum shear = **1.4** kN — Minimum shear F_{min} = **-1.4** kN

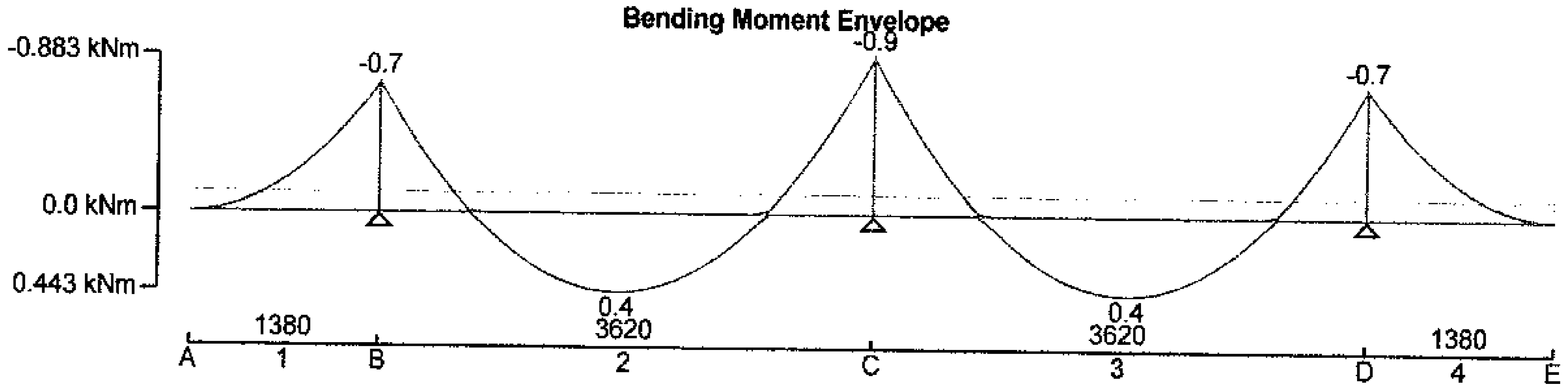
Maximum moment = **0.4** kNm — Minimum moment = **-0.9** kNm

Maximum deflection = **0.1** mm — Minimum deflection = **0.0** mm

Span Max/Min results - Combination Summary

Span 1	Maximum shear = **0.0** kN at **0.000** m	Minimum shear = **-1.0** kN at **1.380** m
	Maximum moment = **0.0** kNm at **0.000** m	Minimum moment = **-0.7** kNm at **1.380** m
	Maximum deflection = **0.1** mm at **0.000** m	Minimum deflection = **0.0** mm at **1.232** m
Span 2	Maximum shear = **1.3** kN at **0.000** m	Minimum shear = **-1.4** kN at **3.620** m
	Maximum moment = **0.4** kNm at **1.752** m	Minimum moment = **-0.9** kNm at **3.620** m
	Maximum deflection = **0.1** mm at **1.756** m	Minimum deflection = **0.0** mm at **0.000** m
Span 3	Maximum shear = **1.4** kN at **0.000** m	Minimum shear = **-1.3** kN at **3.620** m
	Maximum moment = **0.4** kNm at **1.868** m	Minimum moment = **-0.9** kNm at **0.000** m
	Maximum deflection = **0.1** mm at **1.864** m	Minimum deflection = **0.0** mm at **0.000** m
Span 4	Maximum shear = **1.0** kN at **0.000** m	Minimum shear = **0.0** kN at **1.380** m
	Maximum moment = **0.0** kNm at **1.380** m	Minimum moment = **-0.7** kNm at **0.000** m
	Maximum deflection = **0.1** mm at **1.380** m	Minimum deflection = **0.0** mm at **0.148** m

Bending Moment Envelope



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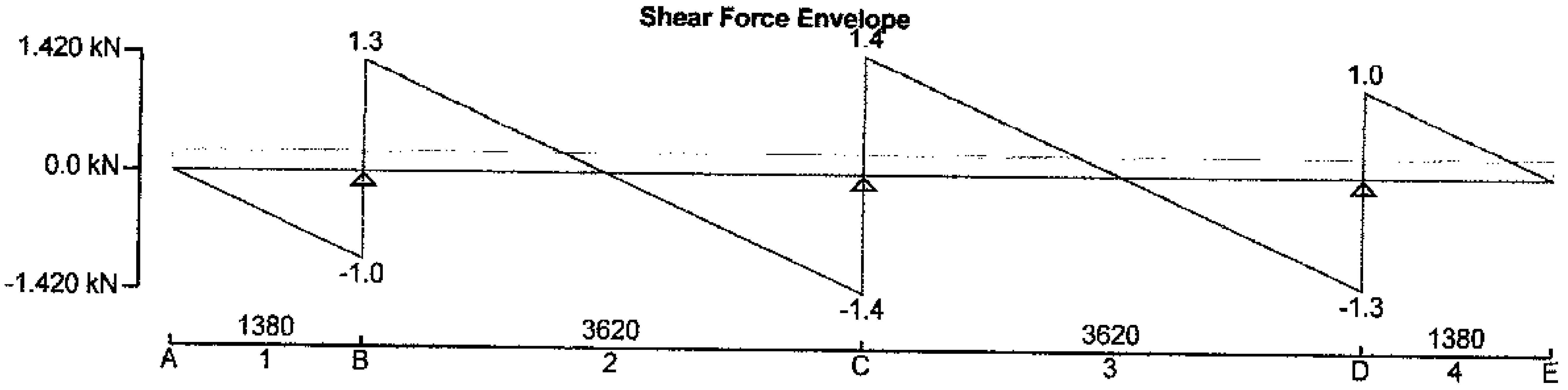
Shear Force Envelope



SPAN RESULTS - SPAN 1

x (m)	M_{max}(kNm)	M_{min}(kNm)	F_{max} (kN)	F_{min} (kN)	δ_{max} (mm)	δ_{min} (mm)
0.000	0.00	0.00	0.00	0.00	0.1	0.0
0.173	0.00	-0.01	0.00	-0.13	0.1	0.0
0.345	0.00	-0.05	0.00	-0.26	0.1	0.0
0.518	0.00	-0.10	0.00	-0.39	0.0	0.0
0.690	0.00	-0.18	0.00	-0.52	0.0	0.0
0.863	0.00	-0.28	0.00	-0.66	0.0	0.0
1.035	0.00	-0.41	0.00	-0.79	0.0	0.0
1.208	0.00	-0.55	0.00	-0.92	0.0	0.0
1.232	0.00	-0.58	0.00	-0.94	0.0	0.0
1.380	0.00	-0.72	0.00	-1.05	0.0	0.0

SPAN RESULTS - SPAN 2

x (m)	M_{max}(kNm)	M_{min}(kNm)	F_{max} (kN)	F_{min} (kN)	δ_{max} (mm)	δ_{min} (mm)
0.000	0.00	-0.72	1.33	0.00	0.0	0.0
0.453	0.00	-0.20	0.99	0.00	0.0	0.0
0.905	0.17	0.00	0.64	0.00	0.1	0.0
1.358	0.38	0.00	0.30	0.00	0.1	0.0
1.752	0.44	0.00	0.00	0.00	0.1	0.0
1.756	0.44	0.00	0.00	0.00	0.1	0.0
1.810	0.44	0.00	0.00	-0.04	0.1	0.0
2.263	0.34	0.00	0.00	-0.39	0.1	0.0
2.715	0.09	0.00	0.00	-0.73	0.1	0.0
3.168	0.00	-0.35	0.00	-1.08	0.0	0.0
3.620	0.00	-0.88	0.00	-1.42	0.0	0.0

SPAN RESULTS - SPAN 3

x (m)	M_{max}(kNm)	M_{min}(kNm)	F_{max} (kN)	F_{min} (kN)	δ_{max} (mm)	δ_{min} (mm)
0.000	0.00	-0.88	1.42	0.00	0.0	0.0
0.000	0.00	-0.88	1.42	0.00	0.0	0.0
0.453	0.00	-0.32	1.08	0.00	0.0	0.0
0.905	0.09	0.00	0.73	0.00	0.1	0.0

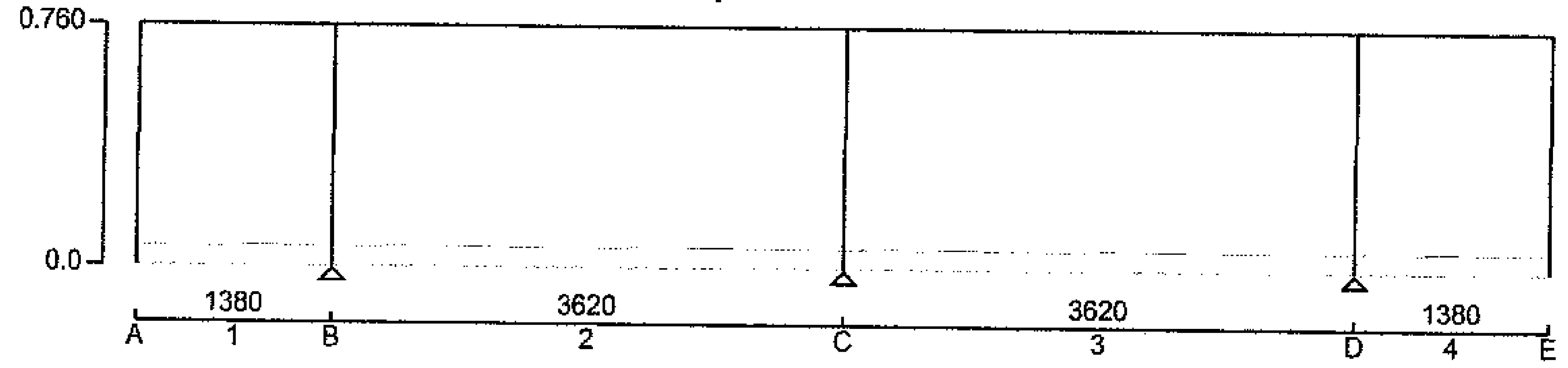
x (m)	M_{max}(kNm)	M_{min}(kNm)	F_{max} (kN)	F_{min} (kN)	δ_{max} (mm)	δ_{min} (mm)
1.358	0.34	0.00	0.39	0.00	0.1	0.0
1.810	0.44	0.00	0.04	0.00	0.1	0.0
1.864	0.44	0.00	0.00	0.00	0.1	0.0
1.868	0.44	0.00	0.00	0.00	0.1	0.0
2.263	0.38	0.00	0.00	-0.30	0.1	0.0
2.715	0.17	0.00	0.00	-0.64	0.1	0.0
3.168	0.00	-0.20	0.00	-0.99	0.0	0.0
3.620	0.00	-0.72	0.00	-1.33	0.0	0.0

SPAN RESULTS - SPAN 4

x (m)	M_{max}(kNm)	M_{min}(kNm)	F_{max} (kN)	F_{min} (kN)	δ_{max} (mm)	δ_{min} (mm)
0.000	0.00	-0.72	1.05	0.00	0.0	0.0
0.148	0.00	-0.58	0.94	0.00	0.0	0.0
0.173	0.00	-0.55	0.92	0.00	0.0	0.0
0.345	0.00	-0.41	0.79	0.00	0.0	0.0
0.518	0.00	-0.28	0.66	0.00	0.0	0.0
0.690	0.00	-0.18	0.52	0.00	0.0	0.0
0.863	0.00	-0.10	0.39	0.00	0.0	0.0
1.035	0.00	-0.05	0.26	0.00	0.1	0.0
1.208	0.00	-0.01	0.13	0.00	0.1	0.0
1.380	0.00	0.00	0.00	0.00	0.1	0.0

RESULTS FOR COMBINATION 1

Load Envelope - Combination 1



Support Reactions and Deflections - Combination 1 :

Support A	Reaction = **0.0** kN	Moment = **0.0** kNm	Deflection = **0.1** mm	Rotation = **-0.01** deg
Support B	Reaction = **-2.4** kN	Moment = **0.0** kNm	Deflection = **0.0** mm	Rotation = **0.00** deg
Support C	Reaction = **-2.8** kN	Moment = **0.0** kNm	Deflection = **0.0** mm	Rotation = **0.00** deg
Support D	Reaction = **-2.4** kN	Moment = **0.0** kNm	Deflection = **0.0** mm	Rotation = **0.00** deg
Support E	Reaction = **0.0** kN	Moment = **0.0** kNm	Deflection = **0.1** mm	Rotation = **0.01** deg

Beam Max/Min results - Combination 1 :

Maximum shear = **1.4** kN — Minimum shear = **-1.4** kN

Maximum moment = **0.4** kNm — Minimum moment = **-0.9** kNm

Maximum deflection = **0.1** mm — Minimum deflection = **0.0** mm

Span Max/Min results - Combination 1 :

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Span 1 Maximum shear = **0.0** kN at **0.000** m — Minimum shear = **-1.0** kN at **1.380** m

Maximum moment = **0.0** kNm at **0.000** m — Minimum moment = **-0.7** kNm at **1.380** m

Maximum deflection = **0.1** mm at **0.000** m — Minimum deflection = **0.0** mm at **1.232** m

Span 2 Maximum shear = **1.3** kN at **0.000** m — Minimum shear = **-1.4** kN at **3.620** m

Maximum moment = **0.4** kNm at **1.752** m — Minimum moment = **-0.9** kNm at **3.620** m

Maximum deflection = **0.1** mm at **1.756** m — Minimum deflection = **0.0** mm at **0.000** m

Span 3 Maximum shear = **1.4** kN at **0.000** m — Minimum shear = **-1.3** kN at **3.620** m

Maximum moment = **0.4** kNm at **1.868** m — Minimum moment = **-0.9** kNm at **0.000** m

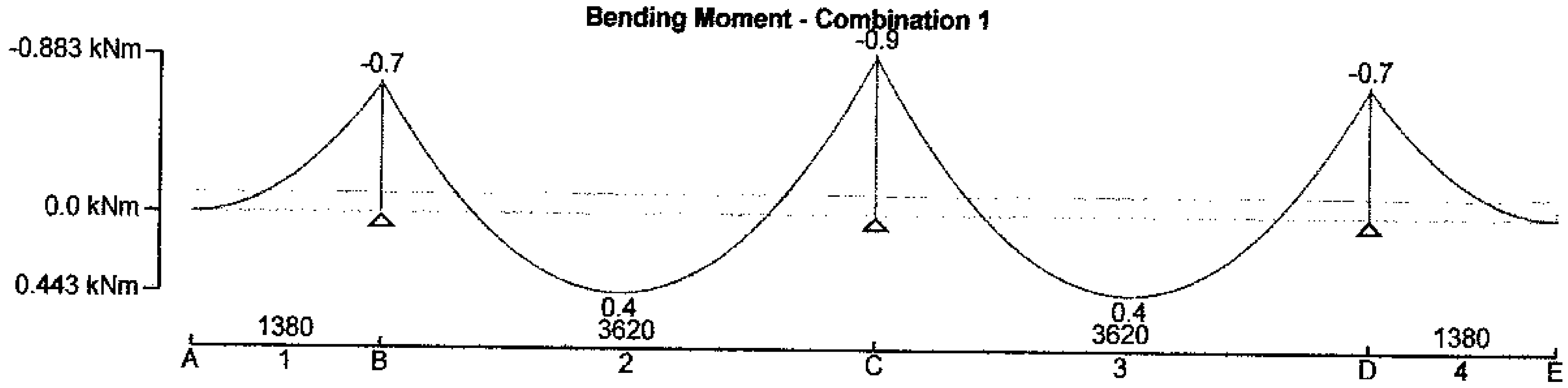
Maximum deflection = **0.1** mm at **1.864** m — Minimum deflection = **0.0** mm at **0.000** m

Span 4 Maximum shear = **1.0** kN at **0.000** m — Minimum shear = **0.0** kN at **1.380** m

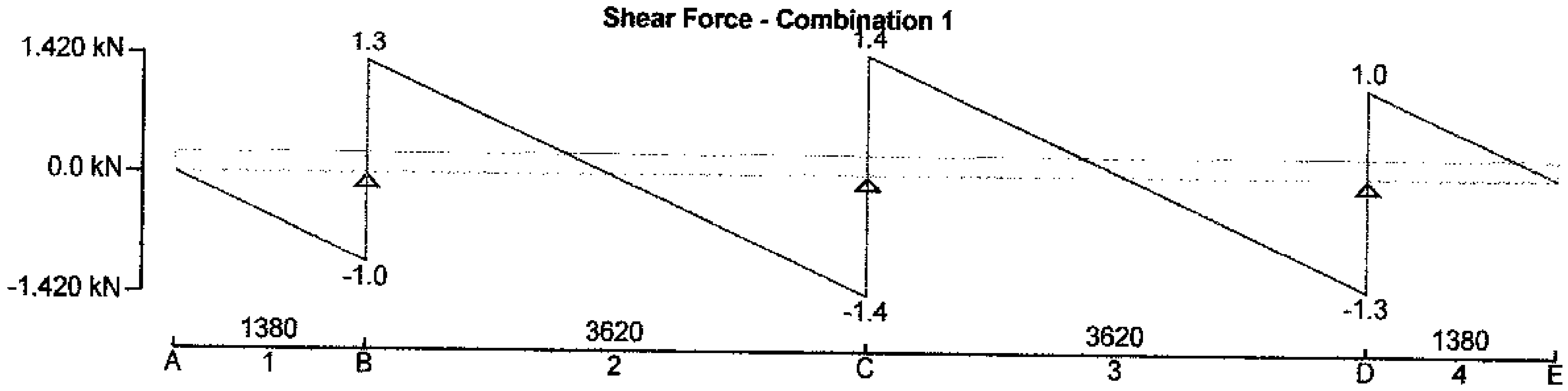
Maximum moment = **0.0** kNm at **1.380** m — Minimum moment = **-0.7** kNm at **0.000** m

Maximum deflection = **0.1** mm at **1.380** m — Minimum deflection = **0.0** mm at **0.148** m

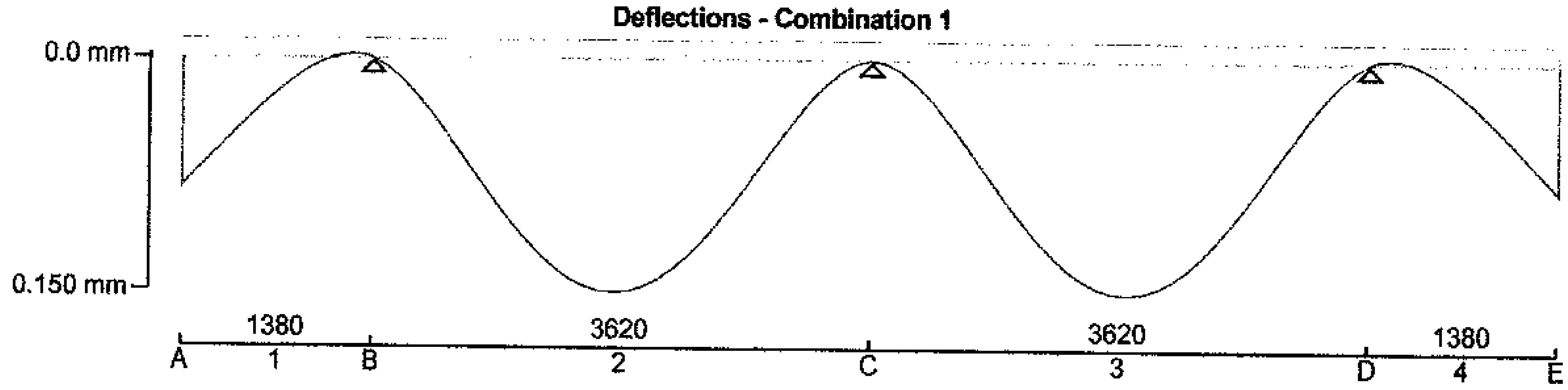
Bending Moment - Combination 1



Shear Force - Combination 1



Deflections - Combination 1



Span Results - Span 1 - Combination

x (m)	F_{left} (kN)	F_{right} (kN)	M (kNm)	δ (mm)
0.000	0.00		0.00	0.1
0.345	-0.26		-0.05	0.1
0.690	-0.52		-0.18	0.0
1.035	-0.79		-0.41	0.0
1.232	-0.94		-0.58	0.0
1.380	-1.05		-0.72	0.0

Span Results - Span 2 - Combination

x (m)	F_{left} (kN)	F_{right} (kN)	M (kNm)	δ (mm)
0.000	1.33		-0.72	0.0
0.905	0.64		0.17	0.1
1.752	0.00		0.44	0.1
1.756	0.00		0.44	0.1
1.810	-0.04		0.44	0.1
2.715	-0.73		0.09	0.1
3.620	-1.42		-0.88	0.0

Span Results - Span 3 - Combination

x (m)	F_{left} (kN)	F_{right} (kN)	M (kNm)	δ (mm)
0.000	1.42		-0.88	0.0
0.905	0.73		0.09	0.1
1.810	0.04		0.44	0.1
1.864	0.00		0.44	0.1
1.868	0.00		0.44	0.1
2.715	-0.64		0.17	0.1
3.620	-1.33		-0.72	0.0

Span Results - Span 4 - Combination

x (m)	F_{left} (kN)	F_{right} (kN)	M (kNm)	δ (mm)
0.000	1.05		-0.72	0.0
0.148	0.94		-0.58	0.0
0.345	0.79		-0.41	0.0
0.690	0.52		-0.18	0.0
1.035	0.26		-0.05	0.1
1.380	0.00		0.00	0.1

DESIGN FOR WORST CASE

Member design checks for a simply-supported steel beam to BS 5950 (with LTB)

Summary of results

Material	Grade = **"S275"**	p_y = **275** N/mm^2	
Section	**"UC 152x152x23"**	Classification	**"Semi-Compact"**

Check	*Load*	*Capacity*	*Notes*	*Result*
Deflection	δ_{y_max} = **0.1** mm	δ_{lim} = **10.0** mm	**Span / 360 or 10.0 mm**	**Pass**
Shear	F_{vy} = **1.4** kN	P_{vy} = **145.8** kN	**Low shear**	**Pass**
Moment	M_x = **0.9** kNm	M_{cx} = **45.1** kNm	**Low shear**	**Pass**
LTB	M_{LT} = **0.9** kNm	M_b / m_{LT} = **33.1** kNm	L_{E_LT} = **3.6** m m_{LT} = **1.00**	**Pass**

Appendix C

Photos

EXISITNG UPSTAND BEAMS

EXISITNG SAFETY HANDRAIL

EXISITNG MASONRY ENCLOSURE WALL AND SAFETY HANDRAIL

GENERAL VIEW OF EXISITNG ROOF

EXISTING DOWNSTAND BEAM BELOW ROOF SLAB

EXISTING DOWNSTAND BEAM BELOW ROOF SLAB

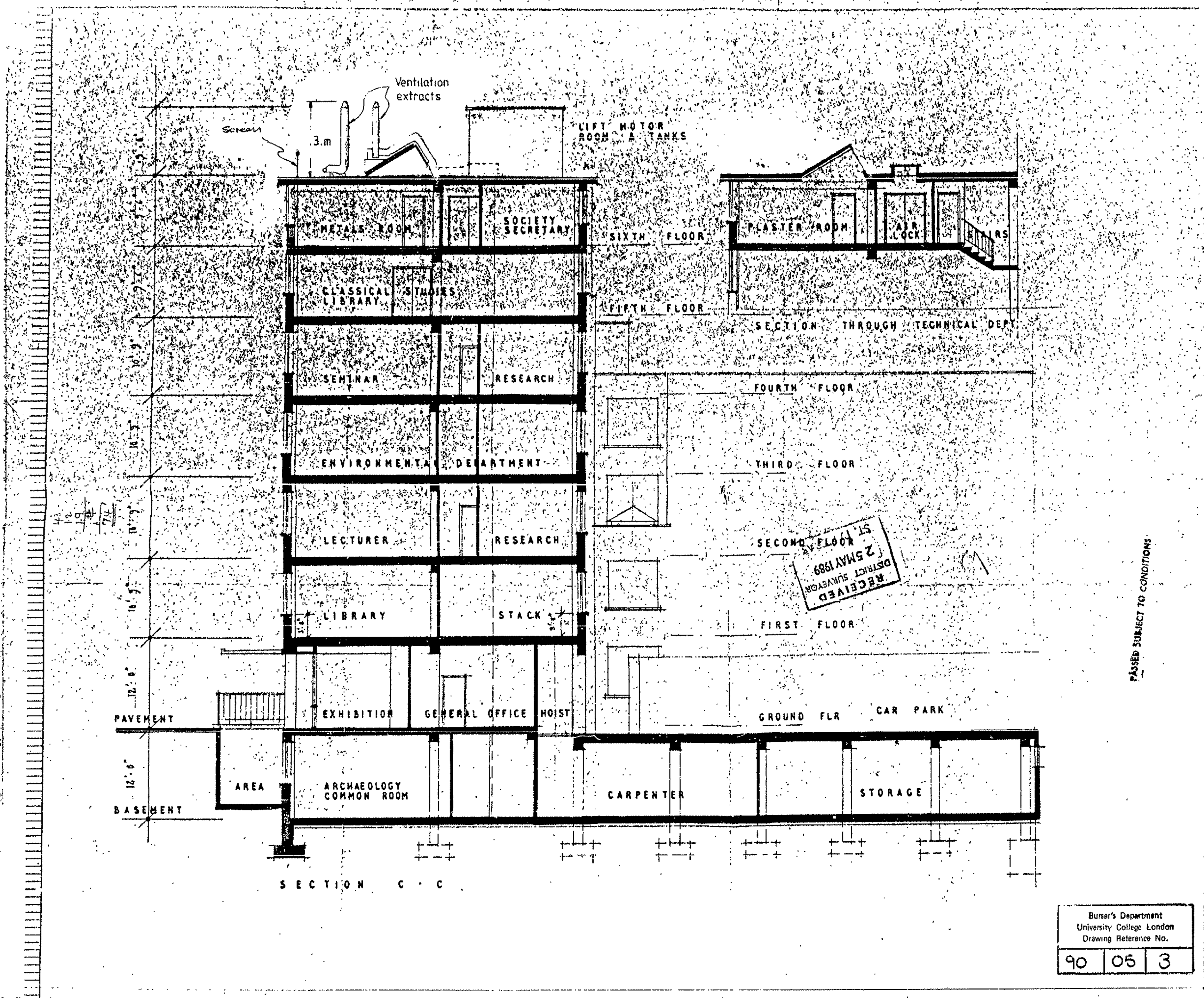
<u>GENERAL VIEW OF EXISITNG ROOF</u>

<u>EXISTING ROOF SLAB CONSTRUCTED FROM REINFORCED CONCRETE BEAM RIBS SPACED APART BY HOLLOW CLAY POTS</u>

EXISTING CIRCULAR CLOUMN BELOW ROOF LEVEL

Appendix D

Archive Drawings Supplied By The Client



NOTES

DO NOT SCALE OFF THIS DRAWING

ALL DIMENSIONS TO BE CHECKED ON SITE PRIOR TO ANY WORK BEING PUT IN HAND

ANY DISCREPANCIES TO BE BROUGHT IMMEDIATELY TO THE ATTENTION OF THE SUPERVISING OFFICER

REVISIONS

Letter	Description	Date
	P478 /89S	

THE BURSAR'S OFFICE
UNIVERSITY COLLEGE LONDON
GOWER STREET WC1E 6BT

drawn:	traced: G.P.J.
date: MAY 89	scale: 1/100

INSTITUTE OF ARCHAEOLOGY
29-35 GORDON SQUARE
SECTIONS

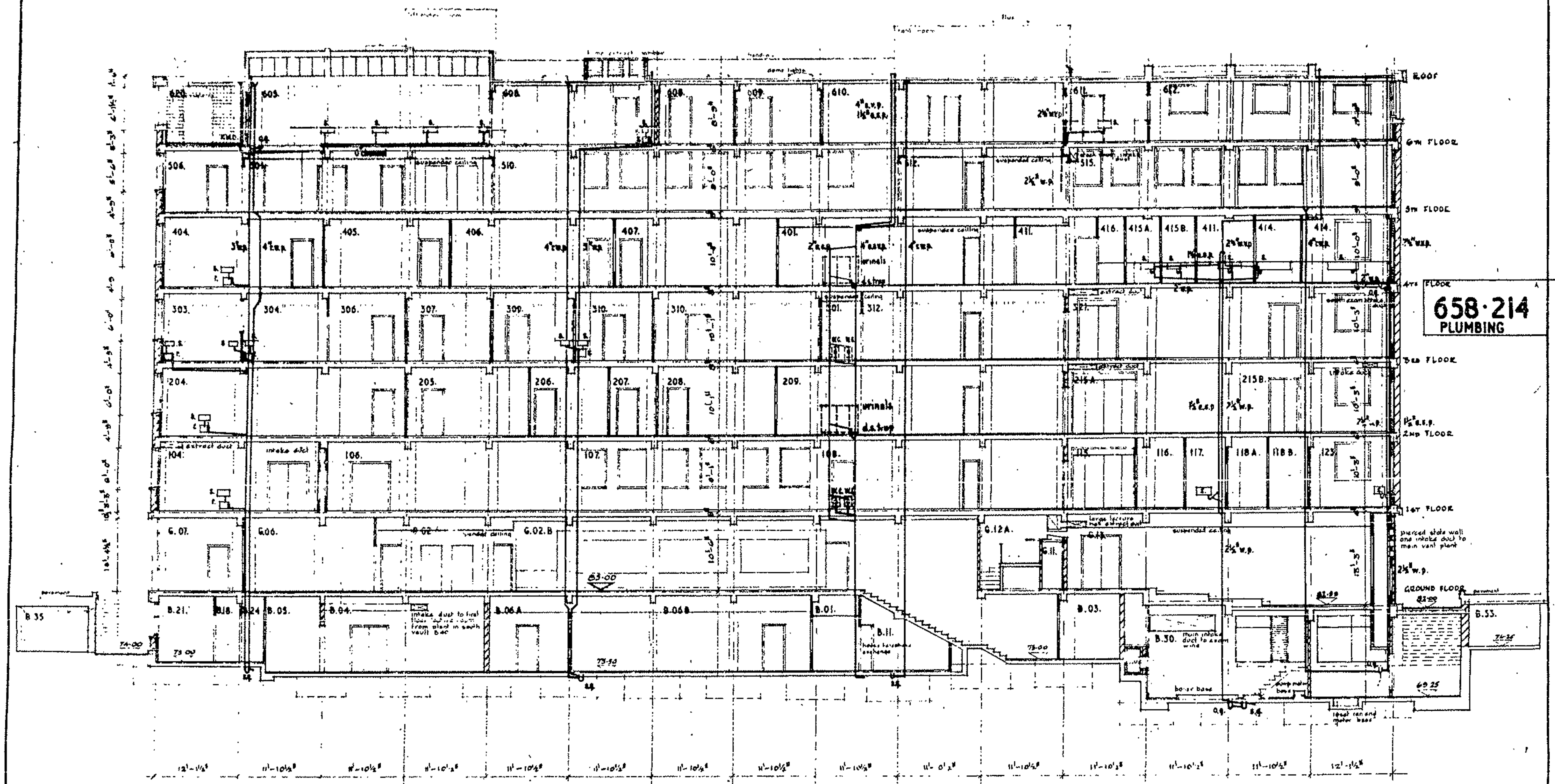
Bursar's Department
University College London
Drawing Reference No.

90	05	3

drg no. [redacted]

SCALE

658·214
PLUMBING



THIS DRAWING IS INTENDED ONLY TO SHOW THE SERVICES
MARKED [illegible] IN THE BOTTOM RIGHT CORNER

PLUMBING

BOOTH & LEDEBOER CHARTERED ARCHITECTS 14 LINCOLNS INN FIELDS W.C.2. & 9 BLACKHALL ROAD OXFORD	INDEX DATE INITIAL REVISION	DRAWN J.E.K.G. TRACED	CHECKED	DATE 14.0.1955 SCALE 1/8" = 1'-0"	UNIVERSITY OF LONDON BUILDING FOR THE INSTITUTES OF ARCHÆOLOGY & CLASSICAL STUDIES AND UNIVERSITY EXAMINATIONS 29-35 GORDON SQUARE	LONGITUDINAL SECTION THROUGH MAIN BLOCK A-A DRG. NO. 90/04/4