MOMENTUM
structural engineers

Design note

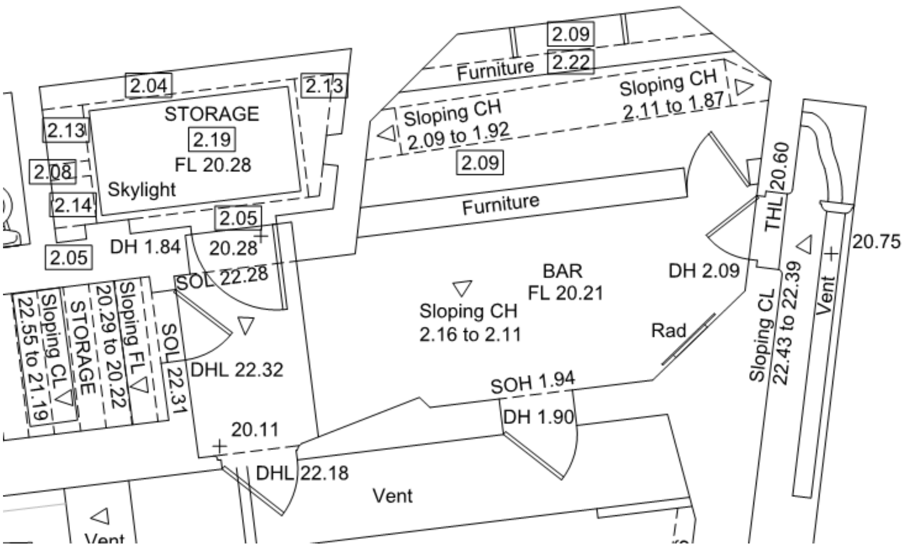
Project	Date	By	Reference
Ambassadors Theatre, London	10 Nov 2022	RR	5208_DNT_1

Title
Remedial works, structural design philosophy (rev A)

Introduction

This design note discusses the need for remedial works and the structural design philosophy adopted for the Ambassadors Theatre in West Street, London. The reason that this has come to light now is that the strip out has only happened when the construction stage started, revealing the full extent and the condition of the existing structure.

This note refers to the bar and storage area (turned into an accessible toilet), both located at lower ground floor (i.e. stalls level).



Excerpt from the stalls level (LGF) survey drawing showing the bar and existing storage area

Condition of Existing Structure

It has only been possible to establish the condition of the existing structure, following the strip out of the bar and storage area. In general the structure is in a good condition with the exception of the overhead structure, supporting the pavement and the skylights. In general, the overhead structure consists of steel beams with a concrete infill (a so-called filler joist slab).

Generally speaking the structure has been affected by moist (be it dampness or even water ingress) which has in turn resulted in the corrosion of the steel beams. As a consequence, the remedial works will need to incorporate an updated waterproofing solution to prevent the issue from occurring again.

In the storage area the steel beams have severely corroded with a significant amount of concrete that has spalled. The corrosion of the beams varies from severe delamination of the flanges to most of the actual section being lost. As a consequence, the existing structure can no longer be relied upon, and remedial works are required to make the structure safe again.

In the bar area the steel beams turned out to be in a better condition than in the storage. However, some local corrosion (and delamination) was found on the steelwork which could be inspected. It is worth noting that most of the steelwork in this area is encased in concrete: therefore, it is not possible to inspect the full extent of the steelwork (without demolishing the existing structure) and establish the full extent of the corrosion. As a consequence, it has been decided to undertake remedial works, based on the worst case scenario, I.e. the existing structure ultimately failing.


Existing overhead structure in the storage area: large parts of the steel sections have been lost, and delamination is occurred throughout. Additionally, the concrete has spalled due to the corrosion expansion.


Existing overhead structure in the bar area: there is some corrosion on the existing steelwork, but this had been limited in extent. Additionally, the concrete has spalled due to the corrosion expansion.

Remedial Works

The remedial works proposed for the storage are the replacement of the original steel beams by new reinforced concrete beams. Using new RC beams - with the correct grade and rebar cover - will ensure that the durability of the structure is guaranteed.

The remedial works proposed for the bar area are based on strengthening of the existing structure, rather than replacing it. The reason for this is that the existing structural steel is still in a reasonable condition. Additionally, the steel is completely encased in concrete (making it impossible to extract the steel without demolishing the GF slab as a whole in this area) and it forms a transfer structure for the curved facade above. Based on the above a new steel structure has been designed that supports the existing: a number of transfer beams is introduced, supporting by steel columns. The structural zones have been kept to a minimum, ensuring the space below is only slightly affected.

Furthermore, the surface corrosion on the existing steel beams in the bar area will be removed, followed by the application of a zinc-rich paint to prevent further corrosion. Lastly, it is recommended to install a new waterproofing system to ensure that the structure is not affected by moisture-related issues in the future.

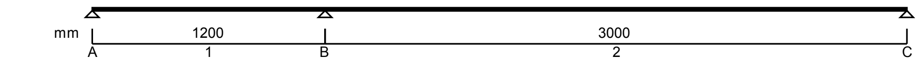
For full structural details of the remedial works please refer to Momentum's drawing set.

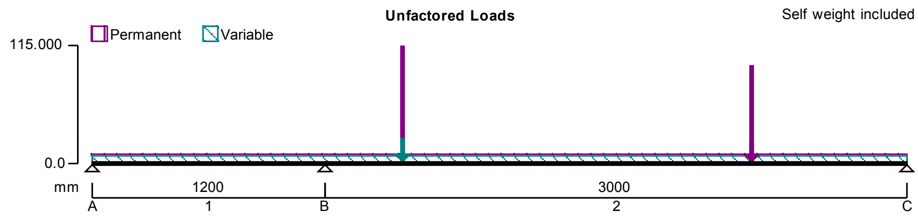
Appendix: structural calculations related to remedial works.

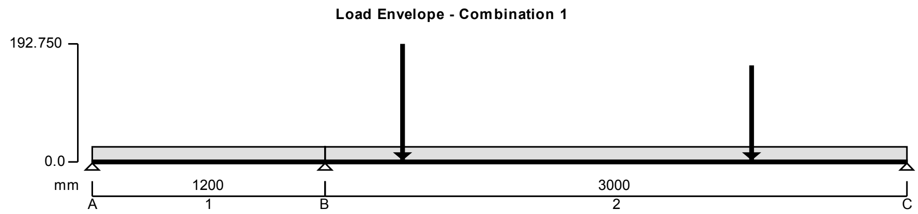
STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

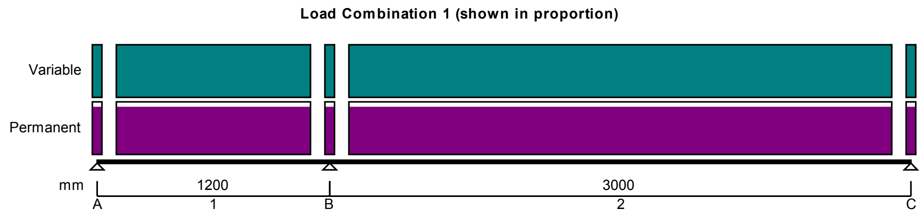
In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex

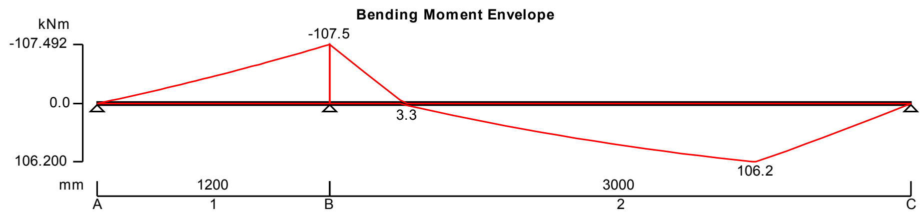
TEDDS calculation version 3.0.13



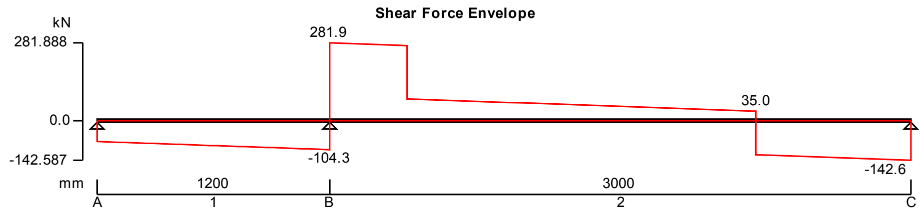








MOMENTUM structural engineers Momentum 90 Walcot St Bath BA1 5BG	Project Ambassadors Theatre, London				Job no. 5208	
	Calcs for Steel remedial works, bar				Start page no./Revision 2	
	Calcs by RR	Calcs date 06/10/2022	Checked by	Checked date	Approved by	Approved date



Support conditions

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

Applied loading

Beam loads	Permanent self weight of beam × 1
	w1 - Permanent full UDL 9 kN/m
	w1 - Variable full UDL 7.5 kN/m
	F1 - Permanent point load 115 kN at 1600 mm
	F1 - Variable point load 25 kN at 1600 mm
	F2 - Permanent point load 96 kN at 3400 mm
	F2 - Permanent point load 21 kN at 3400 mm

Load combinations

Load combination 1	Support A	Permanent × 1.35
		Variable × 1.50
	Span 1	Permanent × 1.35
		Variable × 1.50
	Support B	Permanent × 1.35
		Variable × 1.50
	Span 2	Permanent × 1.35
		Variable × 1.50
	Support C	Permanent × 1.35
		Variable × 1.50

Analysis results

Maximum moment	M_{max} = **106.2** kNm	M_{min} = **-107.5** kNm
Maximum moment span 1	M_{s1_max} = **0** kNm	M_{s1_min} = **-107.5** kNm
Maximum moment span 2	M_{s2_max} = **106.2** kNm	M_{s2_min} = **-107.5** kNm
Maximum shear	V_{max} = **281.9** kN	V_{min} = **-142.6** kN
Maximum shear span 1	V_{s1_max} = **-74.8** kN	V_{s1_min} = **-104.3** kN
Maximum shear span 2	V_{s2_max} = **281.9** kN	V_{s2_min} = **-142.6** kN
Deflection	δ_{max} = **0.1** mm	δ_{min} = **0** mm
Deflection span 1	δ_{s1_max} = **0** mm	δ_{s1_min} = **0** mm
Deflection span 2	δ_{s2_max} = **0.1** mm	δ_{s2_min} = **0** mm
Maximum reaction at support A	R_{A_max} = **-74.8** kN	R_{A_min} = **-74.8** kN

Unfactored permanent load reaction at support A $R_{A\_Permanent}$ = **-49.1** kN

Unfactored variable load reaction at support A $R_{A\_Variable}$ = **-5.7** kN

Maximum reaction at support B R_{B_max} = **386.2** kN R_{B_min} = **386.2** kN

Unfactored permanent load reaction at support B $R_{B\_Permanent}$ = **228.7** kN

Unfactored variable load reaction at support B $R_{B\_Variable}$ = **51.6** kN

Maximum reaction at support C R_{C_max} = **142.6** kN R_{C_min} = **142.6** kN

Unfactored permanent load reaction at support C $R_{C\_Permanent}$ = **93.9** kN

Unfactored variable load reaction at support C $R_{C\_Variable}$ = **10.5** kN

Section details

Section type **2 x UKB 356x171x45 (Tata Steel Advance)**

Steel grade **S275**

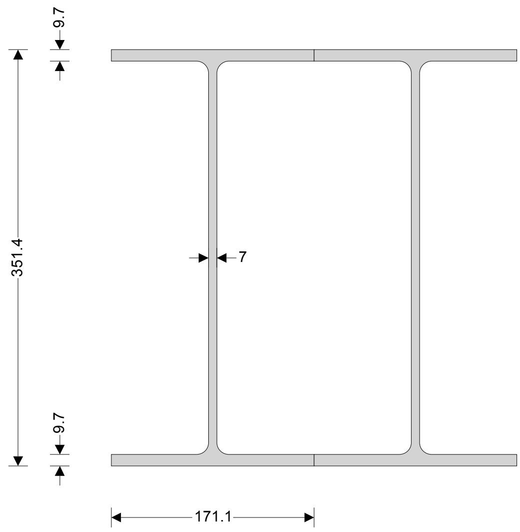
EN 10025-2:2004 - Hot rolled products of structural steels

Nominal thickness of element $t = \max(t_f, t_w)$ = **9.7** mm

Nominal yield strength f_y = **275** N/mm^2

Nominal ultimate tensile strength f_u = **410** N/mm^2

Modulus of elasticity E = **210000** N/mm^2



Partial factors - Section 6.1

Resistance of cross-sections γ_{M0} = **1.00**

Resistance of members to instability γ_{M1} = **1.00**

Resistance of tensile members to fracture γ_{M2} = **1.10**

Lateral restraint

Span 1 has full lateral restraint

Span 2 has full lateral restraint

Effective length factors

Effective length factor in major axis K_y = **1.000**

Effective length factor in minor axis K_z = **1.000**

Effective length factor for torsion $K_{LT.A}$ = **1.000**

$K_{LT.B}$ = **1.000**

$K_{LT.C}$ = **1.000**

Classification of cross sections - Section 5.5

$\varepsilon = \sqrt{[235 \text{ N/mm}^2 / f_y]}$ = **0.92**

Internal compression parts subject to bending - Table 5.2 (sheet 1 of 3)

Width of section $c = d =$ **311.6** mm

$c / t_w = 48.2 \times \varepsilon <= 72 \times \varepsilon$ Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section $c = (b - t_w - 2 \times r) / 2 =$ **71.9** mm

$c / t_f = 8.0 \times \varepsilon <= 9 \times \varepsilon$ Class 1

Section is class 1

Check shear - Section 6.2.6

Height of web $h_w = h - 2 \times t_f =$ **332** mm

Shear area factor $\eta =$ **1.000**

$h_w / t_w < 72 \times \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force $V_{Ed} = \max(\text{abs}(V_{max}), \text{abs}(V_{min})) =$ **281.9** kN

Shear area - cl 6.2.6(3) $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) =$ **2679** mm^2

Design shear resistance - cl 6.2.6(2) $V_{c,Rd} = V_{pl,Rd} = N \times A_v \times (f_y / \sqrt{[3]}) / \gamma_{M0} =$ **850.7** kN

PASS - Design shear resistance exceeds design shear force

Check bending moment at span 2 major (y-y) axis - Section 6.2.5

Design bending moment $M_{Ed} = \max(\text{abs}(M_{s2\_max}), \text{abs}(M_{s2\_min})) =$ **107.5** kNm

Design bending resistance moment - eq 6.13 $M_{c,Rd} = M_{pl,Rd} = N \times W_{pl.y} \times f_y / \gamma_{M0} =$ **426** kNm

PASS - Design bending resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to variable loads

Limiting deflection $\delta_{lim} = L_{s2} / 360 =$ **8.3** mm

Maximum deflection span 2 $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) =$ **0.129** mm

PASS - Maximum deflection does not exceed deflection limit

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Title

Date
Oct. '22

By
RR.

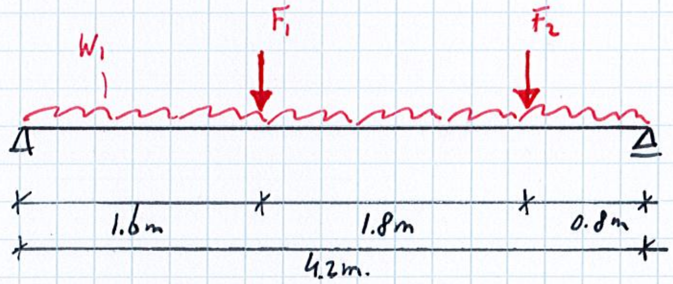
Reference

Review of double steel beams supporting curved facade (bow area)

* Starting points / assumptions

- 2no. existing UBs (depth unknown) support curved facade above, from GF to roof.

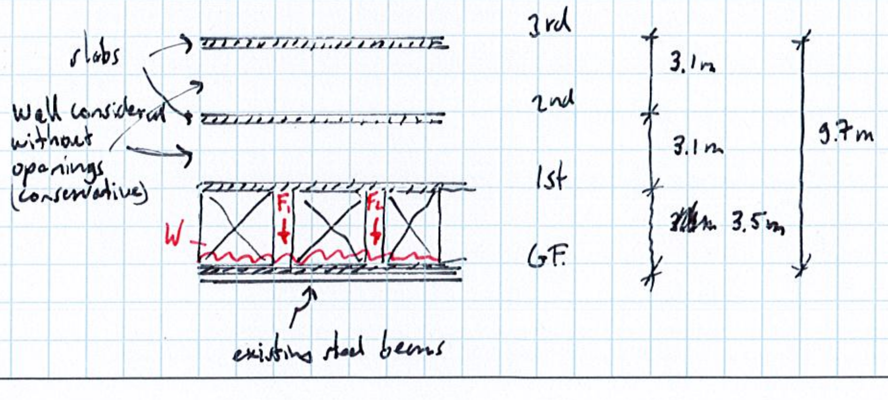
* Existing scheme



* Loads

basic assumptions:
- slabs 200thk + finishes G = 6 kN/m²
- liveload congregation/offices Q = 4 " (average)
- 330 thk wall masonry: G = 6 "

simplified, developed elevation of facade



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	G (kN/m')	Q (kN/m')	
W'_1 from G.F. 1.5m × 6 and 5 =	9	7.5	Wd = 23 kN/m
line loads above G.F.	G (kN/m')	Q (kN/m')	
• 1st floor slab 1.5m × 6 and 4 =	9	6	
• 2nd " " 1.5m × 6 " 4 =	9	6	
• 3rd roof slab 1.5m × 6 " 1 =	9	2	
• wall 1st-roof: 6.2m × 6 =	37		
	64	14	Wd = 107 kN/m
F_1 load width is 1.8m (due to curve)	G (kN)	Q (kN)	
1.8m × 64 and 14 =	115	25	Fd = 193 kN.
F_2 load width is 1.5m (due to curve)			
1.5m × 64 and 14 =	96	21	Fd = 161 kN.

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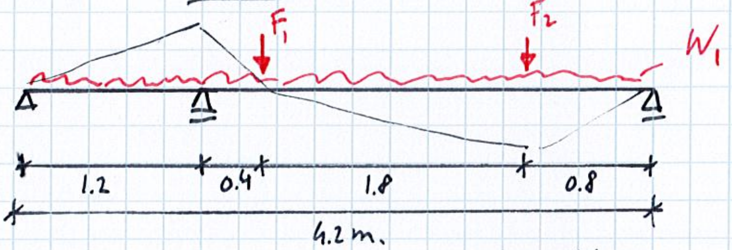
Scheme 1   1 add'l colm + 2 double beams

0.4m / 1.8m / 0.8m

$M_d = 108$ kNm



$M_d = 106$ kNm

beam options – UC 152 x 23 $M_{Rd} = 58$ kNm x 2 = 116 kNm > 108 kNm ✓

– RHS ~~200 x 12 x~~ 300x 100x 12.5 $M_{Rd} = 126$ kNm.

resin injected.

New middle column: $N_d = 262 + 104 = 366$ kN

→ SHS ~~[illegible]~~ 120 x 8   ($N_{Rd} = 947$ kN)

Foundation middle column $N_{req} = \frac{366}{1.4} = 261$ kN.

$A_{req} = \frac{261}{100} = 2.6\ m^2$ → 1.6 m x 1.6 m. depth to match existing foundations.

Side columns $N_d = 143$ kN.

→ PFC 150x 75 x 18   ($N_{Rd} = 804$ kN)

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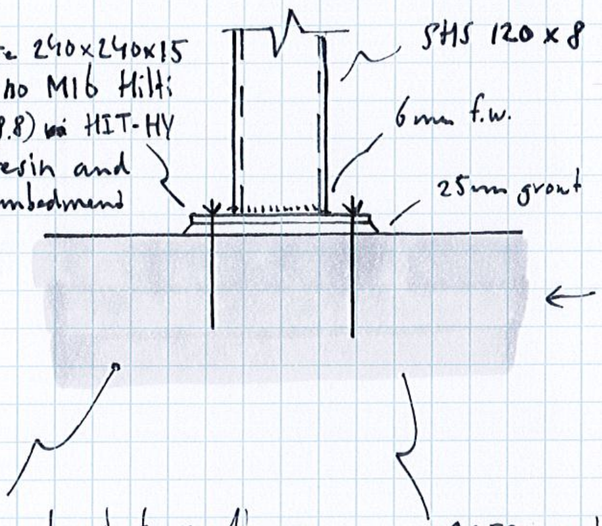
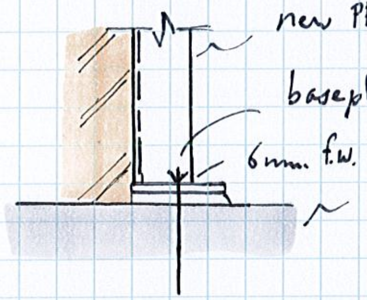
Middle column baseplate

$N_d = 366$ kN.

$A_{req} = \frac{366 \cdot 10^3}{18} = 20{,}333 \text{ mm}^2 \rightarrow$ say 143×143.

Choose practical size $240 \times 240 \times 15$





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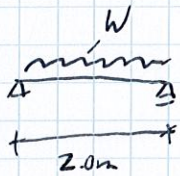
Date Oct. 22 | **By** R.R. | **Reference**

Secondary transfer beam

Dead load from skylight:

$G = \sim 0.5\,m \times 20 =$ $10\ kN/m^2$

$Q =$ $10\ kN/m^2$



$P_d = 28.5\ kN/m^2$ $w_d = 2.5\,m \times 28.5 = 71\ kN/m$

$M_d = 1/8 \times \cancel{28.5}\,71 \times 2^2 = 36\ kNm$

→ RHS $150 \times 100 \times 6.3$ ($M_{Rd} = 39\ kNm$)