Structural Calculation Package for

Victoria Warehouse - Manchester

Booth King Partnership Limited

Sep 2023

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Rev 00	27th Sep '23	First issue.
Rev.01	26th Sep '23	Minor clarification updates to the conclusion.

Contents

1. Introduction

1.1. Scope

A prohibition notice was served at Victoria Warehouse for all activities on the first-floor structure with the following specific justification:

'There is no comprehensive assessment of the first-floor loading including a detailed analysis that takes into account dynamic loading resulting from crowd movement. Therefore, there is a risk of overloading and collapse.'

Booth King Partnership Limited (BKPL) have been appointed by Live Nation as Structural Engineers for checking the design of the existing Victoria Warehouse 'mezzanine' floor in Manchester to satisfy the concern stated above.

The building consists mainly of a 'warehouse' portion, an open industrial space converted for use as a concert venue containing a steel balcony structure (see separate BKPL reports on structural assessment and vibration), that abuts onto a 'mezzanine' structure, also used for spectating the concert venue - the remainder of the first floor is of a similar construction to this 'mezzanine', and can be confidently taken as representative of the entire structure.

BKPL attended site on 12th Sep 2023 to carry out a structural review and obtained measurements from areas of the structure readily visible - the member sizes adopted in this report represent the most conservative dimensions observed.

This report contains an evaluation of the analysis/design of the existing 'mezzanine' structure and its constituent members (slab, beams and columns), along with a modal analysis of the 'mezzanine' sub-frame and accompanying footfall analysis. This is to be read in conjunction with previously issued reports for structural assessment and vibration checks of the steel balcony structure.

1.2. Materials

All Structural Steel members are conservatively assumed to be grade S235 (while not directly applicable when for historic steel, this allows for a baseline for checking).

Concrete present in slabs is conservatively taken as C25/30.

1.3. Software

The following software was used in the structural design of the project:

- Scia Engineer 22.1
- CADS Hub – Footfall Analysis

1.4. Disclaimer

This report is prepared for the use of Live Nation in connection with this appointment. It is not intended for and should not be relied upon by any third party.

The opinions expressed are based on the conditions as readily seen, and our interpretation of the evidence does not benefit from long-term observation. Our inspection has only covered the major exposed aspects of the structure that it was possible to inspect.

We undertook localised 'breaking out' to determine the size of the typical steel beams/columns; however, did not undertake widespread breaking out to view the condition of those parts of the structure which are covered, unexposed or inaccessible - and are therefore unable to report that any such part is free from defect.

A detailed inspection of the structure has not been carried out for rot and infestation. This is specialist work and is not within the scope of this investigation and report.

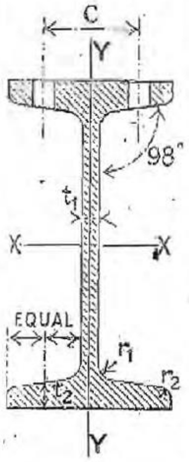
2. Modelling

2.1. Structure

The main structural slab is comprised of a ribbed concrete slab with deep clay pots used to form ‘voids’ between ribs – although the pots are meant to form air voids, practically, a good portion of these have been infiltrated by concrete during construction.

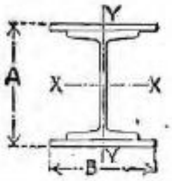
The slab is taken as overall depth 200mm, with a 100mm deep x 90mm wide rib @ approx. 350mm c/c. - non-structural screed of 30mm was also observed.

The concrete slab sits on steel I-beams – these are determined to be Historic Reference Mark NBSB 13 sections (15”x6”):



Reference Mark	Size Inches	Weight per foot lbs.	Standard Thicknesses: Web t_1	Standard Thicknesses: Flange t_2	Radii: Root r_1	Radii: Toe r_2	Centres of Holes C Inches
"A" GIRDER SECTIONS.							
NBSB 18	24×7½	90	·52	·984	·73	·36	4·5
" 17	22×7	75	·50	·834	·69	·34	4·0
" 16	20×6½	65	·45	·820	·65	·32	3·75
" 15	18×6	55	·42	·757	·61	·30	3·5
" 14	16×6	50	·40	·726	·61	·30	3·5
" 13	15×6	45	·38	·655	·61	·30	3·5
" 12	14×5½	40	·37	·627	·57	·28	3·25
" 11	13×5	35	·35	·604	·53	·26	2·75
" 10	12×5	30	·33	·507	·53	·26	2·75
" 9	10×4½	25	·30	·505	·49	·24	2·5
" 8	9×4	21	·30	·457	·45	·22	2·25
" 7	8×4	18	·28	·398	·45	·22	2·25
" 6	7×3½	15	·25	·398	·41	·20	2·0
" 5	6×3	12	·23	·377	·37	·18	1·5
" 4	5×2½	9	·20	·347	·33	·16	..
" 3	4½×2	7	·19	·322	·29	·14	..
" 2	4×1¾	5	·17	·239	·27	·13	..
" 1	3×1½	4	·16	·249	·25	·12	..

Columns are conservatively taken as Compound Stanchions of Historic Reference mark S150 - 10”x6” beam section with ½” plates riveted to the outside face of flanges (although BKPL noted 2no. plates riveted to each flange at ground floor, only a single plate is present at the head at the beam-column interface, and up to first floor):



COMPOUND STANCHIONS.

DIMENSIONS & PROPERTIES IN INCH UNITS.

Reference Mark	Size Inches A × B	Weight per foot lbs.	DIMENSIONS Inches		Area Square Inches	Radii of Gyration Inches		Section Moduli	
			Beam	Each Flange		About X - X	About Y - Y	About X - X	About Y - Y
S 121	14 ×12	222	10×8	12×2	64·18	5·63	3·13	290·4	105·1
" 122	13 ×12	181	"	12×1½	52·18	5·34	3·05	228·6	81·1
" 123	12½×12	160·5	"	12×1¼	46·18	5·19	3·00	198·7	69·[illegible]
" 124	12 ×12	140	"	12×1	40·18	5·03	2·92	169·5	57·1
" 125	11½×12	120	"	12× ¾	34·18	4·87	2·82	140·8	[illegible]5·1
" 126	11 ×12	99·5	"	12× ½	28·18	4·69	2·66	112·7	33·1
" 127	13 ×10	160·5	"	10×1½	46·18	5·28	2·57	197·9	60·9
" 128	12½×10	143·5	"	10×1¼	41·18	5·13	2·53	173·3	52·6
" 129	12 ×10	126·5	"	10×1	36·18	4·97	2·47	149·2	44·3
" 130	11½×10	109·5	"	10× ¾	31·18	4·81	2·40	125·7	35·9
" 131	11 ×10	92·5	"	10× ½	26·18	4·64	2·30	102·6	27·6
" 132	10¾× 9	81·5	"	9× ⅝	22·93	4·53	2·00	87·5	22·3
S 133	13 ×12	166	10×6	12×1½	47·77	5·42	3·08	215·7	75·6
" 134	12 ×12	125	"	12×1	35·77	5·11	2·94	155·5	51·6
" 135	11 ×12	83	"	12× ½	23·77	4·75	2·64	97·4	27·6
" 136	13 ×10	144·5	"	10×1½	41·77	5·37	2·55	185·0	54·4
" 137	12 ×10	110·5	"	10×1	31·77	5·05	2·44	135·3	37·7
" 138	11 ×10	76·5	"	10× ½	21·77	4·70	2·20	87·4	21·0
" 139	10¾× 9	65	"	9× ⅜	18·52	4·57	1·91	71·9	15·0
S 140	12 ×10	154·5	9×7	10×1½	44·71	4·82	2·55	173·4	58·0
" 141	11½×10	137·5	"	10×1¼	39·71	4·68	2·50	151·0	49·7
" 142	11 ×10	120·5	"	10×1	34·71	4·52	2·44	129·1	41·4
" 143	10½×10	103·5	"	10× ¾	29·71	4·36	2·36	107·7	33·0
" 144	10 ×10	86·5	"	10× ½	24·71	4·19	2·24	86·8	24·7
" 145	9¾× 9	75	"	9× ⅝	21·46	4·08	2·00	73·1	19·1
S 146	11 × 9	129	8×6	9×1½	37·30	4·42	2·33	132·6	44·8
" 147	10½× 9	114	"	9×1¼	32·30	4·28	2·29	114·1	38·1
" 148	10 × 9	98·5	"	9×1	28·30	4·12	2·23	96·2	31·3
" 149	9½× 9	83	"	9× ¾	23·30	3·97	2·16	78·8	24·6
" 150	9 × 9	68	"	9× ½	19·30	3·79	2·04	61·7	17·8
" 151	8¾× 7	55	"	7× ⅝	15·55	3·65	1·62	47·4	11·7

Note that the main structural steel frame (beams and columns) is generally encased in unreinforced concrete – in some locations this has been cut back on the columns for aesthetic reasons. For simplicity, the stiffening effect of concrete has not been considered in either of the following analyses - this is a conservative approach as, in reality, the concrete will increase the strength and rigidity of the structure.

2.2. Loading

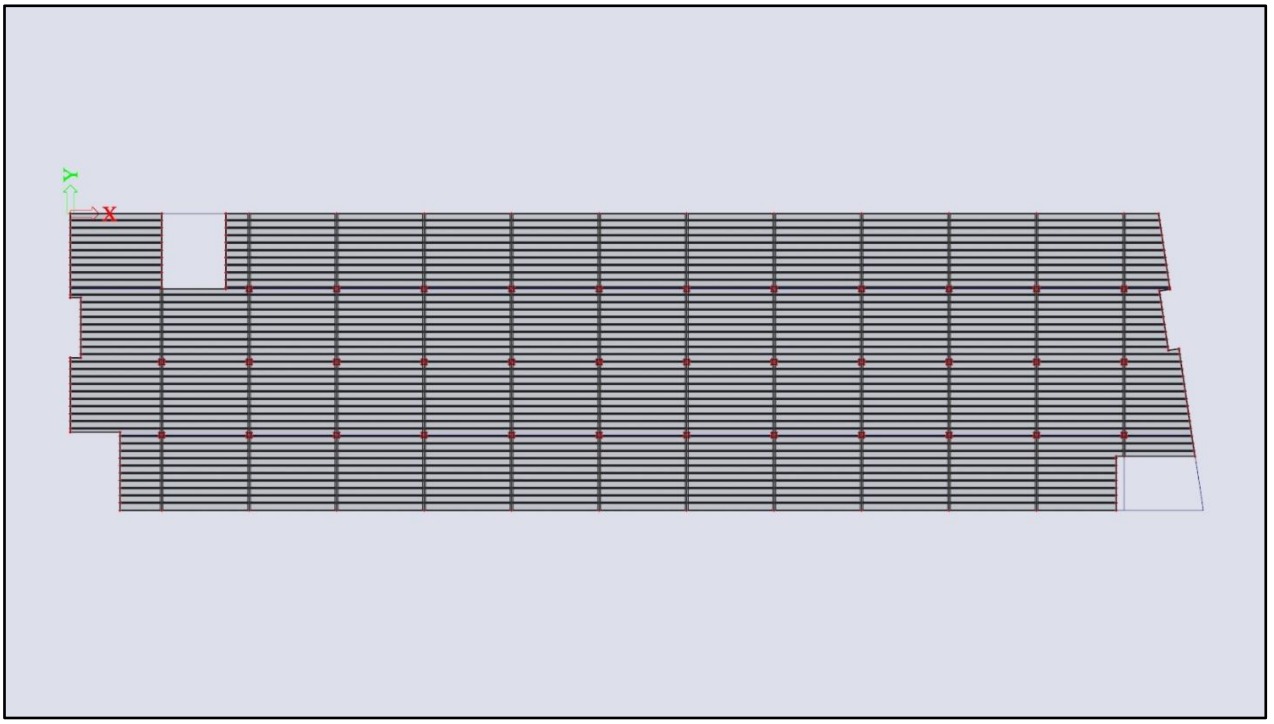
Self-weight of the structure is calculated by the software.

It is estimated that a conservative Superimposed Dead Load of 2.5kN/m^2 is sufficient in accounting for the combined weight of concrete infiltration present in the pots, non-structural screed, and sparse/lightweight finishes to the structure.

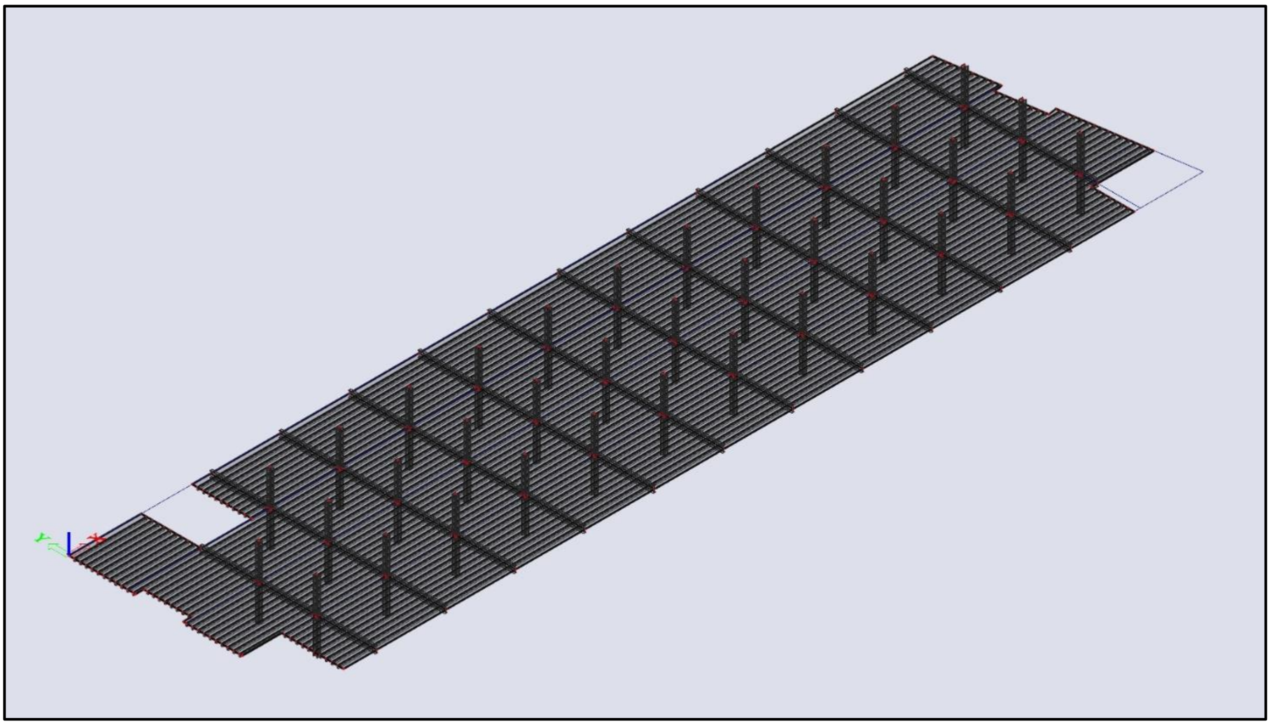
Imposed Load is taken as 5.00kN/m^2 according to uniform imposed action Sub-category C51 (assembly areas without fixed seating, concert halls, bars and places of worship), specified in Table NA.2 of NA to BS EN 1991-1-1:2002. Note, this is the same as category C5 in BS6399, which was the standard in use at the original change of use.

2.3. Structural Layout

BKPL's spot site measurements were used to correlate the provided plan layout to recreate the mezzanine floor structure as a sub-frame in SCIA Engineer 22.1.



Plan View

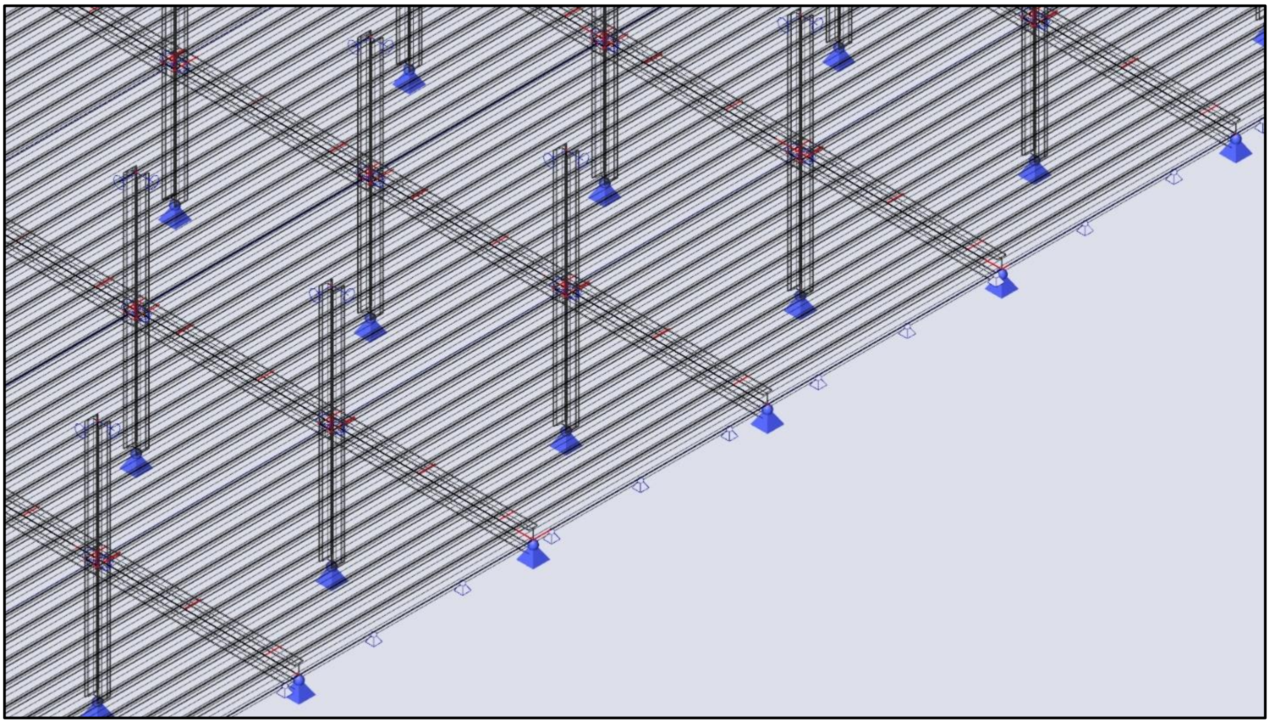


Isometric View

Room C2, 2nd Floor, Planetree House, 21-31 Oldham Street, Manchester, M1 1JG
Tel: 0161 694 7087
Email: office@booth-king.co.uk
Website: www.booth-king.co.uk

Beams are treated as fully rotationally free at the ends, columns are rotationally released from the slab, and masonry walls are represented as fully rotationally released line supports.

Slabs are treated as fully continuous in both directions, with ribs continuous along their length through the entire structure, except at column heads. Note, that the concrete ribs have reinforcement bars that extend over the supports, hence the assumption of continuity.

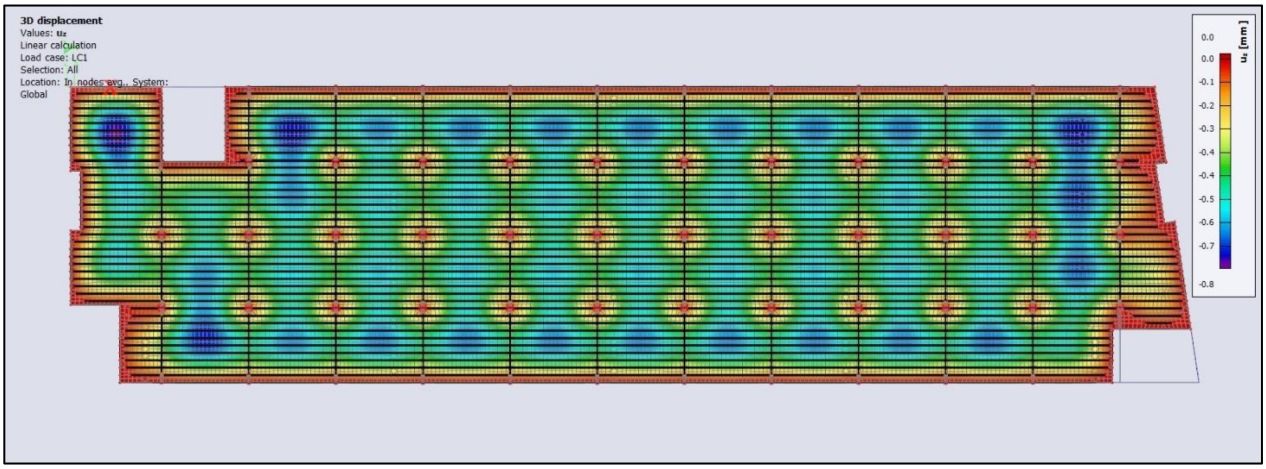


Restraints and Releases

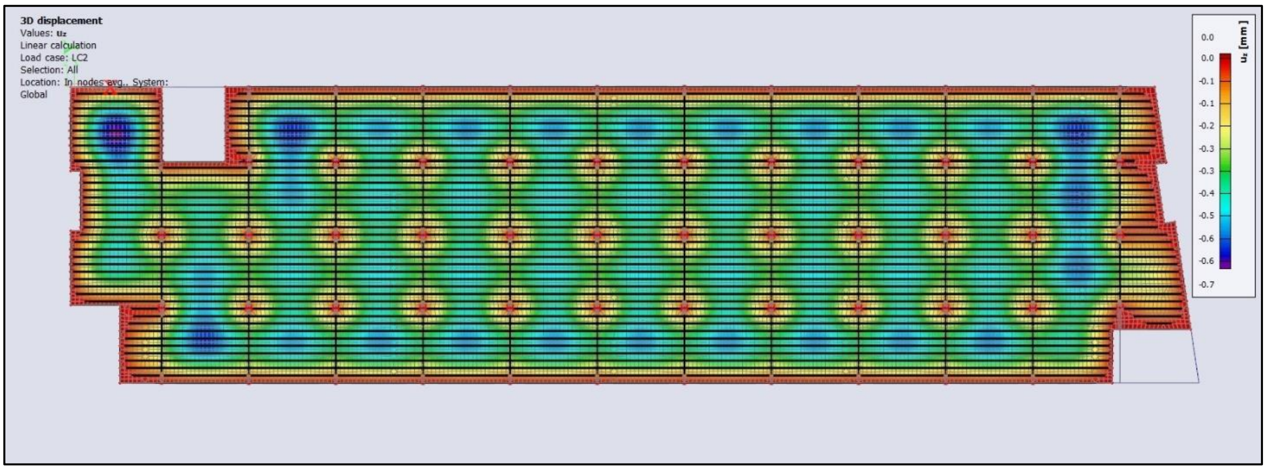
3. Analysis

3.1. Limit State Analysis

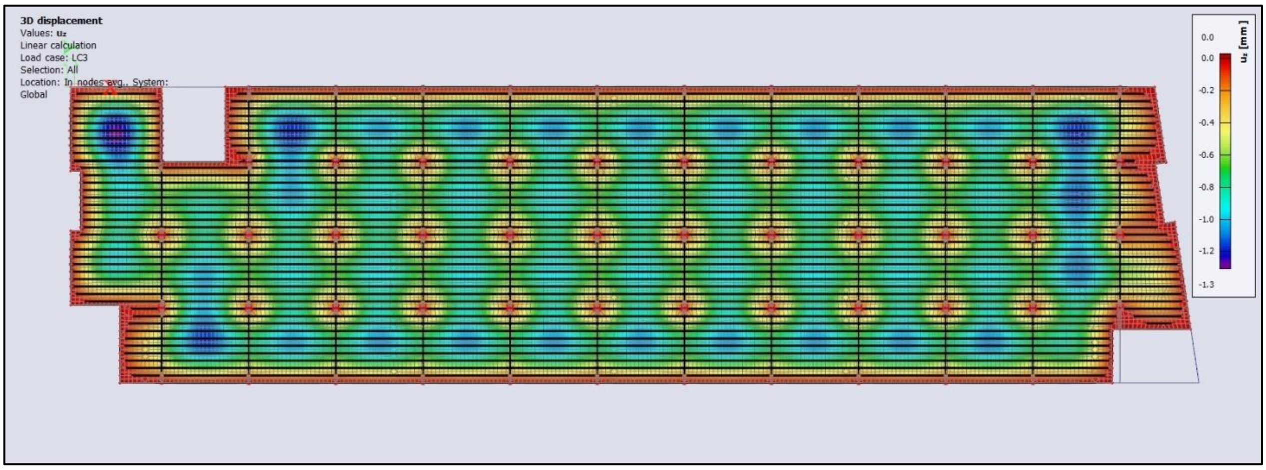
Elastic deformation of the characteristic structure is used to verify accurate behaviour of the model.



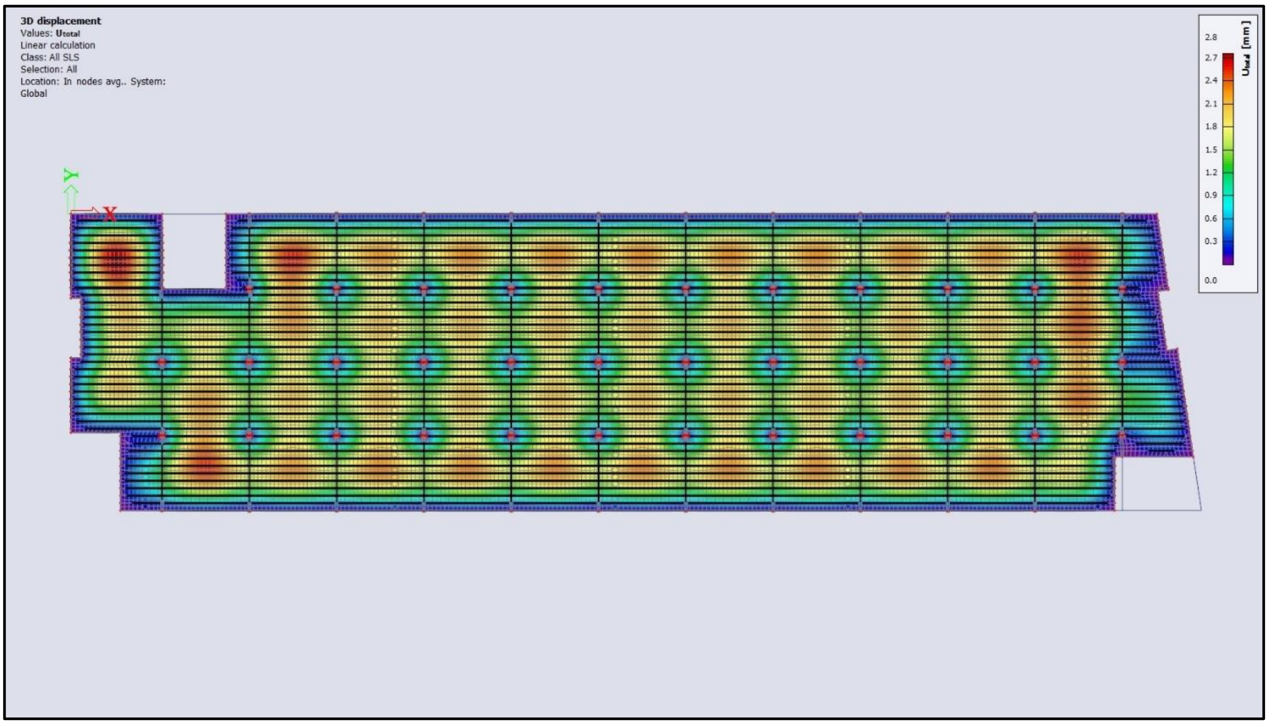
Maximum Self-weight Deflection = 0.8mm



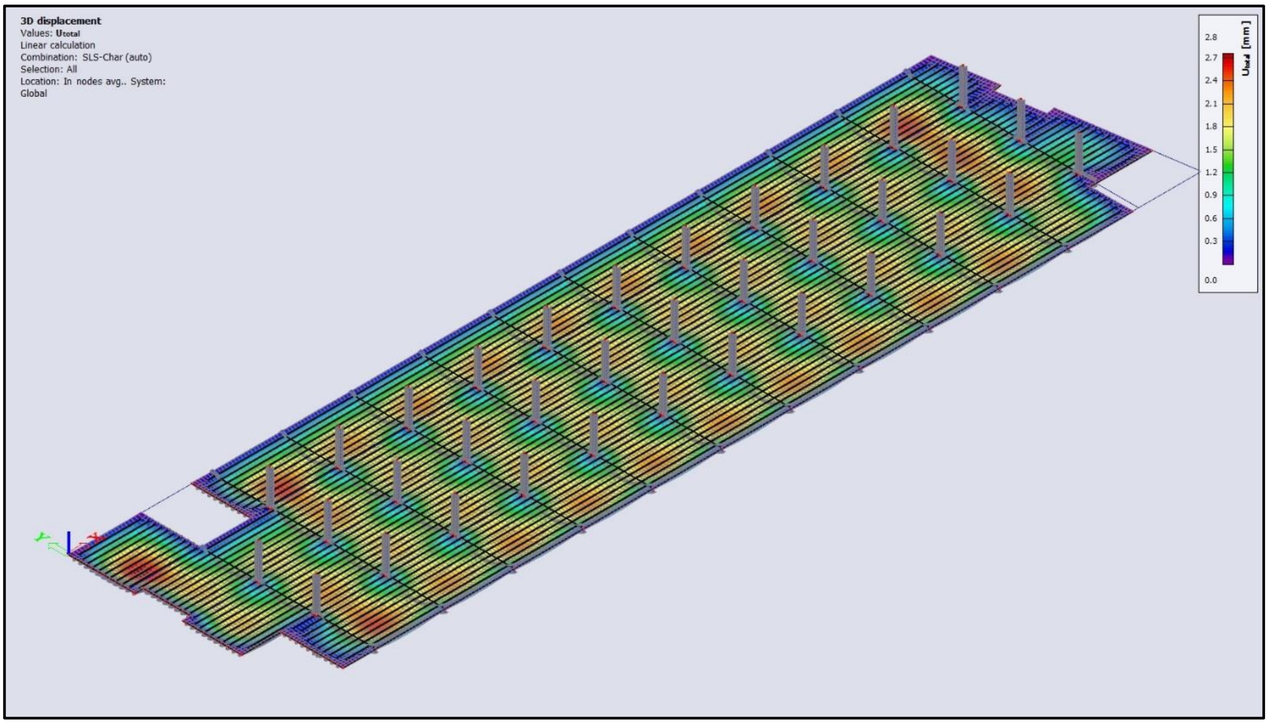
Maximum Superimposed Dead Load Deflection = 0.7mm



Maximum Imposed Load Deflection = 1.3mm



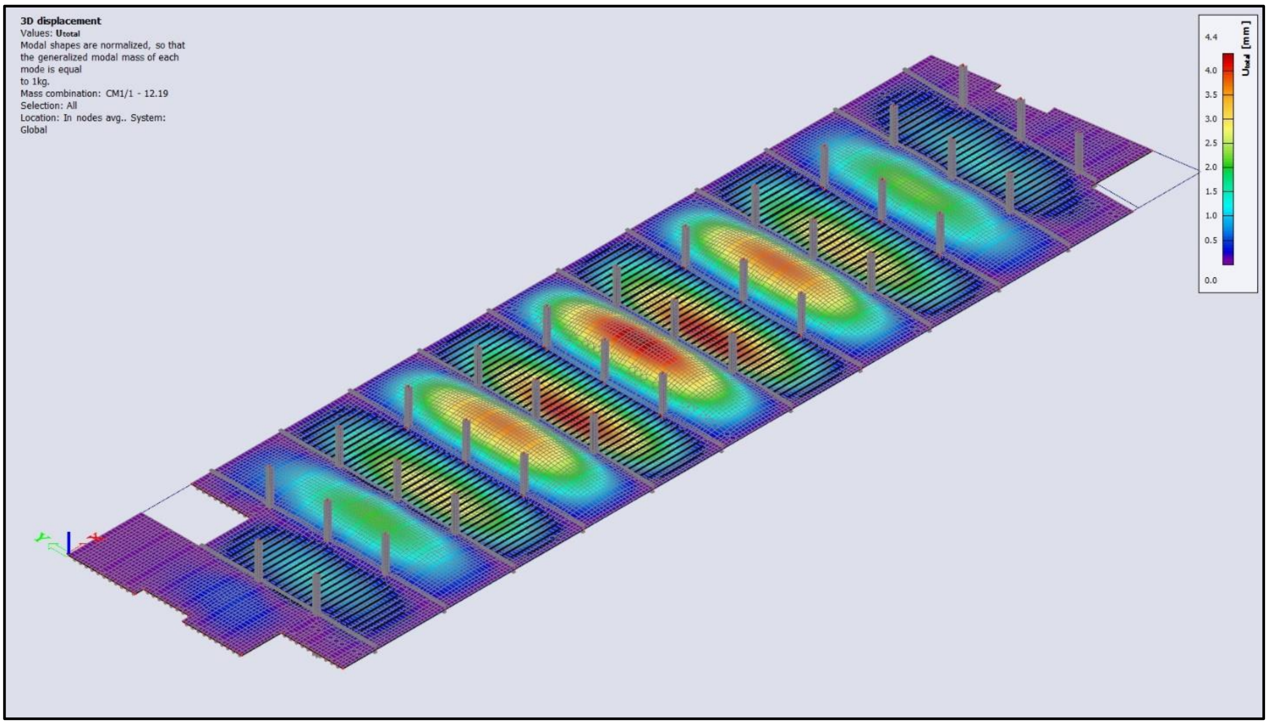
Total 3D Elastic Deformation (Plan View)



Total 3D Elastic Deformation (Isometric View)

3.2. Modal Analysis

Modal analysis indicates that the behaviour/deformation of the structure is as expected, with a sinusoidal waveform propagating along the length of the ribbed slab in the first Eigenmode.



First Eigenmode Deformation (12.19Hz)

Eigen frequencies

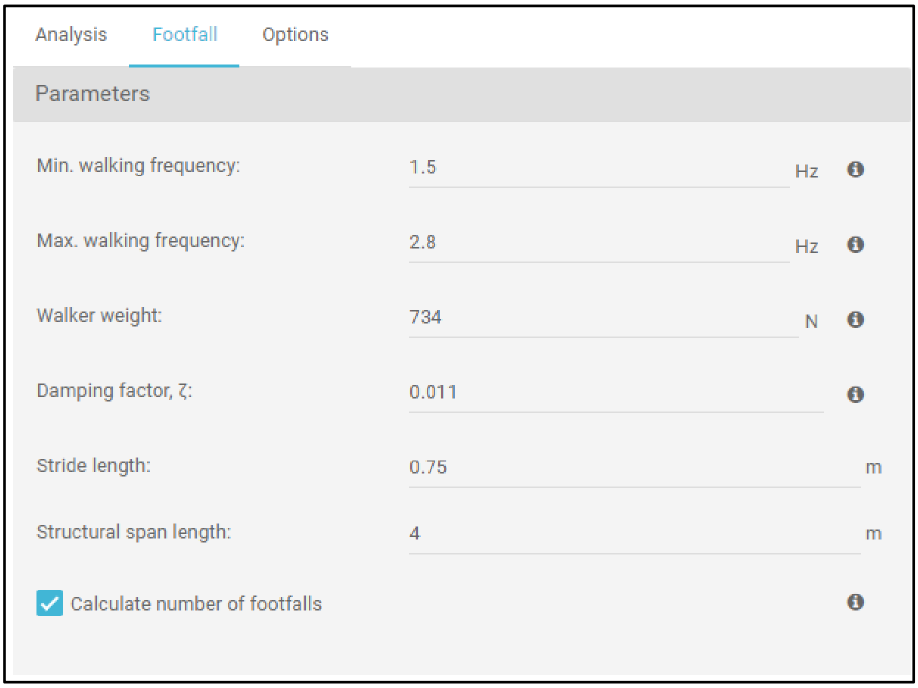
N	f [Hz]	ω [1/s]	ω² [1/s²]	T [s]
Mass combination : CM1				
1	12.19	76.57	5863.37	0.08
2	12.51	78.57	6173.43	0.08
3	12.97	81.48	6638.93	0.08
4	13.21	83.01	6890.03	0.08
5	13.47	84.63	7161.79	0.07
6	13.53	85.04	7231.80	0.07
7	13.91	87.40	7638.93	0.07
8	14.07	88.40	7815.00	0.07
9	14.41	90.51	8192.26	0.07
10	14.59	91.69	8407.10	0.07

The calculated fundamental frequency (12.19Hz) is greater than the limit (8.4Hz) prescribed by BS 6399-1:1996 (Annex A) – SCI P354 reiterates that resonant effects need not be evaluated if this is the case.

3.3. Footfall Analysis

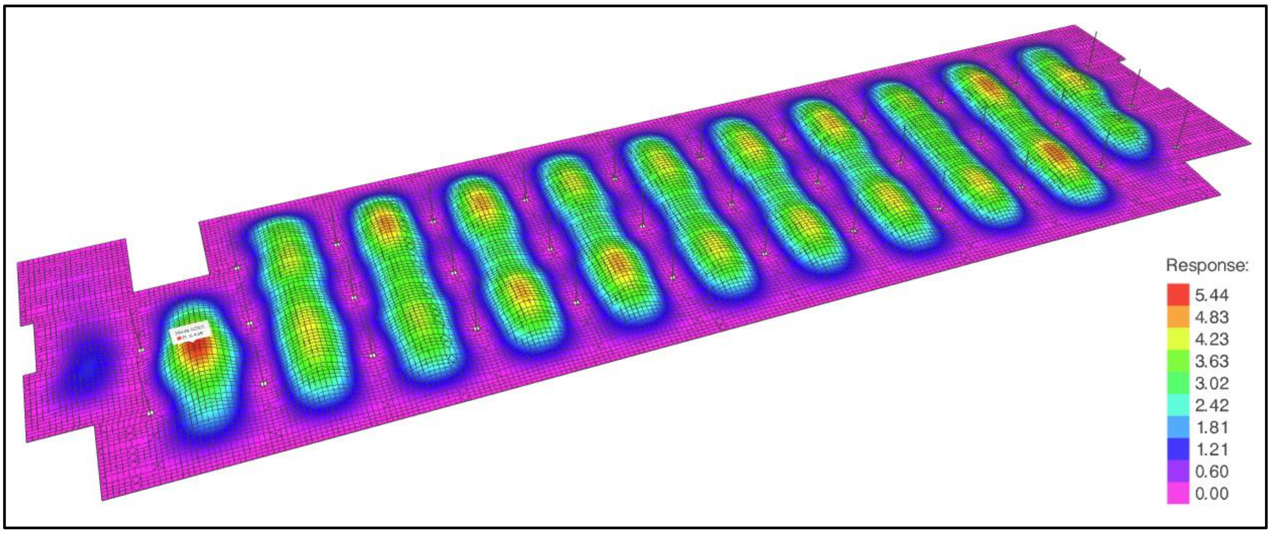
Although proven in the previous section that the structure is designed to avoid resonance, a Footfall Analysis is also carried out to demonstrate the extent of compliance.

Footfall analysis is carried out according to SCI P354 recommendations for High Impact Rhythmic Activities for groups (1.5-2.8Hz) with a damping factor of 1.1% using CADS Hub – Footfall Analysis:



Footfall Input Parameters

'High Impact Rhythmic' activities has been chosen for the 'mezzanine' area because this is primarily a bar area, where it is unlikely that extended periods of crowd jumping will occur, as would occur at the stage and potentially the balcony viewing area.

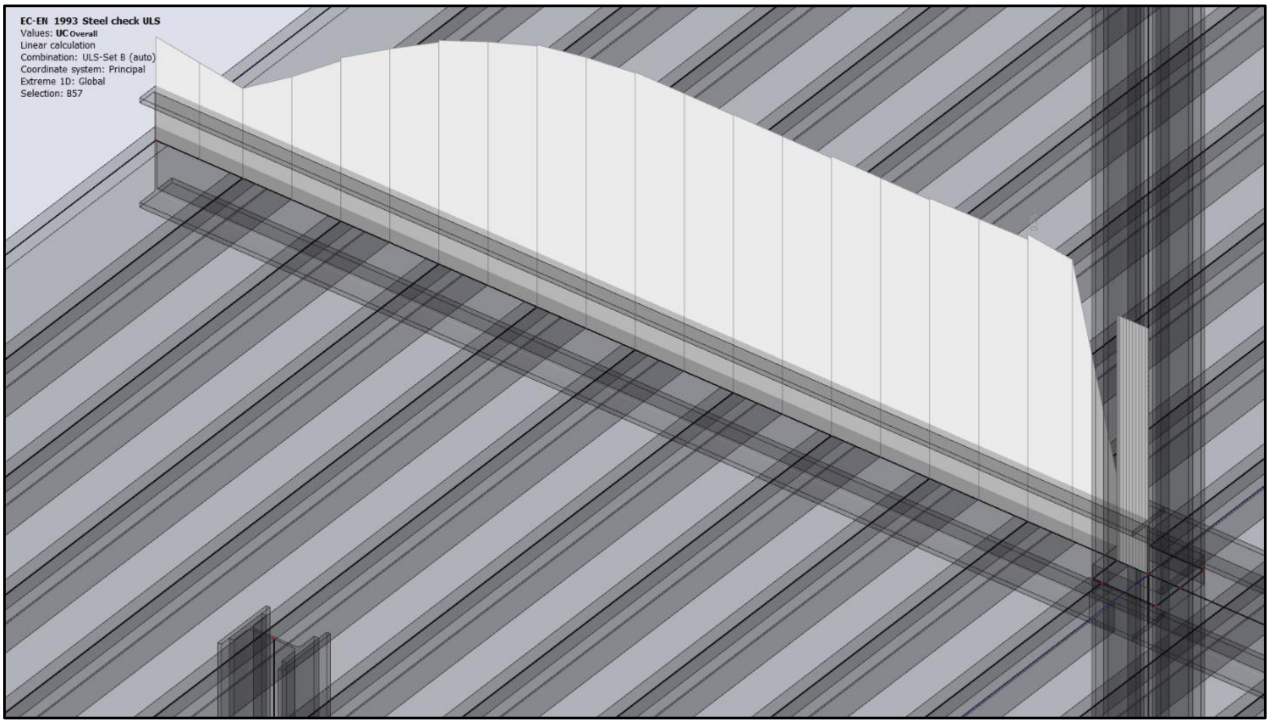


Resonant Response

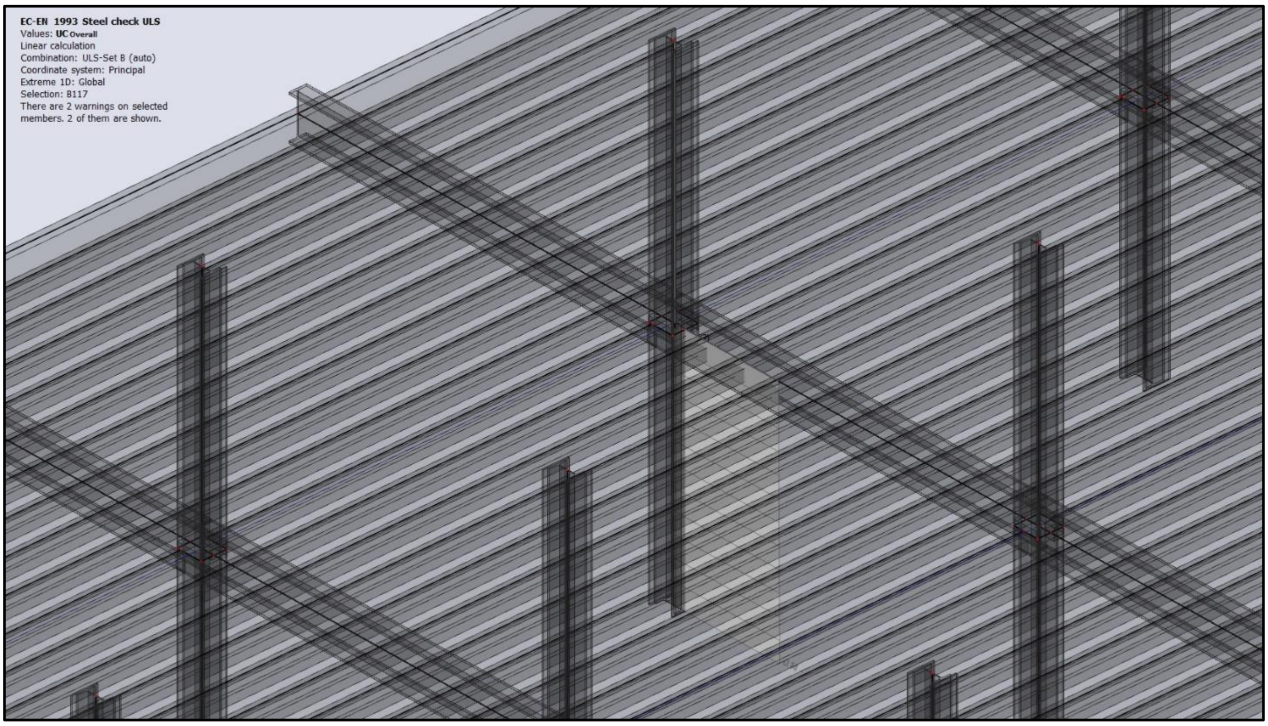
The maximum response is significantly less than SCI P354's prescribed limit of 69.4 (5.4).

4. Design

4.1. Steel Utilisation & Design Summary



Representative Beam



Representative Column

EC-EN 1993 Steel check ULS

Values: **$UC_{Overall}$**
Linear calculation
Combination: ULS-Set B (auto)
Coordinate system: Principal
Extreme 1D: Global
Selection: B57

EN 1993-1-1 Code Check
National annex: British BS-EN NA

Member B57	**3.110 / 3.540 m**	**I (381; 152; 17; 10; 6)**	**Rolled**	**S 235**	**ULS-Set B (auto)**	**0.25 -**

Combination key
ULS-Set B (auto) / 1.25*LC1 + 1.25*LC2 + 1.50*LC3

Partial safety factors		
Resistance of cross-sections	γ_{M0}	1.00
Resistance to instability	γ_{M1}	1.00
Resistance of net sections	γ_{M2}	1.10

Material			
Yield strength	f_y	235.0	MPa
Ultimate strength	f_u	360.0	MPa

Section checks
Section is classified as Class 1

Section checks	Design force	Value	Unit	Resistance	Value	Unit	Unity check [-]
Compression	N_{Ed}	-12.23	kN	$N_{c,Rd}$	1988.68	kN	0.01
Shear V_y	$V_{y,Ed}$	-0.03	kN	$V_{pl,y,Rd}$	708.97	kN	0.00
Shear V_z	$V_{z,Ed}$	-42.97	kN	$V_{pl,z,Rd}$	510.60	kN	0.08
Bending M_y	$M_{y,Ed}$	31.64	kNm	$M_{pl,y,Rd}$	287.09	kNm	0.11
Bending M_z	$M_{z,Ed}$	0.01	kNm	$M_{pl,z,Rd}$	47.36	kNm	0.00
Torsion	τ_{Ed}	0.0	MPa	τ_{Rd}	135.7	MPa	0.00

Combined section checks

Combined section checks	Unity check [-]
Bending, Axial force and Shear	0.01

Stability checks
Decisive position for stability classification: 3.110 m
Section is classified as Class 1
Buckling group : Default

Buckling axis	k	L [m]	N_{cr} [kN]	M_{cr} [kNm]	λ_{rel}	χ
y-y	4.74	16.230	1599.55		1.12	1.00
z-z	0.75	2.570	3087.45		0.80	1.00
LTB	1.00	3.427		475.47	0.78	1.00

Combined stability checks

Interaction factors	k_{yy}	k_{yz}	k_{zy}	k_{zz}
Value	1.01	0.81	0.53	1.01

Maximum moment $M_{y,Ed}$ is derived from beam B57 position 1.710 m.
Maximum moment $M_{z,Ed}$ is derived from beam B57 position 2.060 m.

Combined stability checks	$M_{y,Ed}$ [kNm]	$M_{z,Ed}$ [kNm]	Unity check [-]
Bending and Axial Compression	69.00	0.01	0.25

Representative Beam

EC-EN 1993 Steel check ULS

Values: **UC$_{Overall}$**
Linear calculation
Combination: ULS-Set B (auto)
Coordinate system: Principal
Extreme 1D: Global
Selection: B117

EN 1993-1-1 Code Check
National annex: British BS-EN NA

Member B117	0.000 / 4.200 m	General cross-section	General	S 235	ULS-Set B (auto)	0.14 -

Combination key
ULS-Set B (auto) / 1.25*LC1 + 1.25*LC2 + 1.50*LC3

Partial safety factors		
Resistance of cross-sections	γ_{M0}	1.00
Resistance to instability	γ_{M1}	1.00
Resistance of net sections	γ_{M2}	1.10

Material			
Yield strength	f_y	235.0	MPa
Ultimate strength	f_u	360.0	MPa

Warning: Strength reduction in function of the thickness is not supported for this type of cross-section.

Section checks
Section is classified as Class 3

Section checks	Design force	Value	Unit	Resistance	Value	Unit	Unity check [-]
Compression	N_{Ed}	-230.84	kN	$N_{c,Rd}$	2947.65	kN	0.08
Shear V_y	$V_{y,Ed}$	-0.01	kN	$V_{pl,y,Rd}$	1334.51	kN	0.00
Shear V_z	$V_{z,Ed}$	-0.01	kN	$V_{pl,z,Rd}$	283.43	kN	0.00

Combined section checks

Combined section checks	Unity check [-]

Stability checks
Section is classified as Class 3
Buckling group : Default

Buckling axis	k	L [m]	N_{cr} [kN]	M_{cr} [kNm]	λ_{rel}	χ
y-y	2.00	4.205	13670.44		0.46	0.80
z-z	1.00	4.200	4114.95		0.85	0.55
LTB	1.00	4.200		1376.25	0.42	1.00

Stability checks	Design force	Value	Unit	Resistance	Value	Unit	Unity check [-]
Flexural buckling	N_{Ed}	-230.84	kN	$N_{b,Rd}$	1626.47	kN	0.14

Combined stability checks

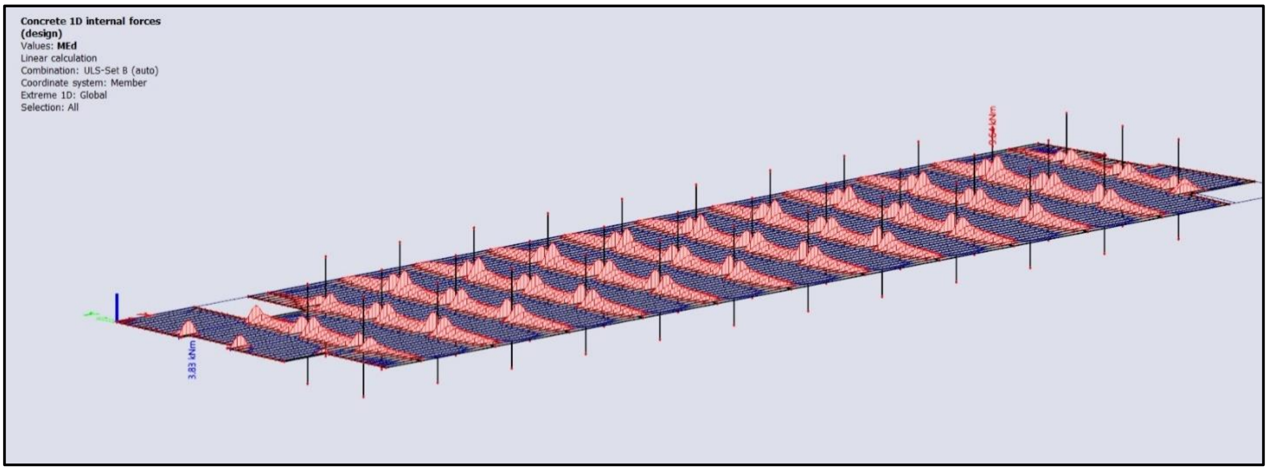
Interaction factors	k_{yy}	k_{yz}	k_{zy}	k_{zz}
Value	0.81	1.05	0.79	1.02

Maximum moment $M_{y,Ed}$ is derived from beam B117 position 2.100 m.
Maximum moment $M_{z,Ed}$ is derived from beam B117 position 2.275 m.

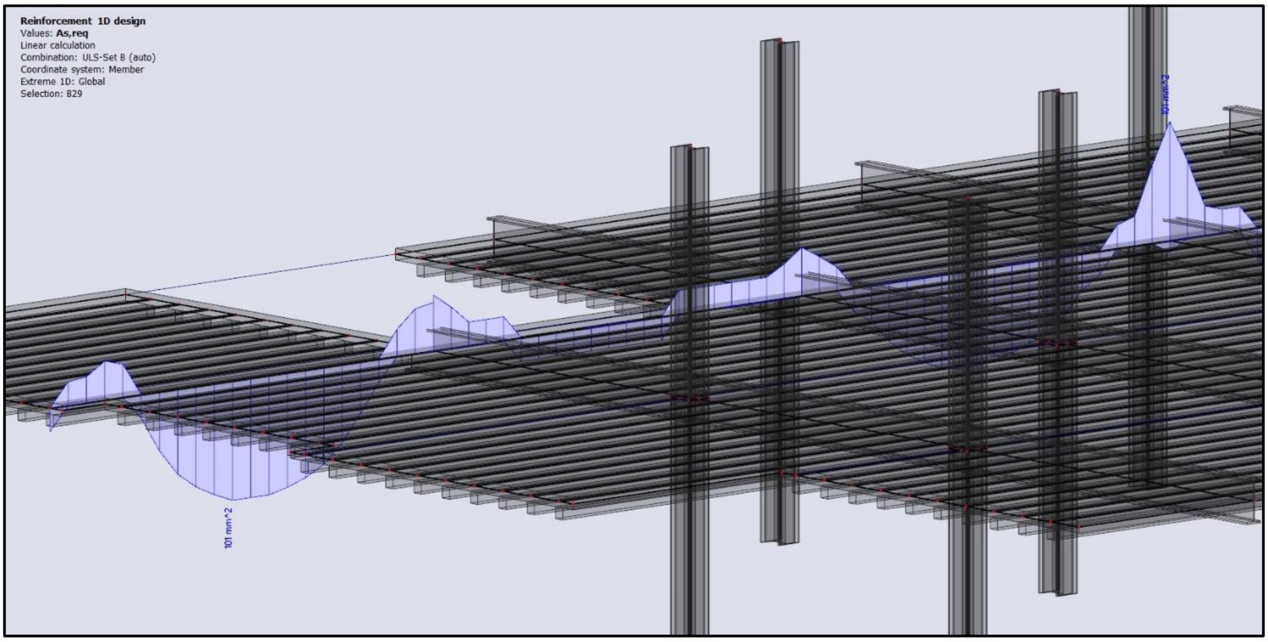
Combined stability checks	$M_{y,Ed}$ [kNm]	$M_{z,Ed}$ [kNm]	Unity check [-]
Bending and Axial Compression	-0.02	0.12	0.14

Representative Column

4.2. Concrete Design Check



Rib Bending Moments



Representative Rib – Required Reinforcement

BKPL observed 2xH20s T+B per rib (>101mm^2), therefore OKAY by inspection.

5. Key Notes & Conclusion

- Key structural elements were checked and deemed satisfactory in static design at the ultimate and serviceability limit states.
- A structural survey was undertaken to the full ground and first floor space, and the structural frame is virtually identical throughout. Whilst the checks have been specifically carried out in the bar area of the mezzanine (as this is the area that is most likely to see the highest loads and excitation from a crowd), the rest of the first floor can be taken as equivalent.
- A dynamic analysis was carried out to determine the natural frequency of the floor build-up (12.19Hz) and thereby demonstrate compliance with the prescribed minimum limit of 8.4Hz.
- A supplementary footfall analysis has also been carried out to demonstrate the extent of compliance with the maximum recommended limit (5.44 <<< 69.4).
- <u>BKPL have no concerns with the structure for the current use as a music venue</u>, or bar area. All design checks undertaken show that the structure is well within capacity for the ongoing usage.
- BKPL are aware of *Harry Seymour & Associates* and *Acoustic & Engineering Consultants Limited*'s assessments of the structure and have reviewed their reports.
  Whilst there are some numerical variations between the reports, these can be attributed to the differences in extent and conservativeness of either approach - for example, the finite element modelling undertaken by BKPL ignores the stiffening effect of concrete encasement in comparison to the frequency measurements undertaken by AEC; AEC's approach uses a simplified calculation for natural frequency. Even with these differences, <u>all outcomes, from all assessments, satisfy compliance</u>.